HERBERT HOOVER DIKE

MAJOR REHABILITATION EVALUATION REPORT

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Table of Contents

SUBJECT	PAGE NO.
1. Project Authorization	1
2. Location and Description	1
3. Current Operation and Use	3
4. Problem Identification	5
5. Project History	9
6. High Water Events and Emergency Repairs	20
High Water Event, 1995	20
High Water Event, 1998	36
Failure of Herbert Hoover Dike Tieback Levee, 1974	37
7. History of Project Costs	38
8. Major Rehabilitation Evaluation Approach	44
9. Economic Considerations	47
Federal Interest	47
Base Condition	47
With Rehabilitation Condition	47
Economic Life Cycle Model	48
Summary of Costs and Benefits	49
10. Reliability Analysis	51
11. Physical Consequences of Dike Failure	55
12. Engineering Evaluation of Alternatives	60
Base Condition	60
Modification of Lake Okeechobee Regulation Schedule	62
Increased Outlet Capacity	63
Relief Wells	63
Sub-Levees	63
Landside Toe Ditch Water Level Control	64
Landside Drainage Berm	64
Impervious Cutoff Wall	65
13. Environmental Considerations	65
Environmental Effects	66
Coordination and Correspondence	66
Reports and Studies	66
NEPA Documentation	67
Preparers	67

Table of Contents (continued)

SUBJECT	PAGE NO.
14. Assessment of Alternatives	69
Critical Assumptions and Key Variables	69
15. Recommended Plan	72
16. Major Rehabilitation Classification	75
17. Project Cost Estimate	77
18. Cost Sharing Considerations	77
19. Summary and Conclusions	77
Recommended Plan	77
Costs and Benefits	78
Conclusions	78

APPENDICES

A. Risk and Reliability Analysis.
B. Economic Analysis.
C. Environmental Documentation.
D. M-CACES Cost Estimate.
E. Project Cooperation Agreement.
F. Project Management Plan.
G. Schedule of Fully-Funded Project Costs by Fiscal Year.
H. Geotechnical Analysis.
I. Hydrology and Hydraulics Analysis.
J. Real Estate Issues.
K. History of Operation and Maintenance Expenditures.
L. Expert Witness Report and Resumes.

List of Plates

1. Herbert Hoover Dike
2. Structure 154 Vicinity Map

Table of Contents (continued)

List of Figures

SUBJECT	PAGE NO.
1. Location sketch.	2
2. Lake Okeechobee 1% flood stage hydrograph.	4
3. Herbert Hoover Dike problem sketch.	6
4. South Florida conditions, 1856.	10
5. Monument to those killed in 1926 and 1928 hurricanes.	10
6. Conditions near Pahokee, prior to dike construction.	11
7. Small dragline constructing a retaining dike near Pahokee.	12
8. Draglines building parallel retaining dikes.	12
9. Dipper and pipeline dredges at work.	13
10. Levee construction between retaining dikes.	13
11. Hydraulic dredging operation, circa 1935.	14
12. Gabion and riprap revetment near Port Mayaca.	18
13. Lake Okeechobee stage hydrograph during 1995 and 1998 high water events.	21
14. Ponded seepage at Lake Harbor site.	21
15. Piping and sandbagging at Lake Harbor site.	22
16. Sandbagged pipe exit points at Lake Harbor site.	22
17. Piping in toe ditch at Culvert 10.	23
18. Toe sloughing and piping at Culvert 10.	23
19. Sinkhole on levee crest at Lake Harbor site.	24
20. Sinkhole on levee crest at Lake Harbor site.	24
21. Landward of Lake Harbor sinkhole site.	25
22. Sandbag stockpiling near Lake Harbor site.	25
23. Sandbag weir to increase tailwater at Lake Harbor site.	26
24. Excavation of pipes above sandbag sites.	27
25. Excavation of pipes above sandbag sites.	27
26. Berm repair and toe ditch excavation at Lake Harbor site.	28
27. Stripping of peat layer from berm at Lake Harbor sinkhole site.	28
28. Berm at Lake Harbor site, after stripping of peat layer.	29
29. Backfilling of berm and toe ditch at Lake Harbor site.	29
30. Replacing topsoil at Lake Harbor site.	30
31. Repaired berm at Lake Harbor site, ready for grassing.	30
32. Drilling of vertical drains at Belle Glade site.	31

Table of Contents (continued)

List of Figures

SUBJECT	PAGE NO.
33. Belle Glade site; filling of vertical drains with No. 10 stone.	31
34. Belle Glade site; excavation of drain cap and laterals to drainage ditch.	32
35. Belle Glade site; vertical drains on 15-foot centers.	32
36. Culvert 10 site; cleared ditch looking north.	33
37. Culvert 10 site; seepage drain pipe and junction box.	33
38. Culvert 10 site; 48″ diameter perforated cmp drainage pipe wrapped in geotextile fabric.	34
39. Culvert 10 site; bringing up fill around drainage pipe and manholes.	34
40. Culvert 10 site; headwall construction at end of seepage drain.	35
41. Culvert 10 site; completed seepage berm looking west.	35
42. Culvert 10 site; completed seepage berm looking north.	36
43. Reach designations.	45
44. Location of population centers.	59
45. Typical cross section of recommended plan.	76

List of Tables

1. Operation, Maintenance, Repair and Rehabilitation Expenditures – HH Dike.	40
2. Emergency Repair Costs in Response to 1995 High Water Event.	42
3. Proposed Emergency Repairs in Response to 1998 High Water Event	43
4. Summary of Benefit/Cost Analysis.	49
5. Summary of Herbert Hoover Dike Reliability Analysis.	55
6. Hazard Potential Classification for Civil Works Projects.	61
7. Draft Environmental Impact Statement, List of Preparers.	68
8. Comparison of Recommended Rehabilitation Alternative and Base Condition.	74

1. PROJECT AUTHORIZATION.

Herbert Hoover Dike is a component of the Central and Southern Florida (C&SF) Project for Flood Control and Other Purposes. It is generally understood that the birth of the C&SF Project began with the Flood Control Act of 1948; however, Federal participation in local flood control efforts started much earlier, in response to the disastrous hurricanes of 1926 and 1928. The River and Harbor Act of 1930 authorized the construction of levees, for protection from storm surge-induced flooding, along the north and south shores of Lake Okeechobee. The 1948 Act created the C&SF Project and included authorization for enlargement of the existing levees and construction of additional levees along the northeast and northwest shores. The Flood Control Act of 1960 authorized the name of all levees around the shore of Lake Okeechobee to be “Herbert Hoover Dike”, in honor of the former President and his role in implementing levee construction.

2. LOCATION AND DESCRIPTION.

Herbert Hoover Dike is an earthen embankment system located along the perimeter of Lake Okeechobee, a large (724 mi^2 surface area) freshwater lake in south Florida (Figure 1). The lake is located about 30 miles west of the Atlantic Ocean and 60 miles east of the Gulf of Mexico. The lake and surrounding drainage area encompass approximately 5,600 square miles. The dike was constructed primarily to provide local flood protection. Components of the embankment system have been built intermittently since the early 1900’s. Federal involvement began in the 1930’s with the construction of dikes (for flood protection) along portions of the north and south shores. In the 1960’s, the crest elevations of those dikes were increased and additional embankments were constructed on the northwest and northeast shores. As a result, the Herbert Hoover Dike system now encircles Lake Okeechobee entirely, except in the vicinity of Fisheating Creek on the western shore (Plate 1). The existing embankments total about 140 miles in length with crest elevations ranging from 32 to 46 feet, National Geodetic Vertical Datum (NGVD)[1]. Adjacent land elevations typically range from 10 to 20 feet, NGVD. Lakeside levee slopes vary from 1V:3H to 1V:10H and landside slopes range from 1:2 to 1:5. Typical plan and elevation views of the dike are provided in Appendix H.

[1] - All elevations herein are referenced to NGVD.

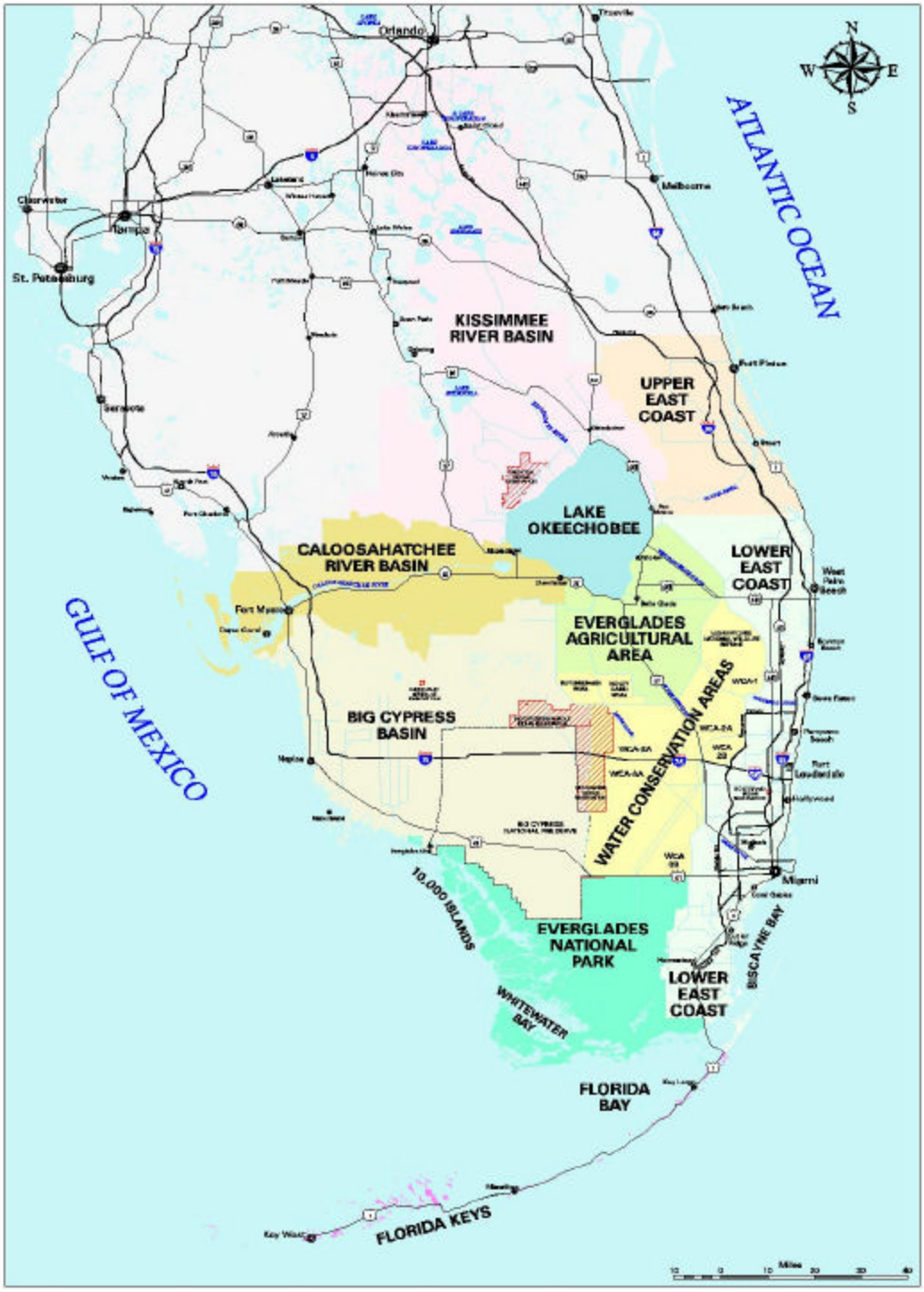


Figure 1. Project location.

In addition to the earthen embankments, the Herbert Hoover Dike system also includes numerous water control structures (e.g., culverts, spillways, locks, and pump stations) along its length. The lake, dike system, and related features are integral components of the C&SF Project, with functions related to flood protection, navigation, agricultural and municipal water supply, prevention of saltwater intrusion, recreation, and the enhancement of environmental resources.

3. CURRENT OPERATION AND USE.

Herbert Hoover Dike provides flood protection to urban and agricultural areas in five counties which border Lake Okeechobee. Water levels in the lake are operated according to a regulation schedule developed through the cooperative efforts of Federal, State, and local interests. The current schedule was established with consideration of many various issues, including area meteorology, hydrologic and hydraulic characteristics of the C&SF Project, water supply demands, and environmental factors. The primary objective of the regulation schedule is to provide enough storage capacity at the beginning of the wet season (June through October) to provide adequate flood protection for adjacent areas. This must be accomplished while simultaneously storing enough water during the wet season to meet agricultural and urban water demands during the subsequent dry season. Other major considerations include protection of estuaries at the downstream end of Lake Okeechobee outlet channels, and preservation of biological habitat provided by the lake’s littoral zone.

The current regulation schedule, referred to as RUN 25, was implemented in 1994. Its goal is to manage water control facilities in a manner that will result in typical lake levels between 15.65 and 16.75 feet. The schedule utilizes multiple operation zones that vary flood releases over a wide range before reaching maximum release rates. Inflow and outflow features of Lake Okeechobee are depicted in Plate 1. Inflows occur through the Kissimmee River, Fisheating Creek, Nubbin Slough (via S-191), and Taylor Creek (via S-133). Also, nine pump stations (S-2, S-3, S-4, S-127, S-129, S-131, S-135, S-153, and S-154) pass local drainage flows collected from areas around the lake; and, of course, rainfall over the lake (in excess of evapotranspiration) contributes to lake inflows. Flood releases are accomplished by passing water east through St. Lucie Canal (via S-308), west through the Caloosahatchee

River (via S-77), and south through five canals which lead to water conservation areas (via S-309, S-352, S-351, and S-354). The operation zones were designed to reduce damaging flows to the St. Lucie Canal and Caloosahatchee River estuaries without sacrificing the flood control or water supply benefits derived from the lake.

Regardless of the operating schedule and water management strategies in place, capabilities for discharge of floodwaters from Lake Okeechobee are limited. Lake Okeechobee flood stages are dependent on flood inflow rates, the lake stage prior to flood conditions, outlet structure and outlet channel discharge capacities, and local inflows to outlet channels (i.e., in excess of Lake Okeechobee discharges). Figure 2 depicts the Lake Okeechobee stage hydrograph for the 1% chance of exceedance flood condition.

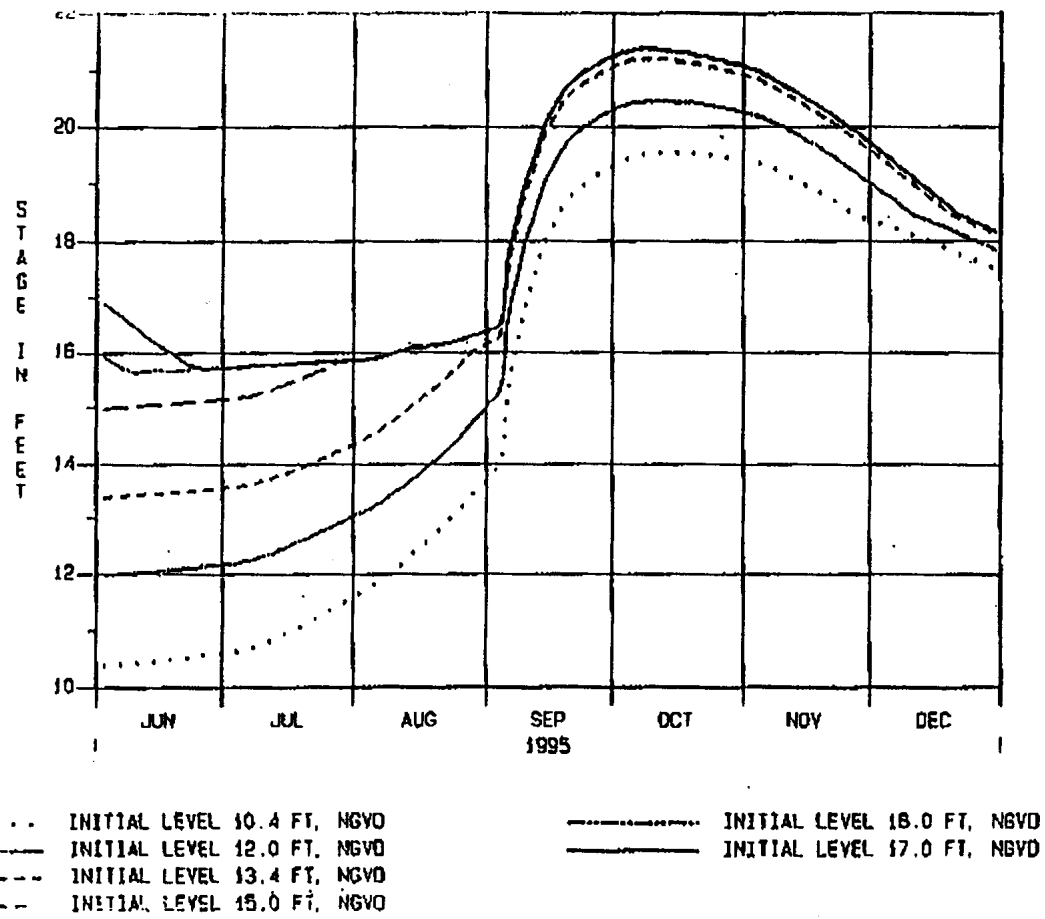


Figure 2. Lake Okeechobee 1% Flood Stage Hydrograph

It reflects realistic assumptions of inflow rates, discharge strategies, and local inflow demands on outlet channels. It demonstrates that under such conditions, the surface of Lake Okeechobee can be lowered at a rate of about 0.4 inches per day. Therefore, it is clear that during periods of high lake levels, and corresponding high hydraulic loadings on the dike, reduction of those loads by rapid reduction in lake level is not possible. Thus, it is important that Herbert Hoover Dike be capable of withstanding severe hydraulic loads for extended lengths of time, during which gradual reduction of lake stage can be accomplished. Herbert Hoover Dike, due to problems related to piping and internal erosion, does not possess that capability in its present state.

4. PROBLEM IDENTIFICATION.

This evaluation was undertaken to address problems related to piping, through-seepage, and structural stability of the Herbert Hoover Dike system. Since 1984, CESAJ reports have documented concerns related to embankment sections that are prone to seepage and stability problems. The problems are primarily a result of the heterogeneous nature of both the embankment and its foundation. They are best summarized by identification of the features depicted in Figure 3. A description of each numbered item is provided below:

(1) Peat-limestone interface – Subsurface conditions are often characterized by a peat layer underlain by a shallow six-inch thick layer of limestone. This situation creates a potential pathway through which seepage and erosion can occur.

(2) Toe-ditch blowout – In some areas, piezometric data indicates that the landside toe ditch is directly connected to the deep limestone.

(3) Artesian flow – Piezometric data indicates that artesian flow conditions exist in some areas of the dike toe.

(4) Toe ditch drawdowns – In some landside toe ditches, pumps can rapidly draw down water levels in the ditch, thereby increasing the differential head across the dike.

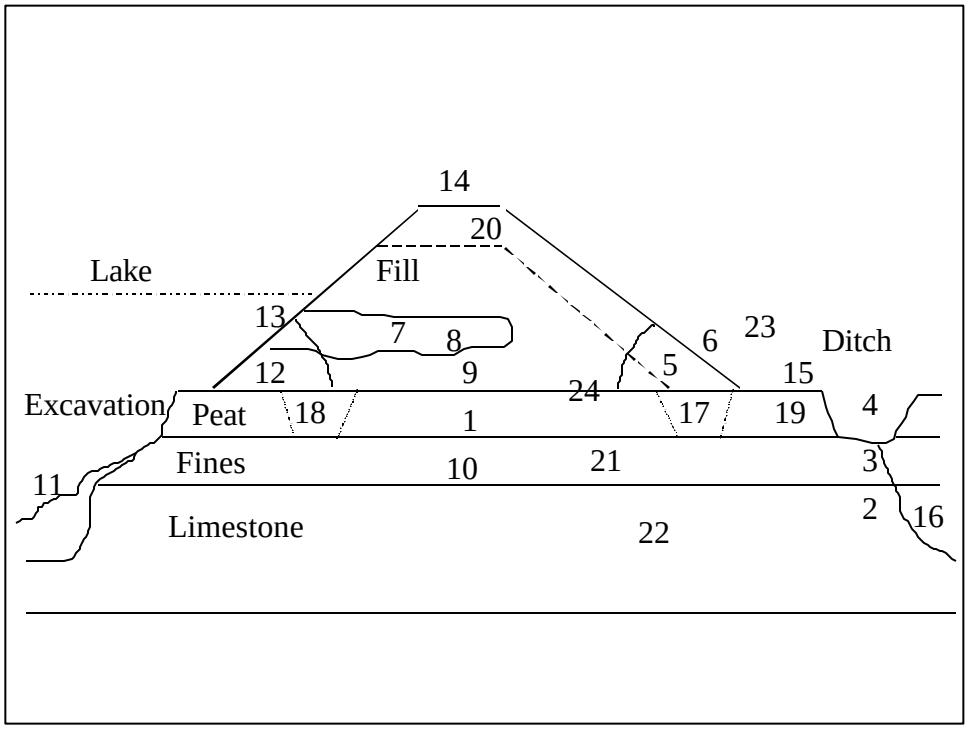


Figure 3. Herbert Hoover Dike problem sketch.

(5) Blocked landside toe – Some dike sections were constructed using landside containment dikes composed of muck and clay. Those dikes can block through-seepage, thereby raising water levels within the embankment.

(6) Soft toe and evidence of springs – The blocked landside toe can result in the presence of soft toe conditions and visual evidence of seepage exit points.

(7) Gravel layers – As a result of hydraulic dredge and fill construction methods, gravel and shell layers exist within the embankment. These layers are potential seepage paths.

(8) Soft embankment – Soft embankment sections, which may indicate the occurrence of internal erosion, have been observed in some areas.

(9) Soft foundation – Soft foundation conditions, which may indicate the occurrence of foundation erosion, have been observed in some areas.

(10) Pervious zones – Piezometric data have identified the presence of highly pervious dike sections. These may be related to gravel and/or shell layers, erosion, or other voids.

(11) Filtercake control – In some areas, lakeside excavations exposed deep limestone layers, which may become potential seepage pathways. The accumulation of muck and eventual covering (filtercake) of the exposed strata may have prevented additional seepage events from worsening.

(12) Blocked lakeside toe – Like the landside toe, some lakeside toe sections were characterized by clayey containment dikes, that block or divert seepage flows to higher elevations in the dike.

(13) Lakeside overflow – The blocked lakeside toe may result in a low piezometric surface until water overtops the clay layer, where it immediately increases the piezometric conditions inside the dike.

(14) Sinkholes – Numerous sinkholes have been observed along the crest of the dike.

(15) Quick conditions – In some areas, low blow counts at the levee toe have indicated an active quick condition that extends all the way to the depth of the limestone.

(16) Deep excavations – In some areas, deep excavation immediately adjacent to the landside toe have exposed deep, porous, limestone layers.

(17) Fire toe trench – In some areas, a fire toe trench was excavated in the organic peat layer and filled with any available less flammable material. This could result in a direct connection (through the fill) between the limestone and the base of the peat.

(18) Seepage trench – In some areas, this unusual construction feature was observed along the lakeside toe of the embankment. Like the fire toe trench, it provides a potential path connecting the limestone

layers to the base of the peat.

(19) Low density peat – This material along the landside toe may float when tailwater conditions are high.

(20) Two dike construction phases – The fact that portions of the present dike were constructed in two or three different phases (pre-1930, 1930's, and 1960's) lends additional complexity to the dike structure. For example, different construction methods and material sources add to the heterogeneity of the dike.

(21) Other erosion paths – Cemented horizons within the silt layers, stringers of limestone, or other geologic seams are potential seepage paths.

(22) Erodible shell layer – Subsurface investigations often identified the presence of a thin, erodible layer of shell.

(23) Vertical penetration – Other problem sources are vertical penetration of the embankment toe by power poles, abandoned wells, and or improperly filled core borings.

(24) Horizontal penetration – Piping and erosion paths may correspond to the location of pipes, conduits, water supply lines, and culverts (both active and abandoned).

The embankment was originally constructed using hydraulic dredge and dragline techniques; therefore, its composition is characterized by a variety of geologic materials including silts, clays, peat, sand, shell, and gravel. Beneath the embankment, foundation conditions are also non-uniform, with pervious layers of limestone, sand, gravel, and shell providing potential paths for underseepage and erosion. In some locations, embankment construction included excavation of an adjacent navigation channel (rim canal) which parallels the lakeside toe of the dike. Rim canal excavation may have exposed pervious subsurface layers, which could act as conduits for seepage flows. As a result, seepage forces at the landside embankment toe could cause the displacement of embankment materials and the creation of an uninterrupted seepage flow path from

lakeside to landside. If unchecked, such conditions can lead to substantial internal erosion of the embankment (or embankment foundation), a localized collapse of the dike, and breaching.

5. PROJECT HISTORY.

In the 19th century, prior to construction of Herbert Hoover Dike, the elevation of Lake Okeechobee's southern shore was about 21 feet, NGVD. Since that time, significant land subsidence has occurred and typical land elevation along the south shore is now about +14.0 feet. In those earlier days of the late 1800's (see Figure 4), the normal lake stage was about 20.5 feet and lake stages typically fluctuated about 2.5 feet per year (USACE, 1960). During periods of relatively high inflows from the Kissimmee River, Nubbin Slough, Taylor Creek, and Fisheating Creek, the lake waters would rise until the stage exceeded the elevation of the south shore. At that point waters would flow over the south shore and proceed further southward to the Everglades. A lesser portion of the lake's outflow would pass down the Caloosahatchee River to the Gulf of Mexico.

Between 1910 and 1920, local interests constructed the first levees built along the lake's perimeter. The levees extended for about 47 miles along the south shore between Bacom Point and Moore Haven. On the north shore, a short levee section was constructed just north of the lake's confluence with the Kissimmee River. These first levees were composed largely of muck excavated from adjacent borrow canals. Levee crest elevations typically ranged from 21 to 27 feet.

In 1926 and 1928, the Lake Okeechobee area was subjected to devastating hurricanes. Those storms produced wind-induced surge conditions on the lake and in some locations the existing levees were overtopped. The 1926 and 1928 storms claimed 386 and an estimated 2,700 lives, respectively. Monuments to commemorate those losses were later erected in both Clewiston and Belle Glade (Figure 5). These catastrophes demonstrated that the original levees built by local interests were not adequate, and prompted Federal involvement in the provision of flood protection to lakeside communities.

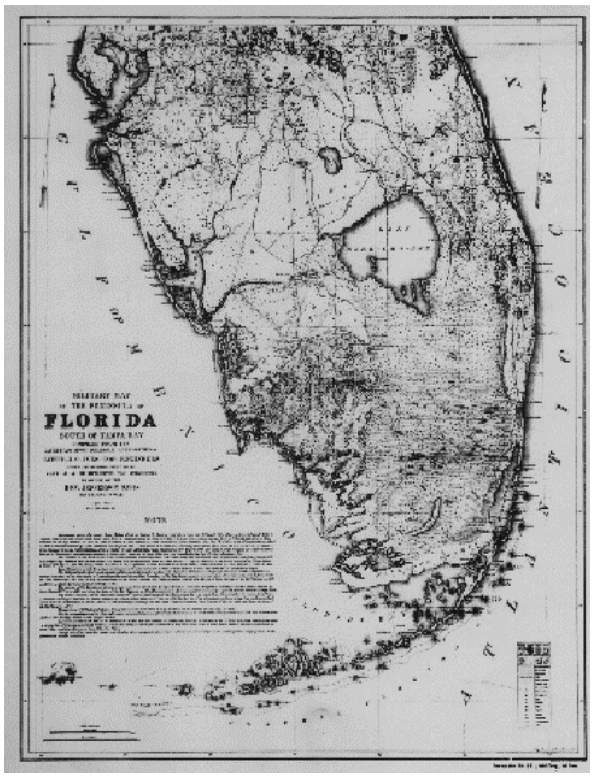


Figure 4. South Florida conditions, 1856.

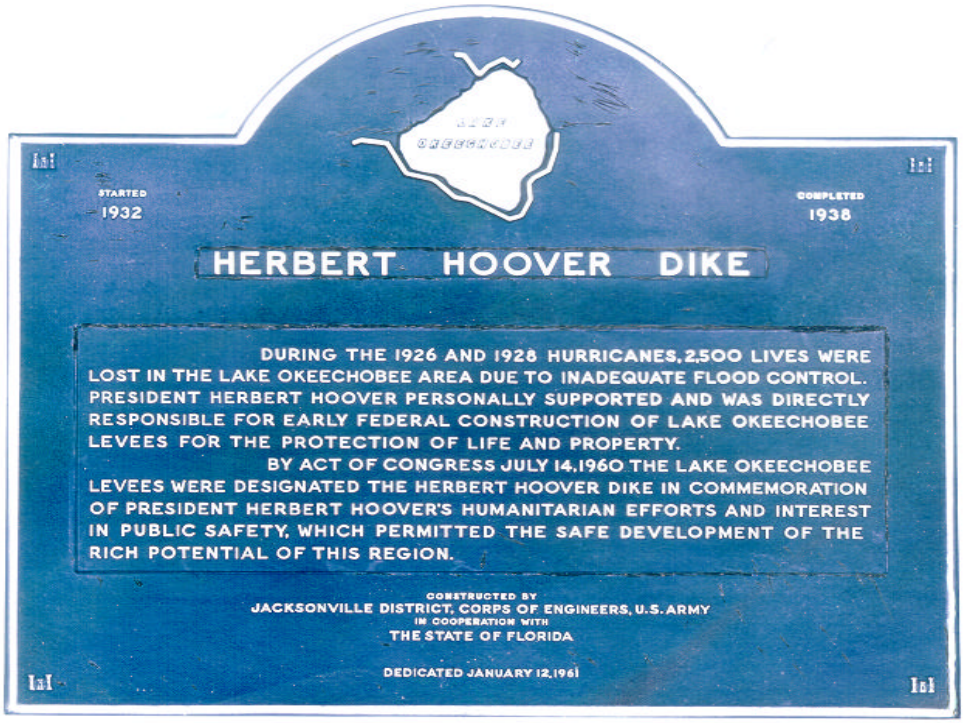


Figure 5. Monument to those killed in 1926 and 1928 hurricanes.

The first Federal embankments were constructed between 1932 and 1938. They consisted of 67.8 miles of levee along the south shore and 15.7 miles along the north shore. Figure 6 depicts an area near Pahokee prior to dike construction.

Figure 6. Conditions near Pahokee, prior to dike construction.

The cuspate shoreline features suggest that wave-induced erosion was occurring, and this 1935 photograph reveals little evidence of the original, locally constructed levees. Construction of the Federal embankments was accomplished with native materials available at the site. Draglines were used to first build retaining dikes as shown in Figures 7 and 8. Then, hydraulic pipeline dredges, dipper dredges, and draglines were used to construct the levee between the two retaining dikes, as shown in Figures 9 and 10.

It should be noted that the dipper dredge, on the right in Figure 9, is the floating counterpart of the familiar land-based mechanical excavating shovel. Because of its great leverage, it was used primarily to excavate hard compact material or rock. Blasting operations were also used to excavate rock material. Marl was also encountered frequently during dike construction. Marl is defined as a

Figure 7. Small dragline constructing a retaining dike near Pahokee.

Figure 8. Draglines building parallel retaining dikes.

Figure 9. Dipper and pipeline dredges at work.

Figure 10. Levee construction between retaining dikes.

loose or crumble earthy deposit that contains a substantial amount of calcium carbonate. After blasting or cutting through the rock and marl, hydraulic dredges were used to suck the material from the lake bottom and pump it to the construction site. As the water drained off, the rock, marl, and other materials were left to be shaped by draglines and bulldozers into the completed dike. The major point of this discussion is that a wide variety of native materials were used to construct the dike. In areas where the larger diameter rock, gravel, and marl were deposited, the dike is characterized by pockets of very porous material. Such areas are prone to excessive seepage. Evidence of this construction method is provided in Figure 11, which depicts a hydraulic dredge exit point. Although the quality of the photograph is poor, one can discern that the material immediately below the pipeline is 2- to 6-inch diameter rock. This material was probably placed by dragline or dipper dredge prior to installation of the hydraulic dredge pipeline. No impervious core or seepage control features were included during dike construction; therefore, sections such as that shown in Figure 11 are very porous and prone to seepage problems.

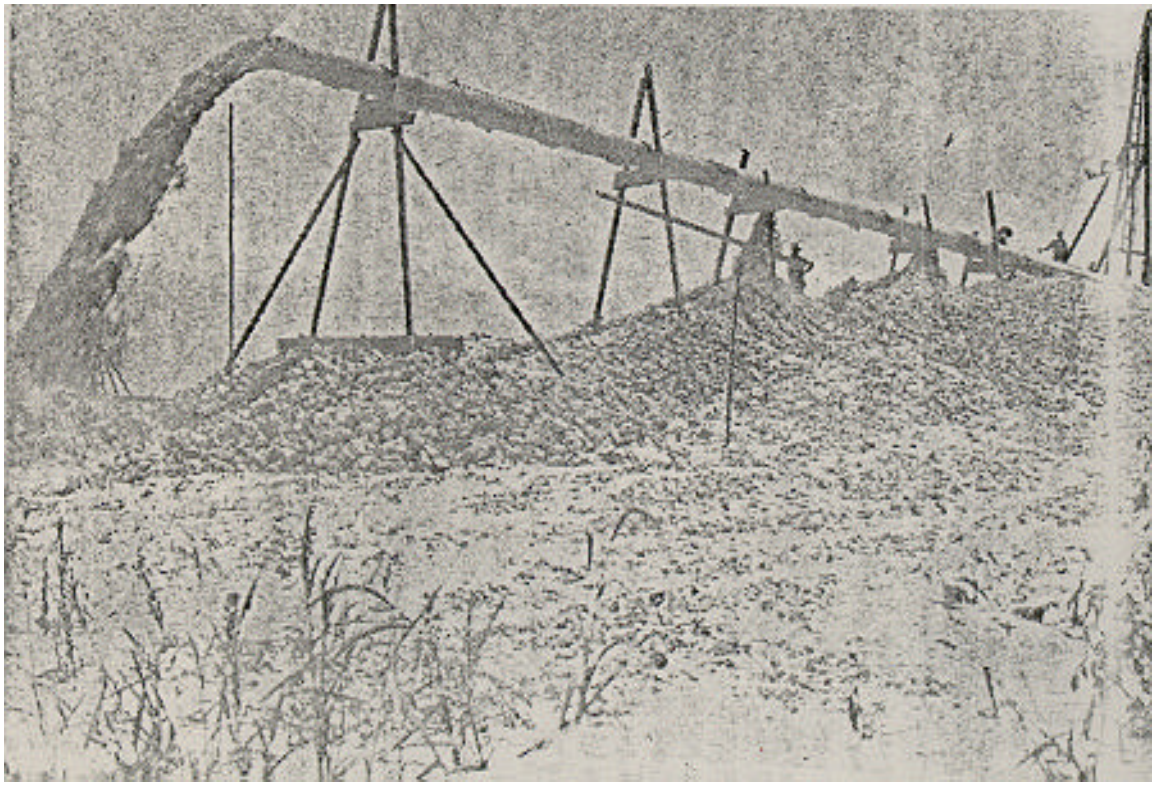


Figure 11. Hydraulic dredging operation, circa 1935.

Construction of the 67.8-mile long south shore dike was completed in 1936. The 15.7-mile long north shore dike was completed in 1938. The average crown elevation (32.5 feet) of the finished dike rose about 17-18 feet above the adjacent land elevation. Lake side slopes varied from 1V:3H to 1V:10H while landside slopes ranged from 1V:3H to 1V:4H. The cost of the completed dike was estimated at $14,500,000.

South Florida experienced extremely dry conditions between 1931 and 1945. As a result, groundwater levels decreased and saltwater intrusion of municipal water supply wells was experienced in Miami and other coastal cities. Also, land which in the past had regularly flooded, was now drying out and subsiding. The dry conditions sparked fires which claimed thousands of acres of peaty, organic, Everglades soil. During these dry years, Floridians realized that future water management efforts must consider not only drainage and flood control, but water conservation as well.

In 1947, the drought ceased as 100 inches of rain fell on south Florida. Three tropical cyclones inundated the Lake Okeechobee area between 17 September and 12 October. Flood damages were estimated at more than $59,000,000. Storm surge on Lake Okeechobee was as great as 6 feet along the south shore, which resulted in a short-duration, peak lake stage of about 22 feet, NGVD; however, the dike performed well and there was no surge-induced flooding or loss of life. This series of extreme drought and flood events prompted the U.S. Army Corps of Engineers to formulate a comprehensive water management program for south Florida. That program, the Central and Southern Florida (C&SF) Project, was authorized by the Flood Control Act of June 30, 1948. Its purposes included flood prevention, navigation, water level control, water conservation, prevention of saltwater intrusion, and fish and wildlife preservation. The design and construction of C&SF Project improvements related to Herbert Hoover Dike were accomplished in the 1950’s and 1960’s.

A 1959 General Design Memorandum (USACE, 1959) concluded that if Lake Okeechobee is to be operated as a multi-purpose reservoir, a levee system which will protect developed areas and provide sufficient outlet capacity to permit the lake to be regulated within safe limits must be provided. The plan of improvement included (a) construction of levees on the northwest and northeast

shores of Lake Okeechobee, and (b) raising of existing levees. It was recommended that the design of project works be based on the following hydraulic conditions: probable maximum hurricane on a 17.5-ft pool, standard project hurricane on a 21.6-ft pool (the 30-day average 100-year flood stage at that time), and moderate hurricane on a 24.8-ft pool (the 30-day average Standard Project Flood stage at that time). This resulted in levee crown elevations which ranged from 32 to 46 feet, NGVD. Typical elevations of adjacent lands ranged from about 10 to 18 feet, NGVD.

From 1959 through 1965, a series of Design Memoranda documented design efforts related to the dike improvements. They are summarized here to identify the extent of slope stability analyses undertaken as part of those improvements.

(1) For the new northeast shore levees, a stability analysis was performed with lake stage assumed at +17.5 feet. A landside slope of 1V:4H was required in Reach B (a distance of about 13,000 feet beginning about 3.5 miles north of St. Lucie Canal) where it was found that a slope stability safety factor of 1.39 was obtained with that slope. A landside slope of 1V:3H was recommended for Reaches A and C (the remainder of the northeast shore). The General Design Memorandum (USACE, 1963) stated that in these areas, the predominately sand embankment and foundation would permit a steeper slope, but the 1:3 was provided for ease of maintenance. Lakeside slopes were identified as 1V:6H. These slopes were not required for stability, but were dictated by hydraulic considerations (i.e., wave attenuation and reduction of wave runup). Seepage drainage was to be conveyed by the landside borrow pit to Pumping Station 135. Levee toe drains were not considered with respect to seepage because heads against the levee would be reasonably low except during hurricane conditions, at which time the water level would be at its highest, but only for short periods. The designers concluded that: “the levee also has, by design, a broad base which further reduces the need for levee toe drains” (USACE, 1963).

(2) For the new northwest shore levees, a stability analysis for dike sections that would be built on a sand foundation was performed with lake stage assumed as +23.5 feet. This analysis indicated that a 1V:2H landside slope

would be stable, with a factor of safety of 1.9. Along L-50, a 1V:4H landside slope was recommended due to the presence of soft peat in the upper part of the foundation. A 1:6 lakeside slope was required because of wave and runup considerations (USACE, 1959).

(3) For the existing southwest shore levees, a stability analysis was performed for a proposed landward slope of 1V:3H (assumed water surface elevation unknown). Results indicated that the 1:3 side slope would be stable with the provision of a 12-foot wide trench cut through the peat under the levee toe and backfilled with borrow material. Lakeside slopes (1V:5H to 1V:10H) were dictated by wave and runup conditions (USACE, 1961).

(4) For the existing southeast shore levees, a stability analysis was performed with lake stage assumed as +17.5 feet. The existing (pre-raised) 1V:4H levee backslope was found to have a safety factor less than 1.1. The levee raising was designed with 1:4 slope where peat layers were less than 6 feet thick, and 1:5 where peat layers were more than 6 feet thick. This resulted in a safety factor of 1.4. In some reaches, the available right-of-way would not contain these slopes. In those areas, steeper slopes were recommended, and as a result, the safety factor was reduced to a minimum of 1.25 for most critical cases (USACE, 1962).

During the 1970’s and 1980’s, CESAJ constructed revetments along certain sections of the Herbert Hoover Dike system, as depicted in Figure 12. The revetments were designed to protect the lakeside slope of the dike from erosion due to boat wakes and storm waves. A stability analysis was performed during the design of the east shore revetments. That analysis resulted in a safety factor of 1.3 with a lake stage of 17.5 feet, and a safety factor of 1.1 with a lake stage of 25.0 feet. It was stated that EM 1110-2-1902 requires a minimum safety factor of 1.5. This 1984 report contains the first documentation of the Jacksonville District’s (CESAJ) concerns related to dike stability. Two alternatives for improving dike stability were proposed. One involved construction of a stability berm along the landward toe; however, this was not recommended due to right-of-way limitations. The second alternative was a slurry-trench cutoff wall, which would increase the factor of safety to 1.46. This report recommended that CESAJ

undertake further study of methods to control seepage and increase dike stability (USACE, 1984).

Figure 12. Gabion and riprap revetment near Port Mayaca.

In response to the 1984 report, the South Atlantic Division (CESAD) indicated that further study of the overall stability of the dike system should be undertaken; and that the appropriate guidance was ER-1130-2-417, Major Rehabilitation Program and Dam Safety Assurance Program. That regulation requires preparation of a reconnaissance report that addresses the need and justification of any modifications for dam safety and funding authorization. It should be noted that a reconnaissance report is typically prepared with existing information; therefore, the report was to be based on historical surveys and existing subsurface geotechnical data (USACE, 1984b).

As instructed by CESAD, CESAJ prepared a reconnaissance report (USACE, 1986) addressing seepage and stability concerns along the dike system. That report indicated that factors of safety less than 1.3 were obtained for lake levels greater than 17.5 feet. It also pointed out that USACE's minimum required factor of safety for steady seepage is 1.5. The report recommended that a thorough seepage and stability analysis, including core borings and piezometer installation, be conducted.

In 1992, CESAJ completed a Letter Report (USACE, 1992) identifying Herbert Hoover Dike culverts which were in critical need of repair or replacement. Culvert locations were considered as weak links in the embankment system, due to their age and the fact that their barrels were not lengthened when the embankment was raised in the 1960's. Instead of lengthening the barrels, steeper slopes were constructed, and as a result, the shortest seepage paths through the embankment correspond to those culvert locations. Preparation of the Letter Report included stability analyses performed for lake elevations of 17.5, 21.6, and 25.0 feet, NGVD. The results indicated that factors of safety, for all conditions investigated, were below recommended threshold values. Culvert repairs were proposed at 18 individual culvert sites. Those repairs included barrel grouting and abandonment, installation of liners, installation of sheetpile wingwalls, installation of gabion revetments, excavation and backfilling for slope improvements, and wingwall foundation repairs. From July 1993 through July 1998, improvements were completed at the 18 problem sites, at a cost of about $6.5 million.

In 1993, CESAJ completed preparation of a Special Report which further documented the Corps of Engineers' concerns related to embankment seepage and stability problems (USACE, 1993). The results of the Special Report indicated that there is a high potential for levee instability due to seepage pressures along several dike sections. Safety factors related to heave indicated that potential heave problems exist at all sections analyzed. Analysis of creep ratios indicated that the potential for piping failures exists at all sections considered. It should be noted that the prolonged period between preparation of the 1986 Reconnaissance Report and the 1993 Special Report was due, in part, to the unusually low lake levels that existed between 1986 and 1991. In fact, drought conditions in 1989 and 1990 resulted in a 2.5-year long period in which the Lake Okeechobee stage did not exceed 14.0 feet, NGVD.

In June 1993, a General Design Conference was held to brief CESAD and Headquarters (HQUSACE) personnel on the contents of the Special Report. Findings of the Special Report and CESAJ's concerns related to dike stability were discussed. A major goal of the conference was to obtain CESAD and HQUSACE concurrence with the Jacksonville District's plan of action for additional efforts to quantify and remedy seepage and stability problems along the dike. CESAJ

proposed that those efforts be supported with Construction General (CG) funding under the Dam Safety Assurance Program, and cost-shared with a non-Federal sponsor.

In August 1994, HQUSACE indicated that the Dam Safety Assurance Program, and CG funding, were not the appropriate channels to support the proposed efforts (USACE, 1994). They did provide approval to proceed, under the Major Rehabilitation Program, with additional investigations to determine the scope and develop a recommended solution for the seepage and stability concerns. CESAJ was instructed to use O&M funding for preparation of a Major Rehabilitation Evaluation Report.

In summary, the Corps of Engineers has been addressing seepage and stability problems along Herbert Hoover Dike since 1984. From August 1994 to present, CESAJ has proceeded with preparation of this Major Rehabilitation Evaluation Report. Efforts to quantify the severity of seepage and stability problems have indicated that there is a very serious risk of dike failure due to piping.

6. HIGH WATER EVENTS AND EMERGENCY REPAIRS.

High Water Event, 1995 – In the late summer and early fall of 1995, Lake Okeechobee rose to a maximum elevation of 18.6 feet (Figure 13). Portions of the Herbert Hoover Dike system along the south and southeast shores exhibited evidence of substantial distress, without breaching, during that high water event. Nine major problem areas were identified. Significant visual distress indicators were observed along approximately 6,500 feet of dike near Lake Harbor. Other problem areas included about 500 feet of the dike north of Culvert 10, a 100-foot long dike section north of Pahokee, a 250-foot section of the dike west of Belle Glade, and a 2200-foot long section near S-352. At each of those sites, inspection teams discovered excessive seepage, piping of dike material, and sinkhole formation. Of particular concern were those areas where piping of dike and/or foundation materials was occurring. Cloudy pipe flows and deltas of fine sands and silts at pipe exits indicated that internal erosion was occurring. Immediate response measures included sandbagging of boils and piping exit points, and sandbagging of landside toe ditches to increase water levels on the landward side of the dike. Those conditions are depicted in Figures 14 through 23.

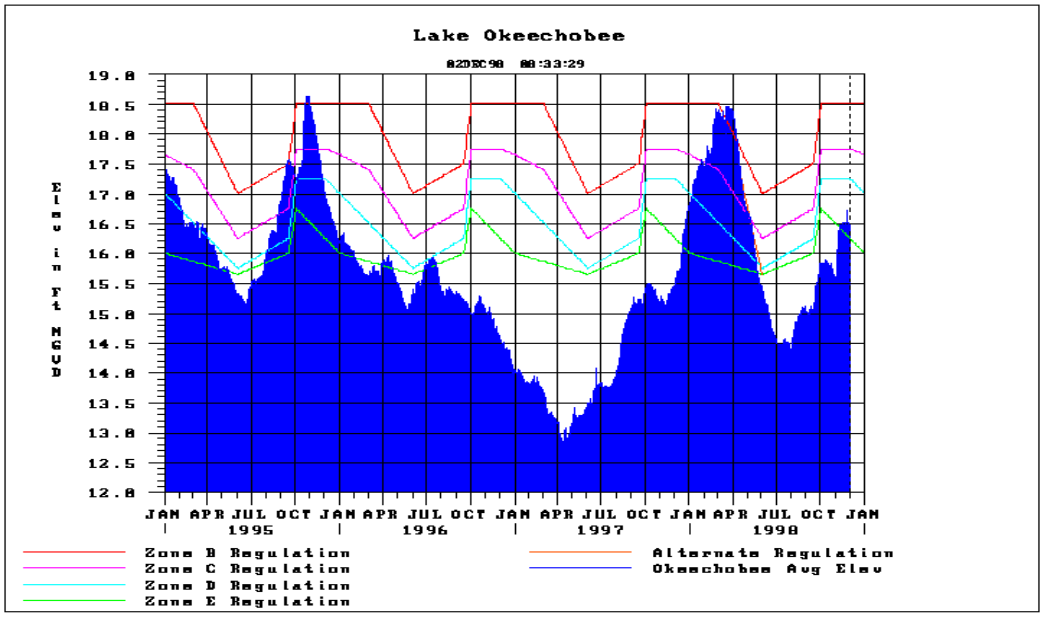


Figure 13. Lake Okeechobee stage hydrograph during 1995 and 1998 high water events.

Figure 14. Ponded seepage at Lake Harbor site.

Figure 15. Piping and sandbagging at Lake Harbor site.

Figure 16. Sandbagged pipe exit points at Lake Harbor site.

Figure 17. Piping in toe ditch at Culvert 10.

Figure 18. Toe sloughing and piping at Culvert 10.

Figure 19. Sinkhole on levee crest at Lake Harbor site.

Figure 20. Sinkhole on levee crest at Lake Harbor site.

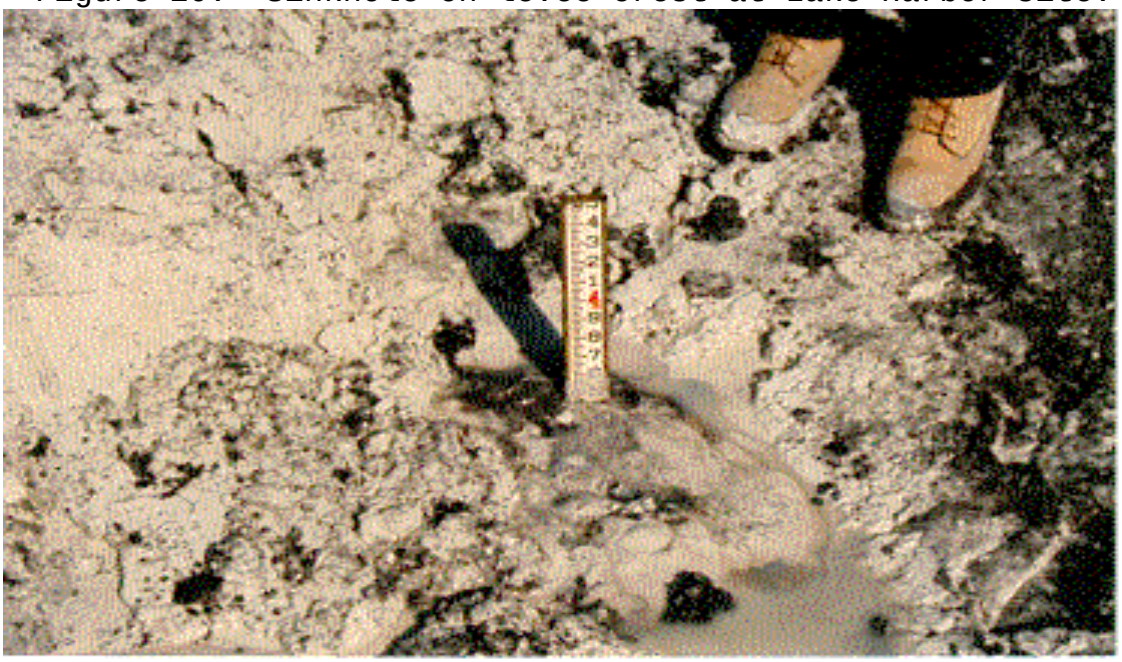


Figure 21. Landward of Lake Harbor sinkhole site.
Note liquefied material (4-ft survey rod pushed into berm).

Figure 22. Sandbag stockpiling near Lake Harbor site.

Figure 23. Sandbag weir to increase tailwater elevation at Lake Harbor site.

Remedial work at eight of the nine critical sites was accomplished in 1996. That work consisted primarily of excavation of pipes, stripping peat layers from the landside berm, reconstruction of the landside berm, and construction of vertical relief drains. It should be noted that these activities were designed as emergency repairs. A major objective was completion of repairs prior to the 01 June start of the 1996 hurricane season. This was accomplished at three of the five sites. The repairs were not intended nor designed to be a permanent solution to the seepage and stability problems. They only offer a level of protection comparable to the event that prompted their construction (i.e., roughly a 3% chance of exceedence event). In contrast, the measures constructed at Culvert 10 were essentially a prototype installation of the rehabilitation plan that CESAJ is proposing as a permanent solution. It consists of a landside seepage berm constructed of graded material, a relief trench, and a drainage system to collect and convey seepage flows. A more detailed description of the proposed rehabilitation

plan is provided in Section 15 (Recommended Plan), and Appendix H of this report. Photographs of the interim repairs, and Culvert 10 repairs are presented in Figures 24 through 42. Total cost of the 1996 emergency repairs was $1,266,358.

Figure 24. Excavation of pipes above sandbag sites.

Figure 25. Excavation of pipes above sandbag sites.

Figure 26. Berm repair and toe ditch excavation at Lake Harbor site.

Figure 27. Stripping of peat layer from berm at Lake Harbor sinkhole site.

Figure 28. Berm at Lake Harbor site, after stripping of peat layer.

Figure 29. Backfilling of berm and toe ditch at Lake Harbor site.

Figure 30. Replacing topsoil at Lake Harbor site.

Figure 31. Repaired berm at Lake Harbor site, ready for grassing.

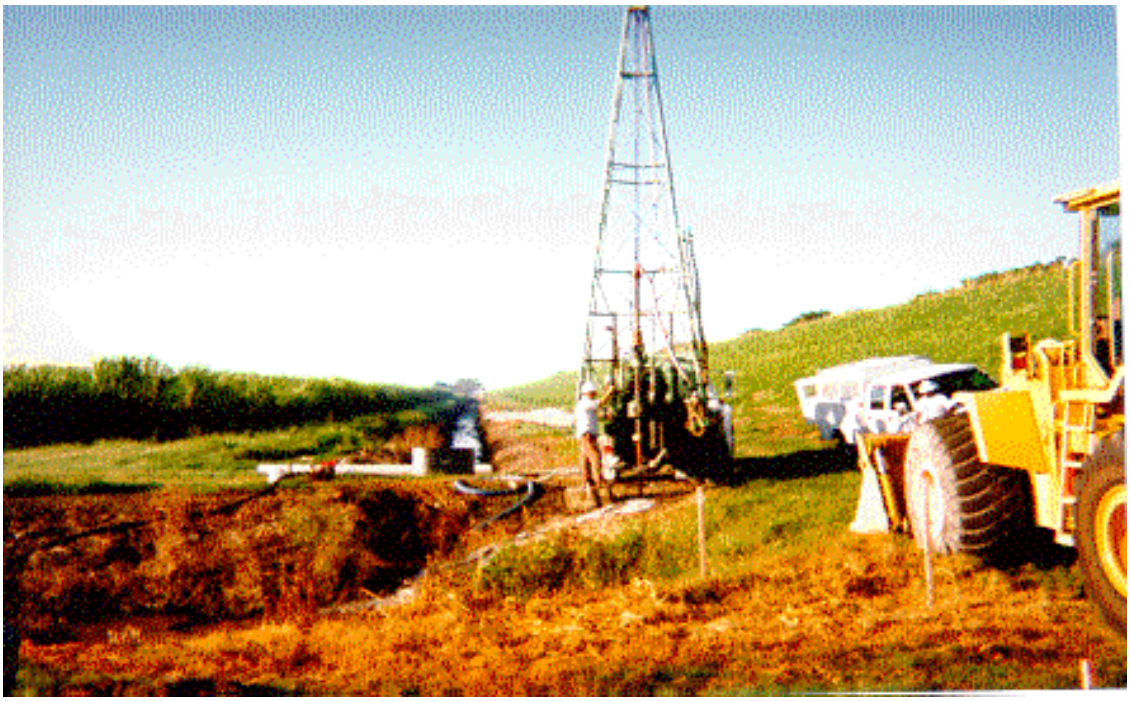


Figure 32. Drilling of vertical drains at Belle Glade site.

Figure 33. Belle Glade site; filling of vertical drains with No. 10 stone.

Figure 34. Belle Glade site; excavation of drain cap and laterals to drainage ditch.

Figure 35. Belle Glade site; vertical drains on 15-foot centers.

Figure 36. Culvert 10 site; cleared ditch looking north

Figure 37. Culvert 10 site; seepage drain pipe and junction box.

Figure 38. Culvert 10 site; 48″ diameter perforated cmp drainage pipe wrapped in geotextile fabric.

Figure 39. Culvert 10 site; bringing up fill around drainage pipe and manholes.

Figure 40. Culvert 10 site; headwall construction at end

of seepage drain.

Figure 41. Culvert 10 site; completed seepage berm looking west.

Figure 42. Culvert 10 site; completed seepage berm looking

north.

High Water Event, 1998 – In March 1998, the Lake Okeechobee pool elevation rose to 18.5 feet; therefore, intensive monitoring and troubleshooting activities were again necessary. The 1995-1996 emergency repairs constructed at Lake Harbor (four sites) and the seepage berm/drainage constructed at Culvert 10 performed well. Some noticeable seepage was exiting at the westernmost repair site at Lake Harbor; however, the wet berm conditions did not worsen. The repair site at Belle Glade did not perform adequately. The berm seepage problem was controlled relatively well; however, boils were regularly observed emerging from the landside toe ditch. Although there was evidence of recent excavation in the toe ditch, the relief drains should have relieved the hydraulic pressure and prevented boil formation if they had performed as expected. It should also be noted that the extent of boil formation along this ditch was much greater than had been observed in 1995. This suggests that an overall worsening of conditions is occurring at this site, presumable due to cumulative damage with each successive high water event. Although performance of relief wells at the Belle Glade site was unacceptable, the opportunity to utilize this alternative and observe its performance provided valuable information. Observations at this site have demonstrated the need for inclusion of the continuous relief trench as a feature of the recommended rehabilitation.

In response to the problems observed during the 1998 high water event, additional emergency repairs have been planned. Those repairs would include installation of a seepage berm, like the one constructed at Culvert 10, along approximately 1.5 miles of embankment near Belle Glade. Other proposed repairs would include modification of existing berms and installation of stop log risers to improve control of water levels in problem area toe ditches. The estimated cost of these emergency repairs is $2,414,000.

Failure of Herbert Hoover Dike Tieback Levee, 1974 - Historical evidence of dike failure with breaching is provided by the May, 1974 failure of a Herbert Hoover Dike-Kissimmee River tieback levee near structure S-154 (see Plate 2); however, that failure occurred under unusual

circumstances. Those circumstances are related to the fact that very dry conditions existed at the time of the breach, and the hydraulic gradient was opposite of that characteristic of a typical hydraulic loading. In other words, the landside water level was higher than the lakeside water level; therefore, the failure resulted in a flood release that flowed from the landside canal into the Kissimmee River (and Lake Okeechobee). As a result, only the water control structure and adjacent tieback levee were damaged and no flood-related damages occurred. The cost associated with rehabilitation of S-154 and the Herbert Hoover Dike tieback levee was $4,080,000.

The tieback levee extends from Lake Okeechobee, for a distance of about 6.5 miles along the north bank of the Kissimmee River. The tieback levee provides flood protection for the area north of the Kissimmee River and west of the town of Okeechobee. Some of the rainfall that collects in this protected area is passed into the Kissimmee River through S-154. In May 1974, dry conditions existed and the water level in Lake Okeechobee (and Kissimmee River near the lake) stood at an elevation of about 11.5 feet. Water levels in the canal on the north side of the tieback levee are maintained at an optimum elevation of 25.0 feet. At the time of the dike failure,

the canal water level was 20.5 feet; however, watermarks on S-154 indicated that at some time prior to failure, the north side water level had been about 25.7 feet. Thus, the difference in water levels on each side of the dike (i.e., the head, or hydraulic gradient) was estimated to be as high as 14.2 feet prior to failure, and 9.0 feet at the time of failure. As a result of this hydraulic loading, the tieback levee failed due to piping. The resulting breach in the levee was about 100 feet wide at the crown and 75 feet wide at the water line. Fortunately, due to the low lake level, the breach resulted in a flood release that flowed from the landside canal into the Kissimmee River, and subsequently on into Lake Okeechobee. In a more typical failure scenario, the water in the lake would be higher than the water on the landside of the dike; and a dike failure under those conditions would result in floodwaters released from Lake Okeechobee onto the adjacent lands. Under those conditions, damages would be much more severe than those experienced during the 1974 failure.

7. *HISTORY OF PROJECT COSTS.*

This section has been included to provide, on an annual basis, a summary of historical operation, maintenance, repair, and rehabilitation costs for the Herbert Hoover Dike system. Records of Operation and Maintenance (O&M) activities were studied in an effort to identify those items directly related to the levee system. Detailed information on specific activities is provided in Appendix K. Expenditures for each year were totaled and updated based on a 1998 cost index to approximate the present value of the repair expenses. A summary of the updated costs is presented in Table 1, which indicates that the average annual cost of levee maintenance, repair, and rehabilitation over the life of the project is approximately $5,370,000 per year. Table 1 reveals large expenditures in the years 1939, 1940, and 1941. In 1938, the east shore levee sustained considerable damage due to wave attack. Those damages were repaired during the subsequent three-year period. The nature of those repairs is described in greater detail in Appendix M.

Of particular importance are the costs specifically related to seepage and stability problems. Such information is available only for the years 1995 and 1998, in which high water conditions and related problems demanded emergency construction of seepage berms and relief wells. In

response to the high water event of 1995, emergency repairs were constructed at nine individual sites. Table 2 summarizes expenditures at each site, and indicates that 9,550 feet of repairs were accomplished at a cost of $1,266,000. It should be noted that these emergency repairs were limited in scope and do not provide the desired level of protection. They are regarded as interim repair measures, designed to improve conditions at specific sites until a more comprehensive, permanent solution can be constructed. In response to the 1998 high water event, interim repairs have been planned for construction at seven additional sites. The estimated cost of those repairs is $2,414,000. More detailed information is provided in Table 3. The designation of individual reaches of the dike system is explained in Section 8. The proposed rehabilitation measures would eliminate the need to implement interim repairs after each high water event, thereby reducing future O&M expenditures.

Table 1. Operation, Maintenance, Repair and Rehabilitation Expenditures – HH Dike.
(Expenditures for each year were updated based on a 1998 cost index)

Year	Reach 1	Reach 2	Reach 3	Reach 4	Reach 5	Reach 6	Reach 7	Reach 8	Total	Maximum Stage (ft)
1937	$9,374,129	$219,920	$72,117	$0	$0	$0	$0	$0	$9,666,166	17.3
1938	$2,278,532	$0	$0	$0	$417,186	$0	$0	$0	$2,695,718	16.55
1939	$18,410,820	$497,360	$163,196	$925,999	$355,535	$0	$0	$0	$20,352,910	17.34
1940	$40,588,723	$1,024,158	$336,152	$788,122	$732,113	$0	$0	$0	$43,469,268	17.22
1941	$20,949,113	$791,144	$259,544	$418,875	$389,108	$0	$0	$0	$22,807,784	16.65
1942	$419,942	$383,947	$125,982	$295,459	$274,462	$0	$0	$0	$1,499,792	16.64
1943	$260,841	$757,901	$78,252	$183,520	$170,478	$0	$0	$0	$1,450,992	15.36
1944	$78,412	$274,696	$23,524	$55,169	$51,248	$0	$0	$0	$483,049	15.19
1945	$536,297	$619,426	$160,889	$377,323	$689,102	$0	$0	$0	$2,383,037	17.22
1946	$337,762	$1,068,514	$350,453	$237,640	$906,221	$0	$0	$0	$2,900,590	16.9
1947	$151,084	$112,149	$36,799	$86,302	$80,169	$0	$0	$0	$466,503	18.77
1948	$303,964	$946,089	$462,215	$193,480	$750,470	$0	$0	$0	$2,656,218	17.77
1949	$206,817	$1,794,552	$461,823	$145,510	$1,715,965	$0	$0	$0	$4,324,667	15.53
1950	$0	$84,705	$18,847	$0	$0	$0	$0	$0	$103,552	15.16
1951	$5,199,123	$777,330	$1,700,751	$165,422	$153,666	$0	$0	$0	$7,996,292	15.97
1952	$708,521	$574,430	$188,390	$83,695	$77,747	$0	$0	$0	$1,632,783	15.81
1953	$114,195	$104,406	$34,258	$202,728	$74,634	$0	$0	$0	$530,221	17.66
1954	$94,626	$86,515	$28,388	$66,576	$61,845	$0	$0	$0	$337,950	16.01
1955	$235,416	$153,725	$50,419	$104,819	$97,370	$0	$0	$0	$641,749	14.39
1956	$371,094	$339,124	$110,950	$60,782	$56,463	$0	$0	$0	$938,413	12.85
1957	$235,640	$215,442	$70,692	$165,789	$154,007	$0	$0	$0	$841,570	15.45
1958	$178,045	$162,784	$53,414	$125,268	$116,365	$0	$0	$0	$635,876	16.18
1959	$83,532	$76,372	$25,060	$58,771	$54,594	$0	$0	$0	$298,329	15.93
1960	$289,817	$264,975	$86,945	$203,907	$189,416	$0	$0	$0	$1,035,060	17.69
1961	$201,158	$243,180	$60,347	$141,529	$131,471	$0	$0	$0	$777,685	15.3
1962	$261,550	$239,033	$78,481	$183,966	$134,087	$0	$0	$0	$897,117	15.15
1963	$8,613,973	$7,861,190	$2,586,908	$6,041,660	$108,307	$396,938	$0	$124,969	$25,733,945	14.72
1964	$10,923,138	$10,064,399	$3,280,400	$7,675,113	$129,362	$380,141	$0	$149,264	$32,601,817	14.63
1965	$4,499,135	$4,113,818	$1,350,024	$3,165,520	$2,940,516	$212,477	$0	$127,486	$16,408,976	15.13
1966	$165,539	$151,667	$49,939	$163,230	$108,201	$208,079	$0	$124,848	$971,503	15.94
1967	$154,510	$141,562	$46,612	$108,761	$100,992	$194,216	$0	$116,530	$863,183	14.9

Table 1. Operation, Maintenance, Repair and Rehabilitation Expenditures – HH Dike. (continued)

Year	Reach 1	Reach 2	Reach 3	Reach 4	Reach 5	Reach 6	Reach 7	Reach 8	Total	
1968	$228,769	$208,613	$67,852	$117,876	$109,884	$139,853	$131,861	$126,866	$1,131,574	15.8
1969	$183,962	$178,519	$54,427	$128,447	$189,460	$152,395	$143,686	$138,244	$1,169,140	16.8
1970	$226,661	$206,570	$67,154	$158,353	$133,757	$170,236	$160,508	$282,821	$1,406,060	16.77
1971	$210,589	$192,003	$62,663	$147,394	$164,657	$107,491	$101,348	$97,509	$1,083,654	14.44
1972	$282,019	$234,550	$76,271	$296,902	$164,343	$188,754	$177,968	$171,227	$1,592,034	14.07
1973	$335,333	$285,948	$90,078	$211,607	$120,878	$190,092	$110,112	$172,532	$1,516,580	15.17
1974	$352,332	$321,060	$104,240	$246,007	$229,329	$291,873	$275,194	$264,770	$2,084,805	15.6
1975	$431,446	$393,152	$127,647	$301,246	$280,823	$357,411	$336,988	$324,223	$2,552,936	14.63
1976	$472,167	$430,258	$139,694	$329,678	$307,327	$391,144	$368,793	$354,823	$2,793,884	14.81
1977	$374,779	$341,514	$110,881	$261,680	$243,939	$310,468	$292,727	$281,638	$2,217,626	14.58
1978	$359,986	$328,034	$106,505	$251,351	$234,310	$298,213	$281,172	$270,522	$2,130,093	16.81
1979	$305,683	$278,551	$90,439	$213,435	$198,965	$253,228	$238,758	$229,714	$1,808,773	17.62
1980	$1,834,426	$186,939	$60,695	$143,239	$1,192,829	$169,945	$160,234	$154,164	$3,902,471	17.47
1981	$354,757	$323,269	$104,958	$247,700	$230,907	$293,881	$277,088	$266,592	$2,099,152	13.84
1982	$219,269	$199,807	$64,872	$153,099	$142,719	$181,643	$171,263	$164,776	$1,297,448	17.64
1983	$408,350	$372,106	$120,813	$285,120	$265,790	$338,278	$318,948	$306,866	$2,416,271	18.26
1984	$6,806,636	$342,637	$111,246	$262,540	$244,741	$311,488	$293,689	$282,564	$8,655,541	16.71
1985	$4,252,591	$224,617	$72,928	$172,109	$160,441	$204,197	$1,527,894	$185,236	$6,800,013	15.34
1986	$2,558,643	$1,037,887	$94,713	$223,523	$208,369	$265,196	$996,213	$240,571	$5,625,115	15.13
1987	$2,441,340	$2,122,589	$94,305	$222,560	$207,471	$264,054	$1,310,260	$239,535	$6,902,114	16.09
1988	$1,098,814	$1,927,783	$73,327	$173,052	$161,320	$205,316	$1,044,552	$186,251	$4,870,415	16.36
1989	$2,908,854	$2,880,087	$95,891	$226,304	$210,961	$268,496	$2,584,741	$243,564	$9,418,898	14.4
1990	$5,853,303	$5,808,745	$148,529	$350,527	$326,763	$415,880	$5,743,392	$377,262	$19,024,401	13.06
1991	$3,011,043	$2,979,050	$106,642	$251,674	$234,611	$298,596	$2,932,128	$270,870	$10,084,614	16.47
1992	$228,493	$208,213	$67,602	$159,540	$148,724	$189,284	$178,468	$171,708	$1,352,032	16.32
1993	$364,323	$331,986	$107,788	$254,379	$237,133	$301,806	$284,560	$273,781	$2,155,756	16.42
1994	$437,632	$398,789	$129,477	$305,566	$284,849	$362,536	$341,819	$328,872	$2,589,540	17.52
1995	$504,364	$459,598	$149,220	$352,160	$328,285	$417,817	$393,941	$379,019	$2,984,404	18.63
1996	$356,505	$324,862	$105,475	$248,920	$232,045	$295,329	$278,453	$267,906	$2,109,495	16.3
AVG	$2,727,142	$961,204	$252,635	$485,349	$306,946	$265,493	$739,888	$226,398	**$5,369,626**	

Table 2. Emergency Repair Costs in Response to 1995 High Water Event.

Table 2. Emergency Repair Costs in Response to 1995 High Water Event

(Total costs have been adjusted to reflect 1998 values)

SITE (&REACH)		Length (feet)	Material (tons)	Actual Unit Cost / ton	Materials Cost ($)	Other Material Cost ($)	Equipment Rental ($)	E&D Cost ($)	SFOO Labor ($)	Total Cost 1998($)
Lake Harbor (1000') Initial	(R3)	1000	9500	9.75	92,625	0	11,000	20,657	52,356	187,236
Lake Harbor (500') Remaining	(R3)	500	5000	9.75	48,750	0	8,000	10,328	26,178	98,851
C-10 Cond. 4	(R1)	500	4500	6.28	28,260	34,532	0	10,328	26,178	105,256
Miller Cond. 4	(R1)	100	400	6.28	2,512	0	0	2,066	5,236	10,402
Lake Harbor Pt. 2 Cnd. 3	(R3)	1000	9000	5.45	49,050	0	14,000	20,657	52,356	144,227
Lake Harbor Pt.5,6,6A Cnd 3	(R3)	1500	13500	5.45	73,575	0	21,000	30,985	78,534	216,340
Lake Harbor Pt 6E, F,G	(R3)	2500	22500	5.45	122,625	$0	35,000	51,641	130,890	360,565
Reach 3, Pt. 4, Cond. 3	(R3)	250	1100	5.73	6,303	0	0	5,164	13,089	26,029
S-352 N & S Sites, Cond. 3	(R1)	2200	3200	6.83	21,856	0	0	45,444	115,183	193,433
TOTALS		9550	68700		$445,556	$ 34,532	$ 89,000	$197,270	$500,000	$ 1,342,339

Table 3. Proposed Emergency Repairs in Response to 1998 High Water Event.

Priority and Reach	Site Name	Problem Description	Recommended Repair	E&D Cost	Construction Cost	Comments
1 - R3	South S-351	Boils in ditch bottom; seeps and piping in ditch side slopes	Phase 1 install stop log risers to control water levels in ditch Phase 2 construct seepage berm, filter and drainage system.	$80,000	$2,000,000	Estimated length of repair is 1.5 miles
2 - R1	R1-10 Box Site	Existing core boring or well on berm	Excavate and regrout well or boring, backfill with granular filter stone	$3,000	$15,000	AE drill crew
3 - R3	R3-4; Old sand bag site from 1995 event	Piping and boils in ditch bottom	Construct seepage berm, excavate peat, backfill with drainage system and granular filter	$6,000	$240,000	Estimated length of repair is 1,000 feet
4 - R1	Quarry site	Possible piping damage	Inspect quarry side slope with divers	$3,000	$6,000	Divers from Operations Office required
5 - R3	Lake Harbor old hump site from 1995 event	Excess seepage and pore pressures	Modify existing seepage berm, excavate peat in embankment toe, backfill with granular material	$6,000	$24,000	Estimated length of repair is 1,000 feet
6 - R1	S-352 Canal Point	Boils in ditch bottom	Install two stop log risers and culverts to control water levels in ditches	$6,000	$15,000	
7 - R2	South Florida Operations Off		Replenish stockpiles of sand and granular material		$10,000	

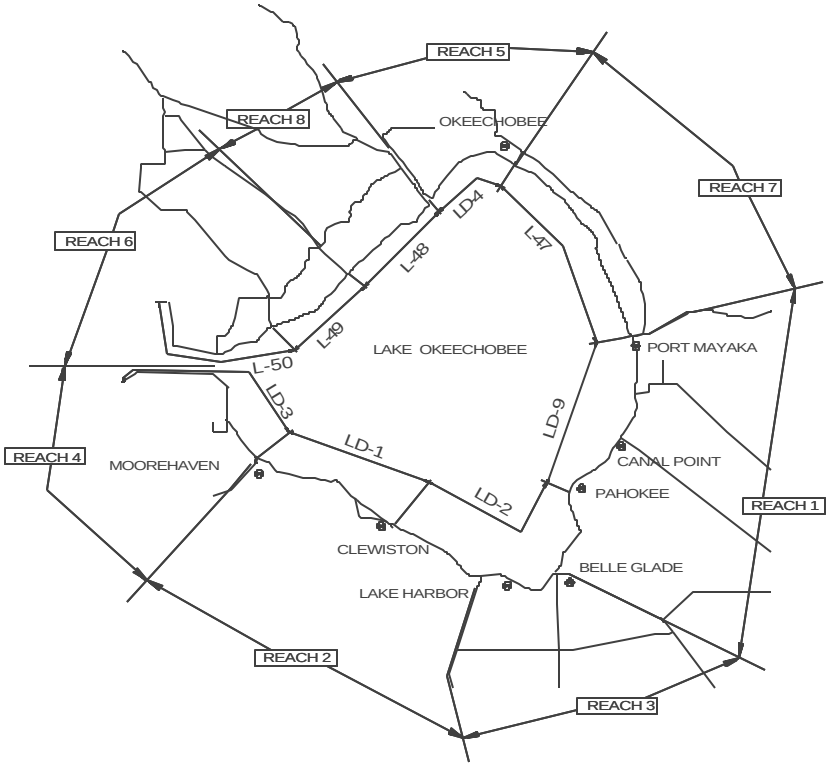
8. MAJOR REHABILITATION EVALUATION APPROACH.

The primary goal of this Herbert Hoover Dike Major Rehabilitation Evaluation is to ensure that a reliable levee system is provided along the perimeter of Lake Okeechobee. Due to the extensive length of the dike system, and the variability of conditions that exist along its length, the study area was divided into eight reaches, as depicted in Figure 43. The delineation of those reaches was based on factors such as physical characteristics of the dike, foundation conditions, drainage features, and the location of population centers. Each reach was assigned a priority rating (1 through 8), which reflects the assumed severity of potential seepage and stability problems within that reach. Reach 1 (highest priority) encompasses the southeast lake shore and extends for 22.35 miles between S-308 at Port Mayaca and S-351 at Belle Glade. Reach 2 is 20.42 miles long and extends from S-354 at Lake Harbor to S-77 at Moore Haven. The 6.70 mile long embankment section between S-354 and S-351 is designated as Reach 3. Reach 4 extends from S-77 for a distance of 15.69 miles top the L-43 tieback levee at Fisheating Creek. Reach 5 is located at the north end of the lake and includes 14.54 miles of levee between the Kissimmee River and Nubbin Slough. The 27.99 mile long Reach 6 lies along the west shore Between Fisheating Creek and Indian Prairie Canal. Reach 7 is 17.55 miles long and extends from Nubbin Slough to S-308 at Port Mayaca; and Reach 8 (lowest priority) includes the 12.5 miles of dike between Indian Prairie Canal and the Kissimmee River.

This Major Rehabilitation Evaluation employed a comprehensive approach, with engineering, economic, and environmental analyses performed for the entire Herbert Hoover Dike system. However, during geotechnical field investigations, more detailed, site-specific information was obtained for the engineering analysis of Reach 1. This approach has allowed preparation of a Major Rehabilitation Evaluation Report which:

(a) determines if rehabilitation measures related to seepage and stability problems are warranted,
(b) provides economic justification for the rehabilitation measures,
(c) addresses environmental issues related to the proposed rehabilitation,

FIGURE 43


REACH 5
REACH 8
OKEECHOBEE
REACH 7
REACH 6
LD4
L-48
L-47
L-49
LAKE OKEECHOBEE
PORT MAYAKA
L-50
LD-3
LD-9
LD-1
REACH 4
MOOREHAVEN
CANAL POINT
PAHOKEE
LD-2
REACH 1
CLEWISTON
BELLE GLADE
LAKE HARBOR
REACH 2
REACH 3

(d) serves as the technical supporting document for a comprehensive Project Cooperation Agreement, and
(e) allows direct progression into preparation of Plans and Specifications for rehabilitation of Reach 1.

The evaluation has indeed indicated that rehabilitation efforts are warranted; therefore, upon approval of this report, a series of additional efforts will be initiated if appropriate funding is available. First, efforts to prepare Plans and Specifications for the rehabilitation of Reach 1 will be undertaken. This report is directed toward obtaining a 2001 construction start for rehabilitation of Reach 1. Concurrent with Reach 1 Plans and Specifications efforts, preparation of the next Major Rehabilitation Evaluation Report, addressing Reaches 2, 3, and 7 will be initiated. The analysis of those three reaches is being combined into one Major Rehabilitation Evaluation Report. That decision was based on information gained as a result of the comprehensive engineering analysis of the dike system (documented herein), and observations of performance during the 1995 and 1998 high water events. Those efforts and events have demonstrated the urgency of rehabilitation needs along the southern and eastern shores of Lake Okeechobee. In response to that urgency, it is recommended that Herbert Hoover Dike remedial measures in Reaches 1, 2, 3, and 7 be undertaken in the most efficient manner possible. Additional Major Rehabilitation Evaluation Reports addressing potential problems in the remaining reaches (4, 5, 6, and 8) will follow.

It should be noted that the scope and level of detail of this first Major Rehabilitation Evaluation Report will facilitate efficient, timely preparation of subsequent reports for the other reaches. As mentioned previously, the economic, environmental, and engineering analyses have been performed for the entire dike system. The economic life cycle analysis model has been developed in such a way that it can immediately be applied to any rehabilitation scenario in any or all reaches. The major tasks remaining for evaluation of the other reaches involve more site specific geotechnical engineering, at a level of detail commensurate with that accomplished for Reach 1, as documented in this report.

9. ECONOMIC CONSIDERATIONS.

Federal Interest – Through the Flood Control Act of 1936, the U.S. Congress established as a nationwide policy that flood control (i.e., flood damage reduction) on navigable waters or their tributaries is in the interest of the general public welfare. Therefore, flood control efforts, in cooperation with State and Local entities, are proper activities of the Federal government. That Act provided that the Federal government may participate in improvements "for flood control purposes if the benefits to whomsoever they accrue are in excess of the estimated costs, and if the lives and social security of people are otherwise adversely affected." Herbert Hoover Dike provides flood control benefits to approximately 40,000 people (1996 census) who reside in protected areas. Rehabilitation of the levee system is necessary to ensure that an adequate level of flood protection is provided. Also, flood control efforts continue to be a primary project purpose for U.S. Army Corps of Engineers Civil Works activities, as defined in Engineering Circular EC 11-2-175, "Army Programs – Annual Program and Budget Request for Civil Works Activities, Corps of Engineers, Fiscal Year 2000." Therefore, there is a Federal interest in the rehabilitation of Herbert Hoover Dike.

Base Condition – The base condition is synonymous with the "without project" condition. It is the alternative against which all other plans will be measured and compared. The base condition assumes that the Jacksonville District will continue to operate the dike in the most efficient manner possible (within the limits of available funding) without the proposed rehabilitation. Under the base condition, the reliability concerns and threat of dike failure which exist today will continue to exist in the future; therefore, persons and property protected by the dike will continue to be subjected to an unacceptable risk of dike failure and the catastrophic consequences of such a failure.

With Rehabilitation Condition - The primary difference between the base condition and all other plans considered is related to dike performance. For any component of the dike, the probability of unsatisfactory performance (PUP) is directly related to (a) characteristics of that dike component (i.e., geometry, composition, foundation condition, etc.), and (b) the Lake Okeechobee pool elevation. The probability of a dike failure is the

product of a dike component's PUP at a given lake stage, and the probability of that lake stage. If a component of the dike system were rehabilitated, then the risk of dike failure that currently exists would be reduced or eliminated. In other words, that dike component's PUP would change. The extent of the reduction in failure probability would depend on the characteristics of the proposed rehabilitation alternatives. For example, a simple berm or relief well alternative, such as the 1996 interim repair measures, would reduce probabilities of unsatisfactory performance by some amount. However, construction of the more substantial seepage berm with drainage system, such as that constructed at Culvert 10, would provide a much greater reduction in the chance of failure.

It should be noted that not only were different rehabilitation alternatives considered, but many various rehabilitation strategies were also investigated. Due to the wide variation in conditions that exist along the length of the dike system, some sections are more problematic than others. Also the consequences of a dike failure differ greatly, depending on the failure location. As a result, the dike system was divided into individual components. A matrix of various strategies involving the rehabilitation of individual components and combinations of multiple components was developed and analyzed. Consideration was also given to the severity of problems along different sections of the dike, the suitability of different rehabilitation measures for different dike sections, and the preferred timing of proposed rehabilitation activities.

Economic Life Cycle Model – In an effort to adequately address the uncertainties inherent to this seepage and dike stability evaluation, a life cycle analysis model was developed. The model employed a risk-based approach to simulate (a) hydraulic conditions on Lake Okeechobee, (b) the potential breaching of the dike in response to those hydraulic conditions, and (c) socio-economic damages associated with a breach. A detailed description of the life cycle model and all aspects of the risk and uncertainty analysis is provided in Appendix A (Risk and Reliability Analysis).

The life cycle model was used to quantify benefits and costs over a specified project lifetime for the base

condition and all rehabilitation strategies. Major economic benefits of a rehabilitation strategy were measured as the difference between the present discounted value of life cycle costs associated with a rehabilitation alternative, and those associated with the base condition. As a result, benefit cost ratios could be generated for comparison of the economic justification of various rehabilitation strategies.

Summary of Costs and Benefits – The estimated first cost for rehabilitation of the 22.35-mile length of Reach 1 is $73,805,605. Interest during construction is estimated to be $8,536,592 resulting in a total project investment of $82,342,197. The estimated benefit to cost ratio is 1.07 to 1.

The primary economic consequences of a breach were considered to be flood damages in those areas normally protected by the dike system. Those damages included residential and non-residential structures, non-seasonal and seasonal agricultural damages, roadway damages, railroad damages, public utility damages, and damages to the dike system itself. These are the damage categories which were monetarily quantified and used in the economic

Table 4. Benefit Summary

Annual Cost	$5,685,205
Annual Benefits	
Life Cycle Model	$5,371,253
Insurance	
Administrative cost savings	$ 733,650
Total Annual Benefits	$6,104,903
Net Benefits	$ 419,698
Benefit to Cost	1.07

comparison of project costs and benefits; however, it should be noted that other types of damages exist which were not quantified monetarily. Although they were not included in the economic analysis, these damages should be considered in a qualitative sense when issues related to

justification of the proposed rehabilitation are weighed. Those damages include:

(a) Emergency Operation Center (EOC) Costs – Costs associated with establishing, staffing, and operating an EOC in response to a dike failure would be significant.
(b) Litigation Costs – It would be realistic to assume that costs related to litigation would be significant in the wake of a dike failure.
(c) Sugarcane Rail Costs – Sugarcane fields are often equipped with rail systems to facilitate planting and harvest efforts. Damages to these rail systems were not quantified.
(d) Estuarine Damages – Even controlled releases of Lake Okeechobee waters have allegedly had negative impacts on the estuarine ecosystems of the St. Lucie and Caloosahatchee Rivers. Those impacts include fish kills, fish lesions, shellfishery damages, and damage to seagrass beds and other benthic resources. If harmful releases were not already being discharged at water control structures, a dike failure which resulted in uncontrolled flooding of these estuaries would have similar impacts. These environmental damages were not quantified as part of the economic analysis.
(e) Human Suffering – The potential for human suffering and loss-of-life as a result of a dike failure is significant. The population (40,000, 1996 estimate) residing in areas protected by the Herbert Hoover Dike system consists primarily of six communities, all of which are located immediately adjacent to the dike system. Five of the six communities are located along the south and east shores where seepage and stability concerns are greatest; therefore, the population at risk is substantial. Flooding studies indicate that flooding of these communities would be severe and warning times would be limited. Such impacts were not included in the economic analysis, but should be considered in decisions regarding project justification.

10. RELIABILITY ANALYSIS.

Guidance applicable to the Major Rehabilitation Program requires that probabilistic benefit-cost analyses be used to determine if a project is economically justified. This involves assessing the differences in benefits between an existing levee and a proposed rehabilitated levee. To support this type of analysis, geotechnical engineers must characterize the reliability of the levees in question. The reliability is typically expressed in a probabilistic form as a *reliability index*, β, *probability of unsatisfactory performance*, Pr(U), or *probability of failure,* Pr(f). For the purposes of this report, Pr(U) and Pr(f) are synonymous. Unsatisfactory performance and failure both imply the occurrence of a levee breach, which results in an uncontrolled flood release of water from Lake Okeechobee onto adjacent lands.

Reliability indices provide a relative measure of the current dike condition and provide a qualitative estimate of expected performance. Embankments with relatively high reliability indices will be expected to perform well. Embankments with low reliability indices will be expected to perform poorly and present major rehabilitation problems. If the reliability indices are very low, the dike may be classified as hazardous. In compliance with ETL 1110-2-547 (USACE, 1995), the following target reliability indices were used to measure the condition of Herbert Hoover Dike components.

Target Reliability Indices

Expected Performance Level	Beta	Probability of Unsatisfactory Performance
High	5.0	0.0000003
Good	4.0	0.00003
Above Average	3.0	0.001
Below Average	2.5	0.006
Poor	2.0	0.023
Unsatisfactory	1.5	0.07
Hazardous	1.0	0.16

The following paragraphs summarize the methods employed to calculate reliability indices and corresponding probabilities of unsatisfactory performance for individual

components of the dike system. It should be noted that guidance related to the development and execution of those analysis tools was obtained from the following sources.

(a) ETL 1110-2-547, 30 Sep 1995, Introduction to Probability and Reliability Methods for Use in Geotechnical Engineering.
(b) EC 1110-2-554, 27 Feb 1998, Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies.
(c) ETL 1110-2-532, 1 May 1992, Reliability Assessment of Navigation Structures.
(d) Wolff, T.E., 1995. "Probabilistic Analysis in Geotechnical Engineering," Short Course for the U.S. Army Engineer District, Jacksonville, March 7-10, 1995.
(e) Wolff, T.E., Demsky, E.C., Schauer, J., and Perry, E., 1996. "Reliability Assessment of Dike and Levee Embankments." Proceedings of Uncertainty '96.

The probability of unsatisfactory performance is the probability that the value of the performance function will approach the limit state, or that an unsatisfactory event will occur. The natural log of the factor of safety, ln(FS), is taken as the performance function and the condition, ln(FS)=0, is taken as the limit state. β incorporates the information inherent in the factor of safety, but additionally provides a measure of the relative certainty or uncertainty regarding parameter values. The calculation of β involves five steps:

(1) Identifying a performance function, typically ln(FS) for geotechnical analyses.
(2) Identifying the random variables which contribute uncertainty to the performance function.
(3) Characterizing the random variables in terms of their expected values E[X], coefficients of variation, V_x, and, where necessary, their correlation coefficients, ρ_{xy}.
(4) Determining the expected value and standard deviations of the performance function.
(5) Evaluating β from the results of step 4.

For step 1, the safety factor against slope failure was determined using the UTEXAS3 computer program (Edris and Wright, 1987). The safety factor for seepage failure was

taken as the ratio of the critical exit gradient, i_c, to the exit gradient at the landside toe, i_o, predicted by methods developed by the Corps of Engineers (USACE, 1956). Exit gradients were calculated using the computer program LEVEEMSU (Wolff, 1989). It should be noted that measurements and observations made during the 1995 and 1998 high water events were used to calibrate the performance function models. More detailed information on this subject, and other aspects of the reliability analysis is provided in Appendix H.

For step 2, random variables related to slope stability included the shear strength parameters, c and ϕ. Random variables considered in the seepage analysis included the horizontal and vertical permeability values for all materials, tailwater elevation, gravel pocket distributions, and critical gradient.

For step 3, the probabilistic moments were calculated by standard statistical means when available data allowed. For some variables, expected values, variance, and correlation coefficients were based on limited data, assumed probability distributions, and engineering judgement.

For step 4, the expected value and standard deviation of the performance function was determined using the Taylor's series first-order, second-moment (FOSM) mean value approach.

For Step 5, β values were calculated as the number of standard deviations by which the expected value of the performance function exceeds the limit state.

The reliability index is useful for comparing the relative reliability of one structure to another, the relative reliability of a structure for different performance modes, and the relative change in reliability of a structure subjected to changing loads. However, for incorporation into the Economic Life Cycle Analysis Model, the β values had to be expressed as probabilities of unsatisfactory performance, Pr(U). This allowed them to be utilized as multipliers in concert with corresponding water level probabilities and economic consequences of dike failures. For this analysis, Pr(U) was taken as the cumulative probability for the standard normal distribution evaluated

at $-\beta$ standard deviations, or $Pr(U)= \Phi(-\beta)$. The individual failure probabilities for each failure mode (i.e., slope stability, underseepage, and through seepage) were then systematically combined into one composite probability of failure function. This was accomplished by consideration of a dike component failure as a series system. This implies that a dike component will fail if inadequate performance is demonstrated by any one of the failure modes, or,

$$Pr(f_{total})= 1-(1-Pr(f_{ss}))(1-Pr(f_{us}))(1-Pr(f_{ts}))$$

where $Pr(f_{total})$= total failure probability of a component.
$Pr(f_{ss})$= slope stability failure probability
$Pr(f_{us})$= underseepage failure probability.
$Pr(f_{ts})$= through seepage failure probability.

As mentioned previously, the entire dike system was divided into eight reaches. However, variation in dike conditions within some reaches led to the further subdivision of a reach into dike components. For example, the increased level of detail of geotechnical information for Reach 1 led to the subdivision of Reach 1 into five individual dike components. Those components exhibited differences in physical characteristics that warranted a separate reliability analysis. A similar subdivision was carried out for Reach 6; therefore, the final reliability analysis of Herbert Hoover Dike considered 13 individual dike components. They are referred to as Components 1A, 1B, 1C-a, 1C-b, 1C-c, 2, 3, 4, 5, 6A, 6B, 7, and 8. A summary of the reliability assessment for Herbert Hoover Dike is provided in Table 5. The estimated failure probabilities and corresponding expected performance levels clearly demonstrate the severity of seepage and stability problems along Herbert Hoover Dike. The 18- and 21-foot pool elevations included in Table 5 correspond roughly to the 3% and 1% chance of exceedance flood stages. The 26-foot stage is approximately equivalent to the Standard Project Flood (SPF) elevation on Lake Okeechobee. The reliability analysis suggests that along three dike components (1C-a, 2, and 3) the expected performance is categorized as hazardous, at lake levels as low as 18.0 feet. At 21.0 feet, performance ratings along the entire dike system range from poor to hazardous. Of particular concern is the expected performance along all of the south and east shore sections of the dike. This includes all of the levee system from Moore Haven east and north to Nubbin Slough.

At the SPF elevation, expected performance levels for the base condition are unacceptable for the entire dike system. This reliability analysis, and the observed occurrence of

Table 5. Summary of Herbert Hoover Dike Reliability Analysis

Base Condition Probabilities of Dike Component Failure, Pr(f), at Various Lake Stages

Dike	Lake Okeechobee Stage, (ft, NGVD)					
Component	18	EPL	21	EPL	26	EPL
1A	0.00750	Below Average	0.45000	Hazardous	1.00000	Hazardous
1B	0.00750	Below Average	0.39000	Hazardous	1.00000	Hazardous
1C-a	0.12000	Hazardous	0.91000	Hazardous	1.00000	Hazardous
1C-b	0.03000	Poor	0.49000	Hazardous	1.00000	Hazardous
1C-c	0.07880	Unsatisfactory	0.68000	Hazardous	0.88000	Hazardous
2	0.12000	Hazardous	0.89000	Hazardous	1.00000	Hazardous
3	0.14000	Hazardous	1.00000	Hazardous	1.00000	Hazardous
4	0.00250	Above Average	0.05000	Unsatisfactory	0.13400	Hazardous
5	0.00250	Above Average	0.05000	Unsatisfactory	0.13400	Hazardous
6A	0.00130	Above Average	0.01950	Poor	0.05500	Poor
6B	0.00200	Above Average	0.03120	Poor	0.08000	Unsatisfactory
7	0.05960	Unsatisfactory	0.99000	Hazardous	1.00000	Hazardous
8	0.00250	Above Average	0.05000	Unsatisfactory	0.13400	Hazardous

EPL = Expected Performance Level (based on ETL 1110-2-547 target reliability indices)

seepage problems during two recent high water events, indicates that the likelihood of a levee failure is unacceptable during periods of high lake stage. As a result, corrective rehabilitation measures should be efficiently implemented to reduce that risk of failure and ensure that acceptable levels of flood protection are provided by the Herbert Hoover Dike system.

11. PHYSICAL CONSEQUENCES OF DIKE FAILURE.

The term “dike failure” implies a catastrophic breaching of some portion of the Herbert Hoover Dike system. This situation would result in flooding, as waters from Lake Okeechobee passed through the breach and onto adjacent lands. A failure would be initiated by the continuous uncontrolled seepage of waters from one side of the levee to the other. The seepage could occur as either through-seepage or under seepage. If seepage flows increased to a rate that begins to displace material from the dike, or its foundation, then this piping can erode material from within, or beneath, the levee. This erosion could eventually create large voids in or beneath the dike. If the voids become large enough, the dike is weakened, and

sections of the levee could collapse. Such a collapse would reduce the levee crown elevation in the immediate area to a point where lake waters would overtop the dike. At that point lake water flows through the breach would be uncontrollable, levee erosion would continue, and flooding of adjacent areas would result.

The physical consequences and extent of damages associated with a dike breach are primarily influenced by the three following parameters:

(1) Breach Location – Although some levee sections are more reliable than others, a breach could possibly occur at any location along the 140-mile long Herbert Hoover Dike system. The consequences of a failure along the south shore, for example, are different than those corresponding to a failure at the north end of the lake. To account for this, breaching analyses and corresponding flood inundation mapping were done to simulate failures at each of the thirteen dike components. Multiple breaches (i.e., the simultaneous failure of more than one component) were also considered.

(2) Lake Okeechobee Pool Elevation – The severity of flooding after a breach occurs is directly related to the lake stage which exists at the time of the breach. Breaching analyses and corresponding flood inundation mapping were accomplished for three pool elevations, 18-, 21-, and 26-feet. Those elevations correspond to the 30-year, 100-year, and SPF Lake Okeechobee flood stages, respectively.

(3) Closure Time – The severity of flooding after a dike failure is also related to the time required to accomplish closure of the breach. Based on published information, experience, and engineering judgement, two possible breach closure scenarios were evaluated. One assumed that conditions following a breach would allow closure to be accomplished in a period of ten days. The other scenario assumed that breach closure efforts would require 45 days for completion. For any given breach, the determination of which scenario would apply was dependent on the magnitude of the hydraulic gradient. A high gradient would result in

more rapid flow rates which would hinder the closure operations.

More detailed information concerning the breaching analysis is provided in Appendices A (Risk and Reliability Analysis) and I (Hydrology and Hydraulics Analysis). All inundation maps and flood stage hydrographs for the affected areas are also included in Appendix I. Those maps and hydrographs provide information related to time of flood arrival and depth of flooding for specific locations throughout the inundated areas. This information was subsequently used in the economic analysis to quantify flood-related damages.

Economic consequences of a dike failure include damage to the levee itself. Damages to the dike include costs associated with closing the breach as quickly as possible. Emergency closure measures considered included closure at the existing levee line, crescent-shaped closure on the levee's exterior, and closure on the levee's interior. The consideration of specific closure methods included, riprap placement, pile driving, barge sinking, and closure using site-specific natural features such as nearby tree islands. Other costs included repairing the dike to a more suitable condition after the emergency closure was accomplished, flooding was stopped, and water level conditions had receded to a point where construction operations were possible. The economic analysis assumed that the dike would be repaired to a state comparable to that which existed prior to its failure.

The major economic consequences of a breach are, of course, related to flood damages in those areas normally protected by the dike system. Those damages included residential structures, non-residential structures, non-seasonal (i.e., fruit tree) agricultural resources, seasonal (row and truck crops) agricultural resources, roadway damages, railroad damages, and damage to public utilities. The extent of damages is related to the size of the flooded area and depth of flooding, as identified by the flood inundation maps and flood stage hydrographs. More detailed information related to the damage calculations is provided in Appendix B (Economic Analysis).

Of utmost importance is the real potential for human suffering and loss of life which could result from a dike failure. The U.S. Army Corps of Engineers typically utilizes a "hazard potential classification" system to

evaluate the hydrologic hazard potential of dams, as stipulated in ER 1110-2-1155, Engineering and Design, Dam Safety Assurance Program (USACE 1997). That classification system is provided in Table 6. The guidance states that:

> "If there is certainty that one or more lives will be lost due to failure or incorrect operation of the project, the project should be classified as high hazard. This certainty should be due to extensive residential or industrial development in the flood plain downstream of the project, and should be confirmed by inundation mapping which considers population at risk, time of flood wave travel and warning time."

The population residing in areas protected by the Herbert Hoover Dike system consists primarily of six communities. They are Pahokee (pop. 6,993), Belle Glade (pop. 16,656), Clewiston (pop. 6,645), Moore Haven (pop. 1,439), Okeechobee (pop. 4,831), and South Bay (pop. 5,043). Populations were estimated based on 1996 census results. As shown in Figure 44, all of those communities are located immediately adjacent to the dike system, and five of the six communities are located along the south and east shores where seepage and stability concerns are greatest; therefore, the population at risk is substantial. Inundation mapping and flood stage hydrographs indicate that flooding of these communities would be severe and warning times would be limited. In light of these considerations, it is imperative that efforts to rehabilitate the Herbert Hoover Dike system be initiated in the most efficient manner possible.

Figure 44

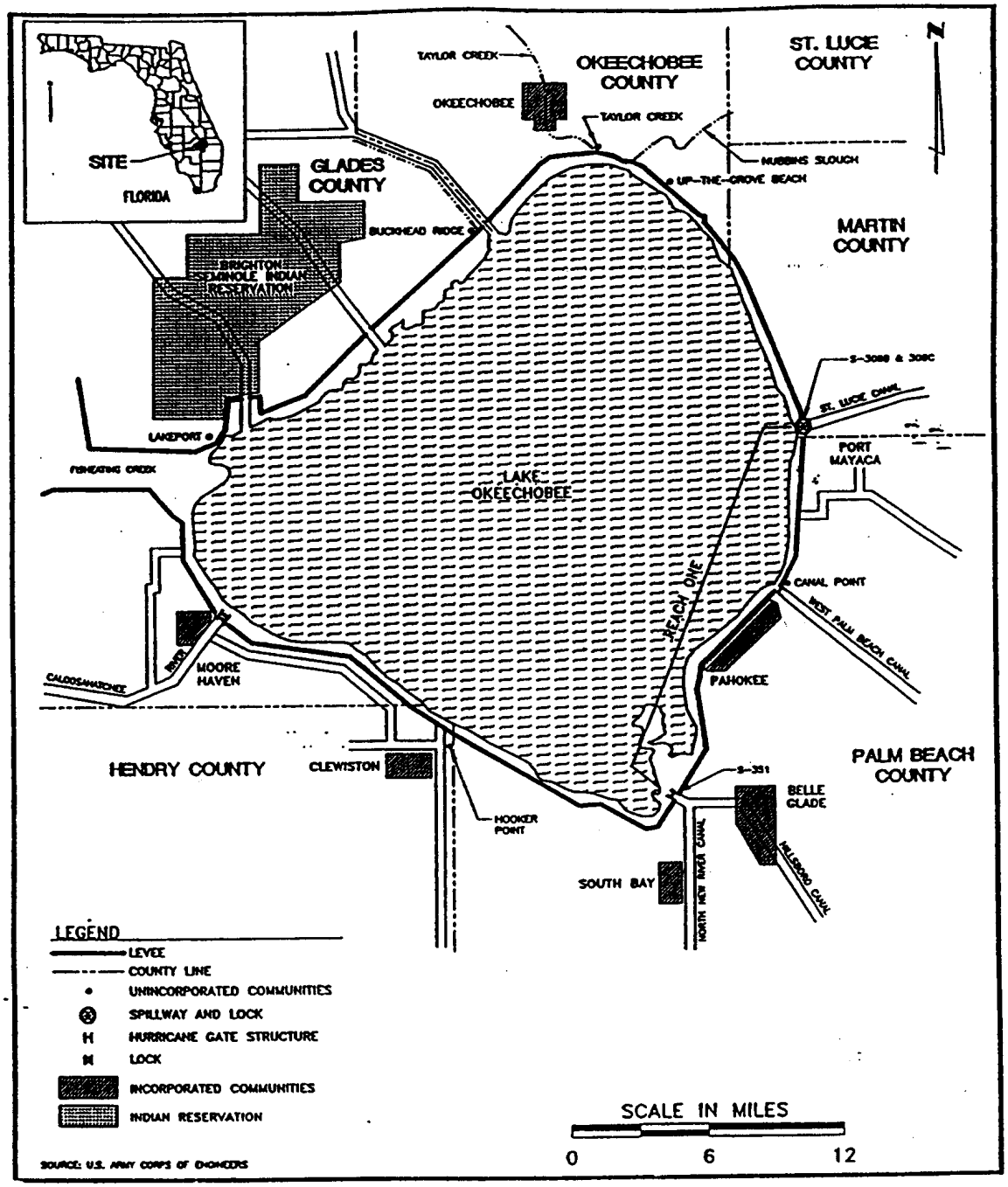


Figure 44. Location of population centers.

12. ENGINEERING EVALUATION OF ALTERNATIVES.

The following paragraphs summarize the rehabilitation alternatives that were considered. Some alternatives were rejected early in the evaluation process due to excessive costs, inadequate levels of protection provided, or unacceptable environmental consequences. Such omissions will be identified in the descriptions provided below. A summary of rehabilitation alternatives considered is provided in Table 6a.

a. Base Condition.

The base condition is also referred to as the “without project” condition. It is the alternative against which all other rehabilitation plans will be measured and compared. The base condition assumes that the Jacksonville District will continue to operate the dike in the most efficient manner possible (within the limits of available funding) without the proposed rehabilitation. Under the base condition, the reliability concerns and threat of dike failure which exist today will continue to exist in the future; therefore, persons and property protected by the dike will continue to be subjected to an unacceptable risk of dike failure and the catastrophic consequences of such a failure. Such an event, with subsequent flooding, would result in extreme socio-economic damages; however, of paramount importance is the real potential for significant human suffering, including loss of life. In light of these considerations, the base condition alternative is unacceptable.

Table 6. Hazard Potential Classification for Civil Works Projects.

CATEGORY[1]	LOW	SIGNIFICANT	HIGH
Direct Loss Of Life[2]	None Expected (due to rural location with no permanent structures for human habitation	Uncertain (rural location with few residences and only transient or industrial development	Certain (one or more extensive residential, commercial, or industrial development
Lifeline Losses[3]	No disruption of services – repairs are cosmetic or rapidly repairable damage	Disruption of essential facilities and access	Disruption of critical facilities and access
Property Losses[4]	Private agricultural lands, equipment and isolated buildings	Major public and private facilities	Extensive public and private facilities
Environ-Mental Losses[5]	Minimal incremental damage	Major mitigation required	Extensive mitigation cost or impossible to Mitigate

Notes:

1. Categories are based upon project performance and do not apply to individual structures within a project.
2. Loss of life potential based upon inundation mapping of area downstream of the project. Analyses of loss of life potential should take into account the extent of development and associated population at risk, time of flood wave travel and warning time.
3. Indirect threats to life caused by the interruption of lifeline services due to project failure, or operation, i.e., direct loss of (or access to) critical medical facilities or loss of water or power supply, communications, power supply, etc.
4. Direct economic impact of value of property damages to project facilities and downstream property and indirect economic impact due to loss of project services, i.e., impact on navigation industry of loss of a dam and navigation pool, or impact upon a community of the loss of water or power supply.
5. Environmental impact downstream caused by the incremental flood wave produced by the project failure, beyond which would normally be expected for the magnitude flood event under a with-project condition.

Table 6a. Rehabilitation Alternatives

Alternative	Description	Warrants Further Consideration	Comment
a	Base condition	Not Applicable	Existing conditions; not a rehabilitation alternative
b	Modification of lake regulation schedule	No	Unacceptable level of protection
c	Increased outlet capacity	No	Unacceptable cost and environmental consequences
d	Relief wells	No	Unacceptable level of protection
e	Sub-levees	Yes	A component of the final tailwater control Alternative A, suitable for some reaches
f	Landside toe ditch water level control	Yes	Final Alternative A; suitable for use in Reaches 4,5,6,7 and 8
g	Landside drainage berm	Yes	Final Alternative B; suitable for use in Reaches 1,2 and 3
h	Impervious cutoff wall	Yes	Final Alternative C; suitable for use in some reaches; however, characterized by excessive cost and unknown groundwater impacts

b. Modification of Lake Okeechobee Regulation Schedule.

A non-structural solution alternative, which would ensure that high lake levels could not occur, was considered. This would involve modification of the lake regulation schedule; however, the ability to lower the lake stage is limited by the capacity of available outlet facilities. During extreme rainfall events, the amount of water entering Lake Okeechobee is much greater than the amount of water we can discharge from the lake. Because of this, the lake will rise during these events, even though CESAJ will be discharging water from the lake as rapidly as possible. Reasonable modifications of the regulation schedule do reduce the peak flood elevations on Lake Okeechobee; however, the extent of reduction is not an adequate solution to the seepage and stability problems. For example, under the current regulation schedule (Run 25), the peak 100-year flood elevation on Lake Okeechobee is 21.3 feet. If a recently proposed lower regulation schedule (Run 22D) were put into effect, the 100-year flood elevation would be reduced to 20.5 feet. This is a beneficial change, which as the numbers indicate, reduces the lake stage and corresponding likelihood of failure associated with a 100-yr flood on Lake Okeechobee. But the

extent of the reduction in stage and failure likelihood is not adequate. An unacceptable risk of dike failure would still exist at the lowered stage of 20.5 feet. A regulation schedule that resulted in extremely low lake levels, or draining the lake entirely, would have catastrophic socio-economic and environmental consequences. Lake Okeechobee waters are used in many ways, and they provide south Florida with a wide variety of benefits related to municipal water supply, agricultural water supply, recreation, navigation, the prevention of saltwater intrusion, and the protection and enhancement of environmental resources. Lake levels must be maintained within reasonable levels; therefore, modification of the regulation schedule is not a stand-alone solution to the seepage and stability problems at Herbert Hoover Dike.

c. Increased Outlet Capacity.

Increased capabilities related to the discharge of waters from Lake Okeechobee would allow the Corps of Engineers to maintain lower lake stages. That would require either (1) enlargement of existing outlet channels (primarily the St. Lucie Canal and Caloosahatchee River), or (2) construction of a new outlet channel from the lake to either the Atlantic Ocean or Gulf of Mexico. The costs of either option would be greater than the costs associated with the proposed seepage berm. Also, any measures that would increase the outlet capacity would have severe environmental consequences, both along the length of the outlet channel, and at the channel outlet estuaries.

d. Relief Wells.

Relief wells would be constructed along the landside toe of the levee to prevent excessive uplift pressures and piping through the foundation. Each well would consist primarily of a boring, a riser pipe to conduct water to the surface, and a drainage ditch to convey the seepage flows above ground. This alternative may be suitable for use in Reaches where conditions are less severe; however, in Reach 1, the level of protection provided by a relief well solution was determined to be inadequate.

e. Sub-Levees.

Sub-levees could be constructed to control seepage by storing water on the landside of the levee. This area of

storage provides a higher tailwater elevation and, thereby reduces the hydraulic gradient across the levee. A sub-levee system would consist of an area designed to store water near the landside base of the levee. This storage would be accomplished by constructing an embankment around the area's perimeter. In some locations along Herbert Hoover Dike, large landside borrow canals parallel the toe of the levee. Sub-levees may be viable rehabilitation alternatives in these locations; however, for Reach 1, a sub-levee solution would not provide an adequate level of protection. Also, such a solution would be more costly than more traditional solution measures (e.g., berm construction, relief wells, cutoff walls, etc.).

f. Landside Toe Ditch Water Level Control.

This alternative would address seepage and stability problems by reducing the magnitude of the hydraulic conditions to which the dike is subjected. This reduction in hydraulic loads would be accomplished, not by reducing the lake elevation (or headwater), but by increasing the water level on the landward side of the dike (or tailwater), thereby reducing the hydraulic gradient (or head) across the dike. This alternative would involve toe ditch clearing and regrading, and installation of gated culverts and pumps to control the toe ditch water levels. Additionally, a 3- to 4-foot thick peat layer would be excavated from the landside toe of the levee. In its place, a 25-foot wide by 5-foot thick stability berm would be constructed. The stability berm would reduce uplift forces at the toe, provide increased resistance to seepage flows, and allow better inspection access at the dike toe. This alternative was investigated in significant detail during the economic analysis; however, in Reach 1, the estimated level of protection provided by this alternative was not adequate. In addition, this alternative increases the local flooding potential in areas immediately adjacent to the dike; and presently, local agricultural interests control most of the Herbert Hoover Dike toe ditches. The estimated cost of this alternative is $1,049,300 per mile.

g. Landside Drainage Berm.

This alternative would involve excavation of a 4-foot thick peat layer from the landside levee toe, and construction of a drainage berm. The drainage berm would consist of a 5-foot thick layer of filter sand and gravel. The berm would

contain a 48-inch diameter perforated culvert wrapped in geotextile fabric. A 2-foot wide, 25-foot deep, gravel-filled relief trench would be constructed under the drainage berm. The drainage berm would prevent the piping of sands and silty material from the embankment or foundation. The relief trench would control uplift pressures and prevent heaving at the levee toe. The entire drainage system will reduce artesian pressures in the foundation and will collect and convey seepage flows to a controlled outlet. Gated culverts would be installed at various locations to drain the 48-inch diameter culvert. Based on economic, environmental, and engineering considerations, this alternative was selected for implementation along the 22.35 miles of dike in Reach 1. Estimated cost of this alternative is $3,024,800 per mile.

h. Impervious Cutoff Wall.

An impervious cutoff wall consists of a physical barrier that prevents seepage by blocking the seepage path beneath and/or through the levee. It is typically the most effective method of seepage control as it reduces both uplift pressures and through seepage. The wall would be constructed on the lakeside slope of the dike. It would consist of a 3-foot wide, 60-foot deep excavated trench filled with a soil-cement mixture. The top of the wall would be located at an approximate elevation of 25 feet, NGVD. Although this alternative is very effective, it is expensive, with an estimated cost of $13,847,200 per mile. Also, this alternative will affect the upper aquifer and could have a detrimental impact on groundwater flows immediately adjacent to the dike.

13. ENVIRONMENTAL CONSIDERATIONS.

The following paragraphs provide a brief description of the project area’s environmental characteristics and effects of the proposed rehabilitation. Coordination related to environmental issues is also summarized. More detailed information is provided in Appendix C (Environmental Documentation).

Environmental Effects – Lake Okeechobee is a major hydrologic feature of south Florida and the Everglades ecosystem; therefore, its waters play a critical role in the protection and enhancement of environmental resources. Fish and wildlife species are numerous and utilize the many natural areas around the lake. Sixteen faunal species known to occur in the vicinity of the lake are currently listed as threatened or endangered by the U.S. Fish and Wildlife Service. Twenty-nine faunal species known to occur in the region are currently listed as threatened, endangered, or as species of special concern by the Florida Game and Freshwater Fish Commission.

Implementation of the proposed rehabilitation alternative would cause minimal short-term disturbance to, and displacement of, components of the human and natural environments. These impacts include minimal soil, vegetation, and wetland disruption during excavation and fill activities. Minimal effects to existing water resources and foraging habitat for wading birds and listed species are expected as well. A detailed discussion of the affected environment, with specific references to physical, biological, social, and economic resources, is provided in Appendix C.

Coordination and Correspondence – A Notice of Intent to prepare a Draft Environmental Impact Statement for this Major Rehabilitation Evaluation Report was published in the Federal Register Volume 61, No. 190 on September 30, 1996. A Scoping Letter describing the proposed project and soliciting comments was sent to government agencies, non-government agencies, Indian Tribes and interested public on December 6, 1994. A list of all recipients is provided in Appendix C. Written responses to the Scoping Letter were submitted to the Corps of Engineers and served to assist in identifying potential environmental and planning issues throughout the study. Those responses included concerns regarding impacts to scenic trails, wetlands, threatened and endangered species, navigation, erosion control, and Lake Okeechobee water levels. Each of these concerns was addressed during preparation of the Draft Environmental Impact Statement.

Reports and Studies – The Herbert Hoover Dike Major Rehabilitation Report, Fish and Wildlife Planning Assistance Report was prepared in April 1998. The Draft Environmental Impact Statement (Appendix C) was completed

in December 1998. It includes detailed information related to studies conducted to evaluate the effects of the proposed rehabilitation plan. The subjects of those studies include topography, geology, soils, hydrology, water supply, water quality, water management, vegetation, wetlands, fish and wildlife, threatened and endangered species, noise, air quality, hazardous, toxic and radioactive wastes, land use, aesthetic resources, recreational resources, and cultural resources.

NEPA Documentation – Environmental information related to this project has been compiled and the Draft Environmental Impact Statement, Herbert Hoover Dike Major Rehabilitation Evaluation Report, dated December 1998, has been prepared. A systematic interdisciplinary approach to planning has been utilized. Alternatives have been studied, developed, and described, and ecological information has been developed and utilized. This Draft Environmental Impact Statement was circulated prior to finalization in accordance with the National Environmental Policy Act of 1969, as amended.

Preparers – The individuals primarily responsible for contributing to the preparation, review, and technical editing of this Draft Environmental Impact Statement are listed in Table 7.

TABLE 7
DRAFT ENVIRONMENTAL IMPACT STATEMENT
LIST OF PREPARERS

Name	Affiliation	Discipline/Expertise	Role in Preparing Document
Mark T. Ziminske	US Army Corps of Engineers	Ecologist	Project Management; Technical Review Environmental Compliance
David L. McCullough	US Army Corps of Engineers	Archaeologist	DEIS Preparation; Cultural Resources
Paul C. Stevenson	US Army Corps of Engineers	Residential Landscape Architect	DEIS Preparation; Aesthetics/Recreation
Peter H. Besrutschko	US Army Corps of Engineers	Environmental Engineer	DEIS Preparation; HTRW Evaluation
Renee Thomas	Lotspeich & Associates Inc.	Contract Program Manager	Technical Review, QA/QC
David B. Culver	Golder Associates Inc.	Environmental Scientist	DEIS Primary Author; Study Manager
Denise Froese	Golder Associates Inc.	Economist	DEIS Preparation; Socioeconomics
Rosemary G. Mora	Golder Associates Inc.	Aquaculture & Fisheries Management	DEIS Preparation; Introduction, Alternatives
Kelly Murray	Golder Associates Inc.	Hydrogeologist	DEIS Preparation; Water Resources
Richard Zwolak	Golder Associates Inc.	Environmental Planner	Senior Technical Review

14. ASSESSMENT OF ALTERNATIVES.

The economic analysis of various rehabilitation strategies was accomplished through the execution of a risk-based Economic Life Cycle Analysis Model. The model provides a structured framework for developing and storing the required input data, performing the needed calculations, evaluating results, and comparing the base condition with rehabilitation alternatives from a benefit/cost perspective. The model simulates the possible breaching of the Herbert Hoover Dike system in response to changing water levels over a life cycle. Damages are accrued based on destruction of dike components and flooding behind the dike. Rehabilitation of the dike reduces the probability of dike failure and associated damages.

Critical Assumptions and Key Variables – As stated previously, Herbert Hoover Dike is an earthen embankment surrounding Lake Okeechobee. The lake's location lies in the potential path of tropical cyclones, which may originate in either the Gulf of Mexico or Atlantic Ocean; therefore, the dike is occasionally subjected to storm surges. When the difference in lakeside and landside water surface elevations is great, the dike is subject to seepage and piping; and the potential for breaching of the dike through seepage- or piping-induced failure of a levee component exists. Such failure would lead to flood inundation of areas protected by the dike, and corresponding damages associated with residential and agricultural land use, dike damages, and affected populations. The following paragraphs provide an overview of probabilistic aspects of the problem and critical assumptions made in the model development. More detailed information is provided in Appendix A (Risk and Reliability Analysis).

The primary probabilistic aspects of the model are (1) variation in lake stage over time, (2) the frequency and type of hurricane activity, (3) the variation in tailwater (i.e., landside water levels) over time, and (4) the probability of breaching along different sections of the embankment. These important sources of uncertainty were identified and described statistically. At each point within the simulation where descriptive data for an uncertain variable is required, the statistical distribution for that variable is used to set a value. This is accomplished through random (i.e., Monte Carlo)

sampling from the distribution. Therefore, the values of these variable parameters will differ from each simulation cycle to the next. Many simulation cycles are run and the resultant overall statistics are used in the decision-making process. This approach combines uncertainties in the many variables that describe a system, to obtain a statistical description of the system's behavior as a whole.

Rehabilitation alternatives are directed at reducing the likelihood of a dike breach. The primary forcing mechanism that may cause a breach is a large hydraulic gradient between the lake side and tailwater side (i.e., the landward side) of the levee. Lakeside elevation is established as a combination of static lake stage, which varies slowly based on Lake Okeechobee inflows and outflows, and hurricane storm surge, which varies rapidly in response to the passing of a hurricane. The susceptibility to breaching is a function of the levee's physical condition, which varies along its length.

To capture the geotechnical variation of the dike system, the dike was divided into a number of reaches, and reaches were further divided into components, which are contiguous sections of the dike. The component is the part of the system that is susceptible to breaching; therefore, failure and repair are simulated at the component level. The performance of a dike component (breaching or not breaching) is modeled stochastically through the use of Probability of Unsatisfactory Performance (PUP) functions. The PUPs estimate the probability of a given component breaching as a function of the lake stage at that component. The impact of rehabilitation alternatives is reflected primarily through modification of the PUP functions. In other words, if a component is rehabilitated, the with-rehabilitation PUP function will define a lower probability of failure for a given lake stage, when compared to the base condition PUP function.

If a component breaches, it can fail in one of two modes. The modes of failure differ depending on the magnitude of the hydraulic gradient that causes failure, and the corresponding flow velocities of the floodwaters through the breach. A relatively small hydraulic gradient will result in a *low-velocity breach*. Flood flow velocities at the breach would be less than 10 feet per second (fps) and conditions would allow breach closure in approximately ten

days. A large hydraulic gradient would result in a *high-velocity breach*. Expected flood flows would be greater than 10 fps and closure efforts would be more difficult. It was estimated that approximately 45 days would be required to close a high-velocity breach.

Areas outside the dike system, which could be inundated by floodwaters, are referred to as *damage cells*. A breach of a component can result in the inundation of one or more damage cells. Monetary and non-monetary impacts are directly related to the inundation of these damage cells. Economic damages are a function of the land use within a damage cell, lake stage when a breach occurs, mode of failure (high or low velocity), and the time passed since a previous inundation. The model stores user-entered functions that determine the impacts of a breach as a function of lake stage. The problem is further complicated because a simple one-to-one relationship between a component breach and inundated damage cells does not exist. The same damage cell can be flooded by a breach in one of several different components. Also, simultaneous breaching of individual components is possible, with overlap of affected damage cells. In such a case, the impacts of each individual component breach are identified separately; then the worst case damages are counted and the others are disregarded. This prevents the model from overestimating damages incurred in a single damage cell, which result from multiple breaches.

Hurricane season extends from June 1 to November 30 of each year. The historical record indicates that the tracks of tropical cyclones affecting Lake Okeechobee can be divided into two basic categories. Storms from the southeast (i.e., from the Atlantic Ocean) typically produce the greatest surges along the southeast quadrant of the lakeshore. Storms from the southwest (i.e., from the Gulf of Mexico) typically produce the greatest surges along the northwest lakeshore quadrant. Storm surge frequency relationships were developed for each dike component; and these were used to stochastically identify storm surge heights throughout the simulation procedure.

A period-based (as opposed to event-based) modeling approach was selected, based primarily on the continuous nature of the available lake stage data. A four-season model was used to capture variations in the probability of

hurricane occurrence, and variation in the characteristics of croplands during different times of the year.

Provisions were also made for the reduction in lake stage (water lost through the breach) that would follow the occurrence of a breach. Although the effect on the Lake Okeechobee stage would be minor, it was included in an effort to incorporate the greatest degree of realism in the simulation procedure.

Economic impacts of a dike failure include the cost of breach closure, the cost of dike repair to its pre-breach condition, increased operation and maintenance costs, and damages due to flood inundation. Provisions were also made for the consideration of population affected by a breach. The effects of flood inundation were quantified by use of functions that relate flood damages to static lake stage.

The Economic Life Cycle Analysis Model was typically run for a 50-year project life span, with each year divided into four periods. Including the period for land acquisition and construction, a 223 period 50-year life cycle was constructed for each simulation. This process was typically repeated for 5,000 iterations to generate an adequate population of resulting data, from which statistical parameters could be calculated. Those parameters served as the basis of comparison, for the evaluation and rating of individual rehabilitation alternatives.

15. RECOMMENDED PLAN.

The recommended rehabilitation plan for Reach 1 is construction of a seepage/drainage berm along 22.35 miles of the southeast shore embankment between S-308 at Port Mayaca and S-351 at Belle Glade. A typical cross section is provided in Figure 45. The estimated total cost of the proposed Reach 1 rehabilitation is $82,342,200. Economic Analyses indicated that the Benefit/Cost ratio would be 0.945 to 1 without flood insurance benefits and 1.07 to 1 with flood insurance benefits added. The estimated total construction time associated with this Reach 1 rehabilitation is 4.5 years. A summary of the major benefit and cost items for this plan, and a comprehensive rehabilitation plan (described below), is provided in Table 8. The Reach 1 rehabilitation involves

Table 8

Comparison of Recommended Rehabilitation Alternatives and Base Condition
(dollars in present value)

	REHABILITATION ALTERNATIVE		
BENEFIT/COST ITEM	Base Condition	Rehab Reach 1	Comprehensive Rehab
Damage Cost (mean)	154,816,129	87,676,796	47,765,570
Repair Cost (mean)	32,781,114	22,097,601	11,705,240
O&M Cost (mean)	50,520,718	50,548,519	50,568,566
Investment Cost (mean)		82,342,197	158,549,783
Benefits (mean)		77,795,044	128,078,584
Net Benefits (mean)		-4,547,153	-30,471,199
B/C Ratio (mean)		0.945	0.808
B/C Ratio with flood insurance administration cost savings in Reach 1		1.070	0.875

excavation of a 4-foot thick layer of peat material from the landside toe of the existing dike. This excavated area would be backfilled with a 5-foot thick layer of graded, granular fill, to act as a filter and drainage blanket. A 2-foot thick drainage blanket would extend further up the dike slope to an elevation of +23.0 feet. Its function would be to intercept through seepage. In order to reduce uplift pressures and intercept underseepage, a 2-foot wide relief trench would be excavated along the bottom of the existing toe ditch. This trench would be filled with a graded, granular, filter stone. The relief trench would extend down vertically to the top of the lower subsurface limestone layer, a depth of about 25 feet. A 48-inch diameter perforated culvert, wrapped in geotextile fabric, would be installed at the same elevation as the existing toe ditch, and covered with graded, granular, filter stone. The dike-parallel culvert would collect and convey excessive seepage flows. Drop inlets would be constructed on 400-foot centers along the top of the berm/culvert system to drain surface runoff into the culvert. At each location where the drainage system meets an existing drainage canal, a mechanism for discharging flows from the 48-inch diameter pipe into the canal is required. This will be accomplished by construction of a drainage pool and stoplog riser at each outlet point. The drainage pool will collect and hold flows emerging from the seepage drain system. The pool will be lined with a gabion mattress to prevent erosion of, and seepage through, its banks. From the drainage pool, flows will be introduced into the adjacent canal or drainage ditch by means of a stoplog riser. This system will provide control over the water level within the seepage drain.

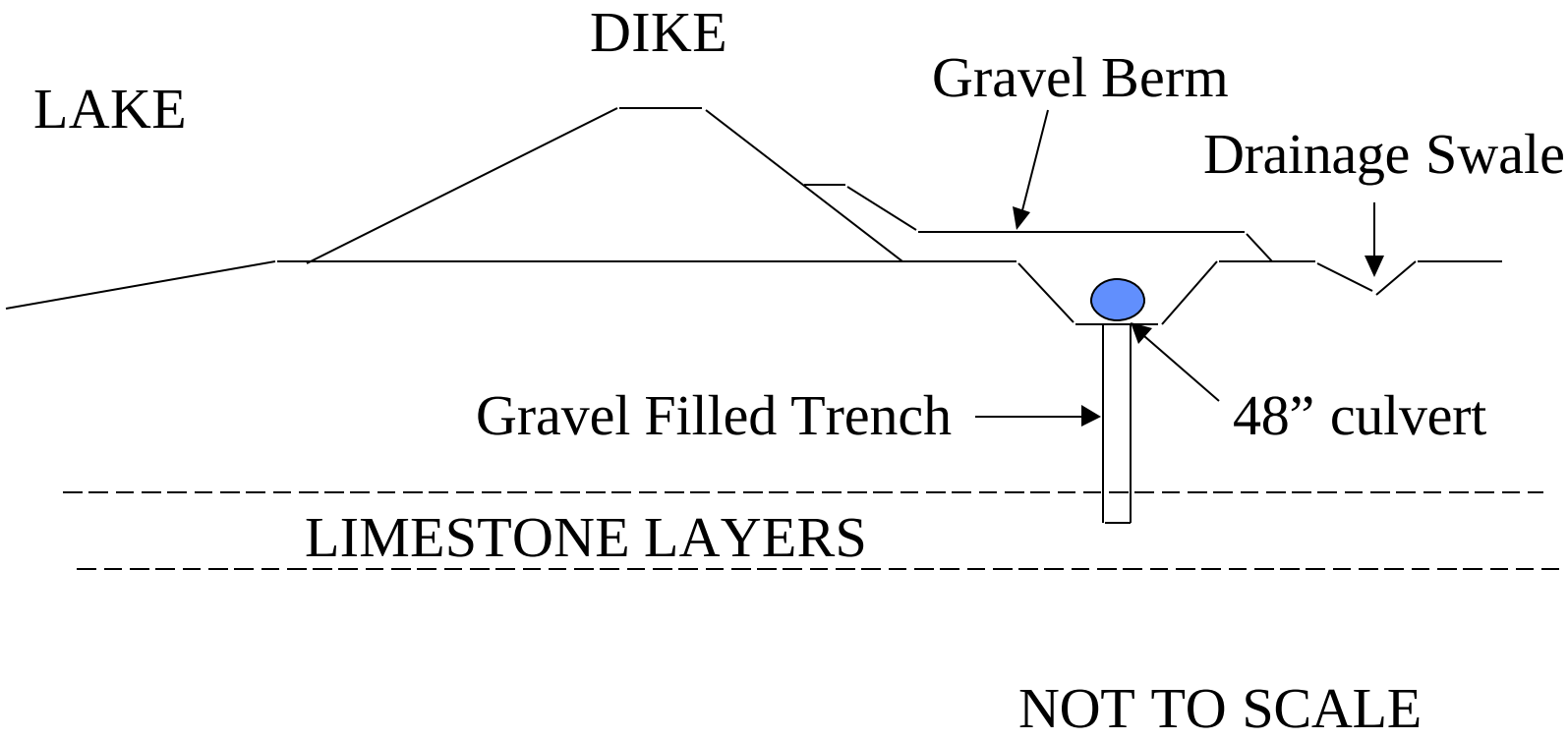
Another objective of this analysis was the preparation of a preliminary estimate of the rehabilitation efforts that would be required to remedy seepage and stability problems along the entire length of Herbert Hoover Dike. That comprehensive rehabilitation plan would involve construction of a seepage/drainage berm (Alternative A) along a 36.5-mile length of embankment in Reaches 1, 2, and 3. The plan would also implement the tailwater control measures (Alternative A) in Reaches 5 and 7, and portions of Reaches 4, 6, and 8. The total length of embankment along which Alternative A is proposed is 54.5 miles; therefore, the comprehensive rehabilitation plan involves some type of rehabilitation effort along 91 miles of the 142-mile long dike system. The estimated total cost of

these proposed improvements including land acquisition tabulated for Reach 1 is $173,210,620. Economic analyses indicated that corresponding present value of the investment cost would be $158,549,800 and benefits would total $128,078,584, resulting in a Benefit/Cost ratio of 0.808 to 1 without flood insurance benefits tabulated for Reach 1. Including these benefits would result in a Benefit/Cost ratio of 0.875 to 1. The estimated total construction time associated with this comprehensive rehabilitation is twelve years. Although the benefit to cost ratio for the comprehensive rehabilitation plan was less than 1, the preliminary analysis was not at the level of detail used in Reach 1. Engineering calculations have not been refined to the extent of Reach 1 and additional benefits have not been computed for infrastructure, transportation delays, flood insurance administrative cost savings and roads. Further detailed investigation for other reaches is warranted.

16. MAJOR REHABILITATION CLASSIFICATION.

The proposed rehabilitation of Herbert Hoover Dike meets the requirements applicable to the *Reliability* rehabilitation category, as specified in Engineering Pamphlet EP 1130-2-500, Project Operations, Partners and Support (Work Management Guidance and Procedures). The work is non-recurring in nature and is intended to restore an acceptable level of flood protection along the southeast shore of Lake Okeechobee. The proposed rehabilitation consists of structural improvement of a facility constructed, operated, and maintained by the U.S. Army Corps of Engineers. The improvements would increase the reliability of the dike, and reduce the likelihood of an embankment failure and associated catastrophic socio-economic impacts. The proposed plan of rehabilitation is the most viable alternative, as it will significantly extend the physical life of the dike system and is economically justified as documented by the benefit/cost analysis.

Figure 45


DIKE
LAKE
Gravel Berm
Drainage Swale
Gravel Filled Trench
48” culvert
LIMESTONE LAYERS
NOT TO SCALE

17. PROJECT COST ESTIMATE.

A schedule of fully funded project costs is provided in Appendix G. M-CACES cost estimates for rehabilitation alternatives and the recommended plan of rehabilitation in Reach 1 is provided in Appendix D (M-CACES Cost Estimate).

18. COST SHARING CONSIDERATIONS.

The latest guidance from HQUASCE, 6th Endorsement of the Special Report dated 09 August 1994, states as follows: "After approval of the evaluation report by HQUSACE, required major rehabilitation work should be budgeted under the CG appropriation for construction as a fully federally funded project." Therefore, a Local Cooperation Agreement is not applicable to the subject project. CESAJ will continue to work cooperatively with the local sponsor to inform them of the project status. Operation and maintenance of the embankment system will follow the guidance contained in the existing agreement between the U.S. Army Corps of Engineers and the local sponsor, South Florida Water Management District.

19. SUMMARY AND CONCLUSIONS.

This report documents a comprehensive effort to identify seepage and stability problems, define their locations, and quantify their severity. Alternative solutions are proposed and a recommended solution is identified, based on engineering and socio-economic decisions. The report addresses consequences of a dike failure, which include population impacts and loss of life, as well as economic and environmental damages. A probabilistic risk and uncertainty model was developed to complement the more traditional analysis methods, and provide an additional decision-making tool.

Recommended Plan - The recommended plan for Major Rehabilitation of Reach 1 involves the construction of a seepage berm, with relief trench and drainage system, along the landside toe of the dike. The 5-foot thick berm will consist of filter sand and gravel, and will contain a perforated culvert for the collection and conveyance of seepage flows. The berm will prevent the piping of material from the embankment and foundation. The relief

trench will control uplift pressures and prevent heaving at the landward toe of the embankment; and the vertical relief drainage system will reduce artesian pressures in the foundation and will intercept and transport seepage which would otherwise emerge uncontrolled landward of the embankment.

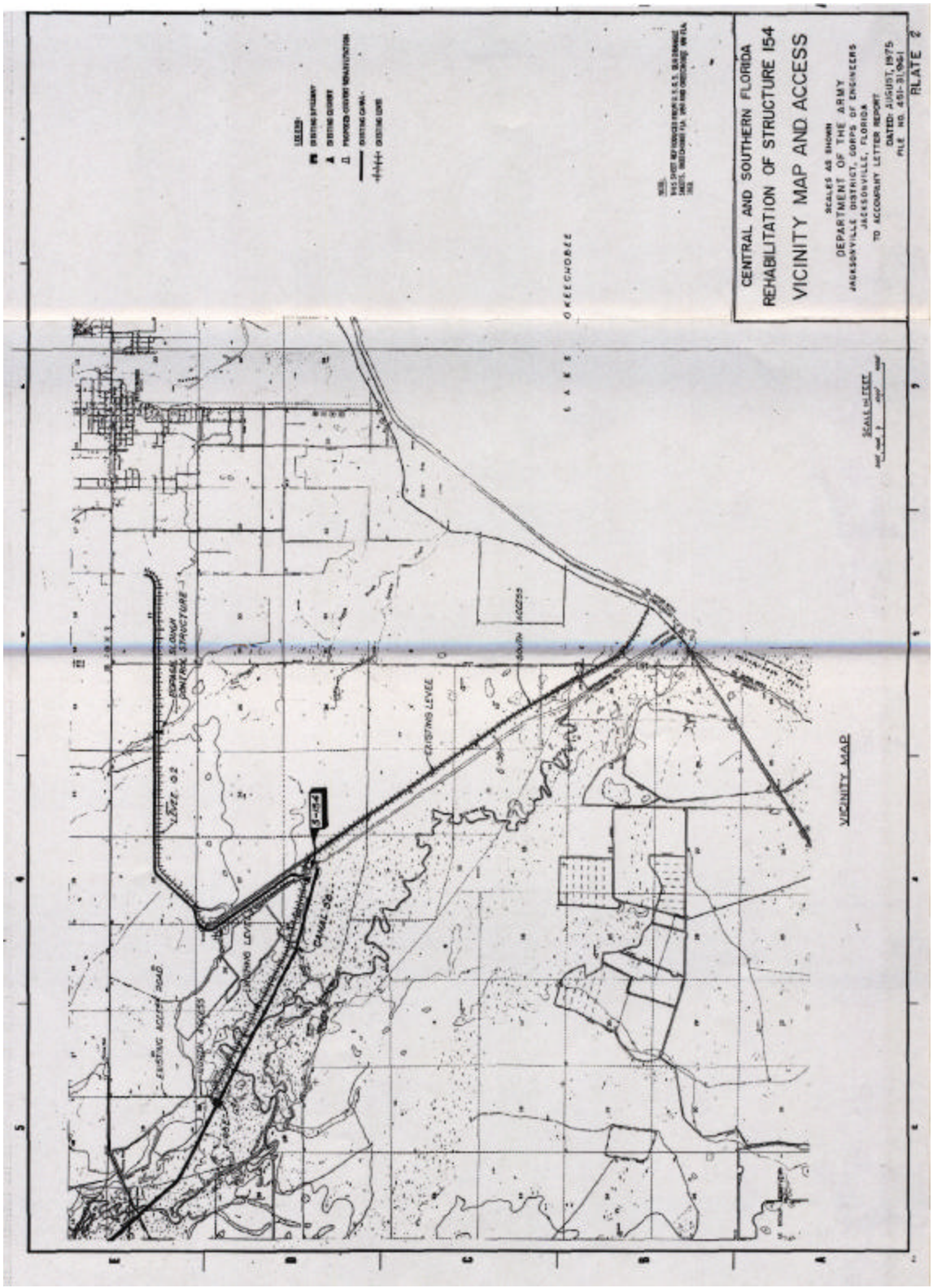
Costs and Benefits – The estimated first cost for rehabilitation of the 22.35-mile length of Reach 1 is $73,805,605. Interest during construction is estimated to be $8,536,592 resulting in a total project investment of $82,342,197. The estimated benefit to cost ratio is 1.07 to 1.

Conclusions – This report documents the existence of seepage, piping, and erosion problems along the Herbert Hoover Dike system. Efforts to quantify the severity of those problems have indicated that there is a very serious risk of catastrophic dike failure due to piping. Such an event, with subsequent flooding, would result in extreme socio-economic and environmental damages; however, of paramount importance is the real potential for significant human suffering, including loss of life. In light of these considerations, it is imperative that efforts to rehabilitate the Herbert Hoover Dike system be initiated immediately; therefore, it is recommended that this report be approved and efforts to initiate rehabilitation of Reach 1 be undertaken in the most efficient manner possible. It is also recommended that efforts to prepare the second Major Rehabilitation Evaluation Report, addressing Reaches 2, 3, and 7, be initiated as soon as possible. This will ensure that expedient measures are taken to restore acceptable levels of flood protection in those areas as well.

Plate 1.

LEGEND
COMPLETED OR UNDER CONSTRUCTION
REMAINDER OF PROJECT
CENTRAL AND SOUTHERN FLORIDA
(FLOOD CONTROL AND OTHER PURPOSES)
1968 AUTHORIZATION
54-8

Plate 2. Vicinity Map



1. INTRODUCTION.

This appendix was prepared to document the preparation and usage of the Herbert Hoover Dike Life Cycle Analysis (LCA) Model. The model was developed by the Institute for Water Resources (IWR) of the U.S. Army Corps of Engineers, at the request of, and in close cooperation with, the U.S. Army Engineer District, Jacksonville (CESAJ). Computer programming tasks during model development were conducted by Planning and Management Consultants, Ltd., of Carbondale, Illinois, and RMM Technical Services, Inc., Cincinnati, Ohio.

The model accomplishes a risk-based (Monte Carlo) simulation of potential dike breaching, associated economic damages, and population affected. The model considers the effects of Lake Okeechobee stage and hurricane-induced storm surge as the forcing parameters that may cause a breach. A quarterly (i.e., 3-month) time scale is used to capture variations in hurricane probabilities and the state of agricultural resources during different times of the year. The model is equipped to consider multiple land uses, various flood severity, monetary and non-monetary damages, the consequences of repeated flooding in relatively short time periods, simultaneous breaches at more than one location, and the cost of repair and rehabilitation efforts. It is a tool designed to identify and compare the relative costs and benefits associated with the base (without-project) condition and various rehabilitation (with-project) conditions.

2. BACKGROUND INFORMATION.

Herbert Hoover Dike is an earthen embankment surrounding Lake Okeechobee in south Florida. The lake lies in the potential paths of Gulf and Atlantic hurricanes; therefore, during hurricane season, the lake waters may be influenced by hurricane-induced storm surge. The dike is subject to seepage and slope stability problems under conditions characterized by a large hydraulic gradient (i.e., when the difference in water levels on each side of the dike is great). A fault tree depicting the two failure mechanisms (piping and slope stability) is provided in Figure A-1. A failure, or breach, of a dike component would lead to flood inundation of areas protected by the dike, and corresponding socio-economic damages.

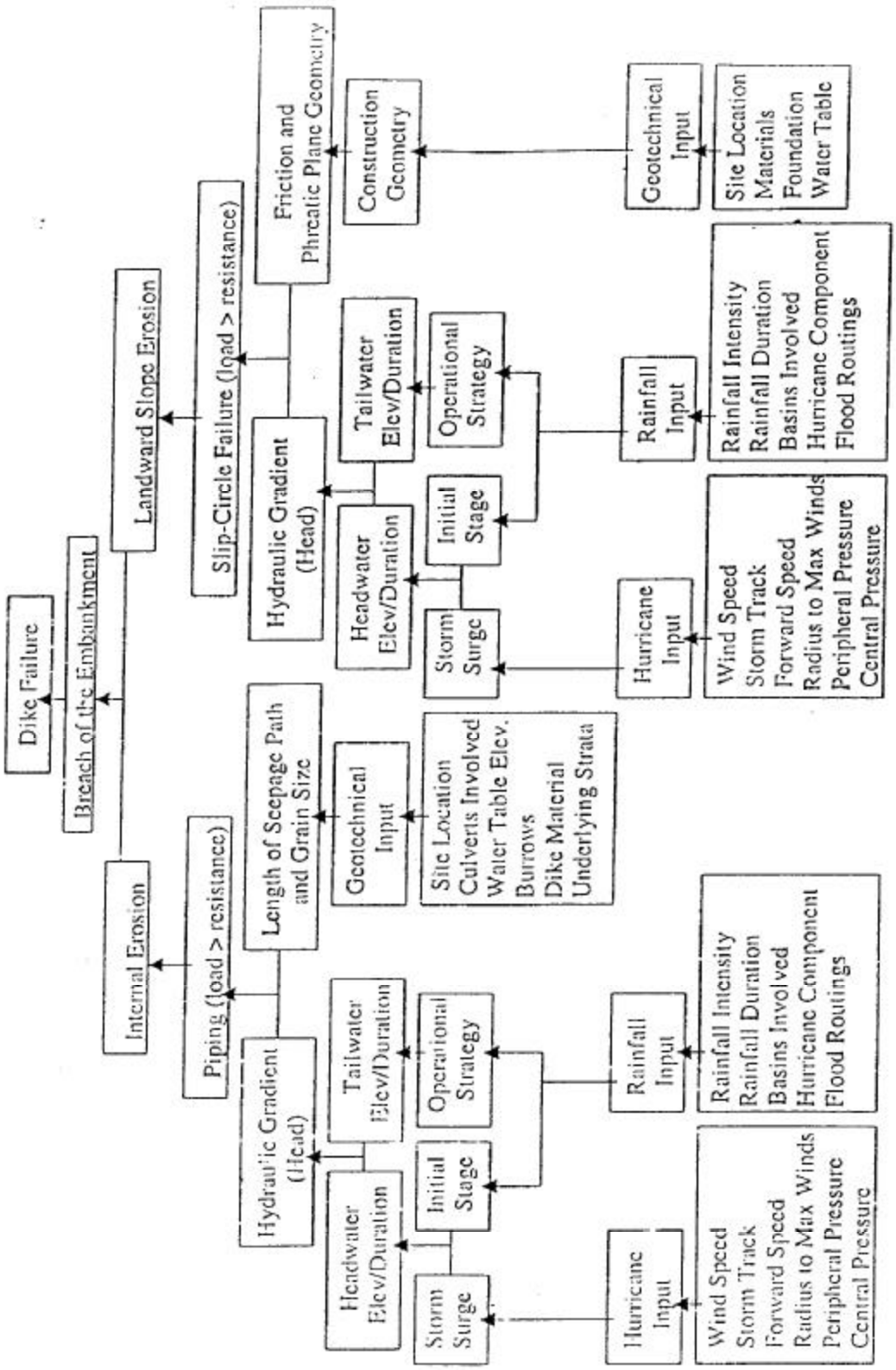


Figure A-1. Herbert Hoover Dike fault tree.

The LCA model considers the uncertainty and variability in a number of key parameters, which influence the performance of the Herbert Hoover Dike system. Those probabilistic parameters include (1) the variation in lake stage over time, (2) the frequency of occurrence and characteristics of tropical cyclones in the area, (3) the variation in tailwater stage over time (i.e., water levels on the landward side of the dike), and (4) the variation in failure probability associated with different sections of the dike and different hydraulic loadings. Instead of assigning discrete values to these parameters, they are described by probability distributions which reflect the range of possible values which may apply, and the frequency of occurrence (or likelihood) of those values. This is the basis of the risk-based economic analysis, which must be conducted during investigations of rehabilitation project justification. The LCA model uses Monte Carlo simulation techniques, which combine the uncertainties that describe a system, to obtain a statistical description of the entire system's behavior. This accomplished through repeated runs of the LCA model, and variation of the input data based on the statistical descriptions of the uncertain parameters. Thus, the first task involves identifying, and describing statistically, the important sources of uncertainty for a given problem. This data is then used within the simulation model developed for the problem. At each point within the simulation at which descriptive data for uncertain variables is required, the statistical distribution is used to set a value (through random sampling from the distribution). As a result, the values assigned to a particular variable will differ from simulation run to simulation run. Many simulation runs are then executed, and the resulting overall statistics associated with the problem are used in the process of identifying the best solution.

3. TECHNICAL OVERVIEW.

Rehabilitation alternatives are directed at reducing the likelihood of a levee failure. The forcing mechanisms, which would cause a failure, are related to the difference in water levels on each side of the dike (i.e., the hydraulic gradient, or head). Lakeside elevation is a combination of static lake stage, which varies slowly (e.g., weeks) based on Lake Okeechobee inflows and outflows, and hurricane-induced storm surge, which varies rapidly (hours) in response to the passing of a tropical

cyclone. The susceptibility to breaching is a function of the physical characteristics of the levee and its foundation, which vary along its length.

For purposes of modeling the geotechnical variation along the dike system, it was divided into thirteen components. A component is a section of the dike system that is susceptible to breaching; therefore, dike failure and repair are simulated at the component level. Breaching behavior is modeled in a stochastic fashion, through the use of Probability of Unsatisfactory Performance (PUP) functions. PUP functions define, for each dike component, the probability of failure and its relationship to lake stage. Lake stage is established based on the static lake stage, and if a hurricane is occurring, the storm surge. The impact of rehabilitation alternatives is reflected primarily through modification of the PUP functions. In other words, if a component is rehabilitated, the PUP function for that component will define a lower probability of failure for a given lake stage.

If a dike failure occurs, it is defined as either a high-velocity or low-velocity breach. The difference in breach modes is related to the expected floodwater flow velocity that would exist at the breach site. This flow velocity is directly related to the magnitude of the hydraulic gradient that exists at the time of breaching. A high gradient (i.e., large difference in headwater and tailwater elevations) would result in more rapid flow rates (>10 feet per second (fps)), which would hinder efforts to close the breach. A low gradient would result in less severe floodwater flow velocities (<10 fps), and the breach could be closed with less difficulty. As a result, there is a difference in the time, equipment, materials, and manpower required for closure of the two breach types. Likewise, there is a difference in the severity of flooding that would occur as a result of the two breach types.

A breach of a dike component ca result in the inundation of one or more areas outside the levee. These individual flooded areas are referred to as damage cells (see Figure A-2). Monetary and non-monetary impacts are associated with flooding of these damage cells. The economic damages associated with a damage cell are related to the following factors: (1) land use within the damage cell, (2) lake stage when a breach occurs, (3) type of breach (high- or low-velocity, and (4) time passed since any previous

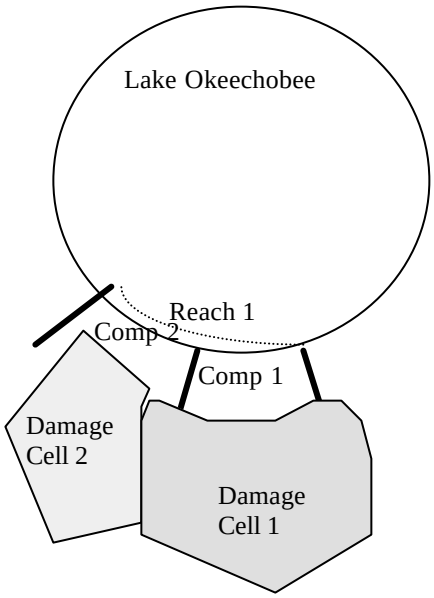


Figure A-2. Physical structure of the solution approach.

inundation. Item (4) was addressed to account for the persistence of some agricultural crop damages (e.g., fruit trees), which in some cases may extend over a period of ten years following a flood. The problem is further complicated by the fact that there is no simple one-to-one relationship between a component breach and the damage cell(s) that are inundated. The same damage cell can be flooded from breaches in more that one component; and depending on the flood severity, a breach of one component may impact more than one damage cell. The model stores a large array of user-input damage functions that determine, based on lake stage, the impacts associated with the many different breach scenarios that might occur.

The LCA model employs a period-based modeling approach. This type of approach was selected primarily because of the nature of the continuous hydrologic Lake Okeechobee stage data. Such data does not correspond well to an event-based modeling approach. Four periods (seasons or quarters) were used to capture variations in hurricane probabilities and crop conditions during different times of the year. An autoregressive moving average (ARMA) model, developed from historical lake stage data, provided the synthetic static lake stage hydrology required for the simulations. This approach allowed the model to account for the fact that the

lake stage in a given period *is* related, to a degree, by the lake stage in the prior period.

Frequency relationships describing storm surge at each dike component were developed. In the event of a hurricane, the LCA model estimates the storm surge at an index location, and then uses functional relationships between that location and all other locations around the lake, to estimate the height of the storm surge at each dike component.

Provisions were also made in the model to account for reduction in lake stage if a breach occurs, due to the loss of lake waters through the breach. Due to the large volume of Lake Okeechobee, this lake stage reduction effect was minor; however, it was included in the LCA model to achieve more realistic simulations of lake behavior.

Economic impacts of a dike failure include costs to close the breach, costs to repair the damaged dike component, costs to pump floodwaters from inundated areas, Operation and Maintenance (O&M) costs, rehabilitation costs, and damages which occur in the flooded damage cells. In addition, census data for each damage cell is incorporated in the model, to provide an estimate of the populations affected by various flooding scenarios. The impacts of flood inundation are quantified based on an array of functions that relate damages to static lake stage at the time of breaching.

4. MODEL PROCESSING.

The overall processing flow of the model is relatively straightforward. A simplified event tree is provided in Figure A-3, which represents a dike system composed of 3 components. The process can be divided into three major tasks: (1) establish the hydraulic loads to which the dike is subjected, (2) evaluate the dike’s performance in response to those loads, and (3) identify the consequences of the dike’s performance. The modeler must first provide all of the required input data (e.g., frequency distributions, damage functions, cost functions, PUP functions, simulation duration, number of iterations,

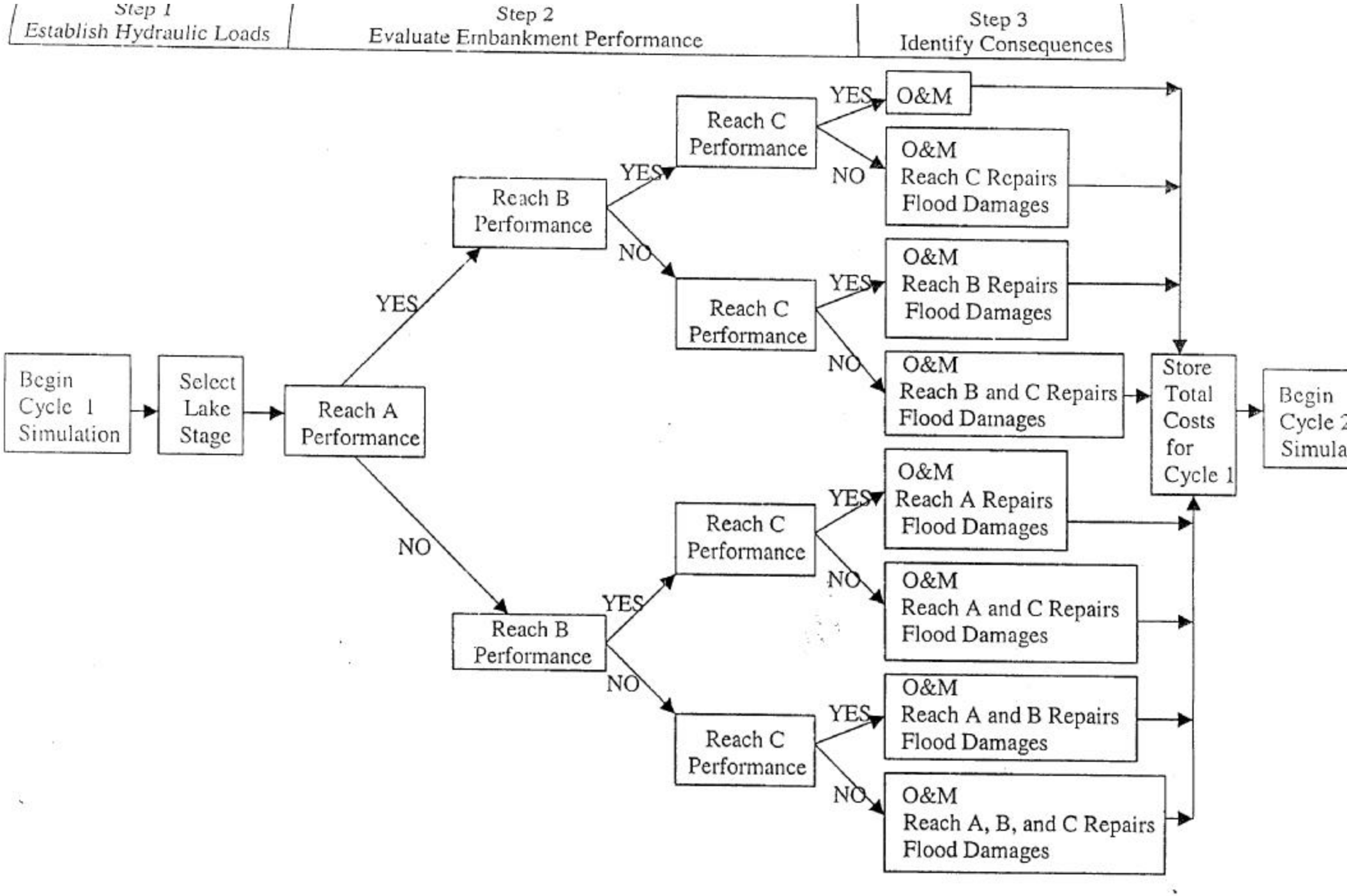


Figure A-3. Simplified event tree.

interest rate, etc.). Then, in each model iteration (i.e., in each period), the following processes are executed:

(a) determine the static lake stage in the current period, based on the ARMA hydrologic model.
(b) determine if a hurricane occurs in the current cycle, based on input probabilities of tropical cyclone activity.
(c) evaluate each component, in turn, for breaching based on the lake stage at that component. Stage is identified as static lake stage, plus storm surge if a hurricane is simulated. When the stage is established, that stage is used to determine the corresponding probability of failure (from the PUP functions) at each component. This probability of failure is compared with a randomly generated number (0 to 1), to determine if a breach takes place. If a breach occurs, the breach mode (high- or low-velocity) must be determined. This is done by first obtaining a random value of the tailwater elevation, based on input triangular tailwater distributions which describe each component. The difference between static lake stage and tailwater elevations defines the hydraulic gradient. This gradient is calculated and compared with an input critical gradient (13 feet). If the calculated gradient is greater than or equal to the critical gradient, a high-velocity breach condition exists; otherwise, a low-velocity breach is simulated.
(d) identify the consequences of dike performance. If no breach occurs, the costs incurred are normal O&M costs, which are defined for each component as a function of static lake stage. If a breach does occur in this cycle, the corresponding monetary and population impacts are identified, based on input stage-damage relationships for each damage cell affected. The impacts are then totaled. If prior inundation conditions exist, then damage reduction factors are applied to the damages, based on damage reduction functions specified for specific land uses.

This process is then repeated for each cycle of the iteration, for as many iterations as the user specified. The model then arranges the results in a variety of tabular and graphical displays, which may be used to evaluate the performance of the existing dike, or rehabilitation

scenario that was modeled. Each *Base Condition* or *Rehabilitation Alternative* modeling effort was typically run to represent a 50-year project life after completion of construction; therefore, depending on construction time, the number of cycles (periods) was between 200 and 250. Each modeling effort was typically run for 5,000 iterations, to ensure that an adequate population of data was generated from which statistical information could be obtained.

5. ANALYSIS FRAMEWORK.

Physical System – The basic physical segments of the dike system and surrounding area are reaches, components, and damage cells. A reach is a contiguous portion of the dike, having certain common characteristics. Each reach consists of one or more components, which are further subdivisions into contiguous portions of the reach. Each reach must have at least one associated component. Components within a reach have certain uniform characteristics while other characteristics will differ. For example, the O&M cost versus lake stage function is common among all components within a given reach; however, differences in physical characteristics between individual components in a given reach are reflected by items such as repair costs and PUP functions. The component is the smallest unit of the dike to which data is assigned. A damage cell is an area of land lying outside (landward) of the dike. Damages cells are areas that are susceptible to flooding damages in the event of a dike failure. The project area was physically segregated into 8 reaches, 13 components, and 14 damage cells. The relationship between components and reaches is as follows:

REACH	COMPONENT
1	1A
	1B
	1C-a
	1C-b
	1C-c
2	2
3	3
4	4
5	5
6	6A
	6B
7	7
8	8

The fourteen damage cells are referred to as Damage Cells 1A,1B,1C,2,3A,3B,4,5,6A,6B,6C,7,and 8. There is an association between a breach in a component, and the damage cells where flooding can potentially occur. Those relationships are as follow:

Breached Component	Damage Cells Potentially Flooded
1A	1A,1B,1C
1B	1B,1C,3B
1C-a	1B,1C,3A,3B,3C
1C-b	1B,1C,3A,3B,3C
1C-c	1B,1C,3A,3B,3C
2	2
3	3
4	4,2
5	5
6A	6A,6B,6C,8
6B	6A,6B,6C,8
7	7
8	6A,6B,6C,8

<u>Breaching</u> – In the LCA model, each component is treated as a homogeneous, independent structure. Under high lake levels, a component can breach through piping or slope stability failure. Breaching of a component is based on user-input Probability of Unsatisfactory Performance (PUP) functions. PUP functions, for each component, define the probability of failure as a function of water surface elevation. PUP functions were developed to represent the base condition and all rehabilitation alternatives. The process by which PUP functions were developed included

consideration of the uncertainty in tailwater elevation; therefore, there is no need to explicitly incorporate tailwater elevations into the determination of breaching. Tailwater estimates are required, however, to determine the type of breach which occurs. It is assumed that time requirements for breach closure are 45 days for a high-velocity breach, and 10 days for a low-velocity breach. Multiple components can potentially breach simultaneously. It is assumed that if a breach occurs, it will be repaired prior to the start of the next simulation period. Breaches are repaired to match the dike conditions that existed prior to the breach; therefore, the probabilities of unsatisfactory performance for pre-breach and post-repair conditions are the same. Probabilities of unsatisfactory performance for a component are, of course, modified if that component is rehabilitated.

As mentioned previously, tailwater estimates at each component are required to determine the breach type (high- or low-velocity). Tailwater estimates are obtained through specification of a minimum, maximum, and most likely tailwater elevation for each component. That information defines a component-specific triangular tailwater probability distribution. A single tailwater probability distribution is established for each component. It is not influenced by factors such as period, lake stage, or hurricane occurrence. Some rehabilitation alternatives included measures to control tailwater elevations in the toe ditches; therefore, the LCA model included provisions that establish a modified tailwater probability distribution to represent post-rehabilitation conditions for some alternatives.

Tropical Cyclones – Each year is characterized by a hurricane season that extends from June 1 through November 30. Tropical cyclones affecting Lake Okeechobee can be divided into two basic categories, those that approach from the Atlantic Ocean, and those that approach from the Gulf of Mexico. An analysis of historical storm surge data indicated that Atlantic storms typically produce the maximum surges along the southeast and east lakeshores, while Gulf storms typically produce the greatest storm surges along the north and northwest lakeshores. The user must provide input relative to the probability of any type of hurricane in a given period. The user must also provide the conditional probability that, if a hurricane occurs, it will be a Gulf storm. Otherwise, it is an Atlantic storm.

Cycle Specification – The LCA model considers each year of simulation as four seasons, or quarters. The seasons are defined as December through February, March through May, June through August, and September through November; therefore, the hurricane season is represented by two seasons. This approach allows the model to capture variation in hurricane probabilities throughout the year, and the season-dependent damages associated with flooding of agricultural resources.

Static Lake Stage – The model assumes that at a given time, there exists a Lake Okeechobee water surface elevation that is the same at all locations (i.e., a flat pool elevation). This is referred to as the static lake stage. The static lake stage varies throughout the year, due to wet and dry seasons, storms, droughts, etc.; however, the static lake stage from one season to the next is serially correlated. In other words, the magnitude of the stage in a given season impacts, to some extent, the magnitude of the stage in the following season. Historical records of Lake Okeechobee stage data were compared to autoregressive and mixed autoregressive moving average models (ARMA) in an effort to identify the most effective stage prediction tool. Preliminary investigations confirmed the stationarity of the series. The ARMA model that performed best was characterized by the following processes:

$$S(t)=m+r[S(t-1)-m]+sd[e(t)-qe(t-1)] \qquad (1)$$

where: S(t)= lake stage at time, t.
m = estimated historical mean lake stage.
r = estimated autocorrelation parameter.
sd = estimated historical standard deviation of lake stage.
et = random error term, normally distributed with mean of 0, and standard deviation, se, determined in equation (2) below.
q = estimated moving average parameter.

and:

$$se=[1-(r*r)]/[1+(q*q)-2rq] \qquad (2)$$

Therefore, the stage at time, t, was based on the historic mean, the departure of the prior period’s stage from the mean, and a random error term calculated for the current and prior periods.

Equation (1) was used to generate the synthetic stages, and the required parameters were estimated based on characteristics of the historical time series of stage data. Those parameter values were as follows:

$$m = 14.379$$
$$r = 0.5350$$
$$sd = 1.5153$$
$$q = -0.4856$$
$$\text{and} \quad se = 0.6376$$

In testing this formulation, the series was started at the historic mean stage. Equation (1) was used, with a normal distribution for et, to generate synthetic stage data. Other probability distributions for et were investigated, and the best fit to historical data was accomplished with a lognormal distribution. The capability for placing physical constraints on the static lake stage estimates was also provided, by allowing specification, if desired, of maximum and minimum allowable stages.

Storm Surge – Hurricane-induced storm surge characteristics vary between components, due to factors such as the geometric characteristics of the shoreline, water depths, fetch lengths, and storm tracks. The storm surge at each component should be cross-correlated, since they are all a result of the same hurricane; therefore, it would not be appropriate to independently determine the storm surge at each component for a given storm. The component surges are not independent events. Instead, the LCA model determines the surge at a single component (the index component), and based on that value, derives the surges at all other components. Component 1C-c was selected as the index component.

The storm surge at the index component is obtained from an input surge-frequency distribution, which represents conditions at component 1C-c. Separate surge-frequency distributions were established for Gulf and Atlantic storms. The frequency distributions were developed to reflect the water surface elevation that corresponds to a duration of four hours (see Figure A-4). This approach was based on the fact that the instantaneous peak surge magnitude would not have a significant impact on seepage and slope stability processes. It was assumed that, in order to contribute to those processes, the rise in water level must last for a period of four hours or more. This

rise in water level, which is of sufficient duration to effect seepage and stability processes, is referred to as the *effective surge*.

The storm surges at all components other than the index component have both a deterministic part and a stochastic part. The deterministic part is calculated by application of a coefficient, which is multiplied by the index component surge. The stochastic part is retrieved from a simple triangular distribution. Both the deterministic and stochastic parts are time-invariant and independent of lake stage. Therefore, the functional relationship between the

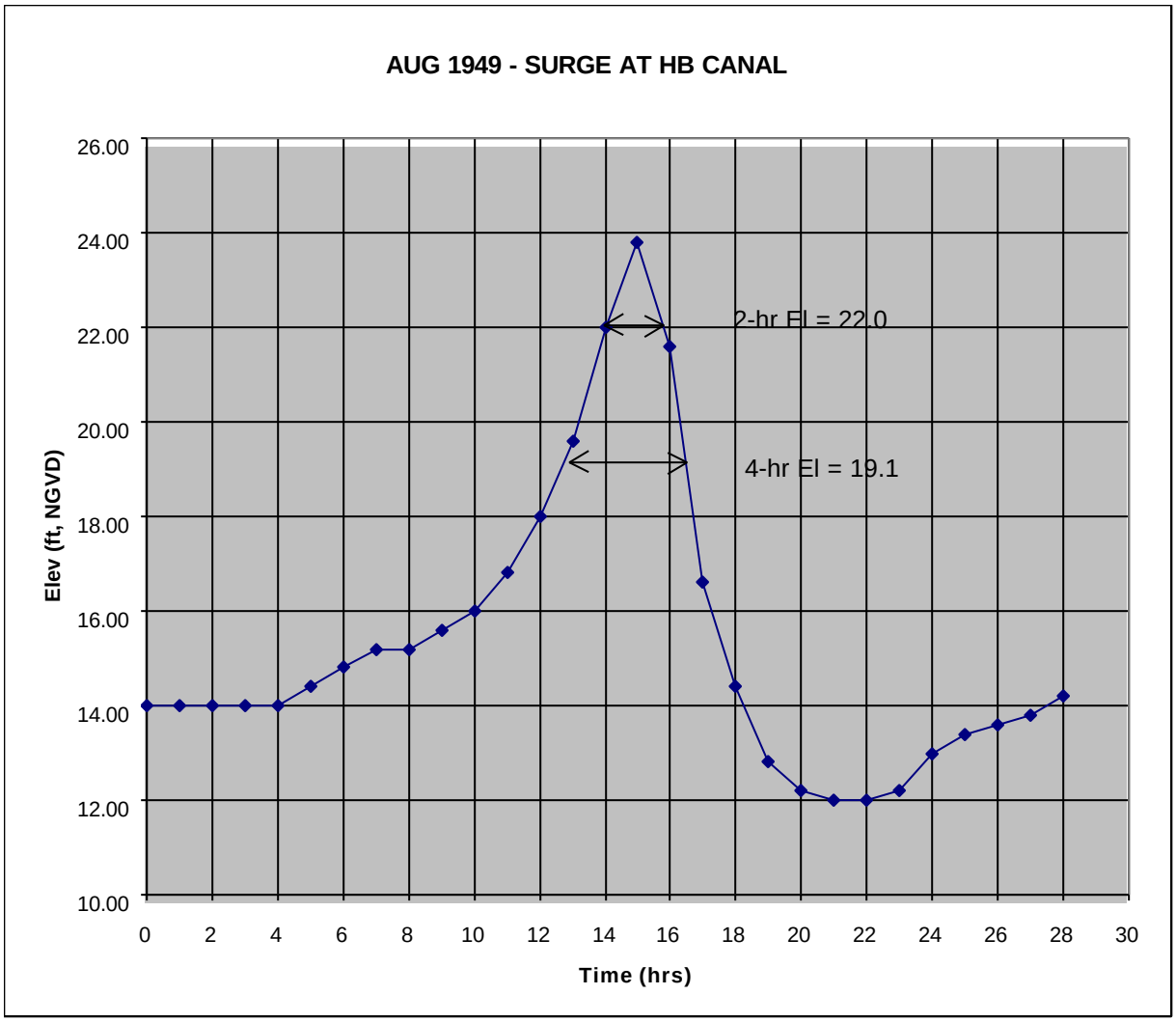


Figure A-4. Typical storm surge hydrograph.

effective surge at a derived component and effective surge at the index component is:

$$ES(d) = [ES(i) \times K(d,hurr)] + ST(d,hurr) \qquad (3)$$

where: ES(d) = effective surge at derived component.
ES(i) = effective surge at index component.
K(d,hurr) = coefficient, based on the component in question and type of hurricane.
and ST(d,hurr) = stochastic term, based on the component in question and type of hurricane.

In summary, establishment of storm surge heights involves determination of hurricane type, quantifying the surge at the index component, and calculation of the derived surges at all other reaches. Those effective surge values are then added to the static lake stage to obtain the total water surface elevation. That total lake stage defines the hydraulic loading, which in concert with the PUP functions, is used to evaluate the dike's performance in each cycle.

6. Impacts of Flood Inundation.

A breach of the dike system would result in flooding of adjacent land areas. The impacts that were quantified by the LCA model are defined in the following paragraphs.

Breach Closure Costs – In an effort to estimate costs associated with breach closures, three dike failure scenarios were considered. For all cases, it was assumed that the final breach width would be about 1,000 feet.

a. *Case 1*. Under this scenario, initial efforts would be directed toward armoring (with stone) the ends of the breach to prevent further erosion. Expected flow velocities suggest that the median armor stone weight should be about 180 pounds. After the ends are armored, 10-foot diameter culverts (assume six pipes, each 50 feet in length) would be placed in the breach and workers would gradually place more stone until closure is accomplished. It is estimated that 14,000 cubic yards (cy) of stone will be required. This stone is located at the South Florida Operations Office (SFOO). Material and equipment haul distances from SFOO to a breach in each of the eight embankment reaches are estimated below.

To Reach 1 - 40 miles
To Reach 2 - 5 miles
To Reach 3 - 15 miles
To Reach 4 - 20 miles
To Reach 5 - 75 miles
To Reach 6 - 40 miles
To Reach 7 - 50 miles
To Reach 8 - 60 miles

Equipment needs include dump trucks for stone hauling, and earth-moving equipment (assume four D-7 bulldozers and two large track-hoes).

b. *Case 2*. If an extremely high lake stage existed at the time of a breach, flow velocities through the breach could exceed 10 to 12 feet per second. In this case, breach closure as described above (in Case 1) would not be possible due to the massive stone requirements; therefore, another alternative was proposed. That alternative involves the construction of an “eyebrow levee” in the immediate vicinity of the breach. Pile-driving equipment would be positioned on barges at some distance from the breach, where flow velocities are manageable. Pilings would be driven in a semicircular array. Then, trees (e.g. melaleuca), stone, soil, sandbags, etc. would be placed upstream of the pilings to block the flow. It is assumed that approximately 30 pilings would be required. Estimated equipment and material requirements include four barges, one pile-driving rig, four bulldozers, two track-hoes, two skidders, and dump trucks for hauling earth and stone. The volume of stone required for closure is 18,000 cy. This breach closure approach would only be viable in Reaches 1; therefore, the appropriate distance for estimating mobilization from SFOO is 40 miles.

c. *Case 3*. If breach flow velocities were extremely high, and a breach site were located anywhere along the dike between Moore Haven and Bacom Point, the tree islands could be used as a foundation for emergency closure. Stone and fill material would be transported by barge and dumped to raise the elevation of the tree island immediately upstream of the breach. Both ends of the rim canal would then be blocked to complete the closure. This breach closure approach would be considered only in Reaches 2 and 3; therefore, haul distances from SFOO are 5 and 15 miles, respectively. The volume of stone required for closure is estimated as 14,000 cubic yards.

During emergency situations, needed earthen fill materials could be obtained by excavating sections of the nearest tieback levees, which would be repaired later. Other fill sources include the C-43 and Nubbin Slough spoil banks. Additional materials and resources, such as armor stone, trucks, earth-moving equipment, lighting, barges, vessels, etc., would be available from the SFOO, the National Guard Construction Fleet, Miami area contractors, and/or local contractors.

Based on the assumptions described above, the following breach closure costs were used as input to the LCA model.

HERBERT HOOVER DIKE BREACH CLOSURE COSTS					
		LOW VELOCITY BREACH		HIGH VELOCITY BREACH	
	Length	Repair Cost	Repair Cost	Repair Cost	Repair Cost
Reach	(feet)	($)	($/ft)	($)	($/ft)
1	118000	$766,483	$6.50	$1,304,574	$11.06
2	107800	$639,083	$5.93	$1,304,574	$12.10
3	35350	$675,483	$19.11	$1,304,574	$36.90
4	82818	$693,683	$8.38	$1,304,574	$15.75
5	76791	$893,883	$11.64	$1,304,574	$16.99
6	147768	$766,483	$5.19	$1,304,574	$8.83
7	92678	$802,883	$8.66	$1,304,574	$14.08
8	89011	$839,283	$9.43	$1,304,574	$14.66

<u>Breach Repair Costs</u> – In the event of a dike failure, the primary objective would be to close the breach as quickly as possible; however, the post-closure condition of the dike in the vicinity of the breach would be poor. To improve these poor conditions, immediate efforts would be initiated to restore an acceptable level of protection along the damaged dike. For the base condition, it was assumed that the dike would be restored to its original, pre-breach condition. The LCA model also considered costs associated with restoration of a rehabilitated dike component, even though the likelihood of a rehabilitated dike failure would be remote.

Repair efforts would include construction of a sheetpile cofferdam and pumping to establish a suitable working environment. Restoration of the dike would then be accomplished through typical levee construction activities. Repair costs input to the LCA model are provided below in Table 1.

Table 1. Dike Repair costs.

1. EXISTING CONDITIONS (these values reflect existing conditions, I.e., repair of the dike as it now exists).

		LOW VELOCITY BREACH		HIGH VELOCITY BREACH	
Component	Length (feet)	Repair Cost ($)	Repair Cost ($/ft)	Repair Cost ($)	Repair Cost ($/ft)
1A	25,500	$18,432,750	$722.85	$18,549,030	$727.41
1B	21,500	$18,406,750	$856.13	$18,504,790	$860.69
1C-a	24,000	$18,423,000	$767.63	$18,532,440	$772.19
1C-b	23,000	$18,416,500	$800.72	$18,521,380	$805.28
1C-c	24,000	$18,423,000	$767.63	$18,532,440	$772.19
2	107,800	$18,906,254	$175.38	$19,571,380	$181.55
3B	35,350	$18,942,539	$535.86	$19,571,415	$553.65
4	82,818	$18,961,015	$228.95	$19,571,384	$236.32
5	76,791	$16,747,847	$218.10	$17,158,679	$223.45
6A	76,800	$16,252,592	$211.62	$16,532,144	$215.26
6B	70,968	$16,222,324	$228.59	$16,480,647	$232.23
7	92,678	$19,069,591	$205.76	$19,571,906	$211.18
8	89,011	$16,693,374	$187.54	$17,158,901	$192.77

2. ALTERNATIVE A - TAILWATER REGULATION (this alternative would not impact the breach repair costs; therefore, they are the same as the existing condition repair costs).

		LOW VELOCITY BREACH		HIGH VELOCITY BREACH	
Component	Length (feet)	Repair Cost ($)	Repair Cost ($/ft)	Repair Cost ($)	Repair Cost ($/ft)
1A	25,500	$18,432,750	$722.85	$18,549,030	$727.41
1B	21,500	$18,406,750	$856.13	$18,504,790	$860.69
1C-a	24,000	$18,423,000	$767.63	$18,532,440	$772.19
1C-b	23,000	$18,416,500	$800.72	$18,521,380	$805.28
1C-c	24,000	$18,423,000	$767.63	$18,532,440	$772.19
2	107,800	$18,906,254	$175.38	$19,571,380	$181.55
3B	35,350	$18,942,539	$535.86	$19,571,415	$553.65
4	82,818	$18,961,015	$228.95	$19,571,384	$236.32
5	76,791	$16,747,847	$218.10	$17,158,679	$223.45
6A	76,800	$16,252,592	$211.62	$16,532,144	$215.26
6B	70,968	$16,222,324	$228.59	$16,480,647	$232.23
7	92,678	$19,069,591	$205.76	$19,571,906	$211.18
8	89,011	$16,693,374	$187.54	$17,158,901	$192.77

(continued next page)

Table 1 (continued)

3. ALTERNATIVE B - SEEPAGE BERM

Rehab cost = $4.5 million/mile, therefore 1,000-ft repair = $852,273.
Therefore, add $852K to each of the existing condition values.

		LOW VELOCITY BREACH		HIGH VELOCITY BREACH	
Component	Length (feet)	Repair Cost ($)	Repair Cost ($/ft)	Repair Cost ($)	Repair Cost ($/ft)
1A	25,500	$19,285,023	$756.28	$19,401,303	$760.84
1B	21,500	$19,259,023	$895.77	$19,357,063	$900.33
1C-a	24,000	$19,275,273	$803.14	$19,384,713	$807.70
1C-b	23,000	$19,268,773	$837.77	$19,373,653	$842.33
1C-c	24,000	$19,275,273	$803.14	$19,384,713	$807.70
2	107,800	$19,758,527	$183.29	$20,423,653	$189.46
3B	35,350	$19,794,812	$559.97	$20,423,688	$577.76
4	82,818	$19,813,288	$239.24	$20,423,657	$246.61
5	76,791	$17,600,120	$229.20	$18,010,952	$234.55
6A	76,800	$17,104,865	$222.72	$17,384,417	$226.36
6B	70,968	$17,074,597	$240.60	$17,332,920	$244.24
7	92,678	$19,921,864	$214.96	$20,424,179	$220.38
8	89,011	$17,545,647	$197.12	$18,011,174	$202.35

4. ALTERNATIVE C - CUTOFF WALL

Rehab cost = $15.7 million/mile, therefore 1000-ft repair = $2,973,485.
Therefore, add 2.3M to each of the existing condition values.

		LOW VELOCITY BREACH		HIGH VELOCITY BREACH	
Component	Length (feet)	Repair Cost ($)	Repair Cost ($/ft)	Repair Cost ($)	Repair Cost ($/ft)
1A	25,500	$21,406,235	$839.46	$21,522,515	$844.02
1B	21,500	$21,380,235	$994.43	$21,478,275	$998.99
1C-a	24,000	$21,396,485	$891.52	$21,505,925	$896.08
1C-b	23,000	$21,389,985	$930.00	$21,494,865	$934.56
1C-c	24,000	$21,396,485	$891.52	$21,505,925	$896.08
2	107,800	$21,879,739	$202.97	$22,544,865	$209.14
3B	35,350	$21,916,024	$619.97	$22,544,900	$637.76
4	82,818	$21,934,500	$264.85	$22,544,869	$272.22
5	76,791	$19,721,332	$256.82	$20,132,164	$262.17
6A	76,800	$19,226,077	$250.34	$19,505,629	$253.98
6B	70,968	$19,195,809	$270.49	$19,454,132	$274.13
7	92,678	$22,043,076	$237.85	$22,545,391	$243.27
8	89,011	$19,666,859	$220.95	$20,132,386	$226.18

Operation and Maintenance Costs – O&M costs are specified by reach. An analysis of annual O&M expenditures (Y) versus annual peak lake stage (X) provided the following relationships:

Reach 1 - Y = 76,595(X) – 586,446
Reach 2 - Y = 24,304(X) + 82,554
Reach 3 - Y = 10,727(X) – 61,578
Reach 4 - Y = 21,585(X) – 131,669
Reach 5 - Y = 22,123(X) – 50,299
Reach 6 - Y = 19,397(X) – 71,768
Reach 7 - Y = 22,371(X) – 136,896
Reach 8 - Y = 18,269(X) – 70,637

These relationships were modified to reflect quarterly O&M costs for input into the seasonal LCA model.

The model also provides for the fact that higher O&M costs are incurred in years following hurricane inundation. A multiplicative factor was established based on an analysis of historical data. Therefore, if a hurricane occurs during the simulation, the O&M costs for that period and the subsequent three periods are computed as the original O&M cost (defined by the relationship above) times the hurricane-related multiplier.

Rehabilitation Costs – Rehabilitation alternatives are introduced into the LCA model by changing the component data to reflect the with-rehabilitation conditions, and by introducing a rehabilitation cost schedule; therefore, a rehabilitation alternative consists of the two following parts:

(a) a spending schedule which defines, by quarter, the expenditures associated with a given rehabilitation plan. This information is used to calculate the present value of that plan.
(b) changes to the component data that reflect all impacts of the rehabilitation plan. These include post-rehabilitation PUP functions, repair costs, and tailwater distributions.

In effect, a rehabilitation plan primarily involves resetting the parameter values for the rehabilitated dike component. These changes are assigned an effective cycle to account for the time required to construct various rehabilitation features. With this approach, different

components can be rehabilitated at different times or simultaneously. Also, individual components can be rehabilitated more than once. The model effectively separates the physical consequences of the rehabilitation from the economic consequences. This allows the user to account for items such as mobilization costs, economies of scale, and specifically scheduled construction, as compared to simply assigning a rehabilitation cost to an individual component.

Socio-Economic Impacts – The LCA model is capable of assigning various damages to specific damage cells, based on specific breach scenarios. This is accomplished by use of an extensive array of lake stage versus damage relationships. Impacts can be monetary or non-monetary. All monetary impacts are summed to one total damage value, while non-monetary impacts are individually managed.

The economic database includes four land use types (residential urban, non-residential urban, agricultural fruit tree cops, and agricultural row and truck crops), one "other benefits" category, and one non-monetary "population affected" category. Impacts vary depending on the particular components that breach, the type of breach, the season of the year, flooding in prior seasons, and the lake stage at the time of the breach.

The impact calculation problem involves determining which damage cells are inundated, and the economic damages in those cells. The problem is complicated by the possibility of multiple simultaneous breaches, as one individual damage cells can be affected by breaches in more than one component; thus, double counting of damages must be prevented. The solution method involves maintaining a list of all damage cells that are inundated for the current period. At the start of each period, the list is always cleared. Each component that breaches is processed in order. For each component, the damage cells affected are known, as are the particular damage functions for the period, land use type, and breach velocity. As each component is examined, if we encounter a new damage cell it is added to the list of affected damage cells. The appropriate stage-damage function for each cell is selected, based on the current component, period, and breach type, for each of the land uses. The damage value is calculated, based on static lake stage, and adjusted for prior period damages. Static lake stage is used for the

damage determination, as the storm surge component is characterized by a short duration and would have no significant impact on the flooding severity.

For each impact type, the list is examined to see if previously recorded damage (due to inundation from another breached component) exists. If so, then the model tests to see if the damage associated with the current component is greater than the damage currently recorded for that land use type. If so, the previous value is replaced with the current value. If not, the previous value is left in place, thereby reflecting that higher damages have already been recorded for this land use in the current damage cell. The ultimate result is a list of all damage cells, and numbers for each of the impact types, expressing the maximum damages associated with inundation from a breach in any component. This information (by land use type and damage cell) is then recorded for the current period, and appropriately discounted based on present value factors. In addition, the list of inundated damage cells is used to update information on the last cycle of inundation for each damage cell to the current cycle, for use in subsequent periods for damage reduction.

7. MODEL INPUT REQUIREMENTS.

Due to the complexity of the physical problem being considered, the LCA model requires a substantial amount of user-provided input. Information is needed to describe reaches, components, damage cells, inter-relationships between components and damage cells, and damages associated with dike failures. A summary of those input requirements is provided below.

<u>Description of the Physical System</u> – This information describes the physical characteristics of the dike system and surrounding lands.

a. Reach Information – Each of the 8 reaches is identified by number, length, location, and associated O&M cost versus lake stage relationship. An example of the LCA model’s Reach Management Form is displayed in Figure A-5.
b. Component Information – Each of the 13 dike components is identified by alphanumeric name, length, location, parent reach, PUP function, breach repair cost, breach closure cost, tailwater frequency

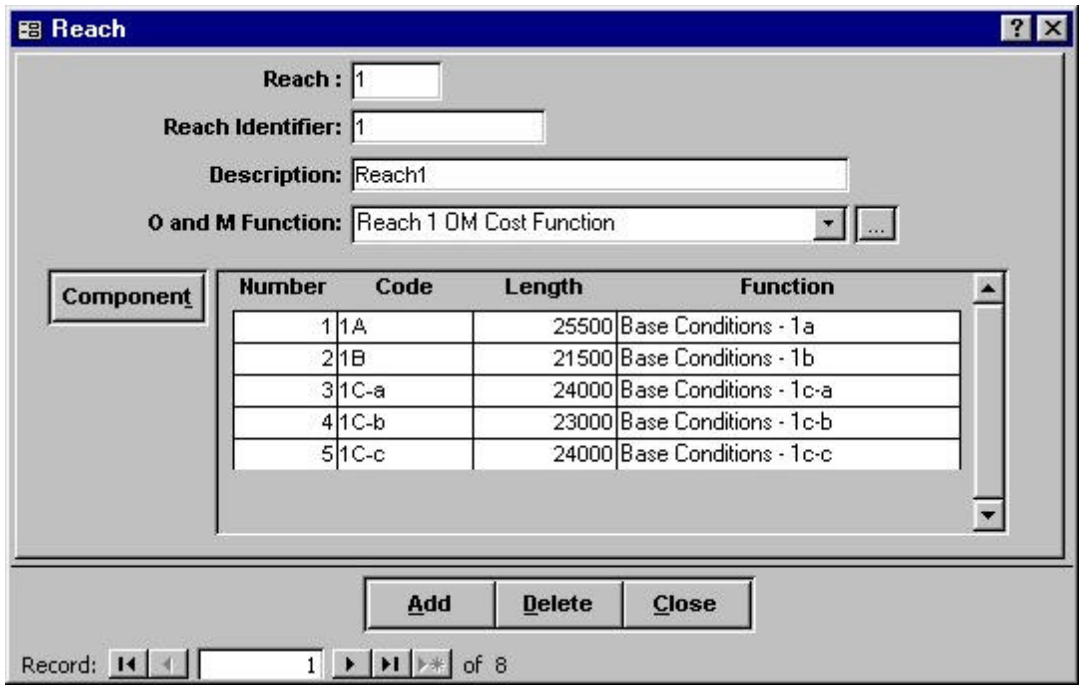


Figure A-5. Reach management form.

c. distribution, storm surge multiplier coefficient, frequency distribution for stochastic storm surge component, and lake stage reduction factors.
d. Damage Cell Information – alphanumeric name, and locations identify each of the 14 damage cells.
e. Component-Damage Cell Relationships – For each component, a list of all damage cells which may (depending on lake stage at time of breach) be affected must be identified.

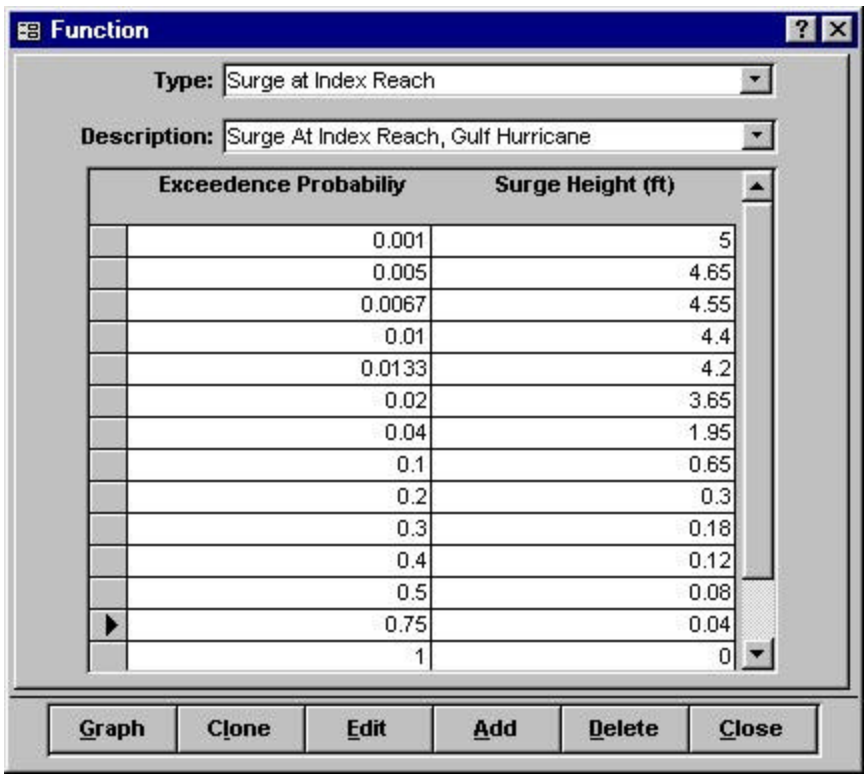
Period (Seasonal) Information - Each of the four periods (i.e., seasons, quarters, cycles) must be assigned an identifying number. The number of days in a period must be specified; and the probabilities associated with any hurricane occurrence, a Gulf hurricane occurrence, and an Atlantic hurricane occurrence must be identified.

Lakeside Water Surface Elevation – The lakeside water surface elevation consists of two components, static lake stage and hurricane-induced storm surge.

a. Static Lake Stage – Computation, in each cycle, of the static lake sage is accomplished through use of

the previously described ARMA model. Parameters required as user input include, the historical mean lake stage, the historical standard deviation of lake stage, the estimated serial autocorrelation parameter, the mean of the random error term, maximum possible lake stage, minimum possible lake stage, and an initial lake stage to begin the modeling process.

b. Storm surge at Index Component – A storm surge frequency distribution, for each storm type (Atlantic and Gulf) is required. These frequency distributions represent conditions at the index component. Figure A-6 depicts the LCA model interface for introduction of storm surge data at the index component.



Exceedence Probability	Surge Height (ft)
0.001	5
0.005	4.65
0.0067	4.55
0.01	4.4
0.0133	4.2
0.02	3.65
0.04	1.95
0.1	0.65
0.2	0.3
0.3	0.18
0.4	0.12
0.5	0.08
0.75	0.04
1	0

Figure A-6. Function management form for storm surge input.

<u>Economic Information</u> – The model requires input data related to stage damage relationships, costs of breach closures, costs of breached dike repairs, and rehabilitation costs as described below.

a. Damages – Much of the damage information is provided as piece-wise linear functional relationships, which

express damages as a function of static lake stage. The model selection and application of a particular function depends on the breached component, land use, affected damage cell, season, and breach type. Each function is assigned a unique identifier. As an example, consider the stage damage curves for truck/row crops. These functions must include seasonal variation; therefore, separate curves for each damage cell, and each season, are required. To achieve consistency in the data processing methodology, quarterly curves must be identified for other land uses as well; however, if there is no seasonal variation relative to that land use, then the same function may be used for each quarter. This approach is depicted in Figure A-7, which depicts the models Stage Damage Management Form.

Hoover Dike - [Stage Damage]

File

Component	Damage Cell	Land Use	Period	Velocity	Function Description
1A	1A	Urban Residential	1	Low	Function90
1A	1A	Urban Residential	2	Low	Function90
1A	1A	Urban Residential	3	Low	Function90
1A	1A	Urban Residential	4	Low	Function90
1A	1A	Non-Residential Urban	1	Low	Function7
1A	1A	Non-Residential Urban	2	Low	Function7
1A	1A	Non-Residential Urban	3	Low	Function7
1A	1A	Non-Residential Urban	4	Low	Function7
1A	1A	Fruit Tree Crops	1	Low	Function91
1A	1A	Fruit Tree Crops	2	Low	Function91
1A	1A	Fruit Tree Crops	3	Low	Function91
1A	1A	Fruit Tree Crops	4	Low	Function91
1A	1A	Row-Truck Crops	1	Low	Function92
1A	1A	Row-Truck Crops	2	Low	Function93
1A	1A	Row-Truck Crops	3	Low	Function94
1A	1A	Row-Truck Crops	4	Low	Function95
1A	1A	Population Affected	1	Low	function257
1A	1A	Population Affected	2	Low	function257
1A	1A	Population Affected	3	Low	function257
1A	1A	Population Affected	4	Low	function257
1A	1A	Other	1	Low	Dummy Other Benefits-Stage Function
1A	1A	Other	2	Low	Dummy Other Benefits-Stage Function

Record: 1 of 2160

Figure A-7. Example of stage damage management form.

b. Closure and Repair Costs – Emergency repair costs associated with breach closure and damaged dike repair were input at the component description level.
c. O&M Costs – O&M costs versus lake stage relationships were input at the component description level. A

global multiplicative factor is also introduced to account for higher O&M costs in periods following a hurricane occurrence.

d. Rehabilitation Costs – Costs associated with a rehabilitation plan must be specified as a spending schedule, which identifies the dollars expended in each quarter.

Physical Rehabilitation Information – In addition to the cost cycle data, for each rehabilitation scenario proposed, the user must supply information which describes the features, performance, and impacts of that plan. That information includes post-rehabilitation PUP functions, repair costs, and tailwater frequency distributions. It is introduced to the model through use of the Rehabilitation Plan Management Form, as depicted in Figure A-8.

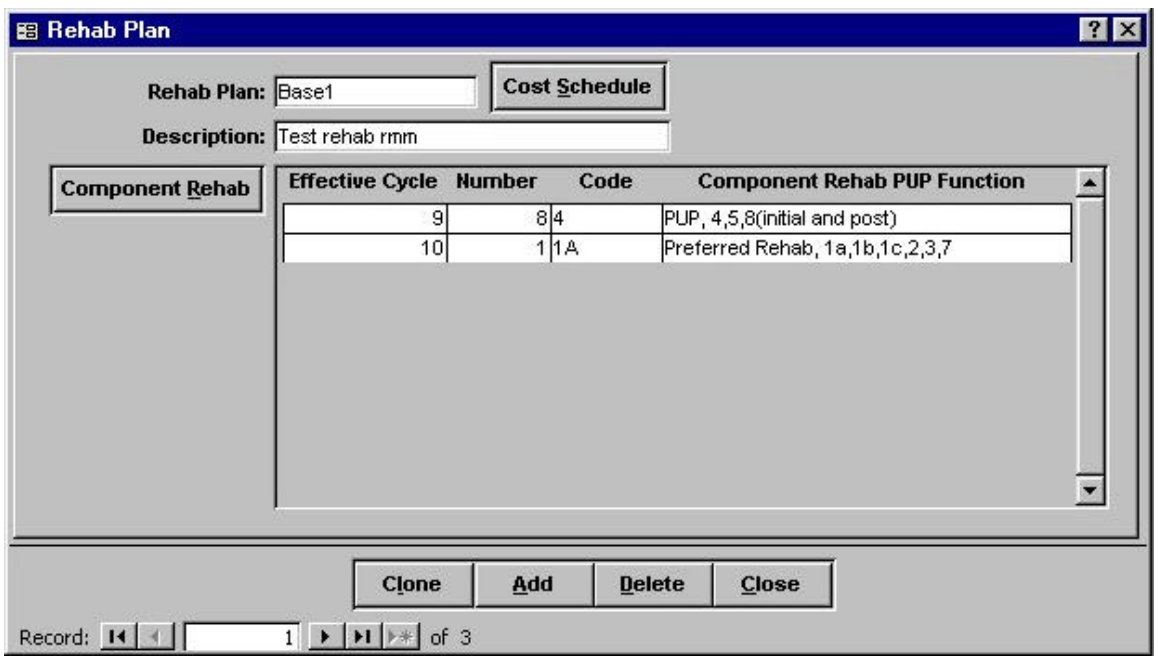


Figure A-8. Rehabilitation plan management form.

General Information – General information required for each life cycle simulation includes the duration of the simulation (typically 50 years + design/construction time), the number of life cycle iterations (typically 5,000), interest rate, simulation start year, economic analysis base year, critical head differential, and initial water surface elevation. This information is input through use

of the Scenario Management Form. An example is provided in Figure A-9.

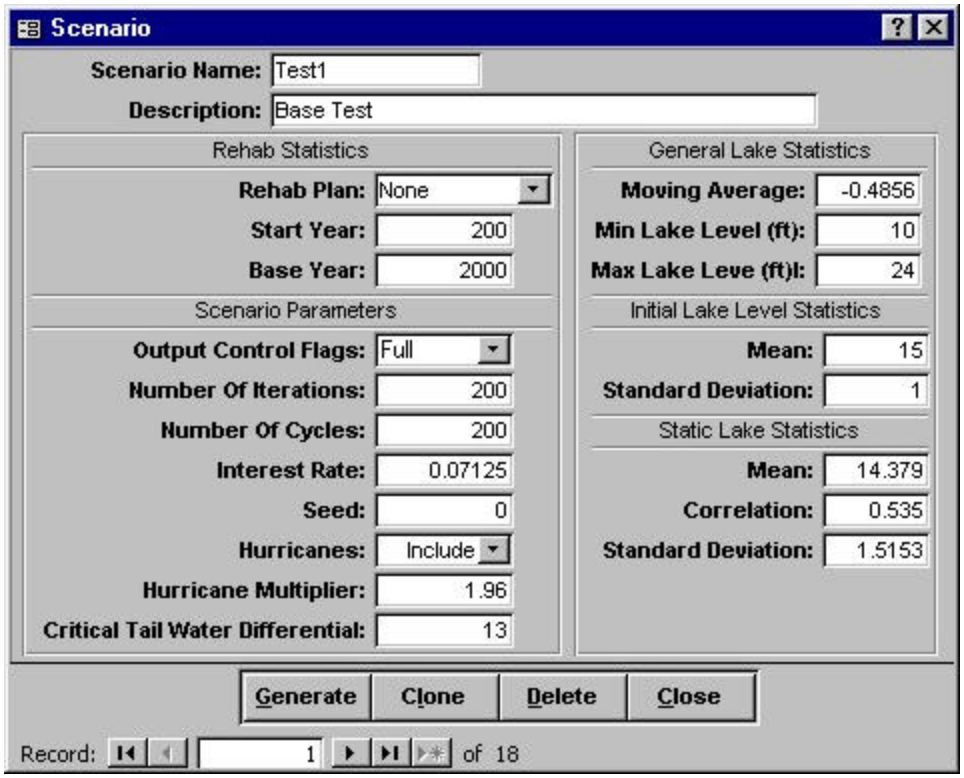


Figure A-9. Scenario managementfForm.

8. MODEL RESULTS.

Results of the LCA model simulations may be viewed in either graphical or tabular form. The graphics features may be used to display any of the input function data. Results that may be displayed graphically include the following.

<u>Component Breach Information</u> – This display allows the user to gauge the frequency of breaching that took place in a given simulation. The user may choose to evaluate breaching frequency for a single scenario but with multiple components. Another option allows graphical display of breaching frequency for multiple scenarios with a single component. An example is provided in Figure A-10, which displays the frequency of breaches for all components during a 1,000-iteration life cycle simulation.

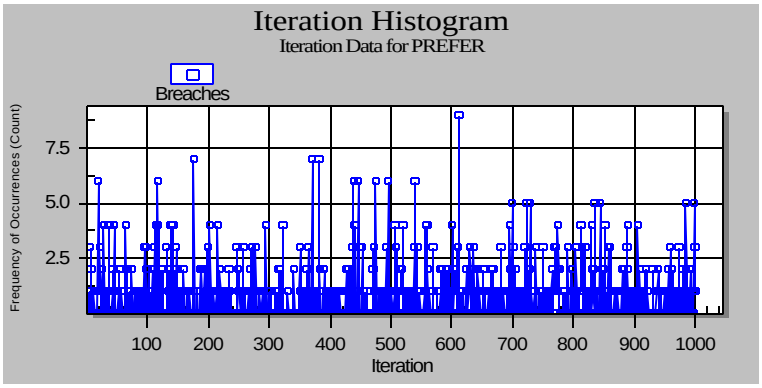


Figure A-10. Breach frequency plot.

Lake Stage Information – Lake stage data can be viewed graphically to evaluate simulated stage frequency. Options include evaluation of a single scenario and multiple periods, or multiple scenarios for one period. An example of a single scenario lake stage histogram is provided in Figure A-11.

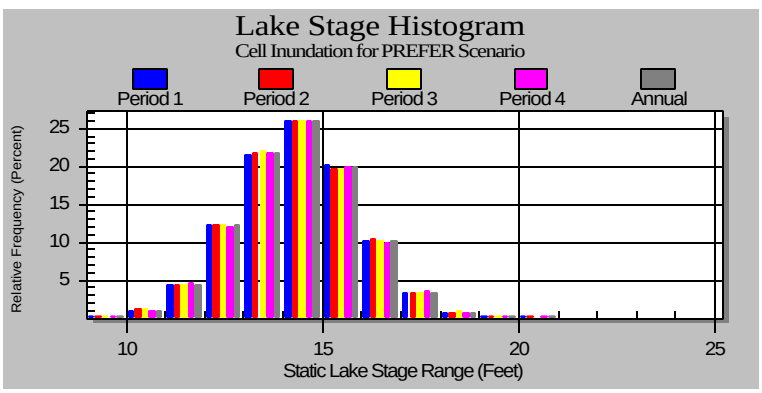


Figure A-11. Lake stage histogram.

Iteration Statistics – Statistics related to costs incurred during a simulation, hurricane occurrences, and breach frequency can be viewed graphically. Examples of iteration histograms related to total costs, hurricane events, and breaching are presented in Figures A-12, A-13, and A-14, respectively.

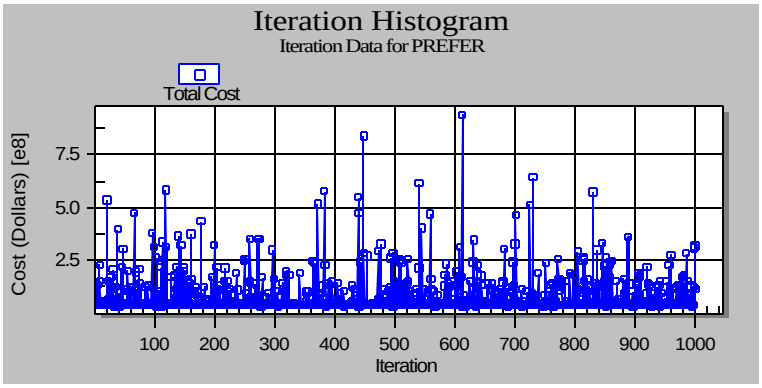


Figure A-12. Total cost histogram.

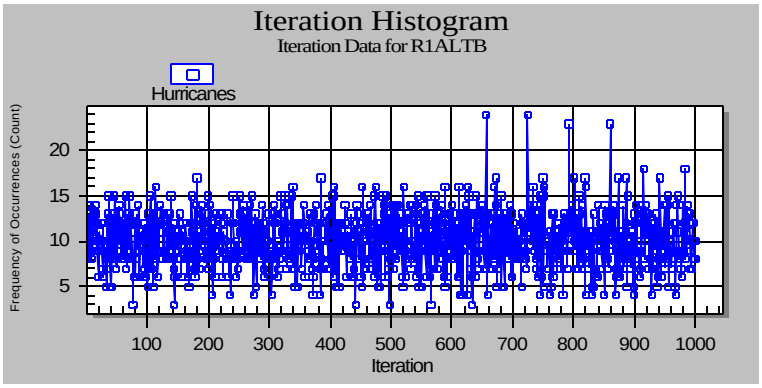


Figure A-13 Hurricane frequency histogram.

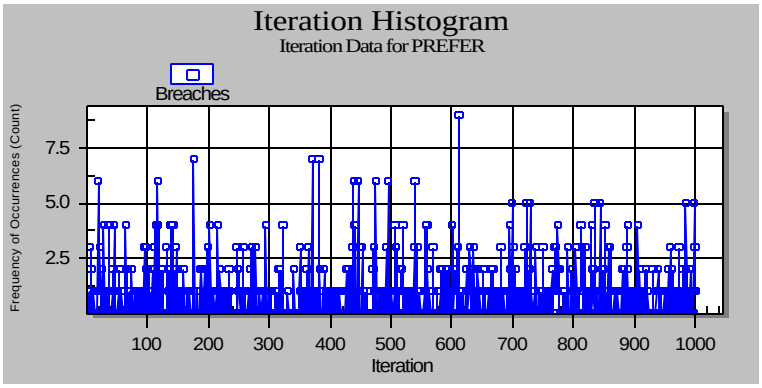


Figure A-14. Breach frequency histogram.

The LCA model also offers a variety of tabular options for presentation of model results. Options for reporting of model input and output data include:

(a) Reach Summary Report – This table displays the characteristics applicable to an entire reach. It is a compilation of reach-associated input data. No model results are included. An example is provided in the Sample Tables section at the end of this appendix.

(b) Reach All Report – This table includes the information included in the Reach Summary Report, plus detailed damage cell and stage-damage function data. Due to the length of the Reach All Report (70+/- pages), it was not included in the Sample Tables section.

(c) Function Data Report – These reports summarize the values that define various user-input functions utilized by the LCA model. Function data reports are available for component PUP functions, damage cell stage-damage functions, damage reduction based on prior period inundation, lake stage reduction functions, reach O&M cost functions, and the storm surge function at the index reach. An example of a function data report, which provides base condition PUP functions is provided in the Sample Tables section at the end of this appendix.

(d) Scenario Description Summary Report – This report summarizes the cost statistics associated with a given scenario simulation. An example is provided in the Sample Tables section at the end of this appendix.

(e) Scenario Simulation Comparison Report – This report allows the user to compare the results of one or more rehabilitation scenarios to the Base Condition. It provides statistical data relative to benefits and costs, and presents a benefit/cost ratio. An example is provided in the Sample Tables section at the end of this appendix.

(f) Scenario Land Use Results – This report summarizes, for a given simulation, the damages and damage statistics, by land use, for all damages incurred in each damage cell. An example is provided in the Sample Tables section at the end of this appendix.

ASCII output files are also available for printing. They are identified by the following subscripts: *.PRN – output

print file with all run statistics, *.ITR – iteration statistics with one line per iteration, *.DBG – debug information file was used by model developers to track items during LCA model evolution, and *.ECH – a file that echos the input data used for a given simulation.

9. SUMMARY.

The Herbert Hoover Dike Life Cycle Analysis Model is a menu driven system, containing simplified data entry forms, exceptional graphics capabilities, statistical reports, and a Monte Carlo simulation process. The model is capable of simulating severe weather, calculating lake levels, and estimating damages and costs for the base condition and various dike rehabilitation proposals. Results generated during model simulations are used to evaluate expenditures and assist in the decision-making processes associated with potential rehabilitation plans.

Herbert Hoover Dike

Major Rehabilitation Evaluation Report

Life Cycle Analysis Model

Sample Tables

Herbert Hoover Dike

Major Rehabilitation Evaluation Report

Appendix B

Economic Analysis

APPENDIX B

ECONOMIC ANALYSIS

TABLE OF CONTENTS

STUDY AREA 1
BACKGROUND 3
DAMAGE TO DEVELOPMENT - DATA COLLECTION 3
General 3
Reach determination 4
Determination of flood stages 9
Determination of flood duration 10
Determination of topographic elevations 10
Urban Land use determination 11
Urban Structure Value and First Floor Elevations 12
Urban Content Values 25
Urban Damage Relationships 26
Summary 27
Agricultural Land Use Determination 29
Agricultural Damage Relationships 30
EVALUATION OF FLOOD DAMAGE POTENTIAL 37
Urban damage susceptibility: 37
Agricultural damage susceptibility: 37
Data Reduction: 40
Emergency Response Costs 45
Road Damages 47
Infrastructure 48
Automobile Damages Prevented 48
Estimated HTRW Related Costs Due To Flooding 50
HOOVER DIKE MODEL 56
General Description 56
Acknowledgements 57
Background 57
Technical Overview 58
Model Processing 61
Analysis Framework 62
a) Physical System 62
b) Breaching 64
c) Hurricanes 65
d) Seasons 65
e) Water Surface Elevation 66
Static Lake Stage 66
Storm Surge 67
f) Impacts of Inundation 67
Repair Cost 68

O&M Costs 68
Rehabilitation Costs 68
Impacts 69
2) Simulation Process 71
Hurricane Occurrence 72
Storm Surge 73
3) Method of Handling of Rehabilitation 76
4) Data Requirements of the Model 77
Model Outputs 80
Sensitivity to Topography 101
Benefits Not Computed 105
Comprehensive Rehabilitation Plan 108

APPENDIX B

ECONOMIC ANALYSIS

STUDY AREA

Lake Okeechobee is a large natural, shallow, freshwater lake located in south central Florida (Figure 1). It is the second-largest freshwater lake in the continental United States. The lake has a surface area that covers approximately 730 square miles at elevation 14.0 feet. Historically, its depth has ranged from about 10 to 18 feet, and its bottom is near sea level. Its tributary drainage is about 5,300 square miles and it can hold more than one trillion gallons of water. Lake Okeechobee waters are contained within a system of embankment and control structures, which comprise the Herbert Hoover Dike Levee System (HHD). The HHD encircles 143 miles of Lake Okeechobee perimeter. The HHD system provides water supply, flood control, navigation, fish and wildlife, and recreation benefits. In addition, four major canals, which are connected to Lake Okeechobee, assist in the development of rich agricultural lands south of the lake.

Portions of HHD were built in the 1930s under the authorization of the River and Harbor Act of 1930. In 1947 a major hurricane prompted the need for additional flood protection in Florida. In response, Congress passed the Flood Control Act of 1948 which authorized the first phase of a comprehensive plan for flood protection and other water control benefits in central and southern Florida. In the 1980s and 1990s, investigations of the dike system revealed seepage and stability problems. The HHD levee system is critically important for areas immediately landward in terms of flood protection. These areas have potential for severe damages and loss of life if the HHD system performs unsatisfactorily.

Figure 1

BACKGROUND

The Jacksonville District of the U.S. Army Corps of Engineers has undertaken the evaluation of major rehabilitation proposals for the Herbert Hoover Dike. The rehabilitation problem is that the dike may exhibit unreliable performance at high lake levels resulting in failure of one or more dike sections. A failure would flood the drainage compartment corresponding to the failed section causing damage property. Also, emergency repairs would be undertaken to close the breach, incurring additional costs.

The structural performance of the dike is not certain. Engineering analysis has concluded that the likelihood of any failure is dependent upon the differential water levels between the lake and the tailwater. The larger this head differential, the greater the likelihood of piping or sliding failures. The water level at any embankment section is dependent on both the static lake level and surge due to hurricanes. Both of these are independent random variables.

To meet the major rehabilitation study reporting requirements, the base condition and all rehabilitation strategies must be evaluated by quantifying benefits and costs over a specified life-cycle time period. A major source of economic benefit of a major rehabilitation is measured as the difference in the present discounted value of life cycle costs between an alternative and the base condition. Actual yearly performance and cost over the life cycle is uncertain since performance can only be described probabilistically. Life-cycle cost is really a distribution based on the possible series of yearly performances and corresponding costs. Since there are a large possible number of these series, Monte Carlo simulation is used to numerically estimate the distribution of these costs.

DAMAGE TO DEVELOPMENT - DATA COLLECTION

General

To determine potential damage reduction associated with different levels of levee repair, stage-damage curves are utilized to assess damage potential with and without improvements. Building stage-damage relationships are comparable to computing average annual damage and benefits in a traditional flood control study. The only differences are lake stage probabilities and probabilities of unsatisfactory performance for levee break scenarios will be assigned by the risk model

using Monte Carlo simulation.

Separate stage-damage relationships have been prepared for each affected hydrologic reach, for each of four aggregated land use classes. The aggregated land use classes are residential urban, non-residential urban, tree fruit crops and truck crops and pasture. Three storms of different magnitudes have been utilized to produce the relationships based upon lake stages of 18 feet, 21 feet and 26 feet. Separate sets of stage-damage relationships have been prepared for high and low velocity breaches. To prepare stage-damage relationships, certain types of information must be collected for urban and agricultural land use types in the basin. For urban land uses data requirements include reach determination, land use determination, determination of topographic elevations and first floor elevations to determine flood depth in structure, structure and content value determination, and determination of depth-damage relationships. In addition for agricultural land use types, the duration of flooding must be considered. This information is discussed in the following paragraphs.

Reach determination

It is necessary to produce stage-damage functions to coincide with reach delineation used in the risk model. The economic study area is contained by the 25-foot contour surrounding Lake Okeechobee at varying distances within portions of Glades, Hendry, Martin, Okeechobee, and Palm Beach counties. Within the study area, there are eleven damage cells in the model corresponding to the locations of possible levee breaches. These cells are shown in Figures 2, 3 and 4.

Figure 2

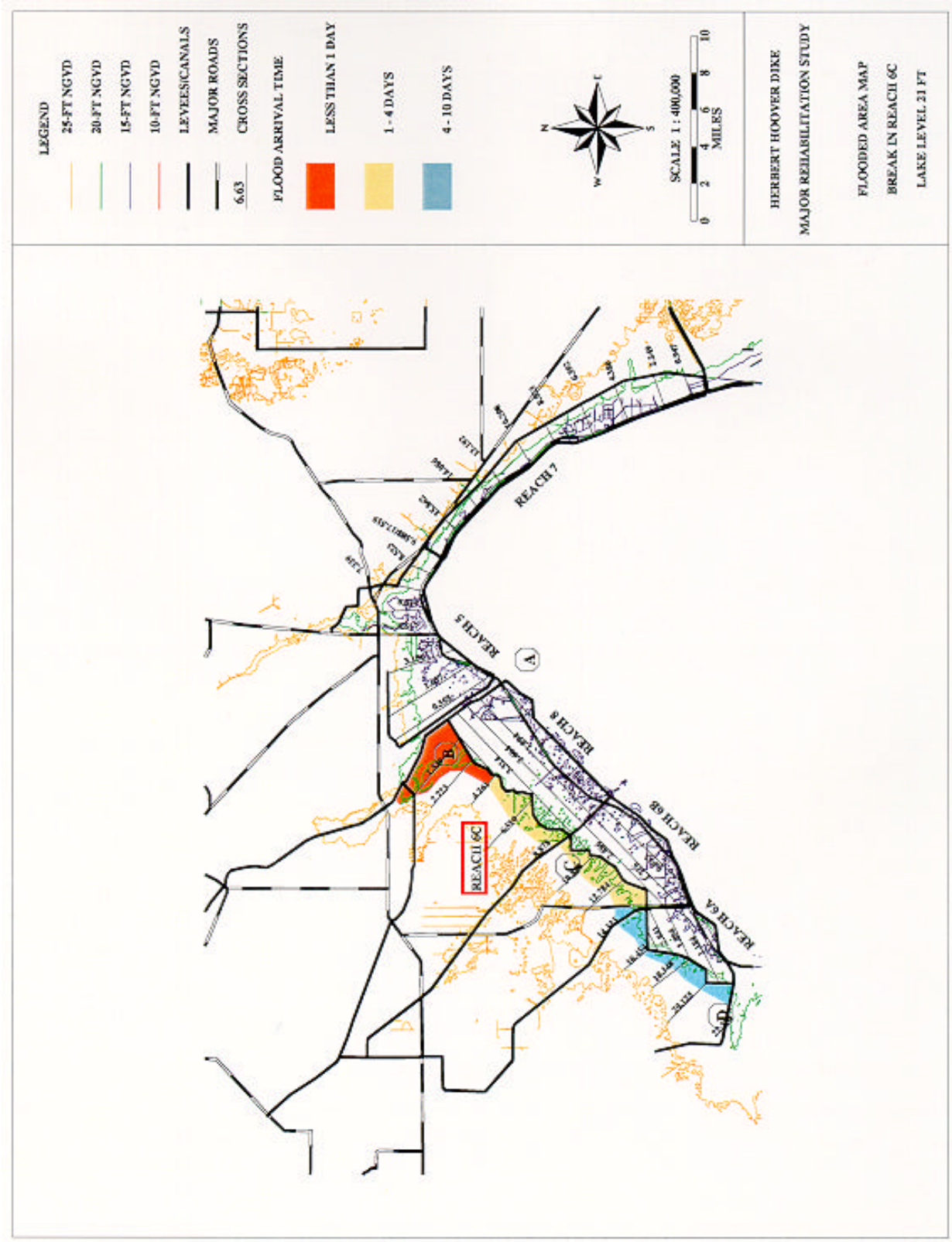

LEGEND
25-FT NGVD
20-FT NGVD
15-FT NGVD
10-FT NGVD
LEVEES/CANALS
MAJOR ROADS
CROSS SECTIONS
6.63
FLOOD ARRIVAL TIME
LESS THAN 1 DAY
1 - 4 DAYS
4 - 10 DAYS
SCALE: 1 : 400,000
MILES
HERBERT HOOVER DIKE
MAJOR REHABILITATION STUDY
FLOODED AREA MAP
BREAK IN REACH 6C
LAKE LEVEL 21 FT
REACH 7
REACH 5
REACH 8
REACH 6B
REACH 6A
REACH 6C

Figure 3

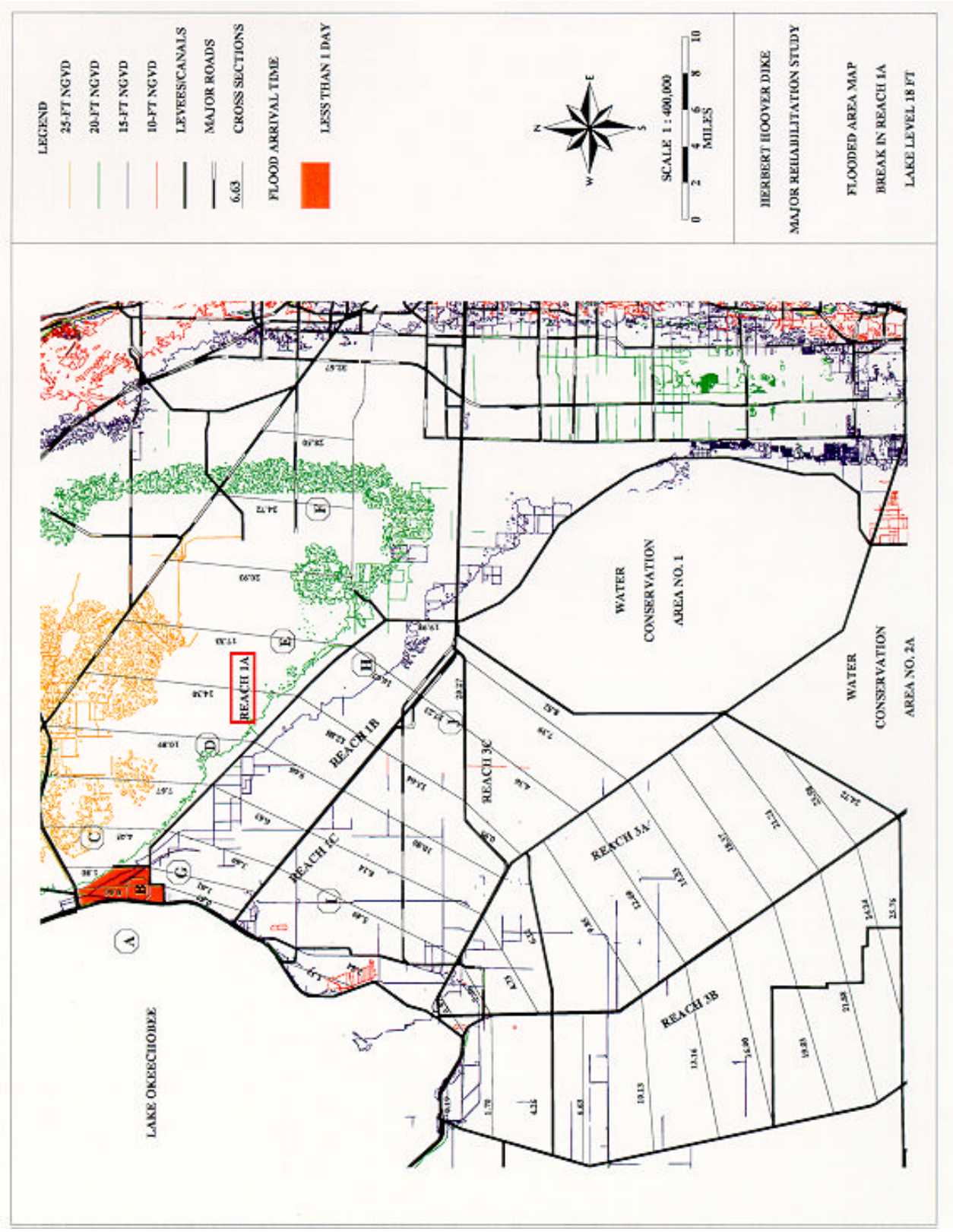
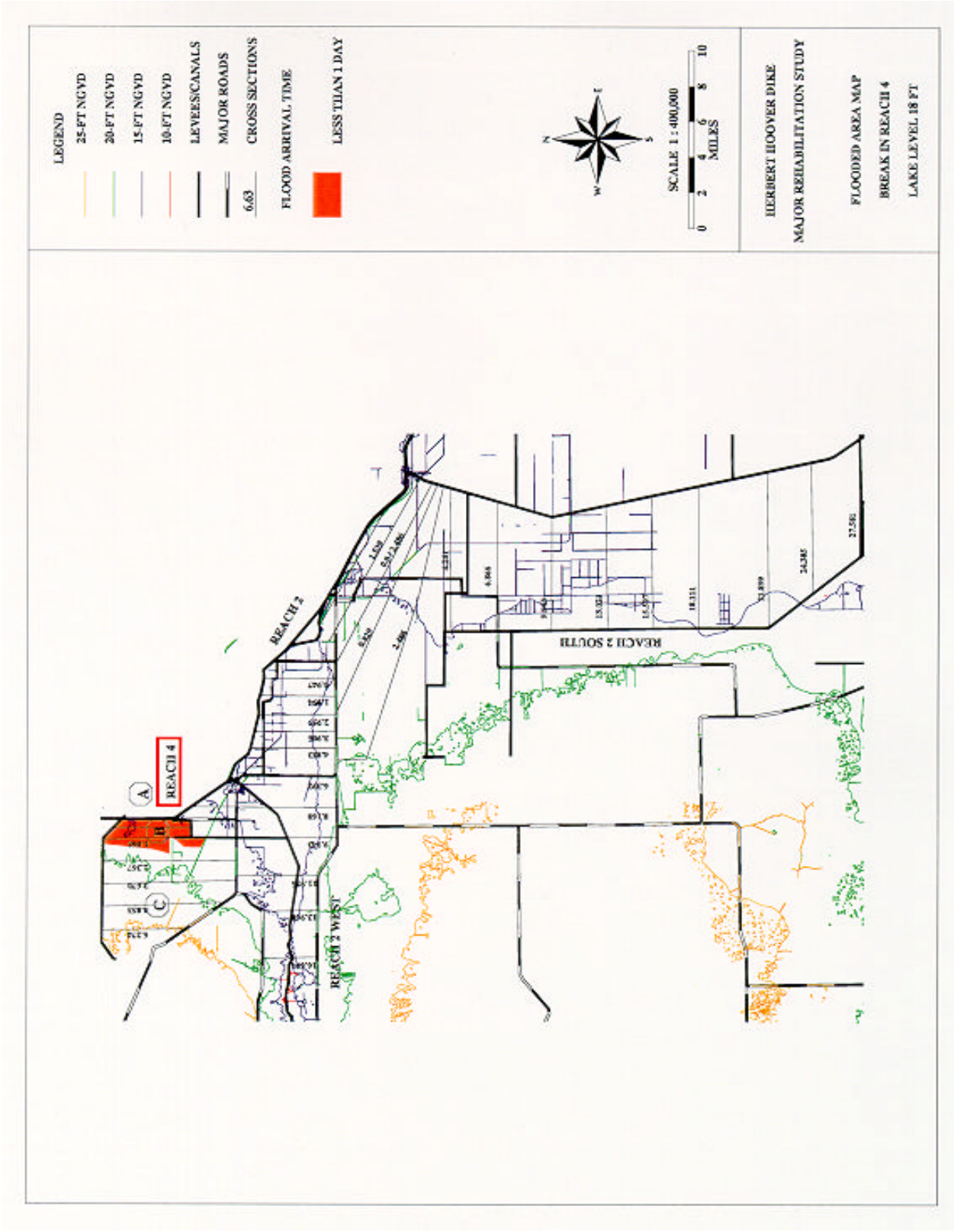

LEGEND
25-FT NGVD
20-FT NGVD
15-FT NGVD
10-FT NGVD
LEVEES/CANALS
MAJOR ROADS
CROSS SECTIONS
FLOOD ARRIVAL TIME
LESS THAN 1 DAY
SCALE 1 : 400,000
MILES
HERBERT HOOVER DIKE
MAJOR REHABILITATION STUDY
FLOODED AREA MAP
BREAK IN REACH 1A
LAKE LEVEL 18 FT
LAKE OKEECHOBEE
WATER CONSERVATION AREA NO. 1
WATER CONSERVATION AREA NO. 2A
REACH 1A
REACH 1B
REACH 1C
REACH 3A
REACH 3B
REACH 3C

Figure 4


LEGEND
25-FT NGVD
20-FT NGVD
15-FT NGVD
10-FT NGVD
LEVEES/CANALS
MAJOR ROADS
6.63
CROSS SECTIONS
FLOOD ARRIVAL TIME
LESS THAN 1 DAY
N
W
E
S
SCALE 1 : 400,000
0 2 4 6 8 10
MILES
HERBERT HOOVER DIKE
MAJOR REHABILITATION STUDY
FLOODED AREA MAP
BREAK IN REACH 4
LAKE LEVEL 18 FT
REACH 4
A
B
C
REACH 2
REACH 2 WEST
REACH 2 SOUTH

In addition, levee breaches in any of these cells can affect adjoining damage cells if lake stages are high enough. If any of the eleven cells are breached, the affected damage cells are as follows:

1. Break in damage cell 6B:

    a. Effects in damage cell 6B
    b. Effects in damage cell 6A
    c. Effects in damage cell 6C
    d. Effects in damage cell 8

2. Break in damage cell 6C:
    a. Effects in damage cell 6A
    b. Effects in damage cell 6B
    c. Effects in damage cell 6C
    d. Effects in damage cell 8

3. Break in damage cell 7 (Effects in damage cell 7 only)

4. Break in damage cell 3
    a. Effects in damage cell 3B
    b. Effects in damage cell 1C
    c. Effects in damage cell 1B
    d. Effects in damage cell 3A
    e. Effects in damage cell 3C

5. Break in damage cell 4
    a. Effects in damage cell 4
    b. Effects in damage cell 2

6. Break in damage cell 5 - (Effects in damage cell 5 only)

7. Break in damage cell 6A
    a. Effects in damage cell 6A
    b. Effects in damage cell 6B
    c. Effects in damage cell 6C
    d. Effects in damage cell 8

8. Break in damage cell 1A
    a. Effects in damage cell 1A
    b. Effects in damage cell 1C
    c. Effects in damage cell 1B

9. Break in damage cell 1B
    a. Effects in damage cell 1B
    b. Effects in damage cell 3C

c. Effects in damage cell 1C
d. Effects in damage cell 3B
e. Effects in damage cell 3A

10. Break in damage cell 1C
    a. Effects in damage cell 1C
    b. Effects in damage cell 1B
    c. Effects in damage cell 3B
    d. Effects in damage cell 3A
    e. Effects in damage cell 3C

11. Break in damage cell 2 - Effects in damage cell 2 only

In order to provide stage-damage information for these damage cells, flood and topographic elevations were applied using detailed hydrologic cross-sections. Different procedures have been utilized for agriculture and urban land use types. The urban area portion of the study area extends approximately 25 miles from the lake. For this area, 59 cross sectional areas have been set up in an urban databank to apply flood elevations. Unlike urban areas, agricultural areas extend throughout the basin. 139 cross sections were utilized to apply flood elevations to agricultural development. Topographic elevations for urban were applied utilizing nineteen topographic mylar overlays at a scale of 1”=600’ or 1:7,200 created from 1:24,000 topographic maps. Topographic elevations for agriculture were applied directly from 1:24,000 topographic maps for each condition.

Determination of flood stages

Peak stages for the high and low velocity condition were obtained by reviewing stage hydrographs for each cross-sectional area. As discussed previously, 59 cross sectional areas were utilized to apply flood elevations in urban areas and 139 cross sections were utilized to apply flood elevations to agricultural development over the entire basin. Separate sets of hydrographs were analyzed for each conditional break scenario for each of three different lake stage events. These events relate to the 18 foot, 21 foot and 26 foot lake levels.

For each of these conditions, low and high velocity condition peak stages were obtained by reviewing the hydrographs to determine what stages would be after 10 days and 45 days of inundation respectively. Peak stage tables for each condition were created. For agricultural parcels, stage information was only used to determine which parcels were damage susceptible.

Damage to agricultural activities is a function of the duration of flooding as discussed below.

Determination of flood duration

Stage-hydrographs were prepared for the 139 cross-sectional areas listed above for the 18 foot, 21 foot, and 26 foot lake stage conditions. From this information, it was possible to associate a peak stage with a high velocity breach with expected repair in 45 days, and a low velocity condition breach with expected repair in 10 days for each of these lake level conditions. However, these hydrographs were truncated at 45 days of flooding, usually after a hydrologic equilibrium condition was reached. Since water would have to be pumped out of the basin, there was no good formal estimate of how long the water would be on the ground. After review of all the hydrographs, it was observed that water would be on the ground for at least 22 days under the smallest event regardless of the topographic elevation of the affected land use. Twenty-two days was then used for all flood damage estimates. This is considered reasonable for urban damage estimates since urban damage due to duration is primarily to lawns, pavement, shrubs and streets and is usually minimal. This is considered reasonable for the agricultural damage estimates since the primary damage susceptible crop in the basin is sugar cane and the Natural Resource Conservation Service, NRCS, indicates sugarcane is dead after 22 days. Pasture would have greater damage susceptibility if flood duration were greater than 22 days. However damage susceptibility for pasture is relatively small when compared to crop susceptibility and although under estimated would have relatively little effect upon the overall totals.

Determination of topographic elevations

Topographic elevations for urban and agricultural areas were created from 1:24,000 topographic maps with an interpolated 2-foot contour interval. In the urban areas, the appropriate maps were then resized to overlay the existing land use maps. Nineteen topographic mylar overlays at a scale of 1:7,200 were used. Topographic elevations were then encoded for each structural record in the urban database and associated with each agricultural parcel in the agricultural database.

Urban Land use determination

Major towns include Pahokee, Belle Glade, and South Bay on the Southeast side of the lake, Clewiston and Moore Haven on the

southwest side and Okeechobee on the north side. Residential development is approximately 94% of the total structures in the basin. The portion of residential that is single family residential is approximately 50% of the total structures. Residential structures in the area are approximately 2,000 square feet, and are relatively old. Approximately 95% of the residential structures in the area utilize wood exterior walls and only 5% utilize masonry. Average structural value for residential is less than $30,000. Values are positively skewed especially in the southern areas around the lake indicating there are more structures less than $30,000 than the opposite. The only place this does not hold true is Palm Beach where values are much higher.

In the numerous RV parks in the area, there are often mixtures of smaller camper trailers, large camper trailers, and small, older trailers (as would be called mobile homes) and any variation of these may have been made to be seemingly permanent. Such additions as skirting, flower beds, entrance porches, decks, or even screened or glassed rooms had been added to some of these trailers. Some of these trailers are on plots that are under long term leases within what would otherwise be a transient campground. It was decided that any trailer, that appeared to have sufficient additions such that it would not be easily possible to hitch a tow vehicle to the trailer and remove it from the site on very short notice, would be inventoried as a permanent residential structure. Most neighborhoods would have a mix of several styles, sizes, ages and conditions of structures. There is also little clustering of commercial activities such as gas stations or fast food restaurants.

A detailed urban land use investigation was conducted by Gulf Engineers and Consultants out of Baton Rouge, Louisiana. The contractor utilized TIGER/Line files prepared by the Department of Commerce, Bureau of Census, to produce a basin map of the entire area, four general area maps at a scale of 1:75,000 and 16 local area maps scale 1:7,200. The four general maps covered the entire study area and the local maps covered various areas that were anticipated to contain concentrations of structures such as in and around the municipalities of Okeechobee, Port Mayaca, Canal Point, Pahokee, Belle Glade, South Bay, Lake Harbor, Clewiston, Moore Haven, and Lakeport. Information on both sets of these maps included the 25 foot contour boundary, general reach delineation, census tract delineation, latitude-longitude in 1 minute increments, streets, water bodies, railroads, townships, county boundaries and other physical features.

Urban Structure Value and First Floor Elevations

The contractor then conducted field investigations to record land use types, count the number of structures, estimate structure value, and record first floor elevations. Each group of structures were recorded on the basemaps and cross-tabulated in a database. Each database record includes a long list of attributes including streets, towns, type of structure, name, foundation etc. Samples of the residential form attributes for 33 structures are shown in Table 1. Samples of the commercial form attributes are shown in Table 2.

Structure value was determined using the replacement less depreciation method. To make this determination, the contractor utilized the Marshall & Swift, (M&S) evaluation methodology. These valuations are keyed on the postal zip code of the location of the structure being calculated. To assure that the values were reasonable, a consistency or calibration check was performed. Local sources of comparative real estate values were consulted for current market values and construction costs within the study area. Realtors in Clewiston, Belle Glade, and Okeechobee were contacted. They were each asked to supply the addresses of higher, middle, and lower price range homes that they currently or recently listed. They were asked for the selling price of the property, the separate land value of the property, and, if available, the relation of the sales price to appraised value. The contractor used the information supplied by these local sources to refine the estimates of values made by use of the valuation software. These properties selected by the Realtors were surveyed by contractor staff for M&S characteristics. The GEC staff had no control of which properties the Realtors submitted and the Realtors had no knowledge of the values that the contractor would be estimating or of the method the contractor was using. The sales price less the land value was compared to the depreciated replacement cost as estimated with the M&S software. While there was a wide range in variation, the cumulative difference indicated the M&S values were within approximately 4 percent of the local information.

The residential forms have the following data fields for coordination and identification of the forms with the geographic locations of the data entered on them as well as providing a means of correlating all of the completed forms from the inventory field effort with the base maps.

Attribute Description

1. Sequence Number - A unique number used for structure location

2. Structure ID - A coded number that indicates the county, spatial area number and form and location number.

3. County - A two-letter abbreviation for the county (GL - Glades, HY - Hendry, MT - Martin, KC - Okeechobee, PB - Palm Beach)

4. Damage Cell - Hydrologic damage cell from Working Map (1-A,

1-B, 1-C, 2, 3-A, 3-B, 4, 5, 6-A, 6-B, 6-C, 7, 8, and S-308

5. Sub-Reach - Working Reach Number for Flood Damage Evaluation

6. Form Number Field form number

7. Date of Survey - Date of observations noted on each form

8. Name of Surveyor - Name of survey team member entering information on form

9. Line Number - There are a possible 9 data records per survey form

10. Multiplier - Number of structures for which the line entry is a typical representative

11. Street Number - Street number (if available) of the representative structure

12. Street Name - Street name (if available) of the representative structure

13. Street Suffix - (ST, AVE, DR, etc.)

14. Town - Name of town that structure is in or near

15. MapG - The label number of the General Area Map on which the representative structure location is marked

16. MapL - The label number of the Local Area Map on which the representative structure location is marked

17. Topography - Pre-subsidence topography encoded from quadrangle sheets (not used)

18. Topography - Adjusted topographic elevations for subsidence

19. HAGL - Estimated Height Above Ground Level of first floor of structure

20. Tract - Census Tract

21. Latitude - (Noted as a 6-digit number, i.e., Lat 26o35'02N" would be entered as 263502 since all locations in

the study area are in the Northern Hemisphere)

22. Longitude - (Noted as a 6-digit number, i.e., Long 80o20'15"W would be entered as 802015 since all locations in the study area are in the Western Hemisphere)

23. Township - (As found on quadrangle maps)

24. Range - As found on quadrangle maps)

25. Section - As found on quadrangle maps)

26. SPA - Structures Per Acre as estimated by survey personnel

27. FLUCCS - Florida Land Use and Cover Classification System Number

The following information is required by the Marshall-Swift Methodology to Compute Structure Value as Replacement less Depreciation:

28. Land Use - Numerical land use classifications system used for flood damage computations

29. Type - Structure Type (1=Single Family; 2=Low-rise apts; 3=Town House End Unit; 4=Town House Inside Unit; 5=Duplex) Used for Marshall – Swift Assessment.

30. Zipcode - Postal Zipcode

31. Qual - Structure Quality (1=Low; 2=Fair; 3=Average; 4=Good; 5=Very Good; 6=Excellent)

32. Finished Sqft - Finished floor square footage

33. Duplicate - Same as 33.

34. Age - Effective Age (50 years minus the number of years the structure would continue to be livable with the apparent present level of maintenance)

35. Cond - Structure Condition (1=Worn Out; 2=Badly Worn; 3=Average; 4=Good; 5=Very Good; 6=Excellent)

36. Style - Structure Style (1=One Story; 2=Two Story; 3=Three Story; 4=Split-Level; 5= 1-1/2 Story Finished; 11=Bi-Level)

37. H/C - Heating and Cooling (Used Heated and Cooled Air for all)

38. ExtWall - Exterior Wall Construction (Wood Frame: 1=Plywood; 2=Hardboard Sheets; 3=Stucco; 4=Siding; 5=Shingle; 6=Masonry Veneer; Masonry: 7=Common Brick; 8=Face Brick; 9=Stone; 10=Concrete Block)

39. Roof - Roof Construction (1=Composition Shingle; 2=Built-Up Rock; 3=Wood Shingle; 4=Wood Shake; 5=Concrete Tile; 6=Clay Tile; 7=Galvanized Metal; 8=Slate; 9=Composition Roll; 10=Plastic Tile)

40. GarType - Type of Garage/Carport (1=Built-in; 2=Attached; 3=Extra or Detached; 9=None)

41. GarBays - Number of Garage or Carport Bays (1=Single; 2=Double; etc; 9=None)

42. Fndn - Foundation (1=Pier; 2=Slab; 3=Masonry Pier)

43. FP(#) - Number of Fireplaces

44. Val - Depreciated Replacement Value to be calculated with M&S

45 Adj. Val - Adjusted value of Residence based upon sales information

46. CSVR - Not used

47. Cont_Val - Not used.

Urban Content Values

Content value was applied as a percentage of structure value. These values were computed using District information for residential structures and information developed by the Galveston District for commercial composite classifications. Residential contents are estimated to be 40% of the value of the structure in this analysis. Past experience has indicated that the value of residential personal (contents), can be considered to be a function of the value of the structure. Interviews with insurance agents in Florida have indicated that personal property value can vary between 30% and 50% of the value of the structure depending upon the structural value. Fifty percent is the default value of coverage used in

most insurance policies. Content value for very expensive homes can exceed 50% due to the addition of selected items not usually found in average homes, which would include antiques and works of art.

Damage susceptible land use classifications other than residential encountered in the study would include various commercial and institutional and industrial activities. Normally the content as a percent of structure ratio applicable to these establishments would have been analyzed by an evaluation of each activity. However, due to the number of structures involved with each activity, this was not possible. Composite content as a percent of structure ratios utilized for commercial and institutional land use classifications were developed using information provided by the Galveston District. The ratios used in the study are as closely related as possible to the individual types of commercial and institutional land use types that are located in the basin. Composite content as a percent of structure ratios utilized for sugar mills and other industrial processing were determined using phone interviews.

Urban Damage Relationships

The single-family residential damage relationships for structural damage and content damage were estimated using two sources. First, actual claims information from the Federal Emergency Management Agency (FEMA) for the State of Florida during the period 1978 to 1996 for slab on grade homes was used. Second, estimates provided by GEC for wooden structures were also used. The composite relationship was weighted based upon the percentage of homes in the basin with each construction type. Mobile home and multi-family damage relationships for structural damage and content damage were also estimated using actual claims information from the Federal Emergency Management Agency (FEMA) for the State of Florida during the period 1978 to 1996. Mixed unit residential is a combination of single family residential and mobile homes in the area. A weighted composite relationship was prepared for this classification based upon the average percentage of each use in the area. Depth-damage relationships for all residential structures and contents have been expressed as a percentage of the value of the structure or content. Residential structure design in the study area is generally homogeneous; single family homes are one story, and multi-family dwellings are generally two stories. In this study, flood depths only effect the first floor of the

structure.

Damage susceptible land use classifications other than residential encountered in the area include various classifications of commercial, industrial and institutional development. Normally the damage to these establishments would have been analyzed by an evaluation of each activity. However, due to the number of structures involved with each activity, this was not possible. The depth-damage relationships utilized for commercial and institutional land use classifications were developed by the Galveston District. The relationships used in the study are as closely related as possible to the individual types of commercial and industrial land use types that are located in the basin.

Two additional classifications in the report are sugar cane processing and other industrial. Types of structure used in the other industrial classification has been checked using the urban database and a composite structure damage relationship was created. A damage relationship for sugar contents was developed with the help of Okeelanta and the Sugarcane Growers Co-operative of Florida.

In addition to damage to structure and content, damage to lawns, pavement, shrubs, and residential streets are calculated for residential development. Estimates of damages to lawns, pavement, shrubs, and streets are based on information obtained from local horticulturists, professional landscaping experts, county agents, and city, county, and state engineers.

Summary

When completed the information discussed in the previous paragraphs was encoded for the computer software Access, and Dbase. The contractors database results were checked for reasonableness by computing minimum and maximum ranges, means, standard deviations and skews for first floor elevations and structural values relating to each land use type in each relevant spatial area. Structure counts were verified using census tract housing counts as a control. The contractor made several trips to the field to verify that the number of structures was correct. Final counts include approximately 22,000 residential structures and 2,100 commercial structures.

After the inventory was completed, the Florida Land Use Cover Classification System was used to determine detailed

land use types from the database. Twenty-two residential types and seventy-two commercial types were identified in the basin. Six classifications of residential and nineteen classifications of commercial were identified after accumulating the detailed types of residential and commercial data in the database. This land use classification list including urban and agricultural land use types is shown in table 3.

TABLE 3
Herbert Hoover Dike Land Use Classification System

Number	Land Use Classification
1	Single Family Residential – Low
2	Single Family Residential – Medium
3	Single Family Residential – High
4	Mobile Homes
5	Mixed Unit Residential
6	Multi-Family Residential
7	Sales and Service – General
8	Shopping Center
9	Service Stations
10	Banks
11	Convenience Stores
12	Restaurants
13	Warehouses
14	Professional Services
15	Culture and Entertainment
16	Hotels and Motels
17	Industrial – Sugar Processing
18	Industrial – Other
19	Educational
20	Religious Activities
21	Health Care
22	Government Services
23	Recreation
24	Transportation, Communication
25	Miscellaneous
26	Small Vegetables
27	Beans
28	Potatoes
29	Sweet Corn
30	Grain Corn
31	Cucumbers
32	Rice
33	Sugar Cane

34	Grapes
35	Melons
36	Tropical Fruit
37	Strawberries
38	Citrus
39	Avocados
40	Pasture
41	Sod
42	Alfalfa
43	Sorghum
44	Nursery
45	Tomatoes
46	Peppers
47	Unspecified

Agricultural Land Use Determination

Agricultural land use was prepared using permit records for agricultural consumptive water use from the South Florida Water Management District. Data included in the Permit record includes the Permit Number, Ownership, Crops Grown, Irrigated Acreage, Water Allocations, and Number of Plantings and Dates. Although the permit data is 10 years old, it is updated upon sale or transfer of property or other modification. The data do not reflect modifications made by the existing owner since the permit was recorded. Also, the data only includes irrigated agricultural use. For parts of the Everglades Agricultural Area (EAA), we have supplemented this information with information for non-irrigated acreage from telephone conversations with landowners. To record this information, agricultural basemaps were prepared using GIS techniques. The basemaps included physical features, the location and the identification number of the permitted areas, damage cells, and cross-sectional information to apply flood elevations. After review of the information, 22 agricultural land uses (26-47) were identified including mainly citrus, pasture, truck crops, and sugarcane. This information is shown in Table 3.

Over 60% of the irrigated agricultural acreage in the basin and over 80% of the acreage in the EAA is in sugar cane. Up to 4 multiple annual harvests (or ratoons) can be expected from each planting. Discussions with small growers indicate they plant sweet corn after the last harvest of sugar cane, or approximately once every 4 years. Therefore, all sugar cane indicated for small growers is expected to be 75% sugar cane and 25% sweet corn in any given year. Larger holders that

include US Sugar and Okeelanta intensively grow sugar cane and minimize the use of other crops. For these producers 92% of the annual crop is classified as sugar cane and 8% as sweet corn or rice.

Agricultural Damage Relationships

Losses to vegetable and fruit crops are very much dependent upon the duration of flooding in root zones. Put simply, crop damage is caused by the crops inability to breathe and the associated diseases which occur due to the growth of micro-organisms when aerobic activity is replaced by anaerobic activity due to lack of oxygen and CO2. Discussions with knowledgeable sources have indicated that damage begins immediately after water enters the root zone and continues until mortality occurs. The beginning elevation of damage susceptibility is calculated as the existing topographical elevation minus the length of the root zone plus the typical bedding height. Expected losses during time frames between the initial flooding and total plant mortality are computed using a linear relationship. The actual period when losses will occur is also dependent upon whether or not water in the root zones is moving or stagnant during the flood and whether root zones are continuously or intermittently flooded

Most of the agricultural damage relationships were computed for this study by the Natural Resource Conservation Service (NRCS) and damage is measured as the loss of yield less any fixed costs involved in the production of the crop. The information utilized to produce a damage relationship for Spring Sweet Corn in January-March is shown in Table 4. When this information was not available, the Jacksonville District developed a conservative estimate of the value of vegetables in the ground was approximated by using operating costs and land rental values less fixed costs, to produce the crop. Tree and Fruit Crops have damage susceptibility year around. Row and truck crops are seasonal in nature, therefore at different times of the year, the crops will have different damage susceptibility. To account for this, four sets of seasonal damage relationships were prepared for each truck crop classification. Since there are only a limited amount of multiple plantings during the year in the basin, only one land use scenario was used which simplified the evaluation.

To capture the differences in damage susceptibility, quarterly seasons were selected. These seasons are December-February, March-May, June-August, and September-November.

This basis of the selection was to accommodate a quarterly time step which also is designed to coincide with the hurricane season for modeling purposes.

