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CENTRAL AND SOUTHERN
FLORIDA PROJECT

FOR FLOOD CONTROL
AND OTHER PURPOSES

PART I, AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54
GENERAL DESIGN MEMORANDUM
APPENDICES

MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

US Army Corps
of Engineers
Jacksonville District

CENTRAL AND SOUTHERN FLORIDA PROJECT FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM

MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

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CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX A

SUMMARY OF COORDINATION

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
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APPENDIX A

COORDINATION

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APPENDIX A

SUMMARY OF COORDINATION

1. Coordination Process. During the preparation of the General Plan, and prior to its approval in January 1985, the Corps of Engineers was involved in intense coordination with a wide range of agencies, groups, and individuals. At that time, Florida Governor Bob Graham had assembled the Everglades National Park/East Everglades Resource Planning and Management Committee. He charged the committee with assessing a myriad of land and water quality issues and to develop a plan for implementation of recommended solutions. The committee's implementation plan was adopted as State policy in April 1985. This coordination process was very helpful in identifying, not only environmental problems in the area, but also potential issues related to agriculture, residential development, municipal and industrial water supply, institutional conflicts and limitations, agency positions, etc.

One of the committee's recommendations was to create the Southern Everglades Technical Committee (SETEC) to provide a forum for the exchange of information. The Corps is represented on this committee and has provided several briefings on ongoing studies. The Corps is also represented on the West Dade Wellfield Technical Committee and the exchange of information on this issue is critical to both developing a water delivery system that will work and siting the wellfield in an acceptable location.

Additionally, the Corps has conducted two series of coordination meetings. In each series, meetings were held with SFWMD, ENP and other environmental groups and agencies, Dade County, agricultural interests, residential interests, and the Miccosukee Tribe of Indians. Prior to all meetings, information was mailed to all attendees that was then explained in detail at the meetings. Comments and questions were requested after the meeting.

At the first round of coordination meetings, descriptions of the original three alternative plans were presented along with predicted hydrologic impacts. Comments received subsequent to these meetings led to the abandonment of the Flow-Through alternative and the more detailed investigation of optimizing to the Rain-Driven alternative. The revised set of various versions of the Rain-Driven alternative and predicted hydrologic impacts were then presented at the final series of meetings.

The last round of coordination involved the Draft GDM and included a public meetings and workshops with public and private interests. The public meeting took place in Miami on 22 October 1990. The workshops were conducted in September and October 1990 with residents of the East Everglades residential area, and recreational boaters that had expressed concern about the proposed S-349 structures blocking the L-67A borrow canal. As result of these meetings, a bypass channel was included with the structures to allow boaters accessing the canal throughout its entire length. The structures and bypass

channels were opposed by some boaters but supported by others. Several other alternatives were analyzed and found unfeasible due to either high cost or safety reasons. Results of this coordination round can be found in the coordination section of the Environmental Impact Statement. Copy of letters received during the first round of coordination are included in this appendix.

2. Views of Study Participants. Comments received as a result of the two series of coordination meetings are paraphrased below.

a. Initial Coordination Meetings.

(1) Everglades National Park (letter dated 20 March 1987).

* Current knowledge of the relationship between hydrology and biology is insufficient to definitely identify optimum hydrologic conditions. However, adverse biological impacts have been documented.

* The following objectives were offered:

Mitigate unseasonal flood releases through S-12;

Restore flows across the full width of Shark River Slough.

Allow timing and volume of water deliveries to fluctuate in response to rainfall and antecedent conditions;

Maintain downstream Shark Slough water levels at pre-project levels; and

Maintain water quality.

* The GDM will provide the opportunity to accomplish many of these objectives.

* Immediate relief from the impacts of flood releases to the Park will be experienced as a result of the GDM plans.

* The operational flexibility afforded by the plan should be utilized in conjunction with long-term ecological research to "fine tune" the water delivery system.

* After reviewing the South Florida Water Management Model calibration results, the Park stated the model is more appropriate for estimating relative differences between alternatives than for determining precise results.

* The following comments on how the alternative plans would meet the above stated objectives were provided:

The Flow-Through and Rain-Driven alternatives would both mitigate unseasonal S-12 flood releases;

Both alternatives would restore water deliveries across the full width of Shark River Slough thereby stimulating marsh productivity, favoring deposition of native peat soils, reducing fire frequency, and slowing the spread of exotic plant species;

ENP stated that the Rain-Driven alternative was better at more closely matching historical response to rainfall;

The Park expressed concern that neither alternative improved the match between historic and modeled stages at P-33; and

The most useful step toward preventing further water quality degradation is maximization of sheet flow.

* Because of the need for additional data to precisely define optimum conditions, the Park does not support recommending a final delivery schedule in the GDM.

* Further testing of the Rain-Driven concept is needed with the proposed structural changes in place.

* Sheetflow from WCA No. 3B should be the primary source of water delivery to NESRS rather than S-333.

* Land acquisition or flood protection (that would not adversely impact water quality in ENP) should be provided for the residential area.

* Land acquisition or a means of removal of any rise in water levels caused by the introduction of flow to NESRS would be supported.

* Potential impacts of the proposed West Dade Wellfield should be evaluated for sites east and west of L-31N.

(2) U.S. Fish and Wildlife Service (letter dated 23 March 1987). The Fish and Wildlife Service (FWS) focused on the L-31N seepage levee and flood control plans that were being considered for residential and agricultural development in the East Everglades, as alternatives to land acquisition. It was noted that construction of the WCA No. 3B and L-31N seepage levees would destroy about 200 acres of high quality wetlands in addition to creating other indirect impacts such as increased susceptibility to invasion by *Melaleuca* (and other exotic plants), isolation of wetlands between the levees, increased access to the area, etc. Also, the levee alignment for flood protection of the agricultural area would pass through critical habitat for the Cape Sable Sparrow. Modified flood protection levee alignments were proposed for consideration. FWS suggested that a pump be installed to return seepage water from the L-31N borrow canal back into the Slough, as a substitute for the seepage levee.

(3) U.S. Environmental Protection Agency (letter dated 5 August 1987).

* EPA supports the concept of improving flows across the breadth of Shark River Slough.

* The Flow-Through and Rain-Driven Plans would both improve water flows to the Park.

* Because of upstream water management actions, the Flow-Through Plan could cause undesirable unseasonal dry season discharges.

* EPA recommends implementation of the Rain-Driven Plan.

* Flood protection plans should be designed to minimize impacts on wetland areas.

* An alternative levee alignment was suggested for the residential and agricultural areas.

* EPA could support their levee alignment for the residential and agricultural areas through the NEPA process.

* EPA supported FWS's recommendation for replacement of the seepage levees in WCA No. 3B and the L-31N seepage levee with a pump station.

* Dade County should choose a site for the proposed West Dade Wellfield in conjunction with this GDM.

(4) South Florida Water Management District (letter dated 10 February 1987).

* The Flow-Through alternative would eliminate flexibility of the ENP/South Dade County Conveyance System.

* The Flow-Through alternative also does not result in flow through WCA No. 3B which is a major failure of the concept.

* The Rain-Driven alternative would accomplish flow through WCA No. 3B.

* SFWMD could support residential area flood protection but not agricultural flood protection.

* If necessary, culverts in L-31N could be provided for flood control of the agricultural area.

* Construction of several small structures or degraded levee sections were suggested for consideration as an alternative to large structures.

* Control structures in the L-67A borrow canal are not supported.

* Detailed studies of the need for seepage levees is required.

* Modeling of the final alternative should include a proposed West Dade Wellfield in a site east of L-31N.

(5) Florida Department of Environmental Regulation (letter dated 23 December 1986).

* DER supports efforts to improve water deliveries to ENP.

* The Flow-Through alternative would not be the most desirable plan because it would not provide natural timing of water deliveries.

* Flood protection should not be provided to the agricultural area.

* Flood protection for the residential area should be provided for the minimum area.

* The proposed West Dade Wellfield should be modeled with the recommended plan.

* The flood control levee alignments should exclude the wellfield site.

* The effects of increased development around the wellfield site should be factored into the economic analysis.

* Water quality impacts of pumping flood runoff from developed areas into the slough should be evaluated.

* Alternative plans considered should investigate other disposal methods for damaging quantities of water and other water sources for coastal areas.

* DER supports ENP's request for continued long-term testing of water deliveries.

(6) Ghioto, Singhofen, and Associates, Incorporated (letter dated 10 February 1987. This is a water resources and civil engineering firm that is providing consulting services to south Dade County agricultural interests. Comments on the C-111 and the Modified Water Deliveries to ENP GDM's were provided for consideration in the same letter.

* Concern was expressed that there was a lack of single flood event analysis for determination of impacts to the agricultural community.

* There was also concern that increased flows from the northern section of L-31N borrow canal as a result of modified water deliveries would increase flood risks in the agricultural area adjacent to C-111.

* Opposition was expressed to the inclusion of the IAP in the Base Condition (No Action Plan).

* Higher water levels indicated in the model results for the alternative plans negatively impact groundwater storage in adjacent agricultural lands.

* It was recommended that the Corps fully evaluate all data collected during the test period.

* Any alternative that did not include agricultural flood protection would be opposed.

* An alternative levee alignment was proposed for agricultural flood protection.

(7) Southern Everglades Technical Committee (letter dated 11 February 1987.

* SETEC supports the concept of improving water deliveries to ENP by structural and operational changes.

* Continued testing should be conducted to determine the appropriate water delivery plan.

* The Rain-Driven alternative will provide better water storage and resource management than the Flow-Through alternative.

* Structural modifications inherent in both alternatives could provide the following benefits: additional delivery options for continued iterative testing; reduction in flooding in the area south of S-12; and sheet flow through Northeast Shark River Slough would improve hydroperiods, reduce dry season fires, and improve water quality.

* SETEC suggested modeling the Rain-Driven alternative with plugs in the L-67A borrow canal.

* Modeling should be performed using target water levels at gage P-33 compared with actual (or computed water levels) to adjust water deliveries.

* Current data, with the IAP and without the West Dade Wellfield, should be simulated in the model for additional verification.

* Extreme flood and drought conditions should be modeled.

* The impact of the IAP on the Base Condition should be evaluated.

b. Second Coordination Meetings. Letters received in response to the second series of coordination meetings are paraphrased below:

(1) Everglades National Park (letter dated April 1, 1988). Water Management objectives stated in these Park's 20 March 1988 letter were reconfirmed.

* The proposed structural modifications should further improve the water delivery system and provide the flexibility to allow additional operational changes as ongoing research may suggest. There is the need for continued experimentation.

* The Park would recommend flood protection for the residential area if it can be accomplished with minimal impacts on adjacent wetlands and if it would not add constraint on water deliveries to NESRS. Flood protection for agriculture is not warranted.

* Flows should be provided uniformly across Shark River Slough rather than concentrated through the S-12's.

* The potential impacts of the proposed west Dade Wellfield should be evaluated.

* The Federal Government through the Secretary of Interior and the State of Florida have committees for acquisition of private lands of NESRS in the East Everglades. The GDM should proceed under the assumption that these lands will be in public ownership.

* If flood protection is not provided (and the developed land is acquired, then the Rain-Driven alternative with S-349 and S-356 is the most appropriate plan. L-67 Ext. should be leveled by pushing the material into the borrow canal.

* Water deliveries to NESRS should be made primarily from WCA No. 3B.

* If a residential area flood protection system is recommended, the Park would prefer a levee/canal system only around the immediate residental areas such as that proposed by the U.S. Fish and Wildlife Service. A pump station, S-356, pumping excess water over the protected area to L-31N borrow canal would be preferred.

* The tentative testing process has contributed to our understanding of the timing of water deliveries and the relationship to upstream rainfall and antecedent condition.

c. Local Sponsor Support. On 23 February 1990, the South Florida Water Management District endorsed the project. They stated:

* "At this point we still endorse the original Rain-Driven Plan. Since the structural changes between that the the MRD Plan do not differ, we do not object to the GDM being completed and submitted for review subject to the condition that Sec. 7 coordination be reinitiated. There are also some design details that need further refinement during development of the Features Design Memorandum prior to the preparation of

plans and specifications. This will allow formulation of a delivery schedule and final design that are acceptable to the broad spectrum of Federal, State, and local agencies and the public. We look forward to participating with the Federal government in a program that is truly in the best interest of all south Florida."

IN REPLY REFER TO:

United States Department of the Interior

NATIONAL PARK SERVICE

EVERGLADES NATIONAL PARK
AND
FORT JEFFERSON NATIONAL MONUMENT
P.O. BOX 279
HOMESTEAD, FLORIDA 33030

MAR 20 1987

Colonel Charles T. Myers, III
Department of the Army
Corps of Engineers
District Engineer
P.O. Box 2970
Jacksonville, Florida 32232-0019

Dear Colonel Myers:

We have taken the opportunity to review your agency's progress on the General Design Memorandum (GDM) for Modified Water Deliveries to Everglades National Park (ENP), and found it to be an important step toward a successful planning document. We greatly appreciate the time and effort put forth by your staff in developing these analyses, and hope we can now take steps to complete the GDM as soon as possible.

As stated in the cover letter accompanying the preliminary hydrologic analyses, the purposes of the final document are to identify an improved water delivery system to ENP as well as to address associated East Everglades issues such as flood protection/acquisition alternatives for developed areas and flowage rights in undeveloped areas. In a separate letter dated November 7, 1986, you also noted that an evaluation of water demands of the area would be included in the final report along with a possible recommendation for a new water delivery schedule for Shark Slough. In this response we will focus on the water delivery system analyses presented so far, but we also will present our views on completion of the GDM process and the future of water delivery to the park.

Shark Slough Water Management Objectives

Our knowledge of the relationships between hydrology and biology in the southern Everglades is insufficient to create a definitive blueprint for restoration of historic biological conditions in the park; however, sufficient information is available to document adverse biological impacts in Shark Slough since completion of L-29 and the S-12 structures in 1962 and the L-67 levee and borrow canal in 1967. Over the last several years, Everglades National Park has articulated these adverse effects on park animal and plant communities (please see letter from Superintendent Morehead to Col. Devereaux dated September 10, 1982, and the June 26, 1984, memorandum from ENP staff to Superintendent Morehead regarding "Staff Recommendations on Improved Water Management for Everglades National Park"). Examples discussed have included a dramatic drop in wood stork nesting success, flooding of established alligator nests, and dispersals of wading bird feeding concentrations.

Evidence suggests that even though wildlife resources, particularly wading bird populations, were already in decline before 1962, the decline was accelerated by construction of the present water management system. Our strategy for reversing the observed declines in park resources has, therefore, been two-tiered. First, since we have a basic understanding of adverse impacts caused by operation of the existing water management system, we have developed the following objectives for improving water delivery:

1) Mitigate high water releases to Shark Slough through the S-12 structures, especially during normal dry season recession periods.

2) Restore flow distribution across the entire width of Shark Slough (roughly L-30 to 40-Mile Bend). This would not only contribute significantly to goal 1, but would also help restore hydroperiods in the eastern portion of the historic Shark Slough basin.

3) Allow volume and timing of deliveries to fluctuate as they did historically in response to rainfall and antecedent conditions in the southern Everglades.

4) Maintain downstream Shark Slough water levels at pre-project levels.

5) Maintain water quality so that resource degradation does not occur.

We have made specific recommendations to the Corps for accomplishing most of these objectives (see aforementioned letters and the letter of March 1983 from Superintendent Morehead to Colonel Devereaux outlining the "7-point plan"). These have included new delivery schedules (rainfall-runoff, Bridge 105), operational procedures (flow-through, re-distribution to northeast Shark Slough via S-333) and structural modifications (removal of L-67 (ext.) levee and borrow canal, control of conveyance in L-67A canal, and facilities to re-establish sheetflow between Conservation Areas 3A and 3B and between Area 3B and northeast Shark Slough). The present GDM will provide the opportunity to accomplish many of these objectives.

As a second tier of this approach, the National Park Service, Florida Audubon Society, South Florida Water Management District, and your agency have embarked on various research projects to investigate relationships between hydrologic and biological resources in the park. Ecological research, however, is quite complex, and synthesis of results into restoration recommendations takes time. It is our belief that with the changes in the water management system requested, we will experience immediate relief from the impacts of flood releases in the park and drainage of northeast Shark Slough occurring since the 1960's. But, we also will have built the operational flexibility into the system to fine-tune water deliveries to the park as results of ongoing research suggest means of further enhancing or "restoring" Shark Slough biological conditions.

Model Calibration

The park must judge the Corps of Engineers' modeling results by the goals stated above to determine if the desired results can been achieved by the structural modifications and operational procedures modeled; however, this judgement must be made within the context of the predictive capability of the model.

Calibration of a regional model may be difficult in remote areas like Everglades National Park due to a shortage of physical hydrology and geology data and knowledge of hydrologic processes over the entire basin. Calibration results for stations P-33 and P-36 presented in South Florida Water Management District (SFWMD) Technical Publication 84-3 show very close concurrence between model output and actual field data during much of the 1969-1975 calibration/verification period. However, the model loses accuracy at the higher and lower extremes. For example, prediction of drydowns was inconsistent, with near perfect results in some years and large underpredictions (P-33) or overpredictions (P-36) in other years. The model also tends to overpredict upper extremes, especially at P-36. These limitations are particularly important in that high and low extremes can have some of the most damaging effects on park biota. For these reasons we view the model results as being best for determining placement of proposed structures and the ability, in broad terms, to divert water from the present system to areas cut off from the Everglades flow-way in 1962. Regarding downstream effects in ENP, model results are more appropriate for estimating relative differences between alternatives than for determining precise results.

Hydrologic and Biological Evaluation

Results of your staff's preliminary analysis provide very encouraging evidence that most of the requested improvements may be achieved via the alternatives tested.

Goal 1: Mitigate high water releases to Shark Slough through the S-12 structures, especially during normal dry season recession periods.

Comparisons of results for the alternatives modeled to actual 1969-1982 conditions suggest that both the flow-through and rain-based options are effective in reducing flood releases to Shark Slough. Clearly, either option will tend to strongly mitigate the adverse biological effects resulting from unseasonal regulatory flood releases into the park, including flooding of alligator nests and dispersal of wading birds.

Goal 2: Restore flow distribution across the entire width of Shark Slough (roughly L-30 to 40-Mile Bend). This would not only contribute significantly to goal 1, but would also help restore hydroperiods in the eastern portion of the historic Shark Slough basin.

Restoration of flow distribution to northeast Shark Slough would not only serve as an outlet for flood waters released from Conservation Area 3A (goal 1) but would also mitigate the significant hydroperiod reduction and

resulting ecological impacts in northeast Shark Slough created by construction of L-29 and L-67 (ext.). Modeling results showed that the flow-through and rainfall-based options are both capable of improving flow distributions across Shark Slough, though under the assumptions used, the rainfall-based option produced a more desirable distribution.

Northeast Shark Slough hydroperiods were significantly increased by both alternatives. This would stimulate marsh productivity by enhancing invertebrate and fish standing stocks, thus increasing wading bird food availability. Longer hydroperiods would also favor deposition of native peat soils rather than the marl soils characteristic of shorter hydroperiod marshes. Fire frequency in the area would be reduced, thereby decreasing the chances of severe, peat-consuming fires and possible resulting changes in native vegetation communities. Reduced fire frequency in the area will also slow the spread of Melaleuca quinquenervia, an exotic tree capable of dramatically altering native Everglades plant communities.

Goal 3: Allow volume and timing of deliveries to fluctuate as they did historically in response to rainfall and antecedent conditions in the southern Everglades.

At this time ENP believes that the rainfall-based alternative is better at adjusting volume and timing of deliveries to more closely match historical response to rainfall. The flow-through method of water delivery also achieves a better response to rainfall than the base condition, but to a lesser extent than the rainfall-based option.

Biological benefits are essentially those listed under goal 1.

Goal 4: Maintain downstream Shark Slough water levels at pre-project levels.

None of the options modeled appear to maintain downstream stages at pre-project levels. (We do not know the extent to which errors inherent in the model played a role in these results). Wet season stages at P-33 predicted by the models appear to be beneficial; however, late spring water levels indicate extremely dry conditions similar to those of the early 1960's when the S-12 gates were closed for nearly two years. These conditions are not acceptable in that the severe drydowns predicted under both alternatives would have devastating effects on marsh soils, fish and wildlife resources, and vegetation communities dependent upon wetter conditions. Examples of deleterious effects include increased risk of major peat-consuming fires and loss of dry season refugia vital to the marsh ecosystem.

Goal 5: Maintain water quality so that resource degradation does not occur.

Data collected by the Corps of Engineers and the South Florida Water Management District indicate violations of phosphate and other standards for quality of water delivered through the S-12 structures during 1984, 1985, and 1986. Nutrient dosing studies recently conducted by the park in

Shark Slough indicate that increasing phosphorus concentrations may result in significant changes in marsh algal and macrophyte communities. In those studies, phosphorus dosing at levels near ENP standards resulted in dramatic increases in algal biomass, loss of the characteristic periphyton mat, and a shift toward a more pollutant tolerant algal species composition. Similar changes have been documented in Water Conservation Areas 1 and 2A in response to nutrient loading.

Once everything feasible is done to control nutrient, herbicide, pesticide, and fungicide runoff at or near their sources, the most useful step toward preventing further water quality problems in Shark Slough is maximization of sheetflow in the system to allow filtration by the marsh. We assume that the redistribution of flow accomplished by the modeled alternatives would provide water quality mitigation.

Summary and Conclusions

The park has asked for relief from large flood events that are caused by management of the upstream water management system. Under the modeling assumptions used, the rainfall-based alternative did a somewhat better job of mitigating effects of flood releases than the flow-through alternative.

A disturbing feature of both alternatives, however, is the extreme drought conditions in central Shark Slough in most years. "Maintaining historic Shark Slough water levels" is a goal of the park, yet we have been unable to define exactly what that means due to a lack of data from before Everglades drainage projects began. Ultimately we will choose desirable downstream hydrographs or other means of establishing water conditions needed for restoration of biological resources, and research toward that end continues. We, therefore, do not support including a final recommendation for a new water delivery schedule in this document.

ENP prefers development of the rainfall-based concept for determining optimum volume and timing of water deliveries to Shark Slough. We feel that a rainfall-runoff relationship developed from pre-1962 data can ascertain what discharge volumes and timing were before construction of L-29 and associated structures. Water now available in the system above that amount must be diverted to avoid flood damages or must be used to supplement the park as needed, should the low water conditions predicted by the model in central Shark Slough become a reality. Further development of pre-1962 rainfall-runoff relationships and determination of the need for supplemental deliveries or diversions of excess water will hopefully be achieved through ongoing research concurrent with the iterative testing process.

Structural modifications included in the rainfall-based alternative hold much promise for providing the requested mitigation/restoration as noted above. Further experimentation with this alternative should include the addition of structures in the L-67A canal and facilities for conveyance from Area 3A to Area 3B. These facilities should be distributed along the length of the levee as modeled for the flow-through alternative. We also recommend detailed studies of the system's capability to move 55-60% of Shark River Slough deliveries to northeast Shark Slough under extreme high

water conditions. Sheetflow from Conservation Area 3B should be the primary source of water delivery here rather than structure S-333. These steps will help to assure the requested distribution and water quality mitigation.

The park recognizes the importance of resolving the East Everglades flood protection issue in order to implement improved water deliveries to ENP. Protection of water quality and maintenance of historic water levels along the park's eastern border are vital issues. From that standpoint, either land acquisition or flood protection facilities that do not adversely impact park water quality or quantity must be provided for the 8.5 square mile residential area. Lacking the latter assurances and benefit/cost data, we are withholding a final opinion. Regarding the agricultural lands south of the residential area, we feel that it is appropriate to consider both land acquisition and the means for removal of any rise in the area's water table that can be attributed to the introduction of flow into northeast Shark Slough.

Proposals for new wellfields in western and southern Dade county are of great concern. We have already noted that downstream stages in the park under the base condition and the alternatives are unacceptable. A 150-200 MGD wellfield west of L-31N will drain ENP wetlands even further and negate the positive effects of enhanced northeast Shark Slough water levels. All scenarios should be run with and without the proposed wellfield in place, but, more importantly the Corps should examine wellfield sites east of L-31N. The intent of the GDM is nullified if sufficient water is not available in the system to compensate for wellfield withdrawals.

Everglades National Park strongly endorses proceeding as quickly as possible with the requested engineering re-arrangements. This would allow us to meet most of our stated goals immediately, and would establish sufficient operational flexibility in the system to allow further fine-tuning of deliveries as new information on biological requirements emerges. The park also strongly endorses continuation of the iterative testing process as the vehicle for further improving water delivery to Shark Slough. Once the structural changes are in place, water delivery experiments unencumbered by the constraints imposed on previous tests should lead to the earliest possible restoration of the park's ecological resources.

Again, we view the progress on the GDM thus far as an important step in assuring improved water delivery to the park while meeting needs of other users. We look forward to working with the Corps of Engineers to incorporate suggested improvements in modeled alternatives and finalize recommendations for structural changes.

Sincerely,

M.V. Finley

Michael V. Finley
Superintendent

United States Department of the Interior

FISH AND WILDLIFE SERVICE

P.O. BOX 2676

VERO BEACH, FLORIDA 32961-2676

March 23, 1987

Colonel Charles T. Myers, III
District Engineer
U.S. Army Corps of Engineers
Post Office Box 4970
Jacksonville, Florida 32232

Dear Colonel Myers:

We have reviewed the flood protection alternatives proposed to protect the "8-1/2 square mile residential area" and additional agricultural lands in the East Everglades, part of the General Design Memorandum (GDM) for Modified Water Delivery to Everglades National Park. We inspected the proposed L-31N seepage levee alignment, which would extend from Water Conservation Area 3B southward to the East Everglades residential area, the proposed levee corridor around the residential area, and that proposed for the adjoining agricultural area. The examination was based upon the diagrams supplied (copies appended), color infrared aerial photographs, and an onsite inspection February 4, 1987. This planning-aid letter is a result of these initial efforts on planning for these structual features common to both water delivery alternatives.

The route of the proposed seepage levee was examined from the western levee of L-31N, viewing soil, water and vegetation up to 1,000 feet to the west. Individual features of interest were inspected more closely (primarily for the purpose of groundtruthing the aerial photographs), and conditions were described at each one-mile increment along this levee. We did not make such a continuous survey of the proposed levee alignment around the residential and agricultural areas, but rather examined such conditions where the levee alignment intersected existing roadways in this area.

The general conditions noted were: 1) presence of several inches of surface water in Area 3B and within the first several miles south of Tamiami Trail. There was no surface water at this time in areas surrounding the residential or agricultural areas. 2) The proposed levee system would cover wetlands predominantly vegetated by sawgrass and spikerush to the north, and sawgrass, muhly grass, and other perennial wetland plants further to the south. 3) Killifishes, apple snails, pigfrogs, and a number of wading birds, including wood storks, were present in the flooded areas near the northern portion of the proposed work during our inspection. We noted crayfish tunnels, and tracks indicating that these areas also host mammals such as marsh rabbit, otter, bobcat, and raccoon. This fauna also would be expected to utilize the higher elevations in the southern part of the proposed levee, under typical wet season conditions.

We understand that the levee would have a bottom width of approximately 70 feet. Material for its construction could come from offsite, but we anticipate that the nearest source of fill, the western levee of L-31, still would be required to isolate the L-31 canal. Therefore, we assume that a borrow pit would be dredged along the proposed levee and would probably be at least 50 feet wide.

Under the above scenario, the seepage levee would fill 127 acres, and the borrow canal would eliminate another 91 acres of wetlands.

In addition to these direct impacts, we expect that the toe of the levee would be high enough to encourage the invasion of Brazilian pepper, Melaleuca, or other exotic plants, and that the wetlands remaining between the seepage levee and the existing L-31 levee would be isolated from the rest of the Everglades system, precluding ingress and egress of aquatic organisms. Another effect would be the disturbance of wildlife and their habitat by persons gaining increased access to this area via the new levee. During our inspection, we were impressed by a number of all-terrain vehicles along and next to the existing L-31 levee.

The levee around the residential area would involve approximately 6.8 miles of work in Everglades wetland, which would cover about 98 acres. The proposed agricultural levee would extend an additional 5.2 miles, destroying another 76 acres. More significantly, existing wetlands internal to the levee system ultimately would be converted to uplands. Thus, within the residential area, approximately 1,660 acres of good quality wetlands (minimally disturbed at present) would be subject to conversion, while over 2,900 acres of such wetlands would be reclaimed or "protected" by the agricultural levee area.

The proposed levee around the agricultural area also would cross through designated critical habitat of the Cape Sable Sparrow, a Federally listed endangered species. Over half of the land within that levee is critical habitat. On the other hand, the proposal for residential protection avoids critical habitat completely.

Our interpretation of the model outputs you supplied is that the degree of threat posed from increased water levels under the proposed delivery schedules would be extremely minor, especially for the agricultural area. The fields in question currently receive some degree of protection from the elevated 237th Avenue and the adjoining canal. Moreover, approximately half of the area within the proposed agricultural levee is currently natural wet prairie, which would only be damaged by "flood protection" measures. On the other hand, we can understand the need to provide year-round protection for the most heavily developed residential area, but believe that this can be done in a manner more compatible with environmental protection and probably at a reduced cost. We have attached diagrams showing the highest quality wetland within the proposed levee alignments (both agricultural and residential), and also show a proposed realignment of this levee to exclude these wetlands. Although the alignment we are proposing has more corners to it, it actually is shorter (approximately 9.1 miles) than the alignment shown in your diagram (9.8 miles) by virtue of excluding these wetland areas.

We understand that you still are considering the construction of detention areas for treatment of storm waters pumped out of the residential area. Instead of constructing these within the slough, we recommend that you consider using one of the wetland areas between your proposed levee alignment and the one that we propose. We assume that construction techniques are available to greatly reduce permeability of the internal portion of the levee. Also, the wetlands within such a detention area could retain their ecological characteristics, or even be enhanced, if the outside levee is equipped with a bleed-down notch or weir structure.

With regard to the proposed seepage levee, we are concerned about the intrusion of such a major feature into northeast Shark River Slough, solely for the occasional need to control inflow to L-31. We question the need for the seepage levee, in light of the anticipated construction of a new wellfield in this area by Dade County, which likely will reduce water levels in L-31. Also, we suggest as a substitute measure that you consider installing a pump to simply return seepage water from L-31 over the existing levee, back into the slough, and thus avoid this major construction effort.

In summary, we are concerned that the protection measures proposed for the East Everglades under the GDM for Water Delivery to Everglades National Park would significantly affect high quality wetlands within Water Conservation Area 3B and in northeast Shark River Slough. The agricultural protection levee also would intrude into critical habitat for the Cape Sable sparrow. We suggest alternative measures, backpumping from L-31 to overcome seepage, and relocation of the flood protection levee in the residential area, which might fully mitigate for any such effects and should be less costly. We appreciate the opportunity to comment on this project and would be most pleased to discuss these or other measures.

Sincerely yours,

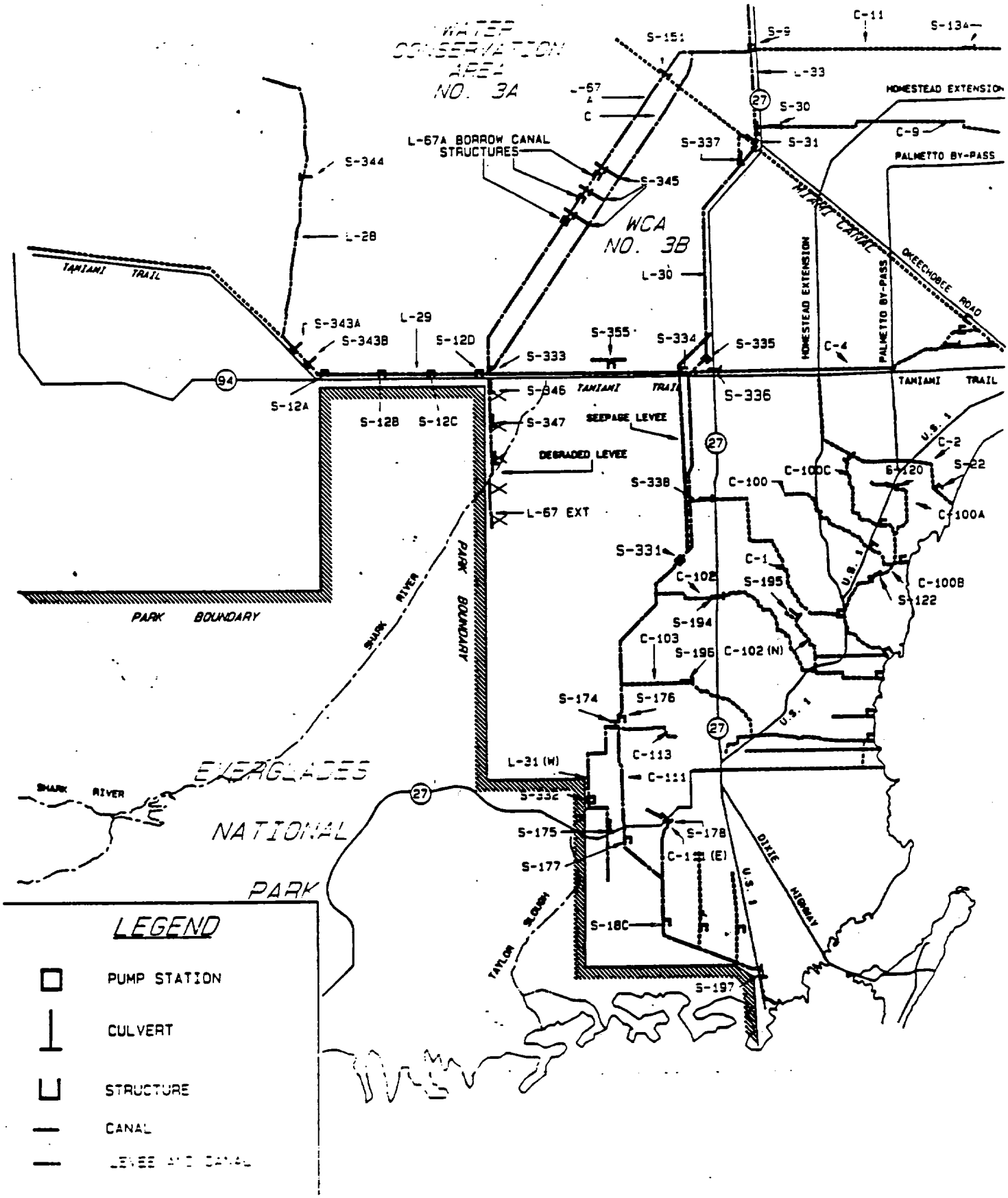
Joseph D. Carroll
Field Supervisor

Enclosures
Proposed Levees (2 figs.)
Internal Wetlands
Alternative Levee Alignment

cc:
EPA, Atlanta, GA.
FG&FWFC, Vero Beach, FL
FG&FWFC, Tallahassee, FL
ENP, Homestead, FL
SE, Jacksonville, FL
SE, Vero Beach, FL

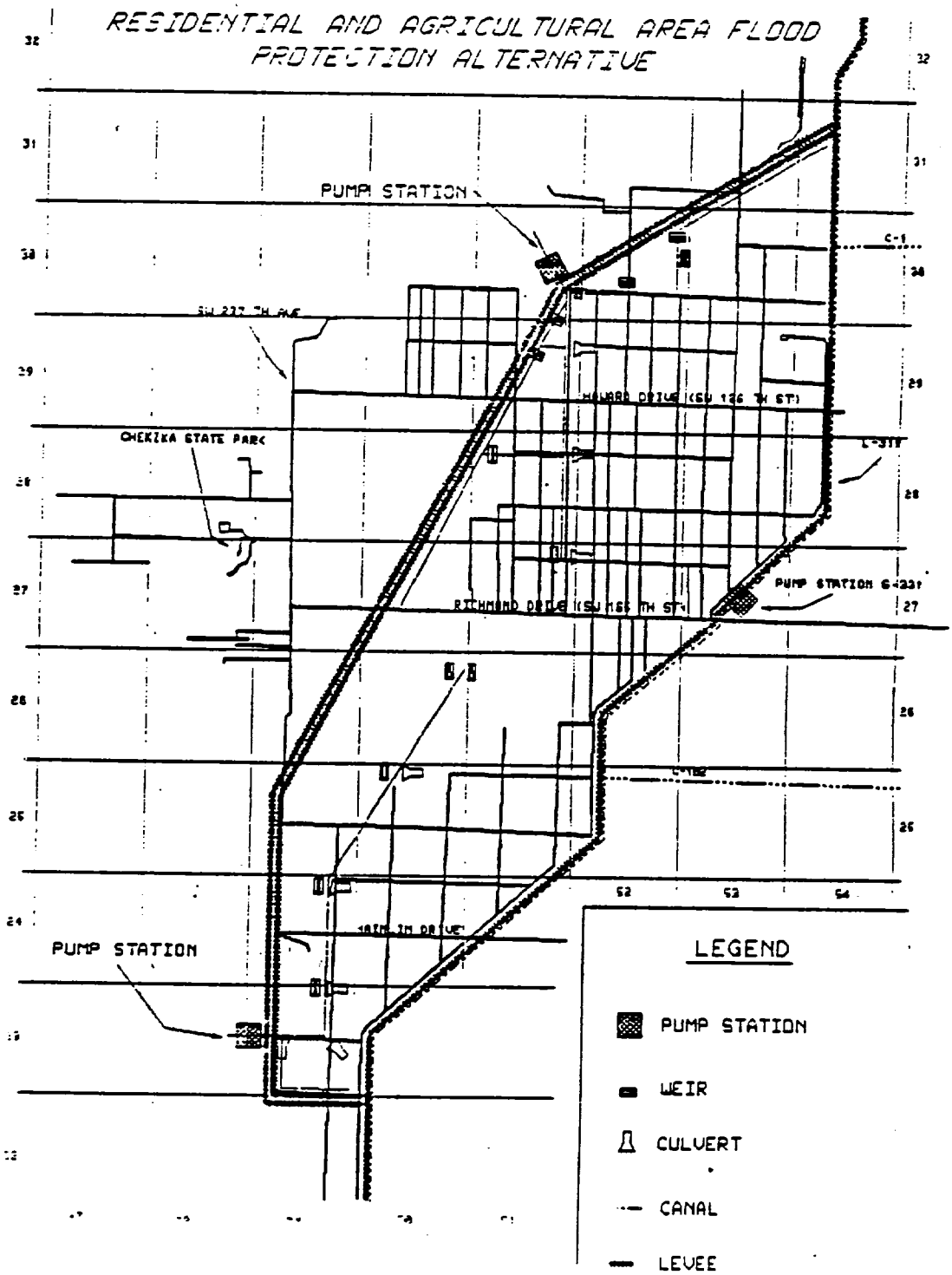
GDM - MODIFIED WATER DELIVERIES TO E.P.

FLOW-THRU WATER DELIVERIES ALTERNATIVE



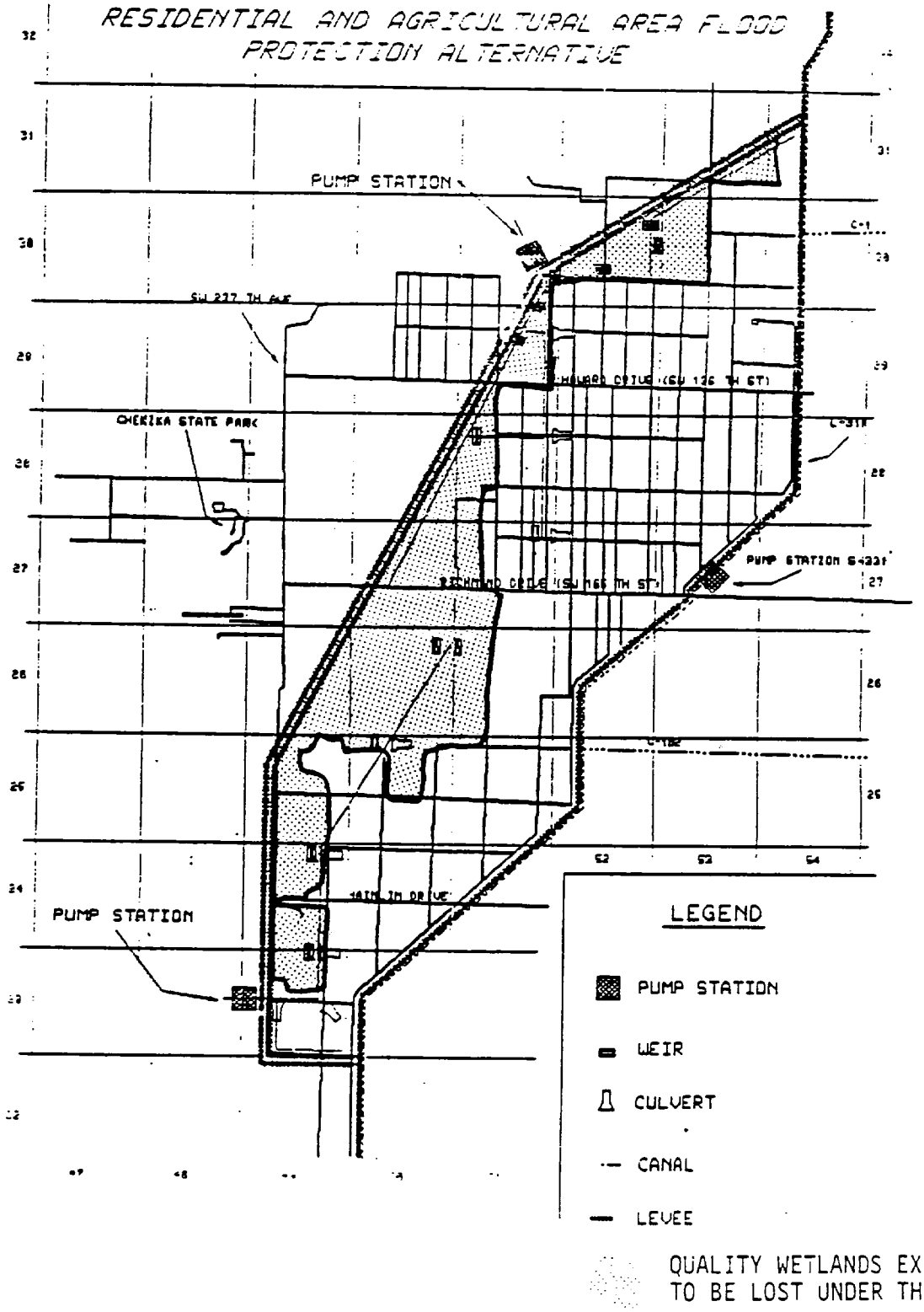
DEVELOPED EAST EVERGLADES
GDM - MODIFIED WATER DELIVERIES TO ENP

RESIDENTIAL AND AGRICULTURAL AREA FLOOD PROTECTION ALTERNATIVE



DEVELOPED EAST EVERGLADES
GDM - MODIFIED WATER DELIVERIES TO ENP

RESIDENTIAL AND AGRICULTURAL AREA FLOOD PROTECTION ALTERNATIVE

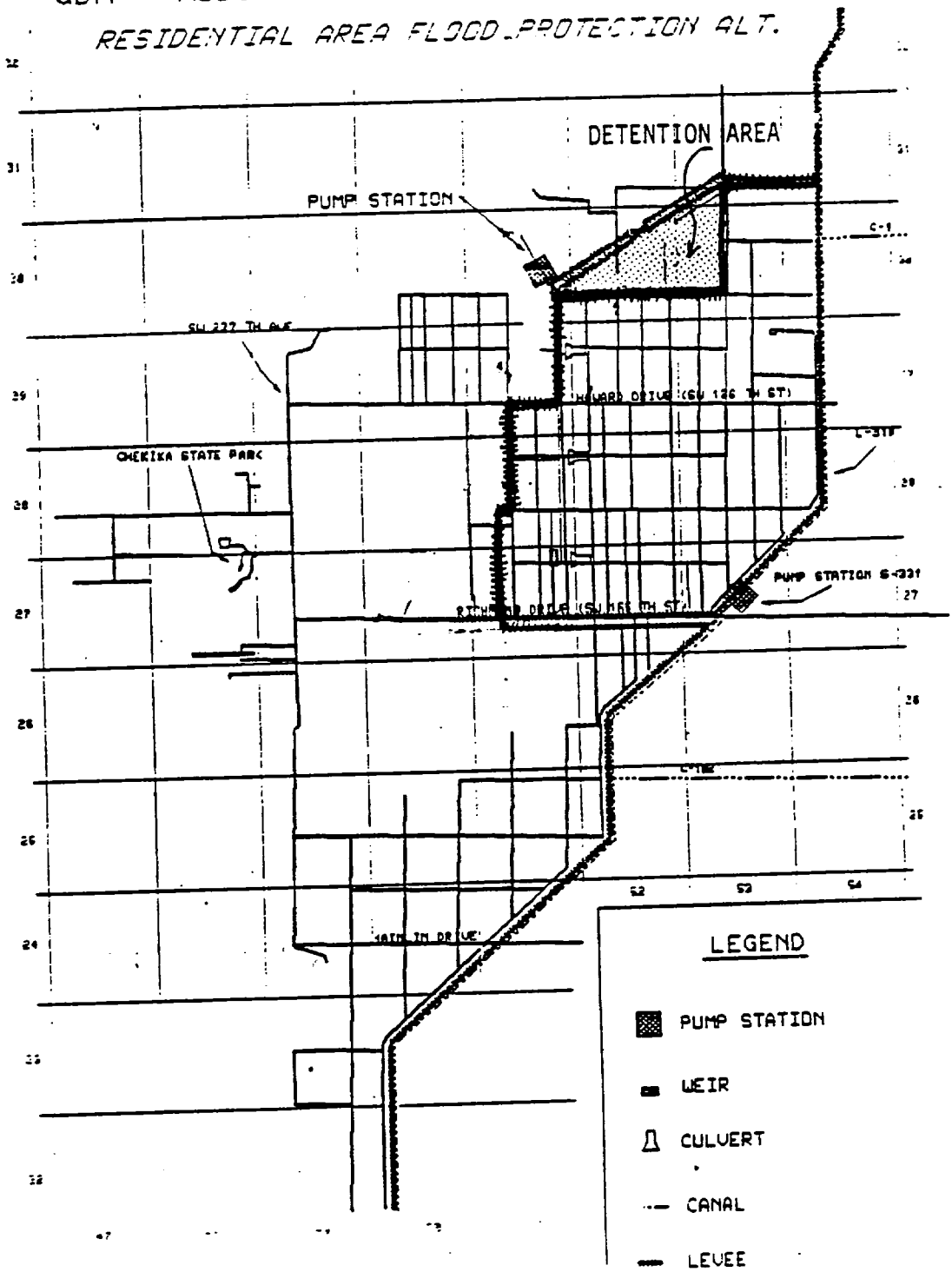


SUGGESTED LEVEE REALIGNMENT AND DETENTION AREA

DEVELOPED EAST EVERGLADES

GDM - MODIFIED WATER DELIVERIES TO ENP

RESIDENTIAL AREA FLOOD PROTECTION ALT.



UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

REGION IV

345 COURTLAND STREET
ATLANTA, GEORGIA 30365

AUG 5 1987

4WMD-WEB/EHH

Colonel Robert L. Herndon
District Engineer
U.S. Army Corps of Engineers, Jacksonville
P.O. Box 4970
Jacksonville, Florida 32232

ATTENTION: John Moulding, Lewis Horning

Dear Colonel Herndon:

On November 24 and 25, 1986, an Environmental Protection Agency (EPA) representative met with members of your Planning Division staff and other state and federal environmental resource agencies at Everglades National Park (ENP) to discuss the General Design Memorandum (GDM) for Modified Water Delivery to ENP. We appreciate the efforts that your staff took to meet with us and provide us with the opportunity to comment on the evolving preferred plan for the GDM for Modified Water Delivery to ENP.

EPA fully supports the concept of improving hydrological flows across the breadth of the Shark River Slough to the ENP as well as restoring the historical seasonal flows to the Park. We have discussed the two proposed water delivery plans, the flow-through alternative and the rain-driven alternative, with ENP officials. Both of these water delivery plans represent significant improvements over the existing water delivery operation which does not allow hydrological sheet flow across the eastern half of the Shark River Slough. The flow-through plan would allow unregulated discharges from the upgradient Water Conservation Areas (WCA) 3A and 3B to flow to ENP. However, due to periodic water pumping activities in upstream agricultural areas, there are concerns that excessive water could be delivered to the Park during the normal winter dry season, resulting in serious ecological impacts. The rain-driven plan would regulate water deliveries to ENP based on normal seasonal water levels in the upgradient WCA's 3A and 3B, incorporating a local rainfall component into the water delivery schedule. We agree with the ENP's recommendation for implementation of the rain-driven alternative.

Of particular interest to EPA Region IV are the preliminary flood protection levee alternative alignments as well as the proposed L-31N seepage levee alignment that were presented at the November 1986 meeting. The first flood protection alternative levee is proposed to protect the 8.5 square mile East Everglades residential area. The second proposal would extend further southward and would decrease flooding of an extensive area of natural everglades in areas of potential agricultural expansion. For purposes of discussion, we will refer to this second alternative as the "agricultural levee plan." We understand that your Planning Division has determined that implementation of a modified water delivery plan could raise seasonal ground water levels in the East Everglades residential and agricultural areas as much as 0.2 to 0.3 feet.

As you are aware, my Wetlands Section staff has worked closely with your Regulatory Division to protect valuable wetlands in the East Everglades from unwise agricultural and residential development. EPA supports some form of flood protection for the 8.5 square mile residential area. However, any flood protection plan should be designed to minimize impacts to natural areas, particularly wetlands. The current 6.8 mile long levee alignment would eliminate approximately 98 acres of wetlands and "levee in" approximately 1,660 acres of high quality wetlands. The agricultural levee alignment would result in filling an additional 76 acres of wetlands and cut off an additional 2,900 acres of high quality wetlands from normal sheetflow.

Wetland impacts associated with the proposed residential levee alignment could be greatly reduced if some relatively minor alignment shifts were implemented. The currently proposed residential levee alignment would, in fact, "levee in" areas of recent Section 404 interest. For example, the 160-acre wetland tract recently proposed for rockplowing in a Section 404 permit application by Mr. Feghali (85IPS-20317) and denied by the Jacksonville District would be hydrologically cutoff from the contiguous Everglades system. We are enclosing a modified residential levee alignment plan which we feel would greatly reduce wetland impacts. EPA could support this levee alignment through the National Environmental Policy Act (NEPA) process.

Recently, EPA has initiated a 404(c) action on the Henry Rem Estates, Harold Becker, et al. and Senior Corporation tracts. The Henry Rem Estates and the adjacent Harold Becker, et al. tracts are in the center of the area of wetlands which would be "leveed in" by the proposed agricultural levee plan. EPA is strongly opposed to the proposed agricultural levee plan due to its severe direct and secondary impacts on wetlands that EPA has determined are valuable and in need of protection. As shown in the attached EPA preferred levee alignment, we could support a levee alignment in the agricultural areas which would provide existing farm fields with flood protection while avoiding additional wetland losses.

The last major point of EPA's concern with the current GDM structural plans pertains to the L-31 seepage levee alignment. Under the current proposal, a levee with an adjacent borrow canal would eliminate approximately 125 acres of prime Everglades. Further, the project, as proposed, could result in significant secondary impacts on adjacent wetlands such as reduction of habitat availability, introduction of exotic tree species and alteration of hydroperiods. EPA fully supports the U.S. Fish and Wildlife Service recommendation for a substitute measure such as an L-31N backpumping station. The pump could return seepage flows to the Shark River Slough without the damage to valuable wetland areas associated with additional levee construction.

As you are aware, Dade County is planning to select a location for its West Dade public drinking water wellfield soon. We feel that any levee alignment planning efforts should include consideration of the proposed West Dade public drinking water wellfield location alternatives and associated impacts. EPA believes that Dade County should choose the location of its West Dade Wellfield in conjunction with the development of the Water Delivery Plan.

In our involvement with the Dade County Wellfield site selection process, we will encourage selection of a site that not only can provide present needs for high quality drinking water, but will also work toward assuring long-term protection of the aquifer and the cone-of-influence. Our concern is that water from any site selected may become degraded unless appropriate development controls are implemented. My staff is coordinating closely with Dade County officials through their alternative site selection process as is your staff. We anticipate that the West Dade Wellfield site selection process will continue through 1987 and 1988.

Thank you for providing us with the opportunity to comment on this matter.

Sincerely yours,

Jack E. Ravan
Regional Administrator

Enclosure

cc: See enclosed

cc: Mr. Dale Twatchmann, Secretary
Florida Department of Environmental Regulation
Twin Towers Office Building
2600 Blair Stone Road
Tallahassee, Florida 32301
Attn: Pam McVety, Randy Armstrong

Mr. Joe Carroll, Field Supervisor
U.S. Fish and Wildlife Service
P.O. Box 2676
Vero Beach, Florida 32906

Mr. James Pulliam, Jr.
Regional Director
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75 Spring Street, S.W.
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Attn: John Baumeister

Mr. Robert Usherson
Metro Dade County Planning Dept.
111 N.W. 1st. Street
Miami, Florida 33128-1972

Mr. Paul Quimbo
Dept. of Community Affairs
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Tallahassee, Florida 32301

Ms. Wendy Lovett
Dept. of Community Affairs
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Tallahassee, Florida 32301
Attn: Ann Deister

Mr. Mike Finley, Supertindent
Everglades National Park
P.O. Box 279
Homestead, Florida 33030

Mr. Tony Clemente
DERM
111 N.W. 1st Street
Suite 1310
Miami, Florida 33128
Attn: George Molnar

Mr. John Wodraska
South Florida Water Mgmt. District
Post Office Box V
3301 Gun Club Road
West Palm Beach, Florida 33406
Attn: Tilford Creel, Tom McVickers

Mr. Karsten Rist
5530 Sunset Drive
Miami, Florida 33143
Attn: Sam Poole

South Florida Water Management District

John R. Wodraska, Executive Director
Tilford C. Creel, Deputy Executive Director

Post Office Box V 3301 Gun Club Road
West Palm Beach, Florida 33402
Telephone (305) 686-8800
Florida WATS Line 1-800-432-2045

IN REPLY REFER TO:

February 10, 1987

Colonel Charles T. Myers III
District Engineer
Jacksonville District
Corps of Engineers
P. O. Box 4970
400 West Bay St.
Jacksonville, Fl. 32232

Dear Colonel Myers:

Subject: Review of East Everglades/ENP GDM, Preliminary Results

We have reviewed the preliminary results of the hydrologic analysis for this G.D.M. and, in addition, have been represented at each of the coordination meetings held over the last few months. We would like to make the following comments on the material that has been submitted to date.

1. Operational Plan.
   The WCA-3A/Shark River Slough (SRS) interface must be controlled through active operation of the outlet structures from WCA-3A and 3B. The flow-through model results show serious storage losses in the south end of WCA-3A that would eliminate the flexibility of the South Dade Conveyance System. The Miccosukee Tribe would surely object because of the loss of the use of their airboat trails for several months almost every year. Although the flow-through plan puts large volumes into WCA-3B, there is virtually no flow through the system due to the high level in L-29 caused by leaving S-333 open. This must be viewed as a major failure of the concept since sheetflow to NESRS through WCA-3B is an important objective of the SRS restoration effort. The rain-driven plan appears to accomplish this objective.

2. Flood Protection.
   The problems of the developed property west of L-31N must be resolved, either through public acquisition or flood protection, before significant progress can be made on improving flow to SRS. Since no hydrologic simulations of the proposed flood protection systems are yet

William E. Sadowski, Chairman - Miami
John F. Flanigan, Vice Chairman - North Palm Beach
Stanley W. Hole, Naples
J. Neil Gallagher, St. Cloud
Nathaniel P. Reed, Hobe Sound
Kathleen Shea Abrams, Miami Shores
Timer E. Powers, Indiantown
Nancy H. Roen, Plantation
Oscar M. [illegible], Ft. Myers

available some general comments are offered. As you know, our Governing Board and the ENP/EE 380 Committee have endorsed flood protection for the 8.5 square mile residential area. For environmental reasons the area protected should be as close to the 8.5 square miles as practical. We, therefore could support the residential area alternative, but do not consider the plan that protects both the residential area and the agricultural land south of Richmond Drive a viable one. Although we have not seen model results for the flood protection systems, we do not feel that the extensive levee and pump system south of Richmond Drive is appropriate. If necessary, that area should be protected with project culverts into the L-31N canal similar to those authorized in 1965, with the necessary improvements downstream to prevent a reduction in the level of flood protection for the southern reaches of L-31N and C-111.

3. Proposed Structural Changes to the Project.

Solving the residential flooding question will open the door to major restoration of the flow to Northeast Shark River Slough (NESRS). The removal of the L-67 extension is probably the single most significant change in terms of influence on the hydroperiod of the slough. It should definitely be a part of this project, and should be given high priority for implementation.

Restoring the hydrologic function of WCA-3B is also an important objective. The Model results indicate that, with the rainfall-based management scheme, the proposed structures, S-345 and S-355, accomplish this. The specific design of these structures should be compatible with the objective of restoring the slough function. Numerous small structures through L-67A and L-29 would be preferable to a few large structures requiring collection and distribution canals. This may or may not be practical, but should at least be given serious consideration in the design process. In addition, degrading L-67C and L-29 should also be considered before control structures are specified.

The proposed seepage control levees will have to be analyzed very carefully. The rain-driven plan shows very little change in the stage-frequency curve for the 3-29 gage. This would make that levee hard to justify if the rain-driven plan is recommended.

The justification for the proposed seepage levee west of L-31N must be documented using detailed model data which includes the residential flood protection system and the operational guidelines for S-331 that will be used after the flood protection system is in. The fact that higher stages can be held in L-31N and that the residential area will have its own protection system may reduce the necessity for the additional levee. Control structures in the L-67A canal are not warranted if a controlled delivery system is used for flow to the Park. Since we do not endorse any type of flow-through system, we recommend against these structures.

Our preference would be for the GDM/DDM process to result in authorization for immediate construction of the residential flood protection system, S-345 and S-355, and the removal of L-67 extension. Authority for the seepage control levees would also be given but only if field experience or detailed hydrologic analysis, with the other changes in place, demonstrated a genuine need.

4. Water Supply.
The final model results should include wellfield pumpage rates and locations compatible with those projected by the Miami Dade Water and Sewer Authority. The proposed West Dade wellfield should be included in the detailed modeling of the East Everglades. Since it is likely that this wellfield will affect flow in NESRS it would be advisable to include a simulation with the wellfield located east of L-31N. A likely location would be south of the Krome Detention Center between L-31N and U.S. 27.

The Water Management District views this G.D.M. as the first step in a process which could ultimately correct the most serious structural problems associated with the Everglades National Park/C&SF Project interface. We see the potential for major environmental benefits, not only for the Park, but for Northeast Shark River Slough and WCA-3B. At the same time we will be preserving the critical water supply function of the Conservation Areas. Our first priority is to authorize and construct these changes. We expect to continue with a testing program to field test improved, controlled, delivery formulas and do not feel that it is possible or even desirable to lock in a specific formula.

I would like to compliment your staff on the work they have done, especially considering the condensed time frame they have set. The efforts of Mr. Lewis Hornung deserve special note. He has taken on a complicated modeling process along with a very difficult coordination program and is doing an excellent job. The early involvement of all interested parties has been well received and will be instrumental in assuring the success of the project.

Please continue to call on us if we can assist you in any way with the continuation of this critical project.

Sincerely,

JRWodraska

John R. Wodraska
Executive Director

JRW:tmd

cc: Mike Finley, Supt. ENP
Anthony Clemente, DERM
Garrett Sloan, MDWSA
Reginald Walters, Dade County Planning
James Murley, DCA
Jill Hoag, East Everglades
James Humble, Fla. Lime & Avocado Growers

STATE OF FLORIDA

DEPARTMENT OF ENVIRONMENTAL REGULATION

TWIN TOWERS OFFICE BUILDING
2600 BLAIR STONE ROAD
TALLAHASSEE, FLORIDA 32301-8241

BOB GRAHAM
GOVERNOR

VICTORIA J. TSCHINKEL
SECRETARY

December 23, 1986

Colonel Charles T. Myers, III
District Engineer
United States Army Corps of Engineers
Post Office Box 4970
Jacksonville, Florida 32232-0019

Dear Charlie:

The department has reviewed the alternative plans for the restoration of sheetflow through Northeast Shark River Slough, and the proposed flood protection designs for residential and agricultural areas of the East Everglades. Consistent with the State of Florida's Save Our Everglades program, the department supports the efforts to improve the timing and amount of water flow into Everglades National Park and adjacent parts of the Everglades system. These are our comments and concerns:

1. Any water delivery plan for the Everglades National Park and adjacent areas should be designed to duplicate as closely as possible the natural timing and quantity of water flows. It appears logical that rainfall should be a major determining factor in such a plan. Since some areas are drained with groundwater levels controlled at an unnaturally low level, the water elevation in Water Conservation Area 3A is not by itself, a good indication of natural conditions. Consequently, a simple flow through system would not seem to be the most desirable.

2. Flood protection should <u>not</u> be provided for the East Everglades agricultural areas south of Richmond Drive. Provision of any flood protection in the East Everglades was a controversial and much debated issue, but the Everglades National Park - East Everglades Resource Planning and Management Committee finally recommended flood protection of the 8.5 square mile residential area so that restoration of the Northeast Shark River Slough

could proceed. Your second flood protection design encompasses an area well over twice the size of the 8.5 square mile residential area, including significant portions of the federally designated Cape Sable sparrow critical habitat and valuable wetland areas. Dade County designated this area for seasonal vegetable production which would not require construction of drainage facilities. Cultivation of crops which violate the designated land use should not cause the designation to be altered. Continued consideration of flood protection for this area will only encourage the planting of unsuitable crops and increase development pressure. This flood protection plan clearly goes far beyond the intent of the committee, and in fact, works counter to the overall objective of restoring the Everglades to more natural conditions--by separating viable, functioning wetland habitat from the Everglades system and opening up previously protected lands to development.

3. The flood protection alternative for the area north of and including Richmond Drive should be limited to protection for 8.5 square miles. The proposed design flood proofs over 12.5 square miles, much more area than was intended for flood protection by the East Everglades 380 committee. The design will destroy valuable wetlands both within and outside of the canal-levee system.

4. Acquisition of the residential area north of Richmond Drive should be given full and equal consideration. Other flood proofing alternatives such as elevating the individual roads and existing homes and septic tanks should also be evaluated.

5. The flood protection designs do not adequately consider the proposed East Everglades wellfield. The Florida Power and Light Co. right-of-way which has been selected as the site for a new regional wellfield in Dade County is within the area designed to receive flood protection and is traversed midway by a proposed internal collector canal. The canal will clearly be within the cone of depression of the new wells. Further, flood protection of the area surrounding the proposed wellfield will promote its development, increasing the threat to the Biscayne aquifer. We request that:

a. the East Everglades wellfield be incorporated into your 1990 model for the Northeast Shark River Slough sheetflow restoration. Although the wellfield may not be operational by that year, it will be in use by the early 1990s and the hydrological effect of its use should therefore be projected.

b. consideration be given to rerouting the flood protection levees and collector canals to exclude the proposed wellfield.

c. the effects of increased development around a proposed regional wellfield be included in your cost/benefit analysis of the flood protection and the land acquisition alternatives.

6. Water quality effects of the pump station(s) located along the western levees of your flood protection designs have not been evaluated. Water pumped from the residential and agricultural areas will adversely affect the waters of Northeast Shark River Slough and Everglades National Park, an Outstanding Florida Water.

7. The water delivery plans should include alternative disposal methods for damaging quantities of excess water as well as alternative water sources for the coastal areas such as reverse osmosis and aquifer recharge.

The ultimate objective of all proposed Everglades water delivery plans should be to maximize the biological and ecological health of the Everglades system. Studies designed to determine the needs of the biological community have not been completed. An understanding of the ecological needs of the system may require years of study. No action should be taken now which would preclude or hamper necessary future modifications in water delivery. Accordingly, we add our support to the Park's request to continue the testing process over a long period of time to further determine which hydrological modifications will most benefit the ecology of the Everglades.

We look forward to your response to these concerns.

Sincerely,

Vicki

Victoria J. Tschinkel
Secretary

VJT/ps
cc: Kathleen Abrams

GHIOTO, SINGHOFEN & ASSOCIATES, INC.
WATER RESOURCES AND CIVIL ENGINEERING

RODNEY D. GHIOTO, P.E.
PRESIDENT

PETER J. SINGHOFEN, P.E.
VICE PRESIDENT

February 10, 1987

Colonel Charles T. Myers III, P.E.
U.S. Army Corps of Engineers
Jacksonville District
400 West Bay Street
P.O. Box 4970
Jacksonville, FL 32232-0019

Subject: Comments from the South Dade Agriculture Trustees on the NESRS and C-111 Project GDM's

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Dear Colonel Myers:

Enclosed are three copies each of the subject comments for consideration by the Corps. It is the strong belief of the Trustees that these two projects are intimately related. Therefore, we request that the comments on both be routed to personnel working on both projects.

We are pleased with the level of consideration that the Corps has given to these projects and sincerely hope that our comments are taken in a constructive manner. Should you have any questions or comments on these materials, please feel free to contact James Humble or myself.

Very truly yours,
GHIOTO, SINGHOFEN & ASSOCIATES, INC.

Rodney D. Ghioto

Rodney D. Ghioto, P.E.
President

RDG/lg
Enc.
cc: James Humble

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COMMENTS ON PRELIMINARY CORPS PROPOSALS

FROM

SOUTH DADE AGRICULTURE TRUSTEES

GDM - C-111 PROJECT

Prepared By

Ghioto, Singhofen and Associates, Inc.
Water Resources and Civil Engineering

January, 1987

This document provides comments on the proposed alternatives for the GDM - C-111 presented by the U. S. Army Corps of Engineers, dated October 31, 1986. These comments are divided into areas of concern to the Florida Lime and Avocado Committees and are presented in the following sections.

INTERFACE WITH L-31N AND THE WATER DELIVERIES GDM

In a separate document, the Committees have provided comments regarding the proposed Water Deliveries alternatives. Those comments point out that there is a significant concern over the lack of single flood event analysis for determination of impacts to the agricultural community. This concern is carried forward to the C-111 study area via the physical connection of C-111 to L-31N at S-331/S-176. The C-111 GDM must rely upon data predicted at this structure for inflows from the north. Based on the kind of analyses conducted for the water deliveries alternatives, it is currently impossible to know what discharges might occur for the storms analyzed for C-111. Without specification of those inflows for each of the specific storm frequencies analyzed, results obtained are likely to be incompatible and unrealistic.

THE NO-PROJECT ANALYSIS

It has already been established that the Rainfall Plan produces conditions that require the use of the S-331 Pump Station for flood control. This was demonstrated during the Two-Year test that is due for completion this summer. Because of this and the fact that a yet-to-be-done analysis of flood events for L-31N may show an even larger need for use of this structure, a question arises regarding the interdependence of the two projects. Should the water deliveries project be implemented and the the C-111 project not be implemented, what increased flood impact potential is there for the C-111 basin?

In the past, the Committees have recommended that these two projects be integrated from an analysis point of view. That recommendation is re-emphasized here. If this can not be accomplished, it is recommended that the single event analyses discussed in the foregoing section for L-31N be conducted and used in a No-Project analysis for the C-111 basin to insure that there are no adverse downstream impacts.

RATING CURVE FOR THE C-111 GAPS

The rating curve presented in Figure 4 appears to be fitted to historical data through multiple backwater analyses. Two points are used from Tropical Storm Dennis, the highest available measurements for comparison. Due to the nature of the gaps and anticipated variations in upstream stages, as well as tailwater conditions, it is highly likely that they do not function as hydraulic "control points" where one would expect to find critical depth conditions. This further indicates that an unsteady rating curve (consisting of a series of loops for individual storms) is more realistic. Although this kind of behavior is more realistic, it is not suggested here that the Corps perform such an unsteady analysis. However, it is suggested that, for design purposes, a more conservative rating curve be used than that shown in Figure 4. Factors to be considered in making this recommendation include the following.

1. Dynamic storage effects should be considered.

2. Variable tailwater conditions and possibly storm surge should be considered.

3. Extrapolation of the curve past historic measured data would be safer.

4. Level of maintenance of the gaps should be considered.

As an additional observation, the upward slope of this curve beyond the 2000 cfs discharge capacity appears to be unrealistic unless there is a significant increase in cross sectional area at the corresponding elevation.

The above discussion is not proffered as a criticism but rather as an indication of our concern that this flow component be dealt with in a conservative manner due to its hydraulic sensitivity. It is an extremely important aspect of the hydraulic performance of the system.

PUMP STATION SIZING

A number of the proposed alternatives utilize pumping to accomplish flood control. Reliance on pump stations under such critical conditions requires a complete investigation of all factors affecting actual discharge capacity. It is our understanding that the preliminary sizings in the GDM provide some consideration of backward seepage from downstream areas. As an additional consideration, it is recommended that the Corps investigate the impacts of local water table drawdown on these proposed

capacities. Large pumps tend to create a localized depression in the water table to which ground water flows migrate. In the Biscayne Aquifer, this groundwater inflow can be significant (due to very high transmissivities) relative to flows induced down the channel. Our concern here is that predicted channel flows, and therefore upstream stages, are realistic in relation to this phenomenon. Our request is that it be considered in the determination of pump station capacity prior to finalization of GDM cost comparisons.

C-111 REGULATION SCHEDULE

Because all of the NED benefits to be derived from this project are related to the avoidance of agricultural damages, it would seem prudent to use all available means, including non-structural methods, to minimize these damages. To this end, it is recommended that a seasonally variable regulation schedule be included in the GDM that would allow for the annual production of tomatoes in the Frog Pond area. This regulation schedule should conform to that used by the SFWMD for the 1986-87 season.

DISCUSSION OF ALTERNATIVES

The document presents nine potential alternative plans for consideration. The first of these is the "no action" alternative. Based on the history of the past 5 years and damages reported in the document, it is obvious to the Committees that the C-111 system is not adequate for the land uses which currently exist throughout the basin. Therefore, the Committees do not support the status quo with regard to the project.

Plans B through D propose the use of pump stations in combination with other facilities as well as the backfilling of portions of C-111. Based on past experience with the S-331 pump station, it is our feeling that pump stations perform well for water supply purposes. However, their use in a flood control mode is risky if all factors are not fully evaluated. As stated in the foregoing, we are concerned about adequate sizing of these facilities. The Committees do not support these plans unless they are coupled to an adequate gravity discharge alternative including an operable structure at S-197.

It is the position of the Committees that Plan E (C-111 Culverts) would cause unacceptably high stages in the Frog Pond area as well as in areas to the north, both east and west of the canal.

Plan F (Extended C-109) is the most costly of all the alternatives and, if selected, will result in an entirely new set of potential economic, environmental, and social issues. Therefore the Committees would prefer pursuit of alternatives that deal with improvement of the existing system. However, should Plan F be selected for further consideration, it should be coupled with Plan G.

Plan G (New Operable Structure at S-197) has had support from the agricultural community since the "temporary plug" was installed. From an engineering and operational point of view it is ludicrous to rely upon the physical excavation of the plug to avoid flood damages. Based on data presented in the document, it appears that the above statement is also true for environmental considerations. It appears that installation of this facility alone would represent major freshwater savings and drastically improve distribution of storm flows to the Park and Florida Bay. The Committees strongly support construction of this facility. However, as will be discussed later, additional work is needed to make the C-111 system perform adequately for flood control.

Plan H (Enlargement of Other Existing Canals) should be included as an option for recovery of system capacity that may be lost as a result of using the S-331 pump station. It is our opinion that pumping will be required under the Water Deliveries GDM.

Elements contained in Plan I are considered necessary and desirable. The Committees therefore support inclusion of these elements. However, Plan I should be combined with Plan G. This should result in a less costly implementation, because operation of a structure at S-197 should lessen the degree of improvement necessary along C-111 and C-111E.