Table 4

FLOOD DAMAGE ANALYSIS STUDY

EVERGLADES AGRICULTURAL AREA

FL-001-001: Spring Sweet Corn

BASIC INPUTS TABLE

TIME	OPERATION MACHINERY/INPUT	COSTS				
		FIXED	VARIABLE			TOTAL
		OWNERSHIP	OPERATING	LABOR	INPUT	
1/3/96	Land & Capital	$ -	$ 155.79	$ -	$ -	$ 155.79
1/3/96	Tractor 210 Hp	$ 0.88	$ 1.17	$ 0.11	$ -	$ 2.16
	Disc 24 Ft	$ 0.41	$ 0.03	$ 0.08	$ -	$ 0.52
1/4/96	Tractor 210 Hp	$ 1.76	$ 2.35	$ 0.22	$ -	$ 4.33
	Chisel Plow 12 Ft	$ 0.55	$ 0.16	$ 0.06	$ -	$ 0.77
1/5/96	Tractor 210 Hp	$ 0.88	$ 1.17	$ 0.11	$ -	$ 2.16
	Disc 24 Ft	$ 0.41	$ 0.03	$ 0.08	$ -	$ 0.52
1/6/96	Tractor 210 Hp	$ 1.50	$ 2.00	$ 0.19	$ -	$ 3.69
	Leveler 20 Ft	$ 0.42	$ 0.09	$ 0.14	$ -	$ 0.65
1/8/96	Tractor 80 Hp	$ 0.46	$ 0.66	$ 0.16	$ -	$ 1.28
	Broadcast Spreader 20	$ 0.20	$ 0.12	$ 0.12	$ -	$ 0.44
	0-15-23	$ -	$ -	$ -	$ 72.00	$ 72.00
1/10/96	Tractor 210 Hp	$ 0.88	$ 1.17	$ 0.11	$ -	$ 2.16
	Disc 24 Ft	$ 0.41	$ 0.03	$ 0.08	$ -	$ 0.52
1/12/96	Tractor 210 Hp	$ 2.41	$ 3.21	$ 0.30	$ -	$ 5.92
	Disc 24 Ft	$ 1.12	$ 0.09	$ 0.22	$ -	$ 1.43
	Culltivator 12 Ft	$ 0.88	$ 0.01	$ 0.22	$ -	$ 1.11
1/15/96	Tractor 100 Hp	$ 1.28	$ 1.61	$ 0.32	$ -	$ 3.21
	Planter 12 Ft	$ 2.96	$ 0.10	$ 0.23	$ -	$ 3.29
	Sweet Corn Seed	$ -	$ -	$ -	$ 67.76	$ 67.76
	5-15-5	$ -	$ -	$ -	$ 10.50	$ 10.50
	Phorate	$ -	$ -	$ -	$ 8.61	$ 8.61
1/15/96	Rat bait	$ -	$ -	$ -	$ 2.50	$ 2.50
	Parathion	$ -	$ -	$ -	$ 10.75	$ 10.75
	Custom Application	$ -	$ -	$ -	$ 2.00	$ 2.00
1/16/96	Tractor 70 Hp	$ 0.10	$ 0.13	$ 0.04	$ -	$ 0.27
	Cross Ditcher 20 Ft	$ 0.18	$ 0.02	$ 0.03	$ -	$ 0.23
1/16/96	Tractor 100 Hp	$ 2.37	$ 2.97	$ 0.58	$ -	$ 5.92
	Herbicide Sprayer 2R	$ 0.84	$ 0.44	$ 0.42	$ -	$ 1.70
	Atrazine	$ -	$ -	$ -	$ 3.50	$ 3.50

Table 4 (continued)

FLOOD DAMAGE ANALYSIS STUDY

EVERGLADES AGRICULTURAL AREA

FL-001-001: Spring Sweet Corn

BASIC INPUTS TABLE

TIME	OPERATION MACHINERY/INPUT	COSTS				
		FIXED	VARIABLE			TOTAL
		OWNERSHIP	OPERATING	LABOR	INPUT	
2/14/96	Tractor 70 Hp	$ 0.83	$ 1.08	$ 0.30	$ -	$ 2.21
	Granular Hopper	$ 0.14	$ 0.04	$ 0.22	$ -	$ 0.40
	Cultivator 12 Ft	$ 0.88	$ 0.01	$ 0.22	$ -	$ 1.11
	Methomyl 5G	$ -	$ -	$ -	$ 7.40	$ 7.40
2/15/96	Tractor 70 Hp	$ 0.63	$ 0.83	$ 0.23	$ -	$ 1.69
	Fertilizer Distributor 3R	$ 1.50	$ 1.26	$ 0.17	$ -	$ 2.93
	Tech Mangam	$ -	$ -	$ -	$ 4.48	$ 4.48
2/15/96	Tractor 70 Hp	$ 0.83	$ 1.08	$ 0.30	$ -	$ 2.21
	Granular Hopper	$ 0.14	$ 0.04	$ 0.22	$ -	$ 0.40
	Cultivator 12 Ft	$ 0.88	$ 0.01	$ 0.22	$ -	$ 1.11
	Methomyl 5G	$ -	$ -	$ -	$ 7.40	$ 7.40
2/15/96	Tractor 70 Hp	$ 0.83	$ 1.08	$ 0.30	$ -	$ 2.21
	Granular Hopper	$ 0.14	$ 0.04	$ 0.22	$ -	$ 0.40
	Cultivator 12 Ft	$ 0.88	$ 0.01	$ 0.22	$ -	$ 1.11
	Methomyl 5G	$ -	$ -	$ -	$ 7.40	$ 7.40
3/1/96	Custom Scouting	$ -	$ -	$ -	$ 4.00	$ 4.00
3/5/96	Manzate 200	$ -	$ -	$ -	$ 3.67	$ 3.67
	Custom Spraying	$ -	$ -	$ -	$ 5.00	$ 5.00
	Lannate L	$ -	$ -	$ -	$ 4.25	$ 4.25
3/11/96	Custom Scouting	$ -	$ -	$ -	$ 4.00	$ 4.00
3/12/96	Manzate 200	$ -	$ -	$ -	$ 3.67	$ 3.67
	Custom Spraying	$ -	$ -	$ -	$ 5.00	$ 5.00
	Lannate L	$ -	$ -	$ -	$ 4.25	$ 4.25
3/18/96	Custom Scouting	$ -	$ -	$ -	$ 4.00	$ 4.00
3/19/96	Manzate 200	$ -	$ -	$ -	$ 3.67	$ 3.67
	Custom Spraying	$ -	$ -	$ -	$ 4.25	$ 4.25
	Lannate L	$ -	$ -	$ -	$ 5.00	$ 5.00
3/25/96	Custom Scouting	$ -	$ -	$ -	$ 4.00	$ 4.00
3/26/96	Manzate 200	$ -	$ -	$ -	$ 3.67	$ 3.67
	Custom Spraying	$ -	$ -	$ -	$ 4.25	$ 4.25
	Lannate L	$ -	$ -	$ -	$ 5.00	$ 5.00
4/2/96	Custom Scouting	$ -	$ -	$ -	$ 4.00	$ 4.00
4/3/96	Manzate 200	$ -	$ -	$ -	$ 3.67	$ 3.67
	Lannate L	$ -	$ -	$ -	$ 4.25	$ 4.25
	Custom Spraying	$ -	$ -	$ -	$ 5.00	$ 5.00
	Lorsban 4E	$ -	$ -	$ -	$ 4.09	$ 4.09
4/9/96	Custom Scouting	$ -	$ -	$ -	$ 4.00	$ 4.00
4/10/96	Manzate 200	$ -	$ -	$ -	$ 3.67	$ 3.67
	Lannate L	$ -	$ -	$ -	$ 4.25	$ 4.25
	Custom Spraying	$ -	$ -	$ -	$ 5.00	$ 5.00
	Lorsban 4E	$ -	$ -	$ -	$ 4.09	$ 4.09

Table 4 (continued)

FLOOD DAMAGE ANALYSIS STUDY						
EVERGLADES AGRICULTURAL AREA						
FL-001-001: Spring Sweet Corn						
BASIC INPUTS TABLE						
		COSTS				
TIME	OPERATION MACHINERY/INPUT	FIXED	VARIABLE			TOTAL
		OWNERSHIP	OPERATING	LABOR	INPUT	
4/15/96	Combine	$ 21.84	$ 3.23	$ 0.49	$ -	$ 25.56
	Combine Head-Corn 6R	$ 5.71	$ 0.18	$ 0.35	$ -	$ 6.24
	Truck 2T	$ 7.87	$ 8.84	$ 0.49	$ -	$ 17.20
	Crates	$ -	$ -	$ -	$ 380.25	$ 380.25
	P-H-P	$ -	$ -	$ -	$ 390.00	$ 390.00
4/30/96	Crop Drying	$ -	$ -	$ -	$ -	$ -
5/1/96		$ -	$ -	$ -	$ -	$ -
5/15/96		$ -	$ -	$ -	$ -	$ -
5/30/96		$ -	$ -	$ -	$ -	$ -
6/1/96		$ -	$ -	$ -	$ -	$ -

Table 4 (continued)

BASIC INPUTS TABLE						
		COSTS				
TIME	OPERATION MACHINERY/INPUT	FIXED	VARIABLE			TOTAL
		OWNERSHIP	OPERATING	LABOR	INPUT	
Jan	Total Cost Summary	$ 20.90	$ 173.35	$ 3.82	$ 177.62	$ 375.69
Feb	Total Cost Summary	$ 7.68	$ 5.48	$ 2.62	$ 26.68	$ 42.46
Mar	Total Cost Summary	$ -	$ -	$ -	$ 67.68	$ 67.68
Apr	Total Cost Summary	$ 35.42	$ 12.25	$ 1.33	$ 812.27	$ 861.27
May	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Jun	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Jul	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Aug	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Sep	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Oct	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Nov	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Dec	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Jan	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Feb	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Mar	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Apr	Total Cost Summary	$ -	$ -	$ -	$ -	$ -
Crop Budget Totals		$ 64.00	$ 191.08	$ 7.77	$ 1,084.25	$ 1,347.10

Table 4 (continued)

FLOOD DAMAGE ANALYSIS STUDY
EVERGLADES AGRICULTURAL AREA
FL-001-001: Spring Sweet Corn

MONTH OF YEAR	FLOOD DURATION 0 Hours	12 Hours	24 Hours	36 Hours	48+ Hours
Jan	0%	100%	100%	100%	100%
Feb	0%	50%	100%	100%	100%
Mar	0%	25%	50%	75%	100%
Apr	0%	100%	100%	100%	100%
May	0%	0%	0%	0%	0%
Jun	0%	0%	0%	0%	0%
Jul	0%	0%	0%	0%	0%
Aug	0%	0%	0%	0%	0%
Sep	0%	0%	0%	0%	0%
Oct	0%	0%	0%	0%	0%
Nov	0%	0%	0%	0%	0%
Dec	0%	0%	0%	0%	0%
Jan	0%	0%	0%	0%	0%
Feb	0%	0%	0%	0%	0%
Mar	0%	0%	0%	0%	0%
Apr	0%	0%	0%	0%	0%

AS crop becomes more mature it is less damage susceptible. Based upon percentages of early, midseason late season planters and growing seasons.

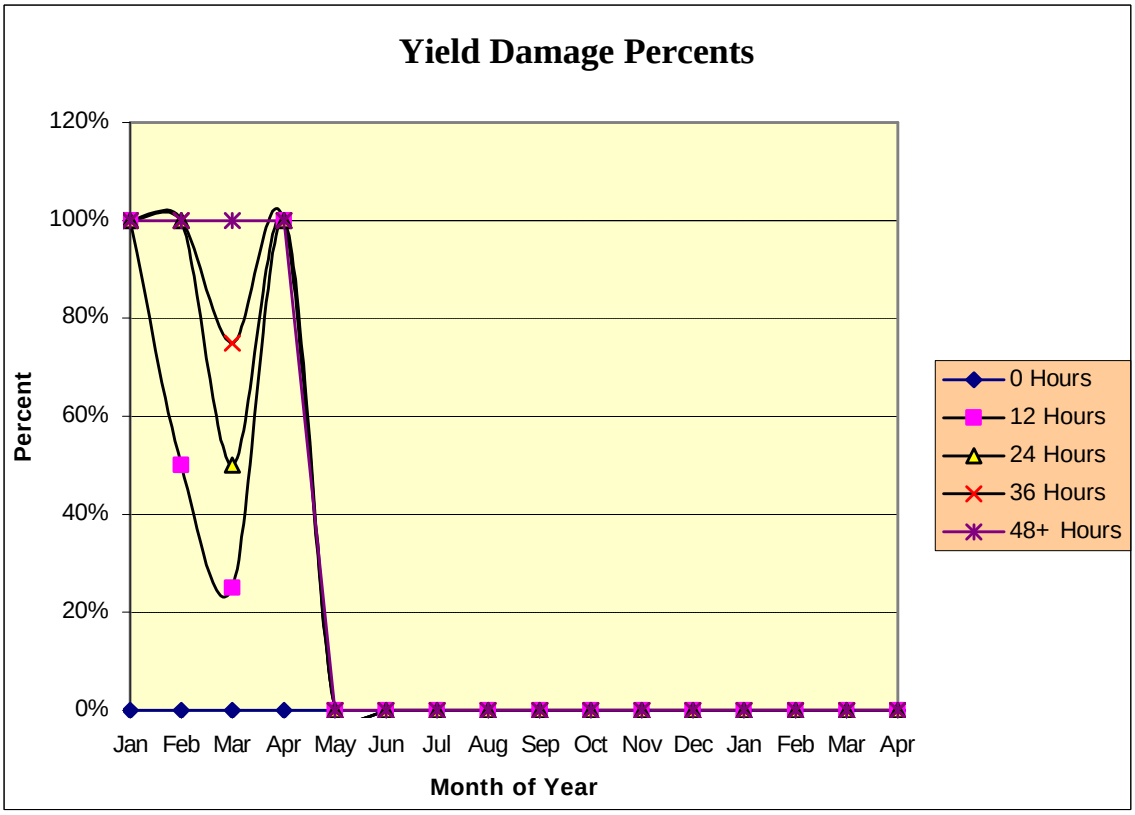


Table 4 (continued)

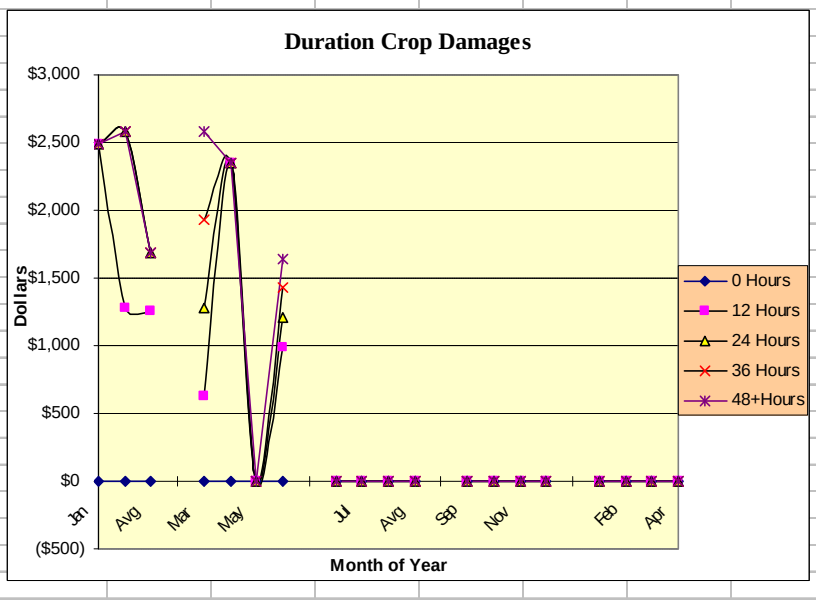
Flood Event Duration		12 Hours								
Month	Monthly Fixed Ownership Costs	Monthly Variable Operating Costs	Monthly Variable Labor Costs	Monthly Variable Input Costs	Monthly Production Costs	Monthly Production Costs	Monthly Costs	Monthly Compiled Total Costs	Monthly Duration Yield Damage Percent	Duration Yield Damage minus Associate Fixed Costs
Jan	$ 20.90	$ 173.35	$ 3.82	$ 177.62	$ 375.69	$ 94.01	$ 469.70	$ 469.70	100%	$ 2,485.09
Feb	$ 7.68	$ 5.48	$ 2.62	$ 26.68	$ 42.46	$ 9.22	$ 51.68	$ 521.37	50%	$ 1,283.10
Mar	$ -	$ -	$ -	$ 67.68	$ 67.68	$ 17.93	$ 85.61	$ 606.99	25%	$ 632.07
Apr	$ 35.42	$ 12.25	$ 1.33	$ 812.27	$ 861.27	$ 218.82	$ 1,080.09	$ 1,687.08	100%	$ 2,345.76
May	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Jun	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Jul	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Aug	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Sep	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Oct	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Nov	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Dec	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Jan	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Feb	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Mar	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Apr	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -

Flood Event Duration		24 Hours								
Month	Monthly Fixed Ownership Costs	Monthly Variable Operating Costs	Monthly Variable Labor Costs	Monthly Variable Input Costs	Monthly Production Costs	Monthly Production Costs	Monthly Costs	Monthly Compiled Total Costs	Monthly Duration Yield Damage Percent	Duration Yield Damage minus Associate Fixed Costs
Jan	$ 20.90	$ 173.35	$ 3.82	$ 177.62	$ 375.69	$ 94.01	$ 469.70	$ 469.70	100%	$ 2,485.09
Feb	$ 7.68	$ 5.48	$ 2.62	$ 26.68	$ 42.46	$ 9.22	$ 51.68	$ 521.37	100%	$ 2,583.10
Mar	$ -	$ -	$ -	$ 67.68	$ 67.68	$ 17.93	$ 85.61	$ 606.99	50%	$ 1,282.07
Apr	$ 35.42	$ 12.25	$ 1.33	$ 812.27	$ 861.27	$ 218.82	$ 1,080.09	$ 1,687.08	100%	$ 2,345.76
May	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Jun	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Jul	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Aug	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Sep	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Oct	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Nov	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Dec	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Jan	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Feb	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Mar	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -
Apr	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	0%	$ -

Table 4 (continued)

FLOOD DAMAGE ANALYSIS STUDY
EVERGLADES AGRICULTURAL AREA
FL-001-001: Spring Sweet Corn

MONTH OF YEAR	FLOOD DURATION				
	0 Hours	12 Hours	24 Hours	36 Hours	48+Hours
Dec	$0	$0	$0	$0	$0
Jan	$0	$2,485	$2,485	$2,485	$2,485
Feb	$0	$1,283	$2,583	$2,583	$2,583
Avg	$0	$1,256	$1,689	$1,689	$1,689
Mar	$0	$632	$1,282	$1,932	$2,582
Apr	$0	$2,346	$2,346	$2,346	$2,346
May	$0	$0	$0	$0	$0
Avg	$0	$993	$1,209	$1,426	$1,643
Jun	$0	$0	$0	$0	$0
Jul	$0	$0	$0	$0	$0
Aug	$0	$0	$0	$0	$0
Avg	$0	$0	$0	$0	$0
Sep	$0	$0	$0	$0	$0
Oct	$0	$0	$0	$0	$0
Nov	$0	$0	$0	$0	$0
Avg	$0	$0	$0	$0	$0
Jan	$0	$0	$0	$0	$0
Feb	$0	$0	$0	$0	$0
Mar	$0	$0	$0	$0	$0
Apr	$0	$0	$0	$0	$0



EVALUATION OF FLOOD DAMAGE POTENTIAL

Urban damage susceptibility:

The effects of various breach scenarios were computed using the existing land use in the study area. The procedure included cataloguing the type of land use, topographic elevation, first floor elevation, and value for each structure and its personal property value. For each condition, the specific type of land use inundated by each condition was then determined and the depth of water each type is exposed to was estimated. Damage to structure and contents is computed from depth-damage relationships prepared in the Jacksonville District on a per structure basis. Damage to lawns, pavement, shrubs and streets associated with residential structures are primarily a function of the duration of flooding. For each breach scenario, the 18 foot, 21 foot, and 26 foot lake stage elevations were evaluated. The 59 urban cross-sectional areas used for applying flood elevations were reduced to 20 where flood stages were similar and the flood damages were computed for each subarea. In addition, flood damage was computed for each of the 25 urban land use types used in the study.

Agricultural damage susceptibility:

The effects of various breach scenarios were computed using the existing land use in the study area. Composite maps were prepared using GIS techniques which included the major sub-basin boundaries, water permit application numbers for identification and corresponding spatial areas, cross-sectional areas for applying flood elevations, and other physical features. Each entry in the urban database is uniquely numbered for spatial identification. A portion of the agricultural database is shown in Table 5. A typical database record for agriculture is as follows:

1. Unique entry number
2. Water Permit Number
3. General Damage Cell Designation – Corresponds to the major damage cells in the study.
4. Model Area Number –Because the agricultural evaluation was completed after the urban evaluation was designed, unique sequential numbers were used to correspond to the model damage cells. Agricultural areas 21-35 correspond to the major damage cells in the study but are used for the purpose of agricultural damage calculation.
5. Cross-section location – Used to assign flood elevations

for each condition, numbered 1 – 139. Flood elevations are used only to determine which agricultural parcels are flooded for each condition. Twenty-two days of duration was used to determine flood damage.

6. Land use name and number.
7. Topography.
8. Root zone and bedding adjustment to topography – Used to determine the elevation of the bottom of the root zone if necessary.
9. Topography adjusted for subsidence.
10. Actual acreage in production.

For each breach scenario, the 18 foot, 21 foot, and 26 foot lake stage elevations were evaluated. As with urban, flood damages for agricultural parcels were computed for each subarea and computed for the remaining 22 agricultural classifications of land use.

Table 5

Entry Number	Permit Number	General Reach	HHD Model Reach	Bolles Model Reach	HHD Cross-Section Locator Counter	Land Use Name	Land Use Number	Topo	Root zone & bedding	Adjusted Topo	Irrigated Acreage in Production
5000	50-00375-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	3,836.53
5001	50-00375-W	3B	27	5	67	SUGAR CANE	33	10.50	0.00	10.50	6,128.48
5002	50-00289-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	320.00
5003	50-00276-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	284.90
5004	50-01591-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	233.00
5005	50-00163-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	221.00
5006	50-00116-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	252.00
5007	50-00239-W	3B	27	1	67	SUGAR CANE	33	10.50	0.00	10.50	295.00
5008	50-00312-W	3B	27	3	67	SUGAR CANE	33	10.50	0.00	10.50	453.56
5009	50-00656-W	3B	27	7	67	SUGAR CANE	33	10.50	0.00	10.50	2,176.00
5010	50-00656-W	3B	27	7	67	RICE	32	10.50	0.00	10.50	384.00
5011	50-00656-W	3B	27	7	68	SUGAR CANE	33	10.50	0.00	10.50	2,176.00
5012	50-00656-W	3B	27	7	68	RICE	32	10.50	0.00	10.50	384.00
5013	50-00656-W	3B	27	3	67	SUGAR CANE	33	10.50	0.00	10.50	1,640.93
5014	50-00656-W	3B	27	3	67	RICE	32	10.50	0.00	10.50	289.58
5015	50-00656-W	3B	27	3	68	SUGAR CANE	33	10.50	0.00	10.50	1,640.93
5016	50-00656-W	3B	27	3	68	RICE	32	10.50	0.00	10.50	289.58
5017	50-01590-W	3B	27	3	70	SUGAR CANE	33	10.50	0.00	10.50	294.40
5018	50-01590-W	3B	27	3	71	SUGAR CANE	33	10.50	0.00	10.50	294.40
5019	50-01590-W	3B	27	3	70	SWEET CORN	29	10.50	0.17	10.33	25.60
5020	50-01590-W	3B	27	3	71	SWEET CORN	29	10.50	0.17	10.33	25.60
5021	50-00238-W	3B	27	3	70	SUGAR CANE	33	10.50	0.00	10.50	286.12
5022	50-00238-W	3B	27	3	70	SWEET CORN	29	10.50	0.17	10.33	24.88
5023	EAA Permit	3B	27	3	71	SUGAR CANE	33	10.50	0.00	10.50	1,760.00
5024	EAA Permit	3B	27	3	70	SUGAR CANE	33	10.50	0.00	10.50	1,760.00
5025	50-00071-W	3B	27	3	71	SUGAR CANE	33	10.50	0.00	10.50	743.36
5026	50-00071-W	3B	27	3	71	SWEET CORN	29	10.50	0.17	10.33	64.64
5027	50-00175-W	3B	27	3	70	SUGAR CANE	33	10.50	0.00	10.50	246.10
5028	50-00175-W	3B	27	3	70	SWEET CORN	29	10.50	0.17	10.33	21.40
5029	50-00175-W	3B	27	3	70	SOD	41	10.50	0.00	10.50	322.50
5030	50-00175-W	3B	27	3	71	SUGAR CANE	33	10.50	0.00	10.50	246.10
5031	50-00175-W	3B	27	3	71	SWEET CORN	29	10.50	0.17	10.33	21.40
5032	50-00175-W	3B	27	3	71	SOD	41	10.50	0.00	10.50	322.50
5033	50-00263-W	3B	27	3	70	SUGAR CANE	33	10.50	0.00	10.50	207.45
5034	50-00263-W	3B	27	3	70	SWEET CORN	29	10.50	0.17	10.33	69.15
5035	50-00184-W	3B	27	3	71	SUGAR CANE	33	10.50	0.00	10.50	1,177.60
5036	50-00184-W	3B	27	3	71	SWEET CORN	29	10.50	0.17	10.33	102.40
5037	50-00070-W	3A	26	2	55	SUGAR CANE	33	10.50	0.00	10.50	382.50
5038	50-00070-W	3A	26	2	55	SWEET CORN	29	10.50	0.17	10.33	127.50
5039	50-00240-W	3B	27	1	65	SUGAR CANE	33	10.50	0.00	10.50	164.31
5040	50-00240-W	3B	27	1	65	SWEET CORN	29	10.50	0.17	10.33	54.77
5041	50-00240-W	3B	27	1	66	SUGAR CANE	33	10.50	0.00	10.50	54.77
5042	50-00240-W	3B	27	1	66	SWEET CORN	29	10.50	0.17	10.33	18.26
5043	50-00217-W	3B	27	1	65	SUGAR CANE	33	10.50	0.00	10.50	68.08
5044	50-00217-W	3B	27	1	65	SWEET CORN	29	10.50	0.17	10.33	5.92

Data Reduction:

In addition to the conditions mentioned above, agricultural and urban damage susceptibility was computed twice, once for a high velocity condition breach and once for a low velocity condition breach. Given the amount of data involved, computer programs were written to reduce the data included in the computer output files and the resulting data were imported into a Lotus spreadsheet. An example of this part of the spreadsheet is shown in Tables 6 and 7. In this example, the row and truck crops are for one quarter only. The reduction process included 1) aggregating model areas and associating the sums with the actual damage cells used in the study and 2) aggregating land use classifications to correspond to the four major classifications used in the risk model. The results of this process are again reduced to produce stage-damage relationships as shown in Table 8. Table 8 reflects four entries for the row and truck crops (one for each quarter). The entire effort required an evaluation of 35 separate spatial areas affected by 11 different breach scenarios. For each of these areas, residential urban, non-residential urban, tree and fruit crops, and quarterly row and truck crop stage-damage relationships were created. In addition, the process was repeated for a high and low velocity condition. Table 8 is an example of the low velocity condition and Table 9 an example of the high velocity condition. Four hundred and ninety stage-damage relationships were created to evaluate damage to development (35 (spatial areas) X 7 (generalized land use classifications) X 2 (high and low velocity condition)). An additional 70 stage-damage relationships were created to evaluate other benefits as discussed below.

Table 6

			Break in Reach 1A 26 Feet			
Reaches	Structure	Conten	LPSS	Total Urban	Agriculture	Tota Urban and Ac
1	37,611,798.99	18,401,259.38	4,877,514.33	60,890,572.70	0.00	60,890,572.
2	1,109,140.25	2,499,716.06	357,534.61	3,966,390.92	0.00	3,966,390.9
3	10,630,969.16	6,193,184.28	3,781,879.42	20,606,032.86	0.00	20,606,032.8
4	24,241,976.18	42,676,014.95	4,671,349.87	71,589,341.00	0.00	71,589,341.0
5	0.00	0.00	0.00	0.00	0.00	0.00
6	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00	0.00	0.00
8	0.00	0.00	0.00	0.00	0.00	0.00
9	0.00	0.00	0.00	0.00	0.00	0.00
10	0.00	0.00	0.00	0.00	0.00	0.00
11	0.00	0.00	0.00	0.00	0.00	0.00
12	0.00	0.00	0.00	0.00	0.00	0.00
13	0.00	0.00	0.00	0.00	0.00	0.00
14	0.00	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00
16	0.00	0.00	0.00	0.00	0.00	0.00
17	0.00	0.00	0.00	0.00	0.00	0.00
18	0.00	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00
20	0.00	0.00	0.00	0.00	0.00	0.00
21	0.00	0.00	0.00	0.00	0.00	0.00
22	0.00	0.00	0.00	0.00	39,345,308.29	39,345,308.
23	0.00	0.00	0.00	0.00	53,426,370.56	53,426,370.
24	0.00	0.00	0.00	0.00	69,955,886.91	69,955,886.
25	0.00	0.00	0.00	0.00	0.00	0.00
26	0.00	0.00	0.00	0.00	0.00	0.00
27	0.00	0.00	0.00	0.00	0.00	0.00
28	0.00	0.00	0.00	0.00	0.00	0.00
29	0.00	0.00	0.00	0.00	0.00	0.00
30	0.00	0.00	0.00	0.00	0.00	0.00
31	0.00	0.00	0.00	0.00	0.00	0.00
32	0.00	0.00	0.00	0.00	0.00	0.00
33	0.00	0.00	0.00	0.00	0.00	0.00
34	0.00	0.00	0.00	0.00	0.00	0.00
35	0.00	0.00	0.00	0.00	0.00	0.00
Sum	73,593,884.58	69,770,174.67	13,688,278.23	157,052,337.48	162,727,565.76	319,779,903.24

Table 7

			Break in Reach 1A							
			26 Feet							
	Urban	Urban	Agriculture	Urban	Agriculture	Urban	Agriculture			
Model Reaches	3	4	24	1	22	2	23	Total	Total	Total
Actual Reaches	1C	1C	1C	1A	1A	1B	1B	1C	1A	1B
Single Family Residential - Low	284,240.64	1,648,257.79	0.00	44,318,505.69	0.00	70,630.54	0.00	1,932,498.43	44,318,505.69	70,630.54
Single Family Residential - Medium	3,069,019.88	13,897,340.24	0.00	14,591,145.38	0.00	546,901.35	0.00	16,966,360.12	14,591,145.38	546,901.35
Single Family Residential - High	10,156,922.14	11,341,323.98	0.00	0.00	0.00	1,052,097.75	0.00	21,498,246.12	0.00	1,052,097.75
Mobile Homes	2,095,543.82	1,025,701.77	0.00	1,980,921.64	0.00	71,220.23	0.00	3,121,245.59	1,980,921.64	71,220.23
Mixed Unit Residential	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Multi-Family Residential	2,498,663.84	1,003,358.32	0.00	0.00	0.00	0.00	0.00	3,502,022.16	0.00	0.00
Sales and Service - General	694,620.74	329,268.31	0.00	0.00	0.00	0.00	0.00	1,023,889.05	0.00	0.00
Shopping Centers	6,247.09	328,649.54	0.00	0.00	0.00	0.00	0.00	334,896.63	0.00	0.00
Service Stations	10,651.34	34,884.54	0.00	0.00	0.00	0.00	0.00	45,535.88	0.00	0.00
Banks	48,091.00	0.00	0.00	0.00	0.00	0.00	0.00	48,091.00	0.00	0.00
Convenience Stores	47,975.98	32,430,667.51	0.00	0.00	0.00	0.00	0.00	32,478,643.49	0.00	0.00
Resturants	128,931.85	21,434.10	0.00	0.00	0.00	0.00	0.00	150,365.95	0.00	0.00
Warehouses	23,350.36	230,460.60	0.00	0.00	0.00	0.00	0.00	253,810.96	0.00	0.00
Professional Services	72,472.15	212,157.24	0.00	0.00	0.00	0.00	0.00	284,629.39	0.00	0.00
Culture and Entertainment	1,944.97	6,365.47	0.00	0.00	0.00	0.00	0.00	8,310.44	0.00	0.00
Hotels and Motels	6,870.92	0.00	0.00	0.00	0.00	0.00	0.00	6,870.92	0.00	0.00
Industrial - Sugar Processing	0.00	2,764,129.74	0.00	0.00	0.00	2,225,541.05	0.00	2,764,129.74	0.00	2,225,541.05
Industrial - Other	92,758.28	4,900,675.41	0.00	0.00	0.00	0.00	0.00	4,993,433.69	0.00	0.00
Educational	651,955.06	588,018.90	0.00	0.00	0.00	0.00	0.00	1,239,973.96	0.00	0.00
Religious Activities	333,136.01	505,396.61	0.00	0.00	0.00	0.00	0.00	838,532.62	0.00	0.00
Health Care	0.00	321,250.92	0.00	0.00	0.00	0.00	0.00	321,250.92	0.00	0.00
Government Services	382,636.78	0.00	0.00	0.00	0.00	0.00	0.00	382,636.78	0.00	0.00
Recreation	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Transportation, Communication	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Miscellaneous	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Small Vegetables	0.00	0.00	625,232.77	0.00	192,261.00	0.00	4,846,515.25	625,232.77	192,261.00	4,846,515.25
Beans	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Potatoes	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Sweet Corn	0.00	0.00	23,860,502.83	0.00	1,088,138.25	0.00	15,155,650.28	23,860,502.83	1,088,138.25	15,155,650.28
Grain Corn	0.00	0.00	128,952.00	0.00	0.00	0.00	0.00	128,952.00	0.00	0.00
Cucumbers	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Rice	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Sugar Cane	0.00	0.00	34,697,524.61	0.00	1,563,594.75	0.00	32,109,897.84	34,697,524.61	1,563,594.75	32,109,897.84
Grapes	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Melons	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Tropical Fruit	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Strawberries	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Citrus	0.00	0.00	0.00	0.00	36,076,114.29	0.00	0.00	0.00	36,076,114.29	0.00
Avocados	0.00	0.00	210,750.00	0.00	0.00	0.00	0.00	210,750.00	0.00	0.00
Pasture	0.00	0.00	4,423.90	0.00	0.00	0.00	0.00	4,423.90	0.00	0.00
Sod	0.00	0.00	10,428,500.81	0.00	0.00	0.00	1,314,307.19	10,428,500.81	0.00	1,314,307.19
Alfalfa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Sorghum	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Nursery	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Tomatoes	0.00	0.00	0.00	0.00	425,200.00	0.00	0.00	0.00	425,200.00	0.00
Peppers	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Unspecified	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Sum:	20,606,032.85	71,589,340.99	69,955,886.92	60,890,572.71	39,345,308.29	3,966,390.92	53,426,370.56	162,151,260.76	100,235,881.00	57,392,761.48
Aggregated Land Use Classes:										
Residential Urban	18,104,390.32	28,915,982.10	0.00	60,890,572.71	0.00	1,740,849.87	0.00	47,020,372.42	60,890,572.71	1,740,849.87
Non-Residential Urban	2,501,642.53	42,673,358.89	0.00	0.00	0.00	2,225,541.05	0.00	45,175,001.42	0.00	2,225,541.05
Sub-total Urban	20,606,032.85	71,589,340.99	0.00	60,890,572.71	0.00	3,966,390.92	0.00	92,195,373.84	60,890,572.71	3,966,390.92
Tree Fruit Crops	0.00	0.00	210,750.00	0.00	36,076,114.29	0.00	0.00	210,750.00	36,076,114.29	0.00
Row + Truck Crops and Agriculture	0.00	0.00	69,745,136.92	0.00	3,269,194.00	0.00	53,426,370.56	69,745,136.92	3,269,194.00	53,426,370.56
Sub-total Agriculture	0.00	0.00	69,955,886.92	0.00	39,345,308.29	0.00	53,426,370.56	69,955,886.92	39,345,308.29	53,426,370.56
Total Urban and Agriculture	20,606,032.85	71,589,340.99	69,955,886.92	60,890,572.71	39,345,308.29	3,966,390.92	53,426,370.56	162,151,260.76	100,235,881.00	57,392,761.48

Table 8. Sample Low Velocity Spreadsheet

	1C	1C	1C	1A	1A	1A	1B	1B	1B						
	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet						
Residential Urban	0.00	0.00	5,795,955.01	3,165,569.93	10,266,175.44	60,890,572.71	0.00	0.00	1,740,849.87						
Non-Residential Urban	0.00	0.00	40,540.09	0.00	0.00	0.00	0.00	0.00	2,225,541.05						
Tree Fruit Crops	0.00	0.00	0.00	4,514,152.11	4,514,152.11	36,076,114.29	0.00	0.00	0.00						
Row + Truck Crops and Pasture	0.00	0.00	47,488,827.43	1,020,768.00	1,234,004.25	3,269,194.00	0.00	0.00	53,426,370.56						
Row + Truck Crops and Pasture	0.00	0.00	60,610,361.82	1,310,680.00	1,518,108.75	3,404,861.25	0.00	0.00	64,249,658.36						
Row + Truck Crops and Pasture	0.00	0.00	42,292,724.60	903,960.00	903,960.00	2,987,652.26	0.00	0.00	54,851,196.19						
Row + Truck Crops and Pasture	0.00	0.00	42,507,379.06	904,704.00	904,704.00	4,222,646.02	0.00	0.00	64,867,174.59						
Emergency Costs	0.00	0.00	2,433,960.90	142,067.52	625,272.48	6,075,656.71	0.00	0.00	430,580.67	0.00	0.00	0.00	0.00	0.00	0.00
Affected People	0	0	2,020	488	2,146	4,458	0	0	403						
Pumping Cost			1,199,220	227,000	250,000	1,344,580			1,090,200						
Total "Other"	0	0	3,633,181	369,068	875,272	7,420,237	0	0	1,520,781	0					
Days flooded			84	14	14	84			84						
Break in Reach 1B	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total
	1C	1C	1C	3B	3B	3B	1B	1B	1B	3A	3A	3A	3C	3C	3C
	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet
Residential Urban	0.00	0.00	52,319,175.13	0.00	0.00	0.00	926,125.43	1,799,851.49	2,173,313.15	0.00	0.00	0.00	0.00	0.00	0.00
Non-Residential Urban	0.00	0.00	54,648,587.63	0.00	0.00	0.00	1,401,862.76	2,285,865.85	2,466,598.73	0.00	0.00	0.00	0.00	0.00	0.00
Tree Fruit Crops	0.00	0.00	210,750.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	0.00	12,241,910.31	69,745,136.92	0.00	0.00	0.00	45,502,704.56	53,426,370.56	53,649,774.60	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	0.00	15,653,457.40	85,812,677.55	0.00	0.00	0.00	53,896,250.50	64,249,658.36	64,466,977.98	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	0.00	11,854,583.18	63,397,612.13	0.00	0.00	0.00	45,096,305.25	54,851,196.19	54,851,196.19	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	0.00	12,044,093.80	65,046,908.35	0.00	0.00	0.00	53,917,791.57	64,867,174.59	64,867,174.59	0.00	0.00	0.00	0.00	0.00	0.00
Emergency Costs	0.00	442,538.34	14,306,321.57	0.00	0.00	0.00	67,150.08	430,580.67	430,580.67	0.00	0.00	0.00	0.00	0.00	0.00
Affected People	0	367	11,871	0	0	0	403	403	403	0	0	0	0	0	0
Pumping Cost		1,444,780	817,740			1,408,330	909,000	1,281,220	726,880			1,181,180			408,870
Total "Other"	0	1,887,318	15,124,062	0	0	1,408,330	976,150	1,711,801	1,157,461	0	0	1,181,180	0	0	408,870
Days flooded		84	140			140	8	84	140			140			140
Break in Reach 1C	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total
	1C	1C	1C	3B	3B	3B	3A	3A	3A	1B	1B	1B	3C	3C	3C
	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet
Residential Urban	25,220,009.48	47,149,512.44	62,562,207.87	0.00	0.00	0.00	0.00	0.00	38,965,600.25	0.00	0.00	830,503.32	0.00	0.00	0.00
Non-Residential Urban	2,284,479.36	32,816,157.18	59,711,668.32	0.00	0.00	0.00	0.00	0.00	23,677,934.07	0.00	0.00	1,302,266.90	0.00	0.00	0.00
Tree Fruit Crops	210,750.00	210,750.00	210,750.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	64,422,882.32	69,745,136.92	69,745,136.92	0.00	0.00	0.00	0.00	0.00	13,151,970.23	0.00	0.00	40,769,905.57	0.00	0.00	35,975,128.13
Row + Truck Crops and Pasture	79,851,169.63	85,812,677.55	85,812,677.55	0.00	0.00	0.00	0.00	0.00	14,403,147.50	0.00	0.00	47,567,541.24	0.00	0.00	32,390,551.46
Row + Truck Crops	59,087,558.60	63,397,612.13	63,397,612.13	0.00	0.00	0.00	0.00	0.00	13,750,697.72	0.00	0.00	40,188,859.44	0.00	0.00	34,632,019.62
Row + Truck Crops and Pasture	59,701,714.21	65,046,908.35	65,046,908.35	0.00	0.00	0.00	0.00	0.00	18,568,260.67	0.00	0.00	49,006,306.71	0.00	0.00	56,772,762.38
Emergency Costs	5,147,755.20	14,059,660.86	14,843,171.37	0.00	0.00	0.00	0.00	0.00	11,785,690.88	0.00	0.00	430,580.67	0.00	0.00	0.00
Affected People	8,834	11,667	12,317	0	0	0	0	0	11,468	0	0	403	0	0	0
Pumping Cost	909,000	1,008,620	817,740			1,408,330		545,200	1,181,180		681,500	726,880		490,680	408,870
Total "Other"	6,056,755	15,068,281	15,660,911	0	0	1,408,330	0	545,200	12,966,871	0	681,500	1,157,461	0	490,680	408,870
Days flooded	28	84	140			140		84	140		84	140		140	140

Table 9

Break in Reach 1A	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total						
	1C	1C	1C	1A	1A	1A	1B	1B	1B	1B						
	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	26 feet						
Residential Urban	0.00	0.00	47,020,372.42	3,165,569.93	10,266,175.44	60,890,572.71	0.00	0.00	1,740,849.87	1,740,849.87						
Non-Residential Urban	0.00	0.00	45,175,001.42	0.00	0.00	0.00	0.00	0.00	2,225,541.05	2,225,541.05						
Tree Fruit Crops	0.00	0.00	210,750.00	4,514,152.11	4,514,152.11	36,076,114.29	0.00	0.00	0.00	0.00						
Row + Truck Crops and Pasture	0.00	0.00	69,745,136.92	1,020,768.00	1,234,004.25	3,269,194.00	0.00	0.00	53,426,370.56	53,426,370.56						
Row + Truck Crops and Pasture	0.00	0.00	85,812,677.55	1,310,680.00	1,518,108.75	3,404,861.25	0.00	0.00	64,249,658.36	64,249,658.36						
Row + Truck Crops and Pasture	0.00	0.00	63,397,612.13	903,960.00	903,960.00	2,987,652.26	0.00	0.00	54,851,196.19	54,851,196.19						
Row + Truck Crops and Pasture	0.00	0.00	65,046,908.35	904,704.00	904,704.00	4,222,646.02	0.00	0.00	64,867,174.59	64,867,174.59						
Emergency Cost	0.00	0.00	14,164,854.40	142,067.52	625,272.48	6,075,656.71	0.00	0.00	430,580.67	430,580.67	0.00	0.00	0.00	0.00	0.00	0.00
Affected People	0	0	11,754	488	2,146	4,458	0	0	403	403						
Pumping Cost			1,199,220	227,000	250,000	1,344,580			1,090,200	1,090,200						
Total "Other"	0	0	15,364,074	369,068	875,272	7,420,237	0	0	1,520,781	1,520,781						
Days flooded			84	14	14	84			84	84						
Break in Reach 1B	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total
	1C	1C	1C	3B	3B	3B	1B	1B	1B	1B	3A	3A	3A	3C	3C	3C
	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet
Residential Urban	0.00	47,964,325.84	52,895,017.53	0.00	0.00	7,610,227.26	1,207,393.38	1,819,835.09	2,173,313.15	2,173,313.15	0.00	0.00	47,481,478.04	0.00	0.00	0.00
Non-Residential Urban	0.00	47,152,107.90	55,855,590.21	0.00	0.00	4,813,547.44	1,645,275.24	2,313,011.85	2,466,598.73	2,466,598.73	0.00	0.00	33,045,046.95	0.00	0.00	0.00
Tree Fruit Crops	0.00	210,750.00	210,750.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	0.00	69,745,136.92	69,745,136.92	0.00	0.00	118,192,460.31	53,426,370.56	53,426,370.56	53,649,774.60	53,649,774.60	0.00	0.00	139,334,970.12	0.00	0.00	59,132,920.17
Row + Truck Crops and Pasture	0.00	85,812,677.55	85,812,677.55	0.00	0.00	144,048,762.16	64,249,658.36	64,249,658.36	64,466,977.98	64,466,977.98	0.00	0.00	158,164,633.94	0.00	0.00	54,922,475.11
Row + Truck Crops and Pasture	0.00	63,397,612.13	63,397,612.13	0.00	0.00	137,721,225.17	54,851,196.19	54,851,196.19	54,851,196.19	54,851,196.19	0.00	0.00	143,010,238.68	0.00	0.00	57,490,354.52
Row + Truck Crops and Pasture	0.00	65,046,908.35	65,046,908.35	0.00	0.00	136,574,855.87	64,867,174.59	64,867,174.59	64,867,174.59	64,867,174.59	0.00	0.00	168,236,243.69	0.00	0.00	84,683,689.97
Emergency Costs	0.00	14,164,854.40	14,306,321.57	0.00	0.00	3,643,812.36	67,150.08	430,580.67	430,580.67	430,580.67	0.00	0.00	11,785,690.88	0.00	0.00	0.00
Affected People	0	11,754	11,871	0	0	3,052	403	403	403	403	0	0	11,468	0	0	0
Pumping Cost		1,444,780	817,740			1,408,330	909,000	1,281,220	726,880	726,880			1,181,180			408,870
Total "Other"	0	15,609,634	15,124,062	0	0	5,052,142	976,150	1,711,801	1,157,461	1,157,461	0	0	12,966,871	0	0	408,870
Days flooded		84	140			140	8	84	140	140			140			140
Break in Reach 1C	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total	Total
	1C	1C	1C	3B	3B	3B	3A	3A	3A	3A	1B	1B	1B	3C	3C	3C
	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet	26 feet	18 feet	21 feet	26 feet	18 feet	21 feet	26 feet
Residential Urban	38,010,267.65	53,492,775.98	62,653,780.74	0.00	0.00	7,610,227.26	0.00	29,618,773.20	47,481,478.04	47,481,478.04	0.00	240,150.66	1,752,641.18	0.00	0.00	0.00
Non-Residential Urban	26,125,319.93	53,372,330.89	60,075,381.27	0.00	0.00	4,796,342.22	0.00	10,434,909.41	33,045,046.95	33,045,046.95	0.00	982,438.64	2,237,606.22	0.00	0.00	0.00
Tree Fruit Crops	210,750.00	210,750.00	210,750.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Row + Truck Crops and Pasture	69,745,136.92	69,745,136.92	69,745,136.92	0.00	0.00	118,192,460.31	0.00	139,334,970.12	139,334,970.12	139,334,970.12	0.00	36,701,416.57	53,426,370.56	0.00	50,782,155.61	59,132,920.17
Row + Truck Crops and Pasture	85,812,677.55	85,812,677.55	85,812,677.55	0.00	0.00	144,048,762.16	0.00	158,164,633.94	158,164,633.94	158,164,633.94	0.00	42,173,745.24	64,249,658.36	0.00	47,490,535.53	54,922,475.11
Row + Truck Crops	63,397,612.13	63,397,612.13	63,397,612.13	0.00	0.00	137,721,225.17	0.00	143,010,238.68	143,010,238.68	143,010,238.68	0.00	36,102,814.44	54,851,196.19	0.00	49,516,906.78	57,490,354.52
Row + Truck Crops and Pasture	65,046,908.35	65,046,908.35	65,046,908.35	0.00	0.00	136,574,855.87	0.00	168,236,243.69	168,236,243.69	168,236,243.69	0.00	44,916,898.71	64,867,174.59	0.00	71,684,739.55	84,683,689.97
Emergency Costs	6,466,703.04	14,306,321.57	14,843,171.37	0.00	0.00	3,643,812.36	0.00	11,785,690.88	11,785,690.88	11,785,690.88	0.00	173,517.58	430,580.67	0.00	0.00	0.00
Affected People	11,098	11,871	12,317	0	0	3,052	0	11,468	11,468	11,468	0	163	403	0	0	0
Pumping Cost	909,000	1,008,620	817,740			1,408,330		545,200	1,181,180	1,181,180		681,500	726,880		490,680	408,870
Total "Other"	7,375,703	15,314,942	15,660,911	0	0	5,052,142	0	12,330,891	12,966,871	12,966,871	0	855,018	1,157,461	0	490,680	408,870
Days flooded	28	84	140			140		84	140	140		84	140		140	140

Emergency Response Costs

After a flood Federal, State, and local governments and relief organizations provide disaster assistance. This type of assistance includes provision of food, clothing, and shelter to those displaced by the flood; monetary aid to displaced victims for food, travel, and temporary lodging; cleanup and removal of debris; provision of essential services and utilities; and monetary assistance to homeowners for essential repairs of their homes. The estimates used for the emergency costs for this analysis are based on documented expenditures from the De Soto County Emergency Management office. An El Nino related storm from November 1997 to April 1998 affected De Sota County, Florida. The resulting flood forced 200 families along the Peace River and Horse Creek areas from their homes. The Federal Emergency Management Agency provides several types of housing assistance to flood victims. The intent of the assistance is to enable households affected by the disaster to address their disaster-related housing needs.

For this analysis emergency costs associated with the 18 foot, 21 foot, and 26 foot lake stage elevations for each breach scenario were evaluated. In the De Soto County flooding 200 families were impacted which resulted in 73 applications for assistance. The following monies were expended on these floods.

$56,100	housing assistance
$36,650	individual grants
$ 2,100	Red Cross assistance
$94,850	

According to “TH TRACKING REPORTS” from the Federal Emergency Management Agency (FEMA) for disaster no. 1195 had a 73.37% approval rate. Assuming 73 applications from Desoto County flooding 73 * 73.37% = 53 applications were accepted. The Desota County Emergency Management office estimated that the average displacement was for 17.5 days.

$56,100	housing assistance
$ 2,100	Red Cross assistance

$58,200 / 53 = $1,098.11 / 17.5 days = $62.75

The $62.75 was used for each household flooded per estimated days flooded. FEMA has guidelines on rental assistance where eligible applicants who were homeowners prior

to the disaster will receive an initial assistance of up to three month's rent. Eligible applicants who were renting prior to the disaster will receive an initial assistance of one month's rent. The number of days used to compute emergency cost was limited to 90 days for homeowners and 30 days for renters. The 1990 Census of Population and Housing – Summary Tape file 3A provides the owner occupied and rented occupied counts for all census tracts. Census track boundaries and flood reaches were overlaid to determine an upper limit of the weighted number of days to compute emergency costs. Engineering Division estimated the maximum number of days flooding for each scenario.

The Individual and Family Grant money is authorized for six categories of need.

1. Housing
   - To repair, replace, or rebuild the primary owner-occupied residence.
   - To provide access to such residence.
   - To clean or make sanitary.
- To remove debris that is a safety hazard to the residence.
- To provide protective measures to protect the residence from immediate threat.

2. Personal Property
- To repair or replace household items, furnishings, appliances.
- To clean or replace clothing
- To repair or replace tools or specialized clothing and equipment required by an employer.
- To move and store to prevent or reduce damage.

3. Transportation
- To repair, replace, or provide privately owned vehicles or to provide public transportation.

4. Medical and dental expenses
5. Funeral expenses
6. Cost of the first year's flood insurance premium, if required.

The Individual and Family Grant (IFG) monies expended for the Desoto County flood was approximately $691 per household. $36,625/53 = $691.04. We assumed that 50 percent of the IFG money was not used to repair or replace primary residence or contents. We used $345.50 (50% * $691) for each affected household. The expected expenditures for IFG money were added to the emergency costs. Additionally, the District's

Engineering Division estimated the costs of pumping out the floodwaters. The pumping cost methodology is detailed in the engineering appendix. These emergency costs and pumping costs for breaks in reaches 1A, 1B, and 1C are displayed in tables 8 and 9 for the low and high velocity condition respectively.

Road Damages

Road damages are the cost of repairing flood damaged roadway embankments and pavements. Under normal conditions roadways can be inundated for a short duration with little damage. In the case of a breach of Herbert Hoover Dike flood duration in Reach 1 could be measured in weeks and months. To estimate the cost of repairs to these roadways a GIS ARC/INFO database was developed. This database utilized ETAK major road classifications to determine the length of roadway in each damage reach. Damages to light duty residential roads were previously included in the Residential Urban damage category and are not included here. The Florida Department of Transportation (FDOT) was contacted to determine what the damages would be to flooded roadways. The Florida Department of Transportation's District 4 office covers Palm Beach County. The District 4 office is familiar with road construction in Belle Glade and was consulted. The FDOT District 4 office of roadway design reviewed the FDOT Transportation Costs 1996-97, published by the Office of Policy Planning (February 1998). Taking into account Reach 1 area estimated damage factors and reconstruction costs, a reconstruction cost of $500,000 per lane mile or $1,000,000 for a two-lane road was estimated. The damage factor to the roadway for increments of flooding duration were as follows:

Factor	Days of Flooding
35%	15
60%	30
75%	45

The Jacksonville District's Engineering Division estimated the cost to be $500,000 for a two-lane road. For this analysis the average figure of $750,000 per mile for a two-lane road was used. The duration and depths of flooding were based on stage hydrographs. The topography of the roads was taken from bench marked elevations from USGS 7.5 minute series quadrangle sheets. From this information it was estimated that at least seventy percent of these roads would be inundated. Seventy-five percent of these roads would be inundated for at least 15 days and twenty-five percent would

be inundated for at least 30 days. The total value of road damage for a breach would be at least $25,500,000 for a low velocity breach and $34,500,000 for a high velocity breach.

Transportation cost savings

The Jacksonville District identified transportation cost savings of road and street related effects from floods in Reach 1 of the study area that project alternatives might impact: closure of roads due flooding and closure of roads during repairs to flood damage roads. These cause detours and delays to vehicles that normally would use the closed streets and highways. The economic cost of these effects is the value of increased vehicle operating costs from detours around the closed roads and the cost of increased travel time due to longer distance and increased congestion.

Using information of elevations of major streets and in the study area and estimates of water surface elevations and duration, the District identified several highways that might be closed or damaged by flooding during one or more of the events. Major highways are shown in figure 1.

Flood detour cause both increased travel distances and increased travel times to the affected vehicles. Thus, damages are composed of the added vehicle operating costs due to increased travel mileage and the value of the driver’s and passengers’ time lost due to the increased travel time. The estimation of these two values for each of the flood events analyzed requires the following information for each of the affected roads:

1. the increased travel distance detour in miles,
2. the congestion adjusted speed of vehicles traversing the detour,
3. the duration of the road closure,
4. the income of the vehicle occupants,
5. the number of occupants in each vehicle, and
6. the number of cars and other vehicles detoured.

The first three items are different for different flood events. For more severe events, there are fewer, and many times, more lengthy alternative routes; the duration closure is longer, and; the vehicle speed will be less due to more congestion. The information on travel distance and vehicle speed was combined to estimate increased travel time for each vehicle. The income of the vehicle occupants and the increased

travel time was used with the recommended values of fraction of driver/passenger income by increased travel time and type of trip using methodology described in IWR report 91-R-12 and updated to current values to estimate the value of time lost per vehicle per day. This income value was based on the county in which the closed road is located. This value was then combined with the number of vehicles per day, the number of occupants per vehicle, and the number of days of closure of an event to estimate the cost of time to vehicle drivers and passenger by the event. Cars and trucks were accounted separately.

The District prepared spreadsheets to simulate traffic re-routings and traffic delays for the each of the flood stages based on each conditional break scenario for each of the three different lake stage events. These events related to the 18 foot, 21 foot, and 26 foot lake levels for each reach. The spreadsheets were run for both the high velocity and low velocity flows from the breach. Base data for each road taken from these spreadsheets included:

1) stage-event tables,
2) duration-event tables,
3) estimates of detour mileage by event,
4) estimates of detour speed,
5) traffic volumes,
6) estimates of the directional splits in detour routes and detour volumes,

Other values not road specific included:

1) proportion of cars compared to trucks,
2) occupants per vehicle,
3) mean income of occupants,
4) the per mile operating costs for cars and trucks.

Infrastructure

The cost to repair damage to the water distribution, sanitary sewer, storm sewer, electrical systems, power lines and other public utility service systems was estimated. A breach in Reach 1 would cause significant flooding to the towns of Canal Point, Pahokee and Belle Glade, Florida. A number of local agencies and utilities were contacted in the Herbert Hoover Dike region to determine potential infrastructure damages from flooding but had no real flood experience. In 1997, Grand Forks, North Dakota and East Grand Forks, Minnesota had significant flooding that resulted

in the St. Paul District of the Army Corps of Engineers being involved in surveying actual flood damages. It was estimated that damages to the water system, sanitary sewer, storm sewer, sewer cleanup, lift station repair, parking lot repair and electrical repair were about $5,818,000 for 20 miles or $290,900 per mile.

Automobile Damages Prevented

An additional benefit has been claimed for the reduction of flood damages to automobiles. The evaluation required the determination of the number of automobiles affected, the value of an average automobile, and the damage susceptibility associated with flood depth. The methods used to evaluate these items are explained in the following paragraphs.

The number of cars affected was first determined by counting the number of low density, medium density, high density residential, multi-family buildings and mobile homes in each floodprone area associated with each different breach scenario in the Herbert Hoover Dike study area. The number of automobiles per home were estimated based upon a study completed for Palm Beach County in 1996. [1] Automobile density was estimated at 1.5 automobiles per structure for single family residential, 1 automobile per structure for mobile homes and 6 automobiles per structure for multi-family structures (4 occupancies X 1.5 automobiles) per structure. Residential development is predominantly single family residential. Given the size of the floodprone areas created by different breaches and the relatively few major evacuation routes, it was estimated that even with sufficient warning time prior to a flood, it is estimated that perhaps 25% of the residents could be evacuated. Some residents may be outside the basin at the time of the event for a multitude of reasons,

[1] In the 1996 study, the number of cars affected was determined by first determining the total acreage of low density, medium density, high density residential, multi-family buildings and mobile homes in Palm Beach County and corresponding dwelling densities of development. This information was determined from planning documents for the County. Dwelling unit densities for residential land use types were multiplied by acreage in the planning documents to determine the number of occupancies in the county by residential land use classification. The computations were adjusted to the 1990 census information, which was used as a control. A value of 1.5 cars per unit for all classifications produced a total of 734,627 cars for Palm Beach County. The total adjusted number of registered automobiles provided by The Department of Transportation in 1990 was 877,731 for Palm Beach County. Therefore, 1.5 cars per structure was selected for the single-family residential structures in the basin.

which would include vacations, business, shopping or recreation. It was assumed that 75% of the autos would be damage susceptible during a flood event. For the purpose of the evaluation, these cars will be located at their primary residence during the flood.

The value of an average automobile was then calculated. The method utilized selecting typical automobiles in three categories, compact, medium and large. Wholesale and retail values were averaged for each automobile in each category for the years 1983 to 1998. Information provided by the Nationwide Personal Transportation Survey indicated the percentage of the total automobiles on the road in the United States for each of the years from 1967 to 1998. The three classes were averaged for each year and the yearly percentages were used to weight the average value of a typical car. The average computed value used in the study is $6,347.46.

It is estimated in the analysis that one foot of depth is required above ground before water gets into a car and causes any appreciable damage. Depth-damage assumed that most vehicles are damaged 10% with one foot of water, 20% with two feet of water, 30% with three feet of water and 80% to 100% when water reaches or exceeds four feet.

Estimated HTRW Related Costs Due To Flooding

A review of typical hazardous and toxic waste or materials found immediately adjacent to the Herbert Hoover Dike indicates the following types of potential contamination:

a. Treatment Storage and disposal facility (TSD) - This type of facility possess the greatest risk of contamination. Drums stored at the facility may be spilled or labels would be washed off as a result of the flood. All unlabelled drums found after the flood recedes would have to be sampled. Typically one can expect about $ 1,000 per drum.

b. Transport of hazardous waste has a very similar profile as the TSD facility but at a lower risk level.

c. Small quantity (SQG) and large quantity generators (LQG) have similar risk as TSD facilities. The LQG may have more drums on hand than the SQG. It should be noted that spills of these wastes are a low probability because under the Resource Conservation

Recovery Act (RCRA), all tanks and drums must be in good physical shape and must be sealed when not being filled.

d. Underground Storage Tanks (UST) and leaking UST have similar risk. It can be assumed the more than 90% of the UST contain hydrocarbons. Also more than 70% of the hydrocarbon consist of gasoline. If we estimate the remaining types of hydrocarbon, then the types and relatives quantities are:

- gasoline 70%
- diesel 20%
- aviation fuel 8%
- heavy fuels 2%

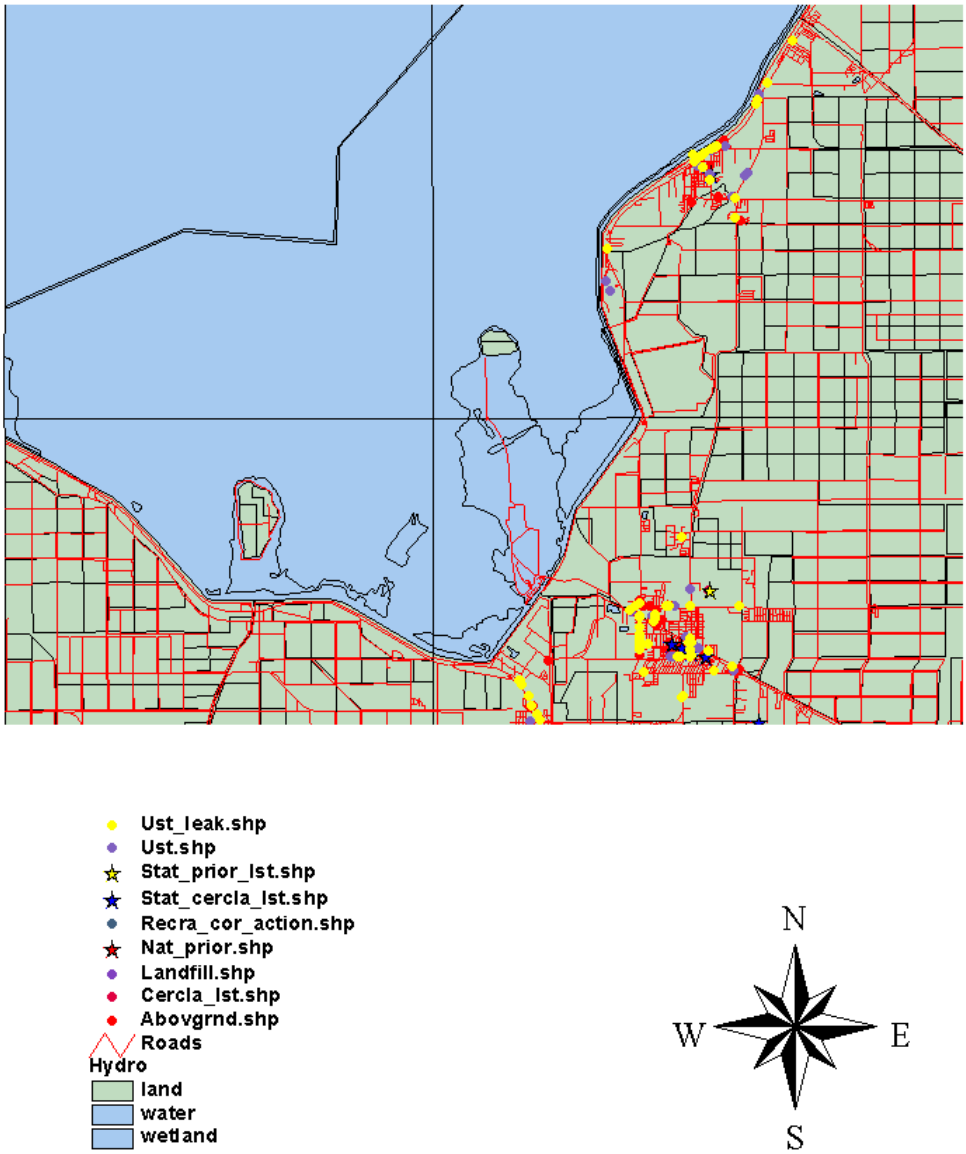
The gasoline and light aviation fuels are anticipated to evaporate after being displaced by the water. Since water is heavier than hydrocarbon it is safe to assume that each tank would be filled with water and the fuel would float away. The diesel and heavier hydrocarbon products tend to adhere to any surface in contact with the floating product.

e. Resource Conservation Recovery Act (RCRA), Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), listed clean-up sites National Priority List [NPL], State Priority List [SPL], Landfills, Spills, etc. type sites have a common condition of soil contamination. Flooded contaminated soils would not disperse the contamination to any significant extent. Therefore we are assuming that any of these types of sites would result in no flood related costs.

A review of several databases when plotted over the Herbert Hoover Dike vicinity shows the potential flood problems. The following figure has been created to provide a relative scale of the quantity and location of each of these HTRW potential problems.

Since all the hydrocarbons will float and adhere to other debris, we can expect negligible cleanup costs for hydrocarbons. The clean-up costs would not be distinguished from other debris cleanup.

HERBERT HOOVER DIKE HTRW DATABASE REVIEW



Contaminated soil sites would not require any cleanup. The flood damage will not significantly spread the contamination.

The only significant potential remaining consists of unlabelled drums floating away from, TSD, SQG, and LQG.

Figure 4 has been enlarged to enable easy counting of these sites. This figure be used to approximate the number drums. Assume $ 1,000 for testing and $ 1,000 for disposal per 55 gallon drum.

Within the southern reach, 25 small quantity generators have been identified. SQG typically store two drums. Therefore 50 times $ 2,000 or $ 100,000 in drum testing and disposal is possible. Only on LQG was indicated. Estimate 50 drums stored or another $ 100,000 in testing and disposal costs.

In conclusion, minimal costs are expected due to HTRW related cleanup because the volumes are very small relative to the wide reach of the flooded plain. Any contaminant when dispersed by the water would be diluted to a great degree and cleanup would be largely due to bioremediation. The worst scenario envisioned would involve the discovery of floating unlabelled drums. Based on the above logic the cost is estimated at about $ 200,000 dollars for the southern reach of the Herbert Hoover Dike. Given the unlikely probability of this event and the relatively low cost of cleanup, benefits for this category have not been included in the evaluation.

Flood Insurance Overhead Costs Saved

An NED benefit can be claimed for the reduction of flood insurance overhead costs. However, after review of the subject report by the Federal Emergency Management Agency (FEMA), the agency stated that the data submitted was not sufficient to warrant a re-delineation of the 1% annual chance floodplain on County Flood Insurance Rate Maps (FIRM). Instead, the floodplain will be delineated as a “Shaded Zone X” and a special note added referring to the flood hazard due to potential dike failure. A “Shaded Zone X” designation will allow residents to purchase flood insurance but the insurance will not be required. Since homeowners will not be required to purchase flood insurance, no additional flood insurance overhead costs are expected without rehabilitation and no benefit is expected with rehabilitation.

HOOVER DIKE MODEL

General Description

The Hoover Dike Economic Life Cycle Model was developed by the Institute for Water Resources (IWR) of the US Army

Corps of Engineers, at the request of, and in close cooperation with, the Jacksonville District (SAJ), to assist in the analyses associated with the Herbert Hoover Dike Major Rehabilitation Evaluation, being carried out by SAJ. The model is a risk-based (Monte Carlo) simulation of potential breaching of the levee surrounding Lake Okeechobee (Hoover Dike), associated economic damages, and population affected. The model takes into account the effect of lake stage and hurricane surge as the driving force for breaching. A 4-season model is used to capture variations in hurricane probabilities and the state of crops during different times of the year. Probabilistic aspects of the model include hurricane occurrence, stochastic lake stage, height of tailwater, and probability of breaching.

The model consists of three related modules: a Microsoft Access 97 database, storing input and output data for the model; a Visual C++ simulation kernel, that performs the actual simulation, reading information from the database, doing the calculations, and storing the results; and a Microsoft Access 97/Visual Basic application that serves as the user interface, allowing the user to input and edit data, launch the simulation runs, and examine output. The model is completely self-contained, and does not require knowledge of Microsoft Access.

Development was started in January of 1997, with an initial version available in September of 1997. Subsequent modifications led to a completed version in August of 1998.

A number of versions of the model were developed, in a process known as iterative development. Initial versions of the model were focussed on simulation of hydrologic behavior (hurricane probabilities and static lake stage). Subsequent versions added geotechnical behavior (breaching in response to stage), and finally economic damages associated with breaches. By January of 1998, the final structure of the model was largely complete. Modifications since that time have been oriented primarily towards improvements in the user interface, and in development of outputs that assist in the analysis.

While the model structure was developed specifically for the Hoover Dike study, it has many general characteristics, potentially allowing it to be used in other situations. The model is a ‘data-driven’ model, i.e. the user inputs information that characterizes the target of the model, in this case Hoover Dike. Thus, the model is somewhat general, while the associated data define the characteristics and behavior of Hoover Dike in particular.

Acknowledgements

The modeling effort was conducted under the direction of Dr. David A. Moser of IWR, and carried out by Planning and Management Consultants, Ltd. (PMCL), Carbondale, Illinois, and RMM Technical Services, Inc., Cincinnati, Ohio. Model structure and functioning was determined in close consultation with representatives of SAJ, who also developed and provided the extensive input data required. Mr. Peter J. Grace of SAJ served as the primary technical and administrative point of contact, and provided technical expertise on the hydrologic aspects of the problem. Mr. Anthony L. DiPiero of SAJ provided input on the geotechnical considerations, and Mr. Thomas G. Arnold and Mr. Dan Peck of SAJ were the key contact on economic matters.

Background

Herbert Hoover Dike is an earthen embankment surrounding Lake Okeechobee in southern Florida. The lake is in the path of Gulf and Atlantic hurricanes during hurricane season, leading to hurricane-induced storm surges. The embankment is subject to seepage and piping under conditions of large head differential. The potential for breaching of the dike through seepage or piping-induced failure of a portion of the levee exists. Such failure would lead to inundation of areas protected by the dike, with attendant damages associated with residential and agricultural land uses, and affected populations. The problem has many probabilistic aspects, including:

- frequency and type of hurricanes;
- lake stage variation over time;
- tailwater distribution over time;
- probability of breaching of different portions of the embankment;

Rehabilitation of selected portions of the embankment will reduce the likelihood of breaching of those portions. The economic feasibility of any given rehabilitation alternative is mandated, by Corps guidelines, to be examined through risk-based economic analysis. Monte Carlo simulation techniques, in which computer models of the phenomenon under study are created and analyzed, are commonly applied to risk-based problems. Monte Carlo simulation combines uncertainties in many variables that describe a system, to obtain a statistical description of the behavior of the system as a whole. This is

accomplished through repeated runs of a simulation model, varying the input data based on the statistical descriptions of uncertain parameters.

The important sources of uncertainty for a given problem are identified and described statistically. These data are then used within the simulation model developed for the problem. At each point within the simulation at which descriptive data for uncertain variables is required, the statistical distribution is used to set a value (through random sampling from the distribution), which will thus differ from simulation run to simulation run. Many simulation runs are made, and the resultant overall statistics are used in the decision-making process. The technique is both flexible and powerful.

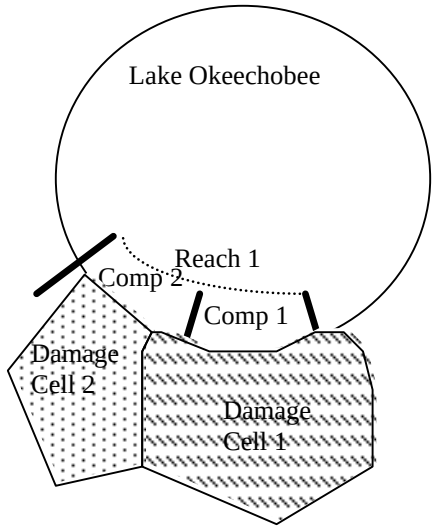
Technical Overview

Rehabilitation alternatives are directed at reducing the likelihood of breaching of the levee. Breaches are anticipated to form in portions of the levee in response to large hydraulic head differentials between the lake side and the tailwater side (outside of the levee). Lakeside elevation is a combination of *static lake stage*, which varies slowly based on normal inflows and outflows to the lake, and *hurricane storm surge*, which varies rapidly (hours) in response to the passing of a hurricane. The susceptibility to breaching is a function of the condition of the levee, which varies along its length.

For purposes of modeling of the geotechnical behavior of the system, Hoover Dike has been divided into a number of *reaches,* and reaches are further subdivided into *components*, which are contiguous sections of the dike. The component is the part of the system that is susceptible to breaching. As with the other models, failure and repair take place at the component level. Breaching behavior is modeled in a stochastic fashion, through Probability of Unacceptable Performance (PUP) functions, giving the probability of a component breach as a function of lake stage at the component. Lake stage is composed of static lake stage, and, if a hurricane exists, hurricane surge. The impact of rehabilitation alternatives is reflected primarily through modification to the PUP functions associated with component breaching, i.e. a rehabilitation alternative will result in a component PUP function showing a lower probability of breaching for the same lake stage.

If a component breaches, it can breach in one of two modes – low velocity (smaller breach) or high velocity (larger breach) with differing associated times and costs of repair, and damages. Breaching behavior is dependent upon the height of tailwater behind the component. If this results in a head differential greater than a critical level, then a high velocity breach is expected. Otherwise, a low velocity breach occurs.

A breach of a component can result in the inundation of one or more areas outside the levee, referred to in the modeling effort as *damage cells*. Monetary and non-monetary impacts are associated with the inundation of these damage cells. The economic damages associated with a damage cell are a function of the land use within the damage cell, lake stage when a breach occurs, type of breach (high/low velocity), and time since previous inundation (due to persistence of agricultural crop damages, sometimes over a period of 10 years following a flood). The problem is complicated further because there is not a simple one-to-one relationship between a component breach and the damage cells that get inundated. The same damage cell can be flooded from breaches in more than one component. The model stores user-entered functions that determine the associated impacts of breaches as a function of static lake stage. The following graphic displays the relationship between reach, components and damage cells.



Hurricane season in the study area is from June 1 through November 30. The historical record indicates that the tracks of tropical cyclones affecting Lake Okeechobee can be divided into two basic categories: from the southeast (Atlantic storm), producing the greatest surges along the southeast quadrant of the lakeshore from the southwest (Gulf of Mexico storm), producing the greatest surges along the northwest quadrant of the lakeshore.

A period-based modeling approach was selected, based primarily on the continuous hydrologic data for lake stage, which does not lend itself to an event-based formulation. A 4-season model is used to capture variations in hurricane probabilities and the state of crops during different times of the year. This formulation restricts the number of hurricanes allowed to one per period. Statistics show that the probability of more than two hurricanes in the 6-month hurricane season is very low, thus this is not seen as being limiting for the Hoover Dike application.

An initial effort at developing synthetic lake stages through an autocorrelation model did not reproduce historical lake stage statistics well. An autoregressive moving average model (ARMA model) was then developed from historical lake stage data, to provide the needed synthetic static lake stage hydrology.

A methodology for estimating the storm surge at each component based on the different types of hurricanes was developed by the Jacksonville District. This method estimates storm surge at an index location, and then uses functional relationships between all other locations and the index location to get the desired surge at a component.

Provision is also made for reduction in lake stage if a breach occurs, due to the loss of water through the breach. Although this effect is minor, given the very large volume of the lake, it is included for completeness of representation.

Economic impacts include repair cost, o&m costs, rehab costs, and economic damages based on land inundation in a damage cell, associated with a breach. In addition, provision is made for non-monetary impacts of damage cell inundation. The effects of inundation in a period are described as mathematical functions of lake stage. For a given lake stage event, the model calculates the expected economic or non-monetary effects.

The model is typically run for a 50-year time span, with each year being divided into four periods, resulting in 200 cycles for a typical simulation. In Monte Carlo simulation, a number of iterations are run, and statistical summaries of behavior over many iterations are developed as the basic model output.

Model Processing

The overall flow of the model is relatively straightforward. The user initially sets all of the needed input parameters (simulation duration, number of iterations, interest rate, etc.). The model is then run for the required number of cycles and iterations. In each iteration, the behavior is as follows:

a) determine the static lake stage in the current cycle, based on the ARMA model;

b) determine whether a hurricane takes place in the current cycle, based on input hurricane probabilities;

c) test each component, in turn, for breaching, based on stage at the component, which is determined as static lake stage plus storm surge (if a hurricane is present). Once the stage has been determined, a PUP value is determined based

on function lookup. This is compared with a generated random variable (0 to 1), to determine if a breach takes place. If a breach condition is found, the velocity of the breach is determined by obtaining a random value of the tailwater from an input triangular distribution of tailwater at each component. The head differential is calculated and compared with an input *critical value* – if the head differential is greater than the critical value, a high velocity breach condition exists, otherwise it is a low velocity breach;

d) if a breach takes place in the cycle, determine the associated impacts (monetary and non-monetary damages), based on function lookups, and accumulate them. The effect of prior inundation is accounted for by applying reduction factors to damages associated with the current cycle, based on damage reduction curves input for specific land uses.

This process is repeated for each cycle of the iteration, and for the number of iterations requested.

Analysis Framework

a) Physical System

The basic physical components of the system are reaches, components, and damage cells. A reach is a contiguous portion of the dike, having certain commonalties in terms of descriptive data, primarily a common O&M cost function. Each reach consists of one or more components, which are further sub-divisions into contiguous portions of the reach. Note that each reach must have at least one associated component.

Components within a reach have certain uniform characteristics (e.g. O&M costs are common amongst components in a single reach, as these are properties of the reach, not of the component), while other characteristics (in particular physical characteristics, repair costs, and PUP functions) will differ. The component is the smallest unit of the dike to which data is assigned.

A damage cell is an area of land lying outside the dike, susceptible to impacts based on inundation. For the Hoover Dike study, 8 reaches, 13 components, and 14 damage cells have been defined. Reaches are numbered 1 through 8. The relationship of components to reaches is as follows:

Reach	Component
1	1A
	1B
	1C-a
	1C-b
	1C-c
2	2
3	3
4	4
5	5
6	6A
	6B
7	7
8	8

The fourteen damage cells are identified as follows: 1A, 1B, 1C, 2, 3A, 3B, 3C, 4, 5, 6A, 6B, 6C, 7, 8. There is an association between a breach in a component, and flooding of a set of damage cells, as follows (there is no significance to the order in which the damage cells are presented):

Component	Damage Cells Potentially Flooded
1A	1A,1B,1C
1B	1B,1C,3B
1C-a	1B,1C,3A,3B,3C
1C-b	1B,1C,3A,3B,3C
1C-c	1B,1C,3A,3B,3C
2	2
3	3
4	4,2
5	5
6A	6A,6B,6C,8
6B	6A,6B,6C,8
7	7
8	6A,6B,6C,8

b) Breaching

In the modeling, each component is treated as a homogeneous, independent structure. Under high lake levels, a component can breach through piping failure. The primary objective of the model is to determine and assess the economic impacts of the breaching of one or more components, in response to hydrologic events (high lake levels). The driving force for the simulation is lake level, determined

stochastically based on statistics on static lake level, which varies throughout the year, and storm surges, which take place during hurricane season. The combination of static lake stage plus storm surge at a component is taken as the water surface elevation (WSE) at the component.

Breaching of a component is determined based on user-input probability curves, called Probability of Unacceptable Performance (PUP) Functions. PUP functions give the probability of breach as a function of water surface elevation at the component. The Geotechnical Branch at SAJ has developed the requisite PUP functions, for both the existing conditions and various rehabilitation alternatives.

Breaching of a component can be in one of two modes - high velocity, or low velocity. The mode of breach is related to the head differential between lake level and component tailwater at the time of breaching. Higher differentials result in greater flow velocities in the event of a breach. A head differential of 13’ is taken as the demarcation between high and low velocity breaches - a head differential of greater than 13’ across a component will result in a high velocity breach and a differential of less than 13’ will result in a low velocity breach. High velocity breaches require large stones (not readily available in the immediate area) to fill.

The PUP functions for components already incorporate the uncertainty in tailwater elevation. Thus, estimates of tailwater level are not incorporated explicitly into the determination of breaching, but do determine the mode of the breach, as described below.

It is anticipated that a high velocity breach will require some 40 days to close, while a low velocity breach can be closed in 10 days. Multiple components can potentially breach simultaneously. It is assumed that, if a breach takes place, it will be repaired by the start of the next period. Breaches are repaired to pre-breach conditions, thus probability of failure pre- and post-repair is the same. Failure probabilities are, of course, changed as a result of rehabilitation of components.

An estimate of tailwater level is required for each component, to determine the speed of the breach (high/low). This is obtained through specification of a minimum, maximum, and most-likely tailwater elevation for each component, which

is used to define a triangular probability distribution. A single distribution is provided for each component, i.e. the tailwater distribution is not a function of period, lake level, or anything other than the specific component. Inasmuch as certain rehabilitation alternatives modify the tailwater at the toe of the dike, the model provides that rehabilitation alternatives may modify the post-rehabilitation tailwater distributions for a component.

c) Hurricanes

Hurricane season in the study area is from June 1 through November 30. The historical record indicates that the tracks of tropical cyclones affecting Lake Okeechobee can be divided into two basic categories: from the southeast (Atlantic storm), producing the greatest surges along the southeast quadrant of the lakeshore and from the southwest (Gulf of Mexico storm), producing the greatest surges along the northwest quadrant of the lakeshore. Both types of hurricanes are handled. The user provides input data giving the probability of any type of hurricane in a given period, and the conditional probability that, given a hurricane exists, it will be a Gulf hurricane. [The only other possibility is an Atlantic hurricane].

d) Seasons

Four seasons are specified (December-February, March-May, June-August, September-November), aligning with the June-through- November hurricane season. This allows sufficient capability to capture both the variation in hurricane probabilities, and the seasonally dependent damages associated with inundation of agricultural row and truck crops. This in effect limits the maximum number of hurricane events that can take place in a year to two (one for each of the two periods representing the hurricane season). The 110-year historical record has been used to estimate the probability of 1 or more storms in a given year, using a Poisson distribution, with results as follows: 1 storm - 0.1773, 2 storms -0.0196, 3 storms - 0.0014. Given the low probability of more than 2 storms per year, the limitation associated with a quarterly model should not be excessively confining.

e) Water Surface Elevation

Water Surface Elevation (WSE) is composed of two parts - a static lake level, uniform for all reaches at a given time,

and a storm surge component during a hurricane. The static lake level varies throughout the year. Static lake level has been shown to be serially correlated quarter-to-quarter.

Static Lake Stage

Dr. David Moser of IWR analyzed lake stage data provided by SAJ, and developed the auto-regressive moving average model (ARMA) used to estimate static lake stage period to period within the Hoover Dike model. From the data, autoregressive and mixed autoregressive-moving average models were investigated. Preliminary investigation confirmed the stationarity of the series.

The ARMA model that performed well has the following process:

$$S(t) = m + r\,[S(t-1) - m] + sd\,[e(t) - q\,e(t-1)] \quad (1)$$

Where

S(t) = stage at time t
m = mean of the process estimated from the historic series
r = estimated autocorrelation parameter
sd = standard deviation of the process estimated from historic series
et = random error term, normally distributed with mean of 0, standard deviation given by term se in equation (2) below
q = estimated moving average parameter

and where

$$se = (1-[r*r])/(1+[q*q]-2rq) \quad (2)$$

That is, the stage at time t is a based on the historic mean, the departure of the prior period stage from the mean, and a ‘random error term’, calculated for the current and prior periods.

This equation (1) is used to generate the synthetic stages. The above parameters were estimated using the Lake Okeechobee historical time series yielding:

m = 14.379
r = 0.5350
sd = 1.5153
q = -0.4856
se = 0.6376

In testing this formulation, the series was started at the historic mean stage and used the process in (1) with a normal distribution for it to generate synthetic sequences. The distribution that best fit the historic data was also determined. A lognormal distribution proved to be the best fit. Using the lognormal distribution, a stage of 18 feet should be exceeded approximately 1.5% of the time.

In order to provide some physical constraints on the static lake stage generation, a maximum and minimum allowable stage are also provided.

Storm Surge

Storm surge during a hurricane varies from component to component, due to the different wind fetch for each component, and depends upon the particular storm track and intensity. The shape of the storm surge hydrograph is also variable. Rather than attempt to handle all of the possibilities related to shape of the surge hydrograph, an external relationship is defined between peak surge and ‘effective surge’, which is a surrogate for the level and duration of the storm surge. Thus, the WSE is calculated as the sum of static lake level and effective surge associated with a storm event.

f) Impacts of Inundation

Economic impacts include repair cost, o&m costs, rehab costs, and economic damages based on land inundation in a damage cell, associated with a breach. In addition, provision is made for non-monetary impacts of damage cell inundation.

Repair Cost

Repair cost is defined on a component basis, as $ per repair, for each of high and low velocity breaches. Revised repair costs are also associated with the data defining rehab alternatives for a component.

O&M Costs

O&M Costs are specified, on a reach basis, as functions giving total O&M dollars in a period, based on static lake stage, and then internally calculated on a quarterly basis.

The data base makes provision for the possibility of higher post-hurricane O&M costs, i.e., if there is a hurricane, greater maintenance costs are incurred. The user can enter a single multiplicative factor (global to the simulation), such that, if a hurricane occurs, the O&M costs for that period and the subsequent three periods are taken as the input O&M costs (which vary by reach) times the global multiplicative factor. If additional hurricanes occur within the year, the effect is multiplicative, i.e. the global factor is applied again for the current period and the subsequent three periods.

At present, there is no capability, within the model, of modifying reach-associated O&M costs post-rehab. Additionally, the application of the O&M multiplier to three subsequent periods is 'hard-wired' in the model, and cannot be changed by the user.

Rehabilitation Costs

Rehabilitation plans consist of two parts: a set of changes to component data, that take place at designated cycles of the simulation; and a rehab cost schedule showing the time-based expenditures associated with the given rehab plan. The model is capable of storing multiple rehab plans.

The model thus separates out the costs associated with rehab from the results of rehab on components. The rehabilitation cost schedule is the list of rehab costs by period, with no reference to the particular component involved. Thus, a generalized cost associated with engineering/design, or mobilization, might be assigned at a certain period, before any work is actually done on the components.

It is the user's responsibility to insure correspondence of rehab costs with the appropriate rehab effects (e.g. post-rehab PUP, post-rehab repair costs, etc.), of a given rehab plan, as they are independently entered.

Damages

The model provides for the capability of assigning categories of inundation impact to damage cells from a specific component breach, described by a functional relationship between the impact and the lake stage. When the model was initially developed, all impacts were associated

with monetary damages based on land use (urban flooding, crop loss, etc.). At a later date in the process, the need for additional impact measures became clear. The same framework was used, i.e. the additional impact measures were treated as pseudo-land uses for damage cells. The specific impact measures are stored in the database, but are not currently modifiable by the user. Each impact can be monetary or non-monetary. Within the model, all the monetary measures are summed to a single monetary impact, while the non-monetary impacts are treated individually.

As currently developed, the Hoover Dike database uses information on four land use types (residential urban; non-residential urban; agricultural fruit tree crops; agricultural row and truck crops), one 'other benefits' type, and an additional non-monetary 'population affected' impact.

Impacts vary depending upon the particular components that breach, the speed of the breach (high or low velocity), the season of the year, the type of impact involved, flooding in prior seasons, and the lake water surface elevation at the time of the breach. This is expressed as a set of piece-wise linear stage-damage relationships, currently envisioned to have a maximum of 50 point pairs per function. While only the agricultural row/truck crops show seasonal differences in damages, this is expressed in the data, rather than internally in the model structure. Thus, all periods for urban residential damages for a given component, damage cell, and breach velocity point to the same function for all four periods, while for the agricultural data, different functions are used for each period. This provides maximum flexibility, allowing any damage impact to vary seasonally if need be.

Breaches of multiple components are specifically considered in the model. If breaching occurs, impacts occur within a 'damage cell' (formerly damage reach), i.e. an area of land. Many damage cells can be associated with a component, and the same damage cell may be inundated by breaches in various components. The inundation of damage cells also depends upon the WSE and speed of the breach. The particular damages depend upon the land use, and whether or not the damage cell has been inundated previously (as long-term crop damage can exist). If multiple reaches breach simultaneously, damage assessment is based on the maximum damages associated with each damage cell from any of the individual component breaches that affect that damage cell.

A complete list of the correspondence between breaching in a component, and effects in damage cells, is shown on page B-29. For each such breach situation of a component and associated damage cells, the appropriate impact function must be identified for each period, impact type, and breach velocity. Thus, 48 such identifiers must be entered (4 periods, 2 breach velocities, 6 impacts [4 land uses, 1 other benefits, 1 population affected]). This does not, however, imply 48 unique functions, because, as noted above, some impacts do not show seasonality, nor do they change with high/low velocity breaches.

Prior-period damages are considered explicitly in the model, to account for damage reduction if a damage cell has previously been inundated. Although this effect is really applicable only for truck and row crops, and fruit and tree crops, the concept is applied to all of the impacts, and is handled by specifying damage reduction curves in the database.

For a situation in which there is inundation in a period, when prior period inundation is still economically affecting the damage cell (which can be a period of years for fruit/tree crops), the model takes into account the lessened damages associated with inundation in the current period. The lessened damages are calculated using adjustment factors based on impact-specific curves (stored in the database) of adjustment factor vs. time since last inundation for the damage cell, and are applied to the unadjusted stage damage curves.

For urban residential and non-residential land uses, it is assumed that damages in a period are independent of anything happening in prior periods. The model keeps track of the time (cycle) of the last inundation of each damage cell. There is no need to keep track of the type of inundation (high or low-velocity), since, for prior-period damages, the effects are the same.

While truck/row, fruit/tree, and residential each have different responses to prior-period flooding, for purposes of ease of modeling, the methodology is generalized, based on user-entered curves of percentage damage assessment in out-periods following inundation vs. time. The differences in treatment of each impact is expressed by differences in data entry, as described below.

For each impact type, the user enters a curve giving the percentage to be applied to the economic damage function for an inundation in the current quarter, based on the time since previous inundation. Note that the interpolation function used with these curves will apply the final value to anything greater than the highest cycle.

g) Simulation Process

The overall simulation process operates by calculating, on a risk-based basis, the probabilities of breaching, and the damages associated with the consequences of breaching, over a simulation life cycle (typically 50 years), divided into periods or time steps (currently quarterly periods, as defined above). As is always true with Monte Carlo simulation, a number of iterations of the simulation are required to determine the distribution of expected damages and benefits associated with rehab.

For each cycle of the simulation duration, the simulation:

1) determines WSE at each component;
2) determines breaching at each component, based on WSE;
3) determines repair costs and damages associated with breaching;

This process is repeated for each cycle, with the simulation model doing the accounting of costs.
A quarterly time step has been selected as sufficient to capture the appropriate economic and physical behaviors.

The details of the process are follows:

For each cycle:

Step 1 - determine WSE

a) determine static lake stage:
An analysis of quarterly lake stage data by Dr. David Moser of IWR led to the development of an autoregressive moving average (ARMA) model described previously. In each quarter, this statistical model is used to estimate the current period lake static stage from the historic mean and the prior period lake stage.

b) determine storm surge portion of WSE at component:

Storm surge is dependent upon a hurricane, and takes place during hurricane season (June 1 - November 30). Thus, the calculation of storm surge will only take place if a hurricane occurs.

Hurricane Occurrence

The determination of hurricane occurrence is based on input data giving the probability of any hurricane occurring in the period, and then, if a hurricane exists, the probability that it is a Gulf or Atlantic-type storm. The model generates a single uniformly-distributed random number, and determines if this number exceeds the input hurricane probability figure. If so, a hurricane occurs, otherwise, there is no hurricane in the period. If there is no hurricane, we do not get any surge in the quarter, and this portion of the calculation is complete.

If a hurricane occurs, the model determines whether it is an Atlantic or Gulf hurricane, in a similar fashion, by again generating a uniform random number, and comparing to the input data probability of a Gulf hurricane. If the random number is less than this value, then a Gulf hurricane exists, otherwise, an Atlantic hurricane occurs.

Storm Surge

With the storm type known, it is necessary to determine a storm surge at each component. The storm surge at each component should be cross-correlated, given that there is only a single hurricane event. It is not appropriate to independently determine the storm surge at each component for a hurricane, because these are not independent events. Instead, the model generates a surge at a single component [the ‘index component’, selected to be component 1C-c], and derives the surge values for all other components from that value.

This is obtained by selecting from an input frequency distribution (by hurricane type) giving storm surge at 1C, which is stored in the database. These curves have been developed to provide the 4-hour duration surge, which has been determined to be the critical time period for surge to have an effect in terms of breaching.

The storm surge at a derived component has a deterministic part and a stochastic part. The deterministic

part is calculated through a coefficient (varying by hurricane type for each component),which is multiplied by the index component surge. The coefficient is time-invariant and independent of lake stage. The stochastic part is also based on time and stage-invariant input data for each component/hurricane type, and is based on a triangular distribution. Thus, the functional relationship between effective surge at a derived component and effective surge at the index component is:

Effective Surge(d) = [Effective Surge(1C-c) * K(d,hurricane type)] + Stochastic Term(d,hurricane type): where d is the derived component, K is obtained from a lookup table, and the Stochastic term is calculated from input statistics.

The surge at multiple components may be the same, but this is handled by assigning the appropriate factors in the above table. As with the issue of prior-period economic damages, this form of generalization of the model is simpler for the simulation, and allows for changes later on simply by changing the data.

In summary, the surge process then involves determining the type of hurricane, developing a number for the surge at the index reach, and calculating the derived surge at other reaches as a multiplier of the index reach surge, plus a stochastic component. This yields the water surface elevation (WSE) for the component.

Step 2 - determination of breaching

For each component, the static stage and surge are added, to get the WSE. Each component is tested for breaching in high/low velocity mode. For each component, a PUP function of breaching probability vs. WSE exists. Given the WSE, the PUP value is interpolated from the appropriate function. A uniform random number (0 to 1) is generated, and if it is greater than the PUP value, then a breach occurs. Note that probable tailwater elevations have already been factored in during the calculation of the PUP functions, thus tailwater is not involved in determining whether or not there is a breach. Determination of high or low velocity breach is based on the head differential between WSE and tailwater. The criterion for a high velocity breach is a head differential of 13’ (this is actually a user input variable). The tailwater elevation

is determined from a triangular distribution of tailwater values (the same for all seasons), derived from minimum, maximum, and most likely value input for each component (independent of lake stage). Knowing the tailwater, the head differential is calculated, determining the breach velocity, which is required for calculation of damages.

A later addition to the model involved the inclusion of the capability of associating a reduction in lake stage with a breach in a component, to represent the loss of water in Lake Okeechobee during the breach. This is accomplished by assigning, for each component, two functions (one for high velocity breach, one for low velocity breach), giving an absolute lake stage reduction (in feet) as a function of static lake stage. Then, if a breach occurs at a component, the appropriate function is selected, and a value is determined, based on the current lake stage, by which the lake stage in the subsequent period is to be decreased, to reflect the loss of water through breach. Note that this is cumulative for the case of multiple component breaches in a period, that is, each component will contribute an incremental reduction in lake stage.

This process is repeated for all components, thus, for a given cycle of the simulation, the state of all components (breach status and velocity of breach) in the dike is known, before proceeding to the calculation of impacts/damages.

Step 3 - determine impacts/damages

At this point, it is known which components have been breached, the WSE for each breach, whether it is a high or low velocity breach, the current quarter, and the current season. As well, the last cycle at which a damage cell was inundated is known for each damage cell.

The impact calculation problem involves determining which damage cells are inundated, and the economic damages/impacts in the cell. The problem is complicated by the possibility of multiple simultaneous breaches, as individual damage cells can be affected by breaches from different components. The problem is to insure that there is no double-counting in this case. The methodology used, as originally proposed by Thomas Arnold of SAJ, is as follows:

A list of all damage cells that are inundated for the current period (the list is always set to empty at the start of each period) is maintained. Each component that has breached is processed in order. For each component, the damage cells affected are known, as are the particular damage functions for the period, land use type, and breach velocity. As each component is examined, if we encounter a new damage cell (i.e. not currently in the list), it is added to the list of affected damage cells. The appropriate stage-damage function for each damage cell is selected, based on the current component, period, and breach type, for each of the impacts (land uses). The damage value is calculated, based on static lake stage, and adjusted for prior period damages. Static lake stage is used for the determination of damages, as the storm surge component is characterized by a very short duration and although it is important in analyzing dike performance (breaching of components), it will have virtually no impact on the severity of flooding.

For each impact type, the list is examined to see if previously recorded damage (due to inundation from another component that breached) exists. If so, then the model tests to see if the damage associated with the current component is greater than the damage currently recorded for the land use type. If so, the previous value is replaced with the current value. If not, the old value is left in place (reflecting that higher damages have already been recorded for this land use in the current damage cell). The ultimate result is a list of all damage cells, and numbers for each of the impact types, expressing the maximum damages associated with inundation from a breach in any component. This information (by land use type and damage cell) is then recorded for the current period, appropriately discounted based on present value factors. In addition, the list of inundated damage cells is used to update information on the last cycle of inundation for each damage cell to the current cycle, for use in subsequent periods for damage reduction.

Step 4 - O&M Costs

O&M costs are determined for each reach as a function of static lake level, independent of breaching (O&M costs are added in even if we have a breach/repair). The global multiplicative factor is used, if a hurricane is present, to adjust the O&M costs for each reach, which are then present-valued and recorded. As previously described, the multiplier

is applied to the current period, and the three subsequent periods.

The overall process (steps 1 through 4) is then repeated for each period, for the duration of the simulation, and for the number of iterations needed to insure statistical stability of the Monte Carlo simulation.

Method of Handling of Rehabilitation

The above process is carried out for the baseline situation, and for various rehab plans. A rehab plan consists of two parts:

a) a spending schedule, given the expenditures, by cycle, associated with the particular rehab plan. This is used to calculate present values of the particular rehab plan.

b) a list of changes to component-oriented data (probability of unacceptable performance function, tailwater distribution factors, component repair cost) that will take place after the rehab is completed for that component.

In effect, a rehab is simply ‘re-setting the parameters’ for the component, at the effective date of the rehab, from those set at the start of the simulation. Note that, under this structure, different components can be rehabbed at different times, and individual components can be rehabbed more than once.

This approach effectively de-couples the physical consequences of the rehab from the economic consequences, allowing for mobilization costs, economies of scale, etc., as compared to simply assigning a rehab cost to an individual component.

Multiple rehab plans can be specified, to test different alternatives.

Data Requirements of the Model

The model is heavily data-intensive. Information is required on reaches, components, damage cells, and their inter-relationships, as well as extensive information giving damages associated with breaches.

a) Reach Information

Identifier and description of each reach, and associated O&M cost curve (O&M cost in period as a function of lake stage)

b) Component Information

Identifier, description, parent reach, length of each component, initial PUP function, repair cost for breach (high and low velocity), tailwater triangular distribution, surge multiplier for index reach surge (Gulf and Atlantic hurricanes), Gulf and Atlantic hurricane random surge distribution (triangular), high and low velocity breach lake stage reduction functions

c) Damage Cell Information

Identifier, description of each damage cell

d) Component-Damage Cell Relationship

List of all damage cells affected by breach in a particular component

e) Period (Seasonal) Information

For each period, an identifier, the number of days in the period, probability of any hurricane, probability of Gulf and Atlantic hurricane (given a hurricane)

Determination of WSE

f) Static Lake Level

Four parameters of the ARMA model are required: the mean, the serial correlation, the standard deviation, and the mean of the error term. As well, an initial lake stage to start the process, and a maximum and minimum lake stage, are required.

g) Surge at Index Component

A frequency distribution for storm surge for each hurricane type at a designated index component is required, in the form of surge vs. exceedence probability.

Determination of Breach

h) PUP

For each component, PUP functions of probability of breach as a function of WSE are required. Functions are required for initial conditions (post-repair PUP is taken to be the same as pre-repair PUP).

Functions are identified by a unique number, such that groups of components can share the same functional relationship for the same purposes, as described above. PUP functions are assumed to be time-invariant. The primary mechanism of dike degradation over time is the dissolution of subsurface limestone layers, which would result in a greater potential for piping. The geotechnical team looked into this issue earlier, and concluded that the estimated rates of limestone dissolution were sufficiently low to allow us to exclude degradation from our analysis; therefore, PUP functions will not change over time.

Economics

i) Damages

Much of the information required is in the form of functional relationships, which are expressed in a common fashion - as piece-wise linear functional representations, up to a current maximum of 50 data points per function. Each such function is assigned a unique identifier, and is associated with a particular function type (e.g. stage-damage function, PUP, etc.). This approach allows for some simplification of data input, in that the same function can be used for more than one situation, rather than requiring being re-entered separately.

For example, for stage-damage curves, truck/row crops typically will have seasonal variation, requiring separate curves for each damage cell in each quarter, while other land use types may not show seasonal variations. For consistency, a quarterly curve needs to be identified for the other land uses as well, but, will be the appropriate curve for each land use type (identifier entered 4 times), e.g.:

Damage Cell	Land Use Type	Quarter	Curve #
1a	Truck	1	1
1a	Truck	2	2

1a	Truck	3	3
1a	Truck	4	4
1a	Fruit	1	5
1a	Fruit	2	5
1a	Fruit	3	5
1a	Fruit	4	5

Piece-wise linear functions for each component, land use, potentially affected damage cell if the component breaches, season, and breach velocity, giving damages in $/stage-event as a function of static lake level;

j) Repair Costs

For each component, emergency repair costs for each breach type must be provided.

k) O&M Costs

For each reach, o&m costs ($ in period) as a function of WSE, global multiplicative factor for post-hurricane, non-breach O&M cost;

l) Rehab Costs

Costs associated with a rehab plan are specified as a spending schedule, giving the cost in specific cycles associated with the rehab (which may include quarters prior to actual construction, to capture E&D costs, etc.)

m) Reach-Related Information

For each rehab plan, post-rehab values for reach annual repair cost, and reach O&M cost functions, with effective quarter, for each reach affected by rehab plan;

n) Component-Related Information

For each rehab plan, post-rehab values for component PUP, repair cost for each breach type, and tailwater distribution parameters (minimum, maximum, and most-likely tailwater)

o) General Information

General information is required for each simulation run, and is organized into ‘scenarios’. For each scenario, information is needed as to the duration of simulation, number

of iterations, interest rate, identification of rehab plan, start year and base year of the simulation, critical head differential (for distinguishing between high and low velocity breach), initial water surface elevation, as well as various parameters that determine the level of detailed output to be provided by the simulation.

Model Outputs

The risk-based Hoover Dike Model was used to evaluate all possible combinations of components and alternatives. Alternative “B” for all of Reach 1 as described in the main report and engineering appendix was selected for the recommended plan. Table 10 shows the Scenario Simulation Comparison between the base condition and rehabilitating Reach 1.

MODEL RESULTS

The estimated first cost for rehabilitation of the 22.35-mile length of Reach 1 is $73,805,605. Interest during construction is estimated to be $6,215,846 resulting in a total project investment of $80,021,451. Based on the resulting annual cost of $5,524,973 taken at a 6.625 percent interest rate for a 50-year project life, and estimated annual benefits of $5,126,183, the resulting benefit to cost ratio is 0.928 to 1. The evaluation begins in the last part of the year 2000 and includes a planning and construction period of 23 quarters and a 50 year project life of 200 quarters. During the construction period for reach 1, none of the components are treated as separable elements. The project life begins at the time the last component is constructed. Interest during construction is accrued from the very beginning of the construction period until the last component is completed. Benefits during construction will also accrue, beginning when each component is completed. A summary of the benefits, cost, and benefit to cost ratio for rehabilitation of all of Reach 1 with alternative “B” are displayed in Table 11.

Risk and Uncertainty

To properly quantify risk and uncertainty, it is important to display and understand the probability distributions which define net economic benefits and the Benefit to Cost ratio. A probability distribution can best be understood by calculating selected moments of the distribution. The mean and standard deviation are the basic moments of a probability distribution and are displayed in Table 12 for each damage category for the without and with project condition. A comparison of the standard deviation as a percentage of the mean indicates that Operation and Maintenance costs have a relatively low degree of risk since some O+M is incurred every year whether or not there is a levee breach. The repair costs, and flood damages have a high degree of risk since these costs are totally dependent upon a project breach. The range of cost incurred in the model for the 5,000 iterations are also shown in Table 12.

Additional risk and uncertainty information pertaining to project justification is shown in Table 10. Given that the errors of the distributions are normally distributed, confidence limits about the true mean can be computed for important distributions if the sample mean and the standard deviation or the standard error of the estimate is known. A 90% confidence

boundary about the true mean of net benefits and benefit to cost ratio has been computed using the following formula:

Upper Limit = mean + 1.645*SEE
Lower Limit = mean – 1.645*SEE where:

1. 1.645 is a critical value of a t statistic for a 2 tailed test at 90% confidence.
2. SEE = Standard Error of the Estimate = Standard Deviation / square root of the sample size (5,000).

Using these formulas, the mean of net benefits is $-5,775,907 with a lower limit of $-5,775,907-1.645*644,709 = $-6,836,453 and an upper limit of $-5,775,907+1.645*644,709 = $-4,715,361. The mean benefit-to-cost ratio is 0.928 with a lower limit of 0.928-1.645*0.008 = 0.915 and an upper limit of 0.928+1.645*0.008 = 0.941.

Even though the above test indicates the population mean is not economically justified using a 90% confidence limit, other information concerning the size, shape and dispersion of the probability distributions is also very important. It is possible that two population distributions can have similar means and confidence limits but have significantly different degrees of dispersion, particularly when the sample size is large. The standard deviation is the primary measure of dispersion as discussed above. Other measures include skew, which measures the symmetry of the distribution and kurtosis which measures the “peakedness” of the distribution are not discussed here. Instead, the attributes of all these features can be graphically viewed in a cumulative frequency histogram shown in Figure 5. As expected with the relatively large degree of dispersion attributable to Repair Costs and Damage Costs, the range of results are from a benefit to cost ratio of 0.0 to a benefit to cost ratio of over 6 to 1. Note the median or midpoint, not the mean, of the distribution is represented at a 50% probability and is equal to 0.465 to 1. Our results indicate that 23% of the time a benefit to cost ratio of 0.0 occurred. This could be expected if no breaches occurred during the project life simulation. Also, if multiple breaches occur in a given simulation, a benefit to cost ratio of over 6.0 to 1 could be expected. If the sample is a true indication of the population, a benefit to cost ratio of over 5.0 to 1 could be equaled or exceeded approximately 2% of the time. Although the project mean Benefit-to-cost ratio would indicate the project is not economically justified, this type of evaluation displays

possible catastrophic consequences that could occur given the uncertainty involved with project calculations. This type of evaluation is of most interest in this type of evaluation when “loss of life” or extreme “loss of property” is involved.

Supporting Information

The following tables and graphs display the outputs from the Hoover Dike Model. Table 12 displays the Scenario Description Summary, which reports more detail of the model runs. Table 13 displays the effective cycle of the expenditures. The total cost in Figures 5 and 6 include damage cost, repair cost, and O&M cost that are referenced in Table 10 for the recommended plan. These cost vary for each iteration. The Iteration Histogram for the without and with project condition in Figures 6 and 7 show total cost and running average. During the 5000 iterations of the model, breaches randomly occur. The frequency (count) and relative frequency (percent) of the breaches are shown in Figures 8 to 11.

Table 11
Benefit-to-Cost Summary
6 5/8% Interest Rate

Benefits

Project Life Benefits	$64,642,614
Benefits During Construction	$ 9,602,930
Total Benefits – Life Cycle Model	$74,245,544
Total Annual Benefits	$ 5,126,183

Costs

Initial Cost	$73,805,605
Construction	$67,604,980
Real Estate	$ 6,200,625
Interest During Construction	$ 6,215,846
Total Project Investment	$80,021,451
Total Annual Cost	$ 5,524,973
Net Benefits	$ -398,790
Benefit-to-Cost Ratio	0.928 to 1

Table 13
Rehabilitation Information

Rehab Plan: R1ALTB-NC

Rehab alternative B in all Reach 1

Effective Cycle	Expenditure	Purpose
4	$1,160,303.57	Begin Real Estate Acquisition and PED
5	$1,160,303.57	
6	$1,160,303.57	
7	$1,160,303.57	
8	$1,160,303.57	
9	$1,160,303.57	
10	$1,160,303.57	End Real Estate Acquisition and PED
11	$5,185,537.89	Begin Reach 1C-a Construction
12	$5,185,537.89	
13	$5,185,537.89	End Reach 1C-a, Start 1C-c
14	$5,185,537.89	
15	$5,185,537.89	End Reach 1C-c, Start 1C-b
16	$5,185,537.89	
17	$5,185,537.89	End Reach 1C-b Construction
18	$5,185,537.89	Start Reach 1B Construction
19	$5,185,537.89	
20	$5,185,537.89	End Reach 1B Construction
21	$5,185,537.89	Start Reach 1A Construction
22	$5,185,537.89	
23	$3,457,025.26	End Reach 1 Construction Base

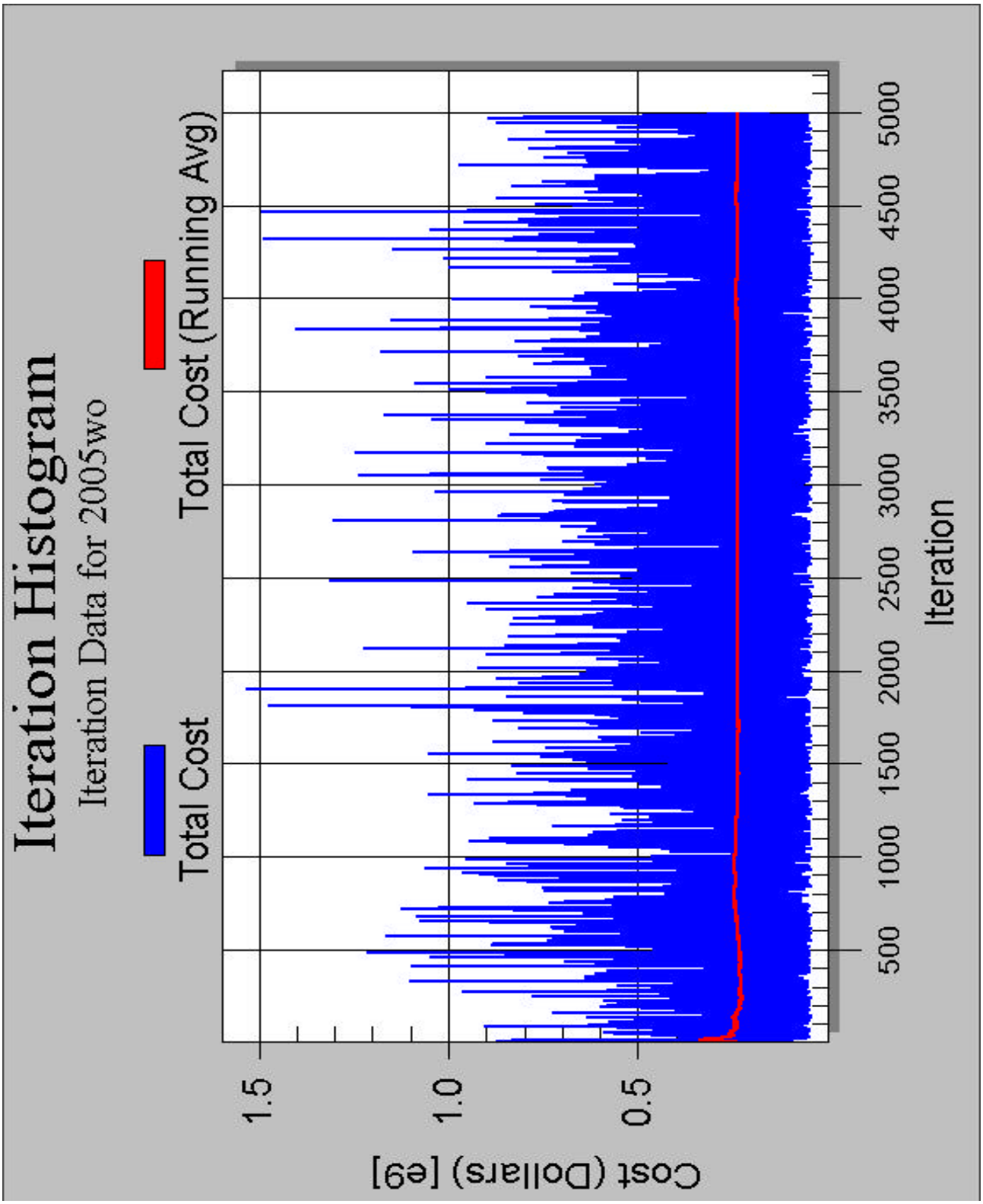

Iteration Histogram
Iteration Data for 2005wo
Total Cost
Total Cost (Running Avg)
Cost (Dollars) [e9]
1.5
1.0
0.5
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Iteration

Figure 6

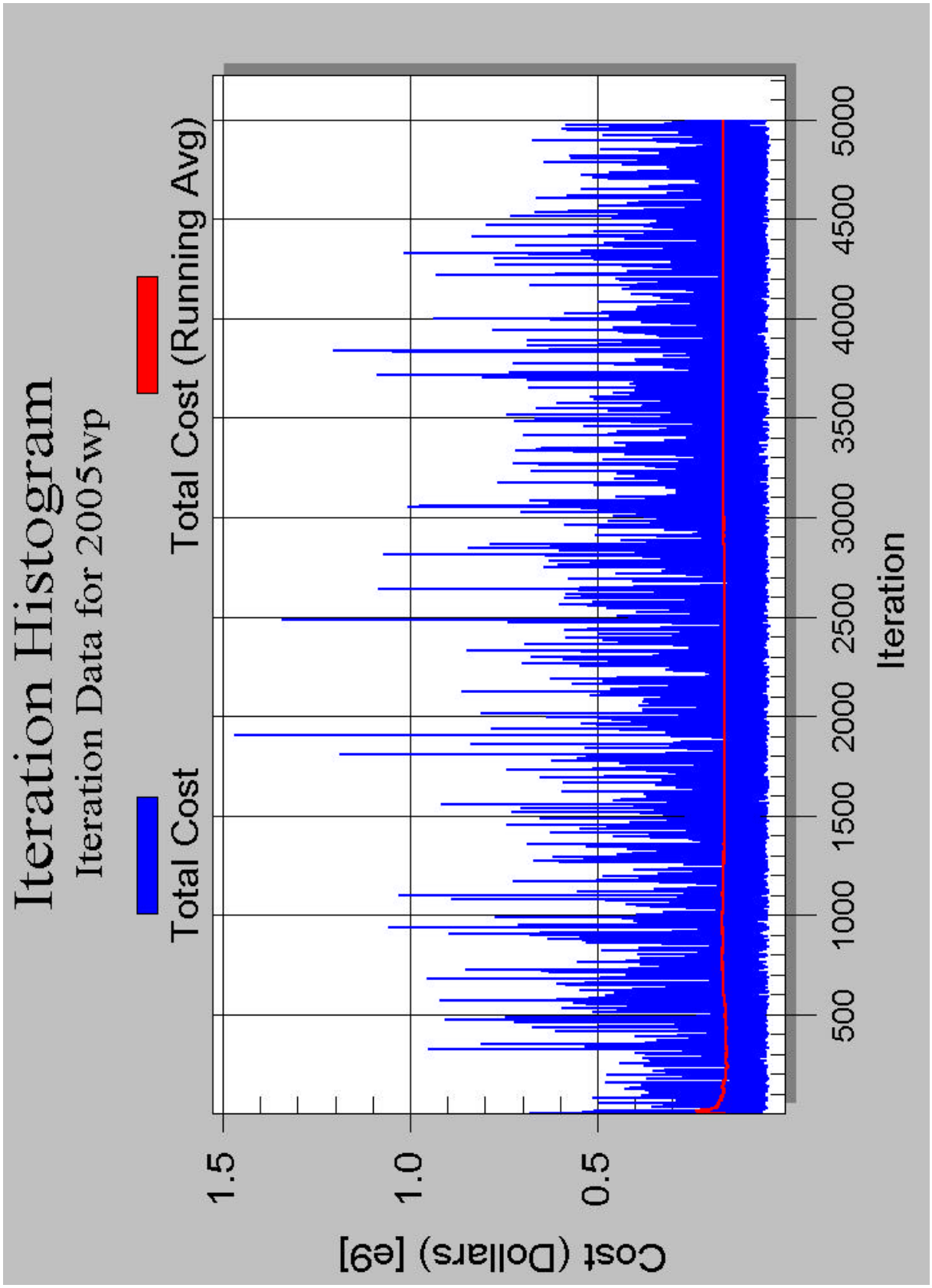

Iteration Histogram
Iteration Data for 2005wp
Total Cost
Total Cost (Running Avg)
Cost (Dollars) [e9]
1.5
1.0
0.5
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Iteration

Figure 7

HERBERT HOOVER DIKE
MAJOR REHABILITATION EVALUATION REPORT

FINAL
ENVIRONMENTAL IMPACT STATEMENT

FINAL ENVIRONMENTAL IMPACT STATEMENT

The Herbert Hoover Dike
Major Rehabilitation Evaluation Report

Proposed Action: Four alternatives are proposed to reduce the probability of a breach of Reach One of the Herbert Hoover Dike which surrounds Lake Okeechobee in south central Florida.

Type of Statement: Final Environmental Impact Statement

Responsible Agencies: U.S. Army Corps of Engineers

Lead Agency: U.S. Army Corps of Engineers

Abstract

The Herbert Hoover Dike (HHD) around Lake Okeechobee in southern Florida was originally constructed as a series of embankments by local interests, circa 1915, in order to provide flood protection to the surrounding communities, and controlled irrigation for local agriculturists. These embankments were improved to the current levee system by the U.S. Army Corps of Engineers (USACE) during the 1930s and 1940s, and major culvert modifications were accomplished in the 1970s. Since then, only as-needed repairs have been made to the HHD. Recent high water events have suggested the need for major rehabilitation, with the occurrence of several boils and pipings around the Dike. The USACE is preparing a series of HHD Major Rehabilitation Evaluation Reports (MRER) which document seepage and stability concerns along the HHD system, and provides rehabilitation options. This initial MRER focuses on Reach One, the southeastern portion of the HHD, and proposes four alternative actions for rehabilitation. The alternatives include (a) no action, which is defined as not making improvements to Reach One, and would cause no physical changes in the study area, (b) construction of a stability berm at the landside toe of the levee and installing culverts with automatic/manual gates and pumps to control the water level in the ditches, (c) construction of an upstream impervious cutoff wall and a landside stability berm at the toe of the levee which would impede groundwater flow and control underseepage, and (d) installation of a seepage berm with relief trench along the lower portion of the landward toe of the embankment, (the Preferred Alternative). A Final Environmental Impact Statement (FEIS) has been prepared to examine the effects of the proposed actions. Based on the analysis of the FEIS, construction of the selected alternative would cause short-term disturbance to and displacement of components of the human and natural environments. Operation of the project would improve slope stability and seepage control and reduce the probability of a breach of Reach One of the HHD. The MRER proposes remedial measures for slope stability and seepage control to reduce the probability of a breach of the HHD at Reach One.

Note: The official closing date for the receipt of comments is September 24, 1999. This report is also available on our web site at: *http://www.saj.usace.army.mil/pd/env-doc.htm.*

For Further Information Contact: Mr. Olice Carter, U.S. Army Corps of Engineers, Jacksonville District, Planning Division P.O. Box 4970 PD-ES, Jacksonville, Florida 32232-0019, Tel: 904/232-1140.

TABLE OF CONTENTS

LIST OF TABLES IV
LIST OF TABLES V
LIST OF ANNEXES VI
LIST OF ACRONYMS VII
1.00 SUMMARY 1
1.01 Background 1
1.02 Scope 1
1.03 Alternatives 2
1.04 Major Conclusions 3
2.00 THE PURPOSE AND NEED FOR THE ACTION 4
2.01 Introduction 4
2.02 Study Authority 4
3.00 ALTERNATIVES CONSIDERED 5
3.01 No Action Alternative 5
3.02 Alternative Actions 5
3.03 Preferred Alternative 6
3.04 Environmentally Preferred Alternative 7
3.05 Mitigation Measures 8
4.00 AFFECTED ENVIRONMENT 8
4.01 Introduction 8
4.02 Location and Climate 8
4.03 Topography, Geology and Soils 10
4.04 Hydrology 12
4.05 Water Supply 13
4.06 Water Quality 14
4.07 Water Management 15
4.08 Vegetation and Cover Types 17
4.09 Wetlands 18
4.10 Fish and Wildlife 20
4.11 Threatened and Endangered Species 22
4.12 Noise 30
4.13 Air Quality 30
4.14 Hazardous, Toxic and Radioactive Wastes 31
4.15 Land Use 31
4.16 Aesthetic Resources 34
4.17 Recreational Resources 36
4.18 Cultural Resources 38
4.19 Socioeconomics 39
5.00 ENVIRONMENTAL CONSEQUENCES 40
5.01 Introduction 40
5.02 Locationand Climate 45
5.03 Topography, Geology and Soils 45
5.04 Hydrology 46
5.05 Water Supply 49

5.06 Water Quality 52
5.07 Water Management 52
5.08 Vegetation and Cover Types 53
5.09 Wetlands 54
5.10 Fish and Wildlife 55
5.11 Threatened and Endangered Species 57
5.12 Noise 60
5.13 Air Quality 61
5.14 Hazardous, Toxic and Radioactive Wastes 62
5.15 Land Use 62
5.16 Aesthetic Resources 69
5.17 Recreational Resources 70
5.18 Cultural Resources 72
5.19 Socioeconomics 73
5.20 Cumulative Effects 74
5.21 Unavoidable Adverse Effects 74
5.22 Relationship Between Short-Term Uses and Long-Term Productivity 77
5.23 Irreversible or Irretrievable Commitment of Resources 77
6.00 PUBLIC COORDINATION 77
7.00 COMPLIANCE WITH ENVIRONMENTAL REQUIREMENTS 77
7.1 NATIONAL ENVIRONMENTAL POLICY ACT 77
7.2 FISH AND WILDLIFE COORDINATION ACT 78
7.3 ENDANGERED SPECIES ACT 79
7.4 NATIONAL HISTORIC PRESERVATION ACT 80
7.5 CLEAN WATER ACT 80
7.6 CLEAN AIR ACT 80
7.7 COASTAL ZONE MANAGEMENT ACT 80
7.8 FARMLAND PROTECTION POLICY ACT 80
7.9 WILD AND SCENIC RIVER ACT 81
7.10 ESTUARY PROTECTION ACT 81
7.11 FEDERAL WATER PROJECT RECREATION ACT 81
7.12 RESOURCE CONSERVATION AND RECOVERY ACT 81
7.13 TOXIC SUBSTANCES CONTROL ACT 81
7.14 MARINE PROTECTION, RESEARCH, AND SANCTUARIES ACT 81
7.15 RIVERS AND HARBORS APPROPRIATION ACT 81
7.16 COASTAL BARRIER RESOURCES ACT 81
7.17 Section 904 of the 1986 WATER RESOURCES DEVELOPMENT ACT 82
7.18 Section 307 of the 1990 WATER RESOURCES DEVELOPMENT ACT 82
7.19 E.O. 11988, FLOODPLAIN MANAGEMENT 82
7.20 E.O. 11990, PROTECTION OF WETLANDS 82
7.21 E.O. 12114, ENVIRONMENTAL EFFECTS ABROAD OF MAJOR FEDERAL ACTIONS 82
7.22 E.O. 12898, ENVIRONMENTAL JUSTICE 83
8.00 LIST OF PREPARERS 83
9.00 REFERENCES CITED 85
10.00 LIST OF RECIPIENTS 88
11.0 INDEX 97
12.00 ANNEXES 103

LIST OF TABLES

TABLE NO.	TITLE	PAGE
1	Average Annual Temperature and Rainfall for Locations Surrounding Lake Okeechobee	9
2	Listed Faunal Species Occurring in Martin and Palm Beach Counties, Florida	23
3	Listed Plant Species Occurring in Martin and Palm Beach Counties, Florida	29
4	1996 Population Estimate for Communities Surrounding Lake Okeechobee	33
5	Summary of Environmental Consequences 42	
6	List of Preparers	83

LIST OF FIGURES

FIGURE NO.	TITLE
1	Location Map
2	Existing Conditions, Typical Cross-Section
3	Alternative No. 1, Typical Cross-Section
4	Alternative No. 2, Typical Cross-Section
5	Alternative No. 3, Typical Cross-Section
6	Culvert System
7	Land Use, Reach One
8	Land Use, Reach 1A
9	Land Use, Reach 1B
10	Land Use, Reach 1C

LIST OF ANNEXES

A - Fish and Wildlife Coordination Act Report

B - Florida Natural Areas Inventory Report

C - Section 404(b)(1) Evaluation

D - Coastal Zone Management Consistency Evaluation

LIST OF ACRONYMS

bls	BELOW LAND SURFACE
CFR	CODE OF FEDERAL REGULATIONS
dB	DECIBELS
EAA	EVERGLADES AGRICULTURAL AREA
EIS	ENVIRONMENTAL IMPACT STATEMENT
EPA	US ENVIRONMENTAL PROTECTION AGENCY
FDACS	FLORIDA DEPARTMENT OF AGRICULTURE AND CONSUMER SERVICES
FDEP	FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
FDOT	FLORIDA DEPARTMENT OF TRANSPORTATION
FEIS	FINAL ENVIRONMENTAL IMPACT STATEMENT
FGFWFC	FLORIDA GAME AND FRESHWATER FISH COMMISSION
FMSF	FLORIDA MASTER SITE FILE
FNAI	FLORIDA NATURAL AREAS INENTORY
HHD	HERBERT HOOVER DIKE
HGS	HURRICANE GATE STRUCTURE
MRER	MAJOR REHABILITATION EVALUATION REPORT
MWL	MINIMUM WATER LEVEL
NEPA	NATIONAL ENVIRONMENTAL POLICY ACT
NGVD	NATIONAL GEODETIC VERTICAL DATUM
NHPA	NATIONAL HISTORIC PRESERVATION ACT of 1966, as amended
NRHP	NATIONAL REGISTER OF HISTORIC PLACES
PSD	PREVENTION OF SIGNIFICANT DETERIORATION
SFWMD	SOUTH FLORIDA WATER MANAGEMENT DISTRICT
SHPO	STATE HISTORIC PRESERVATION OFFICER
USACE	US ARMY CORPS OF ENGINEERS
USFWS	UNITED STATES FISH AND WILDLIFE SERVICE
WCA	WATER CONSERVATION AREA

1.00 SUMMARY

1.01 Background

The Herbert Hoover Dike (HHD) system consists of a series of levees, gated culverts, and locks which encompass Lake Okeechobee, Florida. Construction of this dike began early this century as the first embankments around the Lake were constructed by local interests and were primarily composed of muck, sand, shell, and marl from adjacent borrow canals. These embankments were constructed circa 1915. During the 1930s, Federal interest was initiated as a result of the hurricane tides of 1926 and 1928 overtopping the original embankment. The River and Harbor Act, approved 3 July 1930, authorized the construction of 67.8 miles (109 kilometers (km)) of levee along the south shore of the Lake and 15.7 miles (25.3 km) of levee along the north shore. The typical crest height of those levees ranged from 32 to 35 feet (9.8 meters (m) to 10.7 m) above the National Geodetic Vertical Datum of 1929 (NGVD). These levees were constructed by the USACE between 1932 and 1938. A major hurricane in 1947 prompted the need for additional flood protection work in Florida. In response, Congress passed the Flood Control Act of 1948 authorizing the first phase of the comprehensive plan for flood protection and other water control. Later, major culvert modifications were accomplished in the 1970s.

Recently, only as-needed repairs have been made to the HHD. However, signs of instability such as boils and pipings have occurred during recent years which indicate that major renovations are now necessary, especially along the southeastern portion of the HHD.

The existing HHD system is approximately 143 miles (230 km) long, and is divided into eight segments or "Reaches" for planning purposes. The southeastern segment, Reach One, is the focus of the present study. Reach One is an approximately 22.4 miles (36 km) long segment of the HHD located along the southeast portion of the Lake. This segment extends from the St. Lucie Canal at Port Mayaca, south to the Hillsboro Canal at Belle Glade (Figure 1).

1.02 Scope

The Scoping process as outlined by the Council on Environmental Quality is being utilized to involve Federal, State, and local agencies, affected Indian Tribes, and other interested parties. A Scoping Letter was sent in September of 1996 to interested Federal, State and Local agencies, interested organizations and the public requesting their comments and concerns regarding issues they felt should be addressed in the FEIS.

Responses to the Scoping Letter included concerns regarding impacts to scenic trails, wetlands, threatened and endangered species, and navigation. Additionally, erosion control and increased lake water level were of expressed interest.

The Florida Department of Transportation (FDOT)is concerned about the possible impact of the proposed project on the Lake Okeechobee Scenic Trail segment of the Florida National Scenic Trail, especially with respect to trail alignment, trailhead configurations, location of trail amenities, levee crown surface improvements, and the placement of bicycle/pedestrian bridge structures over navigable locks.

The Florida Department of Environmental Protection (FDEP) recommends that the project include the removal of *Melaleuca quinquenervia* from the project area. The FDEP further recommends an evaluation of the direct and secondary impacts of this project on the existing wetlands where levees and structures are to be constructed, including a consideration of the current condition and function of those wetlands. Mitigation should be included in the project to replace lost wetland functions. Water Quality Certification through the state's Environmental Resources Permitting process would be required for new water flow regulation structures and levees. The FDEP recognizes that construction and hydroperiod modification may threaten populations of the Okeechobee Gourd *(Cucurbita okeechobeensis)*. The FDEP also recommends that land forms and water regulation strategies be structured to meet the ecological restoration objectives of the greater Everglades system.

The Florida Game and Fresh Water Fish Commission (FGFWFC) remarks that the HHD provides major nesting habitat for Lake Okeechobee's aquatic turtles, and the impacts of any dike hardening proposals on turtle nesting habitat should be thoroughly documented and analyzed. Further, the Dike provides nesting habitat for the burrowing owl *(Speotyto cunicularia floridana)*, a state listed species of special concern. The FGFWFC recommends that a detailed population study of burrowing owls on the entire dike system, along with an analysis of the impact of dike improvement options on this population be conducted.

The Okeechobee Waterway Association of Clewiston, Florida, and the Marine Industries Association of Florida, Inc. of Stuart, Florida each responded to the Scoping process with concerns regarding access to and navigation of the Lake, and any impact to these issues that a rehabilitation effort might cause. Erosion control of the dike system was an expressed concern by the City of Pahokee, and well as that of private citizens in the area of Reach One.

During preparation of this FEIS, the responses to the Scoping Letter, were addressed. Further, in order to determine the potential environmental impacts associated with implementing rehabilitative measures along Reach One, a detailed examination of proposed alternatives and the existing environment was accomplished. Specific issues addressed include hydrology, water management, wetlands, flora and fauna, land resources, socio-economics, and others. Additionally, these issues were addressed in lesser detail for the remaining Reaches comprising the entire HHD system.

1.03 Alternatives

The USACE is proposing several alternatives to reduce the probability of a breach of Reach One of the HHD. The No Action Alternative would involve no improvements to the embankment at

Reach One. Alternative No. 1 would involve the construction of a stability berm, improvements to existing drainage ditches, and regulation of the water level in the ditch system. Alternative No. 2 proposes the construction of an impervious cutoff wall and landside stability berm. Alternative No. 3, the Preferred Alternative, involves the installation of a seepage berm with relief trench along the landward toe of the embankment.

1.04 Major Conclusions

The No Action Alternative would cause no physical changes in the study area, nor would it provide acceptable compliance with current regulation requirements of safety factors relative to dike stability. This alternative does not provide adequate protection from the seepage and stability problems that threaten critical areas of Reach One of the HHD. Furthermore, there could be significant socioeconomic implications, as it could allow for continued degradation of the HHD's stability leading to a breach of the Dike. The potential for loss of life and property from a breach is significant to residents along the HHD as well as those that utilize the Dike for recreation. Human well being and safety may be severely impacted and damage to property significant in the event of a levee breach between the Hillsboro and West Palm Beach Canals especially as there is significant urban and residential development very close to the HHD in this area. Finally, in the event of a high velocity breach in the general area between the St. Lucie Canal and the L-8 Canal, in combination with high lake levels, a downstream discharge towards the St. Lucie Estuary would likely cause significant adverse impacts to the estuarine ecosystem. It is reasonable to assume that any levee breach of the HHD system would likely be preceded by significant freshwater releases through the S-308 at Port Mayaca in anticipation of climatic and lake inflow conditions raising water levels and threatening the stability of the HHD. As a result of the above potential adverse scenarios this action was not selected as the preferred alternative.

Implementation of Alternative No. 1 may potentially result in some adverse impacts to the existing environment. Minimal impacts to the soil, vegetation, and wetlands during excavation and fill activities may occur. Moderate impacts to water resources may result from modifications involved in this alternative, and minimal alteration of foraging habitat for wading birds, and listed species along the toe ditch would occur as well. Further, impacts to local agriculture are possible if the local drainage districts and farmers are relieved of the control of irrigation water, through this alternative. This alternative does not provide adequate protection from the seepage and stability problems that threaten critical areas of Reach One of the HHD, as a result, this action was not selected as the preferred alternative.

Implementation of Alternative No. 2 may potentially result in some adverse impacts to the existing environment. These include minimal soil, vegetation, and wetland disruption during excavation and fill activities. Moderate impacts to water resources may result from modifications involved in this alternative, and minimal alteration of foraging habitat for wading birds, and listed species along the toe ditch would occur. Further, impacts to local agriculture are possible if the hydrology is reduced by the cutoff wall proposed by this alternative. This alternative would improve slope stability and seepage

control along Reach One, however, due to the intensive effort of this action and the effects of the cutoff wall to the local groundwater regime, this action was not selected as the preferred alternative.

Implementation of Alternative No. 3 would cause minimal short-term disturbance to, and displacement of, components of the human and natural environments. These include minimal soil, vegetation, and wetland disruption during excavation and fill activities. Only minimal effects to existing water resources, and foraging habitat for wading birds and listed species is expected as well. Implementation of this alternative would improve slope stability and seepage control and reduce the probability of a dike breach within Reach One. Due to these factors, Alternative No. 3 has been selected as the preferred alternative.

2.00 THE PURPOSE AND NEED FOR THE ACTION

The general goal of the HHD MRER is to provide a reliable embankment system around Lake Okeechobee to contain the lake waters for flood protection, water supply, and navigation.

An unreliable embankment system, such as that which currently exists along Reach One of the HHD, could allow for a failure of the system to contain lake waters. Such a failure could result in loss of life, property, and habitat. A reasonable and effective rehabilitative effort is required to eliminate this possibility.

2.01 Introduction

The HHD system consists of approximately 143 miles of levee, surrounding Lake Okeechobee, Florida. The study area around the Lake, comprises five counties: Glades, Hendry, Martin, Okeechobee, and Palm Beach Counties. The study area has been divided into eight segments or reaches. Reach One, the location of the present study, is 22.4 miles (36 km) long and extends from St. Lucie Canal at Port Mayaca, on the east side of the Lake, to the Hillsboro Canal at Belle Glade (Figure 1).

The HHD, constructed largely of local material (e.g., mud, muck, sand, shell fragments) and with a porous limestone bedrock underlying the levee, has been experiencing a high degree of underseepage and seepage through the levee. This seepage resulted in several boils and pipings during the 1995 and 1998 high water events. The most significant occurrences were found along Reach One.

The USACE is conducting a structural and stability analysis study on the HHD which will culminate in an MRER for Reach One. Based on this evaluation, the USACE is proposing several alternatives to reduce the probability of a breach of Reach One of the HHD.

2.02 Study Authority

The Flood Control Act (Act), approved by Congress on 30 June 1948, authorized the first phase of a comprehensive plan to provide flood protection and other water control benefits in central and south Florida. The Act included measures for improving control of Lake Okeechobee by constructing or modifying the spillways and other structures, and enlarging the Lake Okeechobee levees to provide the intended flood protection, water storage and water supply. Levee seepage and stability have a direct effect on the capability of the levee to provide the authorized protection. The authorization for levee repairs and modifications of the Flood Control Act of 1948 justify the proposed renovation to Reach One of the HHD.

3.00 ALTERNATIVES CONSIDERED

The four alternatives currently considered are the No Action Alternative, Alternative No. 1, Alternative No. 2, and Alternative No. 3 (Preferred Alternative). The details of each alternative are presented below.

3.01 No Action Alternative

The No Action Alternative is defined as not making improvements to Reach One. This would cause no physical changes in the study area (Figure 2). The No Action Alternative would not provide acceptable compliance with current regulation requirements of safety factors relative to dike stability. Without acceptable improvements to Reach One of the HHD, the safety of the surrounding human and natural environment may be severely impacted with subsequent effects upon the local and regional economies. A continuation of pipings and boils occurring in this area would increase the potential for local flooding due to rainfall and runoff. In the event of a total breach significant impacts to human life, existing soils, vegetation, water resources, habitat, threatened and endangered species, agriculture and property may result.

The future without project, or the No Action Alternative, does not provide a long-term solution to the seepage and stability problems existing along Reach One. For these reasons, the No Action Alternative is not feasible and will not be considered further.

3.02 Alternative Actions

Alternative No. 1

This alternative includes increasing the water level in the drainage ditches and the construction of a stability berm at the landside toe of the levee (Figure 3). Alternative No. 1 would improve the existing drainage ditches by cleaning out the ditches and re-grading the ditches. Culverts with automatic/manual gates and pumps would be installed to control the water level in the ditches.

During critical high water periods, the water level in the ditches would be raised in order to limit the differential head across the levee. Raising the water levels in the ditches would increase the local flooding potential due to rainfall and runoff. Presently, most of these ditches are controlled by the local drainage districts and farmers.

Additionally, 3 feet (ft) to 4 ft (0.9 m to 1.2 m) of peat would be excavated from the landside toe of the levee. Then a 25 ft (7.6 m) wide, 5 ft (1.5 m) deep stability berm would be constructed. The stability berm would allow access to the toe of the embankment and ditches for inspection.

This alternative does not provide adequate protection from the seepage and stability problems that threaten critical areas of Reach One of the HHD.

Alternative No. 2

Alternative No. 2 involves an upstream impervious cutoff wall and a landside stability berm at the toe of the levee (Figure 4). The cutoff wall would impede groundwater flow. This is the most positive method of underseepage control because it reduces both uplift pressure and through seepage. The wall would consist of a 3 ft (0.9 m) wide, 60 ft (18 m) deep excavation filled with soil-bentonite or soil-cement mixture. The top of the wall would be at an approximate elevation of 25 ft (7.6 m). The cutoff wall would affect the upper aquifer and may lower the groundwater table, thereby affecting local adjacent farms. A landside stability berm as described in Alternative No. 1 would also be constructed. Due to the intensive construction effort and cost of this action and the effects of the cutoff wall to the local groundwater regime, this action was not selected as the preferred alternative.

3.03 Preferred Alternative Action

Alternative No. 3

Alternative No. 3 is the Preferred Alternative and involves the installation of a seepage berm with relief trench and a French drain system along the landward toe of the embankment (Figure 5).

In areas where the embankment toe rests on a peat layer, construction of the seepage berm would begin with excavation of peat material from the landside toe. The extent of excavation would be minimal (typically wide 25 ft (7.6 m), and 4 ft (1.2 m) deep) and would involve only the area in the immediate vicinity of the embankment toe. No excavation would be performed at higher elevations of the embankment slope.

The seepage berm would be constructed along the lower portion of the embankment toe to a point approximately 40 ft (12 m) landward of the intersection of the toe with existing terrain. In areas where a toe ditch now exists, the ditch would be replaced by the proposed seepage berm. The berm

would consist of a 1-foot (0.3 m) thick layer of filter sand overlain by a 5 ft (1.5 m) thick layer of filter stone. The upper surface of the berm would be covered with a sand/soil layer to allow the establishment of grasses. The landward side of the berm would contain a 48-inch (in) (1.2 m) diameter perforated culvert wrapped in geotextile fabric. An approximately 2 ft (0.6 m) wide, 25 ft (7.6 m) deep relief trench would be excavated immediately below the culvert along its entire length. The berm itself would prevent the piping of sands and silts from the embankment and its foundation. The relief trench is designed to control uplift pressures and prevent seepage and piping flows from extending landward of the embankment.

The perforated culvert system would collect and convey seepage flows to controlled outlets which empty into existing drainage canals. At the intersection of the culvert system with drainage canals, simple gated culverts (controlled outlets) would be installed to provide control of flows from the culverts into the canals. Each control structure would typically consist of a vertical half culvert, equipped with a weir or stop-log riser that allows some control over the rate of effluent flows from the seepage berm culvert system into the canal. There are eight points of perforated culvert and canal intersections (Figure 6).

Along the top of the berm, directly above the perforated culvert, drop inlets would be installed on 400 ft (122 m) centers to drain surface runoff into the 48 in (1.2 m) culvert. Drop inlets consist of an opening, with a protective grate, that collects surface runoff from the berm and empties it into the culvert system. Typical dimensions of a drop inlet opening are 2 ft by 2 ft (0.6 m by 0.6 m). A 1-foot to 2-foot (0.3 to 0.6 m) deep drainage swale would also be constructed along the landward toe of the berm to collect and convey surface drainage from each side of the drainage berm.

Just south of culvert C-12, a rock quarry lies adjacent to the landside toe of the embankment (Figures 7 and 10). The quarry is 3,000 ft (914 m) long, 10 ft to 20 ft (3 m to 6 m) deep, and contains water. In this area, there is insufficient space between the embankment toe and quarry bank to allow construction of the proposed drainage berm; therefore, filling of the quarry would be necessary prior to construction in this area. Fill material (approximately 500,000 cubic yards (382,000 cubic meters)) would be obtained from existing USACE stockpiles of suitable spoil material and/or from a licensed commercial upland source. After quarry filling, the drainage berm would be constructed on the reclaimed lands.

3.04 Environmentally Preferred Alternative

The Environmentally Preferable Alternative is that alternative, which if implemented, would result in the least conceivable adverse impacts to the existing environment.

A thorough analysis of the Affected Environment and potential Environmental Consequences as put forth in Sections 4.00 and 5.00 of this document, respectively, has led to the conclusion that Alternative No. 3 is the Environmentally Preferable Alternative for rehabilitation of Reach One of the HHD. Alternative No. 3, if implemented, would result in only minimal adverse impacts to the majority

of the affected environmental components. Moderate impacts to threatened and endangered species through filling of landward toe ditches, and temporary displacement of an existing low voltage transmission line are the only exceptions.

Alternative Nos. 1 and 2 are still viable alternatives for rehabilitation of Reach One of the HHD. These options would likely result in overall minimal to moderate impacts to the existing environment. Environmental components which deem these alternatives less desirable are largely related to increased expected impacts to water resources in this area.

The No Action Alternative is an unsatisfactory alternative, as it allows for continued degradation and instability of the HHD system. As presented in this document, the loss of life, property, and environmental resources stand to be great if no action is taken and a major failure of the system should occur.

3.05 Mitigation Measures

Measures to mitigate against environmental perturbations include erosion and surficial water control. Flexible fencing and/or hay may be used to control erosion during construction activities. Severe surficial water imbalances would be restricted to a minimum by performing the operations during the driest parts of the year.

4.00 AFFECTED ENVIRONMENT

4.01 Introduction

Discussions of the affected environment included in this section describe the environment of Lake Okeechobee and Reach One as it currently exists. Environmental components addressed include physical, biological, social, and economic resources. This Section does not present effects, but puts forth the baseline environment for comparisons in Section 5, Environmental Consequences. Components of the affected environment are presented below.

4.02 Location and Climate

Location

Lake Okeechobee is located in south central Florida, and occupies portions of Glades, Hendry, Martin, Okeechobee, and Palm Beach Counties. The Lake has an area of approximately 720 square miles (1865 square kilometers) with its approximate center near 26° 56′ 55″ north latitude, 80° 56′ 34″ west longitude.

Reach One, the site of the proposed project, is located along the southeastern portion of the Lake. It extends 22.4 miles (36 kilometers) from the St. Lucie Canal near Port Mayaca in Martin County, to the Hillsboro Canal near Belle Glade in Palm Beach County (Figure 1).

Climate

Lake Okeechobee is located in a region characterized by a humid subtropical climate. Summers are long and warm typified by frequent afternoon convection storms. Winters are mild with the temperatures rarely falling below freezing. The summer months constitute the wet season, the winter months the dry season. Prevailing winds in the Lake Okeechobee area vary from southeast to east-northeast, except during winter when winds are from a northwesterly direction. The annual mean wind speed is 9.4 miles per hour (15 km per hour) (USDA, 1978).

The most significant factor affecting the climate of the Lake Okeechobee area is its proximity to large water bodies. Although located on a parallel occupied primarily by arid lands around the world, the maritime effects of the Gulf of Mexico and the Atlantic Ocean on this area result in a significantly modified climate. The climate of lands immediately surrounding the Lake are even further influenced by the Lake itself. Because the Lake stays cooler than the surrounding land during warm days, and warmer than the land at night, the pressure differences and consequent winds significantly affect the local environment. The cooler lake temperatures during the day have a suppression affect on cloud formation over and near the Lake. On remote imagery, the Lake often appears as a hole in the cloud cover, sometimes being cloud free when surrounding areas contain significant cloud cover. Consequently, there is generally up to a 30 percent reduction in annual rainfall over and west of the Lake compared to surrounding areas (Henry et al, 1994). Climate data from points around Lake Okeechobee are presented in Table 1.

The climate of Reach One is comparable to the climate of the Lake Okeechobee region. Data from the Belle Glade Experiment Station are an accurate representation of the conditions typical of Reach One.

Table 1
Average Annual Temperatures and Rainfall
For Locations Surrounding Lake Okeechobee
1961 - 1990
(Southeast Regional Climate Center)

Annual Average:	MinTemp (F)	MaxTemp (F)	AvgTemp (F)	AvgPrcp (Inches)
Canal Point, USDA	62.5°	83.7°	73.1°	50.1°
Belle Glade, ExpStn	61.8°	83.3°	72.6°	51.6°
Clewiston, USACE	64.5°	83.5°	74.0°	45.0°
Moore Haven, Lock 1	62.5°	83.4°	73.0°	43.1°

Okeechobee, Gate 6	63.3°	81.4°	72.1°	NA

NA = Not Available

4.03 Topography, Geology and Soils

Topography

The topography of the lands surrounding Lake Okeechobee is flat to gently sloping with an elevation ranging from 10 ft to 20 ft (3 m to 6 m) NGVD. The area can be divided into three physiographic regions: (1) the Sandy Flatlands to the west and north of the Lake which slope gently towards the Lake; (2) the Eastern Flatlands to the east of the Lake which slope gently towards the Lake; and (3) the Everglades Region to the south, southeast, and southwest of the Lake which generally slope away from the Lake. (Klein et al, 1964; Lichtler, 1960).

Reach One exists entirely within the Everglades physiographic region with typical surface elevations ranging from 12 ft to 14 ft (3.6 m to 4.3 m) NGVD. The elevation at the crest of the HHD in Reach One ranges from 37.8 ft to 38.3 ft (11.5 m to 11.7 m) NGVD. Landward of the Dike crest, the general slope of the Dike is from 20 to 33 percent (%), waterward the general slope is 17%. The mean Lake Okeechobee water surface elevation is 14.5 ft (4.4 m) NGVD, although this level varies from one side of the Lake to another depending upon wind speed and direction. Depths of the Lake within 1 mile (1.6 km) of the Dike range from 1 ft (30 cm) to 11 ft (3.4 m) below the mean water level in natural areas, and are approximately 38 ft (11.6 m) below mean water level in the rim canal.

Geology

The geological formations underlying Lake Okeechobee can be divided into two distinct groups, one which occurs in the Sandy and the Eastern Flatlands region, and one which occurs in the Everglades region.

In the Flatlands region, Pamlico Sand composed primarily of sand and limestone of the Late Pleistocene, occurs from 0 to 10 ft (0 m to 3 m) below land surface (bls). The Anastasia Formation occurs from 10 ft to 230 ft (3 m to 70 m) bls and consists of sand, limestone, and shell beds of the Pleistocene. The next layer of material is the Caloosahatchee Marl which occurs from 230 ft to 330 ft (70 m to 100 m) bls and is made up of shelly sands and shell marl of the Pliocene. Together, the Anastasia Formation and Caloosahatchee Marl comprise the water table or nonartesian aquifer of this region. Underlying these porous layers, there are a series of formations with lower permeability which act as a confining layer. The uppermost of these layers is the Tamiami Formation which occurs from 330 ft to 400 ft (100 m to 123 m) bls. The Tamiami formation is comprised of marly sand, marl, and shell beds of the Miocene. The Hawthorn Formation occurs from 400 ft to 890 ft (123 m to 271 m) bls, and is composed of clayey and sandy marl of the Miocene. The Tampa Formation exists from 890 ft to 940 ft (271 m to 287 m) bls, and is made up of limestone and some marl of the early Miocene. The Tampa Formation exhibits somewhat higher permeability yielding some artesian water.

The remaining known layers are composed of limestone and yield water under artesian conditions with sufficient pressure to flow to the surface. This principal artesian aquifer (Floridan Aquifer) underlies all of Florida and part of southeast Georgia. The layers of this aquifer are the Suwannee Limestone, Ocala Group, and Avon Park Limestone Formations which date back to the Oligocene, Late Eocene, and Late middle Eocene periods, respectively. While the Suwannee Limestone Formation occurs from 940 ft to 1000 ft (287 m to 305 m) bls, the remaining layers vary from 1000 ft (940 m) bls to undetermined depths.

In the Everglades region, the geological formation found at the surface is a thick covering of organic soils. These organic materials started accumulating about 5,000 years ago and range in thickness from 3 ft to 10 ft (1 m to 3 m). The Fort Thompson formation occurs from 8 to 30 ft (2.4 m to 9 m) bls, and is composed of marine and fresh-water sands, marls, limestone, and shell beds of the Pleistocene.

The organic layer and the Fort Thompson Formation of the Everglades region are found in place of the Pamlico Sand and Anastasia Formations of the flatlands. Below these strata, the series of occurrence, composition, and permeability corresponds between the two regions, differing only in relative depths bls (Schroeder et al, 1954).

Reach One lies entirely within the Everglades region, of which, the geological framework is described above.

Soils

For general descriptive purposes, the soils found in the Lake Okeechobee region are grouped based on distinctive patterns of soils, relief, drainage, and natural landscape. There are three predominant soil groups in areas nearest to the HHD, each representing a distinct group of soil classes. These groups are referred to as (1) Soils of the Flatwoods, (2) Soils of Sloughs and Freshwater Marshes, and (3) Soils of the Everglades.

Soils of the Flatwoods are found at various points around the Lake, and are especially predominant in the north. This group is made up of nearly level, poorly drained soils that are sandy throughout, and have organic staining in the subsoil.

The Soils of Sloughs and Freshwater Marshes are common throughout the Lake Okeechobee region. These soils are nearly level and very poorly drained. Most are organic with a sandy substratum, and some have a thin organic surface layer and a loamy subsoil underlain by limestone.

Soils of the Everglades are nearly level and very poorly drained, and are primarily found along the south, southeastern, and southwestern portions of the Lake. This group of soils has a surface layer of muck underlain by limestone.

Reach One occurs within the Soils of the Everglades group. Natural soil classes common within the Everglades group, and found in the vicinity of Reach One include; Torry muck, Adamsville sand-organic subsoil variant, Pahokee muck, and Terra Ceia muck (USDA, 1978). These are primarily moderately permeable soils with a water table within 3 ft (1 m) of ground surface.

No prime farmland soil classes are located in the vicinity of Reach One. However, the areas currently utilized for sugarcane production are classified as unique farmland soils based on use alone.

4.04 Hydrology

Lake Okeechobee is a major hydrologic feature of south Florida and the Everglades ecosystem. The Lake is also the primary reservoir of the Central and Southern Florida Flood Control Project. Lake Okeechobee receives water principally from rainfall and from the Kissimmee River which enters the Lake from Okeechobee County to the north. Smaller tributaries including Fisheating Creek, Harney Pond Canal, Indian Prairie Canal, Taylor Creek, and lesser streams from small drainage basins adjacent to the Lake contribute as well.

Because of Lake Okeechobee's large surface area, much of the surface water is lost to evaporation each year. Water is also released from the Lake through the principal outfall canals which include the West Palm Beach, Hillsboro, North New River, Miami, St. Lucie, and Caloosahatchee River Canals (Figure 6). The Caloosahatchee and St. Lucie canals are the primary outlets for release of flood waters when the Lake is above regulation stages (Fernald and Patton, 1984).

Flow in the major drainage canals is generally from Lake Okeechobee toward the coasts. However, at times the flow in the Lake ends of the canals is toward the Lake owing to various combinations of concentrated rainfall and drainage pumping from farmlands into the canals.

The groundwater throughout the Lake Okeechobee area is usually within 3.28 ft (1 m) of the land surface and extends to about 330 ft (100 m) bls. This water table generally parallels the land-surface features. Differences in ground elevations are so slight that the water table is a relatively uniform surface with few undulations.

The principal source of recharge to the groundwater in this area is derived from local rainfall and by subsurface percolation from the canals into the permeable materials. Discharge from this shallow groundwater reservoir is by evaporation from the land or water surfaces, transpiration by plants, seepage into canals, and pumping from shallow wells. The groundwater flow typically follows a north to south gradient.

The major artesian aquifer underlying this region is the Floridan Aquifer, which occurs from about 1000 ft (300 m) bls to bedrock (Schroeder et al, 1954).

The hydrology within the area of Reach One is much the same as that described for the general area. It is likely though, that because of the vast sugarcane plantations and saturated soil conditions along Reach One, evaporation and transpiration are responsible for an even greater amount of water loss. Major outfall canals along Reach One include the St. Lucie, West Palm Beach, and Hillsboro Canals.

4.05 Water Supply

The surface and groundwaters in the Lake Okeechobee area provide a valuable source of water for public, domestic, industrial, and agricultural use for much of Southeast Florida. Additionally, significant natural areas located in the region receive water from this source as well.

The Lake itself serves as a source of public water supply for Canal Point, Clewiston, Belle Glade, Okeechobee, Pahokee, and South Bay. Local industries such as sugarcane refineries and produce packaging/distribution centers also employ the available groundwater and surface water for their plant operations.

The City of Fort Myers depends upon the Lake to ensure the quantity and quality of the supply of drinking water it withdraws from the Caloosahatchee River. In the future, maintenance of minimum flows and levels within the downstream natural system will place additional demands on the Lake. Urban demands are also expected to steadily increase.

Although the current configuration of the Lake was designed primarily to provide drainage, flood control and water supply benefits, the single largest demand on Lake Okeechobee is to provide water for agricultural irrigation.

Agricultural activities in the Lake Okeechobee area utilize the canals and culverts associated with the Lake as a source of irrigation water for the many sugarcane and truck crops produced in the region. To the south and east of Lake Okeechobee, and adjacent to much of Reach One, the Everglades Agricultural Area (EAA) is one of the most productive farming regions in the country. The EAA relies heavily on water obtained from this resource, notably so along Reach One.

Significant natural areas in the region benefit from water originating in Lake Okeechobee. The Everglades, located south of the Lake, receives a vital allotment of its annual water requirements directly from the Lake and canals along its southern portion. To the south and southeast, there are three Water Conservation Areas (WCA) that receive water from Lake Okeechobee and serve as functional wetlands and municipal water supply.

Located in southeast Palm Beach County, WCA No. 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge), receives water from the West Palm Beach, and Hillsboro Canals which originate from Reach One of the HHD. Located in southeast Palm Beach County and northern Broward County, WCA No. 2 (part of the Everglades Wildlife Management Area), receives water from the Hillsboro, and North New River Canals also originating from Reach One of the HHD.

Located in Broward County, WCA No. 3, receives water from the Miami Canal which originates from the HHD west of South Bay (Figure 6).

These Water Conservation Areas are viable wetland environments and also provide water supply storage for the southeast coast. Additionally, water from WCA No. 3 is discharged to the sloughs and wetlands of Everglades National Park (Fernald and Patton, 1984).

Water released from two of the major outfall canals provides inflow to coastal estuarine ecosystems. The Caloosahatchee Canal feeds the Caloosahatchee River Estuary on Florida's west coast, while the St. Lucie Canal feeds the estuaries associated with the St. Lucie Inlet on the east coast.

4.06 Water Quality

The hydrology of the Lake Okeechobee basin has been greatly modified to control floods and create farmland. Inflow and outflows are largely managed through canals, pumps, and control structures. Intense water quantity management has resulted in a marked decline in water quality.

The water quality of Lake Okeechobee has degraded from its natural state generally as a result of human impacts. In a 1985 basin assessment, the Lake conditions were considered to be moderately eutrophic to eutrophic as a result of nutrient loading primarily from nitrogen and phosphorus. The primary sources of nitrogen and phosphorous include runoff from ranch and dairy operations, back-pumping of runoff from row crops, and may include flooding of perimeter wetlands associated with water level increases within the Lake. Nutrient transport vectors include the Kissimmee River, water pumped from the Everglades Agricultural Area, run-off from

Taylor Creek and Nubbin Slough, and Fisheating Creek (Fernald and Patton, 1984). The current average mean phosphorous concentration in Lake Okeechobee was reported at 0.0643 mg/L (SFWMD, 1998). The current average mean nitrogen concentration in the Lake was reported at 1.3743 mg/L (SFWMD, 1998).

Water in Lake Okeechobee is essentially uniform in chemical composition, moderately hard (hardness 135 ppm) and satisfactory without expensive treatment for practically all uses. Chemical quality of water in the Lake ends of the canals is generally similar to that in Lake Okeechobee whenever water is being discharged from the Lake. Owing to inflow and seepage, the hardness, the total content of dissolved minerals, and the color of water in the canals increases rapidly with distance from the Lake.

The groundwater in Lake Okeechobee is somewhat less than desirable for most uses. There is a general tendency for hardness to increase with depth within the water-table aquifer. The water from shallow wells in the vicinity of Reach One is of such poor quality that it is undesirable for most purposes except irrigation. However, when no other source of water is available, shallow groundwater is used extensively for domestic purposes (Schroeder et al, 1954).

The surface water and groundwater in the area of Reach One have been affected by the disruption of historic water flows. As this area is predominantly used for agriculture, the surface and groundwaters of Reach One have also been affected by elevated nutrient levels including nitrogen and phosphorus; dissolved solids including chloride, sodium and calcium carbonate; trace metals including copper, zinc, lead and iron; and pesticides and/or herbicides (Fernald and Patton, 1984).

4.07 Water Management

Lake Okeechobee is the primary reservoir of the Central and Southern Florida Flood Control Project. The Lake is capable of storing 2.7 million acre-feet of water between stages of 10.5 ft (3.2 m) NGVD and the top of the regulation schedule at 17.5 ft (5.3 m) NGVD.

Water levels in the Lake are managed according to a regulation schedule that was developed by the South Florida Water Management District (SFWMD) and the USACE. The schedule is designed to maintain a low level of 15.5 ft (4.7 m) during the wet season in order to provide storage capacity for excessive amounts of rainfall and to prevent flooding in surrounding areas. The stage at the end of the wet season is regulated at a maximum of 17.5 ft (5.3 m) in order to store water for the dry season. The Caloosahatchee and St. Lucie canals are the primary outlets for release of flood waters when the Lake is above regulation schedule stages. As a note, the Corps always retains ultimate control of the project. The sponsor and others activity must be consistent with the Corps water management plan.

A series of structures are situated around the Lake which provide flood protection, control drainage, and facilitate navigation (Figure 6). The USACE operates the primary structures and navigation locks around the Lake and is responsible for maintenance of the schedule. The SFWMD operates and maintains the secondary water control structures and pump stations (Fernald and Patton, 1984).

Present drainage operations and the regulation of the water stages of Lake Okeechobee have produced a complex water-table pattern in the region. The resistance of peat to lateral groundwater seepage and the relatively impervious character of the marl, which overlies the shallow permeable water-bearing rocks, make water control economically feasible in this area (Shroeder et al, 1954).

Since the Reach One area has very little natural drainage, it therefore depends on large pump stations to prevent floods from heavy rains. Pumps remove excess water from the Everglades Agricultural Area and pump this water into Lake Okeechobee during wet months and release water from the Lake for irrigation during the dry growing season (Fernald and Patton, 1984).

Along Reach One, there are eight gated culverts, two hurricane gate structures, and one lock (Figure 6). Control of waters from these structures is primarily the responsibility of the USACE and SFWMD. However, eight private drainage districts assume control of water flow within the region of

Reach One. These are: 1) Mayaca Groves, 2) Palm Beach Groves, 3) Cloister Farms, 4) U.S. Sugar Corporation, 5) East Beach Drainage District, 6) Pahokee (or 715) Farms, 7) East Shore Drainage District, and 8) South Shore Drainage District.

Under Chapter 373 of the Florida Statutes, Water Management Districts are directed to implement several provisions for water resource protection. The statutes require that surface waters be managed in such a way as to prevent significant harm to the natural resources, including fish and wildlife, and that they lay out tools that may be used to regulate water use. One of the management tools specifically referred to in the statutes is Minimum Water Level (MWL), defined as "the level of groundwater in an aquifer and the level of surface water at which further withdrawals would be significantly harmful to the water resources of the area." The statute further specifies that the minimum levels be established using "best available information" and that they be established within a "reasonable" time period.

The process used to develop minimum water level criteria for Lake Okeechobee based on existing data and the "best available information" was very limited, and the criteria established must be viewed as tentative and subject to validation or perhaps modification as additional research/modeling results become available. A major focus of the current ecological research program on the Lake addresses the issue of system-wide responses to water level variation.

The Lake Okeechobee MWL criteria include three components: (1) minimum depth - a lake water level that, if sustained for a defined period of time, will result in harm to the resource; (2) duration - the estimated period of time that water levels can remain below the specified minimum depth without causing harm to the resource; and (3) return frequency - the frequency of occurrence for events wherein water levels may recede below the minimum depth without causing harm to the resource. The criteria were established in reference to the littoral marsh zone of the Lake, a large region (about 20% of the total lake area) of emergent vegetation along the south and west edges of the Lake. This region of the Lake is the principal spawning area for commercial and recreational fishes, feeding and nesting area for wading birds, and critical habitat for other wildlife, including the American alligator and the endangered snail kite.

Minimum depth criteria were established based on Geographic Information System data regarding littoral zone inundation and drying under different water regimes, information on vegetation spatial distribution, bird and fish use of different vegetation types, and relationships between depth and recreational use of the water resource. Two criteria were established; 11 and 12 ft (3.4 m and 3.7 m) NGVD, each with a specified duration and return frequency. For duration, not more than one time every 7 years below 11 ft NGVD, and not more than one time every 3 years below 12 ft NGVD; and for return frequency, not to exceed the MWL of 11 ft NGVD for longer than 120 days per event, nor the 12 ft NGVD criterion for longer than 180 days per event.

To fully protect the ecosystem from harm, it also is critical that water levels be established at the other extreme, i.e., for the maxima that occur during prolonged high water periods. Recent evidence indicates that those events may be just as harmful to the ecosystem as prolonged lows (Havens, 1998).

Prolonged submersion may damage or eliminate emergent vegetation communities and associated wildlife habitat. This also reduces the Lake's ability to act as a nursery for aquatic organisms (U.S. Department of the Interior, 1997).

4.08 Vegetation and Cover Types

The vegetation and cover types within the Lake Okeechobee region have been greatly altered during the last century. Historically, the natural vegetation was a mix of freshwater marshes, hardwood swamps, cypress swamps, and pine flatwoods. The freshwater marshes were the predominant cover type throughout, but especially along the southern portion of the Lake where it flowed into the Everglades. These marshes were vegetated primarily with sawgrass *(Cladium jamaicense)* and scattered clumps of Carolina willow *(Salix caroliniana)*, sweetbay *(Magnolia virginiana)*,and cypress *(Taxodium* spp.*)*. Hardwood swamps dominated by red maple *(Acer rubrum)*, sweetbay, and sweet gum *(Liquidambar styraciflua)* occurred in riverine areas feeding the Lake, while cypress swamps composed mostly of cypress were found in depressional areas throughout the region. Pine flatwoods composed of slash pine *(Pinus elliottii)*, cabbage palm *(Sabal palmetto)*, and saw palmetto *(Serenoa repens)* were prevalent in upland areas especially to the north. Although some of these natural areas still exist, the introduction of controlled drainage for agriculture and land development has resulted in a significantly different set of cover types.

Landward of the entire HHD, sugarcane plantations, improved pasture, row crops, and urban lands now prevail. The HHD itself is covered with mixed grasses which are mowed on a regular basis. Waterward of the HHD the dominant cover types include open water, freshwater marshes, tree islands, and even some agriculture.

On the landward side of Reach One, the predominant cover types within 500 ft (150 m) of the HHD are urban and agricultural. The city of Pahokee constitutes the urban component which occupies approximately 40% of this area. Residences and businesses within the city of Pahokee, which is located along the central portion of Reach One, often abut the very toe of the Dike. Sugarcane, row crops, and other agricultural areas account for approximately 40% of this area as well. Some of these agricultural areas, especially sugarcane in the southern portion of Reach One, also borders the toe of the Dike. Significant cover types occupying the remainder of this area include small isolated wetlands (see Section 4.09), patches of nuisance vegetation, and a series of limestone quarries.

Beyond 500 ft (150 m) from the HHD, the major cover type along Reach One is agricultural. Sugarcane is the primary crop, with some ornamental groves and row crops. The city of Belle Glade, which is approximately 1.5 mile (2.4 km) southeast of Reach One, comprises the only significant urban component in this range.

The major cover types on the waterward side of Reach One include open water, spoil islands, freshwater marshes, agriculture, and urban. Naturally, open water represents the greatest portion of the area along Reach One. Spoil islands also occur along Reach One, and are especially prevalent along the central and southern portion of the Reach. These islands are not naturally occurring, rather they are narrow strips of dredged material about 16.4 ft (5 m) wide and are approximately 100 ft (30 m) away from the toe of the HHD. These materials were dredged during creation/maintenance of the rim canal. Tree species existing on these narrow islands are mostly nuisance species such as Australian pine *(Casuarina sp.)*, melaleuca *(Melaleuca quinquenervia)*, and Brazilian pepper*(Schinus terebinthifolius)*.

Freshwater marshes occur waterward of Reach One in the low lying areas skirting Kreamer and Torry Islands (Figure 7) which are near the southern end of the Reach. Vegetation in these areas include sawgrass, Brazilian pepper, and cabbage palms.

Agriculture, primarily sugarcane, occurs on Kreamer and Torry Islands, too, occupying several hundred hectares (over 1,000 acres) of land.

The urban component of land cover waterward of Reach One is a small residential area on the southern end of Torry Island.

Nuisance Vegetation

Nuisance vegetation is found all along Reach One. In the northern portion, there is strip of land between the HHD and a transportation corridor (Hwy. 98/441 and the Florida East Coast Railroad) in which many exotics are present. A mixture of melaleuca, Australian pine, and Brazilian pepper is found in this strip of land. In the drainage ditches along the toe of the HHD, nuisance vegetation exists including species such as water hyacinth *(Eichhornia crassipes)* water lettuce *(Pistia stratiotes)*, hydrilla *(Hydrilla verticillata)*, cattails *(Typha sp.)*, and bamboo *(Arundinaria sp.)*.

Limestone Quarries

A series of limestone quarries are located immediately south of Culvert C-12, in the southern portion of Reach One (Figures 7 and 10). These quarries are found immediately at the toe of the HHD and are approximately 3,000 ft (900 m) long. Due to the steep cut of the quarries, no littoral zone vegetation exists. However, some vegetation such as Carolina willow, and Brazilian pepper is found along the very edge where mowers do not reach.

4.09 Wetlands

Wetlands in the Lake Okeechobee area, though greatly reduced through human impacts in both size and number, still exist as functional ecotypes. These wetlands represent an important set of

valuable and productive ecosystems both landward and waterward of the HHD. Additionally, the Lake itself provides a source of water to other significant wetland areas in south Florida (see Section 4.05).

Landward of the HHD, notable wetland types are found in both riparian and isolated depressional areas. In proximity to the HHD, the lower Kissimmee River, Fisheating Creek, Taylor Creek, and Nubbin Slough represent riparian areas supporting viable freshwater marshes as these streams meander into the Lake area. These areas are utilized by a wide variety of wildlife as foraging and nesting habitats. Isolated depressional wetlands in the HHD also provide an important, and often diverse, source of habitat as well.

Waterward of the HHD, large freshwater marshes exist in the shallow littoral zones of the Lake. These are diverse, high quality herbaceous wetlands dominated by sawgrass, bulrushes *(Scirpus sp.*), cattails, eel-grasses *(Vallioneria sp.)*, and hydrilla. These marshes are important habitats for a diverse population of invertebrates and fishes which are, in turn, a food source for other fishes, birds, amphibians, reptiles, and mammals, including humans. These marshes are sensitive to prolonged low water levels, prolonged high water levels, and invasion by exotic species. The larger of these occur where streams enter the Lake providing silt and nutrients. Foremost among the large freshwater marshes is the 150 square mile (400 km^2) marsh on the western shores of the Lake.

Lake Okeechobee supports important wetlands beyond the immediate area by providing them with a source of water. The Florida Everglades to the south, and Water Conservation Areas in Palm Beach and Broward Counties (see Section 4.05) are all recipients of freshwater derived from the Lake.

In the area of Reach One, there are fewer outstanding wetland areas relative to the remainder of the Lake. In this area, the large scale drainage efforts for agriculture and development have perhaps had the most apparent impact.

On the landward side of Reach One, remaining wetlands are typically found along ditches or low lying areas and are usually a result of impoundment rather than natural hydrology. The majority of these are small, isolated freshwater wetlands which are located in the northern portion of Reach One within the strip of land between the HHD and the transportation corridor (Hwy. 98/441 and the Florida East Coast Railroad). Typical vegetation in these wetlands includes Carolina willow, water hyacinth, cattails, water lettuce, and duckweed *(Lemna sp.)*. Along the toe ditch of the HHD, there are a number of places where impoundment of water also occurs. These impoundments are typically small areas occupying less than one hectare (2.47 acre) and host a similar set of hydrophilic vegetation.

Although wetlands present on the landward side of Reach One may not be considered high quality ecosystems, they do host small fishes and invertebrates and provide usable foraging habitat for wading birds.

Waterward of Reach One, there are few wetland areas immediately adjacent to the HHD. Due to dredging activities for the rim canal which parallels the Dike, the littoral zone is narrower than

that found in some of the remaining reaches. The water depth increases rapidly here, providing less habitat, but wading birds are still frequently seen foraging at the toe of the Dike.

Large freshwater marshes are still found waterward of Reach One, primarily around Kreamer and Torry Islands. Located near the southern extent of Reach One, these freshwater marshes are diverse, high quality herbaceous wetlands dominated by sawgrass, bulrushes, cattails, eel-grasses, and hydrilla. These marshes provide over 1,000 acres (several hundred hectares) of valuable habitat for a diverse population of invertebrates and fishes which are, in turn, a food source for other fishes, birds, amphibians, reptiles, and mammals, including humans. These marshes are sensitive to prolonged low water levels, prolonged high water levels, and invasion by exotic species.

4.10 Fish and Wildlife

Fish and wildlife species present in the Lake Okeechobee area, though surely fewer now than a century ago, are still quite numerous and utilize the many natural areas around the Lake.

Fish

The aquatic habitats of the Lake and the freshwater marshes on both the waterward and landward sides of the HHD provide important habitat for a variety of fishes. Within the waters of the Lake, significant populations of fish such as large mouth bass *(Micropterus salmoides)*, catfish *(Ictalurus* spp.*)* black crappie *(Pomoxis nigromaculatus)*, sunshine bass *(Morone chrysops)*, redear sunfish (shellcracker) *(Lepomis microlophus)*, threadfin shad *(Doromosa petense)*, and bluegill *(L. macrochirus)* exist (FGFWFC, 1997). The waterward freshwater marshes provide important nursery grounds for these fishes and many others. The landward freshwater marshes, and ditches provide habitat for numerous smaller fishes. Many of these fishes provide food for amphibians, reptiles, birds, and mammals, including humans.

Amphibians

Important habitat for a variety of amphibians is provided by the aquatic habitats of the Lake and the freshwater marshes on both the waterward and landward sides of the HHD. Bullfrog *(Rana catesbeiana)*, and southern leopard frog *(R. utricularia)* utilize these habitats throughout their life-cycles. Terrestrial amphibians such as southern toad *(Bufo terrestris)*, green treefrog *(Hyla cinerea)*, dwarf salamander *(Eurycea quadridigitata)*, and Everglades dwarf siren *(Pseudobranchus striatus belli)* breed and live their early life stages here . These animals provide a food source for other animals such as fish, birds, reptiles, and mammals (Cowley, 1998).

Reptiles

A variety of reptile species are found within the aquatic habitats of the Lake and the freshwater marshes on both the waterward and landward sides of the HHD. Among these are the American alligator *(Alligator mississippiensis)*, Florida mud turtle *(Kinosternon subrubrum steindachneri)*, common musk turtle *(sternotherus odoratus)*, peninsula cooter *(Pseudemys florida peninsularis)*, Florida snapping turtle *(Chelydra serpentina osceola)*, Florida softshell turtle *(Apelone ferox)*, Florida water snake *(Nerodia fasciata pictiventris)*, and Florida cottonmouth *(Agkistrodon piscivorus conanti)*. Additionally, green anole *(Anolis carolinensis)*, and Cuban anole *(A. sagrei s.)* may be found in the vegetation bordering these habitats (Cowley, 1998). These animals provide a food source for other animals such as fish, birds, amphibians, other reptiles, and mammals.

Birds

The habitats within and surrounding the Lake are frequented by a number of bird species. Birds of prey such as bald eagle *(Haliaeetus leucocephalus)*, osprey *(Pandonion haliaetus)*, snail kite *(Rostrhamus sociabilis plumbeus)*, and northern harrier *(Circus cyaneus)* commonly utilize these areas. Wading birds such as great blue heron *(Ardea herodias)*, great egret *(Casmerodius albus)*, little blue heron *(Egretta caerulea)*, snowy egret *(E. Thula)*, tri-colored heron *(E. tricolor)*, and woodstork *(Mycteria americana)* are seen in relatively large numbers within the aquatic habitats of the Lake and the freshwater marshes on both the waterward and landward sides of the HHD. Other birds commonly utilizing the Lakes resources include anhinga (*Anhinga anhinga)*, double-crested cormorant (*Phalacrocorax auritus)*, and black skimmer (*Rynchops niger)* (Cowley, 1998). Additionally the habitats of Lake Okeechobee and the surrounding area provide cover and foraging habitat for migratory waterfowl such as ring-necked duck *(Aythya collaris)*, canvasback (*A. valisineria)*, and lesser scaup *(A. affinis)* as well as a multitude of neo-tropical migrants.

Mammals

The aquatic habitats of the Lake and the freshwater marshes on both the waterward and landward sides of the HHD provide important habitat for a variety of mammals. Animals such as raccoon *(Procyon lotor)*, river otter *(Lutra canadensis)*, striped skunk *(Mephitis mephitis)*, marsh rabbit *(Sylvilagus palustris)*, and nine-banded armadillo *(Dasypus novemcinctus)* range throughout this area (Cowley, 1998). The Florida manatee *(Trichechus manatus latirostris)* may also utilize the Lake and its waterways as well.

Invertebrates

An important array of invertebrates exist within the Lake and associated habitats. These animals provide an important food base for many fish, amphibians, reptiles, birds and mammals. Apple snail *(itliantus esxtlalupo)* common especially in the freshwater marshes is the sole food source for the endangered snail kite. Other epiphytic snails are common throughout the waters as well. Insect larvae are particularly prevalent throughout the aquatic habitats of the Lake and the freshwater marshes on both the waterward and landward sides of the HHD. Among these are water fleas *(Chydorus sphaericus)*, mayflies *(Ephemereoptera* spp.*)*, dragonflies and damselflies *(*Order Odonata*)*, and mosquitoes (Order Diptera).

Along Reach One, the potential for fish and wildlife is somewhat reduced. On the landward side of the HHD, the area is dominated by urban and agricultural lands. As a result, there is little continuous habitat available for many wildlife

species. Waterward, due to the dredged conditions of the rim canal, the littoral zone is narrower than would naturally occur, providing less than optimal wildlife habitat. Even with the less than ideal conditions, wading birds are commonly observed on the landward and waterward sides of the HHD which indicates a viable population of small fishes and invertebrates along either toe of the Dike.

4.11 Threatened and Endangered Species

Fauna

Sixteen faunal species which are known to occur in Martin and Palm Beach Counties are currently listed as threatened or endangered by the United States Fish and Wildlife Service (USFWS). Twenty-nine faunal species known to occur in these counties are currently listed as threatened, endangered, or as species of special concern by the Florida Game and Freshwater Fish Commission (FGFWFC) (Table 2).

Although ideal habitat conditions no longer exist for wildlife in the area of Reach One, listed faunal species are still observed in this area.

The USFWS has determined that five listed faunal species which occur in the vicinity of Reach One, could be affected by the proposed project. These species include the indigo snake, bald eagle, wood stork, snail kite, and West Indian manatee. The western shore of the Lake, and the entire littoral zone on this shore is designated as critical habitat for the snail kite. This includes the marshes located along the segment of the Lake from the Hurricane Gate at Clewiston to the mouth of the Kissimmee River. The USFWS has further determined that although critical habitat has been designated for the West Indian manatee in certain Florida waters, the waters of Lake Okeechobee are not included in that designation (USFWS, February 1996).

The Florida Natural Areas Inventory (FNAI) has records of observations for five listed faunal species within one mile (1.6 km) of Reach One. These include the American alligator, eastern indigo snake, bald eagle, wood stork, and West Indian manatee (FNAI, 1998). Although this may not be a complete representation of listed species utilizing this area, the FNAI provides the best reliable database for listed species incidence in Florida.

TABLE 2
LISTED FAUNAL SPECIES
OCCURRING IN MARTIN & PALM BEACH COUNTIES, FLORIDA
(FNAI, 1998)

Scientific Name	Common Name	Federal Status[1]	State Status[2]
AMPHIBIANS			
Rana capito	gopher frog		S
REPTILES			
Alligator mississippiensis	American alligator	T/SA	S
Caretta caretta	loggerhead	T	T
Chelonia mydas	green turtle	E	E
Dermochelys coriacea	leatherback	E	E
Drymarchon corais couperi	eastern indigo snake	T	T
Eretmochelys imbricata	hawksbill	E	E
Gopherus polyphemus	gopher tortoise		S
Lepidochelys kempii	Atlantic ridley	E	E
Pituophis melanoleucus mugitus	Florida pine snake		S
BIRDS			
Aphelocoma coerulescens c.	Florida scrub jay	T	T
Caracara plancus	crested caracara	T	T
Egretta caerulea	little blue heron		S
Egretta thula	snowy egret		S
Egretta tricolor	tricolored heron		S
Falco peregrinus	peregrine falcon		E
Grus canadensis pratensis	Florida sandhill crane		T
Haliaeetus leucocephalus	bald eagle	T	T
Mycteria americana	wood stork	E	E
Pelecanus occidentalis	brown pelican		S
Picoides borealis	red-cockaded woodpecker	E	T
Rostrhamus sociabilis plumbeus	snail kite	E	E
Speotyto cunicularia floridana	Florida burrowing owl		S
Sterna antillarum	least tern		T
MAMMALS			
Felis concolor coryi	Florida panther	E	E
Podomys floridanus	Florida mouse		S
Sciurus niger shermani	Sherman's fox squirrel		S
Trichechus manatus	West Indian manatee	E	E

[1]Federal Legal Status (US Fish and Wildlife Service)
E = Listed as an Endangered Species in the List of Endangered and Threatened Wildlife and Plants under the provisions of the Endangered Species Act. Defined as any species which is in danger of extinction throughout all or a significant portion of its range.
T = Listed as a Threatened Species. Defined as any species which is likely to become an endangered species within the foreseeable future throughout all or a significant portion of its range.
T/SA = Threatened due to similarity of appearance.

[2]State Legal Status (Florida Game and Fresh Water Fish Commission)

E = Listed as an Endangered Species. Defined as a species, subspecies, or isolated population which is so rare or depleted in number or so restricted in range of habitat due to any man-made or natural factors that it is in immediate danger of extinction or extirpation from the state, or which may attain such a status within the immediate future.
T = Listed as a Threatened Species. Defined as a species, subspecies, or isolated population which is acutely vulnerable to environmental alteration, declining in number at a rapid rate, or whose range or habitat is decreasing in area at a rapid rate and as a consequence is destined or very likely to become an endangered species within the foreseeable future.
S = Listed as a Species of Special Concern. Defined as a population which warrants special protection, recognition, or consideration because it has an inherent significant vulnerability to habitat modification, environmental alteration, human disturbance, or substantial human exploitation which, in the foreseeable future, may result in its becoming a threatened species.

In addition to species listed by the USFWS and the FNAI, three additional listed species were observed along Reach One during the site survey for this report. Each of these species were birds seen foraging along the toe ditches or at the edge of the Lake. These were the brown pelican, little blue heron and tri-colored heron, each of which is listed as a species of special concern by the FGFWFC.

A description of each species reported by the USFWS and the FNAI follows.

American alligator *Alligator mississippiensis*

The American alligator's range extends across the southeastern states of Alabama, Arkansas, North & South Carolina, Florida, Georgia, Louisiana, Mississippi, Oklahoma, and Texas (University of Florida, 1998).

This reptile utilizes freshwater swamps and marshes as its primary habitat, but is also seen in rivers, lakes and smaller bodies of water. Alligators have been shown to be an important part of their ecosystem, and are thus regarded by many as a "keystone" species. This encompasses many areas from control of prey species to the creation of peat through their nesting activities (University of Florida, 1998).

Populations of the American alligator were severely affected in the early parts of this century, due to hunting of the animal for its skin. In 1967, this species was listed as an endangered species which prohibited alligator hunting. As a result, the alligator has undergone a successful recovery.

The occurrence of the American alligator along Reach One is likely on the waterward side of the HHD, as well as many portions of the landward side. In areas where development has occurred, such as Pahokee, it is unlikely.

The American alligator is currently listed as threatened by the USFWS, due to its similarity to the American crocodile *(Crocodylus acutus)*. The American alligator is currently listed as a species of special concern by the FGFWFC.

Eastern Indigo Snake *Drymarchon corais couperi*

The eastern indigo snake is the largest non-venomous snake in North America. It is an isolated subspecies occurring in southeastern Georgia and throughout peninsular Florida.

The eastern indigo prefers drier habitats, but may be found in a variety of habitats from xeric sandhills, to cabbage palm hammocks, to hydric hardwood hammocks (Schaefer and Junkin, 1990).

Indigos need relatively large areas of undeveloped land to maintain population. The main reason for its decline is habitat loss to development. Further, as habitats become fragmented by roads, indigos become increasingly vulnerable to highway mortality as they travel through their large territories (Schaefer and Junkin, 1990).

The occurrence of the indigo along Reach One is possible along the Dike itself, and landward. However, this is not highly likely due to the isolation of the HHD from more ideal habitats.

The eastern indigo snake has been classified as a threatened species by the USFWS since 1978 and by the FGFWFC since 1971.

Bald Eagle *Haliaeetus leucocephalus*

The bald eagle is the only eagle unique to North America. It ranges over most of the continent, from the northern reaches of Alaska and Canada down to northern Mexico.

The bald eagle occurs in various habitats near lakes, large rivers and coastlines. In general, they need an environment of quiet isolation; tall, mature trees; clean waters; and prefer nesting within one-half mile (0.8 km) of water (USFWS, July 1995).

The bald eagle population was decimated in the 19th and early 20th centuries by habitat destruction, hunting, pesticide use and lead poisoning. In 1967, bald eagles were officially declared an endangered species. Due to this and other protective measures, the population has made a tremendous comeback, its populations greatly improving in numbers, productivity, and security in recent years. Its strongest populations are currently found in Alaska and Florida (USFWS, July 1995).

Two active bald eagle nests were recorded along Reach One as late as 1995 (FNAI, 1998). One of these was located on the waterward side of the levee, just south of Pahokee. The other nest was located in the Dupuis Reserve east of Reach One, and south of the St. Lucie Canal (Figures 7, 8, & 10).

The bald eagle is currently listed as a threatened species by both the USFWS and FGFWFC.

Wood Stork *Mycteria americana*

The wood stork is the only stork occurring in the United States. In the U.S., the wood stork's range includes Alabama, Florida, Georgia, Louisiana, Mississippi, South Carolina, and Texas. The only states in which this bird is known to nest, however, are Florida, Georgia and South Carolina (Mazzotti, 1990).

Wood storks are wetland dwellers and use fresh, brackish and saltwater habitats for feeding and nesting. Feeding takes place in shallow ponds, tidal pools, swamps and marshes. Nesting occurs in cypress, hardwood and mangrove swamps. The extreme dependence of the wood stork on naturally functioning wetlands makes it an excellent indicator of the health of wetland ecosystems (Mazzotti, 1990).

Until the last few decades, the wood stork was a common sight in Florida wetlands. However, between the 1930's and 1960's, there was a serious decline in this species. One reason for the decline in population has been the changes in the hydrologic regime of the Everglades, which affected its foraging habitat and food production (Mazzotti, 1990).

Utilization of habitats along Reach One by the wood stork for foraging is highly likely. It is unlikely, however, that this bird will utilize areas in proximity to Reach One for nesting purposes.

Wood storks are currently listed as an endangered species by both the USFWS and FGFWFC.

Snail Kite *Rostrhamus sociabilis plumbeus*

Although previously located in freshwater marshes over a considerable area of peninsular Florida, the range of the snail kite is currently more limited. This bird is now restricted to several impoundments on the headwaters of the St. John's River; the southwest side of Lake Okeechobee; the eastern and southern portions of WCAs 1, 2A and 3; the southern portion of WCA 2B; the western edge of WCA 3B; and the northern portion of Everglades National Park (USFWS, May 1996).

The kite inhabits relatively open freshwater marshes which support adequate populations of apple snail, upon which this bird feeds almost exclusively. Favorable areas consist of extensive shallow, open water such as sloughs and flats, vegetated by sawgrass and spikerush (*Eleocharis* spp). The areas are often interspersed with tree islands or small groups of scattered shrubs and trees which serve as perching and nesting sites. The water level must be sufficiently stable to prevent loss of the food supply through drying out of the surface. The southwest shore of Lake Okeechobee from the Hurricane Gate at Clewiston to the Kissimmee River (excluding deep open water) is considered critical habitat for the snail kite (USFWS, May 1996).

The snail kite is threatened primarily by habitat loss and destruction. Widespread drainage has permanently lowered the water table in some areas. This drainage permitted development in areas

that were once kite habitat. In addition to loss of habitat through drainage, large areas of marsh are heavily infested with water hyacinth which inhibits the kite's ability to see its prey (USFWS, May 1996).

The utilization of waters along Reach One by the snail kite is not likely common, for the waters are typically too deep here for the apple snail. The primary area of concern in Lake Okeechobee is along its southwestern shore.

The snail kite is currently listed as an endangered species by both the USFWS and FGFWFC.

West Indian Manatee *Trichechus manatus*

The West Indian manatee, or sea cow, is a large, plant-eating aquatic mammal that can be found in the shallow coastal water, rivers, and springs of Florida. Florida is essentially the northern extent of the West Indian manatee's range, though some manatees occasionally are reported from as far north as Virginia and the Carolinas (FP&L, 1989).

The West Indian manatee lives in freshwater, brackish, and marine habitats, and can move freely between salinity extremes. It can be found in both clear and muddy water. Water depths of at least 3 to 7 ft (1 to 2 m) are preferred and flats and shallows are avoided unless adjacent to deeper water. During the summer months, manatees range throughout the coastal waters, estuaries, bays, and rivers of both coasts of Florida and are usually found in small groups. During the winter, manatees tend to congregate in warm springs, and outfall canals associated with electric generation facilities (FP&L, 1989).

Over the past centuries, the principal sources of manatee mortality have been opportunistic hunting by man and deaths associated with unusually cold winters. Today, poaching is rare, but high mortality rates from human-related sources threaten the future of the species. The largest single mortality factor is collision with boats and barges. Manatees also are killed in flood gates and canal locks, by entanglement or ingestion of fishing gear, and through loss of habitat and pollution (FP&L, 1989).

The utilization of Lake Okeechobee by the West Indian manatee is not uncommon. The potential for this animal to occur in the waters immediately adjacent to Reach One of the HHD for feeding is reduced due to the relatively low presence of littoral zone vegetation. The manatee may however, utilize this area for travel and other behaviors.

The West Indian manatee is currently listed as an endangered species by both the USFWS and FGFWFC.

Flora

Five floral species which are known to occur in Martin and Palm Beach Counties are currently listed as threatened or endangered by the USFWS. Twenty-eight floral species known to occur in these counties are currently listed as threatened or endangered by the Florida Department of Agriculture and Consumer Services (FDACS) (Table 3).

Although this may not be a complete representation of listed species existing in this area, the FNAI provides the best reliable database for listed species incidence in Florida.

Although normal vegetative conditions no longer exist in the area of Reach One, one listed floral species is still observed in this area.

The USFWS and FNAI have each determined that the Okeechobee gourd is likely to occur in the area of Reach One (USFWS, February 1996) (FNAI, 1998).

A description of the Okeechobee gourd follows.

Okeechobee Gourd *Curbita okeechobeensis o.*

The Okeechobee gourd is a fibrous-rooted, high-climbing vine with tendrils. Its leaf blades are heart- to kidney-shaped with five to seven shallow, angular lobes and irregularly serrated margins. This plant occurs only along the shores of Lake Okeechobee and the St. John's River (USFWS, February, 1997).

The Okeechobee gourd is usually found in pond apple (*Annona glabra*) hammocks, heavily tangled woods, and willow (*Salix* spp.) and elderberry (*Sambucus canadensis*) thickets. The seeds of this gourd germinate on bare, exposed muck and especially on alligator nests where the soil has been disturbed (USFWS, February 1997).

By 1930 at Lake Okeechobee, about 95% of the pond apple forests that had probably been occupied by this gourd were destroyed for agricultural purposes. At that time the gourd was still locally abundant, but since then it has become rare and difficult to find around the Lake (USFWS, February 1997).

FNAI currently lists two recorded locations of the Okeechobee gourd in proximity to Reach One of the HHD (FNAI, 1998). An Okeechobee gourd survey conducted in 1990-1991 found a total of 11 sites along the southeastern shore of Lake Okeechobee (USFWS, 1998). The specific location of known plant locations is sensitive information, so discussion within this text is restricted (FNAI, 1998).

The Okeechobee gourd is currently listed as an endangered species by both the USFWS and FDACS.

TABLE 3
LISTED PLANT SPECIES
OCCURRING IN MARTIN & PALM BEACH COUNTIES, FLORIDA
(FNAI, 1998)

Scientific Name	Common Name	Federal Status[1]	State Status[2]
VASCULAR PLANTS			
Acrostichum aureum	golden leather fern		E
Asclepias curtissii	Curtiss' milkweed		E
Asimina tetramera	four-petal pawpaw	E	E
Chamaesyce cumulicola	sand-dune spurge		E
Coccothrinax argentata	silver palm		E
Conradina grandiflora	large-flowered rosemary		E
Ctenitis sloanei	Florida tree fern		E
Cucurbita okeechobeensis spp *o.*	Okeechobee gourd	E	E
Halophila johnsonii	Johnson's seagrass	PT	E
Jacquemontia reclinata	beach jacquemontia	E	E
Lantana depressa var floridana	Atl. coast Fl. lantana		E
Lechea cernua	nodding pinweed		T
Lechea divaricata	pine pinweed		E
Lilium catesbaei	southern red lily		T
Linum carteri var smallii	Carter's lrg-flowered flax		E
Nemastylis floridana	fall-flowering ixia		E
Okenia hypogaea	burrowing four-0'clock		E
Oncidium bahamense	dancing-lady orchid		E
Panicum abscissum	cutthroat grass		E
Peperomia humilis	terrestrial peperomia		E
Peperomia obtusifolia	blunt-leaved peperomia		E
Polygala smallii	tiny polygala	E	E
Pteris bahamensis	Bahama brake		E
Pteroglossaspis ecristata	wild coco		T
Remirea maritima	beach-star		E
Tillandsia flexuosa	banded wild-pine		E
Vanilla mexicana	scentless vanilla		E
NON-VASCULAR PLANTS			
Cladonia perforata	perforate reindeer lichen	E	E

[1]Federal Legal Status (US Fish and Wildlife Service)
E = Listed as an Endangered Species in the List of Endangered and Threatened Wildlife and Plants under the provisions of the Endangered Species Act. Defined as any species which is in danger of extinction throughout all or a significant portion of its range.
T = Listed as a Threatened Species. Defined as any species which is likely to become an endangered species within the foreseeable future throughout all or a significant portion of its range.
PT= Proposed for listing as a Threatened Species.

[2]State Legal Status (Florida Dept. of Agriculture and Consumer Services)
E = Listed as an Endangered Plant in the Preservation of Native Flora of Florida Act. Defined as species of plants native to the state that are in imminent danger of extinction within the state, the survival of which is unlikely if the causes of a decline in the number of plants continue, and includes all species determined to be endangered or threatened pursuant to the Federal Endangered Species Act of 1973, as amended.

T = Proposed by the FDACS for listing as Endangered Plants.

4.12 Noise

Along Reach One there are a number of existing sources currently contributing to the overall ambient noise level.

The more predominant of these sources include:

- vehicular traffic traveling along nearby highways;
- railroad traffic along the Florida East Coast Railway;
- single engine aircraft utilizing the Pahokee Airport;
- small industry (i.e., produce processing and distribution);
- boat traffic along the rim canal;
- urban activities in Pahokee and Belle Glade;
- agricultural equipment (tractors, trucks, etc.); and
- pumping stations.

Rural areas typically have noise levels of 35-55 db. Sound levels along transportation arteries are typically in the range of 70 dB.

4.13 Air Quality

Existing air quality in the affected environment is good to moderate. Over 90 percent of the project area is in Palm Beach County with only a small portion located in Martin County. This project is in an area which has been designated by the Clean Air Act as a Prevention of Significant Deterioration (PSD) Class II area for U.S. Environmental Protection Agency (EPA) regulated air pollutants except ground level ozone. All of Palm Beach County is classified by the FDEP as an Ozone Attainment/Maintenance Area. This project would not be subject to any PSD incremental requirements for these pollutants since the project would fall under the fugitive emissions exemption, as per Rule FAC 62-212.400(a)(b).

In the area of Reach One, there are a number of existing sources that may affect air quality in the project area. Registered stationary emission sources include thirty stationary air point sources located in Martin County, and close to two hundred stationary air sources in Palm Beach County (FDEP, 1998). Notable registered sources near Reach One include the local sugar processing plants. Namely, the Atlantic Sugar Association plant near Belle Glade, and the U.S. Sugar Corporation plant near Clewiston each contribute to the overall air quality of this area.

In the area of Reach One, the prevailing southeast and east-northeast winds may carry vehicle emissions from US 98/441, State Road 715, and the Florida East Coast Railroad. Although

these mobile source emissions are not significant, they do currently contribute to the air quality in the area.

Additionally, short-term occurrences of elevated levels of airborne particulate matter may occur periodically from natural fires, controlled burns, and other sources. The potentially unaccounted for volatile organic compound emissions coming from nearby agricultural activities may contribute to the existing air quality as well.

4.14 Hazardous, Toxic and Radioactive Wastes

Several site visits were conducted over the past few years, with the most recent Hazardous, Toxic and Radioactive Waste (HTRW) survey having been performed on 12 August 1998. The HTRW database, aerial photography review and site assessment of the existing conditions found the potential of HTRW contamination. The immediate property surrounding Lake Okeechobee consists of the Herbert Hoover Dike which was free of discolored soil or stressed vegetation, or any other indicator which may indicate contamination levels requiring clean-up on the dike. However, close to the dike, several locations have the potential of being a source of contamination. In the municipality of Pahokee, on the east end of the Lake, businesses and private residences approach very close to the back toe of the Herbert Hoover Dike. It appears that the dike has been used as the "backyard fence". In some instances, private residences have installed a property fence creating a secure backyard boundary, the dike. This may have caused residents in the neighborhood to store materials close to the dike. Although no obvious contamination was observed, the potential of having past spills in these areas does exist. The physical inspection was performed by random spot check and driving along the road in the vicinity of the dike. It should be noted that rainfall and the high seepage rates in the area would have flushed-out most hydrocarbon, or smaller molecule chemical spills. Large molecule (PCB's), and metals may be less mobile and these spills may still measure residual levels. During real estate procurement and project construction, further evaluations will be required. The perimeter road has several leaking underground storage tanks and there have been several reported spills around Lake Okeechobee. All of these potential contamination problems are located within towns or along the highways that runs very close to the dike.

4.15 Land Use

The existing land uses in the Lake Okeechobee area are presented in this Section. Land use elements such as agriculture, urban land, transportation, transmission lines, communication, and quarries are discussed especially for the vicinity of Reach One. Figure 7 depicts the general land use features along Reach One. Figures 8, 9, & 10 enlarge Reach One into three sections (Reach 1A, 1B, & 1C, respectively) for better representation of these components.

Agriculture

The primary land use in the Lake Okeechobee region is agriculture. Major agricultural activities in the area include sugarcane plantations, cattle ranching, dairy farming, ornamental nurseries, vegetable production, and citrus groves. Farmland within the counties that surround Lake Okeechobee (Glades, Hendry, Martin, Okeechobee, and Palm Beach) occupies from 50 to 76% of the total land area (Purdum, 1994).

Throughout the Lake area, agricultural activities frequently occur very near the landward toe of the HHD. Agricultural utilization of the land immediately adjacent to the HHD is especially predominant in the south and southeast where the soil is higher in organics, thus more valuable for crop production. Cattle ranching, common to the north of the Lake, is present in near proximity to the HHD as well.

Along Reach One, agricultural land use is especially common immediately adjacent to the HHD. Along the southern third of Reach One, sugarcane plantations occupy the lands right up to the toe ditch. Further agricultural activities in close proximity to the Dike include ornamental nurseries and row crops in the northern portion of Reach One which occur at the very toe of the Dike as well.

Other common land use types in the Lake Okeechobee region are frequently associated with agriculture. Sugarcane refineries, produce packaging and shipping plants, and other support activities constitute a significant land use along with direct agriculture.

Urban Land

Another significant use of the lands in this region is urban development. Six incorporated communities are situated around the Lake and range in population from approximately 1,400 to 16,000 (Table 4). Each of these communities is partially or completely within 1 mile (1.6 km) of the HHD.

Along Reach One, the city of Pahokee and Belle Glade are in close proximity to the HHD. Pahokee parallels the Dike for over 5 miles (8 km) along the central portion of Reach One. All along this stretch, there are places where residences, businesses, and municipalities occur within 100 ft (30 m) of the Dike. Belle Glade occurs in the southern portion of Reach One, and also has areas that occupy land immediately adjacent to the HHD. South Bay, which is actually about 2 miles (3.2 km) beyond the southern end of Reach One, is still relatively close to this area of concern.

TABLE 4
1996 POPULATION ESTIMATES FOR COMMUNITIES SURROUNDING LAKE OKEECHOBEE (US CENSUS BUREAU, 1997)

Community	Population	County
Belle Glade[1]	16,656	Palm Beach
Clewiston	6,645	Hendry
Moore Haven	1,439	Glades
Okeechobee City	4,831	Okeechobee
Pahokee[1]	6,993	Palm Beach
South Bay	5,043	Palm Beach

[1] Located on Reach One

Smaller communities that exist within 1 mile (1.6 km) of the HHD include Canal Point, Hooker Point, Lakeport, Buckhead Ridge, Taylor Creek, and Up-The Grove Beach. Although population estimates are not available for these communities, their relative nearness to the HHD is notable. Among these, Canal Point occurs along Reach One.

Tribal Indian Reservation

The Brighton Seminole Indian Reservation occupies a large area of land west of the Lake in Glades County. The southern end of this reservation is near the HHD just north of Lakeport (Figure 1).

Transportation

Major transportation corridors around the perimeter of Lake Okeechobee include several highways and railroads.

County Road 78 parallels the Lake along its western and northern shores from Moore Haven to Okeechobee. From Okeechobee, State Highway 98/441 follows the northern and eastern portion of the Lake to Pahokee. County Road 715 then follows the HHD from Pahokee to Belle Glade, where State Highway 27 follows the southern Lake area back to Moore Haven and County Road 78. In many cases, these highways are within 1 mile (1.6 km) of the HHD, and are often within 50 ft (15 m).

Railroad corridors in the Lake Okeechobee area include the Florida East Coast Railway and the South Central Florida Railroad. The East Coast Railway is located along the eastern part of the Lake where it comes very near to the HHD in places along Reach One. The South Central travels along the southern end of the Lake, where it comes within 1 mile (1.6 km) of the HHD in a just a few places.

Along Reach One, there are currently a set of transportation corridors that are in relative proximity to the HHD.

Highway 98/441 parallels the Dike along the northern half of Reach One where it is within 328 ft (100 m) of the Dike for about 10 miles (16 km). Highway 715 parallels the southern half of Reach One where it is within 328 ft (100 m) of the Dike for about 6 miles (9.6 km). The nearest point either of these highways comes to the HHD is at the northern end of Reach One where Highway 98/441 is about 150 ft (45 m) from the HHD for about 2400 ft (730 m).

The Florida East Coast Railroad parallels Reach One along the northern half as well, where it travels for about 8 miles (13.5 km) within 150 ft (45 m) of the HHD. In northern Pahokee, this railroad comes within 75 ft (23 m) of the toe of the Dike for about 2 miles (3.2 km). It comes within this distance again for about 1 mile (1.6 km) just north of culvert C-10A north of Pahokee (Figure 7).

Transmission Lines

A low voltage electric transmission line is situated on the landward slope of the HHD in the northern portion of Reach One. This transmission line is located on the lower third of the slope, above the toe ditch (Figure 7).

Communication

South of the West Palm Beach Canal, there is a Florida Highway Patrol communication tower. This tower is 50 to 75 ft (15 to 23 m) beyond the toe ditch (Figures 7 and 10).

Limestone Quarries

A series of limestone quarries are located immediately south of culvert C-12 (near Paul Rardin Park) in the southern portion of Reach One. These quarries are found immediately at the toe of the HHD and are approximately 3000 ft (900 m) in length altogether (Figures 7 and 10).

4.16 Aesthetic Resources

There are five public access points to view Lake Okeechobee from the elevated vantage point of the levee crown in Reach One. The designated Florida National Scenic Trail (FNST) runs atop the HHD around the entire lake, totaling approximately 115 miles (FDOT, 1998). Panoramic lake and surrounding landscape view sheds vary depending on view access and obstruction in the area. The sounds of an occasional boater, airplane, ATV or farm implement can tend to break the otherwise peaceful setting.

The levee crown affords panoramic views of the flat agricultural fields to the east and rim canal and Torry Island to the west. Foreground views are dotted with minor visual impediments such as guardrail, power lines, trees, and small structures. Moderate aesthetic values are experienced in this area from atop the levee crown dependent on the time of year and day.

The proposed Hooker Highway construction access point is amidst agricultural lands in a rural setting of moderate aesthetic value. The levee crown affords a panoramic view of agricultural fields to the east. The foreground view to the west is of the rim canal with a fringe of trees that form an immediate backdrop.

The proposed Paul Rardin Park construction access point, adjacent to Culvert 12, is an oasis of shade and possesses good aesthetic value. The park is covered with sizeable fig trees that provide dense shade and a row of stately royal palms that line the entrance road from SR 715. The good park aesthetics are best experienced by visitors who stop in the park to picnic or relax.

The visual resources in the area of the proposed construction access adjacent to Culvert 12A are of moderate value from atop the levee facing east. The rim canal and panoramic views of Lake Okeechobee are apparent to the west of the levee. These views possess a high aesthetic value and can be spectacular during sunsets.

The Bacom Point (Culvert 10 off of SR 715) proposed construction access point is a pump station nestled in the levee. Few other structures are visible from the levee crown. The view to the east is of expansive agricultural fields that possess good aesthetic value. The City of Pahokee is visible on the horizon. The panoramic views of Lake Okeechobee possess very good aesthetic value, particularly during sunsets.

The City of Pahokee Marina and Campground is located on Lake Okeechobee west of Pahokee. The area is bordered by groups of trees spotted along the east and west side of the linear lakefront park. Views eastward from the levee crown are of adjacent residential back yards with agricultural fields beyond. These moderate aesthetic views are generally screened from the park user by the grassed levee. The park is clean and possesses good aesthetic quality. The lakefront park features and campsites overlook Lake Okeechobee and provide a panoramic view of good aesthetic value.

Canal Point Park is located on the shoreline of Lake Okeechobee adjacent to S-352 and the West Palm Beach Canal. The linear park provides panoramic views of the lake that possess good aesthetics. The landward side of the levee in this area is minimally developed with few homes and businesses. These views are foreground to near background and are of moderate aesthetic value. Agricultural fields comprise the background scenery of the Canal Point when viewed from the levee crown.

The visual resources in the area of the proposed construction access adjacent to Culvert 10A off of SR 15/700 are very rural. Agricultural fields are visible amidst a roadside border of trees that possess good aesthetic value when viewed from the levee crown. Panoramic views of Lake Okeechobee are visible to the west of Culvert 10A.

The visual resources in the area of the proposed construction access adjacent to Culvert 14 off of SR 15/700 are very rural. Agricultural fields are visible amidst a roadside border of trees that possess good aesthetic value when viewed from the levee crown. Panoramic views of Lake Okeechobee are visible to the west of Culvert 14.

Port Mayaca Lock and Spillway is located at the confluence of the St. Lucie Canal and Lake Okeechobee and is the northern limit of Reach One. The area is characterized by very low relief except for a few trees adjacent to the lockmaster's quarters, telephone poles with wires, and the lock structure. The port lands are grassed but possess moderate aesthetic value at best. The view of Lake Okeechobee is panoramic.

4.17 Recreational Resources

A variety of recreation resources are enjoyed year-round on Lake Okeechobee. State Road 717 (near S-351) provides access to Torry Island adjacent to Belle Glade Municipal Golf Course. An existing bikepath is located on the north lane of SR 717 which terminates at the base of the dike (FDOT, 1998). The Belle Glade Recreation Area on Torry Island includes a multi-laned boat ramp, marina and campground. The J-Mark Fish Camp and Slim's Fish Camp are also located on Torry Island (Greater Lake Okeechobee Tourist Alliance, 1997). Kreamer Island is just north of Torry Island and is renowned for its fishing, bird watching and hunting. It is accessible by boat only, except during extremely low lake levels. The rustic recreation facilities in this project area are utilized throughout the year and are important to residents, budget minded tourists, and the local economy.

There are no developed recreation facilities at the proposed construction access point off of Hooker Highway off SR 15. However, the location is a primary FNST trailhead and the dikes' elevation provides an excellent vantage point for bird watching and sightseeing (FDOT, 1998). Plans for an interpretive kiosk to inform visitors of the former sugar barge loading area are underway (GLOTA, 1997).

Paul Rardin Park is approximately 3.5 miles north of SR 717, adjacent to Culvert 12. The park is a main trailhead to the dike and supports the second highest visitation for sightseeing in Reach One (FDOT, 1998). The park maintains paved vehicular access, parking, picnic, and restroom facilities. A paved boat ramp for access to Lake Okeechobee's rim canal is adjacent to the park. It is owned and maintained by Palm Beach County. The park is a refuge from the hot Florida summers and provides an oasis for locals and tourists alike. Park improvements proposed include a small fishing pier with limited cover and an interpretive kiosk (GLOTA, 1997). A borrow pit/quarry is to the immediate south and intense agricultural practices occur in the fields that border the park to the east and north (USGS, 7.5MI, 1990).

There are no developed recreation facilities at the proposed construction access point adjacent to Culvert 12A. Localized bank fishing occurs at this site. The pump station is located at the southern tip of the local airport and is not developed for structured recreation.

Bacom Point (Culvert 10 off of SR 715) proposed construction access point is a pump station that has no developed recreation facilities but does provide local bank fishing access. The City of Pahokee Marina and Campground is located on Lake Okeechobee approximately five miles north of Paul Rardin Park. The park provides lakeside camping, RV hookup, a lighted fishing pier, a marina, boat ramps, bait shop, a scenic overlook, and an educational facility. Restrooms, showers, laundry, and telephones are provided. Vendors provide concession amenities. The linear park has the highest use rate of any park in FDOT-District 4 (FDOT, 1998).

Canal Point Park is located on Lake Okeechobee approximately five miles north of Pahokee Marina and Campground, adjacent to S-352 and the West Palm Beach Canal and serves as a primary FNST access point (FDOT, 1998). The linear park provides expansive views of the lake, offers tree-shaded picnic facilities, shoreline fishing, and a small-hardened boat ramp. Retail services are close by and support the recreation resources at Canal Point. Historical points of interest are in close proximity to the park. Continued emphasis on using the waterway at Canal Point as a recreation and open space activity area was discussed at GLOTA workshops (GLOTA, 1998). The primary point of activity is the north side of the dike where L-10 remains. The remnants of the walls and doors of the historical West Palm Beach Canal Lock are a testament to past activity. Local residents fish from the lock walls, banks, and bridge.

The proposed construction access adjacent to Culvert 10A off of SR 15/700 has no formal recreation facilities but does provide local bank fishing access. The proposed construction access adjacent to Culvert 14 off of SR 15/700 has no formal recreation facilities but provides unstructured bank fishing access for locals. Port Mayaca Lock and Spillway is located at the confluence of the St. Lucie Canal and Lake Okeechobee approximately seven and a half miles north of Canal Point Park where SR 76 intersects with Highway 441. The proposed park area is approximately 250 acres and includes lands on the lake, canal, rim canal, and basin. Many recreation plans have been discussed and coordinated for the Port Mayaca Lock and Spillway site. The lands possess potential for park development of regional significance given the location and proximity to waterway and highway infrastructure. The area is evenly grassed and well maintained with few trees on the property. Views of the lake are some of the best in the region. The port's lands provide bank fishing in the lake, rim canal, St. Lucie Canal, and lock basin. The property provides boat access to the basin, canals, and lake. The property is void of any structured recreation facilities, however the designated FNST crosses the property (USDA, 1986). When the fish are migrating (early spring through early summer) the basin receives intense fishing pressure.

The United State Department of Agriculture, Forest Service, produced the Florida National Scenic Trail Comprehensive Plan, 1986, which proposed a multi-use trail for the top of HHD by authority of the 1968 National Trails System Act (P.L. 90-543, 82 Stat. 9119). A composite analysis of land use and ownership determined a conceptual trail layout from the panhandle across Federal and state owned recreation lands and waterways. Lake Okeechobee is completely surrounded by the USDA's proposal (USDA, 1986). Since that time recreation studies have included dike trail access as an important consideration and vital aspect for further developing potential recreation in the communities

that border the lake and dike. Each year the three day, 110-mile Big "O" Bike Tour, fully supports fat tire bike riders, begins in Okeechobee and heads south around the lake (Chamber of Commerce, 1998).

The economic effects of recreation activities that occur in the Lake Okeechobee region, because of the lake, have been estimated to be approximately $78M in 1996 figures (GLOTA, 1998).

4.18 Cultural Resources

The earliest widely accepted date of occupation of Florida dates from around 12,000 years ago. This earliest cultural period is termed the Paleo-Indian period and lasted until about 7500 B.C. Few Paleo-Indian archeological sites are recorded in Florida, and none are identified by the Florida Master Site Files (FMSF) near Reach One of the HHD. The Archaic period, (ca. 7500 B.C. - ca. 500 B.C.), is thought to be a reflection of man's adaptation to the changing environment at the start of the Holocene, when our basically modern climate and biota were established. Archaic Indians exploited a wider range of resources than Paleo-Indians, probably utilized a more restricted territory, and may have led a more sedentary existence. Seasonally available food resources, including deer and small game, hardwood nuts, freshwater snails, and marine shellfish were used during the Archaic (Milanich 1994). The Archaic is further subdivided into the Early Archaic (7500B.C. to 5000 B.C.), Middle Archaic (5000 B.C. to 3000 B.C.) and Late Archaic (3000 B.C. to 500 B.C). Few Early or Middle Archaic period archeological sites are recorded in south Florida, and known sites are clustered along the Atlantic and Gulf coasts and inland waterways (Milanich 1994). Foraging and hunting are the main subsistence activities throughout the archaic period, with Late Archaic people exploiting a larger territory and wider range of aquatic and terrestrial food resources (Almy 1996). Archaic sites become more numerous during the Late Archaic period, when essentially modern climatic conditions had been established. Crude fiber-tempered pottery first appears in the Late Archaic. No Archaic period sites are located near Reach One, as recorded in the FMSF. Regional cultural diversity becomes apparent in the archeological record by 500 B.C. The clearest indication is that distinctive styles of pottery were made in different parts of the state (Piper Archaeology/Janus Research 1992). In the Okeechobee Basin, the Belle Glades culture sequence (ca. 500 B.C. - A.D. 1500) is subdivided into four periods. Ceramic technology progresses from fiber tempered to fiber and sand tempered to sand tempered ceramics, with St. Johns ceramic types also being used during the Belle Glades culture sequence. Black earth middens, low sand mounds and circular and linear earthworks are Belle Glade site types located near the HHD, as recorded in the FMSF.

During the early historic period, beginning with the first Spanish colonial period (1513 - 1763), the Calusa inhabited southern Florida. Their population was decimated by European-introduced diseases, warfare, enslavement, and migration out of Florida (Archaeological Consultants Inc 1991). The Miccosukee and the Seminole migrated into Florida in the 18th and 19th centuries from Georgia and Alabama. Throughout the mid 1800's the U.S. relentlessly pursued a policy of Indian removal in Florida, and the Seminole, resisting removal, eventually establishing themselves in the Everglades, Big Cypress Swamp, and the Ten Thousand Islands. Several important battles of the Seminole Wars

occurred around Lake Okeechobee including the largest and bloodiest battle of the Second Seminole War, the Battle of Okeechobee on Christmas Day in 1837 (Carr et. al. 1995). The Okeechobee Battlefield site is located at the north end of Lake Okeechobee and is a National Historic Landmark site. Other Seminole battle and habitation sites, predominantly on tree islands, are located near the HHD.

American settlement around Lake Okeechobee began in earnest in the late 19th century when efforts to drain and reclaim the Everglades began. Agriculture began in the Everglades, south of Lake Okeechobee after drainage projects of the 1906-1927 era (Milano 1995). During this period, the first settlements, Okeelanta and Glade Crest were established just south of the lake. By 1921, there were 16 settlements on or near Lake Okeechobee, with a total estimated population of 2,000. Settlement and agricultural activities escalated during the subsequent decades. The West Palm Beach Canal opened in 1917and the town now known as Canal Point was established (Archaeological Consultants, Inc. 1991). In 1918 a school was built in Pahokee. By 1920 mercantile and commercial buildings were springing up along the Lake. As early as 1917 sugar cane was being produced, and quickly became a flourishing industry in the region. The mid 1920's saw the south Florida real estate boom, which was crippled by the great hurricane of 1926. The 1928 hurricane devastated the recovery from the earlier storm with tremendous property damage and the loss of an estimated 1,800 to 2,000 lives (Archaeological Consultants, Inc. 1991). South Florida benefited from the civic and administrative works of Franklin D. Roosevelt's New Deal programs in the 1930's, including the Canal Point School, a structure determined eligible for inclusion on the National Register of Historic Places. After the hurricanes, work was begun locally to build a series of dikes around Lake Okeechobee. In 1935 the Army Corps of Engineers assumed responsibility for the on-going construction. The dike was completed in 1937 and named after President Herbert Hoover. The HHD may be eligible for inclusion on the National Register of Historic Places for its historical significance.

4.19 Socioeconomics

The major socioeconomic resources in the Lake Okeechobee region are related to agriculture, recreation and tourism, commercial navigation, and commercial fishing.

Agriculture in this region is dependent upon the Lake as a source of irrigation water. The regulated lake depths make it possible for farmlands to receive irrigation water year round regardless of rainfall. In the Lake Okeechobee service area, there are an estimated 742,668 acres of irrigated agricultural lands. These agricultural lands and associated activities employ hundreds of people in the area and bring millions of dollars in revenue annually.

Recreation and tourism activities in the area are enhanced by the regulated water levels of Lake Okeechobee. As a result, Lake Okeechobee is the largest recreational resource in the region. The Lake has been an historic tourist destination, and the Lake and its associated waterways and shoreline provide a wide variety of water-based recreation activities for local residents and out-of-state visitors, including: fishing, boating, picnicking, sightseeing, camping, swimming, birding, hunting, air

boating and hiking. The recent trend toward eco-tourism has been encouraged by the planned extension of the Florida National Scenic Trail and creation of the Lake Okeechobee Scenic Trail. In 1996, the annual value of the recreational resources of the Lake was estimated at $78,151,409 (David Miller & Associates, 1998).

Heavy waterfowl utilization of the Lake attracts tourists and recreational enthusiasts. Common waterfowl species include ring-necked duck (*Aythya collaris*), American widgeon (*Anas americana*), northern pintail (*A. acuta*), green-winged teal (*A. crecca*), Florida duck (*A.fulvigula*), and lesser scaup (*Aythya affinis)*

Lake Okeechobee supports a variety and abundance of sport fish. Consequently, sport fishing is a major recreational activity on the Lake and Lake Okeechobee is currently recognized as supporting one of the best recreational fisheries in the nation. Additionally, Lake Okeechobee supports an active commercial fishing industry. This includes several different types of commercial fishing operations and landside support activities, such as marinas and wholesale and retail distribution facilities. The annual value of the wholesale commercial fishing is $2,326,932 and employs 210 people (David Miller & Associates, 1998).

There are also commercial fisheries on the Lake which harvest the American alligator and the Florida soft shell turtle. Alligators are harvested from the Lake population to supplement the stock in alligator farming operations. Soft shell turtles are harvested by commercial fishermen, with some individual yields in excess of 30,000 pounds (13,640 kilograms) annually. The majority of the harvest is prepared for shipment to Japan, or sold locally, primarily to the Miccosukee Tribe (Moler & Berish, 1995).

The increased depth of Lake Okeechobee makes commercial navigation on the Lake possible. Commercial navigation of Lake Okeechobee and associated waterways was used to transport 430,000 tons of freight in 1995. Petroleum products, including distillate fuel oil, residual fuel oil, and liquid natural gas, comprise the majority of tonnage shipped. Other commercial navigation includes fleets of day/dinner cruise vessels that operate during the tourist season from Pahokee and from Ft. Myers and other commercial guided tours.

Agriculture in the vicinity of Reach One is dependent on releases from Lake Okeechobee for crop irrigation. During prolonged droughts, significant volumes of water from Lake Okeechobee are required to supplement local water supplies and to prevent saltwater intrusion into coastal aquifers and wellfields. Most of the land in the vicinity of Reach One is under cultivation. Agriculture in Reach One is dominated by sugarcane, accounting for 90% of land under cultivation. The remaining 10% of cultivated land primarily includes rice, row crops, and sod (David Miller & Associates, 1998).

5.00 ENVIRONMENTAL CONSEQUENCES

5.01 Introduction

This section discusses potential impacts to the existing environment that may result from implementation of each proposed alternative. Assessment of the No Action Alternative includes an increased probability of unsatisfactory performance of the dike system, or dike failure. Assessment of Alternative Nos. 1, 2, and 3 involves impacts associated with construction and utilization of the selected Alternative on the existing environment. A summary of environmental consequences is presented in Table 5 as presented by the USACE.

TABLE 5
SUMMARY OF ENVIRONMENTAL CONSEQUENCES
(Page 1 of 3)

Environmental Components	No Action Alternative	Alternative No. 1	Alternative No. 2	Alternative No. 3
Location & Climate	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)
Topography	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)
Geology	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)
Soils	Potential for displacement of soils nearest dike failure (3)	Displacement of soils during excavation of peat in toe ditch (1)	Displacement of levee soils for cutoff wall, & peat during excavation of toe ditch (1)	Displacement of soils during excavation of toe ditch (1)
Hydrology	Flooding may affect existing evaporation and recharge regime(3)	Elevated water level in toe ditches may result in localized flooding (2)	Cutoff wall may lower water table and recharge rates (2)	Perforated culverts in relief trench should not lower recharge rates (1)
Water Supply	Reduced agricultural water supply at critical times may damage crops (2)	Loss of tail-water control by private interests may cause conflict (2)	Cutoff wall may reduce tail-waters and agricultural water supply (2)	Water Supply not significantly affected (1)
Water Quality	Increased sediments in surface waters due to flooding (2)	Erosion control efforts during construction should minimize impacts (1)	Erosion control efforts during construction should minimize impacts (1)	Erosion control efforts during construction should minimize impacts (1)
Water Management	Short-term alteration of current water management practices likely (2)	Loss of tail-water control by private interests may cause conflict (2)	Current water management practices not significantly affected (1)	Current water management practices not significantly affected (1)
Vegetation & Cover Types	Native vegetation and crops could be damaged by floodwaters (3)	Impacts limited to vegetation along landward side slopes of levee and in/around toe ditches(1)	Impacts limited to vegetation along landward side slopes of levee and in/around toe ditches(1)	Impacts limited to vegetation along landward side slopes of levee and in/around toe ditches(1)
Wetlands	Significant wetland impacts not expected (1)	Significant wetland impacts not expected (1)	Cutoff wall may reduce water supply to landward wetlands nearest dike (2)	Toe ditch to be filled during construction of relief trench (2)

0 = No Consequences 1 = Minimal Consequences 2 = Moderate Consequences 3 = Extensive Consequences

TABLE 5
SUMMARY OF ENVIRONMENTAL CONSEQUENCES
(Page 2 of 3)

Environmental Components	No Action Alternative	Alternative No. 1	Alternative No. 2	Alternative No. 3
Fish & Wildlife	Loss of some wildlife habitat in vicinity of breach (2)	Periodic increase of landward waters may alter some wildlife habitat (1)	Cutoff wall may reduce water supply altering wildlife habitat (2)	Existing toe ditches will be converted to relief trenches (1)
Threatened & Endangered Species	No significant impacts to T & E species expected (1)	No significant impacts to T & E species expected (1)	No significant impacts to T & E species expected (1)	No significant impacts to T & E species expected (1)
Noise	No Consequences Expected (0)	Minimal, temporary, and localized effects due to construction activities (1)	Minimal, temporary, and localized effects due to construction activities (1)	Minimal, temporary, and localized effects due to construction activities (1)
Air Quality	No Consequences Expected (0)	Minimal, temporary, and localized effects due to construction activities (1)	Minimal, temporary, and localized effects due to construction activities (1)	Minimal, temporary, and localized effects due to construction activities (1)
HTRW	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)	No Consequences Expected (0)
Land Use Agriculture	Extensive crop damage possible (3)	Loss of tail-water control by private interests may cause conflict (2)	Cutoff wall may reduce water supply producing localized affects to agriculture (2)	No significant impacts to agriculture is expected (1)
Land Use Urban Land	Loss of property and life possible in worst case dike failure scenario (3)	No significant impacts to urban Land Use expected (1)	No significant impacts to urban Land Use expected (1)	No significant impacts to urban Land Use expected (1)
Land Use Transportation	Flooding may damage roads and railroads (3)	No significant impacts to trans. features expected (1)	No significant impacts to trans. features expected (1)	Seepage berm my extend into Railroad easement. No other impacts expected (1)
Land Use Transmission Lines	Flooding may damage transmission line structures, resulting in power outages (2)	Construction activities may necessitate temporary relocation of transmission lines (2)	Construction activities may necessitate temporary relocation of transmission lines (2)	Construction activities may necessitate temporary relocation of transmission lines (2)

0 = No Consequences 1 = Minimal Consequences 2 = Moderate Consequences 3 = Extensive Consequences

TABLE 5
SUMMARY OF ENVIRONMENTAL CONSEQUENCES
(Page 3 of 3)

Environmental Components	No Action Alternative	Alternative No. 1	Alternative No. 2	Alternative No. 3
Aesthetic Resources	No Consequences Expected (0)	Temporary/Short-term impacts to Localized Areas as a Result of Construction. Possible Vegetation & Tree Removal (1)	Temporary/Short-term impacts to Localized Areas as a Result of Construction. Possible Vegetation & Tree Removal (1)	Temporary/Short-term impacts to Localized Areas as a Result of Construction. Possible Vegetation & Tree Removal (1)
Recreational Resources	No Consequences Expected (0)	Temporary/Short-term Impacts to Parks, Bank Fishing, Bike Trail, Access to Select Lake Side Locations as a Result of Construction (1)	Probably more severe impacts to recreation as a result of construction on lakeside face of levee. Possible lake access restrictions, bank fishing, bike trail impacts (2)	Temporary/Short-term Impacts to Parks, Bank Fishing, Bike Trail, Access to Select Lake Side Locations as a Result of Construction (1)
Cultural Resources	Potential Significant Adverse Effects In Event Of Dike Failure (2)	Minimal, Non-Adverse Effects (1)	Minimal, Non-Adverse Effects (1)	Minimal, Non-Adverse Effects (1)
Socioeconomics	Flooding may result in loss of property and life (3)	No Adverse Consequences Expected. Possible Beneficial Impacts to Local Economy due to Construction Project (0)	No Adverse Consequences Expected. Possible Beneficial Impacts to Local Economy due to Construction (0)	No Adverse Consequences Expected. Possible Beneficial Impacts to Local Economy due to Construction (0)

0 = No Consequences 1 = Minimal Consequences 2 = Moderate Consequences 3 = Extensive Consequences

5.02 Location and Climate

No impact to the location and climate of Reach One, or the Lake Okeechobee region is expected to occur as a result of implementing any of the alternative actions. Nor will taking no action affect this component of the existing environment.

5.03 Topography, Geology and Soils

Topography

No Action Alternative

Selection of the No Action Alternative will cause no significant changes to the topography of Reach One or the Lake Okeechobee region. Although the potential for failure of the Dike system persists under this alternative, major topographic alterations would not occur as a result of a project failure.

Alternatives No. 1, 2 & 3

No significant impact to the topography of Reach One, or the Lake Okeechobee region is expected to occur as a result of implementing any of the alternative actions. Minor changes will occur in the immediate areas where excavation and fill activities take place, but these will cause only minimal changes to the overall topography of these areas.

Geology

No impact to the geology of Reach One, or the Lake Okeechobee region is expected to occur as a result of implementing any of the alternative actions. Nor will taking No Action affect this component of the existing environment.

Soils

No Action Alternative

Although the No Action Alternative would not cause physical changes in the study area, the current instability problems would persist. In the event of dike failure, surging waters could displace mass volumes of soils in the areas nearest the failure. Given the importance of agriculture in the Lake Okeechobee area and its dependence on soil, this would be a significant loss.

Alternative No. 1

Minimal soil disturbance would occur as a result of implementing Alternative No. 1. Plans for this alternative include excavation of 3 ft to 4 ft (0.9 m to 1.2 m) of peat along the landward slope of the Dike prior to construction of a stability berm. Any other soil disturbances such as those associated with movement of heavy equipment will also be minimal.

Alternative No. 2

Minimal soil disturbance would occur as a result of implementing Alternative No. 2. Excavation of the channel for the cutoff wall will result in soil displacement near the crest of the Dike. Plans for this alternative further include excavation of 3 ft to 4 ft (0.9 m to 1.2 m) of peat along the landward slope of the Dike prior to construction of a stability berm. Any other soil disturbances such as those associated with movement of heavy equipment will also be minimal.

Alternative No. 3

Soil impacts resulting from implementing Alternative No. 3 would be minimal. Excavation of an area 25 ft (7.6 m) wide, and 4 ft (1.2 m) deep along the length of Reach One would be required prior to construction of the seepage berm, and would involve only the area in the immediate vicinity of the embankment toe. Any other soil disturbances such as those associated with movement of heavy equipment will also be minimal.

5.04 Hydrology

Impacts to hydrologic features and processes in the Lake Okeechobee area could possibly occur through implementation of a selected alternative. The No Action Alternative allows for the greatest conceivable consequences due to the continued potential for the Dike system to fail. Alternative Nos. l, 2, and 3 each would cause only minimal to moderate impacts if undertaken.

Evaporation

Because of Lake Okeechobee's large surface area, much of the surface water is lost to evaporation each year as a result of natural processes.

No Action Alternative

The No Action Alternative would have a moderate affect on evaporation. During a flood event, the lake water would cover a larger area. An increase in surface area would result in a corresponding increase in evaporation, especially if it were to occur during the summer months. Implementation of this alternative could have similar consequences in Reach One as in the remaining Reaches.

Alternative No. 1

Implementation of Alternative No. 1, increasing the water level in the drainage ditches, would have minimal affect on evaporation as the water surface area would not increase significantly.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2, construction of an impervious cutoff wall to impede groundwater flow, would have minimal effect on evaporation as the water surface area would not increase.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3, installation of a seepage berm, would have minimal effect on evaporation as the water surface area would not increase.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Outfall

Water is released from Lake Okeechobee through the principal outfall canals which include the West Palm Beach, Hillsboro, North New River, Miami, St. Lucie, and Caloosahatchee River Canals (Figure 6). The Caloosahatchee River and St. Lucie canals are the primary outlets for release of flood waters when the Lake is above regulation stages (Fernald and Patton, 1984).

No Action Alternative

In the event of a major failure of the HHD, consequential flooding could have extensive effects on the hydrologic regime of Reach One and the Lake Okeechobee region. The No Action Alternative allows current stability problems of the HHD to persist, which could lead to a major breach of the Dike during a substantial high water event.

Alternative No. 1

Implementation of Alternative No. 1 is expected to result in moderate impacts to the hydrology along Reach One. Raising the water levels in the ditches will increase the potential for localized flooding.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to result in moderate impacts along Reach One. The installation of an impervious cutoff wall and to impede groundwater flow, the possible lowering of the water table due to the cutoff wall may reduce the amount of water available for discharge.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is expected to result in minimal impacts along Reach One. Installation of seepage berms is not expected to significantly affect discharge during high-water events.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Recharge

The groundwater throughout the Lake Okeechobee area is usually within 3.28 ft (1 m) of the land surface and extends to about 330 ft (100 m) bls. This water table generally parallels the land-surface features. Differences in ground elevations are slight and the water table reflects a relatively uniform surface with few undulations. The principal source of recharge to the groundwater in this area is derived from local rainfall and by subsurface percolation from the canals into the permeable sediments.

No Action Alternative

The No Action Alternative would have minimal effect on recharge along Reach One. Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 1

Implementation of Alternative No. 1 is expected to result in minimal impacts along Reach One. Improvement to the existing ditches and controlled water levels are not anticipated to affect recharge.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to result in moderate impacts along Reach One. The installation of a cutoff wall impeding groundwater flow and lowering the water table will affect the principal source of recharge in this area and decrease the effectiveness of subsurface percolation from the canals into the permeable sediments.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is expected to result in minimal impacts along Reach One. The installation of a seepage berm and a relief trench will control artesian pressures, allowing for more effective percolation through the sediments and more effective recharge to surficial aquifer systems.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

5.05 Water Supply

Impacts to water supply in the Lake Okeechobee area could possibly occur through implementation of a selected alternative. The No Action Alternative allows for the greatest conceivable

consequences due to the continued potential for the Dike system to fail. Alternative Nos. l, 2, and 3 each would cause only minimal to moderate impacts if undertaken.

Public Water Supply

Lake Okeechobee serves as a source of public water supply for Canal Point, Clewiston, Belle Glade, Okeechobee, Pahokee, and South Bay. Pahokee's water supply facility is located within 100 ft (30 m) of the HHD along Reach One. Local industries such as sugarcane refineries and produce packaging/distribution centers also employ the available groundwater and surface water for their plant operations.

No Action Alternative

The No Action Alternative would have a minimal effect on existing current water supply issues. In the event of a levee breach in the HHD, even a major one, water levels are expected to recede slowly so that implementing water conservation measures should not be needed.

Alternative Nos. 1, 2, and 3

Implementation of Alternative Nos. 1, 2, or 3 are expected to have minimal impacts on current public water supply issues along Reach One. The proximity of the Pahokee water facility to the HHD may present some permitting issues if a water quality issue is perceived by concerned agencies. The facility is far enough away from the proposed construction area to avoid direct impacts.

Implementation of any of these alternatives should not result in any significant impacts along the remaining reaches of the HHD.

Agricultural Water Supply

Agricultural activities in the Lake Okeechobee area utilize the canals and culverts associated with the Lake as a source of irrigation water for the many sugarcane and truck crops produced in the region. To the south and east of Lake Okeechobee, and adjacent to much of Reach One, the EAA is one of the most productive farming regions in the country. This area relies heavily on water obtained from this resource, notably so along Reach One.

No Action Alternative

In the unlikely event of a failure of the HHD, consequential flooding could have significant effects on agricultural lands in the area of the failure. The No Action Alternative allows current stability problems of the HHD to persist, which could lead to a major breach of the Dike during a substantial high water event.

The disruption of agricultural water supply at a critical time during the growing season could have detrimental effects on the local economy.

Additionally, loss of crops in the vicinity of the breach could be substantial if the breach were to occur in a heavily farmed area.

Selection of the No Action Alternative could result in extensive consequences to agricultural lands around Lake Okeechobee, and along Reach One.

Alternative No. 1

Implementation of Alternative No. 1 is expected to result in only moderate impacts to the agricultural water supply along Reach One. Presently, the water levels in the ditches are controlled by local drainage districts and farmers. Under this alternative, these entities would no longer have this control. Culverts with automatic/manual gates and pumps would be installed to control the water level in ditches along Reach One. Water levels could thus be raised as needed to decrease differential head, and increase the stability of the HHD. The overall influence of altered water levels in these ditches is significant if it is perceived as loss of irrigation water by the local agriculturists.

Implementation of this alternative should not result in any significant impacts to agricultural water supply in the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to result in moderate impacts to agricultural water supply along Reach One as well. Installation of a cutoff wall along the length of this Reach could result in reduced tail-waters. The extent of this reduction, if any, is unknown, but any reduction in irrigation waters could adversely affect irrigated crops in this area.

Implementation of this alternative should not result in any significant impacts to agricultural water supply in the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is expected to result in minimal impacts to the agricultural water supply along Reach One. The perforated culvert system included in the seepage berm design will collect and convey water into existing drainage canals, just as the existing toe ditches do. No reduction in irrigation water is expected to occur as a result of this alternative.

Implementation of this alternative should not result in any significant impacts to agricultural water supply in the remaining reaches of the HHD.

5.06 Water Quality

Impacts to water quality in the Lake Okeechobee area could possibly occur through implementation of a selected alternative. The No Action Alternative allows for the greatest conceivable consequences due to the continued potential for the Dike system to fail. Alternative Nos. l, 2, and 3 each would cause minimal to moderate impacts if undertaken.

No Action Alternative

The No Action Alternative would have moderate effects on existing water quality due to increased sediments in the surface waters nearest a breach. Implementation of this alternative should not result in any significant impacts outside of the immediate area of the breach. It is fair to envision though, in the event of a high velocity breach in the general area between the St. Lucie and L-8 Canals, in combination with high lake levels, that downstream discharge towards the St. Lucie Estuary may cause significant adverse impacts to the estuarine ecosystem. It is reasonable to assume that any levee breach of the HHD system would likely be preceded by significant freshwater releases through the S-308 at Port Mayaca in anticipation of climatic conditions raising water levels and threatening the HHD. These releases too, would have acute detrimental impacts to sensitive estuarine water quality far downstream.

Alternative Nos. 1, 2, and 3

Implementation of Alternative Nos. 1, 2, or 3 is expected to have minimal to moderate impacts on the water quality along Reach One. Construction activities may result in increased sediment load in nearby surface waters. Silt screens and other control erosion control measures should be implemented during and after construction to reduce this potential.

Implementation of these alternatives should not result in any significant impacts to water quality in the remaining reaches of the HHD.

5.07 Water Management

Impacts to water management in the Lake Okeechobee area could possibly occur through implementation of a selected alternative. The No Action Alternative allows for the greatest conceivable consequences due to the continued potential for the Dike system to fail. Alternative Nos. l, 2, and 3 each would cause only minimal to moderate impacts if undertaken.

No Action Alternative

The No Action Alternative would have moderate effects on the existing water management of Reach One. In the vicinity of the breach and resulting flooded area, water levels resulting from a major dike failure would necessitate implementation of significantly different management activities by those entities responsible for, and relying upon, the Lake's water resources.

Alternative No. 1

Implementation of Alternative No. 1 is expected to have moderate impacts on the water management along Reach One. Increased water levels in the drainage ditches would continue to be managed according to a regulation schedule that was developed by the SFWMD and the USACE. However, the loss of control by the local management districts could create political issues.

Implementation of this alternative should not result in any significant impacts to water management in the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to have minimal impacts on water management along Reach One.

Implementation of this alternative should not result in any significant impacts to water management in the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is expected to have minimal impacts to water management along Reach One. The culvert system for collecting seepage flows and surface runoff will continue to be managed according to a regulation schedule that was developed by the SFWMD and the USACE.

Implementation of this alternative should not result in any significant impacts to water management in the remaining reaches of the HHD.

5.08 Vegetation and Cover Types

No Action Alternative

The No Action Alternative and the continued potential for dike failure could result in extensive impacts to the vegetation landward of the HHD.

Loss of vegetation landward of the Dike would occur in the immediate area of the failure. This loss would result from surging water which would impact sugarcane plantations, ornamental orchards, field crops, and natural vegetation. Further, an interruption in the current hydrology, such as less irrigation over the long-term, could severely inhibit the ability to grow commercial crops. Additionally, de-vegetation of these areas could provide an area for invasive plant species to expand into.

Loss of vegetation waterward of the Dike, in the event of dike failure, would be minimal. Some loss of aquatic vegetation would be expected in the immediate vicinity of the breach; however, the overall reduction in lake levels would be relatively small and no impacts to vegetation would be experienced in areas away from the breach site.

Alternative Nos. 1, 2 & 3

Implementation of Alternative Nos. 1, 2, & 3 would cause only minimal impacts to existing vegetation, could be temporary in nature, and limited to the immediate areas of construction. Along the landward side of the HHD where major construction occurs, existing vegetation would be disturbed. This would impact mixed grasses along the slope of the Dike, and vegetation in the toe ditches. Much of the ditch vegetation is considered nuisance vegetation, such as water hyacinth, water lettuce, and hydrilla.

Implementation of these alternatives should not result in any significant impacts to the vegetation and cover types in the remaining reaches of the HHD.

5.09 Wetlands

No Action Alternative

Selection of the No Action Alternative would lead to minimal wetland impacts if there should be a failure of the Dike system to hold water. These impacts would result from increased water levels due to flooding landward of the HHD.

Landward, wetland impacts in the area of Reach One would be minimal because of the relatively few quality wetlands remaining in this area. Wetland systems in the remaining reaches would not be affected due to the relatively small change in lake level that would result from a breach of the HHD.

Waterward, wetland impacts due to lower lake levels would be minimal. Conditions under which a breach would be likely, and corresponding breach-related reduction in lake levels, would not result in levels that approach the minimum water level criteria (Section 4.07). In the event of a prolonged scenario in which repairs of a large breech take more than a month to repair, water levels would still not fall below the front toe of the levee.

Alternative No. 1

Wetland impacts resulting from implementation of Alternative No. 1 would be minimal and limited to Reach One. This alternative involves raising water levels landward of the Dike during high lake water events. This practice would reduce differential head, and increase dike stability during these times. The landward wetlands involved could be altered, though not necessarily resulting in a negative outcome. Wetland areas could potentially enlarge as a result of this increased water level, providing additional wildlife habitat. Considering the low quality of wetlands existing along Reach One, negative impacts, if any, would be minimal.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 2

Wetland impacts resulting from implementation of Alternative No. 2 would be moderate and limited to Reach One. This alternative involves construction of a cutoff wall which could potentially alter the amount of water that wetlands landward of the HHD receive. This is an uncertain consequence though, and considering the low quality of wetlands existing along Reach One, negative impacts, if any, would be moderate.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

Alternative No. 3

Wetland impacts resulting from implementation of Alternative No. 3 would be moderate and limited to Reach One. This alternative involves converting existing toe ditches to a controlled system of covered culverts as part of a seepage berm. This activity would eliminate the foraging potential along these ditches. A variety of wading birds, small fishes and invertebrates utilize the ditches. Although these areas provide less than optimal habitat, impacts would be relatively significant.

Implementation of this alternative should not result in any significant impacts to the remaining reaches of the HHD.

5.10 Fish and Wildlife

No Action Alternative

The No Action Alternative would cause no direct impact to fish and wildlife in the Lake Okeechobee area. However, the current instability problems would persist, and probability of

unsatisfactory performance of the HHD could increase. In the event of Dike failure, minimal loss of wildlife habitat would occur waterward of the HHD. This is due to the relatively small reduction in lake level that would result from a failure. Modeling results demonstrate less than 0.75 ft (0.23 m) drop in lake levels within 45 days at 18 ft (5.5 m) NGVD, in the event of a breach in Reach One (USACE, 1998).

The implications to fish and wildlife landward of the HHD that may result from dike failure would be limited to the areas of the breach and surrounding habitats. In the area of Reach One, fish and wildlife habitat is marginal. However, those animals most significantly affected by extensive flooding include those with limited mobility. Amphibians, reptiles, and small mammals would be impacted to a moderate degree.

Alternate No. 1

Fish and wildlife impacts resulting from implementation of Alternative No. 1 would be minimal and limited to Reach One. Since this alternative involves raising water levels landward of the Dike periodically, wildlife areas (primarily ditches) would be impacted to some extent. Considering the low quality of fish and wildlife habitat existing along Reach One, negative impacts, if any, would be minimal.

Implementation of this alternative should not have any significant impacts to the fish and wildlife along the remaining reaches of the HHD.

Alternate No. 2

Fish and wildlife impacts resulting from implementation of Alternative No. 2 would be moderate and limited to Reach One. This alternative involves construction of a cutoff wall which could potentially alter the amount of water that wetlands landward of the HHD receive. These are virtually the only remaining wetland habitats landward of Reach One, and although these are low quality habitats, negative impacts would be moderate.

Implementation of this alternative should not have any significant impacts to the fish and wildlife along the remaining reaches of the HHD.

Alternate No. 3

Fish and wildlife impacts resulting from implementation of Alternative No. 3 would be minimal and limited to Reach One. This alternative involves converting existing toe ditches to a controlled system of covered culverts as part of a seepage berm. This activity would eliminate the foraging potential along these ditches. But, considering the low overall quality of these ditches as wildlife

habitat, and the proximity of miles of additional ditches throughout the adjacent agricultural areas, impacts to fish and wildlife would be minimal.

Implementation of this alternative should not have any significant impacts to the fish and wildlife along the remaining reaches of the HHD.

5.11 Threatened and Endangered Species

American alligator *Alligator mississippiensis*

No Action Alternative

The American alligator should incur only minimal short-term impacts in the event of a dike failure both waterward and landward of the HHD. Flexibility in habitat usage and mobility should allow this animal to survive in the Lake Okeechobee region even in the event of major water level drop. If a dike failure should occur during nesting season, the impacts waterward should be minimal since water levels are not expected to decrease significantly during such an event. However, the potential for impacting existing nests landward of the Dike exists to those nests located in the immediate vicinity of a breach.

Alternative Nos. 1, 2 & 3

Impacts to the American alligator resulting from implementing Alternative Nos. 1, 2, or 3 would be minimal to moderate. Any impacts would be limited to the immediate area of construction, and considering the low quality of existing habitat, should be minor.

Implementation of a selected alternative should not have any significant impacts to the American alligator along the remaining reaches of the HHD.

Eastern Indigo Snake *Drymarchon corais couperi*

No Action Alternative

The eastern indigo snake would likely only be affected minimally in the event of a dike failure. Low utilization of areas waterward of the HHD, would limit potential impacts. The levee itself provides useable habitat for the indigo snake, but a dike failure would only directly affect animals in the immediate vicinity. Landward, this animal is rarely observed due to sub-optimal habitat. Any impacts would be minimal, and only in the immediate area of the dike failure.

Alternative Nos. 1, 2 & 3

Impacts to the eastern indigo snake resulting from implementing Alternative Nos. 1, 2, or 3 would be minimal to moderate, and limited to the immediate area of construction. Considering the quality of existing habitat for the indigo snake along the lower third of the Dike, construction impacts may occur, but impacts to snakes may be mitigated by proper implementation of an environmental protection plan.

Implementation of a selected alternative should not have any significant impacts to the eastern indigo snake along the remaining reaches of the HHD.

Bald Eagle *Haliaeetus leucocephalus*

No Action Alternative

The slightly lower water levels resulting from a dike failure should impact the bald eagle to a minimal extent. The expected decrease in water level is too minor to significantly affect its foraging activities around the Lake.

Alternative Nos. 1, 2 & 3

Impacts to the bald eagle resulting from implementing Alternative Nos. 1, 2, or 3 are expected to be minimal. However, the existence of an active bald eagle nest could alter construction plans. An active nest within 1500 ft (457 m) of the Dike would restrict construction activities during nesting season. As recently as 1995, one active bald eagle nest was reported within 1500 ft (450 m) of the HHD. This nest was located on the waterward side of the levee along Reach One, south of Pahokee.