PREFERRED ALTERNATIVE

The Committees propose that the Corps selects the following alternative for solution of flooding and water balance problems in the C-111 Basin.

1. An operable structure at S-197 (Plan G).

2. Selected improvements to channel and structure conveyance along C-111 and C-111E. These improvements are to be predicated on the use of the S-197 structure (Item 1 above) during storm events.

3. Potential improvement of discharge capacity through the C-111 Gaps.

4. Improvements to the C-102 Canal to offset increased discharges from the S-331 pump station.

5. Revised regulation schedules for the ridge structures which would require that they be opened when S-331 is operated during flood conditions (non-water delivery modes).

6. A revised regulation schedule that would allow for seasonal drawdown in the vicinity of the Frog Pond.

COMMENTS ON PRELIMINARY CORPS PROPOSALS

FROM

SOUTH DADE AGRICULTURE TRUSTEES

GDM - MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

Prepared By

Ghioto, Singhofen and Associates, Inc.
Water Resources and Civil Engineering

January, 1987

INTRODUCTION

This document provides comments on the proposed alternatives for the GDM - MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK, presented by the U. S. Army Corps of Engineers in October 1986. These comments are divided into areas of concern related to either concepts or physical facilities and are presented in the following sections.

SELECTION OF "BASE CONDITION"

We take strong exception to the selection of a Base Condition that includes the Interim Action Plan for Lake Okeechobee based on the following arguments.

1. The Base Condition is the benchmark by which effectiveness of alternatives are measured with respect to both performance and benefit/damage accounting. It typically consists of a collection of physical facilities and possibly operational guidelines. Because the Interim Action Plan (IAP) is a product of State alterations to the operational strategy of a Federal Project which directly changes performance characteristics of that project, it is felt that it is not representative of the original design intent and is therefore inappropriate.

   Inclusion of the IAP therefore tends to mask (and potentially increase or decrease) the benefits to be derived by Everglades National Park (ENP). Likewise, it is possible that potential damages to the public and private sectors, resulting directly from the IAP, are implicitly being ignored and reducing the viability of the proposed alternatives for both water deliveries and flood protection.

2. The IAP is a non-constant set of operational guidelines that has changed since its inception. Within the past year, policy decisions at the State level have drastically increased volumes of water that have been released to the south over quantities that have been released in the past. These changes in policy have not been included in the Corps' computations for the project. Therefore, it has already been established that the IAP is a "moving target" that is difficult to quantify with any degree of precision. However, its impacts on facility design criteria as well as operational strategies can be very significant.

3. If it is the intent of the Corps to include modifications to the way that water is managed in the Everglades Agricultural Area, the Corps should establish a set of rigid and predictable guidelines for

the area which will be physically and operationally consistent with the proposed alternative plans for deliveries to ENP. These modifications should then become part of the proposed alternatives and not part of the "Base Condition".

4. To our knowledge, the IAP has never been subjected to public review by the Corps as a proposed modification to project operation. Its inclusion in the base condition is therefore an explicit project alteration without the benefit of formal public hearings.

PERFORMANCE OF THE PROPOSED L-31N SEEPAGE LEVEE

Computed maximum monthly stages in L-31N appear to be higher than the base condition for both the Rain Driven and Flow Through alternatives at times. It is not known if this behavior is the result of local numerical instability (in the model) or computed increased seepage to the canal. Inspection of minimum stages indicates that these are also higher and that these higher levels persist for much longer periods of time. Higher stages in the L-31N canal translate to higher groundwater stages to the east and therefore are of great concern to us. This is true for minimum stages as well as maximum stages, because the minimums are an indication of reduced aquifer storage for absorption of rainfall.

<u>It is the position of the Committees that any increase in water table elevations east of L-31N (regardless of season) is totally unacceptable.</u>

FLOOD EVENT ANALYSIS

The Corps has conducted a study using a continuous simulation procedure to evaluate the efficacy of the two delivery options. This modeling covers a period of 14 years (1969 through 1982). The model employed is an improved version of the model employed by the SFWMD in its earlier analyses of the same area. The position taken by the Committees then was that the model was fine for comparison of long-term water supply alternatives. However, it is not a good measure of the effectiveness of flood control works. Even if it were, the reporting of end of the month stages would not be sufficient to evaluate the results.

Use of regional continuous modeling has a number of advantages with regard to evaluation of the complex interaction of many hydrologic variables in a "real life" situation and how these variables change over long periods of time. That is why such an approach is desirable for water supply planning. The primary drawback in their use

of the current test. It is our opinion that conditions will be worse even with the construction of the seepage levee.

It is recommended that the Corps fully evaluate all data collected thus far during the two-year test, use it for verification of the computer model, and evaluate predicted performance in relation to actual field conditions. It would indeed be a shame if these data, that are considered so crucial to resolving disputes over system operation were to be ignored for the design of this project.

FLOOD PROTECTION ALTERNATIVES WEST OF L-31N

The Corps has proposed three alternatives to "mitigate" flooding west of L-31N as a result of the water deliveries options. These are:

1. Purchase all lands affected.

2. Protect the 8.5 square-mile residential area and purchase remaining affected lands.

3. Protect the residential area and some farm lands.

The Committees find that none of the alternatives proposed are acceptable for mitigation of flood impacts for the following reasons.

Alternatives 1 and 2, by removal of existing and permissible agricultural operations, have a definite negative impact on the agricultural community in Dade County through reduction of already scarce farmable land that is not recoverable in other areas of the County.

Alternative 3, while proposing to protect some lands, is not broad enough to protect lands currently in production let alone those where agricultural activities are permissible. Of the areas that lie within the proposed levee system, it is likely that areas between the levees and the internal seepage control canals will be unusable for agriculture because of anticipated wet conditions. These areas amount to a significant percentage of the entire leveed area.

No analyses have been presented to determine the effectiveness of the proposed systems. This casts a questionable light on the degree of protection to be provided as well as performance characteristics and costs.

In lieu of the proposed alternatives, the Committees propose that the Corps evaluates the conceptual plan provided in Attachment 1 to these comments. This plan

for flood control evaluation is related to their inability to model phenomena on a refined scale and to the statistically short periods of records used. From a statistical point of view, an 18-year period of record can be used to predict a 10-year flood with an accuracy of only +/- 25 percent at the 95% confidence level. To obtain +/- 10 percent accuracy, a record of 90 years is required. Therefore, the modeling done to date is not considered statistically valid for this purpose.

Because of these drawbacks, it is recommended that the Corps initiate a single-event flood analysis of the proposed delivery options to evaluate flood control effectiveness of the L-31N and L-31 canal systems. This analysis should include any increase in antecedent water table levels as well as increased seepage created by the proposed options. We do not object to use of the continuous model to set up these antecedent conditions.

The single-event analysis should be done during the GDM phase of the project to insure that results do not alter major project components or operational strategies. This analysis will also potentially affect the validity and design criteria for the C-111 project.

TWO-YEAR TEST DATA

Field tests were initiated after it was established that the original modeling by SFWMD was inappropriate for determination of flood impacts associated with the diversion of waters to NESRS. Two years of testing with the rainfall formula will be concluded in the summer of 1987. Although canal level criteria are lower than those used for the GDM and there are cutoff criteria for releases to the slough, the Corps has chosen not to evaluate data collected to date in relation to accuracy of the model or with regard to field conditions. The value of these data lies not only in the field testing of the rainfall formula, but also in the degree to which the monitoring network has been expanded to develop a refined understanding of system behavior. This refinement would be invaluable for the verification and improvement of modeling efforts.

Even with the constraints on the use of S-333, there have been times when agriculture both east and west of L-31 have been in jeopardy as a result of system operations. Even if flood protection west of the canal is provided by the project, its levels can not be allowed to rise as a result of project implementation. In our opinion, this can not be avoided with either delivery option without increased use of the S-331 pumping station or system improvements to allow for increased easterly discharges. This problem has been demonstrated and documented during the first 18 months

should be designed to provide total mitigation of effects of the deliveries options. In addition, internal works should be designed to provide year-round drainage appropriate for "permanent-set" as well as "seasonal" agricultural operations. It is our opinion that mitigation in combination with maximized production capability should greatly enhance the cost-effectiveness of the project. All lands west of the proposed levee system should be purchased. It is also proposed that the GDM contain analysis results for flood protection as well as design and performance criteria for later use in a DDM for the project.

cc: W Lovett

SOUTHERN EVERGLADES TECHNICAL COMMITTEE

February 11, 1987

RECEIVED
FEB 13 1987
BUREAU OF STATE
RESOURCE PLANNING

Colonel Charles T. Myers III
U.S. Army Corps of Engineers
Jacksonville District
400 West Bay Street
P.O.Box 4970
Jacksonville, Florida 32232-0019

Dear Colonel Myers:

The Southern Everglades Technical Committee (SETEC) is an advisory organization which was established by the Everglades National Park/East Everglades Resource Planning and Management Committee in April 1985 for the purposes of reviewing water quantity and water quality test designs; coordinating hydrological and ecological monitoring; and identifying future research needs. The Governor and Cabinet recently authorized the continuation of SETEC's work. SETEC is composed of representatives of entities conducting hydrological and ecological research in the Southern Everglades.

On December 3 and December 10, 1986, SETEC met and reviewed the Army Corps of Engineers' General Design Memorandum on Modified Water Deliveries to Everglades National Park. Lewis Hornung represented the Corps at these meetings. Below are SETEC's findings and recommendations on the GDM. These comments are in the form of responses to specific questions posed to SETEC by Mr. Hornung. These comments are supplementary to the comments which are being sent directly to you by local, state and regional agencies who are also represented on SETEC.

1. Does SETEC support the process that is being undertaken to improve hydrologic conditions to benefit the Everglades ecosystem with the Water Conservation Areas, Everglades National Park, and the East Everglades?

SETEC supports efforts to improve the timing and amount of water flow into the East Everglades and Everglades National Park. SETEC supports the construction of the new structures being proposed to

improve water deliveries and to distribute the water in a more natural flow pattern.

SETEC members agree that it is not possible to guarantee that these structures will improve biological conditions in the Everglades under either of the proposed delivery systems. Determining the appropriate water delivery program will require continued testing under the iterative testing process. The structural modifications proposed by the Corps will provide additional delivery options to assist in the implementation of the iterative testing process and will permit increased flexibility in managing the quantity and distribution of flow to Everglades National Park.

2. Would either, or both, of the alternative plans being considered benefit the Everglades ecosystem; and if so, how?

SETEC members believe that the rain driven model will provide better water storage capability and will allow for better resource management in the Everglades. More generally, SETEC members determined that there would be specific benefits to the Everglades if the basic structural changes which are inherent in both alternatives were to be made. Benefits identified by SETEC members are as follows:

-As noted above, these structural changes will provide additional delivery options to assist in the implementation of the iterative testing process.

- These structural changes will reduce flooding south of the S-12 structures. Large flooding releases have in the past disrupted alligator breeding cycles.

-The proposed changes will broaden the distribution of water flow through northeast Shark River Slough. Increased hydroperiods may increase peat production in the Everglades and would reduce the impact of dry season fires. Also, wider distribution of water will help improve water quality in the area by permitting increased filtration.

3. Are there other alternatives (or modifications to the current alternatives) that should be considered at this time?

SETEC members recommend that the following additional testing and modeling be done to address unanswered questions:

-The rain driven alternative should be studied with the placement of plugs in L-67A (as in the flow through model). These plugs might improve the flow of water to WCA-3B and might improve the flow of water from WCA-3B to S-355. This modification may also have the effect of lowering the amount of water flowing from WCA-3A through the S-12 structures and may reduce the impact of increased water levels in the agricultural and residential areas that might result from the proposed plan.

-The Corps should compare the water levels produced by its model, particularly at P-33, with historic data from that site. It is recommended that the Corps use water level data from 1950 to 1963 as a best guess of how the hydroperiod fluctuated prior to human disturbance. Good data on water levels from 1950 to 1963 exist and could be used for this modeling. Additionally the model should be used to determine what the water delivery requirements are to establish an annual hydroperiod at P-33 which would most closely resemble conditions in the 1950s. It is specifically recommended that this be done using the rain driven model.

-The most recent Dade County projections for the West Dade Wellfield should be incorporated into the base conditions. The 1990 model should be run again to determine the impact of the wellfield on water levels.

-The model should be run using present conditions (i.e., with the IAP and without the West Dade Wellfield) to determine how well the model would have predicted water levels at P-33 for the past two to three years. This will provide some guidance on how to interpret the model's data.

-The Corps should run some "worst case" scenarios showing what will happen to water levels and what kind of hydroperiods will result for different extreme years (e.g., one hundred year flood and one hundred year drought). Based on these predictions, it may be necessary to make further modifications to these plans to avert serious impacts, either flooding or drought.

-The base conditions computed by the Corps include operating criteria established in the Interim Action Plan (IAP) for Lake Okeechobee. However, the structural system was not initially designed by the Corps to operate under these conditions. Therefore, the Corps should make model runs using base conditions data that do not include the the IAP and then evaluate the impact of removing the IAP from the base conditions computation.

-Additionally, further information was requested regarding the ability of the model to accurately predict the impacts of changes in the percent of water distributed between the S-12 structures and the S-355 structures.

We appreciate the opportunity to comment on the GDM. We look forward to continuing to work with you on this important effort. In particular, if you believe that SETEC can be of help to consultants in quantifying the ecological benefits of these proposed modifications, please contact me. We believe continued cooperation among the agencies responsible for managing the Everglades and its resources is essential if the Everglades are to be restored.

Sincerely,

Ane Deister

Ane Deister, Chairman
Southern Everglades Technical
Committee

AD/wld

cc: SETEC members

United States Department of the Interior

NATIONAL PARK SERVICE

EVERGLADES NATIONAL PARK
AND
FORT JEFFERSON NATIONAL MONUMENT
P.O. BOX 279
HOMESTEAD, FLORIDA 33030

IN REPLY REFER TO:

L54

APR 01 1993

Colonel Robert Herndon
District Engineer
U.S. Army Corps of Engineers
P.O. Box 4970
Jacksonville, Florida 32232

Dear ~~Colonel~~ Bob Herndon:

My staff and I have completed our review of the revised structural alternatives for the General Design Memorandum (GDM) for improved water deliveries to Shark Slough. First, I would like to thank you and your staff for the time and effort you have spent in revising these alternatives. The coordination meetings your staff has conducted with other agencies and groups interested in improved water deliveries to Everglades National Park have led to significant improvements in the design alternatives. We hope that this extensive review process will lead to widespread support for the proposed modifications and the completion of the GDM as soon as possible.

Shark Slough Water Management Objectives

Our goal in requesting the GDM was to develop an improved water delivery system for the Shark Slough basin which could mitigate many of the observed declines in park resources. The park's specific water management objectives for the basin were outlined in a March 1987 letter to your agency detailing five major objectives for improving water deliveries to Shark Slough. These were:

1. Mitigate high water releases to Shark Slough through the S-12 structures, particularly during dry season recession periods.

2. Restore flow distributions across the entire width of the Shark Slough flow section (L-30 to 40-Mile-Bend).

3. Allow the volume and timing of deliveries to fluctuate naturally in response to upstream rainfall and antecedent conditions.

4. Maintain downstream stages in Shark Slough near the levels which occurred prior to the water management changes of the Central and South Florida Project.

5. Maintain the quality of water deliveries so that resource degradation does not occur.

The park has made many specific recommendations to your agency for accomplishing the above objectives, beginning with the "7 Point Plan" as outlined in the letter of March 1983 from Superintendent Morehead to Colonel Devereaux. Much progress has been made toward reaching these objectives through the iterative testing process coupling the development of new delivery schedules (Flow Through and Rainfall Based delivery experiments) with the redistribution of surface water flows into Northeast Shark Slough via S-333. The proposed structural modifications should further improve the water delivery system and provide the flexibility to allow additional operational changes as ongoing research may suggest. We would like to emphasize that the completion of specific structural modifications does not reduce the need for continued experimentation regarding the development of improved water delivery schedules and possible changes in operation of the system.

Park Concerns Related to the Structural Modifications in the Developed Areas of the East Everglades

Prior to discussing the proposed structural modifications, a few general comments are in order. First, on the question of acquisition or flood protection for the developed areas of the East Everglades, the park would recommend that flood protection be provided if this can be accomplished with as little impact as possible on the adjacent wetlands and would not otherwise compromise the Northeast Shark Slough restoration plans. If analysis demonstrates that the requirements for flood protection within the East Everglades developed areas would place operational constraints on the proposed structural modifications or otherwise reduce the environmental benefits to be provided by the Shark Slough GDM, then the National Park Service would object to providing flood protection to the 8.5 square mile area and request acquisition. If there are no operational constraints and no reduction in environmental benefits then we would support flood protection for the existing developed residential area within the 8.5 square mile area and not for the agricultural areas on the periphery or to the south. Providing additional flood protection in these agricultural areas is not warranted given the relatively minor water level increases suggested by the model output. In addition, at least half of the area within the originally proposed southern agricultural levee system is a natural wet prairie and would be damaged by the proposed flood protection.

A second major issue of concern to the park is the need for reductions in the flows through the S-12 structures during periods of high water. The results of the last two and one half years of the Rainfall Based delivery experiment have shown that under high water conditions the S-12 structures must still remove the largest portion of the excess water in Water Conservation Area 3A (WCA 3A). We hope that the proposed improvements to structure S-334 and the Tamiami Trail, along with acquisition or flood protection for the residential areas in the East Everglades, will remove any remaining conflicts with the operation of structure S-333. We urge your staff to keep in mind our objective of distributing flows to Shark Slough across the entire flow section as uniformly as possible, particularly during periods of large regulatory releases.

A third major concern is the impact of the proposed West Dade Wellfield on our efforts to restore hydroperiods in Northeast Shark Slough. The modeling work completed to date by Dade County is sufficient to indicate the volumes of water needed in the adjacent canals to support the proposed wellfield pumpages and estimated water level drawdowns in Northeast Shark Slough. We would like your staff to incorporate these surface water demands and the estimated water level drawdowns into your future modeling efforts in the Detailed Design Memorandum. This would not only tell us what the future impacts of this wellfield would be on Shark Slough but also what structural modifications might be needed to reduce the effects of these pumpages.

Acquisition of the Private Lands in Northeast Shark Slough

The Federal government through the Secretary of the Interior and the State of Florida through the Governor and Cabinet have jointly committed to the public acquisition of the private lands in Northeast Shark Slough in order to restore historic flows to the entire Shark Slough basin. This process has been initiated by the Governor through the promulgation of an executive order establishing a study group and the Department of the Interior has demonstrated its commitment by the appointment of Federal representatives to this study group. Additionally, the proposed acquisition and potential incorporation of Northeast Shark Slough lands into Everglades National Park enjoys widespread Congressional support.

We believe that this study group will fully address the issue of land acquisition in the East Everglades and will develop appropriate recommendations and acquisition cost estimates. Therefore, we believe the GDM process should proceed under the assumption that the lands in Northeast Shark River Slough that will be subjected to increased hydroperiod will be in public ownership. This realistic assumption appropriately eliminates the land acquisition costs from detailed GDM analysis and renders the taking issue moot.

For the remainder of our analysis we have made a distinction between the type of water delivery schedule in use and the proposed structural modifications. We have done this for two reasons. First, the National Park Service believes that selection of the most appropriate water delivery formula is premature and the experimental program of deliveries must continue so the results of ongoing research can be incorporated. Second, the proposed structural alternatives must be designed with the flexibility to allow for future delivery formula changes as more information is compiled on the volume, distribution, timing, and water quality needs of the basin.

Structural Alternatives with Land Acquisition in the East Everglades Developed Areas

If the developed areas of the East Everglades are acquired and flood protection is not provided, then the Rain-Driven alternative with the S-349 structures and an L-31N pump station appears to be the most appropriate choice. We would prefer that the L-67 extension canal be removed by backfilling the levee material and returning the area to its original grade. This would restore the area back to its original condition and eliminate any east to west variation in water levels produced by the canal and levee system. The addition of the gated spillway structures in the L-67A canal would serve to divert canal flow out into the WCA 3A marsh. This would produce more uniform water levels in WCA 3A as well as increasing travel times in the marsh, thereby improving the quality of delivery waters. Under normal operating conditions the S-349 structures should remain closed to increase stages on the upstream side of the proposed S-345 culverts and increase flows into Water Conservation Area 3B (WCA 3B). The use of a pump station in the L-31N canal to move seepage water northward into the L-29 canal system would be preferable to a seepage levee. This would reintroduce the seepage water back into the wetlands as far north as possible. This would also reduce the loss of valuable wetlands that would occur by construction of a seepage levee west of the L-31N canal.

Your modeling results suggest that replacing the proposed gated spillway structures (S-355) in the L-29 levee with degraded sections of the levee would not produce any significant improvements. On this point, we are more concerned that a surface water connection is made between WCA 3B and Northeast Shark Slough to meet the current

50 to 55 % split as established in the Rainfall Based formula. We feel that deliveries to Northeast Shark Slough should be made primarily from WCA 3B with supplemental flows from S-333 when needed. This would further increase the travel time in the marsh producing additional water quality benefits.

Structural Alternatives with Flood Protection in the East Everglades Developed Areas

If acquisition of the developed areas is not feasible and flood protection must be provided, we would prefer the Rain-Driven alternative with a levee and canal system only around the immediate residential areas such as that proposed by the U. S. Fish and Wildlife Service. We would also recommend the addition of the S-349 structures and an L-31N pump station which transfers the seepage water northward into the L-29 canal. This proposed alternative eliminates the need for a seepage levee west of the L-31N canal. The second alternative of placing a pump station within the protected residential area, which would discharge directly into Northeast Shark Slough, does not appear to provide any additional improvements. It also would have the disadvantage of reintroducing the seepage water collected from the L-31N canal well below the Tamiami Trail, eliminating any potential benefits to the upper slough. Any flood protection system that will remove water from the developed portions of the 8.5 square mile area may cause additional water quality concerns to the park if this water is reintroduced into Shark Slough. For this reason, additional water quality monitoring stations would be needed to safeguard park delivery waters.

Summary and Conclusions

The iterative water delivery tests and their related environmental studies have contributed to our understanding of the factors controlling the downstream conditions in Shark Slough. The experimental delivery program has added to our understanding of the timing of deliveries and their relation to upstream rainfall and antecedent conditions. The testing has also helped to describe the impacts of the redistribution of flows across the entire Shark Slough flow section. The determination of the basin's total delivery volume requirements is more difficult to access because of the lack of data describing the preconstruction historical conditions in the Everglades system and the subsequent changes in the spatial scale of the remaining wetlands. The proposed structural modifications will produce a more flexible water delivery system which will allow future experimental changes to be incorporated as the results of our ongoing environmental studies are evaluated.

The proposed structural modifications should allow larger discharges to be made from WCA 3A into WCA 3B and Northeast Shark Slough to distribute water over as large an area as possible, particularly during high water conditions, if future peak flows continue to be detrimental to park resources. Surface water flows from WCA 3B should be the primary source of delivery water to Northeast Shark Slough rather than structure S-333. This will help establish a more natural sheetflow system and provide additional water quality protection. We recommend that the L-67 extension canal be removed by backfilling the levee material and returning the area to its original grade. This will further restore a more natural water depth distribution across the entire width of the slough.

The National Park Service recommends that the existing developed areas within the 8.5 square mile area of the East Everglades be provided flood protection only if this can be accomplished without compromising the goals or reducing the potential benefits of the

Shark Slough restoration plan. If this overall restoration plan appears to be impacted by the proposed flood protection plans, then the Park Service requests acquisition of the existing developed areas in the East Everglades.

The park is quite concerned about the impact of the proposed West Dade Wellfield on our efforts to restore hydroperiods in Northeast Shark Slough. We recommend that all available data on the wellfield be incorporated in your future modeling. We believe that as currently modeled, the western alignment of the proposed wellfield has serious detrimental effects on the restoration program. The National Park Service recommends that the COE make all possible efforts to encourage Dade County to place the wellfield in an area that will not impact or otherwise compromise our restoration efforts.

The park requested that the GDM recognize the current commitment by the Department of the Interior and the state to acquire the private lands within the East Everglades and make the assumption that those costs will not be attributable to the project. The park fully supports your agency's plans for structural modifications to the water delivery system of Shark Slough and we hope that the required design and construction can be completed as soon as possible. The proposed modifications should remove all of the current operational restrictions to allow the maximum possible flexibility in the operation of the water delivery system.

M.V. Finley

Michael V. Finley
Superintendent

cc: John Wodraska, SFWMD
Joseph D. Carroll, Jr., USFWS
Sheppard Moore, USEPA
Bradly J. Hartman, FG&FWFC
Brian Barnett, FG&FWFC
G. Louis Ducret, Jr., USGS
Tony Clemente, DERM
Herb Zebuth, DER

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CON 13-04; RF: 90160

February 23, 1990

Colonel Bruce A. Malson
District Engineer
U.S. Army Corps of Engineers
Jacksonville District
P.O. Box 4970
Jacksonville, FL 32232-0019

Dear Colonel Malson:

This is in response to your February 7, 1990 letter concerning the General Design Memorandum on Modified Water Deliveries to Everglades National Park. This District has supported this effort from the beginning, but we nevertheless are uncomfortable with the position in which we find ourselves at this point. We are disappointed in the process that led to the apparent selection of a recommended plan. Considering the importance of this program, we expect to be fully involved in future discussions and decisions on this project. After two years of plan development and coordination, and some truly exceptional work by your staff, a proposal was identified that had very broad support from federal, state and local agencies and the general public. From that point the plan apparently went into more than a year of reformulation by the Fish and Wildlife Service (FWS). We were never a party to these efforts and have still not received the information which was the basis of that agency's jeopardy opinion.

We cannot agree with the Modified Rain Driven (MRD) Plan as the recommended plan without a thorough review of all the information that led to its selection. There are some major technical and policy questions concerning water supply that are obvious from just a cursory review of your February 7 package. Page four of the Biological Opinion implies that the Modified Rain Driven plan uses less water from Lake Okeechobee than the base condition, but your table 1 shows that on average the MRD plan requires 91,000 acre feet per year more from the Lake than the base condition. Based on your description of the plan, I believe your table is more correct than the biological report but we need to review all the data to clear this up.

When the minimum delivery schedule for Everglades National Park (ENP) was enacted in 1970, it was recognized that for the federal government to appropriate what was considered a state resource (water) to a federal use (ENP) was an issue that could only be settled through federal legislation. The MRD plan now says that in times of drought we are to take state water out of South Florida's only effective reservoir and dedicate it to maintaining an artificial impoundment on state land for the theoretical benefit of an endangered species. This is obviously not a decision that should be made by a federal biologist in a closed door process. We are in the second year of a severe drought. The way we read the MRD plan we would have been expected to have moved water out of the Lake into WCA-3A for the last

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Tilford C. Creel, Deputy Executive Director
Thomas K. MacVicar, Deputy Executive Director

thirteen months to attempt to satisfy the needs of the snail kite at the expense of the other human and environmental water users in the system.

You are also well aware of the present discussions with ENP and EPA on proposed water quality standards for flow into the Park. While we do not know what the final standards will look like, it is clear that an important element of any plan to ensure that the Park continues to receive good quality water will be to promote as much sheetflow (and as little canal flow) as possible in WCA-3A. The MRD plan does just the opposite by requiring canal flow from Lake Okeechobee to the south end of WCA-3A during droughts. Considering the role of the FWS in the present lawsuit, I find the water quality implications of their biological opinion ironic to say the least.

At this point we still endorse the original Rain Driven Plan. Since the Structural changes between that and the MRD plan do not differ, we do not object to the GDM being completed and submitted for review subject to the condition that Sec. 7 coordination be reinitiated. There are also some design details that need further refinement during development of the Features Design Memorandum prior to the preparation of plans and specifications. This will allow formulation of a delivery schedule and final design that are acceptable to the broad spectrum of federal, state and local agencies and the public. We look forward to participating with the federal government in a program that is truly in the best interest of all of south Florida.

Sincerely,

John R. Wodraska
Executive Director

JRW/TKM/smg

c: Robert Chandler, Everglades National Park
David Ferrell, U.S. Fish & Wildlife Service

United States Department of the Interior
NATIONAL PARK SERVICE
EVERGLADES NATIONAL PARK
AND
FORT JEFFERSON NATIONAL MONUMENT
P.O. BOX 279
HOMESTEAD, FLORIDA 33030

IN REPLY REFER TO:
N22

March 6, 1990

Bruce A. Malson, Colonel
U.S. Corps of Engineers
Jacksonville District
P.O. Box 4970
Jacksonville, Florida 32232-0019

Dear Colonel Malson:

In response to your letter of February 7, 1990, we mention briefly below the points of the Parks' long-term ecological monitoring (LTEM) program that seem applicable to the draft GDM regarding modified water deliveries. Please note that the core elements of our LTEM program have been in place since at least the mid-1980's, without interruption. Thus substantial databases exist against which the effects of the modified water deliveries on several ecological parameters may be measured.

Our existing LTEM studies that appear to be directly relevant to the draft GDM include the following:

1) NESTING OF COLONIAL WADING BIRDS

PURPOSE: Examine timing, location, species composition, numbers, and nesting success of wading birds in the major marsh and estuarine colonies of Everglades National Park, the Northeast Shark Slough, and Water Conservation Areas (WCA) 2 and 3.

LENGTH OF RECORD: For key species, such as the endangered Wood Stork, an essentially unbroken record extends back into the 1950's. Records for the more common species are less detailed up to the mid-1970's, but substantially complete since then. The current colony survey program is a cooperative venture with the National Audubon Society Research Department, and covers the entire Everglades region south from WCA-2. In about 10 nesting seasons from the 1970's to date, detailed investigations of foraging range, food habits and nestling growth and survival have been carried out on particular species and colonies.

2) EVERGLADES MARSH FISHES, ETC.

PURPOSE: By means of monthly throw-trap samples at nine stations in Shark Slough and Northeast Shark Slough, determine the standing stock of marsh fishes (density and biomass by species) in relation to water conditions. Major aquatic macro-invertebrate forage organisms, such as freshwater prawns, are also adequately sampled by this method. Populations of larger fishes are monitored semi-annually by electrofishing in five alligator ponds in northern Shark Slough.

LENGTH OF RECORD: Data from the present throw-trap sampling regime are available since 1978. Records of earlier quantitative sampling of marsh fishes, such as the U.S. Geological Survey pull-traps, extend back into 1965.

3) SRF STUDIES OF FORAGING WADING BIRDS

PURPOSE: Determine numbers and distribution by species of wading birds foraging in the interior and estuarine wetlands of Everglades National Park by the systematic reconnaissance flight (SRF) technique. This method permits an estimate of total numbers as well as of numbers in each 2x2-kilometer cell of a grid covering the entire area. Currently flights are made monthly, December through May, in the dry season and in August in the middle of the wet season.

LENGTH OF RECORD: SRF wading bird coverages have been carried out since January, 1985.

4) SRF DEER STUDIES

PURPOSE: Determine numbers, distribution, recruitment, and sex/age composition (as feasible) of White-tailed Deer by SRF coverage of grassland areas of Everglades National Park. Currently, two coverages are flown each year in the late wet season and the late dry season.

LENGTH OF RECORD: Continuous since 1985.

5) SRF STUDIES OF ALLIGATOR NESTING

PURPOSE: Obtain an estimate of the density and distribution of alligators in grassland areas of the Park by counting nests each July along the SRF grid and determining the fate of a representative sample (about 20 percent) of nests by frequent (every two to three weeks) ground visits.

LENGTH OF RECORD: Continuous since 1985.

6) ENDANGERED SPECIES, INDICATOR SPECIES MONITORING PROGRAM

PURPOSE: Determine long-term population trends and distribution patterns for key endangered species/indicator species. New, more systematic protocols are now being developed for census coverage of the following: Florida Panther, Manatee, Wood Stork, Roseate Spoonbill, Great White Heron, Brown Pelican, Bald Eagle, Osprey, Snail Kite, Limpkin, Cape Sable Sparrow, Am. Crocodile, Loggerhead Sea Turtle, etc.

LENGTH OF RECORD: Baseline data for several of these species exist as far back as the 1960's - 1970's; all will be censused at appropriate intervals in the future.

Current National Park Service (NPS) funding of the above LTEM studies is approximately $300,000 per year. In addition, the South Florida Water Management District (SFWMD) has contributed to studies of wading bird foraging and nesting success within the park through contracts with the National Audubon Society and the University of Florida, both now completed. The studies listed above are clearly the best means we posses for estimating the ecological effects of water delivery modification. Subject, as always, to the availability of funds, it is the firm intention of NPS to continue the six listed studies as long as may be necessary to yield definitive answers regarding water deliveries to the Park. Various recommended modifications and expansions of present studies, as well as additional studies, will be undertaken to the extent that we are able to fund them.

For example, we have no studies in place to examine changes in soils or vegetation that may result from the changed water deliveries, but it is plain that such studies would be highly desirable.

Obviously, the responsibility for monitoring the ecological effects of altered water-delivery patterns must be shared with other agencies. Any changes that occur in the Park are likely to reflect other changes in Water Conservation Areas 3A and 3B and elsewhere in the Everglades system. These changes also need study but are far beyond our authority or capacity to undertake. In some instances, such as the SRF studies for wading birds, results obtained from work within the Park will be difficult to interpret without a regional view of these mobile populations. Yet, present cooperative arrangements for SRF coverage of the WCAs, the Big Cypress, the Everglades agricultural area, and Lake Okeechobee appear likely to expire at the end of the current fiscal period. We strongly suggest that all of the agencies concerned should join in drafting a comprehensive LTEM program for the entire region that may be affected by the modified water deliveries.

Please let us know if you have need of additional materials or information.

Sincerely,

Robert S. Chandler
Superintendent

FLORIDA DEPARTMENT OF STATE
George Firestone
Secretary of State
DIVISION OF HISTORICAL RESOURCES
R.A. Gray Building
Tallahassee, Florida 32399-0250
(904) 488-1480

December 29, 1986

In Reply Refer to:

Mike Wisenbaker
Historic Sites Specialist
(904) 487-2333

Mr. A.J. Salem
U.S. Army Corps of Engineers
P.O. Box 4970
Jacksonville, Florida 32232-0019

RE: Your Letter of December 15, 1986
Cultural Resource Assessment Request
Proposed alternative plans for a modified water delivery system to Everglades National Park, Dade County, Florida

Dear Mr. Salem:

In accordance with the procedures contained in 36 C.F.R., Part 800 ("Procedures for the Protection of Historic and Cultural Properties"), we have reviewed the above referenced project for possible impact to archaeological and historical sites or properties listed, or eligible for listing in the National Register of Historic Places. The authorities for these procedures are the National Historic Preservation Act of 1966 (Public Law 89-665) as amended by P.L. 91-243, P.L. 93-54, P.L. 94-422, P.L. 94-458 and P.L. 96-515, and Presidential Executive Order 11593 ("Protection and Enhancement to the Cultural Environment").

Since the scale of the maps you provided us with is so small, it is difficult to make any specific comments regarding the location of significant, or potentially significant, cultural resources within the project areas. In general, however, it is our recommendation that any new dredge and fill activities which occur on tree islands should be preceded by a systematic, professional archaeological survey. Data collected from tree islands in south Florida indicate that archaeological and historic sites, especially the former, are likely to occur in these areas. It is, therefore, the opinion of this office that

there is a reasonable probability of project activities impacting archaeological and historic sites and properties potentially eligible for listing in the National Register of Historic Places, or otherwise of national, state or local significance.

Since potentially significant archaeological and historical sites may be present on the tree islands, it is our recommendation that, prior to any dredging or filling within these areas, they should be subjected to a systematic, professional cultural resources survey. The purpose of the survey will be to locate and assess the significance of cultural resources present. The resultant survey report should be forwarded to this agency in order to complete the process of reviewing the impact of this project on archaeological resources. However, project activities may proceed in the remainder of the areas without further involvement with this office.

If you have any questions concerning our comments, please do not hesitate to contact us.

Your interest and cooperation in helping to protect Florida's archaeological and historical resources are appreciated.