Implementation of a selected alternative should not have any significant impacts to the bald eagle along the remaining reaches of the HHD.

Wood Stork *Mycteria americana*

No Action Alternative

Impacts to the wood stork in the event of a dike failure would be minimal. Slightly lower lake levels could result in slightly less foraging habitat around the Lake. Any nesting colonies could be deserted if de-watered at a critical nesting time during the year; however, reduction in lake level due to breaching would be minimal.

Alternative Nos. 1, 2 & 3

Impacts to the wood stork resulting from implementing Alternative Nos. 1, 2, or 3 would be minimal to moderate. The wood stork could potentially utilize the toe ditch and adjacent wetlands for foraging activities. However, considering the relatively low quality of these habitats impacts should not be significant.

Implementation of a selected alternative should not have any significant impacts to the wood stork along the remaining reaches of the HHD.

Snail Kite *Rosthrhamus sociabilis plumbeus*

No Action Alternative

Impacts to the snail kite's significant habitat around Lake Okeechobee would be minimal if there should be a major dike failure. The southwest shore of Lake Okeechobee from the Hurricane Gate at Clewiston to the Kissimmee River (excluding deep open water) is considered critical habitat for the snail kite. The water level must be sufficiently stable to prevent loss of the apple snail through drying out of the surface. Water loss in this area, in the event of a dike failure would not be great enough to seriously affect successful foraging of the highly mobile snail kite.

Alternative Nos. 1, 2 & 3

Impacts to the snail kite resulting from implementing Alternative Nos. 1, 2, or 3 would be minimal, and restricted to the immediate area of construction. Construction activities will be limited to the levee itself and the landward side of the levee where this animal doesn't forage extensively. Aside from temporal disturbance caused by the operation of heavy equipment, no impact is expected waterward either. Due to the relatively narrow littoral zone here, this area provides minimal snail kite foraging habitat, so impacts are unlikely.

Implementation of a selected alternative should not have any significant impacts to the snail kite along the remaining reaches of the HHD.

West Indian Manatee *Trichechus manatus*

No Action Alternative

Minimal impacts to the West Indian manatee are expected to occur in the event of a dike failure. Expected water level reductions would not be great enough to affect the animal's food supplies or exposure to boat-related injury or death.

Alternative Nos. 1, 2 & 3

Impacts to West Indian Manatee resulting from implementing Alternative Nos. 1, 2, or 3 would be minimal to none. Construction activities will be limited to the levee itself and the landward side where this animal does not occur.

Implementation of a selected alternative should not have any significant impacts to the West Indian manatee along the remaining reaches of the HHD.

Okeechobee Gourd *Curbita okeechobeensis o.*

No Action Alternative

Okeechobee gourd plants that are currently known to exist in the Lake Okeechobee region are limited to the shores of the Lake inside of the HHD. Slightly lower lake levels resulting from a major dike failure would have minimal impact to the existing Okeechobee gourd population in this area. However, given its limited range and habitat requirements, any alteration in the hydrology where this plant currently exists could significantly damage the population. Impacts to these gourds would most likely occur with sustained high water events, rather than low.

Alternatives Nos. 1, 2 & 3

Implementation of Alternative Nos. 1, 2, or 3 would not likely cause any impacts to the Okeechobee gourd. The occurrence of this plant along the landward extent of Reach One has not been recorded in recent years.

Implementation of a selected alternative should not have any significant impacts to the Okeechobee gourd along the remaining reaches of the HHD.

5.12 Noise

No Action Alternative

The No Action Alternative would not increase ambient noise levels in the vicinity of the HHD. Therefore no impacts are expected to result due to selection of this alternative.

Alternatives Nos. 1, 2 & 3

The implementation of Alternative Nos. 1, 2, and 3 could potentially result in some noise impacts, but would be limited to the sites directly associated with construction activities. Occasional

heavy machinery activity in these areas would produce noise levels above 70 dB in localized areas, but would occur sporadically and should not lead to reduced attenuation of animal species or humans living near the area. Staging areas which will be established at suitable locations within the USACE right-of-way may also experience potential noise impacts, as well as access routes to the crown road. Such routes include the following:

a. County Road 717 near S-351
b. Hooker Highway off State Road 15
c. Paul Rardin Park of State Road 715
d. Culvert 12A off State Road 715
e. Culvert 10 off State Road 715
f. Pahokee State Park off State Road 715
g. S-352 off State Road 715
h. Culvert 10A off State Road 15/700
i. Culvert 14 off State Road 15/700
j. Port Mayaca (S-308) off State Road 15/700.

Implementation of a selected alternative should not result in any increased noise issues in the remaining reaches of the HHD.

5.13 Air Quality

No Action Alternative

Selection of the No-Action Alternative would not impact air quality in the vicinity of the HHD.

Alternatives Nos. 1, 2 & 3

Emissions associated with each of the three Alternatives would be largely generated from heavy machinery operating for short periods in the area of Reach One. Construction activities will cause minor short-term air quality impacts in the form of fugitive dust or airborne particulate matter from earthwork and unpaved roads accessed for the project.

The area is rural and the existing air quality is good to moderate, additional short-term loadings of internal-combustion engine gases would not substantially impact the quality of the air in the vicinity of the HHD.

Every Federally funded project must be consistent with state plans for implementing the provisions of the Clean Air Act Amendments (State Implementation Plans). This project is in conformance with the State Implementation Plan because it will not cause violations of the National Ambient Air Quality Standards.

Implementation of a selected alternative should not result in any air quality issues in the remaining reaches of the HHD.

5.14 Hazardous, Toxic and Radioactive Wastes

A Phase I HTRW Site Assessment was conducted within and around the study area in conformance with the scope and limitations of ASTM Practice E 1527 and Engineer Regulation ER-1165-2-132 The findings and conclusions provided below reflect existing HTRW conditions based on database searches, aerial photography, review of available records, site inspections and interviews. These findings and conclusions are of existing conditions at this time. The project conditions assume that any HTRW found during any phase of the project would be remediated in accordance with local, state and Federal laws. Therefore, it can be assumed that conditions at future construction sites will be contamination free or of low levels, which would include de minimis conditions that generally do not present a material risk of harm to public health or the environment.

Within the agricultural areas there are numerous temporary pump sites and fuel storage areas. These makeshift portable tanks are not reported, and therefore are not presented in the HTRW database. In addition, pesticide/chemical-mixing areas may also exist. Agricultural fields, outbuildings, equipment fueling and agricultural processing facilities are expected to be outside of the immediate area of construction and should not pose an HTRW concern. Project implementation requires that any HTRW problems revealed during the real estate acquisition or actual project construction require full remediation.

The database and Phase-1 assessment results are maintained at the U.S. Army Corps of Engineers, Planning Division, as a series of maps and GIS overlays. It is not appropriate to include detailed information in this report and EIS. However, the data will be available to planners for consultation during development of specific project features, and is expected to aid in avoiding sites with major remediation needs.

5.15 Land Use

Impacts to existing land use in the Lake Okeechobee area could possibly occur through implementation of a selected alternative. The No Action Alternative allows for the greatest conceivable consequences due to the continued potential for the Dike system to fail. Alternative Nos. l, 2, or 3 would each cause only minimal to moderate impacts if undertaken.

Major land use elements in the Lake Okeechobee area include agriculture, urban land, transportation, transmission lines, communication, and quarries. The anticipated consequences to these elements are presented in this Section with emphasis on the vicinity of Reach One.

Figure 7 depicts the general location of land use features along Reach One. Figures 8, 9, & 10 enlarge Reach One into three sections (Reach 1A, 1B, & 1C, respectively) for better representation of these features.

Agriculture

No Action Alternative

The No Action Alternative could potentially have the greatest impact on agriculture in the Lake Okeechobee area. In the event of a major structural failure of the HHD, consequential flooding could have detrimental effects on crops in the area of the failure. In the vicinity of Reach One, sugarcane, ornamental nurseries, and vegetable production are the major production crops. Flood waters from a breach of Reach One could result in immediate and long-term damage to crops in this area. The extent of agricultural damage in total acreage would be dependent upon the location of the breach in relation to agricultural activities, the lake levels at the time of the breach, and duration of flooding.

Alternative No. 1

Implementation of Alternative No. 1 is expected to have moderate impacts on the agriculture along Reach One. Currently, tail-water in the agricultural areas is controlled by local drainage districts and farmers. Under this alternative, these entities would no longer have this control. Culverts with automatic/manual gates and pumps would be installed to control the water level in ditches along Reach One. Water levels could thus be raised as needed to decrease differential head, and increase the stability of the HHD. The overall influence of altered water levels in these ditches is significant if it is perceived as loss of irrigation water by the local farmers.

Implementation of this alternative should not result in any significant impacts to agriculture in the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to have moderate impacts on the agriculture along Reach One. Installation of a cutoff wall along the length of this Reach could result in reduced tail-waters. The extent of this reduction, if any, is unknown, but any reduction in irrigation waters could adversely affect irrigated crops in this area.

Implementation of this alternative should not result in any significant impacts to agriculture in the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is expected to cause only minimal impacts to agriculture along Reach One. The perforated culvert system included in the seepage berm design will collect and convey water into existing drainage canals, just as the existing toe ditches do. No reduction in irrigation water is expected to occur as a result of this alternative.

Implementation of this alternative should not result in any significant impacts to agriculture in the remaining reaches of the HHD.

Urban Land

No Action Alternative

In the event of a major failure of the HHD, consequential flooding could have significant effects on urban lands in the area of the failure. The No Action Alternative allows current stability problems of the HHD to persist, which could lead to a major breach of the Dike during a substantial high water event. Loss of life and property in the vicinity of the breach could be substantial if the breach were to occur in a heavily populated area.

Along Reach One, the city of Pahokee is situated along the very toe of the HHD. This city, with a population of nearly 7000 people, has homes and businesses occupying land immediately adjacent to the Dike. These people and properties are subject to immediate danger if large scale flooding should occur due to dike failure. The city of Belle Glade, with a population of over 16,000 people, is near enough to be influenced by large scale flooding as well. Although the central portion of the city is about one mile (1.6 km) southeast of the HHD, portions of the city occupy land in immediate proximity to the HHD.

Impacts to Pahokee and Belle Glade would be most significant in the event of a high velocity breach associated with a large storm or hurricane event. These consequences may be even greater should a high velocity breach occur during a storm event, at an unseemly hour of the morning, and evacuations do not go as planned.

Selection of the No Action Alternative could result in significant consequences to urban lands within the remaining Reaches around Lake Okeechobee.

Alternative No. 1

Implementation of Alternative No. 1 is expected to result in only minimal impacts to the urban lands along Reach One. Construction activities will be limited to the HHD easement, and should not affect homes or businesses in this area.

Implementation of this alternative should not result in any significant impacts to urban lands in the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to result in only minimal impacts to the urban lands along Reach One as well. Construction activities will be limited to the HHD easement, and should not affect homes or businesses in this area.

Implementation of this alternative should not result in any significant impacts to urban lands in the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is also expected to result in only minimal impacts to the urban lands along Reach One. Construction activities will be limited to the HHD easement, and should not affect homes or businesses in this area.

Implementation of this alternative should not result in any significant impacts to urban lands in the remaining reaches of the HHD.

Transportation

No Action Alternative

Major transportation corridors around the perimeter of Lake Okeechobee include several highways and railroads. If selection of the No Action Alternative and continued levee degradation were to result in a major failure of the HHD, impacts to existing transportation structures would be extensive.

Along Reach One, Highway 98/441, Highway 715, and the Florida East Coast Railroad parallels the HHD for some portion of the Reach (see Section 4.15). In the event of major flooding, the structures nearest the breach could be destroyed. Any travelers or freight moving along these transportation lines would be endangered as well. Even moderate flooding from a low velocity breach is likely to cause road closures and traffic delays.

Alternative No. 1

Implementation of Alternative No. 1 is expected to result in only minimal impacts to the transportation corridors along Reach One. Construction activities are not expected to impact highway or rail structures in the vicinity. Highway traffic can be expected to increase slightly due to movement of construction equipment and personnel, but should result in only minimal impacts.

Implementation of this alternative should not result in any significant impacts to transportation corridors in the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 is expected to result in only minimal impacts to the transportation corridors along Reach One. Construction activities are not expected to impact highway or rail structures in the vicinity. Highway traffic can be expected to increase slightly due to movement of construction equipment and personnel, but should result in only minimal impacts.

Implementation of this alternative should not result in any significant impacts to transportation corridors in the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 is expected to result in only minimal impacts to the transportation corridors along Reach One.

The Florida East Coast Railroad parallels Reach One along the northern half of the Reach and comes within 75 ft (23 m) in places. Although the railroad structure is far enough removed for construction of the seepage berm, railroad drainage ditches and property easements could be encroached.

Highway traffic in the area of Reach One can be expected to increase slightly due to movement of construction equipment and personnel, but should result in only minimal impacts.

Implementation of this alternative should not result in any significant impacts to transportation corridors in the remaining reaches of the HHD.

<u>Transmission Lines</u>

No Action Alternative

The No Action Alternative and continued dike stability problems could result in damage to existing electrical transmission lines. A low voltage electric transmission line is situated on the landward slope the HHD in the northern half of Reach One. A dike failure in this area could result in structure damage and power outages down-flow of this event. For this reason, selection of the No Action Alternative could lead to moderate impacts to existing transmission lines and local power lines.

Alternative No. 1

Implementation of Alternative No. 1 would result in moderate impacts to existing electric transmission lines. Those lines physically located upon the landward slope in the northern portion of Reach One may require temporary relocation. Construction of the proposed stability berm and equipment access to the toe of slope may be inhibited by the presence of transmission lines.

Implementation of this alternative should not result in any significant impacts to transmission lines in the remaining reaches of the HHD.

Alternative No. 2

Implementation of Alternative No. 2 would result in moderate impacts to existing electric transmission lines. Those lines physically located upon the landward slope in the northern portion of Reach One may require temporary relocation. Construction of the proposed stability berm, and equipment access to the toe of slope may be inhibited by the presence of the transmission lines.

Implementation of this alternative should not result in any significant impacts to transmission lines in the remaining reaches of the HHD.

Alternative No. 3

Implementation of Alternative No. 3 would result in moderate impacts to existing electric transmission lines. Those lines physically located upon the landward slope in the northern portion of Reach One may require temporary relocation. Construction of the proposed seepage berm, and equipment access to the toe of slope may be inhibited by the presence of transmission lines.

Implementation of this alternative should not result in any significant impacts to transmission lines in the remaining reaches of the HHD.

Communication

The only significant communication structure in proximity to Reach One is a Florida Highway Patrol communication tower south of the West Palm Beach Canal. This structure is approximately 50 to 75 ft (15 to 23 m) beyond the back toe of the levee.

No Action Alternative

A breach in the Dike could potentially damage this communication tower if it occurred near enough to the structure. Negative implications could be loss of communication between law enforcement officials in this area.

Alternative No. 1

No impact to the communication tower is expected to result from implementation of Alternative No. 1.

Implementation of this alternative should not result in any significant impacts to communication in the remaining reaches of the HHD.

Alternative No. 2

No impact to the communication tower is expected to result from implementation of Alternative No. 2.

Implementation of this alternative should not result in any significant impacts to communication in the remaining reaches of the HHD.

Alternative No. 3

No impact to the communication tower is expected to result from implementation of Alternative No. 3.

Implementation of this alternative should not result in any significant impacts to communication in the remaining reaches of the HHD.

Limestone Quarries

No Action Alternative

The No Action Alternative will cause no significant impact to the limestone quarries in southern portion of Reach One.

Alternative No. 1

Implementation of Alternative No. 1 would cause no significant impact to the limestone quarries in southern portion of Reach One.

Alternative No. 2

Implementation of Alternative No. 2 would cause no significant impact to the limestone quarries in southern portion of Reach One.

Alternative No. 3

Implementation of Alternative No. 3 could potentially cause some alteration to the existing limestone quarries in the southern portion of Reach One. To allow adequate space for construction of the seepage berm in this area, some filling along the western edges of these quarries may be required. Fill material would be stabilized with gabions along the far back toe of the newly constructed berm, precluding the development of a shallow water littoral zone.

As these quarries are no longer being mined along the toe of the HHD, and considering the low practical quality of these quarries as habitat, impacts to this area as a result of implementing Alternative No. 3 would be minimal.

5.16 Aesthetic Resources

Impacts to aesthetic resources within the project area will be due to construction activities and/or access of construction equipment through lands designated for staging, access and construction. There are no adverse impacts to aesthetic resources due to the no-action alternative. Impacts resultant from the alternatives considered, including the preferred alternative, would be similar in scope, magnitude, and duration. Therefore the discussion below will reference the preferred alternative as a surrogate for impacts associated with construction activities and/or access of construction equipment associated with all action alternatives.

Residents and visitors to Torry Island will experience temporary construction impacts to the view shed aesthetics while County Road 717 near S-351 is used as a construction access point. The construction impacts should return to normal after the project is completed. It appears the 50-foot construction right-of-way will not adversely affect aesthetic resources in the area.

The proposed Hooker Highway construction access point off of S.R. 15 will not adversely affect aesthetic resources found in this area. No residents are located in the area.

The proposed Paul Rardin Park construction access point could adversely affect aesthetic resources in the project area. The heavy equipment access over the park entrance road will temporarily increase noise and dust in the area. The recommended plan's estimated 50 foot wide construction right-of-way would require park vegetation and picnic facilities to be removed from the park property. The tree removal adjacent to the levee toe could expose the park visitor to quarry operations to the south. Stately royal palms that border the entrance road would also be adversely affected due to their proximity to the dike toe. Heavy equipment traffic traversing the park entrance road and construction activities would disrupt the quiet park setting during construction.

The proposed construction access at the Culvert 12A location should not adversely affect aesthetic resources in the area. The access route is screened from adjacent property owners by trees on both sides of the existing road. Some temporary impact to aesthetic resources within the area can be expected during construction. Trees within the 50-foot right-of-way of the levee toe will be removed in this area.

The proposed construction access at the Bacom Point (Culvert 10) area should not adversely affect aesthetics. The proposed access road is through adjacent agricultural fields. Some temporary increase in dust and noise from the heavy equipment can be expected in the project area. It appears the 50-foot construction right-of-way will not adversely affect aesthetic resources in the area either.

The proposed construction access at Pahokee Marina and Campground will temporarily disrupt the peaceful park setting. Construction traffic noise will greatly reduce the aesthetic appeal of the lakefront park north and south of the entrance. The view of Lake Okeechobee should not be obstructed by the construction of the recommended plan. Existing vegetation and residential properties along the 50-foot construction right-of-way could be adversely affected in the project area.

The proposed construction access at S-352 off of S.R. 715 will utilize the Canal Point Park entrance road and paved levee crown for construction vehicle access. The temporary construction access and construction will affect the aesthetics of the park area. Some temporary affects during construction could include additional noise, dust, and air pollution. Pre-existing conditions should return once the project has been completed. Exotic vegetation could be affected along the 50-foot construction right-of-way.

The proposed construction access at Culvert 10-A, Culvert 14, and at Port Mayaca off of S.R. 15/700 are in relatively isolated locations with minimal residential development in the area. Temporary construction impacts will not be significant and pre-existing conditions should return once the project has been completed.

5.17 Recreational Resources

Impacts to recreation resources within the project area will be due to construction activities and/or access of construction equipment through lands designated for staging, access and construction. There are no adverse impacts to recreation resources due to the no-action alternative. Impacts resultant from the alternatives considered, including the preferred alternative, would be similar in scope, magnitude, and duration. Therefore the discussion below will reference the preferred alternative as a surrogate for impacts associated with construction activities and/or access of construction equipment associated with all action alternatives.

Access to construction areas from County Road 717 near S-351 would necessitate heavy equipment traversing and working in the area of a public golf course, fishing, camping, boating, sightseeing, and picnicking facilities that are located in the area. Use of these recreation resources is experienced throughout the year. Increased noise and air pollution could be expected during construction activities. Pre-construction conditions would return upon completion of the project construction. No permanent adverse impacts to recreation resources in the project vicinity are expected to occur as a result of the recommended plan. Projected adverse impacts to the local

economy due to a decline in recreation resource use during construction activities is expected to be moderate in magnitude for the duration of the recommended plan construction.

No structured recreation resources exist at the proposed Hooker Highway construction access point off of SR 715. Some temporary impacts to bank fishing use may occur during the construction of the recommended plan. Construction staging areas are projected to interrupt some recreation activities in the area.

Construction activities would significantly impact the Paul Rardin Park. Seasonal park visitor sight-seeing experiences would be adversely affected by increased construction traffic, noise, dust, and air pollution expected during the project. Adverse impacts to or removal of existing park trees and amenities may result due to the recommended plan construction or construction equipment traffic.

No structured recreation resources exist at the proposed Culvert 12A and Culvert 10 construction access point adjacent to SR 715. Some temporary impacts to bank fishing may occur during the recommended plan construction. Construction staging areas would have no impacts to recreation resources in the area.

Construction activities and equipment access may adversely affect the City of Pahokee Marina and Campground facilities. Park fishing, boat ramp, picnicking, and camping access would be interrupted by the construction vehicle traffic and project construction. Park visitors would experience increased noise, dust, and air pollution during the construction of the recommended plan. Excessive wear of the park's paved roads could occur. The construction project and its traffic would adversely affect many park users. Disruption of recreation resources could pose an adverse impact to the local economy of the City of Pahokee.

The proposed access and construction of the recommended plan near S-352 from SR 715 would interrupt the popular Palm Beach County, Canal Point Park fishing spot. The remnants of the walls and doors of the historical West Palm Beach Canal Lock are a testament to past activity. Local residents fish from the lock walls, banks, and bridge.

The recommended plan construction project and access to the dike crown near Culvert 10A from SR 15/700 would result in some temporary disturbance of local bank fishing in the area.

The access and construction of the recommended plan near Culvert 14 from SR 15/700 could result in temporary restriction of bank fishing in the area. Local residents fish in the area and a secondary trailhead linkage to the Florida National Scenic Trail has been proposed to include access for pedestrians and bicyclists and provide parking for up to five cars (FDOT, 1998). When project construction has been completed, recreation use in the area can be expected to return to pre-construction conditions.

The proposed access and construction of the recommended plan at Port Mayaca (S-308) would adversely affect park visitors during the project construction. Basin bank fishing and dike sightseeing access could be interrupted during project construction. An increase in noise, dust, and air pollution would also be experienced. The Florida National Scenic Trail access would be interrupted during construction. When construction of the recommended plan has been completed, recreation use in the area can be expected to return to pre-construction conditions.

The construction of the recommended plan for the Herbert Hoover Dike Major Rehabilitation Evaluation Project will adversely affect recreation resources in the project area. Most of the construction impacts will result in a temporary disruption due to increased noise, dust and heavy equipment traffic. Other impacts may have a longer lasting affect.

Paul Rardin Park, Pahokee Marina and Campground, and Canal Point Park facilities will be affected the most. Trees, roadway, and picnic shelters may fall within the 50-foot wide construction right-of-way at Paul Rardin Park. Construction equipment travel over the park's entrance road may accelerate maintenance surfacing schedules. Park amenities may require accelerated maintenance schedules.

Pahokee Marina and Campground may experience accelerated roadway maintenance surfacing schedules. Campground amenities may require accelerated maintenance schedules. Trees may be lost that screen adjacent properties, provide shade or frame views due to compaction, stress, or removal.

Canal Point Park's entrance road may experience an accelerated maintenance-resurfacing schedule due to construction equipment travel. The popular historic locks and lock wall fishing area may fall within the 50-foot wide construction right-of-way, which would permanently deny access.

5.18 Cultural Resources

The HHD (FMSF #8PB2028) is historically significant and may be eligible for inclusion on the National Register of Historic Places (NRHP). All alternatives, including the No Action Alternative will have an effect on the historic significance of the HHD. The No Action Alternative could have extensive consequences to this historic property and create an adverse effect. However, the COE has determined, in compliance with the National Historic Preservation Act of 1966, as amended (NHPA), that Alternatives 1, 2, and 3 will cause minimal impacts and the effect will not be adverse. In a letter dated August 14, 1998, the Florida State Historic Preservation Officer (SHPO) concurred with this determination of no adverse effect. A number of historic structures and significant archeological resources are located near the HHD, although none are located within the HHD right-of-way. The No Action Alternative, with its continued potential for dike failure and catastrophic flooding, would have moderate to extensive consequences on these historic properties. Alternatives 1, 2, and 3 would cause only minimal to moderate short-term impacts if implemented. The COE has determined, in compliance with the NHPA, that Alternatives 1, 2, and 3 will have no effect on properties listed on or eligible for

listing on the NRHP. The SHPO concurred that historic properties outside the HHD right-of-way will not be affected.

5.19 Socioeconomics

No Action Alternative

If the No Action Alternative is selected, there could be significant socioeconomic implications, as it could allow for continued degradation of the HHD's stability leading to a breach of the Dike. The potential for loss of life and property from a breach is significant to residents along the HHD as well as those that utilize the Dike for recreation. This alternative does not provide adequate protection from the seepage and stability problems that threaten critical areas of Reach One of the HHD. Human well being and safety, as well as property could be severely impacted in the event of a levee breach between the Hillsboro and West Palm Beach Canals especially as there is significant urban and residential development very close to the HHD in this area.

Alternatives No. 1, 2 & 3

It is not anticipated that there will be any long-term socioeconomic impacts as a result of implementing Alternatives 1, 2, or 3. Indeed, the positive monetary benefits realized by those firms and persons involved with construction of the project may offset any short-term negative impacts that might be experienced. Potential negative implications of the construction phase could include some short-term impacts on traffic and tourism. However, this will be dependent on the size of the construction workforce residing in the area, and the timing of the construction process (for example, whether during peak tourism season). As the continued operation/maintenance of the preferred Alternative is not expected to require the relocation of a significant number of individuals to the local study area, no socioeconomic impacts are anticipated during the operation/maintenance phase of the project.

Construction and ongoing operation/maintenance of either of the first three Alternatives will generate positive economic impacts for the local region, and the state of Florida as a whole. Directly, the construction and any ongoing operation/maintenance of the project will have a positive impact on employment, labor income, gross domestic product and government revenues. The effect on these variables will be even higher once the impacts resulting from the procurement of goods and services and the spending of additional labor income are taken into account (the indirect and induced effects of the capital spending).

Labor will be required to design/engineer the project and to actually construct it. Construction of the preferred alternative will also require tradespersons with a variety of skills. All of the construction employment will take place in the local region. While it is anticipated that most of the construction employment will be filled by individuals residing in the local study area, some may come from other regions of Florida. The direct project employment only represents the direct employment impact in the local region. It does not include the indirect or induced employment that will be generated

in the local study area, and the State of Florida, as a result of the project. Not all the employment is confined to the construction industry, as the project will create significant employment opportunities in other industries as well. Industries likely to see the largest boost in employment are community/business/personal services, manufacturing and retail trade.

In addition to the impact on employment, the construction and operation/maintenance of the preferred alternative will have a positive impact on Florida's gross domestic product. Another important economic benefit for the local study area and the state of Florida as a whole would be an increase in labor incomes.

In addition to private sector and individual households, county, state and Federal governments will benefit from the construction and operation/maintenance of the proposed alternative through higher revenues. These higher revenues will come about through a number of avenues including personal income taxes, employee and employer contributions to unemployment insurance plans, and other indirect taxes on goods and services that may be purchased.

5.20 Cumulative Effects

Cumulative Effects are the impacts on the existing environment which result from the incremental impact of the action when added to other past, present, and reasonably foreseeable future actions regardless of what agency (Federal or non-Federal) or person undertakes such other actions. Cumulative impacts can result from individually minor, but collectively significant, actions taking place over a period of time.

Foreseeable actions that may affect the Lake Okeechobee region in ways similar to that of the proposed improvements along Reach One, are improvements of similar nature along one or more of the remaining reaches. Considering that each reach delineates a separate watershed, landward impacts to existing environmental conditions should be separate and not cumulative. Waterward impacts of implementing an improvement alternative has no significant negative impacts per individual reach, and should not if additional improvements are conducted. In fact, the cumulative impacts of further improvements stand to be positive rather than negative, increasing the stability and safety of the HHD system, and enhancing water resource capabilities to meet all existing needs.

5.21 Unavoidable Adverse Effects

As discussed under each resource element in Section 5.0, the negative impacts associated with implementing the preferred alternative action would not be significant. Unavoidable adverse effects that would result from implementation of this alternative are expected to be minimal to moderate in severity. A summary of unavoidable negative impacts follows.

<u>Topography, Geology and Soils</u>

No significant adverse impacts to the topography, geology, and soils are likely to occur due to implementation of the preferred alternative. Minimal impacts to soils as a result of excavation and filling is expected to occur.

Water Resources

Minimal to moderate adverse impacts to the hydrology, water supply, water quality and water management are expected to occur as a result of implementing the preferred alternative.

Vegetation and Cover Types

No significant adverse impacts to the vegetation and cover types are likely to occur due to implementation of the preferred alternative. Minimal short-term impacts to vegetation as a result of construction and minor excavation for this alternative is expected.

Wetlands

Some unavoidable permanent and direct adverse impacts to wetlands are likely to occur due to implementation of the preferred alternative. Excavation and fill of low quality wetlands will be required along the landward toe of the Dike in order to accommodate construction of the proposed seepage berm with relief trench and French drain system involved with this Alternative. Negative consequences should be minimal to moderate and may be completely compensated for under the recommended plan by construction of a new seepage collector canal at the extreme back toe on the landward side of the HHD.

Fish and Wildlife

Moderate adverse effects to fish and wildlife are likely to occur due to implementation of the preferred alternative. The foraging habitat for wading birds in the landward toe ditches would be altered through implementation of this alternative. Additionally, existing reptiles, amphibians, and fishes utilizing these ditches would be lost during this activity. This is a moderate loss, but considering the low quality of these ditches as foraging habitat, and the availability of an extensive network of comparable ditches in the area, not significant in extent.

Threatened and Endangered Species

Minor unavoidable adverse impacts to threatened and endangered species are likely to occur due to implementation of the preferred alternative. The foraging habitat for listed wading birds (e.g.

wood storks, tri-colored heron, little blue heron) in the landward remnant wetlands would be excavated and filled through implementation of this alternative requiring these animals to forage elsewhere. The severity of this loss is minimal to moderate considering the low quality of these ditches as foraging habitat, and the availability of an extensive network of comparable ditches in the area.

Noise

Minor localized noise related impacts during construction operations are expected to occur due to implementation of the preferred alternative.

Air Quality

Minor and localized air quality impacts during construction operations is expected to occur due to implementation of the preferred alternative.

Land Use

Some unavoidable adverse impacts to existing land use elements are likely to occur due to implementation of the preferred alternative. Alteration of local hydrology could affect local agriculturists if the availability of irrigation water is affected. Temporary relocation of electrical transmission lines may be required to conduct construction activities associated with this alternative.

If construction of the seepage berm involved with the preferred alternative requires more land area than the current HHD easement provides, then unavoidable impacts to homes, businesses, roads, and railroads may occur.

Aesthetic Resources

Limited, short-term adverse impacts associated with construction activities will be imposed on aesthetic resources within the project area. These impacts may be mitigated by implementation of a well planned aesthetic measures plan which would account for unavoidable tree and native vegetation removal and dust from earth moving equipment among others. These impacts would be expected to be especially serious at or near to parks, natural areas, residential or urban areas.

Recreation Resources

Limited, short-term adverse impacts associated with construction activities will be imposed on recreation resources within the project area. These impacts may be mitigated by implementation of a well planned recreation measures plan which would account for the cost of pavement resurfacing at parks and other areas used for staging and equipment access, tree replacement, and park amenity replacement, rehabilitation, or repair. An inventory of park amenities and utilities prior to construction would facilitate a rapid return to pre-construction state for those areas so impacted.

5.22 Relationship Between Short-Term Uses and Long-Term Productivity

As discussed under each resource element in Section 5.0, the negative impacts (short-term uses) associated with implementing a selected alternative action would not be significant.

The environmental impacts of this effort are insignificant in terms of the human environment, and the costs to the natural environment. In fact, a positive net benefit to human safety and environmental quality both locally and regionally is expected to result from implementation of Alternative Nos. 1, 2, or 3.

5.23 Irreversible or Irretrievable Commitment of Resources

Significant Federal funding will be irretrievably expended during the implementation of Alternative Nos. 1, 2, and 3. In terms of natural resources, the commitment of small, low quality wetland areas landward of the HHD is irreversible, but can be offset through mitigation. Long-term displacement of some wading bird habitat is probably not a reversible action.

6.00 PUBLIC COORDINATION

A Notice of Intent to prepare a Draft Environmental Impact Statement for the HHD MRER was published in the Federal Register Volume 61, No. 190 on September 30, 1996.
A Scoping Letter describing the proposed project and soliciting comments was sent to government agencies, non-governmental agencies, Indian Tribes and the interested public on December 6, 1994. Written responses to the Scoping Letter were submitted to the Corps and served to assist in identifying potential environmental and planning issues throughout the study. A copy of the Scoping Letter and written responses are on file at the U.S. Army Corps of Engineers, Jacksonville District.

This Draft EIS is being circulated for a period not less than forty-five (45) days for public review. Comments received and responses to those comments by the Corps will be incorporated into the Final EIS as an Annex.

7.00 COMPLIANCE WITH ENVIRONMENTAL REQUIREMENTS

The alternative plans were considered in relation to compliance with Federal environmental review and consultation requirements.

7.1 NATIONAL ENVIRONMENTAL POLICY ACT of 1969, as amended

Environmental information on the project has been compiled and the Draft Environmental Impact Statement for the Herbert Hoover Dike Major Rehabilitation Evaluation Report, dated July

1999, has been prepared. A systematic interdisciplinary approach to planning has been utilized; alternatives have been studied, developed and described, and ecological information has been developed and utilized. This Draft Environmental Impact Statement is being circulated for a period of not less than 45 days prior to finalization in accordance with the National Environmental Policy Act.

7.2 FISH AND WILDLIFE COORDINATION ACT of 1958, as amended

In response to the requirements of this Act, the U.S. Army Corps of Engineers has and will continue to maintain continuous coordination with the U.S. Fish and Wildlife Service and the Florida Game and Fresh Water Fish Commission (FGFWFC) during all stages of the planning and implementation of this project. On October 30, 1998 the U.S. Fish and Wildlife Service submitted a draft Fish and Wildlife Coordination Act Report (CAR) for this study. This report had been coordinated with the state agency responsible for the management of game and fish, the FGFWFC. The findings and recommendations in the draft CAR have received concurrence from the FGFWFC in a letter dated November 12, 1998. The draft CAR is presented in Annex A.

The USFWS principal concern as presented in the draft CAR is to avoid environmental impacts to the lake itself, including avoidance of disposal of fill material or armoring of shoreline on the lake side of the HHD. A secondary concern is the potential for erosion of soils and associated nutrients into the lake from construction activities on the lake side face of the levee. This impact is largely alleviated since all construction activities, it is understood, will occur on the landward side of the levee. A third concern is the indirect impact of disturbance, due to construction activities, on significant fish and wildlife resources, including Federal and state listed species eg. the bald eagle, Eastern indigo snake, and the burrowing owl.

The following paragraphs contain the recommendations as presented in the CAR and Corps responses to these recommendations. Wherein the Corps concurs to the below recommendations, concurrence constitutes agreement in principal, and does reflect a legal commitment on the part of the Corps at this time. Environmental commitments that the Corps and USFWS ultimately agree to will be communicated in the final EIS and specified in the Record of Decision.

7.2.1 U.S. Fish and Wildlife Service draft Fish and Wildlife Coordination Act Report Recommendations

Recommendation #1 – Compensatory wetland mitigation will be provided for unavoidable losses of wetlands.

Response – The Corps agrees with the USFWS assessment that some marginal quality wetlands will be impacted by implementation of the recommended plan. These wetlands are essentially composed of the man-made toe ditch on the landward side of the HHD, and in some low areas, intermittently inundated areas immediately surrounding the toe ditch. This toe ditch provides occasional feeding areas of marginal to poor quality for wading birds and marginal habitat for other wildlife, notably aquatic invertebrates and herptiles eg. turtles, snakes, crayfish as well as small fishes. It should be noted that as

a part of the recommended plan, a landward collector canal of slightly smaller size, but relatively broader in slope, will replace the existing toe ditch. This collector canal will function as a collection and diversion structure, channeling surface runoff from the back slope of the HHD, through a series of conveyance canals, away from the project. Furthermore, the recommendation that wetlands mitigation be carried out within the existing inundated quarries is unfortunately, not feasible as proposed. While the recommended plan will require extensive fill material to backfill the quarries, the plan proposes the use of gabions as a back support structure at the extreme landward side of the project. This will, in essence, create a hardened, vertical wall within the quarries, which precludes the development of a shallow water "shelf" or littoral zone, as a part of the project.

Recommendation #2 – Control of exotic vegetation will be carried out in perpetuity in the compensatory wetlands.

Response – This recommendation is not applicable (see response to recommendation #1, above).

Recommendation #3 – Construction will be scheduled to avoid activity within 1500 feet of any active bald eagle nest during the nesting season.

Response – Concur. The Corps agrees to conduct a pre-construction survey to determine actual locations of bald eagle nests within the immediate vicinity of Reach One prior to detailed design and prior to issuance of any construction contracts. Results will be coordinated with the USFWS, Vero Beach office.

Recommendation #4 – Standard protective measures will be carried out to avoid wounding or killing of Eastern indigo snakes.

Response – Concur. Standard protection measures regarding the Eastern indigo snake will be included in the environmental protection plan when the Corps proceeds to the plans and specifications phase for this project.

Recommendation #5 – If burrowing owls are found to be present in the project area, impacts will be minimized by altering construction schedules to avoid the nesting season and/or burrows will be cordoned off to avoid their direct destruction.

Response – Concur. The Corps will conduct a survey for burrowing owls commensurate with that for bald eagle nests, prior to detailed design and prior to issuance of any construction permits. The Corps will consult with the FGFWFC regarding adopting standardized protection measures should any owls be identified within Reach One. Results will be coordinated with the USFWS and FGFWFC.

7.3 ENDANGERED SPECIES ACT of 1973, as amended

Informal consultation with the USFWS was initiated on September 1, 1995, with a letter requesting a listing of Federally listed species thought to occur within the study area.
In a letter dated February 27, 1996 the USFWS indicated six listed species were thought to occur within the study area. A determination of effect was made by the Corps and sent to USFWS in a letter dated November 30, 1998. The Corps determined that the project recommended plan was not likely to adversely effect Federal and state listed species. The USFWS is expected to provide its Opinion on the Corps' determination. It is not expected that implementation of the recommended plan should necessitate Formal Consultation under the Endangered Species Act. This project has been and will continue to be coordinated under the Endangered Species Act; therefore, the project is in partial compliance with the Act. As the project moves into subsequent planning phases continued coordination will be undertaken to ensure that those recommendations mutually agreed upon between the USFWS and the Corps are carried out.

7.4 NATIONAL HISTORIC PRESERVATION ACT of 1966, as amended

The study is in full compliance with this Act. Consultation with the Florida SHPO has been completed. The SHPO has concurred with the Corps determination of no adverse effect by letter dated August 14, 1998. Cultural resources investigations are ongoing to determine effects to historic properties. When completed, results will be coordinated with the State Historic Preservation Officer and the Advisory Council on Historic Preservation.

7.5 CLEAN WATER ACT of 1972, as amended

The study is in partial compliance at this stage. Full compliance will be achieved with issuance of Water Quality Certification under Section 401 from the State of Florida. A Section 404(b)(1) Evaluation is included in this report as Annex C.

7.6 CLEAN AIR ACT of 1972, as amended

Coordination with the U.S. Environmental Protection Agency and the Florida Department of Environmental Protection, Air Quality Division is ongoing. No permits will be required at this stage of planning. Full compliance will be achieved with receipt of comments on the Final Environmental Impact Statement from the U.S. Environmental Protection Agency.

7.7 COASTAL ZONE MANAGEMENT ACT of 1972, as amended

The study is in partial compliance at this time. Full compliance would be achieved with receipt of comments from the Florida State Clearinghouse. A Federal consistency determination in accordance with 15 CFR 930 Subpart C is included in this report as Annex D.

7.8 FARMLAND PROTECTION POLICY ACT of 1981

The study is in full compliance. No farmland will be affected by the proposed project.

7.9 WILD AND SCENIC RIVER ACT of 1968, as amended

The study is in full compliance. No rivers designated under the Act are in the study area.

7.10 ESTUARY PROTECTION ACT of 1968

The study is in full compliance. No estuaries under the Act are in the study area.

7.11 FEDERAL WATER PROJECT RECREATION ACT of 1965, as amended

The project is in full compliance at this stage. The effects of the proposed action on outdoor recreation have been considered and are presented in the Draft EIS. Continued recreation planning will be performed during detailed project engineering and design.

7.12 RESOURCE CONSERVATION AND RECOVERY ACT of 1976

This law has been determined to be not applicable, as there are no items regulated under this act either being disposed of or affected by this project.

7.13 TOXIC SUBSTANCES CONTROL ACT of 1976

This law has been determined to be not applicable, as there are no items regulated under this act either being disposed of or affected by this project.

7.14 MARINE PROTECTION, RESEARCH, AND SANCTUARIES ACT of 1972, as amended

This Act is not applicable. Ocean disposal of dredged material is not proposed as a part of the HHD Major Rehabilitation Evaluation Feasibility Report.

7.15 RIVERS AND HARBORS APPROPRIATION ACT of 1899

The study is in full compliance. The proposed work would not obstruct navigable waters of the United States.

7.16 COASTAL BARRIER RESOURCES ACT

This Act is not applicable. The study area is not in a designated Coastal Barrier Resources Act unit.

7.17 Section 904 of the 1986 WATER RESOURCES DEVELOPMENT ACT

Section 904 of the 1986 Water Resources Development Act requires that the plan formulation and evaluation process considered both quantifiable and unquantifiable benefits and costs of the quality of the total environment, and preservation of cultural and historical values, The study and report are in full compliance.

7.18 Section 307 of the 1990 WATER RESOURCES DEVELOPMENT ACT

Section 307 of the 1990 Water Resources Development Act establishes, as part of the water resources development program, an interim goal of no overall net loss of the Nation's remaining wetlands, and a long-term goal of increasing the quality and quantity of the Nation's wetlands. The recommended plan is in partial compliance. Several acres of remnant, medium to poor quality wetlands are likely to be effected. Mitigation for effected wetland acreage will ensure there is no net loss of wetland function.

7.19 E.O. 11988, FLOODPLAIN MANAGEMENT

The study is in full compliance. While the considered alternatives have no impact on avoidance of development in the flood plain, the recommended plan will directly support a reduction in hazards and risks associated with floods and will minimize the impact of floods on human safety, health and welfare. The recommended plan will have no impact on the restoration and preservation of the natural and beneficial values of the base flood plain.

7.20 E.O. 11990, PROTECTION OF WETLANDS

The study is in partial compliance. The nature of the recommended plan entails permanent filling of the landward toe ditch, a man-made, yet functional wetland of moderate to poor functional value. However, as stated in paragraph 7.2.1, above, any loss of functional wetlands value will be compensated for by the construction of a landward collector canal of similar size to that of the existing toe ditch. This canal is tentatively designed to have 1:4 side slopes, which makes it broader and shallower than the existing toe ditch, possibly allowing for greater relative wading bird foraging area than the existing canal which has steeper 1:2 side slopes.

7.21 E.O. 12114, ENVIRONMENTAL EFFECTS ABROAD OF MAJOR FEDERAL ACTIONS

This executive order is not applicable to this study. The study area does not include lands outside the United States.

7.22 E.O. 12898, ENVIRONMENTAL JUSTICE

Executive Order 12898 requires the Federal government to review the effects of their programs and actions on minorities and low income communities. The Recommended Plan that was formulated for the Herbert Hoover Dike Major Rehabilitation Evaluation Report will help to ensure the safety of those communities within the study area (eg. Belle Glade and Pahokee) as well as residents living within the area expected to be impacted in the event of a project failure. In addition to ensuring the safety and well being of residents and their property, implementation of the recommended plan may have a significant beneficial effect on local communities through job creation, increased sale of construction material and other goods necessary to sustain a large construction force for the duration of the project. The study area is known to contain an important percentage of low income and minority individuals.

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Archaeological Consultants, Inc.. 1991. A Cultural Resources Survey of a Segment of State Road 15 from State Road 729 to State Road 700 at Canal Point in Palm Beach County, Florida. Ms on file, FMSF, Tallahassee, Florida.

Carr, Robert S., W.S. Steele, and Joe Davis. 1995. Archaeological Assessment and Survey of the Rowland Tract, Okeechobee County, Florida. AHC Technical Report #114. Ms on file, U.S. Army Corps of Engineers, Jacksonville, Florida.

Cowley, Marianne. 1998. Florida Wildlife, Species Accounts. Internet Resource: <http://www.nsis.org/wildlife.html>

David Miller & Associates. March 1998. Final Report; Economic Impact Evaluation, Lake Okeechobee Regulation Schedule Study.

FGFWFC. 1997. Everglades Region Fishing Guide. Florida Game and Fresh Water Fish Commission. Tallahassee, Florida.

FNAI. March, 1998. Florida Natural Areas Inventory: Element Occurrence Records. Tallahassee, Florida.

FP&L. 1989. The West Indian Manatee In Florida. Written by Victoria Brook Van Meter for Florida Power & Light Company. Miami, Florida.

Fernald, Edward A., and Donald J. Patton. 1984. Water Resources Atlas of Florida. Florida State University.

Florida Department of Environmental Protection. March 1998. Air Resources Management System, Air Index Reports for Martin and Palm Beach Counties, Florida.

Havens, K.E. 1998. Minimum Water Level Criteria for Lake Okeechobee. Okeechobee Research Division, South Florida Water Management District. West Palm Beach, Florida. Internet Resource: <http://www.ces.fau.edu/library/flms/20.html>

Henry, James A., Kenneth M. Portier, and Jan Coyne. 1994. The Climate and Weather of Florida. Pineapple Press, Inc. Sarasota, Florida. 279 pp.

Klein, Howard, M. C. Schroeder, and W. F. Lichtler. 1964. Report of Investigations No. 37, Geology and Ground-Water Resources of Glade and Hendry Counties, Florida. U.S. Geological Survey. Tallahassee, Florida.

Lichtler, William F. 1960. Report of Investigations No. 23, Geology and Ground-Water Resources of Martin County, Florida. Water Resources Division, U.S. Geological Survey. Tallahassee, Florida.

Mazzotti, Frank J. December, 1990. University of Florida, Florida Cooperative Extension Service. Publication SS-WIS-12: Wood Storks (Mycteria americana). Gainesville, Florida.

Milano, Karen Webster. 1995. City of Belle Glade Historic Site Survey. Prepared for the City of Belle Glade. Ms on file, FMSF, Tallahassee, Florida.

Moler, Paul E., and Joan E. Berish. 1995. Impact of Commercial Exploitation on Softshell Turtle Populations. Florida Game and Fresh Water Fish Commission. Tallahassee, Florida.

Piper Archaeology/Janus Research. 1992. Cultural Resource Assessment Survey of the Proposed Levee-Midway Transmission Line, St. Lucie, Martin, Palm Beach, Broward, and Dade Counties, Florida. Ms on file, FMSF, Tallahassee Florida.

Purdum, Elizabeth. Et al. 1994. Florida County Atlas and Municipal Fact Book. Florida State University.

Schaefer, Joe, and Janet Junkin. December, 1990. University of Florida, Florida Cooperative Extension Service. Publication SS-WIS-24: The Eastern Indigo Snake: A Threatened Species. Gainesville, Florida.

Schroeder, M. C., D. L. Milliken and S. K. Love. 1954. Report of Investigations No. 13, Water Resources of Palm Beach County, Florida. Water Resources Division, U.S. Geological Survey. Tallahassee, Florida.

SFWMD. January, 1998. Technical Memorandum #356, Surface Water Quality Monitoring Network. South Florida Water Management District.

Southeast Regional Climate Center, South Carolina Department of Natural Resources Land, Water & Conservation Division. Internet Resources. Climatological Normals 1961-90.

U.S. Army Corps of Engineers. 1998. MRER, Modeling Results of Breach in HHD. Jacksonville, Florida.

U.S. Census Bureau. November, 1997. Estimates of the Populations of Places: Annual Time Series, July 1, 1994 To July 1, 1996. Internet Resource: <http//www.census.gov/agfs/qfr/view/qfr931mg.txt>

U.S. Department of the Interior. July 1997. Land Acquisition Programmatic Environmental Assessment, Everglades Agricultural Area. Internet Resource: <http://www.nps.gov/planning/ever/eaa/wildlife.htm>

USFWS. February, 1996. U.S. Fish and Wildlife Service, Official Correspondence.

USFWS. February, 1997. U.S. Fish and Wildlife Service Division of Endangered Species. Okeechobee Gourd, Species Account. Internet Resource: <http://www.fws.gov/r9endspp/i/qad.html>

USFWS. July, 1995. Bald Eagle, Wildlife Species Biologue. Internet Resource: <http:/www.fws.gov/r9extaff/biologues/bio_eagl.html>

USFWS. May, 1996. U.S. Fish and Wildlife Service Division of Endangered Species. Everglade Snail Kite, Species Account. Internet Resource: <http://www.fws.gov/r9endspp/i/b/sab0v.html>

USFWS. March, 1998. Multi Species Recovery Plan for the Threatened and Endangered Species of South Florida, Volume I of II, Okeechobee Gourd. U. S. Fish and Wildlife Service, Atlanta, Georgia.

United States Department of Agriculture, Soil Conservation Service. December 1978. Soil Survey of Palm Beach County Area, Florida.

University of Florida. 1998. Florida Museum of Natural History, Species Account. Internet Resource: <http//www.flmnh.ufl.edu/natsci/herpetology/brittonerocs/csp_amis.htm>

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A
Acer rubrum, 19
Aesthetic Resources, 37, 42, 65, 69
Affected Environment, 8
AFFECTED ENVIRONMENT, 9
Agricultural, 34
agricultural water supply, 48, 49
Agricultural Water Supply, 48
agriculture, 3, 4, 6, 19, 20, 22, 34, 35, 37, 41, 43, 59, 60
Agriculture, 20, 31, 32, 34, 37, 39, 59, 74
air quality, 33, 34, 58
Air Quality, 33, 41, 58, 69
airboating, 38
Alligator mississippiensis, 26, 27, 54
Alternative No. 1, vii, 3, 5, 6, 43, 52, 53, 54, 55, 56, 59, 61, 62, 63, 64, 67, 68, 69
Alternative No. 2, ii, vii, 3, 4, 5, 6, 44, 52, 53, 60, 61, 62, 63, 64, 65
Alternative No. 3, vii, 3, 4, 5, 7, 44, 52, 54, 60, 61, 62, 63, 64, 65
Alternatives, 3, 43, 51, 54, 55, 56, 57, 58, 65, 67
ALTERNATIVES CONSIDERED, 5
american alligator, 25, 27, 38, 54
American alligator, 26, 27, 54
american crocodile, 27
american widgeon, 38
Anas acuta, 38
Anas americana, 38
Anas crecca, 38
Anas fulvigula, 38
Annona glabra, 31
apple snail, 56
aquifer, 6, 11, 12
Ardea herodias, 24
Arundinaria sp., 21
australian pine, 20, 21
Aythya affinis, 38
Aythya collaris, 38

B
bald eagle, 24, 25, 26, 28, 55
Bald Eagle, 28, 55, 73
bamboo, 21
Belle Glade, 1, 4, 9, 10, 20, 33, 35, 36, 60
black crappie, 23
bluegill, 23
boating, 38
boils and pipings, ii, 1, 5
brazilian pepper, 20, 21
breach, ii, iii, 5, 53, 59, 60, 62, 64
Brighton Indian Seminole Reservation, 36
brown pelican, 26

C
cabbage palm, 19, 28
Caloosahatchee River, 13, 14, 15, 45
camping, 38
Canal, 9, 10, 13, 14, 15, 35, 37, 47, 64
Canal Point, 10, 35
canals, 1, 7, 13, 14, 15, 16, 17, 30, 45, 46, 48, 49, 60
carolina willow, 19, 21, 22
Casmerodius albus, 24
Casuarina sp., 20
cattails, 21, 22
cattle ranching, 34
Central and Southern Florida Flood Control Project, 13, 17
citrus orchards, 34
Cladium jamaicense, 19
Clewiston, 10, 25, 29, 33, 35, 56
Climate, 10, 40, 72, 73
commercial fishing, 37, 38
Communication, 37, 64
communication tower, 37, 64
COMPLIANCE WITH ENVIRONMENTAL REQUIREMENTS, 70
construction employment, 66
County Road 715, 36
County Road 78, 36

Crocodylus acutus, 27
Cucurbita okeechobeensis, 2, 32
Cultural Resources, 37, 42, 65, 69
culvert, 7, 8, 37, 60
Cumulative Effects, 67
Curbita okeechobeensis okeechobeensis, 31, 57
cutoff wall, ii, 3, 4, 6, 7, 44, 52, 53, 60
cypress, 19, 29
cypress swamps, 19

D
drainage, ii, 3, 6, 7, 8, 12, 19, 21, 22, 30, 59, 60, 62
Drymarchon corais couperi, 26, 27, 54

E
E. tricolor, 24
eastern indigo snake, 27
Eastern Indigo Snake, 27, 54, 73
Egretta caerulea, 24, 26
Eichhornia crassipes, 21
EIS, ix, 70
elderberry, 31
electric transmission, 37, 63
Eleocharis spp, 29
employment, 66
ENVIRONMENTAL CONSEQUENCES, 39, 40, 41, 42
Environmental Impact Statement, ii, iii, 70
Environmental Resource Permit, 2
Environmentally Preferable Alternative, 4, 8
evaporation, 13, 14, 44, 45
Evaporation, 44
everglade snail kite, 18, 24, 25, 26, 29, 30, 56
Everglade Snail Kite, 29, 56, 73
Everglades, 2, 11, 12, 13, 19, 22, 29

F
failure, 4, 43, 51, 53, 54, 55, 56, 57, 59, 60, 61, 63
Farmland, 34
fauna, 3
Fauna, 25, 54
faunal species, 25, 31
FGFWFC, ix, 25, 27, 28, 29, 30
fish and wildlife, 25, 53, 54, 68
Fish and wildlife, 23
Fish and Wildlife, viii, 23, 26, 32, 53, 68, 73
Fisheating Creek, 21
fishing, 30, 38
Flood Control Act of 1948, 1, 5
flood protection, 1, 4, 5
flora, 3
Flora, 31, 32, 57
floral species, 31
Florida Department of Environmental Protection, 2, 72
Florida Department of Transportation, 2
florida duck, 38
Florida East Coast Railroad, 21, 22, 34, 36, 62
Florida East Coast Railway, 36
Florida National Scenic Trail, 2, 38
Florida Natural Areas Inventory, viii, 25, 72
florida soft shell turtle, 38
Floridan Aquifer, 12
FNAI, 25, 26, 27, 31, 32, 72
foraging, 3, 4, 21, 22, 27, 29, 52, 54, 55, 56, 68
freshwater marshes, 19, 20, 21, 22, 29

G
gated culverts, 1, 7, 17
geological, 11, 12
Geology, 11, 40, 43, 67, 72
Glades, 4, 9, 34, 35, 36
great blue heron, 24
great egret, 24
green-winged teal, 38
gross domestic product, 66
groundwater, iii, 4, 6, 7

H
Haliaeetus leucocephalus, 24, 26, 28, 55
hardwood swamps, 19
Hazardous, Toxic and Radioactive Wastes, 34, 58, 69

Hendry, 4, 9, 34, 35, 72
hiking, 38
hunting, 27, 28, 30, 38
Hwy. 98/441, 21, 22
hydrilla, 21, 51
Hydrilla verticillata, 21
hydrologic, 29
hydrology, 3, 22, 51, 57, 69
Hydrology, 13, 40, 44

I
improved pasture, 19
incorporated communities, 35
index, 75
Indian Reservation, 36
indigo snake, 25, 26, 27, 28, 54
instability, 1, 43, 53
Irreversible or Irretrievable Commitment of Resources, 70
irrigation water, 3, 37, 59, 60, 69

K
keystone, 27
Kissimmee River, 21, 25, 29, 56
Kreamer, 20, 22

L
labor, 66
Lake Okeechobee Scenic Trail, 2, 38
land resources, 3
land use, 34, 35, 59, 69
Land Use, vii, 34, 41, 59, 69
large mouth bass, 23
Lepomis macrochirus, 23
lesser scaup, 38
limestone quarries, 20, 21, 37, 64, 65
Limestone Quarries, 21, 37, 64
Liquidambar styraciflua, 19
LIST OF PREPARERS, 70, 71
List of recipients, 75
little blue heron, 24, 26, 27, 68
littoral zone vegetation, 21
Location, vii, 9, 40, 43
Location and Climate, 9, 43

M
Magnolia virginiana, 19
manatee mortality, 30
Martin, vi, 4, 9, 25, 31, 33, 34, 72
melaleuca, 20, 21
Melaleuca quinquenervia, 2, 20
Miami, 72
Micropterus salmoides, 23
minimum water level, 18
Mitigation Measures, 9
Morone chrysops, 23
MRER, ii, iii, ix, 4, 5
municipalities, 35
Mycteria americana, 24, 26, 29, 55, 72

N
National Environmental Policy Act, iii, 70
natural areas, 11, 19, 23
natural landscape, 12
navigation, 4, 37, 39
NEPA, iii, ix, 70
nesting habitats, 21
No Action Alternative, 3, 5, 6, 43, 51, 53, 54, 57, 59, 60, 61, 63, 64, 65
noise, 57
Noise, 33, 41, 57, 68
northern pintail, 38
Nuisance vegetation, 21
Nuisance Vegetation, 21

O
Okeechobee, ii, vi, 1, 2, 4, 5, 9, 10, 11, 12, 13, 19, 21, 22, 23, 25, 29, 30, 31, 34, 35, 36, 37, 38, 39, 43, 53, 54, 56, 57, 59, 61, 67, 72, 73
okeechobee gourd, 31, 32, 57
Okeechobee Gourd, 2, 31, 57, 73
open water, 19, 20, 29, 56
operation/maintenance, 66
ornamental nurseries, 34, 59
osprey, 24

outfall, 13, 14, 15, 30, 45
Outfall, 45
outfall canals, 15

P
Palm Beach, vi, 4, 9, 22, 25, 31, 33, 34, 35, 37, 64, 72, 73, 74
Pandonion haliaetus, 24
Pelecanus occidentalis, 26
picnicking, 38
pine flatwoods, 19
Pinus elliottii, 19
Pistia stratiotes, 21
Pomoxis nigromaculatus, 23
pond apple, 31
Preferred Alternative, 3, 5, 7
produce packaging, 35
PUBLIC COORDINATION, 70
Public Water Supply, 47

R
Railroad corridors, 36
rainfall, 10, 37
recharge, 14, 46, 47
Recharge, 46
recreation, 4, 37, 38
Recreation, 38
Recreational Resources, 37, 42, 65
red maple, 19
REFERENCES CITED, 72
Relationship Between Short-Term Uses and Long-Term Productivity, 69
relief, iii, 7, 12
relief trench, iii, 7
ring-necked duck, 38
River and Harbor Act, 1
riverine, 19
Rosthrhamus sociabilis plumbeus, 29, 56
Rostrhamus sociabilis plumbeus, 24, 26
row crops, 19, 20, 34, 39

S
Sabal palmetto, 19
Salix caroliniana, 19
Sambucus canadensis, 31
saw palmetto, 19
sawgrass, 19, 20, 29
Schinus terebinthifolius, 20
sea cow, 30
seepage, iii, 5, 6, 7, 44, 52, 54, 60, 62, 63, 65
Serenoa repens, 19
SFWMD, ix
shipping plants, 35
sightseeing, 38
slash pine, 19
socioeconomic impacts, 65
socioeconomic resources, 37
socioeconomics, 3
Socioeconomics, 37, 42, 65, 71
soils, 5, 12, 13, 43, 67
Soils, 11, 12, 13, 40, 43, 67
Soils of Sloughs and Freshwater Marshes, 12, 13
Soils of the Everglades, 12, 13
Soils of the Flatwoods, 12
South Bay, 35
South Central Florida Railroad, 36
spike rushes, 29
sport fishing, 38
St. John's River, 29, 31
St. Lucie, 1, 4, 9
St. Lucie Canal, 1, 4, 9
stability berm, ii, 3, 6, 44, 63
State Highway 27, 36
State Highway 98/441, 36
stationary air sources, 33
Study Authority, 5
sugar processing plants, 33
sugarcane, 13, 19, 20, 34, 39, 51, 59
sugarcane plantations, 34
Sugarcane refineries, 35
sunshine bass, 23
sweet gum, 19
sweetbay, 19
swimming, 38

T
Taxodium spp., 19
Taylor Creek, 21, 35
THE PURPOSE AND NEED FOR THE ACTION, 4
Threatened and Endangered Species, 25, 54, 68
Topography, 11, 40, 43, 67
Torry Islands, 20, 22
tourism, 37, 38, 65
traffic, 62, 65
Transmission Lines, 37, 41, 63
transportation, 21, 22, 34, 36, 59, 61, 62, 63
Transportation, 36, 41, 61
transportation corridors, 36, 61, 62, 63
tree islands, 19, 20, 29
Trichechus manatus, 26, 30, 56
tri-colored heron, 24, 27, 68
Typha sp., 21

U
U.S. Sugar Corporation, 33
Unavoidable Adverse Effects, 67
Urban Land, 35, 60
urban lands, 19, 60, 61
USACE, ii, iii, ix, 1, 3, 5, 8, 34, 37, 57, 58, 65, 69, 70, 75
USFWS, 25, 27, 28, 29, 30, 31, 73

V
vegetable production, 34, 59
vegetation and cover types, 19, 68
Vegetation and Cover Types, 19, 51, 68

W
wading birds, 3, 4, 22, 24, 25, 27, 68
Water Conservation Areas, 22
water hyacinth, 21, 22, 30, 51
water lettuce, 21, 22, 51
water management, 3
Water Management, 17, 40, 50
Water Quality, 16, 40, 49
Water Supply, 14, 40, 47
waterfowl, 38
WCA, ix, 29
west indian manatee, 25, 30, 56
West Indian Manatee, 30, 56, 72
wetland, 2, 3, 4, 21, 22, 29, 51, 52, 70
wetlands, 2, 3, 20, 21, 22, 29, 51, 52, 53, 68
Wetlands, 21, 40, 51, 68
wildlife, 21, 23, 25, 52, 53, 54
wood stork, 25, 26, 29, 55
Wood Stork, 29, 55
woodstork, 24, 25

12.00 ANNEXES

A - Draft Fish and Wildlife Coordination Act Report

B - Florida Natural Areas Inventory Report

C - Section 404(b)(1) Evaluation

D - Coastal Zone Management Consistency Evaluation

1. This appendix provides the M-CACES cost estimates that were computed for three major rehabilitation alternatives, as described below:

(a) Alternative A would address seepage and stability problems by reducing the magnitude of the hydraulic conditions to which the dike is subjected. This reduction in hydraulic loads would be accomplished, not by reducing the lake elevation (or headwater), but by increasing the water level in drainage ditches along the landward side of the dike (or tailwater). This would reduce the hydraulic gradient (or head) across the dike. This alternative would involve toe ditch clearing and regrading, and installation of gated culverts and pumps to control the toe ditch water levels. Additionally, a 3- to 4-foot thick peat layer would be excavated from the landside toe of the levee. In its place, a 25-foot wide, 5-foot thick stability berm would be constructed. The stability berm would reduce uplift forces at the toe, provide better resistance to seepage flows, and allow better inspection access at the dike toe. This alternative was investigated in significant detail during the economic analysis; however, in Reach 1, engineering analyses indicated that the estimated level of protection provided by this alternative was not adequate. In addition, this alternative would increase the likelihood of flooding farmlands adjacent to the dike. Local agricultural interests presently control water levels in most of the Reach 1 toe ditches. Their objective is to maintain toe ditch water levels at the lowest possible levels, in order to prevent flooding of their farmlands. The estimated cost of this alternative is $1,049,300 per mile.

(b) Alternative B is the landside seepage/drainage berm, which was selected as the recommended plan of improvement in Reach 1. It consists of peat excavation from the levee toe and construction of a drainage berm. The drainage berm would consist of a 5-foot thick layer of filter sand and gravel. The berm would contain a 48-inch diameter perforated culvert wrapped in geotextile fabric. A 2-foot wide, 25-foot deep, gravel-filled relief trench would be constructed under the drainage berm. The drainage berm would prevent the piping of sands and silty material from the embankment or foundation. The relief trench would intercept underseepage flows, control uplift pressures, and prevent heaving at the levee toe. The entire drainage system would reduce artesian pressures in the

foundation and would collect and convey seepage flows to a controlled outlet. Gated culverts would be installed at various locations to drain the 48-inch diameter culvert. Based on economic, environmental, and engineering considerations, this alternative was selected for implementation along the 22 miles of reach 1. Estimated cost of this alternative is approximately $3,024,800 per mile.

(c) Alternative C would consist of an impervious cutoff wall, which would act as a physical barrier that prevents seepage by blocking the seepage path beneath and/or through the levee. It is typically the most effective method of seepage control as it reduces both uplift pressures and through-seepage. The wall would be constructed on the lakeside slope of the dike. It would consist of a 3-foot wide, 60-foot deep, excavated trench filled with a soil-cement mixture. The top of the wall would be located at an approximate elevation of 25 feet, NGVD. The top of the wall would be located at an approximate elevation of 25 feet, NGVD. A minor stability berm will be constructed along the exterior toe of the embankment. The berm will consist of excavating the 3- to 4-foot thick layer of peat from the landside toe of the embankment. This peat layer exists between the dike toe and the invert of the drainage ditch, a distance of about 25 feet. After peat excavation, a 25-foot wide, 5-foot thick stability berm will be constructed. The stability berm will also provide improved inspection access to the dike toe and adjacent ditches. It should be noted that this alternative could affect the upper aquifer and have detrimental impacts on groundwater flows immediately adjacent to the dike. The estimated cost of this alternative is $13,847,200 per mile.

The following pages provide M-CACES cost estimates for the three alternatives described above. All estimates include a standard contingency rate of 15 percent.

Wed 17 Feb 1999 U.S. Army Corps of Engineers TIME 09:08:23
Eff. Date 02/16/99 PROJECT HHD801: HERBERT HOOVER DIKE TITLE PAGE 1

HERBERT HOOVER DIKE

MAJOR REHABILITATION EVALUATION REPORT

Designed By: JACKSONVILLE DISTRICT OFFICE
Estimated By: EpC

Prepared By: E.P.CAMPA

Preparation Date: 02/16/99
Effective Date of Pricing: 02/16/99

Sales Tax: 6.00%

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LABOR ID: L75ST5 EQUIP ID: RG0395 Currency in DOLLARS CREW ID: NAT95B UPB ID: NAT95A

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Eff. Date  02/16/99  PROJECT HHD801:   HERBERT HOOVER DIKE
PROJECT NOTES                                                    TITLE PAGE 2

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Contingencies used were the standard 15% based on the level of design.

E&D and S&A were obtained from project management.

LABOR ID: L75ST5 EQUIP ID: RG0395 Currency in DOLLARS CREW ID: NAT95B UPB ID: NAT95A
Wed 17 Feb 1999 U.S. Army Corps of Engineers TIME 09:08:23
Eff. Date 02/16/99 PROJECT HHD801: HERBERT HOOVER DIKE
SUMMARY PAGE 1

** PROJECT OWNER SUMMARY - Contract **

		QUANTITY	UOM	CONTRACT	CONTING	TOTAL COST	UNIT COST
1	HERBERT HOOVER DIKE - Alt A			20,493,371	2,957,769	23,451,140	
2	HERBERT HOOVER DIKE - Alt B			59,144,265	8,460,715	67,604,980	
3	HERBERT HOOVER DIKE - Alt C			270,875,219	38,610,558	309,485,777	

LABOR ID:L75ST5 EQUIP ID:RG0395 Currency in DOLLARS CREW ID: NAT95B UPB ID: NAT95A

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Wed 17 Feb 1999          U.S. Army Corps of Engineers          TIME 09:08:23
Eff. Date  02/16/99  PROJECT HHD801:   HERBERT HOOVER DIKE
                                                              SUMMARY PAGE 2
                     ** PROJECT OWNER SUMMARY - Userdefi **

-------------------------------------------------------------------------------
                      QUANTITY UOM CONTRACT    CONTING   TOTAL COST UNIT COST
-------------------------------------------------------------------------------

1  HERBERT HOOVER DIKE - Alt A

1- A  CONSTRUCTION COST

1- A/11  Levees and Floodwalls

1- A/11.01  Levees

1- A/11.01.01  Mob, Demob & Preparatory Work  399,795   59,969   459,765
1- A/11.01.02  Drainage


1- A/11.01.03  Care & Diversion of Water

1- A/11.01.03/ 1 EXCAVATION                  893,223  133,984 1,027,207
1- A/11.01.03/ 2 culvert pipe                198,030   29,704   227,734
1- A/11.01.03/ 3 regrade ditches437806.00 CY 645,851   96,878   742,728 1.70
1- A/11.01.03/ 4 pumps                       105,106   15,766   120,872
1- A/11.01.03/ 5 construct berm
               w/common borrow655600.00 CY6,711,109 1,006,666 7,717,775 11.77
1- A/11.01.03/ 6  quarry fill             6,135,348   920,302 7,055,650
                                         --------- --------- ----------
   TOTAL Care & Diversion of Water       14,688,666 2,203,300 16,891,966

1- A/11.01.99  Associated General Items
                                         ---------- --------- ----------
   TOTAL Levees                           15,088,462 2,263,269 17,351,731
                                          ---------- --------- ----------
   TOTAL Levees and Floodwalls            15,088,462 2,263,269 17,351,731
                                          ---------- --------- ----------
   TOTAL CONSTRUCTION COST                15,088,462 2,263,269 17,351,731


1- B  NON-CONSTRUCTION COST

1- B/01  lands and damages                 2,780,000   694,500  3,474,500
                                          ---------- --------- ----------
   TOTAL lands and damages                 2,780,000   694,500  3,474,500

1- B/30   Planning,engineering and design

1- B/30.01Planning,engineering and design  1,920,000         0  1,920,000
1- B/30.02  Environmental Monitoring            1,500         0      1,500
                                          ---------- --------- ----------
   TOTAL Planning,engineering and design   1,921,500         0  1,921,500

1- B/31  Construction management              703,409         0    703,409
                                          ---------- --------- ----------
```

TOTAL NON-CONSTRUCTION COST	5,404,90	694,500	6,099,409
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LABOR ID:L75ST5 EQUIP ID:RG0395 Currency in DOLLARS CREW ID:NAT95B UPB ID: NAT95A

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Wed 17 Feb 1999          U.S. Army Corps of Engineers          TIME 09:08:23
Eff. Date  02/16/99  PROJECT HHD801:   HERBERT HOOVER DIKE
                                                              SUMMARY PAGE 3
                     ** PROJECT OWNER SUMMARY - Userdefi **

-------------------------------------------------------------------------------
                      QUANTITY UOM CONTRACT    CONTING   TOTAL COST UNIT COST
-------------------------------------------------------------------------------

   TOTAL HERBERT HOOVER DIKE - Alt A    20,493,371 2,957,769 23,451,140

2  HERBERT HOOVER DIKE - Alt B

2- A  CONSTRUCTION COST

2- A/11  Levees and Floodwalls

2- A/11.01  Levees

2- A/11.01.01  Mob,Demob&Preparatory Work 2,013,966  302,095 2,316,061
2- A/11.01.02  Drainage


2- A/11.01.03  Care & Diversion of Water

2- A/11.01.03/ 2  EXCAVATION
       peat/haul away  262240.00 CYS    3,394,240   509,136  3,903,376  14.88
2- A/11.01.03/ 3  EXCAVATION
       for 48"dia pipe 209792.00 CYS      376,764    56,515    433,279   2.07
3- A/11.01.03/ 4  EXCAVATION
       for drainage swale 87413.00 CYS    442,920    66,438    509,358   5.83
2- A/11.01.03/ 5  48"dia perforated
                  pipe    118483.00 LF 10,214,569 1,532,185 11,746,754  99.14
2- A/11.01.03/ 6  filter
                  sand    218533.00 CYS 2,260,749   339,112  2,599,861  11.90
2- A/11.01.03/ 7  filter
                  stone   655600.00 CYS19,974,379 2,996,157 22,970,536  35.04
2- A/11.01.03/ 8  EXCAVATE RELIEF
                  TRENCH  218533.00 CYS   367,232    55,085    422,317   1.93
2- A/11.01.03/ 9  gravel fill for relief
                  trench  218533.00 CYS 5,808,717   871,307  6,680,024  30.57
2- A/11.01.03/11  quarry fill
                  W/GABIONS 17500.00 SF   728,207   109,231    837,438  47.85
2- A/11.01.03/12  GATED CULVERT 16 EA[stop
                  log we]16.00 EA          59,932     8,990     68,921 4307.58
2- A/11.01.03/13  DROP INLETS
                        290.00 EA         127,911    19,187    147,098 507.24
2- A/11.01.03/14  pool liner
                  w/gabions 341.00 SY       9,733     1,460     11,193  32.82
2- A/11.01.03/15  HAUL IN COMMON BORROW
                  2 MIL RT 437066.00 CY 4,456,125   668,419  5,124,543  11.72
2- A/11.01.03/16  GEO TECH FABRIC
                  - quarry  14333.00 SY    32,860     4,929     37,789   2.64
2- A/11.01.03/17  GEO TECH FABRIC pool
                  liner       448.00 SY     1,088       163      1,251   2.79
2- A/11.01.03/18  GEO TECH FABRIC 22
                  mile cap 655600.00 SY 1,505,374   225,806  1,731,180   2.64
```

TOTAL Care & Diversion of Water	49,760,799	7,464,120	57,224,919

LABOR ID:L75ST5 EQUIP ID:RG0395 Currency in DOLLARS CREW ID:NAT95B UPB ID: NAT95A

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Wed 17 Feb 1999        U.S. Army Corps of Engineers            TIME 09:08:23
Eff. Date  02/16/99  PROJECT HHD801:   HERBERT HOOVER DIKE
                                                              SUMMARY PAGE    4
                     ** PROJECT OWNER SUMMARY - Userdefi **

-------------------------------------------------------------------------------
                      QUANTITY UOM CONTRACT    CONTING   TOTAL COST UNIT COST
-------------------------------------------------------------------------------

2- A/11.01.99 Associated General Items ---------- ----------- -----------
  TOTAL Levee                          51,774,765   7,766,215  59,540,980
                                       ---------- ----------- -----------
  TOTAL Levees and Floodwalls          51,774,765   7,766,215  59,540,980
                                       ---------- ----------- -----------
  TOTAL CONSTRUCTION COST              51,774,765   7,766,215  59,540,980


2- B  NON-CONSTRUCTION COST

2- B/01  Lands and Damages              2,778,000     694,500   3,472,500
                                      ----------- ----------- -----------
   TOTAL Lands and Damages              2,778,000     694,500   3,472,500

2- B/30  Planning,engineering and design

2- B/30.01 Planning,engineering
           and design                   1,920,000           0   1,920,000
2- B/30.02 Environmental Monitoring         1,500           0       1,500
                                      ----------- ----------- -----------
   TOTAL Planning,engineering
         and design                     1,921,500           0   1,921,500

2- B/31  Construction Management        2,670,000           0   2,670,000
                                      ----------- ----------- -----------
   TOTAL NON-CONSTRUCTION COST          7,369,500     694,500   8,064,000
                                      ----------- ----------- -----------
   TOTAL HERBERT HOOVER DIKE - Alt B   59,144,265   8,460,715  67,604,980


3  HERBERT HOOVER DIKE - Alt C

3- A  CONSTRUCTION COST

3- A/11  Levees and Floodwalls

3- A/11.01  Levees

3- A/11.01.01 Mob, Demob
              & Preparatory Work        7,645,374   1,146,806   8,792,180
3- A/11.01.02  Drainage


3- A/11.01.03  Care & Diversion of Water

4- A/11.01.03/ 1  EXCAVATE for cut
                  off wall          237,319,550 35,597,932 272,917,482
3- A/11.01.03/ 2  EXCAVATE at
```

```
                  toe 263295.00 CY     1,097,687    164,653   1,262,340  4.79
3- A/11.01.03/ 3  common borrow fill
                  at toe 655600.00 CY  6,711,109  1,006,666   7,717,775 11.77
                                     ----------- ---------- -----------
   TOTAL Care & Diversion of Water   245,128,345 36,769,252 281,897,597
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LABOR ID:L75ST5 EQUIP ID:RG0395 Currency in DOLLARS CREW ID:NAT95B UPB ID: NAT95A

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Wed 17 Feb 1999        U.S. Army Corps of Engineers            TIME 09:08:23
Eff. Date  02/16/99  PROJECT HHD801:   HERBERT HOOVER DIKE
                                                              SUMMARY PAGE    5
                     ** PROJECT OWNER SUMMARY - Userdefi **

-------------------------------------------------------------------------------
                      QUANTITY UOM CONTRACT    CONTING   TOTAL COST UNIT COST
-------------------------------------------------------------------------------
3- A/11.01.99  Associated General Items
                                   ----------- ----------- -----------
   TOTAL Levees                    252,773,719  37,916,058 290,689,777
                                   ----------- ----------- -----------
   TOTAL Levees and Floodwalls     252,773,719  37,916,058 290,689,777
                                   ----------- ----------- -----------
   TOTAL CONSTRUCTION COST         252,773,719  37,916,058 290,689,777


3- B  NON-CONSTRUCTION COST

3- B/01  lands and damages           2,780,000     694,500   3,474,500
                                   ----------- ----------- -----------
   TOTAL lands and damages           2,780,000     694,500   3,474,500


3- B/30  Planning,engineering and design

3- B/30.01  Planning,engineering
            and design               1,920,000           0   1,920,000
3- B/30.02  Environmental Monitoring      1,500           0       1,500
                                   ---------- ----------- -----------
   TOTAL Planning,engineering
         and design                  1,921,500           0   1,921,500

3- B/31  construction management    13,400,000           0  13,400,000
                                   ---------- ----------- -----------
   TOTAL NON-CONSTRUCTION COST      18,101,500     694,500  18,796,000
                                   ---------- ----------- -----------
   TOTAL HERBERT HOOVER DIKE - Alt C 270,875,219  38,610,558 309,485,777
```

LABOR ID:L75ST5 EQUIP ID:RG0395 Currency in DOLLARS CREW ID: NAT95B UPB ID: NAT95A

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Wed 17 Feb 1999       U.S. Army Corps of Engineers              TIME 09:08:23
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ERROR REPORT                                                  ERROR PAGE    1

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LABOR ID: L75ST5 EQUIP ID: RG0395 Currency in DOLLARS CREW ID: NAT95B UPB ID: NAT95A

```
Wed 17 Feb 1999      U.S. Army Corps of Engineers              TIME 09:08:23
Eff. Date  02/16/99  PROJECT HHD801:   HERBERT HOOVER DIKE
TABLE OF CONTENTS                                          CONTENTS PAGE    1


-------------------------------------------------------------------------------

    SUMMARY REPORTS                                                 SUMMARY PAGE

    PROJECT OWNER SUMMARY -
Contract.........................................1
    PROJECT OWNER SUMMARY -
UserDefi........................................2


No Detailed Estimate...



No Backup Reports...


                     * * *   END TABLE OF CONTENTS   * * *
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PROJECT COOPERATION AGREEMENT (PCA)

The latest guidance from HQUASCE, 6th Endorsement of the Special Report dated 09 August 1994, states as follows: “After approval of the evaluation report by HQUSACE, required major rehabilitation work should be budgeted under the CG appropriation for construction as a fully federally funded project.” Therefore, a PCA is not applicable to the subject project. CESAJ will continue to work cooperatively with the local sponsor to inform them of the project status. Operation and maintenance of the embankment system will follow the guidance contained in the existing agreement between the U.S. Army Corps of Engineers and the local sponsor, South Florida Water Management District.

APPENDIX F

PROJECT MANAGEMENT PLAN

CENTRAL AND SOUTHERN FLORIDA PROJECT

HERBERT HOOVER DIKE

MAJOR REHABILITATION EVALUATION REPORTS

PROJECT MANAGEMENT PLAN

DECEMBER 1998

Jacksonville District, Corps of Engineers

EXECUTIVE SUMMARY

This Project Management Plan (PMP) provides a detailed plan for management and execution of the Herbert Hoover Dike Major Rehabilitation Project from approval of the evaluation report for Reach 1 through completion. The PMP also provides a recommendation to prepare a Major Rehabilitation Report for Reaches 2 and 3. The PMP focuses on the engineering, design, and management activities required, and it also summarizes the baseline cost estimate.

The rehabilitation project is proposed as a FY 2001 new start. The final schedules will be determined after approval of the evaluation report for Reach 1. As a FY 2001 New Start, award of the Reach 1 contract could be scheduled for April 2001 and construction completed in May 2004. The fully funded estimated cost of the contract work for Reach 1 is $66,574,347.

TABLE OF CONTENTS

SECTION I - INTRODUCTION

Para.	Topic	Page
1.	Purpose	1
2.	Authority	1
3.	Background and Related Studies	1
4.	Project Area	4
5.	Project Objectives	5
6.	Recommended Plan of Action	6
7.	Alternative Plans	7

SECTION II - SCOPE OF WORK

8.	General Work Scope	8
9.	Scope of Work	8
10.	Embankment Integrity	8
11.	Subsurface Investigations	9
12.	Hydrologic & Hydraulic Investigations	9
13.	Structural Modifications	9
14.	Environmental Analyses	10
15.	Economic Analyses	10
16.	Surveys	10
17.	Real Estate	10
18.	Cost Estimating	11
19.	Project/Study Management	11

SECTION III - WORK BREAKDOWN STRUCTURE

20.	Work Breakdown Structure	12
21.	Major Rehabilitation Report	12
22.	Land Acquisition	13
23.	Permits	13
24.	Project Management	14

SECTION IV - ORGANIZATION BREAKDOWN STRUCTURE

25.	Organization Breakdown Structure	14
26.	Project Team	17

SECTION V - RESPONSIBILITY ASSIGNMENT MATRIX

27. General 18
28. Project Team Responsibilities 18
29. Responsibility Assignment 19

SECTION VI - PROJECT SCHEDULE

30. Project Schedule 19

SECTION VII - BUDGETS AND COST ESTIMATES

31. Project Budgets and Cost Estimate 19
32. Local Cooperation Agreement 20

SECTION VIII - APPENDIXES

A Project Authorizations

B Project Schedule

C Project Map

SECTION IX - LIST OF ABBREVIATIONS

SECTION I - INTRODUCTION

1. PURPOSE The purpose of this Project Management Plan (PMP) is to provide a comprehensive plan identifying the work required for preparation of the Hoover Dike Major Rehabilitation Evaluation Reports. This PMP includes schedules and budgets for all activities required for preparation, review, and approval of the reports addressing seepage and stability concerns in the dike system around Lake Okeechobee. The PMP is not intended to be all-inclusive or to anticipate all possible changes to the project during its continuing development. Rather, it is a dynamic document that will require change and updating as the project progresses.

2. AUTHORITY The Flood Control Act, approved by Congress on 30 June 1948, authorized the first phase of a comprehensive plan to provide flood protection and other water control benefits in central and south Florida. The Act included measures for improving control of Lake Okeechobee by constructing or modifying the spillways and other structures, and enlarging the Lake Okeechobee levees to provide the intended flood protection, water storage and water supply. Levee seepage and stability have a direct effect on the capability of the levee to provide the authorized protection. The authorization for levee repairs and modifications of the Flood Control Act of 1948 justify the seepage and stability analyses identified in this PMP. Recent HQUSACE guidance has suggested that the report be funded using Operation & Maintenance (O&M) funds. This method will require the appropriation of construction dollars from Construction General (CG) funds. Appendix A provides a summary of authorizations related to the Herbert Hoover Dike System.

3. BACKGROUND The first embankments around Lake Okeechobee were constructed by local interests and were primarily composed of muck, sand, shell, and marl from adjacent borrow canals. These embankments were constructed circa 1915. During the 1930's, federal interest was initiated as a result of the hurricane tides of 1926 and 1928 overtopping the original embankments.

The River and Harbor Act, approved 3 July 1930, authorized the construction of 67.8 miles of levee along the south shore of the lake and 15.7 miles of levee along the north shore. The typical crest height of those levees ranged from +32 to

+35 feet, NGVD. The U. S. Army Corps of Engineers (USACE) constructed these levees between 1932 and 1938. A major hurricane in 1947 prompted the need for additional flood protection work in Florida. In response, Congress passed the Flood Control Act of 1948 authorizing the first phase of the comprehensive plan for flood protection and other water control benefits in central and southern Florida. The passing of that Act authorized the Central and Southern Florida (C&SF) Project.

By the late 1960's the design and construction of the new dike system was completed, raising the elevation of the 1930's levees to about +32 to +41 feet, NGVD. The new dike system included additional levee segments to complete the levee construction around the entire lake, except at the mouth of Fisheating Creek. The completed levee system provides continuous embankment flood protection and is known as the Herbert Hoover Dike System. Previous investigations, in the 1980's and early 1990's, of the dike system's potential seepage and stability problems resulted in the identification of two major areas of concern; the seepage and embankment stability at the culvert locations, and the problematic foundation conditions of the dike.

3.1 Environmental Concerns Preliminary data obtained indicates a concern for improving or retaining lake access routes used by fresh water turtles. Other potential areas of concern include historical and archeological resources, effects on recreation, sport, and commercial fishing, and certain species listed under the Endangered Species Act. Examples include the following: snail kite, bald eagle, and wood stork. The burrowing owl is listed by the State of Florida as a species of special concern.

3.2 Lake Regulation The primary objective of the lake regulation schedule is to store enough water during the wet season to provide adequate water supply during the dry season, while simultaneously providing adequate flood protection for areas adjacent to the lake. Local interests established the first regulation schedule in the early 1900’s,, after the completion of the West Palm Beach, Hillsboro, North New River, and Miami Canals. This schedule was designed to maintain lake levels between elevations 14.0 and 17.0 feet, NGVD. USACE adopted this schedule when Corps involvement began in the 1930's. Over the years the schedule has been modified as a

result of extreme weather conditions, project improvements, and changes in management policy.

The current schedule, Run 25, was documented in an Environmental Assessment (EA) prepared by the Jacksonville District in September 1994. Run 25 ranges from 15.65 to 16.75. The purpose of the 15.65 to 16.75 foot regulation schedule is to reduce damaging flows to the nearby estuaries without sacrificing the flood control or water supply benefits derived from the Lake. A two-year field test was initiated in May 1992, which concluded that Run 25 had no identifiable adverse impacts. Run 25 was approved for implementation in December 1994.

In 1968, Public Law 90-483 authorized raising the lake regulation schedule to a range between 19.5 and 21.5 feet, NGVD. Increasing development and corresponding increased water supply demands prompted the change in lake regulation authorization; however, these changes have not been implemented due to their effect on municipal and agricultural water supplies, navigation, recreation, area economics, and environmental concerns.

3.3 Seepage Studies Prior seepage analyses have been performed primarily to assess the effects of modifications to the lake regulation schedule. One such study was performed by the U. S. Geological Survey (USGS) for the Florida DNR in the early 1970's. This study involved the drilling and monitoring of observation wells in five locations along the south shore of the lake, between Caloosahatchee River and St. Lucie Canal. The study concluded that the seepage was predominately underseepage, occurring through beds of sand and shell, and porous limestone underlying the levee at shallow depths. The seepage rates as measured from five sites ranged from 0.1 to 0.9 cfs/mile/foot of head, across the dike. The largest seepage rates were monitored between the cities of Moore Haven and Clewiston.

The SFWMD conducted a study between June 1979 and January 1980, involving two sites along the east shore of the lake. Those sites were identical to sites 4 and 5 of the USGS study. SFWMD findings indicated that growth of a filtercake layer in the navigation canal had lowered the seepage rates from an average of 0.35 cfs/mile/foot in 1965 to 0.25 cfs/mile/foot in 1980.

In 1986, USACE investigated portions of the same area between Clewiston and Port Mayaca. That study concluded that while the volumes of through-seepage were small relative to the volumes of under-seepage, seepage emerging from the levee face at points above the toe could cause piping or erosion of the embankment. The study concluded that the potential for serious subsurface erosion during high lake stages exists. It also recommended a more thorough investigation of seepage and stability concerns.

The Herbert Hoover Dike Seepage and Stability Analysis Special Report, completed in March 1993, concluded that the analyses of exit gradients at the levee toe, heave, piping, creep ratios, and slope stability, yielded factors of safety less than the minimum values recommended by USACE engineering design manuals in some locations of the levee. The 19 original culvert locations present the most critical sites relative to seepage and embankment stability. When the embankments were raised in the 1960's, construction at the culvert sites was accomplished by adding material to the crown of the levee without a corresponding increase in base width, which would have required longer culverts.

3.4 Stability Studies The Special Report mentioned in the above paragraph included a preliminary stability analysis on a typical section of levee on the southeast shore. The calculated factors of safety were 1.3 and 1.1 for lake levels of 17.5 feet and 25.0 feet, respectively. USACE engineering manuals for design of earthen dams require a safety factor of 1.5 for all such structures. Note, authority exists (by Public Law 90-483) to increase the limit to 21.5 feet, NGVD. These higher and possibly long-term lake levels could subject the dike system to conditions more extreme than those for which it was designed.

Although subsurface information (which reveals the characteristics of underlying strata) is limited, some of the available data suggest that there may be sections of the embankment prone to seepage and stability problems. The most serious of these are the areas where a layer of highly permeable limestone underlies peat deposits. In some areas, the limestone layers are in direct contact with the lake by the excavation of a deep rim canal along the lake side toe of the embankment. The layer would act as a conduit for seepage flows, which could result in increased soil pressures. These

pressures may induce loadings on the overlying peat layers, which are characterized by low shear strengths. As a result, uplift forces at the landside embankment toe could cause a failure.

4. PROJECT AREA The geographic area addressed by the Hoover Dike Major Rehabilitation Project is shown in Appendix D. This area is located in southern Florida and consists of the entire levee system encircling Lake Okeechobee, including all structures within the levee system. The system consists of approximately 143 miles of levee, (divided into nine levee segments; L-D1, L-D2, L-D3, L-D4, L-D9, L-47, L-48, L-49, and L-50), the 19 original culverts, six hurricane gates and other hydraulic structures.

Study areas are defined as Reaches 1 - 8, in order by priority. Reach 1 is 22.44 miles long and extends from St. Lucie Canal at Port Mayaca, on the east side of the lake, to Hurricane Gate Structure (HGS)-4 at Belle Glade. It includes levee sections L-D9 and part of L-D2. Reach 2 is 19.10 miles long and extends from HGS-3 at Lake Harbor to HGS-1 at Moore Haven and includes levee L-D1 and part of L-D2. Reach 3 is 7.10 miles and extends from HGS-4 to HGS-3 and includes the remainder of L-D2. Reach 4 is 19.40 miles and extends from HGS-1 at Moore Haven to Fisheating Creek, and includes levee L-D3. Reach 5 is 15.70 miles and extends from the Kissimmee River north to Nubbin Slough and includes levee L-D4. Reach 6 is 20.00 miles long and extends from Fisheating Creek on the west side of the lake, north to Indian Prairie Canal and included levees L-49 and L-50. Reach 7 is 17.70 miles long and is located on the northeast shore and extends from St. Lucie Canal north to Nubbin Slough, and includes levee L-47. Reach 8 extends from Indian Prairie Canal to the Kissimmee River and includes levee L-48.

5. PROJECT OBJECTIVES This PMP outlines the actions needed to prepare the Hoover Dike Major Rehabilitation Evaluation Reports addressing seepage and stability problems along the Herbert Hoover Dike System. The Herbert Hoover Dike Seepage and Stability Analysis Special Report, dated March 1993, indicated that stability problems do exist within the embankment system. Prior analysis has indicated that, at current lake levels, safety factors for portions of the embankment system are less than regulations require. The objectives of the Major Rehabilitation Evaluation Reports will be outlined as follows:

a. Establish the overall engineering condition.

b. Establish current reliability of all major project features including but not limited to the embankment and the following structures: S-310, S-354, S-351, S-352, S-308, S-193, S-77, C-6, C-5, C-5A, C-1, C-1A, C-2, C-3, C-4, C-12, C-12A, C-10, C-13, C-10A, C-14, C-16, C-11, & C-8.

c. Identify reliability problems associated with critical project features.

d. Identify project features, which are NOT unreliable.

e. Identify operational/potential reliability problems.

f. Identify alternative methods to remedy or manage the problems.

g. Develop cost estimates for proposed solutions and estimate total economic costs & benefits, including the base condition and alternative solutions.

h. Identify environmental concerns.

i. Identify recommended plan.

j. Prepare M-CACES cost estimate for the recommended plan.

6. RECOMMENDED PLAN OF ACTION The recommended plan of action involves preparation of a separate evaluation report for each reach of the project. Detailed economic, environmental, hydraulic and hydrologic analyses will be performed for Reach 1. Preliminary information regarding Reaches 2-8 will be contained in the first MRR. Geotechnical findings have revealed that another MRR should be prepared for Reaches 2 and 3. The major objectives of the reports are as follows:

6.1 <u>Major Rehabilitation Evaluation Report 1</u>

a. Prepare detailed economic analyses (including risk and uncertainty) on the entire dike system.

b. Perform detailed geotechnical and related engineering and design work relative to Priority Reach 1.

c. Identify reliability problems within the entire dike system.

d. Perform detailed risk and uncertainty analysis on project features within Priority Reach 1 including but not limited to the embankment and the following structures: S-351, S-352, S-308, C-12, C-12A, C-10, C-13, C-10A, C-14, C-16, &

C-11.

e. Address potential hydrologic/hydraulic, environmental and cultural resources issues for the entire dike system.

f. Prepare appropriate environmental and cultural resources documentation for Priority Reach 1.

g. Prepare detailed time and cost estimates for design and construction of alternatives in Priority Reach 1.

6.2 The recommended solution consists of constructing a seepage berm for the entire 22 miles in Reach 1. The same solution was proposed for Reaches 2 & 3. In areas where the embankment toe rests on a peat layer, construction will begin with the removal of the peat. The berm would consist of a 1-foot thick layer of filter sand overlain by a 5-foot thick layer of filter stone. The berm will prevent the piping of sands and silts from the embankment and its foundation. The landward side of the berm would contain a 48-inch diameter perforated culvert wrapped in geotextile fabric. The purpose of the culverts is to collect and convey seepage flows to controlled outlets which empties into existing drainage systems. A 2-foot wide by 2-foot deep relief trench will be excavated below the culvert. The trench will control the uplift pressures.

6.3 Major Rehabilitation Evaluation Reports 2 and 3: The primary objective of the report addressing Reaches 2 and 3 will be similar to those for Reach 1. The scope of the report for reaches 2 and 3 will be refined after completion of the first Major Rehabilitation Evaluation Report. Subsurface investigations, geotechnical studies, and risk and uncertainty analyses will be performed to assess the reliability of project features and to identify problem areas and solutions.

7. ALTERNATIVE PLANS An array of alternative plans was developed and evaluated based on the economic, engineering, and environmental considerations. The economic analysis of each alternative includes consideration of risks and uncertainties. Cost estimates will be prepared for each alternative. The complete economic evaluation will require that preliminary estimates of repair costs be prepared using available data for all reaches. The alternatives derived may include the possible use of landside seepage berms, relief

wells, impervious cut-off walls, and changes in lake operation strategies.

SECTION II - SCOPE OF WORK

8. GENERAL WORK SCOPE This section provides a description of the products and services to be performed in preparation of the Major Rehabilitation Evaluation Reports. The scope of work is the basis for preparation of the Work Breakdown Structure, the project schedule, and cost estimate. The PMP will be revised to include the scope of work for plans and specifications for Reach 1 if the Major Rehabilitation Report is approved.

9. SCOPE OF WORK The general goal of the Hoover Dike Major Rehabilitation project is to provide a reliable embankment system around Lake Okeechobee to contain the lake waters for storage supply, flood protection, navigation, and recreation. Analysis of the existing dike system has revealed areas where the factors of safety relative to stability are less than current regulation requirements. This section describes, in detail, the work required to address seepage and stability concerns in Priority Reach 1. The first report will define the extent and severity of any unstable condition within Priority Reach 1 and describe necessary remedial action to repair or manage the problem.

10. EMBANKMENT INTEGRITY There are eight priority areas (reaches) of work defined for this project. Seepage and stability issues for each reach will be addressed in separate reports. Risk analyses and evaluation of the probabilities of failure will be performed for each reach. Risk analyses will be required for all reaches of the dike system and will be performed individually for each report.

10.1 Slope Stability. The original design criteria will be identified and compared to the latest design criteria, noting any deficiencies between them. This will allow for more accurate determination of possible design problems. Those problems and corresponding solution alternatives will be identified in the reports. Embankment cross sections, within Reach 1 will be surveyed every 1,000 feet along the levee centerline, and will extend from the bottom of the rim canal to approximately 500 feet landward of the levee. Slope stability at each culvert location will be investigated, and will include the influence of the foundation piles.

10.2 Seepage. Piezometer arrays will be installed in several locations around the lake to measure hydrostatic pressures, specifically beneath and landward of the levees.

11. SUBSURFACE INVESTIGATIONS Detailed subsurface data will be required for Reach 1. Subsurface investigations for the other reaches will be performed during the preparation of future reports. Field and laboratory tests will be performed to adequately define the engineering characteristics of the soil. Recharge tests will be performed during drilling operations.

11.1 Core Borings Core borings will be drilled every 3,000 feet along the centerline of the levee crest, at the landside toe and 300 feet out from the landside toe. The number of borings required may vary depending on the extent of existing data, complexity and subsurface conditions. Borings will be drilled at each culvert site in Reach 1 on the lakeside, landside, and through the centerline of the levee.

11.2 Rim Canal Core borings will be taken at 6000' centers along the rim canal to determine the extent of the less permeable material that may have accumulated.

12. HYDROLOGIC/HYDRAULIC INVESTIGATIONS Drainage basin hydrology and Lake Okeechobee flood routings will be investigated. Stage-frequency curves will be developed and utilized to establish static (i.e. long duration) pool elevations for the lake. Hurricane surge analysis will be performed to quantify the short duration changes in water surface elevations, and corresponding recurrence intervals. Long and short duration components of the pool elevations will be combined to establish the range of design headwater elevations. Possible alternative schedules will be identified and possible operational strategies will be considered to establish the range of tailwater elevations. Dambreak analyses will be performed to quantify the consequences of embankment non-performance for various scenarios. Hydrologic and hydraulic investigations will be executed relative to water supply studies, flood damage analyses, and identification of structural alternatives. Analyses will include quantifying the probabilities of occurrence of events, which result in load conditions causing unsatisfactory performance of the dike system.

13. STRUCTURE MODIFICATIONS Preliminary designs and cost estimates for structural alternatives will be developed. Reliability analyses for the structures in the selected plan will be performed, if necessary. Potential impacts to existing structures in Reach 1 will be investigated and if necessary, designs and cost estimates for modifications will be prepared.

14. ENVIRONMENTAL ANALYSES An evaluation of existing fish and wildlife resources will be made, including an inventory of habitat types, fish and wildlife species, and other factors of concern to this project. Information will be exchanged with all agencies and groups that will contribute to the environmental analysis of alternative plans. An assessment of potential impacts on cultural resources, recreation, water quality, and water supply will be performed. Studies of the navigation and recreation uses of the lake and a flood damage analysis will be performed. A Notice of Intent to prepare an EIS will be prepared and submitted to EPA for publication in the Federal Register. A Biological Assessment of potential impacts on endangered species will be prepared and coordinated with USFWS. Identification will be made of critical hydrologic conditions for resident fish, wildlife and plant species. Impacts resulting from embankment failure will be investigated with respect to fish and wildlife species, and alternative operational strategies.

15. ECONOMIC ANALYSES Existing economic data will be reviewed and additional data will be collected. A detailed economic flood damage analysis will be conducted for the entire lake and will include the identification of existing lands, structural inventory, agricultural inventory, and prior flood damage studies. Water supply and recreation impact studies will also be performed. Reach 1 will be evaluated for with and without project benefits, and risk and uncertainty analyses will be performed to determine the economic efficiency of alternative rehabilitation plans.

16. SURVEYS Survey requirements for the Reach 1 rehabilitation report will include the location of borings and cone penetrometers, embankment cross sections at 1,000-foot intervals, aerial photography, scanning, and photogrammetry.

17. REAL ESTATE A detailed analysis of real estate requirements and costs will be prepared for Reach 1. All necessary rights-of-entry for surveys will be obtained prior

to initiating work in that reach. If necessary, a real estate ownership survey will be conducted for each reach to insure that adequate real estate interest exist to implement the recommended plan in each reach. Cost estimates of necessary land acquisition will be prepared for each alternative, if required. A real estate gross appraisal for Reach 1 will be prepared during the PED phase.

17.1 Relocations of Utilities There are no known utilities in Reach 1, other than the utilities at the structures that will be affected by the project. However, an investigation will be conducted to determine if any utility relocations will be necessary.

18. COST ESTIMATING An M-CACES cost estimate will be prepared for Reach 1 for the recommended plan. Preliminary cost estimates associated with all solutions alternatives will be prepared.

19. PROJECT/STUDY MANAGEMENT: The project will be closely coordinated with all functional elements and other parties or interests. Project Management personnel will monitor actual progress of the project in relation to the project schedule throughout the preparation of the reports. District Hydraulics/Hydrology personnel will be responsible for the technical management/coordination of the project risk and uncertainty analyses.

SECTION III - WORK BREAKDOWN STRUCTURE

20. WORK BREAKDOWN STRUCTURE: Based upon the scope of work, the work breakdown structure (WBS) consists of a hierarchical definition of the products and sub-products required to implement the project. This PMP includes schedules and budgets for all activities required for preparation, review, and approval of the eight Major Rehabilitation Evaluation Reports required under this project. Appendix A is a work breakdown table, which identifies all major activities required in the work breakdown hierarchy. It also identifies which office will have the lead for the activity, the budgeted amount for execution of each activity, and the schedule dates. Note that all scheduling is based on early start/early finish dates.

21. MAJOR REHABILITATION EVALUATION REPORTS HQUSACE guidance has directed the preparation of a Major Rehabilitation Evaluation report for each Reach of this project. The report for Reach 1 will provide engineering designs and cost estimates for improvements to Reach 1. It will also address economic, environmental, and hydrologic/hydraulic issues concerning the entire Lake Okeechobee/Herbert Hoover Dike system. The first evaluation report will also include preliminary cost estimates for rehabilitation efforts in Reaches 2-3. After HQUSACE approval of the evaluation report, any rehabilitation work required in Reach 1 would be budgeted under the Construction General (CG) appropriation. Upon approval of the first evaluation report, plans and specifications will be prepared for Reach 1.

a. Reach 1 Report: This PMP outlines the work required for the Major Rehabilitation Evaluation Report for Reach 1. Economic and environmental analyses will be provided as appendices to this report.

(1) Right-of-Entry: Rights of Entry will be obtained for areas where work will be performed outside the dike system. These lands may be privately owned. Survey and geotechnical crews will require access to areas on the landward side of the dike system for survey and core boring operations.

(2) Surveys: Surveys will be performed for the entire dike system with primary emphasis on Reach 1. The surveys consist of topographic surveys, aerial photography, and cross

sections. All project maps for Reach 1 will be developed at this phase of the project.

(3) Water Supply Features: An assessment of the water supply needs will be performed in the initial report, including navigation and recreation.

(4) H & H Modeling: Detailed hydraulic modeling and evaluation of storm data will be completed. H & H modeling results will be used to determine hydraulic loads and to provide input for geotechnical analysis and risk/uncertainty analysis. Flooded area maps for various failure conditions will be evaluated for use in the economic justification.

(5) Subsurface Features: Original design criteria, relative to seepage stability, will be evaluated and compared to existing criteria and deficiencies noted. Additional subsurface information will be obtained in Reach 1. The location of these borings will be selected to supplement existing subsurface information. These locations will include any culvert or other structure locations within Reach 1.

(6) Structures: Potential structural impacts caused by the design loads on structures within Reach 1 will be investigated. The reliability and unreliability of structures will be identified and solutions will be proposed, if necessary.

b. Reach 2-3 Reports: Reaches 2 - 3 will have a similar scope as that for Reach 1. Economic, environmental, and engineering analyses conducted on the entire lake and dike system in the Reach 1 evaluation report will be utilized in the evaluation reports for Reaches 2-3. During preparation of these reports, economic, environmental, and engineering efforts will be significantly less than those associated with the first report.

22. LAND ACQUISITION Real estate ownership of lands involved will be investigated to determine whether any private property acquisition will be required. This effort will be performed during the PED phase.

23. PERMITS Water Quality Certification may need to be obtained for the project from the Florida Department of Environmental Regulation.

24. PROJECT MANAGEMENT Project management will involve monitoring and coordination of all project activities, scheduling, and budgeting. It is the responsibility of project management to deliver the project in accordance with the PMP through completion of all reports, MRRs, construction, and settlement of outstanding claims.

SECTION IV - ORGANIZATION BREAKDOWN STRUCTURE

25. ORGANIZATION BREAKDOWN STRUCTURE The Organization Breakdown Structure specifies organizations responsible for performing scheduled activities. Office symbols are listed below:

PROJECT MANAGEMENT

DP Deputy District Engineer for Programs and Project Management
DP-A Principal Assistant to DDE(PM)
DP-I Project Manager

CONSTRUCTION/OPERATIONS DIVISION

CO Construction/Operations Division

CONTRACTING DIVISION

CT Contracting Division

ENGINEERING DIVISION

COST ENGINEERING BRANCH
EN-C Cost Engineering Branch

DESIGN BRANCH
EN-DC Specifications Section
EN-DD Drafting Section
EN-DL Levees and Waterways Section
EN-DM Mechanical Section
EN-DS Structures Section

GEOTECHNICAL BRANCH
EN-GS Soils Design Section
EN-GG Geology and Exploration Section
EN-GH Hazardous, Toxic, & Radiological Waste Section

HYDROLOGY AND HYDRAULICS BRANCH
EN-HC Coastal Design Section
EN-HH Hydraulic Data and Design Section
EN-HI Hydrologic Investigation Section
EN-HW Water Management and Meteorology Section

SURVEY BRANCH
EN-S Survey Branch

PLANNING DIVISION

ENVIRONMENTAL RESOURCES BRANCH
PD-EE Environmental Quality Section
PD-ER Environmental Coordination Section
PD-ES Environmental Studies Section

ECONOMICS AND SOCIAL ANALYSIS BRANCH
PD-D Economic and Social Analysis Branch

REAL ESTATE DIVISION

RE Real Estate Division

OTHER CORPS OF ENGINEERS OFFICES

SAD South Atlantic Division
HQUSACE Headquarters, U.S. Army Corps of Engineers

OTHER AGENCIES

SFWMD South Florida Water Management District
USFWS U.S. Fish and Wildlife Service

26. Project Team The following individuals are members of the Herbert Hoover Dike Stability Project Team:

U.S. ARMY CORPS OF ENGINEERS, JACKSONVILLE DISTRICT

Programs & Project Management Division		
Kimberly Brooks-Hall**	Project Management	**904-232-3155**
Public Affairs Office		
Jacquelyn Griffin**	Public Relations	**904-232-1650**
Engineering Division		
Peter Grace**	Coastal Design	2104
Joe Gurule	Coastal Design	1199
Jim Vearil	Water Mgmt & Meteorology	2142
Sue Sofia	Water Mgmt & Meteorology	2785
Mike Choate	Hydrologic Investigation	3143
LeRoy Noerper	Survey	2031
Sam Honeycutt	Geotechnical	1970
Tony Dipiero**	Geotechnical	2030
Bob Ross	Geotechnical	2930
Shashi Makker	Mechanical	1112
Ed Morente	Levees and Waterways	1071
Gene Campa	Cost Engineering	2418
Planning Division		
Jim Riley	Environmental Quality	2117
Mark Ziminske	Environmental Studies	1786
David McCullough	Environmental Studies	3685
Paul Stevenson	Recreation	2130
Eric Raasch	Economics	3680
Dan Peck**	Economics	2784
Martin Gonzalez	Flood Control	1117
Real Estate Division		
Hansler Bealyer**	Real Estate Acquisition	1178
Office of Counsel		
John Pax**	Real Estate Counsel	1168
Information Management Office		
Rory Sutton**	Geographic Information System	2743
Contracting Division		
Deidra Cox**	AE/Construction Contracts	1215
Construction-Operations Division		
Joe Britton**	Construction Services	2078

** Technical Managers for respective offices.

SECTION V - RESPONSIBILITY ASSIGNMENT MATRIX

27. GENERAL The purpose of this section is to describe the organizational responsibilities for performance of the work activities for the project. It is the intersection of the Organizational Breakdown Structure and the Work Breakdown Structure.

28. PROJECT TEAM RESPONSIBILITIES

a. Project Manager The Project Manager is responsible for managing the project parameters (cost, budget, schedule, scope, and quality), as well as interfacing with those involved in the project process (customers, functional elements, government and non-government entities). The PM manages the delivery of the project in accordance with the PMP through construction into initial project operations or transfer to the customer and settlement of outstanding claims. Although the technical elements, through the technical managers, are responsible for the content and quality of technical products, the PM has the responsibility and the authority to challenge technical issues, when necessary.

b. Technical Manager Each functional element is represented by a Technical Manager (TM). The TM's, working under the supervision of the functional chiefs, are responsible for the content and quality of the technical products. While the TM serves on the project management team, the majority of his or her time is spent on technical product management. Specifically, the TM's are responsible for the following:

(1) development of technical products;
(2) interface with PM;
(3) coordination of technical disciplines;
(4) coordination with technical functions;
(5) point of contact for function's supporting contractors;
(6) coordination of function's PMP input;
(7) communication with SFWMD's technical representatives;
(8) representation of technical issues at PRB.

c. Project Team Members In coordination with the PM and TM, the project team members are responsible for development

of schedules and execution of technical products within budget and on schedule. Each team member will serve as the primary contact for activities assigned to their office. Team members are responsible for providing information to the PM and TM that has been coordinated and approved by their respective supervisors.

29. RESPONSIBILITY ASSIGNMENT Each activity, the office responsible for its execution, and the budgeted cost is defined in Appendix A.

SECTION VI - PROJECT SCHEDULE

30. PROJECT SCHEDULE: A detailed project schedule is provided herein as Appendix B. The activities provided in the schedule represent the lowest level provided in the WBS. The schedule details those milestones, which need to be accomplished for construction of Reaches 1-3. Information for Reaches 2 and 3 will be modified should a Major Rehabilitation Report be approved.

SECTION VII - BUDGETS AND COST ESTIMATES

31. PROJECT BUDGETS AND COST ESTIMATES: The fully funded cost estimate is provided below. It is $66.6 million, including contingencies of $8.5 million. The contingencies will be managed by the PM in accordance with ER 5-7-1. The project schedule shown in Appendix B is the basis for the estimates and budgets.

11	Levees and Flooodwalls	$59,632,000
01	Lands and Damages	$ 3,473,000
30	Planning, Engineering, and Design	$ 800,000
31	Construction Management	$ 2,670,000

Project Cost (September 97 Price Level) $66,575,000

b. A scope of work for each task has been included in the estimate. A contingency factor has been applied based upon the level of uncertainty of each item. Funds will be

allocated to the team members in accordance with the budget and schedule.

c. Contingencies will be managed by the PM. The PM has approval authority over certain limited cost/contingency changes. Larger cost/contingency changes will be elevated. The PM is responsible for identifying any inflation changes in the estimate obtained through the annual budget cycle.

32. LOCAL COOPERATION PLAN (LCA)

The latest guidance from HQUASCE, 6th Endorsement of the Special Report dated 09 August 1994, states as follows: “After approval of the evaluation report by HQUSACE, required major rehabilitation work should be budgeted under the CG appropriation for construction as a fully federally funded project.” Therefore, a LCA is not applicable to the subject project. CESAJ will continue to work cooperatively with the local sponsor to inform them of the project status. Operation and maintenance of the embankment system will follow the guidance contained in the existing agreement between the U.S. Army Corps of Engineers and the local sponsor, South Florida Water Management District.

SECTION VIII - APPENDIXES

A	Project Authorizations
B	Project Schedule
C	Project Map

APPENDIX A

PROJECT AUTHORIZATIONS

C&SF PROJECT - AUTHORIZATIONS RELEVANT TO HOOVER DIKE

1930 **Rivers and Harbors Act of 1930**

Senate Document 115, 71st Congress, 2nd Session.
Established Caloosahatchee River and Lake Okeechobee Drainage Areas Project which included about 81 miles of levees to protect developed areas from hurricane driven waters from Lake Okeechobee, improvements to outlet facilities and various other works. Local contribution to be all lands and $2,000,000. Est. cost $9,027,000.

1948 **Flood Control Act of 1948**

House Document 643 (pg 5), 80th Congress, 2nd Session.
Authorized Phase 1 of the comprehensive Central and Southern Florida (C&SF) Project. This phase included levees, channels and water control structures for Lake Okeechobee. It incorporated all works around the lake which were previously authorized in 1930. Local sponsor to provide lands, pay 15% of construction costs and capped local cash contributions at $29,152,000. COE to provide O&M for levees, channels, locks and control works of the St. Lucie Canal, Lake Okeechobee and Caloosahatchee River and the main spillways of the water conservation areas.

1954 **Flood Control Act of 1954**

Public Law 780, 83rd Congress, 2nd Session.
Expanded the C&SF Project to include the Kissimmee and Upper St. Johns river basins. Increased the Lake Okeechobee outlet capacity at the Caloosahatchee River.

1958 **Flood Control Act of 1958**

House Doc. 186 & Senate Doc. 48, 85th Congress, 1st Ses.
Removed monetary cap on local sponsor contributions set in 1948 authorization. Established local share of project costs for 1954 authorization and later features as 20% of construction and S&A costs plus LERRDs plus O&M responsibilities. Corps of Engineers responsible for O&M of Lake Okeechobee outlets.

1960 **Flood Control Act of 1960**

P.L. 86-645, 86th Congress, H.R. 7634
Designated the levees around Lake Okeechobee, Florida as "Herbert Hoover Dike".

1968 **Flood Control Act of 1968**

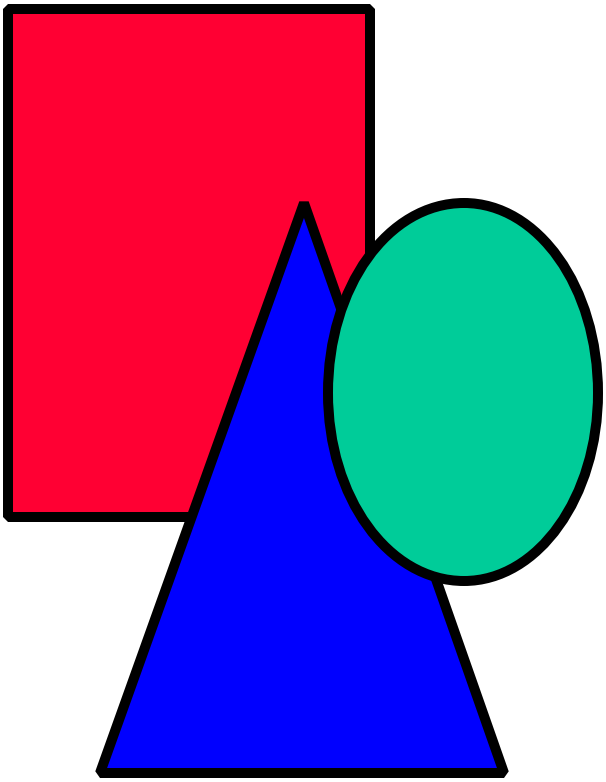
House Document 369, 90th Congress, 2nd Session. Authorized raising upper lake regulation level (maximum water conservation level) from 17.5 feet to 21.5 feet. Authorized raising lake levees and modifications to water control structures to implement same. Estimated cost $49.458M.
NOTE: These modifications have not been initiated. The local sponsor does not support higher lake levels at this time.

APPENDIX B

PROJECT SCHEDULE

APPENDIX C

PROJECT MAP



SECTION IX - LIST OF ABBREVIATIONS

C&SF	Central and Southern Florida
DNR	Department of Natural Resources
EIS	Environmental Impact Statement
FCA	Flood Control Act
FDM	Feature Design Memorandum
GIS	Geographic Information Systems
H&H	Hydrology and Hydraulics
HQUSACE	Headquarters, U.S. Army Corps of Engineers
LCA	Local Cooperation Agreement
M-CACES	Micro Computer Aided Cost Engineering System
P&S	Plans and Specifications
PCA	Project Cooperation Agreement
PM	Project Manager
PMP	Project Management Plan
PRB	Project Review Board
SAD	South Atlantic Division
SFWMD	South Florida Water Management District
SPF	Standard Project Flood
SR	State Road
TM	Technical Manager
USFWS	U.S. Fish and Wildlife Service
WBS	Work Breakdown Structure

1. INTRODUCTION.

1.1. Background.

1.1.1 In the 1920's, agricultural dikes were constructed along the south side of Lake Okeechobee. In the 1930's these levees were raised and extended using the authority provided by the River and Harbor Act of July 3, 1930. In the 1960's, the levees were again raised and extended using the authority of the Flood Control Act of June 30, 1948. The levee system now fully encircles the lake and has crest elevation ranging from about +36 to +40 feet. Due primarily to observed wave generated erosion of the levee, studies were carried out in the 1980's to evaluate the stability of the levee. A Reconnaissance Report in 1986 and a Special Report in 1993 determined there were stability concerns with the Levee.

1.1.2. In 1994, a compressive plan was initiated to acquire Geotechnical data and perform a detailed engineering analysis of the Levee. Due to the 140-mile length of the levee, the levee was subdivided into 8 reaches with priorities of study assigned to each. Priorities were assigned based on the estimated damage potential for each Reach of the levee. Reach #1 would be the initial Reach of the levee studied. The plan was to study each additional Reach of the levee in turn. The study of Reach #1 was under way when a 1995 high water event (+18.6) occurred in which serious seepage and piping events were observed along the southern and southeastern portions of the levee. Limited emergency repairs were made to distressed sections of the levee. A second high water event in 1998 again demonstrated seepage and piping problems.

1.2. Current Study.

1.2.1. This report documents the current Geotechnical study (1994-1998) of the levee. The field explorations and analysis were primarily carried out for Reach #1; but, limited explorations and analysis were also preformed for Reaches 2 through 8, Four geologic sections were analyzed in Reach 1 and an additional geologic section was analyzed for each of the remaining 7 Reaches.

1.2.2. The field work included core borings, classifications of materials, recharge tests, geophysical explorations, piezometer installations, dye tests, down hole video cameras, surveys, and cone penetrometer testing,

1.2.3. Laboratory testing included soil gradations, classifications of materials , loss on ignition, Shelby tubes, pin hole tests, acid leaching, strength tests (R, R Bar, Q tests), shell content, and permeability tests of soils and rock.

1.2.3. Office work includes the creation of geologic models, analysis of piezometric data, evaluation of geophysical data, and the analysis of representative levee sections using FASTSEEP, and UTEXAS3. The results of the analysis were correlated and statistical analysis was performed to generate engineering values for each geologic section.

1.3. Purpose Of Study.

1.3.1. The ultimate purpose of the Geotechnical analysis was to create "PUP" curves (Probability of Unsatisfactory Performance) for each reach of the levee that would be used by others in a Montecarlo simulation which would quantify the risk potential of Herbert Hoover dike.

1.3.2. The current Geotechnical evaluation of the levee has concluded that the levee would breach due to piping at a lake elevation of +21 feet. An engineering firm contracted to perform an independent analysis of the seepage problem also determined there was a significant risk of breach of the levee. A board of consultants reviewed both the Corps of Engineers analysis and the independent engineering firm analysis and concluded that an unacceptable risk of levee failure existed. The consultants recommended rehabilitation of the levee.

1.4. Case Histories of Embankment Failures or Near Failures in the Lake Okeechobee Area. There have been two embankment failures and two near embankment failures in the Lake Okeechobee region. The two failures (breaches) were Structure 154 which located on the north side of Lake Okeechobee in the Reach 5 portion of the embankment (LD-4), and the Florida Power and Light dam located two miles northeast of Port Mayaca. The two near failures were Lake Harbor and Culvert 10 areas of the Herbert Hoover Dike, and several sites along Embankment L-8 which is located southeast of Port Mayaca.

1.4.1. Structure 154**.** Structure 154 is part of the Herbert Hoover Dike embankment along the Kissimmee River. The embankment adjacent to the structure breached (100 foot gap) on 10 -12 May 1974. The structure is in a remote location and the exact time of breaching is unknown. The embankment is about 11 feet wide at the crest and about 120 feet wide at the base. The estimated maximum differential head at the embankment was 14 feet. The geology at the structure is sand deposits which have a few minor shelly layers. An investigation of the failure concluded that the embankment had failed due to piping associated with a shelly layer in the foundation (USACE, 1975). The geology/foundation conditions of the northern portion of Herbert Hoover dike should be similar to the geology/foundations conditions at the Structure 154 embankment.

1.4.1.1. Seepage Analysis of S-154. During our current evaluation (1998) of Herbert Hoover Dike, a seepage analysis of the structure 154 embankment was made using a sandy foundation. The analysis indicated the embankment would not fail. A second seepage analysis of the embankment which included a shell layer in the sand foundation indicated the embankment would fail by piping. Figures H-1.1 and H-1.2 show the differences in the flow nets between a homogenous sand embankment and one with a shelly layer (gravel sized particles). Table H-1.1 shows the critical gradient computations (refer to section “Analysis and results” for definitions). This is an important example in that it illustrates the importance of including relevant features in the analyses.

Table H-1.1. S-154 Gradient Computations.

Embankment	Head	Critical Gradient	Exit Gradient	Factor of Safety
Homogenous sand	14.0	0.843	0.351	**2.4**
Sand with shelly layer	14.0	0.843	0.882	**0.96**

1.4.2. Florida Power & Light Embankment Failure. We have limited data on the FP&L failure because information has been held close due to legal considerations.

The Florida Power and Light embankment failed suddenly and completely resulting in a breach on the evening of 30 October 1979. There was no prior indication that the embankment going to fail. The embankment is similar in size to Herbert Hoover Dike. Due to their proximity, the geology/foundation conditions along the northeastern portion of Herbert Hoover are probably similar to the geology/foundation conditions at the Florida Power and Light embankment failure site.

The embankment is located northeast of Port Mayaca. The embankment was of modern design so it is unlikely that the embankment failed do to through seepage. Therefore the embankment probably failed due to foundation conditions. There are a number of possible failure modes, but the actual failure mode could not be specifically identified.

1.4.2.1. Comments on the Embankment. The consultants' report stated a 1.5-inch pipe is the probable pipe size that initiated the failure. The quantity of material seen discharging from the pipe may appear negligible and still connect the pipe with the lake. If the pipe were smaller, water velocities would be high enough to erode the pipe larger. If the pipe were larger, water velocities would drop and the sands would settle and plug the pipe. The embankment had a soil-cement upstream face. Prior to the failure, boils observed along the toe trench at a number of locations. No boils were specifically noted at the location of the breach. Failure occurred 17 months after the reservoir was filled. Foundation materials are sketchy. A layer of hard pan (cap rock) just below the embankment could have acted as a roof for piping. Sands are located below the hard pan. A shell layer is below the sands. A Limestone horizon is below the shell layer.

1.4.2.2 Five (5) Potential Failure Mechanisms Evaluated by FP&L Consultants in 1980.

1. Piping of material above the hardpan layer to the toe of the embankment.

   ▲H = 17 feet Path Length = 240 feet

Calculations suggested that this would not happen; but, the factor of safety was minimal.

2. Piping of materials below the hard pan layer into the toe ditch.

▲H = 17 feet Path Length = 240 feet

Calculations suggested that this would not happen; but, the factor of safety was minimal.

3. Piping of material into a borrow pond. Aggravated by an estimated 2 foot drop in the borrow pond which was in tern triggered by a 5 foot drop in the L-65 canal. The embankment breached in about 3 days after the pond water was lowered 2 feet.

▲H = 19 feet Path Length = 400 feet

Calculations suggest this failure possible; but, the toe of the embankment should have heaved due to excessive artesian pressures before such a piping failure could have occurred.

4. Piping of material into L-65. Aggravated by a 5-foot drop in the L-65 canal.

▲H = 23 feet Path Length = 800 feet

Calculations suggest this failure unlikely.

5. Vertical downward piping into the shell layer below. Could have created a cavity below the embankment.

They could not model this type of failure.

1.4.2.3. An Additional Potential Theory of the Embankment Failure. The cap rock layer was predominately exposed in the area of the embankment breach. It is likely that artesian pressures below the cap rock broke through the overburden just downstream of the toe of the embankment. The cap rock served as the roof for the pipe and allowed the pipe to increase in size. When the cap rock eventually collapsed, the void below the cap rock was large enough to compromise the embankment. Power poles installed at the toe of the embankment would have penetrated the cap rock and could have completed the piping path.

1.4.3. Near-Failures at Herbert Hoover Dike. During the high water event in 1995, significant piping of materials occurred at the Lake Harbor portion of Herbert Hoover Dike as well as the embankment just north of Culvert Structure 10. Emergency corrective actions by COE personnel prevented a potential breach of the Dike. See Section "High Water Events" for details.

1.4.4. Embankment L-8. Embankment L-8 is a 17-mile long embankment/canal complex located in the agriculture fields just southeast of Port Mayaca and east of Reach 1. During the 1995 high water event, this embankment system also experienced unusually high water levels which put the embankment under distress. Piping of materials occurred over several miles along the embankment. At one location, the embankment was at the point of breaching. The only reason the embankment didn't breach was because the landowner by chance observed the failing embankment and had equipment available to reinforce the embankment and build a containment dike around the point of failure.

1.4.4.1. History of the Failure Point. The failure point was initially observed as a ½-inch pipe flowing clear water. Four (4) hours later, the pipe was observed to be 3 inches. Later it was observed that a portion of the upstream slope of the embankment had collapsed about 2 feet. The land owner brought in heavy equipment and reinforced the weak point.

1.4.4.2. Evaluation of the Failure. An inspection of the embankment established that substantial piping had occurred below a cap rock located at the base of the Peat Horizon. We estimate the maximum differential head experienced was 4.5 feet. It was reported that this cap rock was observed over a 3.5-mile reach of the embankment. Many points along the embankment experienced similar piping of materials just below the peat/cap rock interface. The cap rock acts as a roof for the piping. This is the same type of peat/cap rock interface that exists in Reach 1 of Herbert Hoover Dike. Even though the cap rock was clearly visible in the ditches adjacent to the embankment, investigative core boring had difficulty recovering/documenting the rock. The driller had to use special care in drilling to recover the cap rock. This indicates it is possible for a standard core boring to penetrate the cap rock and not show any indication that it is present.

Figure H-1.1 Homogenous Sand Embankment

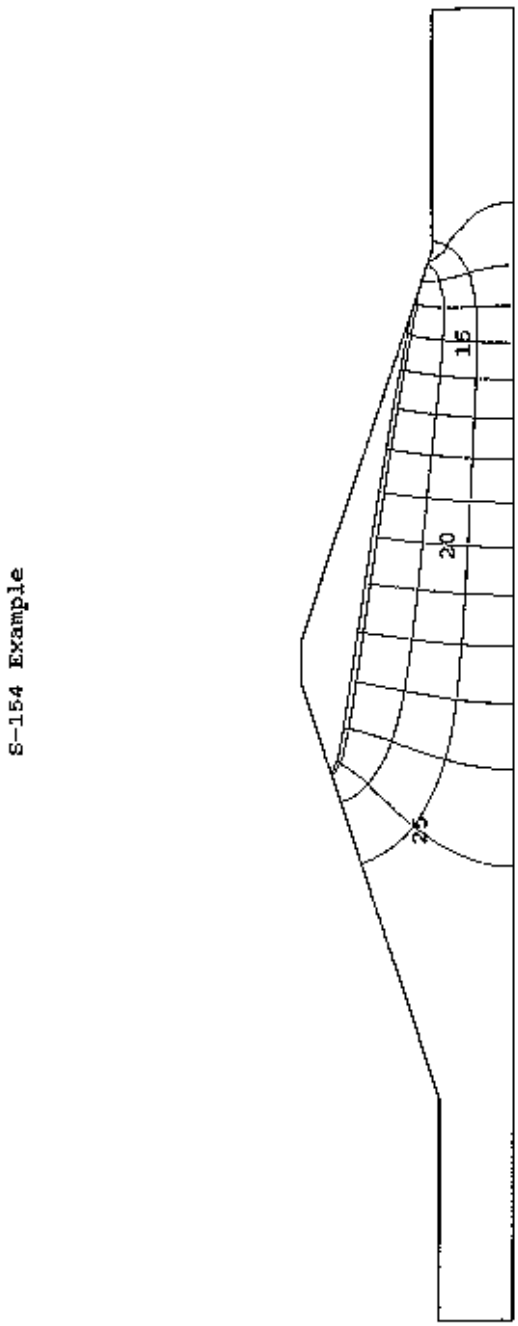
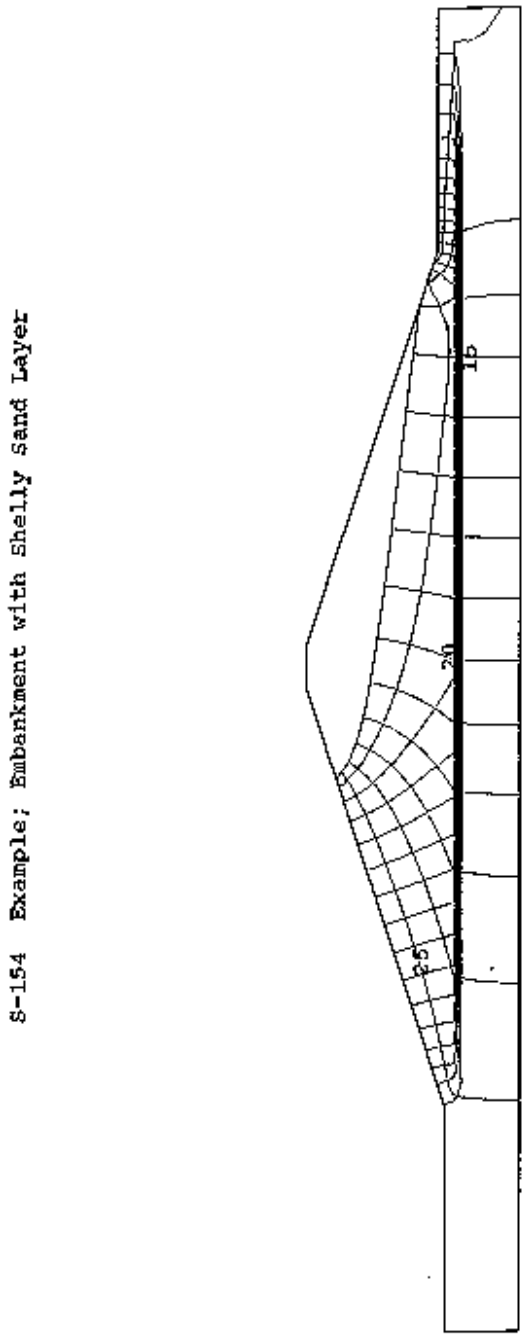


Figure H-1.2 Embankment with Shelly Sand Layer



2. FIELD EXPLORATIONS.

2.1. Core Borings. Core borings (Standard Penetration Testing (SPT) and drive sampling) were the standard tools for the HHD explorations. Core borings provide material samples for inspection and laboratory testing. Another advantage of core borings is that piezometers can be installed for a minimal extra cost. Because core borings are expensive and time consuming, they were drilled for the present MRER seepage exploration on 3,000 foot or greater spacing. This spacing will miss anomalies in the embankment and foundation that may exist between the core boring locations.

2.2. Recharge Tests. Recharge tests were performed primarily in rock. Some tests were done in unconsolidated material. Results are shown in Section "Data Evaluation and Statistics".

2.3. Filtercake Sampling Procedures. The filtercake sampling for a specific location is usually taken after the corresponding water boring has been completed. While the 6" casing is still in place to help stabilize the barges, the length of the work boat added to the length of the barges by securing the work boat directly seaward of the barges. A depth profile across the length of this combined unit is then obtained. The depth profile is taken at approximately one foot intervals from the water's edge as measured with a 100 m tape. Each depth measurement is obtained with a weighted tape to the nearest tenth of a foot. The profile is graphed in the field to determine the top, run, and base of the slope. If the base of the slope is not apparent within the length of the combined unit, the work boat is pulled seaward along a measured length of rope. Additional depth measurements are then obtained until the depth profile flattens, indicating the base of the slope.

Sampling locations are then designated with one at the top of the slope, approximately three along the run, and one at the base of the slope. Samples are located by reoccuping the specified depth. Sampling at a specified location begins with a grab sample obtained with a clamshell sampler. The material recovered is described and if the sample consists of rock or gravel it is discarded. If the sample contains silt, a subsample is collected as well as a duplicate if sufficient sample exists. The duplicate sample is obtained for later laboratory analysis. Additionally, if silt was obtained in the grab sample. An effort is made to determine the thickness of silt at the location. The thickness of silt is determined with a hand held PVC coring device. The PVC apparatus is pushed into the bottom material until resistance is encountered and the total depth of penetration is noted. The sample recovered from the PVC device is also described and if the material is different from that obtained from the grab sampling, a jar sample and duplicate are collected.

2.4. Cone penetrometer testing. The WES cone penetrometer truck was brought on site in March, 1995. Cone Penetrometer Testing (CPT) was expected to produce continuous soil classifications, continuous SPT values, and continuous piezometric values from each of the exploration locations. The continuous classification of materials and the continuous standard blow counts produced by the cone penetrometer were excellent. However, the CPT's were not deemed suitable in this application since they were not able to penetrate the soil horizons of the rocky or gravely materials of most interest, and the truck was not able to operate on soft ground at the toe of the embankment.

2.5. Geophysical Testing. In addition to core borings and CPTs, four geophysical exploration methods (resistivity, ground penetrating radar, self potential, and electromagnetic induction) were used to obtain subsurface data. The objective of the first phase investigation was to determine which method could best characterize materials and locate active or potential seepage paths. The alternative methods were tested for suitability at a test section in Reach 1 just north of the city of Pahokee where documented long term water seeps existed and embankment foundation properties were known. The target depth for the zone of primary investigations was 5 to 20 feet at the toe of the embankment.

2.5.1. Resistivity. Different subsurface materials can be characterized by their electrical resistance properties. The application of the resistivity method consists of placing a series of metal electrodes into the ground arranged in a prescribed manner. An electrical current is passed through two electrodes and the voltage potential is measured across two other electrodes. By varying the distance and geometry between the electrodes, the electrical properties of volumes of material at increasing depths in the subsurface can be established. This can be repeated at a succession of points along a linear array. Newly developed instrumentation for resistivity investigations was utilized in this test effort. Using this equipment, several long and continuous subsurface high resolution (for the method) resistivity profiles were acquired at the test site. The depth of intense investigation was 3 to 30 feet. However, analysis of the data did not reveal that the seepage areas could be discerned from areas where no water permeability problems existed. As a consequence, this method was not applied during the second phase of the investigation.

2.5.2. Ground Penetrating Radar. Areas which are underlain by granular material are often favorable sites for Ground Penetrating Radar (GPR) investigations. The depth of interrogation by this method is highly dependent on the percentage and type of clay contained in the subsurface material. Generally, clay exhibits changes in the dielectric that can greatly attenuate and modify the broadcasted radar wave. The test site proved to be suitable for GPR. This geophysical method involves transmitting a high frequency pulse (50 Kilo Hertz in this case) into the ground. At the test site, GPR penetration depths of 14 to 17 feet were achieved. Subsurface GPR profiles at the test site displayed boundaries in substantial agreement with material boundaries determined by core borings. The reflection coefficients (percentage of the pulse returned) at boundaries varies considerably in a lateral fashion. This effect is interpreted as the result of significant changes in the dielectric properties of the material rather than large resistivity variations. In addition, the absorption characteristics of the strata differs substantially. This implies that significant material changes in the sediments are present which can be illuminated and displayed. Although the results of the GPR investigation were of high quality and displayed notable lateral variations in subsurface conditions, no direct correlation between foundation seepage locations and the GPR stratigraphy could be determined.

2.5.3. Self Potential. The location of seepage under earthen structures can often be determined by self potential techniques. Flowing fluid will create a streaming electrical potential which can be measured. This data is generally collected by employing two special electrodes, one a

fixed reference and the other placed in locations along a traverse. The electrical potential in mill Volts is measured between the electrodes at each place. To achieve a measurable potential, a water head differential of at least several tens of feet is generally required. At the test site, the head differential was significantly less than 10 feet. No measurable electrical potential could be identified that was the result of seepage. Therefore, the self potential technique was not used in further studies.

2.5.4. Electromagnetic. The results of the electromagnetic investigation produced significant results in direct detection of seeps and local high ground water conditions under the embankment. In this application, a Geonics EM-31 was used to broadcast a continuous wave electromagnetic field into the subsurface. This signal interacts with and then induces eddy currents in the subsurface materials and fluids. These currents then generate secondary electromagnetic fields which can then be detected. Contrasting material types and dissimilar soil moisture concentrations produce differing electrical induction effects. As a result, if the lateral subsurface soil resitivity remains reasonably constant for reasonable distances, then any variations displayed in the conductivity results can be largely the result of lateral changes in subsurface moisture conditions. The electromagnetic field investigation was designed to achieve an average depth of significant material influence in the area of 14 to 16 feet in the subsurface. Consequently, it was feasible to gain a qualitative idea of lateral increases or decreases in the ground water table or moisture content along the embankment. Using this method, it was possible to directly detect and measure local seepage sites under the embankment toe.

2.6. Dye Testing. Results are shown in Section "Data Evaluation and Statistics".

2.7. Piezometers. Refer to section "Piezometer Data".

2.8. Bore Hole Camera. Video footage taken of the following borings:

Boring	Start Depth	Finish Depth	Tape Counter	Notes
			Tape #1	
25B	39.5	56.7	0 - 673	Shell@ 45.7; Void @ 43.2
25C	17.8 - 32.0		673 - 1230	Clear water
R2-FB	26.0 - 51.2		1230 - 1610	Clear @ 36.0; cross flow; boil @ bottom
27C	22.0 - 32.2		1610 - 2269	Clear @ 24.5; cross flow @ 31.8
27B	38.7 - 52.0		2269 - 2700	
28B	44.0 - 49.0		2700 - 2859	Dark limestone
28C	20.5 - 33.5		2859 - 3155	Logged bottom to top
10B	44.3 - 49.6		3155 - 4015	Poor visibility, possible flow @ 45.0
10C	29.2 - 32.3		4015 - 4470	Can see particles going into void @ 42.0
7B			4470 -	
31B				

Boring	Start Depth	Finish Depth	Tape Counter	Notes
			Tape #2	
31C				
27B Re-run				Very clear @ 39.5; cross flow @ 49.6

2.9. Surveys and Aerials. Surveys of Reach 1 consisted of 1000 foot long cross sections taken on 1000' centers starting from S-351 and working north to S-308. Every third section was 1500 feet long corresponded to a line of core borings and was labeled "CB-xx". Controlled aerial photography was taken along Reach 1. In 1994 (and prior) topographic and boundary surveys were requested of the entire project. This request was denied. In reaches 2 through 8 only 16 individual cross-sections over approximately 120 miles were obtained.

2.10. Results of Electromagnetic Surveys. (The following is an excerpt from DiPiero et al, 1998.) Approximately 26 miles of embankment was surveyed with EM induction. The objective of the EM survey was to identify active and potential foundation seepage locations and conduits. Areas of greater conductivity were often associated with seepage locations observed during a record high water event in October, 1995. These problems included boils and piping of foundation and embankment materials. One great benefit of this investigation was that large areas of competent dike foundations could be separated from areas containing potential problems requiring additional examination. Figure H-2.1 shows the areas surveyed.

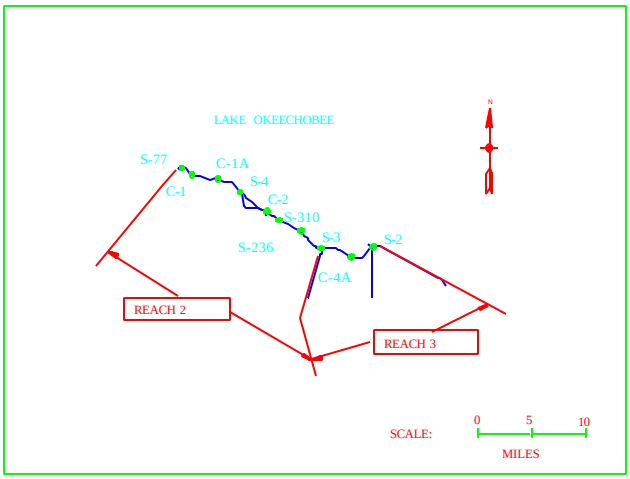


Fig. H-2.1. Area surveyed by EM Induction(Reaches 2 and 3).

2.10.1. Presentation and Analysis of Results. The collected and processed data is presented against distance to demonstrate lateral changes in subsurface moisture content. This is exhibited by the computed conductivity values in millSiems per meter (MS/M). In this investigation, the observed values ranged from 10 to 25 MS/M for the dry to less saturated areas, to

measurements of 40 to over 70 MS/M for the well saturated regions in the subsurface. In addition, an "in-phase" measurement may be extracted from the data. These additional computations are chiefly a response to highly conductive materials in the subsurface such as metal pipes and cables. Very large metal pipes generally generate a very sharp electrical conductivity peak. This feature can be used to easily distinguish metallic conductivity anomalies from responses due to seepage sources within the dike. In this investigation, electrical conductivity values greater than 45 to 50 MS/M indicate the subsurface (within the zone of instrument interrogation) is well saturated. For the purposes of graphical presentation, the values shown in this report represent the average of 50 values plotted as a point. Although individual peaks are not represented, this does not detract from the interpretation of large areas of several thousand feet.

2.10.1.1. Section from S-77 to S-4. This represents the western eight miles of the area of consideration. A typical cross section of the embankment, MRR2-E, is shown in Fig. H-2.2. The geologic conditions in this area are generally sand overlying layers of shell and limestone. The sand also contains varying percentages of shell. The shell and limestone layers are very pervious relative to the sand. Meyer (USGS, 1971) also found this area had the highest transmissivity of five sites studied along the south shore of the lake from Moore Haven east to Canal Point. (See Fig. H-2.2.). The deep excavation on the lake side of the embankment is the borrow canal. The ditch between the embankment and U.S. 27 serves as both a seepage control ditch and surface runoff interceptor. These excavations intersect some of the shell and rock layers in the foundation.

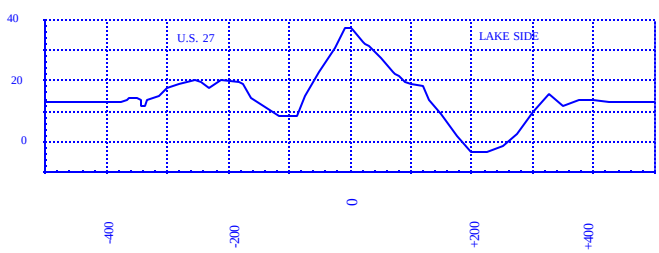


Fig. H-2.2. Cross section MRR2-E. Distances and elevations in feet.

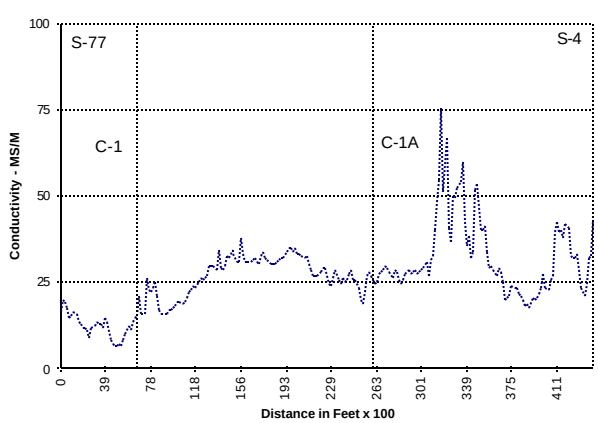


Fig. H-2.3. EM results from S-77 to S-4.

Figure H-2.3 represents data along the section from S-77 to S-4. Only one area in this section was considered for future study. The sustained peaks from distance 301 to 339 represent anomalous conditions in the foundation. Some of these areas registered a high in-phase measurement, indicating metal pipes in the subsurface.

2.10.1.2. Section from S-4 to S-236. Figure H-2.4 represents the middle eight miles of data collected. Cross section MRR2-E is also representative of this section. The geologic conditions in this area are similar with sand overlying layers of shell and limestone. This area represents a transition with increasing silt in the upper sand layers and lenses of peat at the natural ground surface. Anomalies from about distance 20 to 60 and at 135 were noted as significant. Also noted was the increase in background conductivity to about 35 MS/M, indicating a broad lateral change in the subsurface moisture content or changes in the groundwater surface elevation. Anomalies from distance 332 to 395 are characterized similarly, with a noted increase in background conductivity.

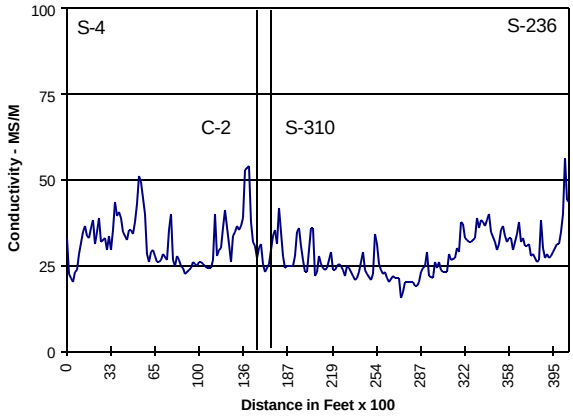


Fig. H-2.4. EM results from S-4 to S-236.

2.10.1.3. Section from S-236 to S-2. This represents the easternmost ten miles of data collected. A typical cross section, MRR3-A, is shown in Fig. H-2.5. The foundation conditions in this section are markedly different from that in the western two sections. A thick surficial layer of peat overlays a layer of silt. Below the silt are interbedded layers of sand and limestone. As in other areas, the percentage of shell in the sand layers can be significant. Also, note the depth of the drainage ditches between U.S. 27 and the embankment.

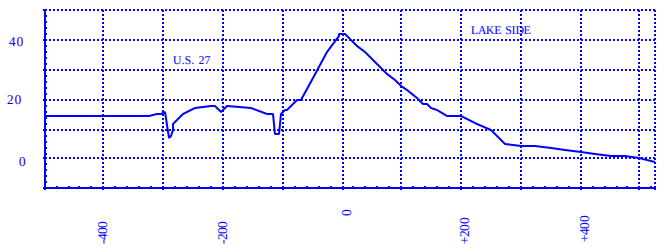


Fig. H-2.5. Cross-section MRR3-A. Distances and elevations in feet.

The most apparent feature of the EM data in Fig. H-2.6 is the pronounced sustained peak at distance 174. These large anomalies with conductivity values from 75 to nearly 100 MS/M indicated largely saturated subsurface conditions. This also inferred that larger hydraulic conductivities existed at depths below the region of inquiry. The background conductivities of 30 - 40 ms/m between S-236 and S-3 are a result of the generally higher ground water table along this portion of the embankment.

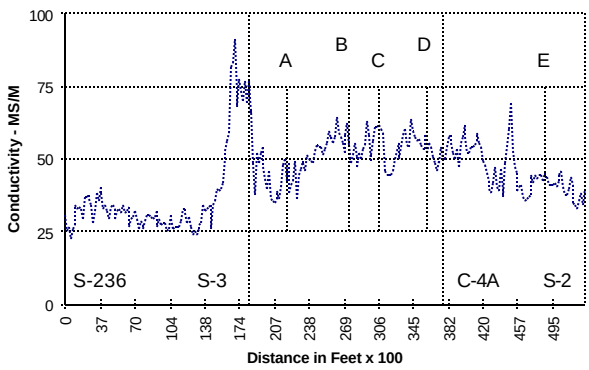


Fig. H-2.6. EM results from S-236 to S-2.

The area at distance 174 was investigated by both core borings and downhole video in September, 1995. The limestone was chalky and very porous with a unit weight of 105 pounds per cubic foot (pcf). Competent limestone in this area is generally 130 to 150 pcf. Visible flow was observed in the video. Clouds of silt and some sand particles were observed to move laterally at different levels within the limestone. Sand and shell that fell into the bottom of the boring were observed to "boil" from flow at the bottom of the hole. Another significant factor at this site was the large borrow ditch shown in Fig. H-2.7 at cross-section MRR2-G.

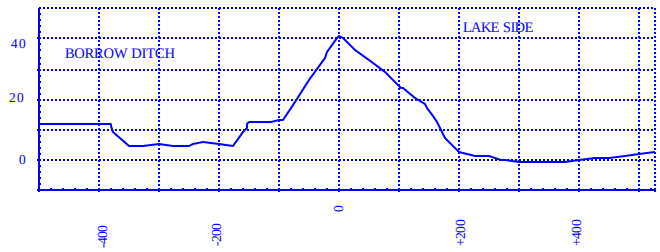


Fig. H-2.7. Cross- section MRR2-G. Distances and elevations in feet.

A broad area of numerous anomalies were encountered east of S-3 in Fig. H-2.8. Conductivities from 45 to 75 MS/M were common, with a background of approximately 50 MS/M. This is representative of very saturated subsurface conditions and an elevated groundwater table. Many of the peaks were located in regions of visible groundwater seepage. Note that at this time, the lake level was at 16.4 feet and the existing ground surface in this area (section MRR3-A) was approximately elevation 15.0. This section was expected to demonstrate the highest hydraulic conductivity.

2.10.2. Conclusions

2.10.2.1. High Water Event of 1995. Two months after completing the second phase geophysical survey of Reaches 2 and 3, Lake Okeechobee reached it's highest level in almost 50 years during late October and early November of 1995. The high daily average lake level recorded during this period was elevation 18.8 feet, NGVD. This corresponded roughly to a "30 year event". Lake elevations are typically between elevation 14 and 15 feet. Daily inspections were performed on the embankment and water control structures. This was in accordance with the Jacksonville District Dam Safety Program and the project's Emergency Action Plan (EAP). Nine general areas of seepage and piping were observed. There were four in Reach 1 and five in Reach 3. The worst of these sites had several pipes in the landside drainage ditch and boils were noted at some of the drainage culverts under U.S. Highway 27.

2.10.2.2. Comparison to High Water Event Damages. Damage to the embankment is defined as excessive seepage or piping. Damage was observed in Reaches 1 and 3. Since Reach 1 was not part of the EM survey, only damage to Reach 3 will be relevant to this comparison with one exception. There were five main areas of damage in Reach 3. They are labeled in Fig. H-2.6 as "A", "B", "C", "D" and "E". Other areas of saturated berms were encountered in Reach 3; but, points A through E were the only locations where there was visible flow exiting the slope or piping of material.

Point A. This was an area where a large volume of seepage was exiting along the downstream berm at elev. 15 ft for about 200 feet. There was no piping of material (flow was clear); however, the quantity of seepage was excessive and cause for concern.

Point B. This was an area about 1500 feet long where there were several dozen pipes exiting into the toe ditch with significant movement of material. This area had the most severe damage. Several of the pipes had caved in, causing depressions to extend from the ditch into the toe of the embankment about 25 feet away. Several small and one large sinkhole developed on the crest of the embankment. The large sinkhole was 3 feet in diameter by 6 feet deep. Probing with a steel rod indicated a cavity for an additional 5 to 6 feet before resistance was encountered. Sinkholes were also found in the crest of the embankment between points B, C, and D.

Point C. This area was approximately 2500 feet long where the downstream slope of the embankment experienced varying degrees of seepage without movement of material.

Point D. This was an area where several seeps exited the berm and a boil developed at elevation 15.5 ft. The boil piped material at first and then flowed clear. It is interesting to note that the embankment section was measured at 280 feet from the lake shore to the boil with a head of approximately three feet. This indicated that there was almost a direct connection to the lake, with less than three feet of head lost in almost 300 feet.

Point E. This was an area where two different types of problems emerged. Through seepage exited the slope at the berm and flowed clear. Also, in the adjacent ditch two areas were flowing into the ditch at the toe of the embankment. There was a strong sulfur odor and a white stain covered the organic soils in the area. This water was likely from below the confining silt layer, indicating flow from the foundation rock below through an open conduit that had formed by fracturing the silt layer below the peat.

Comment on Reach 1. One item of interest was an area of seepage in Reach 1. This area was approximately two miles north of the geophysical test site. During the High Water Event, excessive seepage exited the slope at approximately elevation 14 ft. Subsequent excavation of this area revealed an abandoned 6 inch diameter cast iron water supply pipe. Even though the pipe had been plugged with concrete, it had formed a "roof" for a flow channel which had developed over time. In a structure of this age and size, locating abandoned pipes is an important issue. A pipe of this size and depth would have been detected had this area been surveyed using the same EM methods as were used in Reaches 2 and 3.

2.10.2.3. Comparison to Known Geology. There is an apparent relationship between the EM values, the topography, the foundation materials, and the embankment materials. The higher conductivity values appear to be primarily caused by seepage through the embankment. Five separate cases can be presented for combinations of foundation and embankment conditions which contribute either to the high or low EM readings.

Case 1. In the western portion of Reach 2 from S-77 to S-4, the EM values are low, indicating a dry toe. Both the embankment and foundation are composed primarily of pervious shelly materials which drain the seepage into the foundation and keep the embankment toe dry as shown in Fig. H-2. 8.

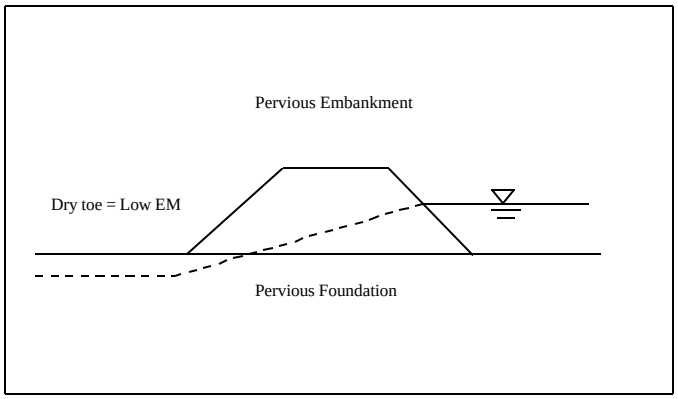


Fig.H-2.8. Case 1. Pervious embankment over pervious foundation.

Case 2. From S-4 to S-236, the readings are increasing slightly. There is a pervious to semi-pervious embankment overlying a confining layer. This is the general condition existing in Reach 3. Water from the lake, seeping into the embankment can't drain into the foundation because of the confining layer. The seepage is forced to exit the toe of the embankment. This results in a wet toe and high EM values as shown in Fig. H-2.9.

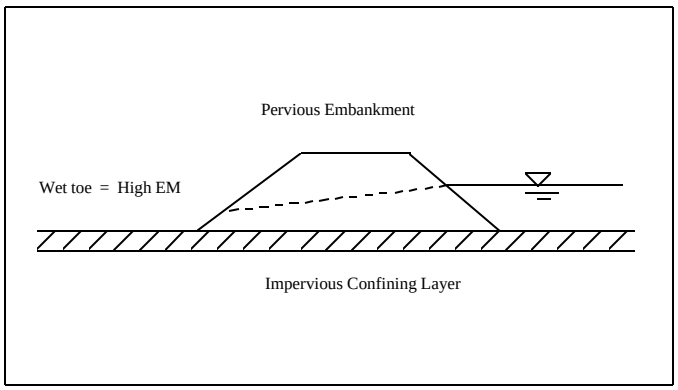


Fig. H-2.9. Case 2. Pervious embankment over impervious foundation.

Case 3. From S-3 to C-4A a dramatic change takes place. Gravely layers within the embankment are highly pervious and serve as conduits to supply seepage water directly to the toe of the embankment. This results in a wet toe and very high EM values as shown in Fig. H-2.10. This condition is also particularly hazardous since a significant head drop occurs in a short distance across the downstream portion of the embankment.

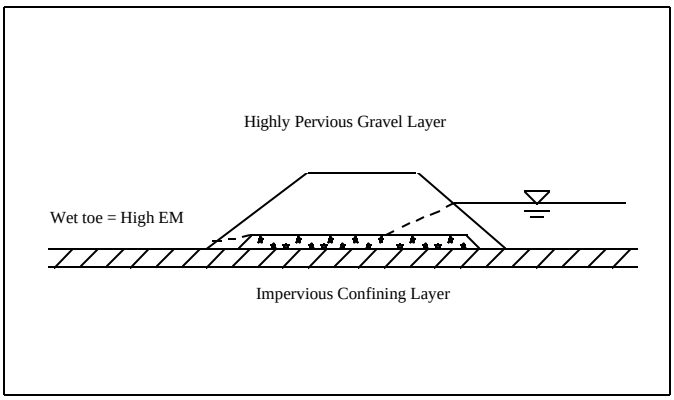


Fig. H-2.10. Case 3. Highly pervious gravel layer in embankment.

Case 4. Based on experiences from core borings and piezometer readings in Reach 1, a fourth condition can be presented. This condition exists sporadically along the embankment where the toe was constructed of clayey or silty materials. This material acts as a downstream impervious blanket which raises the piezometric level within the embankment as shown in Fig. H-2.11. This results in a wet toe and high EM values.

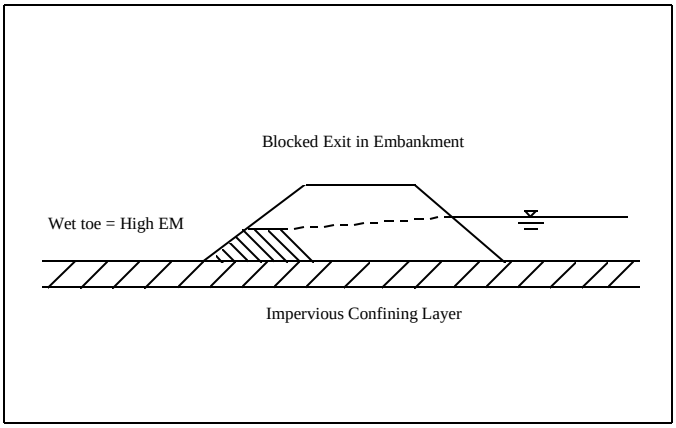


Fig. H-2.11. Case 4. Blocked Exit.

Case 5. Another local anomaly which can be detected is artesian flow from limestone layers beneath the embankment. This occurs where the confining layer has been heaved or fractured (usually in a ditch bottom). These fractures serve as conduits for seepage up through the confining layer into the toe of the embankment. This results in a wet toe and high EM values as shown in Fig. H-2.12.

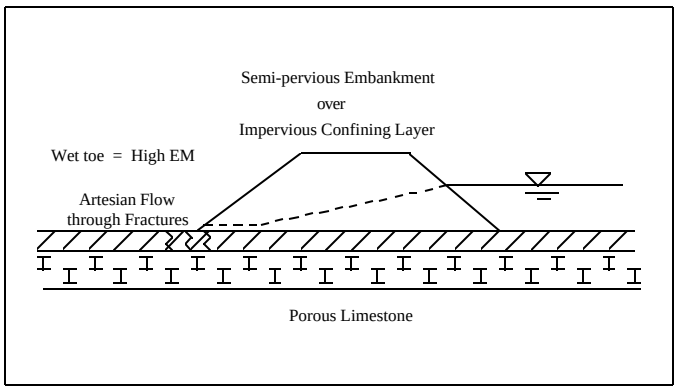


Fig. H-2.12. Case 5. Artesian Flow.

2.10.3. Summary

Geophysical methods can be used to aid in the characterizations of subsurface conditions. These methods are best combined with conventional exploration methods such as core borings and piezometers for a more complete picture of the subsurface conditions. In many cases, however, it is difficult to determine before hand what geophysical method will work best at any given site. At Herbert Hoover Dike, a geophysical test program determined that EM surveying was best suited to aid in the characterization of seepage locations. The subsequent EM survey of Reaches 2 and 3 was able to classify the materials along the levee toe according to their relative moisture contents. This provided additional information which led to the conclusion that a wet toe implies active seepage potential. During the subsequent high water event, the seepage problems seen were in areas where the EM survey had implied high seepage potential. It should also be noted, however, that data from piezometers installed in completed core borings was the single most effective tool in understanding the seepage problems associated with the project.

Some key advantages of the EM method are:

1. EM investigations are useful in differentiating large segments of an earthen embankment dam. This can assist in classifying potential problem areas from those less inclined to have seepage problems.

2. It can provide a continuous log of the moisture conditions (no gaps in the data), it can proceed rapidly (2.5 miles per hour), and it is relatively inexpensive considering the per mile cost. It is the only method available that has the potential to characterize the entire 140 miles of HHD at a minimal cost.

3. It could be used to examine degradation of the embankment and foundation over time to monitor changes in saturated or unsaturated areas. This potential dam safety tool could give an indication of performance after a number of high water events.

2.10.4. Potential for Future Applications

Since it is financially impractical to instrument all of HHD with a close regular spacing of conventional instruments such as piezometers, EM could be used to establish a baseline conductivity around the perimeter of the embankment at a common lake level such as elevation 15 or 16 ft. This would allow examination of areas of the embankment relative to each other. As observed in the Reach 2 to Reach 3 transition, Reach 3 was clearly more conductive than Reach 2 and Reach 3 experienced piping problems at a lake level of 18.5 ft whereas Reach 2 did not. Re-examination after a high water level event could reveal changes in conductivity relating to a worsening condition. Higher EM values could reflect the removal of smaller material from gap graded soils, development of fractures in the confining layer, or lengthening of existing pipes in the embankment.

2.11. Project Photographs

Photo H-2.1 Reach 1, Line 6; Active Quarry at toe of embankment.

Photo H-2.2 Recently excavated material from the quarry (March, 1998).

Photo H-2.3 John Stretch Park in Reach 2. Note large borrow excavations at toe of embankment.

Photo H-2.4 South Bay Park in Reach 3. Note yet another deep excavation at the embankment toe.

Photo H-2.5 Belle Glade Campground beyond S-351. There are permanent structures and people residing within the confines of Herbert Hoover Dike.

Photo H-2.6 Belle Glade Campground, looking north.

Photo H-2.7 Pahokee Marina in Reach 1 looking west across the lake toward the western portion of the embankment.

Photo H-2.8 C-10A Construction as a portion of the culvert structure repair effort. Note width of the embankment.

Photo H-2.9 Typical Drilling operation along embankment toe adjacent to Pahokee Airport (Reach 1, Line 10, booring 10C).

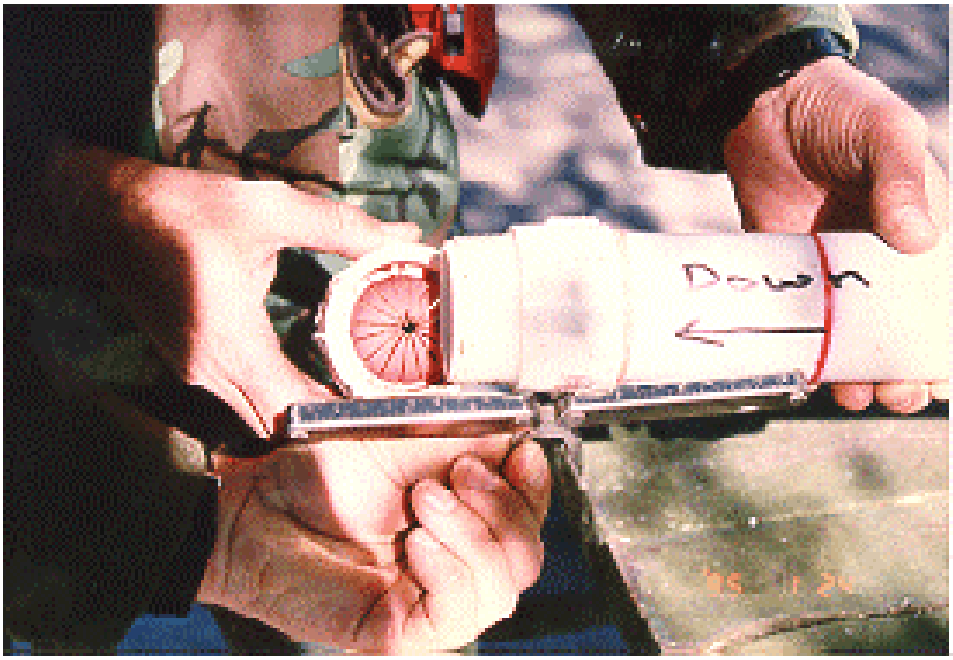


Photo H-2.10 Filtercake sampling; Push apparatus

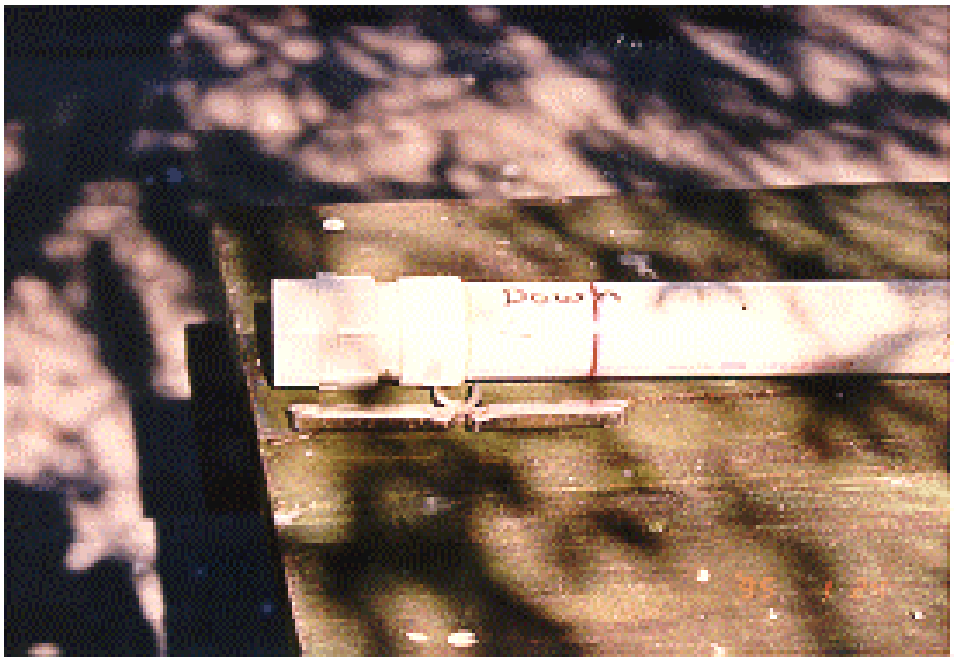


Photo H-2.11 Filtercake sampling; Push apparatus assembled

Photo H-2.12 Filtercake sampling; Grab sampler with sample. Note consistency of sample.

Photo H-2.13 Filtercake sampling; Push sampler with GPS unit on board.

Photo H-2.14 Filtercake sampling; Push sample recovered. Note consistency of sample.

3. LABORATORY TESTING.

3.1. General. The soil properties used for this study came from a coordination of test results of undisturbed and disturbed soil and rock samples. The sample were obtained and tested in 1960's, when the existing embankment was raised, in 1980's, as part of the embankment improvements and in 1995 and 1996 as part of this study.

3.2. Disturbed Samples.

3.2.1. Disturbed samples consist of material obtained from core borings, grab samples of filter cake material from the lake side borrow canal and bag samples of embankment and pipe material. The following tests were performed on representative samples from the core borings and on the grab samples: gradation, water content, Atterberg Limits and specific gravity. Loss on ignitions tests, to test for organic content of the soil sample. Organic matter effects the soil strengths and permeability. Acid leaching tests were performed to determine the mineral content of the soil sample whether the soil was composed of shell or quartz particles.

3.2.2. Bag samples consisting of shell sand and quartz sand were collected from the embankment and from material that piped during the 1995 high water event. Gradation and acid leaching tests were performed on these samples. Test results from gradation and acid leaching are included in section 3.5.

3.2.3. Permeability tests were performed on the shell sand and poorly graded quartz sand samples. As water flows through soil, certain amounts of water are stored or retained within the soil structure. The amount of water stored or retained is a function of the pore-water pressure and the characteristics of the soil structure. Permeability of a soil depends on number of factors, grain size, void ratio, and the shapes and arrangement of soil particles. Shell sand particles are flat and plately in shape and the alignment of these particles will affect the permeability of the soil. Vertical permeability and heaving tests were performed on the shell sand and quartz sand samples. The purpose of the tests was to determine at what head will cause the material to boil and at what head will cause piping of the sand. The sand samples were molded and compacted into 6-inch molds at various densities. Water was forced up through the sand. The time and head at which material started to be removed from the sample was recorded.

3.3. Undisturbed Samples.

3.3.1. General: Undisturbed Shelby Tube samples of the peat, silt, clay and other organic soils were obtained. These samples were obtained in the 1980's and in the 1995 and 1996. The samples were tested for specific gravity, LOI, Atterberg Limits, water content, gradation, permeability, direct shear and Triaxial test for both "Q" and "R" bar tests. Permeability test was performed on the peat in both vertical and horizontal direction.

3.3.2. Pinhole Test: Pinhole erosion tests were performed on peat, a silty, shelly clay and sandy silt samples obtained from undisturbed samples. The purpose of the test was to determine if the soil was erodible. Samples were cut from undisturbed samples, placed in the pinhole erosion apparatus and a small diameter hole was punched through the sample. Water under low hydraulic head was then force through the hole at a given rate for different time interval. Sample was weight before and after each test to determine weight loss. The test results showed that the peat and the clay samples were non-dispersive. However, the sandy silt was highly erodible.

3.4. Rock Core Samples. Rock core samples of the limestone and sandstone layers were obtained in 1995 and 1996. Vertical permeability tests were performed on samples cut from the core parallel to the axis of the core. Horizontal permeability tests were preformed on samples cut perpendicular to the axis of the core. The core samples were obtained from the more competent rock zones and may not be representative of the actual permeability of the intact rock.

3.5 Test Results.

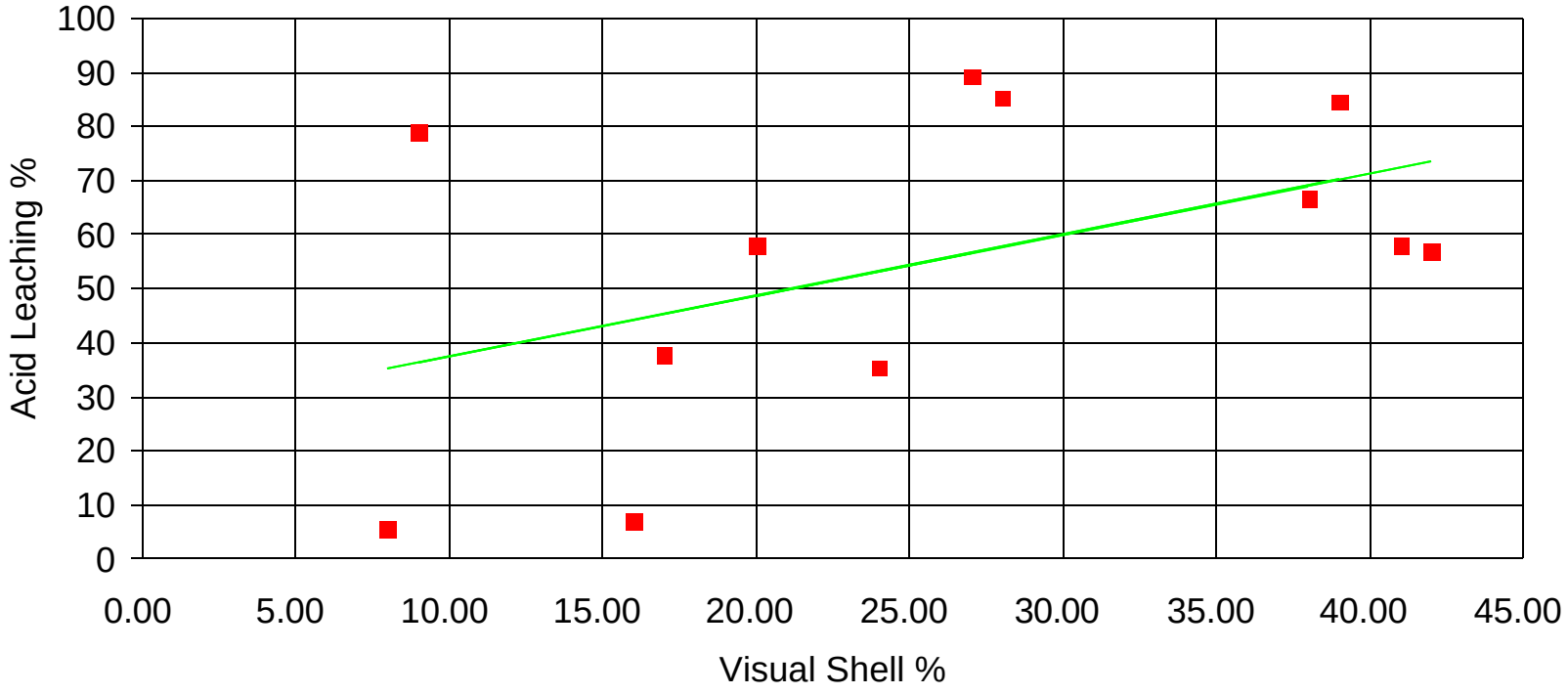
Gradation Data

Boring	Sample	Depth	Elevation	Lab ID #	Classif.	%Pass200	Vis. Shell	S.G.	Acid Leach.
Clean Sands									
CB-HHDR1-1C	#34			73/6903	SC	35			
CB-HHDR1-3C	#26			73/6904	SM	17			
CB-HHDR1-6C	#30			73/6905	SP-SM	12			
CB-HHDR1-7A	#25	44.2-45.7		73/6906	SP-SM	9			
CB-HHDR1-8B	#50	56.0-57.5	-17.2--18.7	73/6907	SP-SM	7			
CB-HHDR1-8C	#26			73/6908	SM	12	17.00	2.70	37.60
CB-HHDR1-10C	#28,29			73/6909	SP-SM	9	41.00	2.73	57.80
CB-HHDR1-11D	#26,27			73/6910	SP-SM	10	42.00	2.72	56.70
CB-HHDR1-14C	#23			73/6911	SC	29			
CB-HHDR1-15D	#25,26			73/6912	SM	12			
Lower Levee									
CB-HHDR1-1B	#16			73/6913	SM	24	28.00	2.77	85.30
CB-HHDR1-3B	#16			73/6914	ML	70			
CB-HHDR1-5B	#15			73/6915	ML	60			
CB-HHDR1-7B	#15	21.0-22.5		73/6916	SC	30	9.00	2.72	78.90
CB-HHDR1-7B	#20	28.5-30.0		73/6917	CL	62			
CB-HHDR1-8B	#25	21.0-22.3	+17.8-+16.5	73/6918	SP-SM	6	27.00	2.76	89.10
CB-HHDR1-9B	#24	21.0-22.5	+17.4-+15.9	73/6919	ML	68			
CB-HHDR1-10B	#16	21.5-23.0		73/6920	SC	37	20.00	2.69	57.90
CB-HHDR1-12B	#15			73/6921	SC	18			
CB-HHDR1-14B	#15			73/6922	CL	73			
Fines									
CB-HHDR1-1D	#9,10			73/6923	SM-SC	48			
CB-HHDR1-3C	#9			73/6924	MH	93			

CB-HHDR1-7A	#6	7.5-9.0		73/6925	ML	52			
CB-HHDR1-7A	#18	32.7-34.2		73/6926	SM	13			
CB-HHDR1-10A	#12	13.3-15.0	+0.6--1.1	73/6930	CL	80			
CB-HHDR1-11A	#10	13.7-15.0	0.0--1.3	73/6931	CL	79			
CB-HHDR1-15A	#10	10.5-13.5	+4.0-+1.0	73/6932	CH	61			
CB-HHDR1-15D	#7			73/6933	CH	88			
Soft (low blow counts)									
CB-HHDR1-3A	#2,3,4			73/6934	SM-H	50			
CB-HHDR1-5A	#2			73/6935	CL	94			
CB-HHDR1-7A	#1	0.0-1.5		73/6936	SM	39	16.00	2.68	7.00
CB-HHDR1-7A	#3	3.0-4.5		73/6937	OH	38			
Rock (high blow counts)									
CB-HHDR1-1A	#13			73/6938	SM	21	3.00		
CB-HHDR1-2D	#22			73/6939	CL	68			
CB-HHDR1-4C	#24			73/6940	SM	14			
CB-HHDR1-5A	#19			73/6941	GC	15			
CB-HHDR1-8B	#43	42.0-42.9	-3.2--4.1	73/6942	SP-SM	9			
CB-HHDR1-12D	#27			73/6943	SM	20			
CB-HHDR1-15C	#15			73/6944	SC	20			
Filtercake									
CB-HHDR1-5A	#4 push			73/6945	MH	94			
CB-HHDR1-7A	#3	9.1		73/6946	MH	89			
CB-HHDR1-7A	#5 push			73/6947	MH	98			
CB-HHDR1-8A	#6	12.2		73/6948	OH	87		2.38	
CB-HHDR1-9A	#6	12.3		73/6949	CH	99			
CB-HHDR1-11A	#6	13.5		73/6950	ML	67		2.41	
Peat									

CB-HHDR1-2D	#2			73/6951	CH	99			
CB-HHDR1-8B	#34	31.5-33.0	+7.3-+5.8	73/6952	OH	75			
CB-HHDR1-9A	#5	4.5-8.3	+8.6-+4.8	73/6953	OH	61			
CB-HHDR1-9B	#33	28.5-30.0	+9.9-+8.4	73/6954	PT	40			
CB-HHDR1-14C	#7			73/6955	PT	76			
Additional									
CB-HHDR1-7B	#22	31.5-33.0		73/6956	PT	47		1.67	
CB-HHDR1-7B	#25	42.5-44.0		73/6957	SM	15			
CB-HHDR1-7B	#30	50.0-51.5		73/6958	SM	20			
CB-HHDR1-7B	#32	57.3-58.8		73/6959	SP-SM	9	8.00		
CB-HHDR1-10B	#20	26.0-27.5		73/6960	SP	1	8.00	2.65	5.60
CB-HHDR1-10B	#25	30.5-32.0		73/6961	PT	39		1.60	
CB-HHDR1-10B	#29	35.0-36.5		73/6962	ML	81			
CB-HHDR1-10B	#30	36.5-38.0		73/6963	ML	79			
CB-HHDR1-10B	#34	42.5-44.0		73/6964	SM	22	38.00		66.46
CB-HHDR1-10B	#39	56.0-57.5		73/6965	SP-SM	10			66.46
Bag Samples									
CB-HHDR1-10B		41.0-45.0		73/6966	SM	22		2.83	44.00
CB-HHDR1-10B		50.0-59.0		73/6967	SP-SM	7			37.00

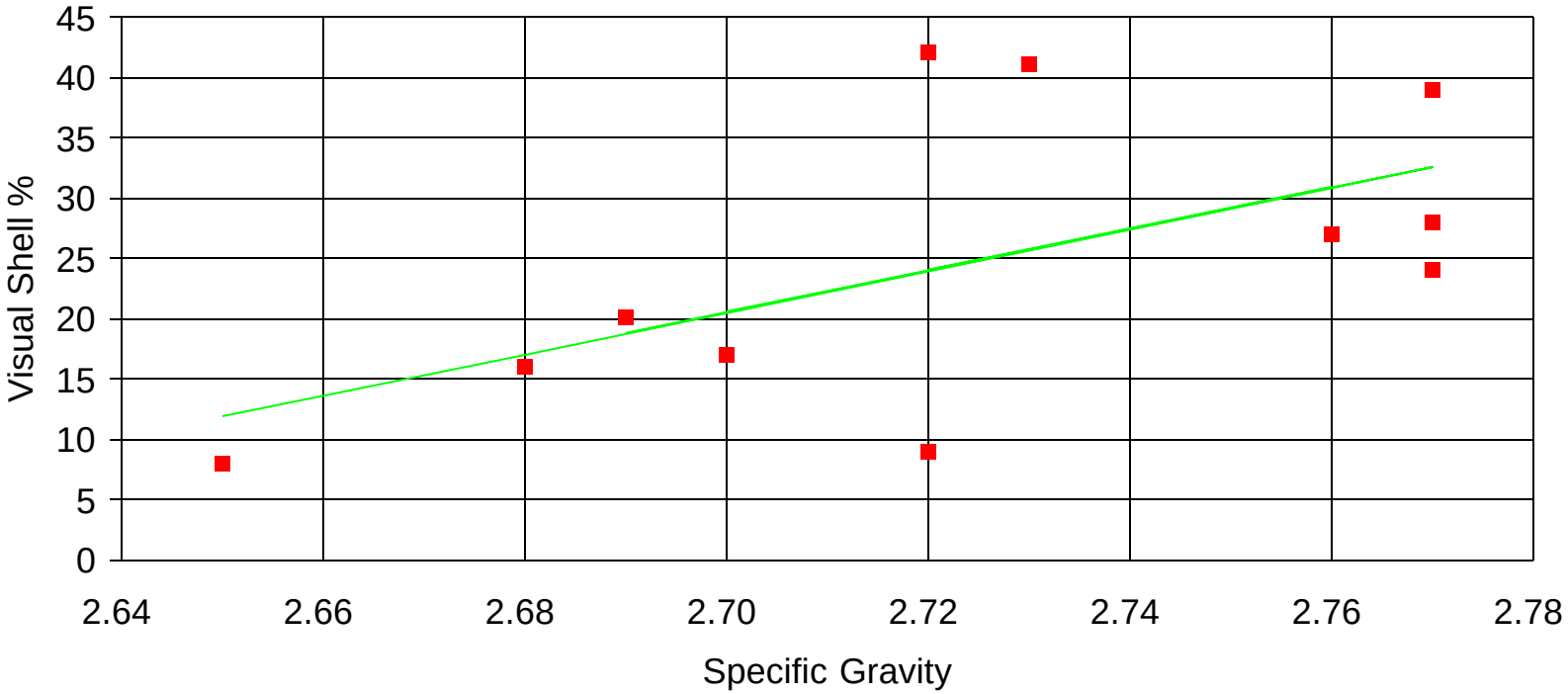
Graph SHL-LEA



Original Data

			Graph name: shl-lea.cgm						
Boring	Classification	Vis. Shell	Acid Leaching	Regression					
CB-HHDR1-8C	SM	17.00	37.60	45.39			Regression Output:		
CB-HHDR1-10C	SP-SM	41.00	57.80	72.25		Constant			26.3583
CB-HHDR1-11D	SP-SM	42.00	56.70	73.37		Std Err of Y Est			26.575846
CB-HHDR1-1B	SM	28.00	85.30	57.70		R Squared			0.2245251
CB-HHDR1-7B	SC	9.00	78.90	36.43		No. of Observations			12
CB-HHDR1-8B	SP-SM	27.00	89.10	56.58		Degrees of Freedom			10
CB-HHDR1-10B	SC	20.00	57.90	48.74					
CB-HHDR1-8A	CH	24.00	35.30	53.22		X Coefficient(s)		1.1192893	
CB-HHDR1-9A	SC	39.00	84.50	70.01		Std Err of Coef.		0.66	
CB-HHDR1-7A	SM	16.00	7.00	44.27					
CB-HHDR1-10B	SP	8.00	5.60	35.31					
CB-HHDR1-10B	SM	38.00	66.46	68.89		Corr Coeff.	0.4738408		

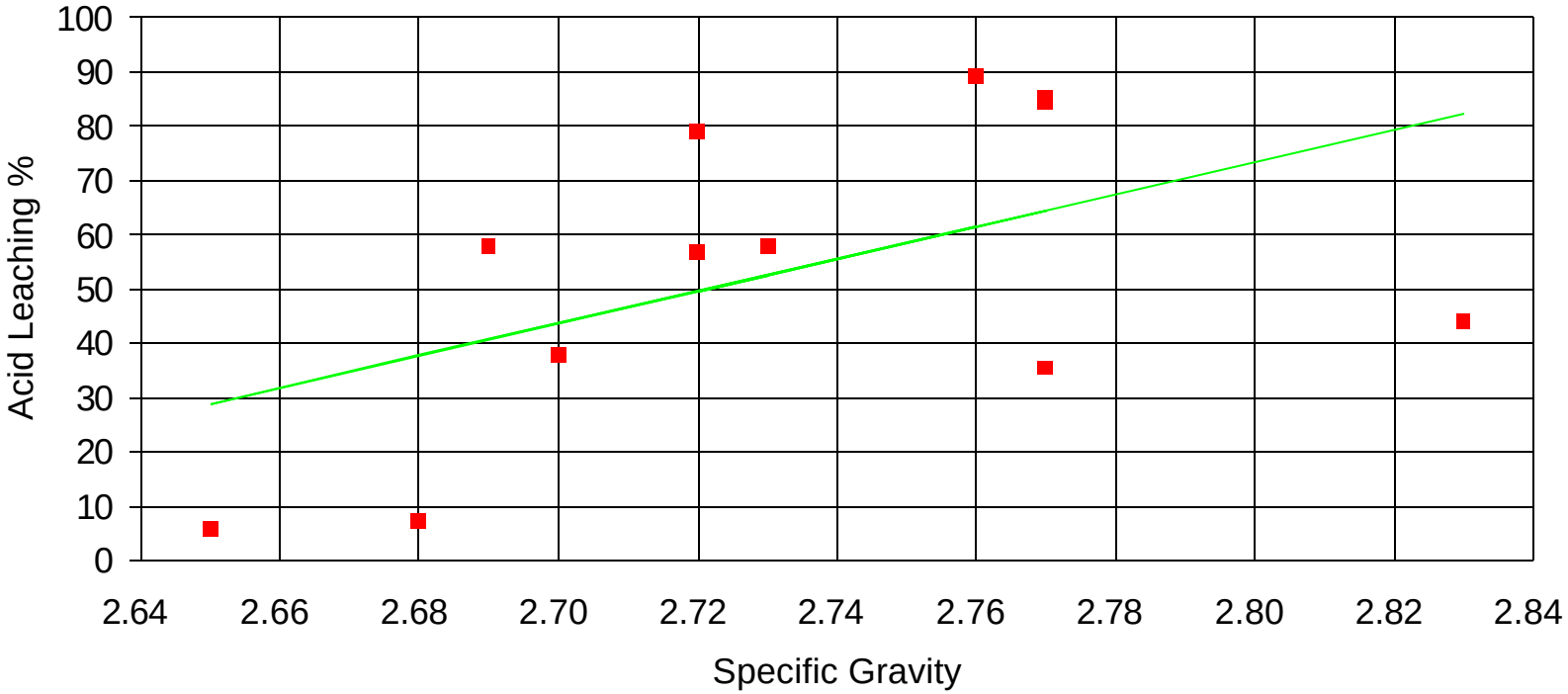
Graph SHL-SG.CGM



Original Data

			Graph Name : shl-sg.cgm						
Boring	Classification	Vis. Shell	S.G.	Regression					
CB-HHDR1-8C	SM	17.00	2.70	20.55			Regression Output:		
CB-HHDR1-10C	SP-SM	41.00	2.73	25.74		Constant			-446.34
CB-HHDR1-11D	SP-SM	42.00	2.72	24.01		Std Err of Y Est			10.35
CB-HHDR1-1B	SM	28.00	2.77	32.65		R Squared			0.34
CB-HHDR1-7B	SC	9.00	2.72	24.01		No. of Observations			11.00
CB-HHDR1-8B	SP-SM	27.00	2.76	30.92		Degrees of Freedom			9.00
CB-HHDR1-10B	SC	20.00	2.69	18.82					
CB-HHDR1-8A	CH	24.00	2.77	32.65		X Coefficient(s)		172.9234	
CB-HHDR1-9A	SC	39.00	2.77	32.65		Std Err of Coef.		79.74126	
CB-HHDR1-7A	SM	16.00	2.68	17.09					
CB-HHDR1-10B	SP	8.00	2.65	11.90		Corr Coeff.	0.585826		

Graph SG-LEA.CGM

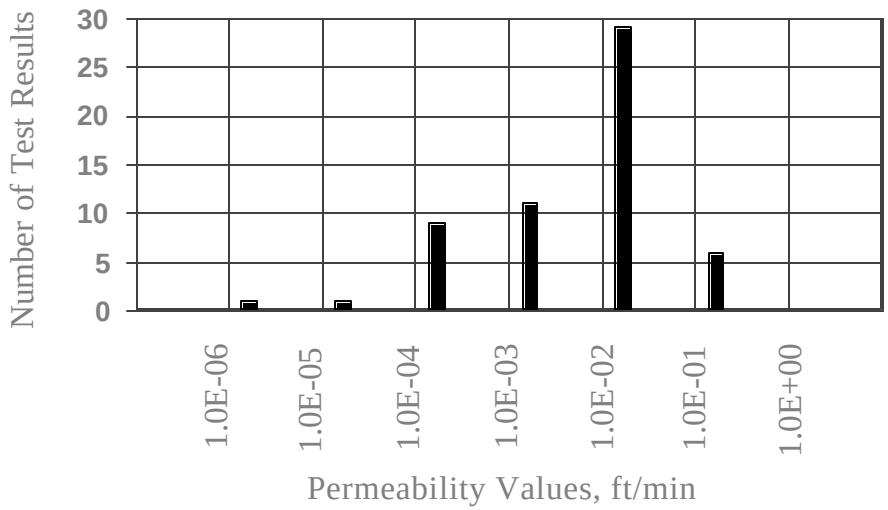


Original Data

			Graph Name: sg-lea.cgm						
Boring	Classification	S.G.	Acid Leaching	Regression					
CB-HHDR1-8C	SM	2.70	37.60	43.66			Regression Output:		
CB-HHDR1-10C	SP-SM	2.73	57.80	52.57		Constant			-757.63
CB-HHDR1-11D	SP-SM	2.72	56.70	49.60		Std Err of Y Est			25.82
CB-HHDR1-1B	SM	2.77	85.30	64.44		R Squared			0.26
CB-HHDR1-7B	SC	2.72	78.90	49.60		No. of Observations			12.00
CB-HHDR1-8B	SP-SM	2.76	89.10	61.47		Degrees of Freedom			10.00
CB-HHDR1-10B	SC	2.69	57.90	40.70					
CB-HHDR1-8A	CH	2.77	35.30	64.44		X Coefficient(s)		296.7769	
CB-HHDR1-9A	SC	2.77	84.50	64.44		Std Err of Coef.		156.4703	
CB-HHDR1-7A	SM	2.68	7.00	37.73					
CB-HHDR1-10B	SP	2.65	5.60	28.82					
CB-HHDR1-10B	SM	2.83	44.00	82.24		Corr Coeff.	0.5143623		

4. DATA EVALUATION AND STATISTICS.

4.1. Detailed Review of Recharge Tests.



4.2. Filtercake Evaluation. This material is that deposited on the rim canal bottom and slopes. It is a very soft silt or clay. There were three lab tests done on this material which gave good agreement. Horizontal and vertical permeability values are expected to be the same due to the young age of the deposit and it's very loose state.

	Filtercake All	Filtercake Final
Expected Value	3.21E-05	3.00E-05
Std. Dev	3.04E-05	3.00E-05
V %	95	100

4.3. Statistics on Permeability Of Soils. These values are based on the compilation of lab data for Reach 1. Some adjustment in Expected Value value for localized features may be necessary for model calibration since these values represent the "general" condition for that material zone to be used in the model. Localized conditions or anomalies will likely be the controlling factor in the model. Unless stated otherwise, all values are in feet per minute. V% is the coefficient of variation, defined as the Expected Value value divided by the standard deviation.

4.3.1. Peat. Recharge and lab tests on peat revealed a significant difference between the horizontal and vertical permeability values. The recharge tests represent the horizontal

permeability values and the lab tests represent the vertical permeability values. Separating the two permeability values and culling two outlying points in the vertical testing (CB-HHDR1-2C and PZ-R1-4C-2) allowed for reasonable lognormal distributions. Note that the "cull" designation indicates where suspect data were not included.

	Peat All	Peat All Vert.	Peat Vert. Cull	Peat Horiz.
Expected Value	4.74E-04	6.77E-04	5.85E-05	2.31E-04
Std. Dev	7.51E-04	1.00E-03	9.61E-06	1.49E-04
V %	158	148	16	65

The final values selected are shown below.

	Peat Vert.	Peat Horiz.
Expected Value	6.00E-05	2.00E-04
Std. Dev	5.00E-05	1.50E-04
V %	83	75

The expected value for both vertical and horizontal permeability are of a reasonable order of magnitude. The standard deviation for the vertical permeability was adjusted higher to accommodate a broader range of uncertainty since there were only four test values to provide data points. We would expect that with a larger sample size the V % would increase to be more in line with that of other materials tested, which was around 100%. Also - the spreadsheet developed by Wolff enables one to "see" the shape of the distribution. This shape can be adjusted by varying the value of standard deviation to give a better shaped lognormal distribution curve.

4.3.2. Organic Silts and Clays. These organic fines are located above the inorganic silts and clays. These are the products of decomposed peat, and are co-mingled with the peat in some areas. The test results are somewhat misleading. Two of three of the recharge tests (Kh) were taken in the same boring. The lower test likely was picking up some influence from the upper limestone and the upper test could have intercepted some damaged area (low blow count, N=2). For whatever reasons, these values are about one order of magnitude higher than expected. One would expect the more the organic material decomposes, the permeability values would be lower, not higher. The vertical permeability values (lab tests) show two tests of 10-5 and one at 10-3. The results of culling the 10-3 test are shown below.

	OL / OH Vertical	OL / OH Horizontal	OL / OH Vertical Cull
Expected Value	7.45E-04	2.11E-03	6.35E-05

Std. Dev.	8.86E-03	8.86E-04	2.33E-05
V %	158.5	42.0	36.6

Due to the amount of co-mingling with peat and the difficulty in interpreting field classifications, it was decided not to separate OL/OH from peat in the models. Since the permeability values are expected to be similar (same order of magnitude), this should not adversely impact the models. It was also felt the uncertainty in separating OL / OH layers from peat given the limited specific information (field visual classifications) would be futile.

4.3.3. Upper Sands. These soils are non-gravel dike fill and sands above the lower limestone unit and include material classified as SP, SM or SP-SM. There is so much interbedding in the foundation and mixing in the embankment these USCS types were grouped as a unit to simplify modeling. Lab values were in good agreement with published typical values (TM 5-818-5, Table 3.4 [US Army, 1983]). Horizontal and vertical permeability values are expected to be the same. Recharge test permeability values were significantly lower. This is likely due to the type of test rather than layering effects of the soils.. Culling CB-12-2 and CB-HHDR1-10C (both in the 10-5 order of magnitude) gave a reasonable result for the combined values. The final Expected Value value was lowered slightly as a starting value for the models. The standard deviation was also adjusted to give a more reasonable shape to the assumed lognormal distribution.

	Upper Sands	Upper Sands	Upper Sands	Upper Sands	Upper Sands
	All	Recharge	Lab	All Cull Final	
Expected Value	1.77E-03	3.91E-04	5.21E-03	2.45E-03	2.00E-03
Std. Dev.	2.89E-03	6.04E-04	3.91E-03	3.24E-03	2.00E-03
V %	163	154	75	132	100

4.3.4. Lower Sands. These are sands below the lower limestone unit and include material classified as SP, SM or SP-SM. Again, there was significant interbedding and the USCS types were grouped as a unit to simplify modeling. There was such a variation in test results that there were not just one or two outlying data points which could be omitted. A check with recognized values (TM 5-818-5, Table 3.4 [US Army, 1983]) revealed the Expected Value was a good estimate. Horizontal and vertical permeability values are expected to be the same. The standard deviation was adjusted lower to give a better fit to the assumed lognormal distribution. This was done to limit the amount of variation in the model for this material. Since this was not a "critical" material, adding unnecessary uncertainty to this part of the model was unnecessary.

	Lower Sands	Lower Sands
	All	Final
Expected Value	6.04E-03	6.00E-03

Std. Dev.	9.65E-03	6.00E-03
V %	160	100

4.3.5. Gravel and Shell. There was only one recharge test value in clean gravel or shell zone in Reach 1. TM 5-818-5, Table 3.4 was consulted again for a reasonable value. Horizontal and vertical permeability values are expected to be the same. Note that the designation as "clean" does not imply only gravel size particles. A small sample of the material may classify as a GP, but over a larger area (a few hundred feet) it is expected the Expected Value property would be more like that of a sandy gravel. The concern here besides higher permeability is the internal instability of this material. Piping of matrix material of sandy gravels has been documented in research and field observations. The removal of even a small amount of matrix material could drastically increase the permeability of this zone. Horizontal and vertical permeability values are expected to be the same. The position of the gravel layer was estimated either through core boring information or adjusted to obtain calibration with piezometer readings. Geophysical data (ground penetrating radar) and historical information of construction methods (hydraulic fill) can provide insights to the nature and extent of this feature. This is especially important when trying to explain consistently high readings (pz reading lake level)in the center of the embankment (pz reading changing consistently with lake level).

	Gravel / Shell Rech. Test	Gravel / Shell Final
Expected Value	1.85E-02	1.00E00
Std. Dev.	N/A	1.00E00
V %	N/A	100

4.3.6. Silts and Clays. These materials are the inorganic fines located below the peat and the upper limestone (Lake Flirt Marl). This material could also be considered marl since it is a mix of calcareous materials, primarily passing the # 200 sieve but also including traces and lenses of larger particle sizes up to and including gravels. These discontinuities and the fact that all the material classified (field and lab) as ML or MH likely account for the permeability values about one order of magnitude higher than published typical values.

	ML / MH Vertical	ML / MH Horizontal	ML / MH Combined
Expected Value	1.10E-04	1.52E-04	1.31E-04
Std. Dev.	1.30E-04	9.45E-05	1.04E-04
V %	118.0	62.4	79.6

Final Values were selected from the actual statistical values from the data. Since these values were four orders of magnitude lower than the underlying limestone there was no incentive to "fine tune"

the expected values or standard deviation. The values were separated as horizontal and vertical permeability values in the first models, but will likely be combined in subsequent models.

	ML / MH Vertical	ML / MH Horizontal	ML / MH Combined
Expected Value	1.10E-04	1.52E-04	1.30E-04
Std. Dev.	1.30E-04	9.45E-05	1.00E-04
V %	118	63	77

4.4. Statistics On Permeability Of Rock. Permeability values (expected value and coefficient of variation) for the limestone used in the models are a composite based on data for lower limestone, upper limestone (Lake Flirt Marl) associated with the peat and any formations encountered as sandstone. Refer to Table H-4.1 for a summary. Table H-4.1 is divided into three groups of permeabilities measured by test type. Recharge testing was accomplished using the USBR Method 7310 procedure for Constant Head Hydraulic Conductivity Tests in Single Drill Holes. Laboratory testing was done in accordance with (get specifics from Coy) performed by SAD lab. The USGS tests results were obtained from the USGS report "Seepage beneath Hoover Dike, Southern Shore of Lake Okeechobee, Florida. These values were obtained from both slug tests and pump tests. Dye tests were performed by WES personnel in September, 1995 at several locations in Reach 1.
The statistics were based on several different groupings of data. The groupings are as follows:

"All" - includes all data from all reaches.

"R1 Only" -is for tests performed only in Reach 1.

"R1- Cull" - is for tests performed only in Reach 1 where four suspect test results were omitted. These were tests at CB-14-3, el. -5.5; CB-HHDR!-18C, el. -8; CB-HHDR!-8C, el. -21.4; CB-12A-3, el. -38.0.

"SS" - presents statistics on rock units identified as Sandstone.

"LS-P" - presents statistics on rock units identified as limestone associated with the base of the peat, also known as Lake Flirt Marl.

"All-H" or "All - V" - includes all data from all reaches tested in the horizontal (H) or vertical (V) directions..

"R1-H or R1-V" - includes all data from all reaches tested in the horizontal (H) or vertical (V) directions.

"All -kH/V" - horizontal to vertical permeability ratio for all tests.

"R1 - kH/V" - horizontal to vertical permeability ratio for tests only in Reach 1.

"R1 Cull - kH/V" - horizontal to vertical permeability ratio for tests only in Reach 1 with two outlying test results culled (CB-HHDR1-10B and CB-HHDR!-7A).

"R1 - Slug" - USGS slug tests in Reach 1.

"R1 - Pump" - USGS pump tests in Reach 1.

Observations by Wolff (fax 19 Dec 96) regarding the data are as follows:

1. Each of the data sets (recharge, lab H, lab V and USGS) had a coefficient of variation from about 70 - 110 % (except the SS) and significantly different mean values

2. The smallest mean values (0.0035) were found on the lab V tests. These tests are likely biased since they are made on intact, coreable samples. These are much less permeable than larger rock masses containing joints or other conductive features for which cores cannot be obtained.

3. The next largest mean values (0.0085) were on the lab H samples. This is logical since horizontal permeability is often considered greater than vertical.

4. The next largest mean values (0.045) were results from the recharge tests. These are an order of magnitude higher, and are likely associated with much larger regions than the intact lab samples. These have a higher likelihood of intersecting water bearing joints or lenses and exhibit a higher permeability. This suggests that the permeability of the formation has a higher permeability than the intact core samples tested in the lab.

5. The largest mean values (0.778) are those of the USGS tests, which are another order of magnitude higher. For those that are pump tests they would influence a larger region than the recharge tests.

(Note from DiPiero) The slug tests may be higher on account of the fact that many values were estimated and the USGS indicated that the "analysis of slug tests generally required a great deal of subjective judgement". Therefore, this Expected Value is based on judgement as well as data.

The rock units classified as sandstone (SS) intermingle with the limestone. The main difference is the varying sand content, the sandstone being "sandier" than the limestone. Their permeability values as measured by recharge testing are similar to each other so they are grouped together with limestone. The coefficient of variation is much less than that for the limestone values. This is to be expected since sandstone was encountered at only three sites (C-10A, C-11 and C-12A). Multiple tests were taken at the same site which represents a smaller sample area. The upper limestone unit (LS-P) also has a permeability similar to the lower limestone and therefore can also be grouped together

from a permeability standpoint. It is our opinion that for modeling purposes, all the rock units may be evaluated as a single unit Typical permeability values for the rock layer as a whole would be:

Expected value:	0.25 ft/min
Std. Dev.:	0.20 ft/min
V_k:	80%
Max.:	1.00 ft/min
Min.:	1×10^{-6} ft/min

Table H-4.1. Rock Permeability Statistical Summary

Statistic	Recharge All Data	Recharge R1 Only	Recharge R1 - Cull	Recharge SS	Recharge LS-P
Expected Value	4.13E-02	4.78E-02	4.61E-02	2.20E-02	3.96E-02
Std. Dev.	4.38E-02	4.59E-02	4.35E-02	1.44E-02	3.49E-02
V %	106.00	96.14	94.42	65.02	88.09
Max.	1.70E-01	1.70E-01	1.46E-01	4.58E-02	8.05E-02
Min.	5.77E-06	1.85E-04	1.85E-04	7.39E-03	8.55E-04

Statistic	Lab All - H	Lab All - V	Lab R1 - H	Lab R1 - V
Expected Value	8.52E-03	3.45E-03	1.01E-02	3.40E-03
Std. Dev.	9.29E-03	2.92E-03	9.43E-03	3.03E-03
V %	109.08	84.42	93.49	89.23
Max.	2.53E-02	6.75E-03	2.53E-02	6.75E-03
Min.	1.15E-05	1.48E-06	1.15E-05	1.48E-06

Statistic	Lab All -kH/V	Lab R1 - kH/V	Lab R1 Cull - kH/V
Expected Value	9.45E+00	1.13E+01	2.63E+00
Std. Dev.	1.87E+01	2.01E+01	1.35E+00
V %	197.80	177.72	51.33
Max.	6.35E+01	6.35E+01	5.06E+00
Min.	1.14E-01	4.86E-01	4.86E-01

Statistic	USGS All	USGS R1 Only	USGS All Slug	USGS All Pump
Expected Value	7.78E-01	8.88E-01	5.67E-01	1.41E+00

Std. Dev.	6.25E-01	8.98E-01	3.91E-01	9.49E-01
V %	80.39	101.07	69.02	67.23
Max.	2.08E+00	2.08E+00	1.04E+00	2.08E+00
Min.	4.58E-02	4.58E-02	4.58E-02	7.41E-01

Statistic	Dye Tests
Expected Value	1.18E+00
Std. Dev.	3.88E-01
V %	32.91
Max.	1.84E+00
Min.	6.40E-01

4.5. Statistic on R, R-Bar, Phi, Theta. These values are based on the compilation of all lab data. Some adjustment in Expected Value value for localized features may be necessary for model calibration since these values represent the "general" condition for that material zone to be used in the model. Localized conditions or anomalies will likely be the controlling factor in the model. There are three parameters to vary for each soil group, phi, c and unit weight. Due to the low V% (less than ~ 30%), phi and unit weight are assumed to be normally distributed. Cohesion will be assumed to be lognormally distributed. In cases where V% is greater than 30%, a normal distribution of the variable can imply negative values. Negative values of cohesion would be meaningless in the slope stability models. Lognormal and normal distributions are similar when V% is small (less than 30%).

Materials that will not be a factor in the stability analysis will not be entered as random variables. For example, the lower limestone layer and soils below this layer since this represents the "firm base". Gravel pockets in the embankment lakeside and filter cake (obviously) will not be a factor in landside failures. This leaves three parameters for each soil type as random variables, as well as tailwater elevation. Units are degrees, tons per square foot for cohesion and pounds per cubic foot for unit weight.

4.5.1. Upper Sands. These soils are non-gravel dike fill and sands above the lower limestone unit. Material classified as SP, SM, SC or SP-SM. There was so much interbedding in the foundation and mixing in the embankment these USCS types were grouped as a unit to simplify modeling. Lab values were in good agreement with recognized values.

	Sands R, phi / c	Sands S, phi / c
Expected Value	23.2 / 0.37	40.6 / 0
Std. Dev.	4.1 / 0.34	3.6 / 0
V %	17.6 / 91.9	8.9 / 0

The final values selected are shown below.

	Sands R, phi / c	Sands S, phi / c
Expected Value	23.0 / 0.37	36.0 / 0
Std. Dev.	2.3 / 0.37	3.6 / 0
V %	10 / 100	10 / 0

The expected values for R were selected as close to that predicted by the lab data. It was felt the value of 40 deg. was too high for all materials, therefore this was reduced. The std dev was adjusted based on a more regular V% value as suggested by Harr. S cohesion will not be a variable.

Unit weight for sands will vary within the embankment due to the heterogeneity. However, the change in unit weight should be captured in the values selected for a saturated material.

	Lab Data Sands	Selected Values Sands
Expected Value	114.4	115
Std. Dev.	10.6	5.75
V %	9.3	5

The expected value was selected based on the lab data as well as "book" values. For example, if sands can range from 100 to 125 pcf, then 112.5 pcf could be an Expected Value value. The std dev was adjusted down toward the V of 3% as suggested by Harr.

4.5.2. Peat and Organic Soils. These soils were grouped together for two main reasons. One, the organic layer below the embankment is a combination of natural deposit, fill as well as decomposed peat which approaches the OL or OH classification. Separating zones would be impractical due to lack of sufficiently accurate field classification data. Also - even if it were separated it is doubtful the results would be much different.

	Peat R, phi / c	Peat S, phi / c	OL / OH R, phi / c	OL / OH S, phi / c
Expected Value	16.4/0.28	31.5/0.18	19.2/0.16	35.3/0.06
Std. Dev.	5.1 / 0.22	9.1 / 0.18	3.9 / 0.16	8.9 / 0.09
V %	31.1 / 78.6	28.9 / 100	20.3 / 100	25.2 / 150

Combined Parameters are as follows:

	PT / OL / OH R, phi / c	PT / OL / OH S, phi / c
Expected Value	17.8 / 0.22	33.4 / 0.12
Std. Dev.	4.7 / 0.20	9.0 / 0.15
V %	26.4 / 90.9	26.9 / 125

The final values selected are shown below.

	PT / OL / OH	PT / OL / OH
	R, phi / c	S, phi / c
Expected Value	18 .0/ 0.22	33.0 / 0.12
Std. Dev.	4.5 / 0.22	8.25 / 0.12
V %	25 / 100	25 / 100

The expected values were selected as close to that predicted by the lab data. The std dev was adjusted based the lab data to a more regular V% for both phi and c.

Unit weight for peat and organic soils will also vary heterogeneity. However, the change in unit weight should be captured in the values selected for a saturated material.

	Lab Data	Selected Values
	PT / OL / OH	PT / OL / OH
Expected Value	66.0	70.0
Std. Dev.	10.5	7.0
V %	15.9	10

The expected value was selected based on the lab data and judgment. It was felt the expected value would be slightly higher than the lab data suggests. Also, the V% is expected to be larger, also based on judgment due to the variability of organic soils.

4.5.3. Silts and Clays. These soils were grouped together since their strength properties were similar. Also, as with the peat and organic soils, separating zones would be impractical due to lack of sufficiently accurate field classification data. And it is also doubtful the results would be different if separation were attempted.

	ML / MH	ML / MH	CL / CH	CL / CH
	R, phi / c	S, phi / c	R, phi / c	S, phi / c
Expected Value	12.2 / 0.40	34.0 / 0.20	13.2 / 0.32	32.9 / 0.1
Std. Dev.	2.8 / 0.20	6.5 / 0.10	5.0 / 0.10	10.3/0.13
V %	22.9 / 50.0	19.1 / 50.0	37.8 / 31.3	31.3 / 130

Combined Parameters are as follows:

	ML / MH	ML / MH
	CL / CH	CL / CH
	R, phi / c	S, phi / c
Expected Value	12.6 / 0.39	33.6 / 0.13
Std. Dev.	2.9 / 0.18	7.6 / 0.11
V %	23.0 / 46.2	22.6 / 84.6

The final values selected are shown below.

	ML / MH CL / CH R, phi / c	ML / MH CL / CH S, phi / c
Expected Value	12.5 / 0.35	33.0 / 0.10
Std. Dev.	3.1 / 0.18	8.3 / 0.10
V %	25 / 50	25 / 100

The expected values were selected as close to that predicted by the lab data. The std dev was adjusted based the lab data to a more regular V% for both phi and c. Even though V% for c is suggested at 40% and S strengths are not supposed to have cohesion. [This may need adjustment after a calibration slope stability run is made to see if the results are reasonable.]

Unit weight for peat and organic soils will also vary heterogeneity. However, the change in unit weight should be captured in the values selected for a saturated material.

	Lab Data ML / MH CL / CH	Selected Values ML / MH CL / CH
Expected Value	98.1	100
Std. Dev.	4.8	5
V %	4.9	5

The expected value was selected based on the lab data as well as published typical values. For example, if sands can range from 70 to 120 pcf, then 95 pcf could be an Expected Value value. The std dev was close to the V of 3% as suggested by Harr.

4.6. Stable / Unstable Materials. Critical gradients vary depending upon material properties. Soils can be described as internally stable or internally unstable based partially upon the concept of uniformity coefficient C_U. C_U is defined as the ratio D_{60} / D_{10} , where D_{60} is the diameter of soil particle at the 60 percent by weight point of the curve, and D_{10} is the diameter of soil particle at the 10 percent by weight point of the curve. This gives an indication to the slope of the grain size distribution curve, the higher the C_U the flatter the curve and the wider the range of particle size within the soil. Internally unstable soils are from those with a high C_U . It must be emphasized that C_U in and of itself does not determine whether a material is internally unstable. The criteria developed by Kenney and Lau (Kenney et al., 1985 & 1984) examines the whole portion of the gradation curve, not just discrete points along the curve. This is important for this project since the original embankment was constructed in part by hydraulic fill (dredging). Hydraulic placement of fill causes segregation of materials by weight at the dredge pipe discharge. Heavier (larger) particles settle out close to the pipe discharge. Finer particles move further away before settling out of suspension. Samples retrieved from the embankment reveal gradations such that the fine sand portion can move within the coarse shell or limestone fragment matrix. This phenomenon is known a

suffusion. (Kovacs 1981) This unstable material differs from other high C_U soils where a less gap-graded situation exists and the soils are self-filtering. In other words, stable soils of high C_U have sufficient particle sizes between the coarse and fine portions to where the fine sand portion is not free to move within the coarse shell or limestone fragment matrix. (Sherrard 1979)

4.7. Summary Spreadsheets for Permeability Data.

Recharge

	Actual	Actual		Actual		Method	USBR 7310		Adjusted	Adjusted	Adjusted	Adjusted	Change			
	Bottom of	Test	Hole Dia	Total	Recharge	K	Soil	Bottom of	Bottom of	Test	K	Soil	from	Drilling	Test	
Boring	Casing El.	Length, Ft	inches	Head, Ft	GPM	Ft/Min	Type	Casing El.	Hole El.	Length, Ft	ft/min	Type	Original	Agency	Date	Notes
CB-12A-2	-12	4.5	3	3	10	5.65E-02	LS/SM	-12	-15	3	7.51E-02	LS	y	COE-GH	Aug 23,93	LS w/ 2' SM
CB-12A-3	-10.5	1.7	3	29.2	28.6	3.20E-02	LS	-10.5	-12.5	2	2.89E-02	LS	y	COE - JG	Oct 25, 91	Bottom 0.2' in Shell
CB-13-3	-10.7	5	3	25.7	0.5	3.05E-04	LS	-10.7	-14.7	4	3.59E-04	LS	y	TET	AUG 27, 93	Very dense cemented shell & LS
CB-14-3	1.5	7	3	24	35.7	1.82E-02	SM / LS			1.5	5.24E-02	LS	y	COE - JG	Oct 29, 91	1.5' LS, 1' ML, 4.5' SM
CB-16-2	-1.6	10.6	3	6	0.19	2.82E-04	LS	-1.6	-12.2	10.6	2.82E-04	LS	n	TET	May 15, 94	Dense to soft chalky LS
CB-3-3	3.2	5	3	27.2	0.01	5.77E-06	LS	3.2	-1.8	5	5.77E-06	LS	n	TET	AUG 28, 93	Bottom 2' of test in SP-SM
CB-4A-2	5.9	5	3	9.4	0.4	6.68E-04	LS	5.9	0.9	5	6.68E-04	LS	n	TET	AUG 31, 93	All in LS
CB-4A-2	-8.04	5	3	9.4	0.07	1.17E-04	LS	-8	-13	5	1.17E-04	LS	n	TET	AUG 31, 93	All in LS
CB-4A-3	1.63	5	3	27.6	0.02	1.14E-05	LS	1.6	-3.4	5	1.14E-05	LS	n	TET	AUG 29, 93	All in LS
CB-GPPN-1	-1.1	4.5	3	5.5	20	6.16E-02	CH	-5.6	-7.1	1.5	1.28E-01	LS	y	COE - RG	FEB 95	
CB-GPPN-1	-7.1	6	3	3.5	20	7.85E-02	LS	-7.1	-10.1	3	1.29E-01	LS	y	COE - RG	FEB 95	All in LS
CB-GPPS-1	-1	4.5	3	5.8	20	5.84E-02	GM/LS		-5.5	4.5	5.84E-02	LS	y	COE - RG	FEB 95	GM/LS (1.0')
CB-GPPS-1	-5.5	1.5	3	5.8	15	9.12E-02	LS	-5.5	-7	1.5	9.12E-02	LS	n	COE - RG	FEB 95	All in LS
CB-GPPS-2	3.1	4.5	3	5	12	4.07E-02	GC	-3.5	-8	4.5	4.07E-02	LS	n	COE - RG	FEB 95	
CB-GPPS-2	-1.4	3.7	3	3.5	17	9.46E-02	GC	-8	-11.7	3.7	9.46E-02	LS	y	COE - RG	FEB 95	
CB-GPPS-2	-5.1	2.3	3	5	12	6.47E-02	LS	-11.7	-14	2.3	6.47E-02	LS	n	COE - RG	FEB 95	All in LS
CB-GPPS-3	3.2	4.5	3	5.5	10	3.08E-02	GC/CH	1.7	-1.3	3	4.10E-02	LS	y	COE - RG	FEB 95	GC/CH (1.0')
CB-GPPS-3	-1.3	4.5	3	5.5	20	6.16E-02	GC	-1.3	-5.8	4.5	6.16E-02	LS	y	COE - RG	FEB 95	
CB-GPPS-3	-5.8	1.5	3	4.1	17	1.46E-01	LS	-5.8	-7.3	1.5	1.46E-01	LS	n	COE - RG	FEB 95	All in LS
CB-GPPS-4	2.9	4.5	3	4.5	15	5.65E-02	CH/GC		-1.6	1.6	1.13E-01	LS	n	COE - RG	FEB 15 95	
CB-GPPS-4	-1.6	7.5	3	3	15	5.81E-02	LS	-1.6	-9.1	7.5	5.81E-02	LS	n	COE - RG	FEB 15 95	
CB-HHDR1-10B	-0.7	4.5	2.875	24.5	30	2.10E-02	SC			0	2.10E-02	LS	y	COE - RG	DEC 94 / JAN 95	SC w/ shell & LS frag.s
CB-HHDR1-10B	-4.2	4.8	5.5	25.5	60	3.17E-02	LS/SC		-10.2	4	3.58E-02	LS	y	COE - RG	DEC 94 / JAN 95	Possible void, 50% SC, 50% LS
CB-HHDR1-10B	-5.2	5	2.9	24.5	20	1.29E-02	LS	-6.2	-10.2	4	1.52E-02	LS	y	COE - RG	DEC 94 / JAN 95	Verify hole dia / all in LS, bottom of hole in GC
CB-HHDR1-18C	-5.3	3	6	5.1	7.3	2.52E-02	SP-SM	-11.4	-12.9	1.5	3.64E-02	LS	y	PSI-WE	mar 27 95	ran test thru 4' of 2" pvc welll screen
CB-HHDR1-22C	-4.9	5	3	3.8	1.4	5.78E-03	LS	-4.9	-9.9	5	5.78E-03	LS	n	PSI-WE	Apr 15 95	incomplete data ??
CB-HHDR1-25B	-8.9	4	4.5	23.5	60	4.16E-02	LS	-11.4	-12.9	1.5	7.53E-02	LS	y			
CB-HHDR1-25C	-4.6	5	4.5	4.5	2.5	7.76E-03	LS	-4.6	-9.6	5	7.76E-03	LS	n			
CB-HHDR1-26B	-16.5	5.5	4.5	25.5	77	3.95E-02	LS	-16.5	-22	5.5	3.95E-02	LS	n	PSI-		
CB-HHDR1-27B	-9.2	2.5	4.5	25	107	9.44E-02	LS	-9.2	-11.7	2.5	9.44E-02	LS	n	PSI-		exceeded pump capacity
CB-HHDR1-28B	-9.9	3	4.5	25	15	1.18E-02	SP	-9.9	-12.9	3	1.18E-02	LS	y			
CB-HHDR1-28B	-12.9	5	4.5	25	111	6.20E-02	SP/LS		-15.9	1.5	1.31E-01	LS	y			exceeded pump capacity
CB-HHDR1-2C	-16.5	5	3.5	6	0.1	2.51E-04	SM	-16.5	-21.5	5	2.51E-04	LS	y	TET	OCT 14, 94	All in SM
CB-HHDR1-3C	-18.5	5	3.5	7.5	0.4	8.02E-04	SM	-18.5	-23.5	5	8.02E-04	LS	y	TET	Oct 17, 94	Classified as SM / SP-SM w/ 100+ blows
CB-HHDR1-4C	-14.5	5.5	3.5	6.5	0.25	5.40E-04	SM / LS	-18	-20	2	1.07E-03	LS	Y	TET	Oct 18, 94	Bottom 1.5' in LS, on log as SM w/ > 100 blows
CB-HHDR1-5C	-17.7	5.3	3.5	7.8	0.1	1.85E-04	shell	-17.7	-23	5.3	1.85E-04	LS	Y	TET	Oct 19, 94	Classified as shell & SM, 3.5' in Shell
CB-HHDR1-6C	-15	5	5	9.3	50	7.27E-02	SM/SP-SM			5	7.27E-02	LS	y	COE-RG	JUL 26, 95	75 % SM and 25 % SP-SM
CB-HHDR1-6D	-17.1	5	3.5	7.1	0.2	4.24E-04	shell	-17.1	-22.1	5	4.24E-04	LS	Y	TET	Oct 25, 94	Class. as cem. sand & shell, shell & SM, 33% ea
CB-HHDR1-7B	6.7	4.5	4.5	32.5	3.5	1.62E-03	LS	6.7	2.2	4.5	1.62E-03	LS	n	COE - RG	NOV 2, 94	no change
CB-HHDR1-7B	2.2	4.5	4.5	33.5	10	4.49E-03	LS	2.2	-2.3	4.5	4.49E-03	LS	n	COE - RG	NOV 2, 94	Verify hole dia / 1.5' SC below LS, 0% rec in LS
CB-HHDR1-7B	-2.3	6	2.875	34.5	15	6.03E-03	SC	-2.3	-5.3	3	9.93E-03	LS	y	COE - RG	NOV 3, 94	Shelly w/ LS frags
CB-HHDR1-7B	-12.8	2.8	4.5	32.5	60	3.79E-02	LS	-12.8	-15.6	2.8	3.79E-02	LS	n	COE - RG	NOV 7, 94	All in LS
CB-HHDR1-8D	-1.4	20	2.875	1	10.7	5.83E-02	ML	-13.9	-18.9	5	1.70E-01	LS	y	TET	JAN 16, 95	Don't use test / see length & log
CB-GPPS-1	14	6	3	5.8	12	2.84E-02	PT/GC	9.5	8	1.5	7.29E-02	LS-P	y	COE - RG	FEB 95	
CB-GPPS-2	7.6	4.5	3	5.5	10	3.08E-02	CH	2	1	1	8.05E-02	LS-P	y	COE - RG	FEB 95	
CB-GPPS-3	7.7	4.5	3	5.5	7	2.16E-02	MH/CH	8.7	7.7	1	5.63E-02	LS-P	y	COE - RG	FEB 95	
CB-GPPS-4	7.4	4.5	3	4	2	8.47E-03	ML/CH	6.9	5.9	1	2.21E-02	LS-P	n	COE - RG	FEB 15 95	
CB-HHDR1-10B	4.8	6	2.9	24.8	2.5	1.40E-03	CH	5.7	4.8	0.9	4.79E-03	LS-P	Y	COE - RG	DEC 94 / JAN 95	Verify hole dia / 66% MH, 33% SC w/ shell
CB-HHDR1-12C	3.1	5.9	2.875	5.8	0.1098	2.66E-04	SM - ML	2.6	1.6	1	8.55E-04	LS-P	y	TET	Jan 5, 95	Test in SP / ML, 50% ea, all w/ shell
CB-12A-3	-17	21	3	29.2	5.4	9.60E-04	SM / SP			6	2.54E-03	LS/SM	y	COE - JG	Oct 25, 91	Very dense SM, N = 50+, 4.5' in SP
CB-10A-3	-7	9	3	10.8	25	2.34E-02	SS / SP		-12.5	5.5	3.39E-02	SS	y	COE - JG	Oct 18, 91	50% SS, 50% SP-SM

	Actual	Actual		Actual		Method	USBR 7310		Adjusted	Adjusted	Adjusted	Adjusted	Change			
	Bottom of	Test	Hole Dia	Total	Recharge	K	Soil	Bottom of	Bottom of	Test	K	Soil	from	Drilling	Test	
Boring	Casing El.	Length, Ft	inches	Head, Ft	GPM	Ft/Min	Type	Casing El.	Hole El.	Length, Ft	ft/min	Type	Original	Agency	Date	Notes
CB-10A-3	-18	7	3	23.3	50	2.63E-02	SM / SS		-25	7	2.63E-02	SS	y	COE - JG	Oct 17, 91	2' in SS, 5' in SM
CB-10A-3	-24	5	3	24.8	38.5	2.44E-02	SM	-24	-26	2	4.58E-02	SS	y	COE - JG	Oct 17, 91	1' in SS, 4' in Sm
CB-11-3	6	4.4	3	21.6	9.43	7.52E-03	SS / SM		3	3	9.84E-03	SS	y	COE - JG	Nov 4, 91	50% SS, 50% SM
CB-11-3	6	4.4	3	21.6	9.43	7.52E-03	SS / SM		3	3	9.84E-03	SS	y	COE - JG	Nov 4, 91	50% SS, 50% SM
CB-11-3	-1	3.5	3	22.9	23.8	2.11E-02	SS / LS		-4.5	3.5	2.11E-02	SS	y	COE - JG	Nov 4, 91	1.6' LS, 2.8' SS
CB-12A-3	-3.7	3	3	30.5	10	7.39E-03	SS	-3.7	-6.7	3	7.39E-03	SS	n	COE - JG	Oct 25, 91	0.7' in SM, 2.3' in SS

			Average					Average			SS Only	
			Std. Dev.					Std. Dev.			Average	2.20E-02
			V %	1.07				V %	1.06		Std. Dev.	1.44E-02
			Max.	1.46E-01				Max.	1.70E-01		V %	65.20
			Min.	5.77E-06				Min.	5.77E-06		Max.	4.58E-02
											Min.	7.39E-03

Laboratory Permeability Testing of Rock Samples

	Type	Boring	Sample	Sample		Ross	Test	Sample		K20 x 10-4	Avg K20x10-4	Avg K20	Unit Wt.	Test	Sample		K20 x 10-4	Avg K20x10-4	Avg K20	Unit Wt.	K ratio	K ratio
Boring	Test	Top El.	Depth	Top El.	Class.	Perm. No.	Dir	Height (cm)	Head (cm)	cm/sec	cm/sec	ft/min	SSD pcf	Dir.	Height (cm)	Head (cm)	cm/sec	cm/sec	ft/min	SSD pcf	h/v	v/h
CB-HHDR1-10-B	Lab	39.8	47.0	-7.2	LS	1	Vert.	10.14	91.44	0.003	0.003	1.48E-06	147.2	Hor.	7.12	91.44	0.611	0.602	9.37E-05	145.6	63.46667	0.015756
CB-HHDR1-31A	Lab	0	16.0	-16.0	LS	1	Vert.	10.05	91.44	0.362		6.95E-05	133.8	Hor.	6.85	88.9	0.072	0.078	3.38E-05	130.1	0.485836	2.058309
CB-HHDR1-29A	Lab	13.6	18.0	-4.4	LS	2	Vert.	10.26	91.44	0.001		4.92E-06	131.6	Hor.	6.54	91.44	0.069		1.15E-05	131.6	2.34	0.42735
CB-HHDR1-7A	Lab	12.4	27.0	-14.6	LS	2	Vert.	10.09	91.44	0.25		4.05E-05	138.6	Hor.	7.2	91.44	4.91		1.16E-03	141	28.63747	0.034919
CB-HHDR2-FB	Lab	40.8	40.0	0.8	LS	2 - lite	Vert.	10.17	91.44	37.9		6.03E-03	103.8	Hor.	3.56	91.44	4.63		1.15E-03	103.5	0.190642	5.245434
CB-HHDR1-19A	Lab		22.0	-22.0	LS	3	Vert.	10.41	91.44	28.1		4.21E-03	105.0	Hor.	6.99	91.44	107.8		2.13E-02	106.5	5.063084	0.197508
CB-HHDR1-8A	Lab	13	29.0	-16.0	LS	3	Vert.	10.16	91.44	31.2		5.70E-03	129.2	Hor.	6.48	91.44	55.8		1.03E-02	133.9	1.803109	0.554598
CB-HHDR2-FC	Lab	12.8	11.0	1.8	LS	3 - lite	Vert.	10.01	91.44	7.81		1.43E-03	112.4	Hor.	6.54	91.44	0.59		1.63E-04	106.4	0.113824	8.785498
CB-HHDR1-17A	Lab		25.0	-25.0	LS	4 - blue	Vert.	10.04	91.44	37.2		6.60E-03	104.4	Hor.	6.88	91.44	78.8		1.55E-02	105.6	2.344776	0.42648
CB-HHDR1-22D	Lab	17.9	42.0	-24.1	LS	4 - blue	Vert.	10.03	91.44	14.1		4.15E-03	114.6	Hor.	4.47	91.44	57.2		1.18E-02	117.8	2.836493	0.352548
CB-HHDR1-25A	Lab	12.6	28.0	-15.4	LS	5	Vert.	10.12	91.44	37.1		6.46E-03	108.1	Hor.	6.73	91.44	78.4		1.54E-02	108.6	2.385671	0.419169
CB-HHDR1-29A	Lab	13.6	27.0	-13.4	LS	5	Vert.	10.06	91.44	43		6.75E-03	104.3	Hor.	6.87	91.44	145.1		2.53E-02	92.2	3.746356	0.266926

Average	3.45E-03	1.19E+02
Std. Dev.	2.92E-03	1.57E+01
V %	84.42	13.11
Max.	6.75E-03	1.47E+02
Min.	1.48E-06	1.04E+02

Average	8.52E-03	1.19E+02	9.45E+00	1.57E+00
Std. Dev.	9.29E-03	1.72E+01	1.87E+01	2.71E+00
V %	109.08	14.51	197.80	172.94
Max.	2.53E-02	1.46E+02	6.35E+01	8.79E+00
Min.	1.15E-05	9.22E+01	1.14E-01	1.58E-02

USGS Field Permeability Tests

Site	Levee	Closest Boring	Type Test	Station	X	Y	Represent. Length, Mi.	Zone	Est. Soil Type	Thickness	Kf, gpd/ft2	T, gpd/ft	K, ft/min	
1	LD-1	CB-LD1-14	Slug	180+00	495595	902524	9.0	A-1	LS	5.0	1200	6000	8.33E-01	
2	LD-2	CB-LD2-2	Slug	60+18	531050	879198	8.5	A-1	LS	8.0	384	3040	2.64E-01	
4	LD-2	CB-LD2-66	Slug	980+00	599879	873474	10.0	A-1	LS - Marl	4.0	550	2200	3.82E-01	Avg. val.
3	LD-2	CB-LD2-34	Slug	480+95	565993	859588	8.5	A-1	LS - sh/sp	15.0	1200	18000	8.33E-01	
3	LD-2	CB-LD2-34	Pump	480+95	565993	859588	8.5	A-1	LS - sh/sp	15.0	n/a	16000	7.41E-01	
5	LD-9	CB-LD9-80	Slug	390+06	622685	925158	14.0	A-1	LS - sh/sp	14.0	65	924	4.58E-02	
4	LD-2	CB-LD2-66	Slug	980+00	599879	873474	10.0	A-2	LS - Shell	8.0	1500	12000	1.04E+00	Avg. val.
4	LD-2	CB-LD2-66	Pump	980+00	599879	873474	10.0	A-2	LS - Shell	8.0	n/a	24000	2.08E+00	

Average	7.78E-01
Std. Dev.	6.25E-01
V %	80.39
Max.	2.08E+00
Min.	4.58E-02

Dye Testing Summary

	Test El.	LS EL.		Test El.	LS EL.	Dist.	Shortest	Velocity
Boring	Range	Range	Boring	Range	Range	B - C	Time, min	ft/min
CB-MRR1-10B			CB-MRR1-10C			87	135	0.64
CB-MRR1-25B	-4.4 to 24.4		CB-MRR1-25C			103	90	1.14
CB-MRR1-27B	-1.7 to -11.7		CB-MRR1-27C	-3.4 to -18.4		93	90	1.03
CB-MRR1-7B	3.2 to -19.8		CB-MRR1-7C			138	75	1.84
CB-MRR1-28B			CB-MRR1-28C	-13.2 to -17.2		92	75	1.23
CB-MRR1-31B	-2.1 to -23.1	0.9 to -23.1	CB-MRR1-31C	2.4 to -12.1	4.9 to -16.6	84	70	1.2

Average	1.18E+00
Std. Dev.	3.88E-01
V %	32.91
Max.	1.84E+00
Min.	6.40E-01

4.8. Summary Spreadsheets for Shear Strength Data.

R & S Tests

				Triaxial Compression					Stress Path Method																
					SAD LAB Values								TOTAL STRENGTHS PARAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
Boring	Top El.	Soil Type	SPT #	C, t/sf	f ^{o}	C, t/sf	f ^{o}	C, t/sf																	Batch
CB-HH-3	12.4	SM-H		0.48	17.0	0.10	39.0	0.00	1.00	1.17	2.17	0.57	1.59	0.59	20.00	21.34	0.00	0.00	1.02	0.59	35.00	44.44	0.00	0.00	84-116
									2.00	2.21	4.21	1.35	3.11	1.11					1.76	1.11					
									5.00	4.84	9.84	3.70	7.42	2.42					3.72	2.42					
CB-HHDR1-13C	13.3	SM	2		27.4	0.70	43.0	0.00	0.50	3.42	3.92	-0.36	2.21	1.71	25.00	27.79	0.70	0.79	2.57	1.71	33.00	40.50	0.00	0.00	95-51
									1.51	5.14	6.65	0.36	4.08	2.57					3.72	2.57					
CB-5A-2	-3.1	SC	3		21.0	0.26	31.0	0.00	0.50	1.02	1.52	0.18	1.01	0.51	22.50	24.47	0.15	0.16	0.83	0.51	30.00	35.26	0.00	0.00	94-161
									1.51	3.05	4.56	0.64	3.04	1.53					2.40	1.53					
									3.02	3.82	6.84	0.86	4.93	1.91					4.07	1.91					
CB-5A-2	-12.1	SC	n/a		11.6	1.41	43.3	0.00	0.50	3.35	3.85	-0.29	2.18	1.68	23.00	25.12	0.60	0.66	2.47	1.68	34.50	43.42	0.00	0.00	94-161
									1.51	5.24	6.75	0.94	4.13	2.62					3.19	2.62					
									3.02	4.32	7.34	1.51	5.18	2.16					3.67	2.16					
CB-5A-3	5.3	SC	10		15.9	0.22	39.1	0.00	0.50	0.97	1.47	0.22	0.99	0.49	16.50	17.23	0.20	0.21	0.77	0.49	32.50	39.57	0.00	0.00	94-161
									1.51	1.73	3.24	1.01	2.38	0.87					1.37	0.87					
				AVE	18.58	0.54	39.08	0.00							AVE	23.19		0.37				40.64		0.00	
				STDEV	5.96	0.54	4.96	0.00							STDEV	4.05		0.34				3.61		0.00	
CB-HHSB-1	11.0	PT		0.27	9.5	0.28	32.3	0.00	0.25	0.88	1.13	0.27	0.69	0.44	10.00	10.16	0.28	0.28	0.42	0.44	20.00	21.34	0.21	0.23	90-67
									0.50	0.74	1.24	0.26	0.87	0.37					0.61	0.37					
									1.01	1.10	2.11	0.53	1.56	0.55					1.03	0.55					
CB-HHSB -5	9.4	PT		0.14	7.6	0.23	27.7	0.00	0.25	0.61	0.86	0.19	0.56	0.31	7.00	7.05	0.24	0.24	0.37	0.31	23.00	25.12	0.00	0.00	90-67
									0.50	0.64	0.83	0.24	0.67	0.17					0.43	0.17					
									1.01	0.83	1.84	0.45	1.43	0.42					0.98	0.42					
CB-HHSB-3	11.3	PT		0.44	17.7	0.39	45.0	0.00	0.25	1.41	1.66	0.07	0.96	0.71	13.00	13.35	0.47	0.48	0.89	0.71	34.00	42.42	0.20	0.27	90-67
									0.50	1.38	1.88	0.46	1.19	0.69					0.73	0.69					
									1.01	1.99	3.00	1.01	2.01	1.00					1.00	1.00					
CB-HH-3	6.7	PT		0.40	29.0	0.00	45.0	0.00	1.00	1.43	2.43	0.75	1.72	0.72	21.00	22.57	0.00	0.00	0.97	0.72	33.00	40.50	0.00	0.00	84-116
									2.00	2.43	4.43	1.30	3.22	1.22					1.92	1.22					

				Triaxial Compression					Stress Path Method																
					SAD LAB Values								TOTAL STRENGTHS PARAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
Boring	Top El.	Soil Type	SPT #	C, t/sf	f ^{o}	C, t/sf	f ^{o}	C, t/sf																	Batch
									4.00	7.48	11.48	2.90	7.74	3.74					4.84	3.74					
									5.00	11.69	16.69	3.70	10.85	5.85					7.15	5.85					
CB-HHDR1-2C	8.1	PT	2		15.3	0.11	18.2	0.00	0.50	0.69	1.19	0.45	0.85	0.35	15.30	15.88	0.11	0.11	0.40	0.35	14.50	14.99	0.24	0.25	95-51
									1.51	1.33	2.84	0.50	2.18	0.67					1.68	0.67					
									3.02	2.48	5.50	0.37	4.26	1.24					3.89	1.24					
CB-HHDR1-3D	9.6	PT	1		22.7	0.00	38.5	0.00	0.50	0.68	1.18	0.30	0.84	0.34	21.00	22.57	0.00	0.00	0.54	0.34	32.00	38.67	0.00	0.00	95-51
									1.51	1.70	3.21	0.84	2.36	0.85					1.52	0.85					
									3.02	3.87	6.89	2.00	4.96	1.94					2.96	1.94					
PZ-R1-4B-2	8.7	PT	7		18.6	0.38	42.6	0.00	0.50	1.56	2.06	0.47	1.28	0.78	17.00	17.80	0.40	0.42	0.81	0.78	30.00	35.26	0.30	0.37	96-72
									1.51	2.45	3.96	1.13	2.74	1.23					1.61	1.23					
									3.02	3.91	6.93	2.07	4.98	1.96					2.91	1.96					
PZ-R1-4C-2	7.7	PT	2		20.4	0.06	34.2	0.00	0.50	0.73	1.23	0.29	0.87	0.37	16.00	16.66	0.34	0.35	0.58	0.37	28.50	32.89	0.06	0.07	96-72
									1.51	2.50	4.01	0.98	2.76	1.25					1.78	1.25					
									3.02	3.43	6.45	1.65	4.74	1.72					3.09	1.72					
CB-HHDR1-12C	6.1	PT	2		21.3	0.04	37.1	0.00	0.50	0.62	1.12	0.34	0.81	0.31	20.50	21.96	0.00	0.00	0.47	0.31	31.50	37.79	0.00	0.00	95-51
									1.51	1.97	3.48	0.97	2.50	0.99					1.53	0.99					
									3.02	3.52	6.54	1.81	4.78	1.76					2.97	1.76					
PZ-R1-25B-2	4.6	PT	12		12.9	0.61	27.3	0.00	0.50	1.71	2.21	0.37	1.36	0.86	13.00	13.35	0.54	0.55	0.99	0.86	20.50	21.96	0.43	0.46	96-72
									1.51	2.61	4.12	1.26	2.82	1.31					1.56	1.31					
									3.02	3.18	6.20	0.46	4.61	1.59					4.15	1.59					
PZ-R1-25C-2	3.4	PT	4		25.7	0.02	40.3	0.00	0.50	0.75	1.25	0.37	0.88	0.38	18.0	18.96	0.56	0.59	0.51	0.38	30.20	35.59	0.30	0.37	96-72
									1.51	2.52	4.03	0.99	2.77	1.26					1.78	1.26					
									3.20	4.64	7.84	1.66	5.52	2.32					3.86	2.32					
					18.25	0.19	35.29	0.00							AVE	16.39		0.28				31.50		0.18	
					6.58	0.20	8.39	0.00							STDEV	5.11		0.22				9.12		0.18	
CB-HHDR1-7D	6.1	OL	3		18.7	0.03	25.7	0.00	0.50	0.65	1.15	0.06	0.83	0.33	18.00	18.96	0.10	0.11	0.77	0.33	24.50	27.11	0.00	0.00	95-51
									1.51	1.38	2.89	1.09	2.20	0.69					1.11	0.69					
									3.02	2.99	6.01	1.07	4.52	1.50					3.45	1.50					
AB-HH4-12A	11.3	OH		0.55	18.5	0.40	44.5	0.00	1.00	2.06	3.06	0.79	2.03	1.03	19.00	20.14	0.35	0.37	1.24	1.03	35.00	44.44	0.20	0.28	85-10
									2.00	2.93	4.93	1.57	3.47	1.47					1.90	1.47					
									5.00	5.92	10.92	3.90	7.96	2.96					4.06	2.96					

				Triaxial Compression					Stress Path Method																	
					SAD LAB Values								TOTAL STRENGTHS PARAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
Boring	Top El.	Soil Type	SPT #	C, t/sf	f^o	C, t/sf	f^o	C, t/sf																	Batch
CB-HD1-53	12.8	OH	2	0.65	26.5	0.00	43.0	0.00	1.00	1.84	2.84	0.75	1.92	0.92	23.00	25.12	0.10	0.11	1.17	0.92	36.00	46.60	0.00	0.00	85-10
									2.00	2.89	4.89	1.45	3.45	1.45					2.00	1.45					
									5.00	8.88	13.88	3.60	9.44	4.44					5.84	4.44					
CB-HH4-13	6.5	OH			15.5	0.15	37.0	0.00	0.50	0.75	1.25	1.25	0.88	0.38	15.50	16.10	0.15	0.16	-0.38	0.38	33.00	40.50	0.00	0.00	85-10
									1.50	1.49	2.99	2.99	2.25	0.75					-0.75	0.75					
CB-HD1-58A	14.0	OH		0.55	18.0	0.20	38.0	0.00	1.00	1.47	2.47	0.61	1.74	0.74	19.50	20.74	0.20	0.21	1.13	0.74	32.00	38.67	0.10	0.13	85-10
									2.00	2.68	4.68	1.45	3.34	1.34					1.89	1.34					
									5.00	5.13	10.13	3.25	7.57	2.57					4.32	2.57					
AB-HH4-12B	13.3	OH		0.90	14.0	0.50	45.0	0.00	1.00	2.04	3.04	0.70	2.02	1.02	14.00	14.44	0.50	0.52	1.32	1.02	36.00	46.60	0.10	0.15	85-10
									2.00	2.64	4.64	1.60	3.32	1.32					1.72	1.32					
									5.00	4.51	9.51	4.10	7.26	2.26					3.16	2.26					
CB-16-2	7.3	OH	2		22.2	0.00	31.8	0.00	0.50	0.75	1.25	0.49	0.88	0.38	22.00	23.83	0.00	0.00	0.39	0.38	26.00	29.19	0.00	0.00	94-161
									1.51	1.54	3.05	0.85	2.28	0.77					1.43	0.77					
									3.02	3.78	6.80	1.30	4.91	1.89					3.61	1.89					
CB-HHDR1-14C	12.7	OH	2		14.7	0.15	20.8	0.00	1.01	1.28	2.29	0.82	1.65	0.64	15.00	15.54	0.10	0.10	0.83	0.64	20.50	21.96	0.00	0.00	95-51
									2.02	1.49	3.51	0.98	2.77	0.75					1.79	0.75					
									4.03	3.22	7.25	1.07	5.64	1.61					4.57	1.61					
CB-HHDR1-19C	7.8	OH	5		21.9	0.00	29.9	0.00	0.50	0.59	1.09	0.35	0.80	0.30	20.50	21.96	0.00	0.00	0.45	0.30	27.00	30.63	0.00	0.00	96-72
									1.51	1.79	3.30	0.63	2.41	0.90					1.78	0.90					
									3.02	3.61	6.63	1.17	4.83	1.81					3.66	1.81					
CB-HHDR1-20C	7.4	OH	1		21.4	0.00	37.6	0.00	0.50	0.54	1.04	0.27	0.77	0.27	20.00	21.34	0.00	0.00	0.50	0.27	31.50	37.79	0.00	0.00	96.72
									1.51	1.83	3.34	0.96	2.43	0.92					1.47	0.92					
									3.02	3.45	6.47	1.92	4.75	1.73					2.83	1.73					
CB-HHDR1-21C	8.4	OH	2		15.2	0.12	24.9	0.00	0.50	0.58	1.08	0.33	0.79	0.29	13.00	13.35	0.20	0.21	0.46	0.29	23.00	25.12	0.10	0.11	96-72
									1.51	1.54	3.05	0.98	2.28	0.77					1.30	0.77					
									3.02	2.39	5.41	1.13	4.22	1.20					3.09	1.20					
				Avg	18.78	0.14	34.38	0.00							Avg	19.23		0.16				35.33		0.06	
				StdDev	3.89	0.17	8.34	0.00							StdDev	3.90		0.16				8.90		0.09	
CB-HH4-13		ML		0.50	7.0	0.80	39.0	0.00	1.00	2.03	3.03	0.58	2.02	1.02	8.00	8.08	0.75	0.76	1.44	1.02	26.50	29.91	0.30	0.35	85-10
									2.00	2.35	4.35	1.51	3.18	1.18					1.67	1.18					
									5.00	3.21	8.21	3.89	6.61	1.61					2.72	1.61					

				Triaxial Compression					Stress Path Method																	
					SAD LAB Values								TOTAL STRENGTHS PARAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
Boring	Top El.	Soil Type	SPT #	C, t/sf	f^o	C, t/sf	f^o	C, t/sf																	Batch
CB-HHDR1-18C	6.7	ML	1		14.5	0.34	39.9	0.00	0.50	1.21	1.71	0.26	1.11	0.61	14.00	14.44	0.38	0.39	0.85	0.61	32.50	39.57	0.02	0.03	96-72
									1.51	1.89	3.40	0.93	2.46	0.95					1.53	0.95					
									3.02	2.89	5.91	2.31	4.47	1.45					2.16	1.45					
CB-HHDR1-16C	3.4	MH	3		12.6	0.22	31.5	0.00	0.50	0.76	1.26	0.32	0.88	0.38	12.50	12.81	0.20	0.21	0.56	0.38	24.50	27.11	0.12	0.13	96-72
									1.51	1.54	3.05	1.04	2.28	0.77					1.24	0.77					
									3.02	2.18	5.20	1.86	4.11	1.09					2.25	1.09					
CB-HHDR1-2B	13.6	MH	3		12.1	0.47	40.2	0.00	0.50	1.32	1.82	0.20	1.16	0.66	13.00	13.35	0.40	0.41	0.96	0.66	32.50	39.57	0.10	0.13	95-51
									1.51	2.15	3.66	0.96	2.59	1.08					1.63	1.08					
									3.02	2.67	5.69	2.35	4.36	1.34					2.01	1.34					
				Avg	9.6	0.6	39.6	0.0							Avg	12.2		0.4				34.0		0.2	
				StdDev	3.2	0.3	4.1	0.0							StdDev	2.8		0.2				6.5		0.1	
CB-HHDR1-9C	-0.2	CH			15.0	0.40	44.9	0.00	0.50	1.38	1.88	0.25	1.19	0.69	16.00	16.66	0.25	0.26	0.94	0.69	35.00	44.44	0.00	0.00	95-51
									1.51	1.68	3.19	1.10	2.35	0.84					1.25	0.84					
									3.02	3.43	6.45	2.37	4.74	1.72					2.37	1.72					
CB-5A-2	14.9	CH	7		13.0	0.28	24.0	0.00	0.50	0.97	1.47	0.29	0.99	0.49	13.00	13.35	0.28	0.29	0.70	0.49	22.50	24.47	0.10	0.11	94-161
									1.51	1.61	3.12	0.36	2.32	0.81					1.96	0.81					
									3.02	2.44	5.46	1.22	4.24	1.22					3.02	1.22					
CB-HHDR1-15C	1.8	CH	0		11.0	0.31	34.8	0.00	0.50	1.00	1.50	0.35	1.00	0.50	9.50	9.63	0.40	0.41	0.65	0.50	26.50	29.91	0.16	0.18	95-51
									1.51	1.47	2.98	1.05	2.25	0.74					1.20	0.74					
									3.02	2.19	5.21	2.20	4.12	1.10					1.92	1.10					
				Avg	13.00	0.33	34.57	0.00							Avg	13.21		0.32				32.94		0.10	
				StdDev	2.83	0.06	7.14	0.00							StdDev	4.97		0.10				10.28		0.13	

R & S Tests (Comb.)

				Triaxial Compression					Stress Path Method																	
					SAD LAB Values								TOTAL STRENGTHS PAAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
Boring	Top El.	Soil Type	SPT #	C, t/sf	f ^{o}	C, t/sf	f ^{o}	C, t/sf																	Batch
CB-HH-3	12.4	SM-H		0.48	17.0	0.10	39.0	0.00	1.00	1.17	2.17	0.57	1.59	0.59	20.00	21.34	0.00	0.00	1.02	0.59	35.00	44.44	0.00	0.00	84-116
									2.00	2.21	4.21	1.35	3.11	1.11					1.76	1.11					
									5.00	4.84	9.84	3.70	7.42	2.42					3.72	2.42					
CB-HHDR1-13C	13.3	SM	2		27.4	0.70	43.0	0.00	0.50	3.42	3.92	-0.36	2.21	1.71	25.00	27.79	0.70	0.79	2.57	1.71	33.00	40.50	0.00	0.00	95-51
									1.51	5.14	6.65	0.36	4.08	2.57					3.72	2.57					
CB-5A-2	-3.1	SC	3		21.0	0.26	31.0	0.00	0.50	1.02	1.52	0.18	1.01	0.51	22.50	24.47	0.15	0.16	0.83	0.51	30.00	35.26	0.00	0.00	94-161
									1.51	3.05	4.56	0.64	3.04	1.53					2.40	1.53					
									3.02	3.82	6.84	0.86	4.93	1.91					4.07	1.91					
CB-5A-2	-12.1	SC	n/a		11.6	1.41	43.3	0.00	0.50	3.35	3.85	-0.29	2.18	1.68	23.00	25.12	0.60	0.66	2.47	1.68	34.50	43.42	0.00	0.00	94-161
									1.51	5.24	6.75	0.94	4.13	2.62					3.19	2.62					
									3.02	4.32	7.34	1.51	5.18	2.16					3.67	2.16					
CB-5A-3	5.3	SC	10		15.9	0.22	39.1	0.00	0.50	0.97	1.47	0.22	0.99	0.49	16.50	17.23	0.20	0.21	0.77	0.49	32.50	39.57	0.00	0.00	94-161
									1.51	1.73	3.24	1.01	2.38	0.87					1.37	0.87					
				AVE	18.58	0.54	39.08	0.00							AVE	23.19		0.37				40.64		0.00	
				STDEV	5.96	0.54	4.96	0.00							STDEV	4.05		0.34				3.61		0.00	
CB-HHSB-1	11.0	PT		0.27	9.5	0.28	32.3	0.00	0.25	0.88	1.13	0.27	0.69	0.44	10.00	10.16	0.28	0.28	0.42	0.44	20.00	21.34	0.21	0.23	90-67
									0.50	0.74	1.24	0.26	0.87	0.37					0.61	0.37					
									1.01	1.10	2.11	0.53	1.56	0.55					1.03	0.55					
CB-HHSB -5	9.4	PT		0.14	7.6	0.23	27.7	0.00	0.25	0.61	0.86	0.19	0.56	0.31	7.00	7.05	0.24	0.24	0.37	0.31	23.00	25.12	0.00	0.00	90-67
									0.50	0.64	0.83	0.24	0.67	0.17					0.43	0.17					
									1.01	0.83	1.84	0.45	1.43	0.42					0.98	0.42					
CB-HHSB-3	11.3	PT		0.44	17.7	0.39	45.0	0.00	0.25	1.41	1.66	0.07	0.96	0.71	13.00	13.35	0.47	0.48	0.89	0.71	34.00	42.42	0.20	0.27	90-67
									0.50	1.38	1.88	0.46	1.19	0.69					0.73	0.69					
									1.01	1.99	3.00	1.01	2.01	1.00					1.00	1.00					
CB-HH-3	6.7	PT		0.40	29.0	0.00	45.0	0.00	1.00	1.43	2.43	0.75	1.72	0.72	21.00	22.57	0.00	0.00	0.97	0.72	33.00	40.50	0.00	0.00	84-116
									2.00	2.43	4.43	1.30	3.22	1.22					1.92	1.22					
									4.00	7.48	11.48	2.90	7.74	3.74					4.84	3.74					

				Triaxial Compression					Stress Path Method																	
					SAD LAB Values								TOTAL STRENGTHS PAAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
									5.00	11.69	16.69	3.70	10.85	5.85					7.15	5.85					
CB-HHDR1-2C	8.1	PT	2		15.3	0.11	18.2	0.00	0.50	0.69	1.19	0.45	0.85	0.35	15.30	15.88	0.11	0.11	0.40	0.35	14.50	14.99	0.24	0.25	95-51
									1.51	1.33	2.84	0.50	2.18	0.67					1.68	0.67					
									3.02	2.48	5.50	0.37	4.26	1.24					3.89	1.24					
CB-HHDR1-3D	9.6	PT	1		22.7	0.00	38.5	0.00	0.50	0.68	1.18	0.30	0.84	0.34	21.00	22.57	0.00	0.00	0.54	0.34	32.00	38.67	0.00	0.00	95-51
									1.51	1.70	3.21	0.84	2.36	0.85					1.52	0.85					
									3.02	3.87	6.89	2.00	4.96	1.94					2.96	1.94					
PZ-R1-4B-2	8.7	PT	7		18.6	0.38	42.6	0.00	0.50	1.56	2.06	0.47	1.28	0.78	17.00	17.80	0.40	0.42	0.81	0.78	30.00	35.26	0.30	0.37	96-72
									1.51	2.45	3.96	1.13	2.74	1.23					1.61	1.23					
									3.02	3.91	6.93	2.07	4.98	1.96					2.91	1.96					
PZ-R1-4C-2	7.7	PT	2		20.4	0.06	34.2	0.00	0.50	0.73	1.23	0.29	0.87	0.37	16.00	16.66	0.34	0.35	0.58	0.37	28.50	32.89	0.06	0.07	96-72
									1.51	2.50	4.01	0.98	2.76	1.25					1.78	1.25					
									3.02	3.43	6.45	1.65	4.74	1.72					3.09	1.72					
CB-HHDR1-12C	6.1	PT	2		21.3	0.04	37.1	0.00	0.50	0.62	1.12	0.34	0.81	0.31	20.50	21.96	0.00	0.00	0.47	0.31	31.50	37.79	0.00	0.00	95-51
									1.51	1.97	3.48	0.97	2.50	0.99					1.53	0.99					
									3.02	3.52	6.54	1.81	4.78	1.76					2.97	1.76					
PZ-R1-25B-2	4.6	PT	12		12.9	0.61	27.3	0.00	0.50	1.71	2.21	0.37	1.36	0.86	13.00	13.35	0.54	0.55	0.99	0.86	20.50	21.96	0.43	0.46	96-72
									1.51	2.61	4.12	1.26	2.82	1.31					1.56	1.31					
									3.02	3.18	6.20	0.46	4.61	1.59					4.15	1.59					
PZ-R1-25C-2	3.4	PT	4		25.7	0.02	40.3	0.00	0.50	0.75	1.25	0.37	0.88	0.38	18.0	18.96	0.56	0.59	0.51	0.38	30.20	35.59	0.30	0.37	96-72
									1.51	2.52	4.03	0.99	2.77	1.26					1.78	1.26					
									3.20	4.64	7.84	1.66	5.52	2.32					3.86	2.32					
CB-HHDR1-7D	6.1	OL	3		18.7	0.03	25.7	0.00	0.50	0.65	1.15	0.06	0.83	0.33	18.00	18.96	0.10	0.11	0.77	0.33	24.50	27.11	0.00	0.00	95-51
									1.51	1.38	2.89	1.09	2.20	0.69					1.11	0.69					
									3.02	2.99	6.01	1.07	4.52	1.50					3.45	1.50					
AB-HH4-12A	11.3	OH		0.55	18.5	0.40	44.5	0.00	1.00	2.06	3.06	0.79	2.03	1.03	19.00	20.14	0.35	0.37	1.24	1.03	35.00	44.44	0.20	0.28	85-10
									2.00	2.93	4.93	1.57	3.47	1.47					1.90	1.47					
									5.00	5.92	10.92	3.90	7.96	2.96					4.06	2.96					
CB-HD1-53	12.8	OH	2	0.65	26.5	0.00	43.0	0.00	1.00	1.84	2.84	0.75	1.92	0.92	23.00	25.12	0.10	0.11	1.17	0.92	36.00	46.60	0.00	0.00	85-10
									2.00	2.89	4.89	1.45	3.45	1.45					2.00	1.45					
									5.00	8.88	13.88	3.60	9.44	4.44					5.84	4.44					
CB-HH4-13	6.5	OH			15.5	0.15	37.0	0.00	0.50	0.75	1.25	1.25	0.88	0.38	15.50	16.10	0.15	0.16	-0.38	0.38	33.00	40.50	0.00	0.00	85-10
									1.50	1.49	2.99	2.99	2.25	0.75					-0.75	0.75					

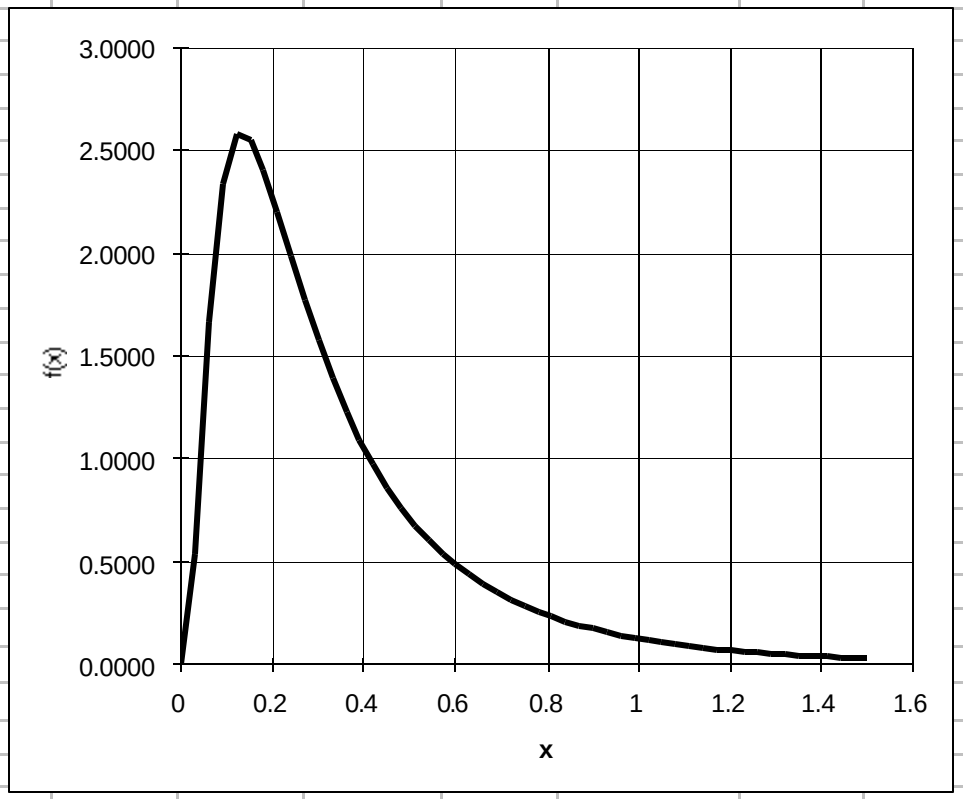
				Triaxial Compression					Stress Path Method																	
					SAD LAB Values								TOTAL STRENGTHS PAAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^0, tot	Φ^0, tot	a tot	c tot	Peff	Qeff	Ψ^0, eff	Φ^0, eff	a eff	c eff	Test
CB-HD1-58A	14.0	OH		0.55	18.0	0.20	38.0	0.00	1.00	1.47	2.47	0.61	1.74	0.74	19.50	20.74	0.20	0.21	1.13	0.74	32.00	38.67	0.10	0.13	85-10
									2.00	2.68	4.68	1.45	3.34	1.34					1.89	1.34					
									5.00	5.13	10.13	3.25	7.57	2.57					4.32	2.57					
AB-HH4-12B	13.3	OH		0.90	14.0	0.50	45.0	0.00	1.00	2.04	3.04	0.70	2.02	1.02	14.00	14.44	0.50	0.52	1.32	1.02	36.00	46.60	0.10	0.15	85-10
									2.00	2.64	4.64	1.60	3.32	1.32					1.72	1.32					
									5.00	4.51	9.51	4.10	7.26	2.26					3.16	2.26					
CB-16-2	7.3	OH	2		22.2	0.00	31.8	0.00	0.50	0.75	1.25	0.49	0.88	0.38	22.00	23.83	0.00	0.00	0.39	0.38	26.00	29.19	0.00	0.00	94-161
									1.51	1.54	3.05	0.85	2.28	0.77					1.43	0.77					
									3.02	3.78	6.80	1.30	4.91	1.89					3.61	1.89					
CB-HHDR1-14C	12.7	OH	2		14.7	0.15	20.8	0.00	1.01	1.28	2.29	0.82	1.65	0.64	15.00	15.54	0.10	0.10	0.83	0.64	20.50	21.96	0.00	0.00	95-51
									2.02	1.49	3.51	0.98	2.77	0.75					1.79	0.75					
									4.03	3.22	7.25	1.07	5.64	1.61					4.57	1.61					
CB-HHDR1-19C	7.8	OH	5		21.9	0.00	29.9	0.00	0.50	0.59	1.09	0.35	0.80	0.30	20.50	21.96	0.00	0.00	0.45	0.30	27.00	30.63	0.00	0.00	96-72
								0.00	1.51	1.79	3.30	0.63	2.41	0.90					1.78	0.90					
								0.00	3.02	3.61	6.63	1.17	4.83	1.81					3.66	1.81					
CB-HHDR1-20C	7.4	OH	1		21.4	0.00	37.6	0.00	0.50	0.54	1.04	0.27	0.77	0.27	20.00	21.34	0.00	0.00	0.50	0.27	31.50	37.79	0.00	0.00	96.72
									1.51	1.83	3.34	0.96	2.43	0.92					1.47	0.92					
									3.02	3.45	6.47	1.92	4.75	1.73					2.83	1.73					
CB-HHDR1-21C	8.4	OH	2		15.2	0.12	24.9	0.00	0.50	0.58	1.08	0.33	0.79	0.29	13.00	13.35	0.20	0.21	0.46	0.29	23.00	25.12	0.10	0.11	96-72
									1.51	1.54	3.05	0.98	2.28	0.77					1.30	0.77					
									3.02	2.39	5.41	1.13	4.22	1.20					3.09	1.20					
				Avg	18.51	0.17	34.84	0.00							Avg	17.81		0.22				33.42		0.12	
				StdDev	5.28	0.18	8.18	0.00							StdDev	4.67		0.20				9.01		0.15	
CB-HH4-13		ML		0.50	7.0	0.80	39.0	0.00	1.00	2.03	3.03	0.58	2.02	1.02	8.00	8.08	0.75	0.76	1.44	1.02	26.50	29.91	0.30	0.35	85-10
									2.00	2.35	4.35	1.51	3.18	1.18					1.67	1.18					
									5.00	3.21	8.21	3.89	6.61	1.61					2.72	1.61					
CB-HHDR1-18C	6.7	ML	1		14.5	0.34	39.9	0.00	0.50	1.21	1.71	0.26	1.11	0.61	14.00	14.44	0.38	0.39	0.85	0.61	32.50	39.57	0.02	0.03	96-72
									1.51	1.89	3.40	0.93	2.46	0.95					1.53	0.95					
									3.02	2.89	5.91	2.31	4.47	1.45					2.16	1.45					
CB-HHDR1-16C	3.4	MH	3		12.6	0.22	31.5	0.00	0.50	0.76	1.26	0.32	0.88	0.38	12.50	12.81	0.20	0.21	0.56	0.38	24.50	27.11	0.12	0.13	96-72
									1.51	1.54	3.05	1.04	2.28	0.77					1.24	0.77					

				Triaxial Compression					Stress Path Method																	
					SAD LAB Values								TOTAL STRENGTHS PAAMETERS						EFFECTIVE STRENGTH PARAMETERS						
	Sample				R - Total		R - Eff		s_3	Ds_1	s_1	u_f	p	q	Ψ^o, tot	Φ^o, tot	a tot	c tot	Peff	Qeff	Ψ^o, eff	Φ^o, eff	a eff	c eff	Test
									3.02	2.18	5.20	1.86	4.11	1.09					2.25	1.09					
CB-HHDR1-2B	13.6	MH	3		12.1	0.47	40.2	0.00	0.50	1.32	1.82	0.20	1.16	0.66	13.00	13.35	0.40	0.41	0.96	0.66	32.50	39.57	0.10	0.13	95-51
									1.51	2.15	3.66	0.96	2.59	1.08					1.63	1.08					
									3.02	2.67	5.69	2.35	4.36	1.34					2.01	1.34					
CB-HHDR1-9C	-0.2	CH			15.0	0.40	44.9	0.00	0.50	1.38	1.88	0.25	1.19	0.69	16.00	16.66	0.25	0.26	0.94	0.69	35.00	44.44	0.00	0.00	95-51
									1.51	1.68	3.19	1.10	2.35	0.84					1.25	0.84					
									3.02	3.43	6.45	2.37	4.74	1.72					2.37	1.72					
CB-5A-2	14.9	CH	7		13.0	0.28	24.0	0.00	0.50	0.97	1.47	0.29	0.99	0.49	13.00	13.35	0.28	0.29	0.70	0.49	22.50	24.47	0.10	0.11	94-161
									1.51	1.61	3.12	0.36	2.32	0.81					1.96	0.81					
									3.02	2.44	5.46	1.22	4.24	1.22					3.02	1.22					
CB-HHDR1-15C	1.8	CH	0		11.0	0.31	34.8	0.00	0.50	1.00	1.50	0.35	1.00	0.50	9.50	9.63	0.40	0.41	0.65	0.50	26.50	29.91	0.16	0.18	95-51
									1.51	1.47	2.98	1.05	2.25	0.74					1.20	0.74					
									3.02	2.19	5.21	2.20	4.12	1.10					1.92	1.10					
				Avg	12.17	0.40	36.33	0.00							Avg	12.62		0.39				33.57		0.13	
				StdDev	2.66	0.19	6.90	0.00							StdDev	2.90		0.18				7.55		0.11	

L25SHRST.XLS Emb, Rc

LOGNORMAL DISTRIBUTION	
Wolff, July 1993	
K is lognormally distributed	
Y = ln K is normally distributed	
Lognormal Parameters	
E[k] =	0.37
σ[k] =	0.37
Vk =	1.0000
E[y] =	-1.3408
σ[y] =	0.8326
k - min =	0.00
k - max =	1.50
Increment =	0.0300
Normalized Parameters	
E[ln k] =	-1.341
σ [ln k] =	0.833
Mean [$e^{\ln k}$] =	0.26
High [$e^{\ln k + \sigma \ln k}$] =	0.60
Low [$e^{\ln k + \sigma \ln k}$] =	0.11

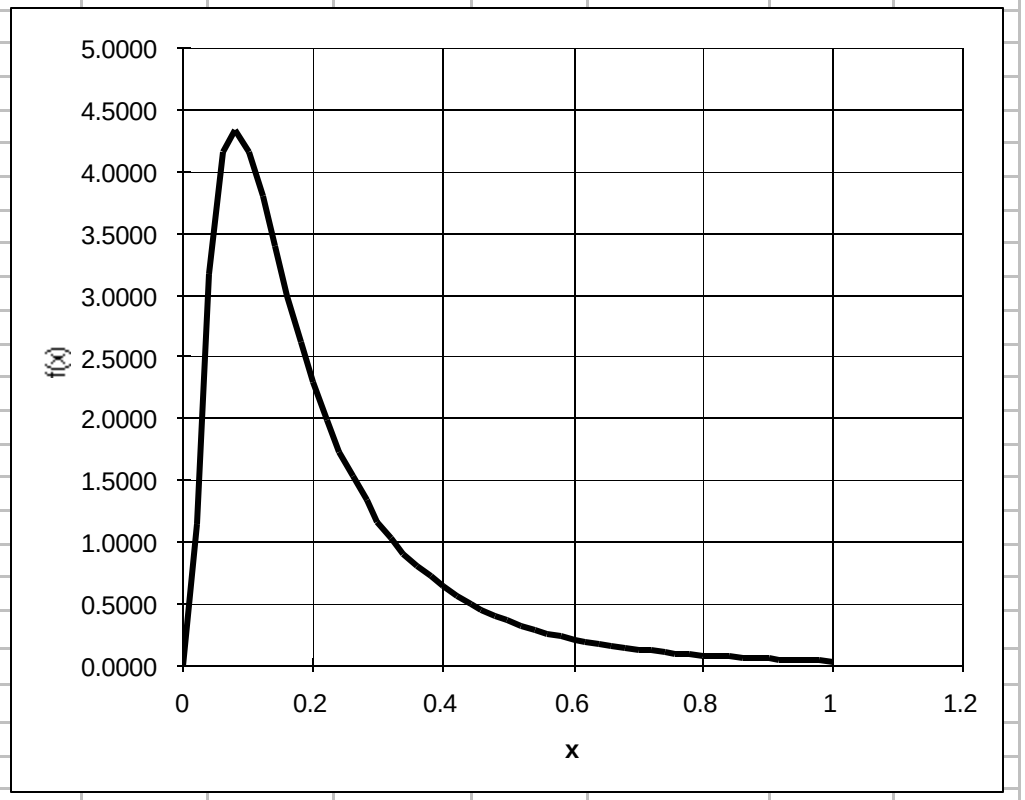
Embankment, R Cohesion



Peat, Rc

LOGNORMAL DISTRIBUTION	
Wolff, July 1993	
K is lognormally distributed	
Y = ln K is normally distributed	
Lognormal Parameters	
E[k] =	0.22
σ[k] =	0.22
Vk =	1.0000
E[y] =	-1.8607
σ[y] =	0.8326
k - min =	0.00
k - max =	1.00
Increment =	0.0200
	.
Normalized Parameters	
E[ln k] =	-1.861
σ [ln k] =	0.833
Mean [$e^{\ln k}$] =	0.16
High [$e^{\ln k + \sigma \ln k}$] =	0.36
Low [$e^{\ln k + \sigma \ln k}$] =	0.07

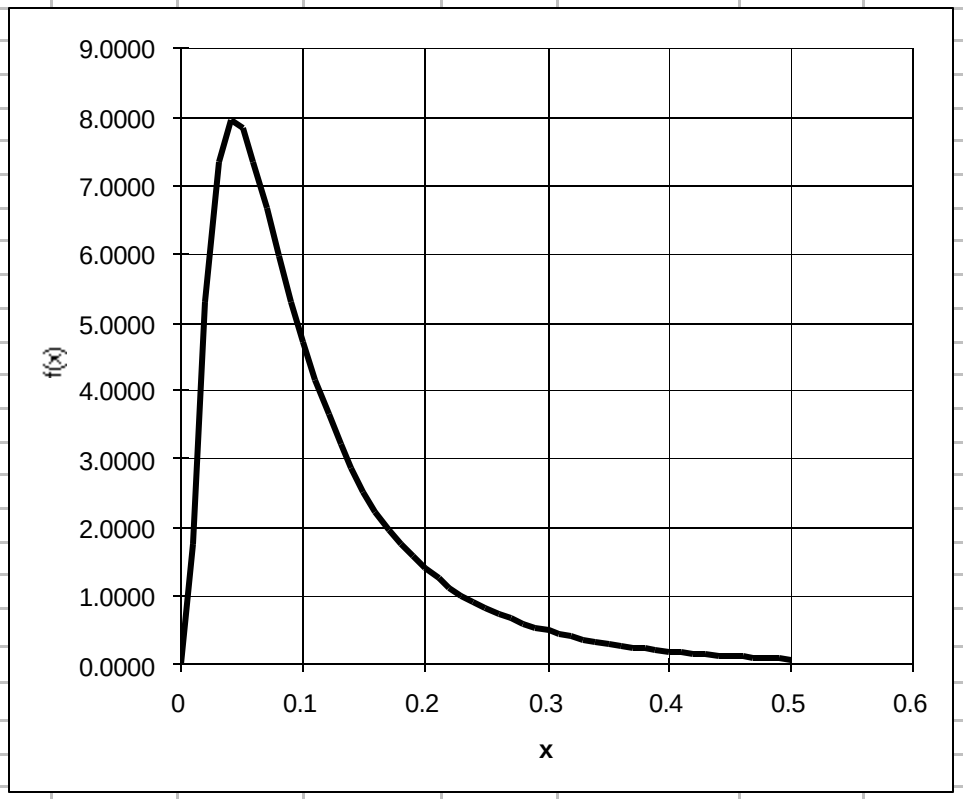
PT/OL/OH, R Cohesion



Peat, Sc

LOGNORMAL DISTRIBUTION	
Wolff, July 1993	
K is lognormally distributed	
Y = ln K is normally distributed	
Lognormal Parameters	
E[k] =	0.12
σ[k] =	0.12
Vk =	1.0000
E[y] =	-2.4668
σ[y] =	0.8326
k - min =	0.00
k - max =	0.50
Increment =	0.0100
Normalized Parameters	
E[ln k] =	-2.467
σ [ln k] =	0.833
Mean [$e^{\ln k}$] =	0.08
High [$e^{\ln k + \sigma \ln k}$] =	0.20
Low [$e^{\ln k + \sigma \ln k}$] =	0.04

PT/OL/OH, S Cohesion



Clay, Rc

LOGNORMAL DISTRIBUTION

Wolff, July 1993

Clays, R Cohesion

K is lognormally distributed

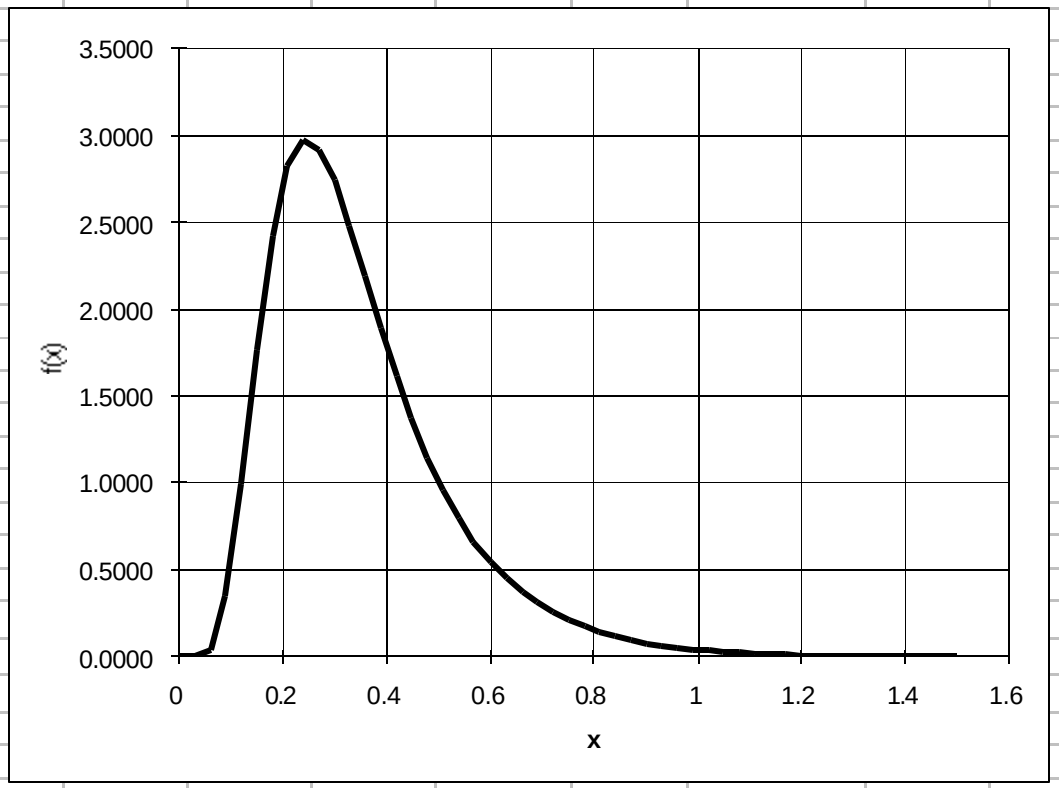
Y = ln K is normally distributed

Lognormal Parameters

E[k] =	0.35
σ[k] =	0.18
Vk =	0.5143
E[y] =	-1.1672
σ[y] =	0.4844
k - min =	0.00
k - max =	1.50
Increment =	0.0300

Normalized Parameters

E[ln k] =	-1.167
σ [ln k] =	0.484
Mean [$e^{\ln k}$] =	0.31
High [$e^{\ln k + \sigma \ln k}$] =	0.51
Low [$e^{\ln k + \sigma \ln k}$] =	0.19



Clay, Sc

LOGNORMAL DISTRIBUTION

Wolff, July 1993

K is lognormally distributed

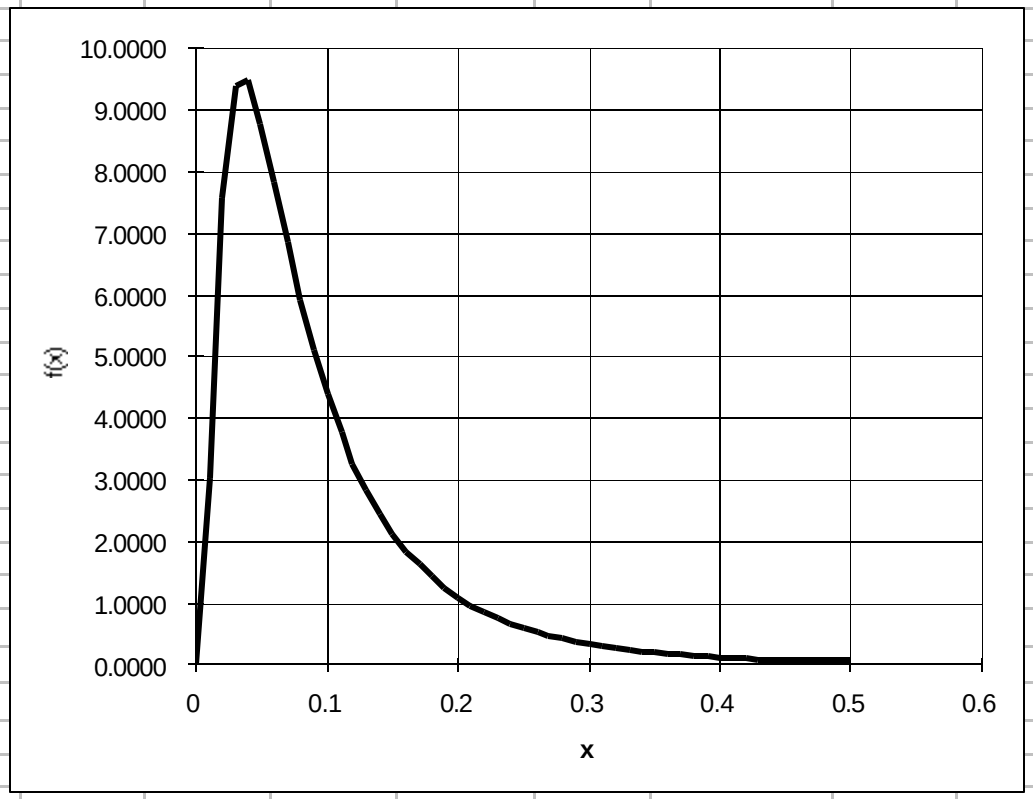
Y = ln K is normally distributed

Lognormal Parameters

E[k] =	0.10
σ[k] =	0.10
Vk =	1.0000
E[y] =	-2.6492
σ[y] =	0.8326
k - min =	0.00
k - max =	0.50
Increment =	0.0100

Normalized Parameters

E[ln k] =	-2.649
σ [ln k] =	0.833
Mean [$e^{\ln k}$] =	0.07
High [$e^{\ln k + \sigma \ln k}$] =	0.16
Low [$e^{\ln k + \sigma \ln k}$] =	0.03



4.9. Summary Spreadsheets for Internal Stability Data

5. PROJECT GEOLOGY

5.1. Geologic History of the Lake Okeechobee Region

Most of the geologic history significant to the current study of Herbert Hoover dike has occurred in the last 1,000,000 years. The lake Okeechobee region is part of the Florida Platform which is a stable structure 400 miles wide and 600 miles long. The lake Okeechobee region has historically been a local depression within that structure. The driving force for the history of the region is the 4 major and numerous minor glaciations that have occurred in the last million years. As each glaciation developed, sea level fell and the lake Okeechobee basin became a fresh water lake in which fresh water sediments were deposited. When the glaciers retreated, sea level rose and the sea covered the Okeechobee basin. Marine sediments were then deposited. This cycle was repeated time and time again forming alternating fresh water deposits and marine deposits. The depositional sequence is imperfect because of erosional periods that would remove some of the previous depositions during each glacial cycle.

The continuous changes in sea level led to conditions that encouraged the formation of caliche, hard pans, and cap rocks within the sediments. This process has created hardened seams, stringers, and rock layers within the sediments at Herbert Hoover Dike. The cap rock and hardened seams are common in the Fines Horizon which typically underlay the peat horizon.

Currently we are recovering from the last glaciation. Sea level is rising and Lake Okeechobee is a fresh water lake. When sea level rises another 20 feet, Lake Okeechobee will once again become a marine environment.

5.1.1. Geologic Units. Geologic Units as reported by the USGS (USGS, 1971) are as follows.

ORGANIC SOILS. Holocene. 0-10 feet thick. Peat. Low permeability.

LAKE FLIRT MARL. Pleistocene. 0-10 feet thick. Sandy marl. Low permeability.

TERRACE DEPOSITS. Pleistocene. 0-10 feet thick. Quartz sands. Low permeability.

FORT THOMPSON FORMATION. Pleistocene. 0-30 feet thick. Alternating marine and fresh-water limestones and/or marls. Variable permeability; low in dense crystalline limestones and high in shelly limestones.

CALOOSAHATCHEE MARL. Pleistocene. 0-30 feet thick. Shell, sandy clay, and sandy limestone. Variable permeability; high in shell beds and low in clay.

TAMIAMI FORMATION. Miocene. 30 to 110 feet thick. Clay, sand, and sandy limestones. Variable permeability; high in sandstones beds and low in sands and clay.

5.2. Geology of Reach 1 Area, Herbert Hoover Dike.

5.2.1. Geologic Units in Reach 1

5.2.1.1. Peat Horizon. Typically black, sometimes brown in color. May be fibrous to intensely decomposed. The peat horizon is sometimes identified as an organic silt. The peat horizon is continuous and widespread. Samples taken in the Everglades by the USGS tested to be 5,000 years old. The Peat Horizon is about 8 feet thick at Belle Glade and gradually thins out both to the northeast and west. An absence of the peat horizon in Reach 1 is usually an indication that it was locally excavated. It sometimes appears to be too thick, too thin, or it is found out of the natural geologic sequence; this is usually the result of local excavations, fill placement, or spoil disposals. The peat's absence at some locations is the result of local topographic highs or sand ridges when the peat horizon was being formed. It is typically the upper natural material present. Any materials overlying the peat horizon in Reach 1 are probably fill.

The Peat Horizon corresponds to the ORGANIC SOILS described by the USGS (USGS, 1946).

5.2.1.2. Fines Horizon: Typically tan calcareous silts and clays. Often contain thin (inch/inches) indurated zones or horizons. The Fines Horizon is continuous and wide spread. It typically forms an impermeable layer between the Peat Horizon and the Rock Horizon. It is 10 to 15 feet thick at the southern end of Reach 1. It gradually thins to the north. At line 22 it has thinned to less than 5 feet thick. From line 22 to line 36, it is typically only a few feet thick and in some places it is absent. The Rock Horizon would then be in contact with the Peat Horizon.

The Fines Horizon generally corresponds to the FORT THOMPSON FORMATION described by the USGS (USGS, 1971).

An important feature of the Fines Horizon is a Limestone layer typically found at the base of the Peat Horizon. USGS has identified this formation as the Lake Flirt Marl. Refer to Figure H-5.1. This is a thin crystalline limestone typically a few inches to 2 feet thick. It is often not identified in core boring logs. In core logs that did not identify this Limestone, its existence can often be inferred by high blow counts encountered at the base of the peat. In core logs not showing the high blow counts, the limestone may be thinner or weathered. It may have been pushed out of the way by the sampler, or it is locally absent. The limestone shows up in the spoil piles throughout the area in ditches excavated through the Peat Horizon. It is exposed in fields where the peat layer has been farmed out. The local people commonly refer to it as the “cap rock”.

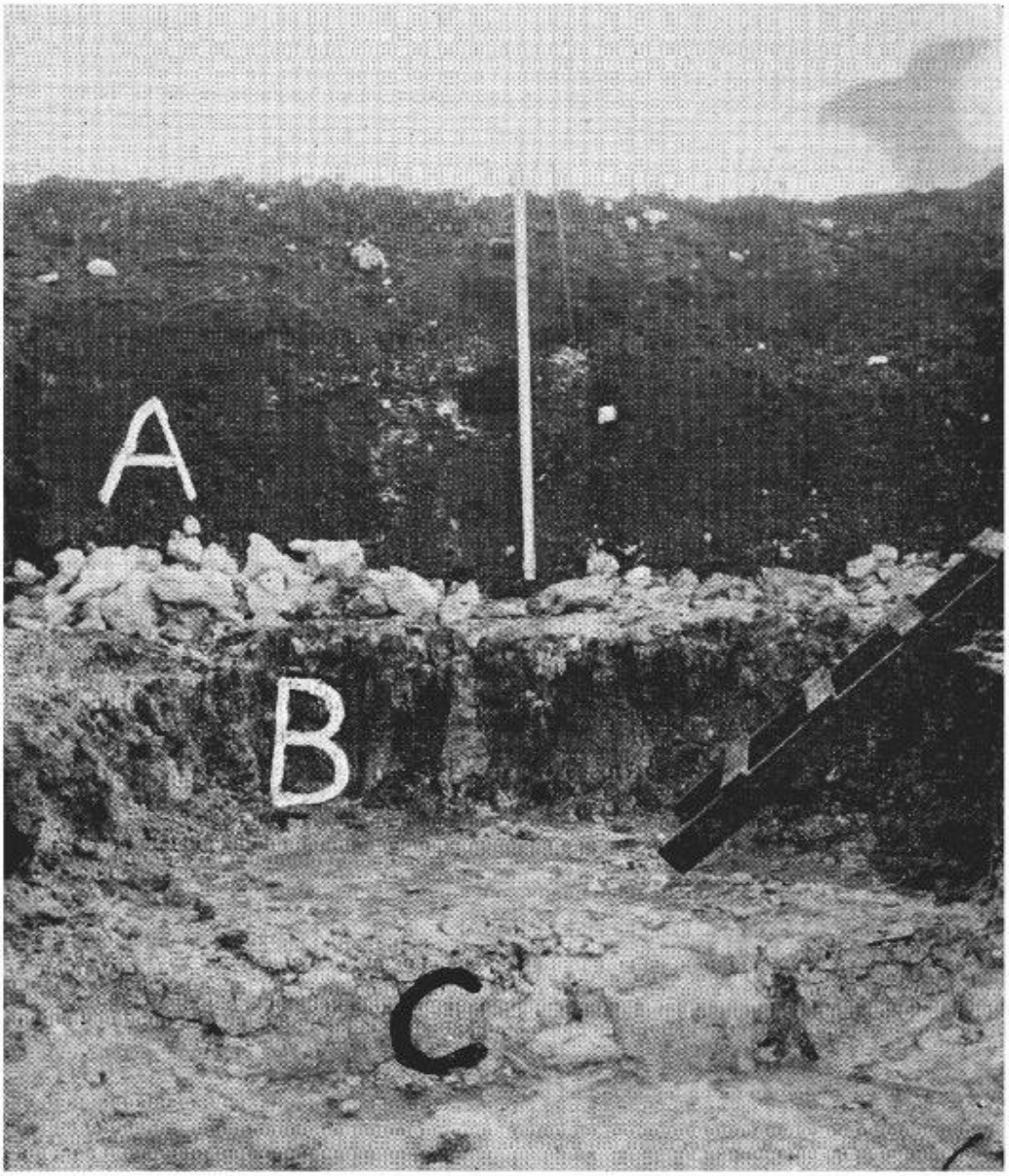


Figure H-5.1.-- Pit showing profile of sediments and peat in the Everglades near the northern part of the Hillsborough Canal. A. Saw-grass peat, 4 feet thick. B. Lake Flirt marl, 20 inches thick. C. Fort Thompson formation limestone.
[Courtesy U.S. Soil Conservation Service, Ft. Lauderdale]

In an investigation of levee L-8 (located east of Herbert Hoover Dike), this limestone was exposed at the base of the peat in a ditch. The limestone layer did not show up in the investigative core borings that were drilled. The driller was then told to use special care when drilling through the base of the peat and then the limestone was recovered. It was reported that the limestone layer was continuous for miles.

5.2.1.3. Rock Horizon. The Rock Horizon is continuous and wide spread. It is typically 10 to 20 feet thick. The top of the Rock Horizon gradually increases in elevation from south to north. At line 1, the top of the Rock Horizon is at elevation -8. At line 36 it risen to elevation +6 feet. The rock horizon is composed of limestone layers and sand layers. The limestones within the Rock Horizon vary from dense crystalline limestones to sandy limestones to shelly limestones. Some of the limestones are essentially impermeable while others are highly permeable, containing voids and solutioning features. There were sometimes sudden and complete losses of drill water when coring the limestone. In some areas the rock horizon is essentially all limestone. In other areas the limestone grades into sand deposits. Typically it appears as interbedded limestone and sand deposits.

The Rock Horizon generally corresponds to the CALOOSAHATCHEE FORMATION described by the USGS.

5.2.1.3. Sand Horizon: Greater than 20 feet thick. Typically fine to medium grained quartz sands and quartz silty sands. Usually has a significant shell component. Occasional shell layers are present. Thin limestone beds are common. Due to it depth, the Sand Horizon is generally not a factor in the evaluation of Herbert Hoover Dike.

The Sand Horizon generally corresponds to the TAMIAMI FORMATION described by the USGS.

5.2.2. Typical geology seen in Reach 1. Refer to Figure H-5.2.

Upper levee fill. 20 feet thick. Sandy/rocky materials

Lower levee fill 10 feet thick. Silty/clayey materials

Peat Horizon. 8 feet thick

Fines Horizon. 10 feet thick. Upper limestone, fines silts/clays, sandy silts and clays

Rock Horizon. 15 feet thick. Composed of limestone and sandy layers. Depending on location the horizon can vary from being predominately a Rock Horizon to being predominately sands.

Sand Horizon. 20+ feet thick. Sands with occasional layers of limestone.

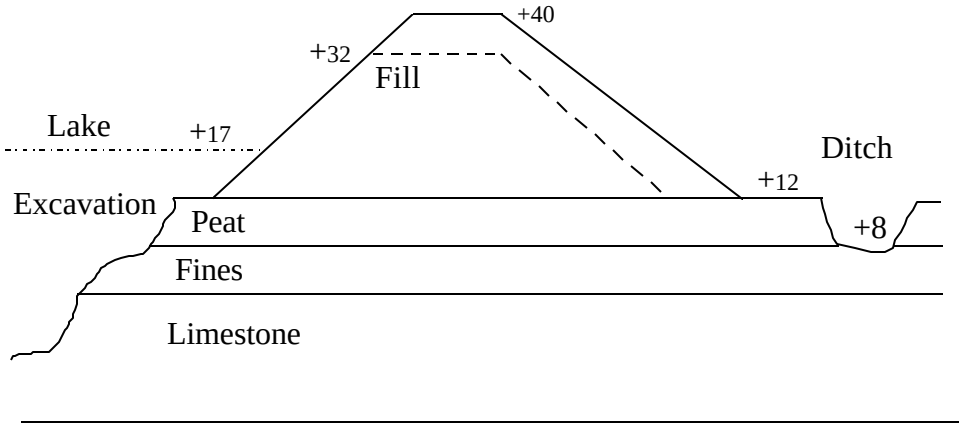


Figure H-5.2. General Geology in Reach 1.

5.3. Evaluation of Soil Classifications.

5.3.1. Comparison of Samples to Core Logs. Inspections of core samples were made at the District warehouse prior to the seepage model calibrations. Comparisons were made between the samples in the jar and the descriptions and classifications on the core logs. Where obvious errors existed, or another interpretation seemed more appropriate, a change to the model was made. The materials have been classified from samples recovered from core borings.

Hundreds of core borings have been drilled over the last 40 years. Care must be used when interpreting the core boring logs because the quality of the drilling/logging of materials can affect how the materials are classified. In general, the geology of the area is well behaved and predictable; but, if one were to select any isolated group of core boring logs, the geology may appear erratic or chaotic. There are 2 principal reasons for the apparent lack of correlation:

First, a significant number of core borings were drilled in disturbed soil horizons. Core borings drilled here would sample fill materials and/or in shoaling that is filling old borrow areas.

Secondly, we are dealing with hundreds of core borings drilled over a 40 year time period, drilled by a dozen different drillers and or agencies, logged by a dozen different geologists, and drilled by different equipment. The same materials can be classified differently.

5.3.2. Examples of Classifications Problems: The same material in the Fines Horizon could be classified as a sandy clay, a sandy silt, a clayey sand, a silty sand, or a marl by different geologists.

Two different core logs may classify an apparent sand layer. In one core boring we may determine that it is actually a limestone layer because it is in a horizon in which other nearby core boring logs have identified the limestone layer. In addition, high blow counts were encountered when drilling through this "sand".

In another core boring, there was also a high blow count; which could suggest that we have a rock layer rather than a sand; but, on closer examination we note that the material is a fine quartz (SP) sand in which it is not unusual to have high blows counts. Additionally, we note that surrounding core boring logs indicate that this is a high blow count sand deposit and not a rock horizon.

A specific example on how materials can be misclassified involves contract drilling done at the Herbert Hoover Dike. The contractor produced a core log (CB-HHDR1-30B) showing a 14 foot layer of limestone. Based on close examination of the core log and our experience of the area geology we determined that this 14-foot limestone layer did not exist. We told the contractor of our concern and required the contractor to redrill the core boring. The redrilled core logs indicated that the 14 Rock Horizon was now 5 feet of limestone and 9 feet of sand. Ultimately we required the contractor to redrill 25 core borings in which we had determined that materials were misclassified. It appears that the misclassifications were closely associated with the name of the person who drilled the core boring.

5.4. Embankment Materials

5.4.1. Excavation. Excavations adjacent to the dike were used to construct the embankment. Therefore, the embankment (fill) was built from the same materials the embankment sits on.

Where the foundation materials are predominately silts/clays, the levee was constructed out of silty/clayey fill.

Where the foundation materials were predominately sand, the levee was constructed out of sandy fill.

Where the foundation materials were predominately rock, the levee was constructed out of rocky/gravelly fill (potentially highly pervious).

Where the foundation materials were predominately shell, the levee was constructed out of shelly fill (potentially highly pervious). Shell (whole or sand to gravel sized fragments) is often present in other materials. There is both sand and sand with shell, silt and silt with shell, clay and clay with shell. There are shell and shelly horizons in which appreciable amounts of shell are present, regardless of the principal classification. Example: shelly sands, shelly silts, shelly clays, and shelly limestones.

5.4.2. Method of Herbert Hoover Dike Construction. The general construction method was to strip and spoil the peat from the continuous borrow area adjacent to the levee and then construct the levee using the borrow trench excavation. An exception to this plan is that some peat** was placed at the land side toe of the levee and some peat was probably also placed at the lake side toe of the levee. This was done to form longitudinal dikes used to contain hydraulically placed fill.

The embankment was placed as hydraulic fill. Hydraulic fill placement will separate coarse materials which settle out quickly from fines which settle some distance away. When dredging the Rock Horizon, the hydraulic fill placement would have created layers and lenses of gravelly materials in the Herbert Hoover embankment that would forms zones of high permeability within the embankment.

note: **The peat horizon was typically left in place beneath the footprint of the dike. Some of the peat under the footprint of the dam was probably pushed up to form the lateral dikes.

5.4.3. Inverted Fill Sequence: A typical section of the levee embankment can often be seen to be an inversion of the natural sequence of foundation materials. If the foundation is layered from top to bottom as :

peat horizon
silt horizon
limestone horizon
sand horizon

the levee fill will be the inverted sequence of that; from top to bottom:

sand horizon
gravel (limestone) horizon
silt horizon
peat horizon

5.5 Ground Water Condition In Reach 1.

5.5.1. General. The report by USGS, 1971 presents a detailed study of the seepage beneath the Dike and its effect on the ground water table.

There are three principal factors controlling the surface ground water table in Reach 1. These are Lake Okeechobee, the drainage canal complex, and rainfall.

Before the construction of Herbert Hoover Dike and the extensive drainage control ditch system, the project area was part of an extensive flow-way which transported water from the Kissimmee River watershed to the Everglades in the south. The project area was an extensive wetland with Lake

Okeechobee enlarging and flooding onto the marshy plains during wet periods. During dry periods, Lake Okeechobee would shrink in size.

With the construction of the Herbert Hoover Dike system, Lake Okeechobee became a flood control reservoir holding back flood waters. With the addition of the extensive drainage canal system, the ground water of the marshlands south of the Lake Okeechobee was lowered to the point where agricultural activities were possible.

The current surface water table is controlled by operations of the drainage ditches which are operated for the benefit of agricultural interests. During rainy periods, when the water table rises, water is directed out of the agricultural lands and pumped into Lake Okeechobee. During dry periods, when the surface water table falls and the fields dry out, water is taken from Lake Okeechobee to irrigate the fields. Excess waters not needed for agriculture are discharged through control structures and principal canals out of the region.

5.5.2. Underseepage from Lake Okeechobee. Underseepage from the lake has minimal affect on the ground water table in the agricultural lands. Normally, Lake Okeechobee is higher than the controlled water tables in the agricultural lands. Seepage would then occur under the Dike towards the agricultural lands. The seepage paths are principally through limestone layers and shelly horizons. The limestones and shelly horizons were exposed to direct contact with the Lake waters by deep borrow areas within the Lake that were excavated for the construction of Herbert Hoover Dike.

There is often a substantial decrease in seepage from the Lake by the silting in of the reservoir. Over time, fines and muck settle in the deep borrow area excavations and form a barrier to seepage. Another process known as filtercaking occurs where seepage waters carry fines with them that plug the voids that are carrying the seepage. Our piezometric studies indicate that the silting in of the reservoir and the filtercake process is significantly restricting seepage under the Dike. For seepage under the Dike, two paths are possible:

The first path is where the seepage in the limestone beds and shell layers is confined by the Fines Horizon and maintains artesian pressures. The seepage is discharged gradually some distance away in the agricultural lands. A variation on this model is where the limestone and shell layers act as drains (no artesian pressures) and the Fines Horizon allows a perched watertable condition to exist. An example of this condition occurs at culvert C-10A where the piezometric pressures below the Fines Horizon are lower than the water level in the canal. The subsurface water is draining toward the fields some distance away.

The second path is where the seepage finds a break in the Fines Horizon at the toe of the Dike and the waters immediately discharges to ditches or lakes or canals located at the toe of the levee. The breaks in the Fines Horizon are caused by excavation (canals, ditches, lakes, a quarry, ditches) through the Fines Horizon, or in some cases by artesian pressures that were sufficient to force open paths through the Fines Horizon.

5.5.3. Landside Water Surface Elevations. This is a complicated issue. For the purposes of this report, the water elevations of concern are those in the ditches running parallel to the embankment. These are called by several names, "toe ditches", seepage collector ditches", etc. Except for very few areas, no ditches are instrumented. Therefore, no comprehensive data exists. In an effort to provide the best estimates of landside water surface elevations between Clewiston and Port Mayaca, a meeting of representatives from CESAJ, South Florida Water Management District (SFWMD), and Chapter 298 Drainage Districts was held at CESAJ's South Florida Operations Office (SFOO) in Clewiston on 24 September 1996. Attendees included Ron Graydon (Superintendent, Ch. 298 Drainage Districts), Steve Ciulla (SFWMD), Marlyn Harn (SFOO), Tony Dipiero and Sam Honeycutt (CESAJ-EN-GS), Jim Vearil and Sue Sofia (CESAJ-EN-HW), and Pete Grace (CESAJ-EN-HC). Results of the meeting and subsequent site inspections are summarized below. It should be noted that topics of discussion included the Everglades Protection efforts. Although proposed water management strategies involve increased flow diversion to stormwater treatment areas south of Lake Okeechobeechobee, it was agreed that local drainage districts would still need to pump waters into the lake under certain conditions.

5.5.3.1. No-Pumping Threshold Lake Level. The Chapter 298 Drainage Districts' pump stations are not operational if the level of Lake Okeechobee equals or exceeds 19 feet, NGVD. These pump stations typically pump water into a relatively small diked containment area adjacent to the landside face of Herbert Hoover Dike. The diked area is connected to the lake by a flap-gated culvert; therefore, as the water surface elevation in the diked containment area increases to a level which exceeds the lake level, the flap gate opens and water passes into the lake. The 19 foot, NGVD, pumping limitation is related primarily to the elevation of the spur dikes which surround the pump outflow containment areas. Due to the spur dike crest elevations, if the lake stage equals or exceeds 19 feet, the water surface in the containment areas can not be raised to levels needed to force open the culvert flapgates; therefore pumping operations would be discontinued under such conditions. It should be noted that during the high water events of 1994, New Hope Sugar pumping operations at Culvert 12A were unable to pump against the 18.6 foot, NGVD, lake stage. As a result, fields in that area were flooded.

5.5.3.2. Head Criteria. Critical heads at specific Chapter 298 pump stations were identified as:

East Shore DD PS at Culvert 12 - 8 feet
South Shore DD (Bean City) PS at Culvert 4A - 12 feet
East Beach DD PS at Culvert 10 - 12 feet
South FL Conservancy Dist PS P-5-N at S-236 - 19 feet
New Hope Sugar PS at Culvert 12A - 18 feet

These heads define the maximum difference in intake and outflow water levels against which the pumps are capable of operating.

5.5.3.3. Clewiston Industrial Canal East to S-236. Landside water levels in this area are dependent on the toe ditch which parallels the dike. Water levels in that toe ditch are controlled by the inverts of approximately 6 culverts which convey water southward under the highway. During wet periods, the least possible toe ditch water level would be equivalent to the lowest culvert invert elevation. CESAJ-EN-GS personnel indicated that invert elevations for specific culverts are available in their records (DOT drawings). During extreme conditions (e.g., when piping problems are a concern), the head across the embankment could be reduced by blockage of the highway culverts, thereby raising the toe ditch water surface elevation. No measured landside water level data is available.

5.5.3.4. S-236 East to S-3. In this area, a toe ditch lies between the embankment and highway 27. This ditch drains westward; therefore, toe ditch water levels are controlled by the invert elevation of a culvert which links the ditch to the C-3 intake basin. Approximately 8 culverts, with invert elevations around +10 feet, NGVD, provide drainage under the highway. Measured water level data should be available from staff gages at S-236, C-3, and South Florida Conservancy PS-5-N. A new staff gage should be installed in the toe ditch east of C-3 when possible. At the east end of this area, the toe ditch follows the highway alignment and a swampy wooded area with a borrow pit, then a park/picnic facility, separates the embankment from the highway. The existence of a drainage connection between the borrow pit/wooded area and toe ditch should be investigated further. Under extreme conditions, blockage of the highway culverts and control of flow (via riser) into the C-3 intake should be considered for raising tailwaters.

5.5.3.5. S-3/S-354 East to Culvert 4A. Landside water levels in this area are again defined by toe ditch conditions. The ditch flows westward from C-4A and empties into Miami Canal through three 60(?)-inch diameter culvert barrels. Measurements recorded at S-354 can be used to identify toe ditch water levels. Approximately 17 culverts, many of which were apparently boarded shut in the fall of 1995 (and are now reopened), provide drainage under the highway. A slag toe berm (crest elevation 18 feet, NGVD) has been constructed along two sections of embankment in this area. The east and west berm sections are about 0.3 and 0.6 miles in length, respectively. During extreme conditions, tailwater elevations could be increased by blockage of highway culverts and control measures at the culverts linking the toe ditch to S-354.

5.5.3.6. Culvert 4A (Bean City PS) East to S-2. A toe ditch drains from S-2 westward and empties through a 78-inch diameter culvert into the intake basin for the Bean City pump station. Toe ditch water levels are controlled by the +3.65 foot, NGVD, culvert invert elevation on the west end. Measured water surface data is available from the intake staff gage at the Bean City pump station. During extreme conditions, toe ditch water levels may be maximized by control of flows through the culvert at Bean City pump station.

5.5.3.7. S-2/S-351 North to Culvert 12. Immediately north of S-351, there is no drainage ditch paralleling the embankment toe. The landside water surface conditions in this area are controlled by the local groundwater elevation. Sugar cane fields which border the embankment toe are drained by ditches which carry flows southward, then westward to Hillsboro Canal. The northern ends

of these north-south drainage ditches originate at the embankment toe; therefore, the best approximation of landside water levels is equivalent to the water level in the north end of these ditches. During the site investigation, water in the ditches suggested that the water table was about 2 to 3 feet below ground elevation at the toe. A staff gage at the north end of one of these north-south drainage ditches would provide valuable information relative to landside water levels in this area, which extends for approximately 6,000 feet north of S-351. North of this area, a toe ditch collects drainage from as far north as Culvert 12 and carries it south to Hillsboro Canal. Pump stations in this area are capable of pumping internal drainage canals down to levels lower than water levels in Hillsboro Canal; therefore, characteristic elevations (invert and top of bank) of the toe ditch should be used to approximate landside water levels in this area. A rock quarry is located at the extreme north end of this area. Water levels in the quarry pit best define the landside conditions immediately south of C-12; however, little information is available.

5.5.3.8. Culvert 12 North to Culvert 12A. A toe ditch extends along this entire area, from Paul Bardin Park to C-12A. This ditch collects seepage and runoff between the dike and highway 715. Site inspection revealed only one culvert which passes flows beneath the highway. A small pump and staff gage are located at the extreme north end of the toe ditch. When operated, this pump passes water from the toe ditch into the intake basin for the New Hope Sugar Pump Station. Measured toe ditch water level data is available from the staff gage mentioned above.

5.5.3.9. Culvert 12A North to Culvert 10. Landside water levels in this area correspond to conditions in a toe ditch which borders the Palm Beach County Glades Airport. The ditch drains to the south and empties through a culvert into the New Hope Sugar Pump Station intake basin; therefore, when toe ditch water levels exceed +7 feet, NGVD, they correspond to measured intake water levels at the pump station. During extreme conditions, tailwaters could be maximized by control of flows through the culvert at the south end of the toe ditch.

5.5.3.10. Culvert 10 North to Okeechobee State Park. A toe ditch extends along the base of the embankment throughout this area. The ditch drains from Okeechobee State Park southward and empties through a 36-in diameter French drain system into the intake basin of the East Beach Water Control District Pump Station 1, where water level measurements are recorded; therefore, when toe ditch water levels exceed +10 feet, NGVD, they correspond to measured levels at the pump station intake. During extreme events, tailwaters could be maximized by control of flows at the culvert which links the French drain system to the pump station intake basin.

5.5.3.11. Okeechobee State Park North to S-352. North of Okeechobee State Park, a toe ditch conveys flows northward to the West Palm Beach Canal at S-352; therefore, S-352 water level measurements will be used as indicators of the toe ditch water surface elevations. Along approximately half of this 3 to 4 mile drainage zone, a Florida East Coast Railroad track parallels the embankment. Under extreme circumstances, measures to raise tailwaters in this area could include ponding of water between the railroad and embankment (i.e., the railroad bed might be used as a sublevee).

5.5.3.12. S-352 North to Culvert 13. In this area, a toe ditch transfers flows from a point just south of C-13 southward directly into the West Palm Beach Canal. S-352 water level measurements may be interpreted as toe ditch water levels.
Tailwater control could be attempted during extreme conditions through blockage of flows at the toe ditch intersection with West Palm Beach Canal.

5.5.3.13. Culvert 13 North to Culvert 10A. A toe ditch controls drainage from C-13 north to C-10A. Flow proceeds from south to north and empties directly into L-8 Canal at C-10A; therefore water level measurements at C-10A may be used to approximate water levels in this toe ditch. It should be noted that the installation of a gated structure (e.g., stop log riser or screw gate) on each side (north and south) of Culvert 13 would provide beneficial toe ditch water level control during extreme events. Since CESAJ controls toe ditch outflows (i.e., at S-352 and at C-10A), the gates would allow for higher tailwaters in the toe ditches with no detrimental effects to the Chapter 298 controlled drainage which is transferred to C-13 from the east.

5.5.3.14. Culvert 10A North to Port Mayaca. Drainage along the base of the embankment is controlled by a toe ditch between C-10A and Port Mayaca (i.e., St. Lucie Canal). In this area, the Florida East Coast Railroad track and/or highway 98 parallel the dike alignment. During periods of excessive rainfall, this area is subject to ponding; therefore, landside water surface elevations will approach (and sometimes exceed) the elevation of the highway crown. During normal conditions, the landside water surface elevation can be approximated as the toe ditch bottom elevation. Measured water level data at culverts C-14, C-16, C-11, and S-308-B (Port Mayaca Lock) may provide additional insight relative to landside water levels in this area.

5.5.3.15. During the 25 September 1996 return to Jacksonville, CESAJ-EN-H personnel made tailwater site inspections at various locations between Clewiston and Okeechobee (i.e., along the west shore). Those areas are typically characterized by a large landside borrow canal in which tailwater elevations are controlled by USACE water control structures.

5.6. Engineering and Geologic Features Seen Along Different Reaches of the Herbert Hoover Dike. Following is a list of significant features we have observed being present at Herbert Hoover Dike. At any location along the levee one or a combination of the listed features may exist.

Refer to Figure H-5.3 to key numbered items to locations.

5.6.1. Peat/Limestone Interface. A 6-inch layer of limestone at the base of the peat is a potential path of seepage/erosion. This peat/limestone condition is very wide spread. Sometimes the limestone is shown on the core boring logs. Sometimes the limestone only shows up as a high blow count seen at the base of the peat. Sometimes there doesn't appear to be any evidence of the limestone layer at the base of the peat. We believe it can exist even where there is no indication of it seen on the core logs.

5.6.2. Artesian Flow. Some piezometers in the limestone at the toe of the dike show artesian conditions. The existence of artesian pressures indicates that the Fines Horizon is not breached. The artesian pressures have a potential to heave the levee toe.

5.6.3. Blown Ditch. Piezometers show direct connection to the deep limestone. The Fines Horizon has been breached.

5.6.4. Ditch Drawdowns. In some areas, pumps can rapidly drawn down land side ditch water levels and increase differential heads between the lake and the ditch. Pumping the ditches down during a high lake stage could fail the levee.

5.6.5. Blocked Land Side Toe. Clayey land side toe fill blocks seepage through the embankment and creates a high piezometric level within the embankment.

5.6.6. Soft Toe, Springs. The blocked land side toe results in a soft toe and water seeps at the land side toe of the embankment. These are indications of high piezometric levels at the toe of the levee. Soft toes can be located by driving a vehicle along the toe of the dike; you have located a soft toe when you bog down. They can also be located by the grass mowing patterns; the tractors avoid these areas and leave patches of unmowed grass.

5.6.7. Gravel Layers. Gravel and shell layers in the embankment create seepage paths. These layers come about because the borrow area used to construct the levee contained rock and/or shell horizons and the material was placed as hydraulic fill.

5.6.8. Pervious Zones. Piezometers show sections of the embankment are highly pervious. This is probably related to gravel/ shell layers, erosion or voids. The size of these pervious zones can not be determined. Their existence is principally a function of the materials that were locally available at any given time when the fill was being placed by hydraulic methods. Shell deposits are common at the site and significant portions of the levee would have been built out of highly shelly material. Additionally, rock deposits placed as fill would produce a gravelly embankment.

5.6.9. Soft Embankment. Soft embankment zones are identified by abnormally low blow counts encountered during drilling. They could be related to non-compaction of the materials due to an arching effect that prevents compaction of the foundation materials. They could also be an indication of internal embankment erosion which has removed material from the foundation creating seepage/piping paths.

5.6.10. Soft Foundation. Soft foundation zones are identified by abnormally low blow counts encountered during drilling. They could be related to non-compaction of materials due to an arching effect or could be an indication of foundation erosion which has removed material from the foundation creating seepage/piping paths.

5.6.11. Filtercake Control. Deep borrow excavations in the lake connect the deep limestone directly to the lake. Subsequent filling of the borrow area excavation by shoaling and muck usually seals off the limestone and limits seepage under the embankment.

5.6.12. Clayey Upstream Toe. Clayey lake side toe acts as an upstream clay blanket. This feature is a result of the original construction methods used to construct the levee. Longitudinal dikes were pushed up from whatever materials (including clayey/silty/peaty materials) were locally available to act as a containment dike to hold the hydraulic fill being placed.

5.6.13. Overflow. Blocked clayey upstream toe acting as a clay blanket causes the embankment to have a low piezometric surface until water overtops the clay. Then there will be a step increase in the water level. A section of levee that looks good at a given elevation can have significantly different performance characteristics when the clayey upstream toe is overtopped.

5.6.14. Sinkholes. Numerous sinkholes are seen along the crest of the levee in the lake Harbor area. This occurred in the same reach of the embankment where significant piping of materials occurred at the toe of the levee during the 1995 high water event. Sand is settling into the gravelly layers deeper in the embankment, filling up voids and/or the material is being piped out of the embankment.

5.6.15. Quick Conditions. Low blow counts at the toe of the levee indicate an active quick condition (unstable) and that materials at the toe are at the critical piping condition. See the Miller Site "C" core borings.

5.6.16. Deep Excavations. Deep excavations at the land side toe of the levee expose the Rock Horizon. This shortens the seepage paths and increases the seepage potential. Additionally, since seepage paths would be under water, we are not able to monitor any damages that may be occurring. A serious erosion problem could be occurring under the Herbert Hoover Dike without any observable signs. A dike failure could occur without any warning.

5.6.17. Fire Toe Trench. A construction feature that breaches the peat to prevent peat fires from burning under the embankment. They provide a direct connection from the fill to the limestone at the base of the peat. This allows a seepage path to jump from the embankment to the peat/limestone interface or visa versa.

5.6.18. Seepage Trench. An unusual construction feature that breaches the peat on the upstream toe of the levee similar to the fire toe trench. This may also allows a seepage path to jump from the embankment to the peat/limestone interface or visa versa

5.6.19. Low Density Peat Horizon. The peat located at the toe of the levee will heave/float with high tail water.

5.6.20. Two Phases of Levee Construction. The embankment was built to elevation +34 in 1937. The embankment was raised to +38 feet in 1964. This adds more complexity to understanding the embankment.

5.6.21. Cemented Horizons. Cemented horizons within the Fines Horizon could serve as roofs that would allow erosion paths to form and propagate horizontal piping paths.

5.6.22. Erodeable Shell Layer. An erodeable shell layer composed of very small rounded shell about the size of coarse sand. This thin, wet layer was recovered, but not logged, in borings CB-HHDR-6B and CB-HHDR-6C (Reach 1, Line 6, quarry site). There is no reason to expect it is limited to this single occurrence. It could easily be overlooked in other core borings. The material has no cohesion and its shape and uniform size would make it easily erodeable.

5.6.23. Power Pole / Shallow Well Installations. These man made features breach the confining layer and provide a path for water under artesian pressure to erode materials. At the Florida Power & Light embankment failure, there were power poles installed at the downstream toe of the embankment in the area where the breach occurred. These power poles would have penetrated a sandstone layer that was probably associated with the embankment failure. (Re. FPL Dike failure location slides - before & after).

5.6.24. Horizontal Conduits Through the Enbankment. Either culverts or water supply pipelines active or abandoned. Some are unknown (S-352N site) and can provide a path for piping.

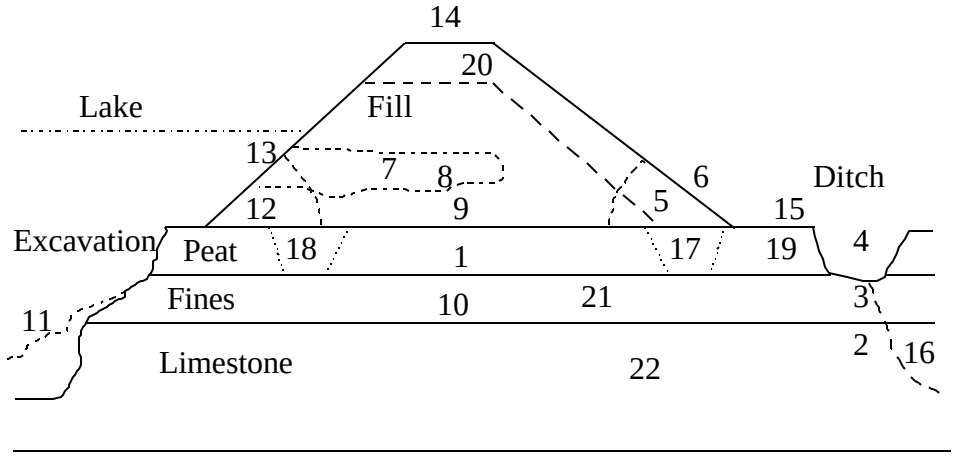


Figure H-5.3. Engineering and Geologic Features.

6. PIEZOMETERS

6.1. Generalized Discussions about the Piezometer. We have installed 61 piezometers at Herbert Hoover Dike in group sizes ranging from 1 to 9 piezometers each in each piezometer line. Typically a piezometer array will have 4 piezometers with 1 shallow and 1 deep piezometer at the centerline of the Levee and 1 shallow and 1 deep piezometer at the toe of the levee. Due to limited resources available for installing piezometers and the great length of the levee, we typically restricted our piezometers to 4 piezometers per line. Four (4) piezometers (we actually usually have 6 data points per line when we include the lake level and the tail water.) provide limited data along any individual levee section. On the other hand we have been able to recover significant data with the 4 piezometers.

We typically used piezometer zones of about 5 feet long. While we would prefer smaller zones (such as 1 foot long), we used 5 feet zones because of uncertainties during installation. Our field people had limited experience installing precision piezometers. The geology as understood at the time of installation was often uncertain. There would be uncertainties about the piezometer being in the target horizon and on whether the benetonite seals would be affecting the target zone. Our installation plan included having representative reading above and below the confining layer, intercepting the peat/limestone interface and/or be placed in the lower limestone horizon.

6.2. Piezometer Evaluation. In the evaluation of the piezometers we considered all the available including information such as:

Piezometer readings
Changes in piezometer reading over time
Changes in piezometer reading as related to changing lake levels
Flow theory
Piezometric computer models
Lake levels
Tail water levels
Construction history of the levee
The geology from the core logs.
As build levee sections
Surface observations during drilling and high water inspections.
Topography.

When all the sources of information were combined, significant interpretations of the piezometric data could be made and detailed inferences could be made about the levee structure and performance. Individual interpretations of the Piezometers/Geology for each Piezometer array are given in Section 6.4.4.

6.3. Piezometric Models. Following is the presentation of 8 models that represent all the basic piezometric conditions we have observed at Herbert Hoover Dike. Each model uses the

typical section of the geology that exists along the south and southeast portions of Herbert Hoover Dike.

6.3.1. Model 1a. The embankment piezometer read the same as the lake elevation. In this model, the piezometer reads the same as the lake because the downstream toe of the levee was constructed out of clayey materials which acts as a downstream clay blanket. The full hydrostatic head of the lake is acting on the downstream toe. In the field we may see springs and or soft ground at the downstream toe of the levee. One way to locate the soft ground is to drive a vehicle along the toe of the dike. When the vehicle bogs down, you have located one of these zones. Another way to locate these areas is to look for unmowed areas. The mowers avoid soft ground.

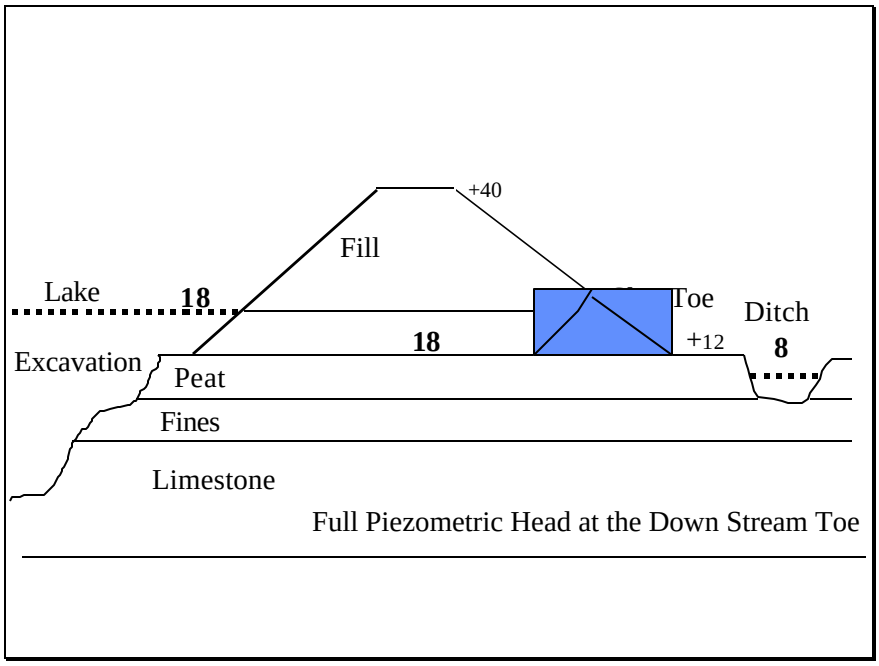


Figure H-6.1. Model 1a, Embankment piezometer reads the same as lake.

6.3.2. Model 1b. This model is similar to Model 1a in that the embankment piezometer reads the same as the lake elevation. In this model the piezometer reads the same as the lake because the core of the levee was constructed out gravelly materials which transmit, without appreciable head loss, the full hydrostatic head of the lake to the down stream toe of the levee. In the field we see wet areas, springs, and piping conditions at higher lake levels.

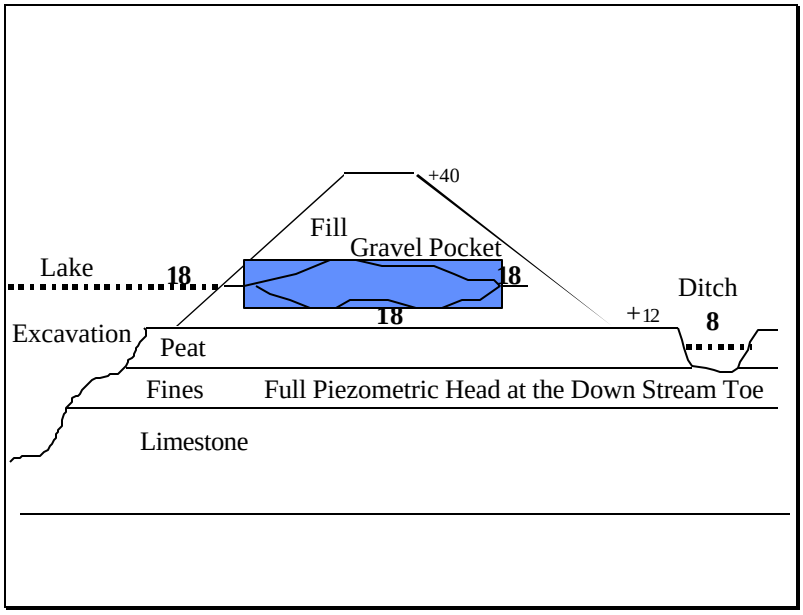


Figure H-6.2. Model 1b, Embankment piezometer reads the same as lake.

The 1a and 1b models appear to explain the conditions observed during the 1995-98 high water events for areas where significant piping of levee materials was observed.

6.3.3. Model 2. In Model 2, we observe artesian pressures at the downstream toe of the levee. The "Fines" horizon acts as a confining layer. Lake waters enter the limestone horizon which transmits artesian pressures to the downstream toe of the Dike. This creates a potential heave situation at the toe of the dike.

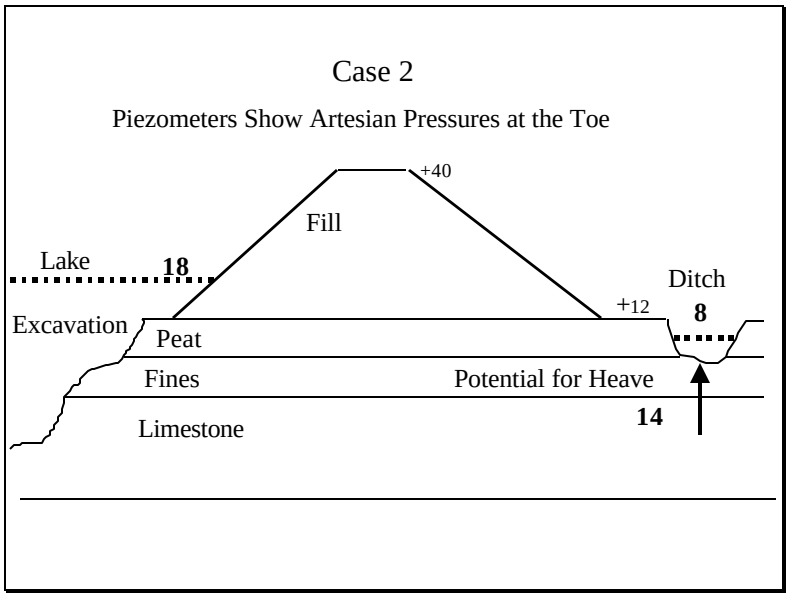


Figure H-6.3. Model 2, Piezometer shows artesian pressures at the toe.

6.3.4. Model 3a (See Section 5). In Model 3a, we observe that the foundation piezometers at the toe of the levee read the same as the tail water. This indicates that the "Fines" horizon has been breached and is no longer acting as a confining layer.

The Fines horizon may have been breached by previous excavations or it may have heaved and/or fractured due artesian conditions. The fracturing/heaving permanently damages the fines horizon and relieves the artesian condition.

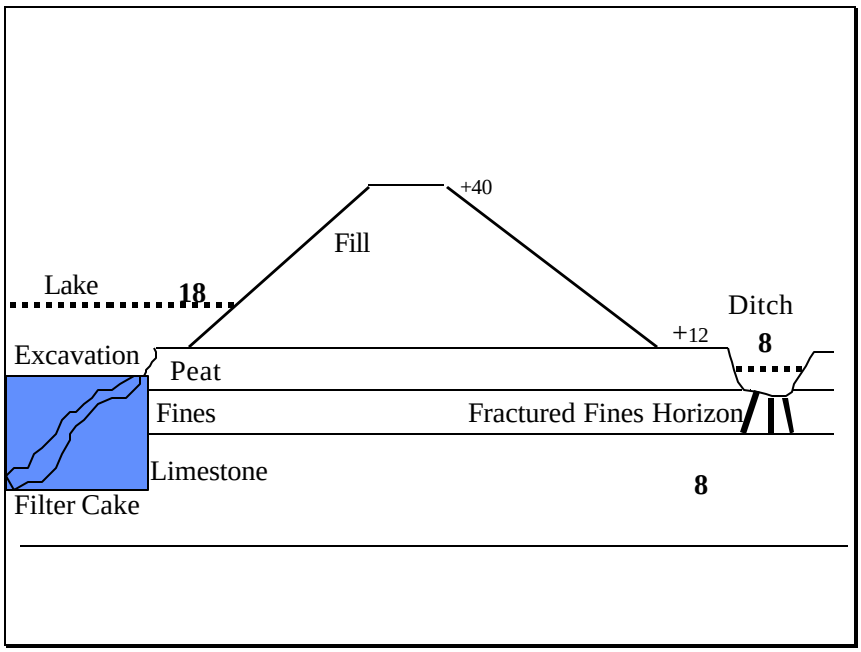


Figure H-6.4. Model 3a, Piezometer readings at the toe the same as the tail water.

6.3.5. Model 3b. In Model 3b all foundation piezometers read the same as the tail water or quarry lake. Deep excavations on both sides of the levee have breached all the geologic units. Silting in of the lake excavation has formed a Filter cake that partially seals the lake waters from the foundation limestones. The lack of head drop between the downstream toe and the centerline piezometer indicates there is piping damage (open pipes) in the foundation. This is a very dangerous situation.

According to modern piping theory, It is difficult for a pipe to grow from the downstream toe of the levee to the centerline of the levee; but, after the open pipe reaches the centerline of the levee, it would be relatively easy for the pipe to progress to the lake. If the pipe reaches the lake, the flow of water through the pipe will increase dramatically. This in turn could lead to massive internal erosion of the foundation and cause the levee to fail.

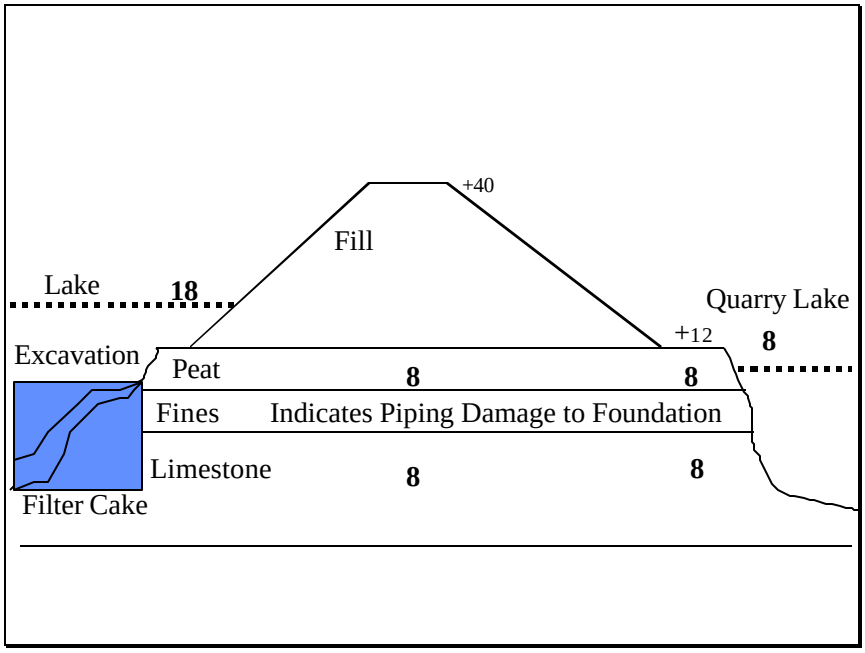


Figure H-6.5. Model 3b, All piezometers read the same as the quarry lake.

6.3.6. Model 4. In Model 4, we observe that the foundation piezometers are independent of both the lake level and the tail water elevation. This indicates the existence of a Filter cake in the lake the seals the limestone horizon from the lake, that the confining layer (Fines horizon) is intact, and that the pervious limestone horizon is acting as a drain to some distant point.

At present we don't know whether this is good or a potential problem situation.

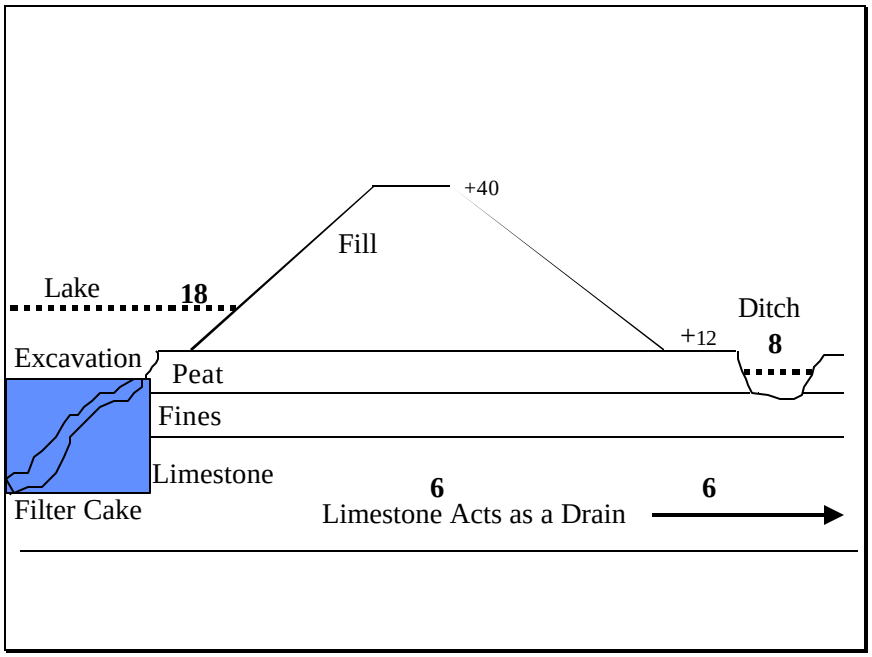


Figure H-6.6. Model 4, Piezometers readings are independent of the lake and tail water

6.3.7. Model 5. In Model 5, essentially all the head drop is occurring between the lake and the centerline piezometers. This is normally a good sign.

When the borrow area in the lake was originally excavated, the lake waters were in direct contact with the limestone horizon. The waters had significant hydraulic potential for piping materials out of the levee foundation. The natural silting/filling in of the borrow area excavation over time has formed a Filter cake that seals the limestone horizon from the lake. We expect the effectiveness of the Filter cake seal to increase over time. This will decrease the potential for piping.

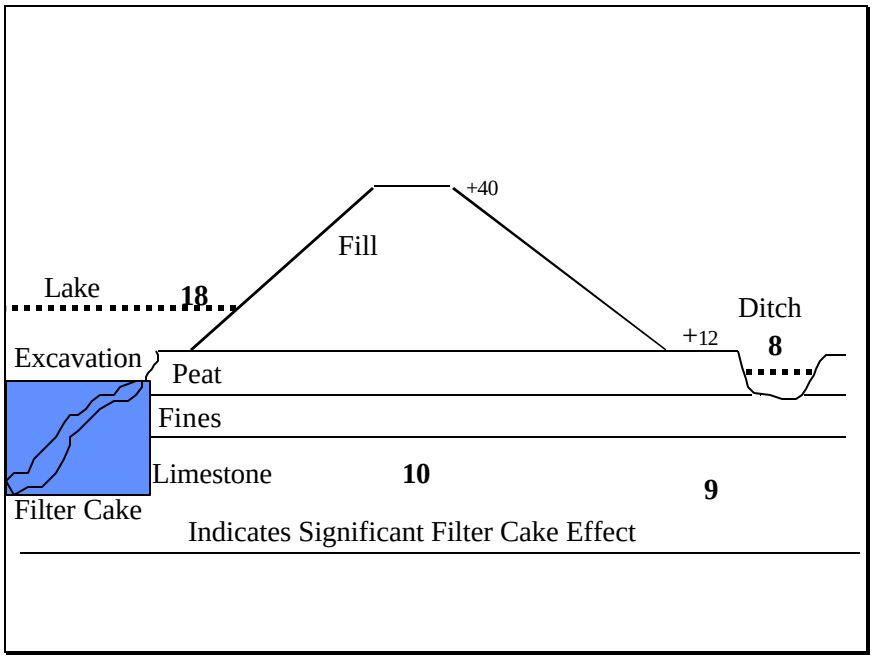


Figure H-6.7. Model 5, Most of the head drop occurs between the lake and the centerline piezometer.

6.3.8. Model 6. In Model 6, all the head drop is occurring between the lake and the centerline piezometer. The lack of head drop between the downstream toe and the centering piezometer indicates there is piping damage (open pipes) in the foundation. This is a very dangerous situation. According to modern piping theory, It is difficult for a pipe to grow from the downstream toe of the levee to the centerline of the levee; but, after the open pipe reaches the centerline of the levee, it would be relatively easy for the pipe to progress to the lake.

If the pipe reaches the lake, the flow of water through the pipe will increase dramatically. This in turn could lead to massive internal erosion of the levee/foundation and cause the levee to fail.

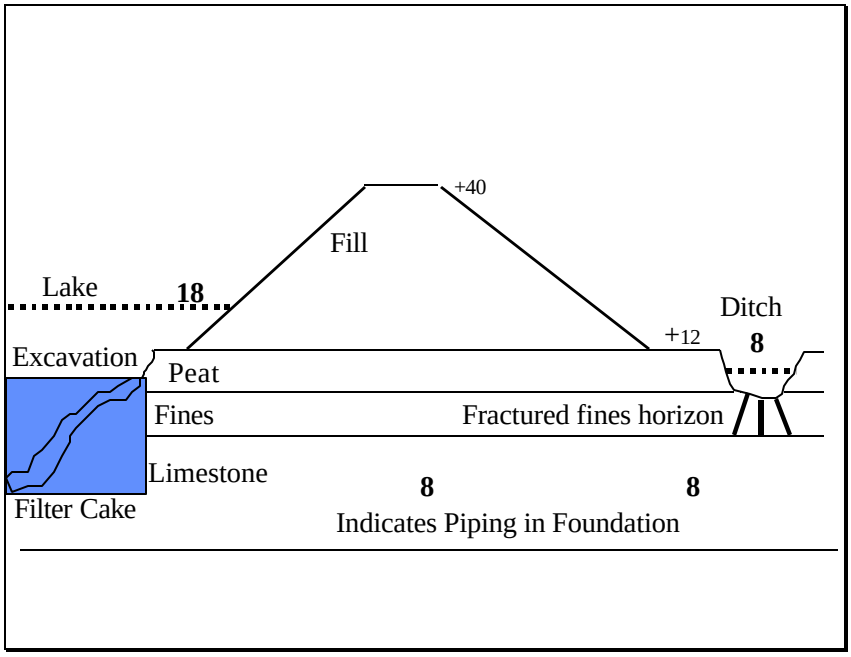


Figure H-6.8. Model 6, All of the head drops occur between the lake and the centerline piezometer.

6.3.9. Model 7. In Model 7, with the lake at elevation of +18 feet, the embankment piezometer show a dry embankment. This condition comes about do to an upstream clayey toe which seals the embankment from the lake waters until a critical elevation (say 19 feet in this model) is reached and the clay is overtopped and water flows into the embankment.

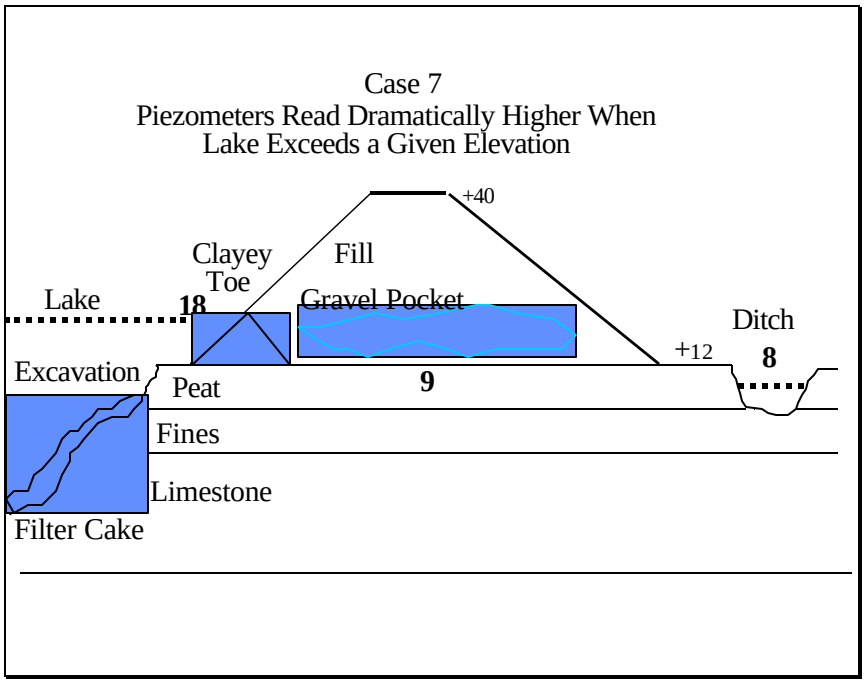


Figure H-6.9. Model 6, Piezometers read dramatically higher when lake exceeds a given elevation.

In this model, based on the historic dry readings of the embankment piezometer, the embankment would be determined to posed no risk of failure, and would not require remediation. However, at a slightly higher lake elevation, the clay would be overtopped, the embankment could fill with water and we could have an embankment on the verge of failure.

The point of this model is to demonstrate that we can never be certain on how the levee will perform at any given lake elevation unless we have experienced that lake elevation and have piezometric data for that lake elevation.

6.4. Piezometer Data.

6.4.1. General Description and Scope.

Piezometers were read once a month during normal conditions, including readings for the water level in ditches where applicable. The monitoring record begins in 1994 for the earliest installed piezometers and includes most by the end of 1995. A high water event occurred from January 1998 through March 1998. During this event piezometers were monitored weekly. The lake level is monitored daily at several structures in the project area.

6.4.2. Summary of Data. A summary is included as Table H-6.1.

Definitions of terms used are as follows:

Y Indicates presence of condition

N Indicates condition not present

? Indicates presence of condition could not be confirmed

R Indicates repair of damage from 95 HWE by construction of a seepage berm. Ditch was re-formed to a much smaller cross section and higher invert elevation. Also indicates presence of US 27 where large amount of peat was excavated and resulting excavation was likely dewatered for placement of road subbase fill.

Embankment Any fill material placed above the natural ground surface

Natural ground surface Top of deposition of natural material, usually identified as the top of natural peat formation

Foundation Top of natural peat formation and below embankment fill.

LS / Peat problem There is clear evidence the section has the Lake Flirt Marl limestone formation under the peat layer. For the purposes of this report this formation will be designated as ls-p.

Blown toe or ditch The toe of the embankment or toe ditch bottom at the toe has heaved and or hydro-fractured and the piezometers indicate a direct connection between the phreatic surface above the confining layer and the piezometric surface below.

Artesian Pressure Water levels in the piezometer are higher than the groundwater table.

Ditch drawdown Water in ditch can be drawn down by pumps or other means outside of COE control.

Blocked exit Relatively impervious (not free draining) zone at the downstream toe of the embankment which holds the phreatic surface high within the embankment.

Soft toe / springs Saturated embankment toe due to blocked exit. Generally associated with the presence of low SPT values (N generally 0 - 3) in the toe ("C") boring for 2 or more consecutive runs (more than three feet). Springs are the visible manifestation of the phreatic surface exiting the embankment.

Gravel layers in embankment Presence of gravel sized particles in centerline ("B") boring of shell or limestone deposited in a relatively homogenous layer by either hydraulic fill or dragline casting. Generally associated with large deposits of shell or limestone in the borrow area. These layers form a "short circuit" for both flow and head.

Loose zones in embankment Presence of low SPT values (N generally 0 - 3) in the embankment in centerline ("B") boring for 2 or more consecutive runs (more than three feet).

Soft zones in foundation Presence of low SPT values (N generally 0 - 3) in the foundation material in centerline ("B") boring for 2 or more consecutive runs (more than three feet).

Pervious upstream toe Presence of gravel noted in borings upstream of centerline.

Filtercake controls Filtercake is the deposit of fines on the upstream borrow canal sideslopes and bottom forming a relatively impervious layer. Filtercake isolates piezometers deep in the limestone from lake by dampening response.

Head upper LS: [Lake - b / b - c]; LL Difference in lake level and upper "B" piezometer reading compared to the difference between the upper "B" piezometer reading and upper "C" piezometer reading at lower lake levels (LL= el. 15.0). Where there is no upper "C" piezometer the elevation is estimated from the tailwater reading plus about 1 foot.

Head lower LS: [Lake - b / b - c]; LL Difference in lake level and lower "B" piezometer reading compared to the difference between the lower "B" piezometer reading and lower "C" piezometer reading at lower lake levels (LL = el. 15.0).

Spillover @ Lake = 18.5 Situation where the piezometers read differently (higher) due to overtopping of a blocked entrance condition. Since there have not been any sustained lake levels higher than elevation 18.5 feet (approximately), this condition can not be confirmed at higher elevations. Therefore, a "NO" indicates only that the condition has not been observed up to elevation 18.5 however, it may exist at higher elevations.

Head upper LS: [Lake - b / b - c]; HL Difference in lake level and upper "B" piezometer reading compared to the difference between the upper "B" piezometer reading and upper "C" piezometer reading at higher lake levels (el. 18.5). Where there is no upper "C" piezometer the elevation is estimated from the tailwater reading plus about 1 foot.

Head lower LS: [Lake - b / b - c]; HL Difference in lake level and lower "B" piezometer reading compared to the difference between the lower "B" piezometer reading and lower "C" piezometer reading at higher lake levels (el. 18.5).

Clayey u.s. toe / blocked entrance Relatively impervious zone at the upstream toe of the embankment which holds the phreatic surface low within the embankment.

Sinkholes A depression in the crest or slope resulting from the loss of material underlying the surface. In this case the crest has a crust about 1 - 2 feet thick resulting from the deposition of calcareous road base material which bridges the void until it reaches sufficient size that the crust collapses. In many cases the only visible manifestation is a depression in the crest such as a rut from vehicle traffic. Some of these depressions were probed and a void or very loose material was encountered.

Quick toe Locations where artesian pressures reduce the effective stresses to near zero, creating a semi-liquid soil which has little to no strength.

Head, lake - emb. @ centerline Difference in lake level and "B" piezometer reading where "B" piezometer is actually up in the embankment and not at the ls-p interface.

Upper Piezometers: Piezometers whose tip is set above the confining layer.

Lower Piezometers: Piezometers whose tip is set below the confining layer.

Confining Layer: The calcareous silt / clay layer below the peat present along most of the southern and eastern portion of the embankment. The significance of this layer is that is an aquaclude and separates the upper phreatic surface from the lower piezometric surface.

6.4.3. Reach 2 and 3 Piezometers.

6.4.3.1. Reach 2 Line F (R2-F). This section includes three piezometers in the vicinity of John Stretch Park. These piezometers were installed in September 1995. Piezometers PZ-R2-FB1 and B2 are located in the same hole in the crest of the levee. Piezometer PZ-R2-FC is located in the landward toe of the levee. Core boring data from four adjacent borings was also included in the analysis. These borings were located approximately 300 feet off the section line. A topographic survey also shows a ditch running along the landward side of the levee.

6.4.3.1.2. Upper Piezometer: PZ-R2-FB1. This piezometer is located in the embankment proper, and not the ls-p layer. Therefore, the piezometric level is indicative of the true water level in the embankment. This piezometer fluctuates with lake level indicating the presence of a blocked exit. Core boring CB-LD2-30 (in the center of the original embankment) indicates a large amount of limestone fragments as well as shelly layers. The borrow area borings indicates a large amount of shell and limestone present for use as fill. It would follow that there are gravel layers in the embankment. Borings also indicate the ls-p layer as present, although the main limestone formation is very close to the bottom of the peat due to a very thin (1-2 foot) confining layer.

6.4.3.1.3. Lower Piezometers: PZ-R2-FB-2 and PZ-R2-FC. Piezometers in the limestone exhibit a dampened response to fluctuating lake levels, indicating the presence of filtercake. The gradient direction of flow is down into the limestone, which appears to act as a drain. There is a deep excavation at the toe of the embankment. PZ-R2-FC does not respond to the lake, indicating that the toe is blown.

6.4.3.1.4. Concerns. There is likely some internal erosion occurring along the ls-p zone. CB-LD2-30 shows rods fell two feet directly above this zone and CB-HHDR2-FB shows zero percent recovery in the same zone. The toe of the embankment is very soft. A slope failure on the berm could breach the blocked exit. Should this occur, the full lake level in the center of the embankment would have a very short distance to drop to tailwater elevation which could lead to serious piping.

6.4.3.2. Reach 3 Line D._ This section includes one piezometer, PZ-ER3D-B, installed in December 1995 in the crest of the levee. Data was also available from two core borings one located at the site of the piezometer and the second at the landward toe of the levee along the same section line. A topographic survey was not available for this particular area, however data from adjacent areas was used.

6.4.3.2.1. Upper Piezometer: PZ-ER3D-B. This piezometer is located in the embankment proper, and not the ls-p layer. Therefore, the piezometric level is indicative of the true water level in the embankment. This piezometer fluctuates with lake level indicating the presence of a blocked exit. This piezometer was installed during the 1995 event at the Point 2 site. During this event heavy seepage was observed exiting the toe of the embankment. The

massive flow produced a very soft toe which was close to quick due to the volume of seepage. The soft toe also shows up in borings.

6.4.3.2.2. Lower Piezometers: N/A

6.4.3.2.3. Conclusions. The 1995 High Water Event produced a large area of heavy seepage at the toe of the embankment. A seepage berm was constructed of ASTM #10 stone as an emergency repair to prevent piping. It is interesting to note that after construction of the berm the phreatic surface in the embankment rose 1.5 - 2 feet to where it reads approximately lake level at lower lake elevations. During the 1998 High Water Event the phreatic surface lagged about 1 foot behind lake level. The berm was constructed to elevation 14.0. Readings indicate that lake levels above 14.0 produce some drainage through the berm. Observations during the 1998 event indicated a saturated berm but no piping. It should be noted that the flow source appears to be concentrated. After the initial repair and during low lake levels a persistent wet spot was noted. This 20 x 20' area received another 2' of #10 stone and the area remained dry, even during the 1998 event.

6.4.3.2.4. Concerns. The toe of the embankment is very soft. A slope failure on the berm could breach the blocked exit. Should this occur, the full lake level in the center of the embankment would have a very short distance to drop to tailwater elevation which could lead to serious piping. The concentrated flows through embankment base provided source for seepage. This would indicate pre-existing pipes or significant internal erosion in embankment. The existing berm was more "permeable" (in general) than the "permeable" drain made of #10 stone. This temporary repair should be upgraded to provide sufficient flow capacity.

6.4.3.3. Reach 3 Line A (Sinkhole Site). This section includes three piezometers; PZ-MRR3-SH-B1 and B2 installed in the crest of the levee and PZ-MRR3-SH-C in the landward toe. These piezometers were installed in October and November 1995. Core boring data from nine adjacent borings were including in the analysis, including the sites of the piezometer installation. A topographic survey also shows a ditch running along the landward side of the levee.

6.4.3.3.1. Upper Piezometer: PZ-MRR3-SH-B1. This piezometers is functioning, although the tip is set high and has remained dry.

6.4.3.3.2. Lower Piezometer: PZ-MRR3-SH-B2. The deep piezometer loosely follows lake. The filtercake still has an impact, however, the excavation for U.S. 27 may function as drain during higher lake levels. This excavation is where the peat was excavated and replaced with other fill to provide a suitable road base.

6.4.3.3.3. Concerns. There is an unusual low blow count encountered in upstream toe. The large sinkhole indicates internally unstable materials. The embankment is highly pervious as evidenced from 95 High Water Event. Highly erodeable materials are present which creates a dangerous situation. The seepage berm places as an emergency repair

should be re-evaluated to determine its adequacy at higher lake levels. The toe of the embankment is very soft. A slope failure on the berm could breach the blocked exit. Should this occur, the full lake level in the center of the embankment would have a very short distance to drop to tailwater elevation which could lead to serious piping.

6.4.3.4. Reach 3 Line E. This section includes one piezometer, PZ-ER3E-B, installed in December 1995 in the crest of the levee. Data was also available from two core borings one located at the site of the piezometer and the second at the landward toe of the levee along the same section line. A topographic survey was not available for this particular area, however, adjacent areas were available. This line has one deep piezometer under the crest (PZ-ER3E-B). This site is very similar to sinkhole site. The main exception is that loose zones were not encountered in the embankment and soft zones were not encountered in the foundation. The seepage and piping observed during the 1995 High Water Event was not as severe at this location. . A seepage berm was constructed of ASTM #10 stone as an emergency repair to prevent future piping. The location between this line and the R3-SH currently ranks as the highest area of concern along the entire embankment system. Had emergency repairs not been made at this site, the R3-D site and the R3-SH site, those areas would logically rank higher.

6.4.3.5. Reach 3 Point 4. This section includes three piezometers, PZ-ER3P4-B1 and B2, installed in the crest of the levee; and PZ-ER-3P4-C installed in the landward toe of the levee. The piezometers were installed in April 1996. Data was also available from two core borings one located at the crest and the second at the landward toe of the levee along the same section line. A topographic survey was not available for this particular area, however, adjacent areas were available.

6.4.3.5.1. Upper Piezometer: PZ-ER3P4-B1. The shallow B piezometer reads the lake, indicating the embankment is fully charged with water. This is due to the combination of a blocked exit and pervious embankment fill. If there was a shallow “C” piezometer it is expected that it would read slightly above the water level in the main canal. Notice there is no head loss to the center of the embankment and all the head loss occurs between this point and the toe.

6.4.3.5.2. Lower Piezometers: PZ-ER3P4-B2 & PZ-ER3P4-C. In this area the filtercake definitely controls the water levels below the confining layer. Notice that in the deep piezometers there is very little head loss from “B” to “C”.

6.4.3.5.3. Conclusions. This area has many features making this area subject to seepage and piping problems as evidenced by both High Water Events. Many boils in the toe ditch observed in 1995 and 1998. White staining around the boils indicates seepage from within the limestone and a breached confining layer. Stone column relief wells and a small seepage berm were constructed in 1996 as a result of the 1995 event.

6.4.3.5.4. Concerns. Boils were observed in the toe ditch during the 1998 event near the relief wells as well as further north in the ditch. Boils were more numerous and severe than in the 1995 event, indicating cumulative damage has occurred. This area ranks among the locations of most concern. Currently over $ 1 million of emergency repairs has been proposed for FY 99.

6.4.4 Reach 1C Piezometers.

6.4.4.1. Reach 1 Line 4. This section includes four piezometers, two in the crest and two at the toe of the levee. The piezometers are set in the same hole at different depths to reach geologic layers of interest. The piezometers were installed in July and August 1995. Data is also available from three core borings in the vicinity across the levee. A core boring sample is not available at either piezometer site. The topographic survey of this section shows a small ditch at the landward toe of the levee.

6.4.4.1.1. Upper Piezometers: PZ-HHDR1-4B2 & PZ-HHDR1-4C2. The increase in head loss at the higher lake levels is an indicator of a blocked entrance. At higher lake levels there is an almost uniform head loss from entrance t o exit above the confining layer. Instrumentation shows that for this project this is the exception rather than the rule.

6.4.4.1.2. Lower Piezometers: PZ-HHDR1-4B1 & PZ-HHDR1-4C1. In this area the filtercake definitely controls the water levels below the confining layer. Notice that in the deep piezometers there is very little head loss from “B” to “C”.

6.4.4.1.3. Conclusions. In this section the piezometers appear to be well behaved. In other words, they read what one would expect them to read based on the geology. The possible presence of ls-p and / or a blocked entrance may be the controlling factors in the behavior of the upper piezometers. Filtercake definitely controls the lower piezometers. Based on the piezometers and geology no seepage problems would be expected at this location. No seepage sites were observed during the 1995 or 1998 High Water events.

6.4.4.2. Reach 1 Line 6. This section includes four piezometers, two in the crest and two at the toe of the levee. The piezometers are set in the same hole at different depths to reach geologic layers of interest. The piezometers were installed in July 1995. Data is also available from five core borings in the vicinity across the levee, including both the crest and toe piezometer sites. The topographic survey of this section shows a large quarry lake that has been excavated on the landward side of the levee close to the toe. The quarry lake depth in this vicinity has been recorded as –14.0 feet. The water level in the quarry lake is monitored using a staff gauge at the same time as the piezometers

6.4.4.2.1. Upper Piezometers: There are no piezometers set above the confining layer.

6.4.4.2.2. Lower Piezometers: The most significant fact is that all piezometers read the same (within one foot of each other) at all recorded lake levels.

6.4.4.2.3. Conclusions. The fact that all piezometers read the same is due to the presence of voids starting at the downstream toe that have progressed back far enough to provide a good hydraulic connection. If it was not for the presence of a good filtercake layer which accounts for about 8 feet of head loss there could really be a problem with high flows and piping under the embankment since there are deep excavations on both sides of the embankment. It is interesting to note that the piezometers read slightly artesian during high lake levels. This can be attributed to sand fill placed during 60's construction to partially shape the existing downstream slope in the then existing quarry.

6.4.4.2.4. Concerns. The piezometers provide evidence of developing voids. Since the first visible evidence would be partial berm collapses at a fairly advanced state of erosion, this section must be monitored closely.

6.4.4.3. Reach 1 Line 7. This section includes two piezometers, one at the crest and one at the toe of the levee. The crest piezometer was installed in December 1994 and the toe piezometer in February 1995. Data is also available from six core borings in the vicinity across the levee, including both the crest and toe piezometer sites. The topographic survey of this section shows a small ditch on the landward toe of the levee running along a roadside.

6.4.4.3.1. Upper Piezometers: There are no piezometers set above the confining layer.

6.4.4.3.2. Lower Piezometers: In this area the filtercake definitely controls the water levels below the confining layer. Notice that in the deep piezometers there is very little head loss from "B" to "C".

6.4.4.3.3. Conclusions. The soft toe indicates a high water level in the embankment. This is likely due to shelly zones in embankment. The toe ditch in this area is very shallow. It is basically a roadside drainage ditch. There are deeper ditches on the other side of the road. Artesian pressure indicates that the confining layer is still intact.

6.4.4.4. Reach 1 Line 9. Piezometers PZ-1 to PZ-9 were installed as a group in February and March 1990. The piezometers were installed in groups of three: three at the crest of the levee, three at the landward toe, and three on the landward side of the toe ditch approximately 70 feet from the toe of the levee. Core borings and a topographic section were later taken across a section 600 feet to the north. The topographic survey shows a ditch on the landward side of the levee.

6.4.4.4.1. Upper Piezometers: (P1, P3, P6). Piezometer P1 is influenced by both the lake and water level in the ditch. The tip of piezometer P3 is set high

(elevation 16) and so far has read dry. Piezometer P6 does not respond to either the lake or the water level in the ditch.

6.4.4.4.2. Lower Piezometers: (P2, P4, P5, P7, P8, P9). These piezometers do not respond to the lake at all. There is however a good correlation between these readings and the ditch water levels. Piezometer P4 does not respond to either the lake or the water level in the ditch.

6.4.4.4.3. Conclusions. At first observation the reading appear strange and do not make sense. All readings fluctuate with tailwater. This would indicate that the confining layer has been breached. Also there is a possible blocked entrance condition as shown by P1 and P3.

6.4.4.4.4. Concerns. The lower piezometers indicate conditions similar to R1-6 only worse due to lack of artesian pressure. This condition would be much worse without the filtercake dropping the heat 8 - 9 feet in the limestone. This line also has a deep ditch (low invert) and a pump connected to the ditch which can rapidly lower water levels 5 feet. The cavity in ls-p may indicate progression of damage under the crest. The piezometers read different from 95 HWE to 98 HWE. the piezometers read lower and closer in 98, indicating progression of damage.

6.4.4.5. Reach 1 Line 10. This section includes piezometers PZ-HHDR1-10 and PZ-PAC both of which include piezometers in the crest of the levee and at the landward toe. The placement of the PZ-HHDR1-10 piezometers, installed in January 1995, corresponds to the core boring locations and topographic survey. The PZ-PAC piezometers were installed in May 1996 approximately 600 feet to the north but are not in the vicinity of other core borings or survey lines. Core borings were taken along the section line at three different times, including the CB-HHDR1-10 group, CB-LD2-77 TO 80, and CB-HH-7-14 and 17. Nine of these core borings are shown on the geologic section. The topographic survey shows a ditch on the landward side of the levee.

6.4.4.5.1. Upper Piezometers: (PZ-HHDR1-10B2 & PZ-HHDR1-10C2).

6.4.4.5.2. Lower Piezometers: (PZ-HHDR1-10B1 & PZ-HHDR1-10C1). One unusual feature, perhaps resulting from this area being built out in old Pelican Bay or from airport construction, is the occurrence of granular material in “C” boring instead of the organics encountered most everywhere else.

6.4.4.5.3. Conclusions. Head losses indicate there is a partially blocked entrance and good filtercake which controls head in both the ls-p and lower limestone layers. The high permeability of the limestone is indicated by basically no head loss between the “B” and “C” piezometers. It is interesting to note that at lower lake elevations (below el. 14) all

piezometers read the same. All piezometers appear not to read the ditch, which would seem to indicate an intact confining layer. However, observations during 1995 HWE revealed white staining and seepage exiting the ditch slope. Overall, this section does not exhibit sighs of distress and examined on it's own, would not be cause for concern at lake levels previously encountered.

6.4.4.6. Reach 1 Pahokee Airport, Point C. This section includes piezometers PZ-HHDR1-10 and PZ-PAC both of which include piezometers in the crest of the levee and at the landward toe. The PZ-PAC piezometers were installed in May 1996 approximately 600 feet to the north but are not in the vicinity of other core borings or survey lines. Core borings were taken along the section line at three different times, including the CB-HHDR1-10 group, CB-LD2-77 TO 80, and CB-HH-7-14 and 17. Nine of these core borings are shown on the geologic section. The topographic survey shows a ditch on the landward side of the levee.

6.4.4.6.1. Upper Piezometers: (PZ-ERPAC-B2 & PZ-ERPAC-C2).

6.4.4.6.2. Lower Piezometers: (PZ-ERPAC-B1 & PZ-ERPAC-C1).

6.4.4.6.3. Conclusions. In contrast to section R1-10 which is approximately 600 feet to the south, head losses indicate there is not a blocked entrance and a poor to non-existent filtercake. In addition, the head drop between the lower "B" and "C" piezometer is uncharacteristically high. This is due to the lower "C" piezometer reading lower than expected. This would indicate some kind of drain or sink nearby.

All piezometers appear to follow the lake and not the ditch. This would indicate an intact confining layer, even though observations during 1995 HWE revealed white staining and seepage exiting the ditch slope. The lower piezometers did read artesian during the HWE.

Overall, above the confining layer the piezometer readings indicate the dike acts as sand embankment.

6.4.4.7. Reach 1 Miller Site. This section includes three piezometers, two set in the crest and one in the landward toe of the levee. These piezometers were installed in April and March 1996. Data is available from two core borings at the site of the piezometer installations. The topographic survey shows a ditch on the landward side of the levee. The aerial survey shows numerous houses in the vicinity of this section with a road running approximately 500 feet from the crest of the levee.

6.4.4.7.1. Upper Piezometer: (PZ-ERM-B1). The upper "B" piezometer reads the lake, a clear indication of a blocked exit. Core borings CB-HHD8-4 & 5 indicate a pervious upstream toe. This piezometer clearly follows the lake. There is also evidence that the ls-p layer is present, but not much on logs.

6.4.4.7.2. Lower Piezometers: (PZ-ERM1-B2 & PZ-ERM1-C). These piezometers also follow the lake and not tailwater.

6.4.4.7.3. Conclusions. The low head losses into the lower limestone indicate a lack of filtercake control in this section. Very low blow counts in the "C" boring and analysis indicate the ditch bottom should have heaved, but water still artesian below the confining layer. Minimal head loss in the lower piezometers was also encountered as large water takes in recharge tests when drilling.

6.4.4.7.4. Concerns. Core borings indicate the embankment in this area is on a very soft layer of peat. The toe has very little strength (approaching quick) likely due to water from the lake being fed through the ls-p layer. Models of this area show an increase in permeability of the downstream toe with higher lake levels. Since the toe is peat, this is likely due to the peat swelling and also the fact that this material has such a low unit weight it nearly floats.

6.4.4.8. Reach 1 Survey Line CS-42, Also Geophysical Test Site, Pahokee South.

This section includes three piezometers, two set in the crest and one in the landward toe of the levee. PZ-R1-GPPS was installed in February 1995 and PZ-R1-S42-B1 and B2 were installed in August 1995. Data is available from two core borings in the vicinity, one at the site of the piezometer installation in the toe of the levee. A crest core boring is not available for this section. The topographic survey shows a ditch on the landward side of the levee. The aerial survey shows numerous houses in the vicinity of this section with a road running approximately 500 feet from the crest of the levee.

6.4.4.8.1. Upper Piezometers: (PZ-R1-S42-B2). Upper piezometer reads the lake level in the center of the embankment.

6.4.4.8.2. Lower Piezometers: (PZ-R1-S42-B2 & PZ-R1-S42, GPPS2).

6.4.4.8.3. Conclusions. This is a classic case of a blocked exit with a pervious upstream toe. Core borings indicate a large amount of shell and gravel are present. Head loss indicates a poor filtercake control. However, the embankment so pervious with gravel that this may overtop the filtercake at very low lake levels. Also, the toe has 10 feet or more of peat, which is thicker than in most areas.

6.4.4.8.4. Concerns. This area was selected for geophysical trials due to the wet, soft toe observed at a lake elevation of approximately 16.4.5. Although this site did not exhibit piping, it could be expected that when the blocked exit overtops that piping of embankment material would be highly likely.

6.4.5. <u>Reach 1A&B Piezometers.</u>

6.4.5.1. Reach 1 S-352 North Site, Point 3. This section includes three piezometers, two set in the crest and one in the landward toe of the levee. These piezometers were installed in April 1996. One core boring is located at the same location as the crest piezometers. Data was also available from several additional core borings taken at a different time approximately 500 feet to the south, coinciding with the topographic survey. The topographic survey shows a ditch on the landward side of the levee at the edge of a road approximately 300 feet from the crest of the levee. The aerial survey shows numerous houses in the vicinity of this section.

6.4.5.1.1. Upper Piezometer: (PZ-ER-352-3B1).

6.4.5.1.2. Lower Piezometers: (PZ-ER-352-3B2 & PZ-ER-352-3C).

6.4.5.1.3. Conclusions. Both "B" piezometers read the same, indicating either a poor bentonite seal (both piezometers in the same hole) or a breached or non-existent confining layer under the center of the embankment. It is possible that construction of the railroad embankment subgrade has had an influence on the piezometers and borings in this section due to possible excavation of peat and other soft material under embankment. The piezometers follow the lake, indicating little filtercake control. The negligible head losses in the lower piezometers indicate the limestone is very permeable.

6.4.5.1.4. Concerns. Artesian pressure indicates the confining layer has not been completely breached. However, boils in the ditch bottom and seepage exiting the slope of the toe ditch indicate that this section is susceptible to problems at lake levels already experienced during both HWE's.

6.4.5.2. Reach 1 S-352 North Site, Points 4 and 5. This section includes four piezometers: PZ-ER1352-4C 200 feet south of the survey line; and PZ-ER1352-5B1, B2, and PZ-ER1352-5C 300 feet north of the survey line. The piezometers were actually set in two groups 500 feet apart, but were combined into the same section using the topographic survey between them. PZ-ER1352-4C was installed in the landward toe of the levee; PZ-ER1352-5B1 and B2 in the crest; and PZ-ER1352-5C in the landward toe. All of the piezometers were installed in April 1996. Data was available from four core borings in the vicinity, including the site of PZ-ER1352-5C. The topographic survey shows a ditch on the landward side of the levee and a road approximately 400 feet from the crest of the levee. The aerial survey shows several houses in the vicinity of this section.

6.4.5.2.1. Upper Piezometer: (PZ-ER-352-3B1)

6.4.5.2.2. Lower Piezometers: (PZ-ER-352-3B2 & PZ-ER-352-3C; PZ-ER-352-4C)

6.4.5.2.3. Conclusions: Both "B" piezometers read the same, indicating either a poor bentonite seal (both piezometers in the same hole) or a breached or non-existent confining layer under the center of the embankment. It is possible that construction of the railroad embankment subgrade has had an influence on the piezometers and borings in this section due to possible excavation of peat and other soft material under embankment. Additional evidence for this is that the "C" piezometers read lower than at Point 3. The piezometers follow the lake, indicating little filtercake control. The negligible head losses in the lower piezometers indicate the limestone is very permeable.

6.4.5.2.4. Concerns: Artesian pressure indicates the confining layer has not been completely breached. However, boils in the ditch bottom and seepage exiting the slope of the toe ditch indicate that this section is susceptible to problems at lake levels already experienced during both HWE's. In addition, the upper "B" piezometer has read near lake level prior to installation of a drainage trench in the embankment toe in this area in 1996.

6.4.5.3. Reach 1 Line 25, R1-25

This section includes four piezometers, two set in the crest and two in the landward toe of the levee. These piezometers were installed in August 1995. Core borings are located at the same locations as the crest and toe piezometers. Data was also available from several additional core borings in the vicinity. The topographic survey shows a ditch on the landward side of the levee. The aerial survey shows railroad tracks approximately 300 feet from the crest of the levee and a road approximately 450 feet from the crest. Several houses are located on the landward side of the road.

6.4.5.3.1. Upper Piezometers: (PZ-R1-25-B2 & PZ-R1-25-C2)

6.4.5.3.2. Lower Piezometers: (PZ-R1-25-B1 & PZ-R1-25-C1)

6.4.5.3.3. Conclusions: The piezometer readings in 1995 behave different from more recent readings, primarily the "spillover" effect observed is no longer present. It is possible that construction of the railroad embankment subgrade has had an influence on the piezometers and borings in this section due to possible excavation of peat and other soft material under embankment. The piezometers follow the lake, indicating little filtercake control. The negligible head losses in the lower piezometers indicate the limestone is very permeable.

6.4.5.4. Reach 1 Line 31, R1-31

This section includes two piezometers, one set in the crest and one in the landward toe of the levee. These piezometers were installed in September 1995. Core borings are located at the same locations as the piezometers. Data was also available from several additional core borings in the vicinity. The topographic survey shows a ditch on the landward side of the levee. The aerial survey shows railroad tracks approximately 200 feet from the crest of the levee and a road approximately 300 feet from the crest.

6.4.5.4.1. Upper Piezometers: None

6.4.5.4.2. Lower Piezometers: (PZ-R1-31-B1 & PZ-R1-31-C)

6.4.5.4.3. Conclusions: The confining layer in this area is very thin to non-existent. Separation of upper and lower piezometric zones should not be expected to the same extent as areas with a confining layer.
It is possible that construction of the railroad embankment subgrade has had an influence on the piezometers and borings in this section due to possible excavation of peat and other soft material under embankment. All pressure drop is between lake bottom and limestone. The piezometers follow the lake, indicating little filtercake control. The negligible head losses in the lower piezometers indicate the limestone is very permeable.

6.4.5.4.4. Concerns: The lack of entrance losses from the filtercake, lack of a confining layer and presence of high permeability rock provide a dangerous combination. The fact that the ground elevation in this area is high (el. 15 and higher) has kept the highest sustained head to below 4 feet. Higher lake levels (el. 21 and above) could produce drastically different performance in this portion of the embankment.

6.4.6. Reach 7 Line A Piezometer, R7-AB, "B" boring

This section includes one piezometers set in the crest of the levee. This piezometer was installed on 23 October 1995. A core boring is located at the same location in the crest although approximately 600 feet to the north. Data was also available from three additional core borings in the vicinity. The topographic survey shows a large ditch on the landward side of the levee.

6.4.6.1. Upper Piezometers: None.

6.4.6.2. Lower Piezometer: (PZ-HHDR7A-B)

6.4.6.3. Conclusions: This portion of the embankment is out of the peat region. The most distinctive feature is the landside borrow canal used to provide fill for the embankment. The bottom of this excavation is down to elevation -10. All of the pressure drop is between lake bottom and limestone since core borings indicate that the limestone very permeable.

6.4.6.4. Concerns: The deep borrow canal at the downstream toe provides an exit for seepage through the foundation, the rock lenses provide a roof for pipe formation, and water levels in the borrow canal prevent direct observation of any piping that would occur. There is only 2 - 3 ‘ of fine quartz sand above rock on lake side keeping water out of highly pervious rock. It should also be noted that a sinkhole was encountered during 98 HWE, indicating the presence of internally unstable material in the embankment.

6.4.7. Piezometer Sections, Core Borings and Readings.

Section(s)	No. Pages	Plate No.s
R2-F	15	1
R3-D R3-SH	30	2,3
R3-C	14	4
R3-E R3-P4	17	5,6
R1-4	13	7
R1-6	17	8
R1-7	21	9
R1-9	32	10
R1-10 R1-PAC	31	-
R1-M	13	11
R1-S42	15	12
R1-352-3	20	13
R1-352-4-5	11	-
R1-25	23	14
R1-31	24	15
R7-AB	13	16

Table H-6.1

SUMMARY OF PIEZOMETER LINE INTERPRETATIONS

	CONDITIONS	LINES	R2-F	R3-D	R3-SH	R3-E	R3-P4	R1-4	R1-6	R1-7	R1-9	R1-10	R1-PAC	R1-M	R1-S42	R1-352-3	R1-352-4-5	R1-25 (Old)	R1-25(New)	R1-31	R7-AB
1	LS / Peat problem		Y	?	?	Y	Y	?	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	N	Y
2	Blown toe or ditch		Y	?	?	?	Y	N	Y	N	Y	N	N	N	N	N	N	Y	N	N	Y
3	Artesian Pressure		N	?	N	N	N	Y	Y*	Y	N	Y	Y	Y	Y	Y	Y	?	Y	Y	N
4	Ditch drawdown		N	R*	R*	R*	Y	N	Y	N	Y	Y	Y	?	?	Y	Y	?	?	?	N
5	blocked exit		Y	Y	Y	Y	Y	N	Y	Y	N	N	N	Y	Y	N	Y	Y	N	?	N
6	soft toe / springs	C	Y	Y	Y	Y	Y	N	N	Y	N	N	N	Y	Y	Y	Y	N	N	Y?	N
7	gravel layers in embankment	B	Y	Y	Y	Y	Y	?	Y	Y	Y	?	?	Y	Y	?	?	Y	Y	Y	Y
8	loose zones in embankment	B	N	Y	Y	N	Y	N	Y	N	N	N	N	Y	N	?	?	N	N	N	N
9	soft zones in foundation	B	N	N	Y	N	Y	N	N	N	Y	N	N	Y	N	?	?	Y	Y	N	Y
10	pervious upstream toe		N	?	?	?	?	N	?	?	?	N	N	Y	Y	?	Y	Y	Y	?	?
11	filtercake controls		Y	?	Y	Y	Y	Y	Y	Y	Y	Y	N	N	N	N	N	N	N	N	n/a
11a	Head upper LS: [Lake - b / b - c]	LL	0/3	1/?	n/a	n/a	0/7	1/5	n/a	n/a	9/1	3/0	2/6	0/3	0/2	2/1	2/3	1/1	1/1	n/a	n/a
11b	Head lower LS: [Lake - b / b - c]	LL	3/1	n/a	2/?	2/?	5/2	4/1	7/0	4/1	9/1	4/0	3/5	2/1	2/0	2/1	2/3	2/0	2/0	2/0	3/0
12	spillover @ Lake = 18.5		N	N	N	N	N	N	N	N	N	N	N	N	N	N	?	Y	N	N	N
12a	Head upper LS: [Lake - b / b - c]	HL	0/*	1/?	n/a	n/a	0/10	4/5	n/a	n/a	4/1	6/0	3/6	0/6	0/5	3/1	4/4	0/4	2/1	n/a	n/a
12b	Head lower LS: [Lake - b / b - c]	HL	4/1	n/a	5/?	6/?	9/1	5/1	8/0	6/1	8/1	5/0	4/6	3/1	3/1	3/1	4/4	2/2	3/1	4/0	5/0
13	clayey u.s. toe / blocked entrance		N	N	N	N	N	Y	?	?	?	Y	N	N	N	N	N	Y	N	?	N
14	sinkholes		N	Y	Y	Y	N	N	N	N	N	N	N	?	N	N	N	N	N	N	Y
15	quick toe		N	Y	N	N	Y	N	N	N	N	N	N	Y	N	N	N	Y	N	N	N
16	Head, lake - emb. @ centerline		0	0 - 1	?	?	?	?	?	?	?	?	?	0	?	?	?	?	?	?	?
	LL= lake @ 15.5	HL = lake @ 18.5	John Stretch Park	E.R. Pt. 2	L.H. Sinkhole	E.R. Pt. 5, 6, 6A			Borrow "lake"		Old PZ's		E.R. Pahokee Apt. Ditch	E. R. Miller Site	Geophysical Site	E.R. S-352 Pt. 3	E.R. S-352 Pt. 4&5				

? = no direct evidence R* in past for roadway excavation

B = b boring (embankment centerline)

C = C boring (embankment toe)

Y* = Artesian flow noticed during high water event

7. HIGH WATER EVENTS

There have been four events since 1938 that can be considered "High Water Events". These were peak lake levels of 18.8 in 1947, 18.6 in 1995, 18.4 in 1998 and 18.3 in 1983. During each of these events the lake was above elevation 18.0 for a sustained period of time (as opposed to a short duration storm surge). The condition of the embankment system was not documented during the 1947 and 1983 events in the same manner as the 1995 and 1998 events. There was a report of a near breach of the embankment south of Pahokee due to wave attack during the 1947 hurricane. However, this was not a seepage problem. The primary focus of this section of the report is to document seepage related distress.

7.1 1995 High Water Event

7.1.1 Hydrology. (Refer to Figure H-7.1.) Two wet tropical events (Tropical Storm Chantal and Hurricane Erin) during the last half of July more than compensated for a dry first half. Rainfall averaged 8.22 inches which was 115% of average. The wet season total of 18.26 inches was 119% of average, while the year-to-date rainfall was 107% of average at 31.63 inches. Lake Okeechobee rose above its non-harmful release schedule this month. The Corps began Level II pulse releases from the lake on 31 July. Because of high stages in the water conservation areas (WCA's), SFWMD did not make releases from the lake into the WCA's. Direct hits from Hurricane and Tropical Storm Jerry combined to produce very heavy rainfall over most of the project. August rainfall averaged 11.65 inches; that was 167% of average. The wet season total of 29.92 inches was 134% of average, while the year-to-date rainfall was 118% of average at 43.28 inches. Lake Okeechobee rose substantially this month. The lake ended the month in Flood Regulation Zone B. The Corps increased regulatory releases from the lake from Level II pulse releases to Zone C releases and then to Zone B releases. A wet first ten days of the month was more than counterbalanced by a relatively dry last half of September. Rainfall averaging 6.43 inches was 86% of the average. The wet season total of 36.35 inches was 122% of average, while the year-to-date rainfall was 113% of average at 49.71 inches. Lake Okeechobee rose to near its Flood Regulation Zone A at the middle of the month. Then the lake began to decline. It ended the month in Flood Regulation Zone C. The Corps decreased regulatory releases from the lake from Zone B releases to Zone C releases. Heavy rains attributable to Hurricane Opal and a stalled front produced one of the wettest October's on record along the west and east coasts, respectively. Rainfall averaging 10.17 inches was 219% of the average. The wet season total of 46.52 inches was 135% of average, while the year-to-date rainfall was 123% of average at 59.88 inches. The extreme rainfall event this month prompted the Corps and SFWMD to make many operational changes during the second half of the month. Lake Okeechobee rose from its Flood Regulation Zone C to Zone A during the second half of the month. The lake peaked at 18.64 feet on 26 October. The Corps increased regulatory releases from the lake from Zone C releases to Zone B and then to Zone A releases. November was the driest month in two years was a welcome relief following record rains during the wet season. A lack of active frontal boundaries and no tropical systems allowed only 0.84 inches, which was 43% of average. Year-to-date rainfall of 60.72 inches was 120% of average. The dry November provided good opportunity to recover from the past wet period. Lake Okeechobee declined from slightly below Flood Regulation Zone A to Level III pulse release zone during the month. The Corps decreased regulatory releases from the lake from Zone A

through Zones B and C releases to Level III pulse releases. The driest month in three years continued a relief trend following some record rains during the wet season. A lack of active frontal boundaries ion December allowed only 0.78 inches to fall; that was 45% of average. Annual rainfall of 61.50 inches was 117% of average. Lake Okeechobee declined from Level III to Level I pulse release zone during the month. The Corps decreased regulatory releases from the lake according to the lake's regulation schedule.

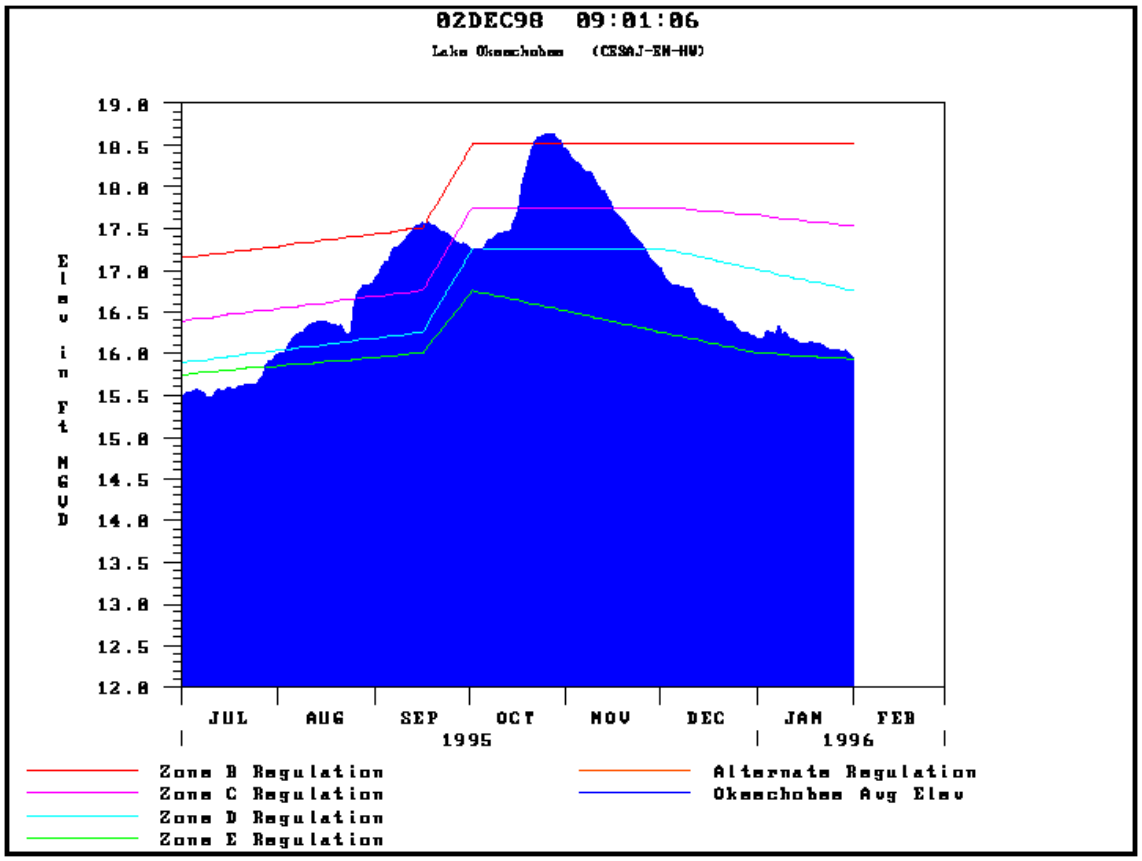


Figure H-7.1. Lake Okeechobee 10-Station Average water surface elevation.

7.1.2 Event Chronology.

This high water event began on 25 August 1995 (Day 1), when the average elevation of Lake Okeechobee exceeded 16.5. The event extended through 20 December 1995 (Day 118) when the average elevation of Lake Okeechobee dropped below EL 16.5.

The provisions of the Emergency Action Plan for monitoring the project were implemented. This consists of three levels of monitoring as follows:

Level I: Between lake elevations of 16.5 and 17.5 feet, monitoring is to consist of monthly inspections of the project by South Florida Operations Office (SFOO) staff.

Level II: For limited local flooding next to Lake Okeechobee and lake levels above 17.5 feet, monitoring is to consist of weekly inspections of the project by SFOO staff and monthly inspection by CESAJ-EN and CESAJ-CO staff.

Level III: For widespread local flooding and lake levels above EL 18.5, monitoring is to consist of daily inspections of the project by SFOO staff and daily inspection by CESAJ-EN and CESAJ-CO staff until the lake recedes back to EL 18.5.

7.1.2.1. Level 1 Monitoring.

During this period, SFOO inspected the Herbert Hoover Dike from Clewiston to Port Mayaca on 5-7 September (Days 12-14). SFOO inspected the entire Herbert Hoover Dike on 28-29 September (Days 35 and 36).

LEVEL 1. The average elevation of Lake Okeechobee is between EL 16.5 and EL 17.5.

Day 1. On 25 August 1995, the average elevation of Lake Okeechobee exceeded EL 16.5.

Days 2 Through 19. The average elevation of Lake Okeechobee continued to rise.

LEVEL 1, SECOND TIME. The average elevation of Lake Okeechobee is between EL 16.5 and EL 17.5.

Day 29. On 21 September the average elevation of Lake Okeechobee dropped below EL 17.5.

Days 30 Trough 50. The average elevation of Lake Okeechobee remained above EL 16.5.

LEVEL 1, THIRD TIME. The average elevation of Lake Okeechobee is between EL 16.5 and EL 17.5.

Day 90. On 22 November the average elevation of Lake Okeechobee dropped below EL 17.5.

Days 91 Through 117. The average elevation of Lake Okeechobee remained above EL 16.5.

7.1.2.2. Level 2 Monitoring.

The first weekly inspection occurred on 21-22 September (Days 28-29). There was an increase in ponded and wet areas since the first Level 1 inspection, but there was no change in the "small water boil". Tony: any weekly inspections by SFOO during Days 69-90 (1 Nov - 22 Nov)? CESAJ-EN and CESAJ-CO staff did not do any monthly inspection since Level 2 monitoring was less than 30 days.

After the lake level dropped below EL 18.5 on Day 68, a daily inspection of the entire Herbert Hoover Dike occurred on Day 69. After that, daily inspections continued from Clewiston easterly towards Port Mayaca to observe the active seeps. Three weekly inspections of the entire Herbert Hoover Dike occurred until the lake dropped below EL 17.5 (Day 90).

LEVEL 2. The average elevation of Lake Okeechobee is between EL 17.5 and EL 18.5.

Day 20. On 13 September the average elevation of Lake Okeechobee exceeded EL 17.5.

Days 21 Through 28. The average elevation of Lake Okeechobee remained above EL 17.5.

LEVEL 2, SECOND TIME. The average elevation of Lake Okeechobee is between EL 17.5 and EL 18.5.

Day 51. On 14 October the average elevation of Lake Okeechobee again exceeded EL 17.5.

Days 52 Through 57. The average elevation of Lake Okeechobee remained above EL 17.5

LEVEL 2, THIRD TIME. The average elevation of Lake Okeechobee is between EL 17.5 and EL 18.5.
Day 68. On 31 October the average elevation of Lake Okeechobee dropped below EL 18.5.
Days 69 Through 89. The average elevation of Lake Okeechobee remained above EL 17.5.

7.1.2.3. Level 3 Monitoring.

A special Dam Safety Comittee meeting was held on 19 October 1995 prior to implementing Level III monitoring. The monitoring was initially accomplished with six inspection teams divided as follows:
Team 1 - Clewiston (S-310) west to Fisheating Creek and Harney Pond Canal (S-131)
Team 2 – Harney Pond Canal (S-131) north to Okeechobee (S-193)
Team 3 - Okeechobee (S-193) south to Port Mayaca (S-308)
Team 4 - Port Mayaca (S-308) south to Canal Point (S-352)
Team 5 - Canal Point (S-352) south to Belle Glade (S-351)
Team 6 - Belle Glade (S-351) west to Clewiston (S-310)

LEVEL 3. The average elevation of Lake Okeechobee is between EL 17.5 and EL 18.5.

Day 58. On 21 October the average elevation of Lake Okeechobee exceeded EL 18.5.
Days 59 Through 67. The average elevation of Lake Okeechobee remained above EL 18.5.

DAY 58. On 21 October the average elevation of Lake Okeechobee exceeded EL 18.5.
DAY 60. Monday morning, 23 October, five engineers from Engineering Division departed from the District office for daily monitoring. SFOO personnel were doing the daily dike inspection; District personnel arrived at Okeechobee at 1500 hours and accompanied SFOO personnel for remainder of the day's inspection. Inspection in several areas was difficult because of high grass. Observed clear seepage north of Spillway S-352.
At approximately 18:15 two COE engineers returning to Clewiston along US.27 spotted several areas where the berm had collapsed between C-4A and Spillway S-354 (Lake Harbor). They stopped immediately to make a closer inspection. The collapses were caused by piping of berm material into the toe ditch. Although the flow rate was small (about 1 gpm per pipe) the volume of material (fine sand) that had piped was alarming (up to about 1 cuyd at some pipes). The Chief, SFOO was contacted immediately, and personnel with sandbags were sent to the site. Chief, Geotechnical Branch was contacted that evening. Extensive sandbagging operations were postponed until the morning due to the safety hazard of working next to U.S. 27 in the dark and the flow rates were not increasing.
DAY 61. Tuesday, 24 October, two representatives from SFWMD arrived in Clewiston to monitor the situation; they accompanied an inspection team. A Corps drill crew mobilized to the area; they began exploratory borings. Two Corps representatives (public affairs and hydraulics) from the District deployed to work in the SFWMD Emergency Operations Center; they provided technical assistance and answered questions from the public about lake releases. Two other Corps representatives (geotechnical) from the District deployed to help SFWMD in their inspection of levees that were in areas of widespread local flooding.
SFOO and District personnel and two SFWMD representatives performed the daily dike inspection. Inspection of several areas was difficult because of tall grass; priority areas to mow were given. The

new inspection teams had difficulty locating previously reported wet areas; the areas were not all staked and their locations not accurately recorded.

A sinkhole was observed on the crest above the area of piping near Lake Harbor. This was cause for concern since it was above the area where the piping and berm collapses were observed

A farmer reported a boil on the east side of S.R. 715, just north of Paul Rardin Park; an inspection team inspected and recorded the boil.

Sandbagging in the morning consisted of building cross dikes in the toe ditch to raise water levels. By late afternoon water in the toe ditch had risen about 6 to 12 inches; piping had slowed in most pipes, but two pipes still were moving material. Additional sandbags were placed around the active pipes and plans were developed to pump water (next day) into the toe ditch to reduce the differential head.

SFWMD established a phone bank on 24 October for citizens to call with questions or concerns.

DAY 62. Wednesday, 25 October, SFOO and District personnel and one SFWMD representative did the daily dike inspection. A citizen reported the location of deposits of sand in the toe ditch; an inspection team inspected and recorded the area. The mowing crews mowed a large area; this made it possible to inspect areas that were previously inaccessible. The Corps accepted SFWMD's offer of manpower and vehicles to help the Corps inspection teams. Another person from the District office deployed for technical assistance to SFWMD for widespread local flooding problems.

DAY 63. Thursday, 26 Oct, Corps and SFWMD personnel together performed the best detailed inspection to date. The locations of condition 1-4 areas were staked and recorded for future inspection teams.

DAY 64. Daily inspections continued.

DAYS 65-66. The Corps staffed a phone line at the Clewiston office over the weekend (28-29 October) for anyone who noticed new and unusual conditions on or around the Herbert Hoover Dike. Daily inspections continued. Many wet spots and ponded areas that were previously recorded were drying.

DAY 67. Daily inspections continued. Wet spots and ponded areas continued to dry.

DAY 68. On 31 October the average elevation of Lake Okeechobee dropped below EL 18.5. Daily inspections continued. Wet spots and ponded areas were drying; active seeps with clear flow were monitored with no changes.

DAY 69. Even though the lake level was below EL 18.5, the entire Herbert Hoover Dike was inspected. Wet spots and ponded areas continued to dry. This was the last time that the entire dike was inspected daily.

DAY 70. Only Areas 3, 4, and 5 (Clewiston to Port Mayaca) were inspected; teams from Areas 1, 2, and 3 helped the other teams. Closer inspection of the toe ditches was possible because of dryer conditions. This enabled the inspectors to locate additional seeps. These seeps were minor and clear. This was the last day that CESAJ-EN staff performed inspections for the high water event.

7.1.3. SFWMD Assistance.

During the first four days of Level 3 monitoring, the Corps did the daily inspections without assistance from SFWMD. On the fourth day, the Corps accepted SFWMD's offer of manpower and vehicles to help the Corps inspection teams; this totaled approximately 384 man-hours (8 days X 12 hours/day X 4 men). Now the inspection teams could consist of a minimum of two people. The two-person teams discovered more seepage locations, staked and recorded those locations, and used the "buddy system"

to stay safe. Additional SFWMD personnel were on site or at SFWMD's headquarters monitoring the situation. The personnel who were on site gained experience in recognizing seeps and knowing what corresponding action to take.

7.1.4. Observations Of Distress.

Consistent evaluation of observed seepage among the different inspectors was extremely important. Very early in the inspection process a method for rating the severity of seepage was implemented. Seepage locations were rated as follows:

Condition 1: Wet spot, saturated ground, no ponded water.
Condition 2: Ponded area with standing water, no visible flow.
Condition 3: Active seep, visible flow, no movement (piping) of material.
Condition 4: Active seep with movement of material.

The project had already been divided into reaches based on work performed for the Major rehabilitation Report (MRR). This system of reaches was ultimately adopted for the numbering sequence of observed distress. The division of reaches is shown in Figure H-7.2.

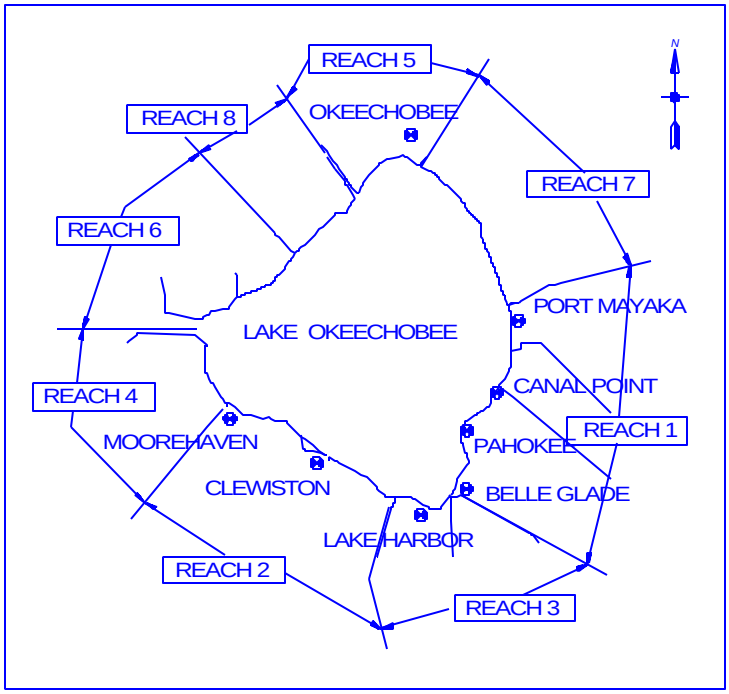


Figure H-7.2. Project Reach Divisions.

7.1.4.1. Reach 3 Sites.

There was one conditon 4 site and five condition 3 sites reported and monitored in Reach 3. Refer to Figure H-7.3.

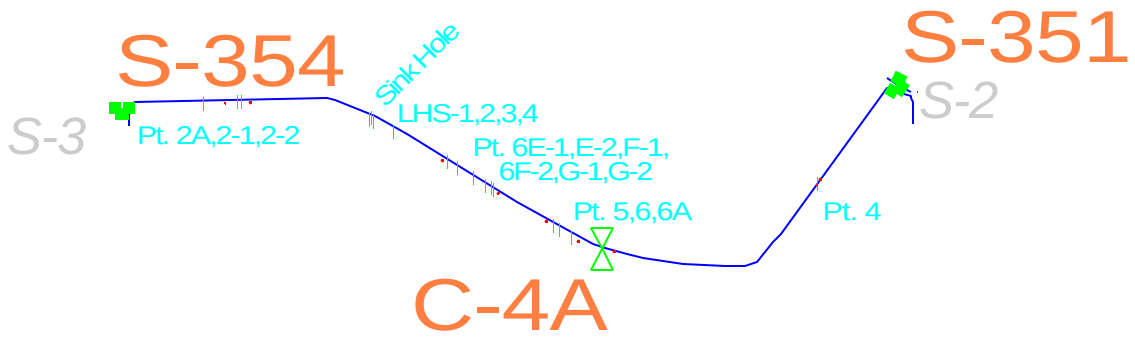


Figure H-7.3. Reach 3 Condition 3 and 4 sites.

Condition 4 Site. The site labeled "Sinkhole" and LHS -1,2,3,4 were located midway between Spillway S-354 and Culvert 4A. The landside berm above the toe ditch was damaged for approximately ¼ mile. The berm had collapsed in several areas and the ditch slopes that were next to the embankment had sloughed in several areas. A sinkhole was observed in the levee crest above the areas where piping occurred. Boils were also observed in the bottom of the toe ditch. No odor or white staining was present in the boils. The piping was considered caused from through seepage.

Condition 3 Sites. There were five condition 3 sites identified in Reach 3. These sites (west to east) were labeled Point 2A, Point 2-1 & 2-2, Point 6 E, F & G, Point 5, 6 & 6A, and Point 4.

Point 2A was an area with a small cond. 3 seep on the berm. Points 2-1 & 2-2 were an area with strong seepage over an area 1000 feet long exiting on the berm at approximately elevation 15. This area was impassible by vehicle or foot.

Points 6 E, F & G: This was an area extending for approximately 2500 feet. The sepage was exiting high on the berm (approximately elevation 15 – 17). The berm was wet, but not saturated. The flow rate was not nearly as great as at point 2.

The point 5 area contained seeps over about a 1500 foot area. This was centered at a boil on the berm which had piped material. The elevation of the boil was measured with a level as 15.7 referenced to the water's edge and an assumed lake elevation of 18.5. In any case, the boil piped on the berm with 2.8 feet of head and a 280 foot seepage length.

Point 4 was located approximately 1 mile south of S-351. Damage consisted of seepage exiting on the berm at elevation 10.6 (based on a lake elevation of 18.5) and several boils in the ditch bottom over approximately 250 feet. Seepage exiting the ditch bottom was unusual in that it caused a white staining

upon contact with peat, and that there was a strong sulfur odor. This was attributed to seepage below the confining layer (silt – clay layer below the peat) reaching the surface through cracks or fractures.

7.1.4.2. Reach 1 Sites.

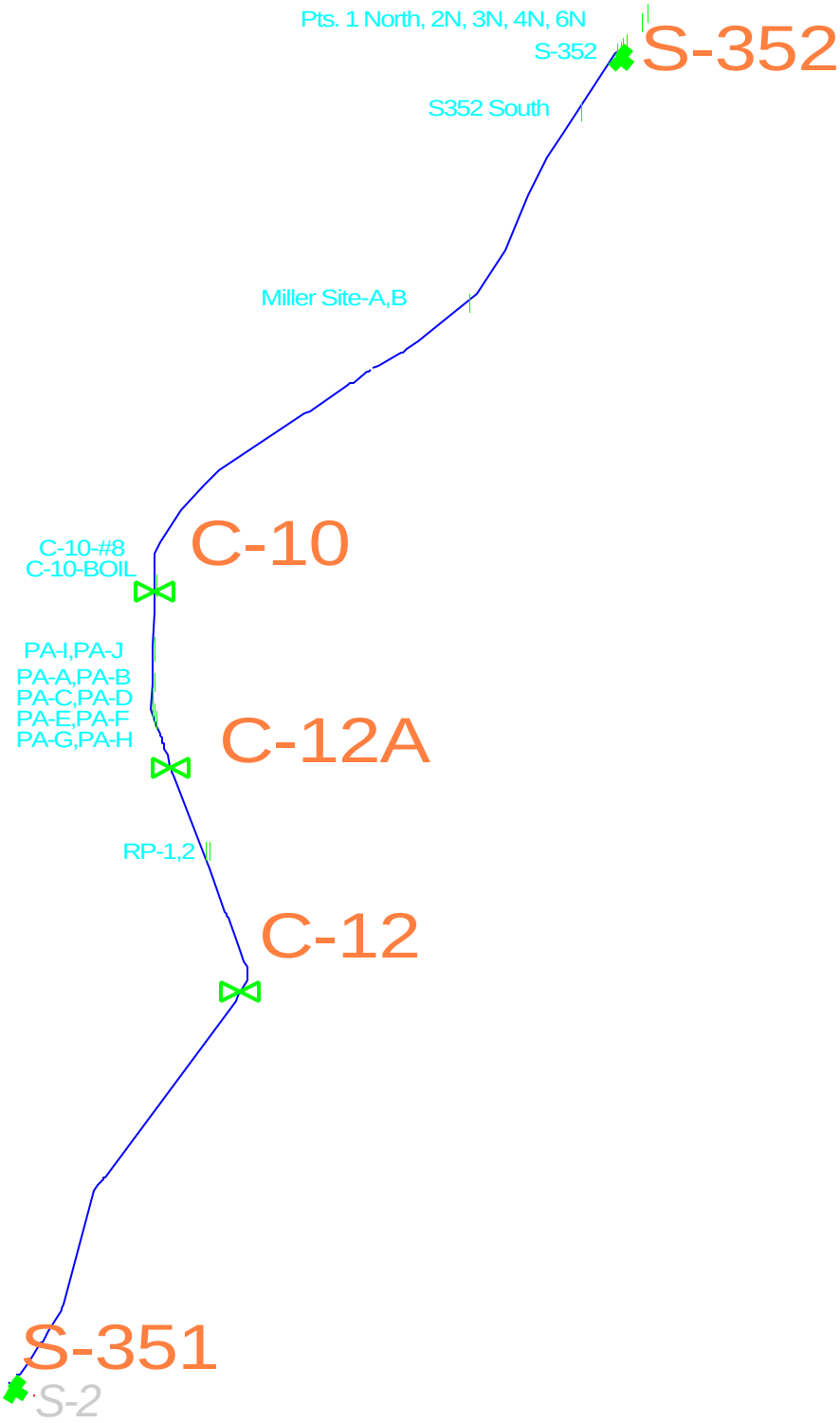


Figure H-7.4. Reach 1 condition 3 and 4 Sites.

Condition 4 Sites.

Culvert 10. This site is just north of culvert structure C-10. The landside toe of the embankment at the bottom of the ditch was damaged due to piping at several locations and collapses of the ditch slope on the lake side of the ditch. No odor or white staining was present in the boils. The piping was considered caused from through seepage.

Miller Site. This site located behind 1565 E. Main Street, Pahokee. The landside slope of the toe ditch was damaged. Three locations were observed where the material that piped out, slid down the slope of the toe ditch; an estimate that 2 to 3 cubic feet of material piped at each point. No odor or white staining was present. The piping was considered caused from through seepage.

Condition 3 Sites.

Pahokee Airport Ditch. This site was located immediately adjacent to the airport property in the ditch between the embankment and airport. Several seeps exited the lakeside slope of the toe ditch at the base of the ditch. There was no detectable sulfur smell, but some white staining was noticed.

Rardin Park. This site was located just south of Pahokee, immediately north of Paul Rardin Park. Two boils were found, one at the toe of the embankment and one across SR 715 on a berm in the cane fields. Upon obtaining the locations with GPS, these boils were determined to be old core borings that were not properly backfilled and grouted.

S-352 North and South sites. Seepage at these sites consisted of blocks of peat being pushed out at the water level in the toe ditch. Generally, a block of peat about one cubic foot in diameter would be pushed out and clear seepage would exit. One such site was located to the south of S-352 and several sites were located to the north of S-352.

7.1.5. Emergency Actions.

Condition 4 Sites. The objective was to stop loss of embankment material.

Lake Harbor. (Sinkhole Site) The area where piping was occurring extended for approximately 1500 feet. Initial attempts to sandbag the individual pipes exiting the sides of the ditch and boils in the bottom of the ditch proved to be ineffective. The sand being piped was so fine that it passed between the sandbags or when some head was built up in the sandbag containment, the head caused material under the sandbags to pipe. Also, there were so many pipe and boil locations these could not be ringed fast enough and working conditions were such that personnel working in the ditch bottom sunk up to their waists and made conditions worse.

A second attempt was to compartmentalize areas containing boils and pipes by constructing small check dams across the ditch bottom. This was partially effective, but the volume of the ditch to fill was such that filling by seepage proved to be too slow and ineffective.

The third and final attempt was construction of higher check dams and setting up a large pump at C-4A to fill up the entire length of toe ditch. Refer to the photos. This method proved to be effective in stopping the piping. Once instructions were given it took approximately two days to set up the equipment and pump enough water to fill the two miles of ditch.

Culvert 10. The area where piping was occurring extended for approximately 500 feet. A similar method was employed at this site. Due to the nature of the pipe (a broad area of piping through a shelly sand seam about 3 inches thick) sandbags were not considered. The ditch bottom was also very soft. Fortunately, a discharge culvert into the local drainage canal was approximately 200 feet downstream.

A gate was fabricated from a sheet of plywood, stakes and sandbags. Water levels rose above the level of the pipes within a few hours.

Miller Site. The piping at the Miller Site had sealed itself off after moving 1 – 2 cubic feet of sand at each of the three pipe locations. This site was monitored, but no remedial action was required.

Condition 3 Sites. The objective was to prevent these sites from becoming condition 4 sites. These sites were monitored, but no remedial action was required. At Pahokee Airport Ditch a culvert at the south end of the ditch was blocked by a gate fabricated from plywood, stakes and sandbags as a preventative measure.

7.1.6. Reporting.

During Level 1 monitoring, CESAD and SFWMD were notified that Level I monitoring was being performed and that several wet areas were identified, including the Lake Harbor area.

During Level 2 monitoring, the District Engineer, CESAD and SFWMD officials, and Local Drainage District and County Emergency Management Officials were briefed on the condition of the Herbert Hoover Dike. The Dam Safety Committee met, and CESAJ-PA made a press release describing Corps and other agencies' coordination of flood protection actions. This level also included pre-positioning of materials, equipment, etc. as a precautionary measure. Corps Emergency Operations officials issued the first situation report one day before starting Level 3 monitoring.

During Level 3 monitoring, interagency coordination intensified with CESAD, SFWMD, Local Drainage Districts, Florida Division of Emergency Management, County Emergency Management Agencies, and County Sheriff's Offices. The Dam Safety Committee met regularly and CESAJ-PA provided press releases on Corps activities. SITREP 7 stated "conference calls were held over the weekend with SFWMD, local, area, and State Emergency Management officials on HHD and L.O. releases. Conference calls were made daily at 1600 hours between SFOO, the District office, and SFWMD for an update on the situation and to discuss the plan of action for the next day. Subsequent situation reports (2 through 8) were issued 22-24, 26-27, 29, and 31 October by Corps Emergency Operations officials.

7.1.7. Media Exposure.

The District Public Affairs office mounted an aggressive media program to inform the public about Corps activities in connection with the high level of water at Lake Okeechobee and the problems at Herbert Hoover Dike. A public affairs specialist was on site to help the technical teams and operations personnel and to work closely with the SFWMD. Media interest was high with most Florida papers and the Los Angeles Times, and many local television stations and CNN carrying the story. A story in The Miami Herald was especially noteworthy. It characterized the on-site project manager as the "little Dutch boy who stuck his finger in the dike to hold back the flood". It also discussed the vigilant efforts that the Corps and others made to ensure safety. Corps Public Affairs officials released news releases about the Level III Monitoring on 26 and 27 October. Corps Public Affairs officials released other news releases about water discharges from Lake Okeechobee on 29 August and 23, 25, 26, and 27 October.

7.1.8. Post High Water Inspection.

On 5 and 6 February 1996, personnel from Headquarters, Division and District Offices, and SFOO participated in the post high water inspection of Herbert Hoover Dike. The purpose of the inspection was to review with higher authority the events of October and November 1995, to discuss the evidence of distress on the embankment, and to convey the need to repair the damages to the project before the next hurricane season.

7.1.9. Implementation Of Repairs.

During the event, Corps drill crews began core boring operations at the condition 3 and 4 sites in the Lake Harbor area. Explorations were also performed at the Reach 3 Point 4 site, Pahokee Airport Ditch, C-10, Miller site and S-352 North sites. Piezometers were installed several sites, and gradations of select samples were obtained. A summary of the E&D effort and repair costs is provided in Table H-7.1. Detailed models were developed for repairs in Reach 3 and Reach 1.

Table H-7.1. 1995 High Water Event Repair Cost Summary.

HERBERT HOOVER DIKE EMERGENCY REPAIRS SUMMARY

SITE	Site	Mat.l	Actual Unit	Mat.l	Other Material	Equip.	E & D	SFOO	Total per
	Length, ft	TONS	Cost / ton	COST	Cost	Rental	COST	Labor (**)	Site
Lake Harbor (1000') Initial	1000	9500	9.75	92625	0	11000	20657	52356	176638
Lake Harbor (500') Remaining	500	5000	9.75	48750	0	8000	10328	26178	93256
C-10 Cond. 4	500	4500	6.28	28260	34532	0	10328	26178	99298
Miller Cond. 4	100	400	6.28	2512	0	0	2066	5236	9813
Lake Harbor Pt. 2 Cond. 3	1000	9000	5.45	49050	0	14000	20657	52356	136063
Lake Harbor Pt.5,6,6A Cond 3	1500	13500	5.45	73575	0	21000	30985	78534	204094
Lake Harbor Pt 6E, F,G Cond 3	2500	22500	5.45	122625	0	35000	51641	130890	340156
Reach 3, Pt. 4, Cond. 3	250	1100	5.73	6303	0	0	5164	13089	24556
S-352 North & South Sites, Cond. 3	2200	3200	6.83	21856	0	0	45444	115183	182484
TOTALS	9550	68700		$ 445,556		$ 89,000	$197,270	$500,000	$ 1,266,358

* Cost based on COEMIS report 5/25/96 (labor, PD, drilling equipment & supplies), total = $197,270 pro rated per foot of site

* *Cost based on total of $500,000 available pro rated per foot of site

7.1.10. Photographs. The following pages are photographs taken of evidence of distress during the event and subsequent repairs.

7.1.10.1. Lake Harbor Sites.

Photo H-7.1 Lake Harbor piping, ditch with sandbags

Photo H-7.2 Lake Harbor piping, ditch with sandbags, close-up

Photo H-7.3 Lake Harbor piping, ditch with sandbags & drill rig

Photo H-7.4 Lake Harbor piping, ditch with sandbags & drill rig, distance shot

Photo H-7.5 Lake Harbor sinkhole, Larry in hole

Photo H-7.6 Lake Harbor sinkhole, Onlookers

Photo H-7.7 Lake Harbor, C-4A pumping

Photo H-7.8 Lake Harbor, C3, sandbag stockpile

Photo H-7.9 Lake Harbor Point 2; overview looking east

Photo H-7.10 Lake Harbor Point 2; overview looking west

Photo H-7.11 Lake Harbor Point 2; overview

Photo H-7.12 Lake Harbor Point 2; closeup of seepage

Photo H-7.13 Lake Harbor Point 6; overview looking west

Photo H-7.14 Lake Harbor Point 5; overview

Photo H-7.15 Lake Harbor Point 5; closeup

Photo H-7.16 Lake Harbor Point 5; large closeup

Photo H-7.17 Lake Harbor Point 2; overview of sandbag weir

Photo H-7.18 Lake Harbor Point 2; overview of sandbag weir, closeup

Photo H-7.19 Lake Harbor Pt 6 EFG Site; berm repair, excavating thru toe ditch

Photo H-7.20 Lake Harbor Pt 6 EFG Site; berm repair, berm peat stripped

Photo H-7.21 Lake Harbor Pt 6 EFG Site; backfilling berm & ditch

Photo H-7.22 Lake Harbor Pt 6 EFG Site; replacing topsoil

Photo H-7.23 Lake Harbor Pt 6 EFG Site; ready for grassing

Photo H-7.24 Lake Harbor Sinkhole site; stripping peat on berm

Photo H-7.25 Lake Harbor Sinkhole site; excavating pipes above sandbag sites

Photo H-7.26 Lake Harbor Sinkhole site; excavating pipes above sandbag sites, close-up

Photo H-7.27 Lake Harbor Sinkhole site; note liquified material (4' survey rod pushed into berm)

7.1.10.2. Reach 3 Point 4 Area.

Photo H-7.28 Reach 3 point 4; Seepage on berm

Photo H-7.29 Reach 3 point 4; overview

Photo H-7.30 Reach 3 point 4; close up of white staining from culvert discharge

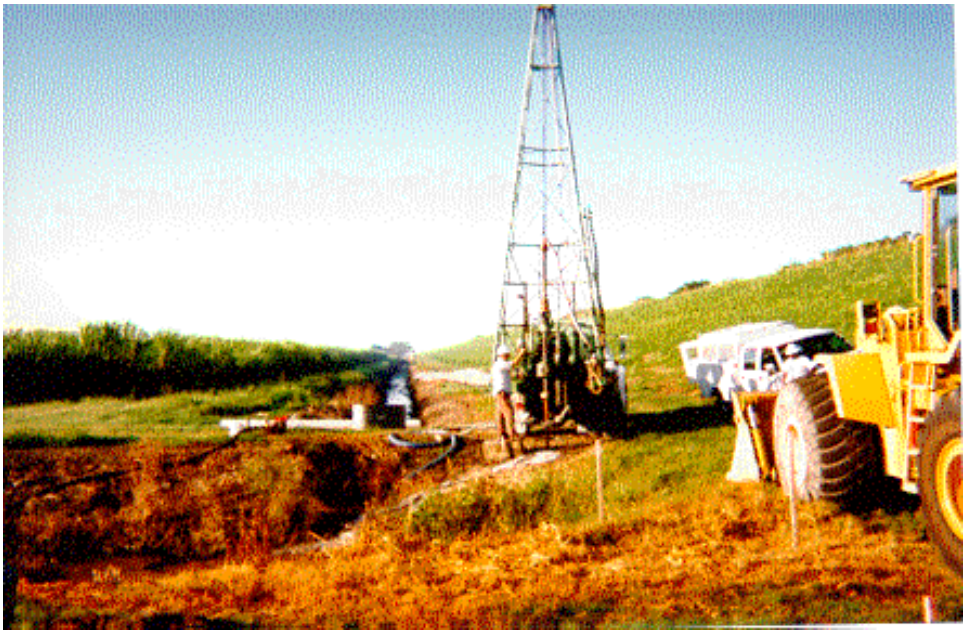


Photo H-7.31 Reach 3 point 4; drilling vertical drains

Photo H-7.32 Reach 3 point 4; filling drains with #10 stone

Photo H-7.33 Reach 3 point 4; excavating drain cap and laterals to ditch

Photo H-7.34 Reach 3 point 4; vertical drains on 15' centers

7.10.1.3. Culvert 10 Area.

Photo H-7.35 C-10 site; cleared ditch looking north (June, '96)

Photo H-7.36 C10; piping in ditch

Photo H-7.37 c10; ditch view north

Photo H-7.38 C10; piping in ditch, sloughing

Photo H-7.39 C10 aerial photo

Photo H-7.40 C-10 site; culvert pipe and junction box

Photo H-7.41 C-10 site; end view of 48” perf. cmp culvert pipe wrapped in geotextile

Photo H-7.42 C-10 site; bringing fill up around pipe and manholes

Photo H-7.43 C-10 site; building headwalls

Photo H-7.44 C-10 site; completed berm looking west

Photo H-7.45 C-10 site; completed berm looking north

7.1.10.4. S-352 Site.

Photo H-7.46 S-352N; seepage on berm

Photo H-7.47 S-352N; collecting and measuring seepage on berm

Photo H-7.48 S-352N; seepage exiting through peat on side of ditch

Photo H-7.49 S-352N; excavating interceptor trench

Photo H-7.50 S-352N; excavating laterals to ditch

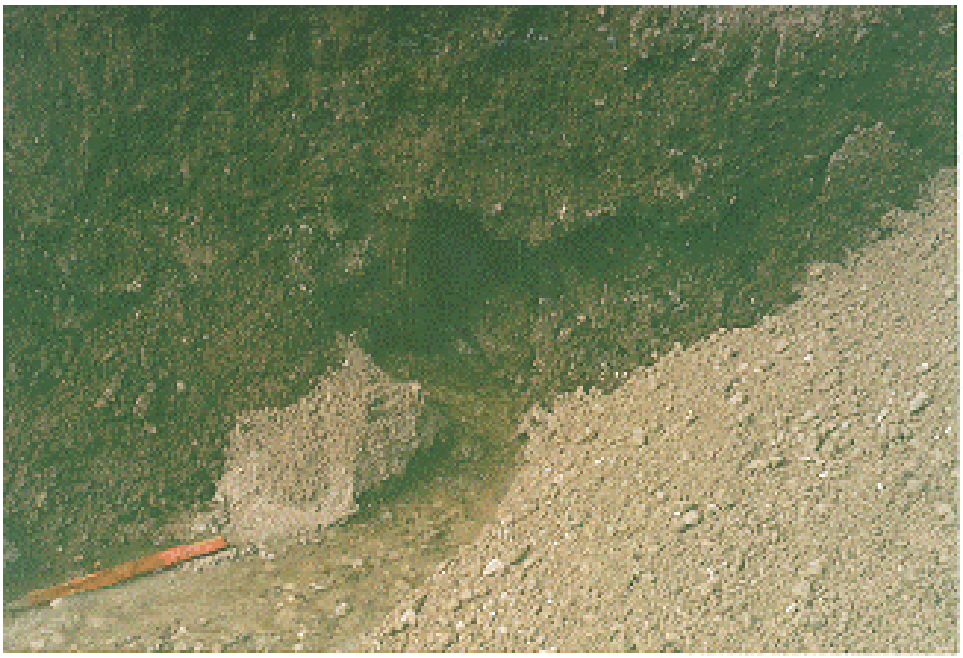


Photo H-7.51 S-352N; abandoned water pipe uncovered, note flow

Photo H-7.52 S-352N; close-up of pipe

Photo H-7.53 S-352N; backfilling interceptor trench

Photo H-7.54 S-352N; covering trench

7.10.5. North Pahokee Site.

Photo H-7.55 Miller Site; south seep next to bridge.

Photo H-7.56 Miller Site; closeup of south seep next to bridge. Notice light colored sand below water level in cleared area.

Photo H-7.57 Miller Site - overview of repair.

Photo H-7.58 Miller Site - cross section of repair.

7.2 1998 HIGH WATER EVENTS.

7.2.1 General. There have been four events since 1938 that can be considered "High Water Events". These were peak lake levels of 18.8 in 1947, 18.3 in 1983, 18.6 in 1995, 18.4 in 1998. During each of these events the lake was above elevation 18.0 for a sustained period of time (as opposed to a short duration storm surge). The condition of the embankment system was not documented during the 1947 and 1983 events in the same manner as the 1995 and 1998 events.

1998 was a wet year for South Florida. In December 1997 starting with a lake level of +15.5 feet, the Lake began to rise. The lake continued to rise through January 1998. During February 1998 the level of the lake exceeded +18 feet. The high lake levels continued through March 1998. Beginning in April 1998, the lake level began dropping. By the end of May the lake had dropped to elevation +15.5. See Figure H-7.2.1. Lake Okeechobee 10-Station Average water surface elevation.

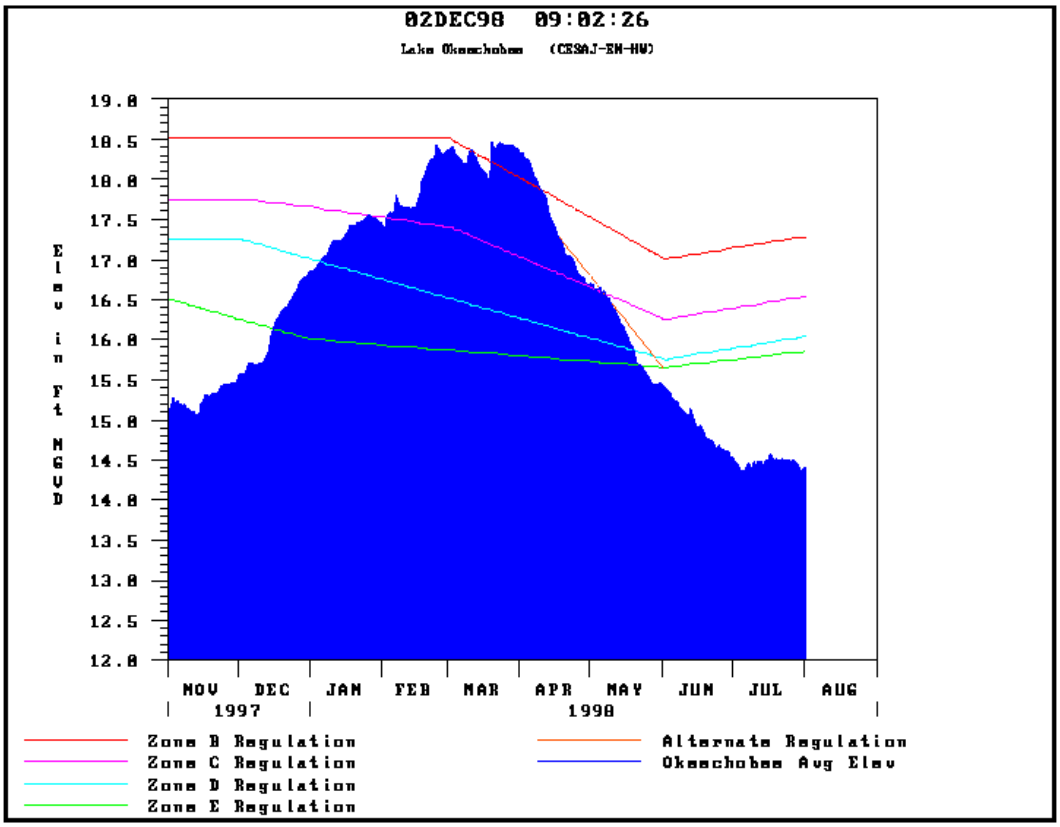


Figure H-7.2.1. Lake Okeechobee 10-Station Average water surface elevation for the 1998 high water event.

7.2.1 Inspection Plan. The COE in accordance with their monitoring plan, instituted inspections of the dike during high water levels and intensified their monitoring for seepage and piping associate with the dike. The three levels of monitoring followed are:

- **Level I:** Between lake elevations of 16.5 and 17.5 feet, monitoring is to consist of monthly inspections of the project by South Florida Operations Office (SFOO) staff.

- **Level II**: For limited local flooding next to Lake Okeechobee and lake levels above 17.5 feet, monitoring is to consist of weekly inspections of the project by SFOO staff and monthly inspection by CESAJ-EN and CESAJ-CO staff.

- **Level III**: For widespread local flooding and lake levels above EL 18.5, monitoring is to consist of daily inspections of the project by SFOO staff and daily inspection by CESAJ-EN and CESAJ-CO staff until the lake recedes back to EL 18.5.

A method for classifying the seepage condition was established during the 1995 high water event. Seepage conditions are rated as follows:

- **Condition 1**: Wet spot, saturated ground, no ponded water.
- **Condition 2**: Ponded area with standing water, no visible flow.
- **Condition 3**: Active seep, visible flow, no movement (piping) of material.
- **Condition 4**: Active seep with movement of material.

The project has been divided into eight (8) reaches based on work performed for the Major rehabilitation Report (MRR). This system of reaches is used to catalog the location of the observed seepage locations. The eight (8) of reaches are shown in Figure *** .

7.2.2 Jacksonville District Office Inspection. A dike inspection was conducted by the Jacksonville District Office on 7-8 April 1998. The EN-GS team members included Mr. Bob Ross, Mr. Sam Honeycutt, and Ms. Candida Koenig. The purpose of the inspection was to perform a scheduled monthly inspection of the Herbert Hoover Dike system during the 1998

high water event. Photographs and a video tape were taken during the inspection. The average lake elevation during the inspection was18.0 feet. Mr. Glen Zubris of the field office assisted the inspection team.

The inspection concentrated on condition 3 sites in Reaches 1, 2, and 3. The 4/01/98 Inspection Report performed by the field office was used to select sites to be visited. The sites in the report were classified by condition, ranging from condition 1 to condition 4. No condition 4 sites were noted in the 4/01/98 inspection.

7.2.2.1 Sites Inspected.

SITE R3-1A. This site showed evidence of piping action with the characteristic delta of white sand and white staining. Water was currently flowing from several of the pipe holes.

SITE R3-3. This site is part of the emergency repairs made in 1995. A portion of the repair, approximately 150 feet in length, is called the hump. Ponding of water was evident on both sides of the hump. The berm was saturated and moved underfoot showing evidence of pumping.

SITE R3-4. This site experienced some piping problems during the 1995 high water event but had not been included in the emergency repairs. Sand bags were still present from the 1995 event. At the location of the sand bags, a piping hole was found approximately 4 inches wide and 2 inches deep. This site showed evidence of piping action with the characteristic delta of white sand and white staining. Water was currently flowing from the pipe hole. Numerous piping holes were found for approximately 100 feet both north and south of the site. The pattern was that about every 10 feet there was a seep (Most were approximately 1-2 inches in diameter) flowing water into the ditch at the toe of the levee. Each seep had an associated delta of piped material. There was no active piping observed during inspections and the total amount of piped materials was minimal (perhaps 1 cubic foot of piped materials in each of the associated deltas).

SITE R3-13.1. Two seeps were located at this site. This site showed evidence of piping action with the characteristic delta of white sand and white staining. Water was currently flowing from the pipe holes. A distinctive channel had formed in the ditch approximately 2 inches deep and 6 inches wide flowing to the south. Mr. Ross calculated the flow to be approximately 20 gal/min in the channel.

SITE R3-13A. This was the site of a cone shaped feature located in the toe ditch as noted in the last inspection report. The site had since been sandbagged around the boil. Water was flowing quite rapidly from several places through the sandbags. The water level inside the sandbag circle was about 10 inches higher than the ditch water.

SITE R3-16. Two seeps were located at this site. This site showed evidence of piping action with the characteristic delta of white sand, peat and white staining. Water was currently flowing from the pipe holes. Some sloughing of the bank had occurred. A crack on the berm was noted about 2 to 3 feet back from the top of the drainage ditch. The crack appeared to continue brokenly for approximately 10 feet in either direction from the location of the seeps. Seepage uplift pressure is creating tension cracks.

SITE R1-10. This location had experienced some past seepage problems and had been boxed with sand and gravel and sandbagged. Some seepage was evident from the base of the box and the toe bank. This seepage showed white staining.

SITE R3-14. This was the site of a boil as noted in the last inspection The site had since been sandbagged around the boil. Water was currently flowing through the sandbags. Just north of the sandbags a channel of flow was estimated to flow 30 gal/min. The direction of flow was to the north, opposite that of the R3-13.1 site still in the same ditch. In this reach of the dike, peat is commonly exposed on the surface and in the ditch. Seeps and boils continuously float out small particles of the peat. The peat is easily erodible.

SITE R3-15. At this site the ditch bank had sloughed off and a hole was evident back into the berm. The hole extended back into the berm for over 5.5 feet, including a large void. Water was flowing out of the hole and a delta had formed with mostly peaty soils and the white residue.

QUARRY LAKE SITE. This site was inspected to monitor the water level in the lake. The quarry lake elevation was at 9.2 feet, giving a current head differential across the levee of 8.8 feet. The toe of the levee next to the quarry lake was saturated.

This is an intermittently active quarry used for the production of sand and gravel.

The elevation of the Quarry Lake is controlled by a culvert that drains lake waters into a ditch. The local authority raises and lowers the lake as needed to facilitate quarrying operations. The culvert is currently closed, thereby maximizing the lake elevation. Opening the culvert would lower the lake by about 2 feet and increase the differential head across the levee from 8.8 feet to 10.8 feet (23% increase). The culvert is 36 inches in diameter and has an estimated invert elevation of 7.2 feet.

SITE R1-10P. A seepage hole was located at this site. This site showed evidence of piping action with the characteristic delta of white sand and white staining. Water was currently flowing from the pipe hole. The hole was angled parallel to the levee, suggesting that the seepage had occurred at a distance from the hole itself and had traveled to a weak spot before breaking through. Tension cracks caused by uplift pressure were evident on the berm showing the direction the flow had taken, flowing to the south and out the pipe hole.

SITE Rl-B4. This site had shown some previous seepage flowing from the berm. Currently the site showed a saturated berm with standing water in the tire tracks but no obvious seeps.

SUMMARY OF INSPECTION. The current average lake level of 18.0 feet represents a drop of 0.4 feet in lake elevation from the recent high average lake level of 18.4 feet on 26 March 1998. With the lack of rain and maximum water releases through the locks and spillways at Port Mayaca and Moore Haven, the lake level continues to drop. Most of the piping activity noted in previous weeks has slowed or stopped as the lake level drops.

7.2.3 Inspection Report of 15 April 1998. Following are summaries of all the inspections accomplished during the high water event and the field inspection report of !5 April 1999. This 15 April 1998 report documents the history of the individual seeps, the location of the individual seeps, and the number of inspections performed during the 1998 high water event.

Summary of 3/4/98 Inspection

New this inspection:		Total Reported:	
Condition One	0	Condition One	13
Condition Two	2	Condition Two	36
Condition Three	7	Condition Three	23
Condition Four	0	Condition Four	0
		TOTAL	72

Summary of 3/4/98 Inspection

New this inspection:		Total Reported:	
Condition One	0	Condition One	12
Condition Two	0	Condition Two	34
Condition Three	1	Condition Three	22
Condition Four	0	Condition Four	0
		TOTAL	68

Summary of 3/18/98 Inspection

Lake Average: 18.01

New this inspection:		Total Reported:	
Condition One	0	Condition One	12
Condition Two	4	Condition Two	39
Condition Three	3	Condition Three	27
Condition Four	1	Condition Four	1
		TOTAL	79

The inspection on 3/18 showed drying and improvement on many of the monitored sites. While many areas showed improvement, eight new sites added to the inspection report Culvert C-3 was sandbagged to 3 feet this inspection period to contain water in the culvert basin and to raise the water level in the toe ditch.
One Condition 4 site was found in Reach 3 that was moving organic material from the boil.
This site was sandbagged to contain the boil and build head pressure over the boil.

Conditions by Area

	Condition 1	Condition 2	Condition 3	Condition 4
Reach 1	2	4	9	0
Reach 2	3	10	9	0
Reach 3	0	10	9	1
Reach 4	0	1	0	0
Reach 5	2	4	0	0
Reach 6	0	1	0	0
Reach 7	0	0	0	0
Reach 8	5	9	0	0
Totals	12	39	27	1

Summary of 3/25/98 Inspection

Lake Average: 18.42

New this inspection:		Total Reported:	
Condition One	0	Condition One	11
Condition Two	0	Condition Two	40
Condition Three	5	Condition Three	32
Condition Four	0	Condition Four	1
		TOTAL	84

During the week 3/18/98 - 3/25/98, 2 to 3 inches of rain was recorded at rain gauges around the lake.
Washed out retention culvert in toe ditch at south end of Glades Airport under repair and staff gauges being installed in toe ditches in critical areas this inspection period.

Conditions by Area

	Condition 1	Condition 2	Condition 3	Condition 4
Reach 1	1	5	14	0
Reach 2	3	10	9	0
Reach 3	0	10	9	1
Reach 4	0	1	0	0
Reach 5	2	4	0	0
Reach 6	0	1	0	0
Reach 7	0	0	0	0
Reach 8	5	9	0	0
Totals	11	40	32	1

Summary of 4/01/98 Inspection

Lake Average: 18.33

New this inspection:		Total Reported:	
Condition One	0	Condition One	11
Condition Two	5	Condition Two	44
Condition Three	5	Condition Three	38
Condition Four	0	Condition Four	0
		TOTAL	93

During the week 3/26/98 - 4/01/98, little or no rain was recorded at rain gauges around the lake. General overview: We are still seeing an increase in berm saturation and increased flow at most of our Condition 3 sites. A few sites have show some drying. As shown above, 10 new sites were reported this inspection with 5 new Condition 3 sites in Reach 3 and 5 new Condition 2 sites in Reach 2. No inspections were performed this week in Reaches 4,5,6 and 8.

Conditions by Area

	Condition 1	Condition 2	Condition 3	Condition 4
Reach 1	1	5	14	0
Reach 2	3	15	9	0
Reach 3	0	9	15	0
Reach 4	0	1	0	0
Reach 5	2	4	0	0
Reach 6	0	1	0	0
Reach 7	0	0	0	0
Reach 8	5	9	0	0
Totals	11	44	38	0

Summary of 4/08/98 Inspection
Lake Average: 17.97

New this inspection:		Total Reported:	
Condition One	0	Condition One	26
Condition Two	1	Condition Two	32
Condition Three	0	Condition Three	36
Condition Four	0	Condition Four	0
		TOTAL	94

During the week 4/01/98 - 4/07/98, no rain was recorded at rain gauges around the lake. General overview: With the dryer weather and falling lake level, many sites are drying up and wehave down-graded 19 conditions this inspection. Many condition 3 sites are showing less flow. One new condition 2 site was reported in Reach 7, which we believe may have been overlooked previously. No inspections were performed this week in Reaches 4 and 6.
Conditions by Area

	Condition 1	Condition 2	Condition 3	Condition 4
Reach 1	1	5	14	0
Reach 2	6	14	7	0
Reach 3	0	9	15	0
Reach 4	0	1	0	0
Reach 5	5	1	0	0
Reach 6	0	1	0	0
Reach 7	0	1	0	0
Reach 8	14	0	0	0
Totals	26	32	36	0

Summary of 4/15/98 Inspection

Lake Average: 17.44

New this inspection:		Total Reported:	
Condition One	0	Condition One	29
Condition Two	0	Condition Two	31
Condition Three	0	Condition Three	29
Condition Four	0	Condition Four	0
		TOTAL	89

During the week 4/08/98 - 4/14/98, no rain was recorded at rain gauges around the lake. General overview: 6 sites were eliminated and 8 were downgraded this inspection. Reaches 4, 5 6 and 8 were not inspected this week. No new sites were reported. As the lake went under 17.50, this will be our last weekly inspection. While we will continue to monitor our Condition 3 sites, overall inspections will now be performed
monthly while the lake remains 17.00 - 17.50.

Conditions by Area

	Condition 1	Condition 2	Condition 3	Condition 4
Reach 1	0	8	8	0
Reach 2	10	9	7	0
Reach 3	0	10	14	0
Reach 4	0	1	0	0
Reach 5	5	1	0	0
Reach 6	0	1	0	0
Reach 7	0	1	0	0
Reach 8	14	0	0	0
Totals	29	31	29	0

Reach 1 Sites:

Site R1-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	1/27/98	17.53	17.90	9.56

location: 1.8 miles south of S-352

LAT: 26-50-552 **LON:** 80-30-879

description: Standing water and saturated soil in an area 100'X10'. Staked at the beginning and end.

flow rate: N/A

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No change	0
2/10/98	17.64			No change	0
2/18/98	18.00			No change	2"
2/26/98	18.31	18.00	10.40	No change	0
3/4/98	18.28	18.20	10.40	No change	0
3/11/98	18.22	18.30	9.80	No change. Grass high	0
3/18/98	18.01	17.80	11.00	No change in condition.	0
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.90	12.10	No change	0
4/8/98	17.44	17.10	12.60	No change	0

Site R1-1A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	3/4/98	18.28	18.20	10.40

location: 1.5 miles south of S-352

LAT: 26-50-620 **LON:** NR

description: Standing water and saturated soil 50 yds from site R1-2

flow rate: N/A

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.30	9.80	No change.	0
3/18/98	18.01	17.80	11.00	No change in condition.	0
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.90	12.10	No change	0
4/8/98	17.44	17.10	12.60	No change	0

Site R1-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	1/27/98	17.53	17.90	9.56

location: 1.4 miles south of S-352

LAT: 26-50-713 **LON:** 80-38-722

description: Standing water in tire tracks overflowing into toe ditch, saturated berm for 1.6 miles

flow rate: .25 GPM

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59	17.90	9.56	No change	0
2/10/98	17.64			No change	0
2/18/98	18.00			No change	2"
2/26/98	18.31	18.00	10.40	No change	0
3/4/98	18.28	18.20	10.40	No change	0
3/11/98	18.22	18.30	9.80	No change.	0
3/18/98	18.01	17.80	11.00	Some drying since last inspection.	0
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change in condition. Seepage all the way to site 2B.	0
4/8/98	17.97	17.90	12.10	No change	0
4/15/98	17.44	17.10	12.60	No change	0

Site R1-2A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/4/98	18.28	18.20	10.40

location: 1.3 miles south of S-352
LAT: 26-50-620 **LON:** 80-38-700
description: Small seep with minimal flow on berm
flow rate: less than 1 GPM
nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.30	9.80	No change.	0
3/18/98	18.01	17.80	11.00	Flow has diminished.	0
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change in condition. Entire area has seepage.	0
4/8/98	17.97	17.90	12.10	No change	0
4/15/98	17.44	17.10	12.60	No change	0

Site R1-2B	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	3/18/98	18.01	17.80	11.00

location: .2 miles south of S-352
LAT: 26-51-380 **LON:** 80-38-204
description: Saturated area with ponding
flow rate: N/A
nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change in condition. Entire area has seepage.	0
4/8/98	17.97	17.90	12.10	No change	0
4/15/98	17.44	17.10	12.60	No change	0

Site R1-2C	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	3/18/98	18.01	17.80	11.00

location: .1 miles south of S-352

LAT: 26-51-649 **LON:** 80-38-090

description: 4'x2' area of ponding water at base of utility pole.

flow rate: N/A

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change.	0
4/8/98	17.97	17.90	12.10	No change	0
4/15/98	17.44	17.10	12.60	Area appears drier. less standing water.	0

Site R1-10	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	2/10/98	17.64	17.35	11.10

location: 2.6 miles north of S-351

LAT: 26-44-193 **LON:** 80-41-519

description: 2" diameter seep in upper bank of toe ditch.

flow rate: .33 GPM

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/18/98	18.00			Flow rate has increased to 1.0 GPM	2"
2/26/98	18.31	18.00	11.00	Flow increased to 2.0 GPM. Size increased to 4". Upgrade to C-4	0
3/4/98	18.28	N/R	N/R	No Change	0
3/5/98	18.26	N/R	N/R	Inspected by Nash and Honeycutt to determine source. Enclosed area with wood frame and sand bags. Filling w/ gravel and monitoring. Possible old drill bore or water pipe.	
3/11/98	18.22	18.30	9.80	Water flowing through toe bank, 1GPM.	0
3/18/98	18.01	17.80	11.00	Water flowing through toe bank, 1GPM. No flow through box.	0
3/25/98	18.42	18.30	9.95	Flow area has expanded and flow has increased to 2 GPM.	0
4/1/98	18.33	18.20	10.60	No change.	0
4/8/98	17.97	17.80	11.90	No change	0
4/15/98	17.44	17.10	12.60	No change	0

Site R1-10M	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	3/25/98	18.42	18.30	9.95

location: 2.4 miles south of Glades Airport
LAT: 26-45-17.64 **LON:** 80-41-03.08
description: 5 - 7 small seeps around culvert that flows from toe ditch under hwy 715.
flow rate: 1 GPM
nearest gauge: C-10

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/1/98	18.33	18.25	11.23	No change.	0
4/8/98	17.97	17.82	?	No change	0
4/15/98	17.44	17.35	12.35	No flow this inspection, downgrade to C-2	0

Site R1-10P	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/25/98	18.42	18.30	9.95

location: 1.5 miles south of Glades Airport, 1.2 miles north of Paul Rardin Park entrance
LAT: 26-45-975 **LON:** 80-41-325
description: 5 - 1 inch seeps flowing from top of toe ditch bank. Very heavy coating of white residue
flow rate: 2 - 3 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/1/98	18.33	18.25	11.23	No change.	0
4/8/98	17.97	17.82	?	No change	0
4/15/98	17.44	17.35	12.35	No change	0

Site R1-10R	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/25/98	18.42	18.30	9.95

location: 1.4 miles south of Glades Airport
LAT: 26-46-06.34 **LON:** 80-41-22.83
description: Numerous small seeps in toe ditch for .2 miles north from site marker.
flow rate: 2 - 3 GPM
nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/1/98	18.33	18.25	11.23	No observable flow.	0
4/8/98	17.97	17.82	?	No change	0
4/15/98	17.44	17.10	12.38	No observable flow.	0

Site R1-10T	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-0 C-3	3/25/98	18.42	18.30	9.95

location: Toe ditch, south end of Glades Airport
LAT: 26-46-52.35 **LON:** 80-41-42.50
description: 2 inch boil in bottom of toe ditch
flow rate: .5 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/1/98	18.33	18.25	11.23	No observable flow.	0
4/8/98	17.97	17.82	11.36	No observable flow.	0
4/15/98	17.44	17.10	12.38	No observable flow. Downgrade to No Condition.	0

Site R1-10V	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-0 C-3	3/25/98	18.42	18.30	9.95

location: Toe ditch, across from Glades Airport, near "crossing log"
LAT: 26-46-58.31 **LON:** 80-41-43.95
description: 4 seeps in toe bank, all about 1 inch in diameter, in 110 yd area. White staining present.
flow rate: .5 GPM to 2 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/1/98	18.33	18.25	11.23	No observable flow.	0
4/8/98	17.97	17.82	11.36	No observable flow.	0
4/15/98	17.44	17.10	12.38	No observable flow. Downgrade to No Condition.	0

Site R1-12	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2 C-3	2/10/98	17.64	17.70	unknown

location: 2.1 miles south of S-352
LAT: 26-50-277 **LON:** 80-38-933
description: Saturated soil and seepage on berm
flow rate: minimal
nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/18/98	18.00			No change	0
2/26/98	18.31	18.00	11.00	No change Vehicle stuck.	0
3/4/98	18.28	18.20	10.40	Marker removed by vandalism. Replaced. No change in condition	0
3/11/98	18.22	18.30	9.80	No change	0
3/18/98	18.01	17.80	11.00	No flow this inspection.	0
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.85	12.90	No change	0
4/15/98	17.44	17.10	12.38	No observable flow. Downgrade to C-2	0

Site R1-13	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2 C-3	3/4/98	18.28	18.20	10.40

location: 2 miles south of S-352, 100 yds from R1-12

LAT: 25-50-340 **LON:** 80-38-913

description: Large wet area on berm with flow into toe ditch.

flow rate: less than 1 GPM

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.30	9.80	No change	0
3/18/98	18.01	17.80	11.00	Area improving.	0
3/25/98	18.42	18.35	10.00	No change in condition.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.85	12.90	No change	0
4/15/98	17.44	17.10	12.38	No observable flow. Downgrade to C-2	0

Site R1B-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	1/28/98	17.51	17.90	9.56

location: North side of S-352

LAT: 26-51-831 **LON:** 80-37-946

description: light flow of muckish material from bottom of toe ditch.

flow rate: unknown

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59	17.90	9.56	No change	0
2/10/98	17.64			No change	0
2/18/98	18.00			Downgrade to C-3, no material moving	2"
2/26/98	18.31	18.20	10.20	No change Renamed to R1B-1 from R1-3 Toe Ditch level up	0
3/4/98	18.28	18.30	10.30	No change	0
3/11/98	18.22	18.20	9.80	No change. Water high and murky in toe ditch.	0
3/18/98	18.01	17.80	10.85	No change. Water high and murky in toe ditch.	0
3/25/98	18.42	18.35	10.00	No change. Water high and murky in toe ditch.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.80	12.45	Can't see site, water high and murky in toe ditch.	0
4/15/98	17.44	17.20	12.50	Can't see site, water high and murky in toe ditch.	0

Site R1B-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	2/26/98	18.31	18.20	10.20

location: 100' south of access rd. north side of S-352

LAT: 26-51-860 **LON:** 80-37-886

description: 5 small seeps in toe ditch

flow rate: minimal

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.30	10.30	No change	0
3/11/98	18.22	18.20	9.80	No change.	0
3/18/98	18.01	17.80	10.85	No change.	0
3/25/98	18.42	18.35	10.00	Flow increased to 1 GPM.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.80	12.45	Can't see site, water high in toe ditch.	0
4/15/98	17.44	17.20	12.50	Can't see site, water high in toe ditch.	0

Site R1B-3	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	1/28/98	17.51	17.90	9.56

location: .1 miles north of S-352

LAT: 26-51-908 **LON:** 80-37-860

description: Seepage through berm flowing into toe ditch.

flow rate: Not measured

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59	17.90	9.56	No change	0
2/10/98	17.64			No change	0
2/18/98	18.00			No change	2"
2/26/98	18.31	18.20	10.20	No change Renamed R1B-3 from R1-4	0
3/4/98	18.28	18.30	10.30	No change	0
3/11/98	18.22	18.20	9.80	No change.	0
3/18/98	18.01	17.80	10.85	No change.	0
3/25/98	18.42	18.35	10.00	No change.	0
4/1/98	18.33	18.20	10.60	No change	0
4/8/98	17.97	17.80	12.45	Can't see site, water high in toe ditch.	0
4/15/98	17.44	17.20	12.50	Can't see site, water high in toe ditch.	0

Site R1B-4	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	2/26/98	18.31	18.20	10.20

location: 100' north of access rd. north side of S-352
LAT: 26-51-976 **LON:** 80-37-816
description: Seepage flowing from berm Condition 4
flow rate: 1 GPM

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.30	10.30	No material moving. Downgrade to C-3	0
3/11/98	18.22	18.20	9.80	No change.	0
3/18/98	18.01	17.80	10.85	No change.	0
3/25/98	18.42	18.35	10.00	No change.	0
4/1/98	18.33	18.20	10.60	Increased flow. Wide-spread across berm.	0
4/8/98	17.97	17.80	12.45	No change.	0
4/15/98	17.44	17.20	12.50	No change.	0

Site R1B-4A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	1/28/98	17.51	17.90	9.56

location: .2 miles north of S-352
LAT: 26-52-012 **LON:** 80-37-769
description: Saturated soil on berm
flow rate:

nearest gauge: S-352

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59	17.90	9.56	No change	0
2/10/98	17.64			No change	0
2/18/98	18.00			No change	2"
3/4/98	18.28			No change	0
3/11/98	18.22			No Report	0
3/18/98	18.01			No Report	0
3/25/98	18.42	18.35	10.00	No Report	0
4/1/98	18.33	18.20	10.60	Increased flow. Wide-spread across berm.	0
4/8/98	17.97	17.80	12.45	Site drying.	0

4/15/98	17.44	17.20	12.50	Site drying.	0

Site R1B-6	**condition**		**date discovered**	**lake elevation**	**local upper and lower**
	C-0 C-1		1/28/98	17.51	
location:	1.4 miles north of S-352				
LAT:	26-56-158		**LON:**	80-36-756	
description:	Saturated soil on berm				
flow rate:	N/A				
		nearest gauge:		S-352	

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59	17.90	9.56	No change	0
2/10/98	17.64			No change	0
2/18/98	18.00			No change	2"
2/26/98	18.31	18.20	10.20	No change Renamed R1B-6 from R1-6	0
3/4/98	18.28			No Change C14 = 18.30 upper 15.80 lower	0
3/11/98	18.22	18.20	16.00	No change.	0
3/18/98	18.01	17.80	10.85	No change.	0
3/25/98	18.42	18.35	10.00	Water ponding at site. Upgrade to condition 2	0
4/1/98	18.33	18.20	10.60	No change.	0
4/8/98	17.97	17.80	12.45	Site dry this inspection.	0
4/15/98	17.44	17.20	12.50	Site dry this inspection. Downgrade to No Condition.	0

Site R2-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.32	12.50

location: .6 mile west of C-1

LAT: 26-50-027 **LON:** 81-04-277

description: Standing water on berm for 50 yds. with water flowing

flow rate: 1 GPM

nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	Water flow slowed to .25 GPM	0
3/11/98	18.22	18.20	12.40	No change.	0
3/18/98	18.01	17.92	11.80	No flow, standing water only	0
3/25/98	18.42	18.40	12.20	No flow, standing water only	0
4/1/98	18.33	18.28	12.27	No flow, standing water only	0
4/8/98	17.97	17.86	12.00	No flow, standing water only. Downgrading to C-2 this inspection.	0
4/15/98	17.44	17.30	12.80	No flow, standing water only. area getting smaller.	0

Site R2-1A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1 C-2	2/26/98	18.31	18.32	12.50

location: .3 mile west of C-1

LAT: 26-49-869 **LON:** 81-04-162

description: Standing water on berm for 50 yds. with water flowing

flow rate: 2 GPM

nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	Water flow slowed to 1.5 GPM	0
3/11/98	18.22	18.20	12.40	No change.	0
3/18/98	18.01	17.92	11.80	Less flow this inspection.	0
3/25/98	18.42	18.40	12.20	Flow increased	0
4/1/98	18.33	18.28	12.27	No change.	0
4/8/98	17.97	17.86	12.00	No flow, standing water only. Downgrading to C-2 this inspection.	0
4/15/98	17.44	17.30	12.80	Saturated soil only. Downgrade to C-1.	0

Site R2-1B	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1 C-2	2/26/98	18.31	18.32	12.50

location: .2 mile west of C-1

LAT: 26-49-769 **LON:** 81-04-047

description: Standing water for 100' on berm.

flow rate: N/A

nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	12.40	No change.	0
3/18/98	18.01	17.92	11.80	No ponding this inspection.	0
3/25/98	18.42	18.40	12.20	Standing water for 100' this inspection	0
4/1/98	18.33	18.28	12.27	No change.	0
4/8/98	17.97	17.86	12.00	Less standing water this inspection.	0
4/15/98	17.44	17.30	12.80	Saturated soil only. Downgrade to C-1.	0

Site R2-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/26/98	18.31	18.32	12.50

location: .2 mile east of C-1
LAT: 26-49-508 **LON:** 81-03-809
description: 25' of standing water
flow rate: N/A
nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	12.40	No change.	0
3/18/98	18.01	17.92	11.80	No ponding this inspection.	0
3/25/98	18.42	18.40	12.20	No change.	0
4/1/98	18.33	18.28	12.27	No change.	0
4/8/98	17.97	17.86	12.00	No ponding this inspection.	0
4/15/98	17.44	17.30	12.80	Saturated soil only.	0

Site R2-3	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/26/98	18.31	18.32	12.50

location: 1.2 miles east of C-1
LAT: 26-49-247 **LON:** 81-02-866
description: Saturated berm
flow rate: N/A
nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	12.40	Area dry.	0
3/18/98	18.01	17.92	11.80	Area saturated this inspection.	0
3/25/98	18.42	18.40	12.20	No change	0
4/1/98	18.33	18.28	12.27	No change	0
4/8/98	17.97	17.86	12.00	Area dry.	0
4/15/98	17.44	17.30	12.80	No change	0

Site R2-3A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1 C-2	3/4/98	18.28	18.20	12.22

location: 3.1 miles east of C-1
LAT: 26-48-828 **LON:** 81-00-400
description: 100 yds of saturated soil and ponding on berm
flow rate: N/A
nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.20	12.40	No change	0
3/18/98	18.01	17.92	11.80	No change	0
3/25/98	18.42	18.40	12.20	No change.	0
4/1/98	18.33	18.28	12.27	Area has expanded.	0
4/8/98	17.97	17.86	12.00	No change.	0
4/15/98	17.44	17.30	12.80	Saturated soil only. Downgrade to C-1.	0

Site R2-3B	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	4/1/98	18.33	18.28	12.27

location: 2.8 miles east of C-1
LAT: 26-48-985 **LON:** 81-00-847
description: 490' of saturated soil and ponding on berm
flow rate: N/A
nearest gauge: C-1

4/8/98	17.97	17.86	12.00	No standing water this inspection. Downgrading to C-1	0
4/15/98	17.44	17.30	12.80	No change.	0

Site R2-4	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/26/98	18.31	18.50	12.52

location: .2 mile east of Uncle Joe's

LAT: 26-48-588 **LON:** 80-58-939

description: 200 sq. ft. of saturated berm

flow rate: N/A

nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	12.40	No change	0
3/18/98	18.01	17.92	11.80	No change	0
3/25/98	18.42	18.40	12.20	No change.	0
4/1/98	18.33	18.28	12.27	No change.	0
4/8/98	17.97	17.86	12.00	Area dry this inspection.	0
4/15/98	17.44	17.30	12.80	No change.	0

Site R2-4A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/26/98	18.31	18.50	12.52

location: .4 mile east of Uncle Joe's

LAT: 26-48-430 **LON:** 80-58-829

description: 200 sq. ft. of saturated berm

flow rate: N/A

nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	12.40	No change	0
3/18/98	18.01	17.92	11.80	No change	0
3/25/98	18.42	18.40	12.20	No change.	0
4/1/98	18.33	18.28	12.27	No change.	0
4/8/98	17.97	17.86	12.00	No change.	0
4/15/98	17.44	17.30	12.80	No change.	0

Site R2-4B	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.50	12.52

location: .5 mile east of Uncle Joe's

LAT: 26-48-348 **LON:** 80-58-753

description: 100 sq. ft. of standing water on berm

flow rate: N/A

nearest gauge: C-1

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	12.40	No change	0
3/18/98	18.01	17.92	11.80	No standing water this inspection. Area still saturated.	0
3/25/98	18.42	18.40	12.20	No change.	0
4/1/98	18.33	18.28	12.27	No change.	0
4/8/98	17.97	17.86	12.00	No change.	0
4/15/98	17.44	17.30	12.80	No change.	0

Site R2-4C	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	4/1/98	18.33	18.52	11.90

location: 1.0 mile east of S-4
LAT: 26-46-801 **LON:** 80-56-913
description: 500' of saturated soil and ponding on berm
flow rate: N/A
nearest gauge: S-4

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	17.90	11.80	No change.	0
4/15/98	17.44	17.30	12.80	No change.	0

Site R2-4D	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	4/1/98	18.33	18.52	11.90

location: 2.3 miles east of S-4
LAT: 26-46-797 **LON:** 81-56-914
description: 500' of saturated soil and ponding on berm
flow rate: N/A
nearest gauge: S-4

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	17.90	11.80	No change.	0
4/15/98	17.44	17.30	12.80	No change.	0

Site R2-5	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/26/98	18.31	18.50	12.52

location: .1 mile east of C-2

LAT: 26-45-802 **LON:** 80-55-469

description: 50 yds X 50ft of standing water on berm

flow rate: N/A

nearest gauge: C-2

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	12.22	No change	0
3/11/98	18.22	18.20	13.38	No change	0
3/18/98	18.01	18.10	11.50	Area improving, area of standing water getting smaller.	0
3/25/98	18.42	18.56	12.20	No change.	0
4/1/98	18.33	18.52	11.90	No change.	0
4/8/98	17.97	17.90	11.80	No standing water this inspection. Downgrading to C-1	0
4/15/98	17.44	17.52	11.70	No change.	0

Site R2-6	**condition**		**date discovered**	**lake elevation**	**local upper and lower**
	C-3		1/23/98	17.53	
location:	1.7 miles NW of S-236				
LAT:	26-44-397		**LON:**	80-52-637	
description:	2 small seeps 18" apart on bank of toe ditch				
flow rate:	.1 GPM				
		nearest gauge:		S-236	

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
1/27/98	17.53			No change	0
2/3/98	17.59			No change	0
2/10/98	17.64	17.70	11.20	No change	0
2/18/98	18.00	18.00	12.10	No change	2"
2/26/98	18.31	18.38	8.30	No change, Renamed from R2-4	0
3/4/98	18.28	18.50	6.90	Adding another boil to this site 75' east, very slow flow.	0
3/11/98	18.22	18.30	7.10	No change	0
3/18/98	18.01	18.10	8.48	Less flow this inspection.	0
3/25/98	18.42	18.60	7.40	No change.	0
4/1/98	18.33	18.52	11.90	No change.	0
4/8/98	17.97	17.90	8.10	No change.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-6A	**condition**		**date discovered**	**lake elevation**	**local upper and lower**
	C-3		1/27/98	17.53	
location:	Toe ditch 50 yds east of Crooked hook				
LAT:	26-44-355		**LON:**	80-52-604	
description:	3" sand boil in toe ditch with white discoloration evident.				
flow rate:	.2 GPM				
		nearest gauge:		S-236	

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No change	0
2/10/98	17.64	17.70	11.20	No change	0
2/18/98	18.00	18.00	12.10	No change	2"
2/26/98	18.31	18.38	8.30	No change, Renamed from R2-5	0
3/4/98	18.28	18.50	6.90	No change	0
3/11/98	18.22	18.30	7.10	No change	0
3/18/98	18.01	18.10	8.48	Less flow this inspection.	0
3/25/98	18.42	18.60	7.40	Increase in flow this inspection.	0
4/1/98	18.33	18.52	11.90	No change.	0
4/8/98	17.97	17.90	8.10	Less flow this inspection.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-7	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.38	8.30

location: 3.6 miles east of S-310 / .9 mile west of S-236
LAT: 26-44-047 **LON:** 80-51-917
description: 50 ft X 20ft of standing water on berm
flow rate: N/A
nearest gauge: C-2

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.50	6.90	No change. Staked beginning and end.	0
3/11/98	18.22	18.30	7.10	No change	0
3/18/98	18.01	18.10	8.48	No change	0
3/25/98	18.42	18.60	7.40	No change	0
4/1/98	18.33	18.52	11.90	No change.	0
4/8/98	17.97	17.90	8.10	No change.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-7A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.38	8.30

location: 3.7 miles east of S-310 / .8 mile west of S-236
LAT: 26-43-911 **LON:** 80-51-803
description: 50 ft X 20ft of standing water on berm
flow rate: N/A
nearest gauge: C-2

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.50	6.90	No change. Staked beginning and end.	0
3/11/98	18.22	18.30	7.10	No change	0
3/18/98	18.01	18.10	8.48	No change	0
3/25/98	18.42	18.60	7.40	No change	0
4/1/98	18.33	18.52	11.90	No change.	0
4/8/98	17.97	17.90	8.10	No change.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-8	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-3	1/27/98	17.53	

location: 3.9 miles east of S-310 .5 mile west of S-236

LAT: 26-43-912 **LON:** 80-51-648

description: 2" sand boil in toe ditch with white discoloration evident.

flow rate: .2 GPM

nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No change	0
2/10/98	17.64	17.70	11.20	No change	0
2/18/98	18.00	18.00	12.10	No change	2"
2/18/98	18.31	18.38	8.30	No change, Previously R2-3 and R2-6	0
3/4/98	18.28	18.50	6.90	No change, Gauges from C-3	0
3/11/98	18.22	18.30	7.10	No change, Gauges from C-3	0
3/18/98	18.01	18.10	8.48	Less flow this inspection.	0
3/25/98	18.42	18.60	7.40	No change	0
4/1/98	18.33	18.30	7.40	Increased to 4" . Flow increased to 4-5 GPM	0
4/8/98	17.97	17.90	8.10	No change.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-8A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-0 C-2	4/1/98	18.33	18.30	7.40

location: .4 mile west of S-236
LAT: 26-43-924 **LON:** 80-51-626
description: .4 mile of saturated soil and ponding on berm
flow rate: N/A
nearest gauge: S-236

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	17.90	8.10	No Change.	0
4/15/98	17.44	17.60	9.80	Area dry. Downgrade to no condition.	0

Site R2-9	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2	1/14/98	17.24	

location: 1.5 miles east of C-3
LAT: 26-43-029 **LON:** 80-50-426
description: Saturated berm with ponding water.
flow rate: N/A
nearest gauge: S-236

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
1/27/98	17.53			No Change	0
2/3/98	17.59			No Change	0
2/10/98	17.64	17.70	11.20	No Change	0
2/18/98	18.00	18.00	12.10	No Change	2"
2/26/98	18.31	18.48	8.30	No change, Previously R3-1	0
3/4/98	18.28	18.50	6.90	No change. Staked beginning and end.	0
3/11/98	18.22	18.40	10.80	No change, Gauges from S-354.	0
3/18/98	18.01	18.10	8.48	No Change	0
3/25/98	18.42	18.60	7.40	No Change	0
4/1/98	18.33	18.30	7.40	No Change	0
4/8/98	17.97	17.90	8.10	No Change	0
4/15/98	17.44	17.60	9.80	No Change	0

Site R2-9A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/4/98	18.28	18.50	6.90

location: 2.5 miles west of S-3
LAT: 26-42-958 **LON:** 80-50-361
description: 1 inch boil in north bank of toe ditch. Sulfur odor and sulfur traces.
flow rate: .25 GPM
nearest gauge: S-236

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.40	10.80	No change, Gauges from S-354.	0
3/18/98	18.01	18.10	8.48	No flow this inspection.	0
3/25/98	18.42	18.60	7.40	Another boil added to site 200' west of first.	0
4/1/98	18.33	18.30	7.40	No change	0
4/8/98	17.97	17.90	8.10	Less flow this inspection.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-9B	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	3/18/98	18.01	18.10	8.48

location: 1.8 miles east of S-236
LAT: 26-42-812 **LON:** 80-50-206
description: Standing water for 40yds. at base of levee.
flow rate: N/A
nearest gauge: S-236

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.60	7.40	No Change	0
4/1/98	18.33	18.30	7.40	No Change	0
4/8/98	17.97	17.90	8.10	No Change	0
4/15/98	17.44	17.60	9.80	No Change	0

Site R2-10	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1 C-2	1/14/98	17.24	

location: 3 miles east of C-3
LAT: 26-42-338 **LON:** 80-49-765
description: Saturated berm
flow rate: N/A
nearest gauge: S-236

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
1/27/98	17.53			No Change.	0
2/3/98	17.59			No Change.	0
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/26/98	18.31	18.48	8.30	Upgrade to C-2, Previously R3-2	0
3/4/98	18.28	18.50	6.90	No Change.	0
3/11/98	18.22	18.40	10.80	No change, Gauges from S-354.	0
3/18/98	18.01	18.10	8.48	Area drying, no ponding this inspection.	0
3/25/98	18.42	18.60	7.40	No Change.	0
4/1/98	18.33	18.30	7.40	No Change.	0
4/8/98	17.97	17.90	8.10	No Change.	0
4/15/98	17.44	17.60	9.80	Area drying, no ponding this inspection. Downgrade to C-1	0

Site R2-10A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/11/98	18.22	18.40	10.80

location: 50 yds. west of site R2-11
LAT: 26-42-137 **LON:** 80-49-574
description: 2" boil in toe ditch
flow rate: N/A
nearest gauge: S-236

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/18/98	18.01	18.10	8.48	No Change.	0
3/25/98	18.42	18.60	7.40	No Change.	0
4/1/98	18.33	18.30	7.40	No Change.	0
4/8/98	17.97	17.90	8.10	No Change.	0

4/15/98	17.44	17.60	9.80	No Change.	0

Site R2-11	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	2/26/98	18.31	18.20	10.31

location: 2.4 miles east of S-236 1.2 miles west of S-3

LAT: 26-42-074 **LON:** 80-49-513

description: 2 boils on N. side of toe ditch. Sulfur odor and discoloration present

flow rate: .25 GPM

nearest gauge: S-3

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	N/R	N/R	No change.	0
3/11/98	18.22	18.40	10.80	No change, Gauges from S-354.	0
3/18/98	18.01	18.10	8.48	No change, Gauges from S-236.	0
3/25/98	18.42			Increase in flow this inspection	0
4/1/98	18.33	18.30	7.40	No change, Gauges from S-236.	0
4/8/98	17.97	17.90	8.10	Adding another small boil to this site. Gauges from S-236.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R2-12	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/4/98	18.28	N/R	N/R

location: 2.5 miles west of S-3, .1 mile east of site R2-11
LAT: 26-41-997 **LON:** 80-49-395
description: 4 inch boil. Sulfur odor and sulfur traces.
flow rate: .50 GPM

nearest gauge: S-3

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.40	10.80	No change, Gauges from S-354.	0
3/18/98	18.01	18.10	8.48	Flow has decreased to .25 GPM. Gauges from S-236.	0
3/25/98	18.42			Increase in flow this inspection	0
4/1/98	18.33	18.30	7.40	No change, Gauges from S-236.	0
4/8/98	17.97	17.90	8.10	No change, Gauges from S-236.	0
4/15/98	17.44	17.60	9.80	Reduce flow to .25 GPM.	0

Site R2-12A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	4/8/98	18.33	N/R	N/R

location: 5 miles west of S-3, to 450' west of S-3
LAT: 26-41-904 **LON:** 80-48-716
description: Saturated berm with ponding.
flow rate: N/A

nearest gauge: S-3

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	17.90	8.10	No change, Gauges from S-236.	0
4/15/98	17.44	17.60	9.80	No change.	0

Site R3-A1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	3/18/98	18.01	17.90	11.40

location: .1 mile east of S-354
LAT: 26-41-894 **LON:** 80-48-151
description: Ponding, no flow in tire tracks on berm, 30'X10'.
flow rate: N/A
nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.45	8.00	No change	0
4/1/98	18.33	18.40	10.90	No change	0
4/8/98	17.97	N/R	N/R	Area drier	0
4/15/98	17.44	17.40	9.30	No change	0

Site R3-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.33	10.92

location: .5 mile east of S-354
LAT: 26-41-893 **LON:** 80-47-892
description: Ponding, no flow
flow rate: N/A
nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.63	11.06	No change	0
3/11/98	18.22	18.40	10.80	No change, Renamed from R3-13	0
3/18/98	18.01	17.90	11.40	Area almost dry.	0
3/25/98	18.42	18.45	8.00	No change	0
4/1/98	18.33	18.40	10.90	Area drier	0
4/8/98	17.97	N/R	N/R	Area drier.	0
4/15/98	17.44	17.40	9.30	No change	0

Site R3-1A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	4/1/98	18.33	18.40	10.90

location: .5 mile east of S-354, eastern edge of tree farm.
LAT: 26-41-875 **LON:** 80-47-825
description: Several small seeps in 2' area of toe ditch bank
flow rate: .3 GPM total
nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	N/R	N/R	No change this inspection.	0
4/15/98	17.44	17.40	9.30	Water level up in ditch. can't see site.	0

Site R3-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2	1/27/98	17.53	

location: 1 mile east of S-354 at 'hump' repair site.
LAT: 26-41-879 **LON:** 80-47-628
description: 150 ' Saturated berm with ponding, no flow Ground spongy near hump.
flow rate: N/A
nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No Change.	0
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/26/98	18.31	18.33	10.92	No Change.	0
3/4/98	18.28	18.50	8.00	No Change.	0
3/11/98	18.22	18.40	10.80	No change, Renamed from R3-5	0
3/18/98	18.01	17.90	11.40	No Change.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.40	10.90	No Change.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.40	9.30	No Change.	0

Site R3-3	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2 C-3	1/27/98	17.53	

location: 1.1 mile east of S-354, just east of site R3-5

LAT: 26-41-881 **LON:** 80-47-584

description: Saturated berm with ponding, and seepage

flow rate: N/A

nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59	N/R	N/R	No Change.	0
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/26/98	18.31	18.33	10.92	Upgrade from C-2 to C-3 small active seep	0
3/4/98	18.28	N/R	N/R	Several small seeps added to site	0
3/11/98	18.22	18.40	10.80	No change, Renamed from R3-6	0
3/18/98	18.01	17.90	11.40	No Change.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.40	10.90	No Change.	0
4/8/98	17.97	N/R	N/R	No change this inspection.	0
4/15/98	17.44	17.40	9.30	No flowing water. Downgrade to C-2.	0

Site R3-3.1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	4/1/98	18.33	18.40	10.90

location: 1.1 miles east of S354, 30' west of DOT culvert 14159 on land-side of toe ditch.

LAT: 26-41-907 **LON:** 80-47-310

description: 6" seep creating heavy sluffing of bank. No sulfur odor or white staining, including several small seeps around culvert at this site.

flow rate: 5 GPM

nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	N/R	N/R	Slower flow this inspection.	0
4/15/98	17.44	17.40	9.30	No Change.	0

Site R3-3.2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	4/1/98	18.33	18.40	10.90

location: 1.4 miles east of S354, DOT culvert 12144.

LAT: 26-41-930 **LON:** 80-46-934

description: numerous small seeps with white staining in culvert basin. including several small seeps around culvert at this site.

flow rate: 1 GPM

nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	N/R	N/R	No change this inspection.	0
4/15/98	17.44	17.40	9.30	No change	0

Site R3-4	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/11/98	18.22	18.50	9.00

location: 3.0 miles east of S-354

LAT: 26-41-691 **LON:** 80-46-454

description: 2" seep in toe ditch, lakeside bank, 4" above water. Still has old sandbags from '95-'96

flow rate: 2 GPM

nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.45	8.00	Increased flow	0
4/1/98	18.33	18.40	10.90	No Change.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.40	9.30	No Change.	0

Site R3-5	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	1/27/98	17.53		
location:	3.4 miles east of S-354				
LAT:	26-41-533	**LON:**	80-46-265		
description:	170' Saturated berm with ponding, no flow				
flow rate:	N/A				
		nearest gauge:	C4-A		

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No Change.	0
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/26/98	18.31	18.33	10.92	No Change.	0
3/4/98	18.28	N/R	N/R	No Change.	0
3/11/98	18.22	18.50	9.00	No change, Renamed from R3-7	0
3/18/98	18.01	17.90	11.40	No Change.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.20	9.30	Area drier this inspection	0
4/8/98	17.97	N/R	N/R	Area drier this inspection	0
4/15/98	17.44	17.40	9.30	No change	0

Site R3-6	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2	1/27/98	17.53	

location: 3.5 miles east of S-354

LAT: 26-41-558 **LON:** 80-46-197

description: Saturated berm with ponding, no flow

flow rate: N/A

nearest gauge: C4-A

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No Change	0
2/10/98	17.64	17.70	11.20	No Change	0
2/18/98	18.00	18.00	12.10	No Change	2"
2/26/98	18.31	18.33	10.92	No Change	0
3/4/98	18.28	N/R	N/R	No Change	0
3/11/98	18.22	18.50	9.00	No change, Renamed from R3-8	0
3/18/98	18.01	17.90	11.40	No Change	0
3/25/98	18.42	18.45	8.00	No Change	0
4/1/98	18.33	18.20	9.30	No Change	0
4/8/98	17.97	N/R	N/R	Area drier this inspection	0
4/15/98	17.44	17.40	9.30	No Change	0

Site R3-7	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2	1/27/98	17.53	

location: 3.6 miles east of S-354

LAT: 26-41-496 **LON:** 80-46-110

description: Saturated berm with ponding, no flow. .25 mile long

flow rate: N/A

nearest gauge: C4-A

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/3/98	17.59			No Change.	0
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/26/98	18.31	18.33	10.92	No Change.	0
3/4/98	18.28	N/R	N/R	No Change.	0
3/11/98	18.22	18.50	9.00	No change, Renamed from R3-9	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.20	9.30	No Change.	0
4/8/98	17.97	N/R	N/R	Area drier this inspection	0
4/15/98	17.44	17.40	9.30	No change, Berm highly saturated.	0

Site R3-8	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/4/98	18.28	18.50	8.00

location: 4.0 miles east of S-354
LAT: 26-41-291 **LON:** 80-45-739
description: 2" seep. No sulfur odor
flow rate: could not tell
nearest gauge: C-4A

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.50	9.00	No change, Renamed from R3-16	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.20	9.30	No Change.	0
4/8/98	17.97	N/R	N/R	Site under water in toe ditch. Could not see.	0
4/15/98	17.44	17.40	9.30	Vegetation in ditch. Could not see.	0

Site R3-9	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.40	10.75

location: 0.3 mile east of C-4A
LAT: 26-40-846 **LON:** 80-44-660
description: Ponding, no flow
flow rate: N/A
nearest gauge: C-4A

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.63	11.06	No Change.	0
3/11/98	18.22	18.50	9.00	No change, Renamed from R3-14	0
3/18/98	18.01	17.82	11.22	Area getting smaller.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.20	9.30	Area has expanded	0
4/8/98	17.97	N/R	N/R	Area getting smaller.	0
4/15/98	17.44	17.40	9.30	Site drying.	0

Site R3-10	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2	1/14/98	17.24	

location: .7 miles east of C-4A

LAT: 26-40-794 **LON:** 80-44-317

description: Saturated berm with ponding, no flow

flow rate: N/A

nearest gauge: C-4A

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
1/27/98	17.53			No Change.	0
2/3/98	17.59			No Change.	0
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/26/98	18.31	18.40	11.75	No Change.	0
3/4/98	18.28	18.50	8.00	No Change.	0
3/11/98	18.22	18.40	10.80	No change, Renamed from R3-10	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.45	8.00	No Change.	0
4/1/98	18.33	18.20	9.30	Area has expanded	0
4/8/98	17.97	N/R	N/R	Area getting smaller.	0
4/15/98	17.44	17.40	9.30	No change	0

Site R3-10A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/18/98	18.01	17.82	11.22

location: .2 miles north of South Bay boat ramp.
LAT: 26-41-180 **LON:** 80-43-639
description: 2 seeps in toe bank at 90 degree juncture of ditch. Each seep 4"X1".
flow rate: 1 GPM each
nearest gauge: S351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.50	10.50	No Change.	0
4/1/98	18.33	18.26	10.72	No Change.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.40	9.30	High water in ditch. Can't see site.	0

Site R3-11	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/4/98	18.28	N/R	N/R

location: .4 mile north of South Bay boat ramp
LAT: 26-41-389 **LON:** 80-43-467
description: 6" X 2" seep next to piezometer P2C E3 P4
flow rate: .2 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.50	9.00	No change, Renamed from R3-17	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.50	10.50	No Change.	0
4/1/98	18.33	18.26	10.72	No change Toe ditch elevation - 7.68	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.40	9.30	No Change.	0

Site R3-12	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C2	2/3/98	17.59	

location: .5 mile north of South Bay boat ramp

LAT: 26-41-394 **LON:** 80-43-437

description: Seep located in toe ditch. Discoloration and sulfur smell.

flow rate: unknown

nearest gauge: S-354

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/10/98	17.64	17.70	11.20	No Change.	0
2/18/98	18.00	18.00	12.10	No Change.	2"
2/18/98	18.31	18.33	10.92	No seepage. Downgrade from C-3 to C-2	0
3/4/98	18.28	18.63	11.06	No Change.	0
3/11/98	18.22	18.54	10.92	No change, Renamed from R3-11	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.50	10.50	Site under water this inspection.	0
4/1/98	18.33	18.26	10.72	Site under water this inspection.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.40	9.30	No Change.	0

Site R3-13	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/4/98	18.28	N/R	N/R

location: .5 mile north of South Bay boat ramp
LAT: 26-41-398 **LON:** 80-43-450
description: 2" seep with light flow
flow rate: could not tell
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/11/98	18.22	18.50	9.00	No change. Renamed from R3-18	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.50	10.50	No Change.	0
4/1/98	18.33	18.26	10.72	Site under water this inspection.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.50	10.50	No Change.	0

Site R3-13.1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/18/98	18.01	17.82	11.22

location: .7 mile north of Lake Harbor boat ramp.
LAT: 26-41-446 **LON:** 80-43-426
description: 2 seeps 1" each 2' apart at bottom of toe bank. White residue present.
flow rate: 1 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.50	10.50	No change	0
4/1/98	18.33	18.26	10.72	Flow increased to 3 GPM.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.50	10.50	No Change.	0

Site R3-13.2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/18/98	18.01	17.82	11.22

location: .7 mile north of Lake Harbor boat ramp.
LAT: 26-41-462 **LON:** 80-43-411
description: 3" seep at bottom of toe bank. White residue present.
flow rate: 2 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.50	10.50	No change	0
4/1/98	18.33	18.26	10.72	Flow increased to 3 GPM.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.50	10.50	No Change.	0

Site R3-13A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	3/18/98	18.01	17.82	11.22

location: .7 mile north of Lake Harbor boat ramp.
LAT: 26-41-458 **LON:** 80-43-395
description: Volcano formation rising 4" from bottom of toe ditch. 12" to 14" in diameter.
All material appears to be organic. Sandbagged site.

flow rate: 5 GPM
nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/25/98	18.42	18.50	10.50	Site contained within sandbags, flow decreased to 1.5 GPM, Behaving as a condition 3.	0
4/1/98	18.33	18.26	10.72	No Change.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.50	10.50	No Change.	0

Site R3-14	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	2/26/98	18.31	18.40	10.75

location: .8 miles south of S-351

LAT: 27-4-84 **LON:** 80-40-17

description: Baseball size boil moving fine white material.

flow rate: could not tell

nearest gauge: S-531

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.63	11.06	Added second boil to site with slight movement of material, Sandbagged to contain and build head pressure.	0
3/11/98	18.22	18.54	10.92	No change. Renamed from R3-15	0
3/18/98	18.01	17.82	11.22	No Change.	0
3/25/98	18.42	18.50	10.50	No Change.	0
4/1/98	18.33	18.26	10.72	Flow increased at both boils.	0
4/8/98	17.97	N/R	N/R	No Change.	0
4/15/98	17.44	17.50	10.50	No Change.	0

Site R3-15	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	4/1/98	18.33	18.26	10.72

location: .9 mile north of South Bay boat ramp.

LAT: 26-41-689 **LON:** 80-43-197

description: Condition 3 seep that has sluffed 2' X 8" into toe bank. Seep area 3". Picking up white residual material with flow.

flow rate: 2 GPM

nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	N/R	N/R	No change. Tried to measure horizontal depth of seepage channel. Goes beyond 6'.	0
4/15/98	17.44	17.50	10.50	Less flow this inspection.	0

Site R3-16	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-3	4/1/98	18.33	18.26	10.72

location: 1.0 mile north of South Bay boat ramp.

LAT: 26-41-737 **LON:** 80-43-175

description: 2 condition 3 seeps in toe ditch, one lake-side, one land-side and 8' north. Lake-side seep has sluffed. 1' of bank, is 4" & flowing 2 GPM with white residue. Land-side seep is 2" and flowing 3 GPM.

flow rate: 5 GPM

nearest gauge: S-351

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/8/98	17.97	N/R	N/R	No change.	0
4/15/98	17.44	17.50	10.50	No change	0

Site R4-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-2	2/26/98	18.31	

location: .2 miles w of S-77
LAT: 26-51-042 **LON:** 81-05-482
description: Standing water on berm
flow rate: N/A
nearest gauge: S-77

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	N/R	N/R	No change	0
3/11/98	18.22	18.00	11.70	No change	0
3/18/98	18.01			No change.	0
3/25/98	18.42			No inspection this period.	0

Site R5-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/27/98	18.31	18.16	13.60

location: .8 mile north of Hwy 78
LAT: **LON:**
description: Swampy area believed to be permanent condition
flow rate: N/A
nearest gauge: S-133

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.12	13.48	No change	0
3/11/98	18.22	18.02	13.75	No change	0
3/18/98	18.01	18.00	13.35	No change	0
3/25/98	18.42	18.35	13.47	No change	0
4/8/98	17.97	18.15	13.48	Ponding gone. Downgrading to C-1	0

Site R5-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/27/98	18.31	18.16	13.60

location: .9 mile east of Lock 7
LAT: **LON:**
description: Saturated berm with ponding
flow rate: N/A
nearest gauge: S-133

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.12	13.48	Upgrade to C-2	0
3/11/98	18.22	18.02	13.75	No change	0
3/18/98	18.01	18.00	13.35	No change	0
3/25/98	18.42	18.35	13.47	No change	0
4/8/98	17.97	18.15	13.48	Ponding gone. Downgrading to C-1	0

Site R5-3	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/27/98	18.31	18.16	13.60

location: 1.2 miles east of Lock 7
description: Saturated berm with ponding
flow rate: N/A
nearest gauge: S-133

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.12	13.48	No change	0
3/11/98	18.22	18.02	13.75	No change	0
3/18/98	18.01	18.00	13.35	No change	0
3/25/98	18.42	18.35	13.47	No change	0
4/8/98	17.97	18.15	13.48	Ponding gone. Downgrading to C-1	0

Site R5-4	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/4/98	17.59	

location: 1 mile west of S-193

LAT: **LON:**

description: Sinkhole on levee, 6" to 8" depression. Probed to 3'

flow rate: N/A

nearest gauge: S-133

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
2/10/98	17.64			No change	0
2/18/98	18.00			No change	2"
2/27/98	18.31			No change, Renamed from R5-1	0
3/4/98	18.28	18.12	13.48	No change	0
3/11/98	18.22	18.02	13.75	No change	0
3/18/98	18.01	18.00	13.35	No change	0
3/25/98	18.42	18.35	13.47	No change	0
4/8/98	17.97	18.15	13.48	No change	0

Site R5-6	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/27/98	18.31	18.16	13.60

location: 1.9 miles east of Lock 7

LAT: **LON:**

description: Saturated berm

flow rate: N/A

nearest gauge: S-133

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.12	13.48	No change	0
3/11/98	18.22	18.02	13.75	No change	0
3/18/98	18.01	18.00	13.35	No change	0
3/25/98	18.42	18.35	13.47	No change	0
4/8/98	17.97	18.15	13.48	Area drier	0

Site R5-7	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/27/98	18.31	18.53	18.52

location: north side of S-191

LAT: **LON:**

description: Pond - 60 yd X 30 yd. Not believed to be seepage.

flow rate: N/A

nearest gauge: S-191

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.48	18.34	13.48	No change	0
3/11/98	18.22	18.31	18.30	No change	0
3/18/98	18.01	18.00	13.35	No change	0
3/25/98	18.42	18.50	18.50	No change	0
4/8/98	17.97	18.35	18.25	No change	0

Site R6-1A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	2/26/98	18.31	18.50	12.80

location: Landside of S-131

LAT: **LON:**

description: Seepage where levee meets road, flowing onto road. 400' long

flow rate: N/A

nearest gauge: S-131

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.34	13.94	No change	0
3/11/98	18.22	18.21	13.30	No change	0
3/18/98	18.01	18.26	13.40	No change	0
3/25/98	18.42			No inspection this period.	0

Site R7-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-2	4/8/98	17.97	17.93	?

location: 1.3 miles north of levee gate at S-308

LAT: 27-00-683 **LON:** 80-37-777

description: Saturated berm with standing water 300' - 400' long

flow rate: N/A

nearest gauge: S-308

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
4/15/98	17.44	N/R	N/R	Site drying.	0

Site R8-1	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: .4 miles south of Hwy 78 on North levee
LAT: 27-03-63 **LON:** 80-58-38
description: Saturated berm with ponding, 100'X20'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	20.90	18.10	No change	0
3/18/98	18.01	21.10		Ponding gone.	0
3/25/98	18.42	20.17	18.40	Ponding again	0
4/8/98	17.97	20.80	18.09	Ponding gone. Downgrading to C-1	0

Site R8-2	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: .6 miles south of Hwy 78 on North levee
LAT: 27-03-70 **LON:** 80-58-23
description: Saturated berm, 300'X30'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	20.90	18.10	No change	0
3/18/98	18.01	21.10		No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	No change	0

Site R8-3	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 1.0 miles south of Hwy 78 on North levee
LAT: 27-03-93 **LON:** 80-57-94
description: Saturated berm, 600X40
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	20.90	18.10	No change	0
3/18/98	18.01	21.10		No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	No change	0

Site R8-4	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 1.4 miles south of Hwy 78 on North levee
LAT: 27-04-16 **LON:** 80-57-74
description: Saturated berm with ponding, 44'X40'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	18.06	13.60	No change, Gauge readings from S-127	0
3/18/98	18.01	21.10		No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	Area dry this inspection. Downgrading to C-1	0

Site R8-5	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 1.9 miles south of Hwy 78 on North levee
LAT: 27-04-42 **LON:** 80-57-42
description: Saturated berm with ponding, 350'X30'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	18.06	13.60	No change. Gauge readings from S-127	0
3/18/98	18.01	21.10		No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	Area dry this inspection. Downgrading to C-1	0

Site R8-6	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 2.1 miles south of Hwy 78 on North levee
LAT: 27-04-56 **LON:** 80-57-28
description: Saturated berm with ponding, 450'X20'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	18.06	13.60	No change. Gauge readings from S-127	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	Area dry this inspection. Downgrading to C-1	0

Site R8-7	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 2.5 miles south of Hwy 78 on North levee
LAT: 27-04-74 **LON:** 80-57-05
description: Saturated berm with ponding, 650'X40'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	More water since last inspection	0
3/11/98	18.22	18.06	13.60	No change. Gauge readings from S-127	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	No standing water this inspection. Downgrading to C-1	0

Site R8-8	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 3.8 miles south of Hwy 78 on North levee
LAT: 27-05-57 **LON:** 80-56-12
description: Saturated berm with ponding, 500'X30'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	18.06	13.60	No change. Gauge readings from S-127	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	20.80	18.09	Area dry this inspection. Downgrading to C-1	0

Site R8-9	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 5.6 miles south of Hwy 78 on North levee
LAT: 27-06-61 **LON:** 80-54-93
description: Saturated berm with ponding, 180'X30'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	18.06	13.60	No change. Gauge readings from S-127	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	18.15	13.48	Area dry this inspection. Downgrading to C-1	0

Site R8-10	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 6.1 miles south of Hwy 78 on North levee
LAT: 27-06-89 **LON:** 80-54-62
description: Saturated berm 60'X20'
flow rate: N/A
nearest gauge: S-72

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	20.28	18.10	No change	0
3/11/98	18.22	18.06	13.60	No change. Gauge readings from S-127	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.17	18.40	No change	0
4/8/98	17.97	18.15	13.48	Area dry this inspection.	0

Site R8-11	**condition**	**date discovered**	**lake elevation**	**local upper and lower**
	C-1	2/26/98	18.31	

location: 6.2 miles south of Hwy 78 on North levee
LAT: 27-06-94 **LON:** 80-54-55
description: Saturated berm .8 mileX30'
flow rate: N/A
nearest gauge: S-127

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	13.46	No change	0
3/11/98	18.22	18.06	13.60	No change	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	18.45	13.62	No change	0
4/8/98	17.97	18.15	13.48	Area dry this inspection.	0

Site R8-11A	**condition**	**date discovered**	**lake elevation**	**local upper and lower**	
	C-1	2/26/98	18.31	18.20	13.46

location: 6.4 miles south of Hwy 78 on North levee
LAT: 27-07-22 **LON:** 80-54-24
description: Ponding, 3' diameter
flow rate: N/A
nearest gauge: S-127

updates:	**lake avg**	**upper**	**lower**	**comments on condition**	**rain amount for previous 24 hrs.**
3/4/98	18.28	18.20	13.46	No change	0
3/11/98	18.22	18.06	13.60	Size has increased to 3' X 8'	0
3/18/98	18.01	18.05	13.55	Ponding has dried up.	0
3/25/98	18.42	18.45	13.62	Ponding has returned.	0
4/8/98	17.97	18.15	13.48	Area dry this inspection. Downgrading to C-1	0

Site	R8-12	condition	date discovered	lake elevation	local upper and lower
		C-1	2/26/98	18.31	

location: .3 miles north of S-127

LAT: 27-07-22 **LON:** 80-53-35

description: Saturated berm 300'X50'

flow rate: N/A

nearest gauge: S-72

updates:	lake avg	upper	lower	comments on condition	rain amount for previous 24 hrs.
3/4/98	18.28	18.20	13.46	No change	0
3/11/98	18.22	18.06	13.60	No change	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.70	18.40	No change	0
4/8/98	17.97	18.15	13.48	Area dry this inspection. Downgrading to C-1	0

Site	R8-13	condition	date discovered	lake elevation	local upper and lower
		C-1	2/26/98	18.31	

location: .9 miles north of S-127

LAT: **LON:**

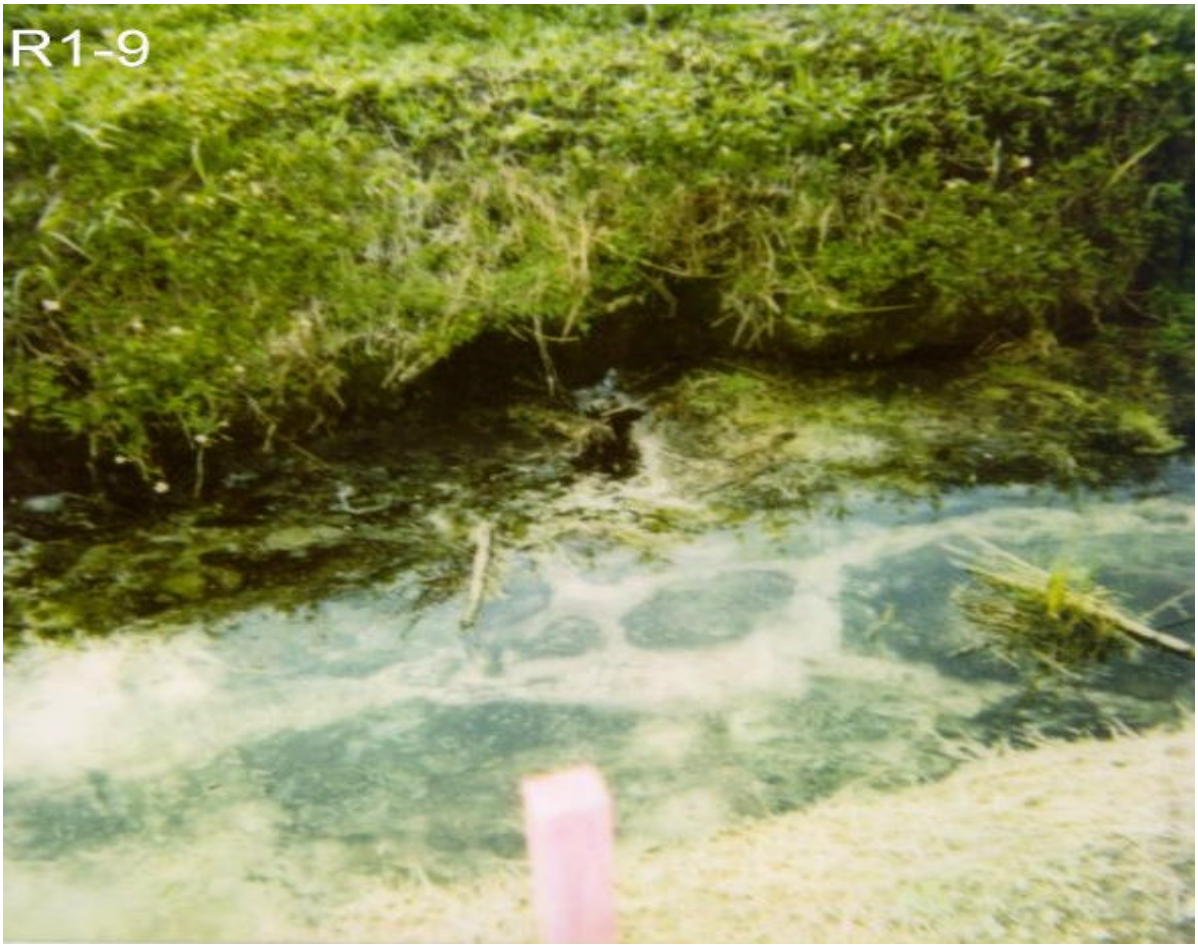
description: Saturated berm 100'X50'

flow rate: N/A

nearest gauge: S-72

updates:	lake avg	upper	lower	comments on condition	rain amount for previous 24 hrs.
3/4/98	18.28	18.20	13.46	No change	0
3/18/98	18.01	18.05	13.55	No change	0
3/25/98	18.42	20.70	18.40	No change	0
4/8/98	17.97	18.15	13.48	Area dry this inspection.	0

7.2.4 Photographs. Following are photographs documenting conditions during the 1998 high water event.



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Photograph of site R1-9

Photograph of site R3-15

Photograph of site R1b-4

Photograph of site R3-2

Photograph of site R2-11

Photograph of site R3-12

Photograph of site R1b-1

Photograph of site approximately 1 mile north of South Bay boat ramp. Ditch water flowing into limestone at base of peat in toe ditch

Close up of previous photograph. Approximately 1 mile north of South Bay boat ramp. Ditch water flowing into limestone at base of peat in toe ditch

Photograph of site R3-13.2

Photograph of site R3-13A

8. ANALYSIS AND RESULTS.

This section describes the criteria applied to interpret the results of both seepage and slope stability analyses.

8.1 Definition Of Failure. In the context of engineering analyses, failure can be defined in many ways. In the context of traditional geotechnical design, failures can be defined as exceeding the limiting equilibrium state. For slope stability, this is generally thought of as exceeding the Mohr-Columb failure envelope for shear strength of a particular soil or rock. For seepage and piping, this is generally thought of as exceeding the critical gradient of the soil resulting in flotation of the soil particles and possible subsequent removal by seepage flow. In order to provide results in a format compatible with the economic analysis, the probabilities of unsatisfactory performance (Pup's) provided must describe the probability of a breach. Stage damage curves represent damage caused by inundation. There is no inundation without breach. For the purposes of this report, breach is defined as follows:

Breach - the condition where some portion of the embankment has been removed such that the crest of the remnant embankment has been lowered below the lake levee, such that overtopping of the remnant embankment can occur with subsequent uncontrolled release of water from the lake.