Sincerely,

for George W. Percy
State Historic
Preservation Officer

GWP/efk

FLORIDA GAME AND FRESH WATER FISH COMMISSION

DON WRIGHT, Orlando | JOSEPH G. SPICOLA, JR., Tampa | MRS. GILBERT W. HUMPHREY, Miccosukee | JOE MARLIN HILLIARD, Clewiston | BEN ROWE, Gainesville

ROBERT M. BRANTLY, Executive Director
ALLAN L. EGBERT, Ph.D., Assistant Executive Director

EVERGLADES REGION
551 North Military Trail
West Palm Beach, FL 33415
(407) 640-6100

August 29, 1991

MEMORANDUM

TO: John Moulding, Biologist
U.S. Army Corps of Engineers

FROM: Jon R. Fury
Biological Scientist III

SUBJECT: Angler Use Survey Results

The purpose of this memo is to provide you with some data we collected during an Angler Use Survey on the L-67A Canal in Water Conservation Area 3. The survey was conducted from 2 December 1990 to 1 June 1991, and was limited to the L-67A Canal. Anglers who were fishing the Miami Canal, L-68A Canal or Tamiami Canal were not interviewed.

We divided the L-67A Canal into two areas (see attached figure). Area A was the section north of the proposed middle water structure (refer to first draft of the GDM on Modified Water Deliveries to Everglades National Park). Area B was the section south. Each area was approximately 21 km in length.

During the survey, a total of 1,451 anglers were interviewed. Results showed a total effort of 50,000 man-hours and a total harvest of 51,000 fish. An estimated $150,637.00 were spent by anglers during the six month period.

As you know, there are two main access points for the L-67A Canal. Seventy-one percent of the anglers interviewed had launched from Holiday Park Recreation Area. Of those anglers, 33% were interviewed in Area B, meaning they had put their boat in at Holiday Park and traveled farther than 21 km to fish. Conversely, 29% of the anglers interviewed had launched at Tamiami Trail. Less than 1% of those anglers traveled north into Area A.

Another concern we had discussed earlier was the utilization of the L-67C Canal. Twenty-six percent of the anglers we asked said that they do fish the L-67C Canal. However, 50% of anglers who accessed the L-67A Canal from the Tamiami Trail boat ramp had said that they fished the L-67C Canal. Of all anglers who answered in the affirmative, 68% reported that they traveled farther than 14.4 km from the boat ramp. This is the approximate point of a proposed structure blocking the canal.

I hope this information is useful to you, and if you have any questions please give me a call.

cc: Morello
Dunford

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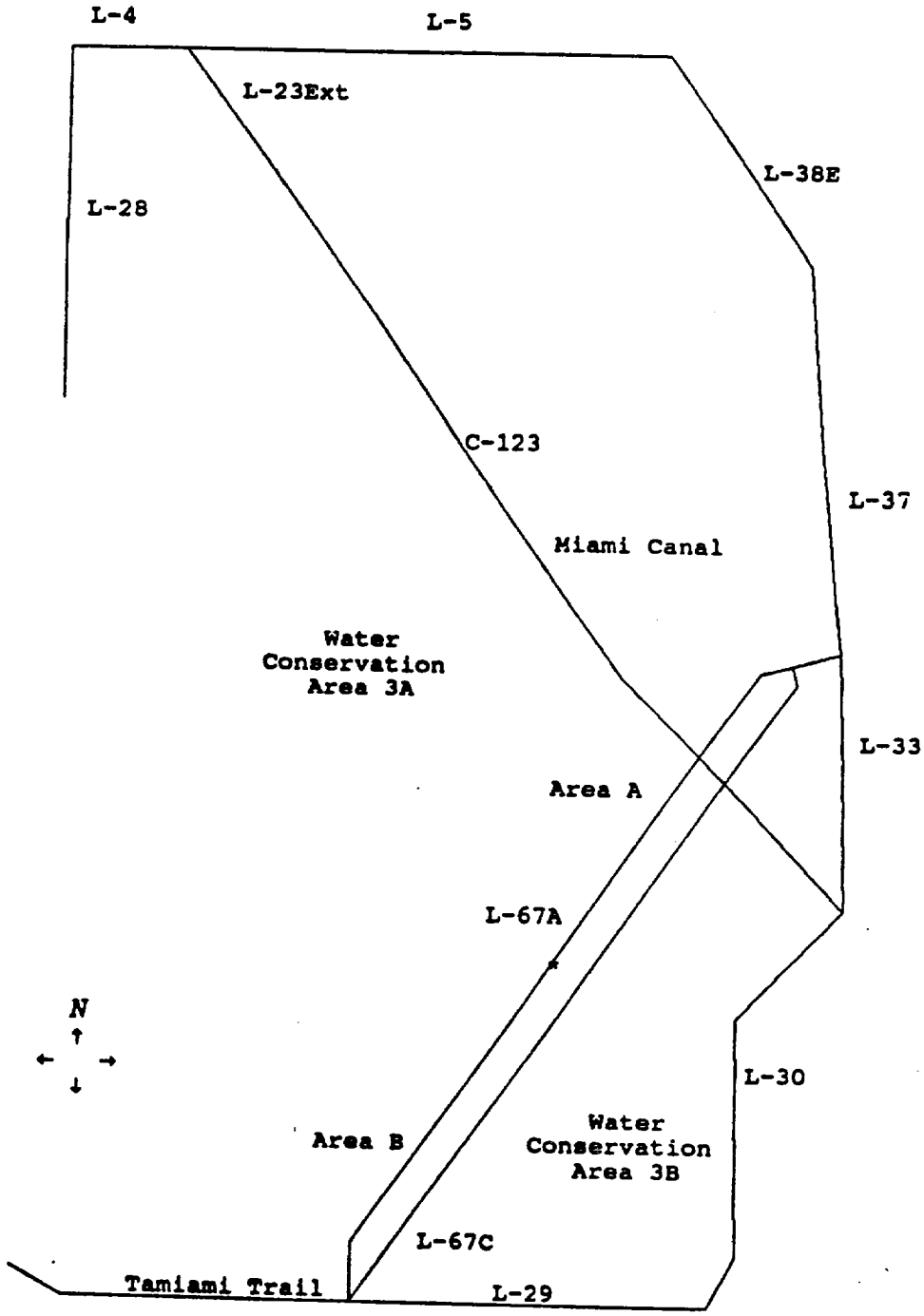

FIGURE 1. - Map showing Water Conservation Area 3 census areas A and B, L-67A Canal during FY 1990-91 (* - denotes boundary between A and B).

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX B

SOUTH FLORIDA WATER MANAGEMENT MODEL

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX B

SOUTH FLORIDA WATER MANAGEMENT MODEL

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APPENDIX B

SOUTH FLORIDA WATER MANAGEMENT MODEL

1. General. The South Florida Water Management Model (SFWMM) is the primary model used in the assessment of regional water supply impacts and of hydrologic changes in the Everglades associated with each alternative plan. Documentation of the model is contained in Technical Publication 84-3 "South Florida Water Management Model Documentation Report", February 1984, South Florida Water Management District. The model was developed by the SFWMD under contract (DACW17-81-C-0035) for the Corps of Engineers. It was developed for use in the Central and Southern Florida Water Supply study but because of its utility, it has been used in this study as well.

The SFWMM is a large scale hydrologic mathematical model capable of simulating the integrated system of surface water and groundwater condition present in south Florida. It is an analytical tool for addressing water management issues related to structural or operational changes to the C&SF Project.

In the eastern portion of the area, the highly permeable surface aquifer and the interaction between the canal system and the aquifer were the primary considerations in the model's development. To the west, the Water Conservation Areas and the Everglades dominate. Here the important processes are overland flow and groundwater movement to the east. Evapotranspiration is important in all areas. To realistically model these phenomena and to be able to delineate the areal extent and duration of flooding, as well as pinpoint excessive drawdown at municipal wellfields, a numerical model based on a distributed nodal network was required.

The scope of the modeling effort favored the selection of mathematical formula that were computationally efficient and whose data requirements could be satisfied without the need for extensive additional field work. Many of the physical variables (aquifer permeability, Manning's overland flow coefficient, etc.) that control hydrologic activity can only be estimated within a fairly large range. A concerted effort was made to obtain the best physical data possible. Variables that were known to be poorly defined were calibrated by comparing the model's results with historical hydrologic measurements.

Plate B-1 shows the area modeled and the grid cells configuration used. Each grid cell is four square miles in size. The model includes 1,703 grid cells.

A time step of one day is used. This is the minimum time increment for which hydrologic data such as rainfall, evaporation, and structure discharge are generally available. Since simulations cover a period of 14 years, a time step of less than one day is impractical.

To simplify programing and to cut down on computer time requirements, there is no iteration between surface and groundwater routines within one time step. An explicit technique is used for the numerical solution of the groundwater

stage distribution for the same reasons. Since all hydrologic processes are modeled independently within one time step, the order of calculation was chosen to handle the most transient phenomenon first.

Conceptually, the SFWMM model can be separated into two major components, the physical or hydrologic model, and the system management model. The hydrologic segment assimilates all the descriptive data concerning the physical system and the time series hydrologic data that drives it. Using a series of mathematical approximations for the processes involved, the water table position at every node in the system is computed. Evapotranspiration at each node and discharge through structures are also estimated by the hydrologic model. This segment is the heart of the model and it is this portion that was tested, adjusted, and verified during the calibration and verification simulations. It is capable of running alone provided the daily discharge rates at the major structures are supplied as input data to the model. For calibration and verification purposes, specifying the daily flow rates was not a problem since historical discharges must be used and these were available for all the major structures in the system. Providing daily flow values becomes a problem when analyzing modifications to the system or changes in the historical operating rules.

The system management routines allow the model to compute daily discharge at major structures. The management model scans the hydrologic conditions throughout the system at the end of each day of simulation. It then computes the next day's flow rate for the structures being managed based on the stage discharge relationship and the specified operational policy for each structure.

a. Physical System Definition. The physical system modeled is defined by the appropriate value for all relevant variables at each node. A series of variables is also defined for each canal in the model area. Initializing these variables is the first task of the computer program. The following variables are defined at each node point; land surface elevation (feet, NGVD), initial stage (feet, NGVD), aquifer thickness (feet), aquifer permeability (feet/day), land use type, surface water flow basin identifier, and rainfall basin identifier. Each canal is defined for the following variables: width (feet), regulation stage (feet, NGVD) (can be fixed or vary seasonally or monthly), hydraulic connectivity coefficient (feet/day/feet of head difference between the canal and aquifer), gate width of outflow structure, canal number receiving outflow, overland flow basin identifier, and the location of each node over which the canal passes.

Other run definition data include the starting year and the number of years for the simulation; maximum number of rows and columns of nodes and the node spacing in miles; number and nodal location of all wellfields; specified structure flow points; land-use types; aquifer storage coefficients and soil infiltration rates for each rainfall basin; total number of canals; surface water detention depth; Manning's coefficients; and evapotranspiration parameters.

b. Time Series Data. The hydrologic simulation model is essentially a system response predictor in which the system consists of the Biscayne aquifer, the major canal network, the Water Conservation Areas, etc. Response is measured in terms of groundwater and surface water stage throughout the area. The model is driven by typical hydrologic activity such as rainfall, evapotranspiration, open channel flow through major structures, and groundwater withdrawals at wellfields. The most obvious hydrologic input is rainfall. The first thing done by the program each model day is to add the day's rainfall to the POND variable at every node. The POND variable stores the surface water depth at each node for use in the channel flow, overland flow, or groundwater flow subroutines. During calibration, historical rainfall was used. The model area is divided into 15 rainfall basins. Up to ten rain gages were used to compute the daily rainfall values for each basin. All nodes in the model are assigned a basin identifier to indicate which rainfall amount is used at the node.

The model has the capability of estimating structure discharge in canals, or daily discharges can be specified and used as a known input to the model. For calibration, as much daily flow data as was available was read into the model. These flows were either added to or subtracted from the appropriate canal volume at the beginning of each time step.

Pumpage amounts at major wellfields must be supplied. A recharge variable is defined for each node. Wellfield withdrawals are subtracted from recharge at the node where the wellfield is located. Actual pumpage figures were used whenever possible for calibration. These data were only available as monthly totals; therefore, a daily average pumpage rate was used for each month at each well.

c. Static Data. Close attention was paid to the accumulation of static data, which includes land surface elevations, aquifer characteristics, land-use types, and drainage basins. Land surface elevations were obtained from contour maps when the grid was superimposed. Aquifer characteristics were obtained from various USGS sources. For Palm Beach County, the information was obtained from the USGS publication, "Hydraulic Conductivity and Water Quality of the Shallow Aquifer, Palm Beach County, Florida," (USGS Water Resources Investigation 76-119). Aquifer depth in Palm Beach County was based upon the USGS report, "Geologic Data from Test Drilling in Palm Beach County, Florida" (Open file report 76-713). For Dade and Broward Counties, "Biscayne Aquifer, Southeast Florida" (USGS Water Resource Investigation 78-107) was the primary source. In addition, Appel (1973) provides much data on the aquifers along the coasts of Dade and Broward Counties. Aquifer parameters were calibrated only when data was sparse or to provide smooth transition between aquifers. The 1973 land use was provided by the Land Resources Division of the SFWMD. Manning's n coefficients used in the computation of overland flow were taken from a Corps report entitled, "Everglades Gaging Program Progress Report for Period Ending December 31, 1956" (Report No. 6, dated July 1957).

d. Open Channel Flow. The method of computing open channel flow in the model was altered after preparation of the model documentation (Technical Publication 84-3). The canals are defined as continuous channel reaches with flow control structures at the upstream and downstream end, and nowhere in

between. The location of the canals is defined by specifying the model node points which the canal passes, beginning at the upstream end and proceeding in order to the downstream end.

Three computations are performed in calculating canal flow: surface water-canal interaction; ground water-canal interaction; and canal outflow. Canal flows depend on each of these; therefore, alternative procedure is used to estimate stages and discharges.

(1) Surface water-canal interaction. Each canal has an overland flow indicator that determines whether surface flow is allowed into or out of the canal. Canals with levees on both sides are not allowed to interact with surface water in the adjacent grid cells. A surface detention depth is defined for each grid cell below which no overland flow occurs.

The overland flow interaction at each node along a canal reach is calculated by Manning's equation. The volume of flow is a function of the calibrated resistivity (Manning's "n") and the head difference between the nodal stage and the canal stage.

(2) Ground water-canal interaction. The groundwater seepage term at each canal node is determined using the groundwater head at the canal node, and:

a co-efficient related to the channel-aquifer hydraulic connectivity. There is very little field work to use as a guide for the selection of this term; consequently it is used as a calibration variable for this model.

(3) Canal outflow. Since the components of the canal budget are a function of canal stage, an iterative procedure is used to estimate the stage on a daily basis. In this methodology, it is not necessary to reduce the simulated daily inflows and outflows of a canal to maintain stability. Thus, the time step used in the computations in the channel flow routine is the same as that of the model.

All components of a canal water budget that are functions of stage (overland flow, groundwater seepage and downstream outflow) are estimated with the same stage during each iteration. A canal is allowed to interact with the surface and groundwater systems only when the conditions for convergence have been met or when the maximum number of iterations have been performed. The number of iterations is variable and dependent on the following factors which are physically based:

* Rainfall

* The initial guess for canal stage

* The choice of increment of stage for the iteration process.

e. Overland Flow. The three processes modeled in this section of the program and the order in which they are computed are soil infiltration, overland flow, and evapotranspiration.

(1) Soil Infiltration. At the beginning of each time step, the rainfall depth for that day is added to the surface ponding variable, POND, at each node point. A simplified approach to soil infiltration based on a constant infiltration rate, defined for each basin, is then used to determine how much of the surface water could possibly infiltrate. The soil column is conceptualized as being totally saturated below the water table and completely dry above.

(2) Overland Flow. Overland flow is an important phenomenon in the WCA's and in the large undeveloped areas away from the coast. Manning's equation is used for surface flow calculations. A roughness coefficient is defined for each land use (or vegetation) type. For urban and agricultural areas a fixed value is used, (.007 and .05, respectively). At open land nodes, Manning's "n" is estimated by an equation of the form n = A x Hb. A and b were estimated initially based on field studies and adjusted in a calibration process using the model. Typical values for A and b are 0.5 and -.77, respectively. H is the average water depth of the two nodes for which the flow is being calculated.

The overland flow volume is converted to depth in feet over a grid cell and added to ponding at the appropriate node. A check is made to insure that the surface water gradient does not change sign in one time step. If the volume calculated in Manning's equation would reverse the gradient, it is reduced to the amount that would allow the stage to reach equilibrium between the two nodes.

A basin indicator value is assigned to every node in the model. The major surface water areas have unique indicators to associate node points within a particular basin. The overland flow computations are carried out only between nodes whose basin indicator values are equal. Unique basin numbers are assigned to all the major water impoundment areas and to the portion of Dade County west of L-31N and C-111. In the Everglades Agricultural Area, basins are defined to represent the drainage basin for a given canal. A special basin number is available for nodes that represent levees that are not part of the Water Conservation Area system. This specification is given to the nodes along the L-67 Extension and the L-31N.

(3) Evapotranspiration. The ET rate is related to the amount of soil moisture available at a specific time and place in the simulation. The method chosen was the concept of potential ET, PET, along with a linear reduction equation to calculate actual ET based on the depth to the water table. The conceptual model is shown in Plate B-2. PET is the maximum ET that would occur when there are no hydrological or meteorological factors to limit plant growth. PET varies with land use, cover type, and time of year. The model

uses a set of 12 monthly PET values for each land use category. Each land use also has a shallow root zone (SRZ) and deep root zone (DRZ) term associated with it that are used to determine the actual amount of ET that will be used.

f. Groundwater Modeling. The groundwater portion of the model is based on the finite difference approximation of the linearized, two-dimensional, transient, subsurface flow equation for unconfined aquifers. Transmissivity values in the x and y directions, the aquifer storage co-efficient, and the recharge term, which includes wellfield pumpage, groundwater seepage to or from canals, and surface ponding and ET are used in the model.

There are several means of solving the discretized groundwater equation. To minimize computer time and provide stability during computations, the Saul'yev method of solution was chosen (Carnahan, Applied Numerical Methods). This technique has the advantage of being an unconditionally stable, explicit procedure that eliminates the need for iteration within a single time step.

The finite difference groundwater equation is solved from four different directions in four succeeding time steps.

Other than the specified boundary conditions, the recharge term includes the major stimuli that could result in groundwater fluctuations. These include ET, groundwater seepage, either through surface ponding or canal-aquifer interaction, and wellfield pumpage. Wellfield pumping is treated as a negative recharge since water is being removed from the system. The volume removed by pumping is converted to an equivalent depth of groundwater and is subtracted uniformly over the entire grid cell. The model does not compute drawdown at the localized well site.

g. System Management Model. A system management model is incorporated into the South Florida Water Management Model (SFWMM) to allow the computation of discharges at any flow point, except boundary inflows. The system management model is conceptually split into three categories: the Everglades Agricultural Area, Lake Okeechobee, and the remainder of the modeling area. Each of these areas apply different procedures in determining the structure discharges.

(1) Structure Modeling. For each structure, discharges are based upon desired operational criteria. In general, operational criteria specify the conditions under which the structure will be operated. These operating rules have as their basis groundwater-surface water stages. Once the determination has been made that the structure is to be opened, a stage vs. discharge relationship is consulted to determine the discharge at the flowpoint.

The system management model has as its basic assumption that groundwater-surface water stages are indicative of the overall hydrologic conditions and, therefore, can be used to manage the outflows. Historical stages and discharges are used to develop stage vs. discharge correlations

where such data are available. Where possible, the stage-discharge relationships are correlations based upon historical stages and discharges. Modified culvert and gated weir relationships have also been applied using the upstream and downstream groundwater-surface water stages.

Detailed descriptions of the structure operation for the alternative plans are provided in other appendices.

The criteria under which many structures are opened are usually fairly well defined, such as a regulation schedule for the Water Conservation Areas, or water conditions at key locations. For other structures, such as S-39, the historical combination of irrigation demand, water supply, and regulatory releases make the operation rule so pragmatic that calibration becomes difficult. It is also possible that construction has altered a structure so as to make the historical stage vs. flow relationship inaccurate. In such cases,the theoretical and/or design operating criteria are used in the modeling.

h. Lake Okeechobee. The system management model has special provisions to manage Lake Okeechobee. Inflows to the lake are based on historical inflows for the period of record (1969-1982). Outflows are computed for each structure based upon operating criteria used for the various alternatives being investigated. Evaporation from the lake is computed by adjusting the historic measured evaporation to compensate for the change in surface area associated with the alternative being modeled. Because much of the Everglades Agricultural Area (EAA) provided water supply from Lake Okeechobee is outside the model area, the volume of agricultural water supply deliveries is adjusted. The total area served by the lake is 48 percent larger than the agricultural area within the model boundary. Therefore, water supply releases made to the EAA are increased by 48 percent.

Regulatory releases from the lake are made to the St. Lucie and Caloosahatchee estuaries and to the WCA's. Water Supply releases are made to the EAA and the WCA's. All transfers from the lake to the WCA's are limited to no more than the maximum capacity of the appropriate agricultural canal based on lake stage and the WCA stage. Regression equations were developed based on historic data to compute the canal capacities under the range of hydrologic conditions for each of the four agricultural canals. These equations were originally prepared for use in a South Florida Regional Routing Model and are documented in Technical Publication 86-3, June 1986, by the South Florida Management District.

i. Everglades Agricultural Area (EAA). Water supply demands and runoff for each basin in the EAA are computed in the AG AREA subroutine. The premise of this subroutine is that water levels in the EAA must remain within a narrow range that fluctuates seasonally. When EAA water levels are above the range, the AG AREA subroutine removes the excess runoff; when water levels are below the range, water supply is introduced to the EAA; and when the water levels are within the range, no action is required for that time step. Water is uniformly added or removed directly to/from groundwater in the AG AREA subroutine.

Routing of the runoff or demands computed in the AG AREA subroutine is performed in the ROUTE subroutine. A series of regression equations were developed by SFWMD (Technical Publication 86-3) to determine the relationship of necessary back pumping to Lake Okeechobee versus total runoff or the Lake Okeechobee Operating Permit. These equations are utilized in the ROUTE subroutine to estimate the necessary backpumping for flood protection based on runoff computed in the AG AREA subroutine. The equations were developed assuming that the Lake Okeechobee Operating Permit was in use.

j. Model Calibration. As much historic data as possible was gathered for rainfall, discharge, and wellfield pumpage. Rain data were obtained for Corps, SFWMD, and U.S. Weather Service gages. Discharge data were obtained from USGS (Water Resources Data for Florida, Volume 2: South Florida) and the SFWMD data base. Wellfield pumpage data were obtained from the local utilities. The period 1969-1975 was used for calibration and the period 1976-1982 was used for verification.

The two major hydrologic unknowns to be calibrated were evapotranspiration rates and friction losses during overland flow. To a much lesser extent, the canal-aquifer connectivity coefficient (CHHC) and the gate width (GWDTH) for the hypothetical weir were also calibrated. As a starting point for agricultural ET, the values from "Supplemental Water Use in the Everglades Agricultural Area", Technical Publication 74-4, South Florida Water Management District, June 1974) were used. All ET rates were essentially calibrated, as were root zone depths.

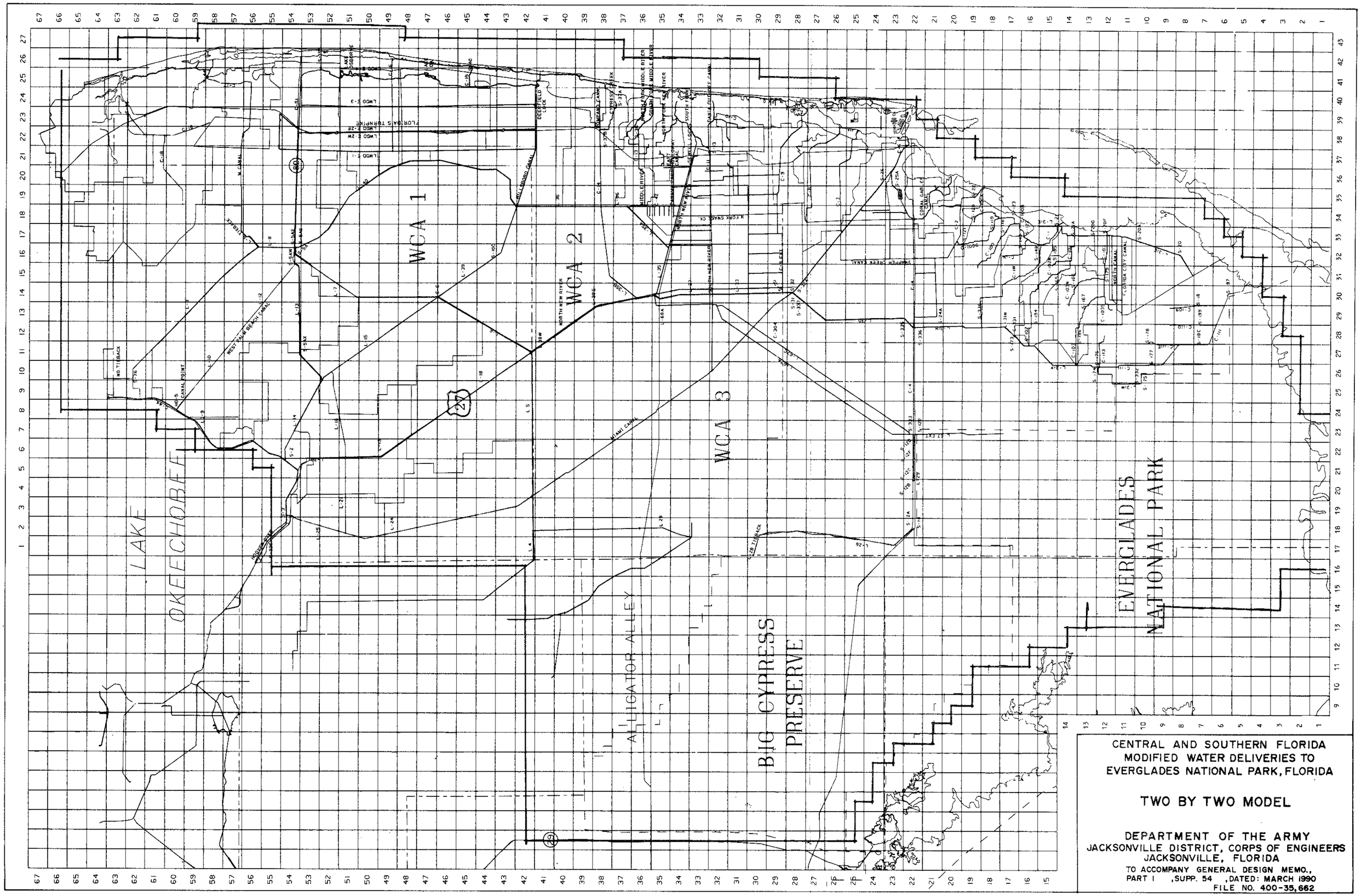
The runoff and water supply needs of the EEA area were calibrated on the assumption that water levels must be maintained within a fairly narrow range. The width and seasonal variation in the allowable range of water levels was adjusted as a calibration factor in an attempt to match historic volumes of water removed from and supplied to the EAA.

The most emphasis was placed on the model being able to accurately predict groundwater-surface water stage. The model was able to compute water levels that matched the historic data quite well in the WCA's. Calibration was acceptable in Palm Beach and Broward Counties although the model had difficulty in matching stages in areas with many small canals that regulate the phreatic surface. In Dade County, the model calibration was good to excellent although in some areas the model slightly over estimates peak stages. Model calibration in ENP and Big Cypress is also quite good. Detailed presentations of the calibration and verification results are provided in Technical Publication 84-3.

k. 1 x 1 Model. A smaller, more detailed version of the SFWMM has been developed by SFWMD. This model computes daily water levels in one-square mile grid cells (vs the four square mile grid cells used in the SFWMM) covering the area south of Tamiami Trail. Other than the grid cell size and the model area, the 1 x 1 Model is identical to the SFWMM. It is used in this report to compute more detailed hydrologic data in the East Everglades for evaluation of potential impacts on developed areas. Boundary conditions along Tamiami Trail for the 1 x 1 Model are computed as an output file of the SFWMM. The Corps has made several modifications to this model. Actual structural operations

for the L-31N canal system were incorporated; a routine for computing laminar and turbulent seepage flow was developed; transmissivity values in the East Everglades were adjusted based on pump tests performed for this report, ground elevations in the developed portion of the East Everglades were adjusted to reflect topographic survey results obtained in this study; and aquifer depths were corrected. The 1 x 1 Model was calibrated using the 1969-1975 period and was verified using the 1976-1982 period. Calibration was good in the East Everglades.

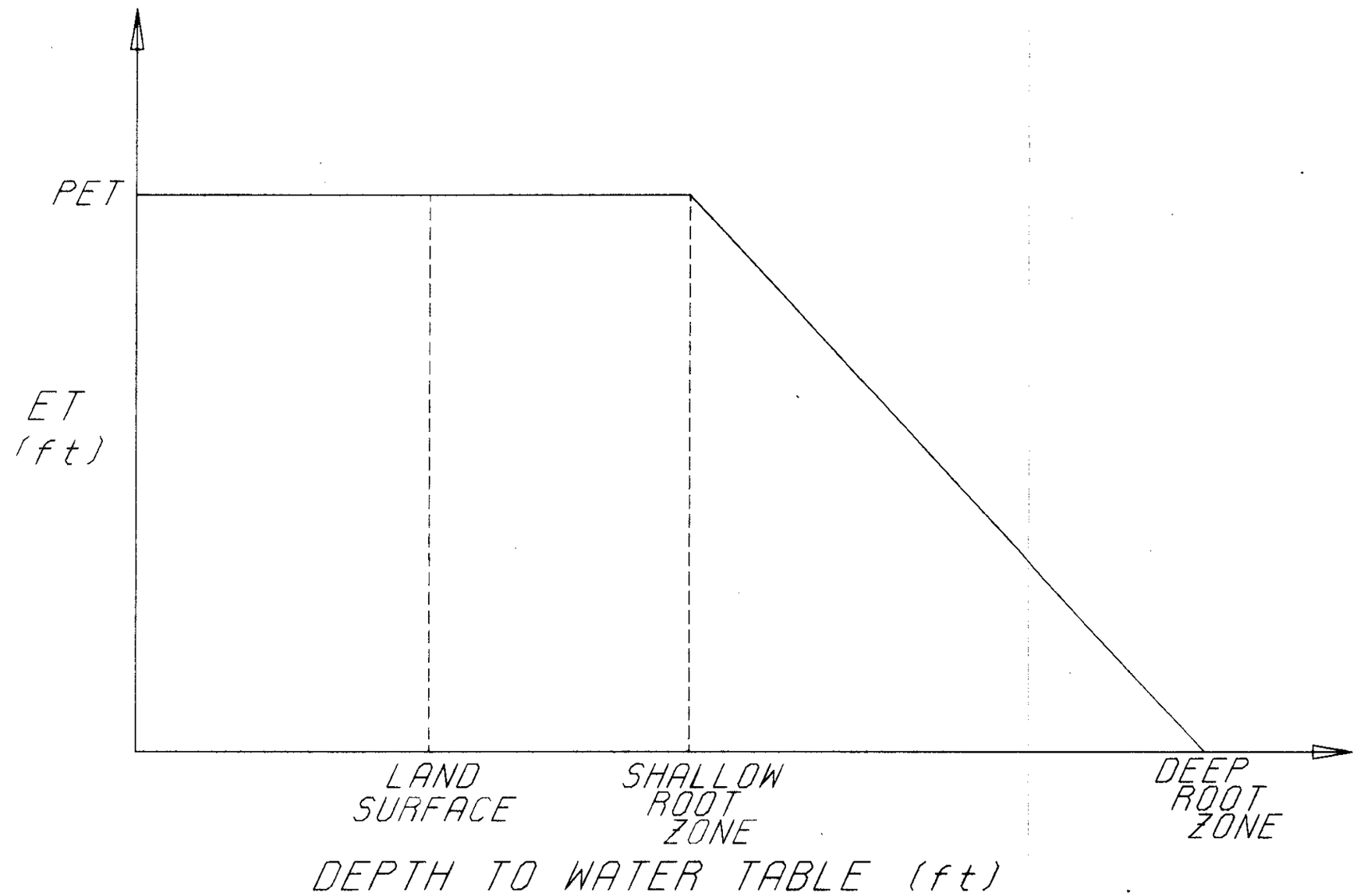
1. HEC-1 Modeling of Interior Drainage. The 1 x 1 Model, although used for establishing boundary conditions, is not refined enough to satisfactorily estimate peak runoff rates from small drainage areas in the flood protection plans considered for the developed East Everglades. The Corps' HEC-1 Flood Hydrograph program was used to compute local runoff within the protected area. It was also used to check whether the addition of flood protection levees caused worse flooding during an extreme event by restricting the escape of runoff into NESRS.



CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK, FLORIDA

TWO BY TWO MODEL

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I ,SUPP. 54 ,DATED: MARCH 1990
FILE NO. 400-35,662



CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK,
FLORIDA

SOUTH FLORIDA WATER
MANAGEMENT MODEL:

ET FUNCTION

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMORANDUM
PART I, SUPPLEMENT 54 DATED MARCH 1990
FILE NO. 400-35,662

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX C

NOT USED

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX D

HYDROLOGIC ANALYSIS OF THE EAST EVERGLADES

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
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APPENDIX D

HYDROLOGIC ANALYSIS OF EAST EVERGLADES

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APPENDIX D

HYDROLOGIC ANALYSIS OF EAST EVERGLADES

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APPENDIX D

HYDROLOGIC ANALYSIS OF EAST EVERGLADES

A. INTRODUCTION

1. General. This appendix specifically addresses the hydrologic modeling efforts and levee design protecting the Rocky Glades area of the East Everglades from the additional flooding which would result from modified water deliveries to Everglades National Park (ENP) via Northeast Shark River Slough (NESRS).

B. BASIN DESCRIPTION

2. General. The East Everglades refers to that area of the Everglades Basin lying between the eastern Park boundary and L-31N and C-111 (Figure D-1). The northern third of the East Everglades lies in Shark River Slough otherwise known as Northeast Shark River Slough which is still undeveloped wetlands.

Shark River Slough constitutes the final 40 miles of the Everglades before it discharges into Florida Bay. The Everglades itself historically was a sheet of very shallow water 40 to 50 miles wide and 100 miles long which slowly filtered southward from Lake Okeechobee to Florida Bay through a sea of sawgrass. An extensive network of canals, levees, control structures and water conservation areas now control the runoff through this area for flood control, water supply, and preservation of fish and wildlife resource benefits. NESRS is extremely flat and is generally between 6 and 7 feet in elevation. Most of the slough has a shallow overlay of organic soil on a porous solution-ridden limerock formation. The vegetation is predominantly sawgrass. There are no defined channels and the area is under water most of the year. Rainfall accounts for most of the accumulated surface water. Prior to the initiation of the Experimental Program of Modified Water Deliveries to ENP, there were no major overland inflows from the north into the East Everglades. Some inflows were the result of seepage from WCA No. 3B through L-29. At times when regulatory releases were made from WCA No. 3A through S-12A, B, C, and D, water would flow around the southern end of L-67 Extension in a northeasterly direction into the East Everglades. Through the middle of the East Everglades is a band of slightly higher rocky terrain known as Rocky Glades (Figure D-2). The eastern part adjacent to L-31N has some low density residential development which would be affected by the proposed new diversions to NESRS. An area approximately 11 sq. mi. in size containing scattered homes is being considered for protection from increased flooding brought on by the diversion plan. In addition, potential impacts to about 6.5 square miles of adjacent agricultural land just to the south in Taylor Slough watershed have been assessed.

3. Climate. The south Florida region has a sub-tropical climate noted for its mild, dry winters and hot, humid summers. The East Everglades receives an average of 54-60 inches of rainfall a year. Approximately 70 percent of the annual rainfall occurs during the May-October rainy season.

4. Soils. Three types of soils predominate in the East Everglades area (Figure D-2). The northern third, Shark River Slough portion is overlain with Everglades Gandy and Loxahatchee peats. The middle third is actually exposed limestone of the Rockland association with little or no soil. The residential and agricultural areas considered for protection fall within this Rockland soils group. A narrow strip of Perrine-Ochopee association loamy marl runs north-south through the residential area and the northeast corner of the agricultural area. The Perrine marls are composed primarily of unconsolidated, finely divided, highly calcareous sediments. In some places layers of organic material 6 to 30 inches thick occur between the loamy layers.