8.1.1 Failure by Seepage. The process of failure by seepage leading to breach for this project is expected to occur by the following three part process.

1. Exceed critical gradient. Seepage forces at an exit point are greater than the critical gradient and seepage flow is sufficient to remove soil particles. Pipe formation begins.

2. Once a pipe (one of many developing simultaneously) reaches a point half way to the lake, it becomes self-propagating to the source. (Kohno et. al., 1987.)

3. Once a pipe (one of many developing simultaneously) reaches the lake, there are no further restrictions to flow. Flow would be expected to increase to a point exceeding the scour velocity for the material. This would remove sufficient material to lower the crest of the remaining embankment below the lake surface elevation.

Therefore, a pipe (one of many developing simultaneously) need only reach approximately half of the total path length to the source (depending upon material type) in order to self-propagate to the source

8.1.2. Failure by Slope Stability. The process of failure by slope stability leading to breach is generally expected to occur when the slope failure is of such a magnitude that the crest of the embankment is lowered below the lake surface elevation. This was not the case for failures modeled on this project. Probabilities for breach by slope stability were evaluated as a conditional probability where the slope failure occurred resulting in the shortening of the seepage path. The probability of Breach (P_B) is the product of the probability of a significant slope failure (P_{SSF})

multiplied by the probability of a breach by seepage given that a slope stability failure has occurred ($P_{SB|SSF}$).

$$P_B = P_{SSF} \times P_{SB|SSF}$$

For this project, slope failures are expected to be a problem only over an area approximately two miles long known as Line 18 in Reach 1. Preliminary information in Reaches 2 - 8 did not indicate a slope stability problem. However, sufficient data ***has not*** been collected in Reaches 2 - 8 to rule out this failure mode in these areas. The failure process by slope stability will be discussed in detail for Reach 1 line 18.

8.2. Stable / Unstable Materials. Critical gradients vary depending upon material properties. Samples retrieved from the embankment reveal gradations where the fine sand portion can move within the coarse shell or limestone fragment matrix. This phenomenon is known a suffusion (Kovacs 1981). Since the finer particles are under little or no confining stresses, they are free to move. Therefore, these materials have much lower critical gradients than internally stable materials.

8.3. Selection of Critical Gradient. There are two principal components to critical gradient, vertical and horizontal.

8.3.1. Vertical Critical Gradient. Generally there are two conditions under which to define i_{cv} .

8.3.1.1. The first condition is the "normal" condition where there are confining stresses prior to pipe development. The vertical critical gradient, i_{cv} , is most widely discussed and analyzed in standard practice. Almost invariably, i_{cv} is defined as

$$i_{cv} = [\, g_{soil} - g_{water} \,] / g_{water}$$

where g_{soil} is the saturated unit weight of the granular soil and g_{water} is 62.4 pcf. The range of unit weights for granular soils which may be encountered (Bowles, 1984) is from 70 to 150 pcf. Based on this range values of i_{cv} could range from 0.12 to 1.4. Keep in mind this is an extreme range for all possible values. Natural sand deposits range from about 100 to 130 pcf. The range of i_{cv} then becomes 0.60 to 1.08. A synopsis of TM 3-424 (USACE, 1956) of observed conditions on the Mississippi River Levees revealed the following values for exit gradient and relative descriptions in Table H-8.1 .

Table H-8.1

Condition	Vertical Exit Gradient
Sand Boils	0.5 - 0.8
Heavy Seepage	0.4 - 0.7

Medium Seepage	0.2 - 0.6
Light / No seepage	0.02 - 0.5

8.3.1.2. The second condition is where the material is under little or no confining stress. Experiments by Skempton & Brogan (Skempton et. al. 1994) show that for sand under little or no confining stress (under a roof or within an unstable material as defined by Kenney & Lau), values for i_{cv} range from 0.2 to 0.34.

8.3.2. Horizontal Critical Gradient. Horizontal critical gradient, i_{ch} , is defined using various theories or empirical methods. Generally there are two conditions under which to define i_{ch} .

8.3.2.1. The first condition is the "normal" condition where there are confining stresses prior to pipe development. Several theories exist, but most either consider the friction angle of the material or the angle of flow to the slope exit face. The WES report eqn. 11 (USACE, 1997) covers this case.

$$i_{ch} = i_{cv} (\cos b \tan q - \sin b)$$

Where q is defined as the angle of internal friction of the material and b is defined as the angle of seepage flow at the exit face.

8.3.2.2. The second condition is where the material is under little or no confining stress. Experiments by Skempton & Brogan (Skempton et. al.) show that for sand under little or no confining stress (under a roof or within an unstable material as defined by Kenney & Lau), i_{ch} for sandy gravels is about 0.16. Flume tests by Schmertman (Schmertmann, 1997) & Townsend (Townsend et. al., 1988) and Sellmeijer (Sellmeijer, 1988) for horizontal piping on materials with a C_U of about 1.5 reveals i_{ch} values of about 0.2. Since the pipes form directly under the flume cover, there is very little confining stress. The combination of these works reveals the following:

1) For unstable materials, the finer portion of the material is free to move under little or no confining stress. The material which pipes has a low C_U.

2) Uniform (stable) materials with low C_U are susceptible to piping at i_{ch} lower than that for materials under confining stresses.

8.4. Piping To Breach. Piping begins when seepage forces at an exit point are greater than the critical gradient and seepage flow is sufficient to remove soil particles. While this is unacceptable from a design standpoint, it does not necessarily mean that a breach is imminent. It does indicate that the forces required for a breach to begin are present. The engineering community has recognized for a long time that the processes involved in a breach are many and complex. Design criteria by Bligh (Bligh, 1910) or Lane (Lane, 1935) using creep ratio, or by factor of safety of critical gradient divided by exit gradient. It is interesting to note that the range of factor of safety in EM 1110-2-1901 is from 1.5 to 15, with an implied recommendation from 2.5 - 5. Were this a design report, we would be using factors of safety on the high end of this range (F.S. = 5) due to the

critical nature of this project. Since this MRER is a document to make an economic decision, not meeting factor of safety criteria is not a valid failure in this MRER process. While we may strongly disagree with this concept, for the purposed of developing damage dollars only a breach is a valid failure mode (as opposed to failing to meet a recommended factor of safety per established official Corps guidance).

8.4.4. Critical Pipe length. Per prior discussion, breach is obtained by one of many simultaneously developing pipes reaching the lake. In order for a pipe to reach the lake, it must reach the half way point from its beginning to the lake. Experiments by Koenders and Sellmeijer (Koenders et. al. 1992) and Khono, Nishigaki and Takeshita (Khono 1987) both show that as the pipe advances to this half way point, the seepage force is at its lowest. If piping is able to advance any further into the embankment, the seepage force increases and the pipe becomes self-propagating without increasing the average head. This can be illustrated by Figure H-8.1 below:

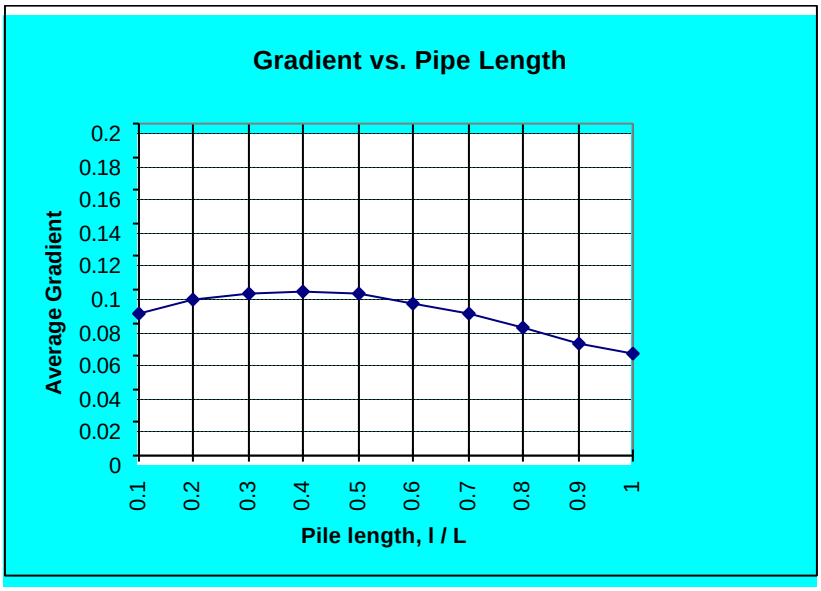


Figure H-8.1. Average Gradient versus pipe length. (From Sellmeijer, 1988).

Where : H = Total head (lake elevation - tailwater elevation)
L = Total seepage length (seepage exit to lake)
l = length of developing pipe

What this figure shows is for a material type (in this case a granular material with L/1000d = 400) the beginning of piping occurs when the average gradient exceeds 0.08. This is observed by the presence of a “sand boil” and subsequent removal of embankment or foundation material. An increase in gradient is necessary for piping to continue (l/L increasing). Once l/L reaches 40 - 50% of the total required pipe length at a gradient of 0.1, no further increase in gradient is necessary. In fact, a sharp reduction in gradient (from 0.1 to 0.06) would be required to stop pipe progression. This explains some observations made at the project during the high water events. First, when piping

begins, it does not necessarily mean a breach will occur. Sometimes piping begins and stops(stabilizes) at a given lake elevation. Also - when pipe development is on the left side of the curve (l/L < 40%) piping can be stopped by a slight decrease in head (ie. by sandbagging the boil). This also indicates for pipes on the right (l/L > 40%) of the curve, remedial efforts would be much more difficult. While the value of remedial efforts cannot be completely discounted, we felt that once multiple pipes over large areas (miles of embankment) were on the far side of the curve, "over the hump", that available resources (SFOO, SFWMD) would be overwhelmed and a breach would be inevitable. At this point it should also be mentioned that a pipe does not necessarily have to remove a large volume of material to connect to the lake. For example, a 1 inch diameter pipe 300 feet long contains a volume of about 1.5 cubic feet. This volume of soil would be about one wheelbarrow full. This volume could be visible, such as exiting on the embankment or ditch slope. It can also occur undetected, such as on a ditch slope under water, within existing solution channels in the limestone (for this we have evidence on borehole video footage) or into ungrouted vertical holes (i.e., old ungrouted core boring holes, utility pole excavations not properly backfilled, old abandoned wells).

8.5 Slope Stability Analysis. Models were analyzed using the UTEXAS3 computer software package. Spencers method was used to examine potential failure surfaces. Failure surfaces were examined that went through a major portion of the embankment.

8.5.1. Basic Approach. In order to perform a probabilistic analysis, multiple runs of the same model would be required where variables could be changed one at a time and their impact on the model computed as total variance. In order to keep the number of runs manageable, the global critical failure surface was located using the same procedure as for a deterministic analysis (multiple start points, varying radius and tangent elevation of the failure arc). Once this surface and starting point was located, this was the basis for the probabilistic analysis. Some have proposed keeping the failure arc constant and using the single arc calculation mode while varying material properties. However, when material properties are changed this changes the location of the critical failure arc slightly. The procedure we selected was to initiate a search with the same starting point and tangent elevation while varying each material variable. Although more time consuming, this ensured evaluating the critical arc for each run.

8.5.2. Pore Pressure Distributions. Ideally, transferring pore pressure values from FASESEEP runs would be the best method of analysis. However, this was attempted and proved to be impractical due to the complexity of the models and software limitations. Piezometric lines were selected to represent pore pressure distributions. Models could be represented by three piezometric lines. One line representing the phreatic surface above the confining layer, one line representing pore pressure distributions within the confining layer, and one line representing pore pressures below the confining layer. Values to define these lines were obtained from the expected value runs from FASTSEEP. There is little variation in pore pressures over large distances (100 feet) between the various fast seep runs. There is significant variation in the vicinity of the ditch, but a failure arc here would not be critical to breach. The larger arcs deeper into the embankment did not experience significant fluctuations in piezometric surfaces due to variations in permeability.

Piezometric lines for each zone, developed from the expected value run at that particular lake level, were used to define pore pressure distributions.

8.5.3. Failure Modes. The failure mode envisioned for slope stability would be as follows. Since no failure surfaces were expected to be of a magnate which would lower the crest below the lake level, a two part failure would be more likely. A significant slope failure would intersect a piping path. This would shorten the path due to disruption and / or dilation of the soil along the lateral portion of the shear surface. From that point, the failure to breach would progress as either a through or under seepage failure. This could be described as follows:

$$P_{B\ SS} = P_{SS} \ \text{x}\ P_{SS\ |\ S}$$

where

$P_{B\ SS}$ = Probability of Breach due to Slope Stability failure

P_{SS} = Probability of Slope Stability Failure

$P_{SS\ |\ S}$ = Probability of breach due to seepage failure given a slope stability failure has occurred.

8.5.4. Overall Assessment. The first slope stability analyses were more involved than subsequent analyses. It became evident early in the analysis process that slope stability was not going to be a major factor. With the exception of Line 18, all other models showed a very low Pf at all lake levels. The order of modeling shows the greater level of detail early on to completely understand the problem. Subsequently analyses and models show a much reduced level of detail in order to shift resources to the seepage evaluation.

8.6 Seepage Analysis. The analysis method chosen was the finite element method (FEM) software package FASTSEEP. This was necessary due to the complexity of the geometry and geology. Had simpler sections been possible, Levee MSU could have been used.

8.6.1. Basic Approach. It should be noted that the order of modeling had a bearing on the approach. The first models (and even analyses) evaluated were done to a greater level of detail than subsequent models or analyses. This was due to several factors, the most significant being that the first evaluations were scrutinized so that a clear understanding of the system behavior could be understood. By understanding what was important to the systems, less significant aspects could be omitted. This is evident in the first seepage analyses were usually done at a lake level of around elevation 18.0. This was done to calibrate to known conditions. In these Taylor series computations all variables were used. In subsequent computations, only the variables contributing significantly to the variance were used.

8.6.2. Failure Modes. The two failure modes examined were through seepage and under seepage. For the purposes of this report, through seepage is defined as seepage above the silt

/ clay confingin layer, regardless of whether seepage is passing through the embankment or foundation. Under seepage is defined as seepage in the foundation below the confining layer.

8.7 Taylor Series Methods. Consultation with Dr. Thomas Wolff n the early stages of the project resulted in selecting the Taylor Series Approach instead of the Point Estimate Method. One advantage of using Taylor Series Approach was that variables could be readily identified which contributed greatly to the problem.

8.7.1. Series and Parallel Systems. In simple terms, the main difference between anseries and parallel system is redundancy. A parallel (redundant) system will fail only when all of its components fail (Harr 1987). In contrast, a series system, like a chain, will fail when only one of its components (links) fail. For simple series systems, all components are considered independent (Wolff, 1995).

8.7.2. Taylor Series Computation. The spreadsheets used were developed by Wolff (Wolff, 1996) and modified to accommodate variables particular to each model. The basic premise of the spreadsheet is a stair step arrangement of the variables in columns and runs in rows. Each row represents a unique combination of variables. Factor of Safety was either computed from exit gradient or by UTEXAS. A probability distribution (normal) of Factor of Safety is then developed based on the summation of variance from the individual runs. From this, a lognormal distribution of Factor of Safety is developed and a Beta value computed. Probability of Failure is computed from the Beta value, which is assumed to be normally distributed.

8.7.3. Combinatorial Reliability. There were four areas where combinatorial reliability was applied. These were independent failure methods within a failure mode, dependent failure methods within a failure mode, combining independent failure modes and combining independent reaches or models.

8.7.3.1. Independent failure methods within a failure mode. For example, this decision is where there are multiple seepage paths for a failure mode. These paths are independent and are treated as a simplified series system (EC 1110-2-554, USACE, 1998). The materials the pipes form in is assumed to have perfectly correlated permeabilities and gradients. This provides one “set” of results for a given lake level. The performance function is such that failure mode occurs if any of the failure methods occur. The probability of failure for the system is taken as the method with the greatest probability of failure.

8.7.3.1.1. Dependent failure methods within a failure mode. This situation was where a two or more step failure process was necessary for breach. For example, for the slope stability mode requires a slope stability failure then a seepage failure to complete the breach.

8.7.3.1.2. Combining independent failure modes. This is a combination of series system probabilities at a given lake elevation. This is the method used to combine the

independent failure modes and develop the combined probability of breach function for each model. The basic equation is:

$$P_{Breach} = 1 - [(1-P_{Through\ Seepage})(1-P_{Under\ Seepage})(1-P_{Slope\ Stability})]$$

8.7.3.1.3. Combining independent reaches or models. Again, this is a combination of series system probabilities at a given lake elevation. This is the method used to combine the independent breach functions for each model and develop a combined probability of breach function for the project. The basic equation is:

$$P_{breach\ Project} = 1 - [(1-P_{breach\ Model\ \#1})(1-P_{breach\ Model\ \#2})...(1-P_{breach\ Model\ \#n})]$$

8.8 Length Effects. The Herbert Hoover Dike embankment is a very long, linear structure. This creates a special concern for which the basic concept is as follows. If one were to construct a chain of a single link with a probability of failure under a specific load of $P_{f\ link}$, the probability of failure of the chain would be $P_{f\ chain} = P_{f\ link}$. If you wanted to construct a longer chain you would naturally need more links. So, if a chain of 10 identical links was constructed with each link having a $P_{f\ link}$ = 10%, the probability of failure of the chain would be greater than 10%. The probability of failure of the chain would be the probability of failure of a series system which would be represented as follows:

$$P_{f\ chain} = 1 - [(1 - P_{f\ link\ 1})(1 - P_{f\ link\ 2}) \ldots (1 - P_{f\ link\ 10})]$$

For this example, the $P_{f\ chain}$ with 10 links each with a $P_{f\ link}$ = 10% would be 65%. Note that this is a considerable increase over the 10% per link. Also note that for small values of $P_{f\ link}$ the equation approaches the sum of $P_{f\ link}$ values. Refer to Figure H-8.2 for a graphical representation.

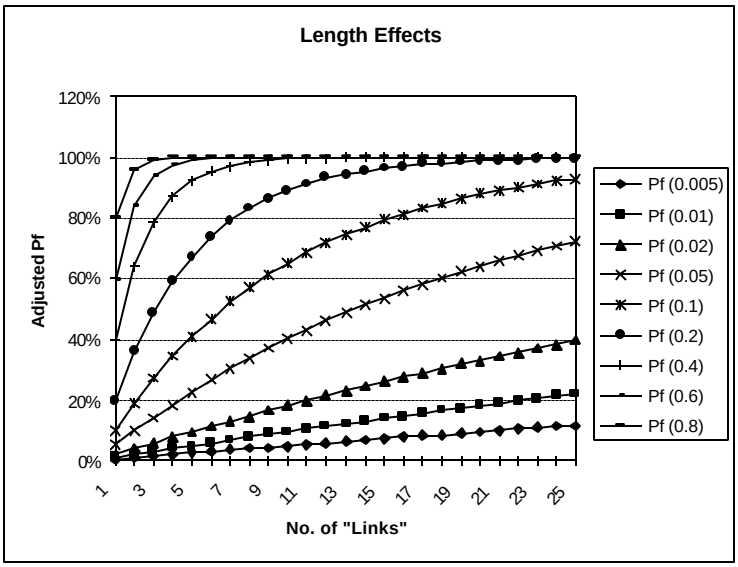


Figure H-8.2

These concepts developed by Vanmarcke and Wolff (Wolff, 1996) were applied to the Herbert Hoover Dike embankment. The fundamental considerations were as follows:

a. Length would have an effect on the final result since the models represented areas of several thousand feet to several miles in length.

b. Some method of incorporating this negative effect, or "penalty" should be used.

c. The final results should be reasonable.

The greatest practical difficulty in applying the concept of a length effect penalty is determining the length of the individual links (unit length of statistically homogeneous embankment). Vanmarcke (Vanmarcke, 1977) and others have proposed estimating a "correlation distance". For link lengths up to this correlation distance the soil properties can be considered highly correlated and statistically homogeneous. Therefore, as illustrated by the example above, the longer this statistically homogeneous embankment is, the greater the number of unit lengths, or "links" in the embankment. Although this may seem to be a new concept, geotechnical engineers routinely select a length of embankment or foundation with similar properties and assign one two dimensional model to represent this entire length. What the length effect penalty is doing in this case is attempting to quantify the increased risk due to the "great length" of the embankment represented by a single model.

8.9. Model For Line 6. This model is designated as "line 6" since it is taken at survey line CB-6, representing the sixth line of core borings taken on 3000' intervals north of S-351. Therefore, this section is approximately 18,000 feet north of S-351 along the dike alignment. This is part of the original levee embankment designated as LD-2 near station 1020+00. As-builts dated August, 1933 show the construction of the original federal project as having a 5 foot crest at elevation 36.0 feet, a 1V:4.5H lakeside slope and a 1V:3H landside slope. Construction of a seepage toe and fire toe through the "muck" was also indicated. One of the most apparent features at this section is the large limestone quarry immediately downstream. This is an active quarry today, and even appeared on surveys in 1962 showed excavation down to elevation -12 feet. When the embankment was raised to its present elevation of 38.0 feet in 1962, the downstream slope was shown to have a compound 1V:5H upper slope and 1V:3H lower slope. The quarry was shown as being partially filled adjacent to the embankment toe with a 1V:3H slope under water. Surveys taken in 1995 show that the compound slope is actually 1V:2H on the lower portion, and the slope under water to be 1V:1.25H. We suspect this 1V:1.25H (38°) slope corresponds to the angle of repose of the slope material. Total volume of material removed is estimated from the 1995 survey to be 350,000 cubic yards. Quarry operations have been observed since 1995, so the current volume removed would be expected to be greater than 350,000 cubic yards.

8.9.1. Selection of Variables and Values. Prior to developing a geologic section for this model, the core samples recovered were examined and compared to the core logs. Significant differences in the embankment materials as compared to core logs were that from elevation 16.0 to 23.0 feet the material was much coarser than would be expected from the logs. Also - a layer of tiny (about 1.5 mm) uniform rounded shells similar to small ball bearings. This layer was continuous under the embankment from elevation -1.0 to -3.0. What was especially significant was this layer was very wet, indicating it was a highly permeable layer.

8.9.1.1. Topographic Features. Topographic features significant to this model were the downstream quarry, seepage toe and fire toe. These can be seen on Figure H-8.3. The quarry was the most dominant feature since it exposed the downstream toe of the embankment. The seepage and fire toes had an impact on the gradients within the embankment, but were not significant contributing factors to breach in this particular model.

8.9.1.2. Geologic Features. Three geologic features were selected as being significant to this model. The filtercake was significant in that it serves as an "entrance valve" into the limestone and sands below the confining layer. Even though the permeability of the material is fairly uniform, gaps in the coverage and layer thickness can account for variations in the permeability of the unit itself. Therefore, the permeability of this material was adjusted to assist in calibrating the model to the piezometer readings. The gravel pocket in the embankment was established as a variable. Borings through the embankment indicated the presence of coarse material. The extent of the pocket was assumed to be a maximum of the width of the original embankment. The geometry of the pocket was included as a variable. Refer to Table H-8.2. The other significant feature were the actual "pipes". The presence and extent of the pipes was inferred from piezometer readings.

8.9.1.3. Slope Stability Variables. Variables and values selected for slope stability were divided into two categories, those with normal distributions and those with lognormal distributions.

Table H-8.2. Normally Distributed variables.

	Shear Strength Summary for Line 6				
		All values in degrees.			
	q	q	Normal Parameters		
Material	Min / Max	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb, R		23.0	2.30	25.3	20.7
Emb, S		33.0	3.30	36.3	29.7
PT/OL/OH, R		18.0	4.50	22.5	13.5
PT/OL/OH, S		33.0	8.25	41.3	24.8
CL / ML, R		12.5	3.10	15.6	9.4
CL / ML, S		33.0	8.30	41.3	24.7

	Material Unit Weights - Saturated			
Material	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb.	115.0	5.75	120.8	109.3
PT/OL/OH	70.0	3.50	73.5	66.5
CL/CH/ML/MH	100.0	5.00	105.0	95.0

Table H-8.3. Lognormally Distributed variables.

	Shear Strength Summary for Line 6			
	All values in degrees or psf.			
		Lognormal Parameters		
Material	C	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Emb, R	740	523	1203	228
Emb, S		no variation, not a variable		
PT/OL/OH, R	440	311	715	135
PT/OL/OH, S	240	170	390	74

CL / ML, R	700	623	1010	383
CL / ML, S	200	141	325	62

8.9.1.3. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability were lognormally distributed.

	Permeability Summary for Line 6				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Sands	2.00E-03	2.00E-03	1.41E-03	3.25E-03	6.15E-04
Gravel	4.00E-01	4.00E-01	2.83E-01	6.50E-01	1.23E-01
Peat	2.00E-04	n/a	1.60E-04	3.12E-04	8.20E-05
	n/a	6.00E-05	4.61E-05	9.53E-05	2.23E-05
Silts & Clays	1.50E-04	n/a	1.27E-04	2.27E-04	7.09E-05
	n/a	1.10E-04	7.11E-05	1.81E-04	2.79E-05
Rock	2.50E-01	n/a	1.77E-01	4.06E-01	7.69E-02
	n/a	2.00E-02	1.41E-02	3.25E-02	6.15E-03
Filtercake	8.50E-05	8.50E-05	6.01E-05	1.38E-04	2.61E-05
Pipe	5.00E+00	5.00E+00	3.54E+00	8.13E+00	1.54E+00

The spatial distribution of the gravel pocked was based both on core boring data and as-built data. The top and bottom elevations of the pocked were estimated from core borings and the width based on as-built topographic data. Table H-8.4 shows the normal distribution assumed for the width of the pocket.

Table H-8.4

Gravel Distribution, Width of Gravel Pocket

Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
145.0	85/205	20.0	165.0	125.0

Transformed to Model Coord.s				
	Center	Lakeside	Landside	Total
Mean [E(x)]	430	-90	55	145
High [E(x) +s]	430	-113	78	165.0
Low [E(x) +s]	430	-67	33	125.0

8.9.2 Selection of Tailwater Distribution. In this case the quarry lake dominated the tailwater scheme. At least one outflow culvert was identified. It is known that the quarry operators can pump to keep water levels low for quarry operations or other reasons. This was done during the High Water Event of 1998. It is also known from topography that the highest practical water level in the quarry lake is elevation 11.0. Above elevation 11.0, surveys show mobile homes in the area would be flooded.

8.9.2.1. Tailwater Distributions for Through Seepage and Slope Stability Analyses. Distributions for these analyses are shown in Table H-8.5. This is a relatively wide range of values for this system based upon probable minimum and maximum values as well as observed water levels and levels inferred from piezometer readings. For these two failure modes, a higher tailwater would be more critical since the corresponding phreatic surface in the upper saturated zone would be higher. This range of values was assumed to be normally distributed.

8.9.2.2. Tailwater Distribution for Under Seepage Analyses. Unlike the through seepage or slope stability failure modes, underseepage was more critical with lower tailwater elevations. Since the quarry lake could be artificially manipulated, a water level close to "normal" was selected for all analyses. Based on all existing data, elevation 8.0 was selected as the "normal" water elevation for the quarry lake. Even though other tailwater levels were evaluated, this scenario (artificial manipulation) was selected as critical since the Corps does not control tailwater elevations.

Table H-8.5.

	Tailwater Elevations for Model at Line 6				
Lake El.	Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
15	7.0	6.0 - 8.0	0.33	7.3	6.7
18	8.0	7.0 - 9.0	0.33	8.3	7.7
21	9.0	8.0 - 10.0	0.33	9.3	8.7
24	10.0	9.0 - 11.0	0.33	10.3	9.7

8.9.3. Selection of Critical Gradients.

8.9.3.1. Piping above the confining layer. For the upper sands i_{ch} was computed as follows using the equation in paragraph 8.3.2.1.

	i_{cv}	i_{ch}(f only)	i_{ch}(f and b)
High values			
g_{soil} = 121 pcf	0.939		
f = 36°		0.682	
b = 0°			0.682
Low values			
g_{soil} = 109 pcf	0.747		
f = 30°		0.431	
b = 10°			0.294

With the range of i_{ch} from 0.682 to 0.294, an average value of 0.488 was selected. Dividing the range from 0.682 to 0.294 by six to obtain three standard deviations on either side of the mean, a standard deviation of s = 0.065 was applied. This gives the following:

Average Value:	0.488
Std. Dev.:	0.065
High Value:	0.533
Low Value:	0.423

This is the critical gradient to start piping. For breach, the gradient has to increase enough to drive the pipe past the half way point, l/L > 40 - 50 %, where l is pipe length and L is total pipe path length. In order to do this, the gradient required for breach is approximately 15% greater than that required to start piping. Refer to Figure H-8.4. The following values for i_{ch} were used in the analysis for through seepage.

Average Value:	0.561
Std. Dev.:	0.075
High Value:	0.636
Low Value:	0.466

8.9.3.1. Piping below the confining layer. The location of the pipes is expected to be directly under the
limestone layers. This would mean the material is under little or no confining stress.

A wide range of values for i_{cv} was considered, from 0.1 to 0.5. This was due the wide range of gradations expected, fluctuations in confining stress conditions and variations in unit weight. o The following values for i_{cv} were used in the analysis for through seepage.

Average Value:	0.300
Std. Dev.:	0.067
High Value:	0.367
Low Value:	0.233

For i_{ch} , the range of values for considered was from 0.15 to 0.30. This was based on the .16 encountered by (Skempton et. al., 1994) and Schmertmann (Schmertmann, 1997) and the 0.3 by Schmertmann (Schmertmann, 1997) for a material with a Cu of about 1.5. The following values for i_{ch} were used in the analysis for through seepage.

Average Value:	0.225
Std. Dev.:	0.025
High Value:	0.250
Low Value:	0.200

For this model, piezometer data indicated pipe formation was near the half way point. Therefore, it would not be appropriate to apply the 15% to the critical gradient computed above. This value is the critical gradient for breach.

8.9.4. Failure Mechanisms. There were two primary failure mechanisms considered for seepage and one for slope stability.

8.9.4.1. Through seepage. Below a lake elevation of 21, the models indicated seepage did not exit the embankment slope. Contour lines indicated the limestone at the peat interface would not be a significant factor. Therefore, the only through seepage failure was expected when seepage exited the slope face. This is expected to be a straight pipe starting in the lower third of the embankment on the downstream slope face and propagating through "stable" sands under normal confining stresses. The critical gradient for breach was selected as 15% greater than that to start piping in order to drive the pipe to the half way point. This implicitly assumes the pipe will propagate to completion (greater than l/L = 50%) in a stable material. In all likelihood, the pipe would seek the path of least resistance and propagate to a part of the gravel pocket where the critical gradient would be lower. This would require an evaluation similar to the two part evaluation at the Line 25 model. Limited resources did not permit this evaluation. Realize however, that the probability of breach would be higher than currently shown. This is due to a) the critical gradient would need to be increased only slightly (perhaps 5% instead of 15%) to have the pipe progress the l/L = 20% to get to the gravel pocket and b) the critical gradient in the unstable gravel is much less, generally about half that in the stable sand.

8.9.4.2. Under seepage. The massive amount of excavation at the toe of the embankment made this the failure mode of most concern. Piping is defined as the removal of embankment or foundation material. Quarry operations were very successful at removing large quantities of foundation materials. In addition to the absence of much of the foundation, the quarry operation was deep enough to pass through the silt / clay confining layer and the upper limestone layer. This provided two "roof" layers for pipe formation and direct access for material to pipe into the quarry lake undected. Evidence for existing "pipes" comes from the piezometer data. All four piezometers read within 1 foot of each other and the quarry lake at various lake elevations. Also, the "B" piezometers under the crest read slightly higher than the "C" piezometers at the toe, indicating gradient towards the quarry lake. The data indicate a very good (low restriction) connection to the quarry lake. A slight separation or pipe formed directly under each roof would provide such a connection. Note that the upper "B" piezometer tip is located in the small rounded shell layer (wet "ball bearings") directly under the confining layer. Of great concern is that data indicates these pipes are at or near the crest of the embankment. This is why the 15% was not added to the critical gradient required for breach due to underseepage. Pipes progressing past the halfway point are self propagating to the source (Lake Okeechobee).

8.9.4.3. Slope Stability. The two part slope stability failure leading to breach was expected for this section. Since the probabilities of breach for this mode were two orders of magnitude less than for seepage failure modes, the seepage portion was omitted.

8.9.5. Model Calibration and Analysis Results.

8.9.5.1. Model Calibration for Seepage Analyses. Once the appropriate features were selected, constant head boundary conditions could be applied. A lake level of 18.5 was selected since data from the 1995 High Water Event was available. No visible evidence of seepage was observed. However, any excessive seepage or piping occurring under the surface of the quarry lake would likely go undetected until a partial collapse of the berm occurred. The model was calibrated to piezometer readings using expected values, not log of expected values. The filtercake material was also used to "calibrate" as an entrance valve so head under the confining layer agreed with the actual piezometer readings. The absence of filtercake drastically alters the head distribution below the confining layer. Pipe lengths were also adjusted to obtain agreement with piezometer readings. Although one could argue for calibration by adjusting the "entrance valve", this would imply a no flow condition. Piezometer readings differ slightly, but all indicate a gradient from the lake to the quarry. In the 1998 High Water Event pumping operations in the quarry stopped when the excavation was complete. Water levels were observed to rise approximately two feet in less than one week with no rainfall during this period. This would confirm that there is indeed flow through the foundation into the quarry lake.

8.9.5.2. Model Calibration for Slope Stability Analyses. The search for the global critical failure arc consisted of examining surfaces tangent to the top of the rock layers and clay layer as well as varying tailwater elevations. The results are shown in Tables H-8.6 and H-8.7. (Note: Models were not completely calibrated; lower sand phi = 30 not 33 and no gravel layer.)

Table H-8.6

For TW = 8

Arc #	X	Y	R	FS	Note
1	527.4	132.2	138.9	1.762	Tan. to top of 1st rock layer
2	527.4	132.2	138.9	1.762	Tan. to top of 1st rock layer
3	527.2	131.0	137.8	1.757	Tan. to top of 1st rock layer
4	551.8	166.8	181.5	2.009	Tan. to top of 2nd rock layer

Table H-8.7

For TW = 10

Arc #	X	Y	R	FS	Note
1	507.4	53.2	49.5	2.022	Tan. to top of clay layer
2	527.4	132.2	139.1	1.730	Tan. to top of 1st rock layer
3	527.2	131.0	137.8	1.725	Tan. to top of 1st rock layer
4	551.4	167.4	182.1	2.002	Tan. to top of 2nd rock layer

8.9.6. Length Effects. For this model a link length of 1500 feet was selected. A shorter than usual length was used due to the unique topographic feature of the quarry excavation.

8.9.7. Pup Curve Development. Table H-8.8 is a summary of the analyses performed on this model. Relevant information for each analysis was listed to show the progression of values with lake level.

8.9.7.1. Through Seepage. This failure mode was not a factor for lake elevations less than 21.0 since seepage did not exit the slope. This mode was difficult to evaluate since seepage would exit the slope for some variable combinations and not for other combinations. This required a slightly different approach for the Taylor Series computation. Probability distributions were developed for both exit gradient and critical gradient based on computed variance for each since the single probability distribution for Factor of Safety could not accept a “zero” value. The exit and critical gradient distributions were combined to develop a third distribution for Factor of Safety. From this point, computations proceeded as described for other Taylor Series calculations. Once seepage exited the slope, the horizontal exit gradient was compared to the horizontal critical gradient. The resulting probabilities are shown in Table H-8.9.

8.9.7.1. Under Seepage. For lake elevations of 18 and 21, underseepage was the predominant failure mode. The upper pipe and lower pipe in the model were examined as locations most critical to piping. The upper pipe was found to be more critical and horizontal piping the predominant mode. Since piping for both pipes occurs in the same material (sand), the

permeability values for the sands are perfectly correlated. Gradients are assumed to be perfectly correlated, so the probability of failure for this mode is taken to be the minimum of the four performance functions evaluated (upper Pipe, horizontal and vertical gradients and lower pipe, horizontal and vertical gradients).

It is interesting to note that the models at 15 run first with tailwater = 7.0. Probabilities of failure were found to be unreasonably high. This failure mode was the one most sensitive to tailwater. As discussed above, a tailwater value of 8.0 was selected for all the analyses in this mode.

8.9.7.1. Slope Stability. Due to the low probability values, only lake elevations of 18 and 24 were analyzed. The probability value for lake elevation 21 was interpolated. The values were two to three orders of magnitude lower than the probability values for seepage modes. Therefore, a simplifying assumption was made that the probability of a seepage failure given a slope stability failure was 1.0. The actual value would be between 0.43 and 1.0 for a lake elevation of 21 and 0.47 and 1.0 for a lake elevation of 24. The slope stability failure probabilities indicated are actually slightly higher than if the combined probability were used. The difference is inconsequential to the final results.

Table H-8.8

Lake El.	Failure Mode	E(FS)	Beta	Sum Variance	P(Failure)	P(Failure) w/ length
15	LP iv	6.295	2.296	24.6987	0.0109	0.0251
15	LP ih	11.764	4.440	43.7834	4.51e-6	1.05e-5
15	UP iv	5.269	1.929	19.2020	0.0268	0.0615
15	UP ih	4.037	1.505	12.0288	0.0662	0.1480
	TW = 7					
15 *	UP ih	4.819	1.987	13.4672	0.0235	0.0539
	TW = 8					
18	LP iv	3.550	1.501	7.5968	0.0667	0.1490
18	LP ih	7.519	2.655	32.5296	3.96e-3	9.22e-3
18	UP iv	3.528	1.670	6.1658	0.0474	0.1070
18 *	UP ih	2.322	1.040	2.5591	0.1490	0.3140
	TW = 8					
21	LP iv	3.158	1.268	6.5108	0.1020	0.2230
21	LP ih	6.250	2.080	29.1813	0.0188	0.0432
21	UP iv	3.191	1.563	4.7678	0.0590	0.1320
21	UP ih	2.064	0.989	1.6295	0.1610	0.3370
	TW = 9					
21 *	UP ih	1.867	0.787	1.3920	0.2160	0.4330
	TW = 8					
21 *	EMB ih	1.369	0.793	0.2128	0.2140	0.4300
	TW = 9					
24	UP ih	1.974	0.855	1.6310	0.1960	0.4000
	TW = 10					
24 *	UP ih	1.512	0.428	0.9036	0.3340	0.6130
	TW = 8					
24 *	EMB ih	1.285	0.705	0.1495	0.2410	0.4740
	TW = 10					
			Slope	Stability	Runs	
18 *	Tan	1.741	3.320	0.0816	4.51e-4	1.05e-3
	el -7					
21 *	Tan				Interpolated	2.13e-3
	el -7					
24 *	Tan	1.682	2.974	0.0829	1.47e-3	3.43e-3
	el -7					

Definitions:

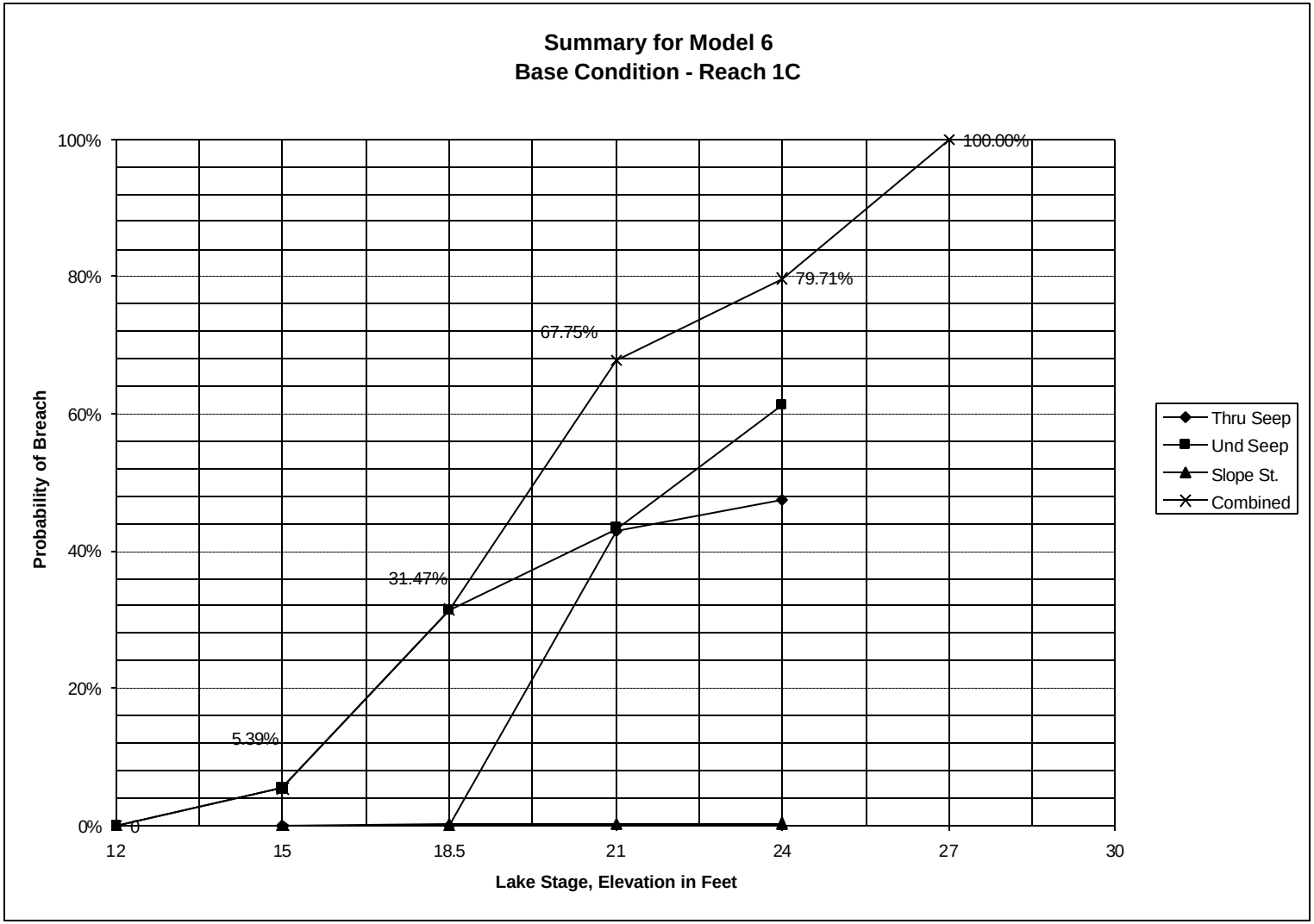
* = Value used in combined probability computation.

LP = Lower Pipe UP = Upper Pipe

iv = vertical exit gradient ih = horizontal exit gradient

8.9.8. Probabilistic Analyses Computation Sheets and Sections.

L6TASUM.XLS


Summary for Model 6
Base Condition - Reach 1C
100%
80%
60%
40%
20%
0%
Probability of Breach
0
5.39%
31.47%
67.75%
79.71%
100.00%
12
15
18.5
21
24
27
30
Lake Stage, Elevation in Feet
Thru Seep
Und Seep
Slope St.
Combined

LEGEND FOR COLOR CROSS SECTIONS

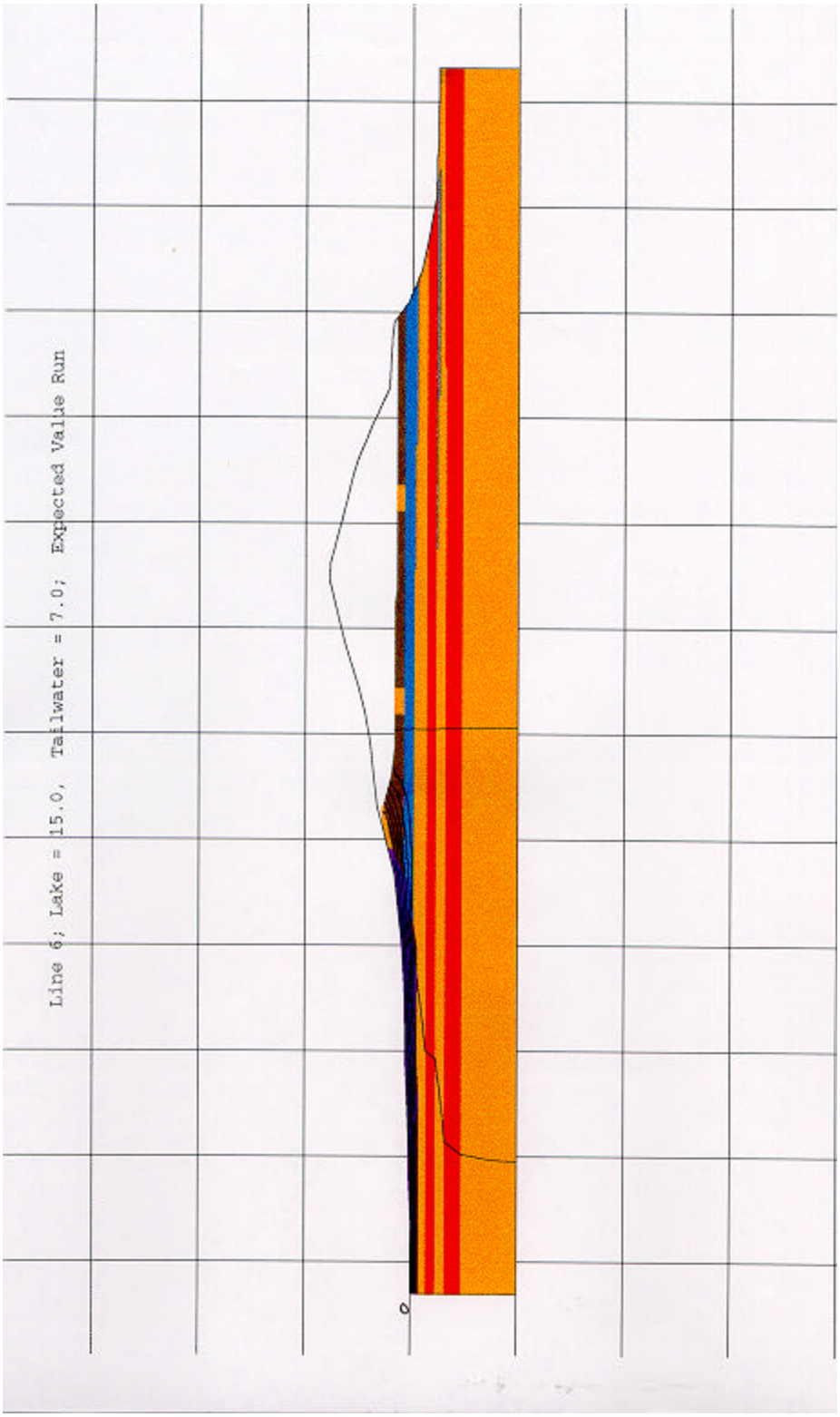
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

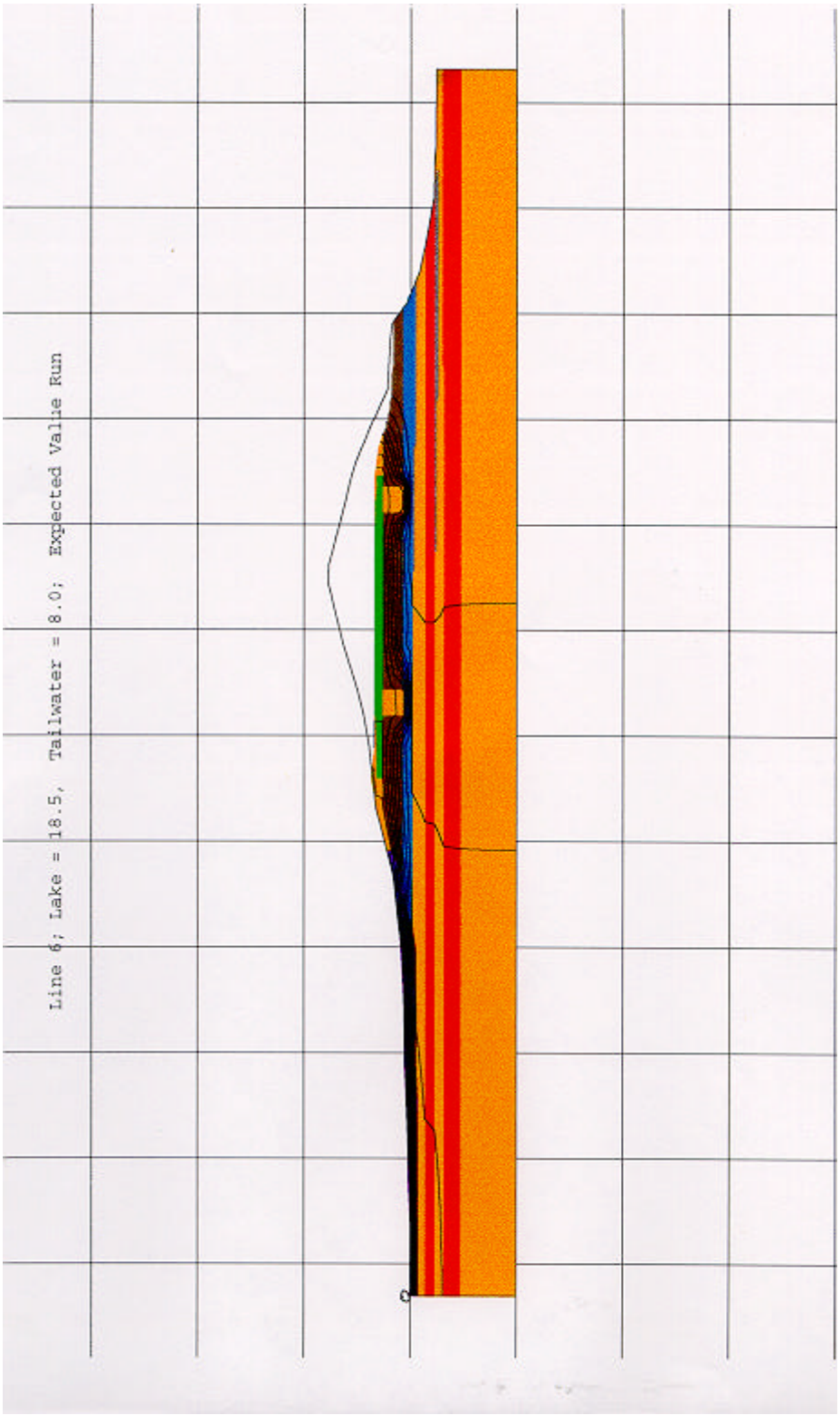
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

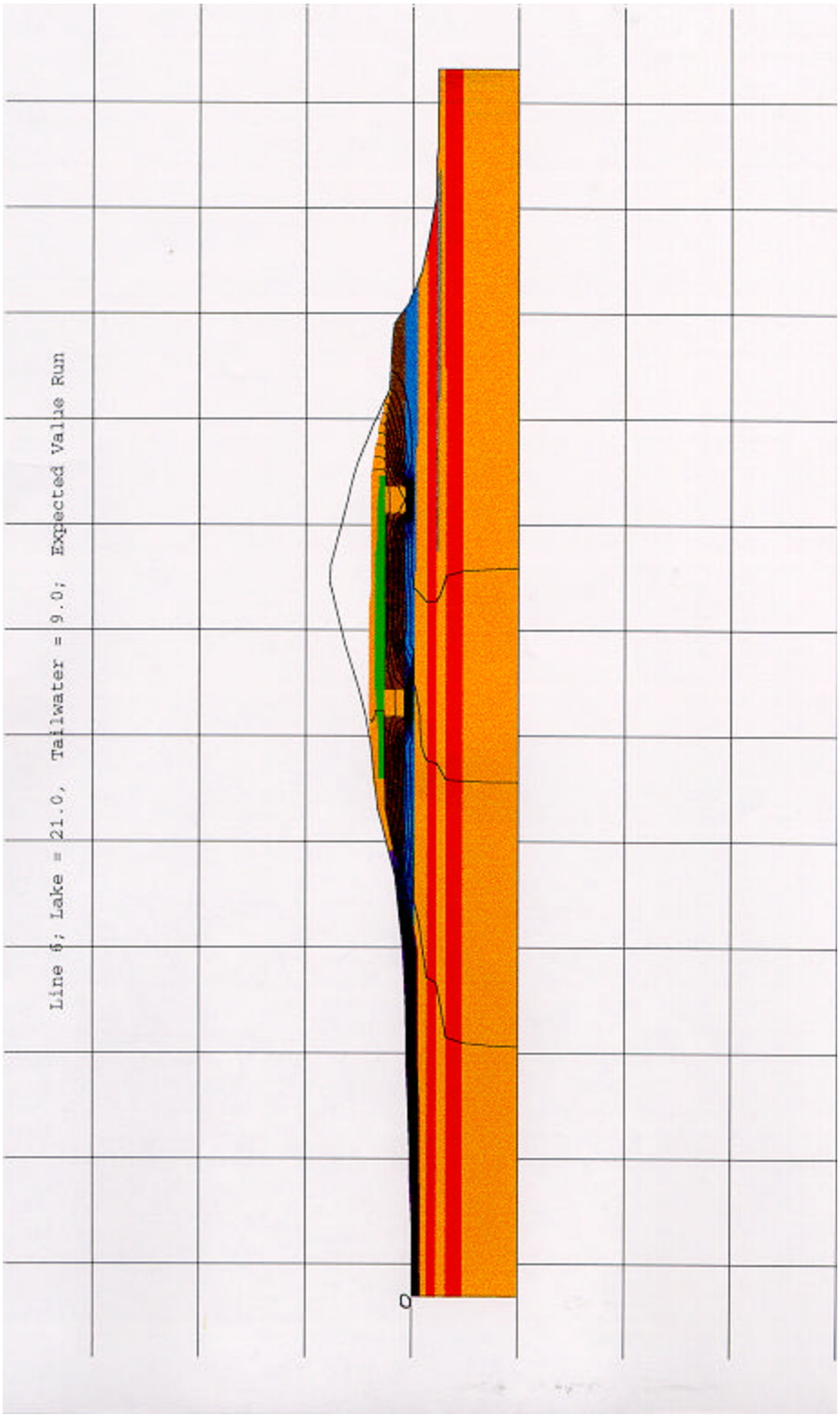
Line 6; Lake = 15.0, Tailwater = 7.0; Expected Value Run


Line 6; Lake = 15.0, Tailwater = 7.0; Expected Value Run
0

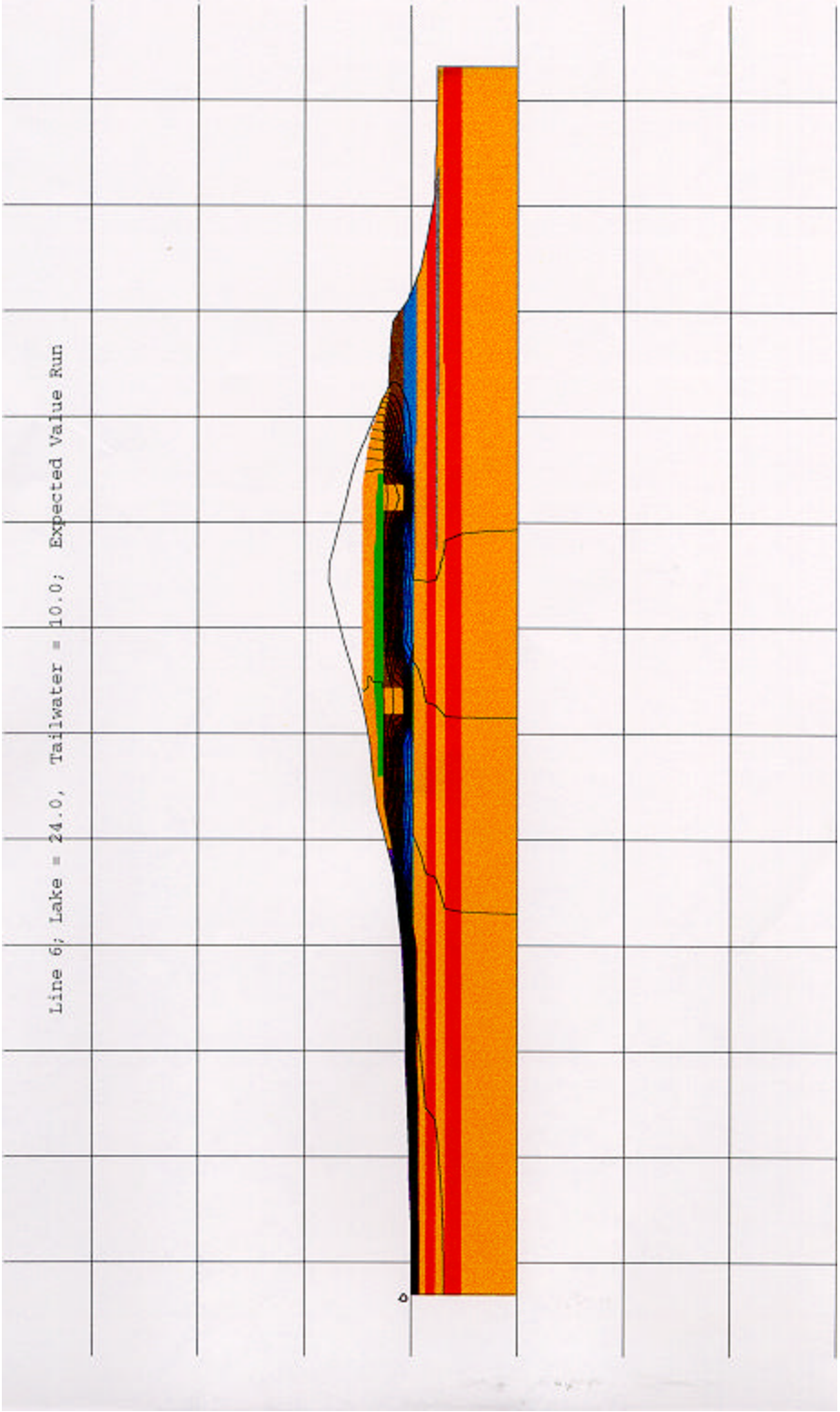
Line 6; Lake = 18.5, Tailwater = 8.0; Expected Value Run



Line 6; Lake = 21.0, Tailwater = 9.0; Expected Value Run



Line 6; Lake = 24.0; Tailwater = 10.0; Expected Value Run


Line 6; Lake = 24.0, Tailwater = 10.0; Expected Value Run

L6-SEEP.XLS LP,iv15

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**15.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	7.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Lower Pipe, iv

	1	4	4	6					
		Rock	Rock	Filter		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.367**	**0.058**	**6.295**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.038**	7.837		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.079**	3.800	4.0754	16.50%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.300	**0.033**	9.150		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.300	**0.077**	3.919	6.8415	27.70%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.300	**0.093**	3.240		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.300	**0.029**	10.300	12.4609	50.45%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.367	**0.058**	6.295		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.233	**0.058**	3.997	1.3209	5.35%
							Sum:	24.6987	100.00%

E[FS] =	**6.295**	E[ln(FS)] =	1.5976	
Var[FS] =	24.6987	Var[ln(FS)] =	0.4844	b **=** **2.296**
s **[FS] =**	**4.9698**	σ[ln(FS)] =	0.6960	Pr(f) = 1.09E-02
V [FS] =	0.7894	V [ln(FS)] =	0.4356	**1 in** **92**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.300	0.300
	High	0.367	0.367
	Low	0.233	0.233

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	9.75E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**2.51E-02**
No. Links =	2.33	**1 in**	**40**

LP,ih15

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**15.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	7.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

	1	4	4	6		**Lower Pipe, ih**			
		Rock	Rock	Filter		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.225**	**0.019**	**11.764**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.017**	13.162		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.023**	9.816	2.8004	6.40%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.225	**0.042**	5.368		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.225	**0.030**	7.568	1.2104	2.76%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.225	**0.031**	7.180		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.225	**0.012**	19.519	38.0640	86.94%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.250	**0.019**	13.071		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.200	**0.019**	10.457	1.7086	3.90%
							Sum:	43.7834	100.00%

E[FS] =	**11.764**	E[ln(FS)] =	2.3276	
Var[FS] =	43.7834	Var[ln(FS)] =	0.2749	b **=** **4.440**
s **[FS] =**	**6.6169**	σ[ln(FS)] =	0.5243	Pr(f) = 4.51E-06
V [FS] =	0.5625	V [ln(FS)] =	0.2252	**1 in** **221811**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.225	0.225
	High	0.250	0.250
	Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	1.00E+00
"Link" Length =	1500	**Adjusted Pr(f) =**	**1.05E-05**
No. Links =	2.33	**1 in**	**95062**

UP,iv15

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**15.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	7.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Upper Pipe, iv

	1	4	4	6					
		Rock	Rock	Filter		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.300**	**0.057**	**5.269**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.040**	7.555		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.072**	4.184	2.8398	14.79%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.300	**0.040**	7.475		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.300	**0.070**	4.295	2.5285	13.17%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.300	**0.093**	3.210		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.300	**0.029**	10.267	12.4491	64.83%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.367	**0.057**	6.445		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.233	**0.057**	4.092	1.3846	7.21%
							Sum:	19.2020	100.00%

E[FS] =	**5.269**	E[ln(FS)] =	1.3989	
Var[FS] =	19.2020	Var[ln(FS)] =	0.5257	b **=** **1.929**
s **[FS] =**	**4.3820**	σ[ln(FS)] =	0.7251	Pr(f) = 2.68E-02
V [FS] =	0.8317	V [ln(FS)] =	0.5183	**1 in** **37**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.300	0.300
	High	0.367	0.367
	Low	0.233	0.233

Adjusted probabilities based on length effect:	**Total Length =**	3,500	Reliability of Section, R = 9.38E-01
	"Link" Length =	1500	**Adjusted Pr(f) =** **6.15E-02**
	No. Links =	2.33	**1 in** **16**

UP,ih15

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**15.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	7.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

	1	4	4	6		**Upper Pipe, ih**			
		Rock	Rock	Filter		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.225**	**0.056**	**4.037**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.039**	5.768		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.062**	3.625	1.1483	9.55%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.225	**0.034**	6.592		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.225	**0.065**	3.443	2.4806	20.62%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.225	**0.080**	2.795		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.225	**0.026**	8.522	8.1987	68.16%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.250	**0.056**	4.485		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.200	**0.056**	3.588	0.2012	1.67%
							Sum:	12.0288	100.00%

E[FS] =	**4.037**	E[ln(FS)] =	1.1190	
Var[FS] =	12.0288	Var[ln(FS)] =	0.5528	b **=** **1.505**
s **[FS] =**	**3.4683**	σ[ln(FS)] =	0.7435	Pr(f) = 6.62E-02
V [FS] =	0.8592	V [ln(FS)] =	0.6644	**1 in** **15**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.225	0.225
	High	0.250	0.250
	Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	8.52E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**1.48E-01**
No. Links =	2.33	**1 in**	**7**

UP,ih15(2)

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **15.0**

Run with TW = 8, not 7

Upper Pipe, ih

	1	4	4	6			Results		Prob Calcs	
		Rock	Rock	Filter	Ditch					
Run	Sands	Horiz.	Vert.	Cake	Tailwater	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**8.0**	**0.225**	**0.047**	**4.819**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	8.0	0.225	**0.034**	6.638		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	8.0	0.225	**0.051**	4.425	1.2238	9.09%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	8.0	0.225	**0.030**	7.467		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	8.0	0.225	**0.056**	3.982	3.0358	22.54%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	8.0	0.225	**0.070**	3.229		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	8.0	0.225	**0.024**	9.190	8.8844	65.97%
15	1.41E-03	1.77E-01	1.41E-02	6.01E-05	**8.3**	0.225	**0.045**	4.978		
16	1.41E-03	1.77E-01	1.41E-02	6.01E-05	**7.7**	0.225	**0.049**	4.595	0.0367	0.27%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	8.0	0.250	**0.047**	5.354		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	8.0	0.200	**0.047**	4.283	0.2867	2.13%
								Sum:	13.4672	100.00%

E[FS] =	**4.819**	E[ln(FS)] =	1.3438
Var[FS] =	13.4672	Var[ln(FS)] =	0.4574
s **[FS] =**	**3.6698**	σ[ln(FS)] =	0.6763
V [FS] =	0.7616	V [ln(FS)] =	0.5033

		For Piping	*For Breach*
Critical Gradient:	Avg	0.225	0.225
	High	0.250	0.250
	Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	9.46E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**5.39E-02**
No. Links =	2.33	**1 in**	**19**

b **=** **1.987**
Pr(f) = 2.35E-02
1 in **43**

LP,iv18

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Line 6

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **18.5**

Tailwater: **8.0**

	1	2	2	3	3	4	4	6	7	9			
		Peat	Peat	Clay	Clay	Rock	Rock	Filter			Ditch	Gravel	
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Cake	Pipe	Gravel	Tailwater	Pocket	i crit
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**3.54E+00**	**2.83E-01**	**8.0**	**145**	**0.300**
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
3	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
4	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
5	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
6	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
7	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
8	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300
9	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.38E-04**	3.54E+00	2.83E-01	8.0	145	0.300
10	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.61E-05**	3.54E+00	2.83E-01	8.0	145	0.300
11	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**8.13E+00**	2.83E-01	8.0	145	0.300
12	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**1.54E+00**	2.83E-01	8.0	145	0.300
13	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**6.50E-01**	8.0	145	0.300
14	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**1.23E-01**	8.0	145	0.300
15	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**8.3**	145	0.300
16	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**7.7**	145	0.300
17	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	165	0.300
18	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	125	0.300
19	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.367
20	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.233

		For Piping	*For Breach*	
Critical Gradient:	Avg	0.300	0.300	
	High	0.367	0.367	
	Low	0.233	0.233	

not used in final summary

E[FS] =	**3.550**	E[ln(FS)] =	1.0311
Var[FS] =	7.5968	Var[ln(FS)] =	0.4718
s **[FS] =**	**2.7562**	σ[ln(FS)] =	0.6868
V [FS] =	0.7764	V [ln(FS)] =	0.6661

Adjusted probabilities based on length effect:

Total Length =	3,500
"Link" Length =	1500
No. Links =	2.33

Lower Pipe, iv

Results		Prob Calcs	
i exit	**FS**	Var	%
0.085	**3.550**		
0.062	4.864		
0.112	2.686	1.1858	15.61%
0.089	3.388		
0.079	3.780	0.0384	0.51%
0.088	3.400		
0.081	3.717	0.0251	0.33%
0.048	6.300		
0.122	2.460	3.6864	48.53%
0.127	2.367		
1686.802	0.000	1.4001	18.43%
0.097	3.105		
0.065	4.586	0.5481	7.22%
0.081	3.688		
0.081	3.688	0.0000	0.00%
0.078	3.844		
0.088	3.417	0.0456	0.60%
0.086	3.488		
0.077	3.881	0.0388	0.51%
0.085	4.343		
0.085	2.757	0.6286	8.27%
	Sum:	7.5968	100.00%

b =	**1.501**
Pr(f) =	6.67E-02
1 in	**15**
Reliability of Section, R =	8.51E-01
Adjusted Pr(f) =	**1.49E-01**
1 in	**7**

LP,ih18

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Lake Level: 18.5

Tailwater: 8.0

	1	2	2	3	3	4	4	6	7	9			
		Peat	Peat	Clay	Clay	Rock	Rock	Filter			Ditch	Gravel	
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Cake	Pipe	Gravel	Tailwater	Pocket	i crit
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**3.54E+00**	**2.83E-01**	**8.0**	**145**	**0.225**
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
3	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
4	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
5	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
6	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
7	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
8	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225
9	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.38E-04**	3.54E+00	2.83E-01	8.0	145	0.225
10	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.61E-05**	3.54E+00	2.83E-01	8.0	145	0.225
11	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**8.13E+00**	2.83E-01	8.0	145	0.225
12	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**1.54E+00**	2.83E-01	8.0	145	0.225
13	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**6.50E-01**	8.0	145	0.225
14	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**1.23E-01**	8.0	145	0.225
15	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**8.3**	145	0.225
16	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**7.7**	145	0.225
17	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	165	0.225
18	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	125	0.225
19	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.250
20	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.200

	For Piping	*For Breach*
Critical Gradient: Avg	0.225	0.225
High	0.250	0.250
Low	0.200	0.200

E[FS] =	**7.519**	E[ln(FS)] =	1.7901
Var[FS] =	32.5296	Var[ln(FS)] =	0.4545
$_s$**[FS] =**	**5.7035**	$_d$[ln(FS)] =	0.6742
V [FS] =	0.7586	V [ln(FS)] =	0.3766

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	9.91E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**9.22E-03**
No. Links =	2.33	**1 in**	**108**

Lower Pipe, ih

Results		Prob Calcs	
i exit	**FS**	Var	%
0.030	**7.519**		
0.028	7.900		
0.035	6.375	0.5814	1.79%
0.032	6.995		
0.030	7.560	0.0797	0.24%
0.033	6.873		
0.025	9.000	1.1313	3.48%
0.017	13.500		
0.051	4.425	20.5889	63.29%
0.045	4.964		
0.021	10.704	8.2355	25.32%
0.032	6.975		
0.026	8.625	0.6806	2.09%
0.029	7.740		
0.027	8.305	0.0797	0.24%
0.030	7.445		
0.033	6.844	0.0905	0.28%
0.033	6.791		
0.028	7.998	0.3641	1.12%
0.030	8.354		
0.030	6.683	0.6979	2.15%
	Sum:	32.5296	100.00%

$_b$ **=**	**2.655**
Pr(f) =	3.96E-03
1 in	**252**

UP,iv18

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **18.5**
Tailwater: **8.0**

Upper Pipe, iv

	1	2	2	3	3	4	4	6	7	9				Results		Prob Calcs	
		Peat	Peat	Clay	Clay	Rock	Rock	Filter			Ditch	Gravel					
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Cake	Pipe	Gravel	Tailwater	Pocket	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**3.54E+00**	**2.83E-01**	**8.0**	**145**	**0.300**	**0.085**	**3.528**		
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.061**	4.888		
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.104**	2.871	1.0169	16.49%
3	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.087**	3.436		
4	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.079**	3.786	0.0305	0.49%
5	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.086**	3.496		
6	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.076**	3.932	0.0475	0.77%
7	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.058**	5.212		
8	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.300	**0.109**	2.764	1.4992	24.32%
9	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.38E-04**	3.54E+00	2.83E-01	8.0	145	0.300	**0.127**	2.362		
10	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.61E-05**	3.54E+00	2.83E-01	8.0	145	0.300	**0.052**	5.780	2.9215	47.38%
11	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**8.13E+00**	2.83E-01	8.0	145	0.300	**0.079**	3.788		
12	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**1.54E+00**	2.83E-01	8.0	145	0.300	**0.076**	3.968	0.0082	0.13%
13	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**6.50E-01**	8.0	145	0.300	**0.083**	3.627		
14	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**1.23E-01**	8.0	145	0.300	**0.083**	3.626	0.0000	0.00%
15	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**8.3**	145	0.300	**0.080**	3.730		
16	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**7.7**	145	0.300	**0.084**	3.564	0.0069	0.11%
17	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	165	0.300	**0.085**	3.535		
18	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	125	0.300	**0.079**	3.774	0.0143	0.23%
19	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.367	**0.085**	4.316		
20	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.233	**0.085**	2.740	0.6208	10.07%
															Sum:	6.1658	100.00%

			For Piping	*For Breach*	**E[FS] =**	**3.528**	E[ln(FS)] = 1.0595
	Critical Gradient:	Avg	0.300	0.300	Var[FS] =	6.1658	Var[ln(FS)] = 0.4024
		High	0.367	0.367	**ϛ[FS] =**	**2.4831**	σ[ln(FS)] = 0.6343
		Low	0.233	0.233	V [FS] =	0.7038	V [ln(FS)] = 0.5987

Adjusted probabilities based on length effect:

not used in final summary

Total Length =	3,500
"Link" Length =	1500
No. Links =	2.33

ϐ **=**	**1.670**
Pr(f) =	4.74E-02
1 in	**21**
Reliability of Section, R =	8.93E-01
Adjusted Pr(f) =	**1.07E-01**
1 in	**9**

UP,ih18

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **18.5**
Tailwater: **8.0**

	1	2	2	3	3	4	4	6	7	9				**Upper Pipe, ih**			
		Peat	Peat	Clay	Clay	Rock	Rock	Filter			Ditch	Gravel		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Cake	Pipe	Gravel	Tailwater	Pocket	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**3.54E+00**	**2.83E-01**	**8.0**	**145**	**0.225**	**0.097**	**2.322**		
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.065**	3.441		
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.116**	1.943	0.5610	21.92%
3	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.102**	2.208		
4	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.078**	2.886	0.1149	4.49%
5	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.093**	2.408		
6	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.081**	2.784	0.0354	1.38%
7	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.063**	3.578		
8	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.225	**0.116**	1.940	0.6705	26.20%
9	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.38E-04**	3.54E+00	2.83E-01	8.0	145	0.225	**0.127**	1.778		
10	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.61E-05**	3.54E+00	2.83E-01	8.0	145	0.225	**0.061**	3.701	0.9247	36.13%
11	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**8.13E+00**	2.83E-01	8.0	145	0.225	**0.088**	2.550		
12	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**1.54E+00**	2.83E-01	8.0	145	0.225	**0.081**	2.772	0.0123	0.48%
13	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**6.50E-01**	8.0	145	0.225	**0.089**	2.533		
14	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	**1.23E-01**	8.0	145	0.225	**0.083**	2.709	0.0077	0.30%
15	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**8.3**	145	0.225	**0.083**	2.700		
16	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	**7.7**	145	0.225	**0.090**	2.491	0.0109	0.43%
17	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	165	0.225	**0.100**	2.240		
18	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	125	0.225	**0.074**	3.028	0.1552	6.06%
19	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.250	**0.097**	2.580		
20	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.54E+00	2.83E-01	8.0	145	0.200	**0.097**	2.064	0.0666	2.60%
															Sum:	2.5591	100.00%

		For Piping	*For Breach*	
Critical Gradient:	Avg	0.225	0.225	
	High	0.250	0.250	
	Low	0.200	0.200	

E[FS] =	**2.322**	E[ln(FS)] =	0.6482
Var[FS] =	2.5591	Var[ln(FS)] =	0.3884
s [FS] =	**1.5997**	σ[ln(FS)] =	0.6232
V [FS] =	0.6889	V [ln(FS)] =	0.9614

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	6.86E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**3.14E-01**
No. Links =	2.33	**1 in**	**3**

b =	**1.040**	
Pr(f) =	1.49E-01	
1 in	**7**	

LP,iv21

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**21.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	9.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Lower Pipe, iv

	1	4	4	6		Results		Prob Calcs	
		Rock	Rock	Filter					
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.300**	**0.095**	**3.158**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.070**	4.286		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.126**	2.375	0.9127	14.02%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.300	**0.053**	5.660		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.300	**0.137**	2.190	3.0113	46.25%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.300	**0.148**	2.027		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.300	**0.061**	4.918	2.0895	32.09%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.367	**0.095**	3.863		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.233	**0.095**	2.453	0.4974	7.64%
							Sum:	6.5108	100.00%

E[FS] =	**3.158**	E[ln(FS)] =	0.8986	
Var[FS] =	6.5108	Var[ln(FS)] =	0.5025	b **=** **1.268**
s **[FS] =**	**2.5516**	σ[ln(FS)] =	0.7089	Pr(f) = 1.02E-01
V [FS] =	0.8080	V [ln(FS)] =	0.7888	**1 in** **10**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.300	0.300
	High	0.367	0.367
	Low	0.233	0.233

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	7.77E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**2.23E-01**
No. Links =	2.33	**1 in**	**4**

LP,ih21

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **21.0**
Tailwater: 9.0

Lower Pipe, ih

Run	1 Sands	4 Rock Horiz.	4 Rock Vert.	6 Filter Cake	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.225**	**0.036**	**6.250**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.031**	7.258		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.039**	5.769	0.5542	1.90%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.225	**0.017**	13.235		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.225	**0.057**	3.947	21.5664	73.90%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.225	**0.053**	4.245		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.225	**0.024**	9.375	6.5785	22.54%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.250	**0.036**	6.944		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.200	**0.036**	5.556	0.4823	1.65%
							Sum:	29.1813	100.00%

E[FS] =	**6.250**	E[ln(FS)] =	1.5536	
Var[FS] =	29.1813	Var[ln(FS)] =	0.5579	b **=** **2.080**
s **[FS] =**	**5.4020**	σ[ln(FS)] =	0.7469	Pr(f) = 1.88E-02
V [FS] =	0.8643	V [ln(FS)] =	0.4808	**1 in** **53**

Critical Gradient:	*For Piping*	*For Breach*
Avg	0.225	0.225
High	0.250	0.250
Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	9.57E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**4.32E-02**
No. Links =	2.33	**1 in**	**23**

UP,iv21

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**21.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	9.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

	1	4	4	6		**Upper Pipe, iv**			
		Rock	Rock	Filter		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.300**	**0.094**	**3.191**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.070**	4.286		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.300	**0.116**	2.586	0.7221	15.15%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.300	**0.064**	4.688		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.300	**0.123**	2.439	1.2639	26.51%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.300	**0.145**	2.069		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.300	**0.059**	5.085	2.2737	47.69%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.367	**0.094**	3.904		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.233	**0.094**	2.479	0.5080	10.66%
							Sum:	4.7678	100.00%

E[FS] =	**3.191**	E[ln(FS)] =	0.9685	
Var[FS] =	4.7678	Var[ln(FS)] =	0.3840	b **=** **1.563**
s **[FS] =**	**2.1835**	σ[ln(FS)] =	0.6196	Pr(f) = 5.90E-02
V [FS] =	0.6842	V [ln(FS)] =	0.6398	**1 in** **17**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.300	0.300
	High	0.367	0.367
	Low	0.233	0.233

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	8.68E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**1.32E-01**
No. Links =	2.33	**1 in**	**8**

UP,ih21

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**21.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	9.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Upper Pipe, ih

	1	4	4	6		Results		Prob Calcs	
		Rock	Rock	Filter					
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.225**	**0.109**	**2.064**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.075**	3.000		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.136**	1.654	0.4527	27.78%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.225	**0.074**	3.041		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.225	**0.138**	1.630	0.4971	30.51%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.225	**0.146**	1.541		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.225	**0.072**	3.125	0.6272	38.49%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.250	**0.109**	2.294		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.200	**0.109**	1.835	0.0526	3.23%
							Sum:	1.6295	100.00%

E[FS] =	**2.064**	E[ln(FS)] =	0.5628		
Var[FS] =	1.6295	Var[ln(FS)] =	0.3238	b =	**0.989**
s **[FS] =**	**1.2765**	σ[ln(FS)] =	0.5691	Pr(f) =	1.61E-01
V [FS] =	0.6184	V [ln(FS)] =	1.0111	**1 in**	**6**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.225	0.225
	High	0.250	0.250
	Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	6.63E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**3.37E-01**
No. Links =	2.33	**1 in**	**3**

UP,ih21(2)

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**21.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	8.0
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

	1	4	4	6		**Upper Pipe, ih**			
		Rock	Rock	Filter		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Cake	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.225**	**0.121**	**1.867**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.081**	2.784		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.149**	1.512	0.4041	29.03%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.225	**0.081**	2.778		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.225	**0.147**	1.532	0.3886	27.92%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.225	**0.158**	1.422		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.225	**0.077**	2.914	0.5563	39.96%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.250	**0.121**	2.074		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.200	**0.121**	1.659	0.0430	3.09%
							Sum:	1.3920	100.00%

E[FS] =	**1.867**	E[ln(FS)] =	0.4562	
Var[FS] =	1.3920	Var[ln(FS)] =	0.3361	b **=** **0.787**
s **[FS] =**	**1.1798**	σ[ln(FS)] =	0.5797	Pr(f) = 2.16E-01
V [FS] =	0.6320	V [ln(FS)] =	1.2708	**1 in** **5**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.225	0.225
	High	0.250	0.250
	Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	5.67E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**4.33E-01**
No. Links =	2.33	**1 in**	**2**

Emb.ih21

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Line 6

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Lake Level: **21.0**

Tailwater: 9.0

	1	4		
		Gravel		Results
Run	Sands	Pocket	i crit	i exit
E	**1.41E-03**	**145**	**0.561**	**0.410**
1	**3.25E-03**	145	0.561	**0.430**
2	**6.15E-04**	145	0.561	**0.173**
17	1.41E-03	165	0.561	**0.433**
18	1.41E-03	125	0.561	**0.001**
19	1.41E-03	145	0.636	**0.410**
20	1.41E-03	145	0.486	**0.410**

E[FS] =	**1.369**	E[ln(FS)] =	0.2600
Var[FS] =	0.2128	Var[ln(FS)] =	0.1076
s **[FS] =**	**0.4613**	σ[ln(FS)] =	0.3280
V [FS] =	0.3371	V [ln(FS)] =	1.2618

		For Piping	*For Breach*
Critical Gradient:	Avg	0.488	0.561
	High	0.553	0.636
	Low	0.423	0.486

b =	**0.793**
Pr(f) =	2.14E-01
1 in	**5**

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	5.70E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**4.30E-01**
No. Links =	2.33	**1 in**	**2**

EXIT GRADIENT

	Prob Calcs	
i exit	Var	%
0.410		
0.430		
0.173	0.0165	26.14%
0.433		
0.001	0.0467	73.86%
0.410		
0.410	0.0000	0.00%
Sum:	0.0633	100.00%

E[i exit] =	**0.410**	E[ln(FS)] =	-1.0512
Var[i exit] =	0.0633	Var[ln(FS)] =	0.3194
s **[i exit] =**	**0.2516**	σ[ln(FS)] =	0.5652
V [i exit] =	0.6135	V [ln(FS)] =	-0.5377
E [i exit]	0.410		
[i exit] +	0.662		
[i exit] -	0.159		

CRITICAL GRADIENT

	Prob Calcs	
i crit	Var	%
0.561		
0.561		
0.561	0.0000	0.00%
0.561		
0.561	0.0000	0.00%
0.636		
0.486	0.0056	100.00%
Sum:	0.0056	100.00%

E[i crit.] =	**0.561**	E[ln(FS)] =	-0.5865
Var[i crit.] =	0.0056	Var[ln(FS)] =	0.0176
s **[i crit.] =**	**0.0748**	σ[ln(FS)] =	0.1326
V [i crit.] =	0.1332	V [ln(FS)] =	-0.2261
E [i crit.]	0.561		
[i crit.]+	0.636		
[i crit.] -	0.486		

UP,ih24

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	FASTSEEP	**Lake Level:**	**24.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS		
Random Variable		**Tailwater:**	10.0
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Upper Pipe, ih

	1	4	4	6			Results		Prob Calcs	
		Rock	Rock	Filter	Gravel					
Run	Sands	Horiz.	Vert.	Cake	Pocket	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**145**	**0.225**	**0.114**	**1.974**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	145	0.225	**0.082**	2.756		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	145	0.225	**0.135**	1.668	0.2959	18.14%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	145	0.225	**0.076**	2.964		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	145	0.225	**0.143**	1.575	0.4824	29.58%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	145	0.225	**0.170**	1.322		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	145	0.225	**0.072**	3.116	0.8044	49.32%
17 / 21	1.41E-03	1.77E-01	1.41E-02	6.01E-05	165	0.225	**0.113**	1.999		
18 / 22	1.41E-03	1.77E-01	1.41E-02	6.01E-05	125	0.225	**0.111**	2.025	0.0002	0.01%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	145	0.250	**0.114**	2.194		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	145	0.200	**0.114**	1.755	0.0481	2.95%
								Sum:	1.6310	100.00%

E[FS] =	**1.974**	E[ln(FS)] =	0.5054
Var[FS] =	1.6310	Var[ln(FS)] =	0.3496
s **[FS] =**	**1.2771**	σ[ln(FS)] =	0.5913
V [FS] =	0.6469	V [ln(FS)] =	1.1700

		For Piping	*For Breach*
Critical Gradient:	Avg	0.225	0.225
	High	0.250	0.250
	Low	0.200	0.200

b **=**	**0.855**		
Pr(f) =	1.96E-01		
1 in	**5**		

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	6.00E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**4.00E-01**
No. Links =	2.33	**1 in**	**3**

UP,ih24(2)

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **24.0**
Tailwater: 8.0

Upper Pipe, ih

Run	1 Sands	4 Rock Horiz.	4 Rock Vert.	6 Filter Cake	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**0.225**	**0.149**	**1.512**		
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.100**	2.254		
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	0.225	**0.183**	1.232	0.2614	28.93%
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	0.225	**0.098**	2.298		
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	0.225	**0.183**	1.232	0.2840	31.43%
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	0.225	**0.196**	1.149		
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	0.225	**0.098**	2.298	0.3299	36.51%
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.250	**0.149**	1.680		
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	0.200	**0.149**	1.344	0.0282	3.12%
							Sum:	0.9036	100.00%

E[FS] =	**1.512**	E[ln(FS)] =	0.2472	
Var[FS] =	0.9036	Var[ln(FS)] =	0.3329	b **=** **0.428**
s **[FS] =**	**0.9506**	σ[ln(FS)] =	0.5770	Pr(f) = 3.34E-01
V [FS] =	0.6285	V [ln(FS)] =	2.3343	**1 in** **3**

Critical Gradient:	*For Piping*	*For Breach*
Avg	0.225	0.225
High	0.250	0.250
Low	0.200	0.200

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	3.87E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**6.13E-01**
No. Links =	2.33	**1 in**	**2**

Emb.ih,24

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Line 6

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **24.0**
Tailwater: 10.0

	1	4	4	6	4		
		Rock	Rock	Filter	Gravel		Results
Run	Sands	Horiz.	Vert.	Cake	Pocket	i crit	i exit
E	**1.41E-03**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**145**	**0.561**	**0.437**
1	**3.25E-03**	1.77E-01	1.41E-02	6.01E-05	145	0.561	**0.430**
2	**6.15E-04**	1.77E-01	1.41E-02	6.01E-05	145	0.561	**0.427**
7	1.41E-03	**4.06E-01**	**3.25E-02**	6.01E-05	145	0.561	**0.433**
8	1.41E-03	**7.69E-02**	**6.15E-03**	6.01E-05	145	0.561	**0.430**
9	1.41E-03	1.77E-01	1.41E-02	**1.38E-04**	145	0.561	**0.430**
10	1.41E-03	1.77E-01	1.41E-02	**2.61E-05**	145	0.561	**0.430**
17 / 21	1.41E-03	1.77E-01	1.41E-02	6.01E-05	165	0.561	**0.437**
18 / 22	1.41E-03	1.77E-01	1.41E-02	6.01E-05	125	0.561	**0.433**
19	1.41E-03	1.77E-01	1.41E-02	6.01E-05	145	0.587	**0.437**
20	1.41E-03	1.77E-01	1.41E-02	6.01E-05	145	0.474	**0.437**

E[FS] =	**1.285**	E[ln(FS)] =	0.2074
Var[FS] =	0.1495	Var[ln(FS)] =	0.0867
$_s$ **[FS] =**	**0.3867**	σ[ln(FS)] =	0.2944
V [FS] =	0.3009	V [ln(FS)] =	1.4194

$_b$ **=** **0.705**
Pr(f) = 2.41E-01
1 in **4**

		For Piping	*For Breach*
Critical Gradient:	Avg	0.488	0.561
	High	0.553	0.636
	Low	0.423	0.486

Adjusted probabilities based on length effect:

Total Length =	3,500	Reliability of Section, R =	5.26E-01
"Link" Length =	1500	**Adjusted Pr(f) =**	**4.74E-01**
No. Links =	2.33	**1 in**	**2**

EXIT GRADIENT

Run	i exit	Prob Calcs Var	%
E	**0.437**		
1	**0.430**		
2	**0.427**	0.00000	33.33%
7	**0.433**		
8	**0.430**	0.0000	33.33%
9	**0.430**		
10	**0.430**	0.0000	0.00%
17 / 21	**0.437**		
18 / 22	**0.433**	0.0000	33.33%
19	**0.437**		
20	**0.437**	0.0000	0.00%
Sum:		0.0000	100.00%

E[i exit] =	**0.437**	E[ln(FS)] =	-0.8285
Var[i exit] =	0.0000	Var[ln(FS)] =	0.0000
$_s$ **[i exit] =**	**0.0029**	σ[ln(FS)] =	0.0066
V [i exit] =	0.0066	V [ln(FS)] =	-0.0080

E [i exit]	0.437
[i exit] +	0.440
[i exit] -	0.434

CRITICAL GRADIENT

Run	i crit	Prob Calcs Var	%
E	**0.561**		
1	0.561		
2	0.561	0.0000	0.00%
7	0.561		
8	0.561	0.0000	0.00%
9	0.561		
10	0.561	0.0000	0.00%
17 / 21	0.561		
18 / 22	0.561	0.0000	0.00%
19	0.636		
20	0.486	0.0056	100.00%
Sum:		0.0056	100.00%

E[i crit.] =	**0.561**	E[ln(FS)] =	-0.5865
Var[i crit.] =	0.0056	Var[ln(FS)] =	0.0176
$_s$ **[i crit.] =**	**0.0748**	σ[ln(FS)] =	0.1326
V [i crit.] =	0.1332	V [ln(FS)] =	-0.2261

E [i crit.]	0.561
[i crit.]+	0.636
[i crit.] -	0.486

Summary

Exit Gradient Computations					**Line 6; Lake = 18.5; Mean TW = 8.0**							
	At pipe entrance					**Upper Pipe**	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**	Y	Head +	Head -	X +	X -	**ih**
E	**427.62**	**8.50**	**8.25**	**-2.00**	**-4.94**	**0.085**	**-2.00**	**8.50**	**8.25**	**427.62**	**425.04**	**0.097**
1	427.62	8.50	8.33	-1.99	-4.76	0.061	-1.99	8.50	8.33	427.62	425.02	0.065
2	427.61	8.45	8.17	-2.00	-4.68	0.104	-2.00	8.50	8.17	427.61	424.76	0.116
3	427.62	8.50	8.28	-2.01	-4.53	0.087	-2.01	8.60	8.28	427.62	424.48	0.102
4	427.62	8.45	8.24	-2.00	-4.65	0.079	-2.00	8.47	8.24	427.62	424.67	0.078
5	427.60	8.53	8.30	-2.01	-4.69	0.086	-2.01	8.57	8.30	427.60	424.71	0.093
6	427.61	8.40	8.21	-1.99	-4.48	0.076	-1.99	8.45	8.21	427.61	424.64	0.081
7	427.61	8.38	8.22	-1.99	-4.77	0.058	-1.99	8.42	8.22	427.61	424.43	0.063
8	427.61	8.60	8.27	-2.01	-5.05	0.109	-2.01	8.64	8.27	427.61	424.42	0.116
9	427.61	8.73	8.34	-2.00	-5.07	0.127	-2.00	8.75	8.34	427.61	424.37	0.127
10	427.60	8.35	8.20	-2.01	-4.90	0.052	-2.01	8.40	8.20	427.60	424.31	0.061
11	427.61	8.35	8.11	-2.01	-5.04	0.079	-2.01	8.35	8.11	427.61	424.89	0.088
12	427.60	8.75	8.53	-2.01	-4.92	0.076	-2.01	8.78	8.53	427.60	424.52	0.081
13	427.60	8.48	8.26	-2.01	-4.67	0.083	-2.01	8.53	8.26	427.60	424.56	0.089
14	427.61	8.48	8.25	-2.01	-4.79	0.083	-2.01	8.81	8.55	427.61	424.48	0.083
15	427.60	8.78	8.55	-2.01	-4.87	0.080	-2.01	8.81	8.55	427.60	424.48	0.083
16	427.58	8.21	7.96	-2.00	-4.97	0.084	-2.00	8.24	7.96	427.58	424.48	0.090
17 / 21	427.59	8.50	8.27	-2.01	-4.72	0.085	-2.01	8.50	8.27	427.59	425.30	0.100
18 / 22	427.61	8.35	8.16	-2.01	-4.40	0.079	-2.01	8.40	8.16	427.61	424.38	0.074
19	427.62	8.50	8.25	-2.00	-4.94	0.085	-2.00	8.50	8.25	427.62	425.04	0.097
20	427.62	8.50	8.25	-2.00	-4.94	0.085	-2.00	8.50	8.25	427.62	425.04	0.097

Exit Gradient Computations					**Line 6; Lake = 18.5; Mean TW = 8.0**							
	At pipe entrance					**Lower Pipe**	At pipe entrance					**Lower Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**	Y	Head +	Head -	X +	X -	**ih**
E	**437.40**	**8.60**	**8.48**	**-12.30**	**-13.72**	**0.085**	**-12.30**	**8.60**	**8.48**	**437.40**	**433.39**	**0.030**
1	437.38	8.60	8.46	-12.31	-14.58	0.062	-12.31	8.55	8.46	437.38	434.22	0.028
2	437.40	8.70	8.48	-12.30	-14.27	0.112	-12.30	8.60	8.48	437.40	434.00	0.035
3	437.38	8.68	8.51	-12.31	-14.23	0.089	-12.31	8.62	8.51	437.38	433.96	0.032
4	437.40	8.60	8.45	-12.31	-14.20	0.079	-12.31	8.55	8.45	437.40	434.04	0.030
5	437.38	8.70	8.52	-12.28	-14.32	0.088	-12.28	8.63	8.52	437.38	434.02	0.033
6	437.39	8.60	8.42	-12.30	-14.53	0.081	-12.30	8.52	8.42	437.39	433.39	0.025
7	437.39	8.50	8.40	-12.30	-14.40	0.048	-12.30	8.46	8.40	437.39	433.79	0.017
8	437.40	8.73	8.48	-12.30	-14.35	0.122	-12.30	8.66	8.48	437.40	433.86	0.051
9	437.38	8.96	8.69	-12.30	-14.43	0.127	-12.30	8.85	8.69	437.38	433.85	0.045
10	437.39	3500.00	8.32	-12.30	-14.37	1686.802	-12.30	8.39	8.32	437.39	434.06	0.021
11	437.40	8.50	8.30	-12.28	-14.35	0.097	-12.28	8.41	8.30	437.40	433.99	0.032
12	437.40	8.87	8.73	-12.30	-14.44	0.065	-12.30	8.82	8.73	437.40	433.95	0.026
13	437.39	8.65	8.48	-12.30	-14.39	0.081	-12.30	8.58	8.48	437.39	433.95	0.029
14	437.79	8.64	8.47	-12.29	-14.38	0.081	-12.29	8.58	8.47	437.79	433.73	0.027
15	437.38	8.92	8.76	-12.30	-14.35	0.078	-12.30	8.87	8.76	437.38	433.74	0.030
16	437.40	8.36	8.18	-12.30	-14.35	0.088	-12.30	8.30	8.18	437.40	433.75	0.033
17 / 21	437.37	8.65	8.49	-12.30	-14.16	0.086	-12.30	8.60	8.49	437.37	434.05	0.033
18 / 22	437.39	8.55	8.39	-12.31	-14.38	0.077	-12.31	8.50	8.39	437.39	433.48	0.028
19	437.40	8.60	8.48	-12.30	-13.72	0.085	-12.30	8.60	8.48	437.40	433.39	0.030
20	437.40	8.60	8.48	-12.30	-13.72	0.085	-12.30	8.60	8.48	437.40	433.39	0.030

Exit Gradient Computations **Line 6; Lake = 21.0; Mean TW = 9.0**

	At pipe entrance					**Upper Pipe**	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**	Y	Head +	Head -	X +	X -	**ih**
E	**427.62**	**9.55**	**9.30**	**-1.99**	**-4.65**	**0.094**	**-1.99**	**9.65**	**9.30**	**427.62**	**424.42**	**0.109**
1	427.60	9.60	9.38	-2.00	-5.14	0.070	-2.00	9.60	9.38	427.60	424.68	0.075
2	427.61	9.55	9.22	-1.99	-4.83	0.116	-1.99	9.65	9.22	427.61	424.44	0.136
7	427.62	9.43	9.26	-1.99	-4.66	0.064	-1.99	9.50	9.26	427.62	424.37	0.074
8	427.60	9.68	9.31	-2.00	-5.01	0.123	-2.00	9.77	9.31	427.60	424.27	0.138
9	427.61	9.82	9.40	-2.01	-4.91	0.145	-2.01	9.86	9.40	427.61	424.45	0.146
10	427.62	9.41	9.23	-2.01	-5.04	0.059	-2.01	9.47	9.23	427.62	424.30	0.072
19	427.62	9.55	9.30	-1.99	-4.65	0.094	-1.99	9.65	9.30	427.62	424.42	0.109
20	427.62	9.55	9.30	-1.99	-4.65	0.094	-1.99	9.65	9.30	427.62	424.42	0.109

	At pipe entrance					**Lower Pipe**	At pipe entrance					**Lower Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**	Y	Head +	Head -	X +	X -	**ih**
E	**437.38**	**9.75**	**9.55**	**-12.30**	**-14.40**	**0.095**	**-12.30**	**9.68**	**9.55**	**437.38**	**433.76**	**0.036**
1	437.39	9.67	9.53	-12.31	-14.31	0.070	-12.31	9.65	9.53	437.39	433.54	0.031
2	437.39	9.80	9.56	-12.30	-14.20	0.126	-12.30	9.70	9.56	437.39	433.83	0.039
7	437.40	9.60	9.47	-12.30	-14.75	0.053	-12.30	9.54	9.47	437.40	433.37	0.017
8	437.38	9.85	9.55	-12.29	-14.48	0.137	-12.29	9.77	9.55	437.38	433.54	0.057
9	437.39	10.10	9.79	-12.31	-14.41	0.148	-12.31	9.98	9.79	437.39	433.82	0.053
10	437.40	9.49	9.37	-12.31	-14.28	0.061	-12.31	9.46	9.37	437.40	433.63	0.024
19	437.38	9.75	9.55	-12.30	-14.40	0.095	-12.30	9.68	9.55	437.38	433.76	0.036
20	437.38	9.75	9.55	-12.30	-14.40	0.095	-12.30	9.68	9.55	437.38	433.76	0.036

	At bottom of slope.							**Emb.**
Run	X+	X-	Head +	Head -	Y+	Y-	Length	**ih**
E	**513.80**	**510.83**	**12.14**	**10.91**	**11.32**	**10.90**	**3.00**	**0.410**
1	513.80	510.83	12.19	10.90	11.32	10.90	3.00	**0.430**
2	513.80	510.83	11.36	10.84	11.32	10.84	3.01	**0.173**
17 / 21	513.80	510.83	12.20	10.90	11.32	10.90	3.00	**0.433**
18 / 22							0.00	**0.001**
19	513.80	510.83	12.14	10.91	11.32	10.90	3.00	**0.410**
20	513.80	510.83	12.14	10.91	11.32	10.90	3.00	**0.410**

Exit Gradient Computations **Line 6; Lake = 21.0; Mean TW = 8.0**

	At pipe entrance					**Upper Pipe**			At bottom of slope.			
Run	Y	Head +	Head -	Y+	Y-	**ih**		Run	X+	X-	Head +	Head -
E	**-1.99**	**8.96**	**8.32**	**427.61**	**422.30**	**0.121**		**E**	**513.80**	**510.83**	**11.93**	**10.90**
1	-2.00	8.84	8.41	427.61	422.29	0.081		1				
2	-2.00	9.02	8.23	427.61	422.30	0.149		2	**513.80**	**510.83**	**12.21**	**10.90**
7	-2.00	8.71	8.28	427.61	422.30	0.081		17 / 21				
8	-2.00	9.12	8.34	427.61	422.30	0.147		18 / 22				
9	-2.00	9.27	8.43	427.61	422.30	0.158		19	513.80	510.83	11.93	10.90
10	-2.00	8.66	8.25	427.61	422.30	0.077		20	513.80	510.83	11.93	10.90
19	-1.99	8.96	8.32	427.61	422.30	0.121						
20	-1.99	8.96	8.32	427.61	422.30	0.121						

Exit Gradient Computations **Line 6; Lake = 15.0; Mean TW = 7.0**

	At pipe entrance					**Upper Pipe**	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**	Y	Head +	Head -	X +	X -	**ih**
E	**427.61**	**7.27**	**7.11**	**-2.00**	**-4.81**	**0.057**	**-2.00**	**7.28**	**7.11**	**427.61**	**424.56**	**0.056**
1	427.61	7.27	7.16	-1.99	-4.76	0.040	-1.99	7.27	7.16	427.61	424.79	0.039
2	427.61	7.26	7.07	-2.00	-4.65	0.072	-2.00	7.25	7.07	427.61	424.71	0.062
7	427.61	7.22	7.10	-1.99	-4.98	0.040	-1.99	7.20	7.10	427.61	424.68	0.034
8	427.60	7.31	7.12	-1.99	-4.71	0.070	-1.99	7.32	7.12	427.60	424.54	0.065
9	427.61	7.48	7.18	-2.00	-5.21	0.093	-2.00	7.44	7.18	427.61	424.38	0.080
10	427.61	7.16	7.07	-2.00	-5.08	0.029	-2.00	7.15	7.07	427.61	424.58	0.026
19	427.61	7.27	7.11	-2.00	-4.81	0.057	-2.00	7.28	7.11	427.61	424.56	0.056
20	427.61	7.27	7.11	-2.00	-4.81	0.057	-2.00	7.28	7.11	427.61	424.56	0.056

	At pipe entrance					**Lower Pipe**	At pipe entrance					**Lower Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**	Y	Head +	Head -	X +	X -	**ih**
E	**437.40**	**7.41**	**7.28**	**-12.30**	**-14.53**	**0.058**	**-12.30**	**7.35**	**7.28**	**437.40**	**433.74**	**0.019**
1	437.38	7.34	7.26	-12.30	-14.39	0.038	-12.30	7.32	7.26	437.38	433.87	0.017
2	437.39	7.45	7.30	-12.31	-14.21	0.079	-12.31	7.38	7.30	437.39	433.90	0.023
7	437.39	7.30	7.24	-12.29	-14.12	0.033	-12.29	7.38	7.24	437.39	434.05	0.042
8	437.39	7.44	7.28	-12.30	-14.39	0.077	-12.30	7.39	7.28	437.39	433.69	0.030
9	437.39	7.65	7.45	-12.31	-14.47	0.093	-12.31	7.56	7.45	437.39	433.88	0.031
10	437.39	7.22	7.16	-12.30	-14.36	0.029	-12.30	7.20	7.16	437.39	433.92	0.012
19	437.40	7.41	7.28	-12.30	-14.53	0.058	-12.30	7.35	7.28	437.40	433.74	0.019
20	437.40	7.41	7.28	-12.30	-14.53	0.058	-12.30	7.35	7.28	437.40	433.74	0.019

	Line 6; Lake = 15.0; Mean TW =8.0					
	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**-2.00**	**8.22**	**8.10**	**427.61**	**425.04**	**0.047**
1	-2.00	8.32	8.14	427.61	422.30	0.034
2	-2.00	8.34	8.07	427.61	422.30	0.051
7	-2.00	8.24	8.08	427.61	422.30	0.030
8	-2.00	8.41	8.11	427.61	422.30	0.056
9	-2.00	8.53	8.16	427.61	422.30	0.070
10	-2.00	8.19	8.06	427.61	422.30	0.024
15	-2.00	8.63	8.39	427.61	422.30	0.045
16	-2.00	8.06	7.80	427.61	422.30	0.049
19	-2.00	8.22	8.10	427.61	425.04	0.047
20	-2.00	8.22	8.10	427.61	425.04	0.047

Exit Gradient Computations **Line 6; Lake = 24.0; Mean TW = 10.0**

	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**ih**
E	**427.60**	**10.66**	**10.35**	**-2.00**	**-4.72**	**0.114**
1	427.62	10.68	10.44	-1.99	-4.93	0.082
2	427.60	10.67	10.26	-2.00	-5.04	0.135
7	427.60	10.54	10.31	-2.00	-5.03	0.076
8	427.60	10.78	10.37	-1.99	-4.86	0.143
9	427.62	10.95	10.46	-2.00	-4.88	0.170
10	427.00	10.47	10.27	-2.00	-4.77	0.072
17 / 21	61.00	10.70	10.35	-2.00	-5.11	0.113
18 / 22	427.60	10.70	10.34	-1.99	-5.23	0.111
19	427.60	10.66	10.35	-2.00	-4.72	0.114
20	427.60	10.66	10.35	-2.00	-4.72	0.114

	At bottom of slope.							**Emb.**
Run	X+	X-	Head +	Head -	Y+	Y-	Length	**ih**
E	**513.80**	**510.83**	**12.21**	**10.90**	**11.32**	**10.90**	**3.00**	**0.437**
1	513.80	510.83	12.20	10.91	11.32	10.90	3.00	**0.430**
2	513.80	510.83	12.19	10.91	11.32	10.90	3.00	**0.427**
7	513.80	510.83	12.20	10.90	11.32	10.90	3.00	**0.433**
8	513.80	510.83	12.20	10.91	11.32	10.90	3.00	**0.430**
9	513.80	510.83	12.19	10.90	11.32	10.90	3.00	**0.430**
10	513.80	510.83	12.19	10.90	11.32	10.90	3.00	**0.430**
17 / 21	513.80	510.83	12.21	10.90	11.32	10.90	3.00	**0.437**
18 / 22	513.80	510.83	12.20	10.90	11.32	10.90	3.00	**0.433**
19	513.80	510.83	12.21	10.90	11.32	10.90	3.00	**0.437**
20	513.80	510.83	12.21	10.90	11.32	10.90	3.00	**0.437**

Exit Gradient Computations **Line 6; Lake = 24.0; Mean TW = 8.0**

	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**ih**
E	**-2.00**	**9.19**	**8.40**	**427.61**	**422.30**	**0.149**
1	-2.00	9.03	8.50	427.61	422.30	0.100
2	-2.00	9.27	8.30	427.61	422.30	0.183
7	-2.00	8.87	8.35	427.61	422.30	0.098
8	-2.00	9.39	8.42	427.61	422.30	0.183
9	-2.00	9.57	8.53	427.61	422.30	0.196
10	-2.00	8.84	8.32	427.61	422.30	0.098
19	-2.00	9.19	8.40	427.61	422.30	0.149
20	-2.00	9.19	8.40	427.61	422.30	0.149

C18.xls Ditch Slope,ih

Probabilistic Through Seepage Analysis:

Herbert Hoover Dike; Line C-18

Deterministic Model: FASTSEEP

LAKE AT 17.5'

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

AVE "N" FOR SM SAND =10 B/F UNIT WGT = 120 PCF

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Ditch Slope, ih

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	ML Horiz	ML Vert	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-05**	**7.11E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**11.2**	**0.82**	**0.521**	**1.574**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.526**	**1.559**		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.531**	**1.544**	0.0001	0.03%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.526**	**1.559**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.526**	**1.559**	0.0000	0.00%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.531**	**1.544**		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.505**	**1.623**	0.0016	0.87%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**7.09E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.578**	**1.418**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**1.81E-04**	**2.79E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.469**	**1.749**	0.0274	15.03%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.547**	**1.499**		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.505**	**1.623**	0.0038	2.10%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	11.2	**0.82**	**0.438**	**1.874**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	11.2	**0.82**	**0.625**	**1.312**	0.0790	43.38%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	11.2	**0.82**	**0.526**	**1.559**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	11.2	**0.82**	**0.521**	**1.574**	0.0001	0.03%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	11.2	**0.82**	**0.411**	**1.993**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	11.2	**0.82**	**0.526**	**1.559**	0.0471	25.86%
17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**11.5**	**0.90**	**0.526**	**1.711**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**10.9**	**0.74**	**0.526**	**1.407**	0.0231	12.69%
																Sum:	0.1822	100.00%

Adjusted probabilities based on length effect:

Total Length =	**9,000**	Reliability of Section, R =	5.46E-01	**E[FS] =**	**1.574**
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**4.54E-01**	Var[FS] =	0.1822
No. Links =	**3**	**1 in**	**2**	**s [FS] =**	**0.4268**
				V [FS] =	0.2711

E[ln(FS)] =	0.3702
Var[ln(FS)] =	0.1674
σ[ln(FS)] =	0.4091
V [ln(FS)] =	1.1051

b =	**0.905**
Pr(f) =	**1.83E-01**
1 in	**5**

Ditch Bottom iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line C-18**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

LAKE AT 17.5'

BASE CONDITION 70 PCF

Ditch Bottom, iv

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	ML Horiz	ML Vert	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater	i crit	Results FS HEAVE	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-05**	**7.11E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**12.0**	**0.86**	**0.456**	**1.886**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.456**	**1.886**		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.456**	**1.886**	0.0000	0.00%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.458**	**1.879**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.456**	**1.886**	0.0000	0.09%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.458**	**1.879**		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.456**	**1.886**	0.0000	0.09%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**7.09E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.435**	**1.975**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**1.81E-04**	**2.79E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.474**	**1.814**	0.0065	48.98%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.470**	**1.830**		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.432**	**1.991**	0.0065	48.46%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.458**	**1.879**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.452**	**1.904**	0.0002	1.18%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	**12.0**	**0.86**	**0.457**	**1.882**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	**12.0**	**0.86**	**0.451**	**1.907**	0.0002	1.19%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	**12.0**	**0.86**	**0.456**	**1.886**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	**12.0**	**0.86**	**0.456**	**1.886**	0.0000	0.00%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.86**	**0.456**	**1.886**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**11.5**	**0.86**	**0.456**	**1.886**	0.0000	0.00%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.94**	**0.456**	**2.061**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.0**	**0.78**	**0.456**	**1.710**	0.0308	230.81%
																Sum:	0.0133	100.00%

Adjusted probabilities based on length effect:

Total Length =	**9,000**	Reliability of Section, R =	1.00E+00	**E[FS] =**	**1.886**
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**7.38E-08**	Var[FS] =	0.0133
No. Links =	**3**	**1 in**	**13543007**	s **[FS] =**	**0.1155**
				V [FS] =	0.0612

E[ln(FS)] =	0.6277
Var[ln(FS)] =	0.0132
σ[ln(FS)] =	0.1151
V [ln(FS)] =	0.1833

b =	**5.455**
Pr(f) =	**2.46E-08**
1 in	**40629020**

L6-SST.XLS L6-18.5

Probabilistic Slope Stability Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model: UTEXAS, Spencer's Method
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
stribution Assumptions: Cohesion lognormal, Phi, gamma & Tailwater normal

Lake Level: **18.5**
Tailwater: **8.0**

Failure surface tangent to first rock layer

	Embankment & Lower Sands			Peat / OL / OH				Silts & Clays			Gravel	Failure Arc	**Computed**	Prob Calcs	
Run	θ	γ moist	γ sat	θ	C	γ moist	γ sat	θ	C	γ sat	Pocket *	X, Y, R	**FS**	Var	%
E	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.741**		
1	36.3	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.4/129.6/136.4	**1.912**		
2	29.7	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.2/131.0/137.8	**1.581**	0.0548	67.11%
3	33.0	112.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.718**		
4	33.0	102.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.2/130.8/137.6	**1.765**	0.0011	1.35%
5	33.0	107.0	121.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.739**		
6	33.0	107.0	109.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.743**	0.0000	0.01%
7	33.0	107.0	115.0	41.3	160.0	57.0	70.0	12.5	620.0	100.0	145	520.2/107.0/113.8	**1.814**		
8	33.0	107.0	115.0	24.8	160.0	57.0	70.0	12.5	620.0	100.0	145	528.0/125.4/132.2	**1.700**	0.0065	7.96%
9	33.0	107.0	115.0	33.0	400.0	57.0	70.0	12.5	620.0	100.0	145	524.2/120.2/127.0	**1.787**		
10	33.0	107.0	115.0	33.0	80.0	57.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.730**	0.0016	1.99%
11	33.0	107.0	115.0	33.0	160.0	63.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.748**		
12	33.0	107.0	115.0	33.0	160.0	51.0	70.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.734**	0.0001	0.12%
13	33.0	107.0	115.0	33.0	160.0	57.0	74.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.750**		
14	33.0	107.0	115.0	33.0	160.0	57.0	64.0	12.5	620.0	100.0	145	527.2/130.6/137.4	**1.726**	0.0003	0.35%
15	33.0	107.0	115.0	33.0	160.0	57.0	70.0	15.6	620.0	100.0	145	527.2/130.6/137.4	**1.763**		
16	33.0	107.0	115.0	33.0	160.0	57.0	70.0	9.4	620.0	100.0	145	527.2/130.6/137.4	**1.719**	0.0010	1.19%
17	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	1020	100.0	145	526.8/134.0/140.8	**1.857**		
18	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	380	100.0	145	520.6/108.2/115.0	**1.687**	0.0145	17.70%
19	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	105.0	145	527.2/130.6/137.4	**1.763**		
20	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	95.0	145	527.2/130.6/137.4	**1.718**	0.0010	1.24%
21	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	165	527.2/130.6/137.4	**1.717**		
22	33.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	125	527.2/131.0/137.8	**1.757**	0.0008	0.98%
													Sum:	0.0816	100.00%

* Relates to channge in piezometric surfaces due to influence of gravel pocket in embankment.

E[FS] =	**1.741**	E[ln(FS)] =	0.5412
Var[FS] =	0.0816	Var[ln(FS)] =	0.0266
$_s$ **[FS] =**	**0.2857**	σ[ln(FS)] =	0.1630
V [FS] =	0.1641	V [ln(FS)] =	0.3012

$_b$ = **3.320**
Pr(f) = 4.51E-04
1 in **2219**

Adjusted probabilities based on length effect:

Total Length =	3,500
"Link" Length =	1500
No. Links =	2.33

Reliability of Section, R = 9.99E-01
Adjusted Pr(f) = **1.05E-03**
1 in **951**

L6-24

Probabilistic Slope Stability Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	UTEXAS, Spencer's Method	**Lake Level:**	**24.0**
Probabilistic Model:	Taylor's Series, Finite Difference	**Tailwater:**	**10.0**
Performance Function:	FS		
Random Variable			
Distribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater normal		
		Failure surface tangent to first rock layer	

Run	Sands θ	Silts & Clays C	Gravel Pocket	Failure Arc X, Y, R	**Computed FS**	Prob Calcs Var	Prob Calcs %
E	33.0	620	145	503.2/68.0/72.5	**1.682**		
1	36.3	620	145	507.6/48.4/44.7	**1.792**		
2	29.7	620	145	503.0/63.0/68.7	**1.548**	0.0298	35.89%
17	33.0	1020	145	499.0/83.2/79.5	**1.776**		
18	33.0	380	145	495.2/58.4/60.1	**1.453**	0.0522	62.89%
21	33.0	620	165	508.4/48.4/44.7	**1.657**		
22	33.0	620	125	503.2/68.8/73.1	**1.702**	0.0010	1.22%
					Sum:	0.0829	100.00%

Since Pf is low, can interpolate for lake el. = 21.

Interpolation for el. 21

18.5		1.05E-03	
21		2.13E-03	
24		3.43E-03	

E[FS] =	**1.682**	E[ln(FS)] =	0.5055
Var[FS] =	0.0829	Var[ln(FS)] =	0.0289
s **[FS] =**	**0.2880**	σ[ln(FS)] =	0.1700
V [FS] =	0.1712	V [ln(FS)] =	0.3363

b **=**	**2.974**
Pr(f) =	1.47E-03
1 in	**680**

Adjusted probabilities based on length effect:	**Total Length =**	3,500	Reliability of Section, R = 9.97E-01
	"Link" Length =	1500	**Adjusted Pr(f) = 3.43E-03**
	No. Links =	2.33	**1 in 292**

8.10 Model For Line 9

8.10.1. Location and Range of Applicability

Line 9 is located approximately 1100 feet south of culvert C-12A. This section is representative of approximately 9000 feet of embankment, lines 7 through 12 . This reach is part of the original levee system designated as levee LD-2. This site is located at old station 1129+80. During the high water event of 1995, seepage and movement of material was noted emerging from the side slope of the drainage ditch and the ditch bottom appeared to be in a quick or fluidized condition. .

8.10.2. Embankment Evaluation.

8.10.2.1 General**:** This reach of the Herbert Hoover Dike system was constructed by hydraulic dredge in 1935. As-build drawings show that the original embankment was constructed to elevation 35 feet, with a crest wide of 10 feet. The interiors slope were 1V on 6H and the exterior slope were 1V on 4H. The embankment was constructed on an existing peat layer up to 5 foot thick. In order to prevent peat fires, a fire toe trenches was excavated at the exterior toe through the peat and muck layers to the sand and limestone layers below. The trench was back filled with sand. In 1965, the embankment within this reach was raised to the landside of the existing embankment by hydraulic dredging and castings from deep borrow pits from within the lake bottom. The embankment was raised to a crest elevation of 38 feet with a crest width of 10 feet. The interior slope is 1V on 6H and the exterior slope is 1V on 4H. A 25 foot wide barm, elevation 8 feet, extends from the land side toe to the top of a deep drainage ditch. Natural ground elevation within this reach range from approximate elevation 8 to 9 feet. The bottom of the drainage ditch is elevation 4 feet. This is lowest natural elevation around the lake.

8.10.2.2. Subsurface Investigation. The subsurface investigation within this reach consist of evaluating borings taken in the 1960, prior to the raising the existing embankment, boring obtained during the dike improvements between 1983 and 1988 and from core borings drilled in 1995.

The 1995 boring for this reach, noted as core boring lines C-7, C-8, C-9 and C-10, were drilled on 3000-foot centers. At this site, the line of borings consists of four core borings. A boring on the interior berm to elevation –40 feet. A boring through the crest to elevation to –22.5 feet, one on the berm at the land side toe to elevation –46 feet and a core boring 150 feet from the land side toe to elevation –32 feet. See Figure H-8.5 for core boring layout.

The borings obtained in the 1960's and boring for the dike improvement in the 80's were drilled on the interior beam of the embankment.

A line of piezometers was installed approximately 560 feet south of line 9. The line of piezometers consist of three piezometers installed through the crest, three piezometers installed at the land side toe and three piezometers installed 50 feet form the landside toe across the drainage ditch. Each piezometer well screen was set at different elevations. Recharge tests were performed in each of the piezometer and in the core boring drilled at the landside toe.

The subsurface investigation in this reach revealed several potential problems:
(1) Through seepage due wet slopes on the landside berm and the piping of materials from the side slope of the drainage ditch during the high water event;
(2) A gravel layer which runs from the lake side to the middle of the embankment thus shorting the seepage path;
(3) Under Seepage: Piezometer readings indicate that peat and silt has
heave and hydrofracture the peat and silt layers. A combination of excess head and seepage can create sand boils and subsurface erosion that may culminate in formation of piping beneath the embankment.

8.10.2.3. Materials Encountered. The geology at this section is similar to the geology for approximately 9000-foot reach. The embankment material consists of fine to coarse quartz and shell sand. A layer of shell and limestone gravel extends from elevation 12.8 to elevation 29 feet with interbedded layers of quartz and shell sand. Below the gravel, is layer of peat to elevation 4.8 feet. Below the peat, is clay/silt layer with interbedded with shell sand layers. Under lying this layer is a thin layer of shell sand overlying a vary porous limestone layer Top of the limestone is elevation –9 feet. Below the limestone are interbedded layers of silty shell sand and shell gravel.

8.10.3. Analysis.

8.10.3.1. General. The embankment was analyzed for slope stability, through seepage and underseepage. Permeability of the various aquifer layers used for this section was determined from laboratory and field test data and was calibrated to match local conditions as determined from piezometers readings. The software program "Fastseep" was utilized to analyze this section for through seepage and under seepage for lake levels of 15 feet, 17.5 feet, 21 feet, and 25 feet and for different tailwater levels. The software program "Utexas3" was utilized to analyze slope stability for lake levels 17.5 feet, and 25 feet. Tailwater elevations were based on highest expected value and on field measurements taken during the 1996 high water even. Pore water pressure and phreatic surfaces were imported from Fastseep.

8.10.3.2. Selection of Critical Gradient.

8.10.3.2.1 Selection of Variables and Values. Shear strengths values and possible range of values used for this section were based on correlation between standard penetration "n" values and laboratory test results on similar materials. Permeability's values and possible range of values for the various aquifer layers were based on laboratory and field test data, recharge tests and were calibrated to match local conditions for this site.

8.10.3.2.2 Topographic Features. Topographic features significant to this model are (1) The deep landside drainage ditch at elevation 4 feet; (2) A shell gravel layer embankment, (3) Deep borrow canal to elevation –3 feet at the lake side toe. This will allow water

access to the highly porous limestone layers: and (4) A fire toe which is a deep 12-foot wide trench excavated through the peat and muck to the limestone and sand layers at the toe of the 1935 embankment.

8.10.3.2.3 Geologic Features. Three geologic features were selected as being significant to this model. The filtercake was significant in that it serves as an "entrance valve" into the limestone and sands below the confining layer. Although the permeability of the material is uniform, gaps in the coverage and layer thickness can account for variations in the permeability of the unit itself. Therefore, the permeability of this material was adjusted to assist in calibrating the model to the piezometer readings. The permeability assign to the fill material in the fire toe trench was varied in order to calibrate the model. The interbedded layers of shell gravel and sand in the embankment. Borings through the embankment indicated the presence of coarse material. The extent of the pocket was assumed a maximum of the width of the original embankment The interbedded limestone and shell sand layers that intercepted the drainage ditch side slopes. Piezometers readings indicated that bottom of the drainage ditch have hydrofracture.

8.10.3.2.4. Slope Stability Variables. Variables and values selected for slope stability were divided into two categories, those with normal distributions and those with lognormal distributions as shown below for the embankment and upper foundation materials. The shear strength values for the limestone and lower sand layers were assume to be constant. For the limestone, R strengths $\phi = 0^o$ and c= 2000 psf and S strengths $\phi = 45^o$ and c= 0 psf. The unit weight for the limestone 145 pcf. For the lower sand, R strengths $\phi = 35^o$ and c= 0 psf with unit weight of 1250 psf was used.

Table H-8.10. Normally Distributed variables.

	Shear Strength Summary for Line 9				
		All values in degrees.			
	Q	q	Normal Parameters		
Material	Min / Max	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb, R		0	0	0	0
Emb, S		30.0	3.00	33.0	27.0
Upper PT/OL/OH, R		17.0	4.7	21.7	12.3
Upper PT/OL/OH, S		33.0	8.25	41.3	24.8
Lower PT/OL/OH, R		17.0	4.5	21.5	12.5

Lower PT/OL/OH, S		28.0	7.0	35.0	21.0
ML, R		14.0	3.5	17.5	10.5
ML, S		39.6	8.3	47.9	31.3
CL , S		12.6	3.1	15.7	9.5
CL , R		32.0	8.3	40.3	23.7

Table H-8.11. Lognormal Distributed variables.

	Shear Strength Summary for Line 9			
	All values in psf.			
		Lognormal Parameters		
Material	C	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Emb, R				
Emb, S		no variation, not a variable		
Upper PT/OL/OH, R	440	311	715	135
Upper PT/OL/OH, S	240	170	390	74
Lower PT/OL/OH, R	200	140	326	60
Lower PT/OL/OH, S	0	0	0	0
ML, R	140	100	195	44
ML, S	60	40	100	20
				383
CL, R	250	215	345	135
CL, S	0	0	0	0

	Material Unit Weights – Saturated (pcf)			
Material	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]

Emb.	115.0	5.75	120.8	109.3
Upper PT/OL/OH	70.0	3.50	73.5	66.5
Lower PT/OL/OH,	100.0	5.00	105.0	95.0
ML	120	5	125	115
Lower SM	125	6.75	131.75	118.25
CL	95	4.75	99.75	90.25

8.10.3.2.5. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability was lognormal distributed.

	Permeability Summary for Line 9				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Sands	2.00E-03	2.00E-03	1.41E-03	3.25E-03	6.15E-04
Gravel	1.00E-00	1.00E-00	7.07E-01	1.63E-00	3.08E-1
Peat	2.00E-04	n/a	1.60E-04	3.12E-04	8.20E-05
	n/a	6.00E-05	4.61E-05	9.53E-05	2.23E-05
Silts & Clays	1.50E-04	n/a	1.27E-04	2.27E-04	7.09E-05
	n/a	1.10E-04	7.11E-05	1.81E-04	2.79E-05
Rock	2.50E-01	n/a	1.77E-01	4.06E-01	7.69E-02
	n/a	2.00E-02	1.41E-02	3.25E-02	6.15E-03
Lower sand	6.00E-03	6.00E-03	4.24E-03	9.75E-03	1.85E-03
Filtercake	3.00E-05	3.00E-05	2.12E-05	4.88E-05	9.23E-06

Pipe	5.00E+00	5.00E+00	3.54E+00	8.13E+00	1.54E+00

8.10.3.2.6. Selection of Tailwater Distribution. The water level in the drainage ditch is control by pumps located at culvert C-12A. These pumps are operated by the local farmers and are used to lower the ground water in their fields. During periods of flooding or high water levels in the lake, the farmers are trying to lower the water in their field by lowering the water in the drainage ditches. The lowest observed water level during the 1995 high water event was elevation 4 or the ditch was pumped dry.

8.10.3.2.7. Tailwater Distributions for Through Seepage and Slope Stability Analyses. For slope stability analysis, a high tail water level was more critical. A tail water of 8 feet was used for all lake levels. For through seepage the tail water level was varied as shown in Table H-8.12. This is a relatively wide range of values for this system based upon probable minimum and maximum values as well as observed water levels and levels inferred from piezometer readings. For these two failure modes, a higher tailwater would be more critical since the corresponding phreatic surface in the upper saturated zone would be higher. This range of values was assumed to be normally distributed.

8.10.3.2.8. Tailwater Distribution for Under Seepage Analyses. Unlike the through seepage or slope stability failure modes, underseepage was more critical with lower tailwater elevations. Since the tailwater is control by local farmers and can be artificially manipulated, a water level close to “normal” was selected for all analyses. Based on all existing data, elevation 5 was selected as the “normal” water elevation. Although other tailwater levels were evaluated, this scenario (artificial manipulation) was selected as critical since the Corps does not control tailwater elevations.

Table H-8.12.

	Tailwater Elevations for Model at Line 9 (ft)				
Lake El.	Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
15	5	4/6	0.33	5.9	4.1
17.5	5	4/6	0.9	5.9	4.1
21	6.5	5/8	1.0	7.9	5.5
25	7.9	6.5/9.5	1.5	9.4	6.5

8.10.4. Selection of Critical Gradients**.** Seepage for the both horizon and vertical exit gradients are sensitive to the tailwater level.

8.10.4.1. Underseepage. The critical or vertical gradient "i_{cv}" used for underseepage was based on (1) The hydrostatic pressure below clay and peat layer high enough to cause heaving in these layers and hydrofracture these layers, (2) The critical gradient of the sand layer below the clay and peat layer and (3) The gradient required to cause the formation of a pipe beneath the embankment.

(1) The Piezometers readings at this site indicate that the bottom of the drainage ditch has heaved and hydrofracture has taken place forming a connection between the bottom of the ditch and the shell and limestone layers below.

The critical gradient for the sand layer below the clay layer is .75 in accordance with EM-1110-2-1901 "Seepage Analysis and Control for Dams" and the Lower Mississippi Method.
This is the gradient required to start piping. Based on H/F analysis (Kenny & Lau) the silty, shell sand foundation material is highly unstable and highly erodible. The gradient required to breach the embankment has to be increased enough to drive the pipe past the half way point, l /L > 40 - 50 %, where l is pipe length and L is total pipe path length. In order to do this, the gradient required for breach should be approximately 15% greater than that required to start piping. See the analysis for horizontal gradient as presented for line 25 for the method used to determine required increase for the critical gradient. By applying this 15% increase, the critical gradient is computed be as shown be low.

Average Value: 0.86
Std. Dev.: 0.08
High Value: 0.94
Low Value: 0.78

8.10.4.2. Through Seepage. The gradient required to cause the highly erodible embankment material to pipe is shown below.
The seepage path length extends from the middle of the embankment to the landside toe, approximate 68.6 meters.

L/1000d= 34.3 d_{70} of 2mm particle size 70 percent finer
L= total seepage path of 68.62 meters

Based on Sellmeijer paper,
H/L= .19 Therefore H= 43@ hump

Therefore, i_c should be .50

Average Value: 0.50
Std. Dev.: 0.05
High Value: 0.55
Low Value: 0.45

8.10.5. Failure Mechanisms. There were two primary failure mechanisms considered for seepage and slope stability.

8.10.5.1. Through seepage. The critical gradient to start the piping was back calculated and this gradient was increased by 15%. This was used as the critical required in order to drive the pipe to the halfway point. This implicitly assumes the pipe will propagate to completion (greater than l/L = 50%) in a stable material. In all likelihood, the pipe would seek the path of least resistance and propagate to a part of the gravel pocket where the critical gradient would be lower. This would require an evaluation similar to the two-part evaluation at the Line 25 model. Limited resources did not permit this evaluation. Realize however, that the probability of breach would be higher than currently shown. The critical gradient would need to be increased only slightly (perhaps 5% instead of 15%) to have the pipe progress the l/L = 20% to get to the gravel pocket and the critical gradient in the unstable gravel is much less than that in the stable sand.

8.10.5.2. Under seepage. The piping will begin as pressure head under the clay/peat layer in the bottom of the drainage ditch increases to a point which will heave and fractures the peat/clay layers and the critical gradient for the sand layer below the peat/clay is met. Piezometers readings indicate that during the high water event artesian pressure was present in the limestone layer.

8.10.5.3. Slope Stability. Evaluating the stability of existing slopes involves a number of difficult choices and the results are often subject to variety of interoperations. Slope failure leading to a breach was assume to occur when the shear failure arc involves approximately one third to one half of the landside embankment. Once this occurs, the shear zones along the failure surface will insects the potential seepage pipe path thus shorting the seepage path and increases the exit gradient.

8.10.6 Model Calibration and Analysis Results

8.10.6.1. Model Calibration for Seepage Analyses. Once the appropriate geological features were selected for a lake level of 17.5 feet. The model was calibrated to match piezometer readings using expected values, not log of expected values. The permeability of the fine material on the lake side slope was “calibrate” as an entrance valve so pressure head under the confining layer agreed with the actual piezometer readings. Pipe lengths were also adjusted to obtain agreement with piezometer readings. Although one could argue for calibration by adjusting the “entrance valve”, this would imply a no flow condition. Piezometer readings indicate that the bottom of the drainage had already hydrofracture.

8.10.6.2. Slope Stability Analyses. The search for the global critical failure arc consisted of examining surfaces tangent to the top of the rock layers and peat/clay layers for lake level 17.5 and 25 feet. Tailwater elevation was assumed 5 feet. Pore pressures and phreatic surface as determine by fastseep was used. The failure arcs selected and identified as critical were arcs which involves approximately one third to one half of the landside embankment with the lowest factor of safety. Only two lake levels were selected to be analysis. For lake level 17.5, the FS varied from ****with an average FS of ***and with a bata b of ***. For lake level of 25 feet, the FS varied from *** with an average FS of *** and with a bata b of ***

8.10.7. Length Effects: For this model, a link length of 1500 feet was selected The distance between core boring lines was 3000 feet. It was assumed that this would represent the geology about half way between the two boring lines.

8.10.8. Pup Curve Developments: Table H-8.13 is a summary of the analyses performed on this model. Relevant information for each analysis was listed to show the progression of values with Lake Level.

8.10.9 Mode of Failure

8.10.9.1 Under Seepage. For Lake Elevations of 21 feet or above, this will be the most likely form of failure. The adjusted Pr(f) of 1.00E+00

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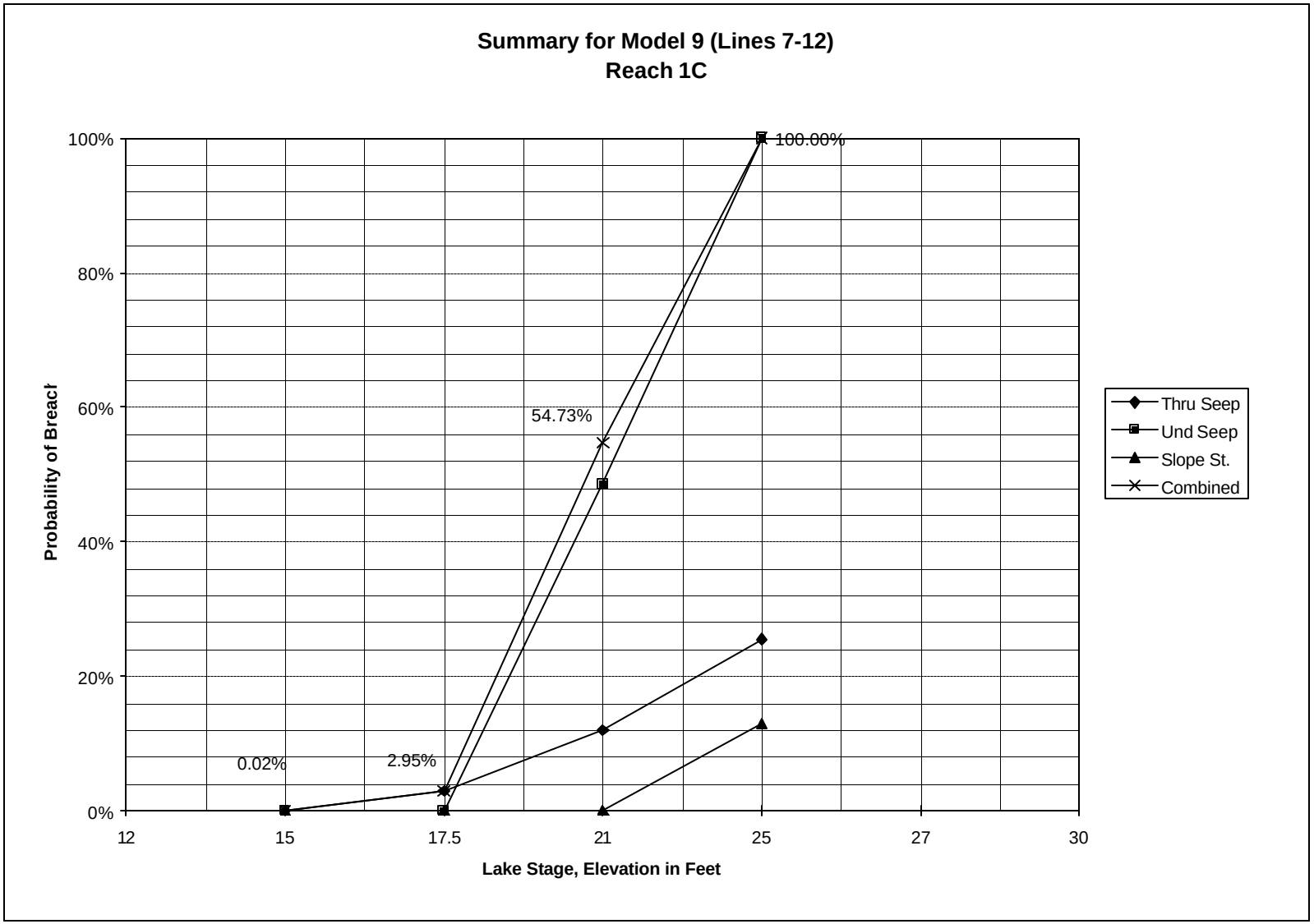
8.10.9.2. Through Seepage. This mode failure was difficult to evaluate since seepage would exit the slope for some variable combinations and not for other combinations. Seepage and movement of materials were noted during the high water event. However, analysis showed that a much higher gradient will be required to drive the pipe to the half way point Once seepage exited the slope, the horizontal exit gradient was compared to the horizontal critical gradient. Based on the analysis, for lake level the adjusted Pr(f) lake level was 7.48E-01

8.10.9.3 Slope Stability. Lake Elevations of 17.5 and 25 feet were analyzed. The probability value for Lake Elevation 21 was interpolated. Slope stability would be the least likely mode of failure. For a lake level of 25 feet, the factor of safety is 1.84.

The resulting probabilities for through seepage, under seepage and slope stability are shown in Tables H-8.13.

8.10.10. Probabilistic Analyses Computation Sheets and Sections.

Line 9 Summary



915.XLS Ditch Slope, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line 9**

Deterministic Model: FASTSEEP **LAKE AT 15'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Ditch Slope, ih

	Upper	GRAVEL	ML		Rock		Lower	Ditch			Results		Prob Calcs	
Run	Sands		Horiz	Vert	Horiz.	Vert.	Sands	Tailwater		i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**7.07E-01**	**1.27E-05**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**5.0**		**0.86**	**0.041**	**20.748**		
3	1.41E-03	**1.63E+00**	1.27E-05	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**		**0.86**	**0.039**	**22.131**		
4	1.41E-03	**3.08E-01**	1.27E-05	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**		**0.86**	**0.041**	**20.748**	0.4783	1.35%
7	1.41E-03	7.07E-01	**2.27E-04**	**7.09E-05**	1.77E-01	1.41E-02	4.24E-03	**5.0**		**0.86**	**0.039**	**22.131**		
8	1.41E-03	7.07E-01	**1.81E-04**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	**5.0**		**0.86**	**0.034**	**25.535**	2.8980	8.21%
11	1.41E-03	7.07E-01	1.27E-05	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	**5.0**		**0.86**	**0.031**	**27.663**		
12	1.41E-03	7.07E-01	1.27E-05	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	**5.0**		**0.86**	**0.047**	**18.442**	21.2572	60.22%
13	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	**5.0**		**0.86**	**0.036**	**23.711**		
14	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	**5.0**		**0.86**	**0.047**	**18.442**	6.9411	19.66%
15	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.8**		**0.86**	**0.026**	**33.196**		
16	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.5**		**0.86**	**0.026**	**33.196**	0.0000	0.00%
17	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**		**0.94**	**0.041**	**22.678**		
18	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**		**0.78**	**0.041**	**18.818**	3.7249	10.55%
												Sum:	35.2996	100.00%

E[FS] =	**20.748**	E[ln(FS)] =	2.9930
Var[FS] =	35.2996	Var[ln(FS)] =	0.0788
s **[FS] =**	**5.9413**	σ[ln(FS)] =	0.2807
V [FS] =	0.2864	V [ln(FS)] =	0.0938

b **=**	**10.661**		
Pr(f) =	**0.00E+00**		
1 in	**#DIV/0!**		

Adjusted probabilities based on length effect:

Total Length =	**9,000**	eliability of Section, R =	1.00E+00
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**0.00E+00**
No. Links =	**3**	**1 in**	**#DIV/0!**

LEGEND FOR COLOR CROSS SECTIONS

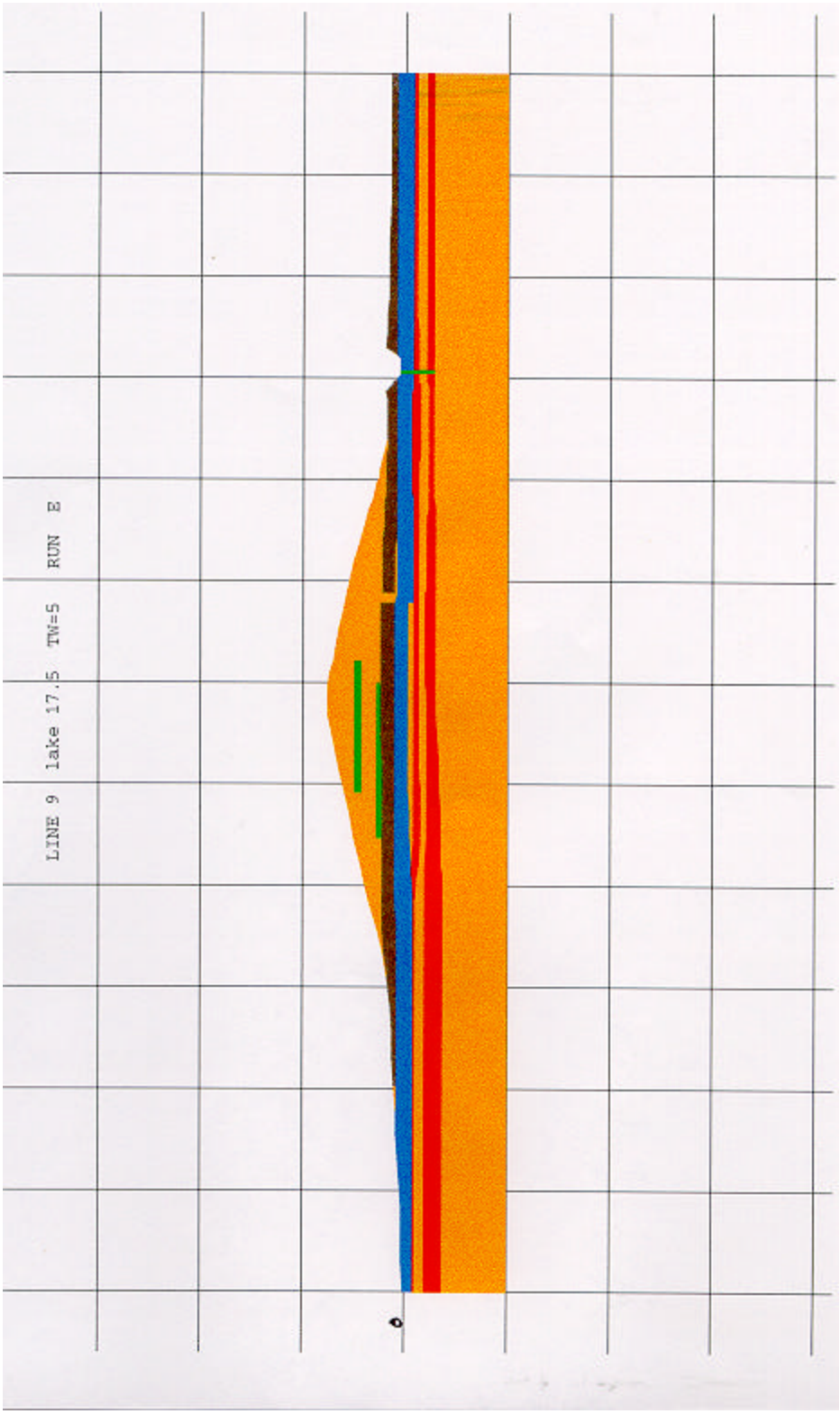
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

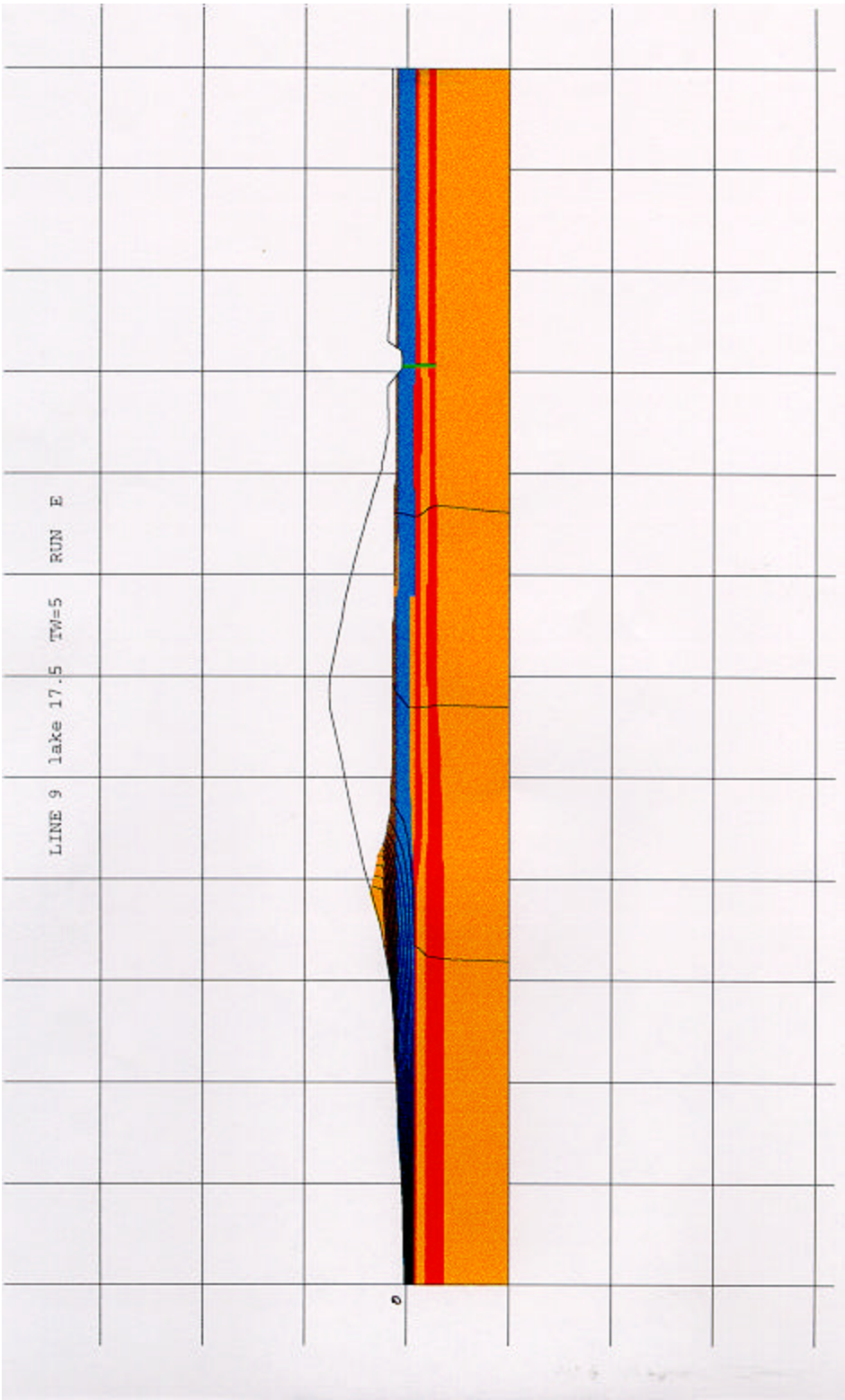
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

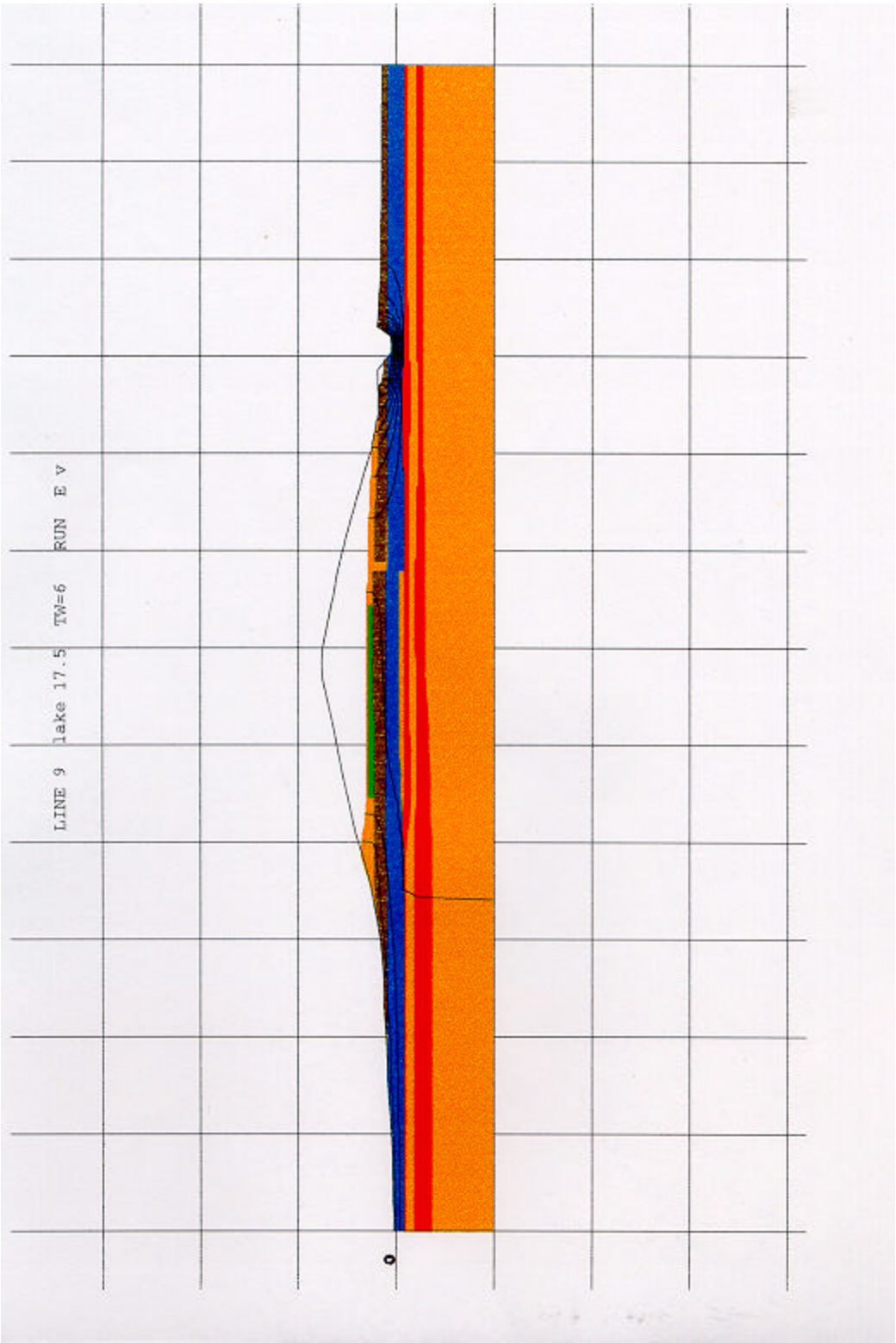
Line 9 lake 17.5 TW=5 RUN E



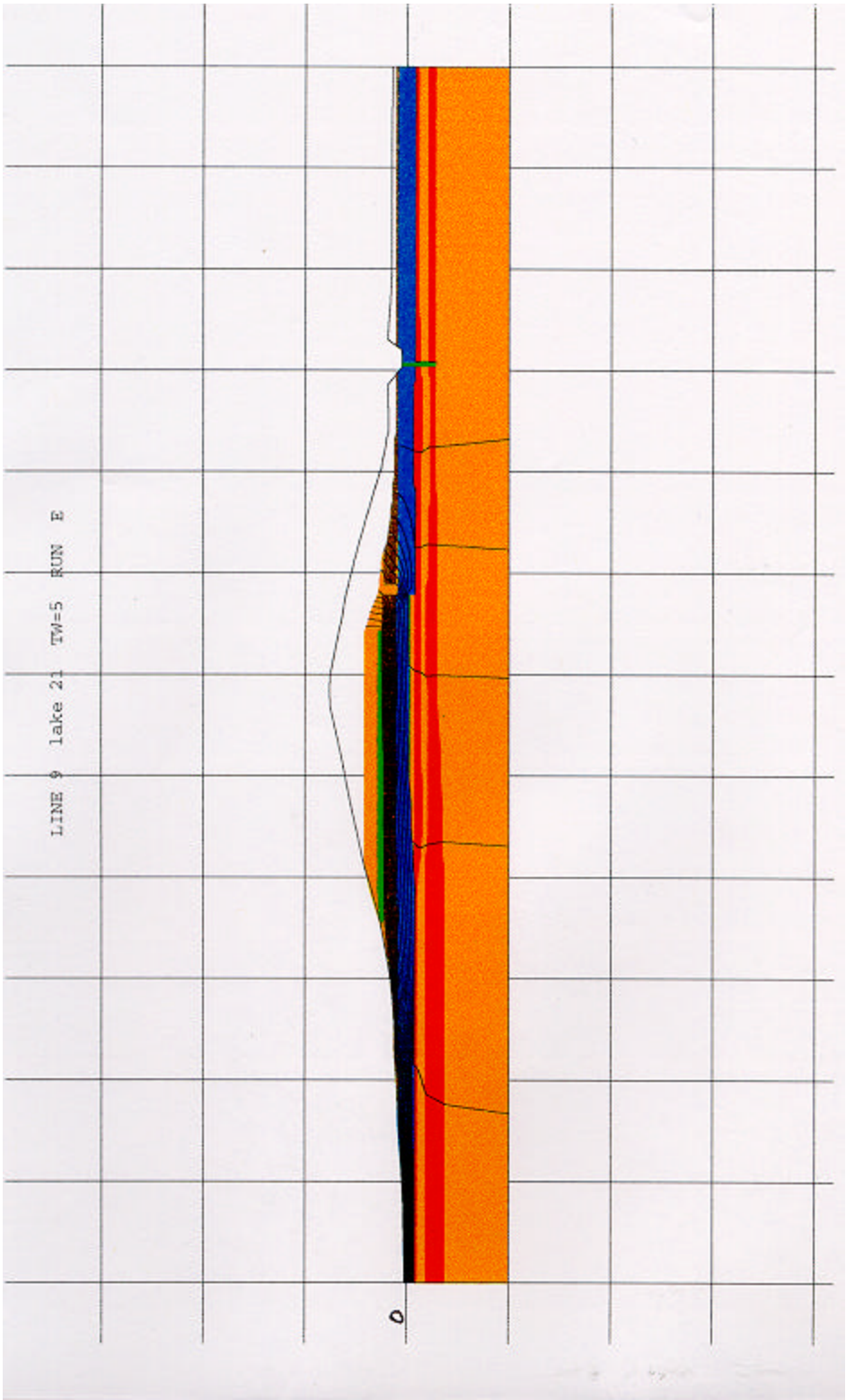
Line 9 lake 17.5 TW=5 RUN E



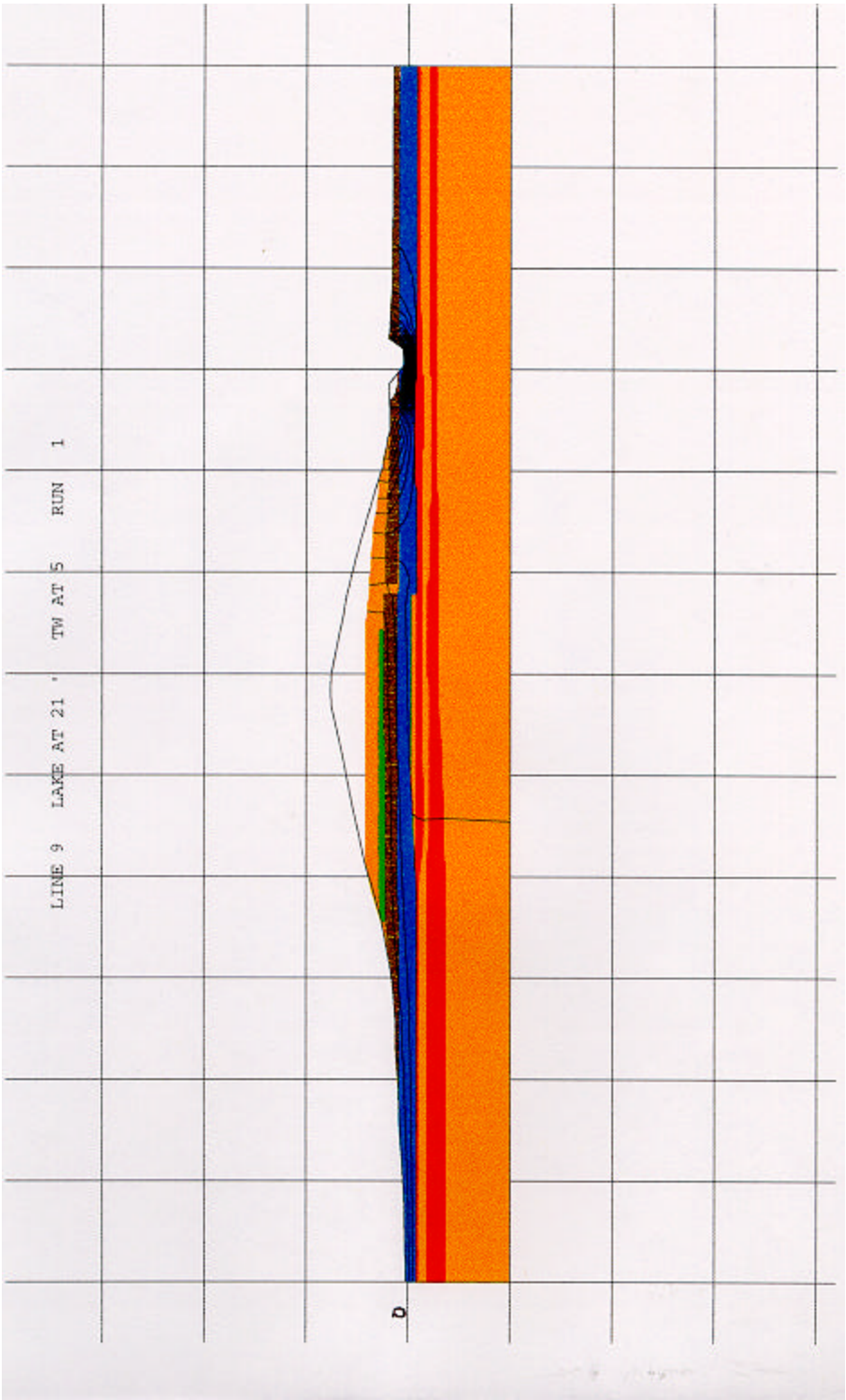
Line 9 lake 17.5 TW=6 RUN E v



Line 9 lake 21 TW=5 RUN E



Line 9 Lake at 21 TW at 5 RUN 1



C921.XLS, Ditch Bottom iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line 9**

Deterministic Model: FASTSEEP **LAKE AT 21'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	Upper		Peat		ML		Clay		LIMESTONE		Lower	Ditch		**Ditch Slope, iv** Results		Prob Calcs	
Run	Sands	Gravel	Horiz.	Vert.	Horiz	Vert	Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	I exit	**FS**	Var	%
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-05**	**7.11E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**5.0**	**0.86**	**2.012**	**0.428**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.022**	**0.425**		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.018**	**0.426**	0.0000	0.00%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.018**	**0.426**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.013**	**0.427**	0.0000	0.00%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.037**	**0.422**		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**1.992**	**0.432**	0.0000	0.40%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**7.09E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.000**	**0.430**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**1.81E-04**	**2.79E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.270**	**0.379**	0.0007	11.52%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.218**	**0.388**		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**1.790**	**0.480**	0.0022	37.92%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	**5.0**	**0.86**	**2.090**	**0.411**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	**5.0**	**0.86**	**1.902**	**0.452**	0.0004	7.32%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	**5.0**	**0.86**	**2.007**	**0.429**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	**5.0**	**0.86**	**2.025**	**0.425**	0.0000	0.07%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.9**	**0.86**	**1.905**	**0.451**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.0**	**0.86**	**2.187**	**0.393**	0.0008	14.90%
17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.94**	**2.012**	**0.467**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.78**	**2.012**	**0.388**	0.0016	27.87%
															Sum:	0.0057	100.00%

E[FS] =	**0.428**	E[ln(FS)] =	-0.8651
Var[FS] =	0.0057	Var[ln(FS)] =	0.0306
s **[FS] =**	**0.0753**	σ[ln(FS)] =	0.1749
V [FS] =	0.1762	V [ln(FS)] =	-0.2021

		b =	**-4.947**
		Pr(f) =	**1.00E+00**
		1 in	**1**

Adjusted probabilities based on length effect:

Total Length =	**9,000**	Reliability of Section, R =	5.36E-20
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**1.00E+00**
No. Links =	**3**	**1 in**	**1**

C921.XLS, Ditch Slope, lh

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line 9**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

LAKE AT 21'

														Ditch Slope, ih			
	Upper	GRAVEL	Peat		ML		Clay		Rock		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz	Vert	Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-05**	**7.11E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**5.0**	**0.50**	**0.049**	**10.105**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.073**	**6.857**		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.047**	**10.667**	3.6281	5.70%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.044**	**11.294**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.057**	**8.727**	1.6472	2.59%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.057**	**8.727**		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.042**	**12.000**	2.6777	4.21%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**7.09E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.049**	**10.105**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**1.81E-04**	**2.79E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.063**	**8.000**	1.1080	1.74%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.070**	**7.111**		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.031**	**16.000**	19.7531	31.04%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	5.0	0.50	**0.031**	**16.000**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	5.0	0.50	**0.070**	**7.111**	19.7531	31.04%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	5.0	0.50	**0.044**	**11.294**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	5.0	0.50	**0.057**	**8.727**	1.6472	2.59%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.5**	0.50	**0.047**	**10.667**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.5**	0.50	**0.138**	**3.623**	12.4046	19.49%
17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	**0.55**	**0.049**	**11.116**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	**0.45**	**0.049**	**9.095**	1.0212	1.60%
															Sum:	63.6401	100.00%

E[FS] =	**10.105**	E[ln(FS)] =	2.0709		
Var[FS] =	63.6401	Var[ln(FS)] =	0.4844	b =	**2.975**
s **[FS] =**	**7.9775**	σ[ln(FS)] =	0.6960	Pr(f) =	**1.46E-03**
V [FS] =	0.7894	V [ln(FS)] =	0.3361	**1 in**	**683**

Adjusted probabilities based on length effect:

Total Length =	**9,000**	Reliability	of Section, R =	9.96E-01
"Link" Length =	**3000**		**Adjusted Pr(f) =**	**4.38E-03**
No. Links =	**3**		**1 in**	**228**

C925.XLS, Ditch Bottom iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line 9**

Deterministic Model: FASTSEEP

LAKE AT 25'

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Ditch Slope, iv

1	2	3		7		4								Results		
	Upper		Peat		ML		Clay		LIMESTONE		Lower	Ditch			Prob Calcs	
Run	Sands	Gravel	Horiz.	Vert.	Horiz	Vert	Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	FS	Var	%
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-05**	**7.11E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**5.0**	**0.86**	**2.455**	**0.350**		
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.453**	**0.351**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.455**	**0.350**	0.0000	0.00%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**7.09E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.432**	**0.354**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**1.81E-04**	**2.79E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.795**	**0.308**	0.0005	3.42%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.692**	**0.320**		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.86**	**2.098**	**0.410**	0.0020	13.22%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	**5.0**	**0.86**	**2.547**	**0.338**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	**5.0**	**0.86**	**2.300**	**0.374**	0.0003	2.12%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	**5.0**	**0.86**	**2.443**	**0.352**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	**5.0**	**0.86**	**2.462**	**0.349**	0.0000	0.01%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.5**	**0.86**	**1.550**	**0.555**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.5**	**0.86**	**2.525**	**0.341**	0.0115	74.34%
17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.94**	**2.455**	**0.383**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.0**	**0.78**	**2.455**	**0.318**	0.0011	6.88%
															Sum:	0.0154	100.00%

E[FS] =	**0.350**	E[ln(FS)] =	-1.1082
Var[FS] =	0.0154	Var[ln(FS)] =	0.1185
s **[FS] =**	**0.1242**	σ[ln(FS)] =	0.3442
V [FS] =	0.3547	V [ln(FS)] =	-0.3106

b =	**-3.220**
Pr(f) =	**9.99E-01**
1 in	**1**

Adjusted probabilities based on length effect:

Total Length =	**9,000**
"Link" Length =	**3000**
No. Links =	**3**

Reliability of Section, R =	2.65E-10
Adjusted Pr(f) =	**1.00E+00**
1 in	**1**

C925.XLS, Ditch Slope, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line 9**

Deterministic Model: FASTSEEP

LAKE AT 25'

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Ditch Slope, ih

	Upper	GRAVEL	Peat		ML		Clay		Rock		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz	Vert	Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-05**	**7.11E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**5.0**	**0.50**	**0.057**	**8.773**		
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.052**	**9.650**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.067**	**7.423**	1.2398	5.56%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**7.09E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.057**	**8.773**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**1.81E-04**	**2.79E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	0.50	**0.067**	**7.423**	0.4554	2.04%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	5.0	0.50	**0.036**	**13.786**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	5.0	0.50	**0.083**	**6.031**	15.0329	67.43%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	5.0	0.50	**0.049**	**10.158**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	5.0	0.50	**0.065**	**7.720**	1.4858	6.66%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**5.5**	0.50	**0.166**	**3.016**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.5**	0.50	**0.075**	**6.655**	3.3116	14.85%
17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	**0.55**	**0.057**	**9.650**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-05	7.11E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.0	**0.45**	**0.057**	**7.895**	0.7696	3.45%
															Sum:	22.2951	100.00%

E[FS] =	**8.773**	E[ln(FS)] =	0.5975		
Var[FS] =	22.2951	Var[ln(FS)] =	3.1482	b =	**0.337**
s[FS] =	**4.7218**	σ[ln(FS)] =	1.7743	Pr(f) =	**3.68E-01**
V [FS] =	0.5382	V [ln(FS)] =	2.9695	**1 in**	**3**

Adjusted probabilities based on length effect:

Total Length =	**9,000**	Reliability of Section, R =	2.52E-01
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**7.48E-01**
No. Links =	**3**	**1 in**	**1**

C9ss.xls, LINE9 SS L=17.5

Probabilistic Slope Stability Analysis: **LINE 9** **LAKE 17.5 FEET**

Deterministic Model:	UTEXAS3
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

	1			2			3			4			5			6					
	EMB	SP-SM			PEAT			CLAY			ML		LOWER		SAND	Rock		Ditch		Prob Calcs	
Run	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	Tailwater	**FS**	Var	%
E	**32.0000**	**0.00**	**115.00**	**33.00**	**240.00**	**70.00**	**12.50**	**780.00**	**100.00**	**28.00**	**0.00**	**120.00**	**32.00**	**0.00**	**130.00**	**45.00**	**2000.00**	**6.00**	**1.84**		
3	32.0000	0.00	115.00	**41.00**	**480.00**	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	1.83		
4	32.0000	0.00	115.00	**24.75**	**0.00**	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	1.82	0.000025	25.00%
5	32.0000	0.00	115.00	33.00	240.00	70.00	**25.90**	**1140.00**	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	1.83		
6	32.0000	0.00	115.00	33.00	240.00	70.00	**9.60**	**420.00**	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	1.82	0.000025	25.00%
7	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	**34.30**	**200.00**	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	1.83		
8	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	**21.70**	**0.00**	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	1.82	0.000025	25.00%
13	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	**7.00**	1.83		
14	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	**5.00**	1.82	0.000025	25.00%
																			Sum:	0.000100	100.00%

Adjusted probabilities based on length effect:

Total Length =	10,000	Reliability of Section, R =	1.00E+00	**E[FS] =**	**1.8400**
"Link" Length =	3000	**Adjusted Pr(f) =**	**0.00E+00**	Var[FS] =	0.0001
No. Links =	3.33	**1 in**	**#DIV/0!**	$_s$ **[FS] =**	**0.0100**
				V [FS] =	0.0054

E[ln(FS)] =	0.6098		
Var[ln(FS)] =	0.000030	$_b$ **=**	**112.1950**
$_d$[ln(FS)] =	0.0054	Pr(f) =	0.00E+00
V [ln(FS)] =	0.0089	**1 in**	**#DIV/0!**

C9ss.xls, LINE 9 SS L=25

Probabilistic Slope Stability Analysis: **LINE 9** **LAKE 25**

Deterministic Model:	UTEXAS3
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

	1			2			3			4			5			6								
	EMB	SP-SM			PEAT			CLAY			ML		LOWER		SAND	Rock		Ditch	x	y	radius		Prob Calcs	
Run	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	UNIT WGT	ϕ	c	Tailwater				FS	Var	%
E	**32.0000**	**0.00**	**115.00**	**33.00**	**240.00**	**70.00**	**12.50**	**780.00**	**100.00**	**28.00**	**0.00**	**120.00**	**32.00**	**0.00**	**130.00**	**45.00**	**2000.00**	**6.00**			84.90	**1.84**		
3	32.0000	0.00	115.00	**41.00**	**480.00**	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	86.00	81.50	84.50	**2.06**		
4	32.0000	0.00	115.00	**24.75**	**0.00**	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	97.00	69.50	70.00	**1.59**	0.055225	23.26%
5	32.0000	0.00	115.00	33.00	240.00	70.00	**25.90**	**1140.00**	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	88.00	81.90	84.90	**1.84**		
6	32.0000	0.00	115.00	33.00	240.00	70.00	**9.60**	**420.00**	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	82.50	93.60	96.60	**1.83**	0.000025	0.01%
7	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	**34.30**	**200.00**	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	78.40	107.40	110.40	**2.43**		
8	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	**21.70**	**0.00**	120.00	32.00	0.00	130.00	45.00	2000.00	6.00	86.10	76.00	79.00	**1.58**	0.180625	76.06%
13	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	**7.00**	92.70	70.00	73.00	**1.76**		
14	32.0000	0.00	115.00	33.00	240.00	70.00	12.50	780.00	100.00	28.00	0.00	120.00	32.00	0.00	130.00	45.00	2000.00	**5.00**				**1.84**	0.001600	0.67%
																						Sum:	0.237475	100.00%

Adjusted probabilities based on length effect:

Total Length =	10,000	Reliability of Section, R =	9.56E-01
"Link" Length =	3000	**Adjusted Pr(f) =**	**4.43E-02**
No. Links =	3.33	**1 in**	**23**

E[FS] =	**1.8400**	E[ln(FS)] =	0.5759	
Var[FS] =	0.2375	Var[ln(FS)] =	0.067792	β = **2.2117**
σ **[FS] =**	**0.4873**	σ[ln(FS)] =	0.2604	Pr(f) = 1.35E-02
V [FS] =	0.2648	V [ln(FS)] =	0.4521	**1 in** **74**

8.11 Model For Line 18

8.11.1. Location and Range of Applicability: Line 18 is located approximately 3800 feet north of Pahokee State Park Marina. This section is representative of approximately 9000 feet of embankment. This reach is part of the original levee embankment system designated as levee D-9 and is near station 185+00. During the high water event of 1995, at the Miller site, which is located 1000 foot north of this line, seepage and movement of material was noted emerging from the side slope of the drainage ditch. About two cubic feet of material was removed.

8.11.2. Embankment Evaluation:

8.11.2.1 General: This reach of the Herbert Hoover Dike system was constructed by hydraulic dredge in 1935. As-build drawings show that the original embankment was constructed to about elevation 35 feet, with a crest wide of 10 feet. The interior slope were 1V on 6H with 1V on 4H exterior slope. The embankment was constructed on an existing peat layer up to 5 foot thick. In order to prevent peat fires, a fire toe trenches was excavated through the peat and muck layers to the sand and limestone layers below at the exterior toe. The trench was back filled with sand. In 1965, the embankment within this reach was raised to the landside of the existing embankment by hydraulic dredging and castings from deep borrow pits from within the lake bottom. The embankment was raised to a crest elevation of 38 feet with a crest width of 12 feet. The interior slope is 1V on 6H and the exterior slope is 1V on 4H. A 25 foot wide barm, elevation 15.9 feet, extends from the land side toe to the top of a deep drainage ditch. Natural ground elevation within this reach is approximate elevation 16.0 feet. The bottom of the drainage ditch is 9.6 feet.

8.11.2.2. Subsurface Investigation. The subsurface investigation within this reach consist of evaluating borings taken in the 1960, prior to the raising the existing embankment, boring obtained during the dike improvements between 1983 and 1988 and from core borings drilled in 1995.

The 1995 boring for this reach noted as core boring lines C-17, C-18 and C-19, were drilled on 3000-foot centers. An additional line of borings was drilled at the Miller site. Each line of borings consists of three core borings. One through the crest to elevation to –21 feet, one at the land side toe to elevation -39 feet and a core boring 100 feet from the land side toe to elevation –28 feet.

The borings obtained in the 1960's and boring for the dike improvement in the 80's were drilled on the interior beam of the embankment.

Two piezometer wells were installed at the Miller site. One well through the crest with two well screens at different tip elevations and one at the land side toe between the toe and the drainage ditch with one well screen. Recharge tests were performed in each of the piezometer and core borings drilled at C-18 at different elevations.

The subsurface investigation in this reach revealed several potential problems: (1) Slope stability problems due to low shear strengths materials encountered at the interface of the embankment and the foundation:

(2) Through seepage due wet slopes and the piping of materials in the drainage ditch side slope during the high water event; (3) A gravel layer which runs from the lake side to the middle of the embankment thus shorting the seepage path; (4) Under Seepage: High hydrostatic pressure in the sand layer under the clay and peat layer could cause these layers to heave and hydrofracture. This could cause the rupture of the clay and peat layers at the weak spots within these layers resulting in a concentration of seepage flow to this area causing sand boils and piping of the sand layer under the clay and peat layer. A combination of excess head and seepage can create sand boils and subsurface erosion that may culminate in formation of piping beneath the structure.

8.11.2.3. Materials Encountered. The geology at this section is similar to the geology in the remainder of this 9000-foot reach. The embankment material consists of a very loose to loose silty, shelly sand with an average blow count. "n" of five. The foundation materials consist of soft clay/silt and peat layers, with interbedded layers of sand/gravel overlying layers of sand and very porous limestone. Low blow count material ("n" from 0 to 2) was encounter in the peat and clay/silt layers at elevation 2 feet. This condition was observed in both the centerline and the land side toe core borings. A seven foot layer of poorly graded shelly gravel extends from the lake side slope to the middle of the original embankment. The bottom of the gravel is elevation is 12 feet. The embankment and foundation condition at this section was considered a " worst case" section because this section had 8 feet of soft material. A highly erodible silty sand layer intersects the slope of the drainage ditch at the land side toe. Seepage and the migration of materials were noted emerging from this layer when the lake elevation was 18.5 feet and the tailwater level was 11.2 feet.

8.11.3. Analysis.

8.11.3.1. General. The embankment was analyzed for slope stability, through seepage and underseepage. Permeability of the various aquifer layers used for this section was determined from laboratory and field test data and was calibrated to match local conditions as determined from piezometers readings. The software program "Fastseep" was utilized to analyze this section for through seepage and under seepage for lake levels of 15 feet, 17.5 feet, 21 feet, and 25 feet and for different tailwater levels. The software program "Utexas3" was utilized to analyze slope stability for lake levels 17.5 feet, and 25 feet. Tailwater elevations were based on highest expected value and on field measurements taken during the 1996 high water even. Pore water pressure and phreatic surfaces were imported from Fastseep.

8.11.3.2. Selection of Critical Gradient.

8.11.3.2.1 Selection of Variables and Values. Shear strengths values and possible range of values used for this section were based on correlation between standard penetration "n" values and laboratory test results on similar materials. Permeability's values and possible range of values for the various aquifer layers were based on laboratory and field test data, recharge tests and were calibrated to match local conditions for this site.

8.11.3.2.2 Topographic Features. Topographic features significant to this model are (1) The deep landside drainage ditch, (2) A gravel layer which extends from the lake side to the approximate middle of the embankment which will allow the water access to middle of the embankment thus shorting the seepage path, (3) A deep borrow pits on the lake side within 300 feet of the toe which will allow water access to the highly porous limestone layers and (4) A fire toe which is a deep trench excavated through the peat and muck to the limestone and sand layers .

8.11.3.2.3 Geologic Features. Three geologic features were selected as being significant to this model. The filtercake was significant in that it serves as an "entrance valve" into the limestone and sands below the confining layer. Although the permeability of the material is uniform, gaps in the coverage and layer thickness can account for variations in the permeability of the unit itself. Therefore, the permeability of this material was adjusted to assist in calibrating the model to the piezometer readings. . The permeability's assign to the fill material in the fire toe trench was varied in order to calibrate the model. The gravel pocket in the embankment was established as a variable. Borings through the embankment indicated the presence of coarse material. The extent of the pocket was assumed a maximum of the width of the original embankment. The geometry of the pocket was included as a variable. Refer to Table H-8.14. The other significant feature was the actual "pipes". The presence and extent of the pipes was inferred from piezometer readings.

8.11.3.2.4. Slope Stability Variables. Variables and values selected for slope stability were divided into two categories, those with normal distributions and those with lognormal distributions as shown below for the embankment and upper foundation materials. The shear strength values for the limestone and lower sand layers were assume to be constant. For the limestone, R strengths f = 0° and c= 2000 psf and S strengths f = 45° and c= 0 psf. The unit weight for the limestone 145 pcf. For the lower sand, R strengths f = 35° and c= 0 psf with unit weight of 1250 psf was used.

Table H-8.14. Normally Distributed variables.

	Shear Strength Summary for Line 18				
		All values in degrees.			
	q	q	Normal Parameters		
Material	Min / Max	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb, R		0	0	0	0
Emb, S		30.0	3.00	33.0	27.0
Upper PT/OL/OH, R		17.0	4.7	21.7	12.3
Upper PT/OL/OH,		33.0	8.25	41.3	24.8

S					
Lower PT/OL/OH, R		17.0	4.5	21.5	12.5
Lower PT/OL/OH, S		28.0	7.0	35.0	21.0
ML, R		14.0	3.5	17.5	10.5
ML, S		39.6	8.3	47.9	31.3
CL , S		12.6	3.1	15.7	9.5
CL , R		32.0	8.3	40.3	23.7

Table H-8.15. Lognormal Distributed variables.

	Shear Strength Summary for Line 18			
	All values in degrees or psf.			
		Lognormal Parameters		
Material	C	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Emb, R				
Emb, S		no variation, not a variable		
Upper PT/OL/OH, R	440	311	715	135
Upper PT/OL/OH, S	240	170	390	74
Lower PT/OL/OH, R	200	140	326	60
Lower PT/OL/OH, S	0	0	0	0
ML, R	140	100	195	44
ML, S	60	40	100	20
				383
CL, R	250	215	345	135
CL, S	0	0	0	0

	Material Unit Weights – Saturated			
Material	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb.	115.0	5.75	120.8	109.3
Upper PT/OL/OH	70.0	3.50	73.5	66.5
Lower PT/OL/OH,	100.0	5.00	105.0	95.0
ML	120	5	125	115
Lower SM	125	6.75	131.75	118.25
CL	95	4.75	99.75	90.25

8.11.3.2.5. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability was lognormal distributed.

	Permeability Summary for Line 18				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Sands	2.00E-03	2.00E-03	1.41E-03	3.25E-03	6.15E-04
Gravel	1.00E-00	1.00E-00	7.07E-01	1.63E-00	3.08E-1
Peat	2.00E-04	n/a	1.60E-04	3.12E-04	8.20E-05
	n/a	6.00E-05	4.61E-05	9.53E-05	2.23E-05
Silts & Clays	1.50E-04	n/a	1.27E-04	2.27E-04	7.09E-05
	n/a	1.10E-04	7.11E-05	1.81E-04	2.79E-05
Rock	2.50E-01	n/a	1.77E-01	4.06E-01	7.69E-02
	n/a	2.00E-02	1.41E-02	3.25E-02	6.15E-03
Lower sand	6.00E-03	6.00E-03	4.24E-03	9.75E-03	1.85E-03

Filtercake	3.00E-05	3.00E-05	2.12E-05	4.88E-05	9.23E-06
Pipe	5.00E+00	5.00E+00	3.54E+00	8.13E+00	1.54E+00

The spatial distribution of the gravel pocked was based both on core boring data and as-built data. The top and bottom elevations of the pocked were estimated from core borings and the width based on as-built topographic data. Table H-8.16 shows the normal distribution assumed for the width of the pocket.

Table H-8.16

Gravel Distribution, Width of Gravel Pocket

Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
145.0	85/205	20.0	165.0	125.0
Transformed to Model Coord.s				
	Center	Lakeside	Landside	Total
Mean [E(x)]	430	-90	55	145
High [E(x) +s]	430	-113	78	165.0
Low [E(x) +s]	430	-67	33	125.0

8.11.3.2.6. Selection of Tailwater Distribution. The tailwater in the drainage ditch is control by the water level in the West Palm Beach Canal at Canal Point. The water level in the ditch is uncontrolled and varies as the water in the canal fluctuates.

8.11.3.2.7. Tailwater Distributions for Through Seepage and Slope Stability Analyses. For slope stability analysis, a high tail water level was more critical. Tail water of 12.5 feet was used for all lake levels. For through seepage the tail water level was varied as shown in Table H-8.17. This is a relatively wide range of values for this system based upon probable minimum and maximum values as well as observed water levels and levels inferred from piezometer readings. For these two failure modes, a higher tailwater would be more critical since the corresponding phreatic surface in the upper saturated zone would be higher. This range of values was assumed to be normally distributed.

8.11.3.2.8. Tailwater Distribution for Under Seepage Analyses. Unlike the through seepage or slope stability failure modes, underseepage was more critical with lower tailwater elevations. Since the tailwater is control by the water in the West Palm Beach Canal and this canal can be artificially manipulated; a water level close to “normal” was selected for all analyses. Based on all existing data, elevation 11.0 was selected as the “normal” water elevation.

Although other tailwater levels were evaluated, this scenario (artificial manipulation) was selected as critical since the Corps does not control tailwater elevations.

Table H-8.17.

	Tailwater Elevations for Model at Line 18				
Lake El.	Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
15	11.0	10/12	0.33	11.3	10.7
17.5	11.2	10.12.2	0.5	11.7	10.7
21	12.2	10/14	0.66	12.9	11.5
24	12.5	10/14	0.66	13.2	11.8

8.11.4. Selection Of Critical Gradients**.** Seepage for the both horizon and vertical exit gradients are sensitive to the tailwater level.

8.11.4.1. Underseepage. The critical or vertical gradient "i_{cv}" used for underseepage was based on (1) the hydrostatic pressure below clay and peat layer high enough to cause heaving in these layers and hydrofracture these layers, (2) the critical gradient of the sand layer below the clay and peat layer and (3) the gradient required to cause the formation of a pipe beneath the embankment.

(1) The total head required to heave the peat and clay layer is shown below

Peat layer 1.4 feet (70/pcf-62.4/pcf) = 10.64 psf
Clay layer 10.2 feet (95/pcf-62.4/pcf) =332.52 psf
Total 343.16 psf

The pressure head required to heave these two layers: 343.16/62.4= 5.5 feet

The critical gradient for the sand layer below the clay layer is .75 in accordance with EM-1110-2-1901 "Seepage Analysis and Control for Dams" and the Lower Mississippi Method.

This is the gradient required to start piping. The gradient required to breach the embankment has to be increased enough to drive the pipe past the half way point, l /L > 40 - 50 %, where l is pipe length and L is total pipe path length. In order to do this, the gradient required for breach should be approximately 15% greater than that required to start piping. See the analysis for horizontal gradient as presented for line 25 for the method used to determine required increase for the critical gradient. By applying this 15% increase, the critical gradient is computed be as shown below.

Average Value: 0.86
Std. Dev.: 0.08
High Value: 0.94
Low Value: 0.78

8.11.4.2. Through Seepage. Based on analysis performed for Lake Level of 18.5 feet and a tailwater of 11.2 feet (the conditions during the 1996 high water event) indicated an exit gradient of .52. This was used as the critical gradient at which the horizontal migration of fines started, i_{exit} was 0.52. In accordance with J. B. Sellmeijer theory and a grain size d_{70} of .2 mm, the critical gradient i_c should be increased to .82 in order to pipe the material and drive the pipe past the half way point.

L/1000d= 34.3 d_{70} of 2mm particle size 70 percent finer
L= total seepage path of 68.62 meters

See Figure H-8.3 (Figure 4).

H/L= .19 Therefore H= 43@ hump
43-18.5= 24.5/43=.57 or the gradient should be increase by 57 percent.

Therefore, i_c should be .82

Average Value: 0.82
Std. Dev.: 0.08
High Value: 0.90
Low Value: 0.74

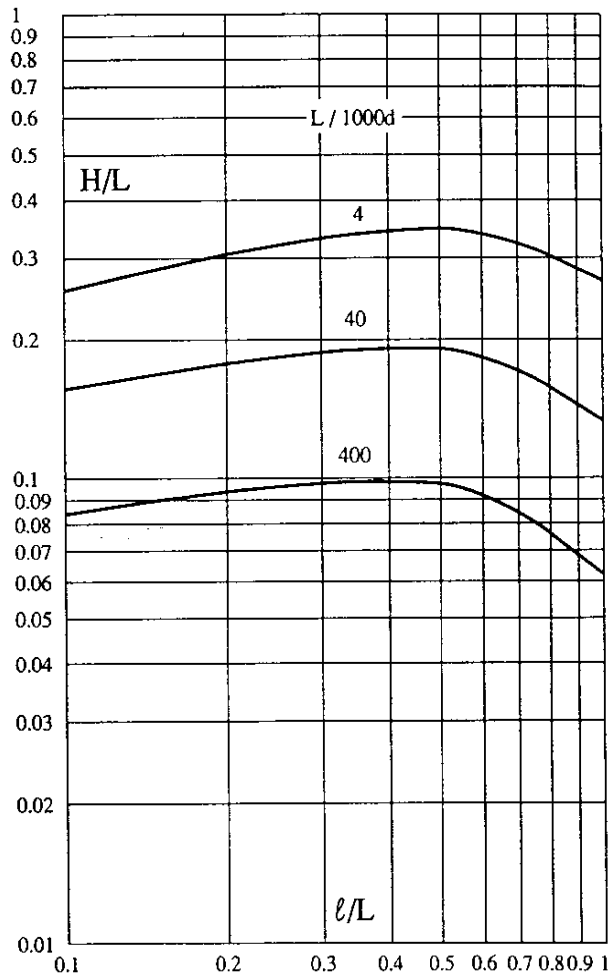


Figure 4. Hydraulic head versus erosion length

Figure H-8.3. From Koenders et al, 1992.

8.11.5. Failure Mechanisms. There were two primary failure mechanisms considered for seepage and slope stability.

8.11.5.1. Through seepage. During the high water event in 1995 seepage and soil was observed piping from the side slope of the landside drainage ditch. The lake was at elevation 18.5 feet and the tailwater was 11.2 feet. Piping started in a fine sand layer sandwiched between a peat and clay layer. The critical gradient to start the piping was back calculated and this gradient was increased by 15%. This was used as the critical required in order to drive the pipe to the halfway point. This implicitly assumes the pipe will propagate to completion (greater than l/L = 50%) in a stable material. In all likelihood, the pipe would seek the path of least resistance and propagate to a part of the gravel pocket where the critical gradient would be lower. This would require an evaluation similar to the two-part evaluation at the Line 25 model. Limited resources did not permit this evaluation. Realize however, that the probability of breach would be higher than currently shown. The critical gradient would need to be increased only slightly (perhaps 5% instead

of 15%) to have the pipe progress the l/L = 20% to get to the gravel pocket and the critical gradient in the unstable gravel is much less than that in the stable sand.

8.11.5.2. Under seepage. The piping will begin as pressure head under the clay/peat layer in the bottom of the drainage ditch increases to a point which will heave and fractures the peat/clay layers and the critical gradient for the sand layer below the peat/clay is met. Piezometers readings indicate that during the high water event artesian pressure was present in the limestone layer.

8.11.5.3. Slope Stability. Evaluating the stability of existing slopes involves a number of difficult choices and the results are often subject to variety of interoperations. Slope failure leading to a breach was assume to occur when the shear failure arc involves approximately one third to one half of the landside embankment. Once this occurs, the shear zones along the failure surface will insects the potential seepage pipe path thus shorting the seepage path and increases the exit gradient.

8.11.6. Model Calibration And Analysis Results.

8.11.6.1. Model Calibration for Seepage Analyses. Once the appropriate geological features were selected, constant head boundary conditions for a lake level of 17.5 feet were applied. The model was calibrated to match piezometer readings using expected values, not log of expected values. The permeability of the fine material on the lake side slope was "calibrate" as an entrance valve so pressure head under the confining layer agreed with the actual piezometer readings. Pipe lengths were also adjusted to obtain agreement with piezometer readings. Although one could argue for calibration by adjusting the "entrance valve", this would imply a no flow condition. Piezometer readings indicate an artesian pressure under the peat in the limestone when the lake was at elevation 18.5 feet.

8.11.6.2. Slope Stability Analyses. The search for the global critical failure arc consisted of examining surfaces tangent to the top of the rock layers and peat/clay layers for lake level 17.5 and 25 feet. Tailwater elevation was assumed 12.5 feet. Pore pressures and phreatic surface as determine by fastseep was used. Although, the slope stability analysis indicator the critical arc had lower factors of safety. The failure arcs selected and identified as critical were arcs which involves approximately one third to one half of the landside embankment with the lowest factor of safety. Only two lake levels were selected to be analysis. For lake level 17.5, the FS varied from 1.14 to 1.32 with an average FS of 1.24 and with a bata b of 1.3. For lake level of 25 feet, the FS varied from 0.80 to 1.13 with an average FS of .99 and with a bata b of 0.008.

8.11.7. Length Effects. For this model, a link length of 1500 feet was selected The distance between core boring lines was 3000 feet. It was assumed that this would represent the geology about half way between the two boring lines.

8.11.8. Pup Curve Development. Table H-8.18 is a summary of the analyses performed on this model. Relevant information for each analysis was listed to show the progression of values with Lake Level.

8.11.9 Mode of Failure.

8.11.9.1 Through Seepage. This mode failure was difficult to evaluate since seepage would exit the slope for some variable combinations and not for other combinations. Seepage and movement of materials were noted during the high water event. However, analysis showed that a much higher gradient will be required to drive the pipe to the half way point Below lake elevation 21 feet, this mode of failure is the most likely. This required a slightly different approach for the Taylor Series computation. Probability distributions were developed for both exit gradient and critical gradient based on computed variance for each since the single probability distribution for Factor of Safety could not accept a "zero" value. The exit and critical gradient distributions were combined to develop a third distribution for Factor of Safety. From this point, computations proceeded as described for other Taylor Series calculations. Once seepage exited the slope, the horizontal exit gradient was compared to the horizontal critical gradient. The resulting probabilities are shown in Table H-8.19.

8.11.9.2. Under Seepage. For Lake Elevations of 18 and 21, underseepage was the least likely mode of failure.

8.11.9.3 Slope Stability. Lake Elevations of 17.5 and 25 feet were analyzed. The probability value for Lake Elevation 21 was interpolated. Above lake elevation slope, stability couple with through seepage will be the most likely mode of failure.

8.11.10. Probabilistic Analyses Computation Sheets and Sections.

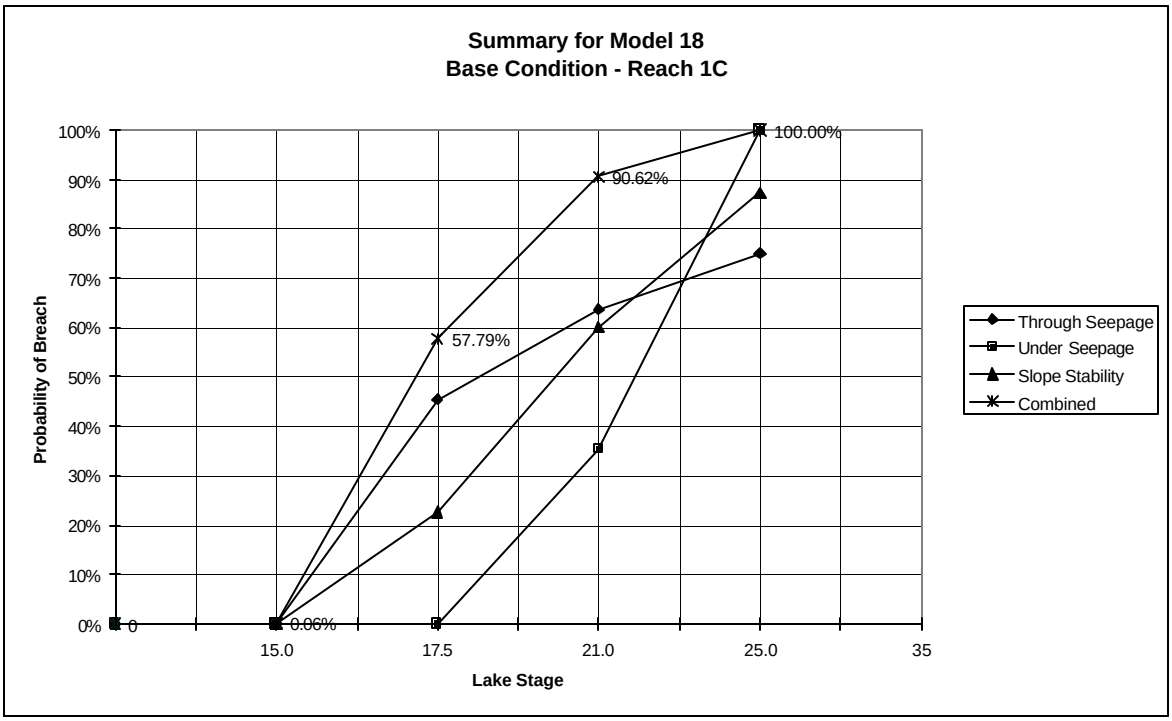
LINE 18 Base Conditions

5/4/96
SRH

	Lake	Through Seepage		Under Seepage		Slope Stability		Judgement		Combined	
		Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
1											
2	15.0	5.77E-04	0.9994	0.00E+00	1.0000	0.00E+00	1.0000		1.0000	0.06%	0.9994
3											
4	17.5	4.54E-01	0.5480	0.00E+00	1.0000	2.27E-01	0.7730		1.0000	57.79%	0.4221
5											
6	21.0	6.37E-01	0.3630	3.54E-01	0.6460	6.00E-01	0.4000		1.0000	90.62%	0.0938
7											
8	25.0	7.50E-01	0.2500	1.00E+00	0.0000	8.73E-01	0.1270		1.0000	100.00%	0.0000
9											
10	35		1.0000		1.0000		1.0000		1.0000		1.0000

Value was extrapolated.

Value was adjusted based on judgment.



LEGEND FOR COLOR CROSS SECTIONS

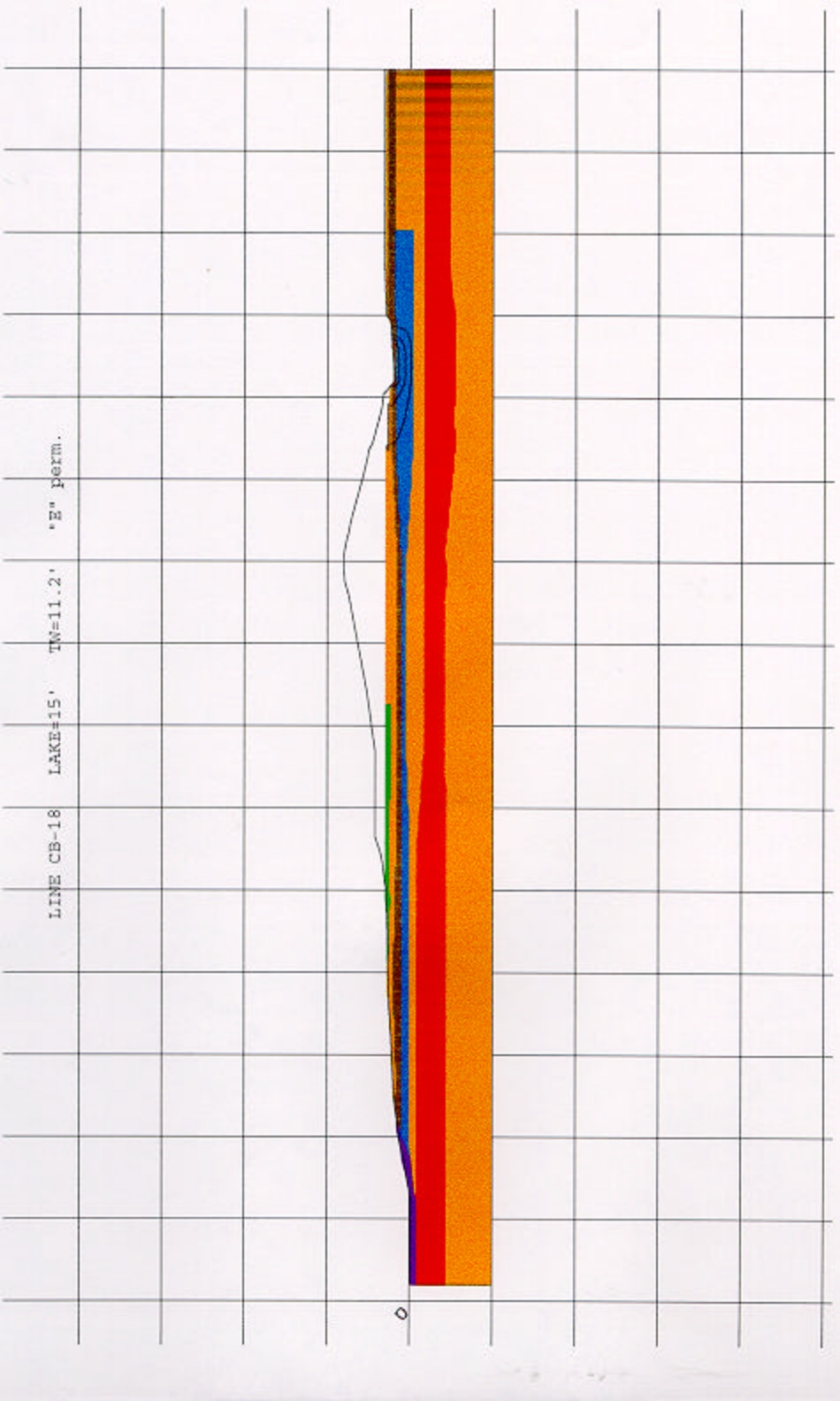
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

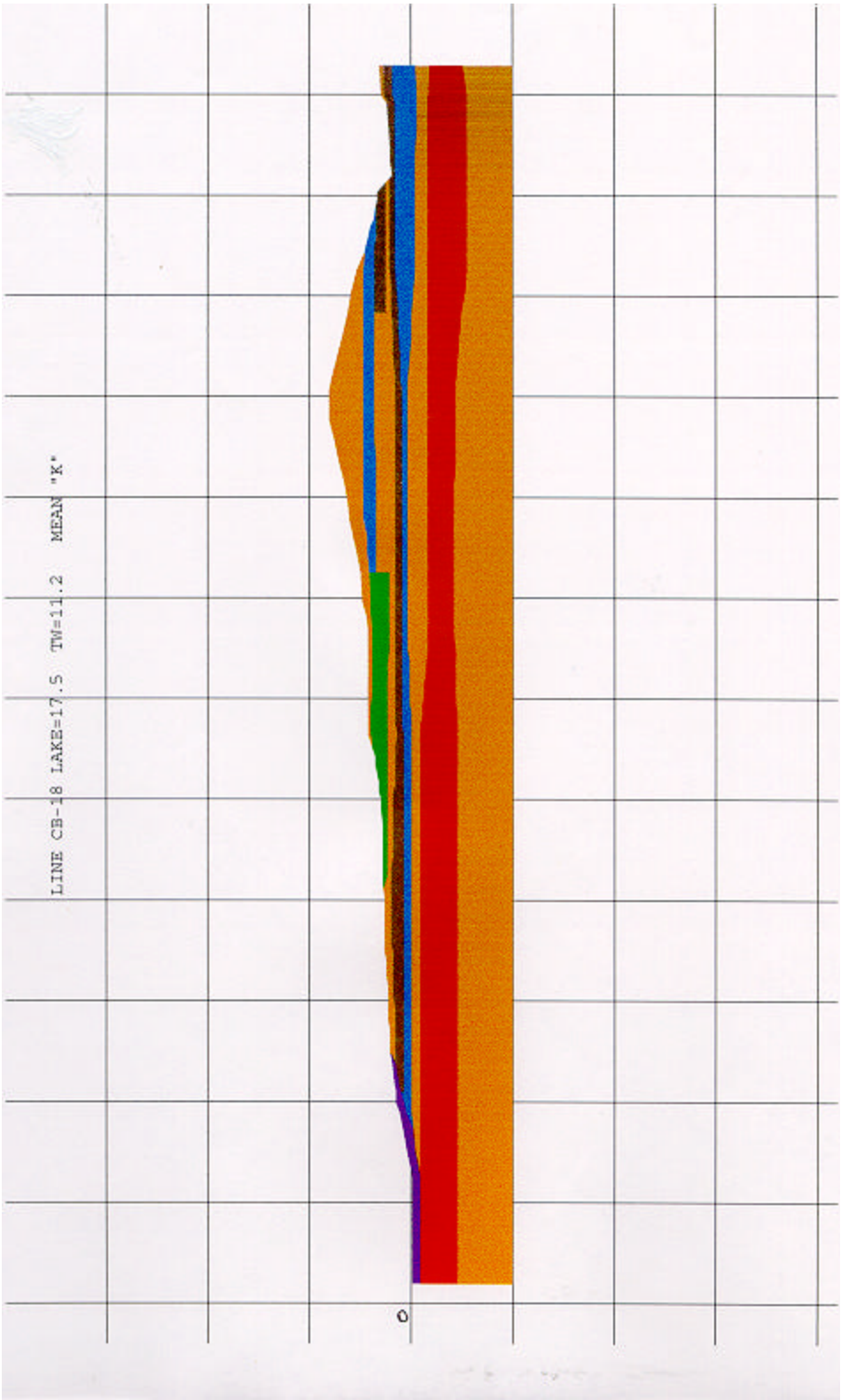
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

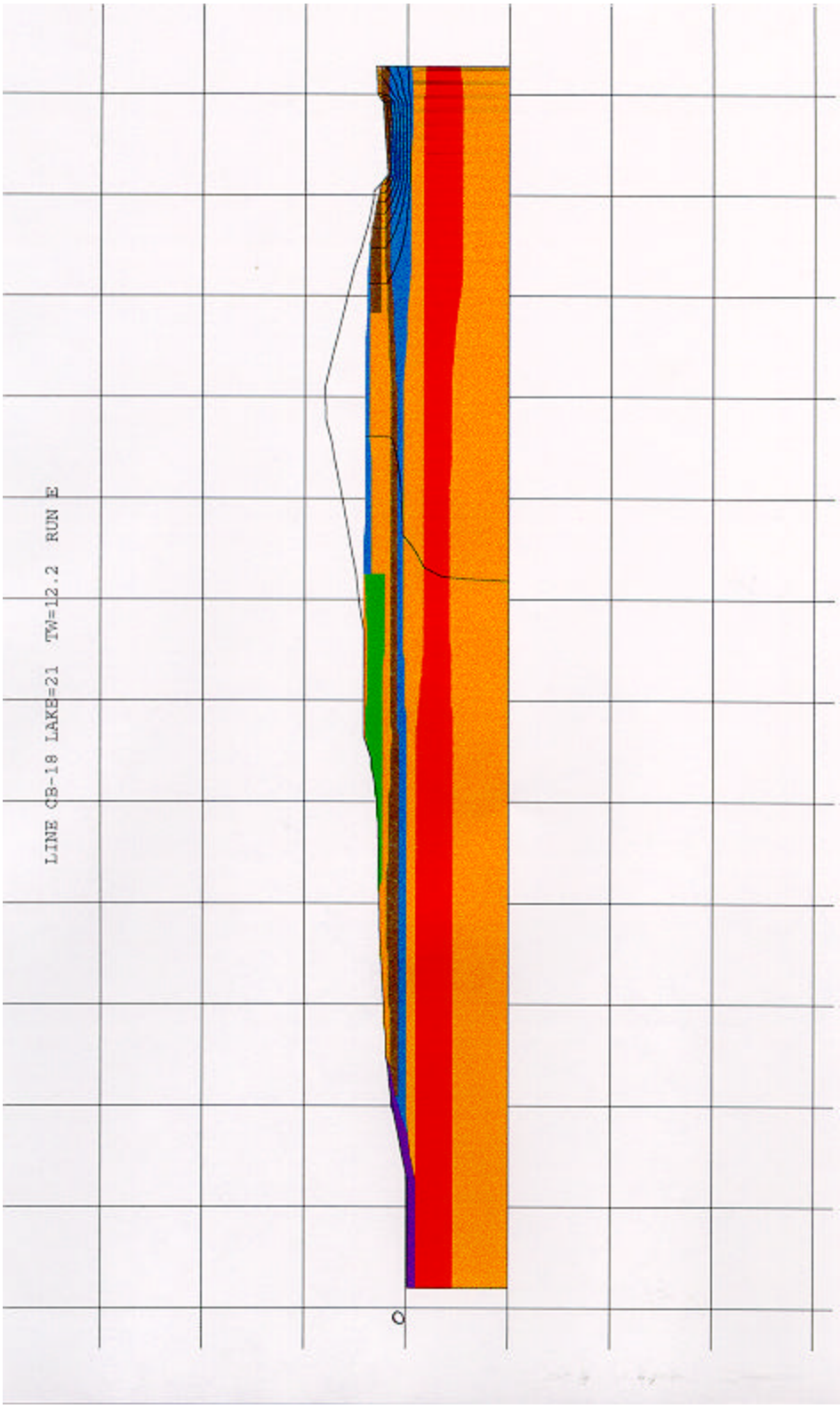
LINE CB-18 LAKE=15’ TW=11.2’ “E” perm.



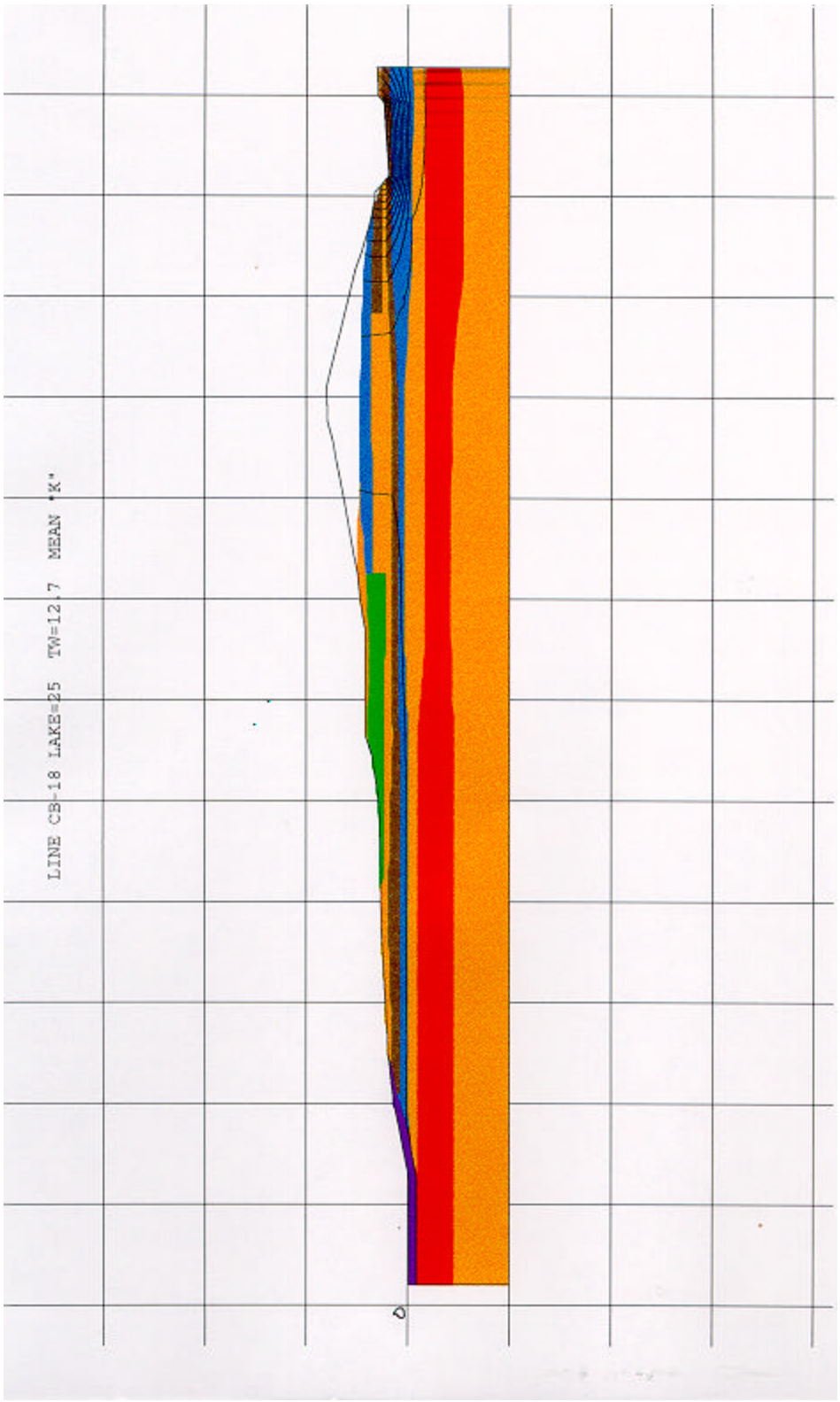
LINE CB-18 LAKE=A7.5 TW=11.2 MEAN “K”



LINE CB-18 LAKE=21 TW=12.2 RUN E



LINE CB-18 LAKE=25 TW=12.7 MEAN"K"


LINE CB-18 LAKE=25 TW=12.7 MEAN "K"

C1815e.xls, Ditch Bottom,iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike;** **LINE C-18**

Deterministic Model: FASTSEEP **LAKE 15**

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS PEAT AVE "N" B/F = 5 UNIT WG 70 PCF

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Ditch Bottom, iv

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater			i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**11.2**			**0.86**	**0.284**	**3.032**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.283**	3.041		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.282**	3.051	0.0000	0.03%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.283**	3.041		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.282**	3.051	0.0000	0.03%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.284**	3.032		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.283**	3.041	0.0000	0.03%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.284**	3.023		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.86**	**0.276**	3.118	0.0022	2.72%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	11.2			**0.86**	**0.284**	3.032		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	11.2			**0.86**	**0.283**	3.041	0.0000	0.03%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	11.2			**0.86**	**0.283**	3.041		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	11.2			**0.86**	**0.283**	3.041	0.0000	0.00%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	11.2			**0.86**	**0.284**	3.032		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	11.2			**0.86**	**0.283**	3.041	0.0000	0.03%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**11.5**			**0.86**	**0.284**	3.032		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**10.9**			**0.86**	**0.282**	3.051	0.0001	0.10%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.94**	**0.284**	3.314		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2			**0.78**	**0.284**	2.750	0.0796	97.04%
19																		
20																		
21																		
22																		
																Sum:	0.0820	100.00%

Adjusted probabilities based on length effect:

Total Length =	**9,000**	liability of Section, R =	1.00E+00	**E[FS] =**	**3.032**	E[ln(FS)] =	1.1049		
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**0.00E+00**	Var[FS] =	0.0820	Var[ln(FS)] =	0.0089	b =	**11.726**
No. Links =	**3**	**1 in**	**#DIV/0!**	s **[FS] =**	**0.2863**	σ[ln(FS)] =	0.0942	Pr(f) =	0.00E+00
				V [FS] =	0.0944	V [ln(FS)] =	0.0853	**1 in**	**#DIV/0!**

C1815e.xls, Ditch Slope, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Line C-18**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

LAKE AT 15 feet

AVE "N" FOR SM SAND =10 B/F UNIT WG 120 PCF

Ditch Slope, ih

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**11.2**	**0.82**	**0.393**	**2.088**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	11.2	**0.82**	**0.393**	2.088		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	11.2	**0.82**	**0.393**	2.088		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	11.2	**0.82**	**0.393**	2.088	0.0000	0.00%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**11.5**	**0.82**	**0.393**	2.088		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**10.9**	**0.82**	**0.393**	2.088	0.0000	0.00%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.90**	**0.393**	2.292		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	11.2	**0.74**	**0.393**	1.885	0.0415	100.00%
														Sum:	0.0415	100.00%

E[FS] =	**2.088**	E[ln(FS)] =	0.7160		
Var[FS] =	0.0415	Var[ln(FS)] =	0.0407	b =	**3.550**
s **[FS] =**	**0.2037**	σ[ln(FS)] =	0.2017	Pr(f) =	1.92E-04
V [FS] =	0.0976	V [ln(FS)] =	0.2817	**1 in**	**5199**

Adjusted probabilities based on length effect:

Total Length =	**9,000**	Reliability of Section, R =	9.99E-01
"Link" Length =	**3000**	**Adjusted Pr(f) =**	**5.77E-04**
No. Links =	**3**	**1 in**	**1733**

C1821.xls, Ditch Slope, ih

Probabilistic Through Seepage Analysis: **Herbert Hoover Dike; Line 18**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

LAKE A 21 feet

Ditch Slope, ih

Run	Upper Sands	Gravel	ML Horiz	Vert	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater	i crit	Results i exit*	FS	Prob Calcs Var	%
E	**1.41E-03**	**7.07E-01**	**1.27E-05**	**7.11E-05**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**12.2**	**0.82**	**0.275**	**2.982**		
1	**3.25E-03**	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.270**	3.037		
2	**6.15E-04**	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.270**	3.037	0.0000	0.00%
3	1.41E-03	**1.63E+00**	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.270**	3.037		
4	1.41E-03	**3.08E-01**	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.270**	3.037	0.0000	0.00%
5	1.41E-03	7.07E-01	**2.27E-04**	**7.09E-05**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.278**	2.955		
6	1.41E-03	7.07E-01	**1.81E-04**	**2.79E-05**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.275**	2.982	0.0002	0.01%
7	1.41E-03	7.07E-01	1.27E-05	7.11E-05	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.280**	2.929		
8	1.41E-03	7.07E-01	1.27E-05	7.11E-05	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.475**	1.726	0.3614	19.81%
9	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.310**	2.645		
10	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.82**	**0.228**	3.604	0.2300	12.61%
11	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	12.2	**0.82**	**0.235**	3.489		
12	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	12.2	**0.82**	**0.195**	4.205	0.1281	7.02%
13	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	12.2	**0.82**	**0.195**	4.205		
14	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	12.2	**0.82**	**0.378**	2.172	1.0332	56.63%
15	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	12.2	**0.82**	**0.270**	3.037		
16	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	12.2	**0.82**	**0.265**	3.094	0.0008	0.04%
`17	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.90**	**0.275**	3.273		
18	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.74**	**0.270**	2.741	0.0708	3.88%
																Sum:	1.8244	100.00%

Adjusted probabilities based on length effect:

Total Length =	9,000	Reliability of Section, R =	3.63E-01	**E[FS] =**	**2.982**	E[ln(FS)] =	0.5734				
"Link" Length =	3000	**Adjusted Pr(f) =**	**6.37E-01**	Var[FS] =	1.8244	Var[ln(FS)] =	1.0383			b =	**0.563**
No. Links =	3	**1 in**	**2**	s **[FS] =**	**1.3507**	σ[ln(FS)] =	1.0190			Pr(f) =	2.87E-01
				V [FS] =	0.4530	V [ln(FS)] =	1.7771			**1 in**	**3**

C1821.xls, Ditch Bottom, iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike;** **LINE -18**

Deterministic Model: FASTSEEP **LAKE LEVEL 21 '**

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS icrit based on .75 in accor lmvd method p 271 increased by .15 to get over the hump

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal **BASE CONDITION**

Ditch Bottom, iv

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater			i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**12.2**			**0.86**	**0.716**	**1.202**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.714**	1.205		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.717**	1.199	0.0000	0.02%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.716**	1.200		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.715**	1.203	0.0000	0.01%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.716**	1.200		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.714**	1.205	0.0000	0.01%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.685**	1.255		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.86**	**0.741**	1.161	0.0022	6.13%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	12.2			**0.86**	**0.630**	1.365		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	12.2			**0.86**	**0.524**	1.641	0.0191	53.47%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	12.2			**0.86**	**0.717**	1.199		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	12.2			**0.86**	**0.708**	1.215	0.0001	0.18%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	12.2			**0.86**	**0.716**	1.202		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	12.2			**0.86**	**0.707**	1.217	0.0001	0.15%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**12.9**			**0.86**	**0.723**	1.189		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**11.5**			**0.86**	**0.709**	1.214	0.0002	0.42%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.94**	**0.716**	1.314		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2			**0.77**	**0.716**	1.076	0.0141	39.60%
19																		
20																		
21																		
22																		
																Sum:	0.0356	100.00%

Adjusted probabilities based on length effect:

				Total Length =	9,000	liability of Section, R =		6.46E-01		**E[FS] =**	**1.202**	E[ln(FS)] =	0.1717				
				"Link" Length =	3000	**Adjusted Pr(f) =**		**3.54E-01**		Var[FS] =	0.0356	Var[ln(FS)] =	0.0244			b =	**1.100**
				No. Links =	3		**1 in**	**3**		s **[FS] =**	**0.1888**	σ[ln(FS)] =	0.1561			Pr(f) =	1.36E-01
										V [FS] =	0.1571	V [ln(FS)] =	0.9090			**1 in**	**7**

C1825.xls, Ditch Slope, ih

Probabilistic Through Seepage Analysis: **Herbert Hoover Dike; Line C-18**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

LAKE AT 25'

BASE CONDITIONS

Run	Upper Sands	Gravel	ML Horiz	Vert	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater	i crit	**Ditch Slope, ih** Results i exit	**FS**	Prob Calcs Var	%
E	**1.41E-03**	**7.07E-01**	**1.27E-05**	**7.11E-05**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**12.5**	**0.82**	**0.414**	**1.980**		
1	**3.25E-03**	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.414**	1.980		
2	**6.15E-04**	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.414**	1.980	0.0000	0.00%
3	1.41E-03	**1.63E+00**	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.414**	1.980		
4	1.41E-03	**3.08E-01**	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.412**	1.991	0.0000	0.01%
5	1.41E-03	7.07E-01	**2.27E-04**	**7.09E-05**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.431**	1.901		
6	1.41E-03	7.07E-01	**1.81E-04**	**2.79E-05**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.370**	2.216	0.0248	4.69%
7	1.41E-03	7.07E-01	1.27E-05	7.11E-05	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.473**	1.733		
8	1.41E-03	7.07E-01	1.27E-05	7.11E-05	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.353**	2.323	0.0870	16.48%
9	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.424**	1.934		
10	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.82**	**0.395**	2.078	0.0052	0.98%
11	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	12.5	**0.82**	**0.304**	2.698		
12	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	12.5	**0.82**	**0.554**	1.480	0.3707	70.22%
13	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	12.5	**0.82**	**0.417**	1.968		
14	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	12.5	**0.82**	**0.412**	1.991	0.0001	0.03%
15	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	12.5	**0.82**	**0.380**	2.158		
16	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	12.5	**0.82**	**0.400**	2.053	0.0028	0.53%
`17	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.90**	**0.414**	2.173		
18	1.41E-03	7.07E-01	1.27E-05	7.11E-05	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.5	**0.74**	**0.414**	1.787	0.0373	7.07%
																Sum:	0.5279	100.00%

Adjusted probabilities based on length effect:

					Total Length =	9,000	Reliability of Section, R =	4.48E-01	**E[FS] =**	**1.980**	E[ln(FS)] =	0.4710				
					"Link" Length =	3000	**Adjusted Pr(f) =**	**5.52E-01**	Var[FS] =	0.5279	Var[ln(FS)] =	0.4239			b =	**0.723**
					No. Links =	3	**1 in**	**2**	s **[FS] =**	**0.7266**	σ[ln(FS)] =	0.6511			Pr(f) =	2.35E-01
									V [FS] =	0.3670	V [ln(FS)] =	1.3825			**1 in**	**4**

C1825.xls, Ditch Bottom, iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike;** **LINE C-18**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

LAKE LEVEL AT 25'

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Filter Cake	Ditch Tailwater	i crit	**Ditch Bottom, iv** Results i exit	**FS**	Prob Calcs Var	%
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**2.12E-05**	**12.2**	**0.86**	**6.973**	**0.123**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.932**	**0.124**		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.925**	**0.124**	0.0000	0.00%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.911**	**0.124**		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.904**	**0.125**	0.0000	0.00%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.938**	**0.124**		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.829**	**0.126**	0.0000	0.33%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**6.568**	**0.131**		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.86**	**7.164**	**0.120**	0.0000	9.78%
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	2.12E-05	12.2	**0.86**	**7.116**	**0.121**		
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	2.12E-05	12.2	**0.86**	**6.555**	**0.131**	0.0000	8.84%
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	2.12E-05	12.2	**0.86**	**6.925**	**0.124**		
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	2.12E-05	12.2	**0.86**	**6.863**	**0.125**	0.0000	0.10%
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**4.88E-05**	12.2	**0.86**	**6.973**	**0.123**		
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**9.23E-06**	12.2	**0.86**	**6.870**	**0.125**	0.0000	0.28%
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**13.5**	**0.86**	**6.425**	**0.134**		
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	**11.9**	**0.86**	**7.637**	**0.113**	0.0001	37.24%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.94**	**6.973**	**0.135**		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	2.12E-05	12.2	**0.78**	**6.973**	**0.112**	0.0001	43.43%
														Sum:	0.0003	100.00%

Adjusted probabilities based on length effect:

Total Length =	9,000	Reliability of Section, R =	0.00E+00	**E[FS] =**	**0.123**	E[ln(FS)] =	-2.1027
"Link" Length =	3000	**Adjusted Pr(f) =**	**1.00E+00**	Var[FS] =	0.0003	Var[ln(FS)] =	0.0197
No. Links =	3	**1 in**	**1**	s **[FS] =**	**0.0174**	σ[ln(FS)] =	0.1405
LENGTH IN FEET				V [FS] =	0.1412	V [ln(FS)] =	-0.0668

b =	**-14.969**
Pr(f) =	1.00E+00
1 in	**1**

8.12. Model for Line 25. This model is designated as "line 25" since it is taken at survey line CB-25, representing the twenty fifth line of core borings taken on 3000' intervals north of S-351. This is part of the original levee embankment designated as LD-9 near station 350+00. As-builts dated November, 1932 show the construction of the original federal project as having a 10 foot crest at elevation 34.0 feet, a 1V:6H lakeside slope and a 1V:4H landside slope. Construction of a fire toe through the "muck" was also indicated. When the embankment was raised to its present elevation of 38.0 feet in 1965, the downstream slope was shown to have a compound 1V:5H upper slope and 1V:3H lower slope.

8.12.1. Selection of Variables and Values. Prior to developing a geologic section for this model, the core samples recovered were examined and compared to the core logs. Comparisons were made between the samples in the jar and the descriptions and classifications on the core logs. Where obvious errors existed, or another interpretation seemed more appropriate, a change to the model was made. Revisions incorporated into the model from core log classification changes as follows:

CB-HHDR1-25B - The layer classified as peat from el. 15.1 to 9.1 was actually SM with some ML.

CB-HHDR1-25B - The CL layer from el. 9.1 to 4.6 was actually SM with some ML.

CB-HHDR1-25B - The SC layer from el 0.1 to -2.9 was actually ML.

CB-HHDR1-25C - The jar samples 1 - 10 classified as PT contained only about one tablespoon of material per jar. Since PT / OL / OH all are modeled with the same permeability and shear strengths, the assumption that the material was all peat was not unreasonable.

CB-HHDR1-25C - The CL layer from 0.4 to -4.1 is most likely ML.

8.12.1.1. Topographic Features. Topographic features significant to this model were the toe ditch and fire toe. The fire toe is a man made feature is shown on the drawings for the original Federal Project. Basically it is a 12 foot wide trench excavated through the peat layer under the landside toe of the original project. The depth varies with the thickness of the peat. The purpose of the fire toe was to prevent a subsurface peat fire from undermining the embankment. The significance of this feature is that it provides a continuous direct connection between the embankment material and the upper limestone below the peat.

8.12.1.2. Geologic Features. Six geologic features were selected as being significant to this model. They are as follows:

Upper Sands - This is a combination of hydraulic fill from the original construction in the 1930's and additional fill placed in the 1960's. The material classifies primarily as sands with varying amounts of fines. A significant portion of the sand is sand size shell particles.

Gravel Layer - a man made feature due to the segregation of coarse materials at a hydraulic fill discharge pipe. The significance of this feature is that it provides intermittent zones of internally unstable high permeability material near the base of the embankment. This feature was encountered in borings CB-HHDR1-25A, CB-HHDI-45 and CB-HHDI-46.

Entrance valve - a natural feature incorporated into the model to aid calibration. There is insufficient detailed information on the exact geologic configuration which allows water into the lower limestone. Piezometer readings at different lake levels were reproduced by assigning the most probable area of water input as a separate material for the model. This material, or "valve" could then be adjusted independently of the rest of the material layers to calibrate the model.

Peat - This natural deposit occurs throughout the area. The peat shows some evidence of consolidation under the embankment. The area at the toe consists of natural deposits and fill in the upper zones.

Silts & Clays - a natural deposit of low plasticity. This material is sometimes referred to as marl. The deposit occasionally contains interbedded sands and shell seams.

Exit Valve - a natural feature incorporated into the model to aid calibration. There is insufficient detailed information on the exact geologic configuration which allows water to exit the lower limestone. Piezometer readings at different lake levels were reproduced by assigning the most probable area of water exit as a separate material for the model. In this case, it is the area below the bottom of the toe ditch. Since both upper and lower "C" piezometers read the same, a connection exists through the silt / clay layer which allows for rapid equalization of the piezometric surfaces above and below the silt / clay layer. This connection is likely due to past hydraulic fracturing, or heave, of the silt / clay layer. This material, or "valve" could then be adjusted independently of the rest of the material layers to calibrate the model.

Pipe - a natural feature incorporated into the model to aid calibration. The "pipe" represents an area damaged by past high water events where some piping or hydraulic fracturing has taken place. The upper pipe length was to calibrate the upper "C" piezometer reading. The lower pipe was not necessary for calibration, but was installed to allow for calculation of internal exit gradients into solution channels in the limestone. The length was based on approximately 20% of the total length (L) of a pipe needed to reach the lake through the entrance valve. Work by Sellmeijer assumes when piping starts the minimum pipe length established is 10% of L. Evidence of widely established solution channels is the basis for selecting a length greater than the minimum 10%. The permeability distribution for this feature was established based on velocities encountered in the dye tests. A slightly higher expected value was used since the model is set up directly connected to one of the larger conduits.

8.12.1.3. Slope Stability Variables. Variables and values selected for slope stability were divided into two categories, those with normal distributions and those with lognormal

distributions. The adjustment to q for sand from the typical values was from 36° to 30° based on low blow counts in the embankment. Except for the upper limerock base course and top 5' with N's of 25, 14 and 14; the remaining 15' has an average N of 5. Based on Bowles (Bowles, 1984) table 3-2, this would be a very loose material with a phi of 27° - 32°. The average of (27 +32)/2 = 29.5, or 30.

Table H-8.19 Normally Distributed variables

	Shear Strength Summary for Line 25				
		All values in degrees.			
	q	q	Normal Parameters		
Material	Min / Max	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb, R	17 / 28	23.0	2.30	25.3	20.7
Emb, S	27 / 45	30.0	3.00	33.0	27.0
PT/OL/OH, R		18.0	4.50	22.5	13.5
PT/OL/OH, S		33.0	8.25	41.3	24.8
CL / ML, R		12.5	3.10	15.6	9.4
CL / ML, S		33.0	8.30	41.3	24.7

Table H-8.20 Lognormally Distributed variables

	Shear Strength Summary for Line 25			
	All values in degrees or psf.			
		Lognormal Parameters		
Material	C	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Emb, R	0.37	0.26	0.60	0.11
Emb, S		no variation, not a variable		
PT/OL/OH, R	0.22	0.16	0.36	0.07
PT/OL/OH, S	0.12	0.08	0.20	0.04
CL / ML, R	0.35	0.31	0.51	0.19
CL / ML, S	0.10	0.07	0.16	0.03

Table H-8.21

	Material Unit Weights			
Material	Mean [E(x)]	s(x)	High [E(x) +s]	Low [E(x) +s]
Emb.	115.0	5.75	120.8	109.3
PT/OL/OH	70.0	3.50	73.5	66.5
CL/CH/ML/MH	100.0	5.00	105.0	95.0

8.12.1.4. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability were lognormally distributed.

	Permeability Summary for Line 25				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Upper Sands	2.00E-03	2.00E-03	1.41E-03	3.25E-03	6.15E-04
Gravel	1.00E+00	1.00E+00	7.07E-01	1.63E+00	3.08E-01
Peat	2.00E-04	n/a	1.60E-04	3.12E-04	8.20E-05
	n/a	6.00E-05	4.61E-05	9.53E-05	2.23E-05
Silts & Clays	1.50E-04	n/a	1.27E-04	2.27E-04	7.09E-05
	n/a	1.10E-04	7.11E-05	1.81E-04	2.79E-05
Rock	2.50E-01	n/a	1.77E-01	4.06E-01	7.69E-02
	n/a	2.00E-02	1.41E-02	3.25E-02	6.15E-03
Lower Sands	6.00E-03	6.00E-03	4.24E-03	9.75E-03	1.85E-03
Entrance Valve	7.50E-02	7.50E-02	5.30E-02	1.22E-01	2.31E-02

Exit Valve	2.00E-01	2.00E-01	1.41E-01	3.25E-01	6.15E-02
Pipe	5.00E+00	5.00E+00	3.54E+00	8.13E+00	1.54E+00

Table H-8.22

Gravel Distribution, Location of end of Gravel Pocket

Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
118.0	0 / 320	53.00	171.0	65.0
Transformed to Model Coord.s				
-32.0	-210 / 110	53.00	21.0	-85.0

8.12.2 Selection of Tailwater Distribution. In this case the toe ditch canal connected to the West Palm Beach canal dominated the tailwater scheme. Tailwater in this area (Lines 23 - 38) is controlled as follows:

S352 north to C-13: In this area, a toe ditch transfers flow from a point just south of C-13 southward directly into the West Palm Beach Canal at S-352. S-352 tailwater measurements may be interpreted as ditch water levels.

C-13 north to C-10A: A toe ditch controls drainage from C-13 north to C-10A. Flow proceeds from south to north and empties directly into L-8 canal at C-10A.

C-10A north to Port Mayaca: Drainage along the base of the embankment is controlled by a toe ditch from C-10A to S-308. In this area, the FEC Railway and / or Highway 98 parallel the dike alignment.

Tailwater elevations are based on the lowest expected value in the ditch associated with that lake stage. Readings obtained downstream of S-352 were used as a basis for determining tailwater elevations in the ditch approximately 6000' north at line 25. For analysis purposes, the lowest probable tailwater was used for seepage and highest probable tailwater for slope stability. It is also known from topography that the highest practical water level is elevation 15.0. Surveys show the existing railroad grade at elevation 16.0.

8.12.2.1. Tailwater Distributions for Slope Stability Analyses. Distributions for these analyses are shown in Table H-8.23. This is a relatively wide range of values for this system based upon probable minimum and maximum values. For this failure mode, a higher tailwater would be more critical since the corresponding phreatic surface in the upper saturated zone would be higher. This range of values was assumed to be normally distributed.

8.12.2.2. Tailwater Distribution for Seepage Analyses. Unlike the slope stability failure modes, underseepage was more critical with lower tailwater elevations. Since the toe ditch could be artificially manipulated, a water level distribution was selected for all analyses.

Table H-8.23

	Tailwater Elevations for Seepage Models				
Lake El.	Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
15	11.0	10 / 12	0.33	11.3	10.7
18	11.5	10 / 13	0.50	12.0	11.0
21	12.0	10 / 14	0.66	12.7	11.3
24	12.5	10 / 15	0.80	13.3	11.7

Table H-8.24

	Tailwater Elevations for Slope Stability Models				
Lake El.	Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
15	11.0	10 / 12	0.33	11.3	10.7
18	11.5	10 / 13	0.50	12.0	11.0
21	12.0	10 / 14	0.66	12.7	11.3
24	12.5	10 / 15	0.80	13.3	11.7

8.12.3. Selection of Critical Gradients.

8.12.3.1. Piping above the confining layer.

For the upper sands and sandy material below the peat, gradients in the model indicated the material did have some confining stresses. Therefore, i_{ch} was computed as follows using the equation derived in the WES report (USACE, 1997): $i_{ch} = i_{cv}$ (cos b tan f -sin b), where f = friction angle and b = the angle of flow (obtained from FastSeep grid).

	i_{cv}	i_{ch}(f only)	i_{ch}(f and b)
High values			
g_{soil} = 121 pcf	0.939		
f = 33°		0.609	
b = 0°			0.609
Low values			
g_{soil} = 109 pcf	0.747		
f = 27°		0.509	
b = 5°			0.314

With the range of i_{ch} from 0.609 to 0.314, an average value of 0.462 was selected. Dividing the range from 0.609 to 0.314 by six to obtain three standard deviations on either side of the mean, a standard deviation of s = 0.049 was applied. This gives the following:

Average Value:	0.462
Std. Dev.:	0.049
High Value:	0.511
Low Value:	0.413

This is the critical gradient to start piping. For breach, the gradient has to increase enough to drive the pipe past the half way point, l/L > 40 - 50 %, where l is pipe length and L is total pipe path length.

The following values for i_{ch} were used in the analysis for through seepage.

Average Value:	0.531
Std. Dev.:	0.056
High Value:	0.587
Low Value:	0.474

8.12.3.2. Piping below the confining layer. For underseepage, the values for i_{cv} were determined from material data as described above and from the Lower Mississippi method. Data indicated a range of i_{cv} from 0.939 to 0.747. Values for i_{cv} observed during the 1937 high water event along the Mississippi River ranged from 0.15 to 0.88, predominantly from 0.5 to 0.8. If we take the range for i_{cv} as 0.5 to 0.8 this gives the following:

Average Value: 0.650

Std. Dev.:	0.050
High Value:	0.700
Low Value:	0.600

Again applying this 15% to the critical gradient computed above, the following critical gradient for breach can be obtained.

Average Value:	0.750
Std. Dev.:	0.058
High Value:	0.808
Low Value:	0.693

8.12.4. Failure Mechanisms.

8.12.4.1. Through Seepage. For through seepage, the piping path begins at the base of the peat in the Lake Flirt marl formation. The sand component of this material pipes out until the pipe reaches the fire toe. The piping between clay and peat was modeled as peat since sand seam is thin (generally less than one foot). It should be noted that piezometer data shows some damage has already occurred between the toe ditch and fire toe. This is labeled as the probability of piping under the peat (Pp). Once the pipe reaches the fire toe it travels up through the sandy material in the fire toe and turns toward the gravel pocket in the embankment. Since the pipe progression through the fire toe is up at an angle greater than the angle of repose of the sandy material, only the flow necessary to transport material is needed to cause piping. This is labeled as the probability of piping through the fire toe (Pft), and is equal to 1. Once the pipe reaches the top of the peat at the back of the fire toe, the pipe progresses horizontally until it reaches the gravel pocket. This is labeled as the probability of piping through the embankment sands (Pp|p), given that piping up to that point has already occurred. Once in the gravel pocket, the pipe is well past the half way point and is self propagating to the source. This is labeled as the probability of piping through the gravel, Pg, and is equal to 1. The entire conditional probability of breach from through seepage Pb is :

$$Pp \times Pft|p \times Pp|p|ft \times Pg|p|ft = Pb \qquad (8.12.1)$$

Substituting Pft = 1 and Pg = 1, equation (8.12.1) reduces to:

$$Pp \times Pp|p = Pb \qquad (8.12.2)$$

8.12.4.2. Under Seepage. For underseepage, the piping path begins as heave and fracturing of the clay and peat in the ditch. Piezometer data shows this has already occurred. Once the gradient through the confining layer is sufficient, piping will occur. This is the probability of vertical piping through the confining layer. This is determined according to the method in TM 3-424 (USACE, 1956), where the gradient is measured across the confining layer. Piping

occurs through the cracks in the confining layer, and progresses back to the source. The critical gradient for breach is used to compute probability of breach, Pb.

8.12.4.3. Slope Stability. For slope stability, a two part process is involved. The first step is the slope failure itself. UTEXAS models using Spencer's method show that the failure arc involves approximately one third to one half of the embankment. This is labeled as the probability of slope stability failure, Ps. Once this shear failure occurs, the shear zones along the failure surface effectively shorten the seepage path. The location of the back of the shear zone is in the approximate location of the fire toe, where the second phase of through seepage failure occurs. This is labeled as the probability of piping through the embankment sands (Pp|s) given that a slope stability failure occurs.. It is calculated the same way as for through seepage and the same values are used. The entire conditional probability of breach from slope stability Pb is :

$$Ps \times Pp|s = Pb \qquad (8.12.3)$$

8.12.5. Analysis Results and Model Calibration.

8.12.5.1. Model calibration for seepage analyses. Once the appropriate features were selected, constant head boundary conditions could be applied. A lake level of 18.5 was selected since data from the 1995 High Water Event was available. Seepage was observed exiting along the ditch slope, and softening and small boils on the ditch bottom were observed. The model was calibrated to piezometer readings using expected values, not log of expected values. The filtercake material and heaved toe ditch bottom were used to "calibrate" as entrance and exit valves, respectively, so head under the confining layer agreed with the actual piezometer readings. The absence of filtercake drastically alters the head distribution below the confining layer. Pipe lengths were also adjusted to obtain agreement with piezometer readings

8.12.5.2. Model calibration for slope stability analyses. The search for the global critical failure arc consisted of examining surfaces tangent to the top of the rock layers and clay layer as well as varying tailwater elevations. The results are shown in Tables H-8.25. For some of the runs for UTEXAS the failure circle "walked out" of the slope. These are identifiable on the spreadsheet by the "Table 14" label on the side. Table 14 is the table from UTEXAS that computes the failure circles at certain tangent points. Individual failure arcs were identified and the "critical" arc selected as the significant arc with the lowest factor of safety.

8.12.6. Length Effects. Each failure mode was adjusted to incorporate the length effect. Due to conditional probabilities a slightly different approach was used. Refer to the computations in Table H-8.26. In this model, a "link" is 3000 feet long. This is based on the approximate spacing of data collected in reach 1 and the general topographic conditions.

8.12.7. Pup Curve Development.

The combined probability of breach function versus lake level is a combination of the three failure modes evaluated. It can be surmised that through seepage and the conditional probability of slope stability and through seepage are impacted buy a large amount of uncertainty which shows as higher probabilities of failure for "high" factors of safety. Under seepage has very little uncertainty associated with this failure mode and is very sensitive to changes in lake level (net head).

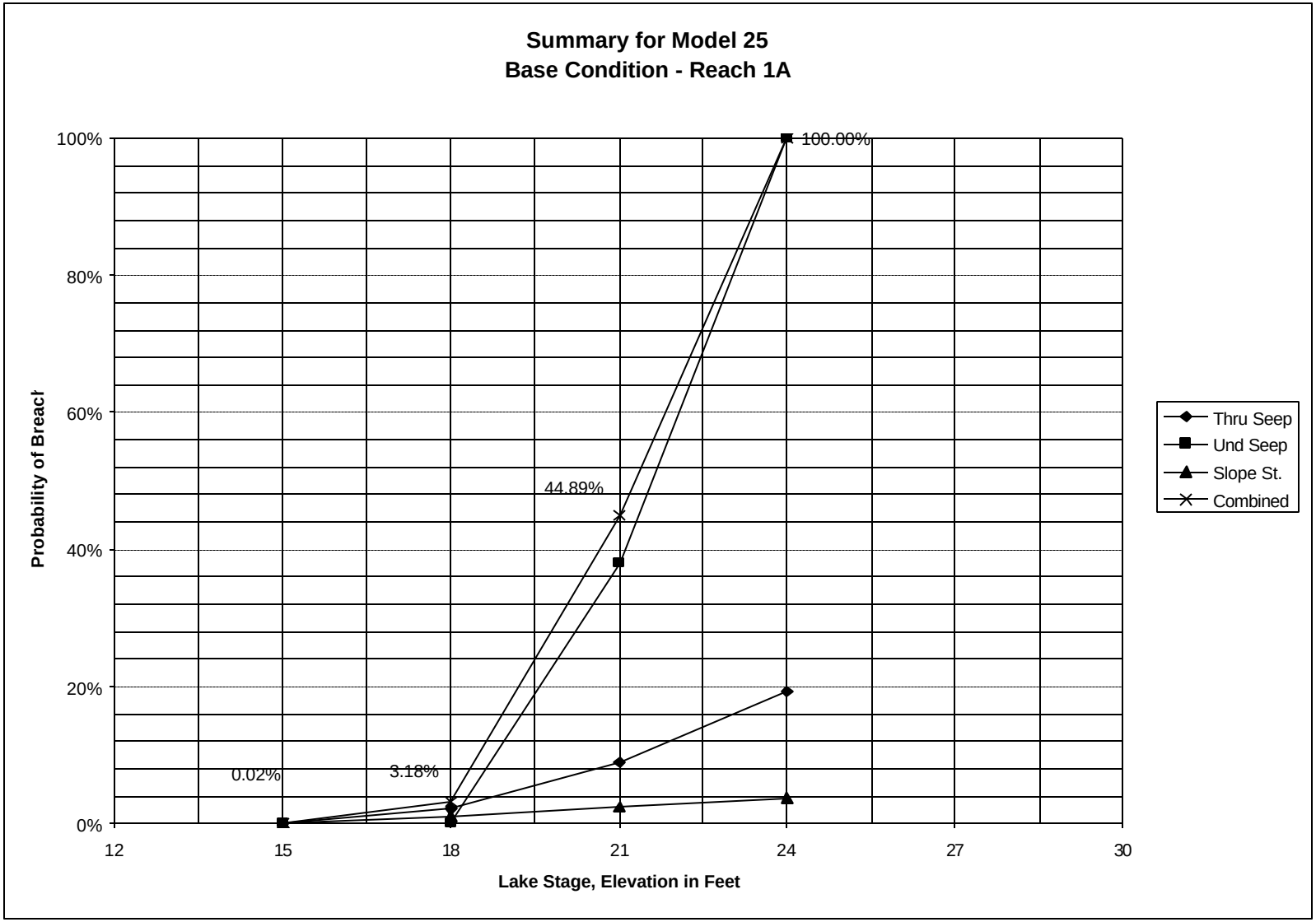
Table H-8.25. Analysis Summary.