Infiltration rates are extremely high for peat and rockland soil types when in an unsaturated condition. Once saturated, infiltration losses cease, and the primary loss of water is through evapotranspiration. The loamy marl is only moderately permeable and intense rainfall during thunderstorms can exceed the infiltration rate of the soil.

C. HISTORICAL FLOW - SHARK RIVER SLOUGH

5. Annual Runoff. The U.S. Geological Survey began measurements of flow across Tamiami Trail in late 1939. By then many drainage canals draining to the East Coast had been constructed across the Everglades to intercept the southward movement of water from Lake Okeechobee through the Everglades National Park. To what extent this reduced the flow southward is not accurately known, but Appendix B of Part I, Supplement 33, [4][1], stated it was doubtful that past depths of flooding exceeded that of the 1947 flood, although duration of flooding was longer. The construction of the east coast protection levee in 1952, which consists of a north-south chain of levees, L-30 through L-40, and the construction of WCA No. 3 in 1962 have each had a great impact on the distribution and quantity of flow to Shark River Slough. Table D-1 presents three 11-year periods of discharge record across the Tamiami Trail, which compare discharge characteristics before, between, and

[1] Numbers in brackets [4] refer to references given at the end of this appendix.

TABLE D-1

COMPARISON OF TAMIAMI TRAIL DISCHARGES FOR THREE 11-YEAR PERIODS

Date	L-30 to L-67A (ac-ft)	L-67A to 40-Mile (ac-ft)	Total (ac-ft)	WCA #3A Precipitation (in)
Before Construction of L-30 thru L-40				
1940	46,420	60,460	106,880	54.5
1941	83,660	87,180	170,840	61.2
1942	40,420	30,010	70,430	51.0
1943	22,930	41,220	64,150	48.2
1944	6,400	11,620	18,020	43.7
1945	77,030	86,260	163,290	51.5
1946	21,970	45,260	67,230	48.5
1947	496,960	344,890	841,850	75.0
1948	373,620	205,060	578,680	57.5
1949	98,780	116,650	215,430	53.6
1950	28,610	60,230	88,840	46.7
Total	1,296,800	1,088,840	2,385,640	591.4
Average	(163 cfs)	(137 cfs)	(300 cfs)	53.76in/yr
After Construction of L-30 thru L-40				
1952	123,880	94,230	218,110	52.9
1953	319,010	190,850	509,860	63.8
1954	547,160	190,940	738,100	55.7
1955	174,470	40,900	215,370	45.0
1956	44,180	31,090	75,270	41.0
1957	359,640	118,700	478,340	63.0
1958	740,080	232,610	972,690	57.1
1959	757,080	307,640	1,064,720	69.0
1960	807,080	379,740	1,186,820	62.9
1961	131,950	44,190	176,140	37.9
1962	40,840	24,420	65,260	50.1
Total	4,045,370	1,655,310	5,701,380	598.4
Average	(509 cfs)	(208 cfs)	(717 cfs)	54.40in/yr
After Construction of WCA #3A and L-29				
1963	39,050	0	39,050	47.02
1964	42,640	4,970	47,610	50.21
1965	47,590	36,940	84,530	42.38
1966	92,020	1,006,460	1,098,480	48.70
1967	53,240	181,380	234,620	46.86
1968	126,980	1,011,400	1,138,380	52.22
1969	123,000	1,764,700	1,887,700	63.92
1970	101,830	1,240,510	1,342,340	45.53
1971	14,440	239,430	253,870	42.11
1972	36,830	312,550	349,380	42.15
1973	32,570	270,380	302,950	37.58
Total	710,190	6,068,720	6,778,910	518.68
Average	(89 cfs)	(763 cfs)	(852 cfs)	47.15in/yr

after major structural controls were built. Construction of the east coast protection levee diverted most of the runoff southward across Tamiami Trail resulting in roughly a doubling of the flow into Shark River Slough. The volume of flow crossing the Tamiami Trail between L-30 and L-67A actually tripled while the flow from L-67A to 40-Mile Bend increased by 52 percent. In 1962, WCA No. 3A and levee L-29 were completed which severely limited flow across the 11-mile stretch of Tamiami Trail between L-30 and L-67A.

Construction of L-67 Ext. in 1967 further reduced flows into the area. From 1962 to initiation of the Experimental Program releases from WCA No. 3 into Shark River Slough passed only through the four S-12 spillways between L-67A and 40-Mile Bend. The overall effect has been to increase total flows southward. Despite the fact that average rainfall was less during the 1963-1973 than for 1952-1962, the volume of flow south was greater mostly due to large transfers of water from Lake Okeechobee between 1966 and 1969. Of more importance, the distribution of the flow across Tamiami Trail from 40-Mile Bend to L-67A versus from L-67A to L-30 has been altered from roughly a 50-50 ratio to a 90-10 ratio.

6. Monthly Flow Regulation. A major function of WCA No. 3A is to maintain adequate water supply storage for water deliveries to Everglades National Park as well as to meet municipal and agricultural demands in south Dade County. Controlling water levels to avoid overtopping the containing levees during a major flood event is another important project purpose. To assure optimum availability of water, the regulation schedule rises from 9.5 feet, NGVD, at the onset of the hurricane season to 10.5 feet, NGVD, at the beginning of the winter dry season. Federal Law (PL 91-282) specifies minimum deliveries of 315,000 acre-feet annually to the park except during water shortages when it will receive 16.5 percent of total deliveries from the project. The minimum monthly delivery schedule is given in Table D-2. When WCA No. 3A water levels rise above regulation schedule, flood releases must be made through structures S-12A through D.

7. Floods of Record in Shark River Slough. The maximum flood stages ever observed in south Florida occurred in October 1947 following the passage of the third hurricane of the season. The year 1947 was also the wettest year on record with annual precipitation nearly 40 percent above normal. Total annual rainfall was 82.76 inches at the Tamiami Trail 40-Mile Bend gage and 94.07 inches at the Homestead Agricultural Research Station gage. The 1947 area-wide average May-October rainfall of 73.54-inch is estimated to have a recurrence frequency of about once in 35 years. Flood stages were reported to be 1 foot higher than the previous worst flood of 1929. The year 1948 also received above normal rainfall and being preceded by the 1947 flood, produced flood stages almost as great as 1947. Table D-3 lists some of the high water marks observed in the study area for the 1947 and 1948 floods. Other notable floods were the 1959-60 and 1968-69 floods.

The completion of the east coast protection levee in 1952 and subsequent works altered the flow distribution making comparison with more recent flood stages difficult. The 1947 high water marks still have not been exceeded south of the Tamiami Trail even though a greater percentage of flow goes south instead of east.

TABLE D-2

MONTHLY FLOW DISTRIBUTION TO EVERGLADES NATIONAL PARK

Month	Percent of Annual Flow	Monthly Volume (Acre-Feet) Shark River Slough	Taylor Slough	Eastern Panhandle
Jan	7.33	22,000	740	360
Feb	3.03	9,000	370	180
Mar	1.36	4,000	185	90
Apr	.63	1,700	185	90
May	.71	1,700	370	180
Jun	4.73	5,000	6,660	3,240
Jul	5.84	7,400	7,400	3,600
Aug	5.27	12,200	2,960	1,440
Sep	15.17	39,000	5,920	2,880
Oct	24.94	67,000	7,770	3,780
Nov	20.48	59,000	3,700	1,800
Dec	10.51	32,000	740	360
Total	100.0	260,000	37,000	18,000
		Average Daily Flow (cfs)		
Jan		358	12	5.9
Feb		162	6	3.2
Mar		65	3	1.5
Apr		29	3	1.5
May		28	6	2.9
Jun		84	112	54.4
Jul		120	120	58.5
Aug		198	48	23.4
Sep		655	99	48.4
Oct		1,090	126	61.5
Nov		992	62	30.2
Dec		520	12	5.9

NOTE: The Taylor Slough flow distribution shown is the revised distribution adopted by the S-332 agreement and permit dated 1 November 1976.

TABLE D-3

1947 and 1948 High Water Marks

Location	Peak Stage (ft. NGVD) 1947	1948
Tamiami Canal at 40-Mile Bend	10.3	10.0
Tamiami Canal at Krome Avenue	9.46	8.53
Tamiami Canal downstream Dade-Broward Levee	8.99	7.76
Tamiami Canal upstream Dade-Broward Levee	---	9.22
L-31N at 168th Street	9.0	8.7

D. ASSESSMENT OF AVAILABLE DATA

8. Stage and Discharge Records. Records of flow to the Everglades National Park have been measured across the Tamiami Trail since November 1939. From 1939 to September 1963 only monthly discharges were published. Beginning in October 1963 daily flows have been published by reaches. Figure D-3 shows these and other surface and groundwater recording stations. The Everglades National Park operates other gaging stations that are not shown. Table D-4 lists selected station numbers and names (or well numbers) shown on Figure D-3 which were used in calibrating the hydrologic model.

For the eastern Rocky Glades area, Well G-596 was used as an indicator of before and after conditions within the protected area. This gaging station was established in July 1949 following two years of record flooding. The land surface datum is 7.28 feet, NGVD. The highest daily water level was 8.30 feet on November 2, 1960. The lowest was 0.56 feet on May 14, 1971.

Well G-1502 located at Chekika State Park was used to compare the impact of the Shark River Slough diversion and the Rocky Glades protective works along the outside of the protective levees. This recording station was established in May 1970. The land surface datum is 8.28 feet, NGVD. The highest recorded stage was 7.89 feet on August 18, 1981. The lowest was 0.49 feet on May 14, 1971.

TABLE D-4

SELECTED WATER-LEVEL GAGING STATIONS
FOR EVERGLADES AREA

Map (1) Index No.	Station Name or Well Number
1	Tamiami Canal Outlets, Levee 67A to 40-Mile Bend
2	Tamiami Canal below S-12C
3	Tamiami Canal Outlets, Levee 30 to Levee 67A
6	Taylor Slough near Homestead (TSHO)
7	Taylor Slough at Royal Palm (TSRP)
10	Everglades P-33
11	Everglades P-38
12	Levee 67 Extended Canal (L-67 Ext)
13	Everglades P-36
15	Everglades P-35
20	Everglades P-34
21	Taylor Slough at Context Rd.
23	Northeast Shark River Slough No. 1
24	Northeast Shark River Slough No. 2
31	G-613
42	G-196A
51	G-596
56	G-620
57	G-1502
58	G-861

(1) Station locations shown in Figure D-3.

9. Rainfall. Rainfall in East Everglades is seasonal with the rainy season occurring from May through October and the dry season running from November through April. Seventy percent of the annual rainfall falls during the 6-month rainy season. Rainfall during the dry season is mostly due to frontal systems passing through the area and is usually light. During the wet season, rainfall is mostly due to convective storms and tropical weather disturbances and frequently exceeds 1 inch. When tropical storms or hurricanes move into the area rainfall often exceeds 5 inches in a few days. May and October are transitional months and may be wet or dry.

Table D-5 lists annual rainfalls at four gaging stations near Shark River Slough. The annual rainfall for the Homestead station ranged from 37 inches in 1971 to 94 inches in 1947. Exceptional flooding occurred following consecutive years of above normal rainfall in 1958-60 and 1968-69. The worst drought occurred during 1961-65. Virtually all of the most damaging storms that have occurred in south Florida have been associated with tropical cyclones. In fact, some of the world's heaviest recorded rainfalls have occurred in connection with hurricanes. It should be noted that upper level

low pressure systems combined with a front can also produce large rainfall amounts during any season of the year, including the winter agricultural growing season. For example, this type of weather disturbance produced a total of 12.85 inches of rain during the four-day period from 24 through 27 April 1979 at Homestead. Another example would be the 4.10 inches of rain that occurred at Homestead on 6 November 1959. Large rain events during the winter growing season would probably be associated with this type of system.

Alaka [1] found that more tropical cyclones occur in the north Atlantic Ocean during September than any other month. About 34% of such storms in the north Atlantic Ocean from 1886 through 1980 occurred in September. October is ranked second to September with about 22%. However, tropical cyclones are most likely to affect south Florida during the month of October. There is a sharp decrease in the frequency of tropical storm formation in late October.

The most intensive agricultural activity occurs during the November-May dry season when the demand for winter fruits and vegetables is greatest and the threat of damaging flooding or intense summer heat has largely passed. This period is not without flood risks, however, since late summer floods in Shark River Slough recede very slowly, and water levels may take months to return to non-damaging stages.

Most of the large rainfall events that occur during the winter growing season occur during October and are associated with tropical weather disturbances. The recorded maximum 10-day rainfall totals during the winter growing seasons of the 57 years of record at the Homestead Experiment Station were examined. It was found that all but two of the largest rainfall totals were associated with tropical cyclones. Therefore, growing season storms with recurrence intervals greater than about 5-years are likely to be the result of tropical weather disturbances. Furthermore, the 27 largest 10-day rainfall totals recorded during the growing season were mostly or entirely within October.

TABLE D-5

ANNUAL RAINFALL AT FOUR STATIONS NEAR SHARK RIVER SLOUGH
(Rainfall in Inches)

Year	Homestead Experiment Station	Miami International Airport	Royal Ranger Station	Tamiami Trail at 40-Mile Bend	Average
1952	58.24	48.94	48.75	56.54	53.12
1953	63.99	65.15	54.76	63.49	61.85
1954	67.76	62.26	64.10	49.99	61.03
1955	52.59	41.35	47.30	46.13	46.84
1956	45.37	37.00	28.72	42.97	38.52
1957	63.22	70.87	48.26	68.22	62.64
1958	73.00	71.92	70.46	59.33	68.68
1959	87.09	89.33	76.92	67.18	80.13
1960	82.12	70.26	70.52	73.91	74.20
1961	45.75	41.70	36.52	44.05	42.00
1962	55.56	42.27	51.16	56.06	51.26
1963	62.65	46.08	53.57	62.41	56.18
1964	61.09	60.20	62.78	55.28	59.84
1965	46.30	58.40	49.86	49.49	51.01
1966	59.99	82.06	73.00	61.46	69.13
1967	54.15	66.22	55.97	53.52	57.46
1968	83.67	83.39	85.26	66.84	79.79
1969	83.72	71.49	83.75	75.11	78.52
1970	45.89	44.72	46.13	47.46	46.05
1971	37.20	50.72	37.28	38.42	40.90
1972	64.03	63.11	59.66	51.49	59.57
1973	57.91	53.24	53.13	47.51	52.95
1974	43.43	49.00	48.33	49.55	47.58
1975	51.39	39.10	53.15	39.44	45.77
1976	71.72	55.90	57.66	55.96	60.31
1977	63.32	64.95	58.07	44.16	57.62
1978	64.77	45.83	74.35	56.01	60.24
Average	60.96	58.35	57.39	54.89	57.90
High	87.09	89.33	85.26	75.11	80.13
Low	37.20	37.00	28.72	38.42	38.52

Table D-6 lists the number of various one-day and five-day storm events observed during each month of the year over a 64 year period. It can be seen that two peak periods of more frequent intense rainfall events occur during the year. These peaks are during May - June and September - October. One-day rainfalls greater than 3.0 inches have occurred most frequently during September, with October ranking second. Five-day rainfall totals greater than 5.0 inches have been recorded most frequently during May and June. A second period of slightly less frequent five-day rainfall totals greater than 5.0 inches occurs in September and October. However, flood risks are greater because it occurs at the end of the rainy season while soil conditions are more likely to be wet.

TABLE D-6

STORM CATEGORIES
HOMESTEAD EXPERIMENT STATION

Number of Days with Rainfall Greater or Equal to Values Shown During Period of Record (1914 - 1977)

MONTH	3"	4"	5"	6"	7"	8"
JAN	-	-	-	-	-	-
FEB	-	-	-	-	-	-
MAR	3	2	-	-	-	-
APR	6	3	2	1	-	-
MAY	15	8	4	2	1	-
JUN	21	13	7	4	-	-
JUL	7	1	-	-	-	-
AUG	4	2	-	-	-	-
SEP	25	10	3	2	2	2
OCT	22	10	8	1	1	1
NOV	2	1	1	-	-	-
DEC	-	0	-	-	-	-

Number of 5-Day Total Rainfall Depths Greater Than or Equal to Values Shown (1914 - 1977)

MONTH	5"	6"	7"	8"	9"	10"	11"
JAN	-	-	-	-	-	-	-
FEB	-	-	-	-	-	-	-
MAR	2	-	-	-	-	-	-
APR	5	3	2	1	-	-	-
MAY	24	15	13	8	7	7	4
JUN	27	14	8	7	5	3	1
JUL	11	1	1	1	1	-	-
AUG	7	2	1	-	-	-	-
SEP	22	15	11	6	4	2	1
OCT	22	14	5	3	3	2	1
NOV	2	1	-	-	-	-	-
DEC	-	-	-	-	-	-	-

10. Pan Evaporation. Pan evaporation records, used to estimate evapotranspiration rates, are available at the Tamiami Trail near 40-Mile Bend gage and at the Hialeah gage. The average monthly pan evaporation for these two stations for the period January 1941 through March 1979 is presented in Table D-7.

TABLE D-7

PAN EVAPORATION
SHARK RIVER SLOUGH

Month	Inches	Month	Inches
Jan	3.52	Jul	6.73
Feb	4.11	Aug	6.58
Mar	5.72	Sep	5.69
Apr	6.73	Oct	5.32
May	7.22	Nov	4.10
Jun	6.56	Dec	3.44
		Annual Total	65.72

11. Geology and Groundwater Movement. Dade County is underlain by a very porous limestone known as the Biscayne aquifer. The Biscayne aquifer is one the most permeable in the world and poorly suited for storing water above ground because of high seepage losses. The base of the aquifer slopes west to east from about sea level along the east edge of Big Cypress Swamp to 50-60 feet below sea level along L-30 and L-31 to 130 feet below sea level along Biscayne Bay as shown in Figure D-4. The depth of aquifer contours are based on core borings for L-67 Extension, L-31N, L-29, S-18C, and the two wells at which pumping tests were conducted for this study. Most of the limestone is either exposed at the surface or thinly mantled, generally less than 2 inches, with organic calcareous silt. The upper 3 to 4 feet of limestone is somewhat oolitic and has been extensively attacked by solution.

Since 1962, the flow across the Tamiami Trail between L-30 and L-67A has been from seepage along L-29. The observed annual flows have ranged from 14,443 acre-feet in 1971 to 126,980 in 1968. Seepage from WCA No. 3B to the east beneath L-30, L-33, and L-37 can be 4 or 5 times greater than this due to greater aquifer depth and steeper hydraulic gradient. Prediction of seepage rates beneath levees were based on the results of pumping tests conducted along Levee 29, Section 3, as described in Part I, Supplement 7 Design Memorandum [3].

The direction of groundwater movement is reflected in the water level contours on Figures D-5 and D-6 representing a wet and dry condition, respectively. Groundwater flow in the Rocky Glades area generally is to the south and southeast towards L-31N. Only during a ground-saturating rainfall would groundwater and surface water move briefly in the direction of Shark River Slough. Afterwards, groundwater movement in the southeasterly direction would soon re-establish itself.

E. STAGE AND DISCHARGE FREQUENCIES

12. General. The stage frequency curve for Well G-596, located in the Rocky Glades residential area, is shown in Figure D-7. Although located at a higher ground elevation of 7.28 feet, water levels at G-596 are slightly lower than those experienced in NESRS where average ground level is 6.0 feet. The stage frequency curve is divided into two curves, the period prior to construction of WCA No. 3 and L-29 in 1962, and the period after completion of these works. The record 1947 and 1948 floods are also plotted.

As expected, the construction of L-29 and the resulting ability to store excess summer rainfall in WCA No. 3 has greatly reduced stages and the hydroperiods in NESRS. The fact that flood releases are made several miles west of the natural water course (NESRS) and contained inside the ENP boundary for about 10.5 miles south of Tamiami Trail by L-67 Ext. also contributes to the drop in stages in NESRS. The difference becomes less as the size of the flood event increases because the impacts of direct rainfall increase and WCA No. 3A is forced to make flood releases.

It is not correct to say that stages for the 1952-61 period reflect historic conditions. Following the disastrous floods of 1947-48, the east coast protection levees, L-30 and L-31, were built. Under current conditions, these levees retard the eastward movement of flood waters and increase flood levels in NESRS. The 1947 and 1948 floods would have been higher in NESRS had the east coast protection levees been in place.

F. RAINFALL FREQUENCY

13. General. A frequency analysis was performed of peak annual rainfall in the project area for durations of one through 20 days. A Log-Pearson Type III distribution was utilized to compute rainfall frequencies. Procedures prescribed in "Guidelines for Determining Flood Flow Frequency", Bulletin No. 17B of the Hydrology Committee, United States Water Resources Council, Revised September 1981, were used in this analysis.

14. Data. The Homestead Experiment Station, with a 64-year period of record is the only long-term record rain gage near the project area and was used in this analysis. Another rain gage, Tamiami Trail at 40 Mile Bend, with a 39-year period of record, was used for comparison. Periods of missing data at each gage were checked to determine if a large storm event caused a gage failure. No such cases were identified. Therefore, years with significant periods of missing data were eliminated from the analysis. The maximum cumulative rainfall totals in each year for periods of from one to 20 consecutive days were used for the annual rainfall analysis. In addition, the 184-day May-October rainy season rainfall totals were obtained for Northeast Shark River Slough from rainfall frequency maps published in SFWMD's Tech. Pub. 81-3 [1].

15. Skew Factors. A regional skew analysis was conducted for south Florida. The results of the analysis were used to select skew factors for the rain gages. It was found that skew factors approach zero as the duration increases. Table D-8 shows the skew factors used at the gage for each duration.

TABLE D-8

SKEW FACTORS FOR ROCKY GLADES
RAINFALL ANALYSIS

Gage	Duration (days)					
	1	2	5	10	20	>30
Homestead	.3	.1	.1	.1	.1	0

16. Rainfall Frequency. The results of the annual rainfall frequency computations are shown in Figure D-8.

17. Design Storms. Two sets of storm data were developed. One was for evaluation of interior drainage in the residential and agricultural protection alternatives using HEC-1. The other was to evaluate the impact of the proposed diversions to NESRS using SFWMD's model and to compare the effectiveness of several proposed protective measures for the developed East Everglades areas. For the interior drainage evaluation, a 3-day storm event was selected using data derived for the Homestead gage. Point rainfall was used because the drainage areas were small enough to preclude the need for areal adjustments. The storm was patterned so the maximum rainfall occurred on the third day. Hourly rainfall for the maximum 24-hour rainfall amount was patterned after the SCS Type II storm typical of summer thunderstorms. Table D-9 lists the 3-day rainfall pattern for the 25-year storm. The 2-, 10-, 50-, and 100-year storms were computed using ratios of the 25-year storm. The ratios were 0.491, 0.799, 1.148, and 1.30, respectively. In like manner, the Standard Project Storm (SPS) was developed for a 10 square-mile area using 50 percent of the Probable Maximum Precipitation as given in National Oceanic and Atmospheric Administration publication, Hydrometeorological Report No. 51 [15]. The SPS is defined as the most critical combination of precipitation, minimum infiltration losses and concentration of runoff that is considered "reasonably characteristic" of the region, excluding extraordinarily rare combinations. Table D-10 presents the SPS.

TABLE D-9

25-YEAR* 3-DAY STORM
HOMESTEAD EXPERIMENTAL STATION

Hour	Day 1	Day 2	Day 3
1	.04	.09	.08
2	.04	.09	.08
3	.04	.09	.10
4	.04	.09	.10
5	.04	.09	.12
6	.04	.09	.13
7	.04	.09	.17
8	.04	.09	.21
9	.04	.09	.27
10	.04	.09	.37
11	.04	.09	.58
12	.04	.09	3.38
13	.04	.09	.91
14	.04	.09	.45
15	.04	.09	.33
16	.04	.10	.23
17	.04	.10	.19
18	.04	.10	.17
19	.04	.10	.13
20	.04	.10	.11
21	.05	.10	.10
22	.05	.10	.09
23	.05	.10	.08
24	.05	.10	.07
	1.00	2.25	8.45

*2-, 10-, 50-, and 100-year storms are 0.491, 0.799, 1.148, and 1.30 times the 25-year storm depth, respectively.

TABLE D-10

3-DAY STANDARD PROJECT STORM
ROCKY GLADES

Hour	Day 1	Day 2	Day 3
1	.05	.10	.20
2	.10	.05	.25
3	.10	.10	.30
4	.05	.10	.30
5	.10	.05	.30
6	.05	.10	.35
7	.10	.10	.40
8	.10	.10	.50
9	.05	.10	.70
10	.10	.10	.90
11	.05	.10	1.30
12	.10	.15	2.80
13	.10	.15	8.00
14	.05	.10	2.10
15	.10	.15	.90
16	.05	.15	.80
17	.10	.15	.50
18	.10	.15	.45
19	.10	.15	.40
20	.05	.15	.35
21	.10	.20	.30
22	.10	.20	.30
23	.05	.20	.25
24	.10	.20	.20
	1.95	3.10	22.85

For evaluating impacts outside the flood protected area, a 184-day rainy season storm was used. The 5-day rainfall frequency data for Homestead and the 184-day rainfall frequency data from SFWMD's Tech. Pub. 81-3 [11] were areally adjusted for a 744 sq. mi. area making up the rainfall basin covering the Everglades and East Everglades area. Table D-11 presents the results of this analysis.

The rainfall was temporally patterned after the May-October 1972 rainy season, with the peak 5-day rainfall occurring during the last 5 days of September. The year 1972 was selected because groundwater starting conditions at the end of the dry season (April 30th) were closest to average. Starting water levels in WCA No. 3A at S-12C were 7.66 feet or 0.34 feet below the 2-year frequency stage for April 30th. The 5-day maximum precipitation was placed at the end of September because most hurricanes occur in September and October. Table D-12 lists the monthly totals for the 25-, 100-year, and standard project storms with September broken down into daily amounts. The SPS for long duration storms has traditionally been taken as 125 percent of the 100-year rainfall by the Jacksonville District. This rainfall was applied to all four rainfall basins in the model or all 1566 grid points in the 1x1 model. The net result of this would be a slight overestimate of rainfall in

west Dade County and a slight underestimate in east Dade County. To establish boundary conditions along the north border, the same rainfall was used for WCA No. 3 in the 2x2 model. This is several inches more than would normally be used for the SPS in WCA Nos. 3A and 3B. However, because WCA No. 3A water levels were already 1.4 feet below regulation schedule and since it is customary to assume starting water levels at regulation schedule for the SPF, the rainfall derived for NESRS and applied to WCA No. 3 appears reasonable.

TABLE D-11

MAY-OCTOBER WET SEASON RAINFALL FREQUENCY

	2-Yr.	5-Yr.	10-Yr.	25-Yr.	50-Yr.	100-Yr.
Point Rainfall (1)	44.2	53.5	60.0	69.0	74.0	81.0
Areal Adjustment(2)	0	-1.29	-1.94	-2.57	-3.03	-3.50
Corrected	44.2	52.21	58.06	66.43	70.97	77.5
Smoothed Curve Value	44.2	52.2	58.1	65.8	71.6	77.5

(1) Taken from isohyetal maps published in SFWMD's Tech. Pub. 81-3 [11]. Rainfall point was centered over northeast Shark River Slough.

(2) Point rainfall areal adjustment for a 744 sq. mi. rainfall basin covering most of Everglades National Park and East Everglades area. Areal adjustment was interpolated from Table 5, GDM Part VI Sect. 6 [7].

TABLE D-12

184-DAY DESIGN STORM RAINFALL FREQUENCIES
744 SQUARE MILE EVERGLADES AREA

Month/Day	1972 Rainfall	25-Yr	100-Yr	SPS
May	8.62	11.67	13.62	17.03
Jun	10.78	14.59	17.04	21.30
Jul	5.23	7.08	8.26	10.33
Aug	5.98	8.09	9.45	11.81
Sep 1	.14	.19	.22	.28
2	.29	.39	.46	.57
3	.00	.00	.00	.00
4	.06	.08	.09	.11
5	.22	.30	.35	.44
6	.02	.03	.03	.04
7	.00	.00	.00	.00
8	.29	.39	.46	.58
9	.52	.70	.82	1.02
10	.32	.43	.51	.64
11	.36	.49	.57	.71
12	.31	.42	.49	.61
13	.01	.01	.02	.03
14	.15	.20	.24	.30
15	.27	.36	.43	.54
16	.00	.00	.00	.00
17	.05	.07	.08	.10
18	.02	.03	.03	.04
19	.00	.00	.00	.00
20	.26	.35	.41	.51
21	.47	.64	.74	.93
22	.19	.26	.30	.37
23	.16	.22	.25	.31
24	.64	.87	1.01	1.26
25	.64	.87	1.01	1.26
26	.04	.69	.71	.89
27	.03	.75	.77	.96
28	.01	1.01	.94	1.18
29	.00	2.29	2.75	3.44
30	.00	7.65	9.97	12.46
Sep Total	5.47	19.69	23.66	29.58
Oct	3.46	4.68	5.47	6.83
May-Oct Total	39.54	65.80	77.50	96.88

G. SEASONAL FLOOD OCCURRENCE AND DAMAGE ASSESSMENT

18. General. A study of annual flood peaks at Well G-596 for the period 1962-85 shows that September-October are the peak flood months both in terms of frequency and severity (see Table D-13). Only two floods occurred during the winter growing season when crop damage could be at its highest level. These floods were only about 1-in-3 year events. The February flood can be considered an anomaly since it was preceded by record high November-January stages. Most damaging floods will occur in September-October, the peak season for tropical weather disturbances. Therefore, agricultural damage assessments were based on annual frequency analyses rather than seasonal analyses. Even though all agricultural lands may not be in production at the time of a severe rain event due to high antecedent moisture conditions and foreseeable tropical storms, the damage to the economy still exists as lost production. Therefore, damages were computed for all cultivated lands for all storm events.

TABLE D-13

AVERAGE ANNUAL PEAK STAGE BY MONTH AND SIZE
WELL G-596 (1962 - 1985)

	7.5'-7.9'	7.0'-7.49'	6.5'-6.9'	6.0'-6.49'	5.5'-5.9'	Total
Jan						
Feb		1[1]				1
Mar						
Apr			1			1
May						
Jun		1				1
Jul			1			1
Aug				2		2
Sep	1	3	3	3		10
Oct	1	2	1	3	1	8
Nov						
Dec						
						24

[1] Preceded by record Nov-Jan flood stages. Judged an anomaly.

H. SOUTH FLORIDA WATER MANAGEMENT MODEL (SFWMM)

19. General. The rate and direction of water movement in Dade County is complicated by the many manmade changes to drainage, the extreme flatness of the terrain, and the highly porous surficial limestone aquifer. In the past the best effort attained at predicting water levels for water management purposes was an analog model developed by the U.S. Geological Survey. Recently, the SFWMD developed a digital model of the Lower East Coast under contract to the Corps that greatly simplifies the testing of management alternatives or new structural measures for controlling or delivering water. A second version of the model exists for the area south of Tamiami Canal. This version has 1 square-mile grids (1x1) instead of 4 square-mile grids (2x2). The denser grid network presents a more accurate picture of water levels. Both versions of the model are now in use in the Jacksonville District Corps Office.

20. 2x2 Model. SFWMD's digital surface-ground water model was developed to cover Dade, Broward, and Palm Beach Counties. Big Cypress Swamp in Collier County is now included, also. The model uses a square grid network to model horizontal and vertical movement of water. A two-by-two mile node spacing is used. Surface (overland) flow modeling adds rainfall and surface inflow to beginning storage then subtracts evapotranspiration, infiltration, and surface outflow. One day time steps are used. Evapotranspiration varies with depth of water table below the land surface. Overland flow is computed using Manning's equation with Manning's "n" varying with depth of flow. Infiltration is either at the maximum rate read in at each grid point or the amount available in groundwater storage. Channel flow is calculated by reading in upstream inflow while downstream outflows and seepage are calculated by an iterative process. Outflow is estimated using actual discharge rating characteristics or a simplified weir equation and the head difference at the structure. Operation of the significant water control structures is simulated in the ROUTE subroutine. Groundwater flow is calculated using a two-dimensional flow equation for a porous aquifer. Subroutines are provided in the model to simulate hydrologic responses in the Everglades Agricultural Area and Lake Okeechobee. A detailed documentation of the model is contained in SFWMD's Technical Publication 84-3 [12]. Appendix B of this report also gives a detailed description of the SFWMM model.

The model was calibrated using the period 1969 to 1975. This period was used because it contained both an extremely wet year, 1969, and an extremely dry year, 1971. All measured flows and rainfall were used as inputs to the model and factors such as potential evapotranspiration rate, Manning's overland flow coefficient, and the coefficient of conductivity between canals and the aquifer were adjusted until the stages predicted by the model were in satisfactory agreement with those actually recorded in key locations throughout the area. The calibration is described in the documentation report referenced above. A base condition for the evaluation of alternatives was defined by adding the South Dade Conveyance System, which was not operable during the calibration period. The results of this run became the baseline for comparison with all other model scenarios. The base condition is the current best estimate of the physical and operational water management system that will exist in 1990 with no modification resulting from this study.

Various alternatives for redistributing flow to the Everglades were simulated for the 1969-82 period. Daily stages along the Tamiami Trail were saved for use as the boundary condition for the 1x1 model.

21. 1x1 Model with Corps Modifications. As mentioned previously, the 1x1 model is essentially the same as the 2x2 model but only covers the area of Dade County and Big Cypress Swamp south of Tamiami Trail. The denser 1x1 mile grid network better defines water levels in the East Everglades area. Because the 1x1 model has eliminated the northern two-thirds of the lower east coast drainage basin, it must be assigned the flow or stages for each day for the period 1969-82 along its northern boundary. Daily discharges at S-334 and S-335 are input to the 1x1 model in a file designated FLO2x2. Daily stages are given by file KNSTGS at 35 four-square-mile grid cells along the northern boundary and these in turn are interpolated to obtain the stages for the 71 one-square-mile grid cells making up the northern boundary of the 1x1 model. Discharges are not assigned to the S-12 structures because flow is determined from stages developed along the northern boundary by the 2x2 model and the overland flow is computed from these stages.

Several modifications were made to the 1x1 model by the Corps. Actual structure operating procedures rather than a fixed weir equation were substituted in the program to handle flows through several of the structures. This added the capability of closing off the eastern outlets of L-31N when damaging flood stages exist along the east coast. Procedures for seepage computations across levees for both laminar and turbulent flow were also added to the model.

New input data were developed for the Corps version of the 1x1 model. Core boring logs for C&SF levees and canals were reviewed to obtain more accurate data on the aquifer depths in the East Everglades area. Transmissivities were revised in the Rocky Glades area to reflect the findings of pump tests near pumping station S-331 and at the two pump test sites in this study. Ground elevations were updated in the Big Cypress Swamp, Everglades National Park (ENP), and East Everglades areas using the latest U.S.G.S. quadrangle maps and topographic surveys, and from ground elevations furnished by the ENP for their water level monitoring stations. A separate topographic survey of the Rocky Glades study area done by the Corps of Engineers established average ground elevations for that area. Figure D-9 shows the average ground elevations used in the model in Rocky Glades and NESRS area.

22. Calibration and Verification of the 1x1 Model. Calibration was performed using the 7-year period 1969-75. The most critical hydrologic unknowns to be calibrated were evapotranspiration rates and friction losses for overland flow. In calibrating the 1x1 model, ET values and roughness coefficients were left unchanged from those used by SFWMD. S-173, located midway along L-31N, was assumed closed throughout the 14-year calibration period as was generally true. All flow from L-31N above S-173 was discharged through C-1. Although not in operation until 1969, S-338 was assumed to be in operation which would allow slightly more flow out of L-31N into C-1 than would actually have occurred. It would also close during a 10-year flood or larger until non-damaging flood stages were restored downstream.

Verification was done for the 7-year period 1976-82. There were no changes in the model between the calibration and verification. It was not possible to calibrate S-331, which went into operation in 1983, since all the necessary input data to the model had not yet been developed.

a. Everglades National Park. Plates D-10 and D-11 depict the calibration and verification of gages P-33 and P-36 in Shark River Slough. The P-33 gage near the end of L-67 Extension shows calibration to be slightly low for 1969-75. Verification for 1976-82 becomes progressively lower indicating a possible increase in the roughness coefficient. Gage P-36 calibration was good. Verification becomes progressively lower the last 3 years ending up 1.0 foot low by 1982 also indicating a possible increase in roughness coefficient.

b. Taylor Slough. The calibration and verification were good at predicting peaks but were consistently high during the recession half of the hydrograph (see Figure D-12). Had the small channel which exists there been modeled, stages would have receded more rapidly.

c. Rocky Glades Area. Well G-596, Figure D-13, is a good indicator of impacts on the developed area of Rocky Glades. Well G-1502, Figure D-14, at Chekika Hammock State Park is representative outside the developed area. The calibration was generally very good at both gages. Verification at G-1502 was excellent. At G-596, verification was slightly low.

d. Canals. The prediction of stages for canals in agricultural and urban areas is generally more difficult to attain due to the large transient movement of storm water inflows. As long as their average stage is reasonably accurate, groundwater stages will not be adversely affected except for nodes adjacent to the canal.

L-31N has been subdivided into L-31N north of S-173, also site of pump station S-339, and L-31S south of S-173. The calibration and verification of L-31N and L-31S are given in Figures D-15 and D-16. Calibration of L-31N at S-173 was good for peaks but high during droughts. Towards the end of the verification period, computed stages were somewhat low throughout. However, this same thing occurred elsewhere at monitoring gages P-33 and P-36. The calibration and verification for L-31N at S-174 is reasonably good throughout. C-111 at S-18C likewise was reasonably good (Figure D-17).

The calibration and verification of C-1W at S-149 in Figure D-18 was fairly good except during the two major flood events of 1969 and 1981 when computed stages increased and historic stages dropped due to the manner in which the outlet structure was operated. There was a similar occurrence on C-103 at S-179 (Figure D-19). These divergent incidents could be corrected by modeling actual operation plans instead of using a simple stage-discharge (weir equation) relationship. The model tends to underestimate discharges and overestimate stages at the coastal structures during major flood events.

23. Modifications to the Calibrated 1x1 Model. Modifications to the 1x1 model were made following verification to reflect recent structural changes and additions, and modifications in the water deliveries and canal levels. Among these were the widening of L-31N; the addition of S-331, S-332, and

S-338; and the enlargement of S-194. Most of the canal hypothetical weir crest elevations used to establish low flow water levels were changed along with the effective weir widths and discharge coefficients used to approximate the stage-discharge relationship. Some structural changes occurred towards the later part of the verification period but were not included in it to simplify the modeling effort. This could account for any poorer verification after 1979.

Seepage equations were developed for six different sites and conditions in the South Florida Water Management Model according to depth of aquifer, average base width of levee, initial turbulent flow velocity, and whether single or double levee. These six locations are L-67 Ext., L-31N with existing levee, L-31N with the addition of a seepage levee, Rocky Glades double levee, Agricultural area double levee, State Road 94 through Big Cypress Swamp, and State Road 27 through Everglades National Park. Initial turbulent flow velocity was taken to be 0.0009 fps except for Rocky Glades and the Agricultural area to its south which were 0.003 and 0.0014, respectively. The remaining levees or watershed divides in the model were left unchanged, i.e., no overland flow permitted between divides, and seepage computed as groundwater flow based on the head difference across a 1-mile gradient.

The maximum or design seepage removal rates were computed manually using smaller increments and segments than used in the model. The model results were used to verify that mitigation requirements were met and to determine average seepage volumes, annual pumpage, and impact on stage duration and peak stage frequencies.

24. Future Model Improvements. SFWMD is currently making extensive improvements in both models. More structures are being given operational criteria rather than approximations using weir flow equations. New structures and canals are being added. A special levee seepage subroutine is being added. Topographic, aquifer depth, and permeability data are being updated. The 2x2 version will be recalibrated and ready by May 1991. The 1x1 will take a few months longer. Both will be used for the final design in the FDM.

25. Modeling Long-Term Impact of Water Management Plans. The time of occurrence and duration of flooding over many months and years are of critical importance in evaluating the impact to agriculture and wetlands from the proposed water management plans and flood protection works. For this reason, 14 years of continuous record were modeled using actual rainfall records. Only one notable flood occurred (in 1969) during the 1969-82 period tested. The 1969 flood was roughly a 14-year event.

The Basic Rain Driven Plan with no protective measure for the Rocky Glades area was run on the 1x1 model for the 14 year test period. The stage duration curves were compared to those for Base Condition to see what the impact on water levels would be in that area and for determining flowage easements. Figures D-20, 21 and 22 present the rise in water table levels and additional days water levels are above Base Condition for wet, normal and dry conditions. Figure D-20, the rise in water levels during wet conditions, approximates the amount of mitigation required from implementing the Rain Driven Plan. The actual amount of mitigation can't be known without looking at larger flood events. The frequency and duration of flooding for larger flood events cannot

reliably be predicted from only 14 years of record; therefore, three synthetic storm events, the 25-year, 100-year, and Standard Project Storms extending over the 6-month rainy season were simulated with the model, and the results were used to estimate damages from these events (see Section K). The three synthetic storms were applied to the 2x2 model so that the northern boundary conditions could be established for the 1x1 model.

The 1x1 node sizes were too large to accurately model and compute the seepage for the proposed double levee around the residential area. Therefore, maximum seepage rates were computed manually by levee reach as explained under Levee Seepage. The interior drainage peak runoff rates were determined using the HEC-1 rainfall-runoff program discussed in the next section. The 1x1 model was used to tabulate monthly pumpage values for the protected areas, which were used to estimate average annual pumpage costs.

I. HEC-1 MODELING OF INTERIOR DRAINAGE

26. General. The 1x1 model does not have sufficient definition to estimate runoff rates for the small watersheds behind levees. Flood hydrographs were developed using the Corps of Engineers, HEC-1 Flood Hydrograph program. A minimum of 10-year flood protection was established for sizing the interior drainage canals. A sensitivity analysis showed that a more critical condition existed with high levee seepage rates and moderate rainfall than with average seepage rates and high rainfall intensity. This analysis showed that levee seepage exceeds interior runoff rates by several times. Another purpose of the HEC-1 modeling was to check whether the levees might cause worse flooding by restricting the escape of runoff into NESRS for intense local rainfall events.

27. Antecedent Moisture Condition. Flooding generally occurs during the June-October rainy season. Therefore, the average wet season groundwater level in each sub-basin behind the levee was selected to begin the rainfall-runoff analysis. Rather than being based on historical groundwater levels at nearby wells, the results of the 1x1 model base condition run was considered more indicative of existing conditions. A single reference point was selected. Groundwater averages for each sub-basin were estimated based upon the natural groundwater gradient in the area. Table D-14 shows the derivation of average groundwater level at the reference point for the Rocky Glades residential area. The monthly average was taken to be the average of the previous end of month stage and the present end of month stage. A similar analysis for the agricultural area provided an average groundwater level for June-October of 5.14 feet at node 50-24.

TABLE D-14

AVERAGE WET SEASON GROUNDWATER LEVEL AT ROCKY GLADES PUMP STATION
BASE CONDITION RUN (NODE 52-29)

Year	May 31	Jun 30	Jul 31	Aug 31	Sep 30	Oct 31
1969	5.00	7.40	7.00	6.90	7.10	7.50
1970	6.70	6.60	6.10	5.90	6.00	6.50
1971	3.80	5.40	5.30	6.00	5.80	6.00
1972	5.80	6.10	5.90	6.30	5.90	5.60
1973	2.50	4.90	6.30	6.60	6.40	5.80
1974	3.40	5.60	6.30	6.20	6.10	5.40
1975	5.20	5.70	6.00	5.90	6.40	5.90
EOM Avg	4.63	5.96	6.13	6.26	6.24	6.10
Month Avg.		5.30	6.04	6.20	6.25	6.16
Season Avg.						5.99

The average groundwater table would be slightly lower with the levees and pumps in place and operating under the Rain-Driven Plan. Antecedent groundwater levels for the selected plan, using L-31N to convey excess flood waters north to NESRS, were based on modeled historical record modified for the selected plan. The average wet season stage at node 52-29 was 5.26 feet with-project.

28. Storm Loss Rate. Holtan's soil infiltration method was selected for use with the HEC-1 model. Holtan's equation describes the soil's available moisture capacity and infiltration as expressed below.

Holtan's equation for infiltration (f) in in/hr:

$$f = (GI)A * SA^{BEXP} + FC$$

where (GI)A = Growth index x infiltration capacity (A) in in/hr

SA = storage available in inches (.19 x inches to water table)
FC = constant percolation rate in in/hr
BEXP = empirical exponent usually 1.4

The growth index (GI) for vegetation was taken as 1 or full grown. Infiltration capacity was 20.0 in/hr. Constant percolation rate was assumed to be 0.01 in/hr which is high for NESRS but entirely possible along L-31N due to seepage into the canal. Storage availability is based on the depth to water table and a porosity of 19 percent.

29. Base Flow. Base flow is that part of the stream discharge that is not attributable to direct runoff from rainfall, but comes from drainage of ground water. Normally base flow is suppressed by rising flood waters and can be ignored. Also, in extremely flat terrain the ponding of flood waters results in a very low hydraulic gradient which also minimizes groundwater inflow. However, for an aquifer as porous as the Biscayne aquifer, base flow can exceed surface runoff for small flood events such as the mean annual flood.

Groundwater seepage into a canal from a sudden rise in water table levels was computed from the following equation for transient seepage:

$$q=h\sqrt{\frac{nT}{\pi t}}$$

where q is discharge in cfs/linear foot along one side of the canal
h a constant, is the rise in water table above the water level in the canal
n is the effective porosity of the soil within the range of the water table elevations
T is the transmissivity of the aquifer in cfs/ft.
t is the elapsed time since the sudden rise in water level in seconds

The derivation of this equation is given on page 20 of the GDM, Part V, Suppl. 52 [6]. Although originally developed to compute seepage from a sudden rise in canal level, it is equally applicable to a sudden rise in water table level or a sudden drop in canal level. The discharge was computed for a 1-foot rise in water level at 0.1-day increments using an effective porosity of 0.06, the value obtained for the pump test at S-331. The total seepage which could occur in the first 24-hours was determined and used to estimate the seepage contribution as explained in the following paragraph.

Table D-15 gives an example of channel bank seepage along one side of the canal for the first 24 hours coinciding with peak daily flood stage. For the example in Table D-15, had the average daily surface runoff rate been 80.7 cfs/mile, then base flow would have been taken as zero for the day. Had the surface flow been zero and the soil moisture capacity completely filled, then base flow would have been 40.6 cfs/mile. Thus, for every two cfs of surface runoff, one cfs would be subtracted from the 40.6 cfs base flow.

TABLE D-15

EXAMPLE COMPUTATION OF CHANNEL BANK SEEPAGE

Given: Design Water Level in Canal = 5.15′
Avg. ground elev. (and water level) = 7.00′
Transmissivity = 18 MGD/ft (27.9 cfs/ft)

Time (days)	Seepage Along One Bank (cfs/mi/ft.hd)	(cfs/mi)
.1	41.4	80.7
.2	29.3	57.1
.3	24.0	46.8
.4	20.8	40.6
.5	18.6	36.3
.6	17.0	33.2
.7	15.6	30.4
.8	14.7	28.7
.9	13.9	27.1
1.0	13.1	25.5
	Avg.	40.6

30. Overland Flow Routing. Rainfall excess was routed overland using the kinematic wave equation in HEC-1. In the residential area, there is no existing drainage system so overland flow distances extended the full length of each sub-basin to the proposed collection canals. A Manning's roughness coefficient of 0.3 was used. The resulting peak 24-hour discharge for the 10-year flood was 55.9 cfs/sq. mi. or 2.08 inches/day. This agrees very closely with the corresponding runoff rate for overland flow as published in SFWMD's "Permit Information Manual".

The same procedure was used for the agricultural area. Many of the agricultural plots consist of furrows with elevated beds. To determine the runoff rate, it was assumed that the runoff rate would not exceed the natural runoff rate, so overland flow across undeveloped land was selected. Doing this, however, ignored the backwater effect of Taylor Slough which often resulted in too high a runoff rate. Therefore, in sizing canal capacity the maximum allowable runoff per sub-basin was held to 2 inches/day or 53.8 cfs/sq. mi. This value corresponds to about the 10-year flood runoff rate for overland flow in undeveloped lands, and is also the established minimum acceptable removal rate for truck crops and vegetable crops in a 2 to 3 sq. mi. area for the Everglades Agricultural Area. Table D-16 presents the results of the overland flow routings.

31. Interior Drainage Ponding Depths With and Without Levees. Flood hydrographs from HEC-1 for the various frequency flood events were routed through storage behind the levees to determine whether flood stages would be worsened by the levees. A flat pool was assumed and levee seepage out of the protected area was ignored. Flood stages without the project were estimated by adding the appropriate 3-day rainfall to the average water table level expected during the rainy season in the vicinity of the levees. The point rainfall was reduced to that for a 25-square-mile basin to estimate an average pool level for the residential and surrounding area. The SPF was determined using 50% of the Probable Maximum Precipitation (PMP).

The 3-day, 10-square-mile PMP when averaged over 25 square miles is 89.3 percent of the point rainfall (or 10 sq. mi.) compared to 96.8 percent for the 100-year and smaller storms. For the SPF, an intermediate value of 93 percent was used. The results of this analysis and the flood routings are presented in Table D-17. Table D-17 clearly shows ponding depths behind the levees are less than without the levees.

TABLE D-16

OVERLAND FLOW PEAK DISCHARGES
FROM HYPOTHETICAL FREQUENCY STORMS

Subbasin	Drainage Area (Sq. Mi)	2-Yr (cfs)	10-Yr (cfs)	25-Yr (cfs)	50-Yr (cfs)	100-Yr (cfs)	SPF (cfs)
(8.7 Sq. Mi. Protected Residential Area)							
2	0.50	4	28	50	70	94	312
4EW	0.12	1	4	7	11	14	50
4NS	1.46	3	30	60	87	118	442
6EW	0.94	1	17	35	51	70	263
7AB	0.43	4	35	66	93	123	400
7C	0.93	1	30	64	96	133	512
7D	0.58	0	15	34	51	71	283
7EN	0.49	2	20	38	55	75	270
7FN	0.49	3	21	38	54	72	249
6NSN	0.47	5	31	56	79	104	329
6NSS	1.41	1	28	60	89	123	484
7FS	0.53	0	10	24	38	53	221
7ES	0.33	0	7	18	28	41	171
Total*	8.69	24	276	552	800	1087	3835
(11.3 Sq. Mi. Protected Residential Area)							
1	1.85	5	47	91	131	177	655
2	1.29	22	82	137	184	237	721
3	2.40	17	73	124	169	221	732
4	1.00	37	120	186	237	291	820
5	2.44	22	82	136	183	237	763
6	0.26	32	77	107	131	157	444
7	2.07	44	151	247	327	416	1221
Total*	11.31	170	589	959	1265	1605	4850
(Agriculture Area)							
1E	1.72	5	39	74	105	142	514
1W	0.95	7	44	81	113	150	507
2E	1.84	3	36	69	100	136	503
2W	0.82	5	40	76	107	143	482
3E	1.90	3	35	69	100	135	503
3W	0.50	5	38	71	99	130	418
4E	0.59	2	19	36	51	69	252
4W	0.22	4	22	39	52	68	207
5E	0.58	6	41	77	107	140	443
5W	0.44	6	42	75	102	133	414
Total*	9.56	45	356	665	923	1217	3985

*Before reservoir routing.

TABLE D-17

FLOOD STAGES WITH LEVEE VS. WITHOUT[3]

Rainfall Freq (yr)	With Levee (HEC-1 Stage Inside Levee) 841 cfs Pump (ft)	470 cfs Pump (ft)	Without Levee Avg. Ground Elev. (ft)	- Avail. Soil Moist. Capacity (ft)	3-Day Point Rainfall Depth (ft)	+ Rainfall for 25 Sq Mi (ft)	= Stage (ft)
			(Residential Area)				
2	5.96	5.34	6.55	-.09[1]	.48	.46	6.92
10	6.11	5.78	6.55	-.09	.78	.75	7.21
25	6.33	6.53	6.55	-.09	.98	.94	7.40
50	6.71	6.87	6.55	-.09	1.12	1.08	7.54
100	6.96	7.13	6.55	-.09	1.27	1.23	7.69
SPF	8.09	8.36	6.55	-.09	2.32	2.16	8.62
			(Agricultural Area)				
	404 cfs Pump						
2	5.44		6.6	-.27[2]	.48	.46	6.79
10	5.75		6.6	-.27	.78	.75	7.08
25	6.62		6.6	-.27	.98	.94	7.27
50	6.87		6.6	-.27	1.12	1.08	7.41
100	7.03		6.6	-.27	1.27	1.23	7.56
SPF	8.03		6.6	-.27	2.32	2.16	8.49

(1) Available Soil Moisture Capacity = (Ground elevation of 6.55 - water table of 6.1) x .19.

(2) Available Soil Moisture Capacity = (Ground elevation of 6.6 - water table of 5.2) x .19.

(3) These stages are for comparative purposes only and are lower than long-term rainfall results using SFWMM.

J. HYDROLOGIC DESIGN

32. General. The impact of the basic rain driven plan would be to increase flood stages in NESRS from more than a foot along the Tamiami Trail to about 0.1 foot in the agricultural area (see figure D-20). The residential area of Rocky Glades showed a 0.1-0.3 foot increase. The minimum required level of protection is that necessary to mitigate increased flooding for all frequency flood events for the residential and agricultural areas of Rocky Glades. Any protection beyond that would have to be economically justified.

During preliminary investigations several levee plans were investigated, some of which included detention areas. For the final analysis seven plans were evaluated. Plans A and B were the initial attempts at minimal protection, assumed to be the increased seepage from a uniform 0.2-foot rise in water level outside the levees plus removal of the 2-year flood event. Double levees were assumed necessary because of the high seepage rates. Pumpage was over the levees directly into the slough to the west. Separate protection for the agricultural area as in Plan B was found unnecessary because all the structural alternatives for the residential area lowered water levels in the agricultural area sufficiently to meet mitigation requirements. Plan C represented a higher level of protection sized to protect against the 1969 flood to see if additional flood control benefits were economically justified. The benefits to cost ratio were unfavorable, therefore, all subsequent plans limited the sizing of pumps to mitigation only.

33. Alternative Plans and Basis for Sizing Pumps. Plates 9 through 15 in the main report show the basic protection plans for the Rocky Glades residential area and the agricultural area to the south. The following paragraphs briefly discuss the basic differences between plans. Table D-18 compares some of the physical dimensions and discharge rates for the various alternatives.

a. Plans A and B. Plan A represents the minimum level of protection required for the 1969-1982 period of record. An 840 cfs pump discharging into NESRS and 9.4 miles of double levees were used to protect 11.3 square miles of residential area. Plan B added a levee and a 400 cfs pump station discharging into Taylor Slough for protecting the adjacent 9.6 square miles of agricultural area to the south. The levees were located westward into wetlands to take advantage of lower transmissivities there and to place a greater distance between levee and main drainage canals thus further reducing seepage. When later tested against extreme flood events, mitigation was not met at all locations in the residential area. It was also environmentally unacceptable because of impacts on wetland acreage and a getaway channel within ENP.

b. Plan C. Plan C is identical to Plan A except the pump size was increased to 1332 cfs to provide complete protection from 1969 flood levels plus the 2-year runoff from inside the levee. Flooding was more than mitigated but the plan was unacceptable for environmental reasons.

c. Plan D. Plan D attempted to overcome environmental objections to the proposed double levee along L-31N by removing it and adding a second pump station to return excess seepage beneath L-31N back into NESRS. It is similar to Plan C except the area protected is smaller and a 500 cfs pump is added to

remove excess seepage from L-31N and pump it westward through the residential area to a 1340 cfs pump. It failed to mitigate flooding even for the 1969 flood.

d. Plan E. This plan attempted to overcome environmental objections to a new getaway channel within Park boundaries for the residential pump by pumping east into L-31N where flow would be conveyed north to a proposed new pump station at the L-29 outfall and then backpumped into NESRS through L-29 borrow canal. The need for a double levee along L-31N was eliminated by enlarging the northern most pump to handle the additional L-31N seepage. The residential area protected was also reduced to 8.7 square miles to minimize the taking of wetlands. No levee was needed for the agricultural area because the drawdown in water levels from S-357 extended well south of the residential area. After several trials, a combination of 470 cfs for the residential pump and 900 cfs for the L-29 pump was found to completely mitigate flooding in the residential and agricultural area of Rocky Glades. A single larger pump alternative at the L-29 site capable of gravity draining the residential area was ruled out immediately because of the tremendous increase in seepage that would be incurred and the need for much greater conveyance capacity in L-31N.

e. Plan F. All prior plans called for levee heights sufficient to prevent overtopping from wind setup from a moderate hurricane coinciding with SPF stages in NESRS. Plan F is the same as Plan E except the hurricane flooding was no longer impounded by a southern levee, thus, lowering levee heights several feet to a top elevation of 10.3 feet, NGVD, (about 4.0 feet high).

f. Plan G. This plan is the same as Plan F except the residential area pump station was moved 2.0 miles north to the north end of the seepage collection channel to reduce land acquisition costs and the need for access bridges. This required increasing the pump size to 533 cfs to handle increased seepage along the north reach. The lower transmissivity rates at the new site reduced the average annual pumpage required. S-356 was increased in size to 950 cfs to handle the additional increase in peak discharge from S-357. The savings in capital costs make Plan G the preferred plan. Further geotechnical investigations and modeling efforts are required before the pump station capacities can be designed.

34. Levee-Seepage. Accurate head losses at the levees could not be obtained from the SFWMM for the various reaches. A computer program was written to compute the head loss across two levees and from overland flow starting a half-mile out and ending 300 feet behind the smaller inner levee at the seepage collection channel. Overland flow was computed using Manning's equation and "n" values shown in figure D-23. A range of starting elevations and discharges were run so that the seepage could be interpolated from a tabular printout if water level in the canal and 2000 feet beyond the levee were known. The method of computing seepage is covered in Appendix G, Geology and Soils Investigation.

TABLE D-18

COMPARISON OF ROCKY GLADES FLOOD PROTECTION ALTERNATIVES

PLAN:	A	B	C	D	E&F	G
Area Protected:	Resid.	Resid. + Agric.	Resid.	Resid.	Resid.	Resid.
Pump Sizes (cfs)	840	840+400	1332	500+1340	470+900	533+900
Levee Length (Miles)	9.4	12.1	9.4	9.4	9.4*	7.3
Canal Length (Miles)	11.3	21.0	11.3	9.3	9.3	7.2
Protected Area (Sq.Mi)	11.3	20.9	11.3	8.7	8.7	8.7
Levee Seepage (cfs)	614	614+302	1095	767	296	383
Other seepage (cfs)	66	66+57	66	48	149**	126***
2-year flood (cfs)	161	161+46	161	25	25	24
Avg Pumpage (Ac Ft/Yr)	105,000	105,000 & 15,000	118,000	31,000& 145,000	57,000& 100,000	54,000& 100,000

* 7.3 miles for Plan F since south levee is eliminated.
** 70 cfs is seepage around pump station. 67 cfs is base flow. 12 cfs is seepage from L-31N.
*** 29 cfs is seepage around pump. 40 cfs is base flow. 57 cfs is L-31N seepage.

35. Canal Sizes. Seepage collection canals are provided where necessary. They would be located approximately 250 feet behind the inside levee to collect and relay excess water to a nearby main receiving canal. The design water levels are set as close to average ground elevation as possible to minimize seepage inflows. The canals are sized to handle interior drainage for the 10-year rainfall runoff or the 2-year runoff combined with levee seepage from about a 10-year flood stage in NESRS, whichever is larger. Only Plan C gave full 10-year flood protection for both interior drainage and NESRS flood stages. The remaining alternatives would still experience flooding from excessive levee seepage from 10-year flood stages in NESRS.

Plates 9 through 15 of the main report show the canal alignments for Plans A through G, and Tables 28-32 give discharges by reach. A further breakdown of inflow by source is shown here in Table D-19 for Plan G. Interior drainage is assumed to be sheet flow uniformly distributed along the main receiving canals. The aquifer is so porous that a surface collection system is unnecessary.

36. Weirs and Culverts. Weirs are required in Plans A, B, and C at the ends of all seepage collection channels to maintain upstream water levels as close to natural ground elevation as possible to assure minimum levee seepage rates. A 0.1-foot reduction in canal water level increases total seepage 93 cfs in the residential area and 34 cfs in the agricultural area. The extremely high transmissivity rate precludes any need for lateral ditches or outfall structures.

TABLE D-19

CUMULATIVE INFLOW TO RESIDENTIAL CANAL IN CFS

Plan G

(1) Reach	Canal Length in Feet	Levee Seepage	Base Flow	2-Yr Runoff	L-31N Seepage	Total Inflow	Design Channel Capacity
J-I	5,600	14	0	4	7	25	28 (2)
I-H	1,230	9	0	1	2	12	37
H-G	5,320	60	8	3	2	73	110
G-F	2,620	27	5	1	2	35	145
F-E	5,480	64	3	5	5	77	222
E-D	7,920	104	14	5	8	131	353
D-C	5,310	54	8	1	12	75	428
C-A	4,910	51	2	4	19	76	504
Return seepage 500′ around S-357						29 (3)	533
Totals		383	40	24	57	533	

(1) See Plate 26.
(2) Assuming 70′ of canal below stilling basin is concrete lined.
(3) Canal sized for larger 10-year runoff of 28 cfs.

37. Levee Design Grade. Plans F and G levee grades were set at 1.0 foot above the SPF stage bordering NESRS. Table D-20 specifies the elevation and location of levee grades. For Plans A through E, levee heights were sized to withstand hurricane wind setup and wave runup. This level of protection exceeded the scope of the authorizing document and was not considered in subsequent levee design heights. It was investigated, however, to make sure wind setup was not worsened in the residential area by the recommended plan as described next.

38. Wind Setup with Low Levees. The low levees would be completely overtopped from the combination standard project flood and moderate hurricane wind setup. A comparison of residential water levels under Base Condition and the low levee plan show that peak wind setup stages for all flood events remain similar between with and without project conditions. Higher NESRS antecedent water stages are mitigated for in the low levee plan by initial storage in the residential area and pump capacity provided by the plan. If increase in height and density of vegetative growth in NESRS is considered, wind setup will be lower with the Rain Driven Plan than under Base Condition.

TABLE D-20

REQUIRED LEVEE GRADES FOR F & G

Levee Location	Top of Levee (ft,NGVD)
A	10.2
C	10.2
D	10.1
E	10.3
F	10.3
G	10.3
H	10.1
I	10.1
J	9.8
L-31N at L-29	17.4
L-31N 1 Mile North of A	16.0

(1) Defined in Plate 26

K. MODELING RESULTS

39. Stage Frequency. Figure D-24 compares the 1969-82 historical stage frequency plotting points at Well G-596 with the 1969-82 model calibration and Base Condition runs. Since the 14-year period used in the model is too short for establishing reliable statistics, three synthetic storm events, the 25-year, 100-year and standard project storms were also run on the model and the results plotted with the 14 years of record. From the Base Condition stage-frequency curve, it is evident that the slope increases after the 10-year frequency. For floods below a 10-year frequency, excess flood waters are either stored in WCA No. 3A or easily carried away in the many canals draining the east coast and designed to remove the 10-year flood event. For floods larger than the 10-year event, releases must be made from the water conservation areas, and structures S-331, S-338, S-194, and S-196 along L-31 are closed to protect the east coast urban areas. The stage frequency curve cannot continue to rise in this transition area at this steeper rate but tapers back to a flatter slope as flood size increases. For the very large flood events, the Base Condition would have less and less impact on the outcome of flooding and the curve should finally approximate the 1962-85 curve. The plotting positions for the 25-year, 100-year, and standard project floods seem to bear this out, and the curve as shown for Base Condition is the result.

The curve fitting for peak annual one day stage was done using a frequency plotting program which determined the line of best fit through the 17 plotting positions using the least squares method, a linear-log transformation and second degree polynomial. The peak annual stages for the period 1969-82 were assigned exceedence frequencies based on Weibull's equation M/(N+1) where N = 14 years and M = rank of flood event beginning with 1 for the largest

and ending with 14 for the smallest. The three synthetic flood events were assigned the frequency of the rainfall as stated previously. This same program was used to determine stage-duration frequencies.

The peak annual daily stage frequency for the various Rain-Driven Plans inside the proposed protected areas of Rocky Glades can be compared with Base Condition in the stage-duration-frequency Tables D-21 and D-22. Figure D-25 compares the stage frequency just outside the protected area at Well G-1502 (Chekika Hammock State Park) for two Rain-Driven Plans and Base Condition.

40. Stage-Duration-Frequency. Stage-duration-frequencies were developed for the 1969-82 period of record and supplemented with the 25-, 100-year and standard project flood hydrographs for each of the grids within the area being considered for flood protection. Durations of 1, 5, 10, and 15 days were employed. Table D-21 presents the results for Well G-596 in the residential area for Base Condition and several Rain-Driven alternatives. Table D-22 does the same for a representative node in the agricultural area. As expected, the Rain-Driven plan with no protective measures increases flood stages progressively from 0.13-foot for the 2-year flood (exceedence probability = 0.5) to 0.35-foot for the 500-year flood (exceedence probability = 0.002) in the residential area. Plans C, E, F, and G successfully met or exceeded mitigation requirements at all nodes and frequencies.

41. Long-Term Stage Durations. To evaluate the impact on wetlands, long-term stage durations extending over several years are required. This task was done using the SFWMM 2x2 model and is presented in Appendix E.

42. Pumpage Volumes. The average annual pumpage volumes for the proposed 470 cfs Rocky Glades pump and the 900 cfs backpump between L-31N and L-29 canals are given in Table D-18. Figures D-26 and D-27 show the discharge duration curves for the 14 years of simulated record.

43. Percent of Time Inundated. Figures D-28 through D-30 give an idea of the impact on water levels in and around the Rocky Glades area. Figures D-28 and D-29 show the percent of time each grid node is inundated for Base Condition and for the Rain-Driven Plans E, F, and G. Figure D-30 shows the change between Base Condition and Plans E, F, and G. As expected, the addition of the protective levee around the Rocky Glades residential area and two pump stations drops the water levels inside and adjacent to the levees and raises the water levels in the northeast corner of NESRS.

TABLE D-21

STAGE-DURATION-FREQUENCY
WELL G-596 IN RESIDENTIAL AREA (Node 53 29)

PROBABILITY OF EXCEEDENCE

Duration (days)	.002	.01	.02	.05	.1	.2	.5
			(Base Condition)				
1	9.05	8.40	8.10	7.66	7.32	6.95	6.43
5	8.80	8.25	7.96	7.54	7.17	6.79	6.23
10	8.55	8.05	7.75	7.33	6.97	6.58	6.00
15	8.30	7.79	7.52	7.11	6.76	6.40	5.87
			(Rain-Driven and No Protection)				
1	9.40	8.70	8.37	7.89	7.51	7.11	6.56
5	8.66	8.29	8.05	7.65	7.32	6.93	6.35
10	8.68	8.15	7.85	7.43	7.06	6.67	6.10
15	8.52	7.91	7.62	7.20	6.85	6.49	5.98
			(Rain-Driven With L-31N Double Levee Only)				
1	9.29	8.63	8.29	7.84	7.47	7.09	6.55
5	8.98	8.40	8.10	7.67	7.30	6.91	6.36
10	8.73	8.16	7.86	7.43	7.07	6.69	6.15
15	8.50	7.89	7.60	7.18	6.85	6.50	6.01
			(Rain-Driven Plan C 1332 cfs Pump Double Levee Residential Area and L-31N)				
1	9.02	8.24	7.86	7.34	6.92	6.48	5.87
5	8.89	7.94	7.52	6.95	6.51	6.07	5.46
10	8.63	7.49	7.05	6.49	6.09	5.71	5.23
15	8.31	7.24	6.81	6.29	5.92	5.57	5.13
			(Rain-Driven Plan A 840 cfs Pump, Double Levee Residential Area and L-31N)				
1	9.20	8.43	8.07	7.54	7.10	6.65	6.03
5	8.91	8.08	7.68	7.13	6.70	6.25	5.62
10	8.63	7.71	7.29	6.73	6.31	5.90	5.36
15	8.38	7.43	7.04	6.52	6.13	5.75	5.25
			(Rain-Driven Plan B 840 + 400 cfs Pumps, Double Levee Residential and Agricultural Areas and L-31N)				
1	9.17	8.42	8.05	7.53	7.10	6.65	6.01
5	8.91	8.10	7.70	7.14	6.70	6.25	5.63
10	8.63	7.66	7.22	6.66	6.25	5.86	5.35
15	8.38	7.41	7.02	6.50	6.12	5.75	5.26
			(Rain-Driven Plans E,F&G 470 + 900 cfs Pumps* via L-31N, Double Levee Residential Area Only)				
1	8.96	8.31	7.95	7.40	6.93	6.43	5.71
5	8.78	7.84	7.42	6.84	6.40	5.94	5.34
10	8.50	7.50	7.08	6.53	6.11	5.70	5.16
15	8.25	7.35	6.95	6.44	6.05	5.65	5.11

*533 and 950 cfs, respectively for Plan G.

TABLE D-22

STAGE-DURATION-FREQUENCY
NODE 50 24 IN AGRICULTURAL AREA

PROBABILITY OF EXCEEDENCE

Duration (days)	.002	.01	.02	.05	.1	.2	.5
			(Base Condition)				
1	8.46	8.15	7.90	7.50	7.13	6.72	6.10
5	8.23	7.96	7.73	7.35	6.99	6.58	5.95
10	7.97	7.69	7.45	7.08	6.74	6.35	5.75
15	7.70	7.42	7.20	6.85	6.54	6.17	5.61
			(Rain-Driven and No Protection)				
1	8.60	8.20	7.93	7.52	7.15	6.75	6.15
5	8.32	7.99	7.75	7.35	7.00	6.61	6.00
10	7.95	7.70	7.49	7.13	6.80	6.42	5.84
15	7.67	7.43	7.22	6.88	6.62	6.23	5.70
			(Rain-Driven L-31N Double Levee Only)				
1	8.46	8.16	7.94	7.54	7.19	6.79	6.18
5	8.21	7.95	7.74	7.36	7.02	6.63	6.04
10	7.94	7.67	7.46	7.11	6.78	6.41	5.86
15	7.66	7.41	7.22	6.88	6.58	6.24	5.72
			(Rain-Driven Plan C 1332 cfs Pump, Double Levee Residential Area and L-31N)				
1	8.46	8.12	7.87	7.47	7.11	6.70	6.10
5	8.27	7.95	7.70	7.31	6.95	6.54	5.93
10	7.91	7.63	7.41	7.04	6.70	6.32	5.72
15	7.65	7.38	7.17	6.82	6.50	6.14	5.59
			(Rain-Driven Plan A 840 cfs Pump, Double Levee Residential Area and L-31N)				
1	8.39	8.08	7.85	7.46	7.12	6.72	6.12
5	8.16	7.91	7.70	7.33	6.98	6.58	5.96
10	7.86	7.60	7.40	7.04	6.71	6.34	5.75
15	7.63	7.37	7.16	6.83	6.52	6.16	5.62
			(Rain-Driven Plan B 840 + 400 cfs Pumps, Double Levee Residential and Agricultural Area and L-31N)				
1	8.46	7.92	7.64	7.25	6.91	6.56	6.06
5	7.97	7.40	7.10	6.68	6.33	5.95	5.42
10	7.82	7.06	6.73	6.29	5.96	5.62	5.18
15	7.48	6.74	6.42	6.02	5.71	5.40	4.99
			(Rain-Driven Plans E,F&G 470 + 900 cfs Pumps* via L-31N, Double Levee Residential Area Only)				
1	8.41	8.08	7.84	7.45	7.09	6.68	6.06
5	8.20	7.91	7.68	7.29	6.92	6.50	5.88
10	7.83	7.58	7.36	7.00	6.67	6.28	5.70
15	7.57	7.33	7.12	6.78	6.46	6.10	5.55

*533 and 950 cfs, respectively for Plan G.