Lake El.	Failure Mode	E(FS)	Beta	Sum Variance	P(Failure)	ic
15 *	UP ih	11.484	2.320	156.1575	0.0102	0.531
18 *	UP ih	5.667	1.395	44.4438	0.0815	0.531
18 *	DB iv	1.667	4.429	0.0363	4.74E-6	0.75
21 *	UP ih	3.507	1.034	13.3721	0.1510	0.531
21 *	DB iv	1.211	1.601	0.0197	0.0547	0.75
24 *	UP ih	2.429	0.480	8.2832	0.3160	0.531
24 *	DB iv	0.951	-0.511	0.0110	0.6950	0.75
15 *	UP ih f	12.894	2.930	135.0611	1.69e-3	0.531
18 *	UP ih f	6.544	1.866	41.6811	0.0310	0.531
21 *	UP ih f	4.407	1.462	17.2479	0.0719	0.531
24 *	UP ih f	3.463	1.418	7.6357	0.0780	0.531
			Slope	Stability		Tailwater
18	CL-LS	2.261	3.954	0.2112	3.85e-5	11.5
18	PT-CL	2.042	3.108	0.2102	9.40e-4	11.5
24	PT-CL	1.916	2.746	0.1947	3.01e-3	12.5
15 *	PT-CL	2.031	2.738	0.2607	3.09e-3	13.0
18 *	PT-CL	1.872	1.745	0.3990	0.0405	14.0
21 *	PT-CL	1.854	1.745	0.3797	0.0405	15.0
24 *	PT-CL	1.789	1.580	0.3753	0.0507	16.0

Definitions:
* = Value used in combined probability computation.
DB = Ditch Bottom UP = Upper Pipe
iv = vertical exit gradient ih = horizontal exit gradient
f = conditional probability CL-LS = Clay - Limestone interface
PT-CL = Peat - Clay interface

8.12.8. Probabilistic Analyses Computation Sheets and Sections.

L25TASUM.XLS



LEGEND FOR COLOR CROSS SECTIONS

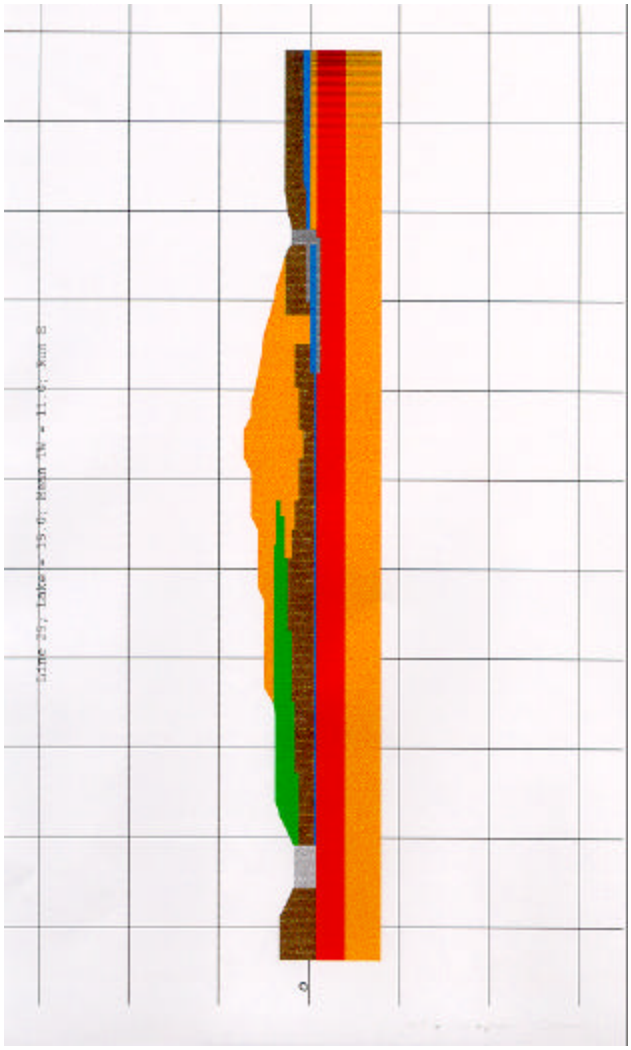
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

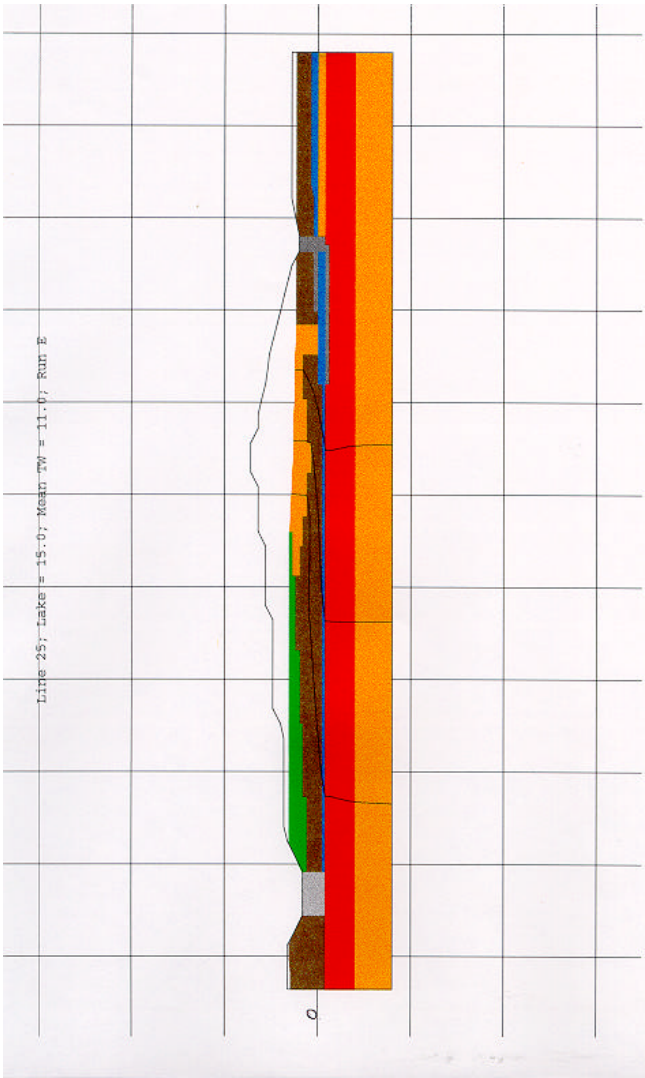
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

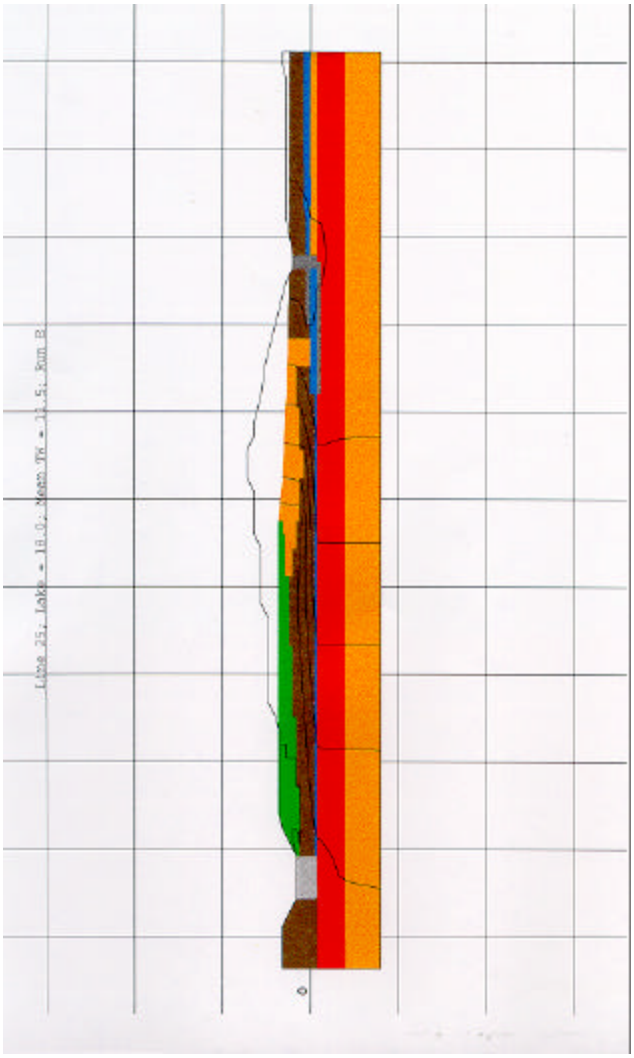
Line 25; Lake = 15.0; Mean TW = 11.0; Run E



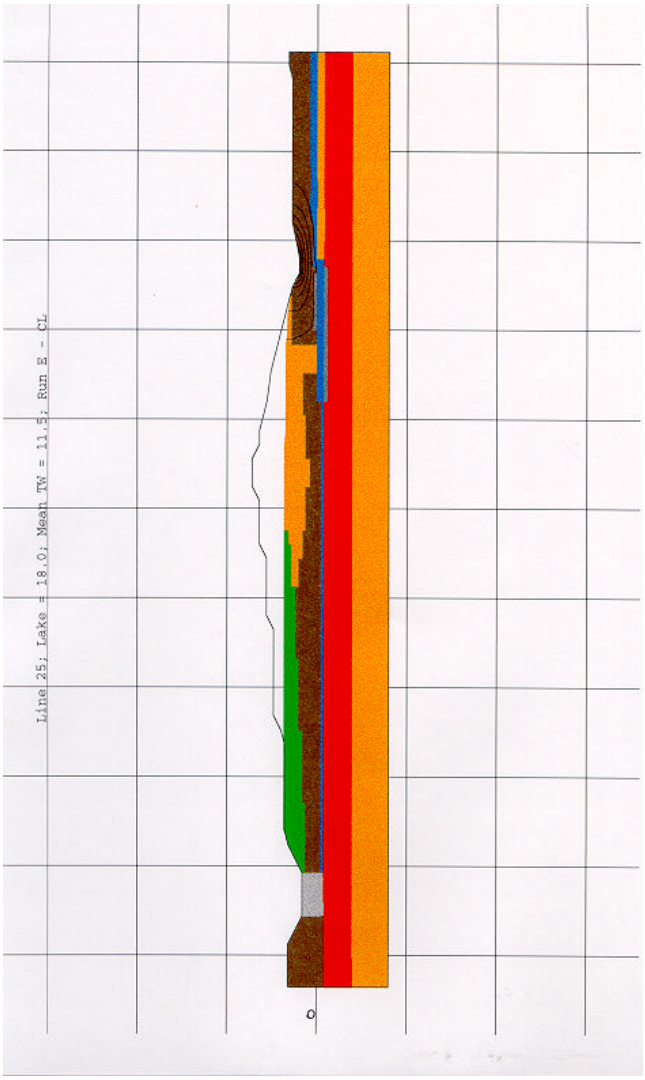
Line 25; Lake = 15.0; Mean TW = 11.0; Run E



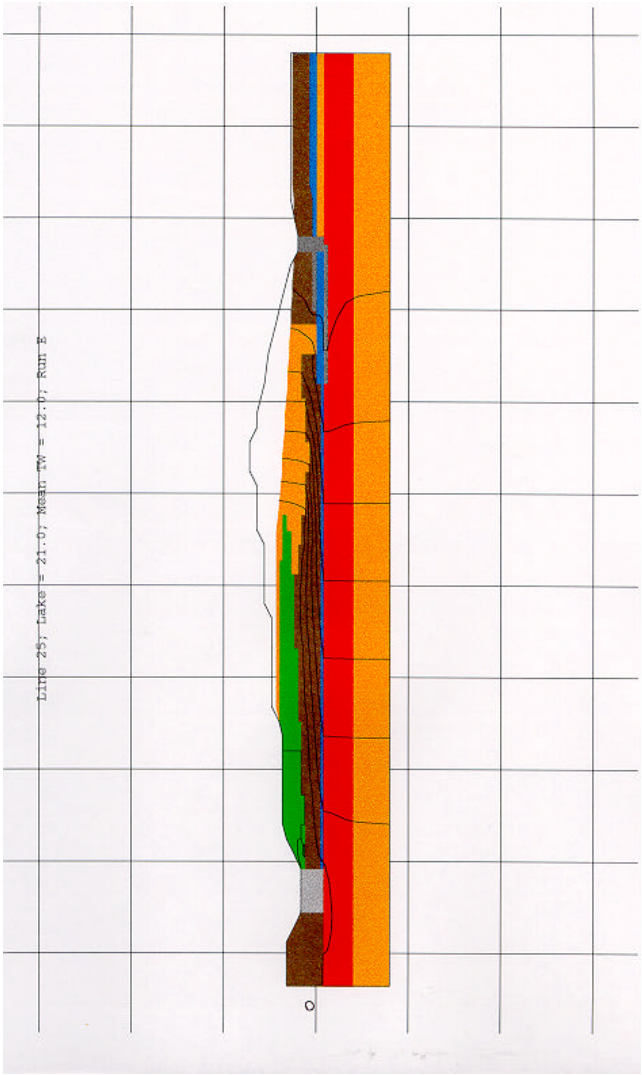
Line 25; Lake = 18.0; Mean TW = 11.5; Run E



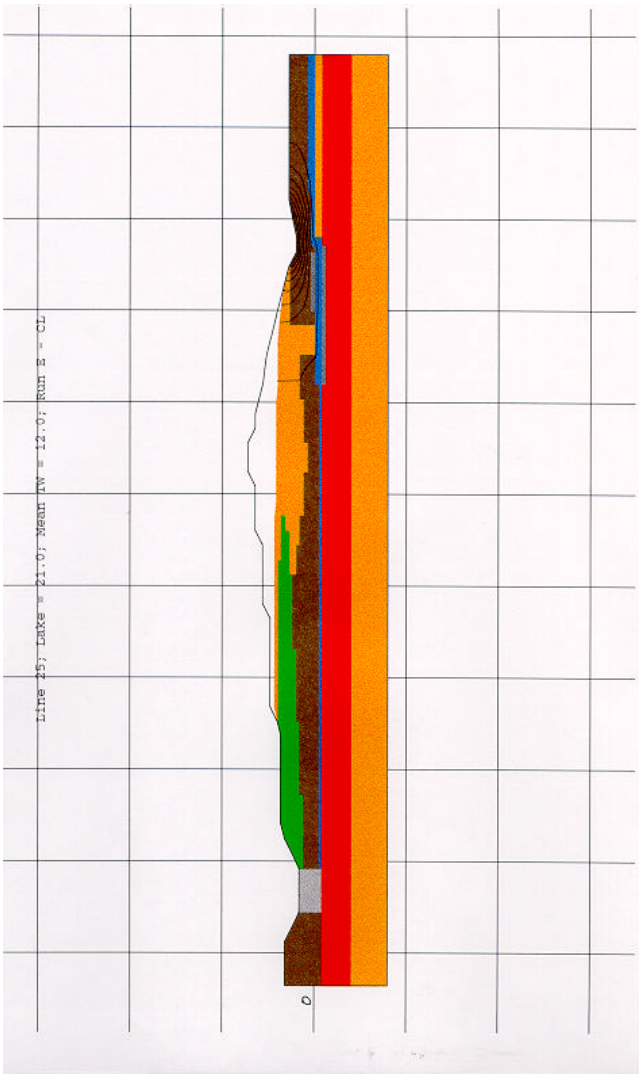
Line 25; Lake = 18.0; Mean TW = 11.5; Run E – CL



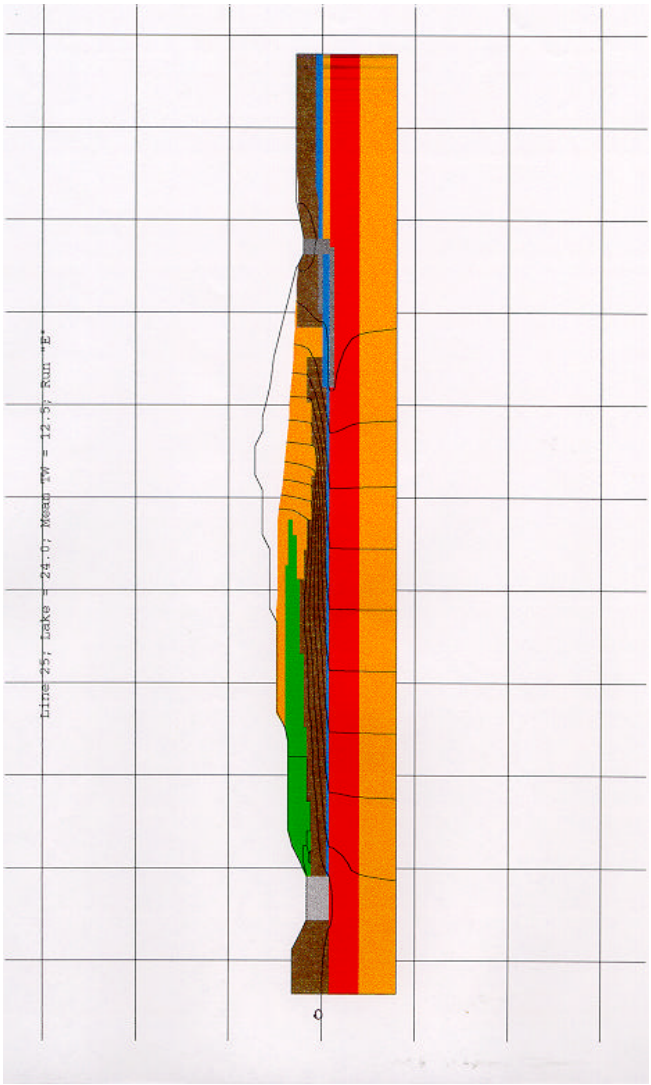
Line 25; Lake = 21.0; Mean TW = 12.0; Run E



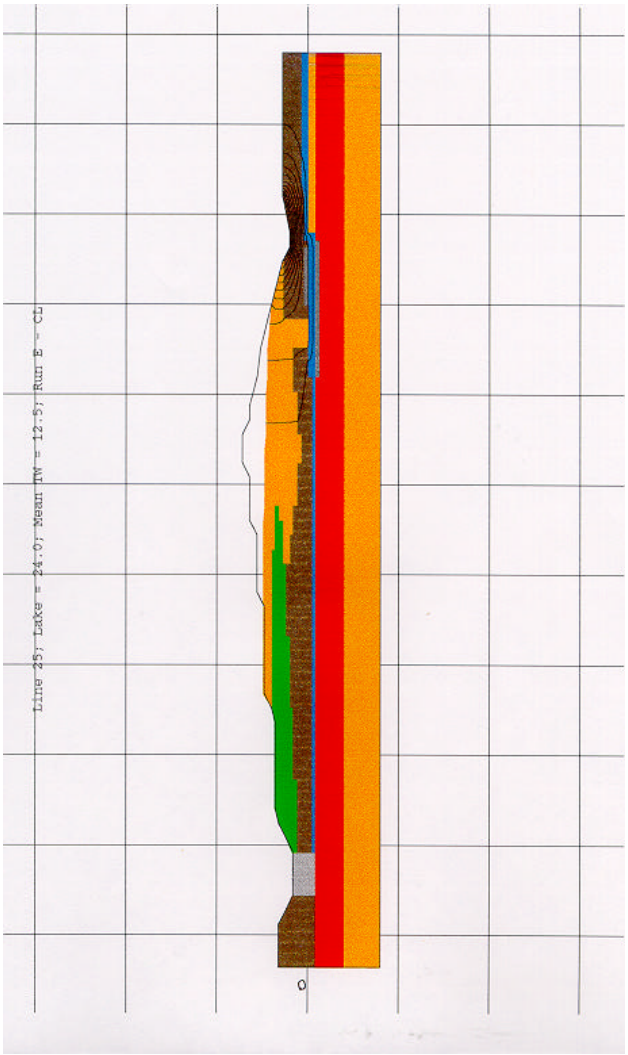
Line 25; Lake = 21.0; Mean TW = 12.0; Run E – CL



Line 25; Lake = 24.0; Mean TW = 12.5; Run “E”



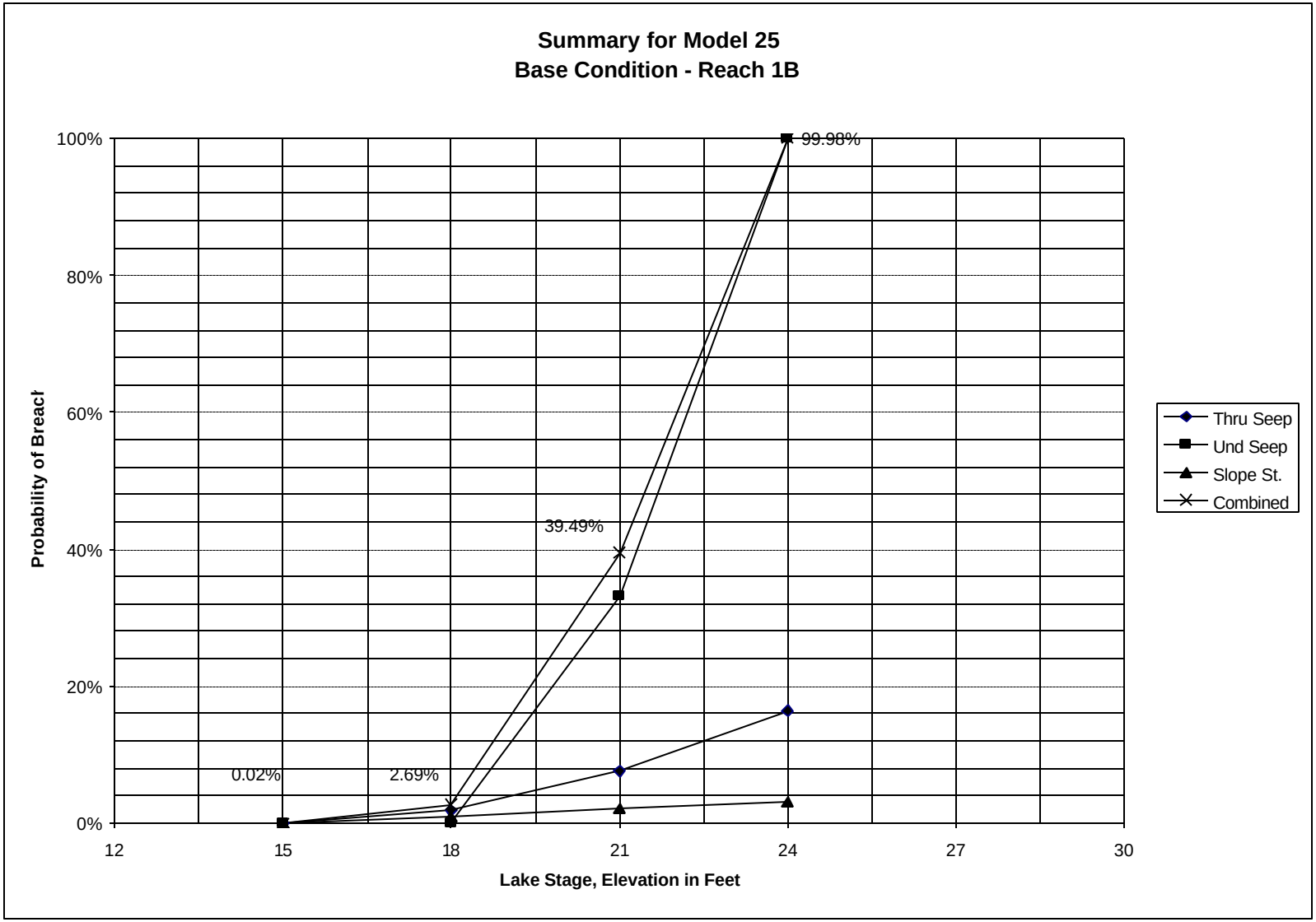
Line 25; Lake = 24.0; Mean TW = 12.5; Run “E” – CL



L25TASUM.XLS, Line 25 North Summary

Adjusted probabilities based on length effect:					Total Length =	25,500									
					Link Length =	3000									
					No. Links =	8.5									
Lake	Through	Seepage			With Length	Under	Seepage	With Lengtl	Slope	Stability			With Lengt	Combined	
Level	Pp	Pp|p	Pb	Rb	Effect	Pb	Rb	Effect	Ps	Pp|s	Pb	Rb	Effect	Pb	Rb
15	1.02E-02	1.70E-03	1.73E-05	1.00E+00	0.01%	0.00E+00	1.00E+00	0.00%	3.09E-03	1.70E-03	5.25E-06	1.00E+00	0.00%	0.02%	0.999808
18	8.16E-02	3.11E-02	2.54E-03	9.97E-01	2.14%	4.74E-06	1.00E+00	0.00%	4.05E-02	3.11E-02	1.26E-03	9.99E-01	1.07%	3.18%	0.9681661
21	1.51E-01	7.20E-02	1.09E-02	9.89E-01	8.87%	5.47E-02	9.45E-01	38.01%	4.05E-02	7.20E-02	2.92E-03	9.97E-01	2.45%	44.89%	0.5510718
24	3.16E-01	7.82E-02	2.47E-02	9.75E-01	19.16%	6.95E-01	3.05E-01	100.00%	5.70E-02	7.82E-02	4.46E-03	9.96E-01	3.73%	100.00%	3.219E-05

L25TASUM.XLS, Line 25 South Summary Chart 2



L25TASUM, Line 25 South Summary

Adjusted probabilities based on length effect:

Total Length =	21,500
Link Length =	3000
No. Links =	7.17

Lake Level	Through Pp	Seepage Pp|p	Pb	Rb	With Length Effect	Under Pb	Seepage Rb	With Lengt Effect	Slope Ps	Stability Pp|s	Pb	Rb	With Lengt Effect	Combined Pb	Rb
15	1.02E-02	1.70E-03	1.73E-05	1.00E+00	0.01%	0.00E+00	1.00E+00	0.00%	3.09E-03	1.70E-03	5.25E-06	1.00E+00	0.00%	0.02%	0.9998381
18	8.16E-02	3.11E-02	2.54E-03	9.97E-01	1.80%	4.74E-06	1.00E+00	0.00%	4.05E-02	3.11E-02	1.26E-03	9.99E-01	0.90%	2.69%	0.9730918
21	1.51E-01	7.20E-02	1.09E-02	9.89E-01	7.54%	5.47E-02	9.45E-01	33.18%	4.05E-02	7.20E-02	2.92E-03	9.97E-01	2.07%	39.49%	0.6050663
24	3.16E-01	7.82E-02	2.47E-02	9.75E-01	16.42%	6.95E-01	3.05E-01	99.98%	5.70E-02	7.82E-02	4.46E-03	9.96E-01	3.15%	99.98%	0.0001631

L25-15TA.XLS Summary

Exit Gradient Computations **Line 25; Lake = 15.0; Mean TW = 11.0**

	At pipe entrance					**Upper Pipe**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**0.13**	**11.50**	**11.24**	**450.07**	**444.45**	**0.0463**
1	0.12	11.50	11.24	450.11	447.41	0.0963
2	0.10	11.34	11.24	450.09	445.97	0.0243
5	0.08	11.40	11.24	450.07	445.30	0.0335
6	0.10	11.50	11.24	450.07	445.84	0.0615
7	0.12	11.39	11.24	450.12	445.51	0.0325
8	0.11	11.50	11.24	450.15	446.27	0.0670
17	0.08	11.40	11.24	450.08	445.91	0.0384
18	0.10	11.50	11.23	450.08	445.67	0.0612
21	0.13	11.50	11.24	450.07	444.45	0.0463
22	0.13	11.50	11.24	450.07	444.45	0.0463
23	0.11	11.50	11.25	450.11	447.95	0.1157
24	0.12	11.35	11.24	450.12	446.23	0.0283

L25-15TA.XLS Upper Pipe, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Lines 23 - 38**

Deterministic Model:	FASTSEEP	**Lake Level:**	**15.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	Model based on data at Line 25	
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Upper Pipe, ih

Run	Upper Sands	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Pipe	i crit	Gravel Pocket	Results i exit	Results **FS**	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**3.54E+00**	**0.531**	**-32.0**	**0.046**	**11.484**		
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.096**	5.517		
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.024**	21.890	67.0123	42.91%
5	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.034**	15.839		
6	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.061**	8.644	12.9439	8.29%
7	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	3.54E+00	0.531	-32.0	**0.033**	16.329		
8	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	3.54E+00	0.531	-32.0	**0.067**	7.929	17.6400	11.30%
13	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**8.13E+00**	0.531	-32.0	**0.038**	13.847		
14	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**1.54E+00**	0.531	-32.0	**0.061**	8.678	6.6799	4.28%
21	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.588	-32.0	**0.046**	12.702		
22	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.475	-32.0	**0.046**	10.266	1.4836	0.95%
23	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	**24.0**	**0.116**	4.590		
24	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	3.54E+00	0.531	**-88.0**	**0.028**	18.789	50.3977	32.27%
										Sum:	156.1575	100.00%

		For Piping	*For Breach*	**E[FS] =**	**11.484**	E[ln(FS)] =	2.0504	
Critical Gradient:	Avg	0.462	0.531	Var[FS] =	156.1575	Var[ln(FS)] =	0.7812	b = **2.320**
	High	0.511	0.588	s**[FS] =**	**12.4963**	σ[ln(FS)] =	0.8838	Pr(f) = 1.02E-02
	Low	0.413	0.475	V [FS] =	1.0881	V [ln(FS)] =	0.4311	**1 in 98**

L25-18TA.XLS Summary

Exit Gradient Computations	
Line 25; Lake = 18.0; Mean TW = 11.5	

	At pipe entrance					**Upper Pipe**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**0.10**	**12.65**	**11.90**	**450.00**	**442.00**	**0.094**
1	0.10	13.46	11.90	450.07	442.24	0.199
2	0.10	12.16	11.89	450.00	444.16	0.046
3	0.10	12.47	11.89	450.08	444.02	0.096
4	0.10	12.46	11.89	450.17	444.00	0.092
5	0.10	12.29	11.90	450.06	444.16	0.066
6	0.10	12.65	11.89	450.09	444.03	0.125
7	0.10	12.27	11.89	450.14	444.06	0.063
8	0.10	12.70	11.89	450.06	444.39	0.143
9	0.10	12.80	12.22	450.06	444.60	0.106
10	0.10	12.19	11.70	450.10	444.32	0.085
11	0.10	12.40	11.90	450.08	444.81	0.095
12	0.10	12.50	11.89	450.10	443.57	0.093
13	0.10	12.42	11.91	450.06	444.67	0.095
14	0.10	12.39	11.86	450.11	444.30	0.091
15	0.10	12.30	11.68	450.09	443.61	0.096
16	0.10	12.80	12.33	450.11	444.84	0.089
`17	0.10	12.40	11.90	450.08	443.88	0.081
18	0.10	12.57	11.88	450.06	444.16	0.117
19	0.10	13.00	12.36	450.10	442.88	0.089
20	0.10	12.00	11.42	450.10	444.31	0.100
21	0.10	12.65	11.90	450.00	442.00	0.09
22	0.10	12.65	11.90	450.00	442.00	0.09
23	0.10	12.40	11.90	450.04	447.89	0.233
24	0.10	12.20	11.89	450.04	444.43	0.055

L25-18TA.XLS Upper Pipe, ih

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **18.0**

Model based on data at Line 25

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Lower Sands	Entrance Valve	Exit Valve	Pipe	Ditch Tailwater
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**4.24E-03**	**5.30E-02**	**1.41E-01**	**3.54E+00**	**11.5**
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
9	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
10	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
11	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**9.75E-03**	5.30E-02	1.41E-01	3.54E+00	11.5
12	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.85E-03**	5.30E-02	1.41E-01	3.54E+00	11.5
13	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**1.22E-01**	1.41E-01	3.54E+00	11.5
14	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	**2.31E-02**	1.41E-01	3.54E+00	11.5
15	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	**3.25E-01**	3.54E+00	11.5
16	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	**6.15E-02**	3.54E+00	11.5
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	**8.13E+00**	11.5
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	**1.54E+00**	11.5
19	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	**12.0**
20	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	**11.0**
21	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
22	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
23	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5
24	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	4.24E-03	5.30E-02	1.41E-01	3.54E+00	11.5

	For Piping	*For Breach*
Critical Gradient: Avg	0.462	0.531
High	0.511	0.588
Low	0.413	0.475

E[FS] =	**5.667**
Var[FS] =	44.4438
s **[FS] =**	**6.6666**
V [FS] =	1.1764

piping of sand between peat and clay

Upper Pipe, ih

i crit	Gravel Pocket	Results i exit	Results **FS**	Prob Calcs Var	Prob Calcs %
0.531	**-32.0**	**0.094**	**5.667**		
0.531	-32.0	**0.199**	2.667		
0.531	-32.0	**0.046**	11.492	19.4706	43.81%
0.531	-32.0	**0.096**	5.551		
0.531	-32.0	**0.092**	5.751	0.0100	0.02%
0.531	-32.0	**0.066**	8.038		
0.531	-32.0	**0.125**	4.236	3.6123	8.13%
0.531	-32.0	**0.063**	8.501		
0.531	-32.0	**0.143**	3.719	5.7162	12.86%
0.531	-32.0	**0.106**	5.002		
0.531	-32.0	**0.085**	6.267	0.4005	0.90%
0.531	-32.0	**0.095**	5.600		
0.531	-32.0	**0.093**	5.688	0.0019	0.00%
0.531	-32.0	**0.095**	5.615		
0.531	-32.0	**0.091**	5.824	0.0109	0.02%
0.531	-32.0	**0.096**	5.553		
0.531	-32.0	**0.089**	5.957	0.0409	0.09%
0.531	-32.0	**0.081**	6.588		
0.531	-32.0	**0.117**	4.543	1.0456	2.35%
0.531	-32.0	**0.089**	5.994		
0.531	-32.0	**0.100**	5.304	0.1190	0.27%
0.588	-32.0	**0.094**	6.268		
0.475	-32.0	**0.100**	4.741	0.5829	1.31%
0.531	24.0	**0.233**	2.285		
0.531	-88.0	**0.055**	9.615	13.4331	30.22%
			Sum	44.4438	100.00%

E[ln(FS)] =	1.3003
Var[ln(FS)] =	0.8687
σ[ln(FS)] =	0.9320
V [ln(FS)] =	0.7168

b =	**1.395**
Pr(f) =	8.15E-02
1 in	**12**

L25-21TA.XLS Summary

Exit Gradient Computations				**Line 25; Lake = 21.0; Mean TW = 12.0**		
	At pipe entrance					**Upper Pipe**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**0.10**	**13.00**	**12.55**	**450.07**	**447.10**	**0.152**
1	0.11	13.50	12.55	450.07	447.02	0.311
2	0.09	13.00	12.54	450.11	443.64	0.071
5	0.09	13.00	12.56	450.03	445.89	0.106
6	0.13	13.50	12.54	450.18	445.34	0.198
7	0.09	13.00	12.55	450.04	445.54	0.100
8	0.10	13.50	12.54	450.05	445.88	0.230
17	0.12	13.00	12.56	450.04	446.76	0.134
18	0.13	13.50	12.53	450.02	444.79	0.185
21	0.10	13.00	12.55	450.07	447.10	0.152
22	0.10	13.00	12.55	450.07	447.10	0.152
23	0.09	14.00	12.56	450.04	446.00	0.356
24	0.07	13.00	12.54	450.08	444.81	0.152

L25-21TA.XLS Upper Pipe, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Lines 23 - 38**

Deterministic Model: FASTSEEP **Lake Level: 21.0**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS Model based on data at Line 25
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	Upper	Peat	Peat	Clay	Clay			Gravel	**Upper Pipe, ih** Results		Prob	Calcs
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Pipe	i crit	Pocket	i exit	**FS**	Var	%
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**3.54E+00**	**0.531**	**-32.0**	**0.152**	**3.507**		
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.311**	1.706		
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.071**	7.473	8.3149	62.18%
5	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.106**	4.999		
6	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.198**	2.679	1.3461	10.07%
7	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	3.54E+00	0.531	-32.0	**0.100**	5.313		
8	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	3.54E+00	0.531	-32.0	**0.230**	2.308	2.2578	16.88%
17	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**8.13E+00**	0.531	-32.0	**0.134**	3.961		
18	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**1.54E+00**	0.531	-32.0	**0.185**	2.865	0.3003	2.25%
21	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.587	-32.0	**0.152**	3.874		
22	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.474	-32.0	**0.152**	3.128	0.1391	1.04%
23	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	**24.0**	**0.356**	1.491		
24	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	3.54E+00	0.531	**-88.0**	**0.152**	3.505	1.0141	7.58%
										Sum:	13.3721	100.00%

		For Piping	*For Breach*						
				E[FS] =	**3.507**	E[ln(FS)] =	0.8867		
Critical Gradient:	Avg	0.462	0.531	Var[FS] =	13.3721	Var[ln(FS)] =	0.7360	b =	**1.034**
	High	0.511	0.587	s**[FS] =**	**3.6568**	σ[ln(FS)] =	0.8579	Pr(f) =	1.51E-01
	Low	0.413	0.474	V [FS] =	1.0428	V [ln(FS)] =	0.9676	**1 in**	**7**

L25-24TA.XLS Summary

Exit Gradient Computations				**Line 25; Lake = 24.0; Mean TW = 12.5**		
	At pipe entrance					**Upper Pipe**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**0.10**	**13.60**	**13.20**	**450.02**	**448.19**	**0.219**
1	0.10	14.00	13.21	450.01	448.16	0.427
2	0.10	13.50	13.19	450.04	447.02	0.103
5	0.10	13.50	13.21	450.04	448.16	0.154
6	0.10	14.00	13.19	450.01	447.22	0.290
7	0.10	13.50	13.20	450.06	447.92	0.140
8	0.10	14.00	13.20	450.07	447.51	0.313
17	0.10	13.50	13.22	450.02	448.60	0.197
18	0.10	13.50	13.18	450.04	448.76	0.250
21	0.10	13.60	13.20	450.02	448.19	0.219
22	0.10	13.60	13.20	450.02	448.19	0.219
23	0.10	15.00	13.22	450.04	446.30	0.476
24	0.10	13.50	13.19	450.06	447.55	0.124

L25-24TA.XLS Upper Pipe, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Lines 23 - 38**

Deterministic Model: FASTSEEP **Lake Level:** **24.0**

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS Model based on data at Line 25

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

									Upper Pipe, ih			
	Upper	Peat	Peat	Clay	Clay			Gravel	Results		Prob	Calcs
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Pipe	i crit	Pocket	i exit	**FS**	Var	%
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**3.54E+00**	**0.531**	**-32.0**	**0.219**	**2.429**		
1	**3.25E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.427**	1.243		
2	**6.15E-04**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.103**	5.173	3.8602	46.60%
5	1.41E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.154**	3.442		
6	1.41E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	3.54E+00	0.531	-32.0	**0.290**	1.829	0.6507	7.86%
7	1.41E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	3.54E+00	0.531	-32.0	**0.140**	3.788		
8	1.41E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	3.54E+00	0.531	-32.0	**0.313**	1.699	1.0906	13.17%
17	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**8.13E+00**	0.531	-32.0	**0.197**	2.693		
18	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**1.54E+00**	0.531	-32.0	**0.250**	2.124	0.0809	0.98%
21	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.587	-32.0	**0.219**	2.686		
22	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.474	-32.0	**0.219**	2.169	0.0668	0.81%
23	1.41E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	0.531	**24.0**	**0.476**	1.116		
24	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	3.54E+00	0.531	**-88.0**	**0.124**	4.299	2.5340	30.59%
										Sum:	8.2832	100.00%

		For Piping	*For Breach*						
				E[FS] =	**2.429**	E[ln(FS)] =	0.4491		
Critical Gradient:	Avg	0.462	0.531	Var[FS] =	8.2832	Var[ln(FS)] =	0.8769	b =	**0.480**
	High	0.511	0.587	s **[FS] =**	**2.8781**	σ[ln(FS)] =	0.9365	Pr(f) =	3.16E-01
	Low	0.413	0.474	V [FS] =	1.1847	V [ln(FS)] =	2.0850	**1 in**	**3**

L25-TACL.XLS Summary

Exit Gradient Computations					**Line 25; Lake = 18.0; Mean TW = 11.5**							
	At clay base					**Ditch Bottom**	At ditch toe					
Run	X	Head +	Head -	Y+	Y-	**iv**	X	Head +	Head -	Y+	Y-	**iv**
E	**488.00**	**17.80**	**11.50**	**10.00**	**-4.00**	**0.450**						**#DIV/0!**
5	488.00	17.68	11.50	10.00	-4.00	0.441						#DIV/0!
6	488.00	17.87	11.50	10.00	-4.00	0.455						#DIV/0!
7	488.00	17.78	11.50	10.00	-4.00	0.449						#DIV/0!
8	488.00	17.82	11.50	10.00	-4.00	0.451						#DIV/0!
9	488.00	17.89	11.50	10.00	-4.00	0.456						#DIV/0!
10	488.00	17.60	11.50	10.00	-4.00	0.436						#DIV/0!
13	488.00	17.81	11.50	10.00	-4.00	0.451						#DIV/0!
14	488.00	17.78	11.50	10.00	-4.00	0.449						#DIV/0!
19	488.00	17.80	12.00	10.00	-4.00	0.414						
20	488.00	17.80	11.00	10.00	-4.00	0.486						
21	488.00	17.80	11.50	10.00	-4.00	0.450						#DIV/0!
22	488.00	17.80	11.50	10.00	-4.00	0.450						#DIV/0!

Exit Gradient Computations					**Line 25; Lake = 21.0; Mean TW = 12.0**							
	At clay base					**Ditch Bottom**	At ditch toe					**Ditch Bottom**
Run	X	Head +	Head -	Y+	Y-	**iv**	X	Head +	Head -	Y+	Y-	**iv**
E	**488.00**	**20.67**	**12.00**	**10.00**	**-4.00**	**0.619**						**#DIV/0!**
5	488.00	20.50	12.00	10.00	-4.00	0.607						#DIV/0!
6	488.00	20.78	12.00	10.00	-4.00	0.627						#DIV/0!
9	488.00	20.83	12.00	10.00	-4.00	0.631						#DIV/0!
10	488.00	20.36	12.00	10.00	-4.00	0.597						#DIV/0!
15	488.00	20.67	12.00	10.00	-4.00	0.619						
16	488.00	20.67	12.00	10.00	-4.00	0.619						
17	488.00	20.65	12.00	10.00	-4.00	0.618						
18	488.00	20.73	12.00	10.00	-4.00	0.624						
19	488.00	20.68	12.70	10.00	-4.00	0.570						
20	488.00	20.67	11.30	10.00	-4.00	0.669						
21	488.00	20.67	12.00	10.00	-4.00	0.619						#DIV/0!
22	488.00	20.67	12.00	10.00	-4.00	0.619						#DIV/0!

L25-TACL.XLS Summary

Exit Gradient Computations					**Line 25; Lake = 24.0; Mean TW = 12.5**							
	At clay base					**Ditch Bottom**	At ditch toe					**Ditch Bottom**
Run	X	Head +	Head -	Y+	Y-	**iv**	X	Head +	Head -	Y+	Y-	**iv**
E	**488.00**	**23.54**	**12.50**	**10.00**	**-4.00**	**0.789**						
5	488.00	23.32	12.50	10.00	-4.00	0.773						#DIV/0!
6	488.00	23.69	12.50	10.00	-4.00	0.799						#DIV/0!
9	488.00	23.76	12.50	10.00	-4.00	0.804						#DIV/0!
10	488.00	23.12	12.50	10.00	-4.00	0.759						#DIV/0!
19	488.00	23.55	13.30	10.00	-4.00	0.732						
20	488.00	23.52	11.70	10.00	-4.00	0.844						
21	488.00	23.54	12.50	10.00	-4.00	0.789	0.00	0.00	0.00	0.00	0.00	0.000
22	488.00	23.54	12.50	10.00	-4.00	0.789	0.00	0.00	0.00	0.00	0.00	0.000

L25-TACL.XLS Ditch Bottom, iv(18)

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Pre-damage Ditch Bottom

Deterministic Model: FASTSEEP

Lake Level: 18.0

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Model based on data at Line 25

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Piping of sand thru fractures in clay

Ditch Bottom, iv

Run	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Rock Horiz.	Rock Vert.	Entrance Valve	Ditch TW	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**5.30E-02**	**12.5**	**0.750**	**0.450**	**1.667**		
5	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	5.30E-02	12.5	0.750	**0.441**	1.699		
6	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	5.30E-02	12.5	0.750	**0.455**	1.648	0.0006	1.77%
7	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	5.30E-02	12.5	0.750	**0.449**	1.672		
8	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	5.30E-02	12.5	0.750	**0.451**	1.661	0.0000	0.08%
9	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	5.30E-02	12.5	0.750	**0.456**	1.643		
10	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	5.30E-02	12.5	0.750	**0.436**	1.721	0.0015	4.21%
13	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.22E-01**	12.5	0.750	**0.451**	1.664		
14	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.31E-02**	12.5	0.750	**0.449**	1.672	0.0000	0.04%
19	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	5.30E-02	**13.3**	0.750	**0.414**	1.810		
20	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	5.30E-02	**11.7**	0.750	**0.486**	1.544	0.0177	48.87%
21	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	5.30E-02	12.5	**0.808**	**0.450**	1.796		
22	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	5.30E-02	12.5	**0.693**	**0.450**	1.540	0.0163	45.03%
											Sum:	0.0363	100.00%

		For Piping	For Breach						
Critical Gradient:	Avg	0.650	0.750	**E[FS] =**	**1.667**	E[ln(FS)] =	0.5043	**b =**	**4.429**
	High	0.700	0.808	Var[FS] =	0.0363	Var[ln(FS)] =	0.0130	Pr(f) =	4.74E-06
	Low	0.600	0.693	**s[FS] =**	**0.1904**	s[ln(FS)] =	0.1139	**1 in**	**210841**
				V [FS] =	0.1142	V [ln(FS)] =	0.2258		

L25-TACL.XLS Ditch Bottom, iv(21)

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Pre-damage Ditch Bottom

Lake Level: **21.0**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Model based on data at Line 25

Piping of sand thru fractures in clay

Ditch Bottom, iv

Run	Peat Horiz.	Peat Vert.	Rock Horiz.	Rock Vert.	Exit Valve	Pipe	Ditch TW	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**1.60E-04**	**4.61E-05**	**1.77E-01**	**1.41E-02**	**1.41E-01**	**3.54E+00**	**12.5**	**0.750**	**0.619**	**1.211**		
5	**3.12E-04**	**9.53E-05**	1.77E-01	1.41E-02	1.41E-01	3.54E+00	12.5	0.750	**0.607**	1.235		
6	**8.20E-05**	**2.23E-05**	1.77E-01	1.41E-02	1.41E-01	3.54E+00	12.5	0.750	**0.627**	1.196	0.0004	1.97%
9	1.60E-04	4.61E-05	**4.06E-01**	**3.25E-02**	1.41E-01	3.54E+00	12.5	0.750	**0.631**	1.189		
10	1.60E-04	4.61E-05	**7.69E-02**	**6.15E-03**	1.41E-01	3.54E+00	12.5	0.750	**0.597**	1.256	0.0011	5.68%
15	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**3.25E-01**	3.54E+00	12.5	0.750	**0.619**	1.211		
16	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**6.15E-02**	3.54E+00	12.5	0.750	**0.619**	1.211	0.0000	0.00%
17	1.60E-04	4.61E-05	1.77E-01	1.41E-02	1.41E-01	**8.13E+00**	12.5	0.750	**0.618**	1.214		
18	1.60E-04	4.61E-05	1.77E-01	1.41E-02	1.41E-01	**1.54E+00**	12.5	0.750	**0.624**	1.203	0.0000	0.16%
19	1.60E-04	4.61E-05	1.77E-01	1.41E-02	1.41E-01	3.54E+00	**13.3**	0.750	**0.570**	1.316		
20	1.60E-04	4.61E-05	1.77E-01	1.41E-02	1.41E-01	3.54E+00	**11.7**	0.750	**0.669**	1.121	0.0095	48.39%
21	1.60E-04	4.61E-05	1.77E-01	1.41E-02	1.41E-01	3.54E+00	12.5	**0.808**	**0.619**	1.305		
22	1.60E-04	4.61E-05	1.77E-01	1.41E-02	1.41E-01	3.54E+00	12.5	**0.693**	**0.619**	1.119	0.0086	43.80%
										Sum:	0.0197	100.00%

		For Piping	*For Breach*							
				E[FS] =	**1.211**		E[ln(FS)] =	0.1848		
radient:	Avg	0.650	0.750	Var[FS] =	0.0197		Var[ln(FS)] =	0.0133	**b =**	**1.601**
	High	0.700	0.808	**s[FS] =**	**0.1403**		s[ln(FS)] =	0.1155	Pr(f) =	5.47E-02
	Low	0.600	0.693	V [FS] =	0.1158		V [ln(FS)] =	0.6246	**1 in**	**18**

L25-TACL.XLS Ditch Bottom, iv(24)

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Pre-damage Ditch Bottom

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Lake Level: **24.0**

Model based on data at Line 25

Piping of sand thru fractures in clay

Ditch Bottom, iv

	Peat	Peat	Rock	Rock	Ditch		Results		Prob Calcs	
Run	Horiz.	Vert.	Horiz.	Vert.	TW	i crit	i exit	FS	Var	%
E	**1.60E-04**	**4.61E-05**	**1.77E-01**	**1.41E-02**	**12.5**	**0.750**	**0.789**	**0.951**		
5	**3.12E-04**	**9.53E-05**	1.77E-01	1.41E-02	12.5	0.750	**0.773**	0.970		
6	**8.20E-05**	**2.23E-05**	1.77E-01	1.41E-02	12.5	0.750	**0.799**	0.938	0.0003	2.34%
9	1.60E-04	4.61E-05	**4.06E-01**	**3.25E-02**	12.5	0.750	**0.804**	0.933		
10	1.60E-04	4.61E-05	**7.69E-02**	**6.15E-03**	12.5	0.750	**0.759**	0.989	0.0008	7.18%
19	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**13.3**	0.750	**0.732**	1.024		
20	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**11.7**	0.750	**0.844**	0.888	0.0046	42.11%
21	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.5	**0.808**	**0.789**	1.025		
22	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.5	**0.693**	**0.789**	0.879	0.0053	48.37%
								Sum:	0.0110	100.00%

E[FS] =	**0.951**	E[ln(FS)] =	-0.0562
Var[FS] =	0.0110	Var[ln(FS)] =	0.0121
s[FS] =	**0.1048**	s[ln(FS)] =	0.1099
V [FS] =	0.1102	V [ln(FS)] =	-1.9559

b = **-0.511**

Pr(f) = 6.95E-01

1 in **1**

	For Piping	*For Breach*
Critical Gradient: Avg	0.650	0.750
High	0.700	0.808
Low	0.600	0.693

L25-TAF.XLS Summary

Exit Gradient Computations					**Line 25; Lake = 18.0; Mean TW = 11.5**							
	At pipe entrance					**Upper Pipe**	At ditch toe					**Ditch Bottom**
Run	Y	Head +	Head -	X +	X -	**ih**	X	Head +	Head -	Y+	Y-	**iv**
E	**8.14**	**12.50**	**11.90**	**425.95**	**418.56**	**0.081**	**484.00**	**11.70**	**11.50**	**10.00**	**5.75**	**0.047**
1	8.13	12.50	11.91	426.07	420.04	0.098	484.00	11.69	11.50	10.00	6.01	0.048
2	8.13	12.21	11.89	426.07	420.40	0.056	484.00	11.70	11.50	10.00	5.80	0.048
3	8.14	12.50	11.89	426.12	418.58	0.081	484.00	11.69	11.50	10.00	5.98	0.047
4	8.12	12.50	11.89	426.06	418.48	0.080	484.00	11.69	11.50	10.00	6.03	0.048
5	8.14	12.50	11.90	426.09	416.46	0.062	484.00	11.70	11.50	10.00	5.89	0.049
6	8.15	12.50	11.89	426.09	419.73	0.096	484.00	11.70	11.50	10.00	5.79	0.048
7	8.13	12.50	11.90	426.09	418.31	0.077	484.00	11.70	11.50	10.00	5.75	0.047
8	8.13	12.50	11.89	426.06	419.01	0.087	484.00	11.70	11.50	10.00	5.82	0.048
17	8.18	12.50	11.90	426.12	418.18	0.076	484.00	11.69	11.50	10.00	6.02	0.048
18	8.13	12.50	11.90	425.98	418.82	0.084	484.00	11.68	11.50	10.00	5.99	0.045
19	8.13	12.50	12.36	426.13	424.15	0.071	484.00	12.17	12.00	10.00	5.97	0.042
20	8.09	12.00	11.43	426.02	419.65	0.089	484.00	11.20	11.00	10.00	6.01	0.050
21	8.14	12.50	11.90	425.95	418.56	0.081	484.00	11.70	11.50	10.00	5.75	0.047
22	8.14	12.50	11.90	425.95	418.56	0.081	484.00	11.70	11.50	10.00	5.75	0.047
23	8.09	13.00	11.92	426.05	421.56	0.241	484.00	11.70	11.50	10.00	5.99	0.050
24	8.21	12.00	11.89	426.01	423.15	0.038	484.00	11.68	11.50	10.00	5.99	0.045

L25-TAF.XLS Summary

Exit Gradient Computations					**Line 25; Lake = 21.0; Mean TW = 12.0**							
	At pipe entrance					**Upper Pipe**	At ditch toe					**Ditch Bottom**
Run	Y	Head +	Head -	X +	X -	**ih**	X	Head +	Head -	Y+	Y-	**iv**
E	**8.11**	**13.50**	**12.55**	**426.09**	**418.21**	**0.121**	**484.00**	**11.70**	**11.50**	**10.00**	**5.75**	**0.047**
1	8.15	13.50	12.57	426.12	419.68	0.144	484.00	11.69	11.50	10.00	6.01	0.048
2	8.16	13.50	12.54	426.14	414.77	0.084	484.00	11.70	11.50	10.00	5.80	0.048
5	8.10	13.50	12.56	426.07	416.25	0.096	484.00	11.70	11.50	10.00	5.89	0.049
6	8.14	13.50	12.55	426.06	419.32	0.141	484.00	11.70	11.50	10.00	5.79	0.048
19	8.14	14.00	13.21	426.08	418.74	0.108	484.00	12.17	12.00	10.00	5.97	0.042
20	8.07	12.50	11.89	426.07	421.57	0.136	484.00	11.20	11.00	10.00	6.01	0.050
21	8.11	13.50	12.55	426.09	418.21	0.121	484.00	11.70	11.50	10.00	5.75	0.047
22	8.11	13.50	12.55	426.09	418.21	0.121	484.00	11.70	11.50	10.00	5.75	0.047
23	8.07	14.50	12.59	426.04	420.61	0.352	484.00	11.70	11.50	10.00	5.99	0.050
24	8.14	13.00	12.54	426.10	418.25	0.059	484.00	11.68	11.50	10.00	5.99	0.045

Exit Gradient Computations					**Line 25; Lake = 15.0; Mean TW = 11.0**							
	At pipe entrance					**Upper Pipe**	At ditch toe					**Ditch Bottom**
Run	Y	Head +	Head -	X +	X -	**ih**	X	Head +	Head -	Y+	Y-	**iv**
E	**8.18**	**11.50**	**11.24**	**426.11**	**419.80**	**0.041**	**484.00**	**11.70**	**11.50**	**10.00**	**5.75**	**0.047**
1	8.13	11.50	11.24	426.11	421.41	0.055	484.00					#DIV/0!
2	8.13	11.40	11.24	426.06	420.55	0.029	484.00					#DIV/0!
5	8.12	11.40	11.24	426.11	421.26	0.033	484.00					#DIV/0!
6	8.14	11.50	11.24	426.09	421.18	0.053	484.00					#DIV/0!
19	8.14	11.68	11.52	426.11	422.12	0.040	484.00					#DIV/0!
20	8.09	11.12	10.96	426.08	422.70	0.047	484.00					#DIV/0!
21	8.18	11.50	11.24	426.11	419.80	0.041	484.00	11.70	11.50	10.00	5.75	0.047
22	8.18	11.50	11.24	426.11	419.80	0.041	484.00	11.70	11.50	10.00	5.75	0.047
23	8.07	11.50	11.25	426.05	424.25	0.139	484.00					#DIV/0!
24	8.18	11.33	11.23	426.13	421.57	0.022	484.00					#DIV/0!

L25-TAF.XLS Summary

Exit Gradient Computations					**Line 25; Lake = 24.0; Mean TW = 12.5**							
	At pipe entrance					**Upper Pipe**	At ditch toe					**Ditch Bottom**
Run	Y	Head +	Head -	X +	X -	**ih**	X	Head +	Head -	Y+	Y-	**iv**
E	**8.11**	**13.50**	**13.21**	**426.12**	**424.23**	**0.153**	**484.00**	**11.70**	**11.50**	**10.00**	**5.75**	**0.047**
1	8.11	13.50	13.24	426.09	424.69	0.186	484.00					#DIV/0!
2	8.09	13.50	13.19	426.04	423.50	0.122	484.00					#DIV/0!
5	8.07	13.50	13.22	426.03	423.97	0.136	484.00					#DIV/0!
6	8.10	13.50	13.20	426.05	424.45	0.187	484.00					#DIV/0!
21	8.11	13.50	13.21	426.12	424.23	0.153	484.00	11.70	11.50	10.00	5.75	0.047
22	8.11	13.50	13.21	426.12	424.23	0.153	484.00	11.70	11.50	10.00	5.75	0.047
23	8.10	14.00	13.26	426.06	424.39	0.443	484.00					#DIV/0!
24	8.11	13.50	13.19	426.07	422.36	0.084	484.00					#DIV/0!

L25-TAF.XLS Upper Pipe, ih(15)

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Failure Mode Model

Lake Level: 15.0

Model based on data at Line 25

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Upper Pipe, ih

Run	Upper Sands	Peat Horiz.	Peat Vert.	Ditch Tailwater	i crit	Gravel Pocket	Results i exit	Results **FS**	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**12.0**	**0.531**	**-32.0**	**0.041**	**12.894**		
1	**3.25E-03**	1.60E-04	4.61E-05	12.0	0.531	-32.0	**0.055**	9.604		
2	**6.15E-04**	1.60E-04	4.61E-05	12.0	0.531	-32.0	**0.029**	18.297	18.8893	13.99%
5	1.41E-03	**3.12E-04**	**9.53E-05**	12.0	0.531	-32.0	**0.033**	16.105		
6	1.41E-03	**8.20E-05**	**2.23E-05**	12.0	0.531	-32.0	**0.053**	10.033	9.2162	6.82%
19	1.41E-03	1.60E-04	4.61E-05	**12.7**	0.531	-32.0	**0.040**	13.249		
20	1.41E-03	1.60E-04	4.61E-05	**11.3**	0.531	-32.0	**0.047**	11.224	1.0257	0.76%
21	1.41E-03	1.60E-04	4.61E-05	12.0	**0.588**	-32.0	**0.041**	14.262		
22	1.41E-03	1.60E-04	4.61E-05	12.0	**0.475**	-32.0	**0.041**	11.527	1.8703	1.38%
23	1.41E-03	1.60E-04	4.61E-05	12.0	0.531	24.0	**0.139**	3.825		
24	1.41E-03	1.60E-04	4.61E-05	12.0	0.531	-88.0	**0.022**	24.227	104.0596	77.05%
								Sum:	135.0611	100.00%

E[FS] =	**12.894**	E[ln(FS)] =	2.2595
Var[FS] =	135.0611	Var[ln(FS)] =	0.5946
s**[FS] =**	**11.6216**	σ[ln(FS)] =	0.7711
V [FS] =	0.9013	V [ln(FS)] =	0.3413

		For Piping	*For Breach*
Critical Gradient:	Avg	0.462	0.531
	High	0.511	0.588
	Low	0.413	0.475

b **=**	**2.930**
Pr(f) =	1.69E-03
1 in	**590**

L25-TAF.XLS Upper Pipe, ih(18)

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Failure Mode Model

Lake Level: **18.0**

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Model based on data at Line 25

Run	Upper Sands	Gravel	Peat Horiz.	Peat Vert.	Clay Horiz.	Clay Vert.	Pipe	Ditch Tailwater	i crit	Gravel Pocket	Upper Pipe, ih Results: i exit	Upper Pipe, ih Results: **FS**	Prob Var	Calcs %
E	**1.41E-03**	**7.07E-01**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**3.54E+00**	**11.5**	**0.531**	**-32.0**	**0.081**	**6.544**		
1	**3.25E-03**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-32.0	**0.098**	5.430		
2	**6.15E-04**	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-32.0	**0.056**	9.414	3.9679	9.52%
3	1.41E-03	**1.63E+00**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-32.0	**0.081**	6.567		
4	1.41E-03	**3.08E-01**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-32.0	**0.080**	6.602	0.0003	0.00%
5	1.41E-03	7.07E-01	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-32.0	**0.062**	8.527		
6	1.41E-03	7.07E-01	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-32.0	**0.096**	5.539	2.2319	5.35%
7	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	3.54E+00	11.5	0.531	-32.0	**0.077**	6.889		
8	1.41E-03	7.07E-01	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	3.54E+00	11.5	0.531	-32.0	**0.087**	6.140	0.1402	0.34%
`17	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**8.13E+00**	11.5	0.531	-32.0	**0.076**	7.031		
18	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**1.54E+00**	11.5	0.531	-32.0	**0.084**	6.340	0.1193	0.29%
19	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	**12.0**	0.531	-32.0	**0.071**	7.514		
20	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	**11.0**	0.531	-32.0	**0.089**	5.938	0.6214	1.49%
21	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	**0.588**	-32.0	**0.081**	7.238		
22	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	**0.475**	-32.0	**0.089**	5.308	0.9313	2.23%
23	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	24.0	**0.241**	2.209		
24	1.41E-03	7.07E-01	1.60E-04	4.61E-05	1.27E-04	7.11E-05	3.54E+00	11.5	0.531	-88.0	**0.038**	13.814	33.6688	80.78%
												Sum:	41.6811	100.00%

				For Piping	*For Breach*	**E[FS] =**	**6.544**			E[ln(FS)] =	1.5387		
		Critical Gradient:	Avg	0.462	0.531	Var[FS] =	41.6811			Var[ln(FS)] =	0.6797	b =	**1.866**
			High	0.511	0.588	s **[FS] =**	**6.4561**			σ[ln(FS)] =	0.8245	Pr(f) =	3.10E-02
			Low	0.413	0.475	V [FS] =	0.9866			V [ln(FS)] =	0.5358	**1 in**	**32**

L25-TAF.XLS Upper Pipe, ih(21)

Probabilistic Underseepage Analysis:

Herbert Hoover Dike; Model for Lines 23 - 38

Failure Mode Model

Lake Level: 21.0

Model based on data at Line 25

Deterministic Model: FASTSEEP

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS

Random Variable

Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Upper Pipe, ih

Run	Upper Sands	Peat Horiz.	Peat Vert.	Ditch Tailwater	i crit	Gravel Pocket	Results i exit	Results **FS**	Prob Calcs Var	Prob Calcs %
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**12.0**	**0.531**	**-32.0**	**0.121**	**4.407**		
1	**3.25E-03**	1.60E-04	4.61E-05	12.0	0.531	-32.0	**0.144**	3.681		
2	**6.15E-04**	1.60E-04	4.61E-05	12.0	0.531	-32.0	**0.084**	6.293	1.7053	9.89%
5	1.41E-03	**3.12E-04**	**9.53E-05**	12.0	0.531	-32.0	**0.096**	5.550		
6	1.41E-03	**8.20E-05**	**2.23E-05**	12.0	0.531	-32.0	**0.141**	3.769	0.7930	4.60%
19	1.41E-03	1.60E-04	4.61E-05	**12.7**	0.531	-32.0	**0.108**	4.936		
20	1.41E-03	1.60E-04	4.61E-05	**11.3**	0.531	-32.0	**0.136**	3.919	0.2585	1.50%
21	1.41E-03	1.60E-04	4.61E-05	12.0	**0.588**	-32.0	**0.121**	4.874		
22	1.41E-03	1.60E-04	4.61E-05	12.0	**0.475**	-32.0	**0.121**	3.940	0.2185	1.27%
23	1.41E-03	1.60E-04	4.61E-05	12.0	0.531	24.0	**0.352**	1.510		
24	1.41E-03	1.60E-04	4.61E-05	12.0	0.531	-88.0	**0.059**	9.067	14.2744	82.75%
								Sum:	17.2497	100.00%

E[FS] =	**4.407**	E[ln(FS)] =	1.1654
Var[FS] =	17.2497	Var[ln(FS)] =	0.6356
s **[FS] =**	**4.1533**	σ[ln(FS)] =	0.7973
V [FS] =	0.9424	V [ln(FS)] =	0.6841

b **=**	**1.462**		
Pr(f) =	7.19E-02		
1 in	**14**		

	For Piping	*For Breach*
Critical Gradient: Avg	0.462	0.531
High	0.511	0.588
Low	0.413	0.475

L25-TAF.XLS Upper Pipe, ih(24)

Probabilistic Underseepage Analysis:							**Herbert Hoover Dike; Model for Lines 23 - 38**		
							Failure Mode Model		
	Deterministic Model:		FASTSEEP				**Lake Level:**	**24.0**	
	Probabilistic Model:		Taylor's Series, Finite Difference						
	Performance Function:		FS				Model based on data at Line 25		
	Random Variable								
	Distribution Assumptions:		K's lognormal, i crit and ditch water level normal						
						Upper Pipe, ih			
	Upper	Peat	Peat		Gravel	Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	i crit	Pocket	i exit	**FS**	Var	%
E	**1.41E-03**	**1.60E-04**	**4.61E-05**	**0.531**	**-32.0**	**0.153**	**3.463**		
1	**3.25E-03**	1.60E-04	4.61E-05	0.531	-32.0	**0.186**	2.861		
2	**6.15E-04**	1.60E-04	4.61E-05	0.531	-32.0	**0.122**	4.353	0.5568	7.29%
5	1.41E-03	**3.12E-04**	**9.53E-05**	0.531	-32.0	**0.136**	3.909		
6	1.41E-03	**8.20E-05**	**2.23E-05**	0.531	-32.0	**0.187**	2.834	0.2890	3.79%
21	1.41E-03	1.60E-04	4.61E-05	**0.588**	-32.0	**0.153**	3.830		
22	1.41E-03	1.60E-04	4.61E-05	**0.475**	-32.0	**0.153**	3.095	0.1349	1.77%
23	1.41E-03	1.60E-04	4.61E-05	0.531	24.0	**0.443**	1.199		
24	1.41E-03	1.60E-04	4.61E-05	0.531	-88.0	**0.084**	6.358	6.6550	87.16%
							Sum:	7.6357	100.00%
	E[FS] =	**3.463**			E[ln(FS)] =	0.9956			
	Var[FS] =	7.6357			Var[ln(FS)] =	0.4928		b =	**1.418**
	s **[FS] =**	**2.7633**			σ[ln(FS)] =	0.7020		Pr(f) =	7.80E-02
	V [FS] =	0.7980			V [ln(FS)] =	0.7051		**1 in**	**13**
				For Piping	*For Breach*				
	Critical Gradient:		Avg	0.462	0.531				
			High	0.511	0.588				
			Low	0.413	0.475				

L25-SSTA.XLS L25-18 Clay

Probabilistic Slope Stability Analysis: **Herbert Hoover Dike; Model for Lines 23 - 38**

Deterministic Model: UTEXAS, Spencer's Method
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
stribution Assumptions: Cohesion lognormal, Phi, gamma & Tailwater normal

Lake Level: **18.0**

Model based on data at Line 25

Failure thgrough the clay tangent to the limestone.
Used higher embankment strength.

	Embankment Sands			Peat / OL / OH				Clays & Silts			Ditch	Failure Arc	**Computed**	Prob Calcs	
Run	θ	γ moist	γ sat	θ	C	γ moist	γ sat	θ	C	γ sat	Tailwater	X, Y, R	**FS**	Var	%
E	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.261**		
1	39.6	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.333**		
2	32.4	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	79,81,85	**2.195**	0.0048	2.25%
3	36.0	112.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.223**		
4	36.0	102.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.302**	0.0016	0.74%
5	36.0	107.0	121.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.249**		
6	36.0	107.0	109.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.273**	0.0001	0.07%
7	36.0	107.0	115.0	41.3	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.442**		
8	36.0	107.0	115.0	24.8	160.0	57.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.111**	0.0274	12.97%
9	36.0	107.0	115.0	33.0	400.0	57.0	70.0	12.5	620.0	100.0	11.5	78,84,88	**2.516**		
10	36.0	107.0	115.0	33.0	80.0	57.0	70.0	12.5	620.0	100.0	11.5	81,75,79	**2.169**	0.0301	14.25%
11	36.0	107.0	115.0	33.0	160.0	63.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.268**		
12	36.0	107.0	115.0	33.0	160.0	51.0	70.0	12.5	620.0	100.0	11.5	80,78,82	**2.254**	0.0000	0.02%
13	36.0	107.0	115.0	33.0	160.0	57.0	74.0	12.5	620.0	100.0	11.5	80,78,82	**2.298**		
14	36.0	107.0	115.0	33.0	160.0	57.0	64.0	12.5	620.0	100.0	11.5	80,78,82	**2.234**	0.0010	0.48%
15	36.0	107.0	115.0	33.0	160.0	57.0	70.0	15.6	620.0	100.0	11.5	80,78,82	**2.348**		
16	36.0	107.0	115.0	33.0	160.0	57.0	70.0	9.4	620.0	100.0	11.5	80,78,82	**2.176**	0.0074	3.50%
17	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	1020.0	100.0	11.5	78,84,88	**2.709**		
18	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	380.0	100.0	11.5	84,66,70	**1.964**	0.1388	65.69%
19	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	105.0	11.5	80,78,82	**2.264**		
20	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	95.0	11.5	80,78,82	**2.258**	0.0000	0.00%
21	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	12.0	80,78,82	**2.250**		
22	36.0	107.0	115.0	33.0	160.0	57.0	70.0	12.5	620.0	100.0	11.0	80,78,82	**2.260**	0.0000	0.01%
													Sum:	0.2112	100.00%

E[FS] =	**2.261**	E[ln(FS)] =	0.7956
Var[FS] =	0.2112	Var[ln(FS)] =	0.0405
s **[FS] =**	**0.4596**	σ[ln(FS)] =	0.2012
V [FS] =	0.2033	V [ln(FS)] =	0.2529

b **=**	**3.954**		
Pr(f) =	3.85E-05		
1 in	**25997**		

Adjusted probabilities based on length effect:	**Total Length =**	15,330	
	"Link" Length =	1500	
	No. Links =	10.22	

Reliability of Section, R =	1.00E+00
Adjusted Pr(f) =	**3.93E-04**
1 in	**2544**

L25-SSTA.XLS L25-18

Probabilistic Slope Stability Analysis: **Herbert Hoover Dike: Model for Lines 23 - 38**

Deterministic Model: UTEXAS, Spencer's Method
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
stribution Assumptions: Cohesion lognormal, Phi, gamma & Tailwater normal

Lake Level: 18.0

Model based on data at Line 25

Failure through the peat tangent to the clay.

	Embankment Sands			Peat / OL / OH				Ditch	Failure Arc	**Computed**	Prob Calcs	
Run	θ	γ moist	γ sat	θ	C	γ moist	γ sat	Tailwater	X, Y, R	**FS**	Var	%
E	30.0	107.0	115.0	33.0	160.0	57.0	70.0	11.5	90,69,69	**2.042**		
1	33.0	107.0	115.0	33.0	160.0	57.0	70.0	11.5	90,68.5,68.5	**2.154**		
2	27.0	107.0	115.0	33.0	160.0	57.0	70.0	11.5	89.5,70.5,70.5	**1.935**	0.0120	5.70%
3	30.0	112.0	115.0	33.0	160.0	57.0	70.0	11.5	90,69,69	**2.022**		
4	30.0	102.0	115.0	33.0	160.0	57.0	70.0	11.5	90,69,69	**2.064**	0.0004	0.21%
5	30.0	107.0	121.0	33.0	160.0	57.0	70.0	11.5	90,69,69	**2.036**		
6	30.0	107.0	109.0	33.0	160.0	57.0	70.0	11.5	90,69,69	**2.049**	0.0000	0.02%
7	30.0	107.0	115.0	41.3	160.0	57.0	70.0	11.5	89,70,70	**2.315**		
8	30.0	107.0	115.0	24.8	160.0	57.0	70.0	11.5	92.5,60.5,60.5	**1.798**	0.0668	31.79%
9	30.0	107.0	115.0	33.0	400.0	57.0	70.0	11.5	89,72,72	**2.545**		
10	30.0	107.0	115.0	33.0	80.0	57.0	70.0	11.5	94,56.5,56.5	**1.833**	0.1267	60.30%
11	30.0	107.0	115.0	33.0	160.0	63.0	70.0	11.5	90,69,69	**2.055**		
12	30.0	107.0	115.0	33.0	160.0	51.0	70.0	11.5	90,69,69	**2.030**	0.0002	0.07%
13	30.0	107.0	115.0	33.0	160.0	57.0	74.0	11.5	90,69,69	**2.102**		
14	30.0	107.0	115.0	33.0	160.0	57.0	64.0	11.5	90,69,69	**1.998**	0.0027	1.29%
15	30.0	107.0	115.0	33.0	160.0	57.0	70.0	12.0	90.68.5,68.5	**2.082**		
16	30.0	107.0	115.0	33.0	160.0	57.0	70.0	11.0	90,69,69	**2.010**	0.0013	0.62%
										Sum:	0.2102	100.00%

E[FS] =	**2.042**	E[ln(FS)] =	0.6893
Var[FS] =	0.2102	Var[ln(FS)] =	0.0492
s **[FS] =**	**0.4585**	σ[ln(FS)] =	0.2218
V [FS] =	0.2245	V [ln(FS)] =	0.3217

b **=** **3.108**
Pr(f) = 9.40E-04
1 in **1063**

Adjusted probabilities based on length effect:

Total Length =	15,330
"Link" Length =	1500
No. Links =	10.22

Reliability of Section, R = 9.90E-01
Adjusted Pr(f) = 9.57E-03
1 in **105**

L25-SSTA.XLS L25-24

Probabilistic Slope Stability Analysis:					**Herbert Hoover Dike; Model for Lines 23 - 38**			
Deterministic Model:	UTEXAS, Spencer's Method				**Lake Level:**	**24.0**		
Probabilistic Model:	Taylor's Series, Finite Difference							
Performance Function:	FS				Model based on data at Line 25			
Random Variable					Failure through the peat tangent to the clay.			
Distribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater normal							
		Embankment Sands	Peat / OL / OH		Failure Arc	**Computed**	Prob Calcs	
	Run	θ	θ	C	X, Y, R	**FS**	Var	%
	E	30.0	33.0	160.0	90,69,69	**1.916**		
	1	33.0	33.0	160.0	90,68.5,68.5	**2.019**		
	2	27.0	33.0	160.0	89,72,72	**1.815**	0.0104	5.34%
	3	30.0	41.3	160.0	89,72,72	**2.166**		
	4	30.0	24.8	160.0	90,60.5,60.5	**1.695**	0.0555	28.48%
	5	30.0	33.0	400.0	88,68,68	**2.428**		
	6	30.0	33.0	80.0	94,59.35,59.5	**1.710**	0.1289	66.18%
						Sum:	0.1947	100.00%
	E[FS] =	**1.916**	E[ln(FS)] =	0.6244				
	Var[FS] =	0.1947	Var[ln(FS)] =	0.0517			b =	**2.746**
	s **[FS] =**	**0.4413**	σ[ln(FS)] =	0.2274			Pr(f) =	3.01E-03
	V [FS] =	0.2303	V [ln(FS)] =	0.3641			**1 in**	**332**
Adjusted probabilities based			**Total Length =**	15,330		Reliability of Section, R =		9.70E-01
on length effect:			**"Link" Length =**	1500			**Adjusted Pr(f) =**	**3.04E-02**
			No. Links =	10.22			**1 in**	**33**

L25-SSHT.XLS L25-15

Probabilistic Slope Stability Analysis:								**Herbert Hoover Dike; Model for Lines 23 - 38**			
Deterministic Model:	UTEXAS, Spencer's Method							**Lake Level:**	**15.0**		
Probabilistic Model:	Taylor's Series, Finite Difference										
erformance Function:	FS							Model based on data at Line 25			
Random Variable											
ribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater normal										
								Failure through the peat tangent to the clay.			
	Embankment Sands			Peat / OL / OH			Ditch	Failure Arc	**Computed**	Prob Calcs	
Run	θ	γ moist	γ sat	θ	C	γ sat	Tailwater	X, Y, R	**FS**	Var	%
E	30.0	107.0	115.0	33.0	160.0	70.0	13.0	90,68.5,68.5	**2.031**		
7	30.0	107.0	115.0	41.3	160.0	70.0	13.0	88.5,60,62.5	**2.212**		
8	30.0	107.0	115.0	24.8	160.0	70.0	13.0	92.5,60.5,60.5	**1.799**	0.0853	32.71%
9	30.0	107.0	115.0	33.0	400.0	70.0	13.0	88.5,60,62.5	**2.389**		
10	30.0	107.0	115.0	33.0	80.0	70.0	13.0	93,59,59	**1.819**	0.1625	62.31%
13	30.0	107.0	115.0	33.0	160.0	74.0	13.0	90,69,69	**2.096**		
14	30.0	107.0	115.0	33.0	160.0	64.0	13.0	90,68.5,68.5	**1.935**	0.0130	4.97%
									Sum:	0.2607	100.00%
				E[FS] =	**2.031**	E[ln(FS)] =	0.6779				
				Var[FS] =	0.2607	Var[ln(FS)] =	0.0613			b =	**2.738**
				s **[FS] =**	**0.5106**	σ[ln(FS)] =	0.2476			Pr(f) =	3.09E-03
				V [FS] =	0.2514	V [ln(FS)] =	0.3652			**1 in**	**324**

L25-SSHT.XLS L25-18

Probabilistic Slope Stability Analysis: **Herbert Hoover Dike; Model for Lines 23 - 38**

Deterministic Model: UTEXAS, Spencer's Method — **Lake Level: 18.0**

Probabilistic Model: Taylor's Series, Finite Difference

Performance Function: FS — Model based on data at Line 25

Random Variable

stribution Assumptions: Cohesion lognormal, Phi, gamma & Tailwater normal

Failure through the peat tangent to the clay.

	Embankment Sands			Peat / OL / OH			Ditch	Failure Arc	**Computed**	Prob Calcs	
Run	θ	γ moist	γ sat	θ	C	γ sat	Tailwater	X, Y, R	**FS**	Var	%
E	30.0	107.0	115.0	33.0	160.0	70.0	14.0	91,65,65	**1.872**		
1	33.0	107.0	115.0	33.0	160.0	70.0	14.0	91.5,63.5,63.5	**1.894**		
2	27.0	107.0	115.0	33.0	160.0	70.0	14.0	90.5,67,67	**1.850**	0.0010	0.24%
3	30.0	112.0	115.0	33.0	160.0	70.0	14.0	91.5,63.5,63.5	**1.855**		
4	30.0	102.0	115.0	33.0	160.0	70.0	14.0	91,65.5,65.5	**1.890**	0.0006	0.15%
5	30.0	107.0	121.0	33.0	160.0	70.0	14.0	91.5,63.5,63.5	**1.864**		
6	30.0	107.0	109.0	33.0	160.0	70.0	14.0	90.5,66.5,66.5	**1.879**	0.0001	0.03%
7	30.0	107.0	115.0	41.3	160.0	70.0	14.0	89.5,70.5,70.5	**2.096**		
8	30.0	107.0	115.0	24.8	160.0	70.0	14.0	92,54,54	**1.666**	0.0925	23.17%
9	30.0	107.0	115.0	33.0	400.0	70.0	14.0	87.5,66,66	**2.393**		
10	30.0	107.0	115.0	33.0	80.0	70.0	14.0	95,53,53	**1.635**	0.2873	72.00%
13	30.0	107.0	115.0	33.0	160.0	74.0	14.0	90.5,67,67	**1.943**		
14	30.0	107.0	115.0	33.0	160.0	64.0	14.0	92,62,62	**1.761**	0.0166	4.15%
21	30.0	107.0	115.0	33.0	160.0	70.0	14.5	90.5,65.5,65.5	**1.892**		
22	30.0	107.0	115.0	33.0	160.0	70.0	13.5	90.5,67,67	**1.937**	0.0010	0.25%
									Sum:	0.3990	100.00%

E[FS] =	**1.872**	E[ln(FS)] =	0.5731
Var[FS] =	0.3990	Var[ln(FS)] =	0.1078
s **[FS] =**	**0.6317**	σ[ln(FS)] =	0.3284
V [FS] =	0.3374	V [ln(FS)] =	0.5730

b **=**	**1.745**
Pr(f) =	4.05E-02
1 in	**25**

L25-SSHT.XLS L25-21

Probabilistic Slope Stability Analysis:								**Herbert Hoover Dike; Model for Lines 23 - 38**				
Deterministic Model:	UTEXAS, Spencer's Method							**Lake Level:**	**21.0**			
Probabilistic Model:	Taylor's Series, Finite Difference											
Performance Function:	FS							Model based on data at Line 25				
Random Variable												
Distribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater norma											
								Failure through the peat tangent to the clay.				
	Embankment Sands			Peat / OL / OH			Ditch	Failure Arc	**Computed**	Prob Calcs		
Run	θ	γ moist	γ sat	θ	C	γ sat	Tailwater	X, Y, R	**FS**	Var	%	
E	30.0	107.0	115.0	33.0	160.0	70.0	15.0	90,66.5,66.5	**1.854**			
7	30.0	107.0	115.0	41.3	160.0	70.0	15.0	89,72,72	**2.068**			
8	30.0	107.0	115.0	24.8	160.0	70.0	15.0	90.5,54.5,54.5	**1.656**	0.0849	22.35%	
9	30.0	107.0	115.0	33.0	400.0	70.0	15.0	87.5,66,66	**2.375**			Table 14
10	30.0	107.0	115.0	33.0	80.0	70.0	15.0	94,56,56	**1.628**	0.2790	73.48%	
13	30.0	107.0	115.0	33.0	160.0	74.0	15.0	89,70.5,70.5	**1.923**			
14	30.0	107.0	115.0	33.0	160.0	64.0	15.0	90.5,62.,62.5	**1.745**	0.0158	4.17%	
									Sum:	0.3797	100.00%	
				E[FS] =	**1.854**	E[ln(FS)] =	0.5650					
				Var[FS] =	0.3797	Var[ln(FS)] =	0.1048			b =	**1.745**	
				s **[FS] =**	**0.6162**	σ[ln(FS)] =	0.3237			Pr(f) =	4.05E-02	
				V [FS] =	0.3324	V [ln(FS)] =	0.5730			**1 in**	**25**	

L25-SSHT.XLS L25-24

Probabilistic Slope Stability Analysis:								**Herbert Hoover Dike; Model for Lines 23 - 38**				
Deterministic Model:	UTEXAS, Spencer's Method							**Lake Level:**	**24.0**			
Probabilistic Model:	Taylor's Series, Finite Difference											
Performance Function:	FS							Model based on data at Line 25				
Random Variable												
Distribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater norma											
								Failure through the peat tangent to the clay.				
	Embankment Sands			Peat / OL / OH			Ditch	Failure Arc	**Computed**	Prob Calcs		
Run	θ	γ moist	γ sat	θ	C	γ sat	Tailwater	X, Y, R	**FS**	Var	%	
E	30.0	107.0	115.0	33.0	160.0	70.0	16.0	89,67,67	**1.789**			Table 14
7	30.0	107.0	115.0	41.3	160.0	70.0	16.0	88.5,70,70	**1.992**			Table 14
8	30.0	107.0	115.0	24.8	160.0	70.0	16.0	89.5,55,55	**1.607**	0.0741	19.75%	
9	30.0	107.0	115.0	33.0	400.0	70.0	16.0	87,64,64	**2.319**			Table 14
10	30.0	107.0	115.0	33.0	80.0	70.0	16.0	93.5,57.5,57.5	**1.564**	0.2850	75.94%	Table 14
13	30.0	107.0	115.0	33.0	160.0	74.0	16.0	88.5,69.5,69.5	**1.860**			Table 14
14	30.0	107.0	115.0	33.0	160.0	64.0	16.0	90,62,62	**1.680**	0.0162	4.32%	
									Sum:	0.3753	100.00%	
				E[FS] =	**1.789**	E[ln(FS)] =	0.5262					
				Var[FS] =	0.3753	Var[ln(FS)] =	0.1109			b =	**1.580**	
				s **[FS] =**	**0.6126**	σ[ln(FS)] =	0.3330			Pr(f) =	5.70E-02	
				V [FS] =	0.3424	V [ln(FS)] =	0.6328			**1 in**	**18**	

9. MODEL DEVELOPMENT & ANALYSIS, REACHES 2-8.

9.1 Available data. Reaches 2 through 8 were investigated in much less detail than Reach 1. A total of 16 topographic cross sections were taken for the remaining 120 miles of embankment. Only a portion of the proposed borings along these 16 sections were able to be obtained due to lack of funding. Some piezometers were installed in Reaches 2, 3 and 7.

9.2 Lessons Learned from Reach 1. Geologic features and piezometer interpretations from Reaches 1 and 3 were applied where applicable to Reaches 2 - 8. This does not eliminate the need for an investigation of these reaches, however this information was used where data was lacking.

9.3 Model For Line Reach 2 Section S-4

9.3.1 Location and Range of Applicability: Reach 2 is located on the south side of Lake Okeechobee between Moore Haven, on the west side, to Structure S-354, Miami Canal, on the east side, approximate 19.6 miles. Reaches 2 consist of old levee system designate as levee LD-1 and portion levee LD-2. Levee LD-1 extends from Moore Haven to Clewiston, 10.6 miles. Levee LD-2 extends from Clewiston to Structure S-354, 9.6 miles. Reach 2 could be divided into two parts. Part 1 extends from Moore Haven to Clewiston, LD-1. This section of embankment has a deep borrow canal, C-20, at the exterior toe and a deep borrow canal, rim canal, to elevation –10 feet at the interior toe. The bottom elevation of C-20 varies from –0 feet to –13 feet. This is the thinnest embankment section on the lake.

Part 2 extends from Clewiston to Structure 354. This section of embankment has a deep borrow canal, rim canal, at the interior toe to elevation –10 feet and a deep drainage ditch to about elevation 9 feet at the exterior toe. The section analyze is designated as section S-4. The section S-4 is located at station 180+00, LD-1 and is representative of the topography and geology for 10.6 miles of embankment within part 1.

9.3.2. Embankment Evaluation.

9.3.2.1 General. This portion of the Herbert Hoover Dike system, LD-1, was constructed in 1935 by hydraulic dredge from deep borrow canal at the interior toe. As-build drawings show that the original embankment was constructed to about elevation 32.5 feet, with a crest wide of 10 feet. The interior slopes were 1V on 6H and the exterior slopes were 1V on 3H. The side slope of the borrow canal was 1H on 3V. In 1965, the embankment within this reach was raised to the landside of the existing embankment by hydraulic dredging and castings from deep borrow canal excavated along the exterior toe. The embankment was raised to a crest elevation that varies from 38 to 36 feet with a crest width of 10 feet. The interior slope is 1V on 6H and the exterior slope is 1V on 3H. A 20-foot wide berm, elevation 14 feet, extends from the land side toe to the top of a deep borrow canal. The exterior borrow canal, C-20, has a 1V on 3H side slopes with bottom wide of 10 feet or more. Natural ground elevation within this reach is approximate 14.0 feet to 15 feet. A 0 to 25 foot wide berm extends from the interior toe to the top of deep borrow canal on the lakeside. The side

slope of the interior borrow canal is 1V on 3H with bottom wide of 100 feet. The bottom elevation varies from 0 to –10 feet

9.3.2.2. Subsurface Investigation. The subsurface investigation within this reach consist of evaluating borings taken in the 1960, prior to the raising the existing embankment, boring obtained during the dike improvements between 1983 and 1988. The 1960 borings consist of a series of four borings, one at the landside toe, at the centerline existing levee, lakeside toe and centerline of the borrow canal. These borings were taken on 6000-foot intervals. In 1971, a study was prepared by the U.S. Geological Survey to determine the amount of seepage beneath Herbert Hoover Dike. Five sites were studied. Site 1 was located at LD-1 section 180+00. The report showed that the greatest amount of seepage occurred at this site, 0.9 cfs per mile per foot of head across the embankment or 404 gal/min per mile per head across the embankment.

9.3.2.3. Materials Encountered. The embankment materials, at this section, consist of a loose, silty, medium to small, shell and quartz sand with some gravel size shell fragments. The foundation materials consist of a thin layer of organic material overlying a calcareous silty, shell and quartz sand layer. This material is overlying a very porous limestone and medium to coarse shell layer. Below this layer is shell sand/gravel layer. The top of the Limestone varies elevation 12.5 feet to 5.4 feet and is about 5 to 10 feet thick. Below the clay layer are layers of sand and limestone. The top of the clay layer is –9 feet. Under the clay is shell sand at elevation –20 feet.

9.3.3 Analysis.

9.3.3.1. General. The embankment was analyzed for through seepage and underseepage. No slope stability analysis was performed for this section. However, slope stability study of this reach presented in a previous report indicated that slope stability should not be a problem. However, additional slope stability analysis should be conducted. The hydraulic characteristics of the different aquifers were determined from laboratory test data and results from the USGS study. The software program "Fastseep" was utilized to analyze this section for through seepage and under seepage for lake levels of 17.5 feet, 21 feet, and 25 feet. The software program "Utexas3" was utilized to analyze slope stability for lake level 25 feet. Tailwater elevations were based on readings for canal C-20 obtained at pump station S-4.

9.3.3.2. Selection of Critical Gradient.

9.3.3.2.1 Selection of Variables and Values. Permeability's values and possible range of values for the various aquifer layers were based on laboratory and field test data USGS study. Shear strengths used in the previous report values were based on correlation between standard penetration "n" values and laboratory test results. Soil shear strengths values of $\phi = 30^{o}$ and c=0 psf for the embankment and foundation shell sand for slope stability.

9.3.3.2.2 Topographic Features. Topographic features significant to this model are (1) The deep landside borrow canal, C-20: (2) A sand/gravel embankment and

foundation: (3) A deep borrow canal, rim canal, at interior toe. This will allow lake water access to the highly porous limestone layers.

9.3.3.2.3 Geologic Features. Two geologic features were selected as being significant to this model. The shell/quartz sand in the embankment and foundation and the highly the porous limestone layer which intersects the interior and exterior borrow canals. A deep clay layer which acts as a confining layer.

9.3.3.2.4. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability was lognormal distributed.

Table H-9.1.

	Permeability Summary for Line Reach 2 S-4				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + s \ln k}$]	Low [$e^{\ln k + s \ln k}$]
Shell Sand	4.20E-01	4.20E-01	2.97E-01	6.83E-01	1.29E-01
Clays	1.50E-04	n/a	1.27E-04	2.27E-04	7.09E-05
	n/a	1.10E-04	7.11E-05	1.81E-04	2.79E-05
Rock	2.50E-01	n/a	1.77E-01	4.06E-01	7.69E-02
	n/a	2.00E-02	1.41E-02	3.25E-02	6.15E-03
Lower sand	6.00E-03	6.00E-03	4.24E-03	9.75E-03	1.85E-03

9.3.3.2.5. Selection of Tailwater Distribution. The tailwater in the C-20 canal is control by pump station S-4 located one mile west of Clewiston and by a gated culvert at Moore Haven. The local drainage districts also operate pump stations at different location along the canal to provide irrigation and flood protection by pumping water into or out of the canal.

9.3.3.2.6. Tailwater Distributions for Through Seepage Analyses. The tail water level assumed for through seepage was varied as shown in Table H-9.2. This was based upon probable minimum and maximum values as well as observed water levels recorded at Pump Station S-4. This range of values was assumed to be normally distributed.

9.3.3.2.7. Tailwater Distribution for Under Seepage Analyses. For underseepage lower tailwater elevations is more critical. Since the water in canal C-20 can be control by pump station S-4 and by the gated culvert at Moore Haven, and can be artificially manipulated, a

water level close to "normal" was selected for all analyses. Based on all existing data, elevation 11.0 was selected as the "normal" water elevation. Although other tailwater levels were evaluated, this scenario (artificial manipulation) was selected as critical since the Corps does not control tailwater elevations.

Table H-9.2.

	Tailwater Elevations for Model Reach 2 Section S-4				
Lake El.	Mean [E(x)]	Min / Max	s(x)	High [E(x) +s]	Low [E(x) +s]
17.5	11.	11.5/10.5	0.5	11.7	10.7
21	12.0	13/11	0.66	12.9	11.5
25	14	15.5/12.5	0.66	13.2	11.8

9.3.4. Selection Of Critical Gradients.

9.3.4.1. General. Seepage for the both horizon and vertical exit gradients are sensitive to the tailwater level.

9.3.4.2. Underseepage. The critical or vertical gradient "i_{cv}" used for underseepage was based on the hydrostatic pressure below the shell sand layer in the bottom of the C-20 canal being high enough to cause the sand to boil and pipe assuming a given tailwater level. The critical gradient for this sand layer is .75 in accordance with EM-1110-2-1901 "Seepage Analysis and Control for Dams" and the Lower Mississippi Method. This is the gradient required to start boils and piping. The gradient required to breach the embankment has to be increased enough to drive the pipe past the half way point, l /L > 40 - 50 % of the embankment width, where l is pipe length and L is total pipe path length. In order to do this, the gradient required for breach should be approximately 15% greater than that required to start piping. See the analysis for horizontal gradient as presented for line 25 for the method used to determine required increase for the critical gradient. By applying this 15% increase, the critical gradient is computed be as shown be low.

Average Value: 0.86
Std. Dev.: 0.08
High Value: 0.94
Low Value: 0.78

9.3.4.3. Piping below the confining layer. The location of the pipes is expected to be directly under the limestone layers. This means that the silty shell sand is under little or no

confining stress. The horizontal gradient to cause piping, i_{ch}, range from .15 to .30. This was based on the .16 encountered by Shmertmann (Shmertmann, 1997) and the .3 by Schmertmann (Shmertmann, 1997). For this model .15 was derived from considerations of the uncertainties in the surveys, soil parameters, Permabilities and calculations.

Average Value:	0.15
Std. Dev.:	0.02
High Value:	0.17
Low Value:	0.13

9.3.5. Failure Mechanisms. There were two primary failure mechanisms considered for seepage and slope stability.

9.3.5.1. Through seepage. The deep excavation at the exterior toe and interior toe of the embankment within this reach made this the failure mode of most concern. Piping is defined as the removal of embankment or foundation material. The deep rim canal and deep borrow canal C-20, is deep enough to allow the lake direct access to the vary porous limestone layer and to the shell sand layers above and below the limestone. The limestone provides a roof to pipe material from the lower formation and direct access for material to pipe above the limestone layer. However, any excessive seepage or piping occurring under the surface of the canal C-20 would likely go undetected until a partial collapse of the berm occurs.

9.3.5.2. Under seepage. The high pore pressure in the bottom of canal C-20 will cause boils and piping of the foundation material. Boils and piping in the bottom of the canal will go undetected until a partial collapse of the berm occurs.

9.3.5.3. Slope Stability. Slope failure leading to a breach occurs when the shear failure arc involves approximately one third to one half of the landside embankment. Once this occurs, the shear zones along the failure surface will insects the potential seepage pipe path thus shorting the seepage path and increasing the exit gradient.

9.3.6. Model Calibration And Analysis Results.

9.3.6.1. Model Calibration for Seepage Analyses. Permeabilities for the various aquifers were based on laboratory test and results of USGS study. Geologic section was based on core borings obtained in the 1960's and 1980's.

9.3.6.2. Slope Stability Analyses. No slope stability analysis was performed as part of this study due to lack of time and funds. However, slope stability analysis perform within this reach in 1993 indicated a factory of safety of 1.3 for a lake level of 25 feet and tailwater of 15 feet.

9.3.6.3. Seepage: No visible evidence of seepage was observed at this site during the 1995 or 1997 high water event. Therefore, the model used was calibrated to show no seepage for lake level 18.5 feet with a tailwater of 11.0 feet.

9.3.7 Length Effects**.** For this model, a link length of 5280 feet was selected. This distance was based on the spacing of know core borings.

9.3.8. Pup Curve Development. Table H-9.3 is a summary of the analyses performed on this model. Relevant information for each analysis was listed to show the progression of values with Lake Level.

9.3.9. Mode Of Failure.

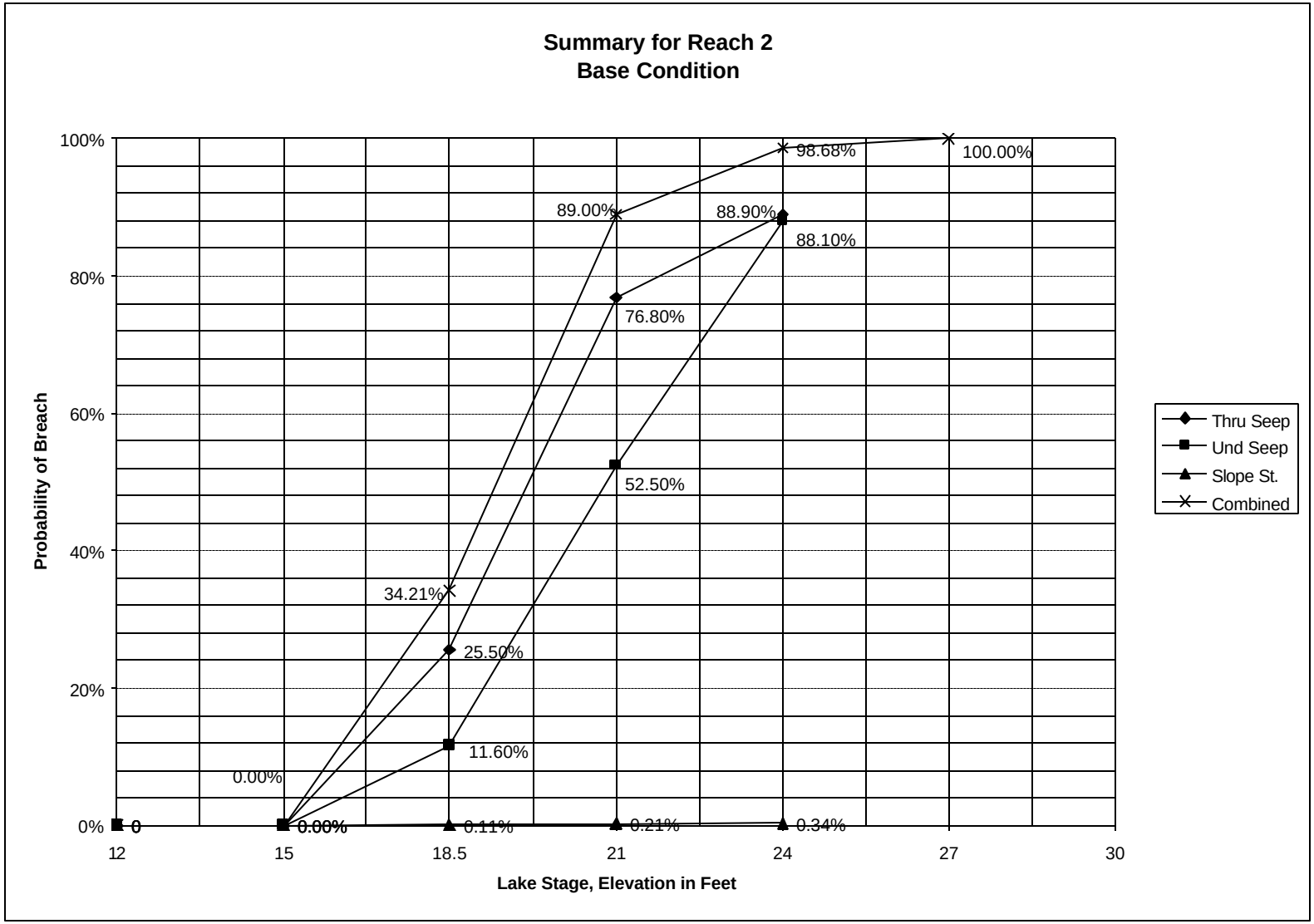
9.3.9.1. Slope Stability. Based on preliminary slope stability analysis from a previous study for a lake elevations 25 feet indicated that this would be the least likely mode of failure within this reach. However, a more detail investigate should be performed with regard to underseepage and slope stability. This should include detail survey, installing piezometers and obtaining additional subsurface stratification data.

9.3.9.2. Through Seepage. This failure mode was not a factor for lake elevations less than 21.0 since seepage did not exit the slope. Two seepage paths were looked at, the first seepage path above the limestone and the second path below the limestone. The first path above the limestone would required a higher exit gradient. The other path was below the limestone layer. This path required a lower exit gradient. The analyzes for his path is presented in this report. Once seepage exited the slope, the horizontal exit gradient was compared to the horizontal critical gradient. The resulting probabilities are shown in Table H-9.3.

9.3.9.3. Under Seepage. For lake elevations of 21 feet or higher, underseepage was the predominant failure mode. The uplift forces in the sand layer in the bottom of C-20 was high enough to cause the sand layer to boil and will cause this material to pipe.

9.3.10. Probabilistic Analyses Computation Sheets and Sections.

Table


Summary for Reach 2
Base Condition
100%
80%
60%
40%
20%
0%
Probability of Breach
12
15
18.5
21
24
27
30
Lake Stage, Elevation in Feet
98.68%
100.00%
89.00%
88.90%
88.10%
76.80%
52.50%
34.21%
25.50%
11.60%
0.00%
0
0.00%
0.11%
0.21%
0.34%
Thru Seep
Und Seep
Slope St.
Combined

LEGEND FOR COLOR CROSS SECTIONS

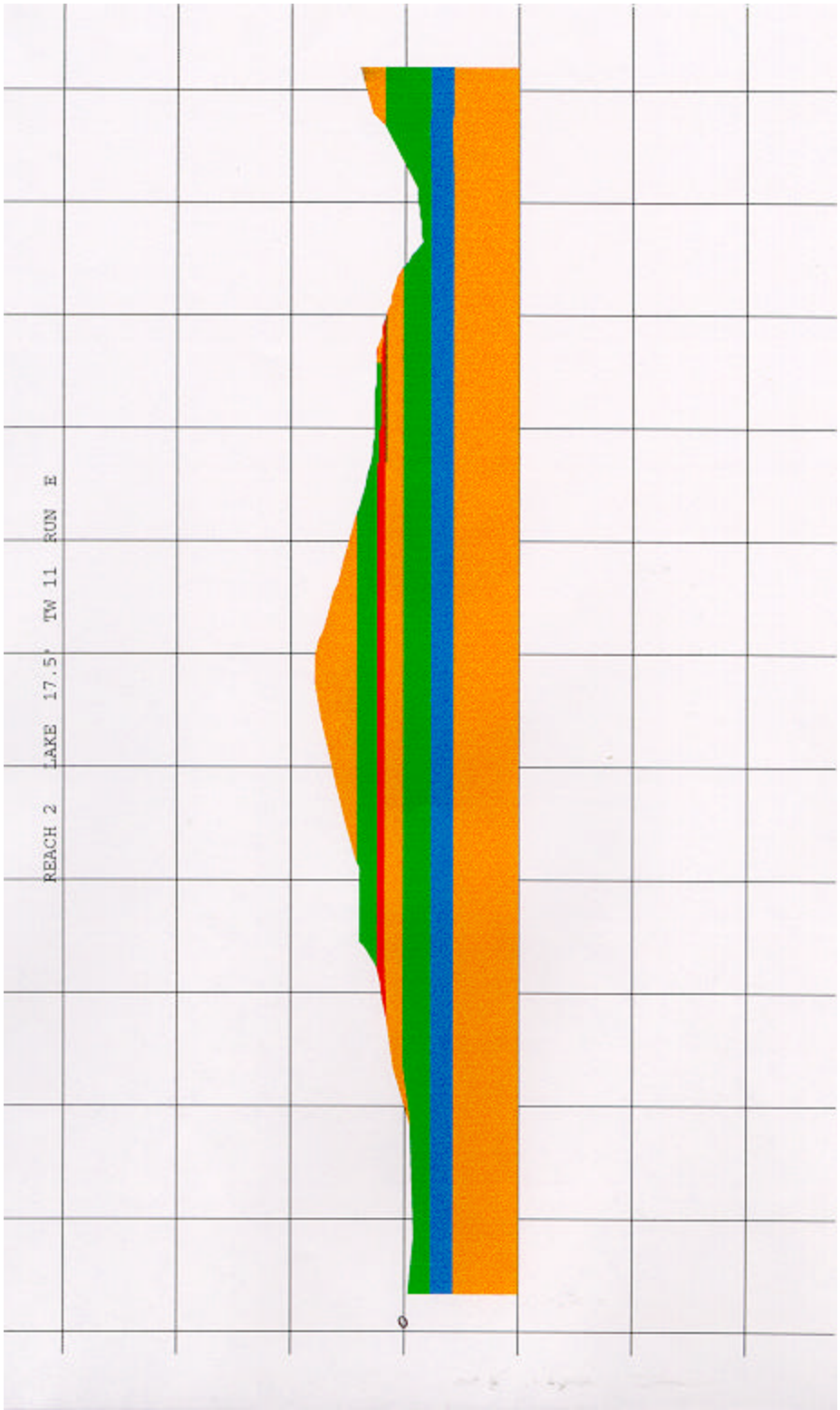
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

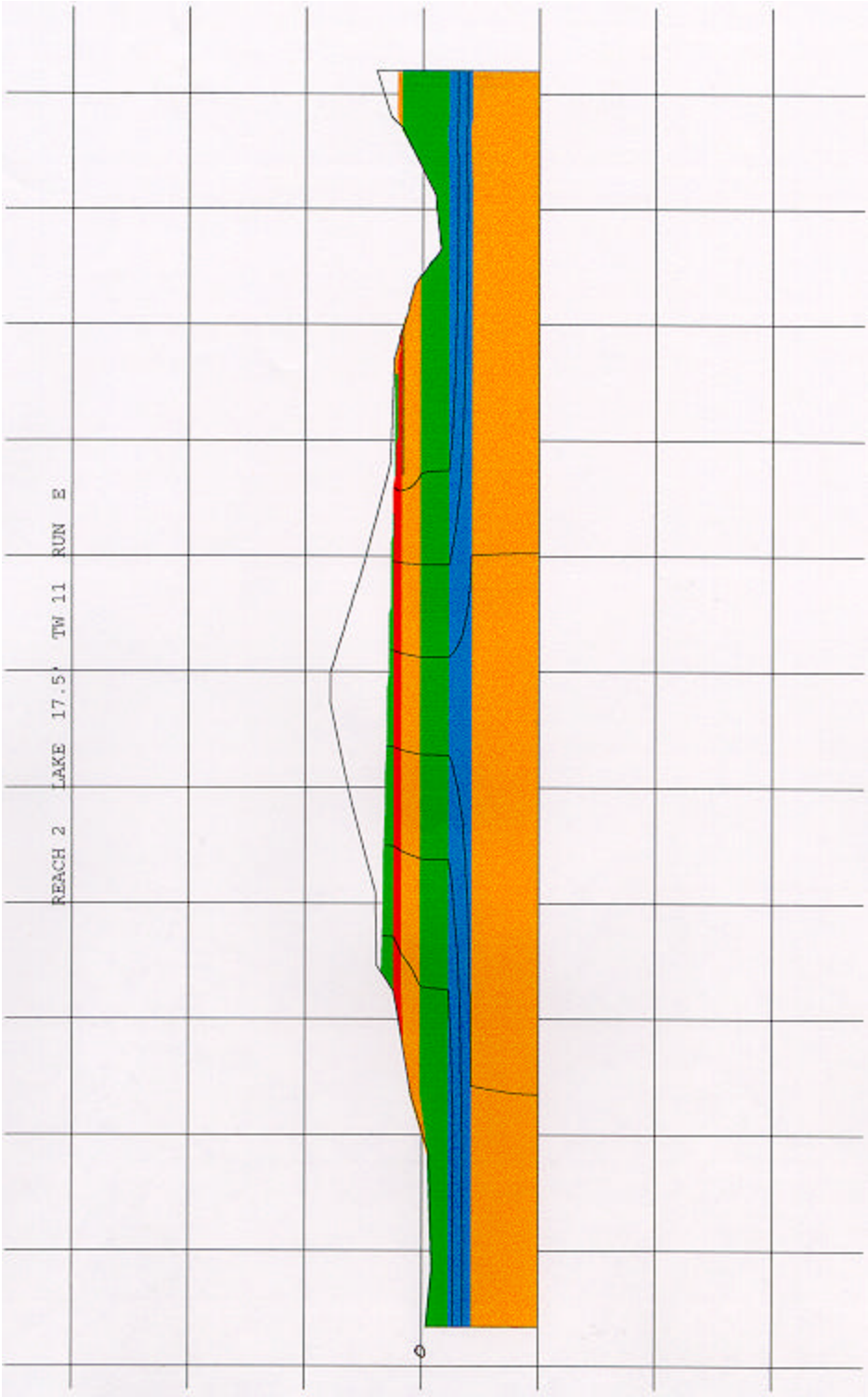
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

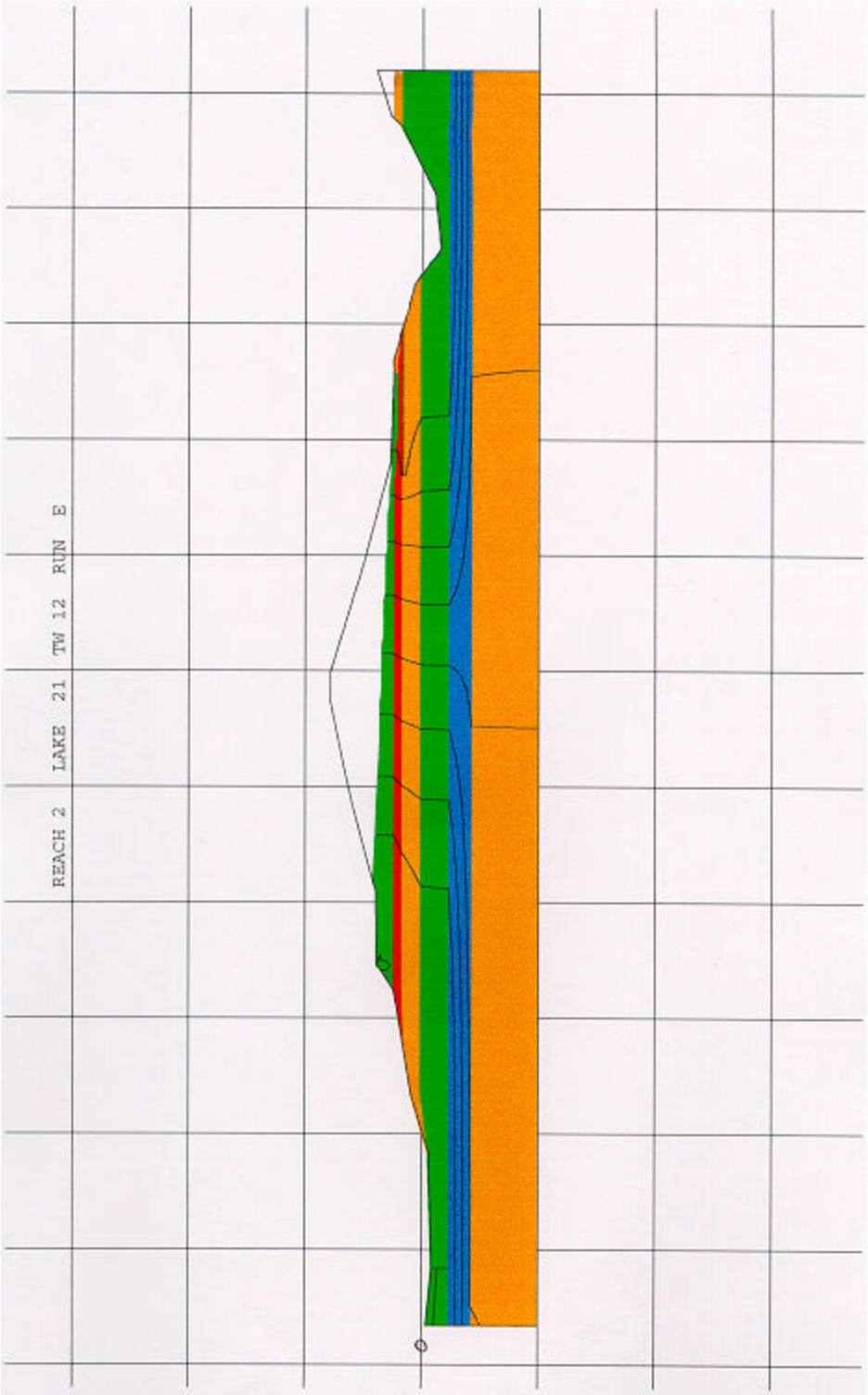
REACH 2 LAKE 17.5’ TW 11 RUN E


REACH 2 LAKE 17.5' TW 11 RUN E
0

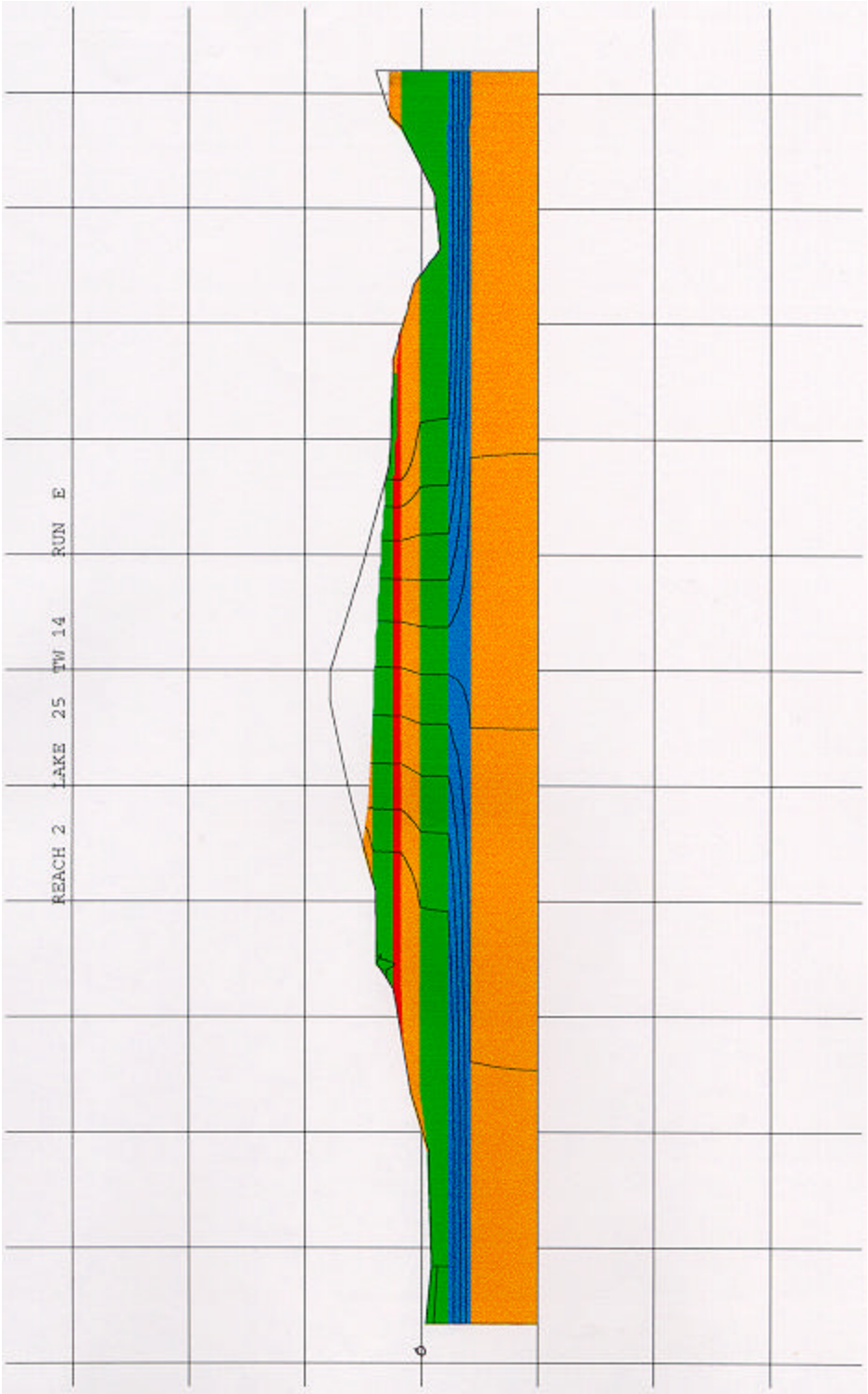
REACH 2 LAKE 17.5′ TW 11 RUN E



REACH 2 LAKE 21 TW 12 RUN E



REACH 2 LAKE 25 TW 14 RUN E



R2175.xls, Ditch Slope, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; REACH 2**

Deterministic Model: FASTSEEP **LAKE AT 17.5'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	1	10	7		3		4			**Ditch Slope, ih**			
	Upper	Shell	Rock		Clay		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**7.00E-03**	**2.97E-01**	**1.77E-01**	**1.41E-02**	**1.27E-04**	**7.11E-05**	**4.24E-03**	**11.0**	**0.15**	**0.086**	**1.750**		
1	7.00E-03	**6.83E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	0.15	**0.100**	**1.500**		
2	7.00E-03	**1.29E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	0.15	**0.073**	**2.042**	0.0734	12.00%
3	7.00E-03	**2.97E-01**	**4.06E-01**	**3.25E-02**	1.27E-04	7.11E-05	4.24E-03	11.0	0.15	**0.073**	**2.042**		
4	7.00E-03	**2.97E-01**	**7.69E-02**	**6.15E-03**	1.27E-04	7.11E-05	4.24E-03	11.0	0.15	**0.100**	**1.500**	0.0734	12.00%
5	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**2.27E-04**	**1.81E-04**	4.24E-03	11.0	0.15	**0.088**	**1.709**		
6	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**7.09E-05**	**2.79E-05**	4.24E-03	11.0	0.15	**0.088**	**1.709**	0.0000	0.00%
7	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**9.75E-03**	11.0	0.15	**0.096**	**1.564**		
8	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**1.85E-03**	11.0	0.15	**0.102**	**1.470**	0.0022	0.36%
9	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**11.5**	0.15	**0.080**	**1.885**		
10	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**10.5**	0.15	**0.247**	**0.607**	0.4078	66.73%
11	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	**0.17**	**0.086**	**1.983**		
12	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	**0.13**	**0.086**	**1.517**	0.0544	8.91%
											Sum:	0.6111	100.00%

E[FS] =	**1.750**	E[ln(FS)] =	0.4686		
Var[FS] =	0.6111	Var[ln(FS)] =	0.1820	b =	**1.099**
s **[FS] =**	**0.7818**	σ[ln(FS)] =	0.4266	Pr(f) =	**1.36E-01**
V [FS] =	0.4467	V [ln(FS)] =	0.9102	**1 in**	**7**

Adjusted probabilities based on length effect:
Length 4 miles links

Total Length =	**19**	Reliability of Section, R =	7.45E-01
"Link" Length =	**9.5**	**Adjusted Pr(f) =**	**2.55E-01**
No. Links =	**2.01052632**	**1 in**	**4**

R2175.xls, Ditch Bottom, iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; REACH 2**

Deterministic Model: FASTSEEP **LAKE AT 17.5'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	1	10	7		3		4			**Ditch Bottom, iv**			
	Upper	Shell	Rock		Clay		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**7.00E-03**	**2.97E-01**	**1.77E-01**	**1.41E-02**	**1.27E-04**	**7.11E-05**	**4.24E-03**	**11.0**	**0.86**	**0.262**	**3.287**		
1	7.00E-03	**6.83E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	0.86	**0.268**	**3.211**		
2	7.00E-03	**1.29E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	0.86	**0.256**	**3.353**	0.0051	0.09%
3	7.00E-03	**2.97E-01**	**4.06E-01**	**3.25E-02**	1.27E-04	7.11E-05	4.24E-03	11.0	0.86	**0.264**	**3.261**		
4	7.00E-03	**2.97E-01**	**7.69E-02**	**6.15E-03**	1.27E-04	7.11E-05	4.24E-03	11.0	0.86	**0.261**	**3.300**	0.0004	0.01%
5	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**2.27E-04**	**1.81E-04**	4.24E-03	11.0	0.86	**0.193**	**4.447**		
6	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**7.09E-05**	**2.79E-05**	4.24E-03	11.0	0.86	**0.305**	**2.819**	0.6627	12.39%
7	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**9.75E-03**	11.0	0.86	**0.194**	**4.424**		
8	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**1.85E-03**	11.0	0.86	**0.099**	**8.663**	4.4927	84.01%
9	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**11.5**	0.86	**0.241**	**3.569**		
10	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**10.5**	0.86	**0.282**	**3.046**	0.0684	1.28%
11	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	**0.94**	**0.262**	**3.593**		
12	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	11.0	**0.76**	**0.262**	**2.905**	0.1183	2.21%
											Sum:	5.3475	100.00%

E[FS] =	**3.287**	E[ln(FS)] =	0.9889		
Var[FS] =	5.3475	Var[ln(FS)] =	0.4021	b =	**1.560**
s **[FS] =**	**2.3125**	σ[ln(FS)] =	0.6341	Pr(f) =	**5.94E-02**
V [FS] =	0.7035	V [ln(FS)] =	0.6412	**1 in**	**17**

Adjusted probabilities based on length effect:
length in miles

Total Length =	**19.1**	Reliability of Section, R =	8.84E-01
"Link" Length =	**9.5**	**Adjusted Pr(f) =**	**1.16E-01**
No. Links =	**2.0**	**1 in**	**9**

R221.xls, Ditch Bottom, iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; REACH 2**

Deterministic Model: FASTSEEP **LAKE AT 21'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	1	10	7		3		4			**Ditch Bottom, iv**			
	Upper	Shell	Rock		Clay		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**7.00E-03**	**2.97E-01**	**1.77E-01**	**1.41E-02**	**1.27E-04**	**7.11E-05**	**4.24E-03**	**12.0**	**0.86**	**0.391**	**2.200**		
1	7.00E-03	**6.83E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	0.86	**0.395**	**2.177**		
2	7.00E-03	**1.29E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	0.86	**0.381**	**2.260**	0.0017	0.03%
3	7.00E-03	**2.97E-01**	**4.06E-01**	**3.25E-02**	1.27E-04	7.11E-05	4.24E-03	12.0	0.86	**0.393**	**2.188**		
4	7.00E-03	**2.97E-01**	**7.69E-02**	**6.15E-03**	1.27E-04	7.11E-05	4.24E-03	12.0	0.86	**0.387**	**2.224**	0.0003	0.01%
5	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**2.27E-04**	**1.81E-04**	4.24E-03	12.0	0.86	**0.292**	**2.949**		
6	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**7.09E-05**	**2.79E-05**	4.24E-03	12.0	0.86	**0.453**	**1.899**	0.2758	5.33%
7	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**9.75E-03**	12.0	0.86	**0.294**	**2.928**		
8	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**1.85E-03**	12.0	0.86	**0.118**	**7.295**	4.7670	92.11%
9	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**13.0**	0.86	**0.340**	**2.528**		
10	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**11.0**	0.86	**0.436**	**1.971**	0.0776	1.50%
11	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	**0.94**	**0.391**	**2.405**		
12	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	**0.76**	**0.391**	**1.944**	0.0530	1.02%
											Sum:	5.1754	100.00%

E[FS] =	**2.200**	E[ln(FS)] =	0.4249		
Var[FS] =	5.1754	Var[ln(FS)] =	0.7272	b =	**0.498**
s **[FS] =**	**2.2749**	σ[ln(FS)] =	0.8527	Pr(f) =	**3.09E-01**
V [FS] =	1.0340	V [ln(FS)] =	2.0070	**1 in**	**3**

Adjusted probabilities based on length effect:
length in miles

Total Length =	**19**	Reliability of Section, R =	4.75E-01
"Link" Length =	**9.5**	**Adjusted Pr(f) =**	**5.25E-01**
No. Links =	**2.01052632**	**1 in**	**2**

R221.xls, Ditch Slope, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; REACH 2**

LAKE AT 21'

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Ditch Slope, ih

	1	10	7		3		4						
	Upper	Shell	Rock		Clay		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**7.00E-03**	**2.97E-01**	**1.77E-01**	**1.41E-02**	**1.27E-04**	**7.11E-05**	**4.24E-03**	**12.0**	**0.15**	**0.147**	**1.021**		
1	7.00E-03	**6.83E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	0.15	**0.161**	**0.930**		
2	7.00E-03	**1.29E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	0.15	**0.124**	**1.205**	0.0188	26.99%
3	7.00E-03	**2.97E-01**	**4.06E-01**	**3.25E-02**	1.27E-04	7.11E-05	4.24E-03	12.0	0.15	**0.120**	**1.246**		
4	7.00E-03	**2.97E-01**	**7.69E-02**	**6.15E-03**	1.27E-04	7.11E-05	4.24E-03	12.0	0.15	**0.145**	**1.035**	0.0111	15.88%
5	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**2.27E-04**	**1.81E-04**	4.24E-03	12.0	0.15	**0.147**	**1.021**		
6	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**7.09E-05**	**2.79E-05**	4.24E-03	12.0	0.15	**0.147**	**1.021**	0.0000	0.00%
7	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**9.75E-03**	12.0	0.15	**0.165**	**0.907**		
8	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**1.85E-03**	12.0	0.15	**0.173**	**0.865**	0.0005	0.65%
9	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**13.0**	0.15	**0.127**	**1.185**		
10	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**11.0**	0.15	**0.167**	**0.896**	0.0209	29.94%
11	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	**0.17**	**0.147**	**1.157**		
12	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	12.0	**0.13**	**0.147**	**0.885**	0.0185	26.54%
											Sum:	0.0698	100.00%

E[FS] =	**1.021**	E[ln(FS)] =	-0.0118		
Var[FS] =	0.0698	Var[ln(FS)] =	0.0648	b =	**-0.046**
s **[FS] =**	**0.2642**	σ[ln(FS)] =	0.2546	Pr(f) =	**5.18E-01**
V [FS] =	0.2588	V [ln(FS)] =	-21.5780	**1 in**	**2**

Adjusted probabilities based on length effect:
Length 4 miles links

Total Length =	**19**	Reliability of Section, R =	2.32E-01
"Link" Length =	**9.5**	**Adjusted Pr(f) =**	**7.68E-01**
No. Links =	**2**	**1 in**	**1**

R225.xls, Ditch Bottom, iv

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; REACH 2**

Deterministic Model: FASTSEEP **LAKE AT 25'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	1	10	7		3		4			**Ditch Bottom, iv**			
	Upper	Shell	Rock		Clay		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**7.00E-03**	**2.97E-01**	**1.77E-01**	**1.41E-02**	**1.27E-04**	**7.11E-05**	**4.24E-03**	**14.0**	**0.86**	**0.164**	**5.252**		
1	7.00E-03	**6.83E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	0.86	**0.164**	**5.228**		
2	7.00E-03	**1.29E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	0.86	**0.159**	**5.404**	0.0077	0.04%
3	7.00E-03	**2.97E-01**	**4.06E-01**	**3.25E-02**	1.27E-04	7.11E-05	4.24E-03	14.0	0.86	**0.163**	**5.277**		
4	7.00E-03	**2.97E-01**	**7.69E-02**	**6.15E-03**	1.27E-04	7.11E-05	4.24E-03	14.0	0.86	**0.164**	**5.228**	0.0006	0.00%
5	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**2.27E-04**	**1.81E-04**	4.24E-03	14.0	0.86	**0.089**	**9.690**		
6	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**7.09E-05**	**2.79E-05**	4.24E-03	14.0	0.86	**0.261**	**3.296**	10.2195	57.47%
7	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**9.75E-03**	14.0	0.86	**0.255**	**3.375**		
8	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**1.85E-03**	14.0	0.86	**0.102**	**8.451**	6.4410	36.22%
9	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**15.5**	0.86	**0.138**	**6.245**		
10	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**12.5**	0.86	**0.194**	**4.443**	0.8116	4.56%
11	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	**0.94**	**0.164**	**5.741**		
12	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	**0.76**	**0.164**	**4.642**	0.3021	1.70%
											Sum:	17.7826	100.00%

E[FS] =	**5.252**	E[ln(FS)] =	1.4099		
Var[FS] =	17.7826	Var[ln(FS)] =	0.4975	b =	**1.999**
s **[FS] =**	**4.2169**	σ[ln(FS)] =	0.7053	Pr(f) =	**2.28E-02**
V [FS] =	0.8029	V [ln(FS)] =	0.5002	**1 in**	**44**

Adjusted probabilities based on length effect:
length in miles

Total Length =	**16**	Reliability of Section, R =	9.12E-01
"Link" Length =	**4**	**Adjusted Pr(f) =**	**8.81E-02**
No. Links =	**4**	**1 in**	**11**

R225.xls, Ditch Slope, ih

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; REACH 2**

Deterministic Model: FASTSEEP **LAKE AT 25'**
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

	1	10	7		3		4			**Ditch Slope, ih**			
	Upper	Shell	Rock		Clay		Lower	Ditch		Results		Prob Calcs	
Run	Sands		Horiz.	Vert.	Horiz.	Vert.	Sands	Tailwater	i crit	i exit	**FS**	Var	%
E	**7.00E-03**	**2.97E-01**	**1.77E-01**	**1.41E-02**	**1.27E-04**	**7.11E-05**	**4.24E-03**	**14.0**	**0.15**	**0.163**	**0.919**		
1	7.00E-03	**6.83E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	0.15	**0.173**	**0.865**		
2	7.00E-03	**1.29E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	0.15	**0.153**	**0.980**	0.0033	1.14%
3	7.00E-03	**2.97E-01**	**4.06E-01**	**3.25E-02**	1.27E-04	7.11E-05	4.24E-03	14.0	0.15	**0.139**	**1.081**		
4	7.00E-03	**2.97E-01**	**7.69E-02**	**6.15E-03**	1.27E-04	7.11E-05	4.24E-03	14.0	0.15	**0.192**	**0.782**	0.0223	7.69%
5	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**2.27E-04**	**1.81E-04**	4.24E-03	14.0	0.15	**0.163**	**0.919**		
6	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	**7.09E-05**	**2.79E-05**	4.24E-03	14.0	0.15	**0.163**	**0.919**	0.0000	0.00%
7	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**9.75E-03**	14.0	0.15	**0.161**	**0.930**		
8	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	**1.85E-03**	14.0	0.15	**0.163**	**0.919**	0.0000	0.01%
9	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**15.5**	0.15	**0.092**	**1.633**		
10	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	**12.5**	0.15	**0.237**	**0.634**	0.2499	85.99%
11	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	**0.17**	**0.163**	**1.041**		
12	7.00E-03	**2.97E-01**	1.77E-01	1.41E-02	1.27E-04	7.11E-05	4.24E-03	14.0	**0.13**	**0.163**	**0.796**	0.0150	5.16%
											Sum:	0.2906	100.00%

E[FS] =	**0.919**	E[ln(FS)] =	-0.2327		
Var[FS] =	0.2906	Var[ln(FS)] =	0.2958	b =	**-0.428**
s **[FS] =**	**0.5390**	σ[ln(FS)] =	0.5439	Pr(f) =	**6.66E-01**
V [FS] =	0.5867	V [ln(FS)] =	-2.3378	**1 in**	**2**

Adjusted probabilities based on length effect:
Length 4 miles links

Total Length =	**19**	Reliability of Section, R =	1.11E-01
"Link" Length =	**9.5**	**Adjusted Pr(f) =**	**8.89E-01**
No. Links =	**2.01052632**	**1 in**	**1**

9.4. Model for Reach 3. This model for Reach 3 is taken at survey line R3-C, based on survey 95-372. This is part of the original levee embankment designated as LD-2 near station 620+00 on the as-builts dated May, 1965 and near section J-J on the as-builts dated May, 1933.. As-builts dated May, 1933 show the construction of the original federal project as having a 20 foot crest at elevation 34.7 feet, a 1V:6H lakeside slope and a 1V:3H landside slope. Construction of a downstream fire toe and an upstream seepage toe through the "muck" were indicated. When the embankment was raised to its present elevation of 41.6 feet in 1965, the slopes and crest remained the same. Even though the as-builts from 1965 show a 10 foot crest, the 1995 survey section revealed a 20 foot crest.

9.4.1. Selection of Variables and Values. Corps of Engineers drill crews were brought on site during the 1995 High Water Event. The explorations for Reach 3 had not yet been performed, so core borings were located at the sites exhibiting signs of distress. Engineers and geologists were on site during much of the drilling, so samples were examined soon after they were recovered.

9.4.1.1. Topographic Features. Topographic features significant to this model were the toe ditch and fire toe. The fire toe is a man made feature is shown on the drawings for the original Federal Project. Basically it is a 12 foot wide trench excavated through the peat layer under the landside toe of the original project. The depth varies with the thickness of the peat. The purpose of the fire toe was to prevent a subsurface peat fire from undermining the embankment. The significance of this feature is that it provides a continuous direct connection between the embankment material and the upper limestone below the peat. The seepage toe is of similar construction.

9.4.1.2. Geologic Features. Six geologic features were selected as being significant to this model. They are as follows:

Sands - This is a combination of hydraulic fill from the original construction in the 1930's and additional fill placed in the 1960's, as well as the sand deeper in the foundation.. The material classifies primarily as sands with varying amounts of fines. A significant portion of the sand is sand size shell particles.

Gravel Layer - a man made feature due to the segregation of coarse materials at a hydraulic fill discharge pipe. The significance of this feature is that it provides intermittent zones of internally unstable high permeability material near the base of the embankment. This feature was encountered in borings CB-E3C-B as well as nearby CB-MRR3-SH-B. It is interesting to note geologist Hess logged the embankment fill in borings CB-LD2-38 and CB-LD2-42 as "limestone". This would indicate a large amount of limestone fragments recovered.

Filtercake - The silting of the rim canal acts as a barrier for lake water entering the deeper limestone exposed during excavation of the rim canal.

Peat - This natural deposit occurs throughout the area. The peat shows some evidence of consolidation under the embankment. The area at the toe consists of natural deposits and fill in the upper zones.

Silts & Clays - a natural deposit of low plasticity. This material is sometimes referred to as marl. The deposit occasionally contains interbedded sands and shell seams. This layer also acts as a confining layer.

Exit Valve - a natural feature incorporated into the model to aid calibration. There is insufficient detailed information on the exact geologic configuration which allows water to exit the lower limestone through the confining layer into the ditch bottom. Piezometer readings at different lake levels were reproduced by assigning the most probable area of water exit as a separate material for the model. In this case, it is the area below the bottom of the toe ditch. This material, or "valve" could then be adjusted independently of the rest of the material layers to calibrate the model.

9.4.1.3. Slope Stability Variables. Variables and values selected for slope stability were divided into two categories, those with normal distributions and those with lognormal distributions.

Table H-9.3 Normally Distributed variables

	Shear Strength Summary for Reach 3				
		All values in degrees.			
	θ	θ	Normal Parameters		
Material	Min / Max	Mean [E(x)]	σ(x)	High [E(x) +σ]	Low [E(x) +σ]
Emb, R		23.0	2.30	25.3	20.7
Emb, S		33.0	3.30	36.3	29.7
PT/OL/OH, R		18.0	4.50	22.5	13.5
PT/OL/OH, S		33.0	8.25	41.3	24.8
CL / ML, R		12.5	3.10	15.6	9.4
CL / ML, S		33.0	8.30	41.3	24.7
Rock		45.0	4.50	49.5	40.5

Table H-9.4 Lognormally Distributed variables

	Shear Strength Summary for Reach 3			
	All values in degrees or psf.			
		Lognormal Parameters		
Material	C	Mean [$e^{\ln k}$]	High [$e^{\ln k + \sigma \ln k}$]	Low [$e^{\ln k + \sigma \ln k}$]
Emb, R	740	523	1203	228
Emb, S		no variation, not a variable		

PT/OL/OH, R	440	311	715	135
PT/OL/OH, S	240	170	390	74
CL / ML, R	700	623	1010	383
CL / ML, S	200	141	325	62

Table H-9.5

	Material Unit Weights			
Material	Mean [E(x)]	σ(x)	High [E(x) +σ]	Low [E(x) +σ]
Emb.	115.0	5.75	120.8	109.3
PT/OL/OH	70.0	3.50	73.5	66.5
CL/CH/ML/MH	100.0	5.00	105.0	95.0
Rock	135.0	6.50	141.5	128.5

9.4.1.4. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability were lognormally distributed.

Table H-9.6

	Permeability Summary for Reach 3				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + \sigma \ln k}$]	Low [$e^{\ln k + \sigma \ln k}$]
Sands	7.00E-03	7.00E-03	4.95E-03	1.14E-02	2.15E-03
Gravel	1.00E+00	1.00E+00	7.07E-01	1.63E+00	3.08E-01
Peat	2.00E-04	n/a	1.60E-04	3.12E-04	8.20E-05
	n/a	6.00E-05	4.61E-05	9.53E-05	2.23E-05
Silts & Clays	1.50E-04	n/a	1.27E-04	2.27E-04	7.09E-05
	n/a	1.10E-04	7.11E-05	1.81E-04	2.79E-05

Rock	2.50E-01	n/a	1.77E-01	4.06E-01	7.69E-02
	n/a	2.00E-02	1.41E-02	3.25E-02	6.15E-03
Filtercake	8.50E-05	8.50E-05	6.01E-05	1.38E-04	2.61E-05
Exit Valve	5.50E-03	5.50E-03	3.89E-03	8.94E-03	1.69E-03

Table H-9.7

Gravel Distribution, Width of Gravel Pocket

Mean [E(x)]	Min / Max	σ(x)	High [E(x) +σ]	Low [E(x) +σ]
			0.0	0.0
Transformed to Model Coord.s				
	Center	Lakeside	Landside	Total
Mean [E(x)]				
High [E(x) +σ]				
Low [E(x) +σ]				

9.4.2 Selection of Tailwater Distribution. Tailwater distributions were based on assumed values. No data exists for water levels in the toe ditch in this area. FDOT culvert inverts in this area (draining under U.S. 27) range from elevation 10 to 12. Ditch bottom elevations are unknown except at survey cross sections due to lack of a current topographic survey of this area.

Table H-9.8

	Tailwater Elevations for Models				
Lake El.	Mean [E(x)]	Min / Max	σ(x)	High [E(x) +σ]	Low [E(x) +σ]
15	11.0		0.33	11.3	10.7
18	12.0		0.33	12.3	11.7
21	12.5		0.33	12.8	12.2
24	13.0		0.33	13.3	12.7

9.4.3. Selection of Critical Gradients.

9.4.3.1. Piping above the confining layer.

For the upper sands and sandy material below the peat, gradients in the model indicated the material did have some confining stresses. Therefore, i_{ch} was computed as follows using the equation derived in the ***WES report: $i_{ch} = i_{cv}$ (cos b tan f -sin b), where f = friction angle and b = the angle of flow (obtained from FastSeep grid).

	i_{cv}	i_{ch}(f only)	i_{ch}(f and b)
High values			
g_{soil} = 121 pcf 0.939			
f = 36°	0.682		
b = 0°		0.682	
Low values			
g_{soil} = 109 pcf 0.747			
f = 30°	0.577		
b = 5°		0.364	

With the range of i_{ch} from 0.682 to 0.364, an average value of 0.523 was selected. Dividing the range from 0.682 to 0.364 by six to obtain three standard deviations on either side of the mean, a standard deviation of s = 0.053 was applied. This gives the following:

Average Value:	0.523
Std. Dev.:	0.053
High Value:	0.576
Low Value:	0.470

This is the critical gradient to start piping. For breach, the gradient has to increase enough to drive the pipe past the half way point, l/L > 40 - 50 %, where l is pipe length and L is total pipe path length.

The following values for i_{ch} were used in the analysis for through seepage.

Average Value:	0.601
Std. Dev.:	0.020
High Value:	0.662
Low Value:	0.541

9.4.3.2. Piping below the confining layer. For underseepage, the values for i_{cv} were determined from material data as described above and from the Lower Mississippi method. Data indicated a range of i_{cv} from 0.939 to 0.747. Values for i_{cv} observed during the 1937 high water event along the Mississippi River ranged from 0.15 to 0.88, predominantly from 0.5 to 0.8. The high end of the range was selected (0.7 - 0.8) due to the high occurrence of observed piping in this area. This gives the following:

Average Value:	0.750
Std. Dev.:	0.050
High Value:	0.800
Low Value:	0.700

Again applying this 15% to the critical gradient computed above, the following critical gradient for breach can be obtained.

Average Value:	0.863
Std. Dev.:	0.057
High Value:	0.920
Low Value:	0.805

9.4.4. Failure Mechanisms.

9.4.4.1. Through Seepage. For through seepage, the piping path begins in the toe ditch in the fill mix comprising the berm. The sand component of this material pipes out until the pipe reaches the gravel pocket in the embankment. This is labeled as the probability of piping through the sand - peat mixture (Psp). Once in the gravel pocket, the critical gradient is about half the value of the critical gradient of the embankment sands. This is labeled as the probability of piping through the gravel, Pg, and is set equal to 1. The entire conditional probability of breach from through seepage Pb is :

$$Psp \times Pg|sp = Pb$$

9.4.4.2. Under Seepage. For underseepage, the piping path begins as heave and fracturing of the silt - clay layer comprising the ditch bottom. Observations of boils in the ditch bottom shows this has already occurred at lake levels above elevation 17.0. Once the gradient through the confining layer is sufficient, piping will occur. This is the probability of vertical piping through the confining layer. This is determined according to the method in TM 3-424, where the gradient is measured across the confining layer. Piping occurs through the cracks in the confining layer, and progresses back to the source. The critical gradient for breach is used to compute probability of breach, Pb.

9.4.4.3. Slope Stability. For slope stability, a two part process is involved. The first step is the slope failure itself. UTEXAS models using Spencer's method show that the failure arc involves approximately one third to one half of the embankment. This is labeled as the probability of slope stability failure, Pss. Once this shear failure occurs, the shear zones along the failure surface effectively shorten the seepage path. The location of the back of the shear zone is in the gravel pocket, where the second phase of through seepage failure occurs. This is labeled as the probability of piping through the embankment sands (Pg|ss) given that a slope stability failure occurs. It is calculated the same way as for through seepage and the same values are used. The entire conditional probability of breach from slope stability Pb is :

$$Pss \times Pg|ss = Pb$$

9.4.5. Analysis Results and Model Calibration.

9.4.5.1. Model calibration for seepage analyses. Once the appropriate features were selected, constant head boundary conditions could be applied. A lake level of 18.5 was selected since data from the 1995 High Water Event was available. Seepage was observed exiting along the embankment slope, and softening and small boils on the ditch bottom were observed. The filtercake material and heaved toe ditch bottom were used to "calibrate" as entrance and exit valves, respectively, so head under the confining layer agreed with the actual piezometer readings. The absence of filtercake drastically alters the head distribution below the confining layer.

9.4.5.2. Model calibration for slope stability analyses. The search for the global critical failure arc consisted of examining surfaces tangent to the top of the rock layers and clay layer as well as varying tailwater elevations. The results are shown in Table H-9.9.

9.4.6. Length Effects. Each failure mode was adjusted to incorporate the length effect. For this model, a "link" length of 20,000 feet was selected. This is based primarily on the more general application of limited data in this area. In other words, only one model was analyzed for the entire reach based on resource constraints. The general nature of the model and length the model is applied to preclude a much shorter link length.

9.4.7. Pup Curve Development.

The combined probability of breach function versus lake level is a combination of the three failure modes evaluated. Individual values are listed in Table H-9.9. These values are presented for evaluation of the impact on the sum of the variance on probability of failure when compared to the computed factor of safety. Note that for through seepage (DS-ih) the value for lake el. 17 is affected buy a large amount of uncertainty which shows as higher probability of failure than at elevation 18.5. Under seepage has very little uncertainty associated with this failure mode and is very sensitive to changes in lake level (net head).

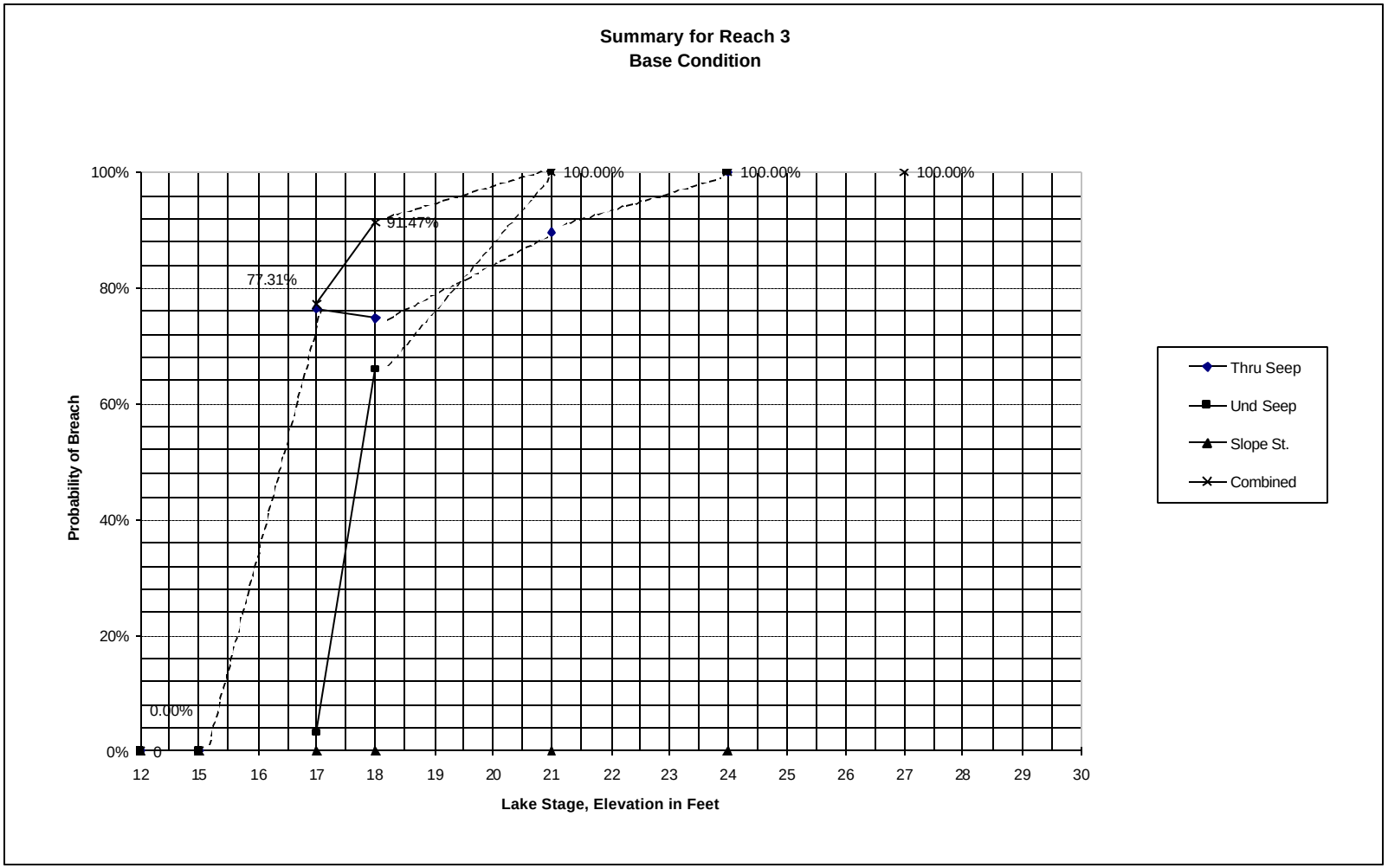
Table H-9.9. Analysis Summary.

Lake El.	**Failure Mode**	**E(FS)**	**Beta**	**Sum Variance**	**ic**	**P(Failure)**	**P(Failure) w/ length**
15	DB-iv-CL	1.679	4.135	0.0429	0.863	1.64e-5	3.08e-5
17	DB-iv-CL	1.301	2.138	0.0244	0.863	0.0163	0.0303
17	DS-ih	1.789	-0.098	9.1797	0.601	0.539	0.766
18.5	DB-iv-CL	1.021	0.158	0.0106	0.863	0.437	0.660
18.5	DS-ih	1.087	-0.054	0.2869	0.601	0.522	0.749
21	DB-iv-CL	0.801	-2.137	0.0073	0.863	0.984	1.00
21	DS-ih	0.950	-0.532	0.0106	0.601	0.703	0.897
			Slope	Stability			
18.5	Tan-rock	2.037	3.834	0.1386	n/a	6.31e-5	1.84e-4
24	Tan-rock	1.439	5.613*	0.0113*	n/a	9.97e-9*	1.87e-8*

Definitions:
DB = Ditch Bottom CL = Confining Layer
iv = vertical exit gradient ih = horizontal exit gradient
Tan-rock = Failure arc tangent to rock layer
* = Values should be higher (Beta lower) - all of variance likely not included in analysis.

9.4.8. Probabilistic Analyses Computation Sheets and Sections.

R3TASUM.XLS


Summary for Reach 3
Base Condition
100%
80%
60%
40%
20%
0%
Probability of Breach
100.00%
100.00%
100.00%
91.47%
77.31%
0.00%
0
12
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
Lake Stage, Elevation in Feet
Thru Seep
Und Seep
Slope St.
Combined

LEGEND FOR COLOR CROSS SECTIONS

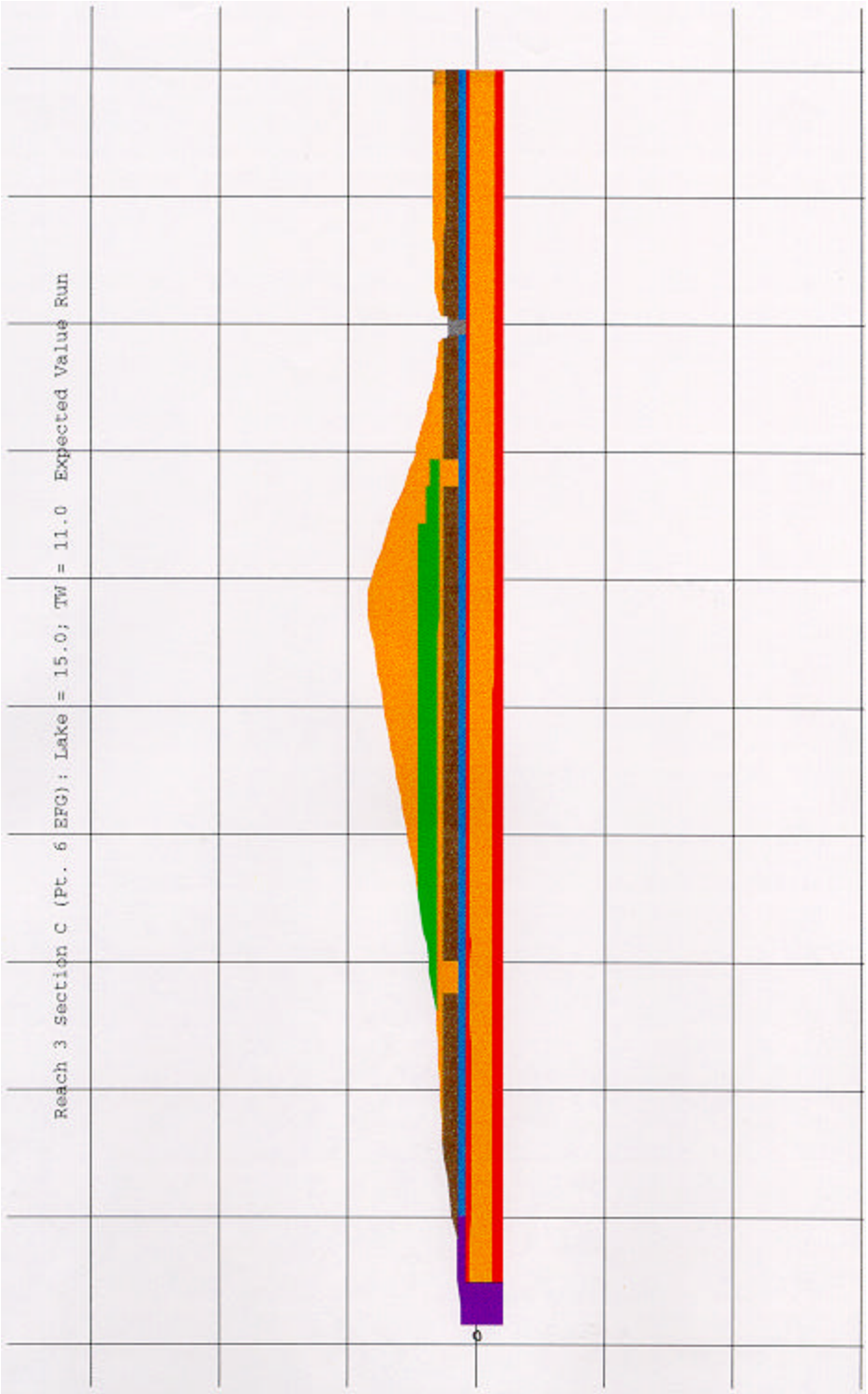
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

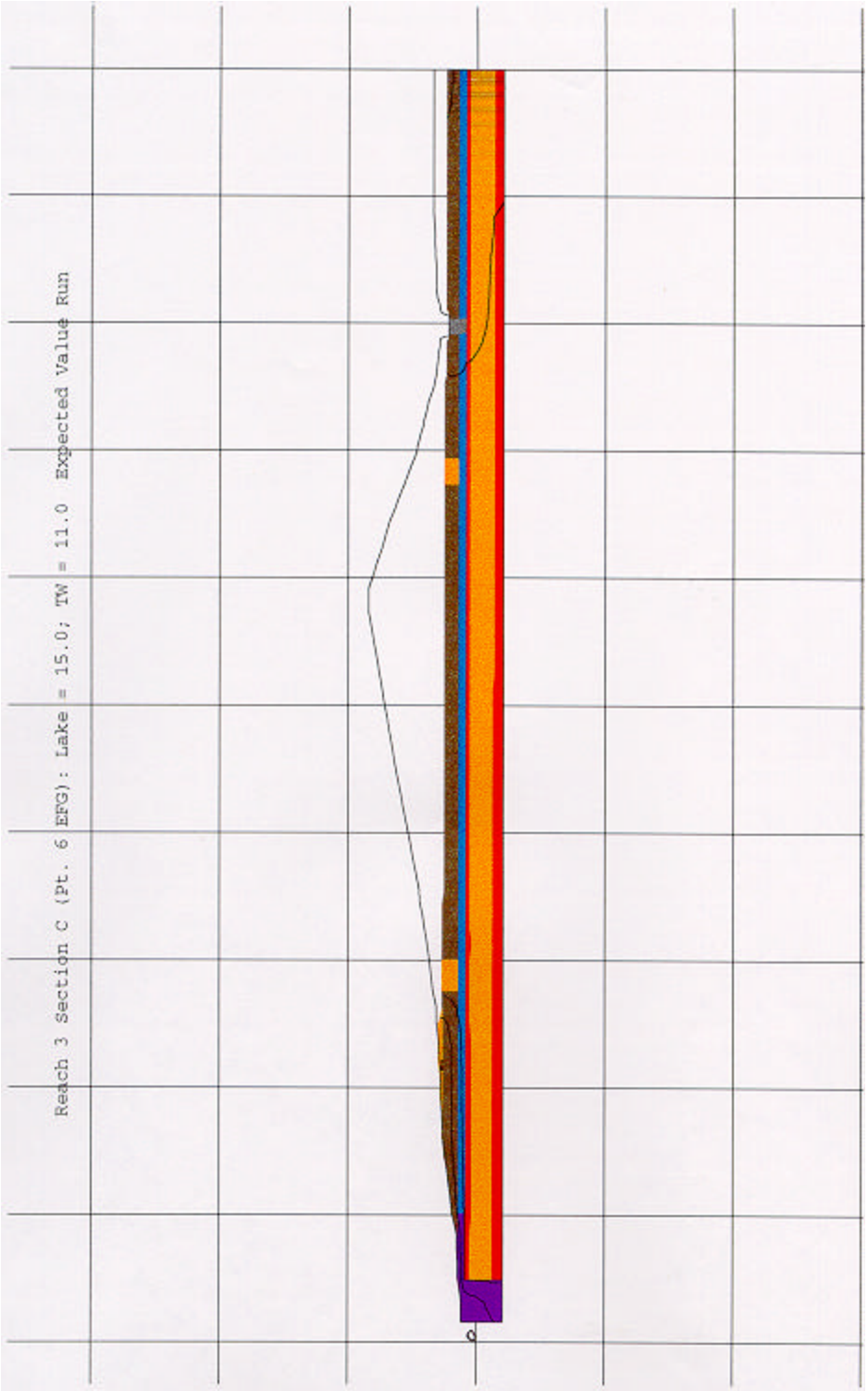
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

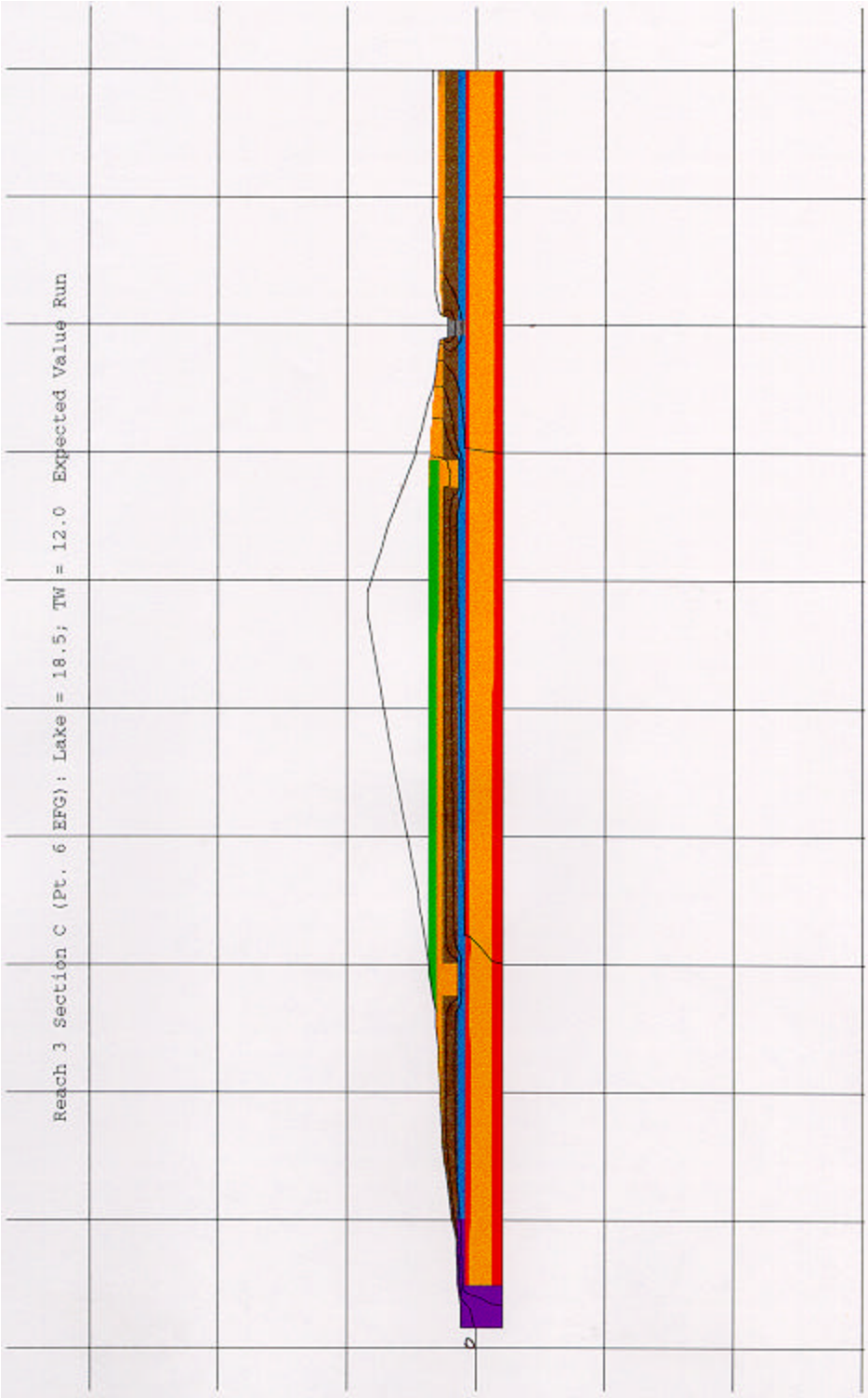
Reach 3 Section C (Pt. 6 EFG): Lake = 15.0; TW = 11.0 Expected Value Run



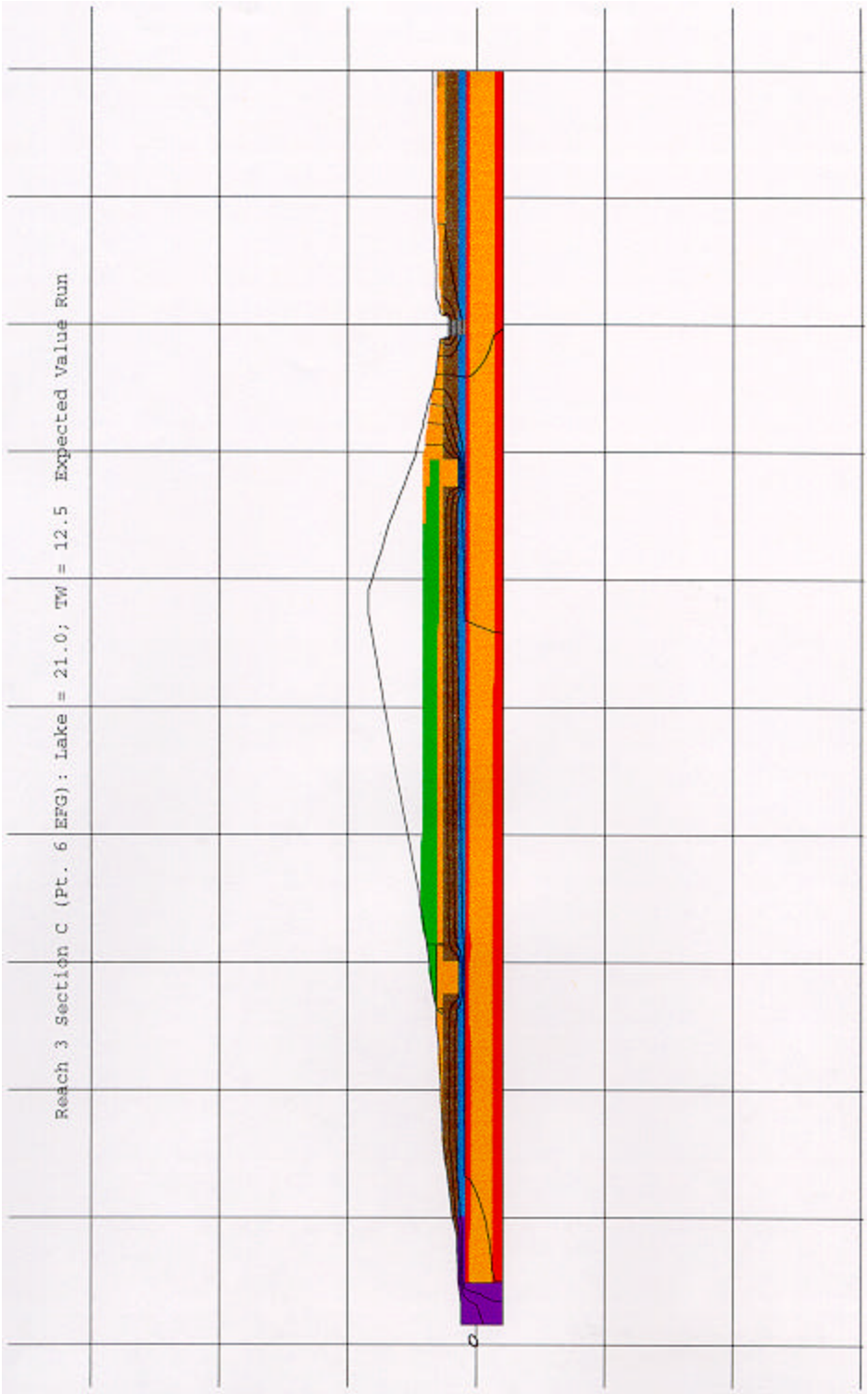
Reach 3 Section C (Pt. 6 EFG): Lake = 15.0; TW = 11.0 Expected Value Run



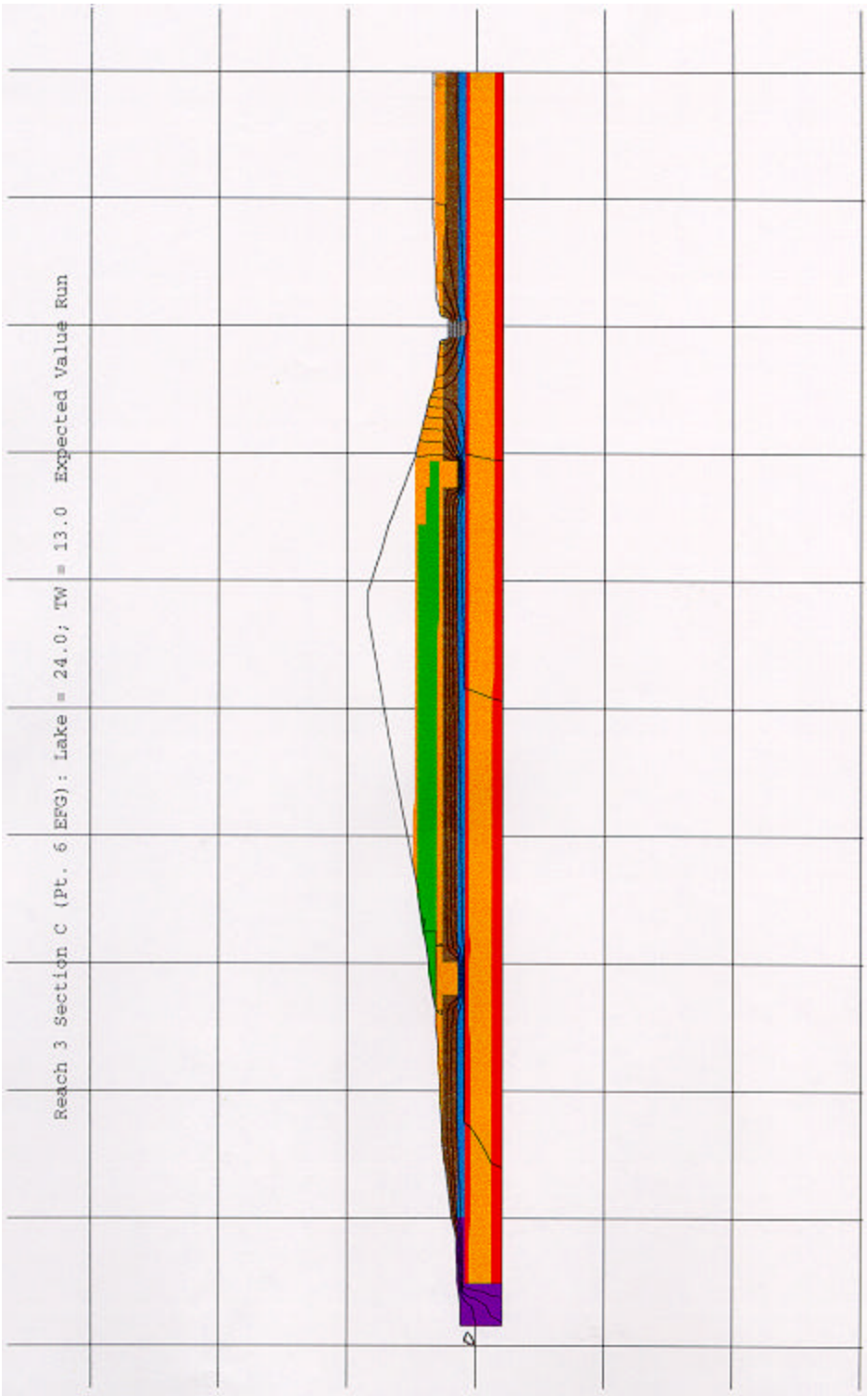
Reach 3 Section C (Pt. 6 EFG): Lake = 18.5; TW = 12.0 Expected Value Run



Reach 3 Section C (Pt. 6 EFG): Lake = 21.0; TW = 12.5 Expected Value Run



Reach 3 Section C (Pt. 6 EFG): Lake = 24.0; TW = 13.0 Expected Value Run



R3-SEEP.XLS, Summary

Exit Gradient Computations			**Reach 3; Lake = 15.0; Mean TW = 11.0**			
				"CL" Series		
	Across ditch bottom.					**Ditch Bottom**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**399.00**	**14.34**	**11.00**	**11.00**	**4.50**	**0.514**
1	399.00	14.34	11.02	11.00	4.50	0.511
2	399.00	14.62	11.01	11.00	4.50	0.555
3	399.00	14.16	11.02	11.00	4.50	0.483
4	399.00	14.48	11.01	11.00	4.50	0.534
7	399.00	14.41	11.02	11.00	4.50	0.522
8	399.00	14.22	11.00	11.00	4.50	0.495
15	399.00	14.35	11.31	11.00	4.50	0.468
16	399.00	14.32	10.87	10.85	4.50	0.543
17	399.00	14.34	11.01	11.00	4.50	0.512
18	399.00	14.29	11.01	11.00	4.50	0.505
19	399.00	14.34	11.00	11.00	4.50	0.514
20	399.00	14.34	11.00	11.00	4.50	0.514

Summary

Exit Gradient Computations			**Reach 3; Lake = 17.0; Mean TW = 12.0**			
				Regular Series		
	At ditch slope face.					**Ditch Slope**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**13.00**	**13.51**	**13.00**	**394.50**	**393.00**	**0.340**
1	13.00	13.56	13.00	394.50	393.00	0.373
2	13.00	13.35	13.00	394.50	393.00	0.233
3	13.00	13.41	13.00	394.50	393.00	0.273
4	13.00	13.54	13.00	394.50	393.00	0.360
11	13.00	13.46	13.00	394.50	393.00	0.307
12	13.00	13.55	13.00	394.50	393.00	0.367
13	13.00	13.55	13.00	394.50	393.00	0.367
14	13.00	13.40	13.00	394.50	393.00	0.267
17	13.00	13.56	13.00	394.50	393.00	0.373
18	13.00	13.12	13.00	394.50	393.00	0.080
19	13.00	13.51	13.00	394.50	393.00	0.340
20	13.00	13.51	13.00	394.50	393.00	0.340
				"CL" Series		
	Across ditch bottom.					**Ditch Bottom**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**399.00**	**16.32**	**12.01**	**11.00**	**4.50**	**0.663**
1	399.00	16.12	12.00	11.00	4.50	0.634
2	399.00	16.36	12.00	11.00	4.50	0.671
3	399.00	16.94	12.00	11.00	4.50	0.760
4	399.00	16.41	12.00	11.00	4.50	0.678
7	399.00	16.38	12.00	11.00	4.50	0.674
8	399.00	16.12	12.00	11.00	4.50	0.634
15	399.00	16.25	12.31	11.00	4.50	0.606
16	399.00	16.20	11.73	11.00	4.50	0.688
17	399.00	16.29	12.00	11.00	4.50	0.660
18	399.00	16.01	12.00	11.00	4.50	0.617
19	399.00	16.32	12.01	11.00	4.50	0.663
20	399.00	16.32	12.01	11.00	4.50	0.663

Summary

Exit Gradient Computations			**Reach 3; Lake = 18.5; Mean TW = 12.0**			
				Regular Series		
	At ditch slope face.					**Ditch Slope**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**13.00**	**13.83**	**13.00**	**394.50**	**393.00**	**0.553**
1	13.00	13.89	13.00	394.50	393.00	0.593
2	13.00	13.67	13.00	394.50	393.00	0.447
3	13.00	13.75	13.00	394.50	393.00	0.500
4	13.00	13.91	13.00	394.50	393.00	0.607
5	13.00	13.82	13.00	394.50	393.00	0.547
6	13.00	13.84	13.00	394.50	393.00	0.560
7	13.00	13.83	13.00	394.50	393.00	0.553
8	13.00	13.82	13.00	394.50	393.00	0.547
9	13.00	13.84	13.00	394.50	393.00	0.560
10	13.00	13.82	13.00	394.50	393.00	0.547
11	13.00	13.77	13.00	394.50	393.00	0.513
12	13.00	13.90	13.00	394.50	393.00	0.600
13	13.00	13.88	13.00	394.50	393.00	0.587
14	13.00	13.75	13.00	394.50	393.00	0.500
15	13.00	13.74	13.00	394.50	393.00	0.493
16	13.00	13.73	13.00	394.50	393.00	0.487
17	13.00	13.92	13.00	394.50	393.00	0.613
18	13.00	13.47	13.00	394.50	393.00	0.313
19	13.00	13.83	13.00	394.50	393.00	0.553
20	13.00	13.83	13.00	394.50	393.00	0.553
				"CL" Series		
	Across ditch bottom.					**Ditch Bottom**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**399.00**	**17.52**	**12.03**	**11.00**	**4.50**	**0.845**
1	399.00	17.35	12.03	11.00	4.50	0.818
2	399.00	17.62	12.01	11.00	4.50	0.863
3	399.00	17.40	12.02	11.00	4.50	0.828
4	399.00	17.69	12.03	11.00	4.50	0.871
5	399.00	17.60	12.02	11.00	4.50	0.858
6	399.00	17.53	12.01	11.00	4.50	0.849
7	399.00	17.66	12.01	11.00	4.50	0.869
8	399.00	17.31	12.01	11.00	4.50	0.815
9	399.00	17.61	12.02	11.00	4.50	0.860
10	399.00	17.49	12.02	11.00	4.50	0.842
13	399.00	17.57	12.01	11.00	4.50	0.855
14	399.00	17.47	12.02	11.00	4.50	0.838
15	399.00	17.49	12.30	11.00	4.50	0.798
16	399.00	17.46	11.72	11.00	4.50	0.883
17	399.00	17.54	12.03	11.00	4.50	0.848
18	399.00	17.29	12.03	11.00	4.50	0.809
19	399.00	17.52	12.03	11.00	4.50	0.845
20	399.00	17.52	12.03	11.00	4.50	0.845

Summary

Exit Gradient Computations			**Reach 3; Lake = 18.5; Mean TW = 12.0**			
	At pipe entrance					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**ih**
E	**-2.00**	**9.19**	**8.40**	**427.61**	**422.30**	**0.149**
1	-2.00	9.03	8.50	427.61	422.30	0.100
2	-2.00	9.27	8.30	427.61	422.30	0.183
7	-2.00	8.87	8.35	427.61	422.30	0.098
8	-2.00	9.39	8.42	427.61	422.30	0.183
9	-2.00	9.57	8.53	427.61	422.30	0.196
10	-2.00	8.84	8.32	427.61	422.30	0.098
19	-2.00	9.19	8.40	427.61	422.30	0.149
20	-2.00	9.19	8.40	427.61	422.30	0.149

Summary

Exit Gradient Computations			**Reach 3; Lake = 21.0; Mean TW = 12.5**			
				Regular Series		
	At ditch slope face.					**Ditch Slope**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**13.00**	**13.95**	**13.00**	**394.50**	**393.00**	**0.633**
1	13.00	13.95	13.00	394.50	393.00	0.633
2	13.00	13.90	13.00	394.50	393.00	0.600
3	13.00	13.92	13.00	394.50	393.00	0.613
4	13.00	13.95	13.00	394.50	393.00	0.633
11	13.00	13.94	13.00	394.50	393.00	0.627
12	13.00	13.98	13.00	394.50	393.00	0.653
13	13.00	13.93	13.00	394.50	393.00	0.620
14	13.00	13.92	13.00	394.50	393.00	0.613
17	13.00	13.96	13.00	394.50	393.00	0.640
18	13.00	13.96	13.00	394.50	393.00	0.640
19	13.00	13.95	13.00	394.50	393.00	0.633
20	13.00	13.95	13.00	394.50	393.00	0.633
				"CL" Series		
	Across ditch bottom.					**Ditch Bottom**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**399.00**	**19.53**	**12.53**	**11.00**	**4.50**	**1.077**
1	399.00	19.03	12.51	11.00	4.50	1.003
2	399.00	19.70	12.50	11.00	4.50	1.108
3	399.00	19.32	12.52	11.00	4.50	1.046
4	399.00	19.79	12.52	11.00	4.50	1.118
7	399.00	19.73	12.52	11.00	4.50	1.109
8	399.00	19.19	12.50	11.00	4.50	1.029
15	399.00	19.47	12.82	11.00	4.50	1.023
16	399.00	19.45	12.23	11.00	4.50	1.111
17	399.00	19.54	12.51	11.00	4.50	1.082
18	399.00	19.39	12.52	11.00	4.50	1.057
19	399.00	19.53	12.53	11.00	4.50	1.077
20	399.00	19.53	12.53	11.00	4.50	1.077

R3-SEEP.XLS, Ditch Bottom, iv15CL

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model:	FASTSEEP	**Lake Level:**	**15.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	**11.0**
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Ditch Bottom, iv

Run	1 Sands	4 Peat Horiz.	4 Peat Vert.	5 Rock Horiz.	5 Rock Vert.	Ditch Tailwater	Gravel Pocket	i crit	Results: i exit	Results: FS	Prob Calcs: Var	Prob Calcs: %
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**1.77E-01**	**1.41E-02**	**12.0**	**207**	**0.863**	**0.514**	**1.679**		
1	**1.41E-02**	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.863	**0.511**	1.689		
2	**2.15E-03**	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.863	**0.555**	1.553	0.0046	10.73%
3	4.95E-03	**3.12E-04**	**9.53E-05**	1.77E-01	1.41E-02	12.0	207	0.863	**0.483**	1.785		
4	4.95E-03	**8.20E-05**	**2.23E-05**	1.77E-01	1.41E-02	12.0	207	0.863	**0.534**	1.616	0.0072	16.81%
7	4.95E-03	1.60E-04	4.61E-05	**4.06E-01**	**3.25E-02**	12.0	207	0.863	**0.522**	1.654		
8	4.95E-03	1.60E-04	4.61E-05	**7.69E-02**	**6.15E-03**	12.0	207	0.863	**0.495**	1.741	0.0019	4.45%
15	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**12.3**	207	0.863	**0.468**	1.844		
16	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**11.7**	207	0.863	**0.543**	1.588	0.0165	38.42%
17	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	217	0.863	**0.512**	1.684		
18	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	187	0.863	**0.505**	1.709	0.0002	0.38%
19	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.920	**0.514**	1.790		
20	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.805	**0.514**	1.567	0.0125	29.21%
										Sum:	0.0429	100.00%

		For Piping	*For Breach*				
Critical Gradient:	Avg	0.75	0.863	**E[FS] =**	**1.679**	E[ln(FS)] =	0.5104
	High	0.800	0.920	Var[FS] =	0.0429	Var[ln(FS)] =	0.0151
	Low	0.700	0.805	$_s$ **[FS] =**	**0.2070**	$_d$ln(FS)] =	0.1229
				V [FS] =	0.1234	V [ln(FS)] =	0.2408

$_b$ **=**	**4.153**		
Pr(f) =	1.64E-05		
1 in	**60941**		

Adjusted probabilities based on length effect:

Total Length =	37,500	Reliability of Section, R =	1.00E+00
"Link" Length =	20000	**Adjusted Pr(f) =**	**3.08E-05**
No. Links =	1.88	**1 in**	**32502**

R3-SEEP.XLS, Ditch Bottom,iv17CL

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Lake Level: **17.0**

Tailwater: **12.0**

Ditch Bottom, iv

	1	4	4	5	5				Results		Prob Calcs	
		Peat	Peat	Rock	Rock	Ditch	Gravel					
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Tailwater	Pocket	i crit	i exit	**FS**	Var	%
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**1.77E-01**	**1.41E-02**	**12.0**	**207**	**0.863**	**0.663**	**1.301**		
1	**1.41E-02**	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.863	**0.634**	1.361		
2	**2.15E-03**	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.863	**0.671**	1.286	0.0014	5.75%
3	4.95E-03	**3.12E-04**	**9.53E-05**	1.77E-01	1.41E-02	12.0	207	0.863	**0.760**	1.135		
4	4.95E-03	**8.20E-05**	**2.23E-05**	1.77E-01	1.41E-02	12.0	207	0.863	**0.678**	1.271	0.0047	19.06%
7	4.95E-03	1.60E-04	4.61E-05	**4.06E-01**	**3.25E-02**	12.0	207	0.863	**0.674**	1.280		
8	4.95E-03	1.60E-04	4.61E-05	**7.69E-02**	**6.15E-03**	12.0	207	0.863	**0.634**	1.361	0.0016	6.68%
15	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**12.3**	207	0.863	**0.606**	1.423		
16	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**11.7**	207	0.863	**0.688**	1.254	0.0071	29.16%
17	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	217	0.863	**0.660**	1.307		
18	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	187	0.863	**0.617**	1.398	0.0021	8.53%
19	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.920	**0.663**	1.387		
20	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.805	**0.663**	1.214	0.0075	30.82%
										Sum:	0.0244	100.00%

		For Piping	*For Breach*				
Critical Gradient:	Avg	0.75	0.863	**E[FS] =**	**1.301**	E[ln(FS)] =	0.2558
	High	0.800	0.920	Var[FS] =	0.0244	Var[ln(FS)] =	0.0143
	Low	0.700	0.805	$_s$ **[FS] =**	**0.1562**	$_{ln}$[ln(FS)] =	0.1197
				V [FS] =	0.1201	V [ln(FS)] =	0.4678

$_b$ =	**2.138**		
Pr(f) =	1.63E-02		
1 in	**61**		

Adjusted probabilities based on length effect:

Total Length =	37,500	Reliability of Section, R =	9.70E-01
"Link" Length =	20000	**Adjusted Pr(f) =**	**3.03E-02**
No. Links =	1.88	**1 in**	**33**

R3-SEEP.XLS, Ditch Slope,ih17

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model:	FASTSEEP	**Lake Level:**	**17.0**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	**12.0**
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Ditch Slope, ih

	1	4	4	7	2			Results		Prob Calcs	
		Peat	Peat	Exit		Gravel					
Run	Sands	Horiz.	Vert.	Valve	Gravel	Pocket	i crit	i exit	**FS**	Var	%
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**3.89E-03**	**7.07E-01**	**207**	**0.601**	**0.340**	**1.769**		
1	**1.41E-02**	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.601	**0.373**	1.611		
2	**2.15E-03**	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.601	**0.233**	2.578	0.2336	2.54%
3	4.95E-03	**3.12E-04**	**9.53E-05**	3.89E-03	7.07E-01	207	0.601	**0.273**	2.200		
4	4.95E-03	**8.20E-05**	**2.23E-05**	3.89E-03	7.07E-01	207	0.601	**0.360**	1.671	0.0702	0.76%
11	4.95E-03	1.60E-04	4.61E-05	**8.94E-03**	7.07E-01	207	0.601	**0.307**	1.961		
12	4.95E-03	1.60E-04	4.61E-05	**1.69E-03**	7.07E-01	207	0.601	**0.367**	1.640	0.0257	0.28%
13	4.95E-03	1.60E-04	4.61E-05	3.89E-03	**1.63E+00**	207	0.601	**0.367**	1.640		
14	4.95E-03	1.60E-04	4.61E-05	3.89E-03	**3.08E-01**	207	0.601	**0.267**	2.255	0.0946	1.03%
17	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	217	0.601	**0.373**	1.611		
18	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	187	0.601	**0.080**	7.518	8.7235	95.03%
19	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.662	**0.340**	1.948		
20	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.541	**0.340**	1.590	0.0321	0.35%
									Sum:	9.1797	100.00%

		For Piping	*For Breach*				
Critical Gradient:	Avg	0.523	0.601	**E[FS] =**	**1.769**	E[ln(FS)] =	-0.1144
	High	0.576	0.662	Var[FS] =	9.1797	Var[ln(FS)] =	1.3695
	Low	0.470	0.541	$_s$ **[FS] =**	**3.0298**	$_d$ln(FS)] =	1.1703
				V [FS] =	1.7127	V [ln(FS)] =	-10.2326

Adjusted probabilities based on length effect:	**Total Length =**	37,500	Reliability of Section, R = 2.34E-01
	"Link" Length =	20000	**Adjusted Pr(f) = 7.66E-01**
	No. Links =	1.88	**1 in 1**

$_b$ **= -0.098**

Pr(f) = 5.39E-01

1 in 2

R3-SEEP.XLS, Ditch Bottom,iv18CL

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model:	FASTSEEP	**Lake Level:**	**18.5**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	**12.0**
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Ditch Bottom, iv

	1	4	4	3	3	5	5	6	2				Results		Prob Calcs	
		Peat	Peat	Clay	Clay	Rock	Rock	Filter		Ditch	Gravel					
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Cake	Gravel	Tailwater	Pocket	i crit	i exit	**FS**	Var	%
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**7.07E-01**	**12.0**	**207**	**0.863**	**0.845**	**1.021**		
1	**1.41E-02**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.863	**0.818**	1.054		
2	**2.15E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.863	**0.863**	0.999	0.0007	6.98%
3	4.95E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.863	**0.828**	1.042		
4	4.95E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.863	**0.871**	0.991	0.0007	6.25%
5	4.95E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.863	**0.858**	1.005		
6	4.95E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.863	**0.849**	1.016	0.0000	0.26%
7	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	6.01E-05	7.07E-01	12.0	207	0.863	**0.869**	0.992		
8	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	6.01E-05	7.07E-01	12.0	207	0.863	**0.815**	1.058	0.0011	10.10%
9	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.38E-04**	7.07E-01	12.0	207	0.863	**0.860**	1.003		
10	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.61E-05**	7.07E-01	12.0	207	0.863	**0.842**	1.025	0.0001	1.14%
13	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**1.63E+00**	12.0	207	0.863	**0.855**	1.008		
14	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**3.08E-01**	12.0	207	0.863	**0.838**	1.029	0.0001	0.97%
15	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	**12.3**	207	0.863	**0.798**	1.080		
16	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	**11.7**	207	0.863	**0.883**	0.977	0.0027	25.20%
17	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	217	0.863	**0.848**	1.017		
18	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	187	0.863	**0.809**	1.066	0.0006	5.50%
19	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.920	**0.845**	1.089		
20	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	7.07E-01	12.0	207	0.805	**0.845**	0.953	0.0046	43.60%
														Sum:	0.0106	100.00%

		For Piping	*For Breach*	**E[FS] =**	**1.021**	E[ln(FS)] = 0.0159
Critical Gradient:	Avg	0.75	0.863	Var[FS] =	0.0106	Var[ln(FS)] = 0.0101
	High	0.800	0.920	$_S$ **[FS] =**	**0.1031**	σ[ln(FS)] = 0.1007
	Low	0.700	0.805	V [FS] =	0.1010	V [ln(FS)] = 6.3406

Adjusted probabilities based on length effect:	**Total Length =**	37,500	Reliability of Section, R = 3.40E-01
	"Link" Length =	20000	**Adjusted Pr(f) = 6.60E-01**
	No. Links =	1.88	**1 in 2**

$_b$ =	**0.158**
Pr(f) =	4.37E-01
1 in	**2**

R3-SEEP.XLS, Ditch Slope,ih18

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model: FASTSEEP
Probabilistic Model: Taylor's Series, Finite Difference
Performance Function: FS
Random Variable
Distribution Assumptions: K's lognormal, i crit and ditch water level normal

Lake Level: **18.5**
Tailwater: **12.0**

	1	4	4	3	3	5	5	6	7	2				**Ditch Slope, ih**			
		Peat	Peat	Clay	Clay	Rock	Rock	Filter	Exit		Ditch	Gravel		Results		Prob Calcs	
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Cake	Valve	Gravel	Tailwater	Pocket	i crit	i exit	**FS**	Var	%
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**1.27E-04**	**7.11E-05**	**1.77E-01**	**1.41E-02**	**6.01E-05**	**3.89E-03**	**7.07E-01**	**12.0**	**207**	**0.601**	**0.553**	**1.087**		
1	**1.41E-02**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.593**	1.014		
2	**2.15E-03**	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.447**	1.347	0.0277	9.65%
3	4.95E-03	**3.12E-04**	**9.53E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.500**	1.203		
4	4.95E-03	**8.20E-05**	**2.23E-05**	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.607**	0.991	0.0112	3.90%
5	4.95E-03	1.60E-04	4.61E-05	**2.27E-04**	**1.81E-04**	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.547**	1.100		
6	4.95E-03	1.60E-04	4.61E-05	**7.09E-05**	**2.79E-05**	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.560**	1.074	0.0002	0.06%
7	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**4.06E-01**	**3.25E-02**	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.553**	1.087		
8	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	**7.69E-02**	**6.15E-03**	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.601	**0.547**	1.100	0.0000	0.02%
9	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**1.38E-04**	3.89E-03	7.07E-01	12.0	207	0.601	**0.560**	1.074		
10	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	**2.61E-05**	3.89E-03	7.07E-01	12.0	207	0.601	**0.547**	1.100	0.0002	0.06%
11	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**8.94E-03**	7.07E-01	12.0	207	0.601	**0.513**	1.172		
12	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	**1.69E-03**	7.07E-01	12.0	207	0.601	**0.600**	1.002	0.0072	2.50%
13	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	**1.63E+00**	12.0	207	0.601	**0.587**	1.025		
14	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	**3.08E-01**	12.0	207	0.601	**0.500**	1.203	0.0079	2.75%
15	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	**12.3**	207	0.601	**0.493**	1.219		
16	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	**11.7**	207	0.601	**0.487**	1.236	0.0001	0.02%
17	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	217	0.601	**0.613**	0.981		
18	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	187	0.601	**0.313**	1.920	0.2204	76.81%
19	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.662	**0.553**	1.197		
20	4.95E-03	1.60E-04	4.61E-05	1.27E-04	7.11E-05	1.77E-01	1.41E-02	6.01E-05	3.89E-03	7.07E-01	12.0	207	0.541	**0.553**	0.977	0.0121	4.23%
															Sum:	0.2869	100.00%

		For Piping	*For Breach*	
Critical Gradient:	Avg	0.523	0.601	
	High	0.576	0.662	
	Low	0.470	0.541	

E[FS] =	**1.087**	E[ln(FS)] =	-0.0253
Var[FS] =	0.2869	Var[ln(FS)] =	0.2174
$_s$ **[FS] =**	**0.5356**	$_d$[ln(FS)] =	0.4663
V [FS] =	0.4928	V [ln(FS)] =	-18.4178

Adjusted probabilities based on length effect:

Total Length =	37,500	Reliability of Section, R =	2.51E-01
"Link" Length =	20000	**Adjusted Pr(f) =**	**7.49E-01**
No. Links =	1.88	**1 in**	**1**

$_b$ **=** **-0.054**
Pr(f) = 5.22E-01
1 in **2**

R3-SEEP.XLS, Ditch Bottom,iv21CL

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Lake Level: **18.5**

Tailwater: **12.0**

Ditch Bottom, iv

	1	4	4	5	5				Results		Prob Calcs	
		Peat	Peat	Rock	Rock	Ditch	Gravel					
Run	Sands	Horiz.	Vert.	Horiz.	Vert.	Tailwater	Pocket	i crit	i exit	**FS**	Var	%
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**1.77E-01**	**1.41E-02**	**12.0**	**207**	**0.863**	**1.077**	**0.801**		
1	**1.41E-02**	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.863	**1.003**	0.860		
2	**2.15E-03**	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.863	**1.108**	0.779	0.0017	22.59%
3	4.95E-03	**3.12E-04**	**9.53E-05**	1.77E-01	1.41E-02	12.0	207	0.863	**1.046**	0.824		
4	4.95E-03	**8.20E-05**	**2.23E-05**	1.77E-01	1.41E-02	12.0	207	0.863	**1.118**	0.771	0.0007	9.70%
7	4.95E-03	1.60E-04	4.61E-05	**4.06E-01**	**3.25E-02**	12.0	207	0.863	**1.109**	0.778		
8	4.95E-03	1.60E-04	4.61E-05	**7.69E-02**	**6.15E-03**	12.0	207	0.863	**1.029**	0.838	0.0009	12.47%
15	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**12.3**	207	0.863	**1.023**	0.843		
16	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	**11.7**	207	0.863	**1.111**	0.776	0.0011	15.12%
17	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	217	0.863	**1.082**	0.797		
18	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	187	0.863	**1.057**	0.816	0.0001	1.18%
19	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.920	**1.077**	0.854		
20	4.95E-03	1.60E-04	4.61E-05	1.77E-01	1.41E-02	12.0	207	0.805	**1.077**	0.748	0.0029	38.93%
										Sum:	0.0073	100.00%

		For Piping	*For Breach*			
Critical Gradient:	Avg	0.75	0.863			
	High	0.800	0.920			
	Low	0.700	0.805			

E[FS] =	**0.801**	E[ln(FS)] =	-0.2277
Var[FS] =	0.0073	Var[ln(FS)] =	0.0114
$_s$ **[FS] =**	**0.0856**	$_{ln}$ [ln(FS)] =	0.1065
V [FS] =	0.1068	V [ln(FS)] =	-0.4679

$_b$ =	**-2.137**		
Pr(f) =	9.84E-01		
1 in	**1**		

Adjusted probabilities based on length effect:

Total Length =	37,500	Reliability of Section, R =	4.44E-04
"Link" Length =	20000	**Adjusted Pr(f) =**	**1.00E+00**
No. Links =	1.88	**1 in**	**1**

R3-SEEP.XLS, Ditch Slope,ih21

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 3**

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Lake Level: **21.0**

Tailwater: **12.5**

Ditch Slope, ih

Run	1 Sands	4 Peat Horiz.	4 Peat Vert.	7 Exit Valve	2 Gravel	Gravel Pocket	i crit	Results i exit	Results FS	Prob Calcs Var	Prob Calcs %
E	**4.95E-03**	**1.60E-04**	**4.61E-05**	**3.89E-03**	**7.07E-01**	**207**	**0.601**	**0.633**	**0.950**		
1	**1.41E-02**	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.601	**0.633**	0.950		
2	**2.15E-03**	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.601	**0.600**	1.002	0.0007	6.56%
3	4.95E-03	**3.12E-04**	**9.53E-05**	3.89E-03	7.07E-01	207	0.601	**0.613**	0.981		
4	4.95E-03	**8.20E-05**	**2.23E-05**	3.89E-03	7.07E-01	207	0.601	**0.633**	0.950	0.0002	2.26%
11	4.95E-03	1.60E-04	4.61E-05	**8.94E-03**	7.07E-01	207	0.601	**0.627**	0.960		
12	4.95E-03	1.60E-04	4.61E-05	**1.69E-03**	7.07E-01	207	0.601	**0.653**	0.921	0.0004	3.62%
13	4.95E-03	1.60E-04	4.61E-05	3.89E-03	**1.63E+00**	207	0.601	**0.620**	0.970		
14	4.95E-03	1.60E-04	4.61E-05	3.89E-03	**3.08E-01**	207	0.601	**0.613**	0.981	0.0000	0.26%
17	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	217	0.601	**0.640**	0.940		
18	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	187	0.601	**0.640**	0.940	0.0000	0.00%
19	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.662	**0.633**	1.046		
20	4.95E-03	1.60E-04	4.61E-05	3.89E-03	7.07E-01	207	0.541	**0.633**	0.853	0.0093	87.30%
									Sum:	0.0106	100.00%

		For Piping	*For Breach*			
Critical Gradient:	Avg	0.523	0.601	**E[FS] =**	**0.950**	E[ln(FS)] = -0.0575
	High	0.576	0.662	Var[FS] =	0.0106	Var[ln(FS)] = 0.0117
	Low	0.470	0.541	$_s$ **[FS] =**	**0.1030**	$_d$ln(FS)] = 0.1081
				V [FS] =	0.1085	V [ln(FS)] = -1.8807

$_b$ = **-0.532**
Pr(f) = 7.03E-01
1 in **1**

Adjusted probabilities based on length effect:

Total Length =	37,500
"Link" Length =	20000
No. Links =	1.88

Reliability of Section, R = 1.03E-01
Adjusted Pr(f) = **8.97E-01**
1 in **1**

R3-SST.XLS, R3-18.5

Probabilistic Slope Stability Analysis:							**Herbert Hoover Dike; Model for Reach 3**			
Deterministic Model:	UTEXAS, Spencer's Method						**Lake Level:**	**18.5**		
Probabilistic Model:	Taylor's Series, Finite Difference						**Tailwater:**	**12.0**		
Performance Function:	FS									
Random Variable										
stribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater normal									
							Failure surface just below first rock lens.			
	Sands	Peat / OL / OH		Silts & Clays		Rock	Failure Arc	**Computed**	Prob Calcs	
Run	θ	θ	C	θ	C	θ	X, Y, R	**FS**	Var	%
E	33.0	33.0	160.0	12.5	620.0	45.0	163.6/85.6/85.3	**2.037**		
1	36.3	33.0	160.0	12.5	620.0	45.0	163.2/95.6/91.1	**2.274**		
2	29.7	33.0	160.0	12.5	620.0	45.0	163.0/83.4/83.7	**1.883**	0.0764	55.13%
3	33.0	41.3	160.0	12.5	620.0	45.0	162.6/84.6/84.9	**2.110**		
4	33.0	24.8	160.0	12.5	620.0	45.0	163.6/87.2/86.7	**1.972**	0.0095	6.87%
5	33.0	33.0	400.0	12.5	620.0	45.0	162.2/886.4/86.7	**2.140**		
6	33.0	33.0	80.0	12.5	620.0	45.0	164.0/87.4/86.7	**2.003**	0.0094	6.77%
7	33.0	33.0	160.0	15.6	620.0	45.0	162.0/86.0/86.1	**2.056**		
8	33.0	33.0	160.0	9.4	620.0	45.0	163.6/85.6/85.3	**2.018**	0.0007	0.52%
9	33.0	33.0	160.0	12.5	1020	45.0	161.8/72.2/74.9	**2.165**		
10	33.0	33.0	160.0	12.5	380	45.0	166.4/96.8/92.3	**1.883**	0.0398	28.68%
11	33.0	33.0	160.0	12.5	1020	49.5	162.2/84.8/85.3	**2.077**		
12	33.0	33.0	160.0	12.5	380	40.5	164.0/87.4/86.7	**2.002**	0.0028	2.03%
								Sum:	0.1386	100.00%
			E[FS] =	**2.037**	E[ln(FS)] =	0.6950				
			Var[FS] =	0.1386	Var[ln(FS)] =	0.0329			b =	**3.834**
			s **[FS] =**	**0.3723**	σ[ln(FS)] =	0.1813			Pr(f) =	6.31E-05
			V [FS] =	0.1828	V [ln(FS)] =	0.2608			**1 in**	**15846**
	Adjusted probabilities based on length effect:					**Total Length =**	37,500	Reliability of Section, R =		1.00E+00
						"Link" Length =	20000		**Adjusted Pr(f) =**	**1.18E-04**
						No. Links =	1.88		**1 in**	**8452**

R3-SST.XLS,R3-24

Probabilistic Slope Stability Analysis: **Herbert Hoover Dike; Model for Line 6**

Deterministic Model:	UTEXAS, Spencer's Method	**Lake Level:**	**24.0**
Probabilistic Model:	Taylor's Series, Finite Difference	**Tailwater:**	**13.0**
Performance Function:	FS		
Random Variable			
Distribution Assumptions:	Cohesion lognormal, Phi, gamma & Tailwater normal		
		Failure surface tangent to rock layer.	

Run	Sands θ	Silts & Clays C	Failure Arc X, Y, R	**Computed FS**	Prob Calcs Var	Prob Calcs %
E	33.0	620	180.4/98.2/90.6	**1.493**		
1	36.3	620	181.4/91.2/83.6	1.547		
2	29.7	620	179.2/104.3/96.8	1.397	0.0113	100.00%
9	33.0	1020	180.5/97.5/89.9	1.494		
10	33.0	380	180.3/99.0/91.4	1.493	0.0000	0.00%
				Sum:	0.0113	100.00%

E[FS] =	**1.493**	E[ln(FS)] =	0.3983
Var[FS] =	0.0113	Var[ln(FS)] =	0.0050
s **[FS] =**	**0.1061**	σ[ln(FS)] =	0.0710
V [FS] =	0.0710	V [ln(FS)] =	0.1782

b **=**	**5.613**		
Pr(f) =	9.97E-09		
1 in	**100335143**		

Adjusted probabilities based on length effect:

Total Length =	37,500	Reliability of Section, R =	1.00E+00
"Link" Length =	20000	**Adjusted Pr(f) =**	**1.87E-08**
No. Links =	1.88	**1 in**	**53512076**

Interpolation for el. 21

18.5	7.94E-04
21	3.97E-04
24	1.25E-07

I think we are missing some of the variance somewhere,
but the Pf is so low it wouldn,t make a difference.

9.5 Model for Reach 4

9.5.1 Location: Reach 4 is located on the southwest side of Lake Okeechobee in Glades County. This reach extends from Moore Haven to Fisheating Creek then west along the south bank of Fisheating Creek to point were the embankment ties into high ground, approximately 19.4 miles. Reach 4 covers portions of the levee system designated as L-D3. The embankment within this reach was constructed in 1933 by hydraulic fill from a deep borrow canal located at the lake side toe. The embankment was constructed to crest elevation of 30 feet. The embankment was raised in 1960s to crest elevation varying from 33 to 37.5 feet by hydraulic fill from deep borrow canal at the land side toe. The canal was excavated to elevation of -10 feet. The natural ground within this area is approximately 14 feet. The interior slope varies 1V on 6H to and the exterior slopes are 1V on 3H.

9.5.2 Subsurface Investigation: Subsurface investigation within this reach consisted primarily of evaluating existing core boring drilled prior to the construction of the embankment in 1933 and core borings obtained to raised the embankment in 1960. TO better define the soil stratification, shear strengths, permeabilities and other engineering properties of the embankment and foundation materials additional core borings and recharge test should be obtained within this reach.

9.5.3 Materials Encountered: Based on limited core boring data the embankment appears to consist of silty sand. and silt. The foundation materials consist of medium to dense, silty sand.

9.5.4 Investigation: Currently we do not have sufficient subsurface information to construct a soil section that will adequately describe all subsurface conditions within this reach. This is due to widely space core borings and lack of borings on the lake side and land side toe of the embankment.

9.5.5 Slope Stability Analysis: No Slope stability analyses were performed for this section.

9.5.6 Seepage Analysis: No seepage analysis was performed on this section.

9.5.7 Probabilistic Analysis: Conditional probability-of-failure versus lake elevation functions presented in this report, for this reach, was based on geometry of the embankment, embankment and foundation materials, observation from the field during periods of high water, past experience with similar earth structures and engineering judgment.

9.6 Model for Reach 5

9.6.1 Location: Reach 5 is located on the north side of Lake Okeechobee in Okeechobee County. This reach extends from Nubbin Slough to Kissimmee River and along the northeast bank of the Kissimmee River to structure S-154, approximately 15.7 miles. Reach 5 covers the portion of the levee system designated as L-D4. The embankment within this reach was constructed in 1933 by hydraulic fill from a deep canal located at the lake side toe. The crest elevation was of 22 feet. The lake side slopes were 1V on 8H and the land side slope was 1V on 3H. The embankment was raised in 1960s to crest elevation of 40 feet by hydraulic fill from deep borrow pit at the land side toe. The pit was excavated to elevation of –14 feet. The natural ground within this area is approximately 14 feet at the embankment and slopes up to elevation 25 feet about 1.5 to 5 miles to the north. The crest elevation varies from 34 to 40 feet. The interior slope varies 1V on 6H to 1V on 8V and the exterior slope are 1V on 3H.

9.6.2 Subsurface Investigation: Subsurface investigation within this reach consist primarily of evaluating existing core boring drilled prior to the construction of the embankment in 1933 and core borings obtained to raised the embankment in 1960.

9.6.3 Materials Encountered: Based on limited core boring data the embankment appears to consist of silty and shelly sand. The foundation materials consist of medium to dense silty and shelly sand.

9.6.4 Investigation: Currently we do not have sufficient subsurface information to construct soil section that will adequately describe all subsurface conditions within this reach. This is due to widely space core borings and lack of borings on the lake side and land side of the embankment. What land side borings available were drilled for the deep borrow canal excavation along the interior toe. A typical soil profile evaluated for this reach was based on existing survey data obtained from as-build drawings and limited subsurface data.

9.6.5 Slope Stability Analysis: Slope stability analyses were perform utilizing WES “UTEXAS2” software program (USACE, 1994). No additional analyses were preformed for this report. Based on the previous analyses for a lake level of 25 feet, the overall factor of safety was 1.36

9.6.6 Seepage Analysis: No seepage analysis was performed on this section.

9.6.7 Probabilistic Analysis: Conditional probability-of-failure versus lake elevation functions presented in this report, for this reach, was based on geometry of the embankment, embankment and foundation materials, observation from the field during periods of high water, past experience with similar earth structures and engineering judgment.

9.7 Model for Reach 6

9.7.1 Location: Reach 6 is located on the West Side of Lake Okeechobee in Glades county. This reach extends from a point were the embankment ties to high ground along Fisheating creek to Indian Prairie Canal, approximately 20 miles. Reach 6 covers portions of the levee system designated as L-50 and L-49. The embankment within this reach was constructed in 1962 by dragline excavating from deep borrow canal to elevation –10 feet that parallels land side toe. The natural ground within this area is approximately 15 feet and slope up to the west. The crest elevation varies from 32 feet, were the embankment tie to existing ground to elevation 39 feet. The interior slopes are 1V on 3H and the exterior slopes are 1V on 6H.

9.7.2 Subsurface Investigation: Subsurface investigation within this reach consisted primarily of evaluating existing core boring drilled prior to the construction of the embankment in 1960 and core borings through the embankment crest obtained in 1986.

9.7.3 Materials Encountered: Based on limited core boring data the embankment appears to consist of silty sand and silt. The foundation materials consist of silty sand with interbedded layers of shell, silt and clay.

9.7.4 Investigation: Currently we do not have sufficient subsurface information to construct soil section that will adequately describe all subsurface conditions within this reach. This is due to widely space core borings along the embankment centerline and lack of borings on the lake side and land side of the embankment. What land side borings available were drilled for the deep borrow canal along the exterior toe. A typical soil profile evaluated for this reach was based on existing survey data obtained from as-build drawings and limited subsurface data. Generally,

9.7.5 Slope Stability Analysis: Slope stability analyses were perform utilizing WES "UTEXAS2" software program (USACE, 1994). No additional analyses were preformed for this report. Based on the previous analyses for a lake level of 25 feet, the overall factor of safety was 1.36

9.7.6 Seepage Analysis: No seepage analysis was performed on this section.

9.7.7 Probabilistic Analysis: Conditional probability-of-failure versus lake elevation functions presented in this report, for this reach, was based on geometry of the embankment, embankment and foundation materials, observation from the field during periods of high water, past experience with similar earth structures and engineering judgment.

9.8. Model for Reach 7. This is part of the levee embankment designated as L-47 between Port Mayaca and Okeechobee. Two cross sections were obtained between S-191 and S-308. Section MRR7-B was obtained north of S-135 and section MRR7-A was obtained south of S-135. The embankment in this area typically has a 20 foot crest at elevation 38.0 feet, a 1V:5H lakeside slope and a 1V:3H landside slope. Construction of a seepage toe or fire toe was not indicated. One of the most apparent features at this section is the large landside borrow canal. The surveys from 1995 show excavation down to elevation -10 feet.

9.8.1. Selection of Variables and Values.

9.8.1.1. Topographic Features. The topographic feature significant to this model was the downstream borrow canal. These can be seen on Figure H-9.1.

9.8.1.2. Geologic Features. The geologic features selected as being significant to this model were the gravel pocket and existing pipes under the limestone lense roofs.. The gravel pocket in the embankment was established as a variable since borings through the embankment indicated the presence of coarse material. The geometry of the pocket was included as a variable. Refer to Table H-9.10. The other significant feature was the beginning of small "pipes". The presence and extent of the pipes was inferred from piezometer readings and geologic features from Reach 1 Line 6.

9.8.1.3. Slope Stability Variables. A slope stability analysis was not performed on this model. Limited resources required reducing the scope of the evaluation. Items for analysis where conditions were not as critical, or as high a risk, were evaluated but not analyzed. The foundation conditions in this area are generally better than areas where models were evaluated and found to be satisfactory.

9.8.1.4. Seepage Analysis Variables. Variables and values selected for seepage analyses were divided into two categories, those with normal distributions and those with lognormal distributions. All values for permeability were lognormally distributed.

Table H-9.10

	Permeability Summary for Reach 7				
	All values in ft/min				
			Lognormal Parameters		
Material	K - Horiz.	K-Vert.	Mean [$e^{\ln k}$]	High [$e^{\ln k + \sigma \ln k}$]	Low [$e^{\ln k + \sigma \ln k}$]
Sands	7.00E-03	7.00E-03	4.95E-03	1.14E-02	2.15E-03
Gravel	1.00E+00	1.00E+00	7.07E-01	1.63E+00	3.08E-01
Silts & Clays	0.00E+00	n/a	-8.97E+00	5.81E-01	0.00E+00
	n/a	0.00E+00	-9.55E+00	9.35E-01	0.00E+00
Rock	0.00E+00	n/a	-1.73E+00	8.33E-01	0.00E+00
	n/a	0.00E+00	-4.26E+00	8.33E-01	0.00E+00
Filtercake	8.50E-05	8.50E-05	6.01E-05	1.38E-04	2.61E-05
Pipe	5.00E+00	5.00E+00	3.54E+00	8.13E+00	1.54E+00

The spatial distribution of the gravel pocked was based on core boring data. The top and bottom elevations of the pocked were estimated from core borings.

9.8.2 Selection of Tailwater Distribution. Distributions for these analyses are shown in Table H-9.11. Tailwater elevations are artificially maintained by pumping. Operation range is between 13.5 - 14.0. However, the canal can be drawn down to 13.0 in advance of a storm. The higher elevation models (el. 18 and above) would likely be associated with storm events. The range of values was assumed to be normally distributed.

Table H-9.11

	Tailwater Elevations for Models				
Lake El.	Mean [E(x)]	Min / Max	σ(x)	High [E(x) +σ]	Low [E(x) +σ]
15	13.0		0.33	13.3	12.7
18	13.0		0.33	13.3	12.7
21	13.0		0.33	13.3	12.7
24	13.0		0.33	13.3	12.7

9.8.3. Selection of Critical Gradients.

9.8.3.1. Piping above the confining layer. Even at a lake elevation of 24, seepage did not exit the embankment or ditch slope. Therefore, through seepage would not be a failure mode.

9.8.3.1. Piping below the confining layer. The location of the pipes is expected to be directly under the limestone layers. This would mean the material is under little or no confining stress.

A wide range of values for i_{cv} was considered, from 0.1 to 0.5. This was due the wide range of gradations expected, fluctuations in confining stress conditions and variations in unit weight. The following values for i_{cv} were used in the analysis for through seepage.

Average Value: 0.350
Std. Dev.: 0.050
High Value: 0.400
Low Value: 0.300

Based on observations by Sellmeijer, the values for critical gradient computed above were increased by 15%. The values to compute the probability of breach for i_{cv} were:

Average Value: 0.403
Std. Dev.: 0.057
High Value: 0.460
Low Value: 0.346

For i_{ch}, the range of values for considered was from 0.15 to 0.30. This was based on the 0.16 encountered by Skempton (Skempton et. al., 1994) and Schmertmann (Schmertmann, 1997) and

the 0.3 by Schmertmann (Schmertmann, 1997) for a material with a Cu of about 1.5. The following values for i_{ch} were used in the analysis for through seepage.

Average Value:	0.225
Std. Dev.:	0.025
High Value:	0.250
Low Value:	0.200

Based on observations by Sellmeijer, the values for critical gradient computed above were increased by 15%. The values to compute the probability of breach for i_{ch} were:

Average Value:	0.259
Std. Dev.:	0.029
High Value:	0.289
Low Value:	0.230

9.8.4. Failure Mechanisms. There was one failure mechanisms considered for seepage.

9.8.4.1. Through seepage. Not considered since seepage did not exit the slope even at lake elevation 24.

9.8.4.2. Under seepage. The massive amount of excavation at the toe of the embankment made this the failure mode of most concern. Piping is defined as the removal of embankment or foundation material. Borrow operations were very successful at removing large quantities of foundation materials. In addition to the absence of much of the foundation, the borrow operation was deep enough to pass through two rock lenses in at least two locations. This provided two "roof" layers for pipe formation and direct access for material to pipe into the borrow lake undected. Evidence for existing "pipes" comes from the piezometer data ar Reach 1, Line 6. All four piezometers there read within 1 foot of each other and the quarry lake at various lake elevations.

9.8.4.3. Slope Stability. Not analyzed.

9.8.5. Model Calibration and Analysis Results.

9.8.5.1. Model Calibration for Seepage Analyses. Once the appropriate features were selected, constant head boundary conditions could be applied. A lake level of 18.5 was selected since data from the 1995 High Water Event was available. No visible evidence of seepage was observed. However, any excessive seepage or piping occurring under the surface of the borrow lake would likely go undetected until a partial collapse of the berm occurred.

9.8.5.2. Model Calibration for Slope Stability Analyses. This analysis was not performed.

9.8.6. Length Effects. For this model a link length of 20,000 feet was selected.

9.8.7. Pup Curve Development. Table H-9.12 is a summary of the analyses performed on this model. Relevant information for each analysis was listed to show the progression of values with lake level.

9.8.7.1. Through Seepage. This failure mode was not a factor . The resulting probabilities are shown in Table H-9.12.

9.8.7.2. Under Seepage. For lake elevations of 18 and 21, underseepage was the predominant failure mode. The upper pipe and lower pipe in the model were examined as locations most critical to piping. The upper pipe was found to be more critical and vertical piping the predominant mode. It is important to note that even though the direction of piping is vertical, partical removal will occur in the horizontal direction. The flow velocity to remove sand particles once they are in suspension is approximately one to two orders of magnatude less than that required to cause erosion. A typical medium sand particle is 1 mm in diameter, and a typical fine sand particle is 0.l1 mm in diameter. This was documented by Hjulstrom. Refer to Figure H-9.2. In addition to this, any vertical gradient would reduce the flow needed even further.

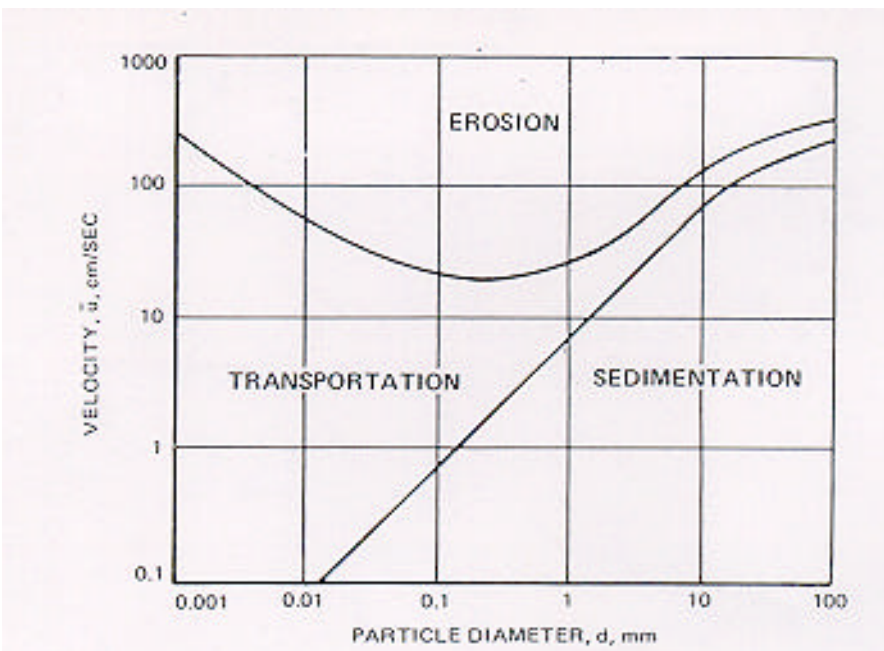


Figure H-9.2. Hjulstrom's curve relating average velocity to grain size for erosion, transportation and sedimentation. [After Hjulstrom, 1935]

9.8.7.1. Slope Stability. This analysis was not performed.

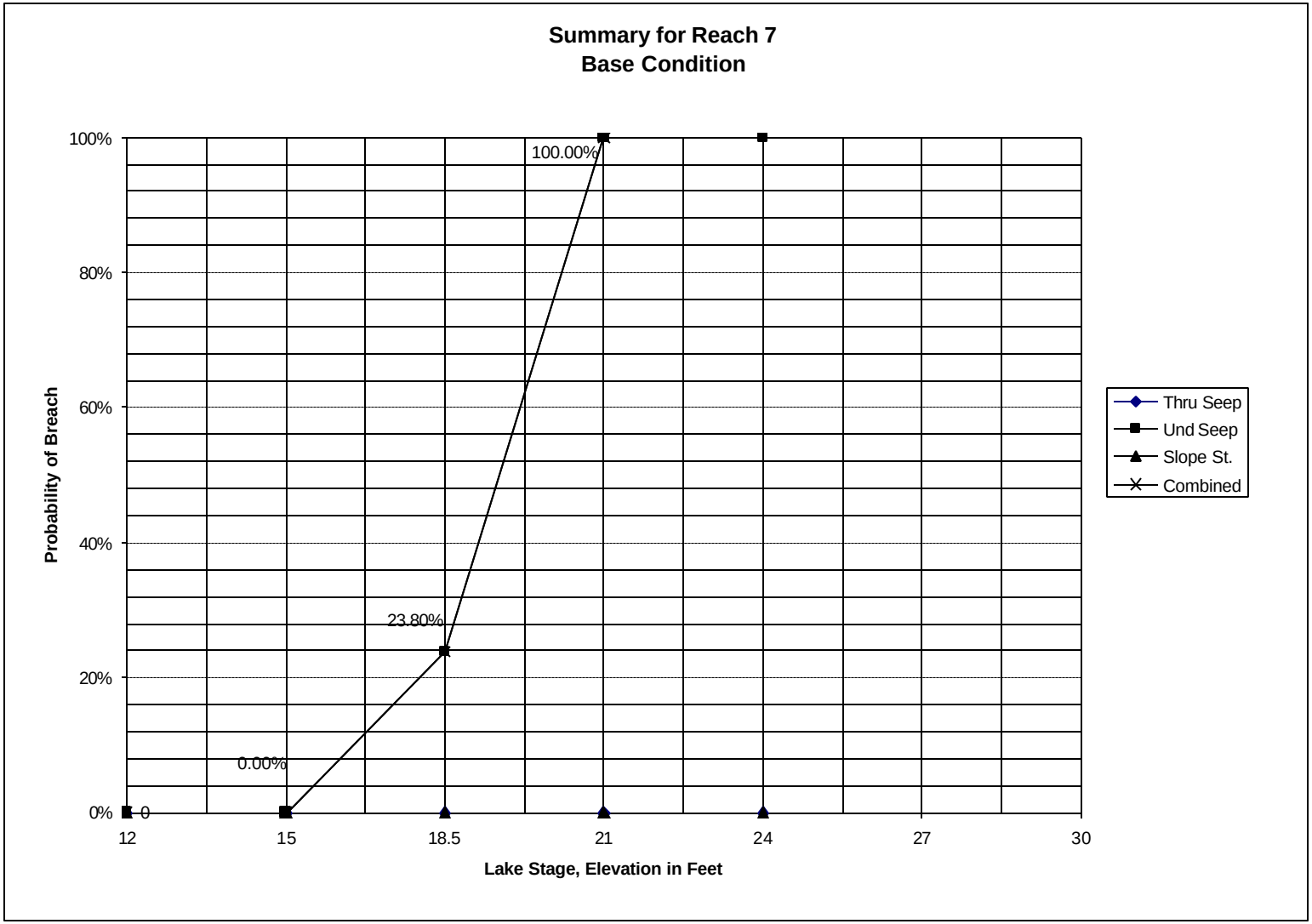
Table H-9.12

Lake El.	Failure Mode	E(FS)	Beta	Sum Variance	P(Failure)	P(Failure) w/ length
18.5	UP iv	1.960	2.296	0.3051	0.0108	0.0496
18.5	UP ih	3.545	5.310	0.702	5.5 e-8	2.5e-7
21.0	UP iv	0.827	-0.987	0.0328	0.838	1.00

Definitions:
* = Value used in combined probability computation.
LP = Lower Pipe UP = Upper Pipe
iv = vertical exit gradient ih = horizontal exit gradient

9.8.8. Probabilistic Analyses computation Sheets and Sections

R7TASUM.XLS, Reach 7 Summary Chart 2



LEGEND FOR COLOR CROSS SECTIONS

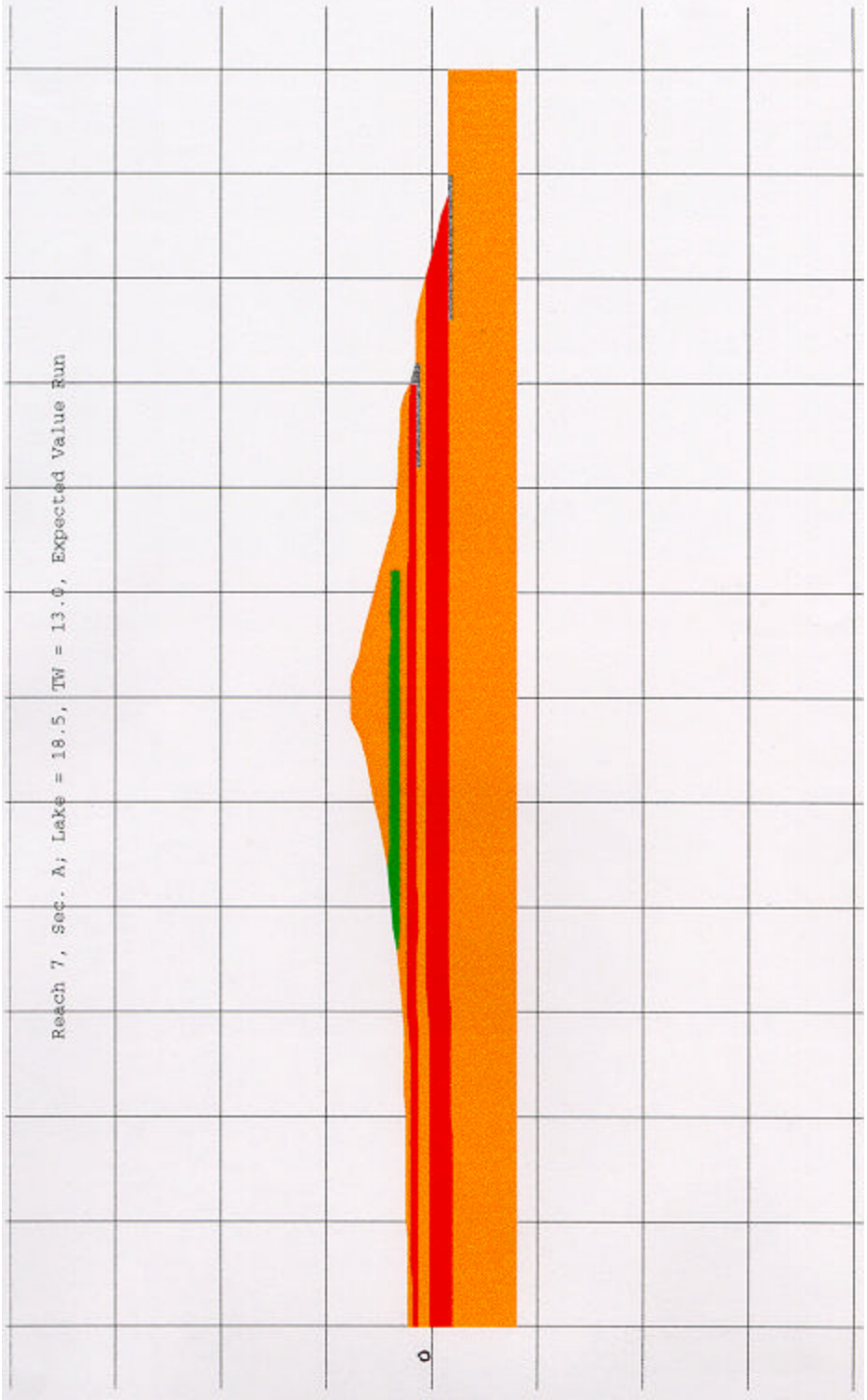
MATERIAL	COLOR
SAND	YELLOW
GRAVEL	GREEN
FILTERCAKE	PURPLE
PEAT	BROWN
SILT / CLAY	BLUE
LIMESTONE	RED
FEATURE (Entrance / exit valve, Pipe)	GRAY

SCALE: EACH SQUARE ON THE GRID IS 50 FEET BY 50 FEET

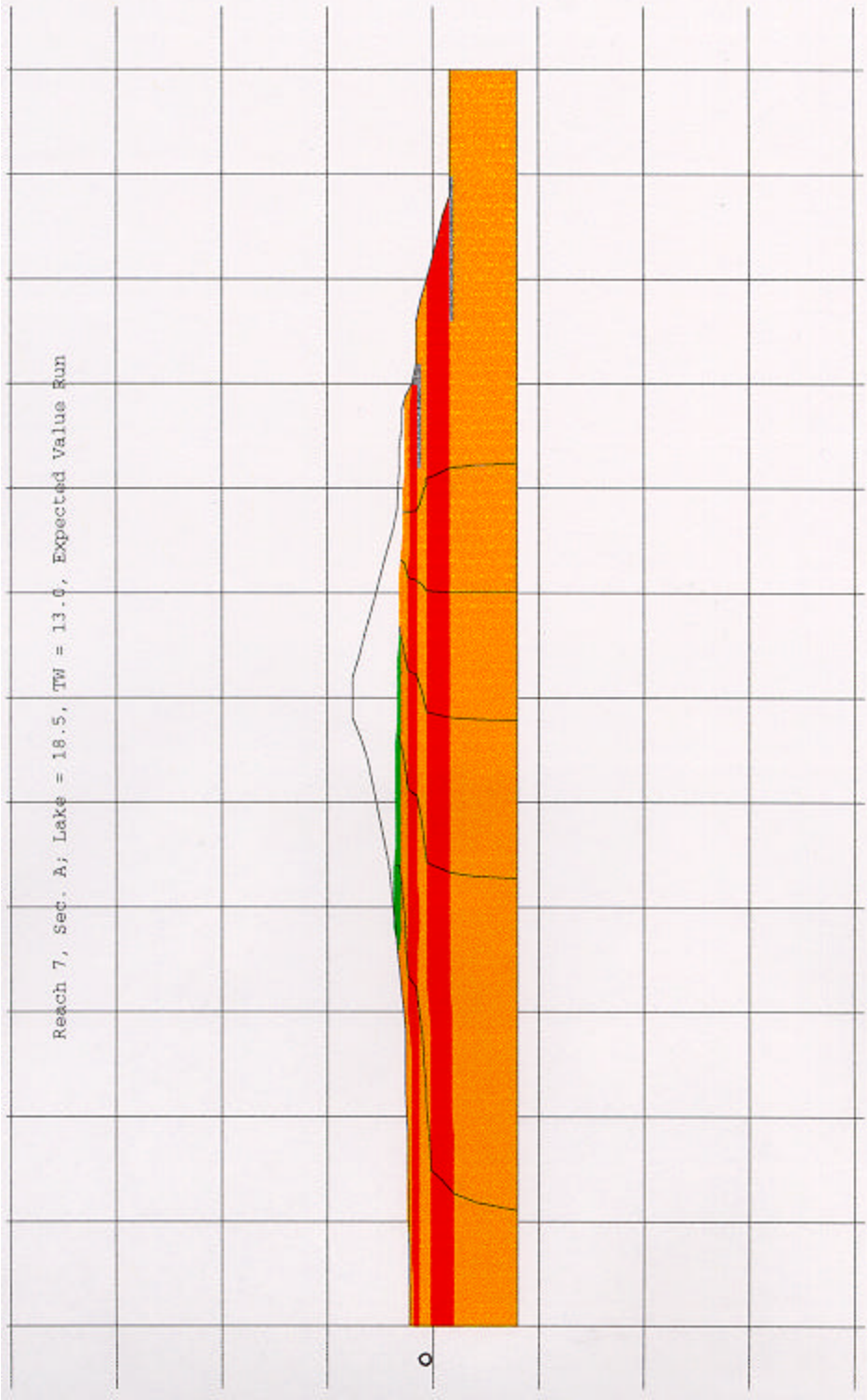
EACH CONTOUR LINE IS ONE FOOT

ELEVATION ZERO IS INDICATED ON EACH SECTION

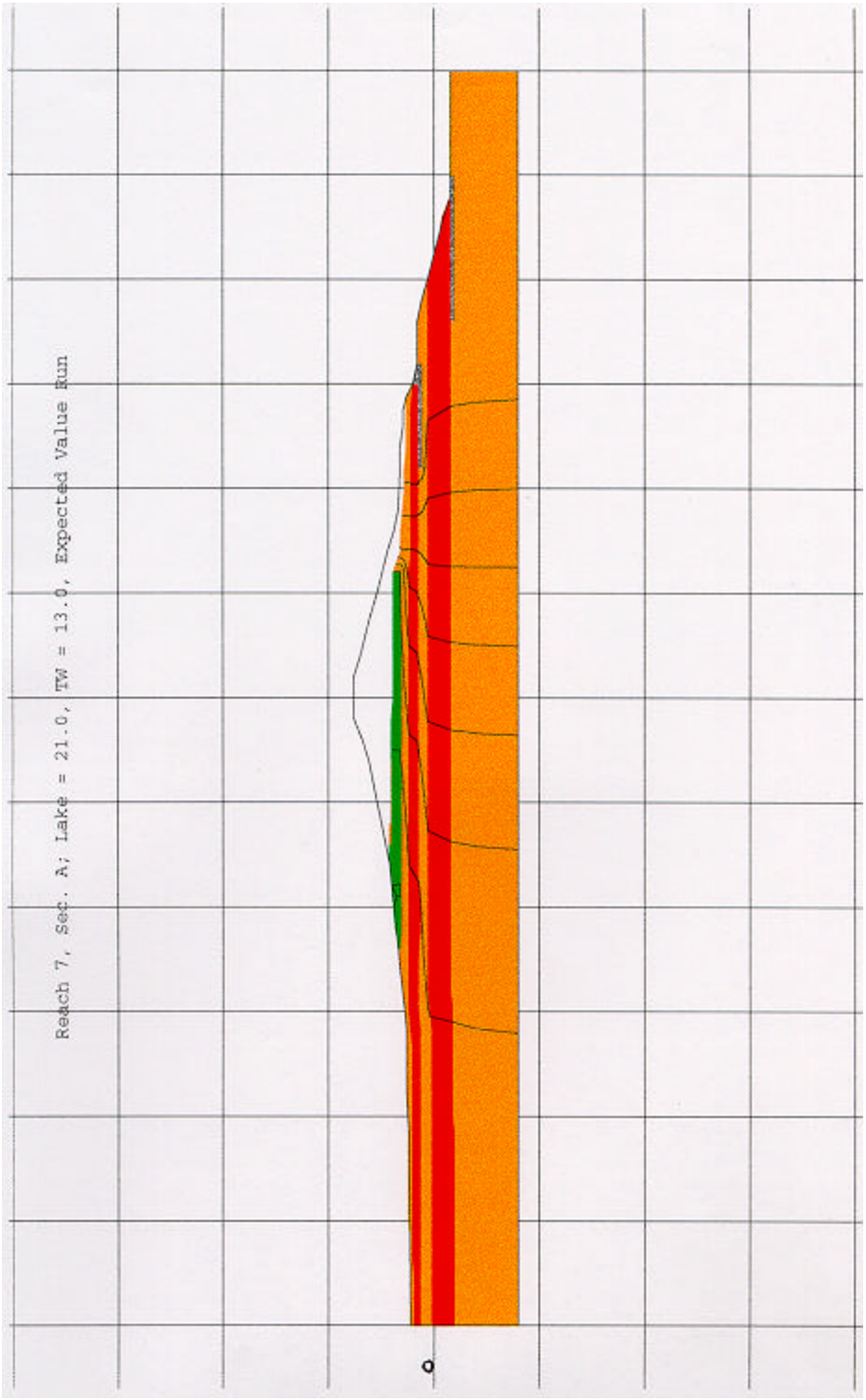
Reach 7, Sec. A; Lake = 18.5, TW = 13.0, Expected Value Run



Reach 7, Sec. A; Lake = 18.5, TW = 13.0, Expected Value Run



Reach 7, Sec. A; Lake = 21.0, TW = 13.0, Expected Value Run



R7-seep.xls, Summary

Exit Gradient Computations					**Reach 7; Lake = 18.5; Mean TW = 13.0**	
				Without Clay Layer		
	At pipe entrance.					**Upper Pipe**
Run	Y	Head +	Head -	X +	X -	**ih**
E	**6.00**	**13.88**	**13.15**	**110.00**	**100.00**	**0.073**
1	6.00	13.84	13.18	110.00	100.00	0.066
2	6.00	13.84	13.12	110.00	100.00	0.072
3	6.00	14.03	13.18	110.00	100.00	0.085
4	6.00	13.77	13.13	110.00	100.00	0.064
7	6.00	13.81	13.12	110.00	100.00	0.069
8	6.00	13.94	13.10	110.00	100.00	0.084
9	6.00	13.83	13.01	110.00	100.00	0.082
10	6.00	13.97	13.30	110.00	100.00	0.067
11	6.00	14.14	13.45	110.00	100.00	0.069
12	6.00	13.62	12.86	110.00	100.00	0.076
13	6.00	13.88	13.15	110.00	100.00	0.073
14	6.00	13.88	13.15	110.00	100.00	0.073
15	6.00	13.88	13.15	110.00	100.00	0.07
16	6.00	13.88	13.15	110.00	100.00	0.07

				Without Clay Layer		
	At pipe entrance.					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**110.00**	**13.90**	**13.15**	**6.00**	**3.00**	**0.250**
1	110.00	13.76	13.18	6.00	3.00	0.193
2	110.00	13.93	13.12	6.00	3.00	0.270
3	110.00	14.03	13.18	6.00	3.00	0.283
4	110.00	13.81	13.13	6.00	3.00	0.227
7	110.00	13.89	13.21	6.00	3.00	0.227
8	110.00	13.84	13.10	6.00	3.00	0.247
9	110.00	13.87	13.07	6.00	3.00	0.267
10	110.00	13.97	13.30	6.00	3.00	0.223
11	110.00	14.16	13.45	6.00	3.00	0.237
12	110.00	13.65	12.86	6.00	3.00	0.263
13	110.00	13.90	13.15	6.00	3.00	0.250
14	110.00	13.90	13.15	6.00	3.00	0.250
15	110.00	13.90	13.15	6.00	3.00	0.25
16	110.00	13.90	13.15	6.00	3.00	0.25

Summary

Exit Gradient Computations					Reach 7; Lake = 21.0;	
					Mean TW = 13.0	
				Without Clay Layer		
	At pipe entrance.					**Upper Pipe**
Run	X	Head +	Head -	Y+	Y-	**iv**
E	**110.00**	**14.54**	**13.27**	**6.00**	**3.00**	**0.423**
1	110.00	14.39	13.34	6.00	3.00	0.350
2	110.00	14.49	13.21	6.00	3.00	0.427
3	110.00	14.64	13.30	6.00	3.00	0.447
4	110.00	14.40	13.24	6.00	3.00	0.387
9	110.00	14.48	13.13	6.00	3.00	0.450
10	110.00	14.65	13.54	6.00	3.00	0.370
15	110.00	14.54	13.27	6.00	3.00	0.423
16	110.00	14.54	13.27	6.00	3.00	0.423

R7-seep.xls, Upper Pipe,ih 18,nc

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 7**

Deterministic Model:	FASTSEEP	**Lake Level:**	**18.5**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	**13.0**
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Without Clay Layer

Run	1 Sands	2 Gravel	5 Rock Horiz.	5 Rock Vert.	6 Pipe	Ditch Tailwater	Gravel Pocket	i crit	Upper Pipe, ih Results i exit	FS	Prob Calcs Var	%
E	**1.41E-03**	**7.07E-01**	**1.77E-01**	**1.41E-02**	**3.54E+00**	**13.0**	**180**	**0.259**	**0.073**	**3.545**		
1	**3.25E-03**	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.259	**0.066**	3.920		
2	**6.15E-04**	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.259	**0.072**	3.594	0.0267	3.80%
3	1.41E-03	**1.63E+00**	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.259	**0.085**	3.044		
4	1.41E-03	**3.08E-01**	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.259	**0.064**	4.043	0.2494	35.53%
7	1.41E-03	7.07E-01	**4.06E-01**	**3.25E-02**	3.54E+00	13.0	180	0.259	**0.069**	3.750		
8	1.41E-03	7.07E-01	**7.69E-02**	**6.15E-03**	3.54E+00	13.0	180	0.259	**0.084**	3.080	0.1121	15.97%
9	1.41E-03	7.07E-01	1.77E-01	1.41E-02	**8.13E+00**	13.0	180	0.259	**0.082**	3.144		
10	1.41E-03	7.07E-01	1.77E-01	1.41E-02	**1.54E+00**	13.0	180	0.259	**0.067**	3.862	0.1289	18.36%
11	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	**13.3**	180	0.259	**0.069**	3.750		
12	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	**12.7**	180	0.259	**0.076**	3.405	0.0298	4.25%
13	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	200	0.259	**0.073**	3.545		
14	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	160	0.259	**0.073**	3.545	0.0000	0.00%
15	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.288	**0.073**	3.938		
16	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.230	**0.073**	3.151	0.1551	22.09%
										Sum:	0.7020	100.00%

Critical Gradient:		*For Piping*	*For Breach*				
	Avg	0.225	0.259	**E[FS] =**	**3.545**	E[ln(FS)] =	1.2382
	High	0.250	0.288	Var[FS] =	0.7020	Var[ln(FS)] =	0.0544
	Low	0.200	0.230	$_s$ **[FS] =**	**0.8379**	$_d$ln(FS)] =	0.2332
				V [FS] =	0.2364	V [ln(FS)] =	0.1883

$_b$ =	**5.310**	Pr(f) =	5.49E-08
1 in	**18227602**		

Adjusted probabilities based on length effect:

Total Length =	93,500	Reliability of Section, R =	1.00E+00
"Link" Length =	20000	**Adjusted Pr(f) =**	**2.56E-07**
No. Links =	4.68	**1 in**	**3898953**

R7-seep.xls, Upper Pipe, iv18,nc

Probabilistic Underseepage Analysis: **Herbert Hoover Dike; Model for Reach 7**

Deterministic Model:	FASTSEEP	**Lake Level:**	**18.5**
Probabilistic Model:	Taylor's Series, Finite Difference		
Performance Function:	FS	**Tailwater:**	**13.0**
Random Variable			
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal		

Without Clay Layer

Run	1 Sands	2 Gravel	5 Rock Horiz.	5 Rock Vert.	6 Pipe	Ditch Tailwater	Gravel Pocket	i crit	Upper Pipe, iv Results i exit	FS	Prob Calcs Var	%
E	**1.41E-03**	**7.07E-01**	**1.77E-01**	**1.41E-02**	**3.54E+00**	**13.0**	**180**	**0.403**	**0.250**	**1.610**		
1	**3.25E-03**	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.403	**0.193**	2.082		
2	**6.15E-04**	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.403	**0.270**	1.491	0.0874	42.44%
3	1.41E-03	**1.63E+00**	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.403	**0.283**	1.421		
4	1.41E-03	**3.08E-01**	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.403	**0.227**	1.776	0.0315	15.32%
7	1.41E-03	7.07E-01	**4.06E-01**	**3.25E-02**	3.54E+00	13.0	180	0.403	**0.227**	1.776		
8	1.41E-03	7.07E-01	**7.69E-02**	**6.15E-03**	3.54E+00	13.0	180	0.403	**0.247**	1.632	0.0052	2.52%
9	1.41E-03	7.07E-01	1.77E-01	1.41E-02	**8.13E+00**	13.0	180	0.403	**0.267**	1.509		
10	1.41E-03	7.07E-01	1.77E-01	1.41E-02	**1.54E+00**	13.0	180	0.403	**0.223**	1.802	0.0214	10.42%
11	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	**13.3**	180	0.403	**0.237**	1.701		
12	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	**12.7**	180	0.403	**0.263**	1.528	0.0074	3.60%
13	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	200	0.403	**0.250**	1.610		
14	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	160	0.403	**0.250**	1.610	0.0000	0.00%
15	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.460	**0.250**	1.840		
16	1.41E-03	7.07E-01	1.77E-01	1.41E-02	3.54E+00	13.0	180	0.345	**0.250**	1.380	0.0529	25.70%
										Sum:	0.2058	100.00%

		For Piping	*For Breach*				
Critical Gradient:	Avg	0.350	0.403	**E[FS] =**	**1.610**	E[ln(FS)] =	0.4380
	High	0.400	0.460	Var[FS] =	0.2058	Var[ln(FS)] =	0.0764
	Low	0.300	0.345	$_s$ **[FS] =**	**0.4537**	$_d$[ln(FS)] =	0.2764
				V [FS] =	0.2818	V [ln(FS)] =	0.6311

Adjusted probabilities based on length effect:	**Total Length =**	93,500	Reliability of Section, R = 7.62E-01
	"Link" Length =	20000	**Adjusted Pr(f) = 2.38E-01**
	No. Links =	4.68	**1 in 4**

$_b$ **= 1.585**

Pr(f) = 5.65E-02

1 in 18

R7-seep.xls, Upper Pipe,iv21,nc

Probabilistic Underseepage Analysis:

Deterministic Model:	FASTSEEP
Probabilistic Model:	Taylor's Series, Finite Difference
Performance Function:	FS
Random Variable	
Distribution Assumptions:	K's lognormal, i crit and ditch water level normal

Herbert Hoover Dike; Model for Reach 7

Lake Level: **21.0**

Tailwater: **13.0**

Without Clay Layer

Upper Pipe, iv

Run	1 Sands	2 Gravel	6 Pipe	i crit	Results: i exit	Results: FS	Prob Calcs: Var	Prob Calcs: %
E	**1.41E-03**	**7.07E-01**	**3.54E+00**	**0.403**	**0.423**	**0.951**		
1	**3.25E-03**	7.07E-01	3.54E+00	0.403	**0.350**	1.150		
2	**6.15E-04**	7.07E-01	3.54E+00	0.403	**0.427**	0.943	0.0107	24.62%
3	1.41E-03	**1.63E+00**	3.54E+00	0.403	**0.447**	0.901		
4	1.41E-03	**3.08E-01**	3.54E+00	0.403	**0.387**	1.041	0.0049	11.27%
9	1.41E-03	7.07E-01	**8.13E+00**	0.403	**0.450**	0.894		
10	1.41E-03	7.07E-01	**1.54E+00**	0.403	**0.370**	1.088	0.0094	21.56%
15	1.41E-03	7.07E-01	3.54E+00	0.460	**0.423**	1.087		
16	1.41E-03	7.07E-01	3.54E+00	0.345	**0.423**	0.815	0.0184	42.55%
						Sum:	0.0434	100.00%

		For Piping	*For Breach*
Critical Gradient:	Avg	0.350	0.403
	High	0.400	0.460
	Low	0.300	0.345

E[FS] =	**0.951**	E[ln(FS)] =	-0.0739
Var[FS] =	0.0434	Var[ln(FS)] =	0.0469
$_S$**[FS] =**	**0.2082**	$_\sigma$[ln(FS)] =	0.2165
V [FS] =	0.2190	V [ln(FS)] =	-2.9294

$_b$ =	**-0.341**
Pr(f) =	6.34E-01
1 in	**2**

Adjusted probabilities based on length effect:

Total Length =	93,500	Reliability of Section, R =	9.15E-03
"Link" Length =	20000	**Adjusted Pr(f) =**	**9.91E-01**
No. Links =	4.68	**1 in**	**1**

9.9. Model Reach 8.

9.9.1 Location: Reach 8 is located on the West Side of Lake Okeechobee in Glades county. This reach extends from Indian Prairie canal to Kissimmee River then north along the south bank of the Kissimmee River to structure S-154, approximately 15.7 miles. Reach 8 covers the portion of the levee system designated as L-48. The embankment within this reach was constructed in 1962 by dragline excavating from deep borrow canal to elevation –10 feet that parallels land side toe. The natural ground within this area is approximately 15 feet and slope up to the west. The crest elevation varies from 32 feet to elevation 39 feet. The interior slopes are 1V on 3H and the exterior slopes are 1V on 6H.

9.9.2 Subsurface Investigation: Subsurface investigation within this reach consisted primarily of evaluating existing core boring drilled prior to the construction of the embankment in 1960 and core borings through the embankment crest obtained in 1986.

9.9.3 Materials Encountered: Based on limited core boring data the embankment appears to consist of silty sand and silt. The foundation materials consist of silty sand with interbedded layers of shell, silt and clay.

9.9.4 Investigation: Currently we do not have sufficient subsurface information to construct soil section that will adequately describe all subsurface conditions within this reach. This is due to widely space core borings along the embankment centerline and lack of borings on the lake side and land side of the embankment. What land side borings available were drilled for the deep borrow canal along the exterior toe.
A typical soil profile evaluated for this reach was based on existing survey data obtained from as-build drawings and limited subsurface data.

9.9.5 Slope Stability Analysis: No stability analyses were performed for this reach.

9.9.6 Seepage Analysis: No seepage analysis was performed on this section.

9.9.7 Probabilistic Analysis: Conditional probability-of-failure versus lake elevation functions presented in this report, for this reach, was based on geometry of the embankment, embankment and foundation materials, observation from the field during periods of high water, past experience with similar earth structures and engineering judgment.

10. EVALUATION OF RESULTS.

An overall evaluation of the project within the context of this study is shown in Figure H-10.1. What this indicates is that based on the explorations and analysis in Reach 1, and the limited explorations and analysis in Reaches 2, 3 and 7, performance problems (including breach) can be expected in these areas above lake elevations of 17.0. Limited explorations and analysis in Reaches 4, 5, 6 and 8 have not revealed problems up to this point. This does not mean a more detailed evaluation would have the same result. Evaluations outside of Reach 1 have been severely limited by lack of a project topographic survey and limited subsurface data. Note that all values other than those for lake elevations 15, 18, 21 and 24 have been interpolated.

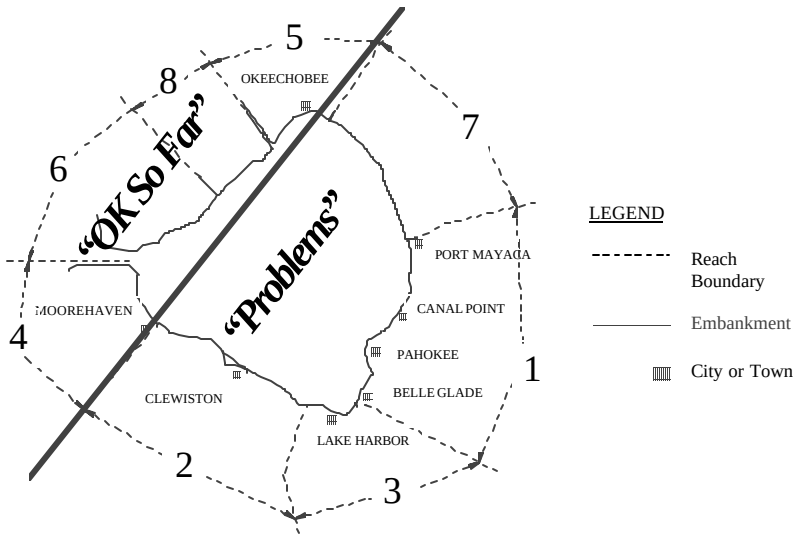


Figure H-10.1. Overall Evaluation.

10.1. Evaluation of Pup Curves. The results of engineering analyses must be viewed with judgement in context with the system being evaluated. In other words, the results must be reasonable. This evaluation consisted of three components; analysis, past performance, and engineering judgement.

10.1.1. Analyses. Results of the analyses for each model are shown in Table H-10.1. These values represent the combined probability of breach for each model (through seepage, under seepage, and slope stability). Also shown are the combined probability of breach for the project at each lake level.

10.1.2. Past Performance. Two areas were evaluated for past performance. These were the performance of the project itself and performance of nearby structures of similar construction and function. Then, an adjustment of results as proposed by Duncan (Duncan et. al., 1983) was used to calibrate the results based on past performance.

10.1.2.1. Project loading. This project was completed in 1935. The time period of evaluation is roughly 60 years. An examination of the lake levels during this period reveals that the embankment has not been tested (this does not include data from the 1998 high water event). In other words, it is a dam that has never had an initial filling. Figure H-10.2 shows the approximate number of months at each lake elevation (10 station average). Significant events are as follows:

YEAR	PEAK ELEVATION
1947	18.9
1995	18.6
1983	18.3

(Note: The 1998 High Water Event produced a maximum water elevation of 18.3.)

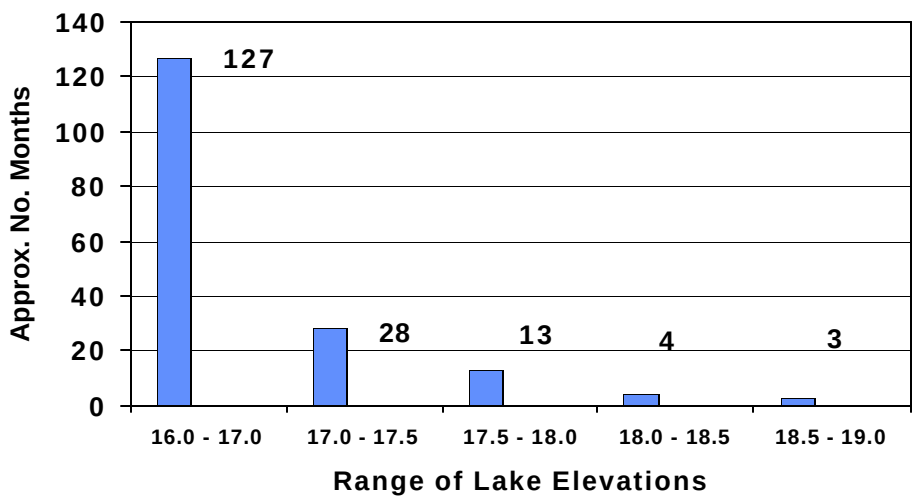


Figure H-10.2. Lake Elevation Histogram.