SELECTED REFERENCES

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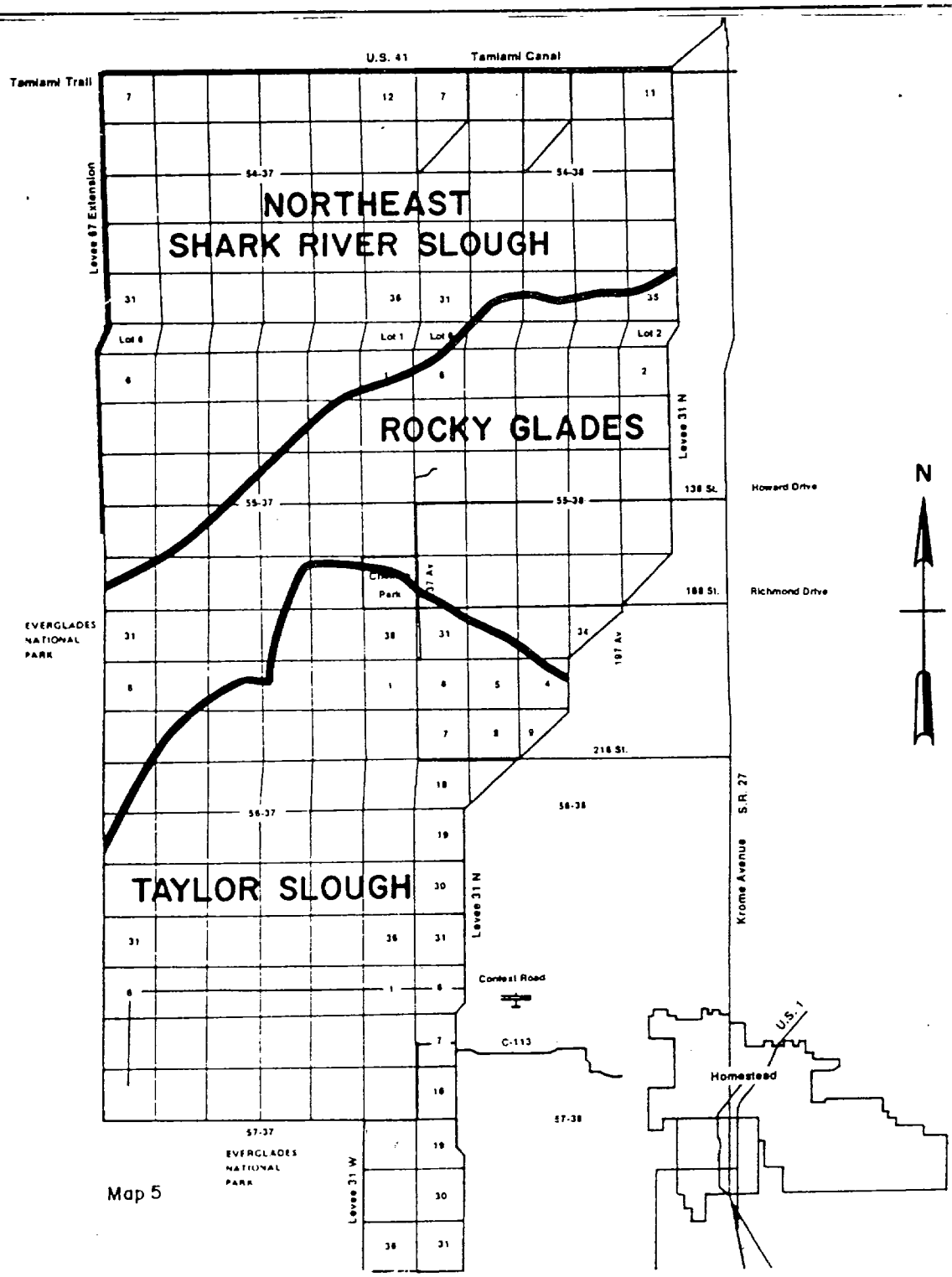


Map 5

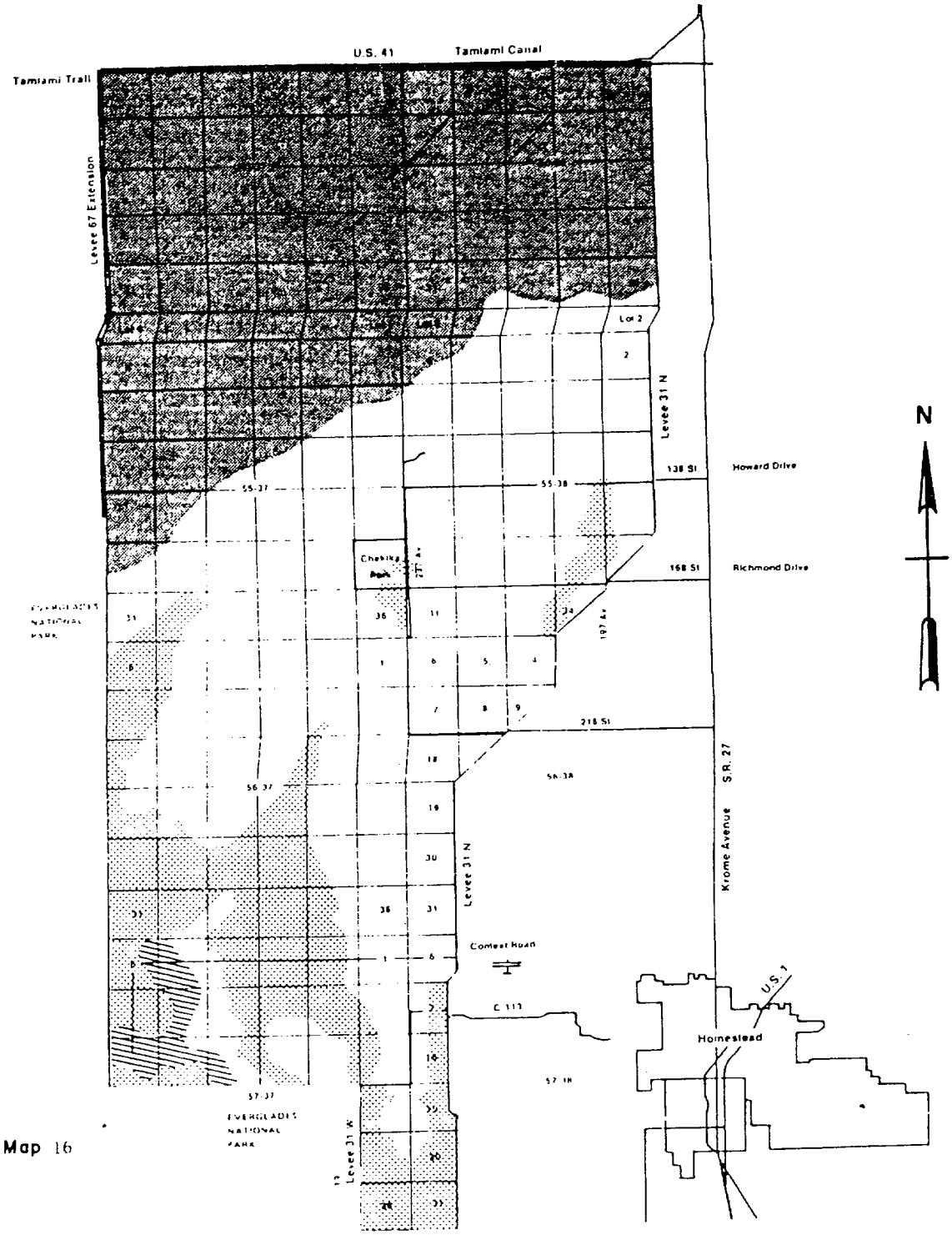
Source: Map 5 of ref. [13].

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

EAST EVERGLADES
PHYSIOGRAPHIC SUB-AREAS

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-1



Map 16

SOILS

- Peats (Everglades, Gandy, Loxahatchee)
- Rockland
- Perrine Marls
- Rockdale Limestone Complex

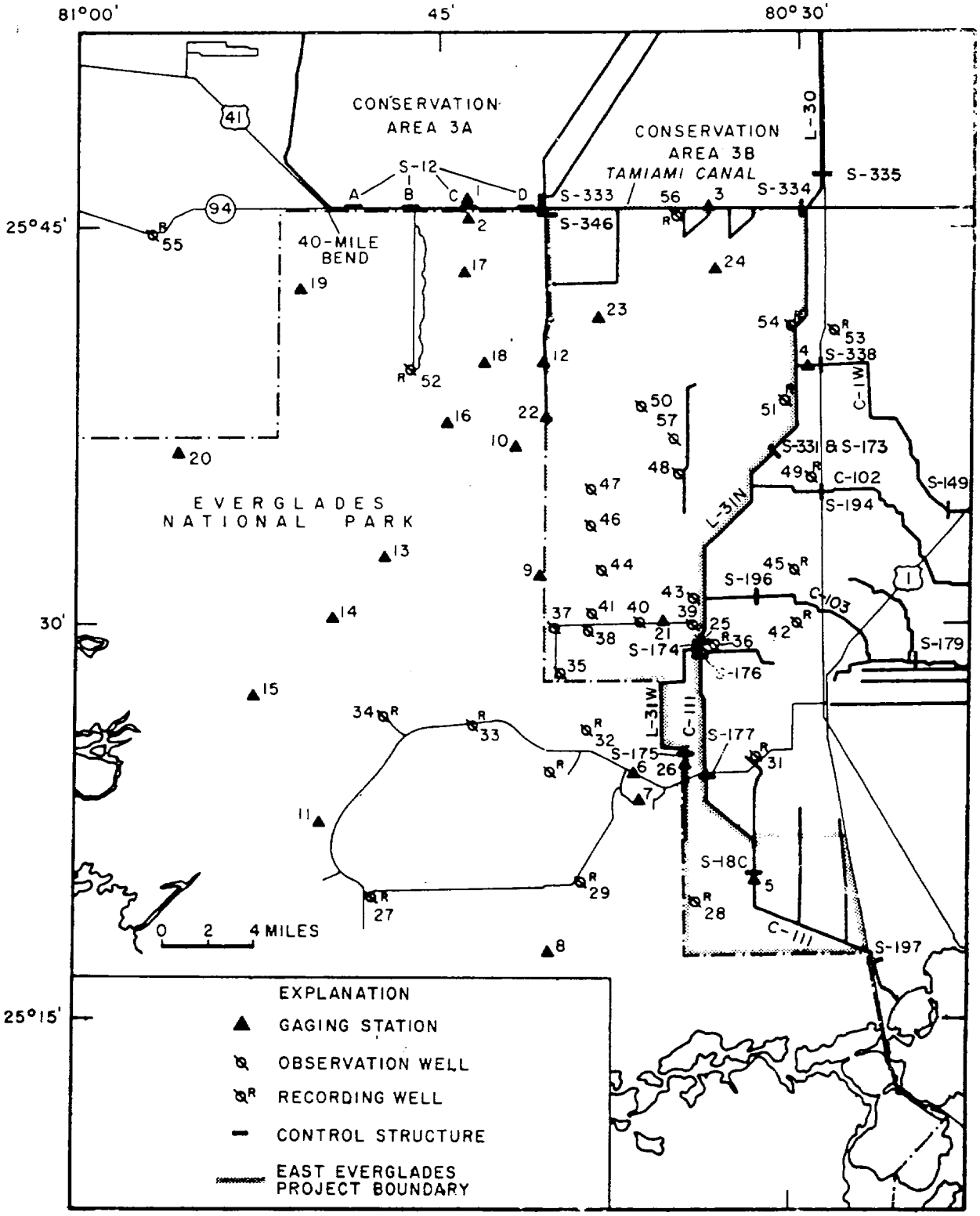
Source: Map 16 of ref. [13].

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

SOILS

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I , SUPP. 54, DATED:
FILE NO.

FIGURE D-2



Source: Fig. 48 of ref. [14].

CENTRAL AND SOUTHERN FLORIDA MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

LOCATIONS OF WATER – LEVEL DATA SITES

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO., PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-3

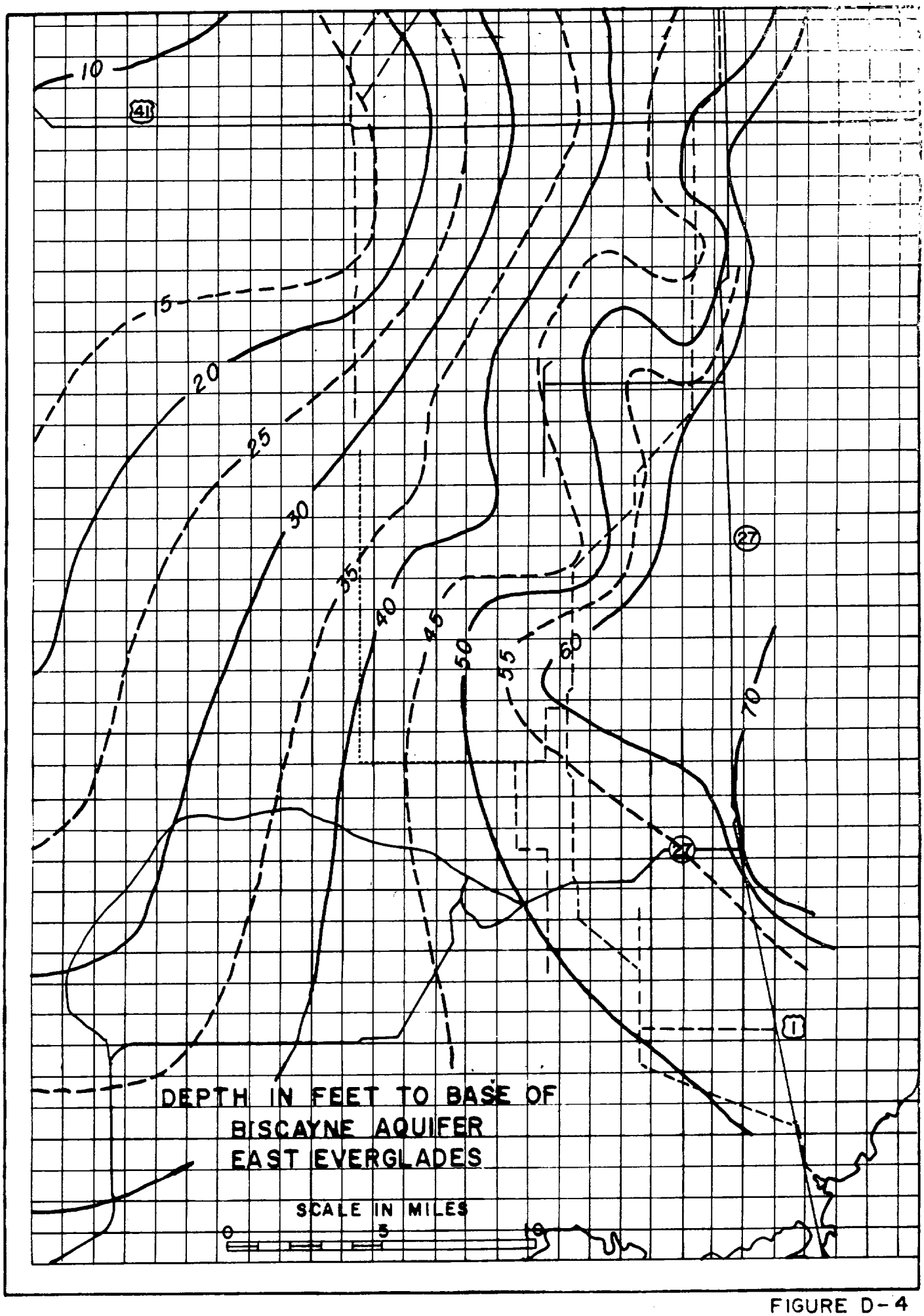
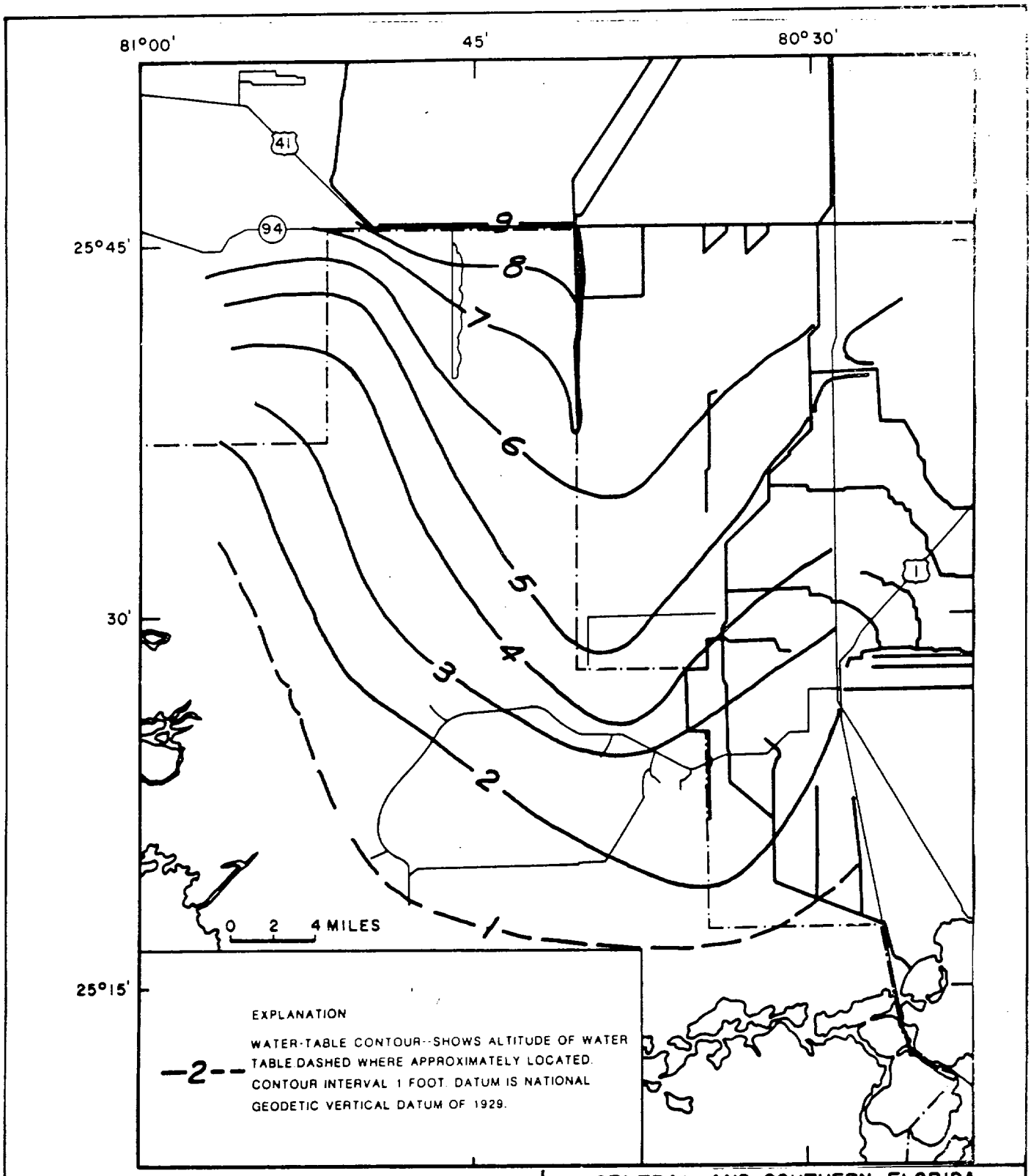

10
15
20
25
30
35
40
45
50
55
60
70
41
27
27
1
DEPTH IN FEET TO BASE OF
BISCAYNE AQUIFER
EAST EVERGLADES
SCALE IN MILES
0
5
10

FIGURE D-4



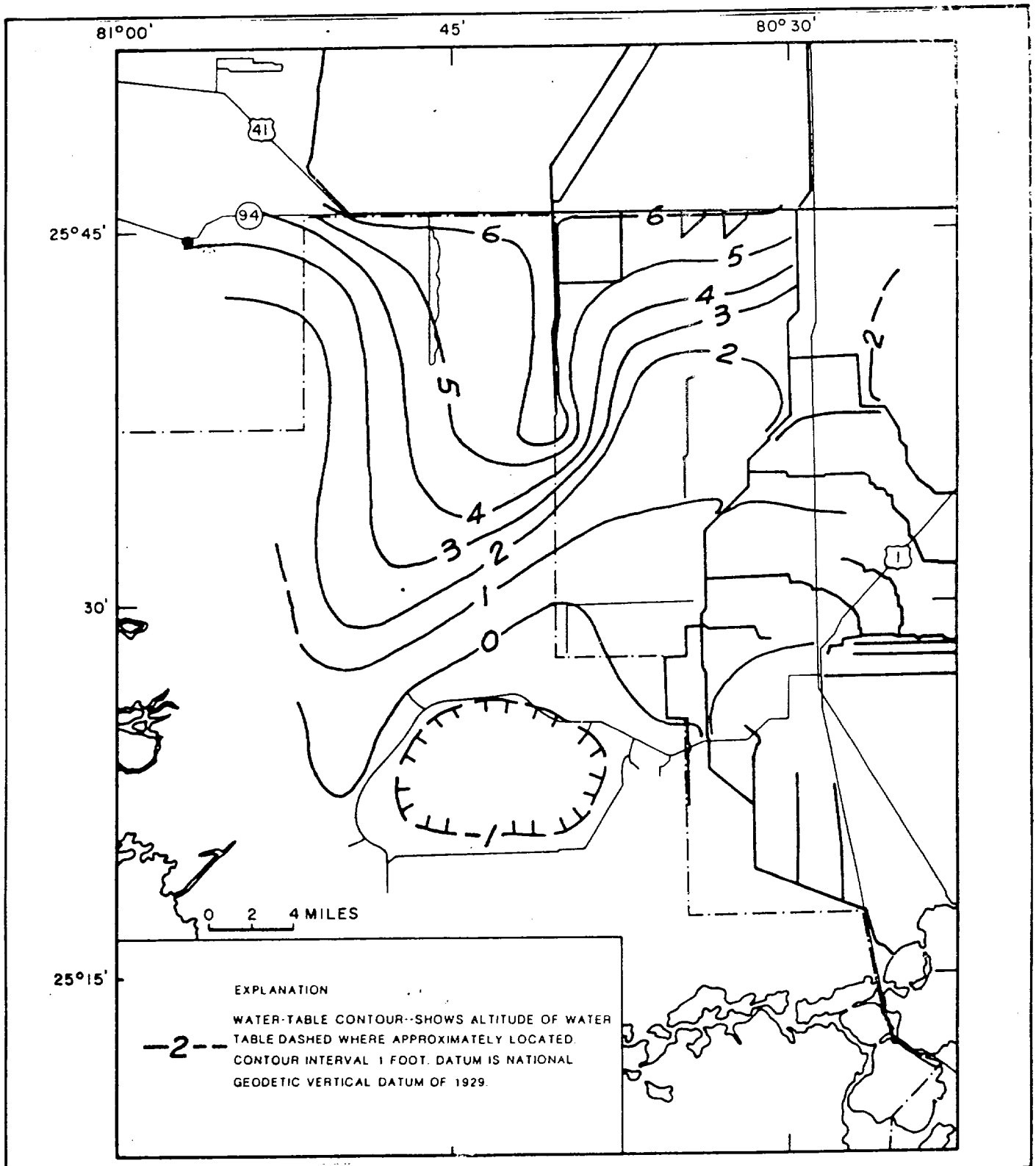
Source: Fig. 49 of ref. [14].

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

WET SEASON WATER TABLE
LEVELS, OCTOBER 27, 1976

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I , SUPP. 54, DATED:
FILE NO.

FIGURE D-5



Source: Fig. 54 of ref. [14].

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

DRY SEASON WATER TABLE
LEVELS, APRIL 24, 1979

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I , SUPP. 54, DATED:
FILE NO.

FIGURE D-6

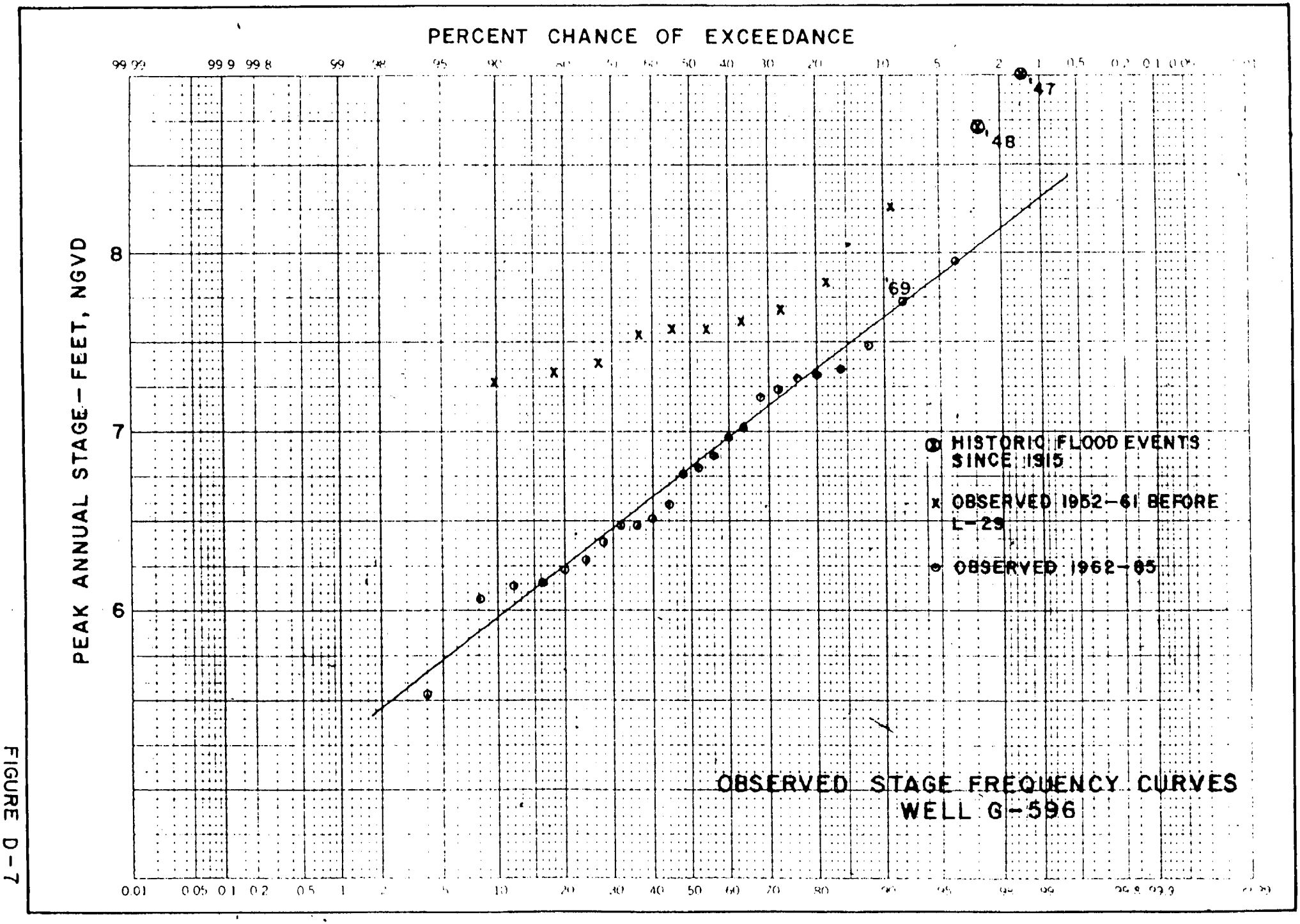

PERCENT CHANCE OF EXCEEDANCE
PEAK ANNUAL STAGE – FEET, NGVD
8
7
6
47
48
69
HISTORIC FLOOD EVENTS SINCE 1915
OBSERVED 1952–61 BEFORE L-29
OBSERVED 1962–85
OBSERVED STAGE FREQUENCY CURVES
WELL G-596

FIGURE D-7

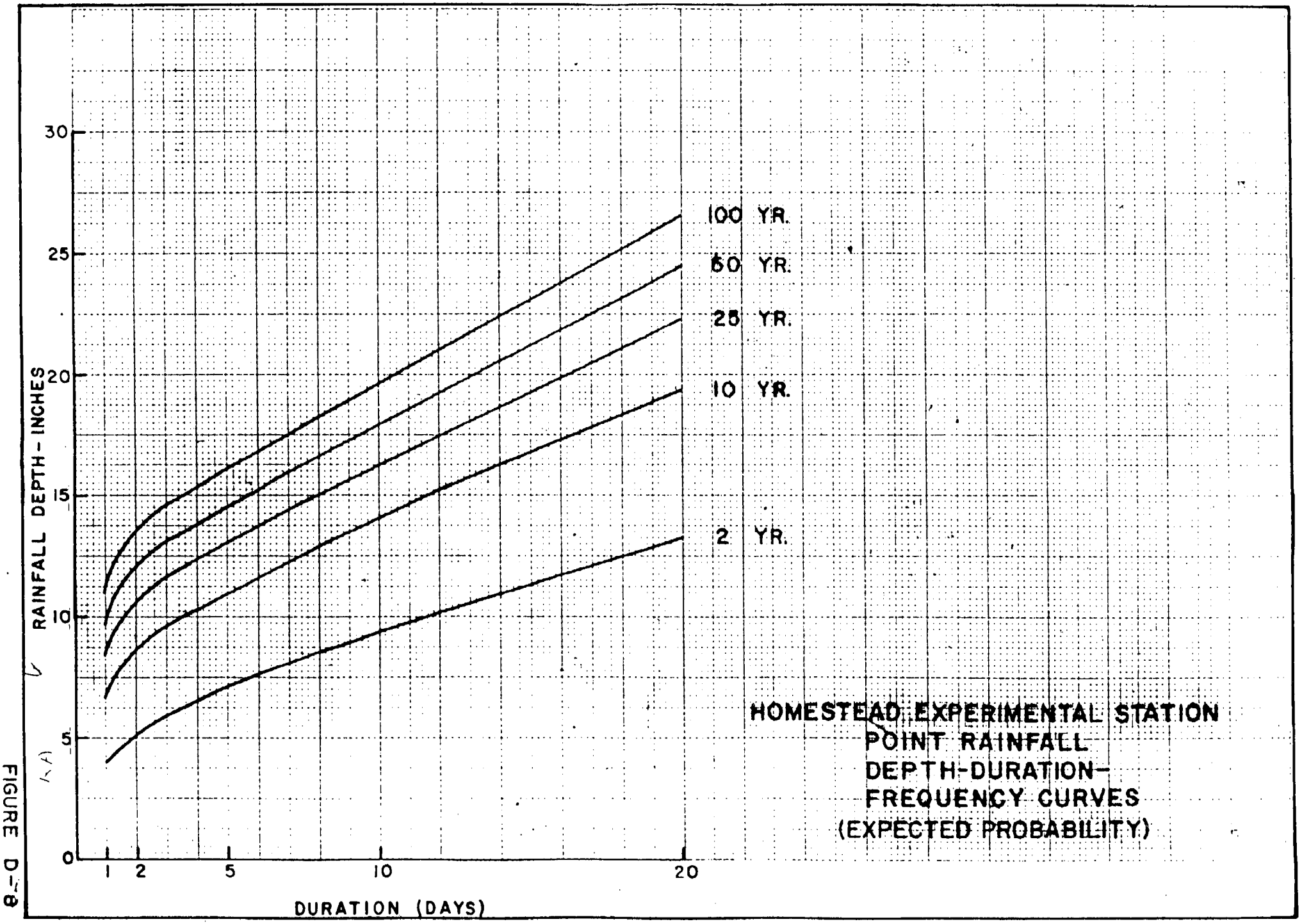

RAINFALL DEPTH - INCHES
30
25
20
15
10
5
0
1
2
5
10
20
DURATION (DAYS)
100 YR.
50 YR.
25 YR.
10 YR.
2 YR.
HOMESTEAD EXPERIMENTAL STATION
POINT RAINFALL
DEPTH-DURATION-
FREQUENCY CURVES
(EXPECTED PROBABILITY)

FIGURE D-8

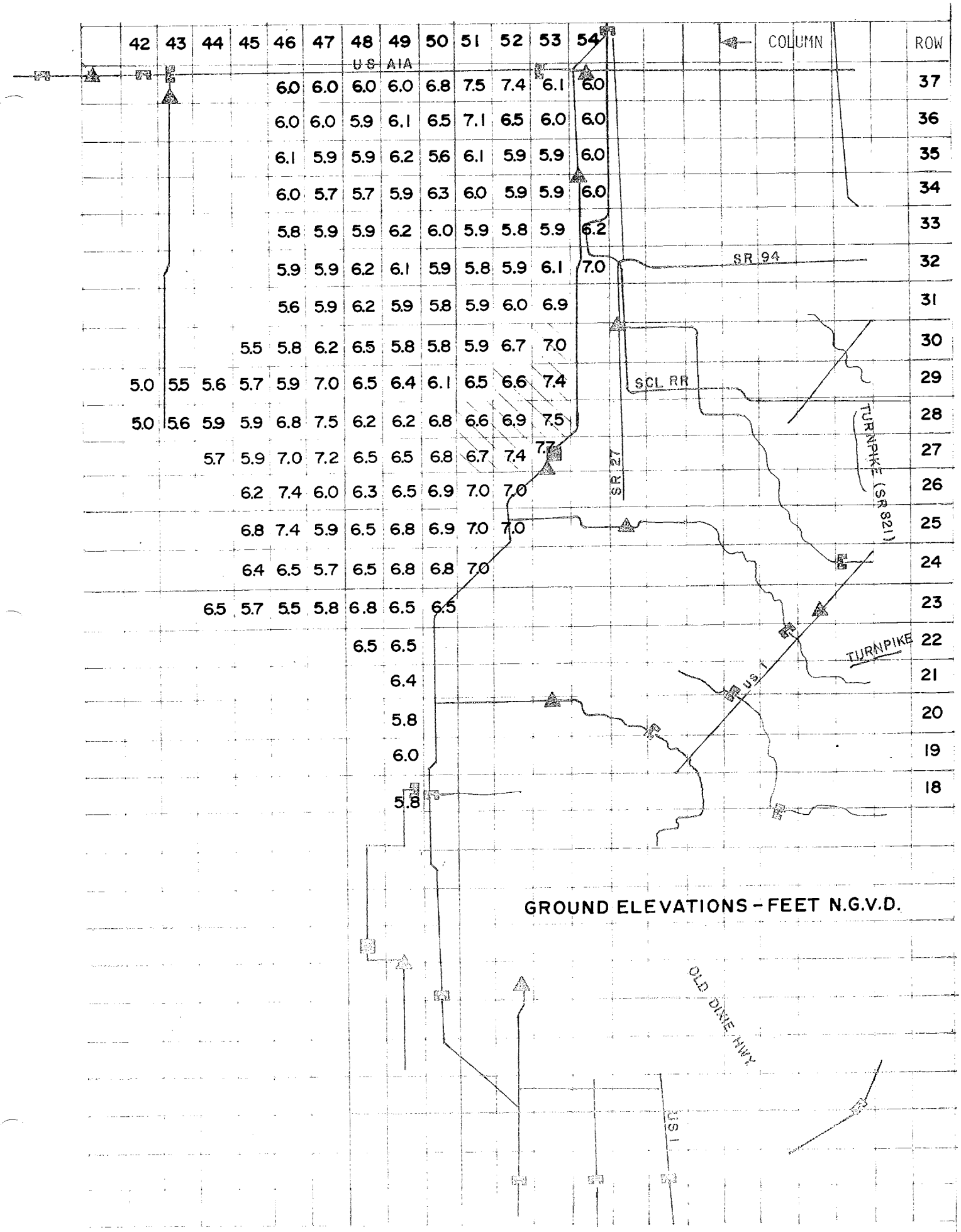
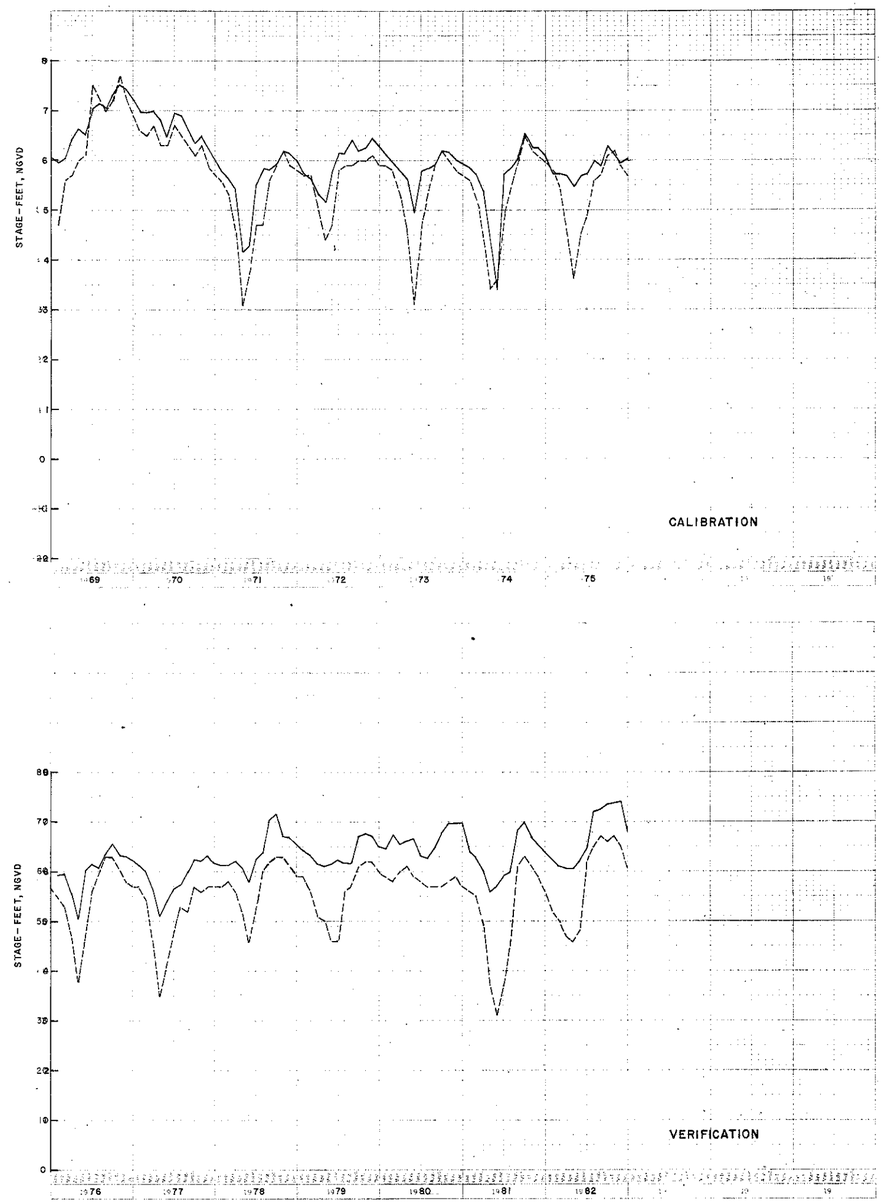

COLUMN
ROW
42 43 44 45 46 47 48 49 50 51 52 53 54
37 6.0 6.0 6.0 6.0 6.8 7.5 7.4 6.1 6.0
36 6.0 6.0 5.9 6.1 6.5 7.1 6.5 6.0 6.0
35 6.1 5.9 5.9 6.2 5.6 6.1 5.9 5.9 6.0
34 6.0 5.7 5.7 5.9 6.3 6.0 5.9 5.9 6.0
33 5.8 5.9 5.9 6.2 6.0 5.9 5.8 5.9 6.2
32 5.9 5.9 6.2 6.1 5.9 5.8 5.9 6.1 7.0
31 5.6 5.9 6.2 5.9 5.8 5.9 6.0 6.9
30 5.5 5.8 6.2 6.5 5.8 5.8 5.9 6.7 7.0
29 5.0 5.5 5.6 5.7 5.9 7.0 6.5 6.4 6.1 6.5 6.6 7.4
28 5.0 5.6 5.9 5.9 6.8 7.5 6.2 6.2 6.8 6.6 6.9 7.5
27 5.7 5.9 7.0 7.2 6.5 6.5 6.8 6.7 7.4 7.7
26 6.2 7.4 6.0 6.3 6.5 6.9 7.0 7.0
25 6.8 7.4 5.9 6.5 6.8 6.9 7.0 7.0
24 6.4 6.5 5.7 6.5 6.8 6.8 7.0
23 6.5 5.7 5.5 5.8 6.8 6.5 6.5
22 6.5 6.5
21 6.4
20 5.8
19 6.0
18 5.8
US AIA
SR 94
SCL RR
SR 27
TURNPIKE (SR 821)
TURNPIKE
US 1
OLD DIXIE HWY
US 1
GROUND ELEVATIONS - FEET N.G.V.D.