10.1.2.2. Project performance. In May 1974, the LD-4 embankment next to culvert structure S-154 (Reach 5) breached due to piping through a shelly sand layer in the foundation. The breach damaged the structure, which required extensive rehabilitation. In October, 1995 the LD-2 and LD-9 embankments experienced significant distress (Reaches 1 and 3). Actions by CESAJ staff (Sandbagging boils, raising tailwater levels in the ditches) prevented probable

breaches in two different reaches. Both of these events (1974 and 1995) occurred within the 60 year span. Therefore, a statistical value of failure of 2/60 or 1 every 30 years could be used to calibrate overall performance.

10.1.2.3. Nearby Structures. The well-documented breach by piping of the FP&L Reservoir in Martin County in October, 1979 (two miles from the embankment in Reach 7) is a relevant event. Another nearby event were several partial breaches by piping of the L-8 embankment along the West Palm Beach canal from 2 to 10 miles from the reach 1 embankment. Although these were not part of the project, we would consider it negligent to not consider the implications of the processes which occurred on these nearby, similar structures.

10.1.3. Judgment. Although judgement cannot be entirely quantified, it was an important part of every part of the analysis and evaluation.

10.2. Presentation of Final Pup Curves for Base Conditions. The adjusted curves are shown in Tables H-10.1 and H-10.2. The major adjustment was for the lake elevation of 18. Based on observations a change in performance occurs above elevation 17. This coincides with the observed signs of distress (piping and seepage). The values at elevation 21 were not adjusted for two main reasons. First, there has been no observed performance at a lake elevation of 21 (never had this event). Second, the analysis results by themselves clearly indicated two extremes in performance from elevation 15 to elevation 21, where performance at elevation 15 was clearly acceptable and at elevation 21 was clearly unacceptable. Somewhere between elevation 15 and 21 project performance degrades significantly. Observations have shown this degradation begins at elevation 17.0. Figures H-10.3 and H-10.4 show a spatial distribution of the combined probabilities of breach for the project based on the scope of this study (limited information outside of Reach 1).

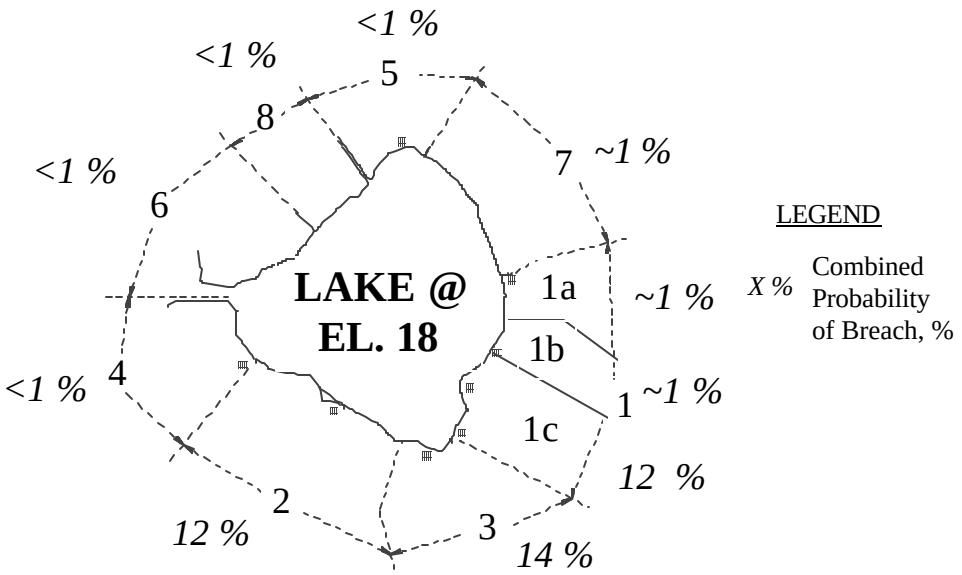


Figure H-10.3. Combined Probabilities of Breach at Lake elevation 18.

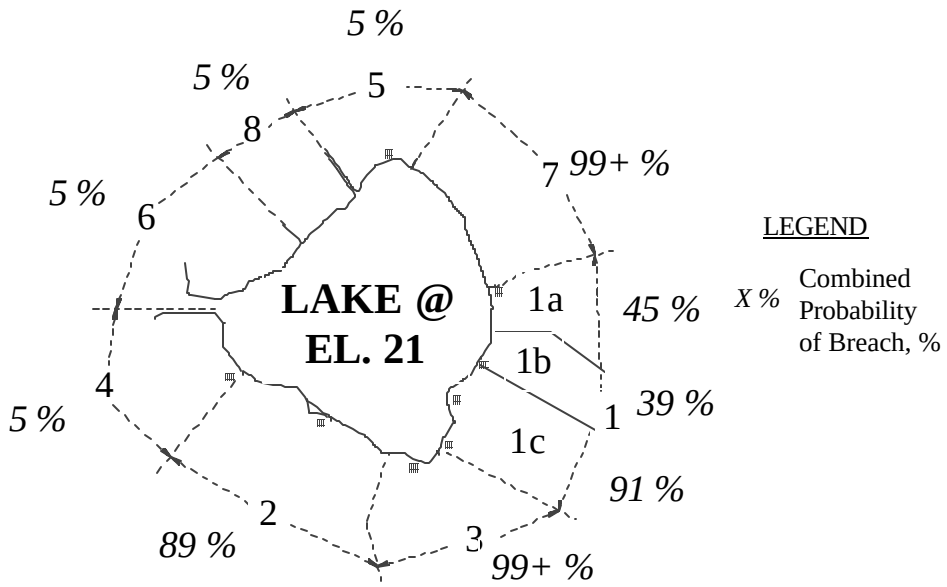


Figure H-10.4. Combined Probabilities of Breach at Lake elevation 21.

10.3. Final Pup Curves for Base Conditions in Risk Analysis Model. Preliminary runs of the Pup functions revealed that there were breaches registered at lake levels below elevation 17. Based on judgement, we recommended the values for the Pup functions for elevations 17 and below be set to zero. The values in the economic model are shown in Table H-10.4.

Table H-10.1 Bcsum-1.xls, Base Conditions

	Base Conditions at Herbert Hoover Dike							
Reach and	Probability of Breach at selected Lake Elevations.							
Section	14	15	16	17	18	21	24	27
Reach 1A, Line 25	0.0030%	0.015%	0.030%	0.120%	3.18%	45.0%	100.00%	100.00%
Reach 1B, Line 25	0.0030%	0.015%	0.030%	0.120%	2.69%	39.0%	100.00%	100.00%
Reach 1C, Line 18	0.0580%	0.290%	0.580%	2.320%	58.00%	91.0%	100.00%	100.00%
Reach 1C, Line 9	0.0000%	0.000%	0.000%	0.000%	0.01%	49.0%	100.00%	100.00%
Reach 1C, Line 6	0.0310%	0.155%	0.310%	1.240%	31.00%	68.0%	80.00%	100.00%
Reach 2	0.1000%	0.500%	1.000%	4.000%	34.20%	99.0%	100.00%	100.00%
Reach 3	0.0770%	0.385%	0.770%	3.080%	77.00%	91.0%	100.00%	100.00%
Reach 4	0.0010%	0.005%	0.010%	0.040%	1.00%	5.0%	10.00%	15.00%
Reach 5	0.0010%	0.005%	0.010%	0.040%	1.00%	5.0%	10.00%	15.00%
Reach 6 A	0.0004%	0.002%	0.004%	0.002%	0.66%	2.0%	4.00%	6.00%
Reach 6 B	0.0006%	0.003%	0.006%	0.003%	1.00%	3.1%	6.40%	9.60%
Reach 7	0.0050%	0.025%	0.050%	0.200%	23.80%	99.0%	100.00%	100.00%
Reach 8	0.0010%	0.005%	0.010%	0.040%	1.00%	5.0%	10.00%	15.00%
Combined Probability	0.2807%	1.398%	2.781%	10.749%	97.00%	100.0%	100.0%	100.0%

Table H-10.2 Bc-final.xls, Base Conditions – Adjusted

	Base Conditions at Herbert Hoover Dike							
Reach and	Probability of Breach at selected Lake Elevations (* = Adjusted).							
Section	14	15	16	17	18 *	21	24	27
Reach 1A, Line 25	0.0030%	0.015%	0.030%	0.120%	0.75%	45.0%	100.0%	100.0%
Reach 1B, Line 25	0.0030%	0.015%	0.030%	0.120%	0.75%	39.0%	100.0%	100.0%
Reach 1C, Line 18	0.0580%	0.290%	0.580%	2.320%	12.00%	91.0%	100.0%	100.0%
Reach 1C, Line 9	0.0000%	0.000%	0.000%	0.000%	3.00%	49.0%	100.0%	100.0%
Reach 1C, Line 6	0.0310%	0.155%	0.310%	1.240%	7.88%	68.0%	80.0%	92.0%
Reach 2	0.1000%	0.500%	1.000%	4.000%	12.00%	89.0%	99.0%	100.0%
Reach 3	0.0770%	0.385%	0.770%	3.080%	14.00%	100.0%	100.0%	100.0%
Reach 4	0.0010%	0.005%	0.010%	0.040%	0.25%	5.0%	10.0%	15.0%
Reach 5	0.0010%	0.005%	0.010%	0.040%	0.25%	5.0%	10.0%	15.0%
Reach 6 A	0.0004%	0.002%	0.004%	0.002%	0.13%	2.0%	4.0%	6.0%
Reach 6 B	0.0006%	0.003%	0.006%	0.003%	0.20%	3.1%	6.4%	9.6%
Reach 7	0.0050%	0.025%	0.050%	0.200%	5.96%	99.0%	100.0%	100.0%
Reach 8	0.0010%	0.005%	0.010%	0.040%	0.25%	5.0%	10.0%	15.0%
Combined Probability	0.2807%	1.398%	2.781%	10.749%	45.46%	100.0%	100.0%	100.0%

Table H-10.3 Bc-final.xls, Detail Summary

	Base Conditions at Herbert Hoover Dike				
Reach and		Probability of Breach at selected Lake Elevations.			
Section		15	18	21	24
Reach 1A, Line 25		0.015%	0.75%	45.00%	100.00%
Und. Seepage		0.000%	0.00%	38.01%	100.00%
Thru Seepage		0.003%	0.50%	8.87%	19.16%
Slope Stab.		0.001%	0.25%	2.45%	3.73%
Reach 1B, Line 25		0.015%	0.75%	39.00%	100.00%
Und. Seepage		0.000%	0.00%	33.18%	99.98%
Thru Seepage		0.003%	0.50%	7.54%	16.42%
Slope Stab.		0.001%	0.25%	2.07%	3.15%
Reach 1C, Line 18		0.289%	12.00%	91.00%	100.00%
Und. Seepage		0.000%	0.00%	35.40%	100.00%
Thru Seepage		0.010%	8.17%	63.70%	75.00%
Slope Stab.		0.000%	4.09%	60.00%	87.30%
Reach 1C, Line 9		0.000%	3.00%	49.00%	100.00%
Und. Seepage		0.000%	0.01%	42.70%	100.00%
Thru Seepage		0.020%	2.94%	10.62%	25.51%
Slope Stab.		0.000%	0.00%	0.00%	12.97%
Reach 1C, Line 6		0.155%	7.88%	68.00%	80.00%
Und. Seepage		1.348%	7.85%	43.30%	61.30%
Thru Seepage		0.000%	0.00%	43.00%	47.40%
Slope Stab.		0.000%	0.03%	0.21%	0.34%
Reach 2		0.500%	12.00%	89.00%	99.00%
Und. Seepage		0.000%	3.83%	52.50%	88.10%
Thru Seepage		0.000%	8.42%	76.80%	88.90%
Slope Stab.		0.000%	0.03%	0.21%	0.34%
Reach 3		0.385%	14.00%	100.00%	100.00%
Und. Seepage		0.000%	6.60%	100.00%	100.00%
Thru Seepage		0.000%	7.49%	89.70%	100.00%
Slope Stab.		0.000%	0.00%	0.00%	0.00%
Reach 4		0.005%	0.25%	5.00%	10.00%
Reach 5		0.005%	0.25%	5.00%	10.00%
Reach 6 A		0.002%	0.13%	1.95%	4.00%
Reach 6 B		0.003%	0.20%	3.12%	6.40%
Reach 7		0.025%	5.96%	100.00%	100.00%
Und. Seepage		0.000%	5.96%	100.00%	100.00%
Thru Seepage		0.000%	0.00%	0.00%	0.00%
Slope Stab.		0.000%	0.00%	0.00%	0.00%
Reach 8		0.005%	0.25%	5.00%	10.00%
Combined Probability		1.397%	45.46%	100.00%	100.00%

Table H-10.4 Bc-final.xls, Base Conditions in Model

	Base Conditions at Herbert Hoover Dike							
Reach and	Probability of Breach at selected Lake Elevations (* = Adjusted).							
Section	14 *	15 *	16 *	17 *	18 *	21	24	27
Reach 1A, Line 25	0.0000%	0.0000%	0.0000%	0.0000%	0.75%	45.0%	100.0%	100.0%
Reach 1B, Line 25	0.0000%	0.0000%	0.0000%	0.0000%	0.75%	39.0%	100.0%	100.0%
Reach 1C, Line 18	0.0000%	0.0000%	0.0000%	0.0000%	12.00%	91.0%	100.0%	100.0%
Reach 1C, Line 9	0.0000%	0.0000%	0.0000%	0.0000%	3.00%	49.0%	100.0%	100.0%
Reach 1C, Line 6	0.0000%	0.0000%	0.0000%	0.0000%	7.88%	68.0%	80.0%	92.0%
Reach 2	0.0000%	0.0000%	0.0000%	0.0000%	12.00%	89.0%	99.0%	100.0%
Reach 3	0.0000%	0.0000%	0.0000%	0.0000%	14.00%	100.0%	100.0%	100.0%
Reach 4	0.0000%	0.0000%	0.0000%	0.0000%	0.25%	5.0%	10.0%	15.0%
Reach 5	0.0000%	0.0000%	0.0000%	0.0000%	0.25%	5.0%	10.0%	15.0%
Reach 6 A	0.0000%	0.0000%	0.0000%	0.0000%	0.13%	2.0%	4.0%	6.0%
Reach 6 B	0.0000%	0.0000%	0.0000%	0.0000%	0.20%	3.1%	6.4%	9.6%
Reach 7	0.0000%	0.0000%	0.0000%	0.0000%	5.96%	99.0%	100.0%	100.0%
Reach 8	0.0000%	0.0000%	0.0000%	0.0000%	0.25%	5.0%	10.0%	15.0%
Combined Probability	0.0000%	0.000%	0.000%	0.000%	45.46%	100.0%	100.0%	100.0%

11. STORM SURGE IMPACTS.

Integrating the geotechnical probability curves to this portion of the event tree proved to be difficult. Instead of developing separate geotechnical Pup functions for storm surge a "pseudo-static" lake level was applied. Therefore, the static lake level plus adjusted surge level would be applied to the Pup functions developed for static lake levels.

11.1. Storm Surge Characteristics. A typical recorded storm surge is shown in Figure H-11.1. Two important aspects are the actual time the lake is above its initial static level the peak surge value. Surge duration has customarily been defined as the time the lake departs from and returns to it's static elevation. This time period also includes the time the lake "sloshes" and the lake level at the point of interest is actually lower than the static elevation.

11.1.1 Surge Duration. Based on historical information (hurricanes in 1949, 1950, and 1979), the duration of the total surge event is about 24 hours. Refer to Tables H-11.1 and H-11.2. Data based on this very limited analysis suggests the following:

1. If you consider the surge duration which is 50% of the total surge duration, the corresponding surge height is about 10% of the peak surge height.
2. The 8-hour long duration is characterized by a surge height that is about 25% of the peak surge height.
3. The 6-hour long duration is characterized by a surge height that is about 35% of the peak surge height.
4. The 4-hour long duration is characterized by a surge height that is about 50% of the peak surge height.
5. The 2-hour long duration is characterized by a surge height that is about 70% of the peak surge height.

11.1.2. Surge Hydrograph. A storm surge elevation is time dependent. For example, a 10-foot storm surge would only maintain peak of 10 feet for a short period of time; but, the 1-foot portion of the surge would continue for hours. Below is a table listing the probable duration of a "10 foot" storm surge for various elevated heads. Refer to Figure H-11.1. The height of the surge is multiplied by the duration of the surge at that elevation. The table suggests that using the damage from a 10 foot surge that lasts for 6 hours could in some ways be considered equivalent to the damage potential done by a 1 foot increase in elevation for 46 hours.

With a 10 foot surge we will have a delta of 8 feet for 1 hour.	8 equivalent hours
With a 10 foot surge we will have a delta of 6 feet for 2 hour.	12 equivalent hours
With a 10 foot surge we will have a delta of 4 feet for 3 hour.	12 equivalent hours
With a 10 foot surge we will have a delta of 2 feet for 4 hour.	8 equivalent hours
With a 10 foot surge we will have a delta of 1 feet for 6 hour.	6 equivalent hours
	---
1 foot equivalent	46 equivalent hours

11.2. Modeling Considerations. A rigorous geotechnical analysis of surge effects would likely consider a time stepped transient seepage analysis. In this analysis, incremental lake levels would be evaluated by transient seepage analyses over specific time increments. Incorporating a probabilistic approach (two complete set of analyses for each variable plus an expected value (deterministic) analysis) would be a major effort. Lacking the tools and resources to accomplish this exhaustive analysis, it was decided an impact, or equivalent lake elevation factor (ELE) would be applied based on a percentage of the maximum surge.

11.3. Geotechnical Considerations. The embankment likely will not breach due to a storm surge alone because of its relatively short duration. More likely, the surge will initiate damage that will cause a failure after the lake returns to the static lake level. For a 10-foot storm surge, the peak 10-foot surge of water would only last for about 1 hour. On the other hand we recognize that the storm surge will increase the probability of levee failure. The problem is how to account for the Storm Surge effect in the model.

11.3.1 Significant Duration. In our judgment it would take at least 4 - 6 hours for the effects of the surge to significantly damage the landside of the embankment. We recognize, however, that artesian pressure is instantaneous. The toe of the embankment can experience damage in as little as one hour if the critical artesian pressure is achieved. Therefore, four hours was selected as the minimum significant duration.

11.3.2. Significant Head. During the 1995 high water event, an increase in head of 1 foot caused significant piping. Based on the 1995 high water event, significant piping can occur in a period of several days (say 36 hours). The direction of the wind will change the elevation of the lake. On any given day, the elevation around the lake may vary by ½ foot. During a major storm, the direction of the wind can remain constant for days. This could result in a 1 foot constant surge for days. A 1 foot increase in head would cause serious piping problems if the lake were already at a critical elevation. A member of the inspection team during the 1998 high water event stated that he could observe increases and decreases in the seepage observed that was affected by wind direction changes.

11.4. Equivalent Lake Elevation Factor. This Equivalent Lake Elevation (ELE) factor, measured in feet, would be added to the static lake elevation prior to obtaining the final lake elevation to be used to obtain probability of breach. A four hour duration corresponds to an elevation approximately equal to 50% of the peak surge elevation. Therefore, 50% of the peak surge elevation was selected as the ELE factor. For example, a static lake elevation of 14.0 is generated In step 1. A tropical storm is generated in step 2. The 10 foot peak duration is multiplied by 50% to obtain 5.0 feet. The ELE factor of 5.0 feet is added to 14.0 to compute the probability of breach at a lake level of 19.0 feet. We feel this is a reasonable approach since it also considers the static lake level prior to surge.

Figure H-11.1 Surge Hydrograph Chart 19

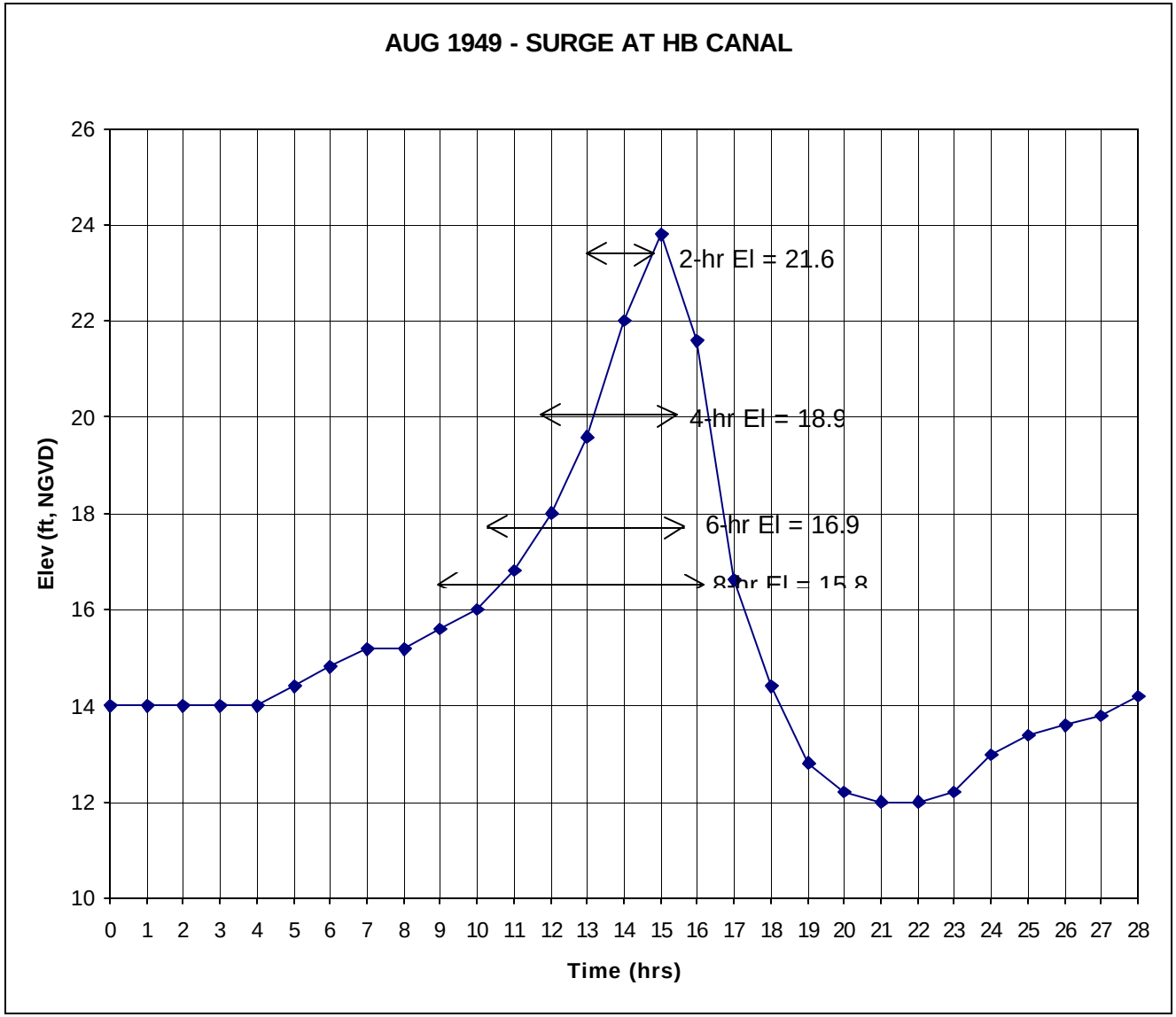


Table H-11.1 Storm Data

			TOTAL DURATION			50% OF TOTAL DURATION			
		IWSE							% OF
STORM	GAGE	(ft, NGVD)	D100%	E100%	H100%	D50%	E50%	H50%	PEAK
777	PTMAYA	14.50	24	17.60	3.10	12	15.00	0.50	16.13
	LS-14	14.50	24	15.90	1.40	12	14.80	0.30	21.43
499	FSHECR	13.00	30	17.40	4.40	15	13.30	0.30	6.82
	IPCANAL	13.00	30	16.00	3.00	15	13.20	0.20	6.67
	KISSIMR	13.00	30	15.80	2.80	15	13.20	0.20	7.14
	OKEECH	13.00	32	16.50	3.50	16	13.20	0.20	5.71
	HILLSBC	13.00	18	14.80	1.80	9	13.10	0.10	5.56
	LS-19	13.00	24	14.00	1.00	12	13.10	0.10	10.00
	C-10A	13.00	24	15.50	2.50	12	13.20	0.20	8.00
	CANALPT	13.00	30	15.50	2.50	15	13.10	0.10	4.00
	HC2	13.00	30	16.40	3.40	15	13.10	0.10	2.94
	MRHAVN	13.00	30	17.40	4.40	15	13.10	0.10	2.27
	LS-18	13.00	24	18.20	5.20	12	13.30	0.30	5.77
	LS-16	13.00	24	15.50	2.50	12	13.20	0.20	8.00
477	CANALPT	14.00	30	20.00	6.00	15	14.80	0.80	13.33
	HILLSBC	14.00	24	23.80	9.80	12	14.70	0.70	7.14
	LAKHBR	14.00	24	21.00	7.00	12	16.00	2.00	28.57
	CLEWST	14.00	24	19.70	5.70	12	15.70	1.70	29.82
	MRHAVN	14.00	30	23.40	9.40	15	14.80	0.80	8.51
	FSHECR	14.00	24	18.30	4.30	12	14.70	0.70	16.28
	LS-14	14.00	16	17.60	3.60	8	14.70	0.70	19.44

Storm 777 - Hurricane David (03 Sep 1979) AVG %
Storm 499 - Hurricane King (18 Oct 1950) 11.12
Storm 477 - Unnamed (27 Aug 1949)

IWSE = Initial water surface elevation
% of Peak = percentage of the peak surge magnitude
D100% = Total duration of the surge event (i.e., from IWSE to IWSE)
E100% = Maximum elevation of the water surface (i.e., peak surge)
H100% = Height of the peak surge above the IWSE
D50% = One half of the total surge duration
E50% = Elevation sustained during time period defined as the 50% duration
H50% = Height of the 50% surge above the IWSE

Table H-11.2 Surge Height and Duration

				SURGE HEIGHTS AT X-HOUR DURATIONS							
X = 2 HR		% OF	X = 4 HR		% OF	X = 6 HR		% OF	X = 8 HR		% OF
EL2	H2	PEAK	EL4	H4	PEAK	EL6	H6	PEAK	EL8	H8	PEAK
17.00	2.50	80.65	16.80	2.30	74.19	16.50	2.00	64.52	16.00	1.50	48.39
15.60	1.10	78.57	15.30	0.80	57.14	15.00	0.50	35.71	14.70	0.20	14.29
16.50	3.50	79.55	15.70	2.70	61.36	14.80	1.80	40.91	14.20	1.20	27.27
15.40	2.40	80.00	14.70	1.70	56.67	14.10	1.10	36.67	13.80	0.80	26.67
14.50	1.50	53.57	13.90	0.90	32.14	13.50	0.50	17.86	13.50	0.50	17.86
14.50	1.50	42.86	13.40	0.40	11.43	13.10	0.10	2.86	13.10	0.10	2.86
13.30	0.30	16.67	13.10	0.10	5.56	13.10	0.10	5.56	13.10	0.10	5.56
13.10	0.10	10.00	13.10	0.10	10.00	13.10	0.10	10.00	13.10	0.10	10.00
14.50	1.50	60.00	13.80	0.80	32.00	13.40	0.40	16.00	13.30	0.30	12.00
14.50	1.50	60.00	14.00	1.00	40.00	13.80	0.80	32.00	13.50	0.50	20.00
15.80	2.80	82.35	14.00	1.00	29.41	13.60	0.60	17.65	13.40	0.40	11.76
16.50	3.50	79.55	16.00	3.00	68.18	15.00	2.00	45.45	14.00	1.00	22.73
17.00	4.00	76.92	16.40	3.40	65.38	15.20	2.20	42.31	14.30	1.30	25.00
15.00	2.00	80.00	14.50	1.50	60.00	14.00	1.00	40.00	13.70	0.70	28.00
18.00	4.00	66.67	17.20	3.20	53.33	16.50	2.50	41.67	15.80	1.80	30.00
21.60	7.60	77.55	18.90	4.90	50.00	16.90	2.90	29.59	15.80	1.80	18.37
19.20	5.20	74.29	17.60	3.60	51.43	16.00	2.00	28.57	15.00	1.00	14.29
19.10	5.10	89.47	18.00	4.00	70.18	17.00	3.00	52.63	16.60	2.60	45.61
22.50	8.50	90.43	21.00	7.00	74.47	19.20	5.20	55.32	18.00	4.00	42.55
17.60	3.60	83.72	16.60	2.60	60.47	16.00	2.00	46.51	15.50	1.50	34.88
16.90	2.90	80.56	15.70	1.70	47.22	14.90	0.90	25.00	14.70	0.70	19.44
		AVG %			AVG %			AVG %			AVG %
		68.73			48.12			32.70			22.74

12. DESIGN OF REMEDIAL MEASURES

12.1 General. Several remedial measures were analyzed for suitability for slope stability and seepage control measures. These measures were analyzed according to their effectiveness, cost availability of right-of-way, constructibility, and compatibility with the environment. Seepage control measures considered are shown below. Other remedial measures looked at were impervious lakeside blanket and relief wells. Slope stability measures considered included flattening of the landside slopes, the construction of a stability berm and relief wells.

12.2. Discussion of Seepage Control Measures.

12.2.1. Alternative A. Alternative A consists of improving the existing drainage ditches by cleaning out the ditches and grading the existing drainage ditches. Refer to Figure H-12.1. Culverts with automatic/manual gates will be installed to control the water level in the ditches. Pumps will be installed at various locations to control the water levels in the ditches. During critical high water period, the water level in the ditches will be raised in order to limit the differential head across the levee. However, raising the water level in the ditches will increase the local flooding potential. Presently, most of these ditches are controlled by the local drainage districts and farmers. A minor stability berm will be constructed along the exterior toe of the embankment The berm will consist of excavating the 3 feet of peat from the landside toe of the embankment between the embankment toe and the toe of the drainage ditch, about 25 feet. Then a 25-foot wide, 5-foot deep stability berm will be constructed. The stability berm will allow access to the toe of the embankment and ditches for inspection. Refer to Table H-12.1 and Figure H-12.2 for a comparison to the base conditions where Alternative A would be implemented in Reach 1.

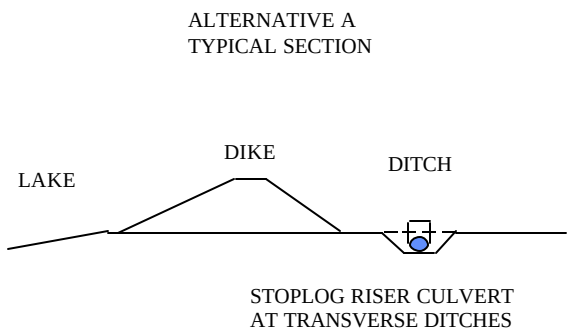


Figure H-12.1

Table H-12.1 bcsum-f.xls, Alternative "A"

Reach 1 - Alternative "A" Comparison

Base Conditions - Reach 1

Probability of Breach at selected Lake Elevations.

* = Adjusted Base Conditions in Model

Reach and Section	15 *	18 *	21	24	27
Reach 1A, Line 25	0.00%	0.75%	45.00%	100.00%	100.00%
Reach 1B, Line 25	0.00%	0.75%	39.00%	100.00%	100.00%
Reach 1C, Line 18	0.00%	12.00%	91.00%	100.00%	100.00%
Reach 1C, Line 9	0.00%	3.00%	49.00%	100.00%	100.00%
Combined Probability	0.00%	15.92%	98.46%	100.00%	100.00%

Alternative "A" in Reach 1

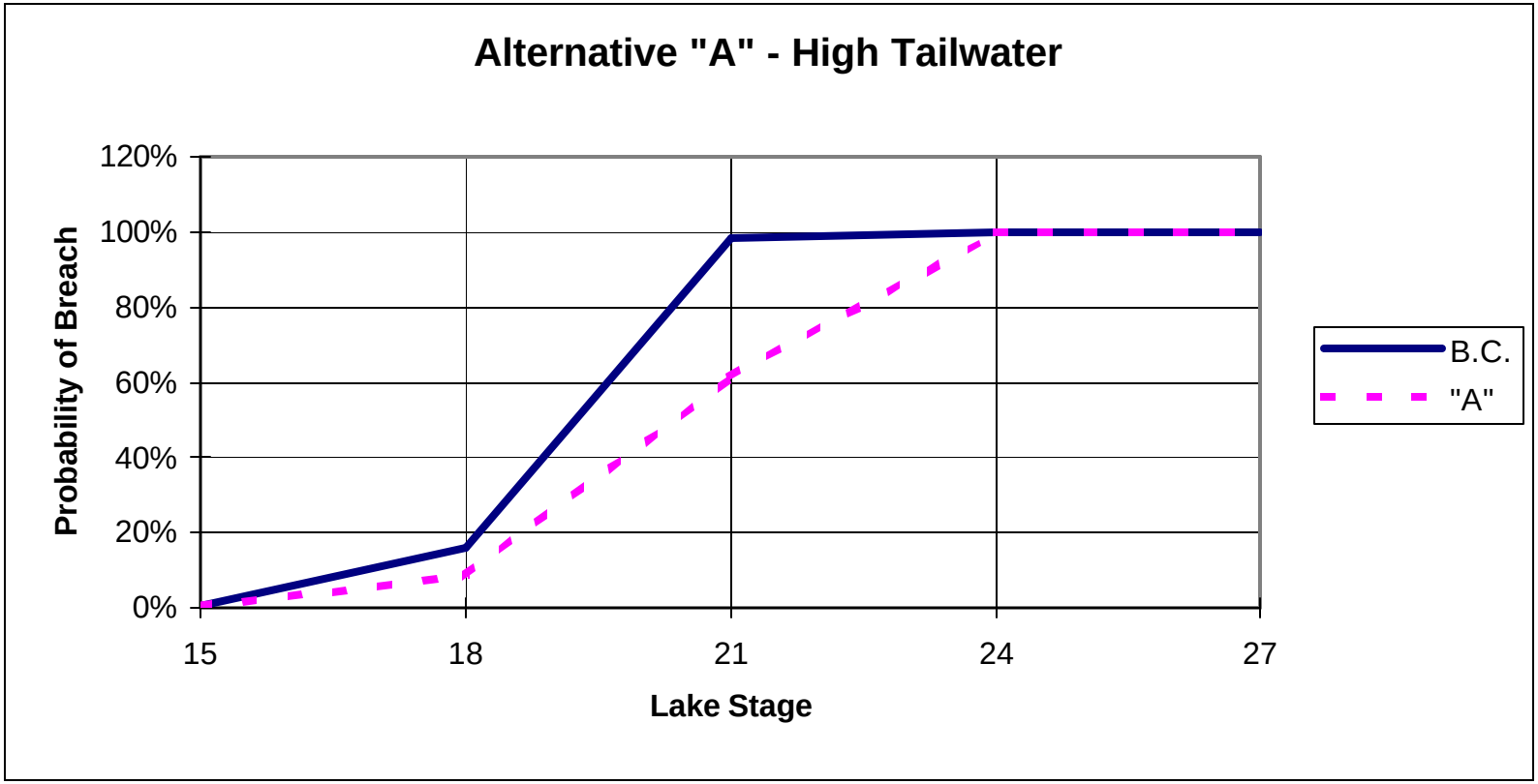
Probability of Breach at selected Lake Elevations.

Reach and Section	15 *	18 *	21	24	27
Reach 1A, Line 25	0.00%	0.01%	1.25%	81.67%	100.00%
Reach 1B, Line 25	0.00%	0.01%	1.06%	76.08%	100.00%
Reach 1C, Line 18	0.00%	7.56%	60.03%	92.49%	100.00%
Reach 1C, Line 9	0.00%	0.98%	1.50%	90.00%	100.00%
Combined Probability	0.00%	8.49%	61.53%	99.97%	100.00%

Alternative A consists of acquiring necessary ROW, stabilizing the berm and installing structures to control water levels in the toe ditches during a high water event.

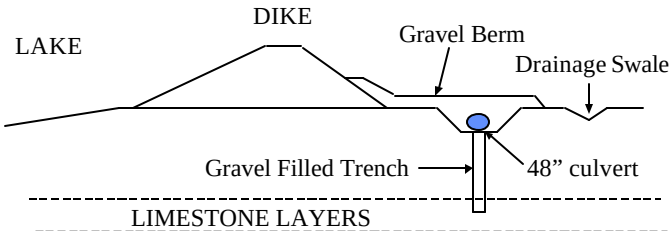
Alt. "A" is not considered to be a feasible option at Reach 1, Line 6.

Figure H-12.2 BCSUM-F.XLS, Alternative “A” Chart 1



12.2.2. Alternative B. Alternative B consists of excavating 4 foot of peat from along the landside toe between the toe of the embankment and the top of the existing drainage ditch. Backfilling this area with 5 foot of graded granular filter drainage blanket. A 2-foot thick drainage blanket will extend up on the embankment to elevation 23 feet. This blanket will intercept through seepage. In order to prevent uplift and to intercept underseepage a 2-foot wide relief trench, fill with a graded granular filter stone will be installed in the bottom of the existing ditch. The relief trench will extend to the top of the lower limestone layer, about 25 feet. A 48 inch perforated culvert wrap with non-woven filter fabric will be installed at the same elevation as the existing ditch and covered with a grader granular filter blanket to carry off excess seepage water. The relief trench/wells will control uplift pressures and prevent heaving at the toe of the embankment. The vertical relief drainage system will reduce artesian pressures in the foundation and will intercept and provide controlled outlet for seepage which otherwise would emerge uncontrolled landward of the embankment. The 48-inch culvert will remove the seepage collected in the berm. Stop log riser culverts will be installed at various locations to drain the 48-inch culvert. Drop inlets will be installed on 400-foot centers to drain surface runoff into the 48-inch culvert. Alternative B solution is not suitable for Reach 7, or at line 6, the quarry site. The vertical relief drain will increase the total seepage from the lake. See Figure H-12.3 for typical section. Refer to Table H-12.2 and Figure H-12.4 for a comparison to the base conditions where Alternative B would be implemented in Reach 1. At each location where the drainage system meets an existing drainage canal, a mechanism for discharging flows from the 48-inch diameter pipe into the canal is required. This will be accomplished by construction of a drainage pool and stoplog riser at each outlet point. The drainage pool will collect and hold flows emerging from the seepage drain system. The pool will be lined with a gabion mattress to prevent erosion of its banks. From the drainage pool, flows will be introduced into the adjacent canal or drainage ditch by means of a stoplog riser. This system will provide control over the water level within the seepage drain.

ALTERNATIVE B
TYPICAL SECTION



NOT TO SCALE

Figure H-12.3

Table H-12.2 BCSUM-F.XLS, Alternative “B”

Reach 1 - Alternative "B" Comparison

Base Conditions - Reach 1

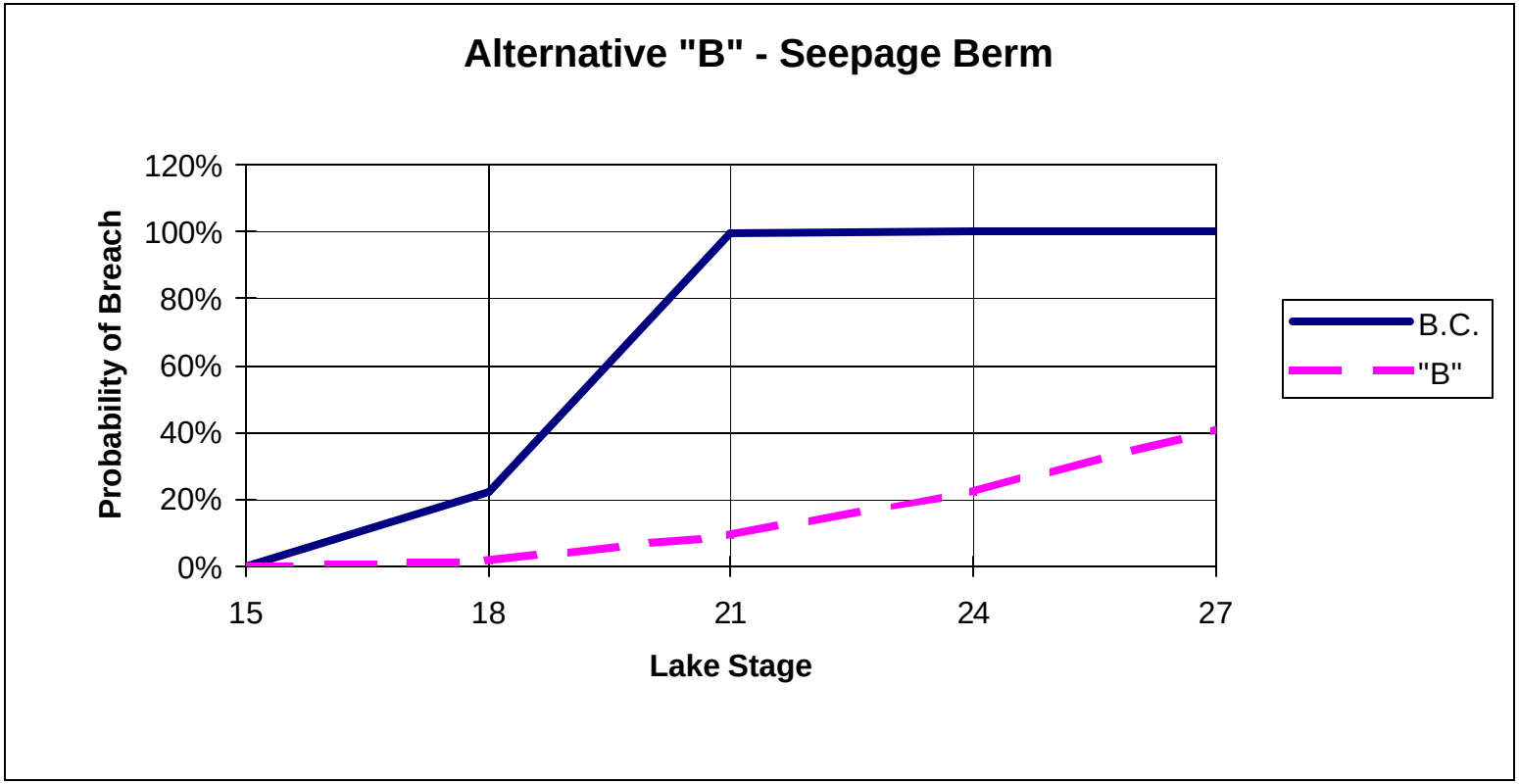
Reach and Section	Probability of Breach at selected Lake Elevations.				
	15 *	18 *	21	24	27
Reach 1A, Line 25	0.00%	0.75%	45.00%	100.00%	100.00%
Reach 1B, Line 25	0.00%	0.75%	39.00%	100.00%	100.00%
Reach 1C, Line 18	0.00%	12.00%	91.00%	100.00%	100.00%
Reach 1C, Line 9	0.00%	3.00%	49.00%	100.00%	100.00%
Reach 1C, Line 6	0.00%	7.88%	68.00%	80.00%	92.00%
Combined Probability	0.00%	22.54%	99.51%	100.00%	100.00%

Alternative "B" in Reach 1

Reach and Section	Probability of Breach at selected Lake Elevations.				
	15 *	18	21	24	27
Reach 1A, Line 25	0.00%	0.33%	2.00%	5.00%	10.00%
Reach 1B, Line 25	0.00%	0.33%	2.00%	5.00%	10.00%
Reach 1C, Line 18	0.00%	0.33%	2.00%	5.00%	10.00%
Reach 1C, Line 9	0.00%	0.33%	2.00%	5.00%	10.00%
Reach 1C, Line 6	0.00%	0.33%	2.00%	5.00%	10.00%
Combined Probability	0.00%	1.65%	9.61%	22.62%	40.95%

Alternative B consists of acquiring necessary ROW and constructing a seepage berm with relief wells and a french drain.

Figure H-12.4 Alternative “B” Chart



12.2.3. Alternative C. Alternative C consists of the constructing an impervious cutoff wall to elevation –40 feet. The wall will be located on the upstream slope of the embankment. The cut-off wall will impede groundwater flow. This is the most positive method of underseepage control because it reduces both uplift pressure and through seepage. The wall will be 3 foot wide and approximately 60 foot deep. The excavation will be filled with soil-bentonite or soil-cement mixture. The top of the wall will be at approximately elevation 25 feet. See Figure H-12.5 for typical section. The negative aspects of the cutoff wall are that must penetrate at least 98 percent of the aquifer or they will not reduce seepage pressures or seepage significantly. The cutoff wall could affect the upper aquifer and the ground water table thereby affecting local adjacent farms. A landside stability berm as described in Alternative "A" will also be constructed. Refer to Table H-12.3 and Figure H-12.6 for a comparison to the base conditions where Alternative C would be implemented in Reach 1.

ALTERNATIVE C
TYPICAL SECTION

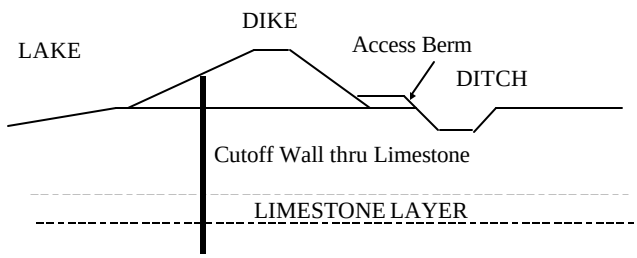


Figure H-12.5

12.2.4. Other Control Methods Considered.

12.2.4.1. Imperious Blanket. A lakeside impervious blanket can be used to reduce the intensity of seepage and pressures below the embankment. However, seepage entry generally occurs in the deep borrow canal and borrow pits located at the interior toe of the embankment, therefore, an imperious blanket would have no effect on underseepage or pressure.

12.2.4.2. Relief Wells. Relief wells can be constructed at the landside toe of the embankment to reduce artesian pressure and to intercept and provide controlled outlets for seepage which otherwise would emerge uncontrolled landward of embankment. Relief wells do not reduce but in fact increase seepage. A drawback with relief wells is that conveyance system must be constructed to remove the seepage. Maintenance on the well system will be high and the wells could stopped up and cause increase pressures.

Table H-12.3 bcsum-f.xls, Alternative “C”

Reach 1 - Alternative "C" Comparison

Base Conditions - Reach 1

Probability of Breach at selected Lake Elevations.

* = Adjusted Base Conditions in Model

Reach and Section	15 *	18 *	21	24	27
Reach 1A, Line 25	0.00%	0.75%	45.00%	100.00%	100.00%
Reach 1B, Line 25	0.00%	0.75%	39.00%	100.00%	100.00%
Reach 1C, Line 18	0.00%	12.00%	91.00%	100.00%	100.00%
Reach 1C, Line 9	0.00%	3.00%	49.00%	100.00%	100.00%
Reach 1C, Line 6	0.00%	7.88%	68.00%	80.00%	92.00%
Combined Probability	0.00%	22.54%	99.51%	100.00%	100.00%

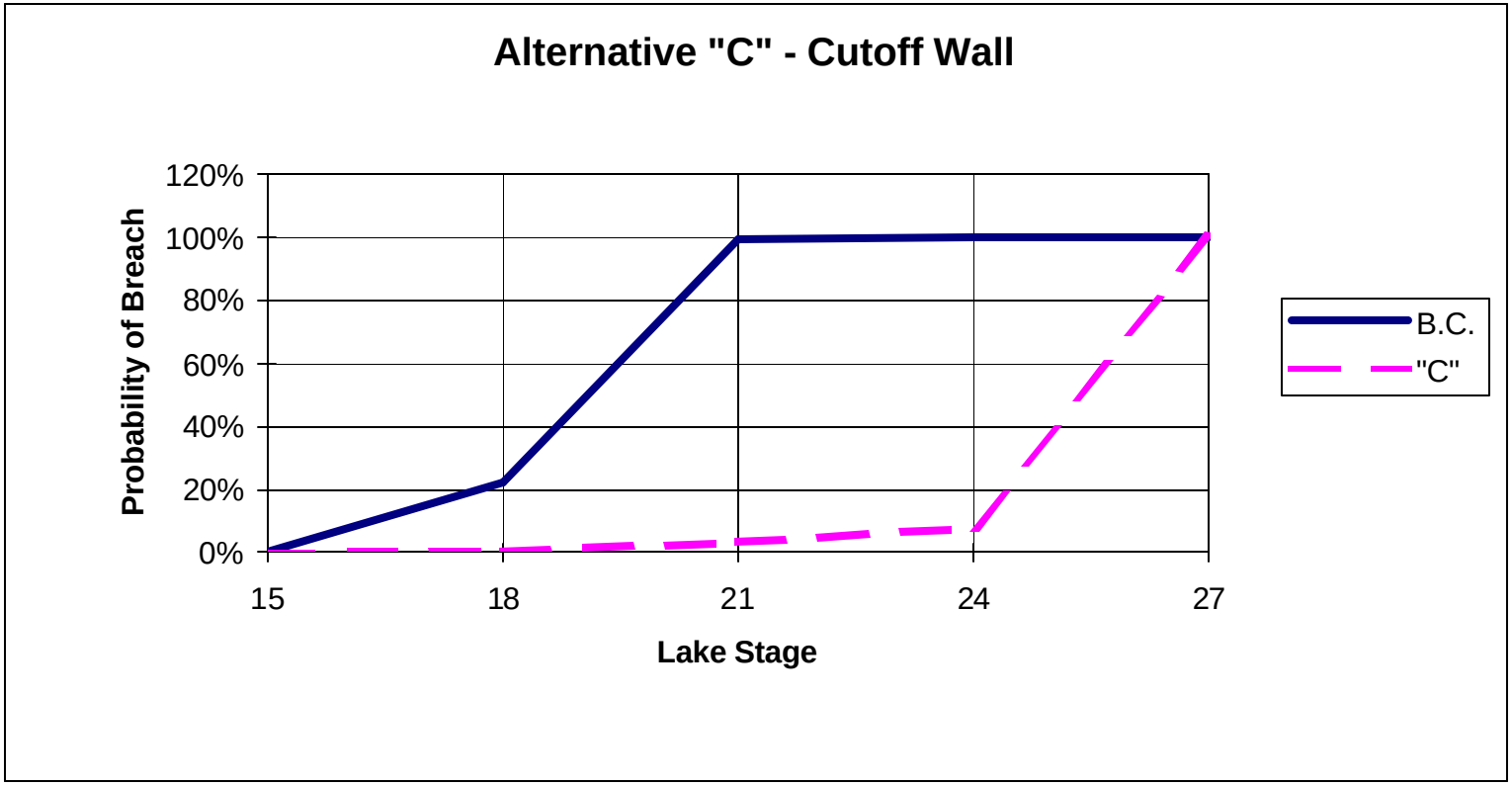
Alternative "C" in Reach 1

Probability of Breach at selected Lake Elevations.

Reach and Section	15 *	18	21	24	27
Reach 1A, Line 25	0.00%	0.11%	0.67%	1.67%	100.00%
Reach 1B, Line 25	0.00%	0.11%	0.67%	1.67%	100.00%
Reach 1C, Line 18	0.00%	0.11%	0.67%	1.67%	100.00%
Reach 1C, Line 9	0.00%	0.11%	0.67%	1.67%	100.00%
Reach 1C, Line 6	0.00%	0.11%	0.67%	1.67%	100.00%
Combined Probability	0.00%	0.55%	3.29%	8.06%	100.00%

Alternative C consists of installing a cutoff wall lakeside of the crest and stabilizing the landside berm. Top of cutoff at el. 25.

Figure H-12.6 Alternative ″C″ Chart 1



12.3 Recommended Alternative Plan. The probability for a breach based on lake levels is shown for Reach 1 for both the base condition and alternative remedial actions.
lists the results of the probabilistic evaluation for all the Reaches and sub-reaches around the lake. If Alternative B is selected as the general treatment for the embankment in Reach 1, the quarry at line 6 in Reach 1 would require backfilling. A cutoff wall would be required for the entire Reach 7 and for approximately 9 miles in reach 2. Refer to Tables H-12.4 and H-12.5 and Figures H-12.7 and H-12.8 for a comparison to the base conditions where the Preferred Plan would be implemented.

Table H-12.4 BCSUM-F.XLS, Preferred Plan

Base Conditions at Herbert Hoover Dike					
Reach and Section	Probability of Breach at selected Lake Elevations.				
	15	18	21	24	27
Reach 1A, Line 25	0.00%	0.75%	45.00%	100.00%	100.00%
Reach 1B, Line 25	0.00%	0.75%	39.00%	100.00%	100.00%
Reach 1C, Line 18	0.00%	12.00%	91.00%	100.00%	100.00%
Reach 1C, Line 9	0.00%	3.00%	49.00%	100.00%	100.00%
Reach 1C, Line 6	0.00%	7.88%	68.00%	80.00%	92.00%
Reach 2	0.00%	12.00%	89.00%	99.00%	100.00%
Reach 3	0.00%	14.00%	100.00%	100.00%	100.00%
Reach 4	0.00%	0.25%	5.00%	10.00%	15.00%
Reach 5	0.00%	0.25%	5.00%	10.00%	15.00%
Reach 6 A	0.00%	0.13%	1.95%	4.00%	6.00%
Reach 6 B	0.00%	0.20%	3.12%	6.40%	9.60%
Reach 7	0.00%	5.96%	99.00%	100.00%	100.00%
Reach 8	0.00%	0.25%	5.00%	10.00%	15.00%
Combined Probability	0.00%	45.46%	100.00%	100.00%	100.00%

	Herbert Hoover Dike - Preferred Plan					
Reach and Section	Probability of Breach at selected Lake Elevations.					Alternative
	15	18	21	24	27	Option
Reach 1A, Line 25	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1B, Line 25	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1C, Line 18	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1C, Line 9	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1C, Line 6	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 2	0.00%	0.33%	2.00%	5.00%	10.00%	B & A
Reach 3	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 4	0.00%	0.25%	3.50%	7.50%	12.50%	A
Reach 5	0.00%	0.25%	3.50%	7.50%	12.50%	A
Reach 6 A	0.00%	0.13%	1.50%	2.50%	5.00%	A
Reach 6 B	0.00%	0.20%	2.00%	5.00%	7.50%	A
Reach 7	0.00%	0.25%	3.50%	7.50%	12.50%	A
Reach 8	0.00%	0.25%	3.50%	7.50%	12.50%	A
Combined Probability	0.00%	3.60%	27.33%	52.65%	75.36%	

Figure H-12.7 Preferred Plan Chart 1

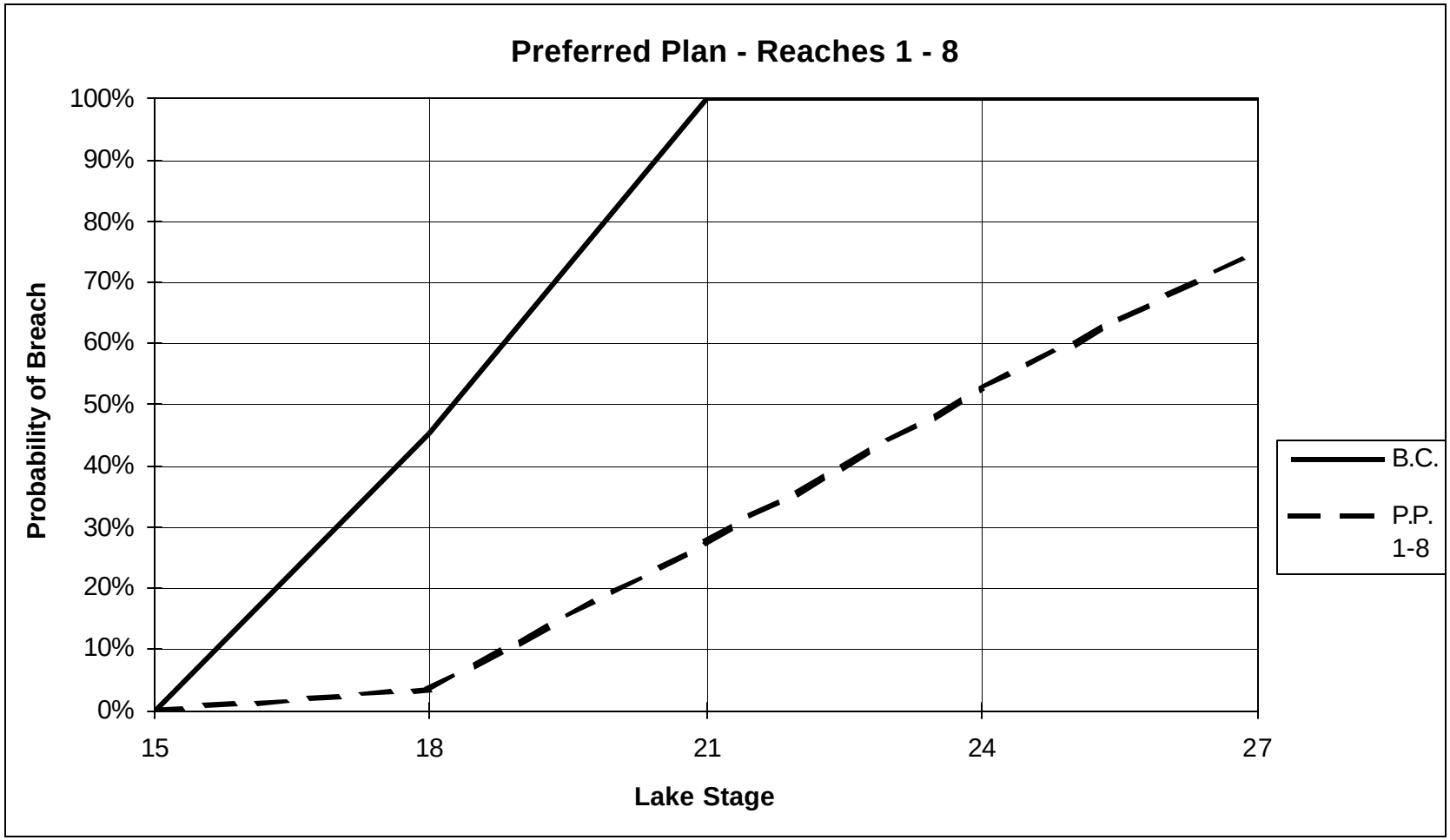
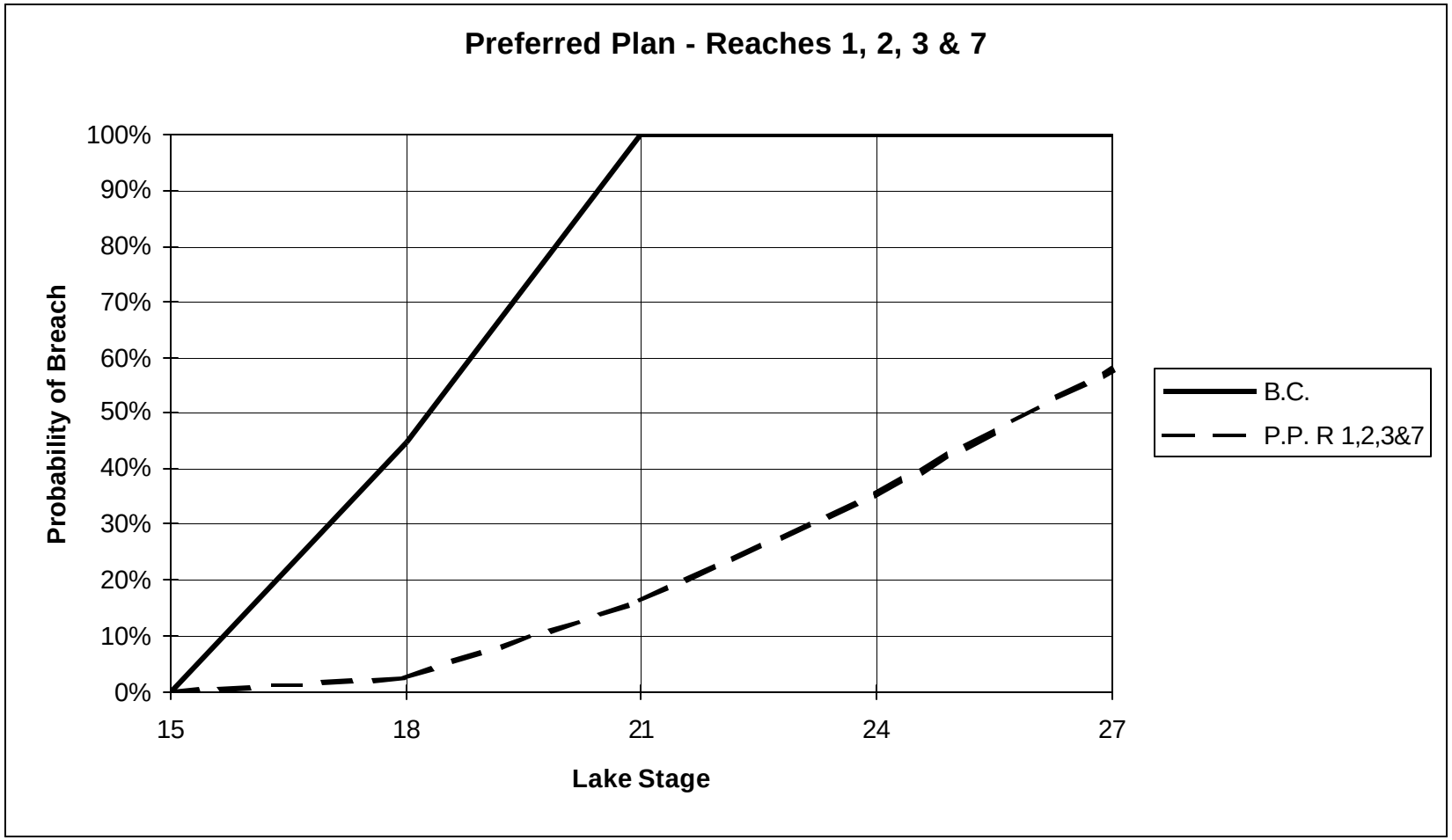


Table H-12.5 bcsum-f.xls, Pref. Plan, R1,2,3&7

Base Conditions at Herbert Hoover Dike					
Reach and Section	Probability of Breach at selected Lake Elevations.				
	15	18	21	24	27
Reach 1A, Line 25	0.00%	0.75%	45.00%	100.00%	100.00%
Reach 1B, Line 25	0.00%	0.75%	39.00%	100.00%	100.00%
Reach 1C, Line 18	0.00%	12.00%	91.00%	100.00%	100.00%
Reach 1C, Line 9	0.00%	3.00%	49.00%	100.00%	100.00%
Reach 1C, Line 6	0.00%	7.88%	68.00%	80.00%	92.00%
Reach 2	0.00%	12.00%	89.00%	99.00%	100.00%
Reach 3	0.00%	14.00%	100.00%	100.00%	100.00%
Reach 7	0.00%	5.96%	99.00%	100.00%	100.00%
Combined Probability	0.00%	44.87%	100.00%	100.00%	100.00%

HHD - Preferred Plan; Reaches 1, 2, 3 & 7						
Reach and Section	Probability of Breach at selected Lake Elevations.					Alternative
	15	18	21	24	27	Option
Reach 1A, Line 25	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1B, Line 25	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1C, Line 18	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1C, Line 9	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 1C, Line 6	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 2	0.00%	0.33%	2.00%	5.00%	10.00%	B & A
Reach 3	0.00%	0.33%	2.00%	5.00%	10.00%	B
Reach 7	0.00%	0.25%	3.50%	7.50%	12.50%	A
Combined Probability	0.00%	2.55%	16.23%	35.40%	58.15%	

Figure H-12.8 Pref. Plan, R1,2,3&7 Chart 1



Herbert Hoover Dike

Major Rehabilitation Evaluation Report

Appendix I

Hydrology and Hydraulics Analysis

1. INTRODUCTION.

This appendix documents hydrologic and hydraulic engineering efforts undertaken in support of the Herbert Hoover Dike Major Rehabilitation Evaluation. Those efforts were directed toward two major objectives. The first objective was identification of the hydraulic loads to which the dike system could be subjected. The second objective was identification of the consequences of a dike failure. Engineering efforts related to those objectives are described below.

2. HYDRAULIC LOADS.

The forcing mechanism that initiates seepage and slope stability problems along the dike system is directly related to the hydraulic gradient. That gradient is defined by the difference in water surface elevations on each side of the dike; therefore, estimates of the hydraulic gradient must consider the Lake Okeechobee pool elevation (headwater) and the elevation of water levels in landside canals and ditches (tailwater). The headwater elevation is a combination of the static lake stage, which varies slowly (weeks) based on Lake Okeechobee inflows and outflows, and hurricane-induced storm surge, which varies rapidly (hours) in response to the passing of a tropical cyclone. The tailwater elevation is dependent on the site-specific characteristics of the toe ditch or canal that exists at a particular location, and the manner in which the water level in that ditch or canal is managed. Frequency of occurrence relationships relative to all three of these processes (i.e., static lake stage, storm surge, and tailwater) were required for this major rehabilitation evaluation. The generation of those frequency relationships is discussed in the following paragraphs.

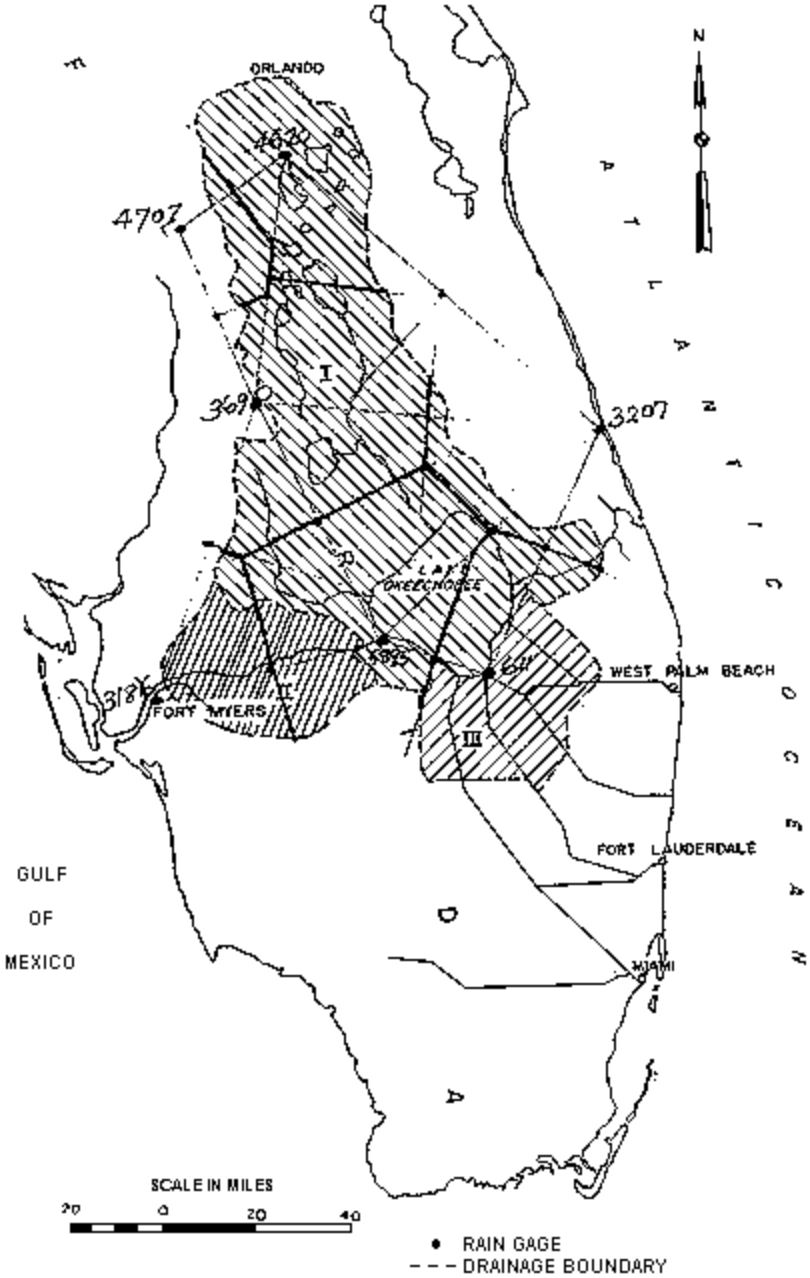
3. LAKE OKEECHOBEE FLOOD ROUTINGS.

Introduction - Flood routings were performed to determine the Lake Okeechobee stage-frequency relationship. The relationship developed herein considers the stage-duration data for the RUN25 lake regulation schedule, rainfall data available for the period 1931 through 1993, and drainage modifications related to the Kissimmee River Restoration Project.

Drainage Basin Information – Figure 1 depicts the drainage area for Lake Okeechobee, St. Lucie Canal, and the

Caloosahatchee River. A total drainage area of about 5,540 square miles contributes discharges to Lake Okeechobee.

Figure 1



Area sub-basins are listed in Table 1.

Table 1. Lake Okeechobee Drainage Areas.

TABLE 1 LAKE OKEECHOBEE DRAINAGE AREAS	
BASIN IDENTIFICATION	AREA SQ MILES
KISSIMMEE RIVER	2456.0
LAKE ISTOKPOGA AREA	989.1
FISHEATING CREEK	550.0
TAYLOR CREEK AREA	188.0
S-153	49.8
S-154	32.4
S-135	21.8
S-133	25.3
S-127	31.2
S-129	18.0
S-131	11.7
S-2	180.0
S-3	129.0
S-4	116.0
LAKE OKEECHOBEE	742.0
TOTAL DRAINAGE AREA	5540.3
CALOOSAHATCHEE RIVER LOCAL AREA	1040.0
ST. LUCIE CANAL LOCAL AREA	245.0

<u>Inflows</u> – An HEC-1 model was used to generate hydrographs for each sub-basin. Inflows to Lake Okeechobee were restricted by the Standard Project Flood (SPF) design capacities of the inlet structures. Discharges from pumped areas were limited to the pump station capacities. Daily rainfall values over Lake Okeeechobee were converted to daily discharges after subtraction of losses due to evapotranspiration. The inflow hydrographs are listed in Table 2 (14 pages). Figures 2 and 3 indicate that the discharges estimated by the current study compare well with

TABLE 2
LAKE OKEECHOBEE INFLOWS
50% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
31-Aug	97	150	111	460	849	1385	2394	26	15	9	21	18	40	26	154	5760
01-Sep	196	285	210	643	1160	1444	5587	49	28	18	40	34	60	39	227	10024
02-Sep	406	585	431	1157	1786	1616	10974	101	58	37	82	70	108	70	409	17899
03-Sep	1183	1674	1234	2924	4386	2141	42299	291	167	108	236	202	279	182	1056	58368
04-Sep	1238	1872	1380	4215	4755	3006	8380	325	187	121	263	226	364	237	1376	27951
05-Sep	437	823	607	3485	4047	3613	2993	141	82	54	115	99	268	174	1015	17960
06-Sep	210	404	298	2515	3518	3767	2394	69	40	26	56	48	188	122	711	14374
07-Sep	167	287	211	1801	3172	3775	2394	49	28	18	40	34	147	95	556	12780
08-Sep	162	259	191	1343	2923	3763	2793	44	25	16	36	31	124	81	468	12267
09-Sep	161	253	186	1084	2722	3751	2594	43	25	16	35	30	109	71	415	11503
10-Sep	161	251	185	920	2562	3739	2993	43	25	16	35	30	101	66	382	11514
11-Sep	161	251	185	829	2415	3728	2993	43	25	16	35	30	93	60	352	11222
12-Sep	161	251	185	787	2287	3717	2594	43	25	16	35	30	84	54	317	10591
13-Sep	133	212	156	719	2087	3697	2195	36	21	13	29	25	72	47	274	9725
14-Sep	101	165	121	622	1907	3654	2195	28	16	10	23	19	60	39	228	9196
15-Sep	97	154	113	554	1773	3597	2594	26	15	10	21	18	52	34	197	9261
16-Sep	97	151	111	512	1654	3539	1995	26	15	9	21	18	48	31	181	8413
17-Sep	97	150	111	487	1537	3484	1397	26	15	9	21	18	46	30	173	7605
18-Sep	97	150	111	474	1433	3431	1796	26	15	9	21	18	44	29	168	7827
19-Sep	97	150	111	466	1339	3380	798	26	15	9	21	18	43	28	165	6671
20-Sep	97	150	111	463	1255	3330	1397	26	15	9	21	18	42	27	162	7128
21-Sep	97	150	111	462	1188	3283	1397	26	15	9	21	18	42	27	159	7010
22-Sep	97	150	111	461	1131	3237	1397	26	15	9	21	18	41	27	158	6905
23-Sep	97	150	111	461	1085	3193	1596	26	15	9	21	18	41	27	157	7012
24-Sep	97	150	111	461	1048	3150	1596	26	15	9	21	18	41	27	157	6932
25-Sep	97	150	111	461	1021	3110	2195	26	15	9	21	18	41	27	157	7464

TABLE 2
LAKE OKEECHOBEE INFLOWS
50% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
26-Sep	97	150	111	461	999	3070	1596	26	15	9	21	18	41	27	157	6803
27-Sep	97	150	111	461	975	3033	1596	26	15	9	21	18	41	27	157	6741
28-Sep	97	150	111	461	954	2997	1596	26	15	9	21	18	41	27	157	6684
29-Sep	97	150	111	461	937	2961	1596	26	15	9	21	18	41	27	157	6632
30-Sep	97	150	111	461	923	2927	1397	26	15	9	21	18	41	27	157	6384
01-Oct	82	131	96	435	871	2890	1397	22	13	8	18	15	38	25	146	6195
02-Oct	66	107	79	388	826	2843	1596	18	10	7	15	13	34	22	131	6161
03-Oct	65	102	75	354	792	2790	1397	17	10	6	14	12	32	21	121	5813
04-Oct	64	100	74	333	764	2737	1397	17	10	6	14	12	30	20	116	5700
05-Oct	64	100	74	320	739	2685	1197	17	10	6	14	12	29	19	113	5404
06-Oct	64	100	74	314	715	2636	1197	17	10	6	14	12	29	19	110	5322
07-Oct	64	100	74	310	696	2589	1197	17	10	6	14	12	28	18	108	5249
08-Oct	64	100	74	308	681	2543	1397	17	10	6	14	12	28	18	107	5384
09-Oct	64	100	74	308	668	2499	1197	17	10	6	14	12	28	18	106	5125
10-Oct	64	100	74	307	657	2457	1397	17	10	6	14	12	27	18	105	5271
11-Oct	64	100	74	307	648	2417	1197	17	10	6	14	12	27	18	105	5021
12-Oct	64	100	74	307	639	2378	1197	17	10	6	14	12	27	18	104	4972
13-Oct	64	100	74	307	628	2340	798	17	10	6	14	12	27	18	104	4524
14-Oct	64	100	74	307	622	2304	1596	17	10	6	14	12	27	18	104	5280
15-Oct	64	100	74	307	623	2269	1596	17	10	6	14	12	27	18	104	5247
16-Oct	64	100	74	307	622	2235	1397	17	10	6	14	12	27	18	104	5012
17-Oct	64	100	74	307	619	2203	1397	17	10	6	14	12	27	18	104	4978
18-Oct	50	81	59	281	575	2168	998	14	8	5	11	9	25	16	94	4399
19-Oct	34	57	42	234	534	2121	1397	9	5	3	8	6	20	13	78	4569

TABLE 2
LAKE OKEECHOBEE INFLOWS
20% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
31-Aug	157	243	179	718	1336	2305	3410	42	24	15	34	29	64	42	244	8850
01-Sep	268	395	291	936	1688	2383	6419	68	39	25	55	47	87	56	328	13092
02-Sep	614	881	649	1722	2722	2623	16247	153	88	56	124	106	162	105	613	26873
03-Sep	1945	2742	2021	4692	6975	3462	61177	477	274	176	386	331	451	294	1704	87112
04-Sep	2000	3028	2232	6796	7451	4874	11233	525	302	196	426	365	587	382	2219	42624
05-Sep	637	1236	911	5494	6201	5831	3209	212	123	81	172	149	419	273	1587	26541
06-Sep	273	553	407	3851	5346	6027	3410	94	55	36	76	66	284	185	1076	21745
07-Sep	227	392	289	2693	4804	5995	3410	67	39	25	54	47	220	143	831	19243
08-Sep	222	355	261	1961	4407	5947	3811	61	35	23	49	42	183	119	691	18173
09-Sep	221	346	255	1549	4080	5900	3610	59	34	22	48	41	160	105	606	17043
10-Sep	221	344	253	1288	3816	5854	4012	59	34	22	48	41	146	95	553	16794
11-Sep	221	343	253	1144	3575	5812	4212	59	34	22	48	41	133	87	504	16493
12-Sep	221	343	253	1079	3366	5770	3811	59	34	22	48	41	118	77	446	15694
13-Sep	207	324	239	1030	3128	5726	3009	56	32	21	45	39	106	69	401	14437
14-Sep	191	301	222	981	2916	5671	3009	52	30	19	42	36	96	62	361	13996
15-Sep	190	296	218	947	2755	5611	3410	51	29	19	41	35	88	57	332	14086
16-Sep	189	294	217	927	2610	5552	2808	51	29	19	41	35	84	55	319	13235
17-Sep	189	294	217	915	2466	5494	2206	51	29	19	41	35	83	54	315	12413
18-Sep	162	256	189	857	2260	5430	2206	44	25	16	36	31	77	50	292	11938
19-Sep	130	210	155	761	2068	5347	1203	36	21	13	29	25	68	44	260	10377
20-Sep	127	199	147	693	1910	5249	1805	34	19	12	28	24	63	41	240	10597
21-Sep	126	197	145	651	1782	5154	1805	34	19	12	27	23	60	39	229	10310
22-Sep	126	196	144	627	1673	5061	1805	34	19	12	27	23	58	38	221	10071
23-Sep	126	196	144	614	1582	4972	2006	34	19	12	27	23	57	37	216	10071
24-Sep	126	196	144	606	1507	4886	2006	34	19	12	27	23	56	36	212	9897
25-Sep	126	196	144	603	1449	4804	2608	34	19	12	27	23	55	36	209	10351

TABLE 2
LAKE OKEECHOBEE INFLOWS
20% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
27-Sep	126	196	144	601	1352	4648	2006	34	19	12	27	23	54	35	206	9490
28-Sep	126	196	144	601	1311	4575	2006	34	19	12	27	23	54	35	205	9374
29-Sep	126	196	144	601	1277	4503	2006	34	19	12	27	23	54	35	204	9268
30-Sep	126	196	144	601	1248	4435	1805	34	19	12	27	23	54	35	204	8969
01-Oct	98	158	117	550	1147	4361	1805	27	15	10	22	19	48	31	184	8599
02-Oct	67	112	82	458	1057	4267	2006	19	11	7	15	13	40	26	154	8340
03-Oct	63	101	74	391	991	4160	2006	17	10	6	14	12	35	23	135	8046
04-Oct	63	98	72	350	938	4054	2006	17	9	6	13	11	33	21	125	7824
05-Oct	63	98	72	326	890	3952	1805	17	9	6	13	11	31	20	118	7438
06-Oct	63	98	72	313	848	3855	1805	17	9	6	13	11	30	19	113	7278
07-Oct	63	98	72	305	813	3760	1805	17	9	6	13	11	29	19	110	7135
08-Oct	63	98	72	302	783	3669	2006	17	9	6	13	11	28	18	107	7208
09-Oct	63	98	72	301	758	3582	1805	17	9	6	13	11	27	18	104	6890
10-Oct	63	98	72	300	736	3498	2006	17	9	6	13	11	27	17	103	6983
11-Oct	63	98	72	300	717	3418	1805	17	9	6	13	11	27	17	102	6682
12-Oct	63	98	72	300	700	3340	1805	17	9	6	13	11	27	17	102	6586
13-Oct	63	98	72	300	682	3265	1404	17	9	6	13	11	27	17	102	6092
14-Oct	63	98	72	300	670	3193	2206	17	9	6	13	11	27	17	102	6811
15-Oct	63	98	72	300	667	3124	2206	17	9	6	13	11	27	17	102	6738
16-Oct	63	98	72	300	662	3057	2006	17	9	6	13	11	27	17	102	6465
17-Oct	63	98	72	300	656	2993	2006	17	9	6	13	11	27	17	102	6396
18-Oct	63	98	72	300	646	2932	1404	17	9	6	13	11	27	17	102	5722
19-Oct	63	98	72	300	637	2872	1805	17	9	6	13	11	27	17	102	6054

TABLE 2
LAKE OKEECHOBEE INFLOWS
10% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
01-Sep	295	432	318	1003	1827	2635	6609	75	43	27	60	52	93	61	353	13890
02-Sep	768	1094	806	2049	3211	2931	19225	190	109	70	154	132	195	127	737	31803
03-Sep	2622	3600	2580	6158	9018	4057	75299	630	360	230	510	435	595	388	2249	108733
04-Sep	2702	3600	2580	9076	9619	5984	13017	630	360	230	510	435	785	511	2963	53004
05-Sep	820	2053	1627	7288	7924	7286	3404	347	205	138	282	248	553	360	2092	34633
06-Sep	337	800	616	5062	6817	7544	3605	135	79	54	110	96	370	241	1401	27273
07-Sep	290	525	393	3520	6130	7503	3605	89	52	34	73	63	286	186	1081	23835
08-Sep	270	441	327	2526	5585	7441	3805	76	44	28	61	53	235	154	888	21941
09-Sep	254	403	297	1935	5129	7373	3605	69	40	26	56	48	201	132	762	20337
10-Sep	252	394	290	1556	4764	7298	4005	68	39	25	55	47	181	118	684	19783
11-Sep	252	392	289	1344	4426	7226	4005	67	39	25	55	47	162	106	614	19056
12-Sep	252	391	288	1247	4134	7156	3605	67	39	25	55	47	141	92	534	18079
13-Sep	238	372	274	1183	3824	7085	3204	64	37	24	52	45	125	82	474	17090
14-Sep	223	349	257	1131	3554	7004	3204	60	34	22	49	42	112	73	425	16546
15-Sep	221	344	253	1096	3346	6918	3605	59	34	22	48	41	103	67	388	16551
16-Sep	220	343	252	1075	3156	6834	3004	59	34	22	48	41	98	64	371	15627
17-Sep	220	342	252	1063	2971	6754	2403	59	34	22	48	41	96	63	366	14740
18-Sep	207	324	238	1031	2769	6672	2804	56	32	21	45	39	93	61	353	14749
19-Sep	191	300	221	981	2586	6583	1802	52	30	19	42	36	88	58	336	13330
20-Sep	189	295	217	946	2429	6488	2403	51	29	19	41	35	86	56	325	13614
21-Sep	189	294	216	925	2301	6396	2403	50	29	19	41	35	84	55	319	13362
22-Sep	189	293	216	913	2191	6306	2403	50	29	19	41	35	83	54	315	13143
23-Sep	189	293	216	906	2099	6221	2603	50	29	19	41	35	82	53	312	13155
24-Sep	189	293	216	902	2025	6138	2603	50	29	19	41	35	82	53	310	12991
25-Sep	189	293	216	901	1968	6059	3204	50	29	19	41	35	81	53	308	13453

TABLE 2
LAKE OKEECHOBEE INFLOWS FOR
10% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
27-Sep	189	293	216	900	1875	5909	2603	50	29	19	41	35	81	53	306	12605
28-Sep	189	293	216	900	1835	5839	2603	50	29	19	41	35	81	52	306	12494
29-Sep	189	293	216	900	1802	5770	2603	50	29	19	41	35	81	52	306	12392
30-Sep	189	293	216	900	1775	5703	2403	50	29	19	41	35	81	52	306	12099
01-Oct	147	237	175	824	1639	5627	2403	41	23	15	33	28	73	47	276	11594
02-Oct	101	168	123	685	1515	5520	2603	29	16	10	23	20	61	39	230	11150
03-Oct	95	151	111	586	1424	5393	2403	26	15	9	21	18	53	35	203	10549
04-Oct	94	148	109	524	1348	5266	2403	25	14	9	20	17	49	32	187	10253
05-Oct	94	147	108	488	1282	5143	2203	25	14	9	20	17	47	30	177	9811
06-Oct	94	146	108	468	1224	5026	2203	25	14	9	20	17	45	29	170	9605
07-Oct	94	146	108	457	1174	4913	2203	25	14	9	20	17	43	28	164	9423
08-Oct	94	146	108	452	1132	4804	2403	25	14	9	20	17	42	27	160	9460
09-Oct	94	146	108	450	1096	4698	2203	25	14	9	20	17	41	27	157	9112
10-Oct	94	146	108	450	1065	4597	2403	25	14	9	20	17	41	26	155	9177
11-Oct	94	146	108	450	1038	4501	2203	25	14	9	20	17	40	26	154	8853
12-Oct	94	146	108	450	1014	4408	2203	25	14	9	20	17	40	26	153	8734
13-Oct	94	146	108	450	990	4318	1802	25	14	9	20	17	40	26	153	8219
14-Oct	94	146	108	450	973	4232	2603	25	14	9	20	17	40	26	153	8917
15-Oct	94	146	108	450	965	4149	2603	25	14	9	20	17	40	26	153	8826
16-Oct	94	146	108	450	956	4068	2403	25	14	9	20	17	40	26	153	8536
17-Oct	94	146	108	450	947	3991	2403	25	14	9	20	17	40	26	153	8450
18-Oct	94	146	108	450	933	3918	1602	25	14	9	20	17	40	26	153	7561
19-Oct	94	146	108	450	919	3846	2003	25	14	9	20	17	40	26	153	7877

TABLE 2
LAKE OKEECHOBEE INFLOWS
5% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOG AREA	KISSIMMEE RIVER	RAINFAL EXCESS LAKE OKEECHOBE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
01-Sep	335	493	363	1189	2192	3390	6675	85	49	31	69	59	111	72	422	1554
02-Sep	933	1325	977	2464	3698	3731	22251	230	132	85	186	160	236	154	894	3746
03-Sep	2805	3600	2580	7989	10352	5188	91431	630	360	230	510	435	777	506	2936	13033
04-Sep	2805	3600	2580	11916	13116	7742	14362	630	360	230	510	435	1032	671	3895	6388
05-Sep	2296	3600	2580	9504	10296	9457	3641	630	360	230	510	435	720	468	2721	4745
06-Sep	569	2049	1681	6551	9110	9775	4046	342	204	141	279	247	476	310	1802	3758
07-Sep	354	865	677	4474	7875	9695	4046	145	86	59	118	104	362	235	1367	3046
08-Sep	327	587	442	3165	7183	9589	4248	100	58	39	81	70	296	193	1117	2750
09-Sep	323	521	386	2429	6514	9488	4046	90	52	34	73	63	255	167	965	2541
10-Sep	323	506	373	1960	6100	9388	4450	87	50	32	71	61	231	151	873	2466
11-Sep	337	522	385	1727	5687	9299	5057	90	52	33	73	63	210	137	796	2447
12-Sep	353	544	401	1658	5375	9225	4652	94	54	35	76	65	187	122	707	2355
13-Sep	355	550	405	1649	5062	9161	4855	95	55	35	77	66	172	113	654	2331
14-Sep	355	551	406	1666	4798	9101	4855	95	55	35	77	66	163	106	615	2295
15-Sep	355	551	406	1678	4588	9043	5259	95	55	35	77	66	155	100	583	2305
16-Sep	355	551	406	1685	4403	8987	4652	95	55	35	77	66	151	98	571	2219
17-Sep	355	551	406	1689	4212	8932	4046	95	55	35	77	66	151	98	571	2134
18-Sep	299	475	350	1587	3908	8862	3237	82	47	30	66	57	140	91	532	1976
19-Sep	235	380	280	1398	3569	8749	2225	65	38	24	53	45	124	81	470	1774
20-Sep	227	358	264	1262	3314	8606	2832	62	35	23	50	43	114	74	434	1770
21-Sep	226	352	260	1178	3096	8465	2832	61	35	22	49	42	109	71	413	1721
22-Sep	226	351	259	1128	2918	8326	2832	60	35	22	49	42	105	69	399	1682
23-Sep	226	351	259	1101	2762	8195	3034	60	35	22	49	42	103	67	389	1670
24-Sep	226	351	258	1086	2636	8067	3034	60	35	22	49	42	101	66	382	1642
25-Sep	226	351	258	1079	2533	7946	3641	60	35	22	49	42	99	64	376	1678

TABLE 2
LAKE OKEECHOBEE INFLOWS
5% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
27-Sep	226	351	258	1076	2371	7715	3034	60	35	22	49	42	97	63	369	15775
28-Sep	226	351	258	1076	2302	7606	3034	60	35	22	49	42	97	63	367	15595
29-Sep	226	351	258	1076	2245	7501	3034	60	35	22	49	42	97	63	366	15431
30-Sep	226	351	258	1076	2197	7398	2832	60	35	22	49	42	96	63	366	15078
01-Oct	169	274	202	972	2033	7284	2832	47	27	17	38	33	86	56	325	14400
02-Oct	106	179	132	783	1834	7127	3034	31	17	11	25	21	69	45	262	13682
03-Oct	83	138	101	621	1670	6938	2023	23	13	9	19	16	57	37	215	11969
04-Oct	67	109	80	489	1501	6739	2023	18	10	7	15	13	47	30	178	11332
05-Oct	64	102	75	406	1373	6537	1821	17	10	6	14	12	41	26	154	10666
06-Oct	64	100	74	358	1258	6345	1821	17	10	6	14	12	37	24	139	10284
07-Oct	64	100	74	331	1163	6158	1821	17	10	6	14	12	34	22	128	9958
08-Oct	64	100	74	317	1080	5978	2023	17	10	6	14	12	31	20	120	9872
09-Oct	64	100	73	310	1011	5805	1821	17	10	6	14	12	30	19	113	9411
10-Oct	64	100	73	308	950	5638	2023	17	10	6	14	12	29	19	109	9378
11-Oct	64	100	73	307	899	5480	1821	17	10	6	14	12	28	18	107	8961
12-Oct	64	100	73	307	854	5326	1821	17	10	6	14	12	27	18	105	8760
13-Oct	64	100	73	307	812	5178	1416	17	10	6	14	12	27	18	104	8164
14-Oct	64	100	73	307	779	5036	2225	17	10	6	14	12	27	18	104	8798
15-Oct	64	100	73	307	757	4899	2225	17	10	6	14	12	27	18	104	8639
16-Oct	64	100	73	307	738	4765	2023	17	10	6	14	12	27	18	104	8284
17-Oct	64	100	73	307	719	4639	2023	17	10	6	14	12	27	18	104	8140
18-Oct	64	100	73	307	702	4517	1618	17	10	6	14	12	27	18	104	7595
19-Oct	64	100	73	307	686	4399	2023	17	10	6	14	12	27	18	104	7866

TABLE 2
LAKE OKEECHOBEE INFLOWS
2% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
01-Sep	594	874	644	2110	3618	5550	6402	151	87	56	123	105	197	128	744	21392
02-Sep	1853	2619	1931	4736	6647	6239	25211	455	261	168	369	316	457	298	1727	53294
03-Sep	2805	3600	2580	17146	19600	9431	112449	630	360	230	510	435	1677	1094	6336	178886
04-Sep	2805	3600	2580	25916	19600	15109	15006	630	360	230	510	435	2243	1460	8468	98955
05-Sep	2805	3600	2580	20296	19600	18864	3001	630	360	230	510	435	1524	991	5760	81187
06-Sep	2805	3600	2580	13735	19600	19479	4401	630	360	230	510	435	987	643	3736	73733
07-Sep	2805	3600	2580	9256	19600	19274	4402	630	360	230	510	435	747	485	2822	67738
08-Sep	2805	3600	2580	6452	19600	19048	4602	630	360	230	510	435	607	397	2293	64151
09-Sep	2805	3600	2580	4889	16446	18837	4401	630	360	230	510	435	521	341	1970	58559
10-Sep	1113	3600	2580	3887	10025	18629	4802	630	360	230	510	435	470	307	1775	49355
11-Sep	704	3600	2580	3362	12131	18440	5402	630	360	230	510	435	422	276	1597	50681
12-Sep	667	1885	1900	3163	9463	18267	5002	284	187	150	236	228	367	239	1383	43426
13-Sep	634	1190	997	3056	9764	18099	4802	195	118	84	160	144	326	215	1238	41031
14-Sep	603	990	758	2979	8408	17915	4802	169	99	66	137	119	297	194	1120	38661
15-Sep	599	943	702	2925	8266	17723	5202	162	94	61	132	114	273	177	1027	38406
16-Sep	598	932	689	2891	7579	17534	4602	161	93	60	130	112	261	170	987	36807
17-Sep	598	929	685	2870	7266	17352	4001	161	92	60	130	112	259	169	980	35673
18-Sep	543	854	630	2758	6727	17161	3601	148	85	55	120	103	248	161	937	34136
19-Sep	481	761	561	2566	6353	16934	2601	131	76	49	106	92	231	150	874	31975
20-Sep	473	740	545	2430	5980	16678	3201	128	74	48	103	89	221	144	835	31696
21-Sep	472	735	541	2347	5688	16433	3201	127	73	47	103	88	215	140	813	31030
22-Sep	472	733	541	2299	5412	16191	3201	127	73	47	103	88	211	137	798	30439
23-Sep	472	733	540	2273	5181	15961	3401	127	73	47	103	88	208	136	788	30137
24-Sep	472	733	540	2258	4982	15738	3401	127	73	47	103	88	206	134	780	29689
25-Sep	472	733	540	2251	4823	15527	4001	127	73	47	103	88	205	133	774	29904

TABLE 2
LAKE OKEECHOBEE INFLOWS
2% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
27-Sep	472	733	540	2248	4569	15123	3401	127	73	47	103	88	203	132	767	28632
28-Sep	472	733	540	2248	4461	14933	3401	127	73	47	103	88	202	132	765	28332
29-Sep	472	733	540	2248	4370	14747	3401	127	73	47	103	88	202	132	765	28054
30-Sep	472	733	540	2248	4296	14568	3201	127	73	47	103	88	202	132	764	27601
01-Oct	431	677	499	2172	4147	14388	3201	117	67	44	95	81	194	126	734	26979
02-Oct	384	608	448	2033	3979	14180	3401	105	60	39	85	73	182	118	689	26391
03-Oct	351	554	408	1883	3804	13945	2601	96	55	36	77	67	170	110	642	24805
04-Oct	319	504	371	1729	3616	13692	2601	87	50	32	70	60	157	102	596	23995
05-Oct	315	492	363	1627	3481	13428	2401	85	49	32	69	59	150	98	567	23221
06-Oct	315	489	361	1566	3359	13179	2401	84	48	31	68	59	145	95	549	22756
07-Oct	315	489	360	1531	3260	12934	2401	84	48	31	68	59	142	92	537	22359
08-Oct	315	489	360	1513	3173	12699	2601	84	48	31	68	59	139	91	528	22205
09-Oct	315	489	360	1504	3101	12472	2401	84	48	31	68	59	138	90	521	21687
10-Oct	315	489	360	1500	3038	12255	2601	84	48	31	68	59	136	89	516	21596
11-Oct	315	489	360	1499	2985	12049	2401	84	48	31	68	59	135	88	513	21131
12-Oct	315	489	360	1498	2937	11849	2401	84	48	31	68	59	135	88	511	20880
13-Oct	315	489	360	1498	2894	11656	2000	84	48	31	68	59	135	88	510	20242
14-Oct	315	489	360	1498	2860	11471	2801	84	48	31	68	59	135	88	510	20822
15-Oct	315	489	360	1498	2837	11293	2801	84	48	31	68	59	135	88	509	20621
16-Oct	315	489	360	1498	2817	11116	2601	84	48	31	68	59	135	88	509	20224
17-Oct	315	489	360	1498	2797	10953	2601	84	48	31	68	59	135	88	509	20042
18-Oct	301	470	346	1473	2745	10790	1800	81	47	30	66	56	132	86	499	18929
19-Oct	285	447	329	1427	2683	10619	2201	77	44	29	62	54	128	83	484	18959

TABLE 2
LAKE OKEECHOBEE INFLOWS
1% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
31-Aug	408	632	466	1910	3434	5863	5186	109	63	41	88	76	172	112	650	19216
01-Sep	615	913	673	2304	3925	6039	6183	158	91	59	128	110	212	138	803	22358
02-Sep	1961	2771	2043	5050	7145	6757	26928	482	277	178	390	334	487	317	1840	56968
03-Sep	2805	3600	2580	19641	19600	10402	130854	630	360	230	510	435	1930	1259	7293	202131
04-Sep	2805	3600	2580	29974	19600	17023	15159	630	360	230	510	435	2596	1690	9800	106995
05-Sep	2805	3600	2580	23251	19600	21367	3590	630	360	230	510	435	1739	1131	6576	88407
06-Sep	2805	3600	2580	15721	19600	22039	5186	630	360	230	510	435	1127	734	4266	79826
07-Sep	2805	3600	2580	10645	19600	21818	5186	630	360	230	510	435	860	559	3250	73069
08-Sep	2805	3600	2580	7399	19600	21578	4986	630	360	230	510	435	696	455	2626	68493
09-Sep	2805	3600	2580	5507	19600	21325	4787	630	360	230	510	435	586	384	2217	65559
10-Sep	2805	3600	2580	4273	19600	21050	5186	630	360	230	510	435	523	342	1974	64100
11-Sep	1982	3600	2580	3640	19600	20799	5784	630	360	230	510	435	467	305	1768	62694
12-Sep	880	3600	2580	3422	14790	20577	5385	630	360	230	510	435	404	264	1524	55595
13-Sep	728	3600	2580	3344	8513	20375	5385	614	360	230	503	435	362	238	1374	48648
14-Sep	696	1870	2003	3324	10936	20170	5385	284	186	155	232	226	334	218	1259	47285
15-Sep	692	1260	1075	3311	8483	19967	5784	208	125	90	169	152	310	200	1162	42995
16-Sep	691	1117	857	3302	9097	19767	5186	191	111	74	155	135	297	194	1124	42303
17-Sep	691	1083	806	3296	7948	19575	4587	187	108	70	151	130	296	193	1120	40251
18-Sep	595	944	698	3116	7682	19361	3391	163	94	61	132	114	278	181	1050	37866
19-Sep	486	781	576	2792	6779	19079	2393	135	78	50	109	94	250	163	944	34716
20-Sep	473	743	548	2559	6490	18744	2992	128	74	48	104	89	233	152	880	34263
21-Sep	471	734	541	2415	6056	18421	2992	127	73	47	103	88	223	145	844	33287
22-Sep	471	732	539	2331	5777	18103	2992	126	73	47	102	88	217	141	819	32566
23-Sep	471	731	539	2285	5467	17800	3191	126	73	47	102	88	212	138	802	32081
24-Sep	471	731	539	2260	5241	17507	3191	126	73	47	102	88	209	136	789	31518
25-Sep	471	731	539	2248	5031	17229	3790	126	73	47	102	88	206	134	779	31601

TABLE 2
LAKE OKEECHOBEE INFLOWS
1% FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
26-Sep	471	731	539	2243	4867	16956	3191	126	73	47	102	88	204	133	772	30550
27-Sep	471	731	539	2242	4713	16698	3191	126	73	47	102	88	203	132	767	30131
28-Sep	471	731	539	2242	4579	16448	3191	126	73	47	102	88	202	132	764	29742
29-Sep	471	731	539	2242	4462	16204	3191	126	73	47	102	88	202	131	763	29380
30-Sep	471	731	539	2242	4368	15969	2992	126	73	47	102	88	202	131	762	28850
01-Oct	430	675	498	2166	4201	15736	2992	117	67	43	94	81	194	126	733	28159
02-Oct	383	606	447	2028	4019	15477	3191	105	60	39	85	73	182	118	687	27506
03-Oct	350	552	407	1879	3834	15193	2593	95	55	36	77	66	169	110	640	26063
04-Oct	318	503	370	1725	3639	14892	2593	87	50	32	70	60	157	102	594	25199
05-Oct	314	491	362	1623	3497	14582	2393	85	49	31	69	59	149	97	566	24374
06-Oct	314	488	360	1562	3371	14289	2393	84	48	31	68	59	145	94	548	23861
07-Oct	314	487	359	1528	3268	14002	2393	84	48	31	68	58	142	92	536	23418
08-Oct	314	487	359	1509	3178	13725	2593	84	48	31	68	58	139	91	527	23219
09-Oct	314	487	359	1500	3103	13459	2393	84	48	31	68	58	137	89	520	22660
10-Oct	314	487	359	1496	3038	13204	2593	84	48	31	68	58	136	89	515	22528
11-Oct	314	487	359	1495	2983	12963	2393	84	48	31	68	58	135	88	512	22026
12-Oct	314	487	359	1495	2934	12727	2393	84	48	31	68	58	135	88	510	21738
13-Oct	314	487	359	1495	2890	12500	1994	84	48	31	68	58	134	87	509	21067
14-Oct	314	487	359	1495	2855	12283	2792	84	48	31	68	58	134	87	508	21612
15-Oct	314	487	359	1495	2831	12074	2792	84	48	31	68	58	134	87	508	21379
16-Oct	314	487	359	1495	2811	11866	2593	84	48	31	68	58	134	87	508	20951
17-Oct	314	487	359	1495	2791	11676	2593	84	48	31	68	58	134	87	508	20741
18-Oct	314	487	359	1495	2770	11490	1994	84	48	31	68	58	134	87	508	19936
19-Oct	314	487	359	1495	2751	11308	2393	84	48	31	68	58	134	87	508	20134

TABLE 2
LAKE OKEECHOBEE INFLOWS
STANDARD PROJECT FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
31-Aug	688	1065	785	3210	5706	9916	7413	184	106	69	149	128	289	189	1094	30999
01-Sep	935	1402	1034	3692	6311	10162	8815	243	140	90	197	169	338	221	1279	35035
02-Sep	2678	3600	2580	7227	10477	11003	34662	630	360	230	510	435	693	452	2618	78157
03-Sep	2805	3600	2580	26453	19600	15211	165095	630	360	230	510	435	2596	1693	9809	251609
04-Sep	2805	3600	2580	40102	19600	23659	19835	630	360	230	510	435	3477	2263	13124	133212
05-Sep	2805	3600	2580	31213	19600	24000	5209	630	360	230	510	435	2344	1524	8862	103904
06-Sep	2805	3600	2580	21229	19600	24000	7413	630	360	230	510	435	1532	998	5795	91717
07-Sep	2805	3600	2580	14470	19600	24000	7413	630	360	230	510	435	1173	763	4434	83005
08-Sep	2805	3600	2580	10170	19600	24000	7012	630	360	230	510	435	956	625	3608	77124
09-Sep	2805	3600	2580	7694	19600	24000	6812	630	360	230	510	435	814	533	3077	73681
10-Sep	2805	3600	2580	6083	19600	24000	7212	630	360	230	510	435	731	478	2760	72017
11-Sep	2805	3600	2580	5244	19600	24000	8014	630	360	230	510	435	657	429	2483	71578
12-Sep	2805	3600	2580	4933	19600	24000	7613	630	360	230	510	435	571	372	2153	70394
13-Sep	2805	3600	2580	4822	19600	24000	7413	630	360	230	510	435	515	339	1953	69793
14-Sep	2805	3600	2580	4800	19600	24000	7413	630	360	230	510	435	478	312	1803	69558
15-Sep	2805	3600	2580	4787	19600	24000	7814	630	360	230	510	435	447	289	1677	69764
16-Sep	1808	3600	2580	4777	19600	24000	7212	630	360	230	510	435	431	280	1626	68083
17-Sep	1102	3600	2580	4772	19600	24000	6611	630	360	230	510	435	429	280	1622	66764
18-Sep	877	3600	2580	4517	19600	24000	5008	630	360	230	510	435	403	263	1523	64539
19-Sep	712	3600	2580	4057	19600	24000	4007	630	360	230	510	435	363	236	1371	62693
20-Sep	691	3346	2580	3726	19600	24000	4608	497	333	230	414	405	339	221	1281	62276
21-Sep	689	1603	2117	3521	13978	24000	4608	257	160	149	211	193	325	212	1229	53257
22-Sep	688	1194	1099	3402	4657	24000	4608	201	119	88	164	144	316	206	1195	42088
23-Sep	688	1098	861	3337	9461	24000	4808	189	109	74	153	132	310	202	1170	46598
24-Sep	688	1075	805	3300	6097	24000	4808	186	107	70	150	130	305	199	1152	43079
25-Sep	688	1070	792	3284	7724	24000	5409	185	107	69	150	129	301	196	1138	45247

TABLE 2
LAKE OKEECHOBEE INFLOWS
STANDARD PROJECT FLOOD EVENT
DISCHARGES IN CFS

DATE	S4	S2	S3	FISHEATING CREEK	ISTOKPOGA AREA	KISSIMMEE RIVER	RAINFALL EXCESS OVER LAKE OKEECHOBEE	S127	S129	S131	S133	S135	S153	S154	S191	TOTAL INFLOW
26-Sep	688	1069	788	3277	6477	24000	4808	185	106	69	150	129	298	194	1127	43372
27-Sep	688	1068	788	3275	6950	24000	4808	185	106	69	150	129	296	193	1121	43833
28-Sep	688	1068	788	3275	6422	24000	4808	185	106	69	150	129	295	192	1117	43299
29-Sep	688	1068	787	3275	6509	24000	4808	185	106	69	150	129	295	192	1115	43383
30-Sep	688	1068	787	3275	6267	24000	4608	185	106	69	150	129	295	192	1114	42939
01-Oct	633	994	733	3175	6133	24000	4608	172	99	64	139	120	284	185	1074	42419
02-Oct	572	902	665	2991	5852	24000	4808	156	90	58	126	109	268	175	1014	41792
03-Oct	537	843	622	2809	5657	24000	4007	146	84	54	118	101	253	165	958	40361
04-Oct	505	792	584	2636	5412	24000	4007	137	79	51	111	95	240	156	907	39719
05-Oct	501	780	575	2522	5252	24000	3806	135	78	50	109	94	231	151	875	39168
06-Oct	501	778	573	2456	5095	24000	3806	134	77	50	109	94	226	147	855	38908
07-Oct	500	777	573	2417	4975	24000	3806	134	77	50	109	93	223	145	841	38728
08-Oct	500	777	573	2398	4866	24000	4007	134	77	50	109	93	220	143	831	38784
09-Oct	500	777	573	2388	4778	24000	3806	134	77	50	109	93	218	142	823	38474
10-Oct	500	777	573	2384	4699	24000	4007	134	77	50	109	93	216	141	817	38584
11-Oct	500	777	573	2382	4634	24000	3806	134	77	50	109	93	215	140	814	38311
12-Oct	500	777	573	2382	4576	24000	3806	134	77	50	109	93	215	140	812	38250
13-Oct	500	777	573	2382	4524	24000	3406	134	77	50	109	93	214	140	811	37796
14-Oct	500	777	573	2382	4482	24000	4207	134	77	50	109	93	214	139	810	38555
15-Oct	500	777	573	2382	4454	24000	4207	134	77	50	109	93	214	139	810	38526
16-Oct	500	777	573	2382	4429	24000	4007	134	77	50	109	93	214	139	810	38301
17-Oct	500	777	573	2382	4405	24000	4007	134	77	50	109	93	214	139	810	38277
18-Oct	487	758	559	2357	4351	24000	3205	131	75	49	106	91	212	138	800	37325
19-Oct	471	735	542	2311	4286	24000	3606	127	73	47	103	88	208	135	785	37524

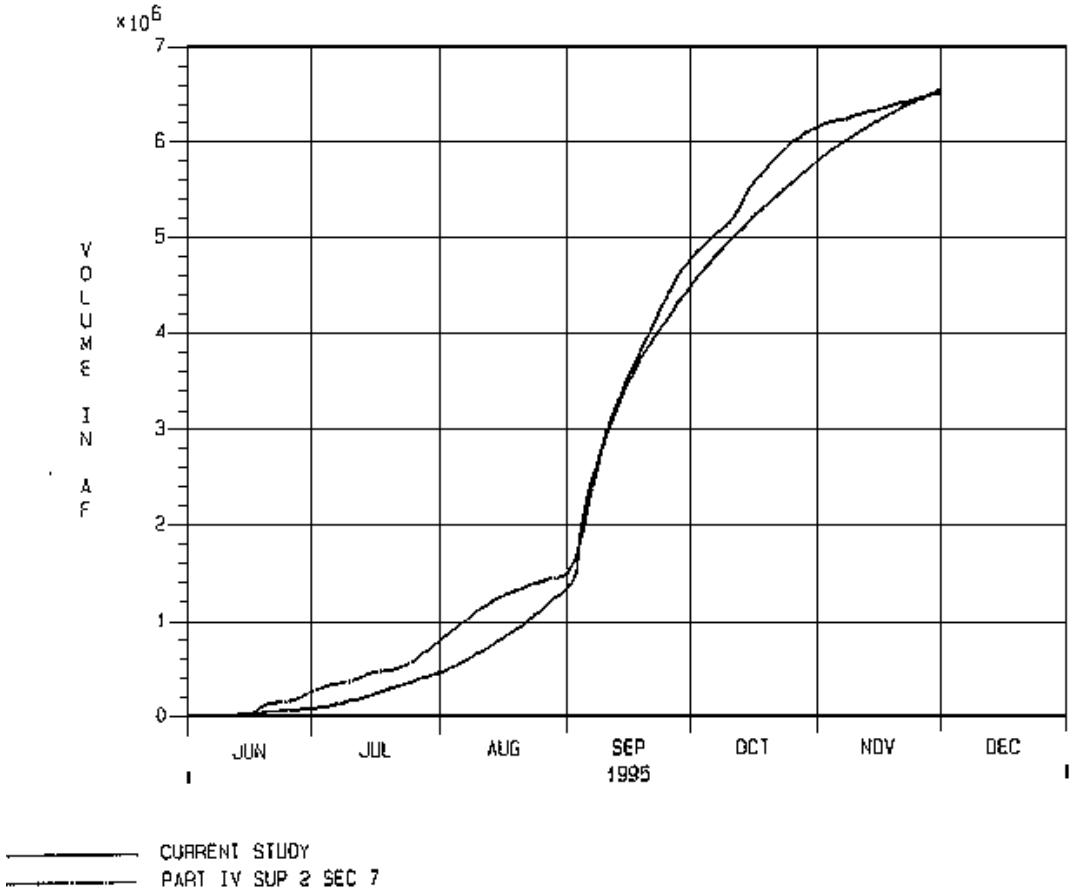


Figure 2. Lake Okeechobee 1% Flood Inflow

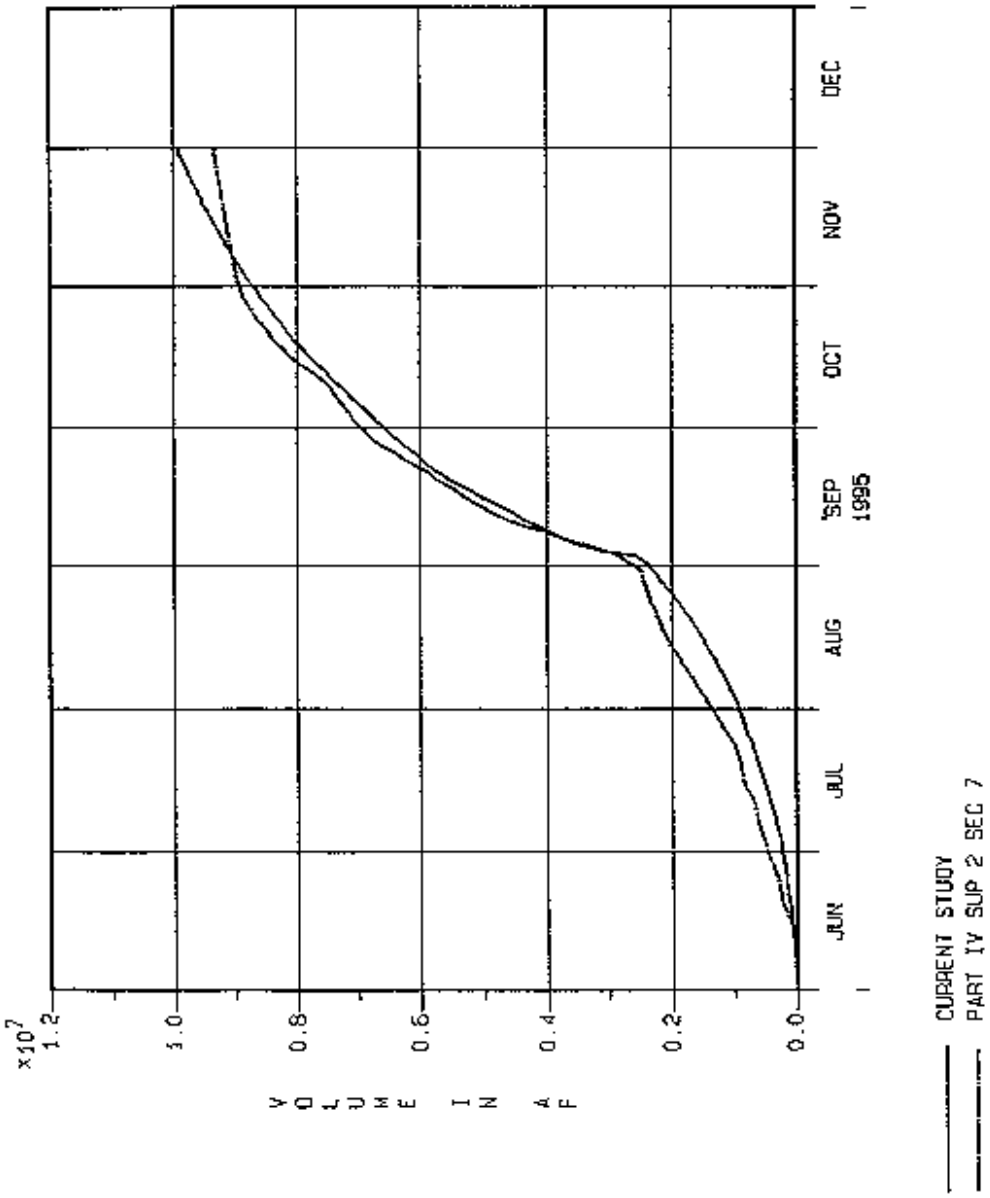
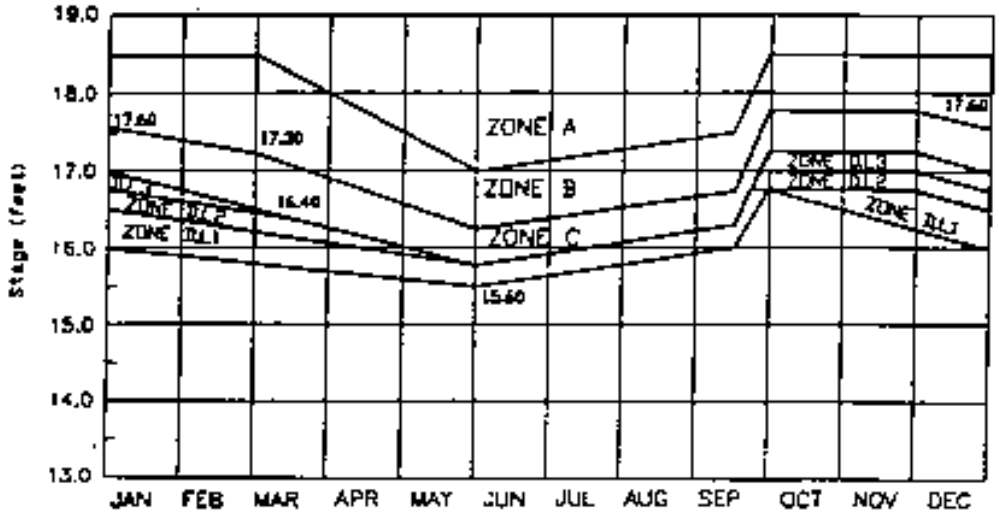


Figure 3. Lake Okeechobee SPF Inflow

an earlier flood routing effort conducted in 1959 (USACE, 1959).

Routings - The current regulation schedule, RUN25, and stage-storage data were incorporated in the routing analysis. The RUN25 schedule, depicted in Figure 3a, ranges from 15.65 feet to 16.75 feet, with multiple operation zones that vary flood releases over a wide range before maximum release rates are attained. For a detailed description of the operational strategy and its components, the reader is directed to the Master Water Control Manual, Lake Okeechobee and Everglades Agricultural Area, Volume 3, June 1996. RUN25 was put into effect in May 1992, in an effort to reduce damaging outflows through the St. Lucie Canal and Caloosahatchee River, without sacrificing the lake's flood control or water supply benefits. It should also be noted that a regulation schedule review study is in progress. The purpose of that study is to determine if a more ecologically beneficial schedule exists which simultaneously ensures that the water-related needs of central and southern Florida continue to be met. A likely candidate for such an alternative regulation schedule is RUN22D, depicted in Figure 3b. It is a lower regulation schedule when compared to RUN25; therefore, its implementation would provide some relief (although slight) to seepage and stability problems along the dike system. A stage-duration curve computed for the winter season, as shown in Figure 4, was used as the initial condition for the routings. The stage-duration curve was considered as a step-wise function with six elevations. Routings were performed with a one-day time step.

Table 3a. RUN25 lake regulation schedule

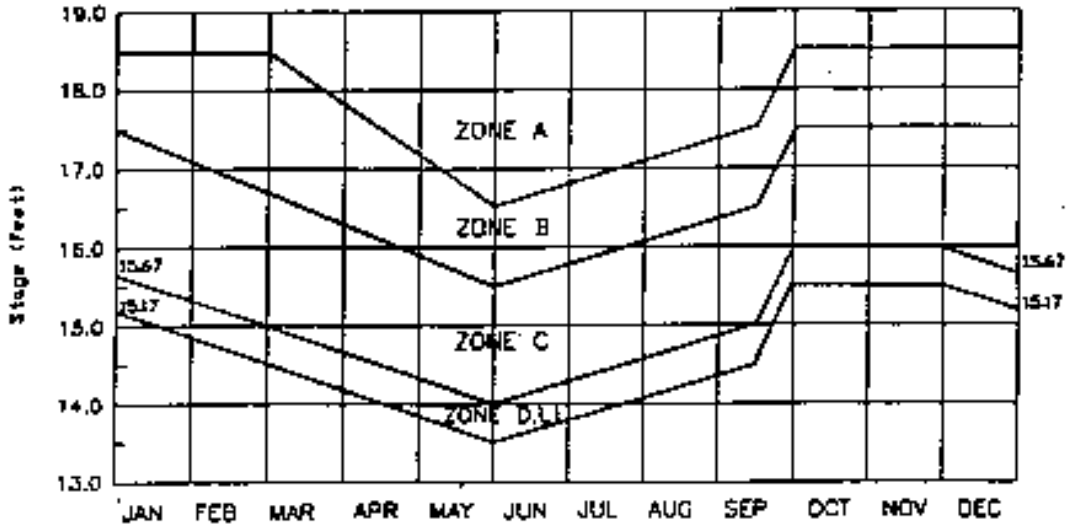


Lake Okeechobee Releases

as used by the South Florida Regional Routing Model

Zone	Agricultural Canals	Caloosahatchee River at S-77	St. Lucie Estuary at S-80 including runoff from C-44 Basin
A	Maximum Practicable Releases to Water Conservation Areas	Up to Maximum Capacity at S-77	Up to Maximum Capacity at S-80
B		6500 CFS	3500 CFS
C		4500 CFS	2500 CFS
D LEVEL 1 LEVEL 2 LEVEL 3		10-day Pulse with a Mean Discharge of: Level 3 = 3000 CFS Level 2 = 2300 CFS Level 1 = 1600 CFS	10-day Pulse with a Mean Discharge of: Level 3 = 1170 CFS Level 2 = 900 CFS Level 1 = 730 CFS

Figure 3b. Proposed RUN22D lake regulation schedule



Lake Okeechobee Releases
as used by the South Florida Regional Routing Model

Zone	Agricultural Canals	Caloosahatchee River at S-77	St. Lucie Estuary at S-80 including runoff from C-44 Basin
A	Maximum Practicable Releases to Water Conservation Areas	Up to Maximum Capacity at S-77	Up to Maximum Capacity at S-80
B		6500 CFS	3500 CFS
C		4500 CFS	2500 CFS
D,L1		10-day Pulse with a Mean Discharge of 1600 CFS	10-day Pulse with a Mean Discharge of 730 CFS

Rainfall - Rainfall data for the period 1931-1993 was provided by the seven rain gages identified in Figure 1. Rainfall data recorded at the Kissimmee and Kissimmee-2 gages were combined to obtain a continuous period of record. Theissen weights were computed for each gage as listed in Table 3. Basin average rainfall values were computed using recorded rainfall at the seven gages and corresponding Theissen weights. Annual maximum basin average rainfall series were computed for durations ranging from 1 day through 364 days. The Log-Pearson Type III method, as incorporated in the HEC flood frequency analysis model, was used to analyze the annual maximum rainfall series. The results provided estimates of statistical rainfall parameters for occurrence frequencies ranging from 50 percent to 1 percent chance of exceedance. The computed rainfalls listed in Table 3 were graphically smoothed to obtain the basin average rainfall for various distributions and frequencies. SPF rainfall values for all durations were established as 125 percent of the 1 percent chance of exceedance storm for the corresponding duration. Balanced storms were used to distribute the completed rainfall over the year-long period, with the highest rainfall occurring in September.

Total Losses – The relationships between monthly rainfall and monthly losses previously developed in the 1959 analysis were adopted for analysis of the 2 percent, 1 percent, and SPF events. A runoff curve number of 56 was estimated from the monthly rainfall and runoff for such rare events with wet moisture conditions. A runoff curve

Figure 4. Lake Okeechobee Stage Duration

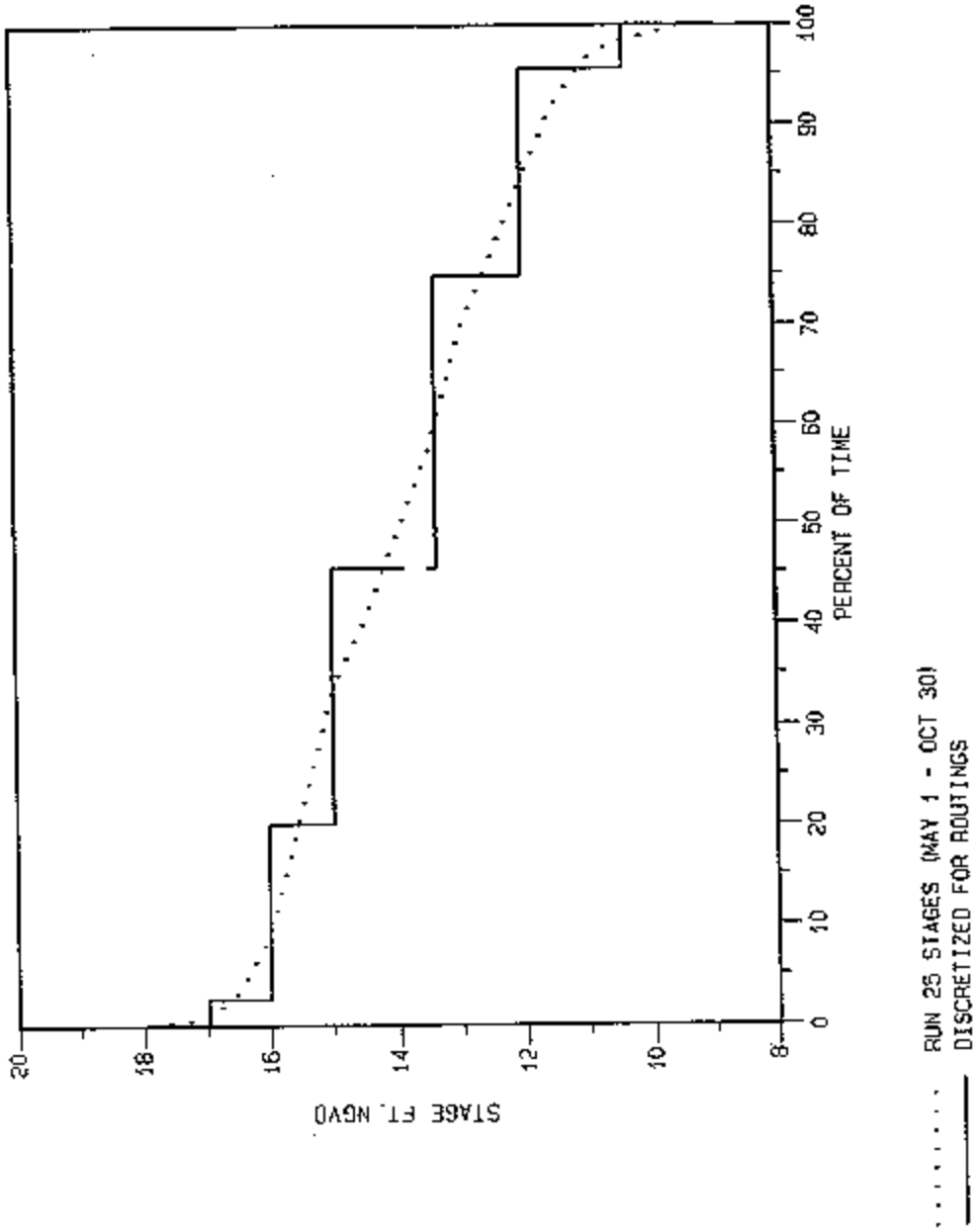


TABLE 3
LAKE OKEECHOBEE BASIN AVERAGE RAINFALL
(RAW DATA PRIOR TO ADJUSTMENTS)

STATION ID	STATION NAME	THEISSEN WEIGHT
369	AVON PARK 2 W	0.239
611	BELLE GLADE EXP STN	0.201
3186	FORT MYERS FAA/AP	0.083
3207	FORT PIERCE	0.046
4620	KISSIMMEE & 2	0.158
4707	LAKE ALFRED EXP STN	0.028
5895	MOORE HAVEN LOCK 1	0.245

DURATION DAYS	NUMBER OF YEARS	HIGH OUTLIERS	LOW OUTLIERS	MEAN	STD DEV	SKEW	RAINFALL IN INCHES PERCENT CHANCE EXCEEDANCE					
							50	20	10	5	2	1
1	63	0	0	0.3178	0.1653	0.6	2.00	2.82	3.45	4.12	5.10	5.93
2	63	0	0	0.4658	0.1649	0.4	2.85	3.99	4.82	5.68	6.90	7.89
3	63	0	0	0.5458	0.1581	0.3	3.45	4.74	5.66	6.59	7.86	8.87
4	63	0	0	0.5969	0.1468	0.3	3.89	5.22	6.15	7.08	8.34	9.34
5	63	0	0	0.6317	0.1371	0.4	4.19	5.54	6.49	7.44	8.75	9.78
10	63	0	0	0.7602	0.1193	0.4	5.65	7.21	8.27	9.31	10.70	11.80
15	63	0	0	0.8590	0.1109	0.2	7.17	8.94	10.10	11.20	12.50	13.60
20	63	0	0	0.9363	0.1057	0.1	8.60	10.60	11.80	13.00	14.40	15.50
30	63	0	0	1.0519	0.1045	0.3	11.10	13.70	15.40	17.10	19.20	20.80
60	63	0	0	1.2584	0.0829	0.2	18.00	21.20	23.20	25.10	27.40	29.10
90	63	0	0	1.3928	0.0782	0.2	24.60	28.70	31.20	33.60	36.40	38.60
120	63	0	0	1.4887	0.0762	-0.1	30.90	35.70	38.50	40.90	43.80	45.70
150	63	0	0	1.5461	0.0741	0.0	35.20	40.60	43.80	46.60	49.90	52.30
180	63	0	0	1.5828	0.0715	0.1	38.20	43.90	47.30	50.40	54.10	56.80
210	63	0	0	1.6140	0.0698	0.0	41.10	47.10	50.50	53.60	57.20	59.80
240	63	0	0	1.6412	0.0701	0.3	43.40	50.00	54.10	57.80	62.50	66.00
270	63	0	0	1.6646	0.0700	0.3	45.80	52.80	57.00	61.00	66.00	69.60
300	63	0	0	1.6805	0.0687	0.3	47.50	54.60	58.90	63.00	68.00	71.60
330	63	0	0	1.6913	0.0681	0.4	48.60	55.80	60.40	64.60	70.00	74.00
364	63	0	0	1.7003	0.0685	0.3	49.80	57.10	61.70	65.80	71.10	74.90

Table 3

number of 36, assuming average moisture conditions, was estimated for other more frequent flood events. Daily total losses were obtained by uniformly distributing monthly losses for all months except September. An evapotranspiration estimate of 3.8 inches was uniformly distributed in September. The difference between the total losses and evapotranspiration losses for September was distributed daily, proportional to the rainfall excess over the evapotranspiration. Monthly values of rainfall, total loss, and rainfall excess are provided in Table 4. Figures 5 and 6 depict the rainfall and rainfall excess over a period of one year for the 1 percent chance of exceedance event and the SPF event, respectively.

Unit Hydrographs – The discharge hydrograph for the 1 percent chance of exceedance flood event, obtained from the Kissimmee River 1135 Project, was optimized by the HEC-1 model to determine a unit hydrograph at S-65D for a drainage area of 2,300 square miles. A unit hydrograph at the same location was also optimized from the SPF for comparison. The unit hydrograph was extrapolated to the mouth for a drainage area of 2,456 square miles. The unit hydrographs for the Lake Istokpoga area, Fisheating Creek, and Taylor Creek were estimated based on the Lake Istokpoga Feasibility Study (USACE 1993). Instantaneous unit hydrographs were used for the lake and other areas serviced by pump stations.

Stage-Frequency Relationships - Figure 8 presents the Lake Okeechobee stage hydrographs for the 1 percent chance of exceedance flood, for six different initial lake levels. Figure 9 provides comparable information for the SPF event. Figure 10 provides the stage hydrographs for flood exceedance frequencies ranging from 50 percent to SPF, for an initial lake level of 17.0 feet. Tables 5 (eight pages) and 6 (eight pages) provide, for an initial lake stage of 17.0 feet, the results of daily routings for the 1 percent and SPF floods, respectively. Peak stage-frequency data were plotted for all six initial lake levels. Table 8 includes the durations for six initial lake levels, stage-frequency data for all initial conditions, and computed combined frequencies. Figure 11 provides the stage frequency data obtained from flood routings and plotted positions of observed stages that have been recorded since 1912. The adopted stage-frequency curve for Lake Okeechobee is also depicted in Figure A-11.

TABLE 4
LAKE OKEECHOBEE
RAINFALL, TOTAL LOSS, AND RAINFALL EXCESS IN INCHES

		PERCENT CHANCE STORM EVENTS						
	MONTH	50	20	10	5	2	1	SPF
RAINFALL	1	0.64	0.64	0.64	0.64	0.64	0.64	0.80
	2	1.02	1.11	1.35	1.63	1.63	1.84	2.30
	3	1.64	1.74	1.88	2.05	2.30	2.69	3.36
	4	2.28	2.63	2.73	3.00	3.30	3.20	4.00
	5	2.61	3.09	3.39	3.54	4.05	4.52	5.65
	6	3.46	3.87	4.19	4.68	5.07	5.36	6.70
	7	6.36	7.15	7.56	7.45	7.87	7.98	9.98
	8	7.64	8.67	9.23	10.13	10.58	10.92	13.65
	9	10.56	12.78	14.21	16.03	17.63	18.98	23.73
	10	6.70	7.52	7.97	7.81	8.29	8.37	10.46
	11	3.76	4.22	4.53	5.05	5.44	5.64	7.05
	12	2.70	3.16	3.52	3.64	4.15	4.77	5.96
	TOTAL	49.38	56.59	61.19	65.64	70.94	74.92	93.65
TOTAL LOSS	1	0.64	0.64	0.64	0.64	0.64	0.64	0.80
	2	1.02	1.11	1.35	1.63	1.63	1.84	2.20
	3	1.64	1.74	1.88	2.05	2.30	2.65	3.10
	4	2.28	2.63	2.73	3.00	3.30	3.20	3.75
	5	2.61	3.09	3.39	3.54	4.00	4.30	5.00
	6	3.46	3.87	4.19	4.60	4.80	5.00	5.70
	7	5.70	6.00	6.30	6.25	6.40	6.50	7.30
	8	6.87	7.52	7.87	8.37	7.50	7.60	8.00
	9	8.56	9.68	10.21	10.91	6.90	7.00	7.00
	10	6.23	6.82	7.08	6.99	5.20	5.20	5.60
	11	3.76	4.20	4.47	4.93	3.80	3.85	4.10
	12	2.25	2.60	2.75	2.85	3.10	3.50	3.80
	TOTAL	45.02	49.90	52.86	55.75	49.56	51.28	56.35
RAINFALL EXCESS	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2	0.00	0.00	0.00	0.00	0.00	0.00	0.10
	3	0.00	0.00	0.00	0.00	0.00	0.04	0.26
	4	0.00	0.00	0.00	0.00	0.00	0.00	0.25
	5	0.00	0.00	0.00	0.00	0.05	0.22	0.65
	6	0.00	0.00	0.00	0.08	0.27	0.36	1.00
	7	0.66	1.15	1.26	1.20	1.47	1.48	2.68
	8	0.77	1.15	1.36	1.76	3.08	3.32	5.65
	9	2.00	3.10	4.00	5.12	10.73	11.98	16.73
	10	0.47	0.70	0.89	0.82	3.09	3.17	4.86
	11	0.00	0.02	0.06	0.12	1.64	1.79	2.95
	12	0.45	0.56	0.77	0.79	1.05	1.27	2.16
	TOTAL	4.36	6.69	8.33	9.89	21.38	23.64	37.30

Figure 5 Basin Average 1% Rainfall

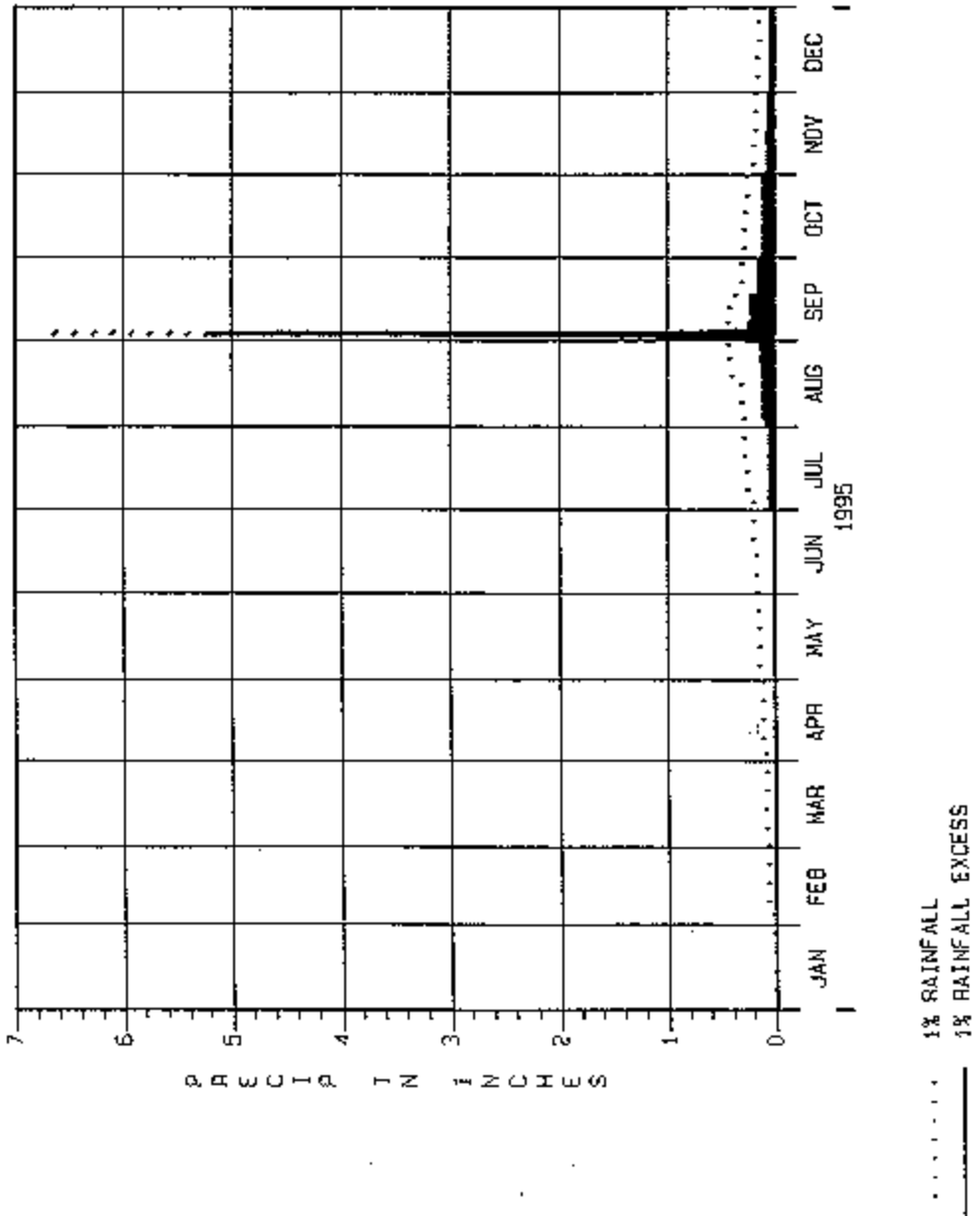


Figure 6 Basin Average SPF Rainfall

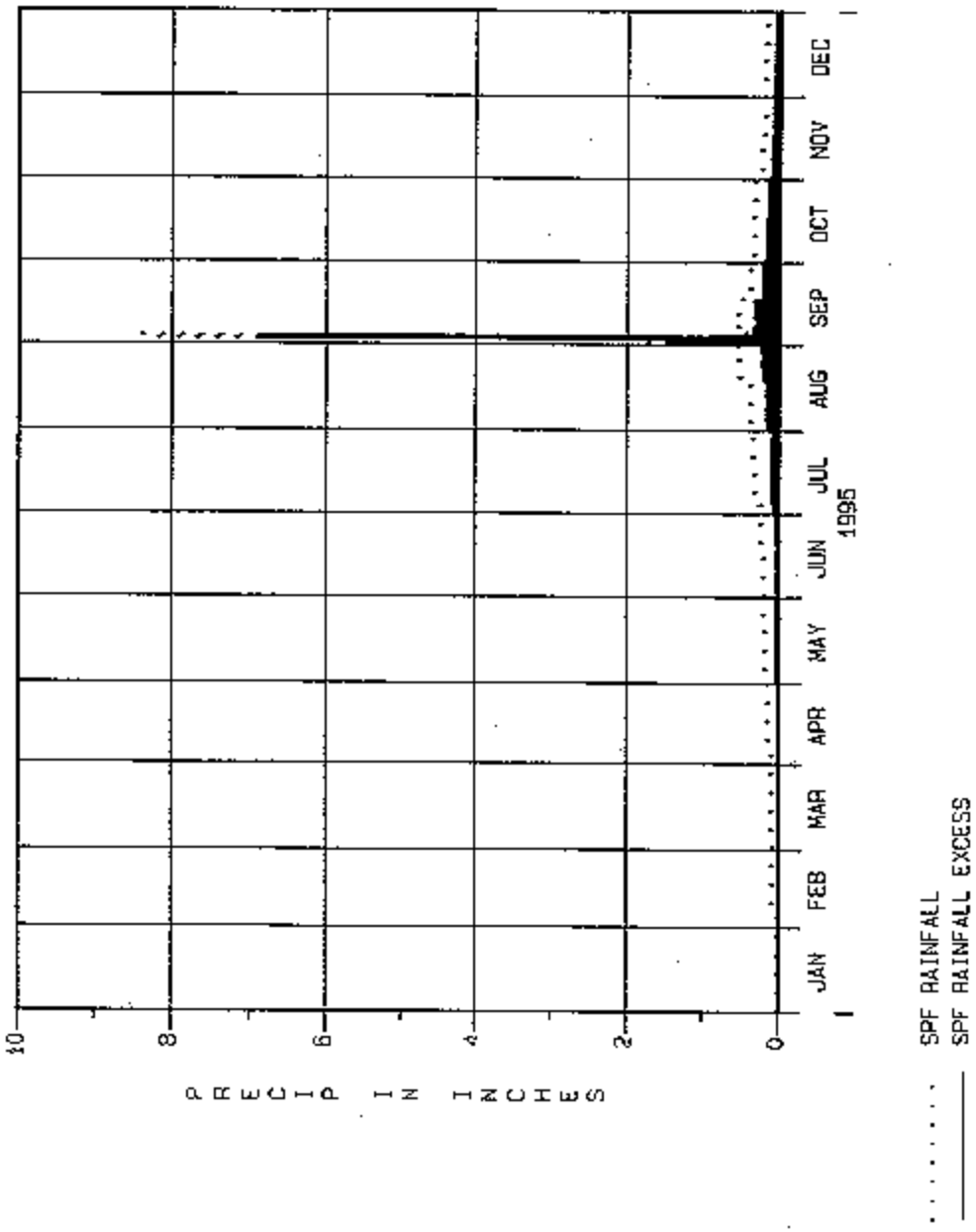


Figure 7 Lake Okeechobee 1% Flood Hydrograph

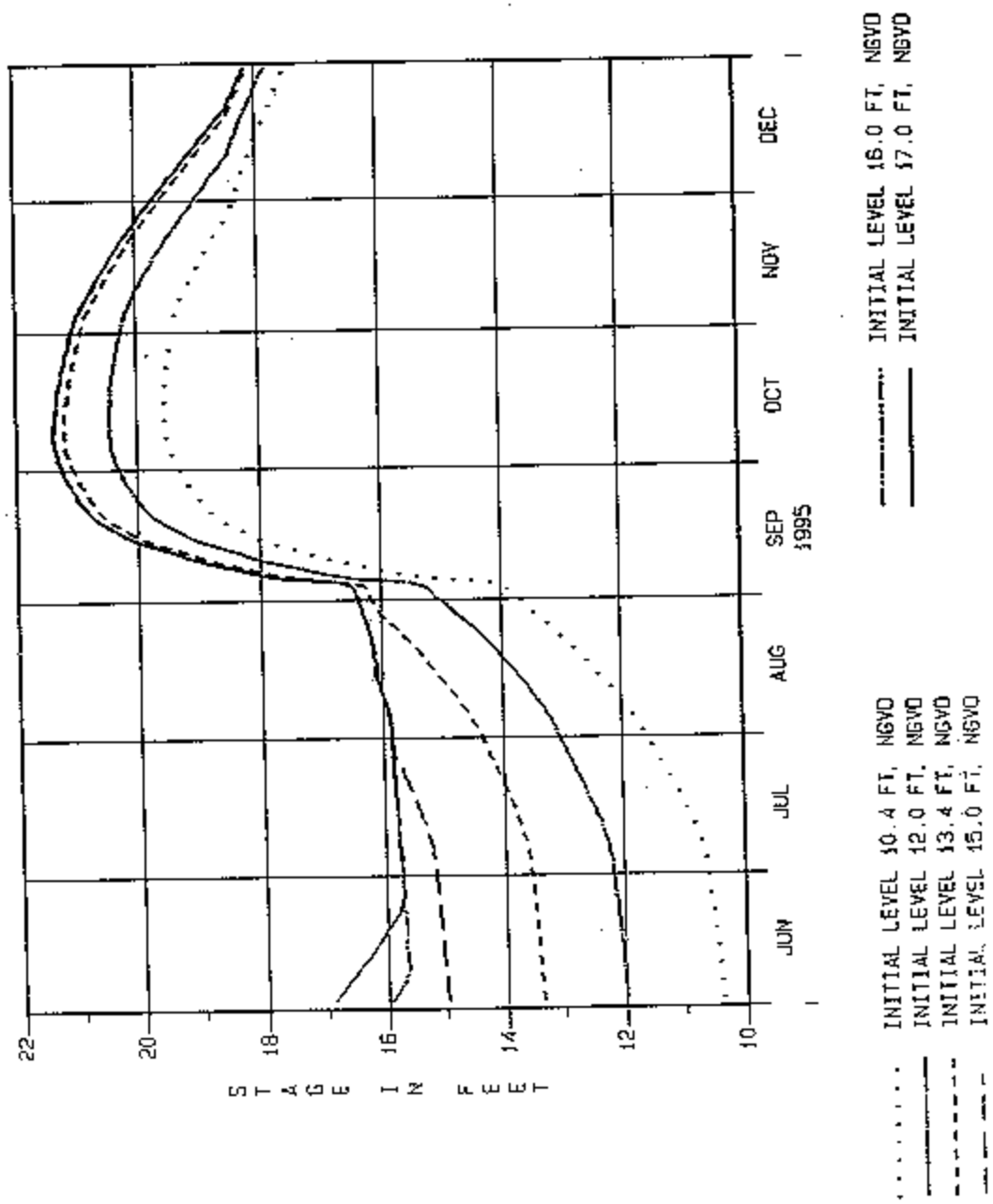


Figure 8 Lake Okeechobee SPF Hydrograph

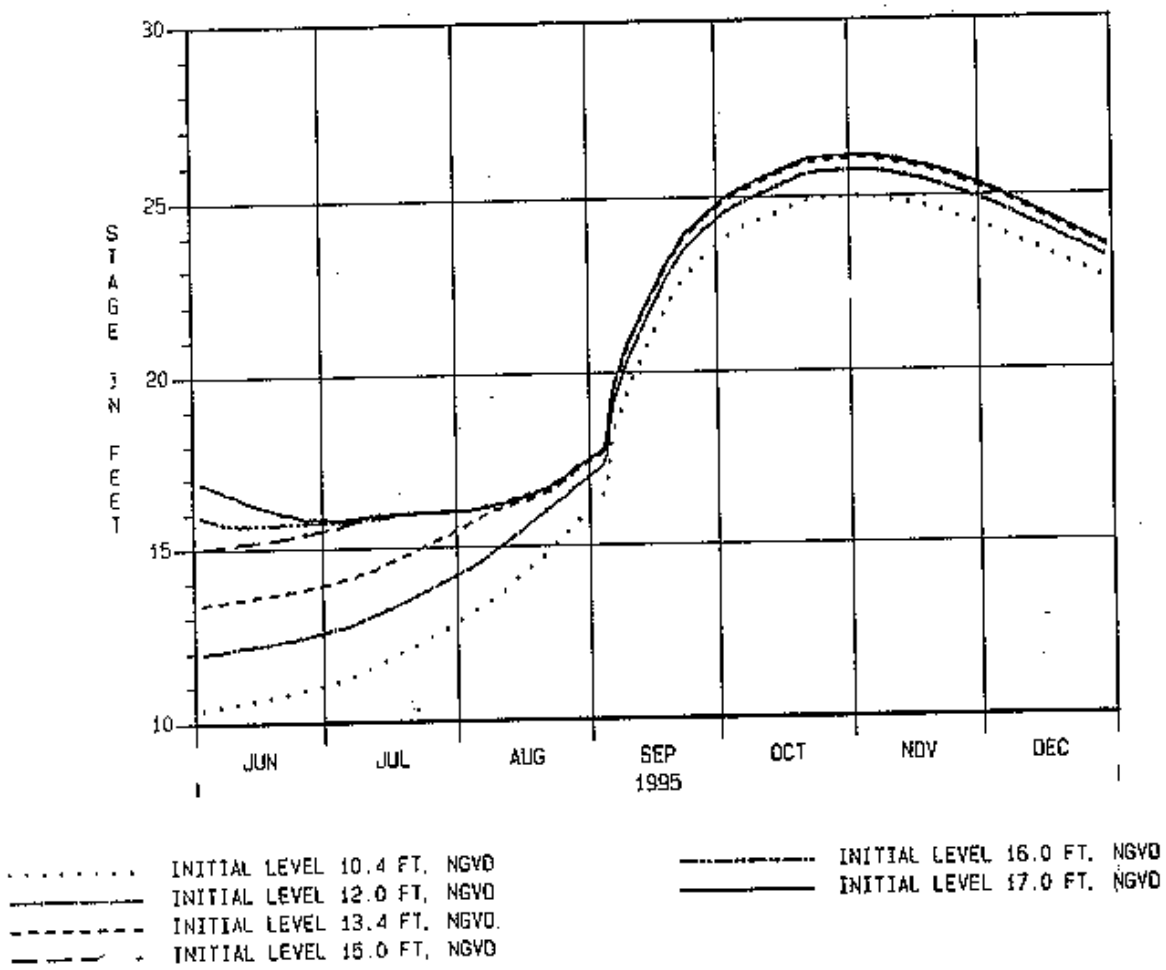


Figure 9. Lake Okeechobee Flood Hydrographs (Initial Level 17 Ft.)

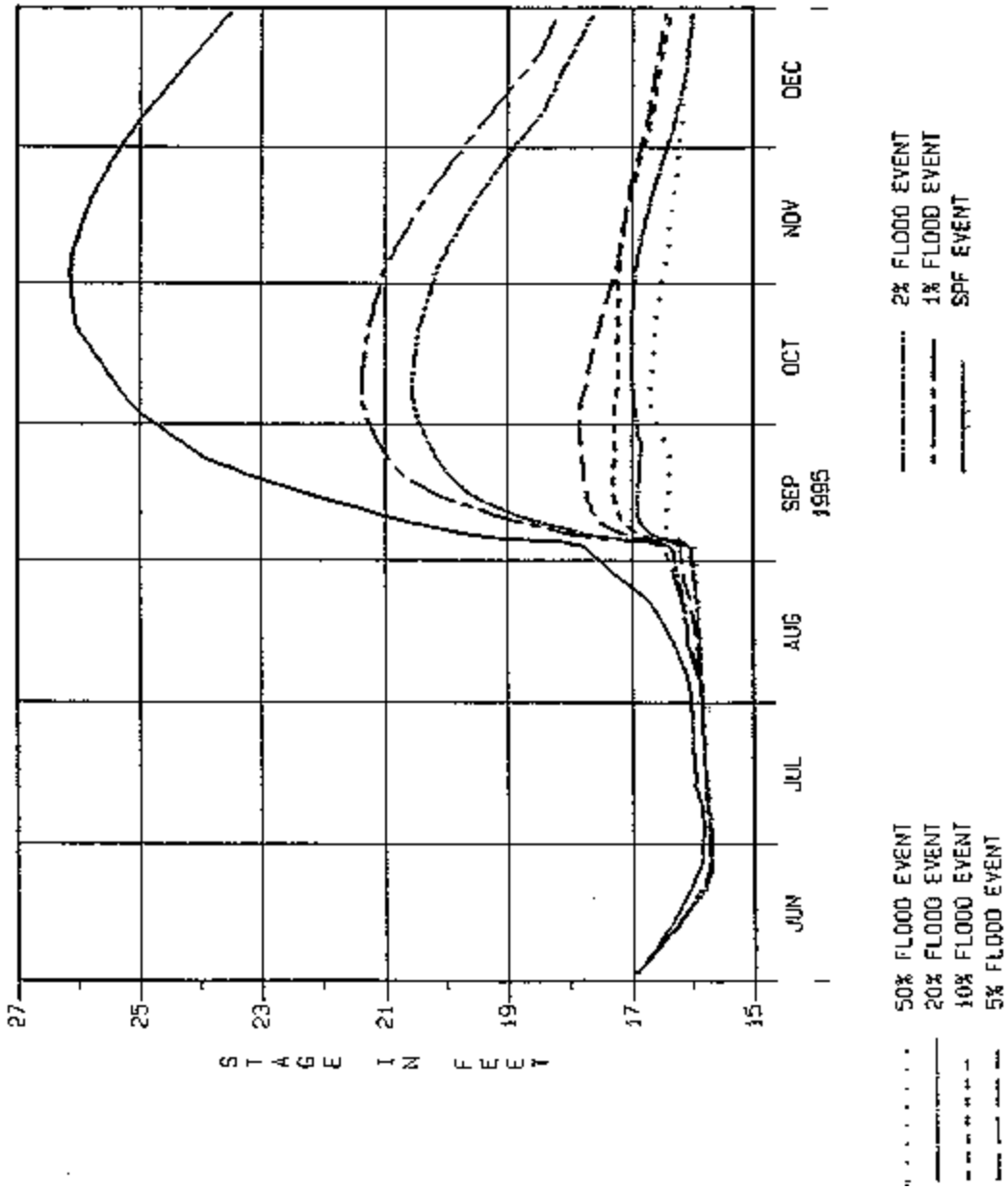


TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
31-May	16.94	B	1077	6500	283	6500	3500	66	3500	5300	49	5300	-28211	4801789
01-Jun	16.87	B	1085	6500	283	6500	3500	66	3500	5300	49	5300	-28195	4773593
02-Jun	16.81	B	1093	6500	283	6500	3500	66	3500	5300	49	5300	-28180	4745413
03-Jun	16.75	B	1100	6500	283	6500	3500	66	3500	5300	49	5300	-28165	4717248
04-Jun	16.69	B	1108	6500	283	6500	3500	66	3500	5300	49	5300	-28151	4689097
05-Jun	16.62	B	1115	6500	283	6500	3500	66	3500	5300	49	5300	-28137	4660960
06-Jun	16.56	B	1121	6500	283	6500	3500	66	3500	5300	49	5300	-28124	4632836
07-Jun	16.50	B	1128	6500	283	6500	3500	66	3500	5300	49	5300	-28111	4604725
08-Jun	16.44	B	1134	6500	283	6500	3500	66	3500	5300	49	5300	-28099	4576627
09-Jun	16.37	B	1140	6500	283	6500	3500	66	3500	5300	49	5300	-28087	4548539
10-Jun	16.31	B	1145	6500	283	6500	3500	66	3500	5300	49	5300	-28076	4520463
11-Jun	16.25	B	1151	6500	283	6500	3500	66	3500	5300	49	5300	-28065	4492398
12-Jun	16.20	C	1156	4500	283	4500	2500	66	2500	5300	49	5300	-22104	4470294
13-Jun	16.15	C	1161	4500	283	4500	2500	66	2500	5300	49	5300	-22094	4448200
14-Jun	16.10	C	1166	4500	283	4500	2500	66	2500	5300	49	5300	-22084	4426116
15-Jun	16.05	C	1171	4500	283	4500	2500	66	2500	5300	49	5300	-22075	4404040
16-Jun	16.00	C	1175	4500	283	4500	2500	66	2500	5300	49	5300	-22066	4381975
17-Jun	15.95	C	1180	4500	283	4500	2500	66	2500	5300	49	5300	-22057	4359917
18-Jun	15.90	C	1184	4500	283	4500	2500	66	2500	5300	49	5300	-22049	4337868
19-Jun	15.85	C	1188	4500	283	4500	2500	66	2500	5300	49	5300	-22041	4315827
20-Jun	15.80	C	1794	4500	283	4500	2500	66	2500	5300	49	5300	-20839	4294988
21-Jun	15.78	D	1605	767	283	767	317	66	317	5300	49	5300	-9480	4285508
22-Jun	15.76	D	1412	767	283	767	317	66	317	5300	49	5300	-9863	4275645
23-Jun	15.73	D	1415	767	283	767	317	66	317	5300	49	5300	-9857	4265788
24-Jun	15.71	D	1819	767	283	767	317	66	317	5300	49	5300	-9054	4256734
25-Jun	15.72	E	1426	0	283	0	0	66	0	0	49	0	2828	4259562
26-Jun	15.73	E	1227	0	283	0	0	66	0	0	49	0	2434	4261996
27-Jun	15.73	E	1226	0	283	0	0	66	0	0	49	0	2432	4264428

TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
28-Jun	15.74	E	1427	0	283	0	0	66	0	0	49	0	2830	4267258
29-Jun	15.74	E	1229	0	283	0	0	66	0	0	49	0	2438	4269696
30-Jun	15.75	E	1230	0	283	0	0	66	0	0	49	0	2439	4272136
01-Jul	15.76	E	2027	0	373	0	0	105	0	0	105	0	4021	4276157
02-Jul	15.74	D	2615	767	633	767	317	165	317	5300	174	5300	-7475	4268682
03-Jul	15.75	E	2747	0	850	0	0	200	0	0	190	0	5448	4274130
04-Jul	15.77	E	2781	0	968	0	0	220	0	0	194	0	5516	4279646
05-Jul	15.79	E	4091	0	1064	0	0	247	0	0	214	0	8114	4287759
06-Jul	15.78	D	4444	767	1187	767	317	276	317	5300	237	5300	-3848	4283911
07-Jul	15.77	D	5090	767	1284	767	317	295	317	5300	242	5300	-2568	4281343
08-Jul	15.80	E	5693	0	1335	0	0	307	0	0	244	0	11291	4292634
09-Jul	15.79	D	4862	767	1366	767	317	316	317	5300	244	5300	-3019	4289615
10-Jul	15.79	D	5400	767	1384	767	317	321	317	5300	244	5300	-1953	4287662
11-Jul	15.81	E	5727	0	1397	0	0	325	0	0	244	0	11359	4299021
12-Jul	15.81	D	5845	767	1404	767	317	328	317	5300	244	5300	-1069	4297952
13-Jul	15.81	D	5751	767	1408	767	317	330	317	5300	244	5300	-1255	4296696
14-Jul	15.80	D	6048	767	1410	767	317	331	317	5300	244	5300	-667	4296029
15-Jul	15.80	D	6139	767	1412	767	317	331	317	5300	244	5300	-487	4295542
16-Jul	15.83	E	6223	0	1413	0	0	331	0	0	244	0	12344	4307885
17-Jul	15.83	D	6101	767	1413	767	317	332	317	5300	244	5300	-562	4307324
18-Jul	15.83	D	6573	767	1413	767	317	332	317	5300	244	5300	374	4307698
19-Jul	15.83	D	6644	767	1413	767	317	332	317	5300	244	5300	517	4308215
20-Jul	15.84	D	7113	767	1413	767	317	332	317	5300	244	5300	1445	4309660
21-Jul	15.84	D	6779	767	1413	767	317	332	317	5300	244	5300	783	4310443
22-Jul	15.84	D	7237	767	1413	767	317	332	317	5300	244	5300	1692	4312135
23-Jul	15.85	D	7697	767	1413	767	317	332	317	5300	244	5300	2604	4314739
24-Jul	15.85	D	7356	767	1413	767	317	332	317	5300	244	5300	1928	4316666
25-Jul	15.86	D	7406	767	1413	767	317	332	317	5300	244	5300	2026	4318693

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DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
26-Jul	15.86	D	6850	767	1413	767	317	332	317	5300	244	5300	924	4319617
27-Jul	15.86	D	7087	767	1413	767	317	332	317	5300	244	5300	1394	4321011
28-Jul	15.87	D	7526	767	1413	767	317	332	317	5300	244	5300	2266	4323277
29-Jul	15.87	D	7369	767	1413	767	317	332	317	5300	244	5300	1953	4325231
30-Jul	15.88	D	7206	767	1413	767	317	332	317	5300	244	5300	1630	4326861
31-Jul	15.88	D	7440	767	1413	767	317	332	317	5300	244	5300	2094	4328955
01-Aug	15.89	D	8272	767	1504	767	317	370	317	5300	300	5300	3745	4332700
02-Aug	15.90	D	8491	767	1763	767	317	430	317	5300	369	5300	4179	4336879
03-Aug	15.91	D	9451	767	1981	767	317	465	317	5300	385	5300	6083	4342962
04-Aug	15.93	D	9784	767	2159	767	317	511	317	5300	426	5300	6744	4349706
05-Aug	15.95	D	10781	767	2398	767	317	564	317	5300	473	5300	8721	4358427
06-Aug	15.97	D	11180	767	2579	767	317	597	317	5300	484	5300	9514	4367941
07-Aug	15.99	D	11293	767	2682	767	317	618	317	5300	487	5300	9736	4377677
08-Aug	16.02	D	12350	767	2738	767	317	632	317	5300	488	5300	11834	4389511
09-Aug	16.05	D	11981	767	2771	767	317	643	317	5300	488	5300	11101	4400613
10-Aug	16.07	D	12580	767	2794	767	317	650	317	5300	488	5300	12289	4412901
11-Aug	16.10	D	12763	767	2806	767	317	656	317	5300	488	5300	12652	4425553
12-Aug	16.10	C	13131	4500	2815	4500	2500	659	2500	5300	488	5300	1648	4427201
13-Aug	16.11	C	12885	4500	2821	4500	2500	661	2500	5300	488	5300	1160	4428361
14-Aug	16.11	C	13224	4500	2824	4500	2500	662	2500	5300	488	5300	1832	4430193
15-Aug	16.11	C	12954	4500	2826	4500	2500	663	2500	5300	488	5300	1296	4431490
16-Aug	16.15	D	13473	767	2827	767	317	663	317	5300	488	5300	14061	4445551
17-Aug	16.15	C	13388	4500	2827	4500	2500	663	2500	5300	488	5300	2159	4447709
18-Aug	16.16	C	13897	4500	2827	4500	2500	663	2500	5300	488	5300	3167	4450877
19-Aug	16.17	C	15470	4500	2887	4500	2500	689	2500	5300	525	5300	6287	4457164
20-Aug	16.19	C	16374	4500	3060	4500	2500	729	2500	5300	571	5300	8080	4465245
21-Aug	16.21	C	16702	4500	3205	4500	2500	752	2500	5300	582	5300	8731	4473977
22-Aug	16.23	C	16757	4500	3283	4500	2500	766	2500	5300	585	5300	8839	4482817

TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
23-Aug	16.25	C	17364	4500	3328	4500	2500	775	2500	5300	585	5300	10044	4492860
24-Aug	16.28	C	17882	4500	3382	4500	2500	794	2500	5300	604	5300	11071	4503931
25-Aug	16.30	C	18242	4500	3485	4500	2500	818	2500	5300	627	5300	11787	4515717
26-Aug	16.33	C	18302	4500	3565	4500	2500	834	2500	5300	632	5300	11904	4527621
27-Aug	16.35	C	17981	4500	3580	4500	2500	830	2500	5300	615	5300	11268	4538889
28-Aug	16.38	C	17965	4500	3520	4500	2500	817	2500	5300	592	5300	11236	4550125
29-Aug	16.41	C	18550	4500	3493	4500	2500	822	2500	5300	606	5300	12396	4562522
30-Aug	16.44	C	19220	4500	3550	4500	2500	838	2500	5300	627	5300	13725	4576247
31-Aug	16.47	C	19216	4500	3605	4500	2500	848	2500	5300	633	5300	13718	4589965
01-Sep	16.51	C	22358	4500	4088	4500	2500	1047	2500	5300	914	5300	19950	4609915
02-Sep	16.74	C	56969	4500	7846	1454	2500	2399	2500	5300	2772	0	105156	4715071
03-Sep	17.60	B	202131	6500	28323	0	3500	9504	3500	5300	3600	0	393985	5109055
04-Sep	18.04	A	106996	9300	56493	0	7963	12773	4127	5300	3600	0	204040	5313094
05-Sep	18.39	A	88407	9300	48894	0	8354	8567	8333	5300	3600	0	158828	5471922
06-Sep	18.69	A	79826	9300	30214	0	8658	5560	8658	5300	3600	0	141162	5613084
07-Sep	18.96	A	73069	9300	19758	0	8934	4234	8934	5300	3600	0	127211	5740295
08-Sep	19.22	A	68493	9300	13907	0	9232	3422	9232	5300	3600	0	117544	5857840
09-Sep	19.45	A	65559	9300	11054	0	9507	2891	9507	5300	3600	0	111180	5969021
10-Sep	19.68	A	64101	9300	8681	619	9766	2578	9766	5300	3600	0	106545	6075566
11-Sep	19.90	A	62694	9300	7922	1378	10015	2309	10015	5300	3600	0	101757	6177323
12-Sep	20.09	A	55596	9300	7482	1818	10253	1992	10253	5300	3600	0	86332	6263655
13-Sep	20.24	A	48649	9300	7239	2061	10451	1794	10451	5300	3600	0	71678	6335333
14-Sep	20.39	A	47285	9300	6900	2400	10612	1643	10612	5300	1870	0	67980	6403313
15-Sep	20.51	A	42996	9300	6551	2749	10765	1517	10765	5300	1260	0	58478	6461791
16-Sep	20.63	A	42304	9300	6262	3038	10897	1465	10897	5300	1117	0	56269	6518060
17-Sep	20.74	A	40252	9300	6233	3067	11024	1461	11024	5300	1083	0	51890	6569949
18-Sep	20.82	A	37867	9300	6016	3284	11152	1370	11152	5300	945	5300	35962	6605912
19-Sep	20.88	A	34716	9300	5406	3894	11255	1231	11255	5300	782	5300	28297	6634209

TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
20-Sep	20.94	A	34264	9300	4897	4403	11337	1148	11337	5300	743	5300	26230	6660439
21-Sep	20.99	A	33288	9300	4622	4678	11412	1101	11412	5300	734	5300	23600	6684040
22-Sep	21.03	A	32567	9300	4467	4833	11480	1069	11480	5300	732	5300	21727	6705766
23-Sep	21.08	A	32082	9300	4382	4918	11543	1047	11543	5300	732	5300	20472	6726238
24-Sep	21.12	A	31518	9300	4325	4975	11602	1030	11602	5300	732	5300	19123	6745362
25-Sep	21.16	A	31602	9300	4296	5004	11658	1017	11658	5300	732	5300	19123	6764484
26-Sep	21.19	A	30551	9300	4275	5025	11713	1007	11713	5300	732	5300	16886	6781370
27-Sep	21.23	A	30131	9300	4261	5039	11762	1001	11762	5300	732	5300	15929	6797299
28-Sep	21.26	A	29743	9300	4251	5049	11808	997	11808	5300	732	5300	15046	6812345
29-Sep	21.29	A	29380	9300	4244	5056	11851	995	11851	5300	732	5300	14226	6826571
30-Sep	21.32	A	28851	9300	4240	5060	11893	995	11893	5300	732	5300	13087	6839658
01-Oct	21.34	A	28159	9300	4150	5150	11931	956	11931	5300	676	5300	11461	6851119
02-Oct	21.36	A	27507	9300	3890	5410	11964	896	11964	5300	607	5300	9586	6860705
03-Oct	21.38	A	26063	9300	3612	5688	11992	835	11992	5300	553	5300	6118	6866822
04-Oct	21.38	A	25200	9300	3322	5978	12009	775	12009	5300	503	5300	3792	6870614
05-Oct	21.39	A	24375	9300	3110	6190	12020	738	12020	5300	491	5300	1715	6872329
06-Oct	21.39	A	23862	9300	2996	6304	12025	715	12025	5300	489	5300	461	6872790
07-Oct	21.39	A	23418	9300	2927	6373	12027	699	12027	5300	488	5300	-557	6872232
08-Oct	21.39	A	23220	9300	2891	6409	12025	687	12025	5300	488	5300	-1020	6871212
09-Oct	21.38	A	22660	9300	2866	6434	12022	678	12022	5300	488	5300	-2174	6869038
10-Oct	21.38	A	22528	9300	2852	6448	12016	672	12016	5300	488	5300	-2451	6866587
11-Oct	21.37	A	22026	9300	2841	6459	12009	668	12009	5300	488	5300	-3454	6863134
12-Oct	21.36	A	21739	9300	2834	6466	11999	665	11999	5300	488	5300	-4018	6859116
13-Oct	21.35	A	21067	9300	2830	6470	11987	664	11987	5300	488	5300	-5335	6853781
14-Oct	21.34	A	21612	9300	2828	6472	11972	663	11972	5300	488	5300	-4228	6849553
15-Oct	21.33	A	21380	9300	2827	6473	11959	663	11959	5300	488	5300	-4667	6844886
16-Oct	21.32	A	20952	9300	2827	6473	11946	663	11946	5300	488	5300	-5489	6839397
17-Oct	21.31	A	20741	9300	2827	6473	11930	663	11930	5300	488	5300	-5875	6833522

TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
18-Oct	21.29	A	19936	9300	2827	6473	11913	663	11913	5300	488	5300	-7438	6826084
19-Oct	21.27	A	20134	9300	2827	6473	11891	663	11891	5300	488	5300	-7003	6819081
20-Oct	21.26	A	19551	9300	2827	6473	11871	663	11871	5300	488	5300	-8119	6810963
21-Oct	21.24	A	19370	9300	2827	6473	11847	663	11847	5300	488	5300	-8432	6802530
22-Oct	21.22	A	19592	9300	2827	6473	11823	663	11823	5300	488	5300	-7943	6794587
23-Oct	21.20	A	19032	9300	2827	6473	11800	663	11800	5300	488	5300	-9008	6785579
24-Oct	21.19	A	19479	9300	2827	6473	11774	663	11774	5300	488	5300	-8069	6777510
25-Oct	21.17	A	19137	9300	2827	6473	11751	663	11751	5300	488	5300	-8702	6768808
26-Oct	21.15	A	19000	9300	2827	6473	11725	663	11725	5300	488	5300	-8924	6759884
27-Oct	21.13	A	18664	9300	2827	6473	11700	663	11700	5300	488	5300	-9539	6750344
28-Oct	21.11	A	18931	9300	2827	6473	11672	663	11672	5300	488	5300	-8954	6741390
29-Oct	21.09	A	18409	9300	2827	6473	11646	663	11646	5300	488	5300	-9939	6731452
30-Oct	21.07	A	18891	9300	2827	6473	11617	663	11617	5300	488	5300	-8925	6722527
31-Oct	21.05	A	18377	9300	2827	6473	11591	663	11591	5300	488	5300	-9893	6712634
01-Nov	21.03	A	18408	9300	2766	6534	11563	637	11563	5300	450	5300	-9896	6702738
02-Nov	21.00	A	16251	9300	2533	6767	11534	571	11534	5300	367	5300	-14579	6688159
03-Nov	20.96	A	15730	9300	2215	7085	11492	508	11492	5300	310	5300	-16159	6672000
04-Nov	20.92	A	14849	9300	1992	7308	11445	471	11445	5300	297	5300	-18257	6653743
05-Nov	20.88	A	14468	9300	1869	7431	11393	449	11393	5300	294	5300	-19153	6634590
06-Nov	20.84	A	14553	9300	1801	7499	11338	434	11338	5300	293	5300	-19010	6615580
07-Nov	20.80	A	14286	9300	1760	7540	11283	423	11283	5300	293	5300	-19511	6596070
08-Nov	20.76	A	13841	9300	1736	7564	11227	414	11227	5300	293	5300	-20331	6575739
09-Nov	20.71	A	13822	9300	1722	7578	11169	408	11169	5300	293	5300	-20281	6555458
10-Nov	20.67	A	13618	9300	1712	7588	11110	403	11110	5300	293	5300	-20590	6534868
11-Nov	20.62	A	13432	9300	1705	7595	11062	400	11062	5300	293	5300	-20877	6513992
12-Nov	20.58	A	13055	9300	1700	7600	11015	399	11015	5300	293	5300	-21540	6492453
13-Nov	20.53	A	13287	9300	1697	7603	10966	398	10966	5300	293	5300	-20989	6471464
14-Nov	20.49	A	13133	9300	1696	7604	10919	398	10919	5300	293	5300	-21203	6450261

TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
15-Nov	20.44	A	12995	9300	1696	7604	10871	398	10871	5300	293	5300	-21382	6428880
16-Nov	20.39	A	12858	9300	1696	7604	10823	398	10823	5300	293	5300	-21559	6407321
17-Nov	20.35	A	12192	9300	1666	7634	10774	385	10774	5300	274	5300	-22842	6384479
18-Nov	20.29	A	11665	9300	1579	7721	10723	365	10723	5300	251	5300	-23957	6360521
19-Nov	20.24	A	11228	9300	1507	7793	10669	353	10669	5300	246	5300	-24861	6335660
20-Nov	20.19	A	11236	9300	1468	7832	10613	347	10613	5300	244	5300	-24812	6310848
21-Nov	20.13	A	10867	9300	1446	7854	10557	342	10557	5300	244	5300	-25478	6285371
22-Nov	20.08	A	10910	9300	1433	7867	10500	339	10500	5300	244	5300	-25303	6260069
23-Nov	20.02	A	10767	9300	1425	7875	10443	337	10443	5300	244	5300	-25488	6234581
24-Nov	19.97	A	10839	9300	1421	7879	10386	335	10386	5300	244	5300	-25241	6209339
25-Nov	19.92	A	10514	9300	1418	7882	10328	333	10328	5300	244	5300	-25776	6183564
26-Nov	19.86	A	10399	9300	1416	7884	10267	332	10267	5300	244	5300	-25888	6157675
27-Nov	19.80	A	10494	9300	1415	7885	10207	332	10207	5300	244	5300	-25582	6132093
28-Nov	19.75	A	10593	9300	1414	7886	10147	332	10147	5300	244	5300	-25271	6106823
29-Nov	19.69	A	10095	9300	1413	7887	10088	332	10088	5300	244	5300	-26142	6080681
30-Nov	19.64	A	9997	9300	1413	7887	10027	332	10027	5300	244	5300	-26215	6054465
01-Dec	19.58	A	9775	9300	1383	7917	9966	319	9966	5300	225	5300	-26593	6027871
02-Dec	19.52	A	9295	9300	1297	8003	9904	299	9904	5300	202	5300	-27595	6000276
03-Dec	19.46	A	9297	9300	1224	8076	9839	287	9839	5300	197	5300	-27606	5972669
04-Dec	19.40	A	9342	9300	1185	8115	9775	280	9775	5300	196	5300	-27467	5945203
05-Dec	19.34	A	9216	9300	1163	8137	9711	276	9711	5300	195	5300	-27633	5917570
06-Dec	19.28	A	9098	9300	1151	8149	9646	273	9646	5300	195	5300	-27764	5889805
07-Dec	19.23	A	8992	9300	1143	8157	9581	270	9581	5300	195	5300	-27862	5861943
08-Dec	19.17	A	8888	9300	1139	8161	9516	268	9516	5300	195	5300	-27947	5833996
09-Dec	19.11	A	8790	9300	1136	8164	9451	267	9451	5300	195	5300	-28017	5805979
10-Dec	19.04	A	8501	9300	1134	8166	9386	266	9386	5300	195	5300	-28465	5777514
11-Dec	18.98	A	8411	9300	1132	8168	9319	266	9319	5300	195	5300	-28516	5748997
12-Dec	18.92	A	8530	9300	1131	8169	9253	265	9253	5300	195	5300	-28150	5720848

TABLE 6
LAKE OKEECHOBEE ROUTINGS
1% FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
13-Dec	18.86	A	8254	9300	1131	8169	9187	265	9187	5300	195	5300	-28566	5692282
14-Dec	18.80	A	8382	9300	1131	8169	9120	265	9120	5300	195	5300	-28181	5664102
15-Dec	18.74	A	8317	9300	1131	8169	9054	265	9054	5300	195	5300	-28178	5635924
16-Dec	18.68	A	8247	9300	1131	8169	8988	265	8988	5300	195	5300	-28187	5607737
17-Dec	18.62	A	7982	9300	1131	8169	8922	265	8922	5300	195	5300	-28581	5579156
18-Dec	18.56	A	7919	9300	1131	8169	8863	265	8863	5300	195	5300	-28590	5550566
19-Dec	18.49	A	7862	9300	1131	8169	8809	265	8809	5300	195	5300	-28594	5521972
20-Dec	18.46	B	7804	6500	1131	6500	3500	265	3500	5300	195	5300	-14869	5507104
21-Dec	18.43	B	7752	6500	1131	6500	3500	265	3500	5300	195	5300	-14972	5492133
22-Dec	18.40	B	7899	6500	1131	6500	3500	265	3500	5300	195	5300	-14680	5477453
23-Dec	18.36	B	7653	6500	1131	6500	3500	265	3500	5300	195	5300	-15169	5462285
24-Dec	18.33	B	7802	6500	1131	6500	3500	265	3500	5300	195	5300	-14872	5447412
25-Dec	18.30	B	7554	6500	1131	6500	3500	265	3500	5300	195	5300	-15364	5432048
26-Dec	18.27	B	7511	6500	1131	6500	3500	265	3500	5300	195	5300	-15449	5416599
27-Dec	18.23	B	7670	6500	1131	6500	3500	265	3500	5300	195	5300	-15133	5401465
28-Dec	18.20	B	7834	6500	1131	6500	3500	265	3500	5300	195	5300	-14808	5386657
29-Dec	18.17	B	7600	6500	1131	6500	3500	265	3500	5300	195	5300	-15273	5371383

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
31-May	16.94	B	2342	6500	563	6500	3500	132	3500	5300	97	5300	-25703	4804297
01-Jun	16.89	B	2480	6500	593	6500	3500	145	3500	5300	116	5300	-25429	4778868
02-Jun	16.83	B	2881	6500	679	6500	3500	165	3500	5300	139	5300	-24634	4754234
03-Jun	16.78	B	2797	6500	751	6500	3500	176	3500	5300	144	5300	-24799	4729435
04-Jun	16.72	B	3285	6500	790	6500	3500	183	3500	5300	145	5300	-23832	4705603
05-Jun	16.67	B	3355	6500	813	6500	3500	188	3500	5300	146	5300	-23693	4681910
06-Jun	16.62	B	3012	6500	825	6500	3500	191	3500	5300	146	5300	-24374	4657537
07-Jun	16.56	B	3460	6500	833	6500	3500	193	3500	5300	146	5300	-23484	4634052
08-Jun	16.51	B	3305	6500	837	6500	3500	195	3500	5300	146	5300	-23792	4610261
09-Jun	16.46	B	3546	6500	840	6500	3500	196	3500	5300	146	5300	-23314	4586947
10-Jun	16.41	B	3382	6500	842	6500	3500	197	3500	5300	146	5300	-23639	4563308
11-Jun	16.35	B	3415	6500	843	6500	3500	198	3500	5300	146	5300	-23574	4539733
12-Jun	16.30	B	3848	6500	844	6500	3500	198	3500	5300	146	5300	-22716	4517017
13-Jun	16.26	C	3276	4500	845	4500	2500	198	2500	5300	146	5300	-17899	4499118
14-Jun	16.23	C	3299	4500	845	4500	2500	198	2500	5300	146	5300	-17853	4481265
15-Jun	16.19	C	3522	4500	845	4500	2500	198	2500	5300	146	5300	-17411	4463854
16-Jun	16.15	C	3546	4500	845	4500	2500	198	2500	5300	146	5300	-17363	4446491
17-Jun	16.11	C	3367	4500	845	4500	2500	198	2500	5300	146	5300	-17719	4428772
18-Jun	16.07	C	3386	4500	845	4500	2500	198	2500	5300	146	5300	-17682	4411090
19-Jun	16.03	C	3807	4500	845	4500	2500	198	2500	5300	146	5300	-16847	4394243
20-Jun	16.00	C	5169	4500	875	4500	2500	211	2500	5300	164	5300	-14144	4380099
21-Jun	15.97	C	5196	4500	961	4500	2500	231	2500	5300	187	5300	-14090	4366009
22-Jun	15.93	C	5132	4500	1033	4500	2500	242	2500	5300	193	5300	-14218	4351791
23-Jun	15.90	C	5231	4500	1072	4500	2500	249	2500	5300	194	5300	-14021	4337770
24-Jun	15.87	C	5712	4500	1094	4500	2500	254	2500	5300	194	5300	-13067	4324704
25-Jun	15.87	D	5381	767	1106	767	317	257	317	5300	194	5300	-1990	4322714
26-Jun	15.86	D	5236	767	1114	767	317	259	317	5300	194	5300	-2278	4320436
27-Jun	15.86	D	5082	767	1118	767	317	261	317	5300	194	5300	-2583	4317853

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
28-Jun	15.85	D	5526	767	1121	767	317	262	317	5300	194	5300	-1701	4316152
29-Jun	15.85	D	5167	767	1123	767	317	263	317	5300	194	5300	-2415	4313737
30-Jun	15.84	D	5202	767	1125	767	317	264	317	5300	194	5300	-2345	4311392
01-Jul	15.84	D	6231	767	1216	767	317	303	317	5300	250	5300	-304	4311088
02-Jul	15.84	D	6846	767	1475	767	317	362	317	5300	319	5300	915	4312004
03-Jul	15.84	D	7005	767	1691	767	317	397	317	5300	335	5300	1231	4313235
04-Jul	15.85	D	7066	767	1808	767	317	417	317	5300	339	5300	1353	4314587
05-Jul	15.86	D	8737	767	1935	767	317	457	317	5300	377	5300	4668	4319255
06-Jul	15.87	D	9308	767	2143	767	317	506	317	5300	423	5300	5799	4325054
07-Jul	15.89	D	10088	767	2311	767	317	536	317	5300	434	5300	7347	4332401
08-Jul	15.91	D	10794	767	2402	767	317	555	317	5300	436	5300	8747	4341148
09-Jul	15.93	D	10039	767	2455	767	317	568	317	5300	437	5300	7249	4348397
10-Jul	15.95	D	10648	767	2485	767	317	577	317	5300	437	5300	8457	4356854
11-Jul	15.97	D	11036	767	2505	767	317	583	317	5300	437	5300	9227	4366081
12-Jul	15.96	C	11210	4500	2516	4500	2500	588	2500	5300	437	5300	-2163	4363918
13-Jul	15.96	C	11165	4500	2524	4500	2500	591	2500	5300	437	5300	-2251	4361667
14-Jul	15.98	D	11508	767	2528	767	317	593	317	5300	437	5300	10164	4371830
15-Jul	15.98	C	11642	4500	2531	4500	2500	594	2500	5300	437	5300	-1306	4370525
16-Jul	15.98	C	11766	4500	2533	4500	2500	594	2500	5300	437	5300	-1059	4369466
17-Jul	15.97	C	11680	4500	2534	4500	2500	594	2500	5300	437	5300	-1230	4368236
18-Jul	16.00	D	12189	767	2534	767	317	594	317	5300	437	5300	11514	4379750
19-Jul	16.00	C	12294	4500	2534	4500	2500	594	2500	5300	437	5300	-12	4379738
20-Jul	16.00	C	12997	4500	2534	4500	2500	594	2500	5300	437	5300	1382	4381120
21-Jul	16.00	C	12693	4500	2534	4500	2500	594	2500	5300	437	5300	780	4381899
22-Jul	16.01	C	13184	4500	2534	4500	2500	594	2500	5300	437	5300	1752	4383651
23-Jul	16.01	C	13673	4500	2534	4500	2500	594	2500	5300	437	5300	2723	4386375
24-Jul	16.02	C	13357	4500	2534	4500	2500	594	2500	5300	437	5300	2096	4388470
25-Jul	16.02	C	13431	4500	2534	4500	2500	594	2500	5300	437	5300	2243	4390714

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
26-Jul	16.03	C	12896	4500	2534	4500	2500	594	2500	5300	437	5300	1182	4391896
27-Jul	16.03	C	13156	4500	2534	4500	2500	594	2500	5300	437	5300	1698	4393594
28-Jul	16.04	C	13618	4500	2534	4500	2500	594	2500	5300	437	5300	2615	4396209
29-Jul	16.04	C	13480	4500	2534	4500	2500	594	2500	5300	437	5300	2340	4398549
30-Jul	16.05	C	13335	4500	2534	4500	2500	594	2500	5300	437	5300	2053	4400603
31-Jul	16.05	C	13588	4500	2534	4500	2500	594	2500	5300	437	5300	2555	4403159
01-Aug	16.06	C	14697	4500	2684	4500	2500	659	2500	5300	530	5300	4753	4407913
02-Aug	16.08	C	15307	4500	3115	4500	2500	758	2500	5300	645	5300	5965	4413877
03-Aug	16.09	C	16499	4500	3476	4500	2500	816	2500	5300	672	5300	8330	4422207
04-Aug	16.12	C	17201	4500	3731	4500	2500	875	2500	5300	715	5300	9721	4431928
05-Aug	16.14	C	18329	4500	4014	4500	2500	937	2500	5300	763	5300	11958	4443885
06-Aug	16.17	C	18840	4500	4218	4500	2500	976	2500	5300	774	5300	12972	4456857
07-Aug	16.20	C	19049	4500	4336	4500	2500	1001	2500	5300	776	5300	13386	4470243
08-Aug	16.24	C	20197	4500	4400	4500	2500	1020	2500	5300	777	5300	15664	4485908
09-Aug	16.27	C	19904	4500	4439	4500	2500	1033	2500	5300	777	5300	15083	4500990
10-Aug	16.31	C	20576	4500	4465	4500	2500	1042	2500	5300	777	5300	16415	4517406
11-Aug	16.34	C	20825	4500	4481	4500	2500	1048	2500	5300	777	5300	16909	4534314
12-Aug	16.38	C	21255	4500	4492	4500	2500	1053	2500	5300	777	5300	17762	4552076
13-Aug	16.42	C	21064	4500	4498	4500	2500	1055	2500	5300	777	5300	17383	4569458
14-Aug	16.46	C	21457	4500	4501	4500	2500	1056	2500	5300	777	5300	18163	4587622
15-Aug	16.50	C	21235	4500	4503	4500	2500	1056	2500	5300	777	5300	17723	4605346
16-Aug	16.54	C	21804	4500	4504	4500	2500	1057	2500	5300	777	5300	18850	4624197
17-Aug	16.58	C	21763	4500	4504	4500	2500	1057	2500	5300	777	5300	18769	4642966
18-Aug	16.63	C	22314	4500	4504	4500	2500	1057	2500	5300	777	5300	19864	4662830
19-Aug	16.68	B	24724	6500	4655	4645	3500	1121	3500	5300	870	5300	22372	4685202
20-Aug	16.74	B	26243	6500	5086	4214	3500	1220	3500	5300	985	5300	26239	4711442
21-Aug	16.82	B	26934	6500	5446	3854	3500	1278	3500	5300	1012	0	38837	4750278
22-Aug	16.91	B	27256	6500	5642	3658	3500	1312	3500	5300	1018	0	39863	4790141

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
23-Aug	17.00	B	28073	6500	5752	3548	3500	1334	3500	5300	1020	0	41703	4831844
24-Aug	17.10	B	28970	6500	5842	3458	3500	1363	3500	5300	1039	0	43661	4875506
25-Aug	17.20	B	29486	6500	5968	3332	3500	1394	3500	5300	1062	0	44934	4920440
26-Aug	17.29	B	29685	6500	6060	3240	3500	1415	3500	5300	1067	0	45511	4965951
27-Aug	17.39	B	29158	6500	6054	3246	3500	1403	3500	5300	1031	0	44454	5010405
28-Aug	17.46	B	29064	6500	5914	3386	3500	1372	3500	5300	986	5300	33479	5043883
29-Aug	17.54	A	29978	9300	5849	3451	7863	1381	7863	5300	1012	0	37022	5080905
30-Aug	17.63	A	30862	9300	5957	3343	7920	1411	7920	5300	1055	0	38874	5119779
31-Aug	17.71	A	31000	9300	6063	3237	7980	1427	7980	5300	1066	0	39240	5159019
01-Sep	17.81	A	35036	9300	6662	2638	8051	1668	8051	5300	1403	0	48290	5207309
02-Sep	18.11	A	78158	9300	11459	0	8147	3412	8147	5300	3600	0	138867	5346176
03-Sep	19.17	A	251610	9300	38375	0	8417	12783	4117	5300	3600	0	490903	5837079
04-Sep	19.74	A	133213	9300	75581	0	9458	17105	0	5300	3600	0	264228	6101306
05-Sep	20.16	A	103904	9300	65546	0	10075	11546	5355	5300	3600	0	195474	6296779
06-Sep	20.51	A	91718	9300	40818	0	10525	7552	9348	5300	3600	0	163381	6460161
07-Sep	20.81	A	83006	9300	26899	0	10893	5778	10893	5300	3600	0	143035	6603196
08-Sep	21.09	A	77124	9300	19118	0	11248	4703	11248	5300	3600	0	130667	6733862
09-Sep	21.36	A	73681	9300	15390	0	11624	4012	11624	5300	3600	0	123090	6856952
10-Sep	21.61	A	72018	9300	12300	0	11981	3603	11981	5300	3600	0	119083	6976035
11-Sep	21.86	A	71579	9300	11301	0	12326	3243	12326	5300	3600	0	117527	7093562
12-Sep	22.10	A	70395	9300	10676	0	12655	2813	12655	5300	3600	0	114526	7208088
13-Sep	22.34	A	69794	9300	10324	0	12961	2548	12961	5300	3600	0	112727	7320815
14-Sep	22.58	A	69558	9300	9884	0	13259	2353	13259	5300	3600	0	111670	7432485
15-Sep	22.82	A	69765	9300	9434	0	13554	2188	13554	5300	3600	0	111493	7543979
16-Sep	23.04	A	68083	9300	9058	242	13857	2121	13857	5300	3600	0	107076	7651054
17-Sep	23.27	A	66765	9300	9022	278	14185	2116	14185	5300	3600	0	103742	7754796
18-Sep	23.47	A	64539	9300	8717	583	14503	1986	14503	5300	3600	0	98091	7852887
19-Sep	23.67	A	62693	9300	7850	1450	14803	1788	14803	5300	3600	0	92114	7945001

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
20-Sep	23.86	A	62276	9300	7127	2173	15086	1671	15086	5300	3346	0	89293	8034294
21-Sep	24.01	A	53258	9300	6737	2563	15360	1604	15296	5300	1603	0	70212	8104507
22-Sep	24.11	A	42088	9300	6516	2784	15576	1558	15342	5300	1194	0	47530	8152038
23-Sep	24.23	A	46598	9300	6395	2905	15722	1527	15373	5300	1098	0	56174	8208211
24-Sep	24.33	A	43080	9300	6314	2986	15895	1503	15397	5300	1076	0	48987	8257198
25-Sep	24.45	A	45248	9300	6274	3026	16046	1484	15416	5300	1070	0	53169	8310367
26-Sep	24.55	A	43372	9300	6244	3056	16210	1470	15430	5300	1069	0	49361	8359728
27-Sep	24.66	A	43834	9300	6224	3076	16362	1462	15438	5300	1069	0	50220	8409948
28-Sep	24.76	A	43299	9300	6209	3091	16516	1456	15444	5300	1069	0	49120	8459068
29-Sep	24.87	A	43384	9300	6199	3101	16668	1454	15446	5300	1069	0	49262	8508330
30-Sep	24.97	A	42940	9300	6194	3106	16820	1453	15447	5300	1069	0	48370	8556700
01-Oct	25.05	A	42419	9300	6073	3227	16900	1401	15499	5300	994	5300	36484	8593184
02-Oct	25.12	A	41792	9300	5729	3571	16900	1322	15578	5300	903	5300	34400	8627584
03-Oct	25.19	A	40362	9300	5380	3920	16900	1249	15651	5300	844	5300	30726	8658310
04-Oct	25.25	A	39719	9300	5052	4249	16900	1183	15717	5300	793	5300	28669	8686979
05-Oct	25.31	A	39168	9300	4819	4481	16900	1142	15758	5300	781	5300	27033	8714012
06-Oct	25.36	A	38909	9300	4693	4607	16900	1116	15784	5300	778	5300	26217	8740229
07-Oct	25.42	A	38729	9300	4617	4683	16900	1098	15802	5300	778	5300	25673	8765902
08-Oct	25.47	A	38785	9300	4577	4723	16900	1084	15816	5300	777	5300	25677	8791579
09-Oct	25.52	A	38475	9300	4549	4751	16900	1073	15827	5300	777	5300	24985	8816564
10-Oct	25.58	A	38584	9300	4533	4767	16900	1066	15834	5300	777	5300	25157	8841721
11-Oct	25.63	A	38312	9300	4520	4780	16900	1061	15839	5300	777	5300	24583	8866304
12-Oct	25.68	A	38250	9300	4512	4788	16900	1059	15841	5300	777	5300	24440	8890744
13-Oct	25.73	A	37797	9300	4507	4793	16900	1057	15843	5300	777	5300	23527	8914271
14-Oct	25.79	A	38555	9300	4505	4795	16900	1057	15843	5300	777	5300	25027	8939298
15-Oct	25.84	A	38527	9300	4504	4796	16900	1057	15843	5300	777	5300	24968	8964266
16-Oct	25.89	A	38301	9300	4504	4796	16900	1057	15843	5300	777	5300	24521	8988787
17-Oct	25.94	A	38278	9300	4504	4796	16900	1057	15843	5300	777	5300	24474	9013261

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
18-Oct	25.99	A	37325	9300	4474	4826	16900	1044	15856	5300	759	5300	22499	9035760
19-Oct	26.04	A	37524	9300	4388	4912	16900	1024	15876	5300	736	5300	22684	9058444
20-Oct	26.07	A	32894	9300	4316	4984	16900	1012	15888	5300	730	5300	13333	9071777
21-Oct	26.07	A	26987	9300	4277	5023	16900	1006	15895	5300	729	5300	1525	9073302
22-Oct	26.09	A	30748	9300	4255	5045	16900	1001	15899	5300	729	5300	8933	9082235
23-Oct	26.09	A	27700	9300	4243	5057	16900	998	15902	5300	729	5300	2858	9085093
24-Oct	26.11	A	29617	9300	4235	5065	16900	996	15904	5300	729	5300	6640	9091733
25-Oct	26.12	A	28174	9300	4231	5069	16900	994	15906	5300	729	5300	3766	9095499
26-Oct	26.13	A	28622	9300	4228	5072	16900	992	15908	5300	729	5300	4646	9100145
27-Oct	26.13	A	27780	9300	4226	5074	16900	991	15909	5300	729	5300	2971	9103116
28-Oct	26.14	A	28263	9300	4224	5076	16900	991	15909	5300	729	5300	3923	9107039
29-Oct	26.14	A	27489	9300	4223	5077	16900	991	15909	5300	729	5300	2386	9109425
30-Oct	26.15	A	28034	9300	4223	5077	16900	991	15909	5300	729	5300	3465	9112890
31-Oct	26.16	A	27381	9300	4223	5077	16900	991	15909	5300	729	5300	2170	9115060
01-Nov	26.16	A	27410	9300	4163	5137	16900	965	15935	5300	692	5300	2058	9117118
02-Nov	26.15	A	24969	9300	3930	5370	16900	899	16001	5300	608	5300	-3374	9113744
03-Nov	26.14	A	24423	9300	3614	5686	16900	836	16064	5300	552	5300	-5210	9108534
04-Nov	26.13	A	23479	9300	3391	5909	16900	800	16100	5300	538	5300	-7598	9100936
05-Nov	26.11	A	23059	9300	3269	6031	16900	777	16123	5300	535	5300	-8716	9092220
06-Nov	26.09	A	23095	9300	3201	6099	16900	762	16138	5300	535	5300	-8811	9083409
07-Nov	26.07	A	22790	9300	3161	6139	16900	751	16149	5300	534	5300	-9516	9073893
08-Nov	26.05	A	22301	9300	3137	6163	16900	743	16157	5300	534	5300	-10551	9063342
09-Nov	26.03	A	22245	9300	3123	6177	16900	736	16164	5300	534	5300	-10703	9052639
10-Nov	26.00	A	22005	9300	3113	6187	16900	732	16168	5300	534	5300	-11208	9041431
11-Nov	25.98	A	21782	9300	3106	6194	16900	729	16171	5300	534	5300	-11670	9029761
12-Nov	25.95	A	21372	9300	3101	6199	16900	727	16173	5300	534	5300	-12496	9017265
13-Nov	25.93	A	21573	9300	3098	6202	16900	727	16173	5300	534	5300	-12105	9005160
14-Nov	25.90	A	21387	9300	3097	6203	16900	726	16174	5300	534	5300	-12476	8992684

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
15-Nov	25.87	A	21219	9300	3097	6203	16900	726	16174	5300	534	5300	-12809	8979875
16-Nov	25.84	A	21056	9300	3097	6203	16900	726	16174	5300	534	5300	-13133	8966742
17-Nov	25.81	A	20028	9300	3037	6263	16900	701	16199	5300	497	5300	-15342	8951400
18-Nov	25.77	A	19282	9300	2864	6436	16900	661	16239	5300	451	5300	-17242	8934158
19-Nov	25.73	A	18707	9300	2720	6580	16900	638	16262	5300	441	5300	-18715	8915443
20-Nov	25.69	A	18613	9300	2642	6658	16900	624	16276	5300	438	5300	-19083	8896360
21-Nov	25.65	A	18162	9300	2598	6702	16900	615	16285	5300	437	5300	-20083	8876277
22-Nov	25.61	A	18132	9300	2574	6726	16900	609	16291	5300	437	5300	-20202	8856075
23-Nov	25.56	A	17926	9300	2558	6742	16900	604	16296	5300	437	5300	-20652	8835423
24-Nov	25.52	A	17939	9300	2550	6750	16900	601	16299	5300	437	5300	-20650	8814773
25-Nov	25.48	A	17561	9300	2544	6756	16900	598	16302	5300	437	5300	-21417	8793356
26-Nov	25.43	A	17395	9300	2540	6760	16900	596	16304	5300	437	5300	-21758	8771598
27-Nov	25.38	A	17447	9300	2537	6763	16900	595	16305	5300	437	5300	-21662	8749936
28-Nov	25.34	A	17504	9300	2535	6765	16900	594	16306	5300	437	5300	-21554	8728382
29-Nov	25.29	A	16963	9300	2534	6766	16900	594	16306	5300	437	5300	-22630	8705752
30-Nov	25.24	A	16826	9300	2534	6766	16900	594	16306	5300	437	5300	-22902	8682850
01-Dec	25.19	A	16569	9300	2504	6796	16900	581	16319	5300	419	5300	-23497	8659353
02-Dec	25.14	A	15926	9300	2387	6913	16900	549	16351	5300	377	5300	-25068	8634285
03-Dec	25.08	A	15705	9300	2229	7071	16900	517	16383	5300	349	5300	-25883	8608402
04-Dec	25.03	A	15611	9300	2118	7182	16900	499	16401	5300	342	5300	-26326	8582076
05-Dec	24.97	A	15380	9300	2057	7243	16900	488	16412	5300	341	5300	-26928	8555148
06-Dec	24.91	A	15176	9300	2023	7277	16900	480	16420	5300	340	5300	-27414	8527734
07-Dec	24.85	A	14994	9300	2003	7297	16880	475	16425	5300	340	5300	-27825	8499909
08-Dec	24.79	A	14825	9300	1991	7309	16794	470	16430	5300	340	5300	-28192	8471717
09-Dec	24.73	A	14665	9300	1984	7316	16707	467	16433	5300	340	5300	-28531	8443186
10-Dec	24.67	A	14319	9300	1979	7321	16619	465	16435	5300	340	5300	-29231	8413955
11-Dec	24.61	A	14177	9300	1975	7325	16529	464	16437	5300	340	5300	-29523	8384432
12-Dec	24.54	A	14247	9300	1973	7327	16438	463	16437	5300	340	5300	-29390	8355043

TABLE 7
LAKE OKEECHOBEE ROUTINGS
STANDARD PROJECT FLOOD EVENT
INITIAL LAKE LEVEL 17.0 FT, NGVD

DATE	LAKE LEVEL FT NGVD	ZONE	INFLOW TO LAKE CFS	CALOOSAHATCHEE RIVER			ST LUCIE CANAL			AGRICULTURAL CANALS			STORAGE VOLUME AF	
				REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	LOCAL INFLOW CFS	ACTUAL FLOW CFS	REGULATORY DISCHARGE CFS	S2 CFS	ACTUAL FLOW CFS	CHANGE	LAKE
13-Dec	24.48	A	13928	9300	1971	7329	16347	462	16347	5300	340	5300	-29849	8325195
14-Dec	24.42	A	14013	9300	1971	7329	16255	462	16255	5300	340	5300	-29498	8295697
15-Dec	24.35	A	13910	9300	1971	7329	16164	462	16164	5300	340	5300	-29523	8266174
16-Dec	24.29	A	13802	9300	1971	7329	16073	462	16073	5300	340	5300	-29556	8236618
17-Dec	24.23	A	13497	9300	1971	7329	15982	462	15982	5300	340	5300	-29980	8206638
18-Dec	24.16	A	13401	9300	1971	7329	15890	462	15890	5300	340	5300	-29988	8176650
19-Dec	24.10	A	13311	9300	1971	7329	15798	462	15798	5300	340	5300	-29982	8146667
20-Dec	24.04	A	13224	9300	1971	7329	15706	462	15706	5300	340	5300	-29972	8116694
21-Dec	23.97	A	13141	9300	1971	7329	15614	462	15614	5300	340	5300	-29954	8086740
22-Dec	23.91	A	13263	9300	1971	7329	15521	462	15521	5300	340	5300	-29530	8057210
23-Dec	23.85	A	12987	9300	1971	7329	15431	462	15431	5300	340	5300	-29897	8027313
24-Dec	23.78	A	13113	9300	1971	7329	15339	462	15339	5300	340	5300	-29465	7997849
25-Dec	23.72	A	12837	9300	1971	7329	15248	462	15248	5300	340	5300	-29832	7968016
26-Dec	23.66	A	12771	9300	1971	7329	15157	462	15157	5300	340	5300	-29782	7938233
27-Dec	23.59	A	12909	9300	1971	7329	15065	462	15065	5300	340	5300	-29326	7908907
28-Dec	23.53	A	13053	9300	1971	7329	14975	462	14975	5300	340	5300	-28862	7880045
29-Dec	23.47	A	12798	9300	1971	7329	14887	462	14887	5300	340	5300	-29193	7850852

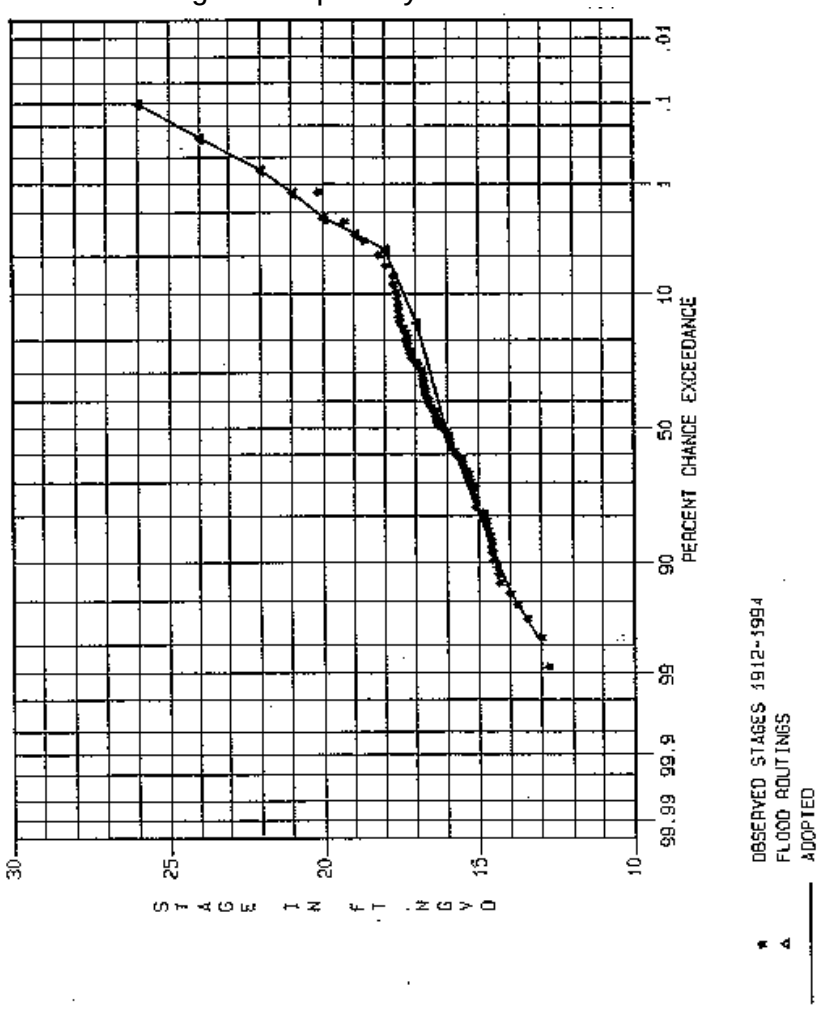
TABLE 8

STAGE-FREQUENCY FOR LAKE OKEECHOBEE

INITIAL STAGE FEET	DURATION PERCENT
10.4	4.3
12.0	20.9
13.4	29.2
15.0	25.6
16.0	17.3
17.0	2.7

ELEVATION FEET	PERCENT CHANCE EXCEEDANCE						
	INITIAL LAKE LEVEL FEET						COMBINED
	10.4	12.0	13.4	15.0	16.0	17.0	
17.0	5.000	11.000	14.000	20.000	20.000	20.000	15.722
18.0	2.800	3.800	4.200	4.900	4.900	4.900	4.375
19.0	1.500	2.500	3.200	3.600	3.600	3.600	3.163
20.0	0.650	1.500	2.200	2.700	2.700	2.700	2.215
21.0	0.550	0.800	1.200	1.500	1.500	1.500	1.225
22.0	0.380	0.530	0.700	0.770	0.770	0.770	0.683
24.0	0.150	0.220	0.290	0.310	0.310	0.310	0.278
26.0	0.060	0.085	0.110	0.120	0.120	0.120	0.107

Figure 10 Stage Frequency for Lake Okeechobee



4. Lake Okeechobee Storm Surge Frequency Functions.

The primary goal of this storm surge analysis effort was the determination of hurricane-induced surge-frequency relationships at selected locations along the shore of Lake Okeechobee. The objective was to employ a probabilistic approach which would yield reliable estimates of storm surge elevations, durations, and frequencies of occurrence. Related efforts included numerical modeling of hurricane wind and atmospheric pressure fields and corresponding lake responses, and the use of a relatively new statistical analysis procedure, the empirical simulation technique (EST), to establish the surge-frequency relationships.

The EST is a statistical procedure designed to develop joint-probability relationships among the various parameters of a multi-parameter system. The user must first establish a database which consists of the forcing and response parameters associated with historical events. The EST then employs resampling, interpolation, and replacement techniques to generate a much larger data base which is statistically similar to the historical data. Further analysis of the expanded data set results in determination of probabilities associated with the response in question.

Introduction - Until recently, many storm-related analysis and design tasks have been approached using traditional joint probability methods, in which parameters descriptive of a hurricane are assigned fixed values based on design criteria; furthermore, the probabilities of occurrence for these values are generally assumed independent. For example, coastal engineering studies may involve the use of synthetic (or hypothetical) hurricanes which are derived based on specification of a number of variables typically used to describe a tropical storm (e.g., maximum wind speed, radius to maximum winds, etc.). The most familiar examples are the Standard Project Hurricane (SPH) and the Probable Maximum Hurricane (PMH). These hypothetical storms were developed by the National Weather Service (1979) in an effort to provide a rational basis for U.S. Atlantic and Gulf coast design applications. They are defined as follows:

a. SPH - The hurricane which represents the most severe combination of hurricane parameters that is reasonably characteristic of a region, excluding extremely rare combinations.

b. PMH - The hurricane having the combination of characteristics which will result in the most severe storm that is reasonably possible in the region involved if the hurricane should approach the point under study along a critical path and at an optimum rate of movement.

The SPH has frequently been used in the design of coastal works where a high degree of protection is required. The PMH has generally been used only in situations where any failure is strictly unacceptable, e.g., in the design of a nuclear power plant proposed for construction in a coastal environment. The occurrence frequencies of the descriptive hurricane parameters are defined by empirical relationships which may not be truly representative of the study area; therefore, the accuracy of the computed return periods associated with the SPH and PMH events are suspect. Although a designer may make a qualitative decision regarding which design storm should be chosen, these definitions do not provide any quantitative information relative to the degrees of safety resulting from that choice. Sensitivity analyses have typically been used as a tool for addressing the levels of uncertainty in a project design; however, this approach presumes that the appropriate ranges of important parameters are known, and that all values within those ranges are equally likely. The results of a sensitivity analysis are often reported as a single, most likely value which is treated as an accurate basis for design.

An alternative to these traditional methods is the use of a probabilistic approach to analysis and design. This approach considers the fact that the values of the important variables considered are not known with certainty; however, the likelihood of a variable taking on a particular value can be described by its frequency of occurrence. The concept of risk involves the probability of an undesirable event, such as a storm surge, and the consequences of the occurrence of that event; therefore, risk analysis incorporates two major activities. One is quantifying the probability of an undesirable event. The second is quantifying the probability of the consequences of that event. The various uncertainties involved are expressed in terms of probabilities. On Lake Okeechobee, variability of the lake stage includes a hurricane-induced surge component. The primary objective of the efforts described herein was the identification of the probability distributions associated with that surge component.

Although previous studies have been performed in an effort to predict storm surge elevations on Lake Okeechobee, a literature review failed to identify any which employed a probabilistic approach. Schmaltz (1986) documented the theoretical development of joint probability methods designed to define Lake Okeechobee surge-frequency relationships; however, implementation of those procedures was never accomplished. That methodology utilized typical joint probability techniques which are based on the assumption that individual hurricane parameters are independent. It assumes that the likelihood of each parameter can be modeled with empirical or parametric relationships. Then, the joint probability of occurrence of a particular storm is computed as the product of the individual storm parameter probabilities by way of the assumed parametric relationships.

In reality, the parameters which describe a storm event are not independent, but are inter-related in some nonlinear sense; therefore, the product of individual parameter probabilities may not be the most appropriate method for establishing the joint probability. Borgman et al. (1992) presented a description of the previously mentioned statistical procedure, the empirical simulation technique, which does not rely on assumed parametric relationships. Instead, EST methods make use of the joint probability relationships characteristic of a data base containing site-specific information relative to individual storm parameters. This data base of historical input and response information is used to develop multiple simulations of future responses; therefore, EST techniques involve the assumption that future events will be statistically similar to past events. Historical events are used as a basis for simulation of multiple future event scenarios. Simulated responses are subsequently used to determine frequencies of occurrence. This effort was undertaken to establish Lake Okeechobee surge-frequency relationships through application of the EST methodology.

<u>Hurricane Surge - General Description</u> - A storm surge is an increase of the water level in a coastal or inland water body which results from forcing by atmospheric weather systems. Surges occur primarily as a result of atmospheric pressure gradients and surface stresses created by wind blowing over a water surface. When the water's momentum carries it beyond the position of static equilibrium, a long-wave phenomenon results in which the water surface elevation increases

downwind and decreases upwind. In addition to the wind speed and duration, the surge is also influenced by water depth, length of fetch, and frictional characteristics of the basin bottom.

Hurricanes are defined as tropical storms characterized by maximum sustained winds of 75 mph or greater. They are typically well-organized with respect to wind patterns and are often described in terms of the following parameters:

a. Latitude and longitude at the eye of the hurricane.
b. Central pressure (atmospheric pressure at the eye).
c. Peripheral pressure (atmospheric pressure at the edge of the hurricane).
d. Track characteristics which define the storm's path.
e. Forward speed of the hurricane.
f. Inflow angle of winds, which accounts for the inward spiraling of horizontal wind velocities about the hurricane center.
g. Radius to maximum winds, measured from the hurricane eye to the isovel circle of maximum wind velocities.
h. Maximum wind speed, either defined explicitly or calculated as a function of the difference between central and ambient pressures.
i. Radial decay factor, which defines the decrease in wind speed with increasing radial distance outward from the radius to maximum winds.
j. Azimuth angle to the maximum velocity vector.

As mentioned previously, engineering studies often involve the use of synthetic hurricanes which are derived based on specification of the parameters listed above. During execution of this analysis, historically based hurricane events were generated and used to compute storm surges. This database of storm parameters and responses (surge) provided input to the empirical simulation technique. The empirical simulation produced multiple repetitions of possible future events and responses with characteristics which differ, within realistic limits, from those of the historical storm data base.

Previous Lake Okeechobee Surge Studies - The first efforts directed toward quantifying storm surges on Lake Okeechobee were begun, to some extent, in response to the catastrophic effects of the hurricanes of 1926, 1928, and 1935. The 1928

storm claimed over 2,000 lives when it passed over the lake, generating a maximum surge of 13.2 feet above the still water level (War Department, 1940). As a result, individuals realized that if lakeside communities were to be a part of that ongoing early development taking place in south Florida, protection from hurricane-induced flooding would be imperative. Thus, began the more detailed efforts to design and construct embankments around the perimeter of Lake Okeechobee. A significant portion of those efforts was directed toward establishing the required crest heights of the proposed embankments. This task identified the need for analytical methods which could be used to predict the magnitude of possible hurricane surges. Investigators realized that meteorological data would also be needed for use in the development and validation of the analytical surge prediction tools. Data collection at Lake Okeechobee began in 1935 with the installation of three lake stage recorders (Schloemer, 1954). By 1936, five stations along the lake shore were equipped with analog devices for measurement of wind speed, barometric pressure, and lake stage. In subsequent years, other stations and instrumentation types were added.

The first tools to analytically compute wind tides on Lake Okeechobee were developed between 1950 and 1956. A brief summary of the contributions made by some of those investigators is provided below:

a. Langhaar (1951) derived several formulas to simulate the effects of winds on large bodies of shallow water. He divided the surge phenomenon into two components, a static surge which would exist if the wind stress persisted indefinitely, and a dynamic surge characterized by the seiching action of the lake surface. The total surge elevation was computed as the sum of those two components. With his formulas, Langhaar calculated surge elevations on Lake Okeechobee and compared them to surge data collected during the hurricanes of September 1948 and August 1949. The results indicated good agreement between predicted and observed maximum surge elevations.

b. Schloemer (1954) performed a comprehensive study of hurricane winds characteristic of the Lake Okeechobee area. In developing a method to synthesize hurricane wind patterns, he compared ten different atmospheric pressure profile formulas to determine which corresponded best to measured pressure data. He also considered wind speed conversion

techniques, wind directions, and critical storm paths in developing the first wind field prediction tool for Lake Okeechobee. His methods were used extensively to produce the wind information required as input to the first surge prediction analyses.

c. Kivisild (1954) performed an extensive study of the inter-relationships between wind velocities, waves, set-up, and bathymetry in shallow bodies of water. The analytical formulas which he developed were used to estimate water movements in Lake Okeechobee during five hurricane events. Kivisild stated that comparison of measured and computed surge elevations indicated that the calculation methods were applicable for design predictions of Lake Okeechobee storm surges.

d. Myers (1954) also studied the characteristics of hurricanes which had affected the Lake Okeechobee area during the period from 1900 to 1949. He established values of central pressure, peripheral pressure, maximum wind speed, and radius to maximum winds, for each storm considered. The primary objective of this effort was to perform a detailed study of existing data suitable for use in designing levees at Lake Okeechobee.

e. A summary of the results of several storm surge studies was prepared by Farrer (1958). The summary also outlined the procedures which were ultimately used by USACE to compute design surge elevations for raising and improving the Lake Okeechobee embankment system. He described the basic equations used in the analytical procedure, important parameters, and methods by which field measurements were used to evaluate unknowns in the wind-tide formula. Two computational methods, a cross section integration procedure and a segmental integration procedure, were discussed. The cross-sectional integration procedure was recommended for shallow lakes where wind velocities are fairly uniform across the cross section and bathymetric variations are relatively constant across the width of the lake. These methods involved an iterative solution technique with computations primarily involving the length and shape of the fetch, depth along each cross section, and wind speed. An empirical method for estimating wind shear stress was developed based on prototype conditions observed at Lake Okeechobee and results of a series of physical model studies performed at the University of California (USACE 1954).

f. During the 1960's and 1970's, significant improvements in storm surge prediction tools were accomplished. These improvements were realized due to the increased efforts of agencies responsible for the provision of flood protection in coastal areas, and due to advancements in computer technology. In 1966, Reid and Bodine (1968) developed a two-dimensional surge model for USACE, Galveston District. This model (SURGE-I) was the first to include algorithms for flooding and drying, barriers, and flow over barriers in the study of surge inundation of low-lying bay areas.

g. Whitaker et al. (1973) modified the SURGE-I algorithms during their application of the model to simulation of Lake Okeechobee surges. Their modifications included numerical methods to better address the surge response as it propagates over the heavily vegetated areas characteristic of Lake Okeechobee's western shore.

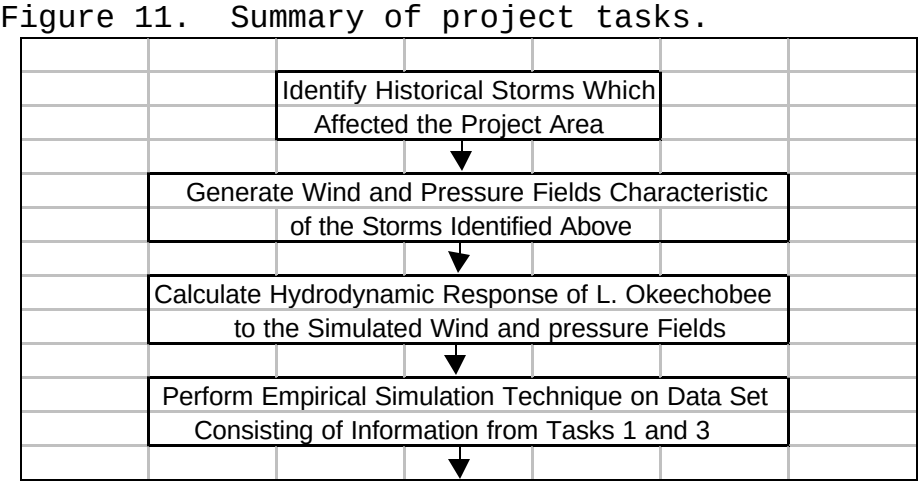
h. In consideration of areas where bays are normally connected to the sea by rivers, improvements to SURGE-I were made by Reid et al. (1977). Their efforts allowed the simulation of overland flooding during a surge event, which can greatly expand the width of the bay-sea connection. Significant modeling improvements were made relative to conditions where frictional resistance forces are dominant, such as at Lake Okeechobee. The program was applied to the simulation of hurricane Carla and synthetic hurricanes in the Sabine-Calcasieu area near Port Arthur, Texas.

i. During the period between June 1982 and July 1985, USACE Waterways Experiment Station performed a numerical investigation of hurricane-induced water level fluctuations on Lake Okeechobee (Schmaltz 1986). This effort involved two major components, the hurricane and hydrodynamic sub-models. The primary function of the hurricane sub-model was generation of the hurricane wind velocity and atmospheric pressure fields which were subsequently used as input for the hydrodynamic model. The modeling package developed by WES included two methods for obtaining the hurricane wind and pressure characteristics. The first was a direct parametric approach, and the second applied a planetary boundary layer (PBL) approach as the solution mechanism. Schmaltz's efforts resulted in surge calculation methods applicable to both design and forecasting situations. Design applications could include either a standard parametric design hurricane approach or a method employing joint probability techniques.

The storm surge modeling efforts summarized above represent only those found which specifically addressed simulation of conditions at Lake Okeechobee. Numerous other investigators have made substantial contributions towards development of state-of-the-art surge modeling techniques; however, a comprehensive discussion of that subject is beyond the scope of this appendix. Individuals seeking more detailed information should consult available references such as Murty (1984).

Project Overview - This effort to develop surge-frequency relationships for Lake Okeechobee can be subdivided into four major tasks as indicated in Figure 12. The initial task was identification of all historical hurricanes which could have forced a surge response on Lake Okeechobee. Task 2 involved the generation of wind and pressure fields associated with those hurricanes. The wind and pressure fields developed in the second task were then used as input for task 3, the simulation of Lake Okeechobee surges corresponding to each of the hurricanes in the data base. Maximum simulated surge elevations were then combined with information describing the storms to formulate the data base used during task 4, execution of the empirical simulation technique. The following section of this appendix will provide a more detailed description of the EST procedures. This will be followed by discussions relative to the numerical models used in tasks 2 and 3, respectively. A final section will summarize how results of tasks 1 through 4 were combined to produce surge-frequency relationships for Lake Okeechobee.

Figure 11. Summary of project tasks.



EMPIRICAL SIMULATION TECHNIQUE - Predictions of the extreme values of a response (e.g. wave heights or surge elevations)

should always be based on a long term data source of measured, observed, or predicted values. The reliability of the predicted extremes is directly related to the accuracy of the available data and the number of years of record; therefore, the longest high quality data source available should be used for an extremal analysis (Thompson, 1994). The development of this type of suitable data source is the objective of the empirical simulation technique.

As previously mentioned, the empirical simulation technique was developed to simulate systems characterized by a number of variables when the actual relationships between those variables is unknown. It accomplishes this through use of a site-specific historical database, containing measured system variables; therefore, the relationships between individual variables, although unknown, are inherently represented in the historical database. With the historical database established, the EST performs a series of resampling with replacement operations which result in the creation of a much larger artificial population of inter-related variables. The resampling with replacement techniques are extensions of mathematical methods, labeled "bootstrap" methods, developed by Efron (1982). In summary, the basic function of the empirical simulation technique is to perform random sampling of a finite length data base, and through interpolation, to generate a larger data base suitable for use in an extremal analysis.

An initial step in the EST process is specification of a set of parameters which describe the dynamics of the physical system being studied. Those parameters must describe both the process being modeled and the effects of that process; thus, the required parameters can be described as input variables and response variables. For this application, the system being studied is a hurricane. The parameters which are used to describe hurricanes (input variables) were identified previously (e.g., radius to maximum winds, central pressure, forward speed, etc.). The parameters describing hurricane effects (response variables) could include maximum storm surge magnitude, surge duration, deepwater wave characteristics, or shoreline response.

Although the response variables are related to the input variables, their inter-relationships involve non-linear correlations which can not be defined; however, the EST assumes that the combined effects of all input variables at a site are reflected by the corresponding response variables at

that site. Therefore, an important step in the analysis procedure is compilation of all available historical data relative to input variables and corresponding response variables for the specific location(s) in question. This information comprises the historical data base. The historical data base for this application included all available input and surge response information relative to hurricanes which have affected Lake Okeechobee.

The next step involves expansion (if appropriate) of the historical data base by selectively creating additional data. The resulting data base is referred to as the "training set." The previously mentioned secondary goal of this project is related to optimization of the training set. During project execution, various alternatives related to augmentation of the historical data set were considered. The additional information is created through inclusion of not only the historical data, but data corresponding to slightly perturbed historical events. For example, additional data might be created in a hurricane data base by inclusion of not only a given historical hurricane's input and response variables, but also the input and response variables resulting from the same hurricane following a slightly altered track. In many cases, more information is available relative to variables describing a system as opposed to variables describing the system responses; therefore, initially, the training set may often consist primarily of input variables. Numerical models, such as the hydrodynamic model used in this application, may be used to generate the missing response variables in a training set.

Once the training set is established, the empirical simulation technique performs N simulations of a T-year sequence of the events being analyzed, each characterized by their individual input and response variables. In other words, T = the length of record (in years) considered appropriate for the type of extremal analysis in question; and N = the number of times the length of record will be simulated. For this application, 100 repetitions of a 200-year sequence of hurricanes was chosen.

Two fundamental assumptions apply to the T-year sequence of events. First, it is assumed that the characteristics of each simulated storm event will be similar to the characteristics of historical events. In other words, the inter-relationships among the input and response vectors must be realistic. For example, a hurricane with a high pressure gradient and low maximum windspeed would not be a reasonable event. Although

the relationship between pressure gradient and maximum windspeed is not known, the two parameters are not independent. The simulation of realistic events is accomplished by using the input and response parameter inter-relationships inherent to the historic data and employing the nearest-neighbor interpolation resampling technique developed by Borgman (Borgman et al 1992). The basic technique can be described in two dimensions as follows. Let X_1, X_2, X_3, ..., X_n be n independent, identically distributed events, each characterized by two descriptive input parameters, Z_1 and Z_2. For example, Z_1 and Z_2 may be pressure gradient and maximum wind speed, respectively.

Each storm event, X_i, has a probability of occurrence, P_i, where $P_i = 1/n$; therefore, a cumulative probability relationship can be developed in which each storm is assigned a probability ranging from 0.0 to 1.0. If each storm has an equal probability of occurrence, then each event may be assigned a corresponding probability segment, S_i, where:

$$[0<S_1<(1/n)] \qquad (1)$$

$$[(1/n)<S_2<(2/n)]$$

$$[(2/n)<S_3<(3/n)]$$

$$[(3/n)<S_4<(4/n)]$$

$$\vdots$$

$$[((n-1)/n)<S_n<1]$$

A random number from 0 to 1 is selected to identify an event from the total population. This procedure is equivalent to drawing and replacing random samples from the total population of events.

Rather than simply resampling a population of historical events, the EST approach is designed to simulate the parameter distributions contained in the training set data base (which contains the population of historical events). The EST method is begun by choosing a sample storm based on a random number selection between 0 and 1. The procedure then performs a random walk from the selected event, X_1, with corresponding response vectors, R_1 and R_2, to the nearest neighbor events. The walk is based on the selection of independent uniform random numbers from -1 to 1. Its purpose is to accomplish the

simulation of responses which are not identical, but are similar, to the historical responses.

The second fundamental assumption is that the number of events selected in the T-year time span must be statistically representative of the number of historical events which have occurred in the study area. A Poisson distribution is used to determine the average number of expected events in a given year, based on the mean frequency of events for the site being studied. The Poisson distribution may be expressed as:

$$P(s:\lambda)=(\lambda^{s}e^{-\lambda})/s! \qquad (2)$$

where: s = number of storms per year, (s = 0,1,2,3,...).
λ = a measure of the number of storms per year based on historical data, [λ = number of events/record length (yrs)]
$P(s;\lambda)$ = the probability of experiencing s storms per year.

The EST initializes a 10,000 element array to the above Poisson distribution. For this project application, ë = 23/104 = 0.2212. The probability of 0 hurricanes per year is 0.8016; therefore, if the 0.0 to 1.0 random number selection is less than or equal to 0.8016, then no hurricanes would occur during that year of simulation. Corresponding probabilities for 1,2, and 3 storms per year are 0.1773, 0.0196, and 0.0014, respectively; therefore, if the random number selection is between 0.8016 and (0.8016 + 0.1773 = 0.9789), one storm is specified to occur during that year of simulation. If the random number selection is between 0.9789 and 0.9985, two storms are specified to occur, etc. When one or more storms are indicated for a given year, they are randomly selected from the training set population and input and response variables are determined by way of the nearest neighbor interpolation technique described earlier.

The EST procedure ultimately results in N repetitions of T-years of simulated event responses. The frequency of occurrence relationships are computed from these responses. That process involves generation of a probability distribution function (pdf) which corresponds to each of the T-year sequences of simulated data.

The initial step in estimating the response-frequency relationships is calculation of a probability distribution

function for the response in question. The cumulative probability distribution function will be of the form:

$$F_x(x)=Pr(X \leq x) \qquad (3)$$

where $Pr(X \leq x)$ represents the probability that the random variable X is less than or equal to some value x, and $F_x(x)$ is the cumulative pdf ranging from 0.0 to 1.0. The difficulty arises in estimating the value of F_x without relying on some parametric relationship to describe the probability. The following procedure (Scheffner et al, 1993) will be adopted because it utilizes the probability laws inherent to the historical data set and does not incorporate any prior assumptions concerning the probability relationship.

Consider a situation where there exists a data set containing ten observations (n=10) of response values for a given site. The n response values are first ranked in order from smallest to largest. Let r denote the rank such that r=1 corresponds to the smallest response and r=10 corresponds to the largest. An empirical estimate of the cumulative pdf is given by Gumbel (1954) as:

$$F_x(x_{(r)})=r/(n+1) \qquad (4)$$

$$\text{for } \{x_{(r)},\ r = 1,2,3,...,n\}$$

This form allows for future values of the response, x, to be less than the smallest observed value, $x_{(1)}$, with a probability of 1/(n+1). Values of x larger than $x_{(n)}$ are also possible, again with a probability of 1/(n+1). Now consider that the cumulative probability for an n-year return event is written as:

$$F(n)=1-(1/n) \qquad (5)$$

where F(n) is the cumulative probability of occurrence for an event with a recurrence interval of n-years. The response frequency of occurrence relationships will be obtained by linearly interpolating a response from equation 4 which corresponds to the probability distribution function associated with a return period specified in equation 5. For this project application, the procedure described above was accomplished for each of 23 locations of interest in and around Lake Okeechobee.

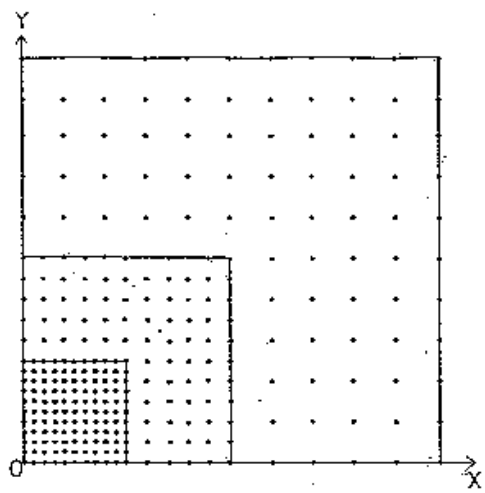
Numerical Modeling Of Wind And Atmospheric Pressures -
A tropical storm planetary boundary layer wind model (HURWIN) was used to simulate hurricane-generated wind and atmospheric pressure fields. HURWIN computes the surface stresses, wind speeds, and wind directions in the planetary boundary layer of a hurricane. The model is based on the solution of the horizontal equation of motion, vertically averaged through the depth of the planetary boundary layer, formulated by Chow (1971). Chow's vortex model was subsequently modified by Cardone et al. (1992) in efforts to improve correlation between modeled winds and available measured wind data. Those modifications included generalization of storm input options and addition of a variable boundary layer depth formulation. HURWIN establishes a fixed pressure gradient in the boundary layer which results in zero local accelerations; therefore, the solution to the motion equation is steady state. A descriptive summary of the model theory, assumptions and solution methods is provided by Thompson and Cardone (1994).

The computational grid is a nested system of rectangular grids which provides greater resolution as one proceeds from the storm's perimeter to the eye (Figure 13). Wind velocities and atmospheric pressures are computed at each node. A nested grid is used in order to achieve a balance between computational efficiency and the desired model resolution. The model uses five grid nests. For this application, the innermost nest consisted of 5 km x 5 km nodes. Remaining nest sizes, proceeding towards the storm periphery, were based on 10 km, 20 km, 40 km, and 80 km node spacings. The resulting entire HURWIN grid covered an area of 1,600 km x 1,600 km.

HURWIN input is provided as a series of data "snapshots". Each snapshot consists of a set of meteorological parameters which characterize the hurricane at given stages of storm development or dissipation. Descriptive parameters include: a) latitude and longitude of the eye, b) track direction and forward speed at the eye, c) radius to maximum winds, d) atmospheric pressure gradient, and e) geostrophic wind speed and direction. For each hurricane simulated, these meteorological parameters were obtained from the hurricane database developed by the National Oceanographic and Atmospheric Administration's (NOAA) National Hurricane Center (NHC) (Jarvinen, et al. 1988). Within this database, NHC has compiled descriptive information relative to all hurricanes and tropical storms which occurred in the North Atlantic Ocean since 1886. For this project, the database encompassing the 104-year period from 1886 through 1989 was used. Hurricane

Andrew (August 1992) conditions were also added to the database. Information, provided at six hour intervals, includes latitude and longitude of the eye, central pressure, and maximum wind speed.

For this investigation, a pre-processing routine was used to approximate the radius to maximum winds. This routine employed an approximation method developed by Jelesnianski and Taylor (1973). The estimate establishes the radius as



Example of nested grid concept employed by HURWIN. (from Thompson and Cardone, 1994)

Figure 12. Example of nested grid concept employed by HURWIN. (from Thompson and Cardone, 1994)

a function of maximum wind speed and atmospheric pressure deficit. Due to the relatively small water surface area modeled in this application, it was necessary to override this routine under certain conditions. This subject will be discussed further in a subsequent section describing model calibration.

For each 6-hour snapshot of input, the governing equations are first solved using the outermost grid nest (i.e., 80 km node spacings). Computed wind velocities were then used as boundary conditions for the second largest grid, and the equations of motion were solved again. This process was continued for the remaining grids with wind velocities from one grid nest used as boundary conditions for the next smallest grid nest. After all the 6-hour snapshots in the data base are processed, the model uses an interpolation

procedure to identify hourly wind and atmospheric pressure fields. This hourly information was then transferred from the nested HURWIN grid to the hydrodynamic model grid for use as input in the storm surge computations.

Numerical Modeling Of Hurricane-Induced Surges **-** The Waterways Experiment Station Implicit Flooding Model (WIFM) was used for simulation of the long wave hydrodynamic processes associated with Lake Okeechobee storm surges. WIFM is a two-dimensional long wave model which computes finite difference approximations of water surface displacements and vertically integrated velocities. The model was originally developed at the USACE, Waterways Experiment Station (Butler, 1978) and has been periodically upgraded through specific applications to problems involving simulation of long waves. An earlier version of WIFM was used during the Lake Okeechobee surge investigation by Schmaltz (1986).

A descriptive overview of the model has been provided by Cialone et al. (1993). Portions of that discussion were used in the following summary of WIFM features. The model uses derivations of the classical Navier-Stokes and continuity equations. Under homogeneous, incompressible fluid conditions, and assuming that vertical water particle velocities are small relative to gravitational acceleration (i.e., pressure under long waves is hydrostatic), the two-dimensional governing equations are provided below as equations (6) and (7). The continuity equation (8) states that the sum of all the net fluid flows into a column of water must be balanced by an increase of fluid in the column. Since incompressible fluid conditions have been assumed, that increase in fluid is reflected by a change in the height of the column's water surface.

$$\delta u/\delta t + u(\delta u/\delta x) + v(\delta u/\delta y) + g[\delta(\eta-\eta_a)/\delta x] - fv - \tau_{sx}/\rho d + \tau_{Bx}/\rho d + A_H[(\delta^2 u/\delta x^2)+(\delta^2 u/\delta y^2)] = 0 \quad (6)$$

$$\delta v/\delta t + u(\delta v/\delta x) + v(\delta v/\delta y) + g[\delta(\eta-\eta_a)/\delta y] + fu - \tau_{sy}/\rho d + \tau_{By}/\rho d + A_H[(\delta^2 v/\delta x^2)+(\delta^2 v/\delta y^2)] = 0 \quad (7)$$

I II III IV V VI VII

$$\delta\eta/\delta t + \delta(ud)/\delta x) + \delta(vd)/\delta y = R \quad (8)$$

where: x,y = independent cartesian space variables (see Figure 14).

t = independent time variable

η = water surface displacement relative to an arbitrary datum

h = satic water depth measured from same datum

η_a = water surface displacement due to atmospheric pressure deficit

d = total water depth (=h+c)

u, v = x and y velocity components, respectively

τ_{Bx}, τ_{By} = bottom friction stresses in the x and y directions, respectively

f = Coriolis parameter

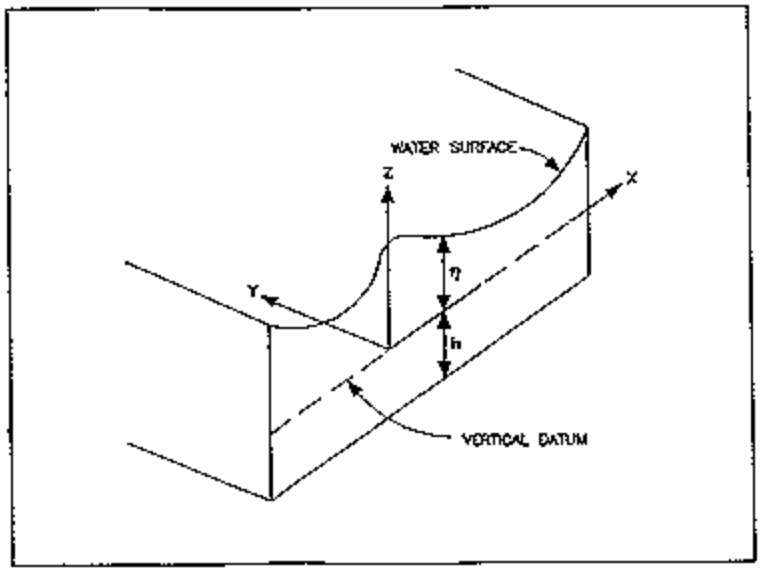
A_H = generalized diffusion coefficient

g = gravitational acceleration

τ_{sx}, τ_{sy} = external surface shear stresses (wind stress) in the x and y directions, respectively

ρ = water density

R = source/sink term to account for mass changes (e.g., rainfall, percolation, evaporation, etc.)



Cartesian coordinate system (from Cialone et al. 1993).

Figure 13. Cartesian coordinate system (from Cialone et al. 1993).

Individual components (indicated by roman numerals) of the governing equations are identified below:

I. Temporal acceleration terms: local flow acceleration terms represent the change in vertically averaged velocity with respect to time.

II. Spatial acceleration terms: inertial (or advective) terms describe the movement of the water due to the fluid motion itself.

III. Barometric pressure gradient terms: describe the principle driving force of the fluid flow, i.e., the slope of the water surface.

IV. Coriolis terms: represent momentum source and sink due to the effect of Earth's rotation. The Coriolis term, f, is defined as:

$$f=2\nu\sin\lambda \qquad (9)$$

where ν is the angular velocity of the earth and λ is the latitude of the study area.

V. External shear stress terms: represent wind (and/or other) forces that contribute to fluid motion. The wind stress term, θ_s, is formulated as:

$$\tau_s = \rho_a C_d |W| W \qquad (10)$$

W is the resultant wind speed in meters/second.

VI. Bottom friction terms: momentum sink terms account for energy dissipation resulting from stress of the fluid layer against the bottom boundary. Shear stresses are represented by quadratic expressions of the form:

$$\tau_{bx}/\rho = [gu(u^2+v^2)^{1/2}]/(C_z)^2 d \qquad (11)$$

C_z is Chezy's friction coefficient. Other terms were defined above. For this application, Manning's n values, expressed as a function of depth, were used in lieu of C_z. The two coefficients are related by the expression:

$$C_z = (d)^{1/6}/n \qquad (12)$$

VII. Horizontal diffusion of momentum terms: account for eddy viscosity and describe the dispersion of momentum due to fluid motion. The diffusion coefficient, A_H, may be considered as a constant or variable parameter. This application employed the variable option, which uses the following method formulated by Vreugdenhil (1973) for determination of A_H:

$$A_H = 6d[g(u^2+v^2)^{1/2}]/C_z \qquad (13)$$

In solving the governing equations, WIFM uses a finite difference approach to approximate those terms represented by partial derivatives; therefore, the continuum is represented by discrete points in time and space. The horizontal plane is discretized through use of a computational grid. Each grid cell, at a given point in time, is characterized by certain flow parameters. WIFM defines the water surface elevation at the center of the cell, and the velocity components are defined at the cell faces. The model is capable of employing either a uniform grid (all cells are of equal size) or a stretched grid. A stretched grid permits more effective simulation of a complex geometry and allows for increased resolution in local areas of interest. The computational grid developed by Schmaltz (1986) was used for this project.

WIFM allows for a variety of boundary conditions depending on characteristics of the study area. Boundary conditions may be classified as one of three general types described below:

a. Open boundaries include the seaward boundaries at the edge of the computational grid and boundaries at flow channels (e.g., rivers) which exit the grid at some point. Water levels and/or flow rates at the boundary points are prescribed as functions of location and time.

b. Land-water boundaries are managed by methods designed to simulate flooding and drying of low-lying terrain. As a storm surge encroaches onto cells that were initially dry, those cells are incorporated into the computational scheme. Initially, water is transferred from one cell to an adjacent dry cell based on discharges computed by the following broad-crested weir formula:

$$q=C_o d^{3/2} \qquad (14)$$

where q is the discharge per unit width, C_o is an admittance coefficient, and d is the difference in average depths between the cells.

The incremental water level increase in the receiving cell is computed as:

$$\eta_r = (\Delta t x q)/\Delta x_r \qquad (15)$$

where η_r is the incremental rise in water level, Δt is the time step, and Δx_r is the width of the receiving cell. When the receiving cell's water level exceeds a small threshold value, it is incorporated into the Navier-Stokes based solution scheme.

To ensure conservation of mass, whenever a cell is flooded, an equal volume of water must be subtracted from the drying cell (i.e., the adjacent cell which provided the water). The algorithm which accomplishes this is essentially the inverse of equation (15), i.e.,

$$\eta d = (\Delta t x q)/\Delta x_d \qquad (16)$$

where η_d is the incremental fall in water level, and Δx_d is the width of the drying cell. If a cell's water level falls below a prescribed threshold value, that cell is removed from the

computational scheme and the remaining water is transferred to an adjoining cell at rates determined by the broadcrested weir formula. These flooding and drying features were of particular importance in the Lake Okeechobee application, where the relatively shallow water body is surrounded by land on all sides.

c. The third boundary type is the barrier. It is used to represent flow field obstructions with widths much narrower than the widths of adjacent grid cells, but with lengths at least as great as a the length of a grid cell. For this application, barriers were used to simulate long narrow islands and the Herbert Hoover Dike system, which encircles the entire lake perimeter. Barriers may be classified as exposed, submerged, or overtopping. Each barrier is allowed to change its classification during the course of a simulation; therefore, a barrier (such as an island) that is submerged during periods of high water levels may be exposed after water levels have receded. Exposed barriers impose a no-flow condition under which flow velocities at the barrier are zero. Submerged barriers are flooded; therefore, they are included in the computational scheme. Overtopping of a barrier occurs when (a) water levels on one side of a barrier are higher than the barrier, while on the opposite side, water levels are lower, or (b) water levels on both sides of the barrier are higher than the barrier, but the levels are less than a prescribed threshold which defines submergence. For both overtopping situations, discharges across the barrier are calculated via a broad-crested weir formula.

WIFM provides several options by which model output may be displayed. Those include (a) time histories of water surface elevations, wind velocities, and/or water velocities at selected grid cells, (b) wind velocity or water velocity vector fields depicting velocity magnitudes and directions over the entire grid at specified times during a simulation, and (c) an output listing which contains a summary of all input parameters and printouts of field arrays, i.e., maximum water levels, or velocities, at each grid cell. WIFM results may be displayed graphically using postprocessing software available through the Coastal Modeling System (Cialone et al. 1993). All available output and postprocessing options were employed during execution of this project.

<u>Calibration And Verification Of Numerical Models</u> **-** Whenever a numerical model is chosen to predict an unknown parameter, it should be constructed in ways that ensure the best available

input information is used. Then the model should be subjected to calibration and verification procedures. These procedures are undertaken to optimize the accuracy of the model predictions. Calibration involves the adjustment of variable model parameters in an effort to maximize the agreement between model predictions and corresponding measured data. When calibration is completed, the model should be used to simulate another event for which measured data exists; however, during this simulation, none of the variable model parameters should be adjusted. Hopefully, this verification procedure will also result in acceptable agreement between the predicted and measured values. If the model fails during verification, calibration should be repeated, with adjustments directed in ways which may compromise the level of agreement of predictions with the first measured data set while improving agreement during verification. The final objective of the construction, calibration, and verification procedures is to maximize the user's confidence that the model can accurately predict the parameter in question. Each of these procedures, relative to this specific application, will be discussed in the following paragraphs.

MODEL CONSTRUCTION. The computational grid used to discretize the study area consists of a 64 X 48 (3072) cell grid encompassing an area of about 3,900 km^2. East-west grid spacings are uniform, while the north-south spacings are stretched to provide larger cell sizes near the lake center, where less resolution is required. All bathymetric and topographic elevations were determined from NOAA nautical chart 11428. Islands, levees, and tree islands were represented by the appropriate boundary conditions, depending on their geometric characteristics and time of construction. Levee construction around the perimeter of Lake Okeechobee took place periodically between 1932 and 1967. Long narrow tree islands were constructed along the southeast and northwest shores during the 1960's; therefore, boundary conditions relative to levee and tree island barriers were established based on the occurrence year of the hurricane being simulated.

MODEL CALIBRATION. Project goals included development of surge-frequency relationships at the 23 Lake Okeechobee stations identified in Figure 15. Fourteen of those sites represent meteorological stations where wind, pressure, and water surface elevations have been recorded at various times. Model calibration was performed using data recorded during the hurricane which crossed the northeast corner of Lake

Okeechobee on 26 August, 1949 (NHC storm 477). While near the lake, the storm was characterized by a central pressure of 965 millibars and wind speeds ranging from 85 to 110 knots. This is the most severe storm to have influenced Lake Okeechobee, for which measured wind, pressure, and stage data are available. During this storm, successful data collection was accomplished at nine of the stations identified in Figure 15.

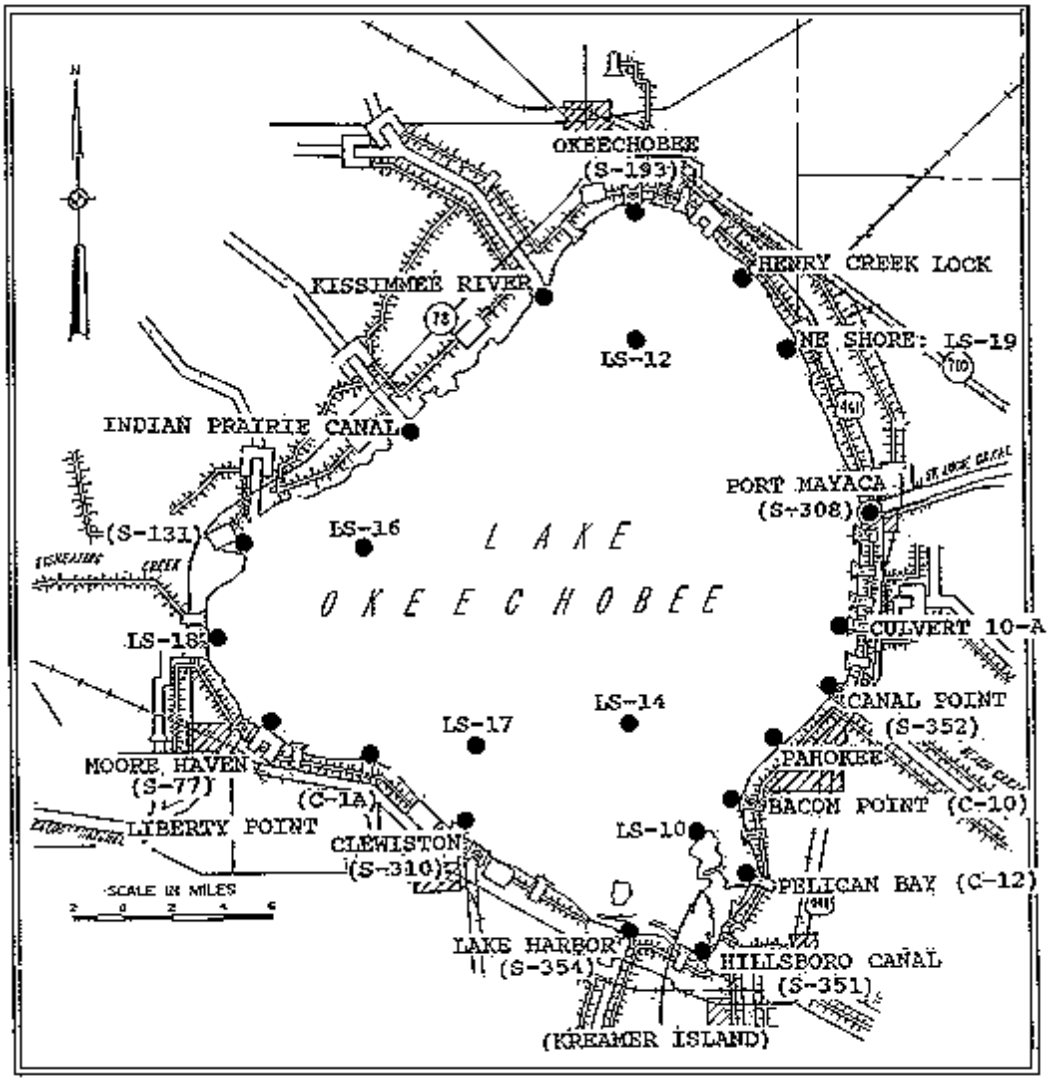


Figure 14. Location of study sites.

HURWIN Calibration. Initial HURWIN runs resulted in poor agreement between measured and predicted wind fields. Snapshots of the wind field velocity vectors indicated that the hurricane track was not accurately represented in the

vicinity of Lake Okeechobee. The NHC data base specifies the storm track in terms of latitude and longitude of the eye, expressed to the nearest tenth of a degree. Due to the relatively small size of the study area, for this investigation, a pre-processing routine was used to approximate the radius to maximum winds. This routine employed an approximation method developed by Jelesnianski and Taylor (1973). The estimate indicated that the resolution of the eye position was not adequate, i.e., the simulated eye was passing approximately 10 nautical miles (n.mi.) north of its actual track. Modification of the hurricane data base was not desired if another remedy could be identified; therefore, the coordinates of the computational grid (i.e., the WIFM grid) were adjusted, as shown in Figure 16, to shift the grid into proper alignment with the simulated storm track. Resulting correlation between measured and predicted wind fields was improved but still not acceptable. This presented a problem since HURWIN input consists entirely of the information in the hurricane database, and modification of those values could not be justified.

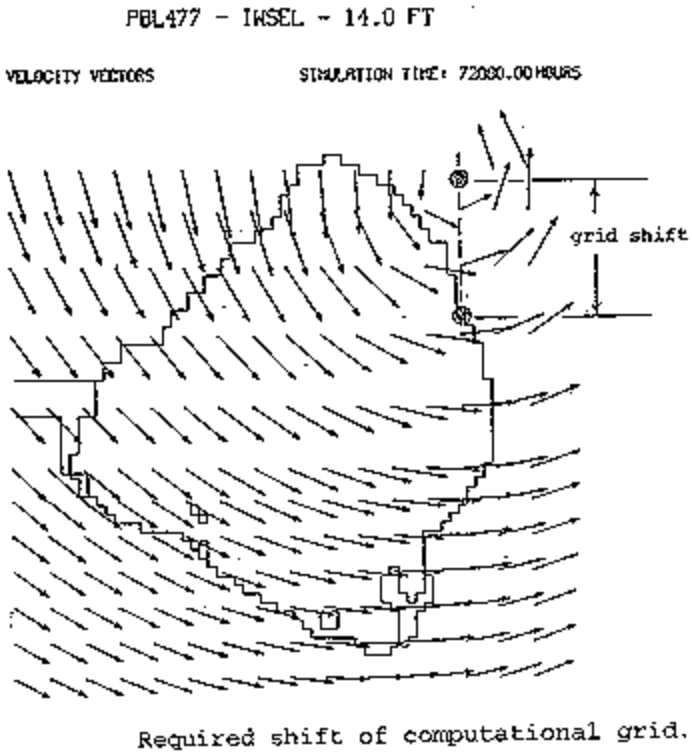


Figure 15. Required shift of computational grid.

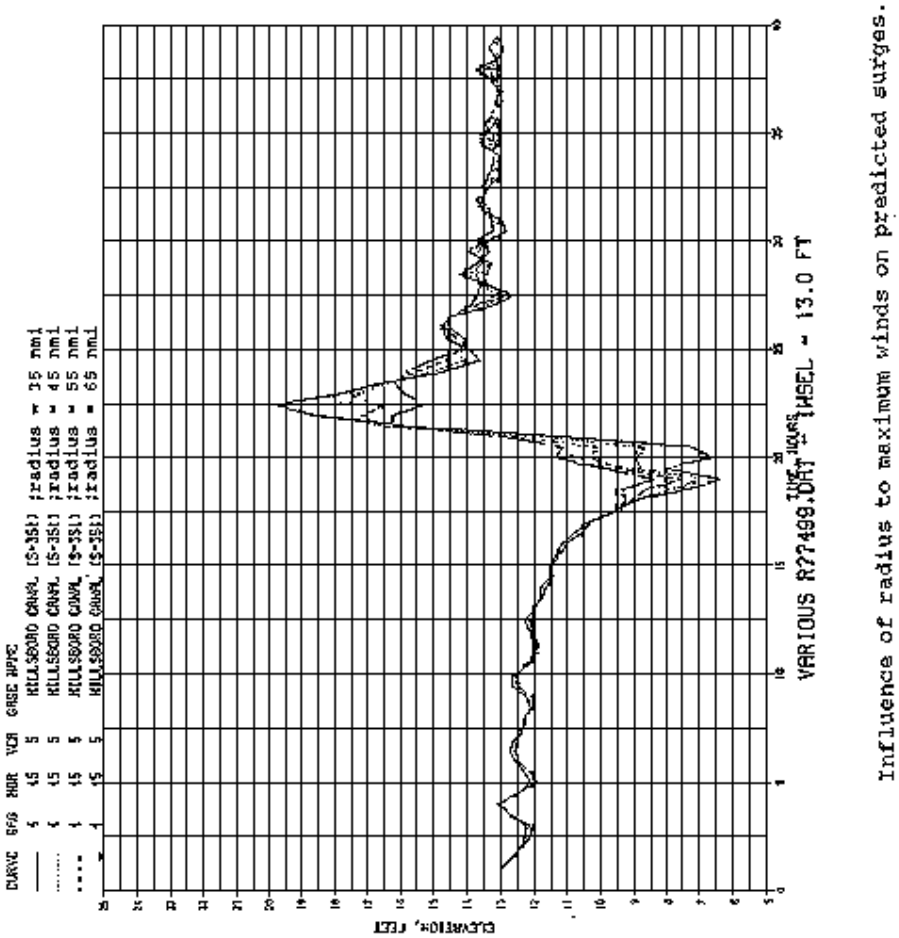
While identifying HURWIN limitations, Thompson and Cardone (1994) indicated that the model does not consider the simulation of transitional stages of the boundary layer across roughness discontinuities, as when a hurricane passes from water to land. Also, as mentioned previously, a pre-processing routine was used to estimate radius to maximum winds based on the pressure deficit and windspeed specified in the database. Since the study area is located approximately 55 km inland, these factors and their influence on the simulated radius to maximum winds were suspected as possible sources of the windfield discrepancies. For the 1949 hurricane, the pre-processing routine results identified the radius to maximum winds at the lake as 9 nautical miles. It is reasonable to assume that after passing overland, energy dissipation due to friction effects would have resulted in a wider radius. HURWIN runs involve the use of a pre-processing routine designed to analyze the NHC data base and obtain the descriptive parameters corresponding to the selected storm. The pre-processing software also contains the software which computes the maximum wind radius. It then compiles the six hour snapshots of hurricane information in a format suitable for use by the HURWIN model.

The most favorable method for adjusting the radius to maximum winds was through manual alteration of the file created by the pre-processing routine. It was decided that although the actual radius to maximum winds was unknown, the wind field could be optimized by the following procedure:

1. Execute pre-processing routine to generate HURWIN input.
2. Manually modify the radius to maximum winds in the HURWIN input file.
3. Execute HURWIN.
4. Execute WIFM.
5. Evaluate the effect of the radius to maximum winds adjustment by inspection of the measured and predicted surge characteristics at all meteorological stations.

Results of this procedure are depicted in Figure 16 which shows the strong influence of variations in the radius to maximum winds on surge elevation at a given site. Although the shape of the surge profile is maintained, the magnitudes of the surge elevations at this site were significantly increased with each reduction of the maximum wind radius. This exercise resulted in selection of a 50 nautical mile radius to maximum winds for simulation of the 1949 hurricane (storm 477); therefore, values in the HURDAT input file were

changed from 9 n.mi. to 50 n.mi. for subsequent use in the WIFM calibration.



Influence of radius to maximum winds on predicted surges.

Figure 16. Influence of radius to maximum winds on predicted surges.

WIFM Calibration. WIFM calibration was accomplished by adjusting bottom coefficients, which are input to the model as Manning's n-values. Coefficients are entered as a function of depth (or elevation for land cells); therefore, at the beginning of a simulation, a coefficient is assigned to each

grid cell based on its bathymetric or topographic value. That friction factor remains constant throughout the simulation, regardless of flooding or drying effects.

The use of WIFM output during HURWIN calibration had indicated that initially selected values of the friction factors were yielding reasonable results; however, further adjustments of the Manning's n-values and corresponding depths were made in order to ensure the best selection. The friction factors ultimately selected ranged from 0.015 in the deepest portions of the lake to 0.030 in heavily vegetated nearshore areas. Comparisons of measured and predicted surges at the nine meteorological stations are depicted in Figures 18 through 26. The figures indicate that fair correspondence between measured and predicted values was achieved. The model underestimated the 23.0 foot surge elevation experienced at Okeechobee, and overestimated the surge peak at Moore Haven. Measured and predicted maximum surge values are presented in Table 9. The percent error (relative to peak surge elevations) ranged from 0% at Canal Point to 21% at Moore Haven. Average error for the all stations considered was 9%. Although more favorable calibration results were desired, further adjustments of the maximum wind radius and/or friction factors failed to improve the correlation.

Difficulties encountered during calibration of the hydrodynamic model soon revealed the complexities involved in the simulation of storm surges on Lake Okeechobee. To reach and influence the study area, storms pass over the ocean, make landfall, proceed some distance overland, and then pass over the lake. The difficulties in simulating the wind and pressure field adjustments that occur over these transitions are significant. Also, the lake is a relatively small body of water (approximately 1900 km^2

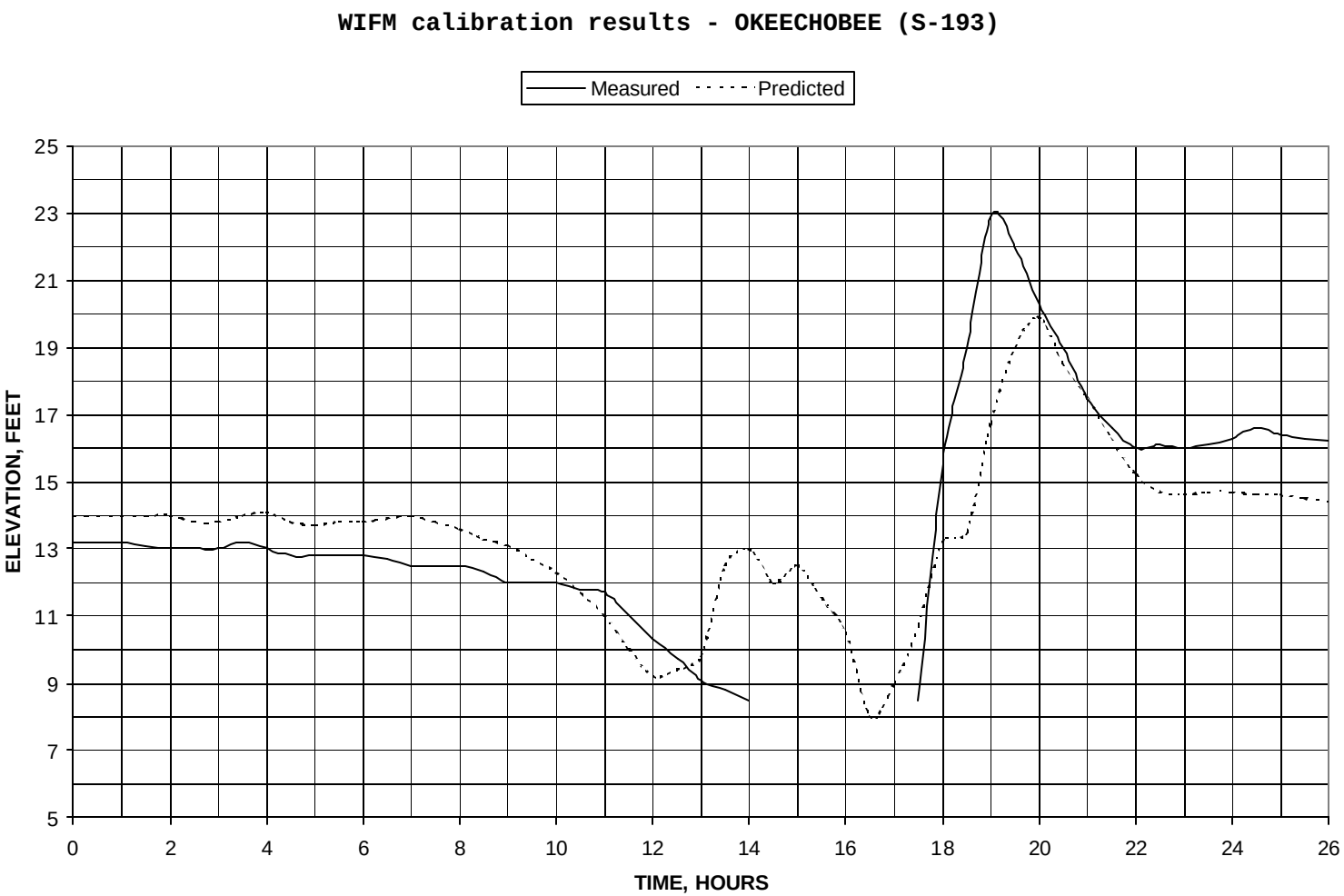


Figure 17. WIFM calibration results – Okeechobee (S-193)

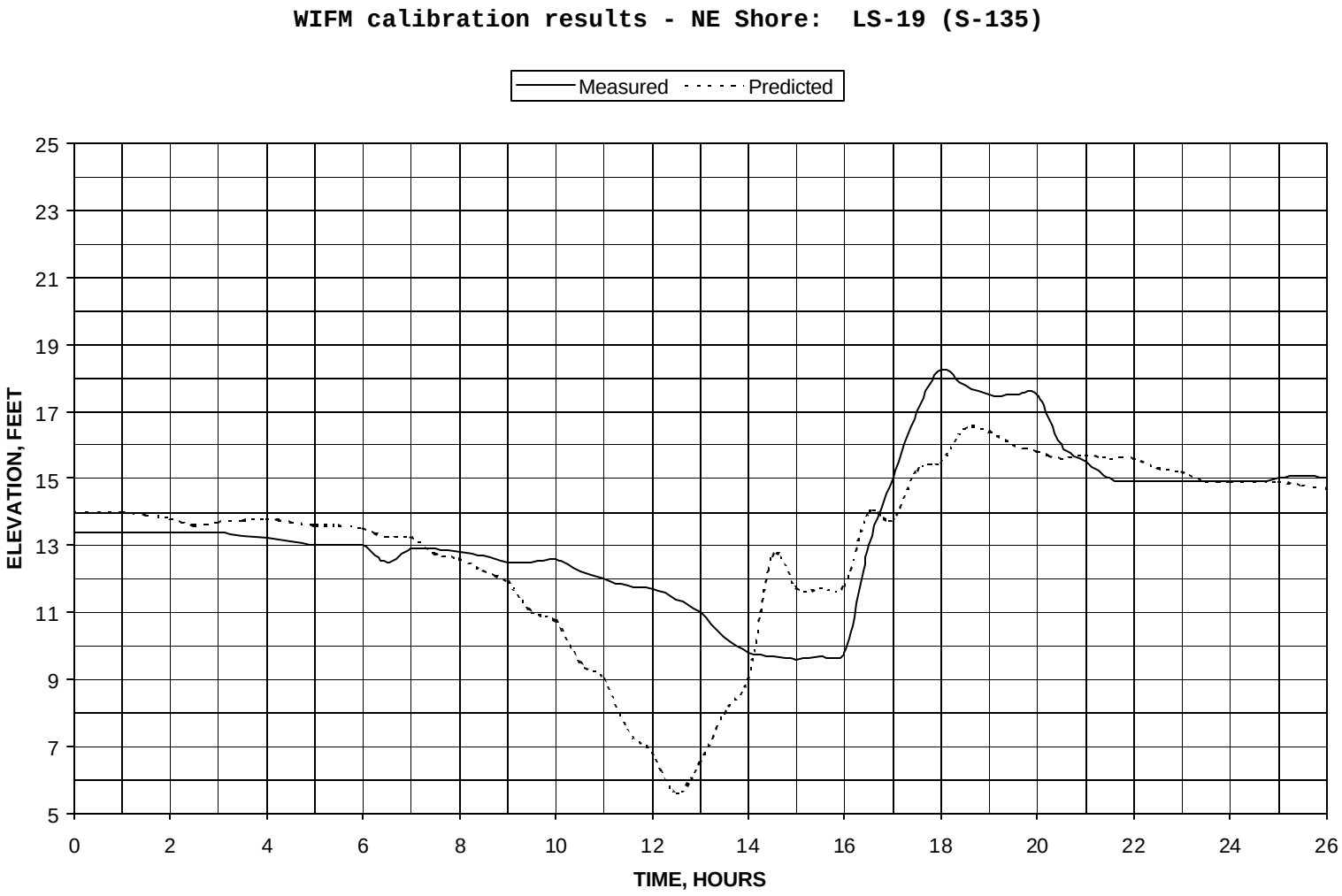


Figure 18. WIFM calibration results – NE Shore: LS-19 (S-135)

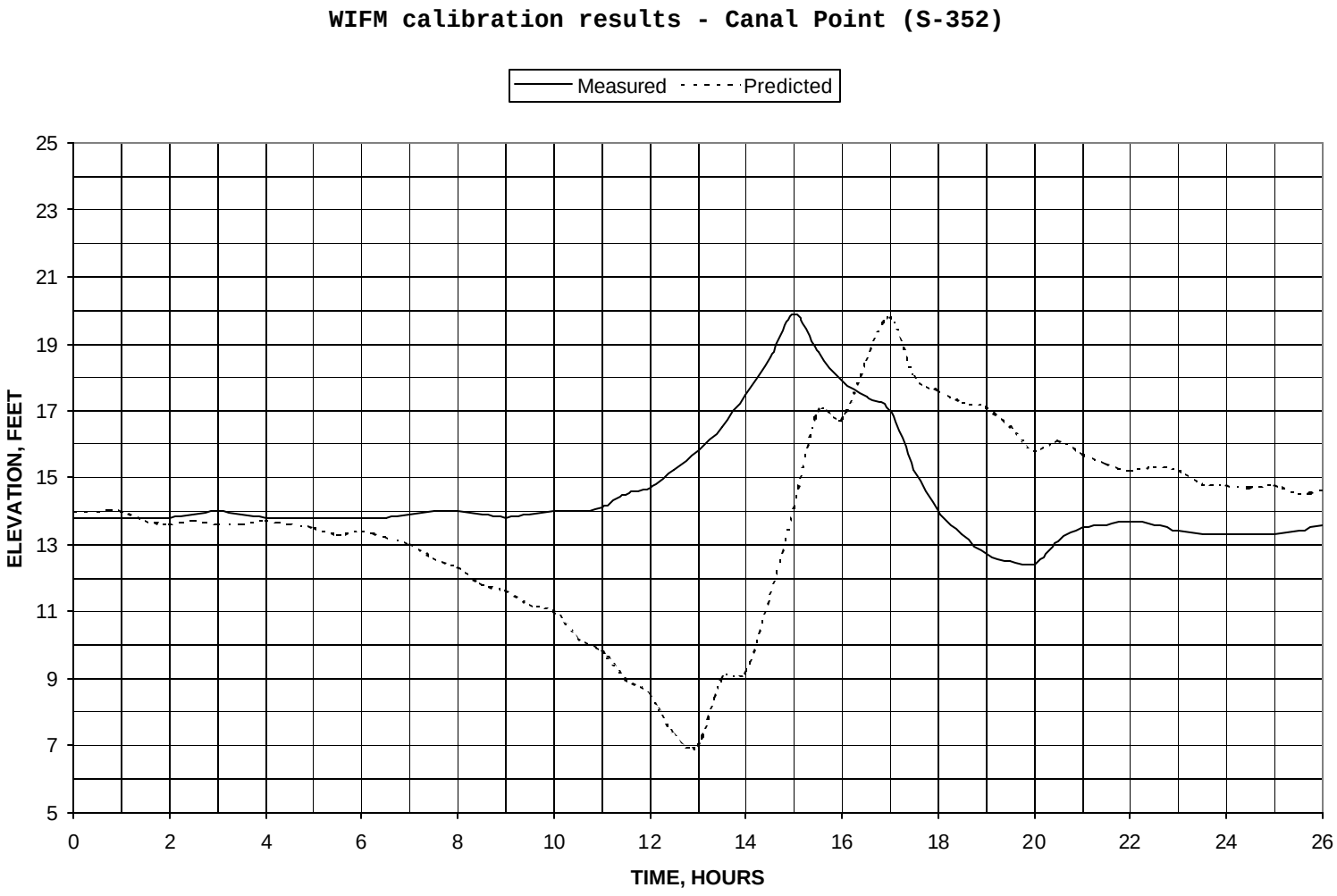


Figure 19. WIFM calibration results – Canal Point (S-352)

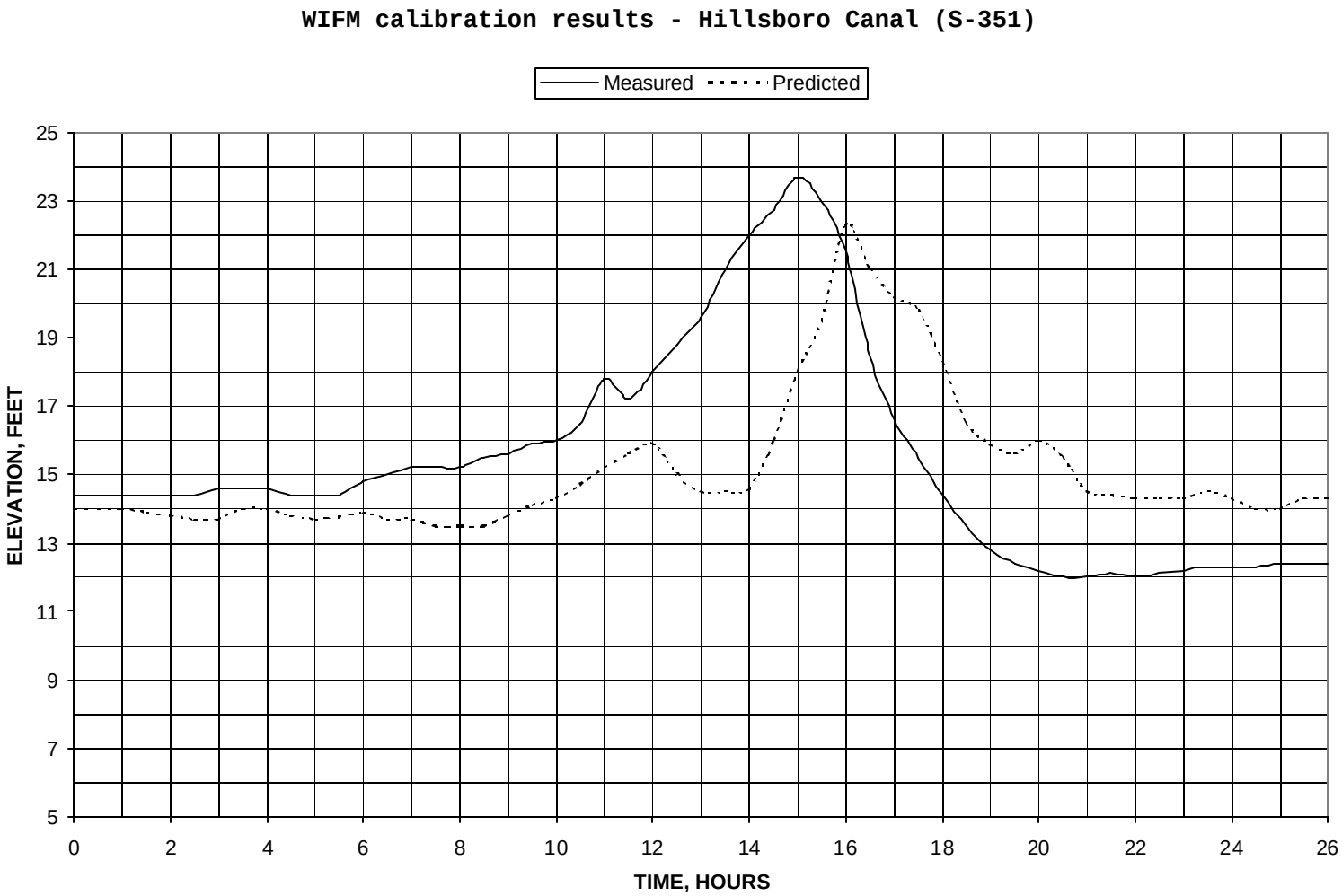


Figure 20. WIFM calibration results – Hillsboro Canal (S-351)

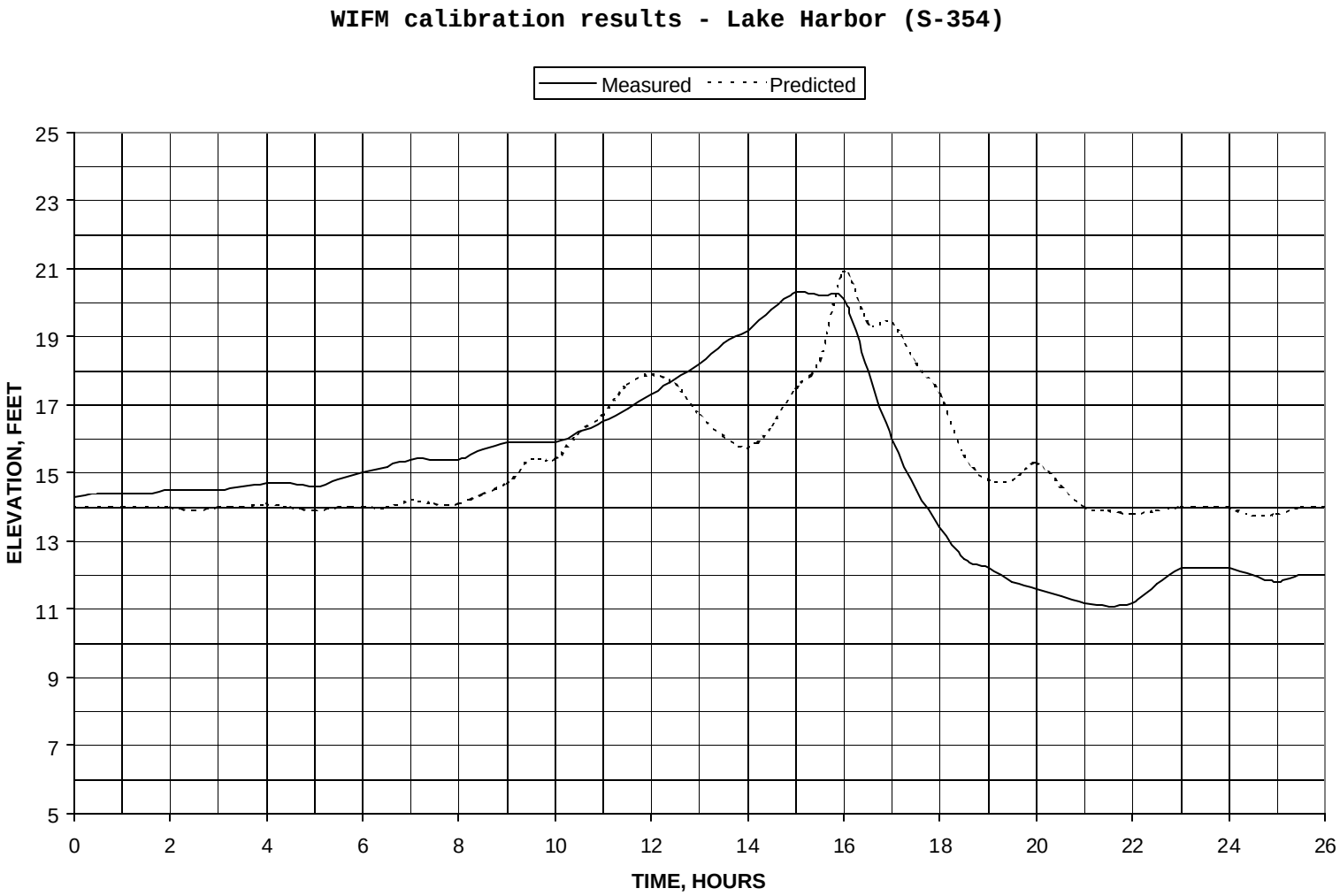


Figure 21. WIFM calibration results – Lake Harbor (S-354)

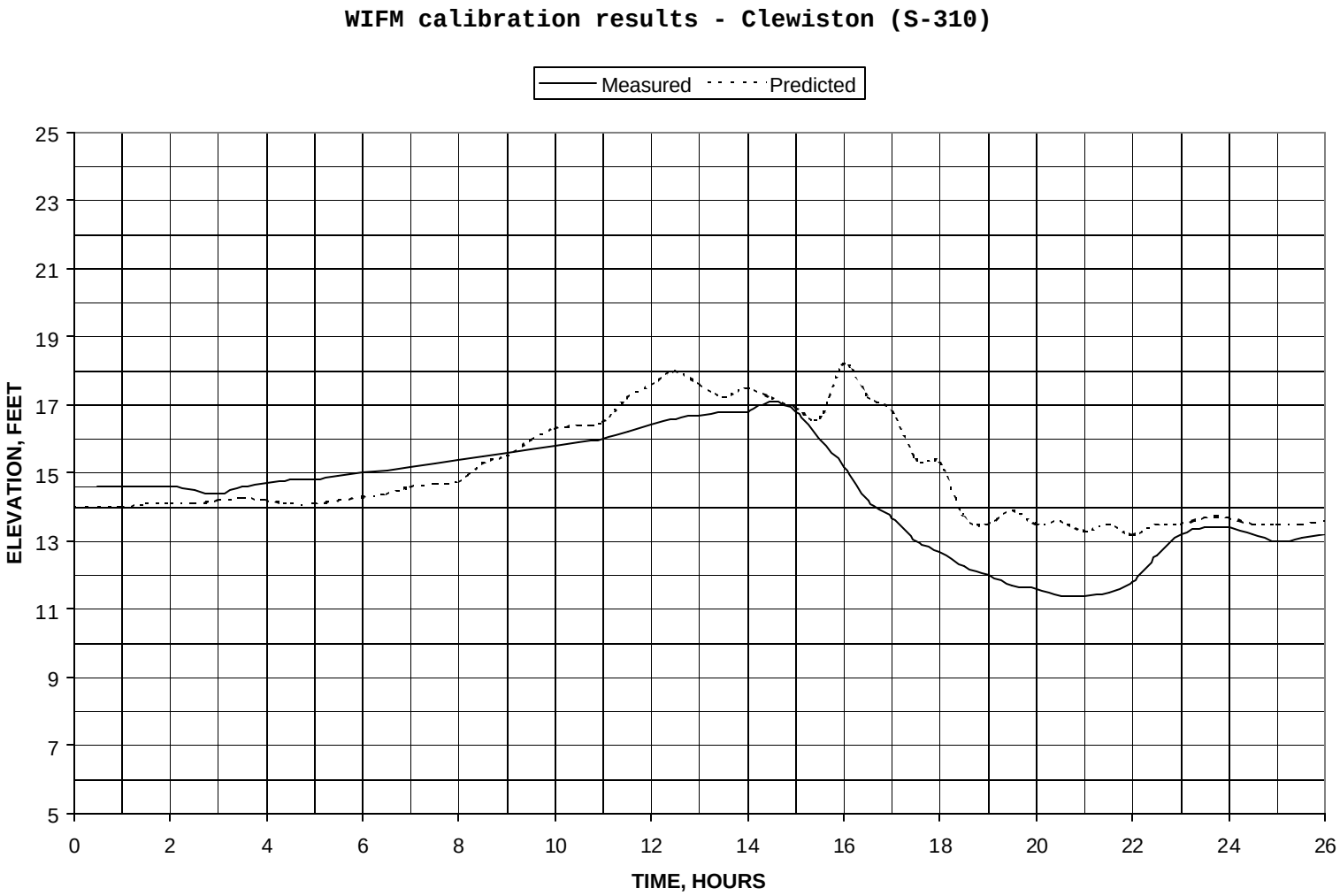


Figure 22. WIFM calibration results – Clewiston (S-310)

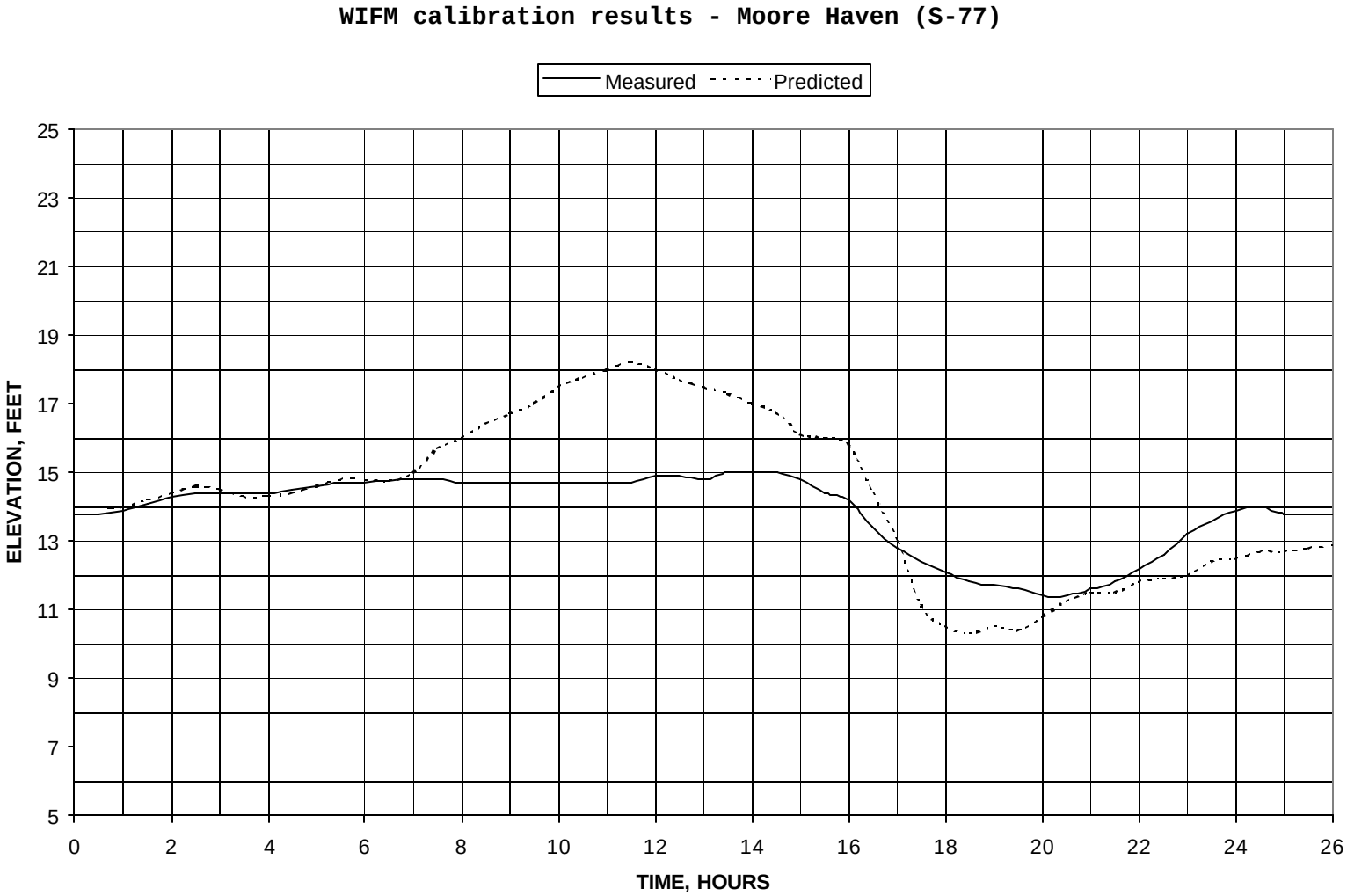


Figure 23. WIFM calibration results – Moore Haven (S-77)

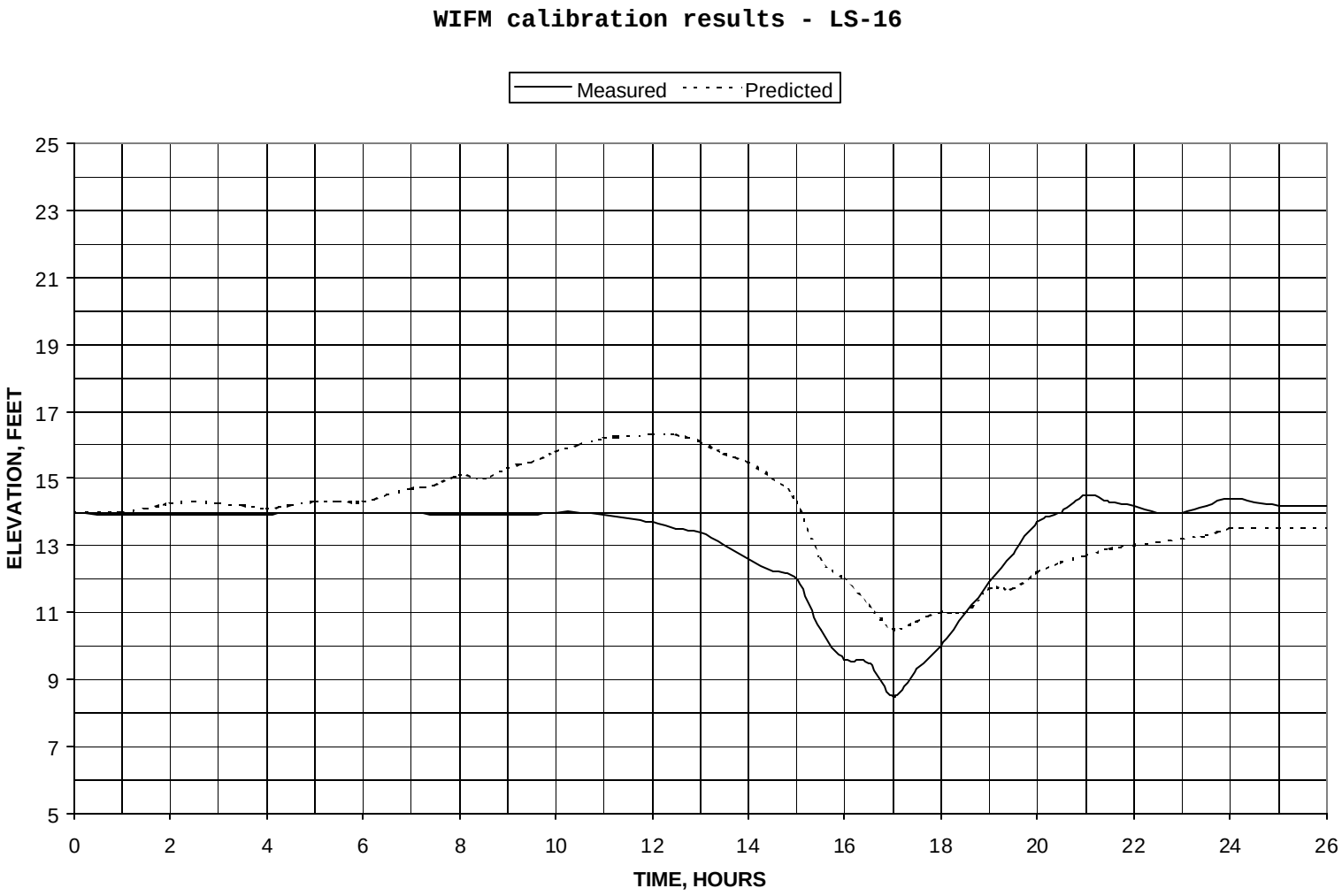


Figure 24. WIFM calibration results – LS-16

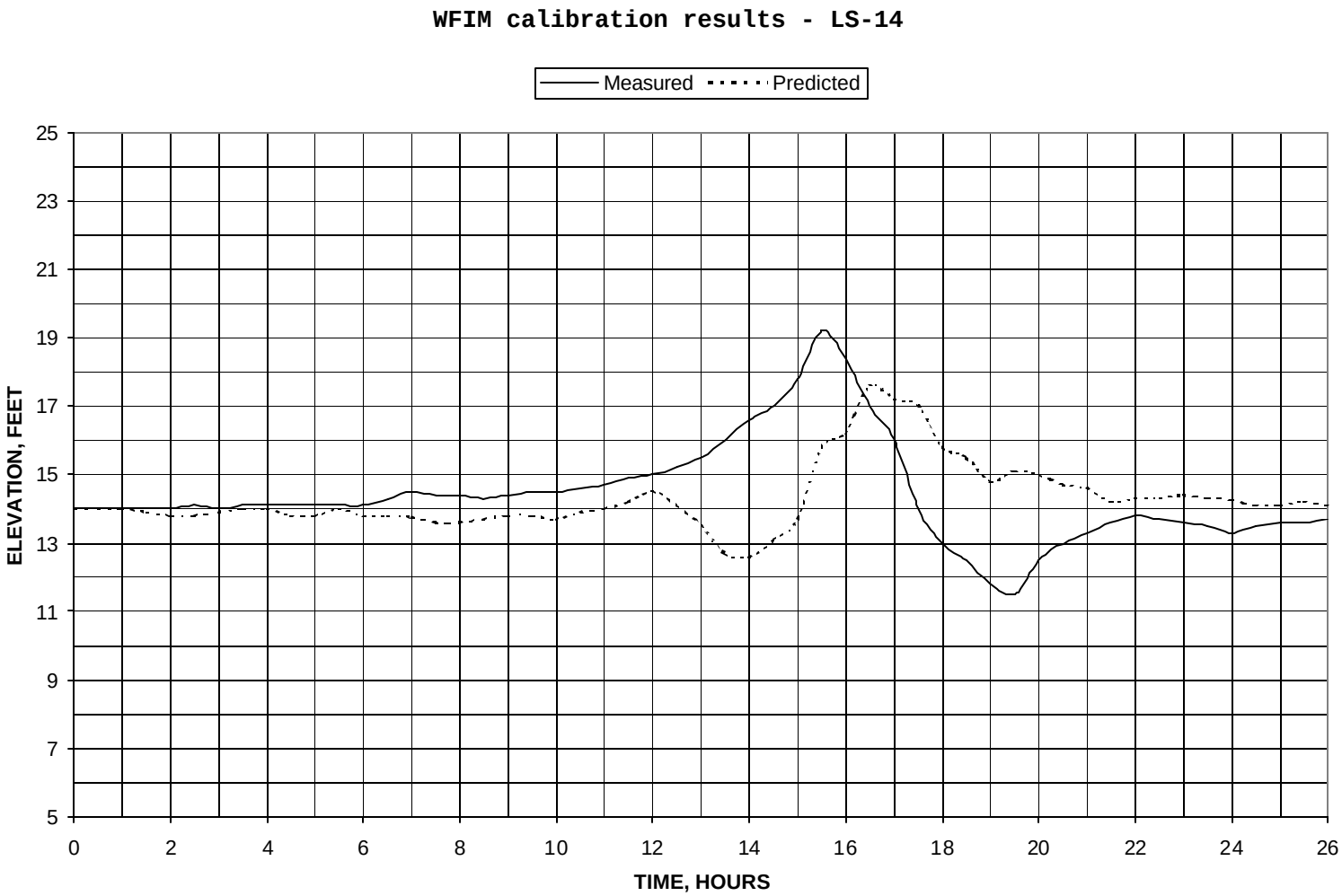


Figure 25. WIFM calibration results – LS-14

Table 9. Calibration results - August 1949 hurricane.

Station	Maximum Surge Elevation (ft, NGVD) Measured	Predicted	% Error
Okeechobee	23.0	20.0	13
LS-19	18.3	16.5	10
Canal Point	20.0	20.0	0
Hillsboro Canal	23.8	22.4	6
Lake Harbor	20.3	20.9	3
Clewiston	17.1	18.3	7
Moore Haven	15.0	18.2	21
LS-16	14.5	16.3	12
LS-14	19.2	17.6	8

surface area) which is extremely sensitive to variations in the rotational windfield. The long period water surface oscillations at a given site on the lake are dependent on all of the input storm variables, especially the storm track, wind speed, and radius to maximum winds.

Although the correlations of measured and predicted surge magnitudes were not exact, the results indicated that the hydrodynamic processes were represented fairly well. Surge time histories showed that the occurrence of water surface recessions (drawdowns) as well as surge phenomena were well matched at most stations. Although the models may not produce accurate surge predictions at all locations for a given hurricane, it is apparent that the results are reasonable. Discrepancies are attributed more to the user's inability to accurately match all of the historical input variables, rather than to capabilities of the hydrodynamic model.

As noted previously, the purpose of the numerical modeling effort was to generate surge responses for inclusion in a data base representative of the project site's historical record. It is not necessary for that data base to contain precise predictions of every surge occurrence at every station for each historical storm; however, it should provide good representative ranges of forcing variables and corresponding surge responses. This ensures that the final full population of hurricane data generated by the EST, from which the statistics are computed, is representative of the historical events. The resulting surge-frequency relationships should therefore be accurate and reliable.

After consideration of the above issues, the calibration results were deemed acceptable and the verification procedure was initiated.

MODEL VERIFICATION - Initial verification of the HURWIN and WIFM models was performed with the hurricane of 16 September, 1928 (storm 292). It was chosen due to the similarity of its track to that of the 1949 calibration storm (storm 477). Although measured wind information does not exist for this storm, surge elevations were estimated at two locations based on post-storm high water marks. The September 1928 hurricane is the most severe storm to have ever influenced the project area. The War Department (1940) reported that this storm "produced conditions believed to approach the most critical that may be expected to occur on the lake as a result of a hurricane." That report estimated maximum wind speeds at Lake Okeechobee to be approximately 110 knots. The NHC data base estimate is 115 knots. At the time of the storm, the few protective levees which existed along the south shore were not adequate. They were overtopped by depths of 2 to 5 feet and over 2,000 people died in the subsequent flooding.

Further model verification was performed using hurricane King (storm 499) and hurricane David (storm 777). These storms were selected due to the availability of corresponding wind, pressure, and surge measurements, and because they represented distinctly different storm tracks when compared to the August 1949 storm. Hurricane King made landfall at Miami and proceeded northward, eventually passing directly over Lake Okeechobee. While in the study area, King was characterized by maximum wind speeds of about 75 knots. Hurricane David skirted the east coast of Florida never actually making landfall in the immediate study area. When directly east of Lake Okeechobee, hurricane winds approached 85 knots.

<u>September 1928 Verification</u> - Calibration with the August 1949 hurricane suggested that the radius to maximum winds was directly related to the overland distance traversed. Both the August 1949 storm and the September 1928 storm traveled approximately 35 n.mi. overland (along nearly identical tracks) before arrival at the study area; therefore, based on the suggestion above, the 50 n.mi. radius to maximum winds established during calibration was applied to both of these storms. HURWIN and WIFM simulations were executed and the resulting track accurately represented the historical event. This implied that the WIFM grid adjustment implemented in the

calibration phase was effective in matching the NHC data base track locations to the historical track locations. The two available historical surge heights (based on high water marks) were reported as 13.2 feet at the south end of the lake and 3.0 feet on the northeast shore. Corresponding predicted surges were 12.5 feet and 3.2 feet, respectively; therefore, model performance was acceptable for this verification test.

Hurricane King Verification - The August 1949 storm (storm 477) and hurricane King (storm 499) traveled overland for approximately 35 n.mi. and 60 n.mi., respectively, prior to arrival at the study area; therefore, one would expect the radius to maximum winds associated with hurricane King to be greater than the 50 n.mi. radius established for the August 1949 storm. HURWIN and WIFM runs were first performed with the 50 n.mi. radius to maximum winds. Predicted time histories of the water surface elevation at all meteorological stations compared relatively well with measured data; however, at most stations the maximum surge value was overpredicted. To evaluate the sensitivity of the results to the maximum wind radius, additional HURWIN and WIFM runs were executed with radii of 35, 45, 55 and 65 nautical miles. The best agreement between measured and predicted surges was accomplished with the 65 n.mi. radius to maximum winds. A summary of the comparison is presented in Table 10. These results supported the earlier assumption that a storm's overland track distance is directly proportional to the maximum wind radius.

Table 10 indicates that the predicted surge elevations (using a 65 n.mi. radius to maximum winds) corresponded reasonably well to the measured data. The percent error ranged from 1% at Okeechobee and LS-12, to 15% at LS-18. Average error for the all stations considered was 8%. The agreement between measured and predicted results obtained during the hurricane King verification was considered adequate. This verification exercise also established the radius to maximum winds for hurricane King at 65 n.mi. while in the study area; therefore; future simulation of storms with tracks similar to hurricane King's, would be performed with a radius to maximum winds of 65 nautical miles.

Table 10. Maximum surge versus radius to maximum winds - Hurricane King

	-------- Surge Elevations (ft, NGVD) ----------					
Station	25 nmi	35 nmi	45 nmi	55 nmi	65 nmi	Measured
Okeechobee	21.9	19.7	18.1	16.9	16.5	16.3
LS-19	17.2	15.2	14.8	14.5	14.2	15.0
Canal Pt	19.0	17.1	16.5	16.1	15.6	13.6
Hllsbro Cn	22.3	19.8	18.1	17.2	16.2	14.5
Lake Harbr	21.1	18.7	17.2	16.4	15.6	17.9
Moore Hvn	20.9	18.9	18.2	17.8	17.4	15.8
LS-18	22.0	20.1	19.5	18.5	18.2	15.7
LS-16	17.9	16.7	16.3	16.0	15.7	15.3
LS-14	17.6	16.3	15.5	14.8	14.6	14.2
LS-12	16.6	15.4	15.1	14.8	14.6	14.8
LS-10	18.0	17.6	16.6	15.6	15.5	14.0

Hurricane David Verification - Hurricane David (storm 777) occurred on 3 September 1979. Levee and tree island construction was completed in the 1960's; therefore, this verification involved model simulations with boundary conditions representative of the present conditions. Hurricane David did not make landfall in Florida; however, it skirted the coastline immediately east of the study area and created a relatively minor surge response on the lake. Since no portions of the track proceeded overland, the radius to maximum winds was suspected to be significantly less than the 50 and 65 nautical mile values characteristic of storms 477 and 499, respectively. Measured data at four meteorological stations was available for comparison. HURWIN and WIFM runs were first performed with the 50 n.mi. radius to maximum winds. Comparison of results with measured surge data indicated that maximum surges were overpredicted. Additional runs were performed with the radius to maximum winds set at 20, 30, 40, and 9 nautical miles. The 9 n.mi. radius represents the value established by the pre-processing routine, and this value resulted in the best agreement between measured and predicted surges. A summary of the comparison is presented in Table 11. Again, these results support the earlier assumption that a storm's overland track distance is directly proportional to the maximum wind radius. Since hurricane David did not proceed overland, the pre-processing algorithm for computing the radius yielded a more effective estimate than it had for storms 292, 477, or 499.

Table 11. Maximum surge versus radius to maximum winds – Hurricane David

Station	20 nmi	30 nmi	40 nmi	50 nmi	9 nmi	Measured
	---------- Surge Elevations (ft, NGVD) ----------					
Port Mayaca	14.9	15.0	15.1	15.2	14.8	17.7
Clewiston	18.5	19.0	19.0	18.8	16.9	15.5
LS-16	16.7	17.0	17.0	17.0	15.9	14.7
LS-14	16.7	17.2	16.9	16.8	15.8	15.8

Table 11 indicates that the predicted surge elevations (using a 9 n.mi. radius to maximum winds) corresponded reasonably well to the measured data. The percent error ranged from 0% at LS-14, to 16% at Port Mayaca. Average error for all stations considered was 8%. Based on consideration of the previous verification and calibration results, verification of hurricane David conditions was considered acceptable.

Development Of Surge-Frequency Relationships - Surge-frequency relationships for Lake Okeechobee were developed by combining the three major analysis tools described in previous sections of this report. As explained in the project overview, the first task involved identification of all historical hurricanes which have influenced the study area. The HURWIN model was executed to develop the wind and atmospheric pressure fields characteristic of those storms. HURWIN output files were then used as input for execution of the hydrodynamic model, WIFM. The hydrodynamic response was recorded for each of the Lake Okeechobee stations where surge-frequency relationships were desired. For each storm, the resulting surge elevations were combined with the input variables describing that storm to form the data base used in the empirical simulation procedure. Execution of the EST then resulted in computation of the surge-frequency relationships. Further discussion of the methods employed is provided below.

Hurricane Selection - Historical hurricanes which may have influenced Lake Okeechobee were identified through analysis of the storm track information in the NHC data base. Any storms which passed within 120 kilometers of the lake center were considered for further analysis. These storms were identified by establishing a rectangular area around the lake with boundaries extending from latitude 25°50' North, longitude 82°00' West, to latitude 28°00' North, 79°40' West. Hurricane tracks which fell within those boundaries identified corresponding storms of interest. This procedure indicated that of the 875 hurricanes and tropical storms in the 104-year

NHC database, 30 storms have occurred which may have influenced the lake. Seven of these storms had weakened considerably by the time they reached the project area; therefore, they were omitted. The remaining 23 hurricanes were included in the historical data base. Characteristics of those historical storms (and two synthetic storms) are summarized in Table 12. Storm dates and maximum wind speeds reflect those conditions when the storm was in the project area.

Table 12. Storms affecting Lake Okeechobee.

Name	NHC Number	Date	Maximum Wind Speed (Kts)
Unnamed	127	08/11/01	40
Unnamed	141	09/11/03	65
Unnamed	194	10/17/10	65
Unnamed	271	07/27/26	90
Unnamed	276	09/18/26	110
Unnamed	289	08/08/28	85
Unnamed	292	09/16/28	115
Unnamed	324	07/31/33	65
Unnamed	331	09/03/33	110
Unnamed	357	11/04/35	65
Unnamed	449	09/16/45	110
Unnamed	461	09/17/47	120
Unnamed	465	10/12/47	65
Unnamed	473	09/18/48	85
Unnamed	477	08/27/49	100
King	499	10/18/50	75
Unnamed	521	08/29/53	35
Hazel	530	10/09/53	60
Donna	597	09/10/60	115
Cleo	629	08/27/64	85
Isbell	635	10/14/64	110
David	777	09/03/79	85
Keith	864	11/23/88	35
*Andrew (easterly track)	876		110
*Andrew (southwesterly track)	878		110

* denotes synthetic hurricanes

The historical data base of hurricanes was expanded by addition of two synthetic storms. Hurricane Andrew struck the Miami area on 24 August, 1992. It followed a predominantly east to west track and passed well south of Lake Okeechobee, creating only a very minor wind surge on the lake; however, a

shift in the storm track of only about 80 kilometers could have brought the eye directly over Lake Okeechobee. Both synthetic storms (876 and 878) were created by applying the hurricane Andrew conditions along adjusted tracks. Hurricane 876 was designed to follow an east to west track over the lake as shown in Figure 27a. The second synthetic storm (878) represented Andrew conditions passing over the lake on a southwest to northeast track, as shown in Figure 27b.

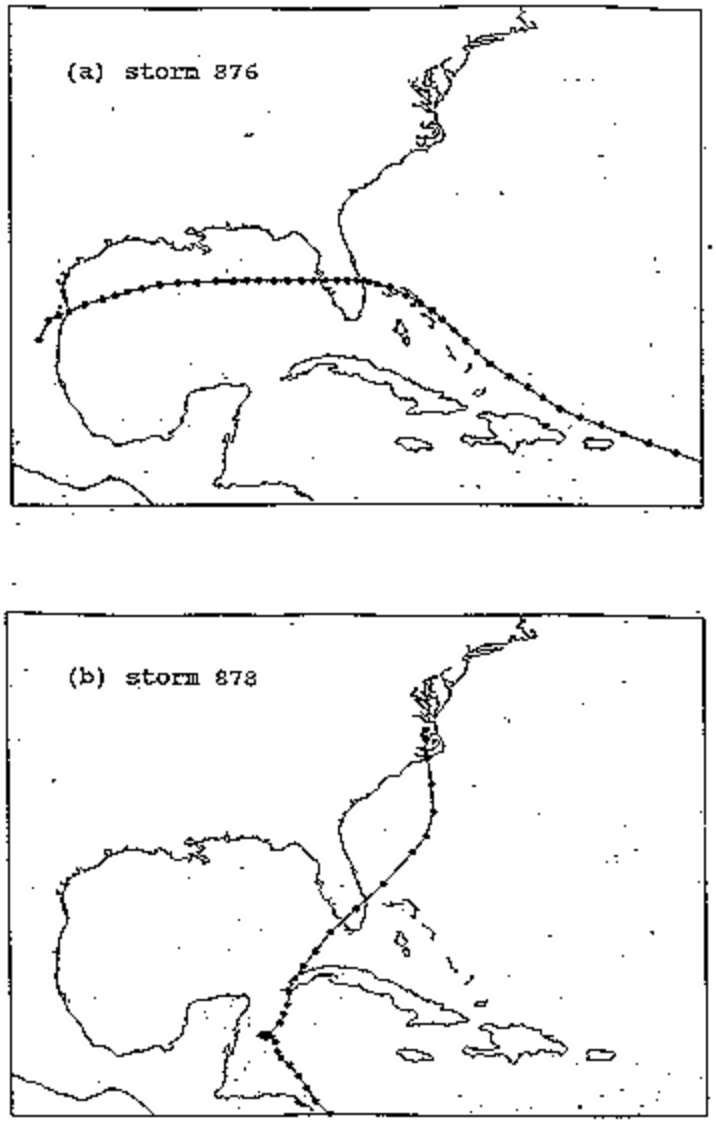


Figure 27. Tracks of synthetic storms 876 and 878.

Figure 26. Tracks of synthetic storms 876 and 878.

HURWIN Modeling - The NHC data base of hurricane information contains 6-hour snapshots of hurricane eye pressure, eye location, and maximum wind speed for the entire life of a selected hurricane. This project required simulation of wind and pressure fields only during those periods when a storm was in the vicinity of south Florida; therefore, for each hurricane, threshold latitude and longitude values were established to capture only the wind and pressure output corresponding to periods when the storm was approaching, influencing, and leaving the project area. For most hurricanes, this resulted in about 35 to 40 hours of wind and pressure data.

HURWIN input files were created by execution of a pre-processing routine. This program was designed to enter the NHC data base and extract the snapshot information associated with an individual storm specified by the user. The program also executed the radius to maximum wind, track angle and forward speed calculations; therefore, the pre-processing routine resulted in development of a HURWIN input file containing six hour snapshots of eye latitude, eye longitude, eye pressure, peripheral pressure, track angle, forward speed, and radius to maximum winds. At this point, the HURWIN input file was inspected to determine if the computed radius to maximum winds corresponded to the chosen values established during calibration and verification. Track types and associated radius values are presented in Figure 28. If the values did not correspond, then the input file was edited to assign the radius to maximum winds a value of 9, 50, or 65 n.mi., depending on the track characteristics. The resulting file was then used as input for the HURWIN simulation.

This procedure was repeated for each of the 25 storms in the augmented historical data set. Resulting wind and pressure fields were generated and saved for use as input to the hydrodynamic model.

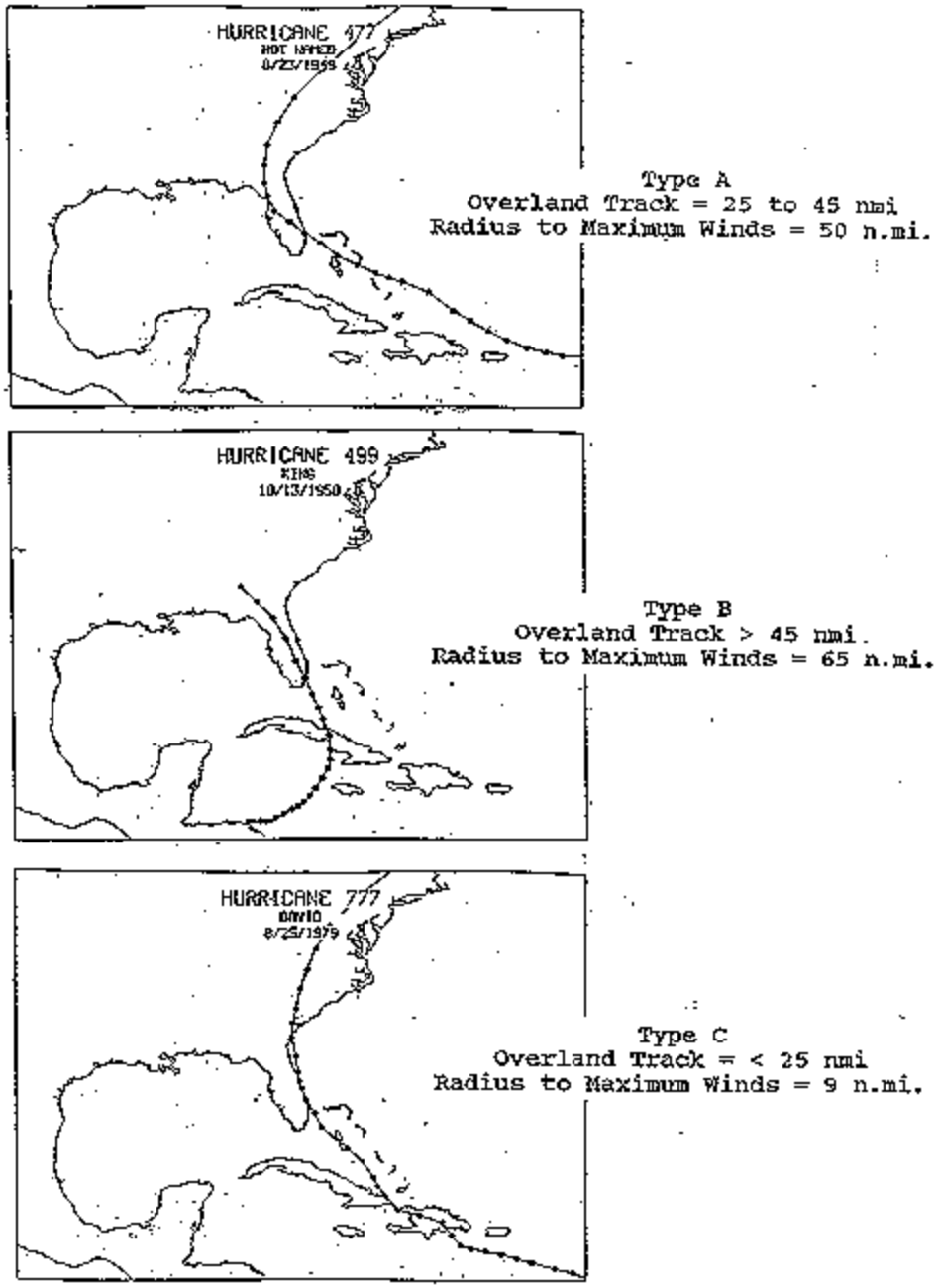


Figure 28. Track types and corresponding maximum wind radii.

Figure 27. Track types and corresponding maximum wind radii.

<u>WIFM Modeling</u> - Preliminary WIFM runs were executed to evaluate the sensitivity of surge predictions to the initial water surface elevation in the lake. Since friction factors are based on bathymetric elevations, not water depths, bottom stresses were not influenced; however, surge variations with

depth would be expected as a result of the flooding and drying processes occurring at the shoreline (including Kreamer Island and the tree islands). This analysis indicated that variations in surge elevation
with depth changes were negligible; therefore, all storms were modeled with an initial water surface elevation of 15.5 feet, NGVD, the average lake stage based on records from 1933 to 1993. The hydrodynamic response of Lake Okeechobee was simulated for each of the 25 storms in the data set. Time series of water surface elevations were recorded at 23 stations in and around the lake, as depicted on Figure 15. Stations were selected at population centers, major USACE navigation and flood control structures, and locations where meteorological instruments have been located. Maximum storm surge values at each station were extracted from the time series for use as response variables in the empirical simulation technique analysis. Snapshots of water surface elevations, and current velocities at each grid cell were also recorded for each storm during periods of maximum influence on the lake.

Empirical Simulation Technique

To begin the EST, the user must specify which storms in the NHC data base are included in the training set (i.e., historical and synthetic storms). The empirical simulation technique is then executed via three major steps, as described below.

1. The first step involves identification of the input variables associated with each storm. The NHC data base is processed to determine the hurricane input variables. This processing includes interpolation of the 6-hour incremental values to compute hourly input variables. For each of the 23 lake stations, input variables are identified at the time when the hurricane is closest to that station. Input variables for the EST are: maximum wind speed, radius to maximum winds, atmospheric pressure deficit, forward speed, track angle, and the minimum distance between the station in question and the hurricane eye.

2. For each storm modeled, maximum surge values at each station were identified from the WIFM results. These surge values represent the response variable which corresponds to the input variables identified in step one above. The second step is simply the combination of corresponding input and response vectors to form the input data set used in the EST.

3. With the input file created, the empirical simulation technique is executed. For this application, the EST performed 100 simulations of a 200-year period to create the data base used for the extremal analysis of surges on Lake Okeechobee. During this procedure, each 200-year sequence of storms is rank-ordered and a cumulative probability function is computed. Frequency of occurrence relationships are then developed based on the methods described previously. Since the analysis involves 100 simulations of the 200-year time span, 100 individual probability density functions and 100 individual surge-frequency relationships are computed for each station. Figure 29 shows the family of surge-frequency relationships developed during analysis of the Fisheating Creek station (S-131). Each family of surge-frequency curves is then averaged and the standard deviation is computed. The results provide a single surge-frequency relationship (for each station) with a measure of variability of the data spread about that mean value; therefore, the full population of data is reduced to a plot of the mean value bracketed by plus and minus one standard deviation. An example corresponding to the family of curves in Figure 29 is shown in Figure 30. This presentation provides a mean frequency estimate for design as well as an error band, or confidence interval.

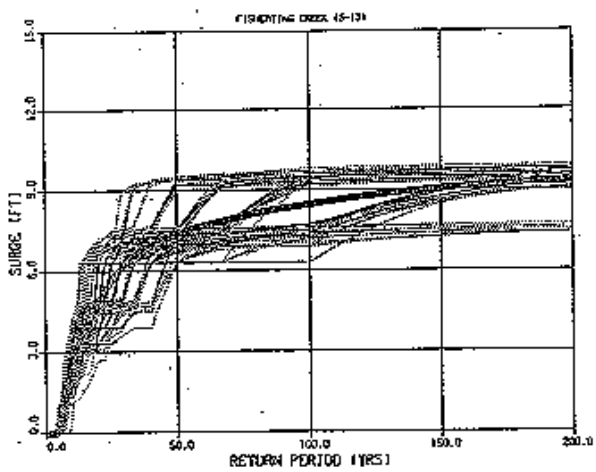


Figure 29. Individual surge-frequency curves - Fisheating Creek

Figure 28. Individual surge-frequency curves - Fisheating Creek

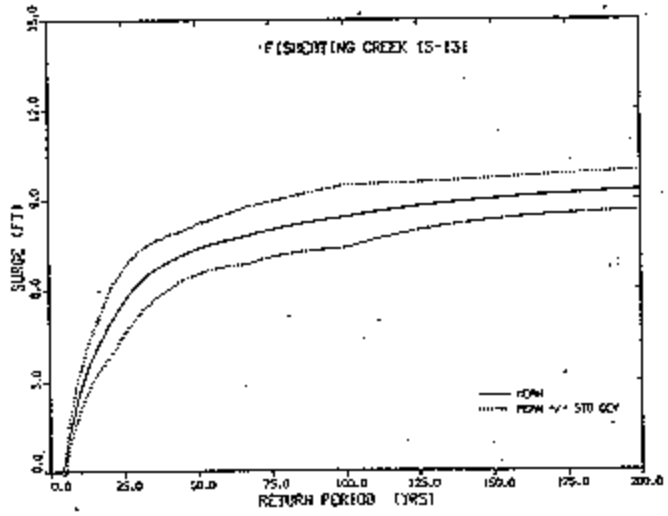


Figure 30. Final surge-frequency relationship - Fisheating Creek

Figure 29. Final surge-frequency relationship - Fisheating Creek

<u>Analysis Of Surge-Frequency Results</u> - The procedure above resulted in generation of surge-frequency relationships for the 23 stations of interest. Tabular and graphical surge-frequency relationships for each station are presented at the end of this appendix.

The results indicate that surge elevations associated with higher return periods (i.e., 100-year and greater) tend to be largest along the lake's southern shore. Comparison of 100-year return periods identified the maximum at Moore Haven, where the 100-year surge is estimated at 13.4 feet above the initial water surface elevation. Other stations exhibiting relatively high 100-year surges include Pelican Bay, LS-10, Hillsboro Canal, Lake Harbor, Clewiston, Liberty Point, and LS-18, all which lie along the southern shore of Lake Okeechobee. The concentration of higher surges along the south shore can be explained based on characteristics of the historical data set. The most severe hurricanes influencing the study area have typically approached along a southeasterly track (i.e., from southeast to northwest), with the eye passing north of the lake center. The cyclonic (i.e. counterclockwise) nature of the wind fields associated with those storms direct the maximum surface stresses toward the

southwest; therefore water is forced to the southwest corner of the lake and, to a slightly lesser extent, to the bay area at the lake's extreme southern end. Geometric characteristics of the lake features also contribute to the high surge levels near Lake Harbor, Hillsboro Canal, and Pelican Bay. The shoreline in this area forms a large bay which is subsequently divided into two lesser bays by the presence of Kreamer Island. Volumes of water being forced south are constricted when they reach these bay-like areas. As the width of the basin decreases, conservation of mass dictates that the water surface must increase.

Surge values characteristic of stations along other sections of the lake were significantly lower. The least severe surge behavior was exhibited at stations on the northwest and northeast shores. Those stations include Indian Prairie Canal, Kissimmee River, Henry Creek Lock, LS-19, and Port Mayaca. Most influential storms have been characterized by wind fields which tended to direct maximum surface stresses away from these areas; therefore, time histories of water surface elevation at these stations often displayed a prominent drawdown followed by some level of surge as the wind direction shifted due to the passing of the storm.

A 100-year surge of 8.5 feet above initial lake stage was established at the northernmost station, Okeechobee. The reason for its greater magnitude relative to adjacent stations is again related to the geometric characteristics of the shoreline. As they proceed northward, the northwest and northeast shorelines converge. Okeechobee lies at the apex of this convergence; therefore, volumes of water driven north are affected by the convergence and as the constriction narrows, a corresponding rise in water surface results.

<u>Investigation Of Est Sensitivity To Hurricane Training Set.</u>
The empirical simulation technique assumes that the forcing and response characteristics of individual storms will be similar to the characteristics of historical events; therefore, the training set, which consists of historical and synthetic storm data must be reasonably representative of the range of hurricane severity expected in the study area. The user must label each storm in the training set to distinguish historical events from synthetic events. This is necessary because the empirical simulation procedure places greater emphasis on the use of historical data during resampling and interpolation routines. If the historical record only contains information corresponding to relatively mild storms,

then it must be augmented with synthetic storms to provide the necessary data associated with extreme events which could occur. To investigate the sensitivity of the surge-frequency results at Lake Okeechobee to the training set, four different scenarios were tested. Each scenario represents a modification of the training set. EST analyses were performed with each training set for comparison of the resulting surge-frequency relationships. Characteristics of the four training sets, and a brief summary of corresponding EST results are presented below.

Training Set 1. The first training set was simply the control set described in all of the previous discussions. It consisted of the 23 historical storms obtained from the NHC data base plus the two synthetic storms created by altering the track of hurricane Andrew.

Training Set 2. The second training set consisted of the 23 historical storms only. No synthetic storms were included.

Plots of the resulting surge-frequency relationships showed no visual evidence of change when compared to the results of the control set, i.e., training set 1. Comparison of tabulated EST results (surge values and return periods) between the two training sets showed variations only on the order of 0.01 foot or less. This suggests that the historical record of hurricanes affecting Lake Okeechobee is a good representation of the range of storm severity characteristic of the study area. The inclusion of severe historical storms such as hurricanes 292 and 477 effectively described the input and response variables characteristic of extreme events; therefore, addition of the synthetic storms was redundant and had no effect on the results.

Training Set 3. As mentioned previously, the empirical simulation technique distinguishes between historical storms and synthetic storms in the training set. The resampling and interpolation procedure places greater emphasis on selection of historical events; therefore, conditions characteristic of historical hurricanes are more heavily reflected in the surge-frequency results. The third training set consisted of the 23 historical storms plus identification of the two synthetic storms as historical events; i.e., it contained 25 storms, all of which were considered as events which have actually occurred in the study area.

At some stations, EST analysis with training set 3 resulted in little change to the control surge-frequency relationship. For example, at Hillsboro Canal, high return period surges were reduced by about 2 percent (see Table 13). At Okeechobee however, surge magnitudes were increased by as much as 25 percent, when compared to the control results.

Storm 876 was a simulation of hurricane Andrew conditions along a westward track. This resulted in wind and surge conditions similar to those associated with several other storms in the training set, including two of the most severe (292 and 477). Although the 876 surge conditions were extreme, they did not have any significant effect on the surge-frequency results because they were redundant.

Table 13. Comparison of training set 3 and control results.

Return Period (yrs)	Hillsboro Canal ---Storm Surge (ft)--- TS1	TS3	% Change	Okeechobee ---Storm Surge (ft)--- TS1	TS3	% Change
25	6.0	6.6	9	4.1	4.6	11
50	10.5	10.3	-3	6.2	7.2	14
75	11.9	11.7	-2	7.7	9.6	20
100	12.4	12.1	-2	8.5	11.1	23
200	12.9	12.7	-2	9.5	12.6	25

On the other hand, no storm similar to storm 878 has ever influenced Lake Okeechobee. Storm 878 subjects the lake to hurricane Andrew conditions approaching and passing directly over the lake center along a northeastward track. This would create the most severe surge conditions at Okeechobee and along the northwest shore. Such conditions don't exist in the true historical record; therefore, the EST analysis resulted in higher surges at those affected stations, including Okeechobee.

Training Set 4. Sections of this report describing calibration and verification procedures indicated that for a given storm and lake station, the predicted hydrodynamic response did not always accurately match the measured response. It was assumed that exact correlation between measured and predicted results was not required. The role of the response model was to provide a good representative range of possible surges. To test this assumption, the EST input

data base was modified by replacing predicted surge elevations with measured surge elevations wherever possible. The fourth training set consisted of the two synthetic storms plus the 23 historical storms, with measured values of the surge response used for storms 477, 499, and 777.

Resulting surge-frequency curves were indistinguishable from the control set curves. Inspection of tabulated EST output files showed variations in surge level on the order of 0.001 ft; therefore, the original training set performed effectively as a basis for the EST generation of the statistical population of storm data.

Summary And Conclusions - Numerical modeling of hurricane induced surges was performed for Lake Okeechobee in south Florida. Results of the numerical modeling effort were subjected to statistical analysis procedures which resulted in the development of surge-frequency relationships at 23 stations on or near the lake shore. Surges were simulated for 23 historical events and 2 synthetic hurricanes. Descriptive information relative to each storm was obtained from the hurricane data base developed by the National Hurricane Center.

The Tropical Storm Planetary Boundary Layer Wind Model (HURWIN) was used to reproduce wind and barometric pressure fields associated with each storm. Model adjustments were made to account for periods of storm progression over land. Results were subsequently used as input to the hydrodynamic model.

The Waterways Experiment Station Implicit Flooding Model (WIFM) was used to simulate the long-wave hydrodynamic response to the wind and pressure fields. WIFM simulations were performed with input provided by HURWIN. Calibration of the hydrodynamic model was based on simulation of the August 1949 hurricane. Verification was performed using three other hurricane events. The calibration and verification procedures yielded additional insight into limitations of the HURWIN model when applied to overland storm progression. WIFM surge results were used to establish the input to the statistical analysis procedure.

The empirical simulation technique was used to compute surge-frequency relationships. Its primary function was to employ

resampling, interpolation, and replacement techniques to a limited data base of historical information. As a result of the EST procedures, a much larger data base was developed for subsequent use in an extremal analysis of hurricane surge. Surge-frequency relationships were developed based on 100 repetitive simulations of a 200-year period of storm activity.

Tabular and graphical presentations of the surge-frequency relationships resulting from this investigation are presented at the end of this appendix. The results indicated that the potential for the greatest surge elevations exists along the southern shore and at Okeechobee. Magnitudes of expected surges and corresponding return periods are provided for 23 stations at various shoreline and open-water locations. The surge magnitudes are expressed as a height above the initial water surface elevation preceding a storm event. This allows for separate consideration of the frequencies of occurrence associated with the hurricane-induced surge and the initial lake stage preceding a hurricane.

Efforts were undertaken to provide additional insight relative to formulation of a storm database for use in the empirical simulation technique. EST analyses were performed with four different historical data sets. The efforts resulted in qualitative information relative to the sensitivity of surge-frequency relationships to characteristics of the historical data set. Results showed that for the Lake Okeechobee study area, the NHC hurricane database provided a thorough listing of influential storms and their characteristics. Combination of the NHC information with corresponding HURWIN/WIFM-generated surge predictions resulted in a historical data base that effectively represented the range of hurricane conditions characteristic of the study area.

The modeling and statistical analysis tools implemented during this study have provided estimates of the storm surge-frequency relationships previously identified as project goals.

5. TAILWATER ELEVATIONS.

Tailwater conditions around the perimeter of Lake Okeechobee vary substantially, due to the characteristics of different toe ditch and drainage canal configurations. Those varying conditions and corresponding locations are summarized in the following paragraphs.

Northeast – The northeast segment of the dike system includes Reach 7. A borrow canal lies adjacent to the landside toe along the entire length of this reach. Water levels in the canal are controlled by project pump stations S-133 and S-135. Optimum water levels are maintained between elevations 13.0 and 14.0 feet, NGVD. During a hurricane alert, the pump stations are operated to lower the canal water level to 12.0 feet.

Northwest – The northwest portion of the dike contains Reaches 5,6, and 8. A continuous borrow canal lies along the landside toe of the levee in these areas. Canal water levels are controlled by project pump stations S-127, S-129, and S-131. Normal canal water levels are maintained within the optimum range of 13.0 to 14.0 feet. During a hurricane alert, the pump stations are operated to lower the canal water level to 12.0 feet.

West – The western lake shore is protected by the dike components in Reach 4. Culverts C-5, C-5a, and Canal 19 control tailwater elevations in this reach. C-5 allows drainage between the lake and Nicodemus Slough as water levels permit. The water levels are typically high in this marchy area. Minimum tailwater elevations of 15.0 to 16.0 feet would be expected when Lake Okeechobee levels were above 18.0 feet; therefore, hydraulic loadings on the dike components in this reach would typically be less severe than at other locations.

Southwest – The southwest lake shore includes dike Reach 2 and a portion of Reach 3. The LD-1 borrow canal lies adjacent to the dike toe. West of Clewiston, canal water levels are controlled by structures S-310, S-235, S-4, and the LD-1 culverts. Canal tailwater elevations typically range from 11.0 to 14.0 feet. If a hurricane threatens, S-235, S-310 and the LD-1 culverts are closed. Pumping is then initiated at S-4 to bring the canal level down to an elevation of 10.0 feet. East of Clewiston, water levels in the Industrial Canal are regulated by project structures S-169, S-310, and S-4. The S-310 lock remains open when the lake level is below 15.5 feet. Otherwise, the lock is closed and runoff is released through the S-169 culverts. S-169 is manually operated to maintain an optimum tailwater elevation of 15.0 feet.

Southeast – The southeast lake shore includes the remainder of Reach 3 and all of Reach 1. Tailwater elevations in these areas are controlled by project structures and by local Drainage Districts.

In Reach 3, project pump station S-236 maintains optimum tailwater elevations between 7.5 and 9.5 feet. If heavy rainfall is expected, pumping is initiated to lower the tailwater stage to 7.5 feet untill the storm has passed. Further east, pump station S-3 controls water levels in the landside toe ditch. Under normal conditions, the optimum tailwater elevation is maintained between 11.5 and 12.0 feet. Durricane a hurricane threat, the S-3 pump station is activated to reduce tailwater elevations to 10.0 feet.

In Reach 1, pump station S-2 and spillway S-351 operate in a similar manner. The optimum tailwater elevations are maintained at 11.5 to 12.0 feet; however, those levels are dropped to 10.0 feet if a hurricane threat exists. Many of the toe ditches in Reach 1 are controlled by local Drainage Districts. During wet conditions, these districts attempt to pump waters from the toe ditches into Lake Okeechobee. Pumping capabilities at specific sites and agricultural drainage goals were considered in detail during the tailwater investigation for Reach 1.

Summary – At different locations along the dike system, water levels at the landside toe of the embankment can vary substantially due to area topography and the presence of water control structures such as locks, toe ditches, borrow canals, pump stations, spillways and culverts. Efforts were undertaken to estimate as accurately as possible the minimum, most likely, and maximum tailwater elevations for each dike component. The results are presented in Table 14 below.

Table 14. Herber Hoover Dike Tailwater Conditions

1. BASE CONDITION			
	Tailwater Elevation (ft, NGVD)		
Component	Minimum	Most Likely	Maximum
1A	10.0	12.0	14.0
1B	10.0	12.0	14.0
1C-a	6.0	10.0	14.0
1C-b	6.0	10.0	14.0
1C-c	6.0	10.0	14.0
2	5.5	7.5	9.5
3B	10.0	12.0	14.0
4	12.0	14.0	16.0
5	12.0	13.0	14.0
6A	12.0	13.0	14.0
6B	12.0	13.0	14.0
7	12.0	13.0	14.0
8	12.0	13.0	14.0

2. ALTERNATIVE A - TAILWATER Control			
	Tailwater Elevation (ft, NGVD)		
Component	Minimum	Most Likely	Maximum
1A	11.0	13.0	14.0
1B	11.0	13.0	14.0
1C-a	7.0	11.0	14.0
1C-b	7.0	11.0	14.0
1C-c	7.0	11.0	14.0
2	6.5	8.5	9.5
3B	10.0	12.0	14.0
4	13.0	14.0	16.0
5	13.0	13.5	14.0
6A	13.0	13.5	14.0
6B	13.0	13.5	14.0
7	13.0	13.5	14.0
8	13.0	13.5	14.0

3. ALTERNATIVE B - SEEPAGE BERM			
	Tailwater Elevation (ft, NGVD)		
Component	Minimum	Most Likely	Maximum
1A	10.0	12.0	14.0
1B	10.0	12.0	14.0
1C-a	6.0	10.0	14.0
1C-b	6.0	10.0	14.0
1C-c	6.0	10.0	14.0
2	5.5	7.5	9.5
3B	10.0	12.0	14.0
4	12.0	14.0	16.0
5	12.0	13.0	14.0
6A	12.0	13.0	14.0
6B	12.0	13.0	14.0
7	12.0	13.0	14.0
8	12.0	13.0	14.0

4. ALTERNATIVE C - CUTOFF WALL			
	Tailwater Elevation (ft, NGVD)		
Component	Minimum	Most Likely	Maximum
1A	10.0	12.0	14.0
1B	10.0	12.0	14.0
1C-a	6.0	10.0	14.0
1C-b	6.0	10.0	14.0
1C-c	6.0	10.0	14.0
2	5.5	7.5	9.5
3B	10.0	12.0	14.0
4	12.0	14.0	16.0
5	12.0	13.0	14.0
6A	12.0	13.0	14.0
6B	12.0	13.0	14.0
7	12.0	13.0	14.0
8	12.0	13.0	14.0

6. DIKE BREACHING ANALYSIS.

The simulation of dike breaching scenarios and resulting floods was crucial to the identification of economic damages associated with a failure and populations at risk. The National Weather Service's Dam-Break Flood Forecasting Model (DAMBRK) was used to simulate the breaching process. The UNET unsteady flow routing program was used to route flows from predetermined breach locations into corresponding damage cells around Lake Okeechobee.

Breaches were assumed to be trapezoidal in shape. Breach geometry was described in terms of breach height, breach width, and the angle of breach side slopes. Breach height differed as the elevation of the embankment crown differs along its length. Based on a literature search for information related to the failure of earthen dams, a 750-foot breach width was assumed. A sensitivity analysis of breach widths ranging from 500 feet to 1,000 feet revealed that the impact of various breach widths on the characteristics of the resulting flood was minimal; therefore, there was no need to incorporate variability of breach width into the analysis. Side slopes on each end of the breach were assumed to be 1V:1H. Floodwave routings were allowed to continue for 45 days. This corresponded to the longest estimated period required for breach closure. Results were summarized as flood inundation maps and flood stage hydrographs. The inundation maps and flood stage hydrographs depict conditions for both high-velocity and low-velocity breaches. The primary difference between the two breach types is flow velocity, and the effect of flow velocity on breach closure activities. The inundation maps provide a plan view depiction of the flooded area boundaries for a variety of conditions. Both the flooded area maps (through colored flood arrival times) and the flood stage hydrographs provide information relative to progression of the flood wave as time passes. The hydrographs also provide information related to depth of flooding at specific locations in the floodplain.

The life cycle analysis model uses the inundation maps and flood stage hydrographs to identify flooding conditions for both high-velocity and low-velocity breaches. If, for a given simulation, the hydraulic gradient exceeds 10 feet, then any breach that occurs under those conditions will be a high-velocity breach. That implies that a 45-day long period will be required to close the breach. The characteristics of the flood, and its progression, during that 45-day period are defined by the appropriate inundation maps and flood stage hydrographs. Those flood characteristics are linked to corresponding economic data by way of site-specific, stage-damage relationships; and those relationships serve as the basis for estimating the economic consequences of a given flood. If, for a given simulation, the hydraulic gradient is less than 10 feet, then any breach that occurs under those conditions will be a low-velocity breach. That implies that a 10-day long period will be required to close the breach. The characteristics of the flood, and its progression, during that 10-day period are defined by the appropriate inundation maps

and flood stage hydrographs. The flood conditions will, of course, be less severe than those conditions associated with a flood that proceeds unchecked for 45 days. Again, the low-velocity flood characteristics are linked to corresponding economic data by way of site-specific, stage-damage relationships; and those relationships serve as the basis for estimating the economic consequences of a given flood.

The area flooded is directly related to the location of the breach. For example, a breach along the north shore levee will flood a particular region that differs from the area that would be flooded by a south shore breach; therefore, an initial task was to identify individual flood cells around the perimeter of the lake. Area topography and the location of secondary levees, roadways, and drainage canals defined the boundaries of those flood cells. This effort identified fourteen (14) individual flood cells.

The severity of flooding is related to the starting water surface elevation (i.e., Lake Okeechobee stage) at the time of levee breaching. Breach analyses were performed with initial lake stages of 18.0, 21.0, and 26.0 feet, NGVD. Flood severity is also related to the size of the breach. The results reflect a 750-foot wide breach that extends from the levee crown down to the natural ground elevation.

To illustrate the use of the inundation maps and flood hydrographs, consider the examples provided in Figures 31 and 32. Figure 31 depicts the expected flood that would result from a breach in Reach 1C with an initial lake stage of 18.0 feet. Figure 31 indicates that during the first 24 hours after breaching, the flood would be confined to the area (defined in red) west of HWY 441. If the breach were not closed, the flood would propagate eastward, and after 3 days would extend to a point about 6.63 miles east of the dike. The cross section numbers reflect the distance in miles from the breach location. The octagons, labeled A through F, identify locations where stage hydrographs are available in Figure 32. For example, Figure 32 indicates that the level of Lake Okeechobee would drop from 18.0 feet to about 17.2 feet, if the breach remained open for a period of 45 days. Figure 31 indicates that site F is located approximately 17 miles from the dike; and Figure 32 indicates that after 25 days of flooding, the flood elevation at site F would be 16.2 feet. Inspection of a topographic quadrangle map would indicate that the ground elevation near site F is about 14 feet; therefore,

the depth of flooding at site F after 25 days is estimated as 2.2 feet.

A comprehensive set of inundation maps and flood stage hydrographs is provided at the end of this appendix. The set identifies characteristics of various flooding scenarios that would result from dike failures at 13 possible dike components, in response to 3 different Lake Okeechobee stages. This information was incorporated in the flood stage-damage analyses that were executed in an effort to evaluate economic justification of the proposed major rehabilitation efforts.

7. ACCURACY OF SIMULATED HYDRAULIC CONDITIONS.

As mentioned previously, one of the primary objectives of the hydrological and hydraulic analyses was the identification of hydraulic loads to which the dike system could be subjected. Due to the probabilistic nature of the economic life cycle analysis (LCA) model, those hydraulic loads were defined in terms of frequency relationships. This approach resulted in the identification of the range of possible loading conditions, and the corresponding frequency of occurrence associated with those loads. Hydraulic loads on the dike were defined primarily in terms of three components, the Lake Okeechobee pool elevation, the increase in pool elevation which may occur in the event of a tropical cyclone, and the water level in ditches and canal adjacent to the dike's landside toe. Therefore, frequency of occurrence relationships were required for all three of these components. Throughout the life cycle simulation process, the LCA model used a random sampling approach to extract loading conditions from those frequency relationships. In an effort to monitor the accuracy of that randomly extracted data, the LCA model was programmed to record the numerous hydraulic loading conditions, which were experienced during a life cycle simulation. That record was then compared to historical records of hydraulic loading conditions to assess the LCA model's performance relative to the simulation of realistic loads.

An example of the LCA's hydraulic loading record is provided in Table 15. The first 25 lines provide information obtained from the model's input file. For example, the scenario modeled was named PREFER2. It simulated the rehabilitation of Reaches 1, 2, 3, and 7. Each project life consisted of 251 cycles (or quarters), and the model executed 5,000 iterations of that 62.75-year project life; therefore, this simulation

represented a total period of 313,750 years. All information following the ==Simulation Output== notation was compiled based on model results. Each will be briefly described below:

Hurricane Statistics – This section allows the user to monitor the frequency of hurricane occurrence. The results indicate that, on the average, the lake was subjected to 7.23 Atlantic hurricanes and 4.75 Gulf hurricanes in each 62.75-year life cycle. This corresponds to the occurrence of 1 hurricane every 5.24 years, which compares well with the historical rate of 1 hurricane every 4.35 years. The relative rate of Atlantic and Gulf storms also compares well with the historical record. Minimum, maximum, and variance of the statistics are also provided.

Lake Stage Statistics – The simulated lake stage statistics are presented both annually and quarterly. The first table indicates that the average annual lake stage during this simulation was 14.4 feet, NGVD. The quarterly data indicates that there was little seasonal variation in lake stage statistics. Again, minimum, maximum, and standard deviation values are also presented.

Annual Histogram – The next five sets of data provide tabulated histograms of lake stage data on an annual basis, and by quarter. For each 1-foot increment in pool elevation, the table provides the corresponding percentage and number of observations. For example, the annual histogram indicates that 3.4 percent of the total lake stages observed were between 17.00 and 18.00 feet. This suggests that, for this simulation, the lake stages were relatively low, since the estimated 3.4 percent chance of exceedance stage on Lake Okeechobee is 18.8 feet.

Component Breach Statistics – This table summarizes the frequency of breaching and categorizes those breaches as high-velocity or low-velocity breaches. For example, the statistics indicate that, for this simulation, a breach occurred in Component 1B an average of 0.05 times every 62.75 years.

Component Breach Histograms – Additional information regarding breaches and individual components is presented in the breach histogram tabulations. They provide insight relative to the lake stages at which breaches are occurring. For example, at Component 1A, 38 breaches (in 313,750 years) occurred with a pool elevation between 18.0 and 19.0 feet.

In summary, it should be noted that the values provided in Table 15 were obtained from a single model simulation. Due to the random sampling methodology incorporated in the LCA model, each simulation will provide different results; however, the statistical nature of those results should be similar. Based on comparison with available historical data and projected extremes, the hydraulic loading conditions simulated by the LCA model were deemed acceptable.

8. ESTIMATION OF PUMPING COSTS.

In the event of a breach of the Herbert Hoover Dike system, flooding would occur. To relieve this flooding, pumps would be mobilized to the flooded area, and floodwaters would be pumped into the nearest available canal(s). Estimates of the costs incurred during these pumping activities was required as input to the economic analysis. Pumping costs were primarily dependent on the required pumping capacity, the mobilization distance, the dimensions and depth of the flooded area, and the pumping duration. Information relative to assumptions made and various pumping scenarios is provided below.

The first possible pumping strategy considered was based on a 600-cfs pumping capacity. This would be accomplished with ten 24-inch electric-powered submersible trailer pumps and ten 12-inch electric-powered submersible trailer pumps. Purchase or rental of the pumps would be required. Mobilization (typical distance of 40 miles) would be carried out from the South Florida Operations Office (SFOO) to the flooded area. Pumping durations of 1, 2, 3, 4, 8, and 12 weeks were considered to cover all flood scenarios where a 600-cfs pumping capacity would be required. The pumps would be operated 24 hours per day, 7 days per week, with 2 individuals manning each array of pumps; therefore, on each 12-hour shift, a total of 8 persons would be required.

The second possible pumping strategy considered was based on a 1200-cfs pumping capacity. This would be accomplished with twenty 24-inch trailer pumps and twenty 12-inch trailer pumps. Mobilization distances of 10, 30, and 50 miles were considered; and pumping durations of 4, 8, 12, 16, and 20 weeks were investigated to cover all flood scenarios where a 1200-cfs pumping capacity would be required. A total of 16 persons would be required to serve on each 12-hour pumping shift. A summary of the estimated pumping requirements and costs is provided in Table 16.

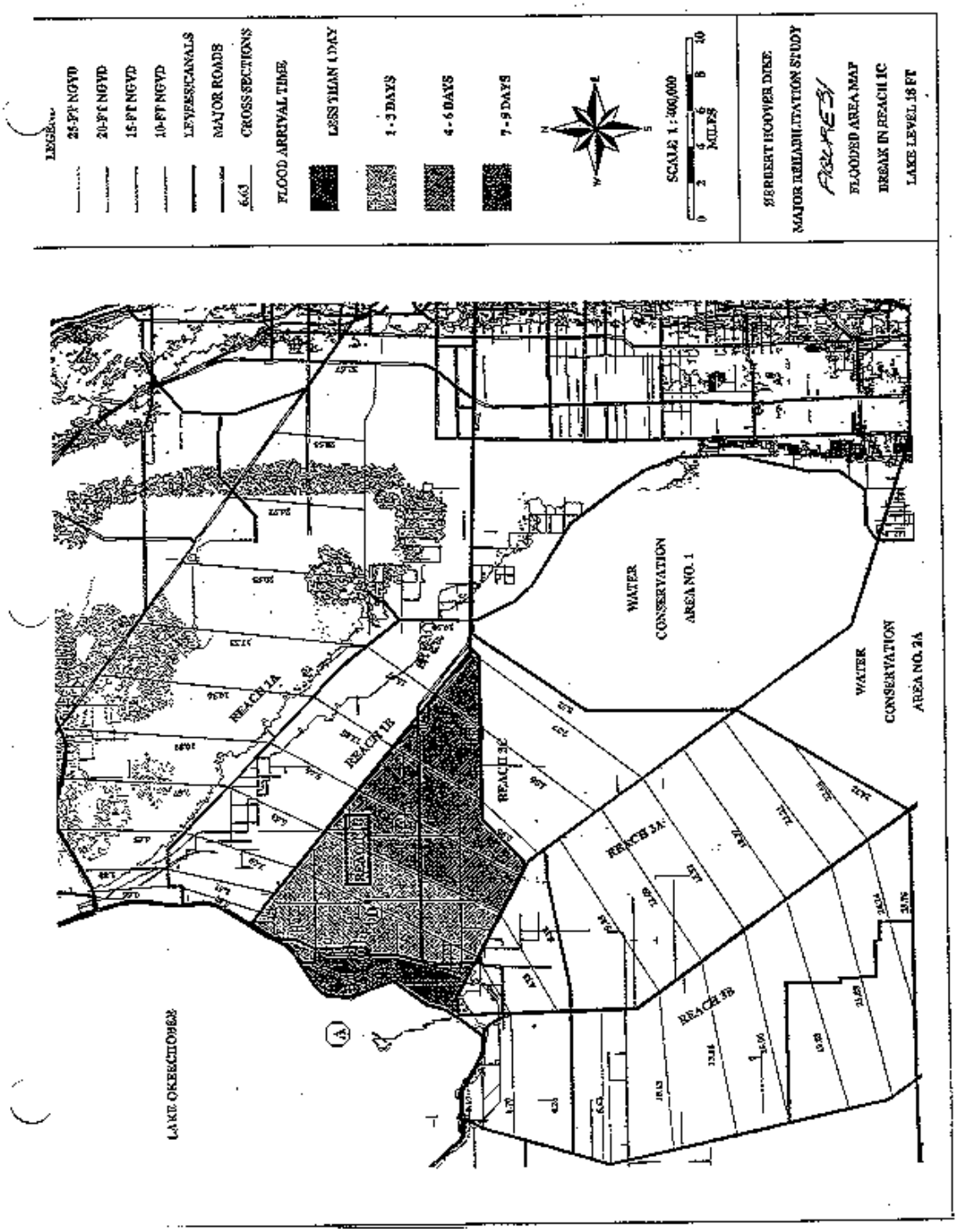


Figure 30. Flooded Area Map

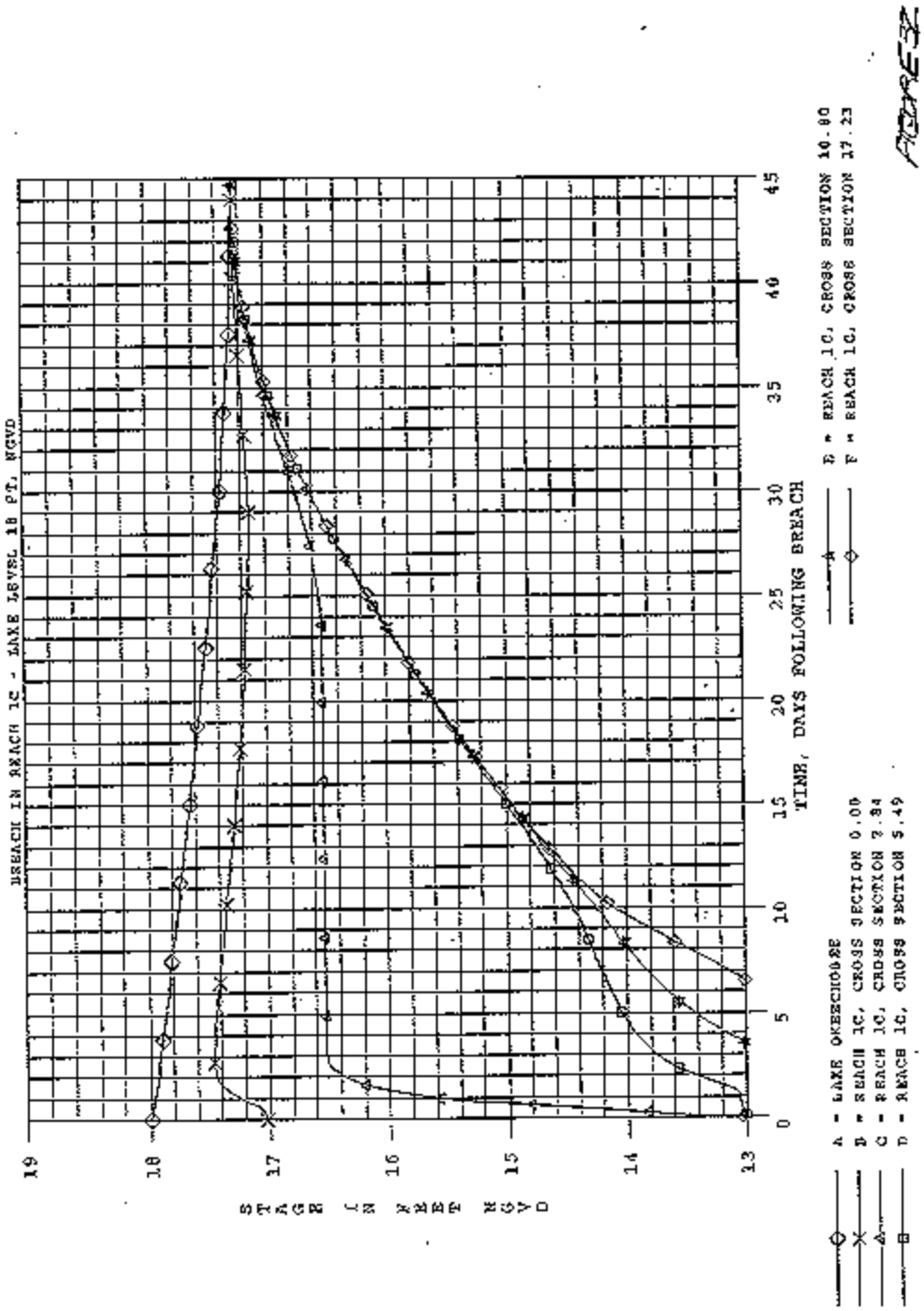

BREACH IN REACH 1C - LAKE LEVEL 18 FT. NGVD
STAGE IN FEET NGVD
19
18
17
16
15
14
13
0
5
10
15
20
25
30
35
40
45
TIME, DAYS FOLLOWING BREACH
A - LAKE OKEECHOBEE
B - REACH 1C, CROSS SECTION 0.00
C - REACH 1C, CROSS SECTION 3.84
D - REACH 1C, CROSS SECTION 5.49
E - REACH 1C, CROSS SECTION 10.80
F - REACH 1C, CROSS SECTION 17.23

Figure 31.

Table 15. Hydraulic Output of the LCA Model

```
Model Run on:02/10/99 09:00:56
Database:C:\Hoover\Hoover.mde
Scenario information: scenario PREFER2
Kernel Version:      0.940
scenario description: Preferred Rehabs - 50-yrs after last construction
Iterations: 5000  Cycles: 251
 Start Year: 1999 Base Year: 2001
 Rehab Plan: PREFER Preferred Rehabs - Reaches 1, 2, 3, & 7
 Critical Head Differential:   10.000
 Hurricane Flag: 1 HURRICANE SIMULATION PERFORMED
 OM Hurricane Multiplier:   1.9600
Lake Stage Generation Parameters
Mean: 14.3790 SD:  1.5153 Correlation:   0.5350 Moving Average:  -0.4856
Random SD:      0.6377
Initial Lake Stage Mean:  15.0000  SD:   1.0000
Simulation Control
Convergence Limit:     5.0000  Min Iterations: 10
Interest Rate:  0.071
Seed: 4560
Period Data
Period  Hurricane Gulf  Atlantic
#:  1 0.057400 0.142900  0.8571
#:  2 0.131300 0.500000  0.5000
#:  3 0.000000 0.000000  0.0000
#:  4 0.000000 0.000000  0.0000

================ Simulation Output ============

Repair Cost Statistics
Observations: 5000 Mean:  10547409  SD:  20206254  Max:  233299250 Min:   0

O&M Cost Statistics
Observations: 5000 Mean: 39657771  SD: 3058617  Max: 54925490 Min: 31896538


Damage Cost Statistics
Observations: 5000 Mean: 39123135  SD: 82390265  Max: 730746884 Min: 0


Total Cost Statistics
Observations:5000 Mean:89328186  SD:101595924  Max:993290800 Min:31896538


Random Lake Stage Statistics
Observations:1255000 Mean: 0.0010  SD: 0.6382  Max: 3.0332 Min: -3.1527


Hurricane Statistics
Code:  0      None Avg: 239.0110 SD:  3.38591 Max: 247.0000 Min: 229.0000
Code:  1  Atlantic Avg:   7.2390 SD:  2.62634 Max:  18.0000 Min:   1.0000
Code:  2      Gulf Avg:   4.7500 SD:  2.23466 Max:  12.0000 Min:   0.0000
```

Lake Stage Statistics
Annual

Annual:		Avg:	14.38495	SD:	1.51410	Max:	21.77387	Min: 7.00142
Period:	1	Avg:	14.39022	SD:	1.51289	Max:	21.63122	Min: 7.00142
Period:	2	Avg:	14.38447	SD:	1.51357	Max:	21.77387	Min: 7.29606
Period:	3	Avg:	14.38156	SD:	1.51391	Max:	20.94385	Min: 7.30880
Period:	4	Avg:	14.38352	SD:	1.51606	Max:	21.16162	Min: 7.61732

Annual Histogram
Number Of Observations: 1254766

Stage Range	Percent	#
[9.00 - 10.00]	0.2	2187
[10.00 - 11.00]	1.1	13633
[11.00 - 12.00]	4.5	56357
[12.00 - 13.00]	12.2	153646
[13.00 - 14.00]	21.9	275240
[14.00 - 15.00]	25.8	324347
[15.00 - 16.00]	19.9	249375
[16.00 - 17.00]	10.1	127320
[17.00 - 18.00]	3.4	42126
[18.00 - 19.00]	0.7	9191
[19.00 - 20.00]	0.1	1226
[20.00 - 21.00]	0.0	110
[21.00 - 22.00]	0.0	8
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0

Period: 1 Lake Stage Histogram
Number Of Observations: 314945

Stage Range	Percent	#
[9.00 - 10.00]	0.2	562
[10.00 - 11.00]	1.1	3326
[11.00 - 12.00]	4.5	14108
[12.00 - 13.00]	12.2	38352
[13.00 - 14.00]	21.8	68618
[14.00 - 15.00]	26.0	81756
[15.00 - 16.00]	20.1	63205
[16.00 - 17.00]	10.1	31731
[17.00 - 18.00]	3.4	10582
[18.00 - 19.00]	0.7	2358
[19.00 - 20.00]	0.1	316
[20.00 - 21.00]	0.0	29
[21.00 - 22.00]	0.0	2
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0

Period: 2 Lake Stage Histogram
Number Of Observations: 314943

Stage Range	Percent	#
[9.00 - 10.00]	0.2	577
[10.00 - 11.00]	1.1	3415
[11.00 - 12.00]	4.5	14073

[12.00 - 13.00]	12.3	38588
[13.00 - 14.00]	22.1	69457
[14.00 - 15.00]	25.8	81140
[15.00 - 16.00]	19.8	62395
[16.00 - 17.00]	10.2	32067
[17.00 - 18.00]	3.4	10599
[18.00 - 19.00]	0.7	2308
[19.00 - 20.00]	0.1	298
[20.00 - 21.00]	0.0	23
[21.00 - 22.00]	0.0	3
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0

Period: 3 Lake Stage Histogram
Number Of Observations: 314943

Stage Range	Percent	#
[9.00 - 10.00]	0.2	534
[10.00 - 11.00]	1.1	3456
[11.00 - 12.00]	4.5	14296
[12.00 - 13.00]	12.3	38641
[13.00 - 14.00]	21.9	68893
[14.00 - 15.00]	25.9	81551
[15.00 - 16.00]	19.8	62509
[16.00 - 17.00]	10.2	32000
[17.00 - 18.00]	3.3	10488
[18.00 - 19.00]	0.7	2208
[19.00 - 20.00]	0.1	337
[20.00 - 21.00]	0.0	30
[21.00 - 22.00]	0.0	0
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0

Period: 4 Lake Stage Histogram
Number Of Observations: 309935

Stage Range	Percent	#
[9.00 - 10.00]	0.2	514
[10.00 - 11.00]	1.1	3436
[11.00 - 12.00]	4.5	13880
[12.00 - 13.00]	12.3	38065
[13.00 - 14.00]	22.0	68272
[14.00 - 15.00]	25.8	79900
[15.00 - 16.00]	19.8	61266
[16.00 - 17.00]	10.2	31522
[17.00 - 18.00]	3.4	10457
[18.00 - 19.00]	0.7	2317
[19.00 - 20.00]	0.1	275
[20.00 - 21.00]	0.0	28
[21.00 - 22.00]	0.0	3
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0

Component Breach Statistics

	Low			High		
	Avg	SD	Max	Avg	SD	Max
1A	0.04	0.21	2	0.00	0.00	0
1B	0.05	0.23	2	0.00	0.00	0
1C-a	0.06	0.25	3	0.01	0.09	1
1C-b	0.05	0.22	2	0.01	0.09	1
1C-c	0.04	0.22	3	0.01	0.09	1
2	0.08	0.28	2	0.12	0.36	4
3	0.16	0.42	4	0.00	0.00	0
4	0.04	0.19	2	0.00	0.00	0
5	0.02	0.16	2	0.00	0.00	0
6A	0.01	0.10	1	0.00	0.00	0
6B	0.02	0.13	2	0.00	0.00	0
7	0.11	0.35	3	0.00	0.00	0
8	0.03	0.18	2	0.00	0.00	0

Component Breach Histograms By Lake Stage

Component: 1A Component Breach Histogram
Number Of Observations: 213

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	29.1	62
[18.00 - 19.00]	38.0	81
[19.00 - 20.00]	17.8	38
[20.00 - 21.00]	9.4	20
[21.00 - 22.00]	3.8	8
[22.00 - 23.00]	0.9	2
[23.00 - 24.00]	0.5	1
[24.00 - 25.00]	0.5	1
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 1B Component Breach Histogram
Number Of Observations: 256

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0

[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	29.7	76
[18.00 - 19.00]	41.0	105
[19.00 - 20.00]	18.8	48
[20.00 - 21.00]	5.5	14
[21.00 - 22.00]	2.0	5
[22.00 - 23.00]	2.3	6
[23.00 - 24.00]	0.8	2
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 1C-a Component Breach Histogram
Number Of Observations: 340

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	36.8	125
[18.00 - 19.00]	34.4	117
[19.00 - 20.00]	15.3	52
[20.00 - 21.00]	6.5	22
[21.00 - 22.00]	4.7	16
[22.00 - 23.00]	2.1	7
[23.00 - 24.00]	0.3	1
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 1C-b Component Breach Histogram
Number Of Observations: 280

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	34.3	96

[18.00 - 19.00]	36.1	101
[19.00 - 20.00]	17.5	49
[20.00 - 21.00]	6.8	19
[21.00 - 22.00]	2.1	6
[22.00 - 23.00]	1.4	4
[23.00 - 24.00]	1.1	3
[24.00 - 25.00]	0.7	2
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 1C-c Component Breach Histogram
Number Of Observations: 264

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	34.8	92
[18.00 - 19.00]	39.4	104
[19.00 - 20.00]	15.5	41
[20.00 - 21.00]	5.7	15
[21.00 - 22.00]	1.9	5
[22.00 - 23.00]	1.1	3
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.8	2
[25.00 - 26.00]	0.8	2
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 2 Component Breach Histogram
Number Of Observations: 1002

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	32.9	330
[18.00 - 19.00]	39.9	400
[19.00 - 20.00]	14.9	149
[20.00 - 21.00]	5.8	58

[21.00 - 22.00]	3.3	33
[22.00 - 23.00]	2.4	24
[23.00 - 24.00]	0.8	8
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 3 Component Breach Histogram
Number Of Observations: 818

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	37.9	310
[18.00 - 19.00]	30.8	252
[19.00 - 20.00]	14.5	119
[20.00 - 21.00]	6.4	52
[21.00 - 22.00]	3.8	31
[22.00 - 23.00]	4.3	35
[23.00 - 24.00]	1.7	14
[24.00 - 25.00]	0.4	3
[25.00 - 26.00]	0.1	1
[26.00 - 27.00]	0.1	1
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 4 Component Breach Histogram
Number Of Observations: 182

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	19.2	35
[18.00 - 19.00]	49.5	90
[19.00 - 20.00]	19.2	35
[20.00 - 21.00]	3.3	6
[21.00 - 22.00]	2.7	5
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	2.7	5

[24.00 - 25.00]	2.2	4
[25.00 - 26.00]	1.1	2
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 5 Component Breach Histogram
Number Of Observations: 123

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	23.6	29
[18.00 - 19.00]	48.0	59
[19.00 - 20.00]	22.8	28
[20.00 - 21.00]	3.3	4
[21.00 - 22.00]	2.4	3
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 6A Component Breach Histogram
Number Of Observations: 53

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	35.8	19
[18.00 - 19.00]	50.9	27
[19.00 - 20.00]	9.4	5
[20.00 - 21.00]	3.8	2
[21.00 - 22.00]	0.0	0
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0

[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 6B Component Breach Histogram
Number Of Observations: 80

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	31.3	25
[18.00 - 19.00]	40.0	32
[19.00 - 20.00]	8.8	7
[20.00 - 21.00]	2.5	2
[21.00 - 22.00]	6.3	5
[22.00 - 23.00]	5.0	4
[23.00 - 24.00]	5.0	4
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	1.3	1
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Component: 7 Component Breach Histogram
Number Of Observations: 573

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	38.6	221
[18.00 - 19.00]	41.9	240
[19.00 - 20.00]	16.1	92
[20.00 - 21.00]	3.0	17
[21.00 - 22.00]	0.5	3
[22.00 - 23.00]	0.0	0
[23.00 - 24.00]	0.0	0
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0

[30.00 - 31.00]	0.0	0

Component: 8 Component Breach Histogram
Number Of Observations: 166

Stage Range	Percent	#
[9.00 - 10.00]	0.0	0
[10.00 - 11.00]	0.0	0
[11.00 - 12.00]	0.0	0
[12.00 - 13.00]	0.0	0
[13.00 - 14.00]	0.0	0
[14.00 - 15.00]	0.0	0
[15.00 - 16.00]	0.0	0
[16.00 - 17.00]	0.0	0
[17.00 - 18.00]	29.5	49
[18.00 - 19.00]	45.8	76
[19.00 - 20.00]	15.1	25
[20.00 - 21.00]	3.6	6
[21.00 - 22.00]	2.4	4
[22.00 - 23.00]	3.0	5
[23.00 - 24.00]	0.6	1
[24.00 - 25.00]	0.0	0
[25.00 - 26.00]	0.0	0
[26.00 - 27.00]	0.0	0
[27.00 - 28.00]	0.0	0
[28.00 - 29.00]	0.0	0
[29.00 - 30.00]	0.0	0
[30.00 - 31.00]	0.0	0

Damage Cell Statistics

Hits

	Avg	SD	Max	Min
1A	0.043	0.211	2	0
1B	0.364	0.634	5	0
1C	0.364	0.634	5	0
2	0.235	0.499	4	0
3A	0.335	0.606	5	0
3B	0.335	0.606	5	0
3C	0.335	0.606	5	0
4	0.036	0.189	2	0
5	0.025	0.156	2	0
6A	0.059	0.247	2	0
6B	0.059	0.247	2	0
6C	0.059	0.247	2	0
7	0.115	0.355	3	0
8	0.059	0.247	2	0

Monetary Damage By Land Use

		Avg	SD	Max	Min
1A	UR	87419	611805	10468953	0
1A	NR	0	0	0	0
1A	AG	76134	512504	6334273	0
1A	RT	18534	129902	2315600	0
1A	OD	8072	59453	1027762	0

1B	UR	22852	140384	2304001	0
1B	NR	34817	202839	3260826	0
1B	AG	0	0	0	0
1B	RT	1144019	6486648	86872457	0
1B	OD	29053	150877	2332676	0
1C	UR	2702967	9009450	99007852	0
1C	NR	861809	4353058	66991137	0
1C	AG	16214	52231	371455	0
1C	RT	6203647	19622736	175299050	0
1C	OD	4480273	14457183	150222552	0
2	UR	662631	1831687	21329625	0
2	NR	150151	472852	6240903	0
2	AG	396984	1017831	6968758	0
2	RT	7517969	19074590	158187270	0
2	OD	1137816	2971352	30269422	0
3A	UR	97417	981229	23607550	0
3A	NR	34382	345929	8317112	0
3A	AG	0	0	0	0
3A	RT	478477	4732837	91171148	0
3A	OD	43756	408906	9836731	0
3B	UR	1053064	3017156	29338945	0
3B	NR	810194	2317311	22097618	0
3B	AG	0	0	0	0
3B	RT	7732664	21518008	162221625	0
3B	OD	748830	2098992	20008442	0
3C	UR	0	0	0	0
3C	NR	0	0	0	0
3C	AG	0	0	0	0
3C	RT	170067	1671496	31567762	0
3C	OD	4514	27065	398685	0
4	UR	0	0	0	0
4	NR	0	0	0	0
4	AG	4161	47916	1440786	0
4	RT	7731	72466	1629796	0
4	OD	2544	21551	497929	0
5	UR	136544	1252842	32605328	0
5	NR	87776	801429	21730615	0
5	AG	90	1069	29640	0
5	OD	26471	258009	6191541	0
6A	UR	14530	205642	6390640	0
6A	NR	1879	26186	698421	0
6A	AG	95	1379	39281	0
6A	RT	192	2715	76796	0
6A	OD	1667	23315	638733	0
6B	UR	8282	105401	2825305	0
6B	NR	987	12581	337711	0
6B	AG	0	0	0	0
6B	RT	0	0	0	0
6B	OD	1739	21993	583929	0
6C	UR	0	0	0	0
6C	NR	0	0	0	0
6C	AG	0	0	0	0
6C	RT	0	0	0	0
6C	OD	0	0	0	0
7	UR	1282481	4178253	52821579	0
7	NR	17003	55966	777549	0
7	AG	138784	506567	4053914	0

7	RT	485453	1611229	14226602	0
7	OD	78362	257502	3536776	0
8	UR	87148	765480	15280997	0
8	NR	4401	37695	665471	0
8	AG	1863	24449	945606	0
8	RT	7408	63116	1065496	0
8	OD	0	0	0	0

Non-Monetary Damage By Land Use

		Avg	SD	Max	Min
1A	PO	28	156	2427	0
1B	PO	19	85	806	0
1C	PO	1345	3590	36788	0
2	PO	890	2169	24855	0
3A	PO	61	538	11468	0
3B	PO	419	1113	10393	0
3C	PO	0	0	0	0
4	PO	0	0	0	0
5	PO	256	1642	21742	0
6A	PO	0	0	0	0
6B	PO	55	233	2040	0
6C	PO	0	0	0	0
7	PO	378	1175	10193	0
8	PO	108	599	7002	0

Table 16. Costs Related to Pumping of Floodwaters

FLOOD CONDITION (elevations = NGVD)	PUMPING COSTS	DURATION (weeks)	TOTAL FLOODED AREA (acres)	DAMAGES BY FLOODCELL						MEAN DEPTH (feet)
				1a	1b	1c	3a	3b	3c	
Breach 1a, LS 18 ft.	$227,000	2	4,500	$227,000						3
Breach 1a, LS 21 ft.	$250,000	2	26,000	$250,000						1
Breach 1a, LS 26 ft.	$3,634,000	12	290,000	$1,344,580	$1,090,200	$1,199,220				9
Breach 1b, LS 18 ft.	$909,000	4	86,000		$909,000					3
Breach 1b, LS 21 ft.	$2,726,000	12	182,000		$1,281,220	$1,444,780				2
Breach 1b, LS 26 ft.	$4,543,000	20	535,000		$726,880	$817,740	$1,181,180	$1,408,330	$408,870	6
Breach 1c, LS 18 ft.	$909,000	4	96,000			$909,000				3
Breach 1c, LS 21 ft.	$2,726,000	12	260,000		$681,500	$1,008,620	$545,200		$490,680	5
Breach 1c, LS 26 ft.	$4,543,000	20	535,000		$726,880	$817,740	$1,181,180	$1,408,330	$408,870	6
				2						
Breach 2, LS 18 ft.	$2,382,000	8	180,000	$2,382,000						1
Breach 2, LS 21 ft.	$2,726,000	12	189,000	$2,726,000						2
Breach 2, LS 26 ft.	$4,543,000	20	221,000	$4,543,000						3
				1a	1b	1c	3a	3b	3c	
Breach 3b, LS 18 ft.	$2,726,000	12	165,000					$2,726,000		2
Breach 3b, LS 21 ft.	$3,634,000	16	165,000					$3,634,000		5
Breach 3b, LS 26 ft.	$4,543,000	20	535,000		$726,880	$817,740	$1,181,180	$1,408,330	$408,870	5
				2	4					
Breach 4, LS 18 ft.	$227,000	2	5,000		$227,000					2
Breach 4, LS 21 ft.	$909,000	4	17,000		$909,000					3
Breach 4, LS 26 ft.	$4,543,000	20	244,000	$4,134,130	$408,870					6

FLOOD CONDITION (elevations = NGVD)	PUMPING COSTS	DURATION (weeks)	TOTAL FLOODED AREA (acres)	DAMAGES BY FLOODCELL						MEAN DEPTH (feet)
				5						
Breach 5, LS 18 ft.	$227,000	2	24,000	$227,000						1
Breach 5, LS 21 ft.	$909,000	8	25,000	$909,000						3
Breach 5, LS 26 ft.	$1,363,000	12	26,000	$1,363,000						7
				6a	6b	6c	8			
Breach 6a, LS 18 ft.	$396,000	1	7,000	$396,000						1
Breach 6a, LS 21 ft.	$400,000	3	7,000	$400,000						4
Breach 6a, LS 26 ft.	$4,543,000	20	90,000	$363,440	$726,880	$2,407,790	$1,044,890			8
				6a	6b	6c	8			
Breach 6b, LS 18 ft.	$227,000	2	14,000		$227,000					1
Breach 6b, LS 21 ft.	$454,000	4	14,000		$454,000					4
Breach 6b, LS 26 ft.	$3,634,000	16	100,000	$254,380	$508,760	$2,107,720	$763,140			5
				6a	6b	6c	8			
Breach 6c, LS 18 ft.	$0	0	0			$0				0
Breach 6c, LS 21 ft.	$341,000	3	25,000			$341,000				1
Breach 6c, LS 26 ft.	$3,634,000	16	100,000	$254,380	$508,760	$2,107,720	$763,140			8
				7						
Breach 7, LS 18 ft.	$396,000	1	6,000	$396,000						1
Breach 7, LS 21 ft.	$454,000	2	12,000	$454,000						1
Breach 7, LS 26 ft.	$909,000	8	16,000	$909,000						6
				6a	6b	6c	8			
Breach 8, LS 18 ft.	$227,000	2	100,000				$227,000			5
Breach 8, LS 21 ft.	$1,817,000	8	21,000				$1,817,000			8
Breach 8, LS 26 ft.	$3,634,000	16	17,000	$254,380	$508,760	$2,107,720	$763,140			1

Plate 1. Levee Culverts

CORPS OF ENGINEERS U.S. ARMY

54-1c

* ONE BARREL AT EACH CULVERT WAS REPLACED WITH SLIDE GATE.

C = LEVEE CULVERT

CULVERTS					
NO.	BARRELS NO.	BARRELS DIA. FT	BARRELS LGTH. FT	CONTROL	INVERT EL. (FT. M.S.L.)
1	2	10	120	PUMP/ FLAP GATES	5.55
1A	3	7	168	VERTICAL SCREW GATES	5.5
*2	6	10	160	FLAP GATES	5.5
3	2	10	120	PUMP/FLAP GATES	5.5
4A	1	10	120	PUMP/FLAP GATE	5.5
*5	3	10	160	FLAP GATES	5.5
*5A	3	10	160	FLAP GATES	5.5
6	1	10	72	SLIDE GATE	5.4
7	3	10	150	FLAP GATES	5.35
8	3	10	150	FLAP GATES	5.44
9	3	10	150	FLAP GATES	5.44
10	2	10	110	PUMP/FLAP GATES	5.5
*10A	5	10	75	FLAP GATES	5.5
11	1	10	96	PUMP/SLIDE GATE	7.2
12	3	10	90	PUMP/FLAP GATES	5.5
12A	1	7	84	PUMP/FLAP GATE	6.0
13	1	10	96	SLIDE GATE	5.5
14	1	10	96	SLIDE GATE	5.5
16	1	10	96	SLIDE GATE	5.5

LEGEND

CANAL
LEVEE
SPILLWAY AND LOCK
SPILLWAY
CULVERTS
PUMPING STATION
HURRICANE GATE STRUCTURE
LOCK

CENTRAL AND SOUTHERN FLORIDA
(FLOOD CONTROL AND OTHER PURPOSES)
LAKE OKEECHOBEE

LEVEE CULVERTS

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

7-24-97

Herbert Hoover Dike

Surge-Frequency Tables

HURRICANE SURGE-FREQUENCY RELATIONSHIPS

Surge values reflect rise in water surface elevation above initial lake stage.

STATION - FISHEATING CREEK (S-131)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	2.7	0.7
25	5.6	1.1
50	7.4	0.9
75	8.0	0.9
100	8.4	1.0
150	9.0	0.7
200	9.3	0.7

STATION - INDIAN PRAIRIE CANAL

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	2.2	0.6
25	3.3	0.3
50	3.8	0.3
75	4.0	0.3
100	4.1	0.3
150	4.3	0.2
200	4.3	0.2

STATION - KISSIMMEE RIVER

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.4	0.6
25	3.1	0.4
50	3.9	0.6
75	4.2	0.5
100	4.4	0.4
150	4.5	0.3
200	4.6	0.3

STATION - OKEECHOBEE (S-193)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.9	0.7
25	4.1	0.9
50	6.2	1.7
75	7.7	1.9
100	8.5	1.9
150	9.2	1.3
200	9.5	1.2

HURRICANE SURGE-FREQUENCY RELATIONSHIPS

Surge values reflect rise in water surface elevation above initial lake stage.

STATION - HENRY CREEK LOCK

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.5	0.4
25	3.1	0.7
50	4.3	0.9
75	4.9	0.9
100	5.3	0.9
150	5.6	0.6
200	5.8	0.6

STATION - NORTHEAST SHORE, LS-19 (S-131)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.4	0.5
25	3.1	0.6
50	4.2	0.8
75	4.7	0.7
100	4.9	0.7
150	5.2	0.4
200	5.3	0.3

STATION - PORT MAYACA (S-308)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.3	0.5
25	3.1	0.9
50	4.5	0.8
75	5.0	0.6
100	5.3	0.5
150	5.6	0.3
200	5.7	0.3

STATION - CULVERT 10-A

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.6	0.4
25	3.8	1.3
50	6.4	1.7
75	7.6	1.7
100	8.4	1.9
150	9.4	1.2
200	9.9	1.2

HURRICANE SURGE-FREQUENCY RELATIONSHIPS

Surge values reflect rise in water surface elevation above initial lake stage.

STATION - CANAL POINT (S-352)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.5	0.4
25	3.9	1.4
50	7.0	1.7
75	8.2	1.7
100	9.0	1.8
150	10.0	1.1
200	10.4	1.0

STATION - PAHOKEE

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.3	0.5
25	4.0	1.6
50	7.2	1.7
75	8.4	1.2
100	8.9	1.0
150	9.3	0.5
200	9.6	0.4

STATION - BACOM POINT (C-10)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.3	0.5
25	3.9	1.6
50	7.3	1.9
75	8.4	1.2
100	8.8	0.9
150	9.1	0.4
200	9.3	0.2

STATION - PELICAN BAY (C-12)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.4	0.5
25	4.7	1.8
50	8.4	2.3
75	9.8	1.5
100	10.3	1.2
150	10.6	0.5
200	10.8	0.6

HURRICANE SURGE-FREQUENCY RELATIONSHIPS

Surge values reflect rise in water surface elevation above initial lake stage.

STATION - HILLSBORO CANAL (S-351)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.4	0.7
25	6.0	2.3
50	10.5	2.3
75	11.9	1.5
100	12.4	1.1
150	12.7	0.5
200	12.9	0.2

STATION - LAKE HARBOR (S-354)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.4	0.6
25	5.5	2.0
50	9.3	2.1
75	10.7	1.5
100	11.2	1.2
150	11.6	0.6
200	11.8	0.3

STATION - CLEWISTON (S-310)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.4	0.5
25	4.8	1.7
50	8.1	2.1
75	9.6	1.5
100	10.1	1.3
150	10.5	0.7
200	10.8	0.4

STATION - LIBERTY POINT (C-1A)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	2.0	0.5
25	5.4	1.9
50	9.0	1.8
75	10.2	1.3
100	10.7	1.1
150	11.2	0.6
200	11.5	0.4

HURRICANE SURGE-FREQUENCY RELATIONSHIPS

Surge values reflect rise in water surface elevation above initial lake stage.

STATION - MOORE HAVEN (S-77)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	2.8	0.6
25	6.9	2.1
50	11.0	2.1
75	12.5	1.9
100	13.4	2.0
150	14.4	1.2
200	14.9	1.1

STATION - LS-18 (North of Moore Haven)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	3.1	0.7
25	7.2	2.0
50	11.1	2.0
75	12.5	1.8
100	13.3	2.0
150	14.4	1.2
200	14.9	1.2

STATION - LS-17 (North of Clewiston)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.1	0.4
25	3.5	1.2
50	5.8	1.4
75	7.0	1.3
100	7.5	1.2
150	7.9	0.8
200	8.1	0.7

STATION - LS-16 (East of Fisheating Creek)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.7	0.4
25	3.6	0.8
50	4.5	0.5
75	4.9	0.6
100	5.2	0.7
150	5.6	0.5
200	5.8	0.5

HURRICANE SURGE-FREQUENCY RELATIONSHIPS

Surge values reflect rise in water surface elevation above initial lake stage.

STATION - LS-14 (Rocky Reef)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	0.7	0.3
25	2.8	1.0
50	4.6	1.0
75	5.4	0.8
100	5.8	0.8
150	6.0	0.5
200	6.2	0.5

STATION - LS-12 (South of Okeechobee)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.0	0.3
25	2.0	0.4
50	2.8	0.7
75	3.2	0.6
100	3.4	0.6
150	3.6	0.4
200	3.8	0.4

STATION - LS-10 (North end of Kreamer Island)

Return Period (yrs)	Storm Surge (ft)	Standard Deviation (ft)
10	1.1	0.4
25	3.9	1.4
50	6.8	1.8
75	8.0	1.3
100	8.6	1.1
150	8.9	0.6
200	9.1	0.4

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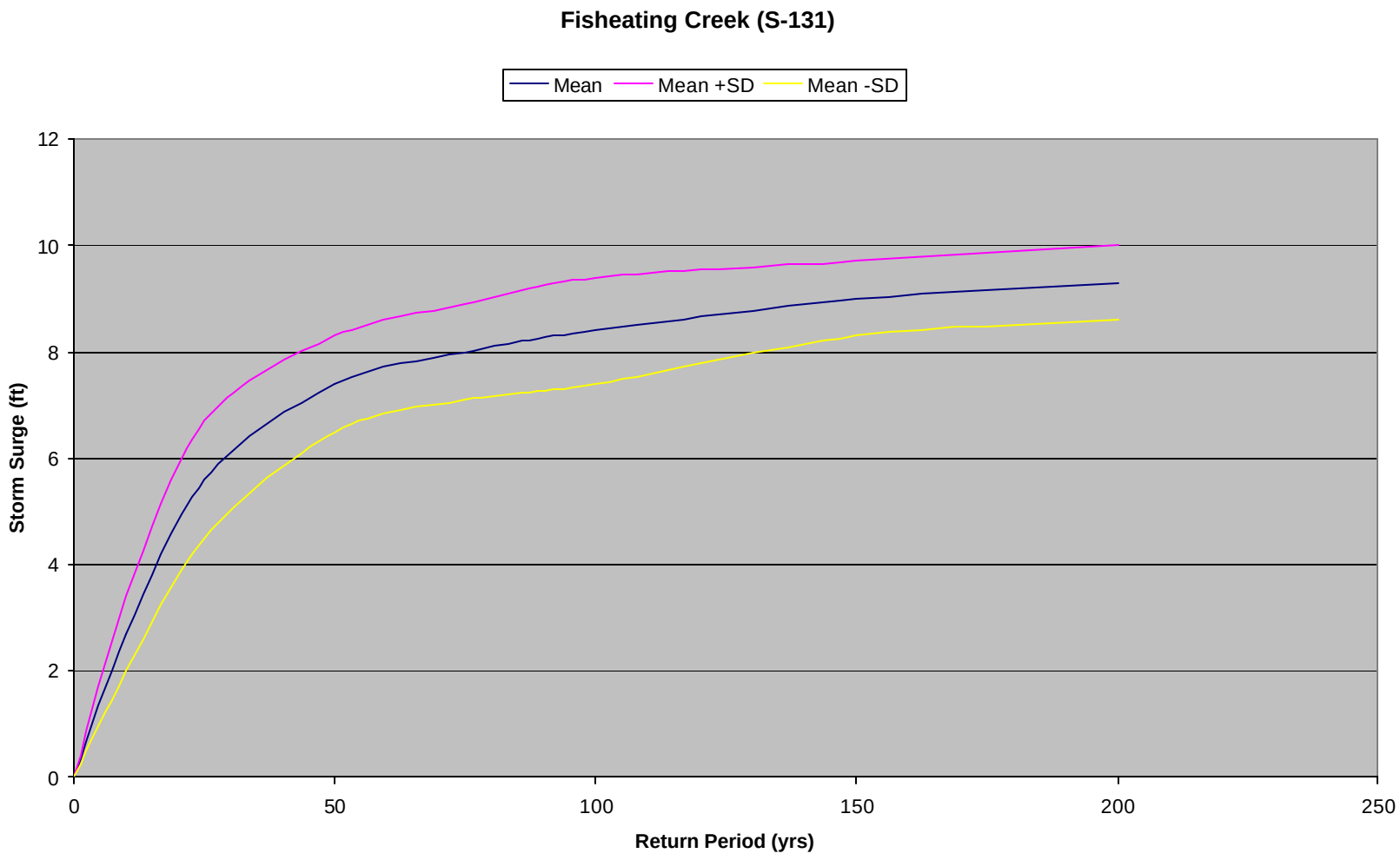
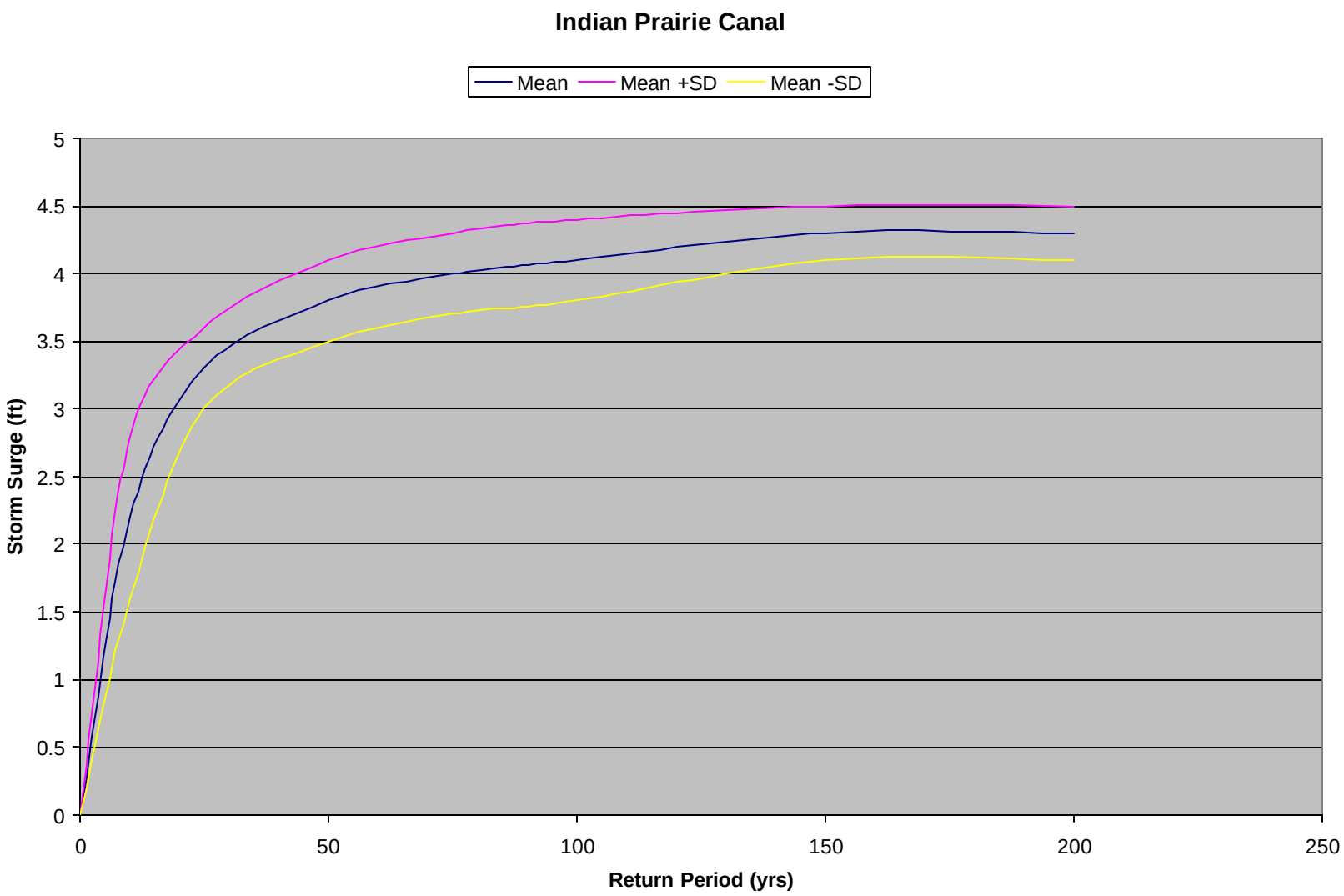
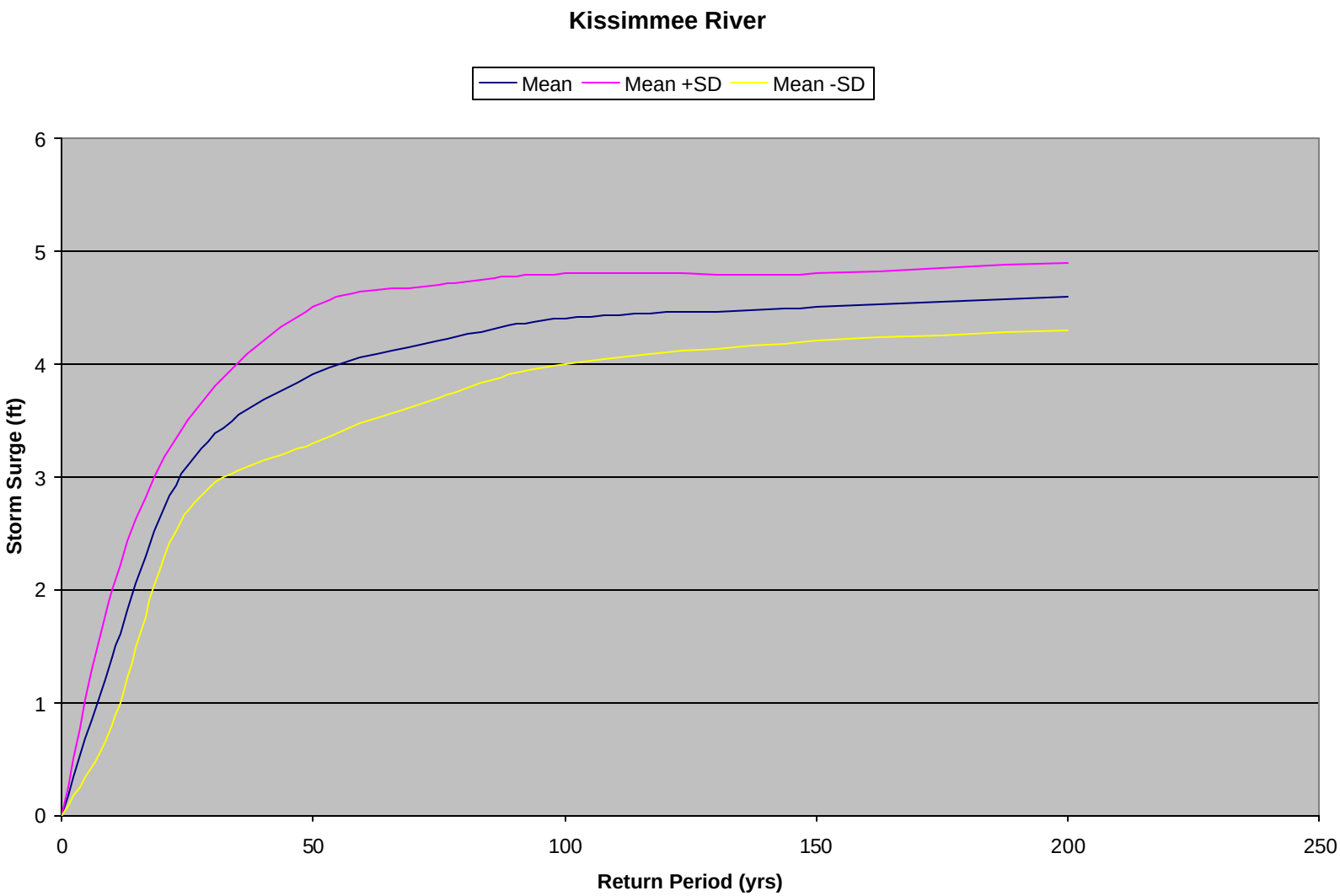
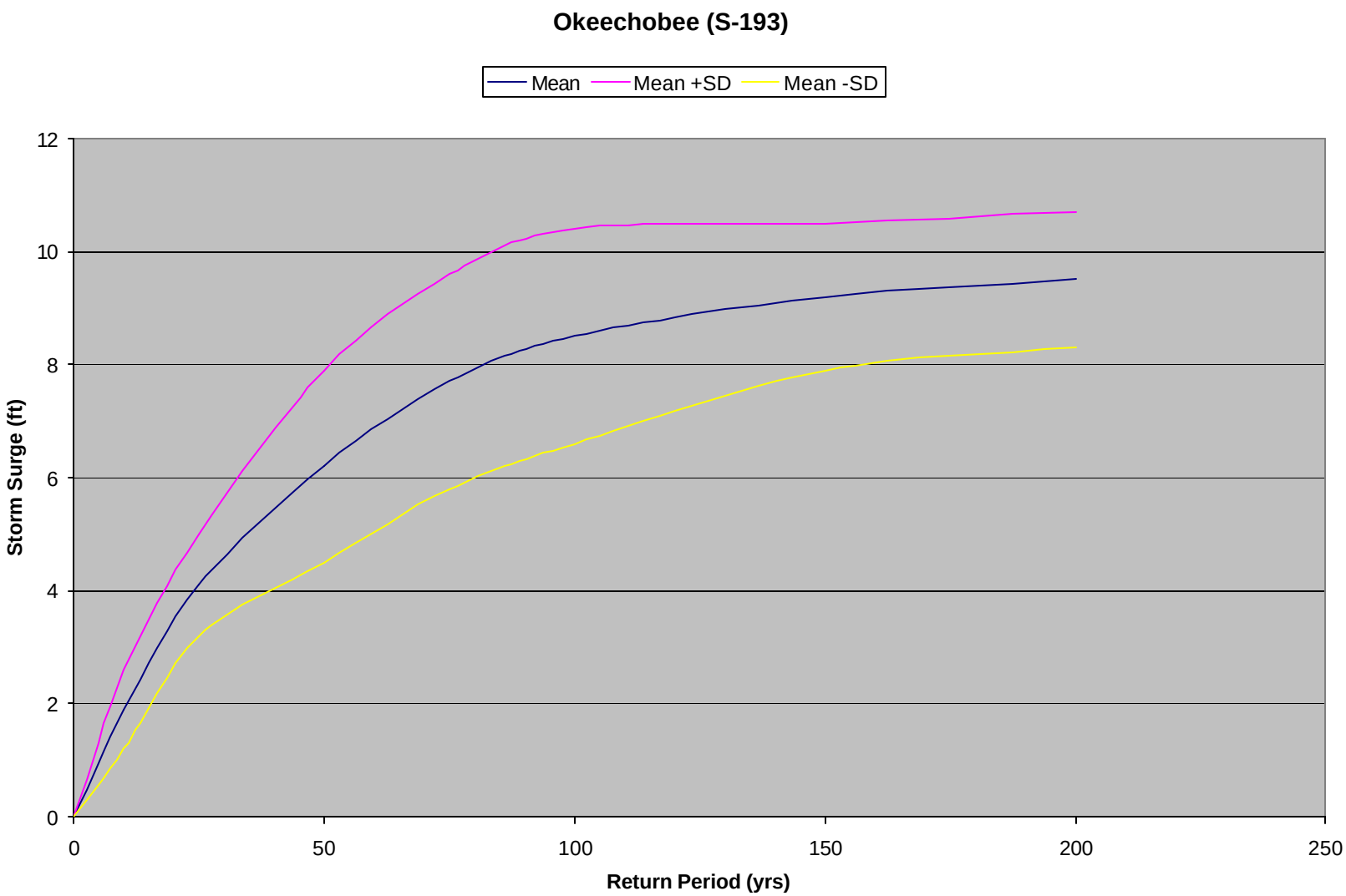
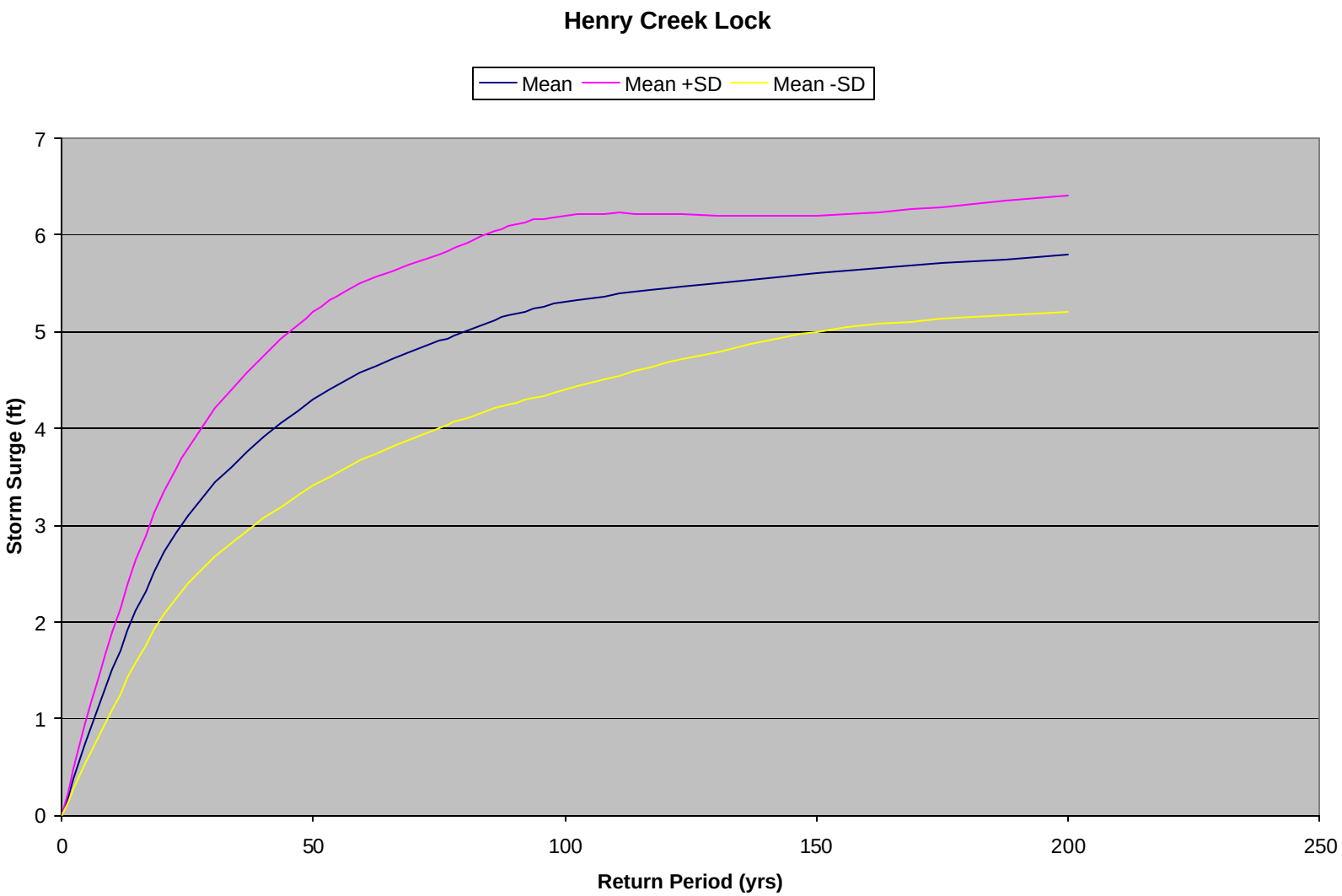
Surge-Frequency Graphs

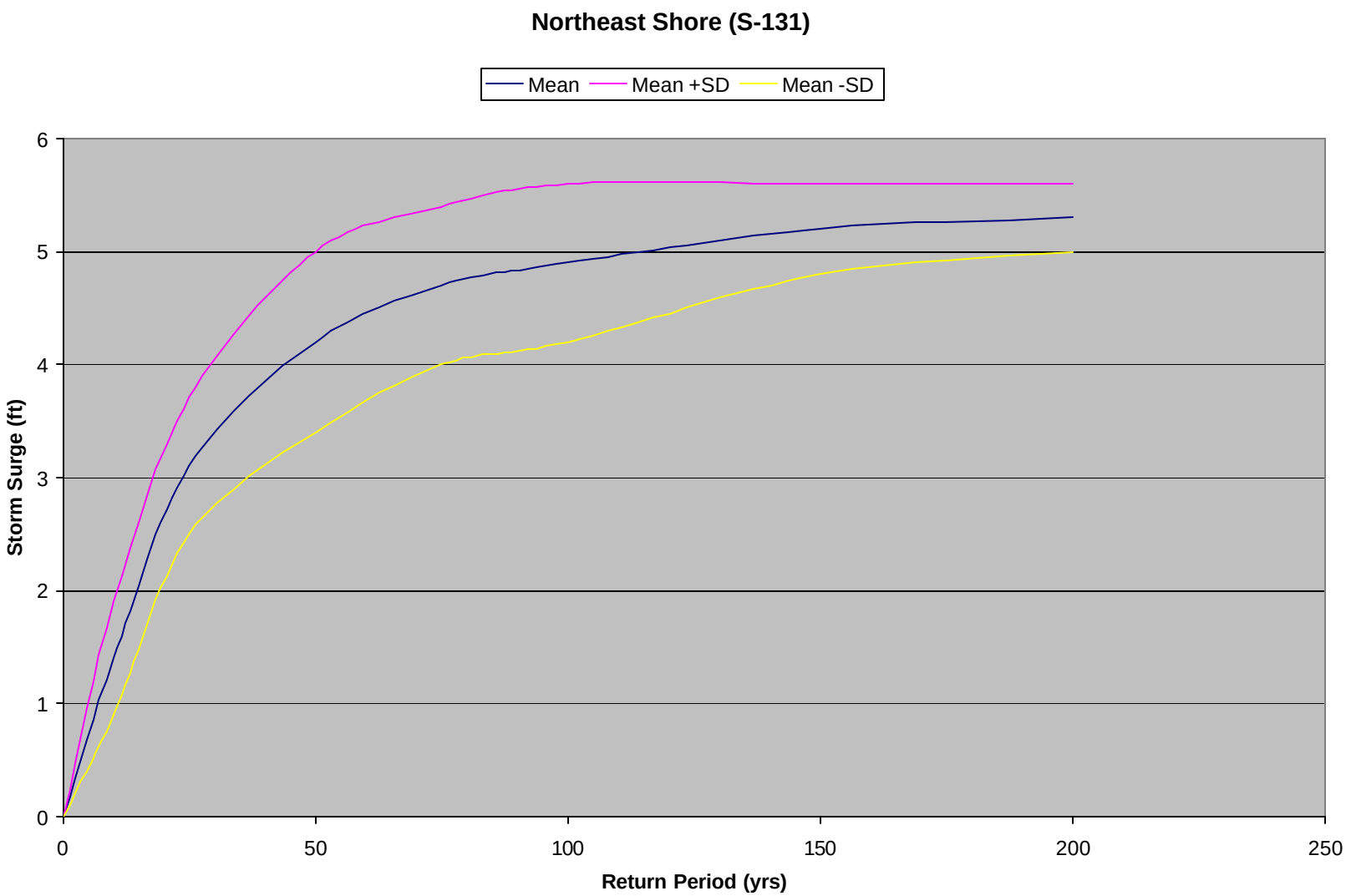

Fisheating Creek (S-131)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

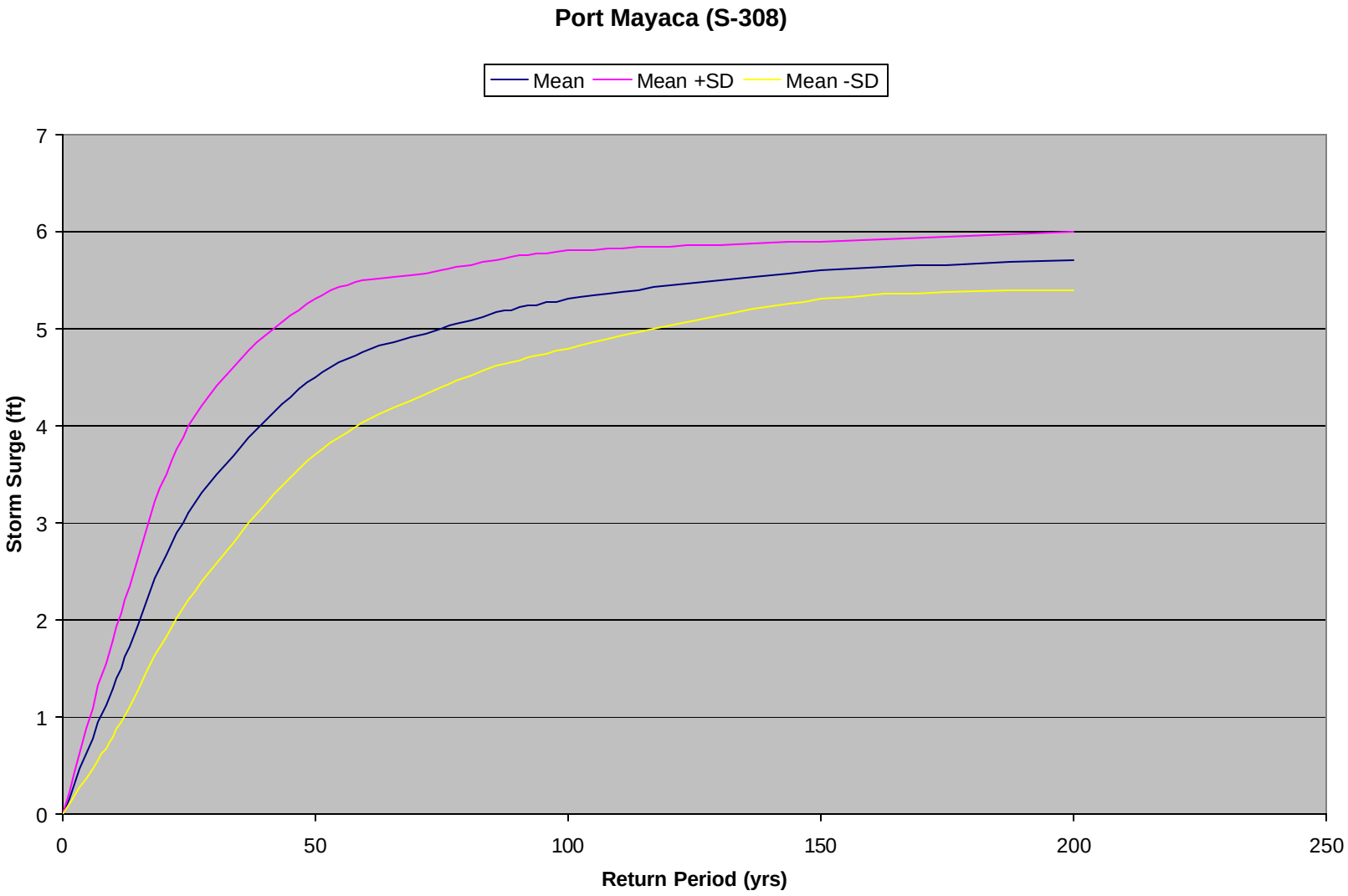

Indian Prairie Canal
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
50
100
150
200
250
Return Period (yrs)

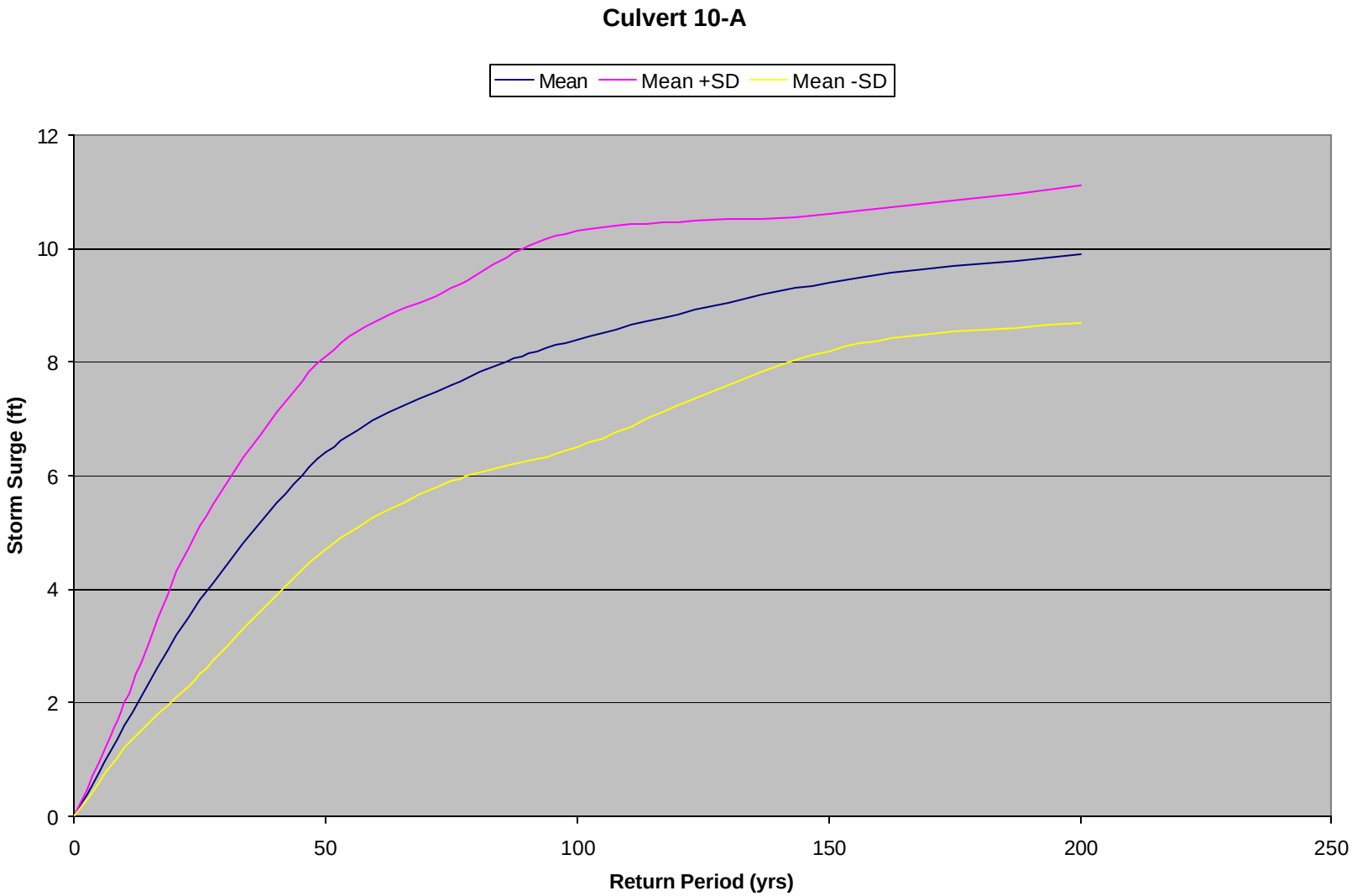

Kissimmee River
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
0
50
100
150
200
250
Return Period (yrs)

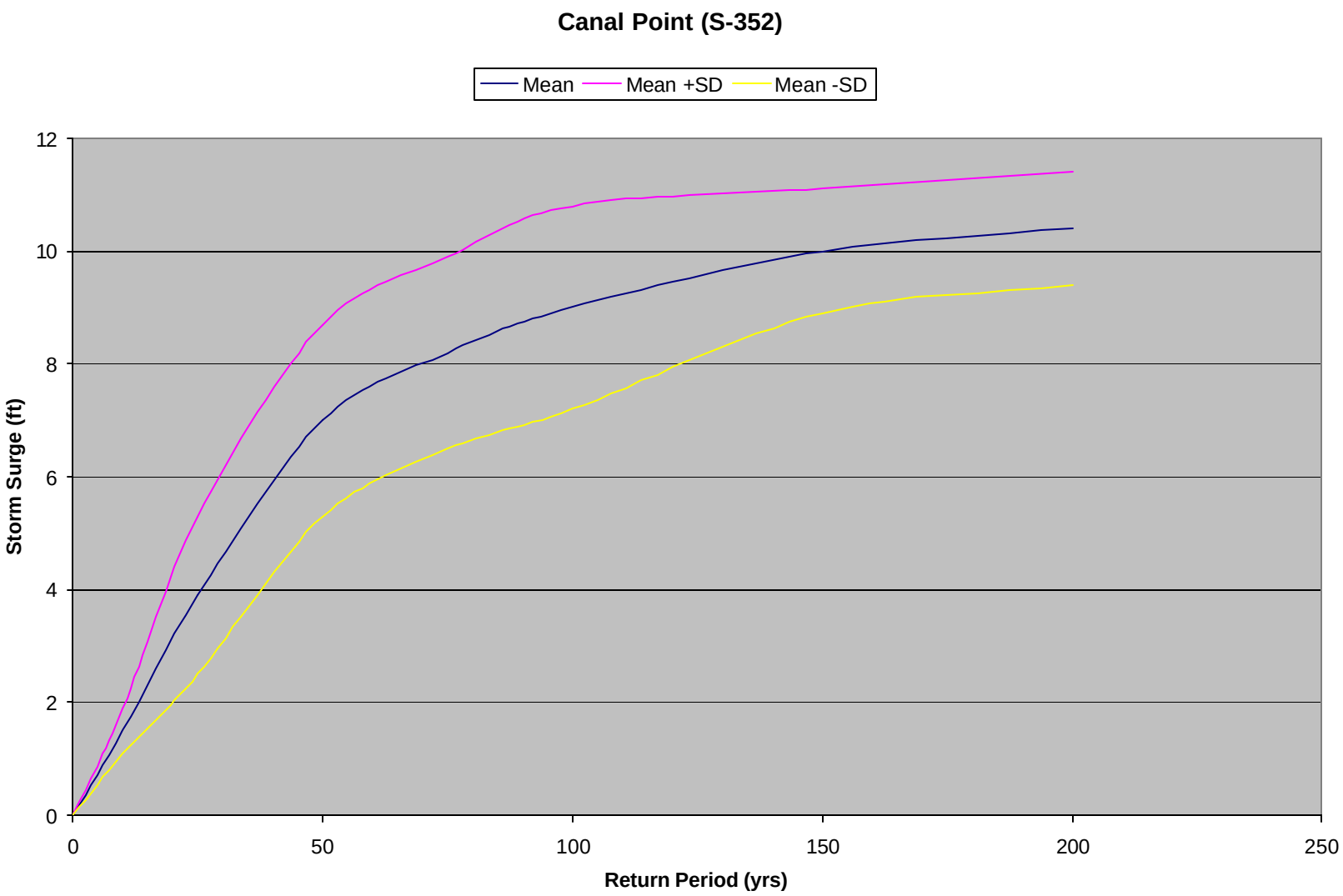

Okeechobee (S-193)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

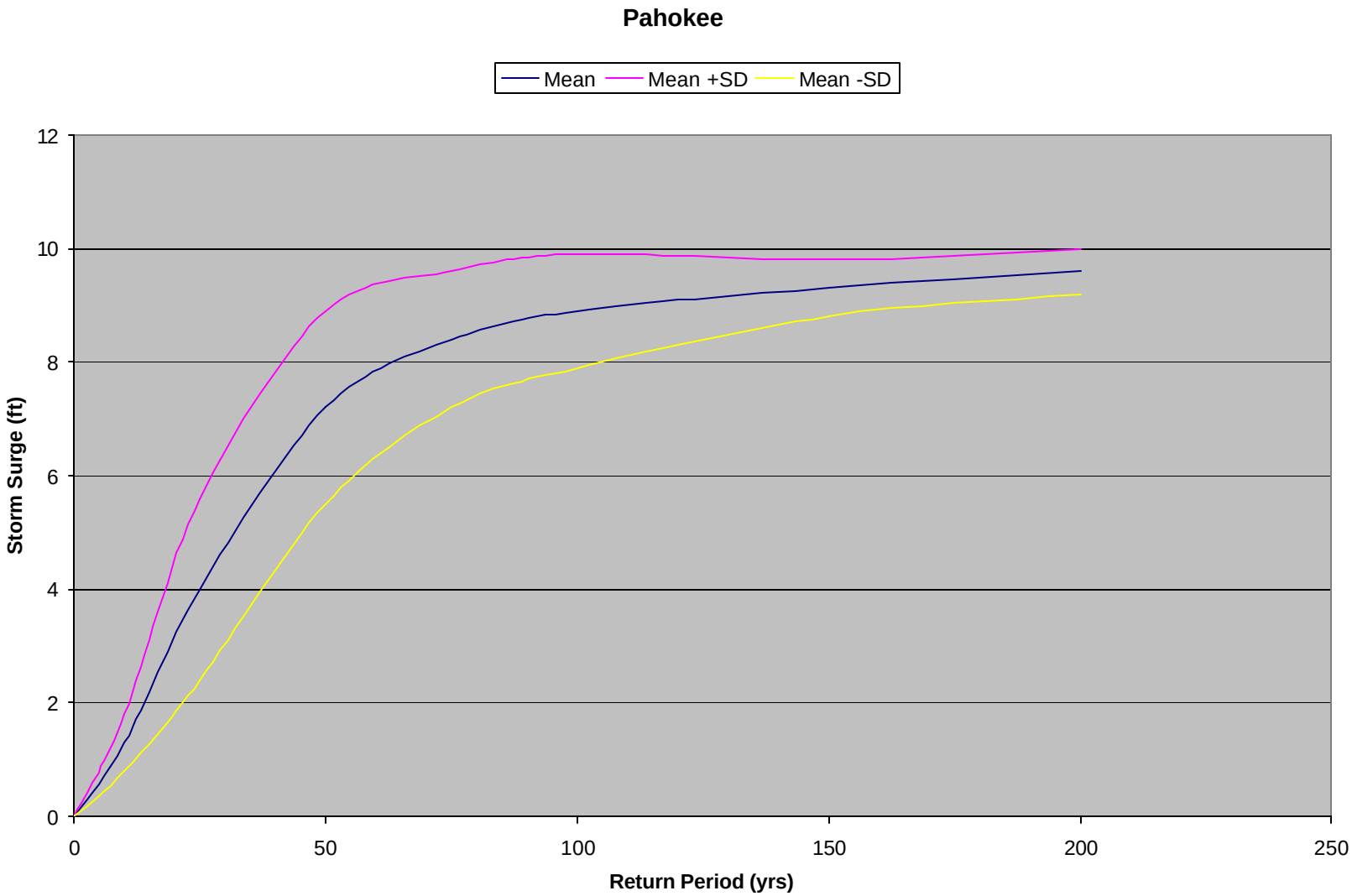

Henry Creek Lock
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
7
0
50
100
150
200
250
Return Period (yrs)

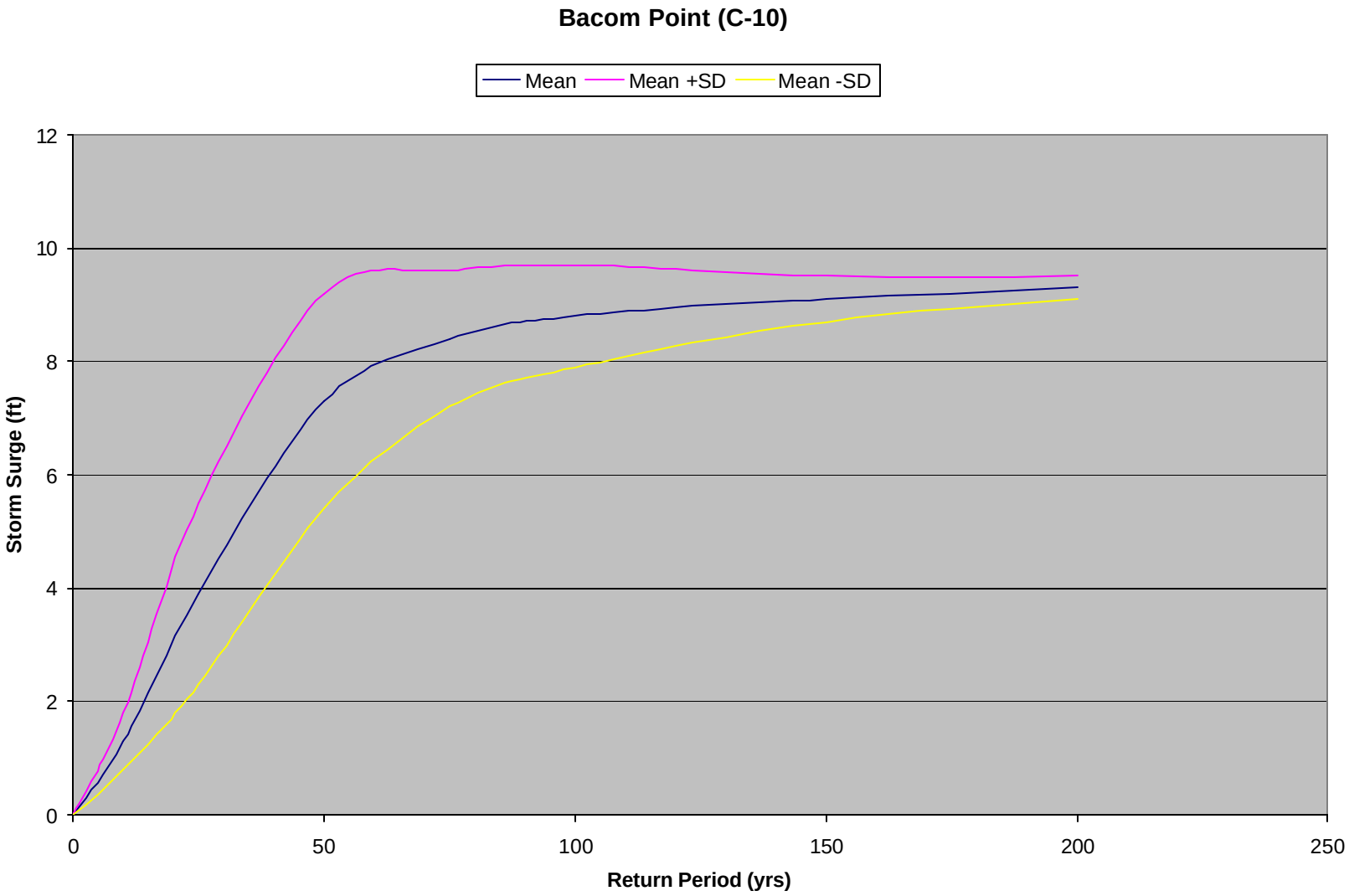

Northeast Shore (S-131)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
0
50
100
150
200
250
Return Period (yrs)

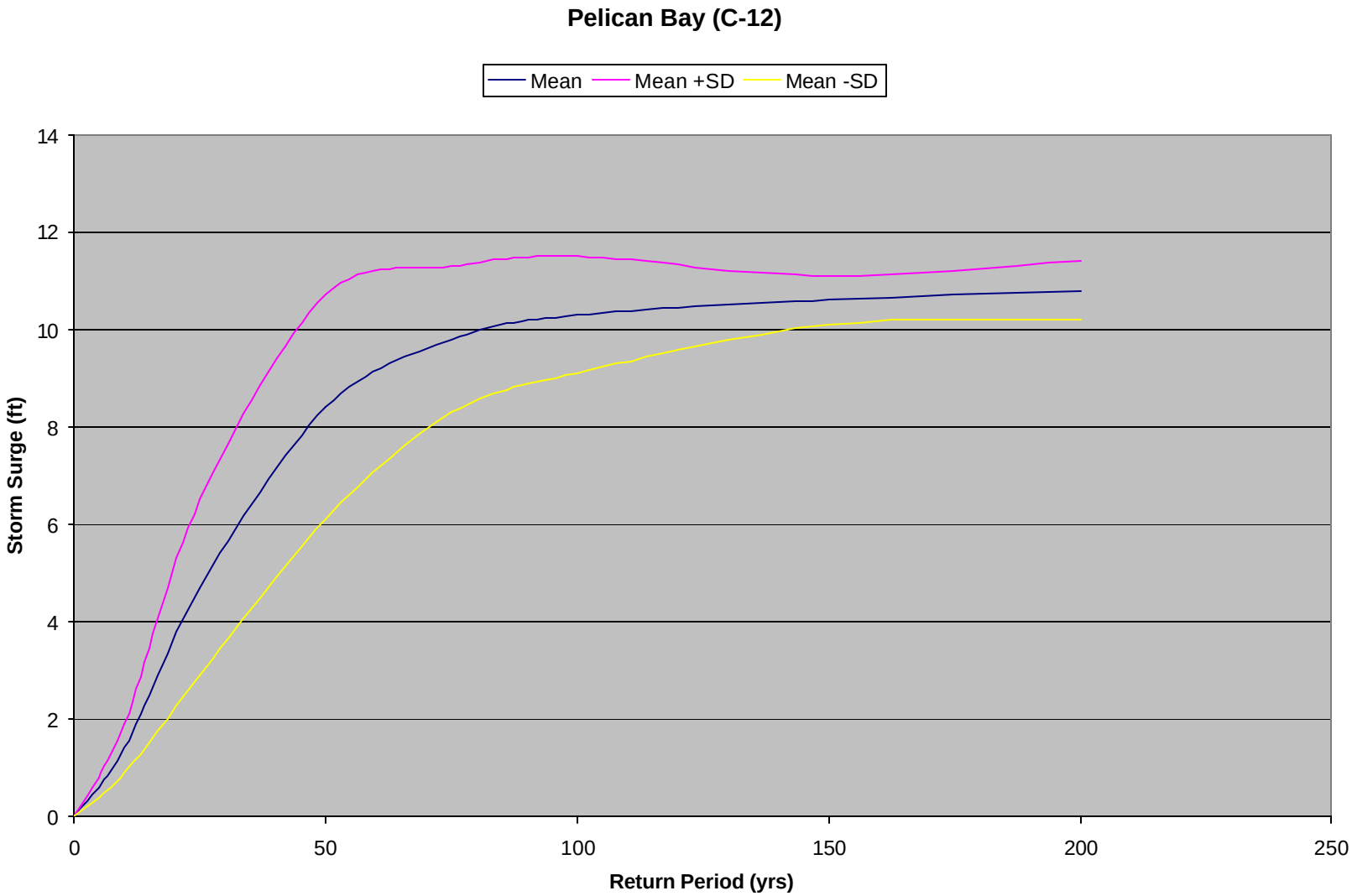

Port Mayaca (S-308)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
7
0
50
100
150
200
250
Return Period (yrs)

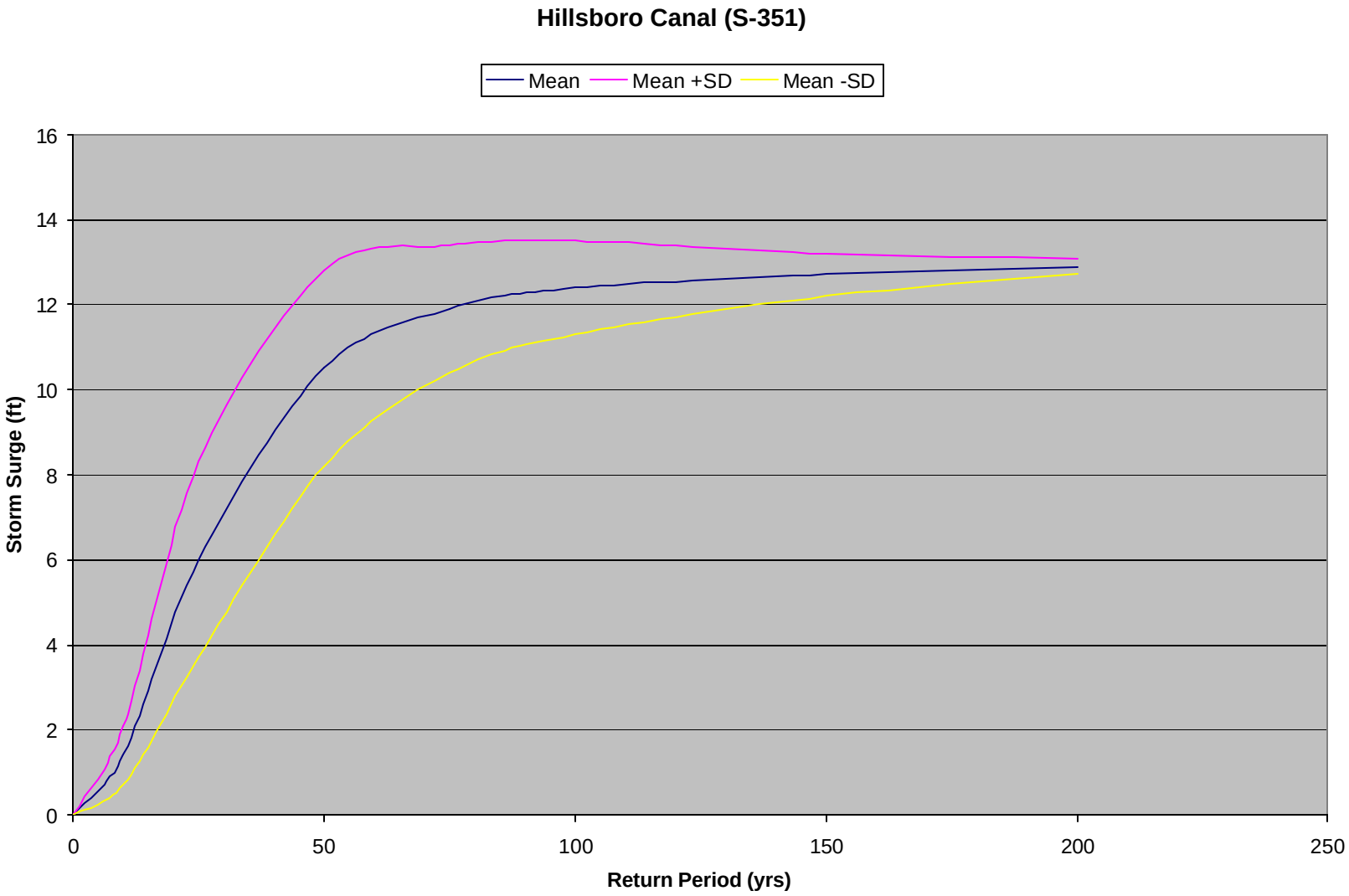

Culvert 10-A
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

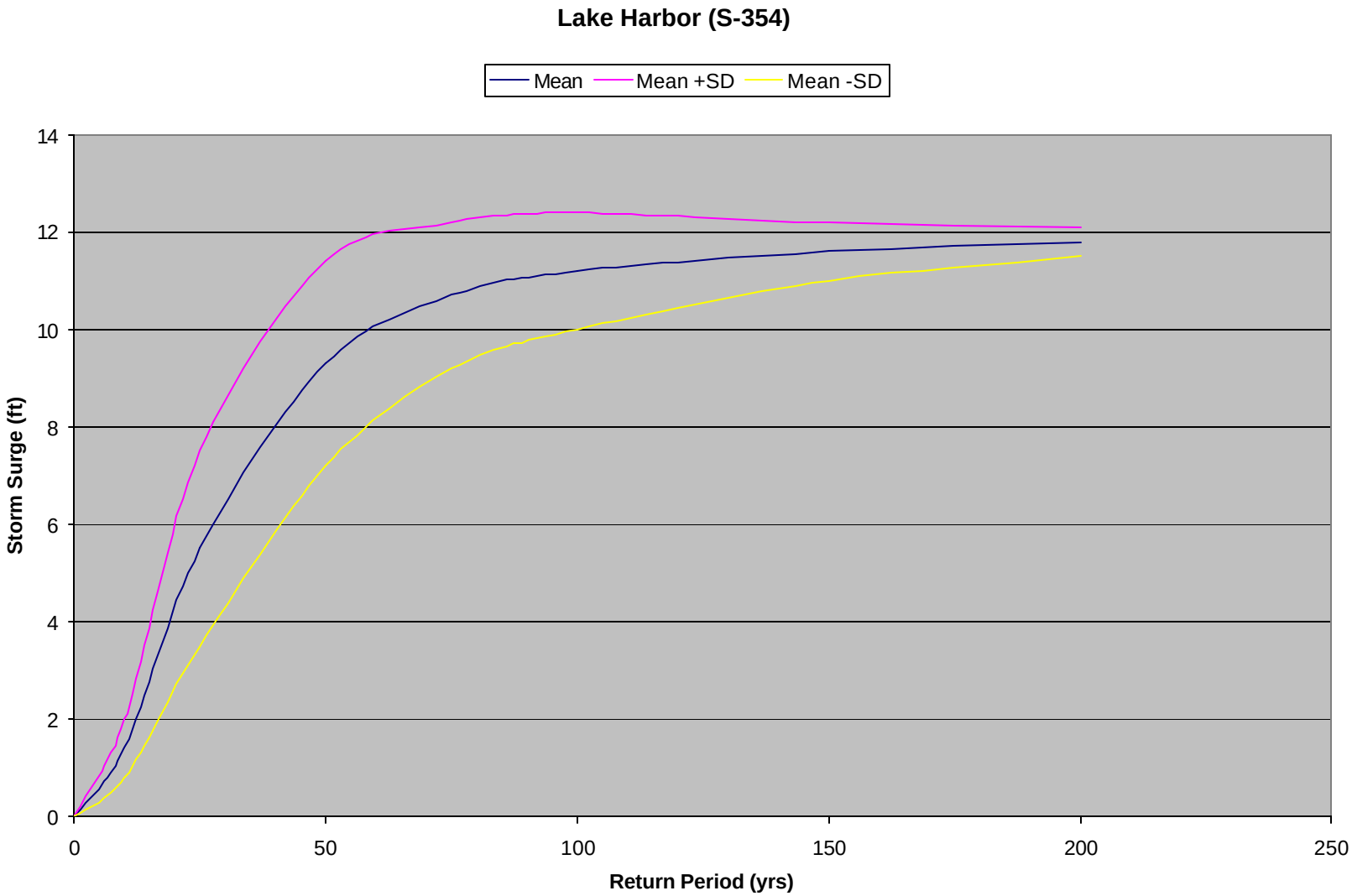

Canal Point (S-352)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

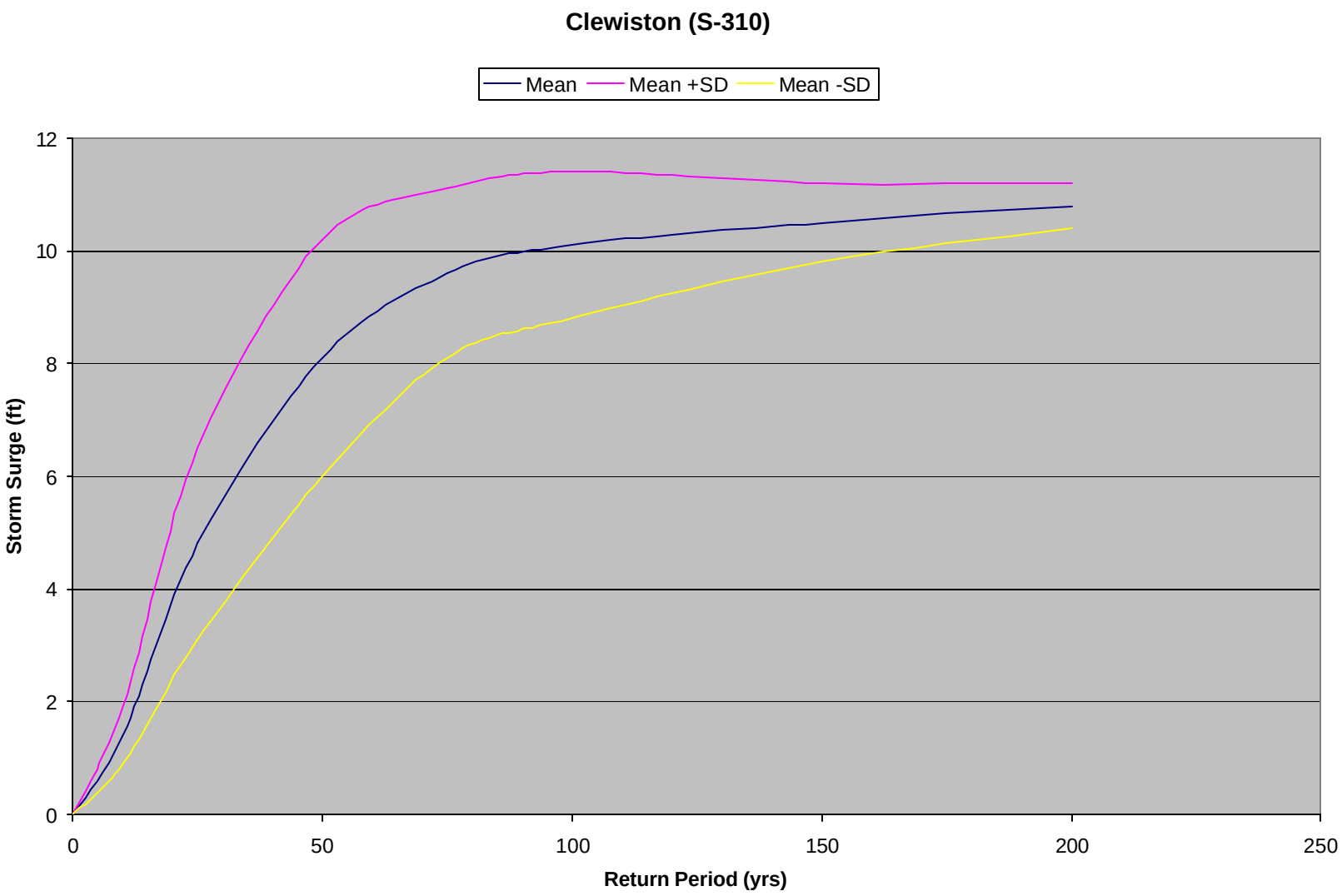

Pahokee
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

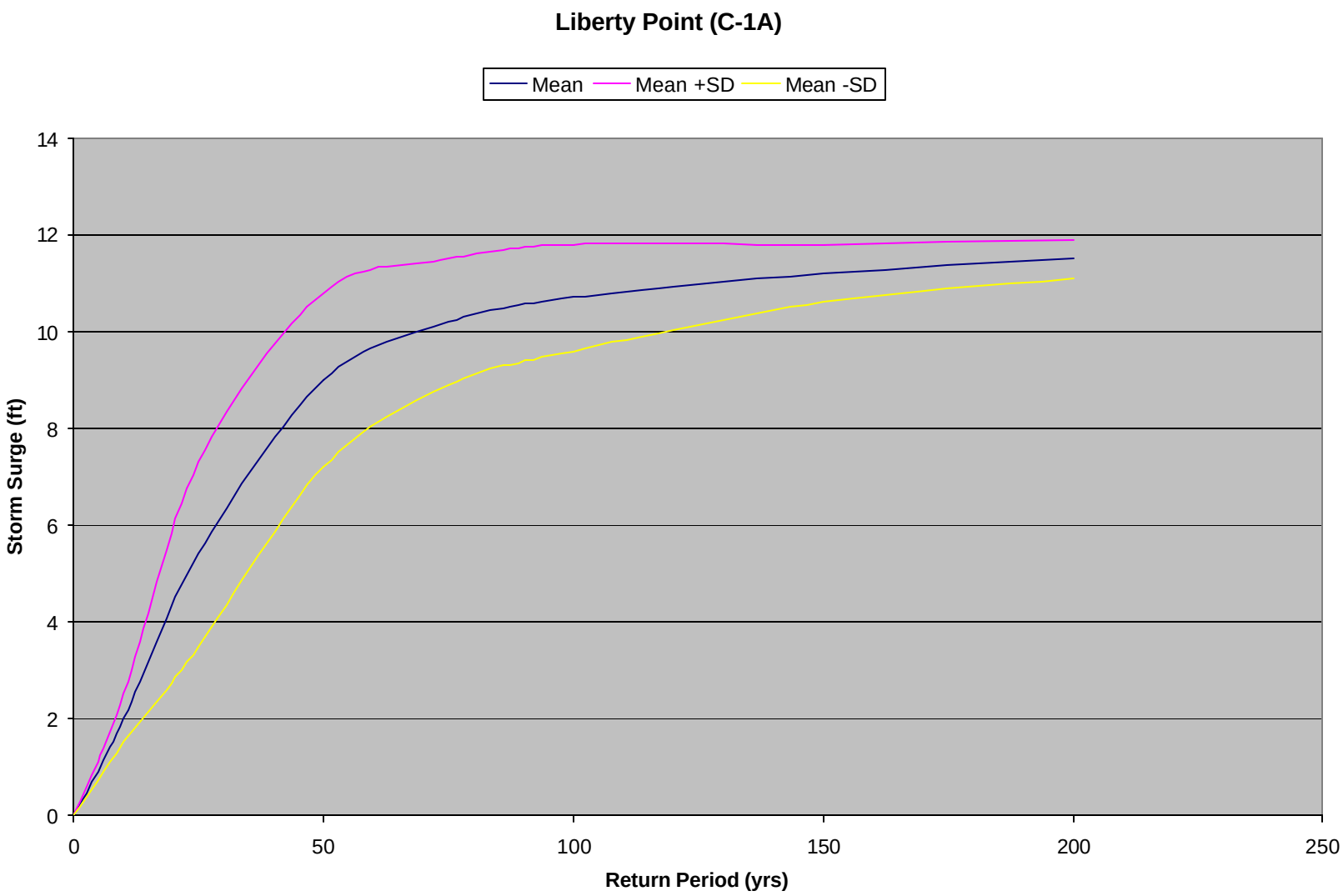

Bacom Point (C-10)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

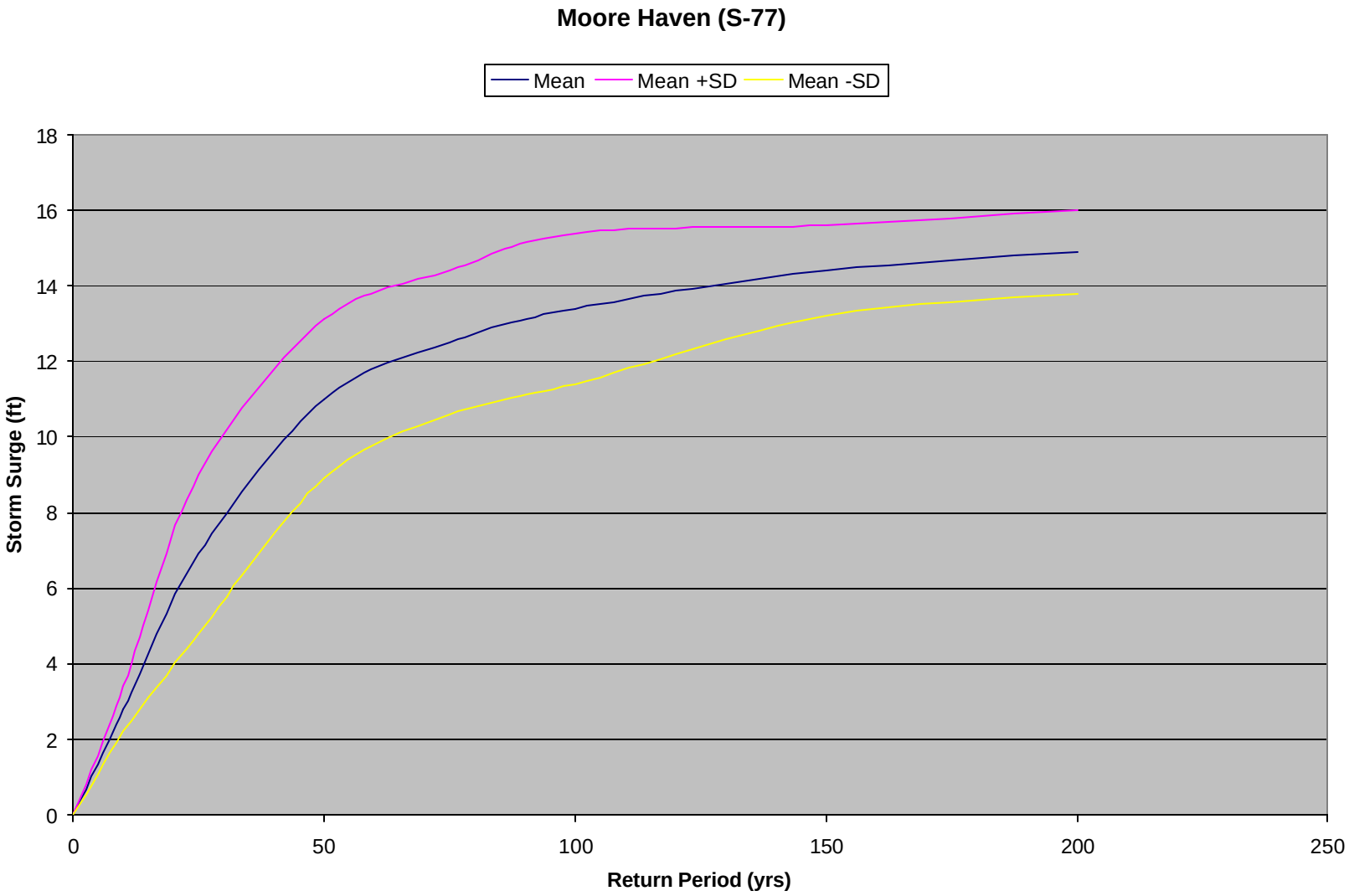

Pelican Bay (C-12)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
14
0
50
100
150
200
250
Return Period (yrs)

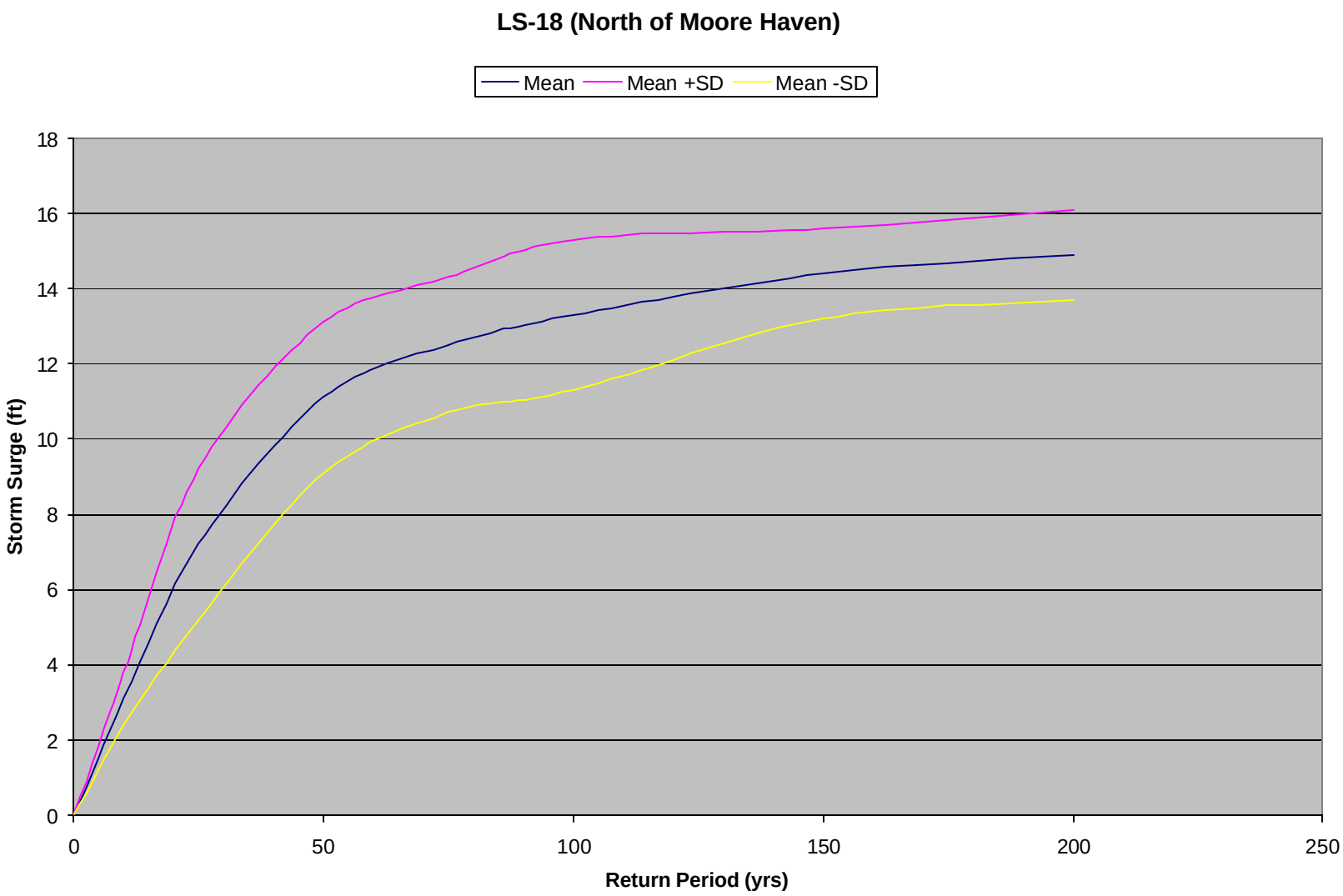

Hillsboro Canal (S-351)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
16
14
12
10
8
6
4
2
0
0
50
100
150
200
250
Return Period (yrs)

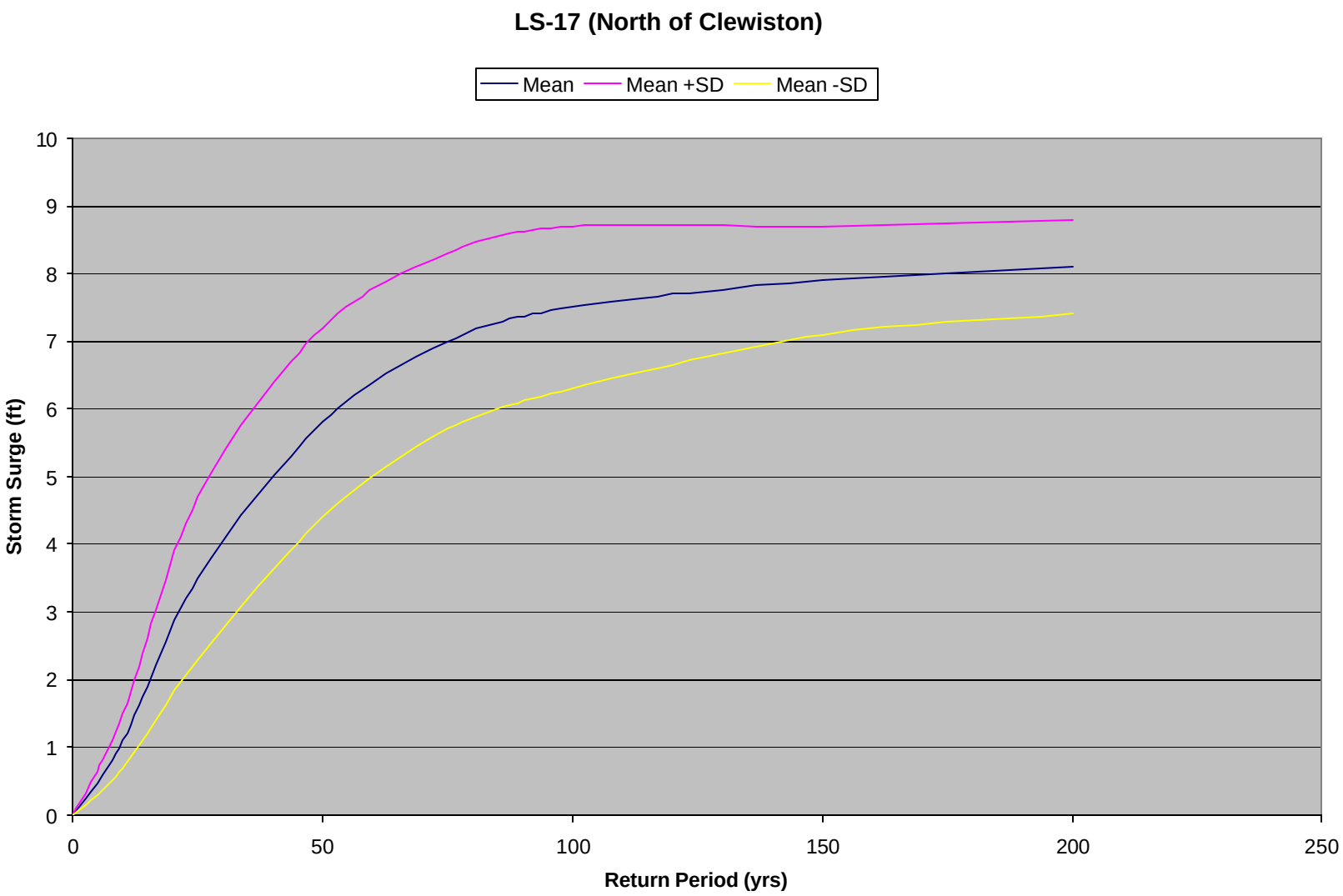

Lake Harbor (S-354)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
14
0
50
100
150
200
250
Return Period (yrs)

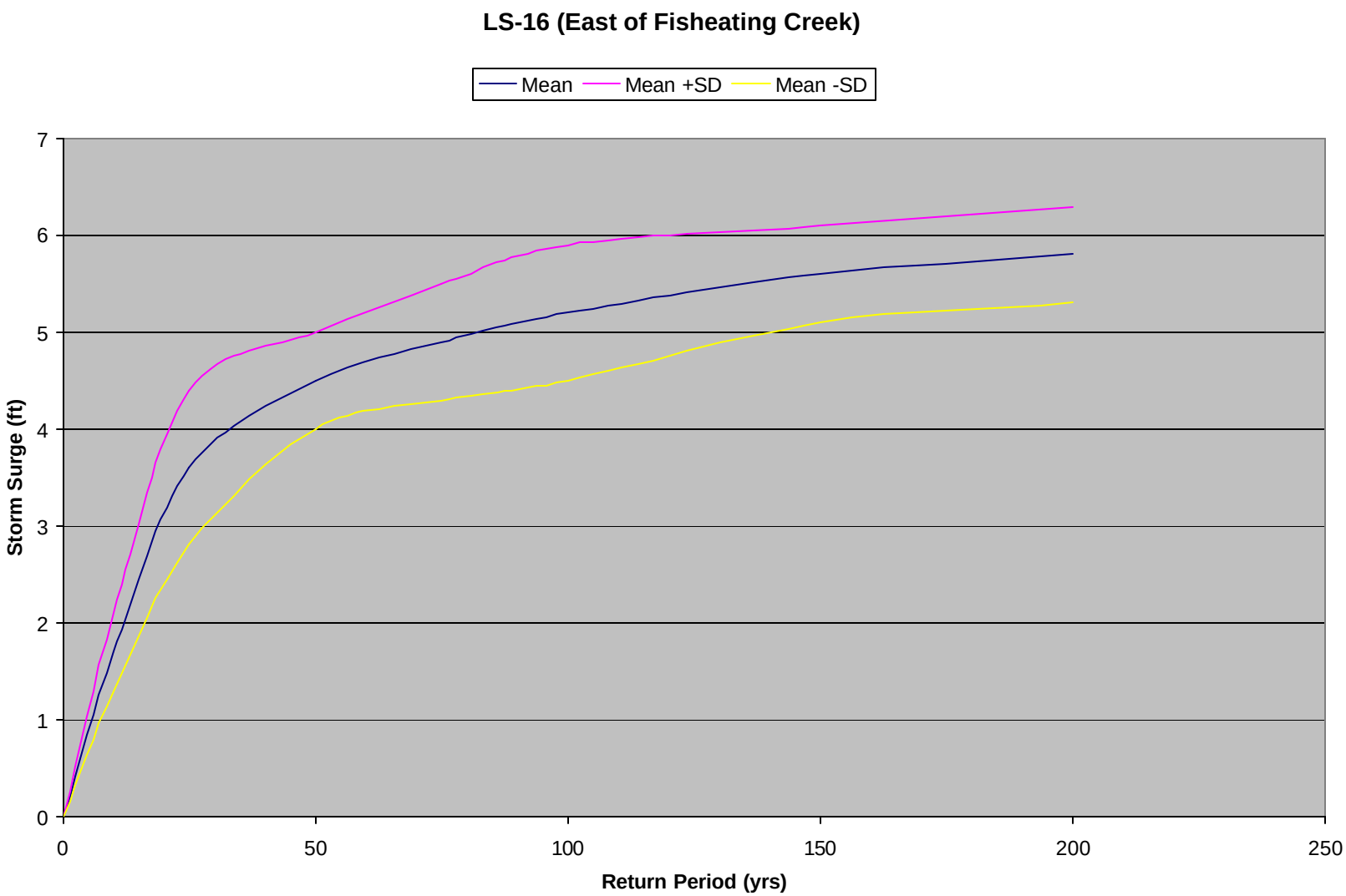

Clewiston (S-310)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

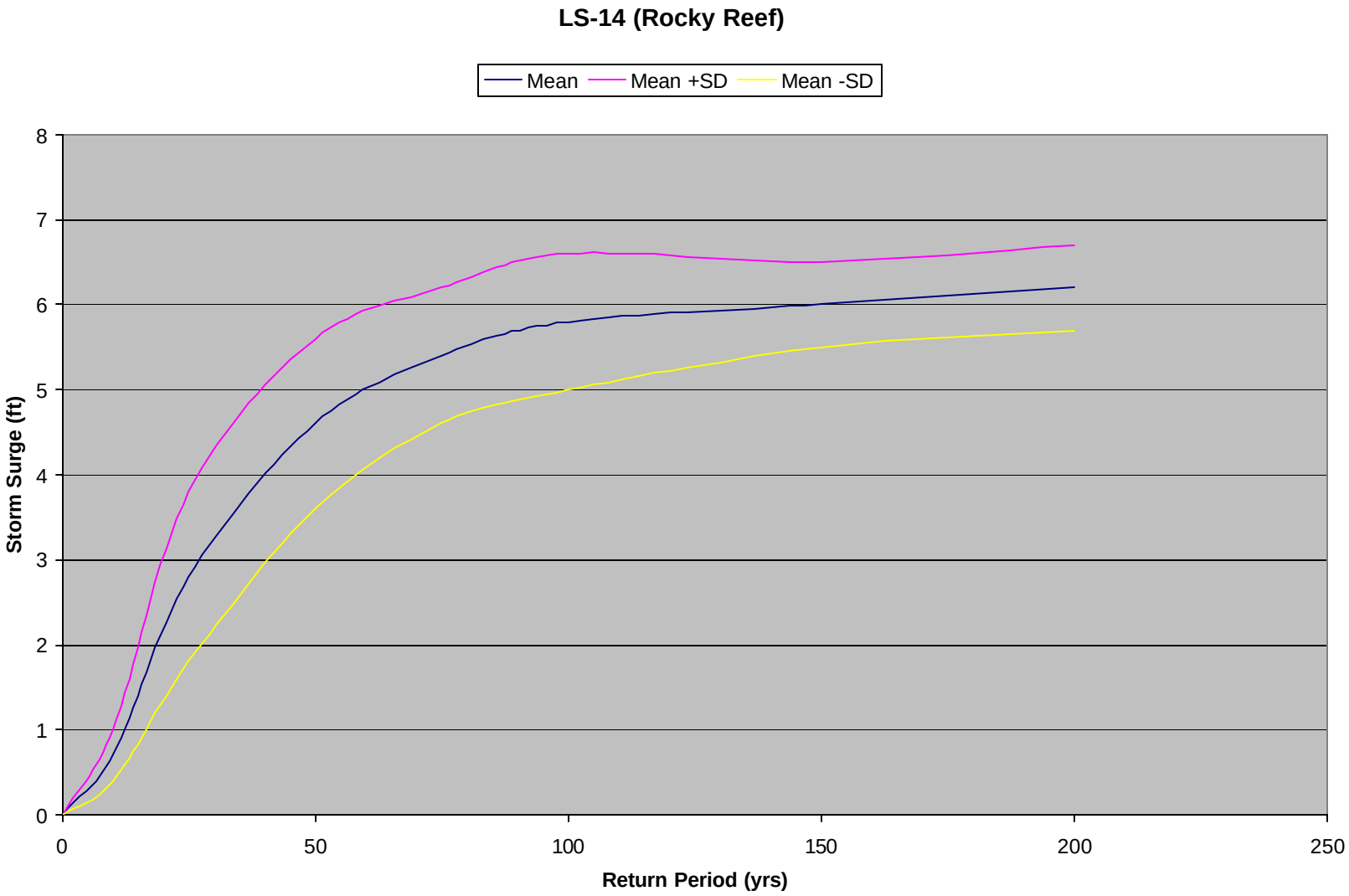

Liberty Point (C-1A)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
14
0
50
100
150
200
250
Return Period (yrs)

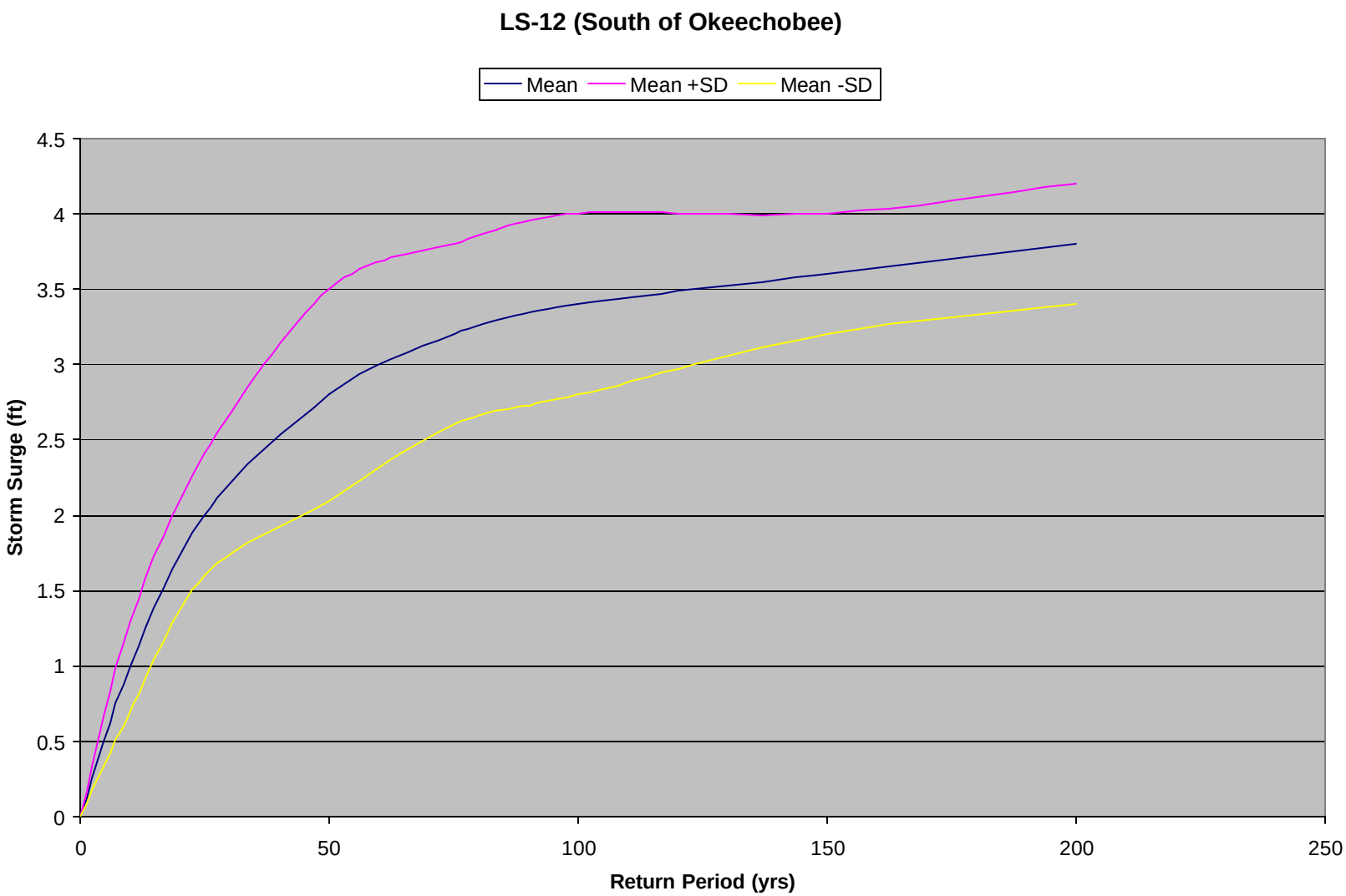

Moore Haven (S-77)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
14
16
18
0
50
100
150
200
250
Return Period (yrs)

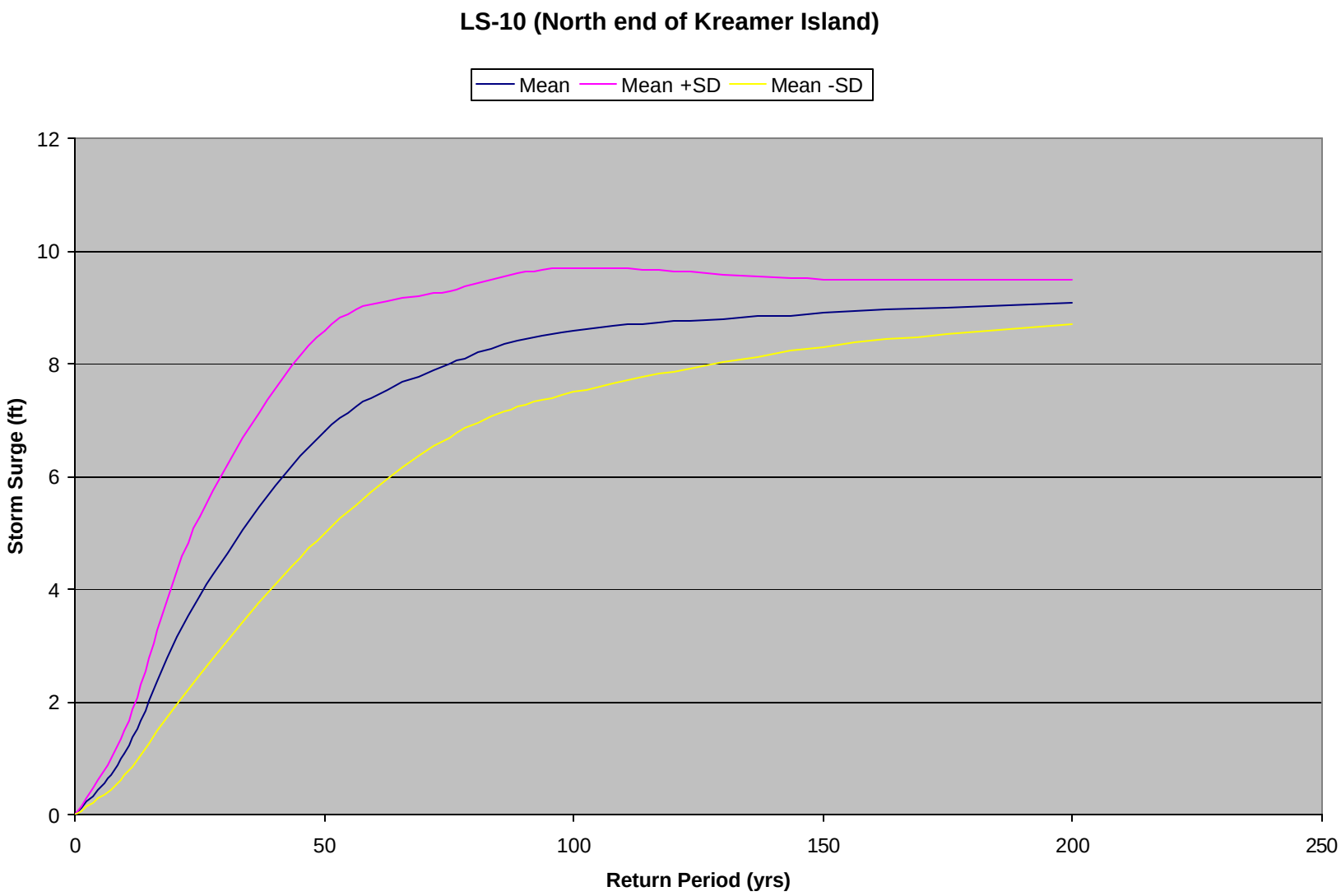

LS-18 (North of Moore Haven)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
14
16
18
0
50
100
150
200
250
Return Period (yrs)


LS-17 (North of Clewiston)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
7
8
9
10
0
50
100
150
200
250
Return Period (yrs)


LS-16 (East of Fisheating Creek)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
7
0
50
100
150
200
250
Return Period (yrs)


LS-14 (Rocky Reef)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
1
2
3
4
5
6
7
8
0
50
100
150
200
250
Return Period (yrs)


LS-12 (South of Okeechobee)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
50
100
150
200
250
Return Period (yrs)


LS-10 (North end of Kreamer Island)
Mean
Mean +SD
Mean -SD
Storm Surge (ft)
0
2
4
6
8
10
12
0
50
100
150
200
250
Return Period (yrs)

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Inundation Mapping and Flood Stage Hydrographs

REFERENCES

Borgman, L.E., Miller, M.C., Butler, H.L., and Reinhard, R.D., 1992. "Empirical Simulation of Future Hurricane Storm Histories as a Tool in Engineering and Economic Analysis," ASCE Proceedings, Civil Engineering in the Oceans V, College Station, TX, 2-5 November 1992.

Butler, H.L., 1978. "Numerical Simulation of Tidal Hydrodynamics: Great Egg Harbor and Corson Inlets, New Jersey," Technical Report H-78-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cardone, V.J., Greenwood, C.V., and Greenwood, J.A., 1992. "Unified Program for the Specification of Hurricane Boundary Layer Winds Over Surfaces of Specified Roughness," Contract Report CERC-92-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Chow, Shu-Hysian, 1971. "A Study of the Wind Field in the Planetary Boundary Layer of a Moving Tropical Cyclone," New York University, Department of Meteorology, New York, NY.

Cialone, M.A., Mark, D.J., Chou, L.W., Leenknecht, D.A., Davis, J.E., Lillycrop, L.S., Jensen, R.E., Thompson, E.F., Gravens, M.B., Rosati, J.D., Wise, R.A., Kraus, N.C., and Magnus Larson, P., 1993. "Coastal Modeling System (CMS) User's Manual," Instruction Report CERC-91-1, U.S. Army Engineers, Coastal Engineering Research Center, Vicksburg, MS.

Efron, Bradley, 1982. "The Jackknife, the Bootstrap, and Other Resampling Plans," Regional Conference Series in Applied Mathematics No. 38, Society for Industrial and Applied Mathematics, Philadelphia, PA.

Farrer, L.A., 1958. "Wind Tides on Lake Okeechobee," Proceedings of the 6th Coastal Engineering Conference, Chapter 7, Council on Wave Resources, The Engineering Foundation, New York, NY.

Garratt, J.R. 1977. "Review of Drag Coefficients over Oceans and Continents," Monthly Weather Review, Vol 105, pp 915-929.

Gumbel, E.J., 1954. "Statistical Theory of Extreme Value and Some Practical Applications," National Bureau of Standards Applied Mathematics Series 33, U.S. Govt. Publication, Washington, D.C.

Jarvinen, B.R., Neumann, C.J., and Davis, M.A., 1988. "A Tropical Cyclone Data Tape for the North Atlantic Basin, 1886-1983: Contents, Limitations, and Uses," NOAA Technical Memorandum NWS NHC 22, National Hurricane Center, Miami, FL.

Jelesnianski, C.P. and Taylor, A.D., 1973. "A Preliminary View of Storm Surges Before and After Storm Modifications," NOAA Technical Memorandum ERL WMPO-3, Weather Modification Program Office, Boulder, CO.

Kivisild, H.R., 1954. "Wind Effect on Shallow Bodies of Water with Special Reference to Lake Okeechobee," Acta Polytechnica, Civil Engineering and Building Construction Series Vol.3 No.1, Royal Swedish Academy of Engineering Sciences, Stockholm, Sweden.

Langhaar, H.L., 1951. "Wind Tides in Inland Waters," Proceedings of the First Midwestern Conference on Fluid Dynamics, pp. 278-296, J.W. Edwards, Ann Arbor, MI.

Murty, T.S., 1984. "Storm Surges - Meteorological Ocean Tides," Bulletin 212, Department of Fisheries and Oceans, Ottawa, Canada.

Myers, V.A., 1954. "Characteristics of United States Hurricanes Pertinent to Levee Design for Lake Okeechobee, Florida," Hydrometeorology Dept. 32, Weather Bureau, U.S. Department of Commerce.

National Weather Service, 1979. "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States," NOAA Technical Report NWS 23, Washington, D.C.

Reid, R.O., Vastano, A.C., and Reid, T.J., 1977. "Development of SURGE II Program with Application to the Sabine-Calcasieu Area for Hurricane Carla and Design Hurricanes," Technical Paper No. 77-13, U.S. Army Engineers, Coastal Engineering Research Center, Vicksburg, MS.

Reid, R.O., and Bodine, B.R., 1968. "Numerical Model for Storm Surges in Galveston Bay", ASCE Journal of Waterways and Harbors Division, No. WWI 1968, pp. 33-57.

Scheffner, N.W., Borgman, L.E., and Mark, D.J., 1993. Applications of Large Domain Hydrodynamic Models To Generate Frequency-of-Occurrence Relationships", ASCE Estuary and Coastline Modeling Conference, Chicago, IL, September 1993.

Schloemer, R.W., 1954. "Analysis and Synthesis of Hurricane Wind Patterns over Lake Okeechobee, Florida," Hydrometeorological Report No. 31, U.S. Government Publication, Washington, D.C.

Schmaltz, R.A., Jr., 1986. "A Numerical Investigation of Hurricane Induced Water Level Fluctuations in Lake Okeechobee," (3 reports), Miscellaneous Paper CERC-86-12, U.S. Army Corps of Engineers, Coastal Engineering Research Center, Vicksburg, MS.

Thompson, E.F., 1994. Unpublished notes on Coastal Engineering.

Thompson, E.F. and Cardone V.J., 1994. "Practical Modeling of Hurricane Surface Wind Fields", ASCE Journal of Waterway, Port, Coastal and Ocean Engineering, (in publication).

U.S. Army Corps of Engineers, Jacksonville District, 1954. "Central and Southern Florida Project, Part IV, Supplement 2, Section 3 (Revised), Design Memorandum, Wind Tides Produced by Hurricanes", Jacksonville, FL.

Vreugdenhil. C.B. 1973. "Secondary Flow Computations", Publication No. 114, Delft Hydraulics Laboratory, Delft, The Netherlands.

War Department, 1940. "Review of Previous Reports on Caloosahatchie River and Lake Okeechobee Drainage Areas, Florida", U.S. Engineers Office, Jacksonville.

Whitaker, R.E., Reid, R.O., and Vastano, A.C., 1973. "Drag Coefficient at Hurricane Wind Speeds as Deduced from the Numerical simulation of Dynamical Water Level Changes in Lake Okeechobee", Texas A&M University, College Station, TX.

Herbert Hoover Dike

Major Rehabilitation Evaluation Report

Appendix J

Real Estate Appendix

TABLE OF CONTENTS

Section No.		Page
1.	STATEMENT OF PURPOSE	4
2.	PROJECT AUTHORIZATION	4
3.	PROJECT LOCATION	4
4.	PROJECT DESCRIPTION	5
5.	GOVERNMENT-OWNED LAND	5
6.	NON-FEDERALLY-OWNED LAND	6
7.	REAL ESTATE REQUIREMENTS	6
8.	OPERATION/MAINTENANCE RESPONSIBILITIES	6
9.	APPRAISAL INFORMATION	7
10.	ATTITUDE OF OWNERS	7
11.	MINERALS	7
12.	HAZARDOUS AND TOXIC WASTE (HTW)	7
13.	RELOCATIONS ASSISTANCE (PUBLIC LAW 91-646)	7
14.	RELOCATIONS, ALTERATIONS, VACATIONS, AND ABANDONMENTS	8
15.	INDUCED FLOODING	8
16.	STANDING TIMBER AND VEGETATIVE COVER	8
17.	RECREATION RESOURCES	8
18.	CULTURAL RESOURCES	8
19.	OUTSTANDING RIGHTS	8
20.	ACQUISITION/ADMINISTRATIVE COSTS	9
21.	SUMMARY OF PROJECT REAL ESTATE COSTS	9
22.	REAL ESTATE ACQUISITION SCHEDULE	9

23. ESTATES TO BE ACQUIRED 9

24. CHART OF ACCOUNTS FOR PROJECT 10

Project Map (Location) . . . . . Exhibit A

Project Map (Reach 1) . . . . . Exhibit B

Project Map (Recommended Plan) . . . . . Exhibit C

REAL ESTATE PLAN
HERBERT HOOVER DIKE
MAJOR REHABILITATION REPORT

1. STATEMENT OF PURPOSE.

a. The purpose of this report is to determine the feasibility of addressing seepage and stability problems along the Herbert Hoover Dike System.

b. This real estate plan is tentative in nature for planning purposes only and both the final real property acquisition lines and estimates of value are subject to change even after approval of this report.

2. PROJECT AUTHORIZATION.

The Flood Control Act, approved by Congress on 30 June 1948, authorized the first phase of a comprehensive plan to provide flood protection and other water control benefits in central and southern Florida. The Act included measures for improving control of Lake Okeechobee by constructing or modifying the spillways and other structures, and enlarging the Lake Okeechobee levees to provide the intended flood protection, water storage and water supply. Levee seepage and stability have a direct effect on the capability of the levee to provide the authorized protection. The authorization for levee repairs and modifications of the Flood Control Act of 1948 justify the seepage and stability analyses identified in this MRR.

The following list consist of authorizations related to the Herbert Hoover Dike System:

a. Rivers and Harbors Act of 1930
b. Rivers and Harbors Act of 1935
c. Flood Control Act of 1948
d. Flood Control Act of 1954
e. Flood Control Act of 1958
f. Flood Control Act of 1960

g. Flood Control Act of 1968

3. PROJECT LOCATION.

The geographic area addressed by this report is located in southern Florida and consist of the levee system encircling Lake Okeechobee, including all structures within the levee system (Exhibit A). The entire system consists of approximately 143 miles of levee,(divided into nine levee segments: L-D1, L-D2, L-D3, L-D4, L-D9, L-47, L-48, L-49, and L-50), the 19 original culverts, six hurricane gates and other hydraulic structures. Study areas are defined as Reaches 1 – 8, in order of priority. The focus of this report is Reach 1. Reach 1 is 22.44 miles long and extends from St. Lucie Canal at Port Mayaca, on the east side of the lake, to Hurricane Gate Structure (HGS)-4 at Belle Glade (Exhibit B). It includes levee sections L-D9 and part of L-D2.

4. PROJECT DESCRIPTION.

The selected plan (Exhibit C) consists of excavating 4 foot of peat from along the landside toe between the toe of the embankment and the top of the existing drainage ditch. Backfilling this area with 5 foot of graded granular filter drainage blanket. A 2-foot thick drainage blanket will extend up on the embankment to elevation 23 feet. This blanket will intercept through seepage. In order to prevent uplift and to intercept underseepage a 3-foot wide relief trench, fill with a graded granular filter stone will be installed in the bottom of the existing ditch. The relief trench will extend to the top of the lower limestone layer, about 25 feet. A 48 inch perforated culvert wrap with non-woven filter fabric will be installed at the same elevation as the existing ditch and covered with a grader granular filter blanket to carry off excess seepage water. The relief trench/wells will control uplift pressures and prevent heaving at the toe of the embankment. The vertical relief drainage system will reduce artesian pressures in the foundation and will intercept and provide controlled outlet for seepage which otherwise would emerge uncontrolled landward of the embankment. The 48-inch culvert will remove the seepage collected in the berm. Stop log riser culverts will be installed at various locations to drain the 48-inch culvert. Drop inlets will be installed on 400-foot centers to drain surface runoff into the 48-inch culvert.

5. GOVERNMENT-OWNED LAND.

The existing project was constructed on lands of which the Federal Government acquired sufficient interests (perpetual right-of-way easements) to support the construction and the operation and maintenance. In general terms, the Federal Governments' right-of-way extends approximately 150 feet from the levee centerline (or about 35 feet landward of the toe).

6. NON-FEDERALLY-OWNED LAND.

The State of Florida and South Florida Water Management District (Local Sponsor for a number of Civil Works Project) hold ownership to significant lands adjoining the existing right-of-way for the Herbert Hoover Dike Project.

7. REAL ESTATE REQUIREMENTS.

The recommended plan (Exhibit C) requires a project right-of-way running parallel with the centerline of the dike and extending 100-foot landward of the toe of the dike. Based on a review of the survey map, an additional 50 to 60 feet of right-of-way, landward of the existing right-of-way will be required to support construction of the recommended plan.

There are a number of public roads providing access to the dike within the proposed area of construction, which will be sufficient to support project construction.

Within the existing right-of-way for operation and maintenance of the dike, there are sufficient lands to allow for staging and storage of vehicles, equipment and personnel to support construction of the recommended plan.

Refer to Paragraph 9 of this appendix for acreage and values.

8. OPERATION/MAINTENANCE RESPONSIBILITIES.

The general division of State and Federal responsibilities for the operation and maintenance of C&SF Project works was established in House Document 643 (80th Congress, 2nd Session) dated 6 May 1948. This document states that operation and maintenance of the levees, channels, locks, and control

works of the St. Lucie Canal, Lake Okeechobee, Caloosahatchee River, and the main spillways designed for control of water levels in conservation areas, is a Federal responsibility. The U.S. Army Corps of Engineers is responsible for operation and maintenance of the levee system around Lake Okeechobee. At inflow points, USACE is, in some cases, responsible for operation and maintenance of project works related to that channel from its point of inflow upstream to the first control structure. This includes nineteen culverts which convey flows beneath the levee at various locations. All project works associated with the three regulatory outlets are also Federal responsibilities. The local sponsor operates and maintains a number of pumps and lock structures located along the levee system.

9. APPRAISAL INFORMATION.

A gross appraisal report for Reach I of the Herbert Hoover Dike Major Rehabilitation Report was completed on 25 February 2000. The report identified various land classifications comprising the lands to be impacted by the proposed project: residential lands, residential improvements, commercial/government lands, commercial/government improvements, agricultural lands, agricultural improvements and severance damages.

The following value is herewith submitted for lands and damages:

Reach I: $ 2,760,500.00

The value is based on the acquisition of approximately 200 acres of land consisting of approximately 250 tracts. The valued estate is the flood protection levee easement.

10. ATTITUDE OF OWNERS.

The project will impact an array of owners and businesses, both private and public, which are landward of the dike. The purpose of the project is to maintain the integrity of the dike and as such, protect any and all individuals and businesses located landward of the dike. It is safely

assumed that any and all individuals and businesses will support the purpose of the project.

11. MINERALS.

There exist no known minerals of value in the project area.

12. HAZARDOUS AND TOXIC WASTES (HTW).

There have not been any hazardous and toxic wastes identified within the project area.

13. RELOCATIONS ASSISTANCE (PUBLIC LAW 91-646).

The gross appraisal identified five single-family residential homes within the proposed project boundaries that will be acquired. The value of said residential homes are included in the total cost for lands and damages. Relocation cost (approx. $150,000.00) associated with Public Law 91-646 has been

included in the acquisition/administrative cost item of the summary of project real estate cost.

14. RELOCATIONS, ALTERATIONS, VACATIONS AND ABANDONMENTS

(Utilities, Structures and Facilities, Cemeteries, and Towns).
There is a portion of a state highway impacted by the project, for which said portion of highway will have to be shifted (relocated). The gross appraisal has included the cost of lands necessary for construction of the relocated highway.

The gross appraisal does not reflect the estimated $3,000,000.00 cost for highway construction cost, as such is not a real estate cost.

A municipal golf course is impacted by the project and the gross appraisal has taken such into consideration and used the cost to cure method for estimating lands and damages.

15. INDUCED FLOODING

There will be no induced flooding directly associated with this project.

16. STANDING TIMBER AND VEGETATIVE COVER.

There exist no timber or unusual vegetative cover in the project area.

17. RECREATION RESOURCES.

There are no separable recreational lands identified for the project.

18. CULTURAL RESOURCES.

There are no known cultural resources that have been identified as being affected by the project.

19. OUTSTANDING RIGHTS.

There are no known outstanding rights in the project area.

20. ACQUISITION/ADMINISTRATIVE COSTS.

Acquisition/Administrative costs:

Federal	$ 200,000.00
Non-Federal (Sponsor)	2,000,000.00
Total	$ 2,200,000.00

21. SUMMARY OF PROJECT REAL ESTATE COSTS.

The following cost figures are subject to change prior to construction:

Lands and Damages	$ 2,760,500.00
Acquisition/Administrative Cost	2,200,000.00

Contingency – 25%	1,240,125.00
Total Estimated Real Estate Cost	$ 6,200,625.00

22. REAL ESTATE ACQUISITION SCHEDULE.

The schedule for acquisition of real estate should allow 18 to 24 months from the time survey work is initiated to identify the individual tracts scheduled for acquisition.

23. ESTATES TO BE ACQUIRED.

Flood Protection Levee Easement

A perpetual and assignable right and easement in (the land described in Scheduled A) (Tracts Nos. ___, ___ and ___) to construct, maintain, repair, operate, patrol and replace a flood protection levee, including all appurtenances thereto; reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may by used without interfering with or abridging the rights and easement hereby acquired: subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

Drainage Ditch Easement

A perpetual and assignable easement and right-of-way in, over and across (the land described in Schedule A) (Tracts Nos. ___, ___ and ___) to construct, maintain, repair, operate, patrol and replace a drainage ditch, reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may be used without interfering with or abridging the rights and easement hereby acquired; subject however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

24. CHART OF ACCOUNTS FOR PROJECT

01	Lands & Damages	$ 2,760,500
01B-	Acquisitions	$ 2,200,000

TOTAL REAL ESTATE COST EXCLUDING

CONTINGENCY $ 4,960,500

REAL ESTATE CONTINGENCY (25%) COST $ 1,240,125

TOTAL PROJECT REAL ESTATE COST $ 6,200,625

Exhibit A

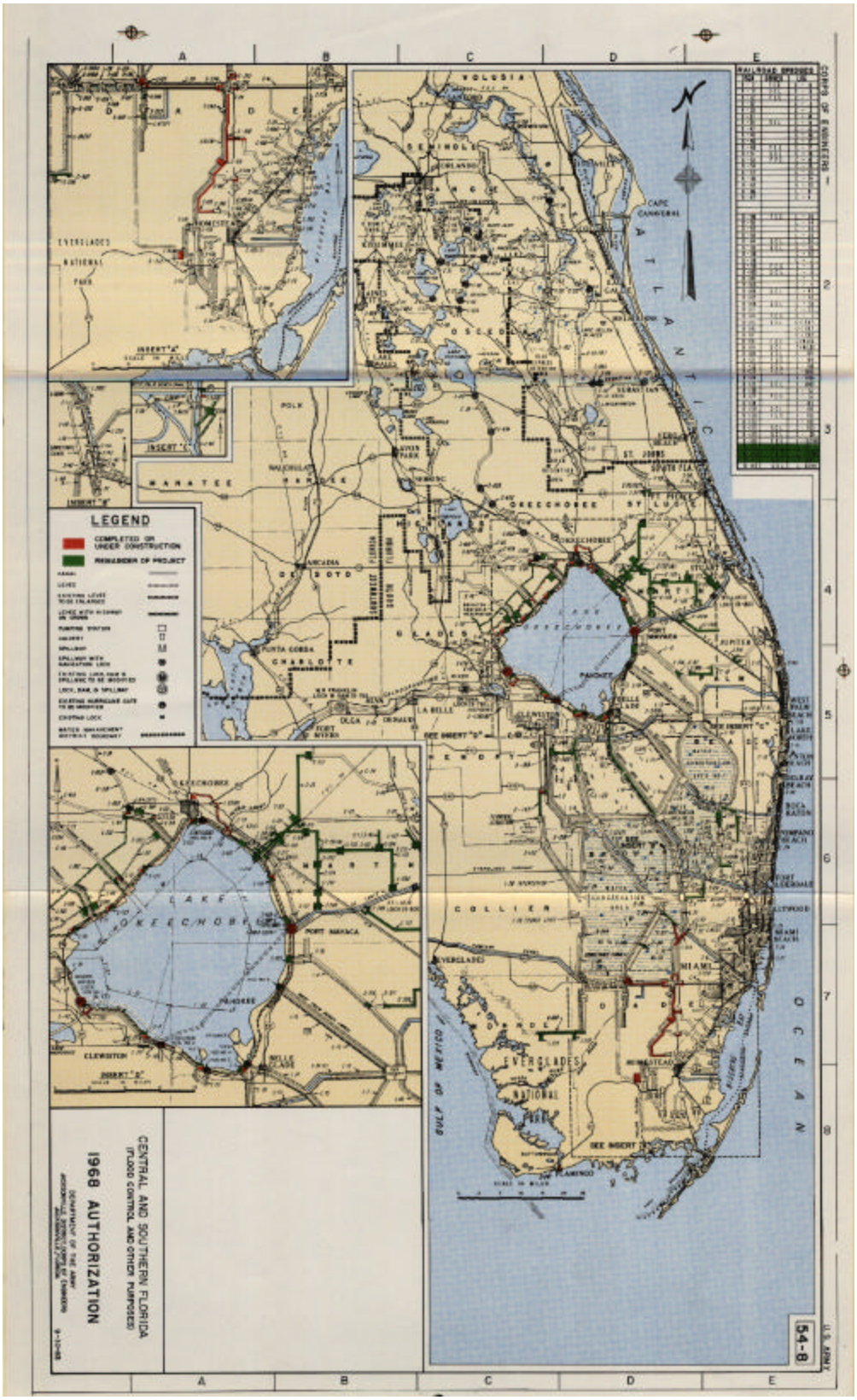

LEGEND
COMPLETED OR UNDER CONSTRUCTION
REMAINDER OF PROJECT
CENTRAL AND SOUTHERN FLORIDA
(FLOOD CONTROL AND OTHER PURPOSES)
1968 AUTHORIZATION
54-8
ATLANTIC
OCEAN
GULF OF MEXICO
LAKE OKEECHOBEE
EVERGLADES NATIONAL PARK
MIAMI
FORT MYERS
PORT MANATEE
CAPE CANAVERAL
INSERT "A"
INSERT "B"
INSERT "C"
INSERT "D"
SEE INSERT "D"
SEE INSERT

Exhibit B

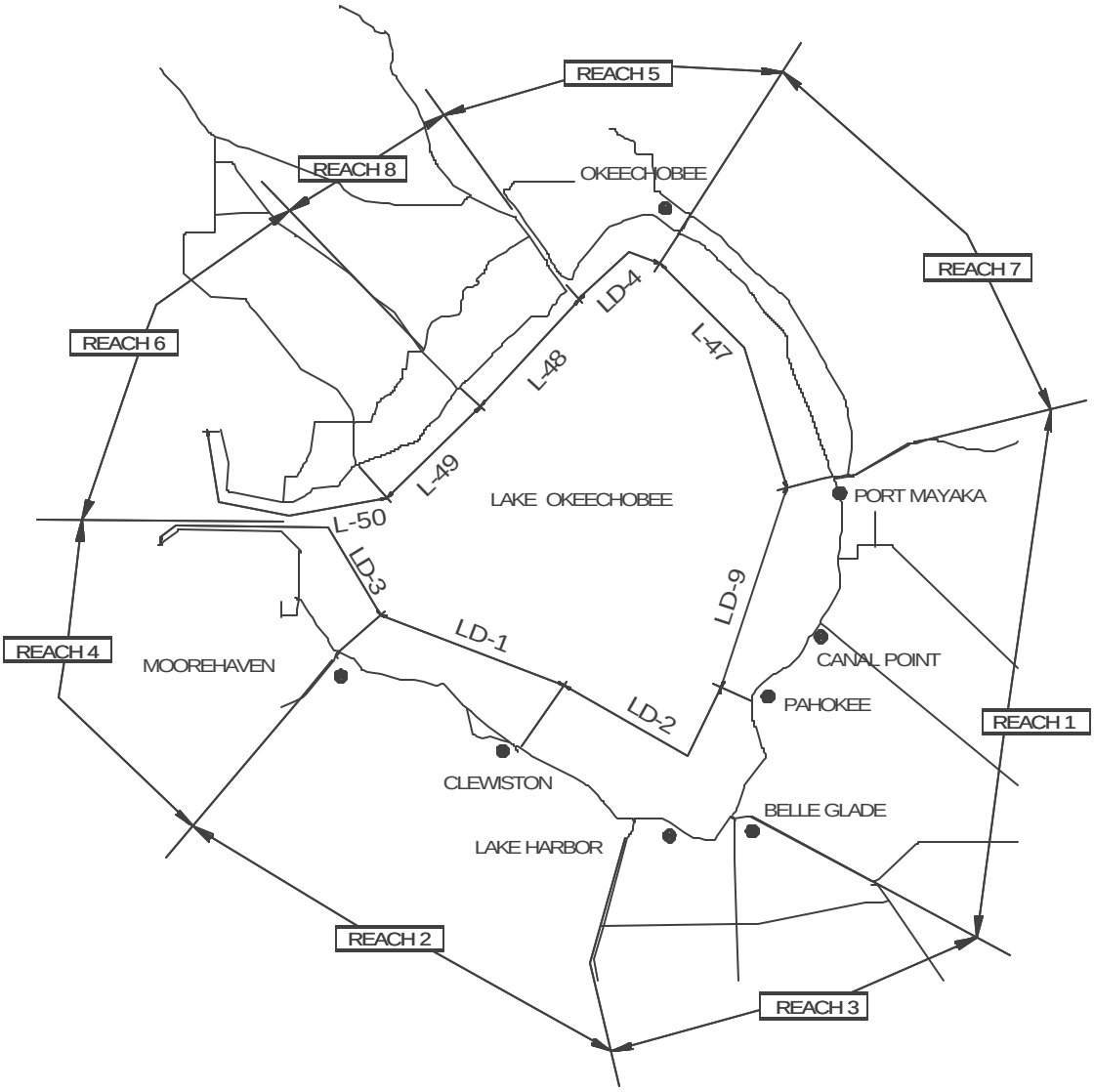
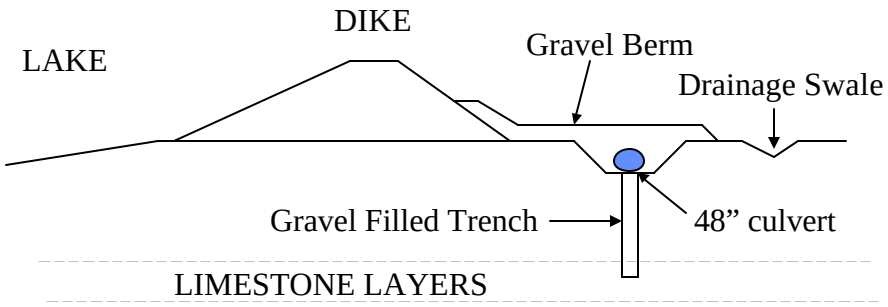

REACH 5
REACH 8
OKEECHOBEE
REACH 7
REACH 6
LD-4
L-47
L-48
L-49
LAKE OKEECHOBEE
PORT MAYAKA
L-50
LD-3
LD-9
LD-1
REACH 4
MOOREHAVEN
CANAL POINT
PAHOKEE
LD-2
REACH 1
CLEWISTON
BELLE GLADE
LAKE HARBOR
REACH 2
REACH 3

Exhibit C

ALTERNATIVE B
TYPICAL SECTION



NOT TO SCALE