FIGURE D-9



LEGEND

HISTORIC

CALIBRATION OR VERIFICATION

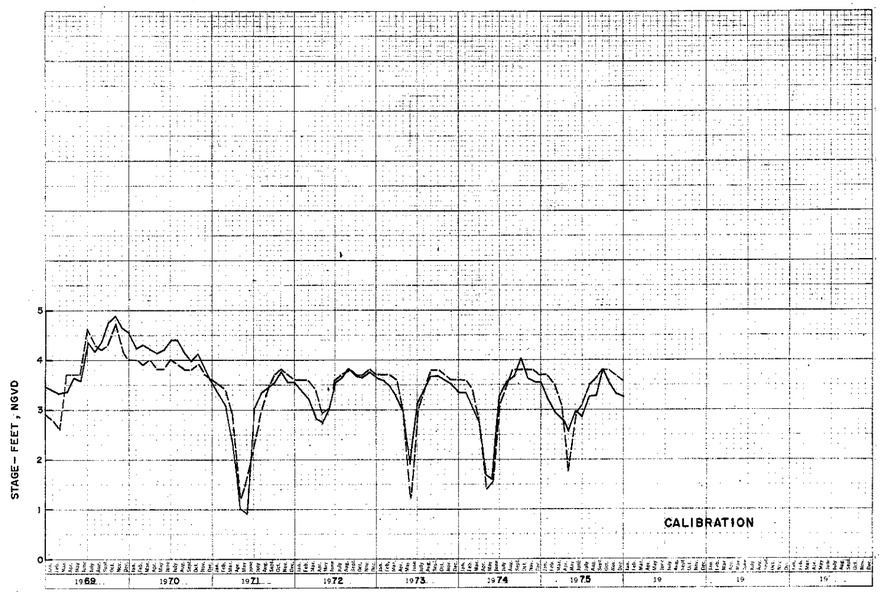
CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
GAGE P-33

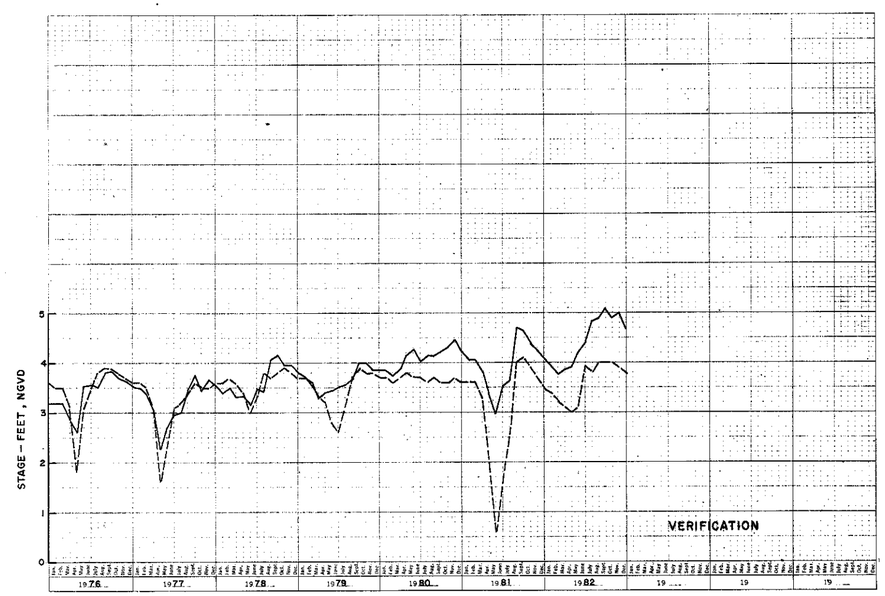
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-10





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CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

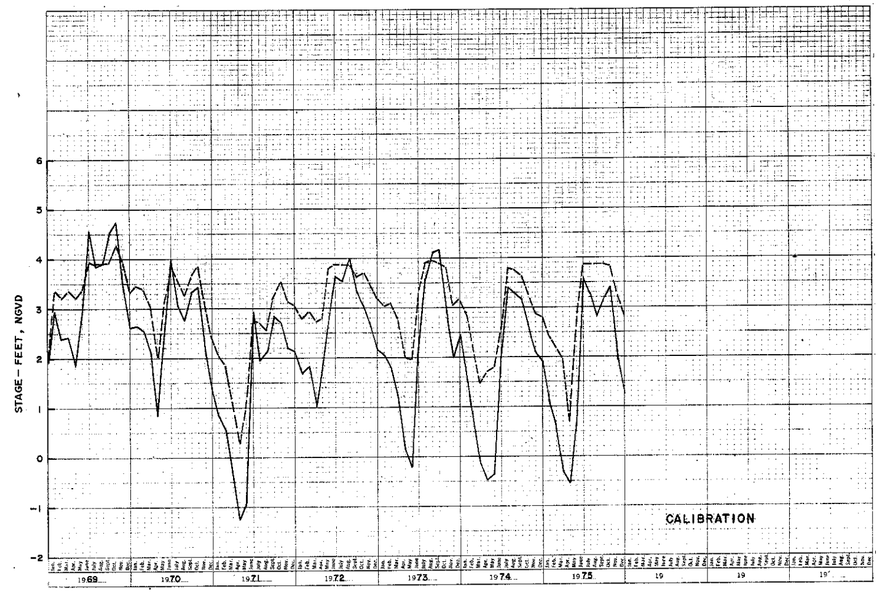
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
GAGE P-36

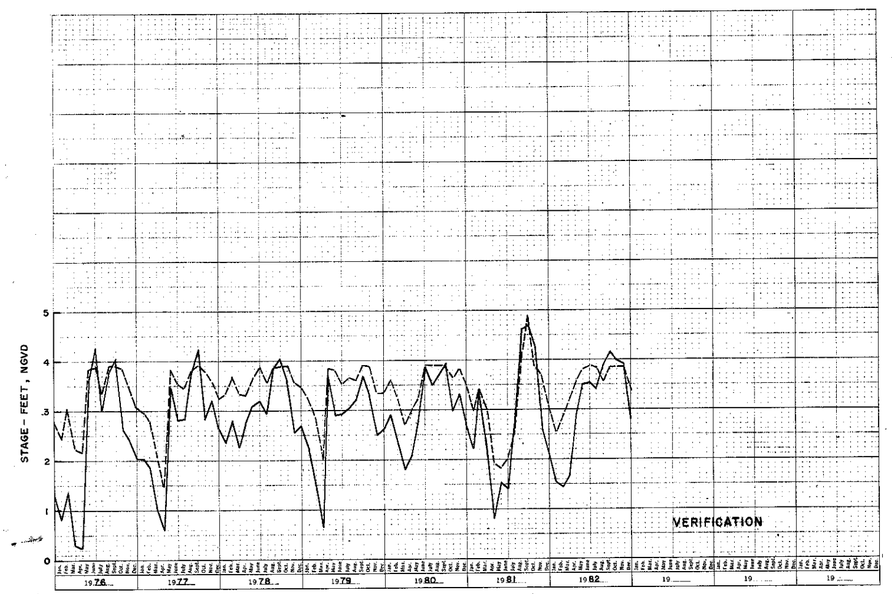
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP 54, DATED:
FILE NO.

FIGURE D-11





LEGEND

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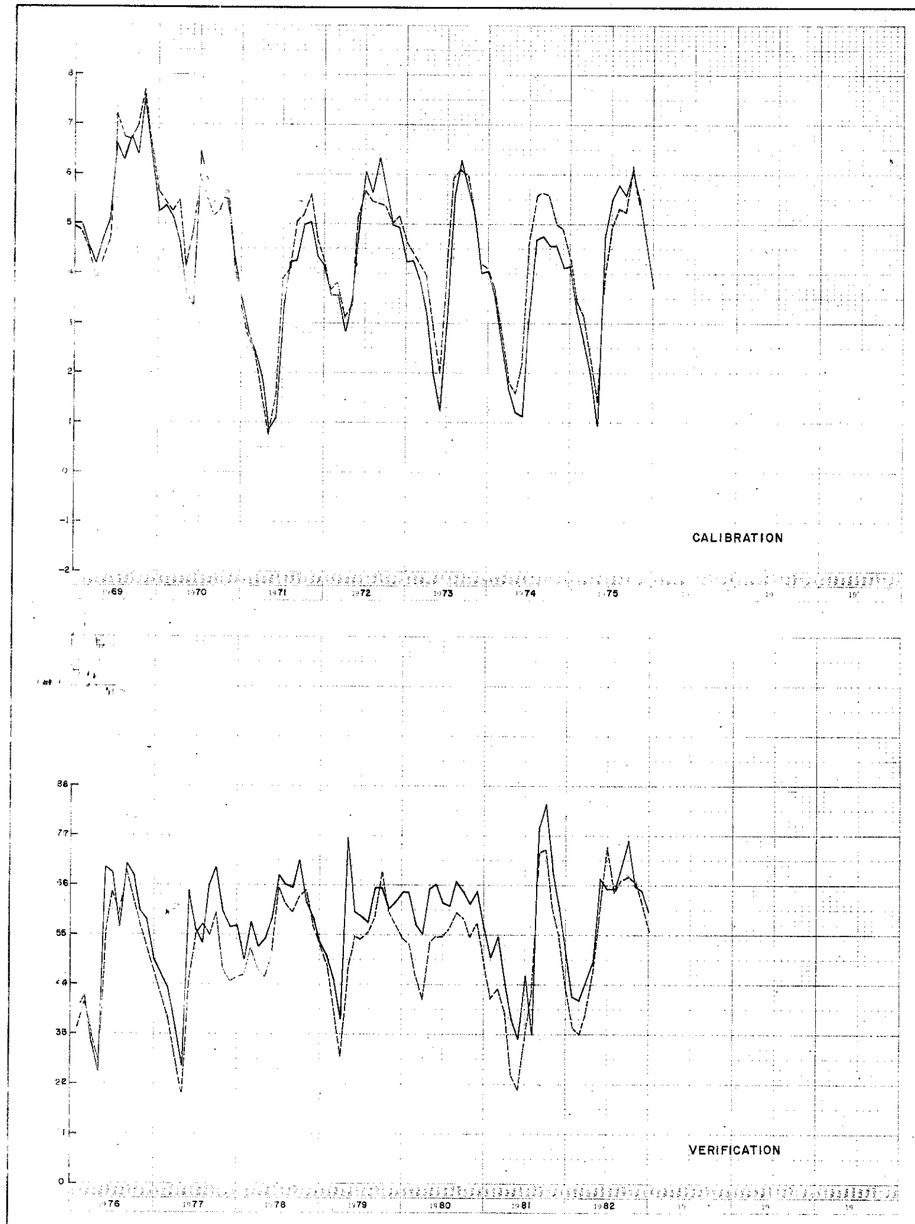
CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
GAGE TSHO

SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP 54, DATED:
FILE NO.

FIGURE D-12



LEGEND

HISTORIC

CALIBRATION OR VERIFICATION

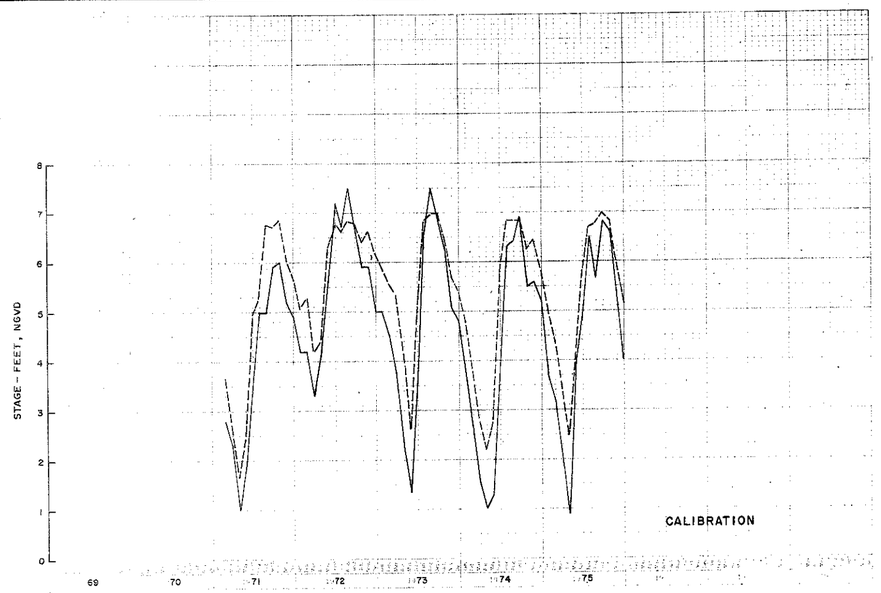
CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
WELL G-596

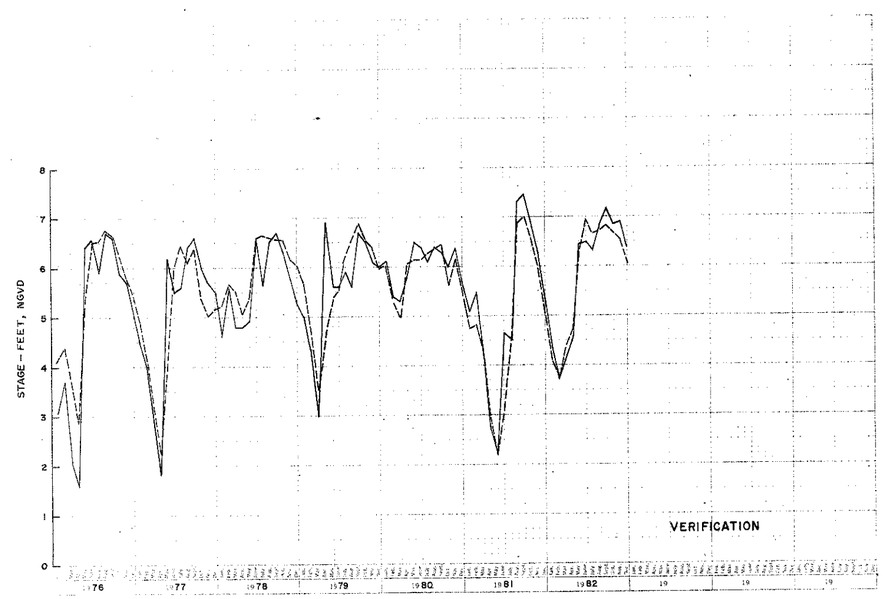
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-13





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CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

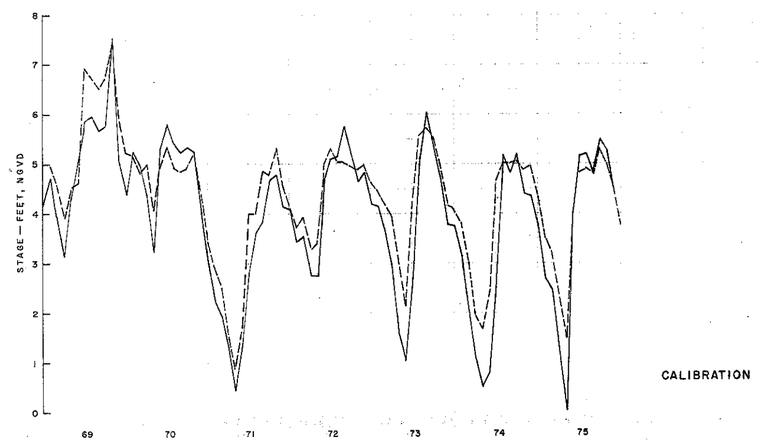
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
WELL G-1502

SCALES AS SHOWN

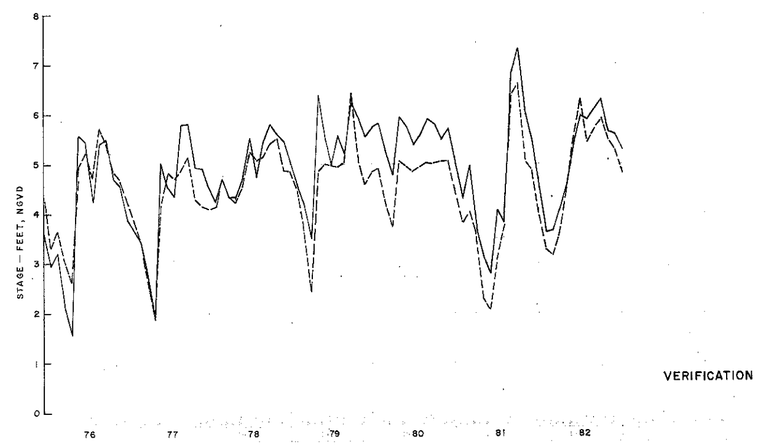
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP 54, DATED:
FILE NO

FIGURE D-14



CALIBRATION



VERIFICATION

LEGEND

HISTORIC

CALIBRATION OR VERIFICATION

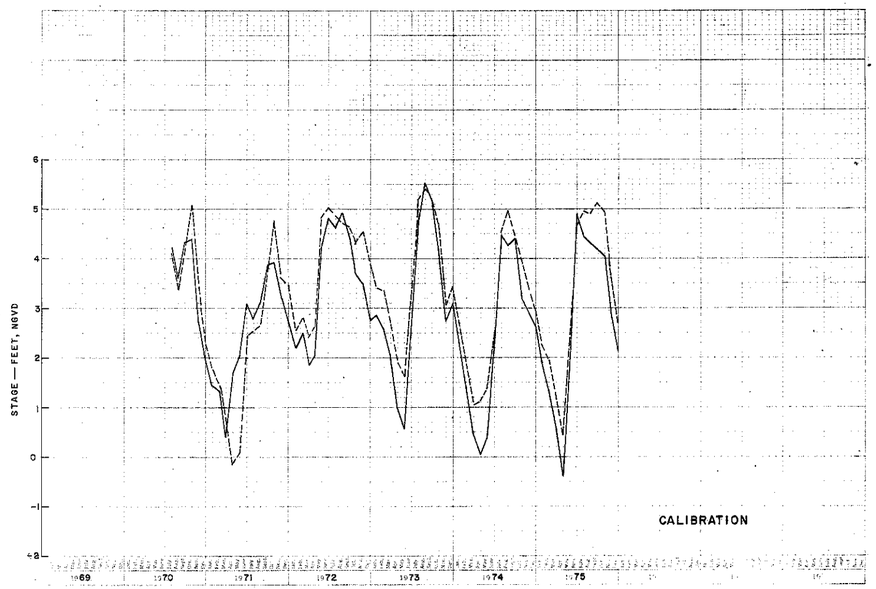
CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK
CALIBRATION — VERIFICATION
STAGE HYDROGRAPH
L-31N AT S-173

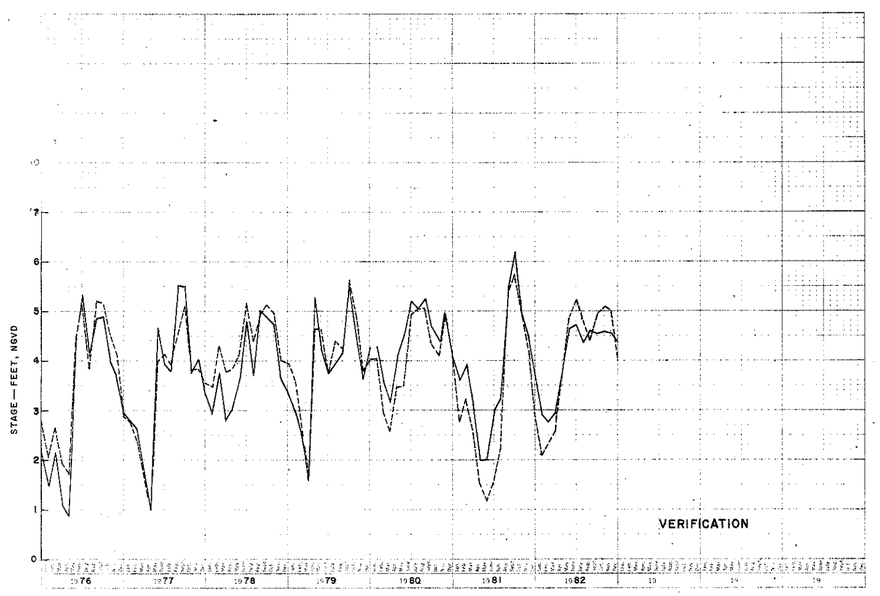
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO

FIGURE D-15





LEGEND

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– – – – CALIBRATION OR VERIFICATION

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

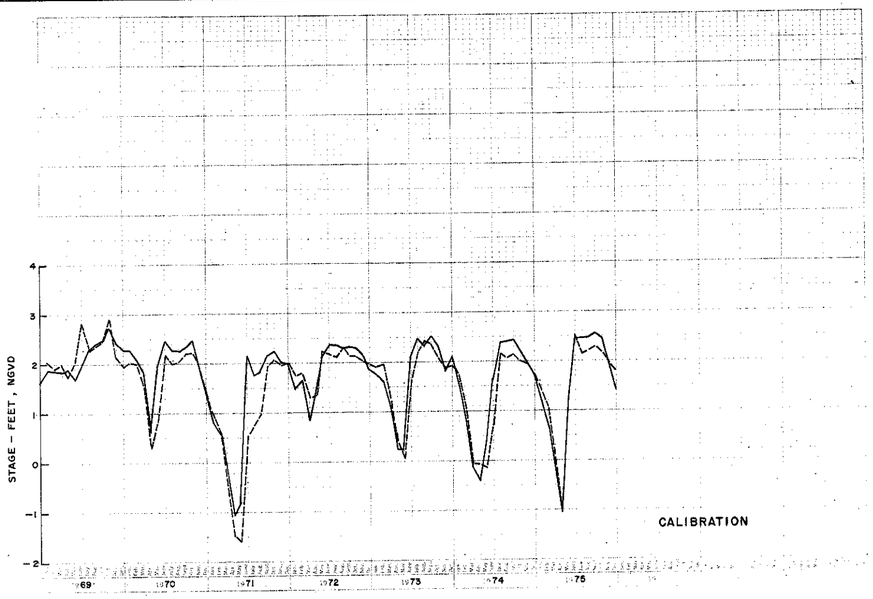
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
L-31N AT S-176

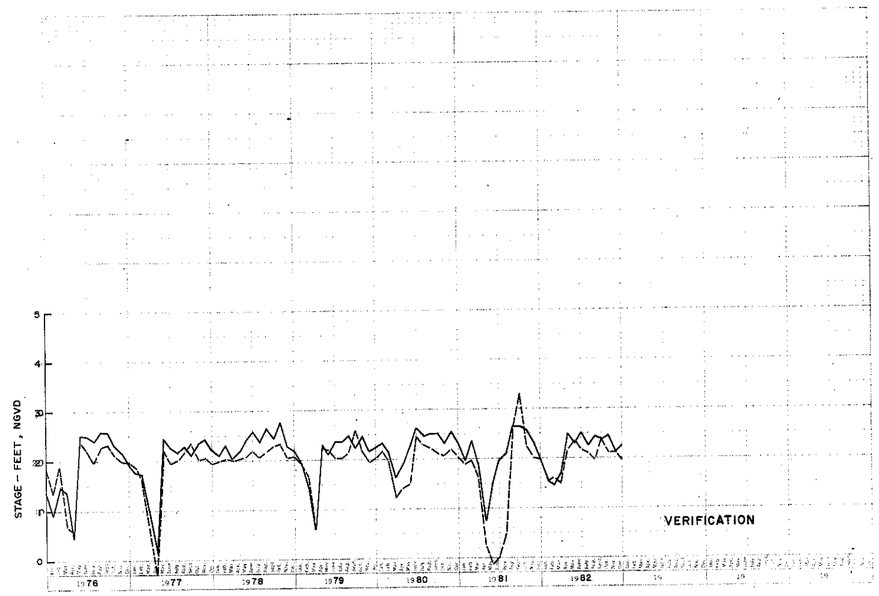
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP 54, DATED:
FILE NO.

FIGURE D-16





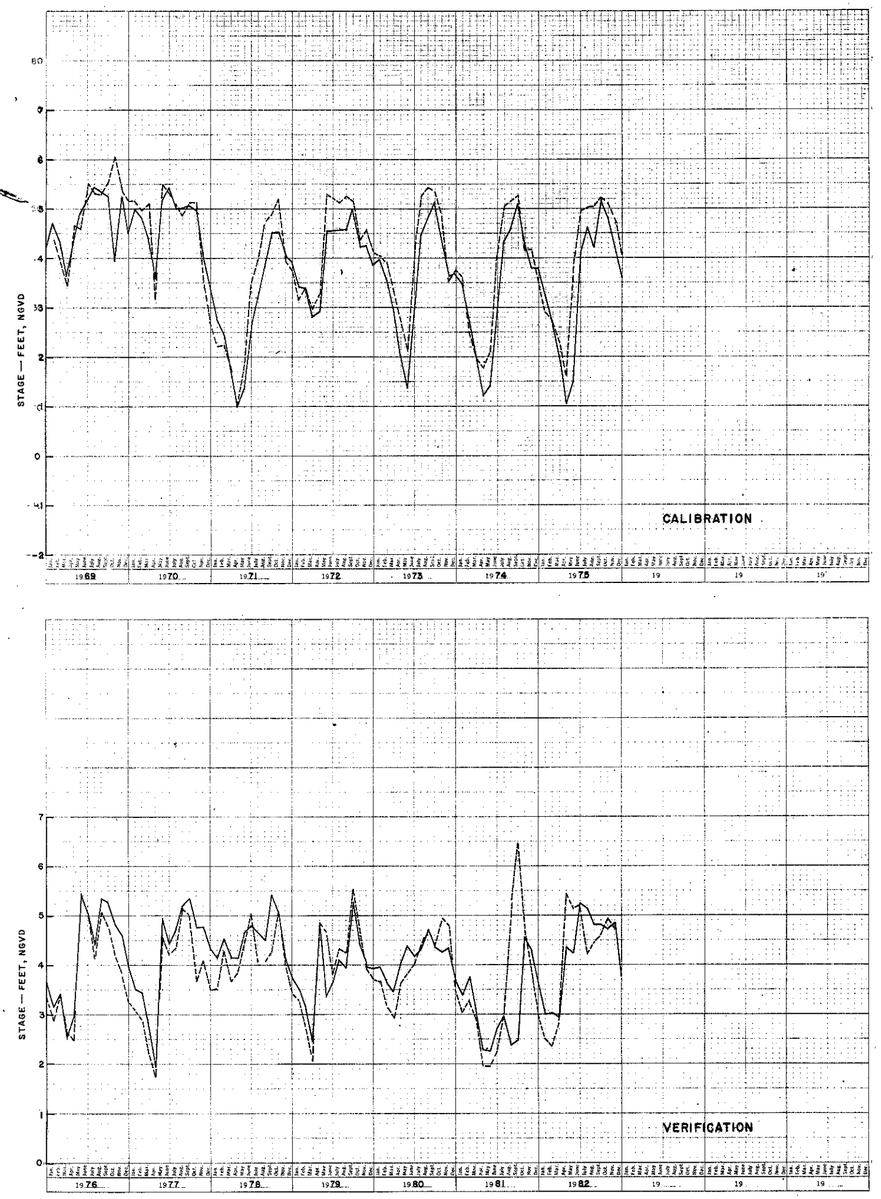
LEGEND

——— HISTORIC

– – – – CALIBRATION OR VERIFICATION

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
C-111E at S-18C
SCALES AS SHOWN
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-17



LEGEND

——— HISTORIC

– – – – CALIBRATION OR VERIFICATION

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

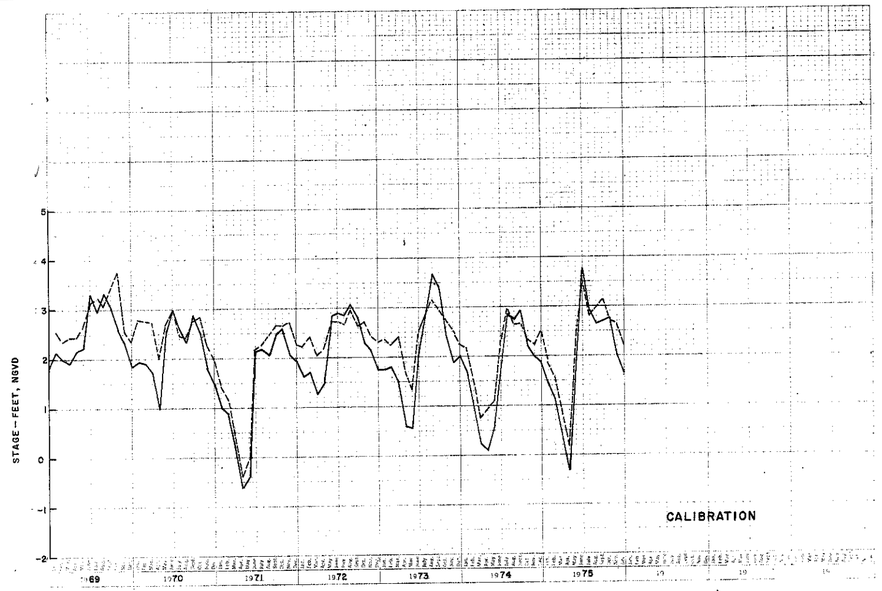
CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
C-IW AT S-149

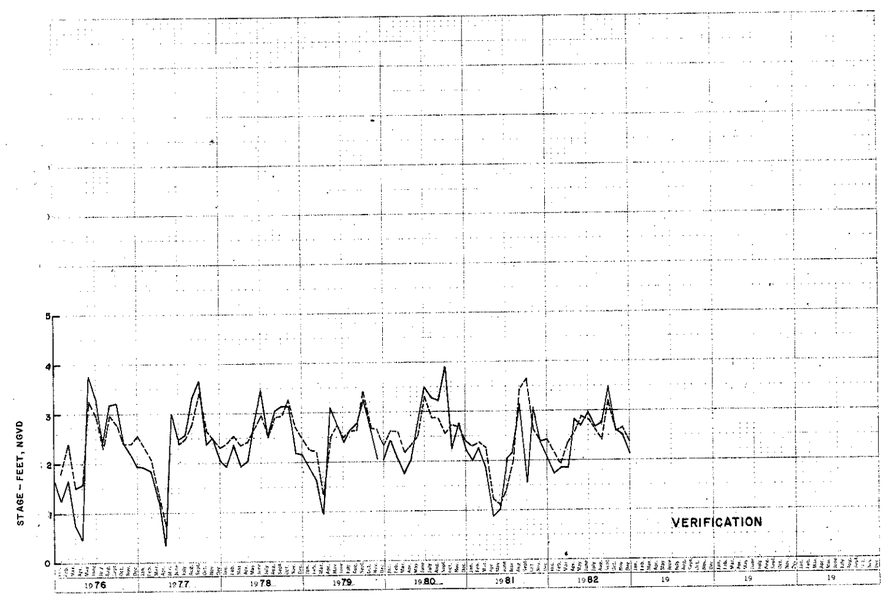
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-18





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——— HISTORIC

– – – – CALIBRATION OR VERIFICATION

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK

CALIBRATION – VERIFICATION
STAGE HYDROGRAPH
C-103 AT S-179

SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I, SUPP. 54, DATED:
FILE NO.

FIGURE D-19

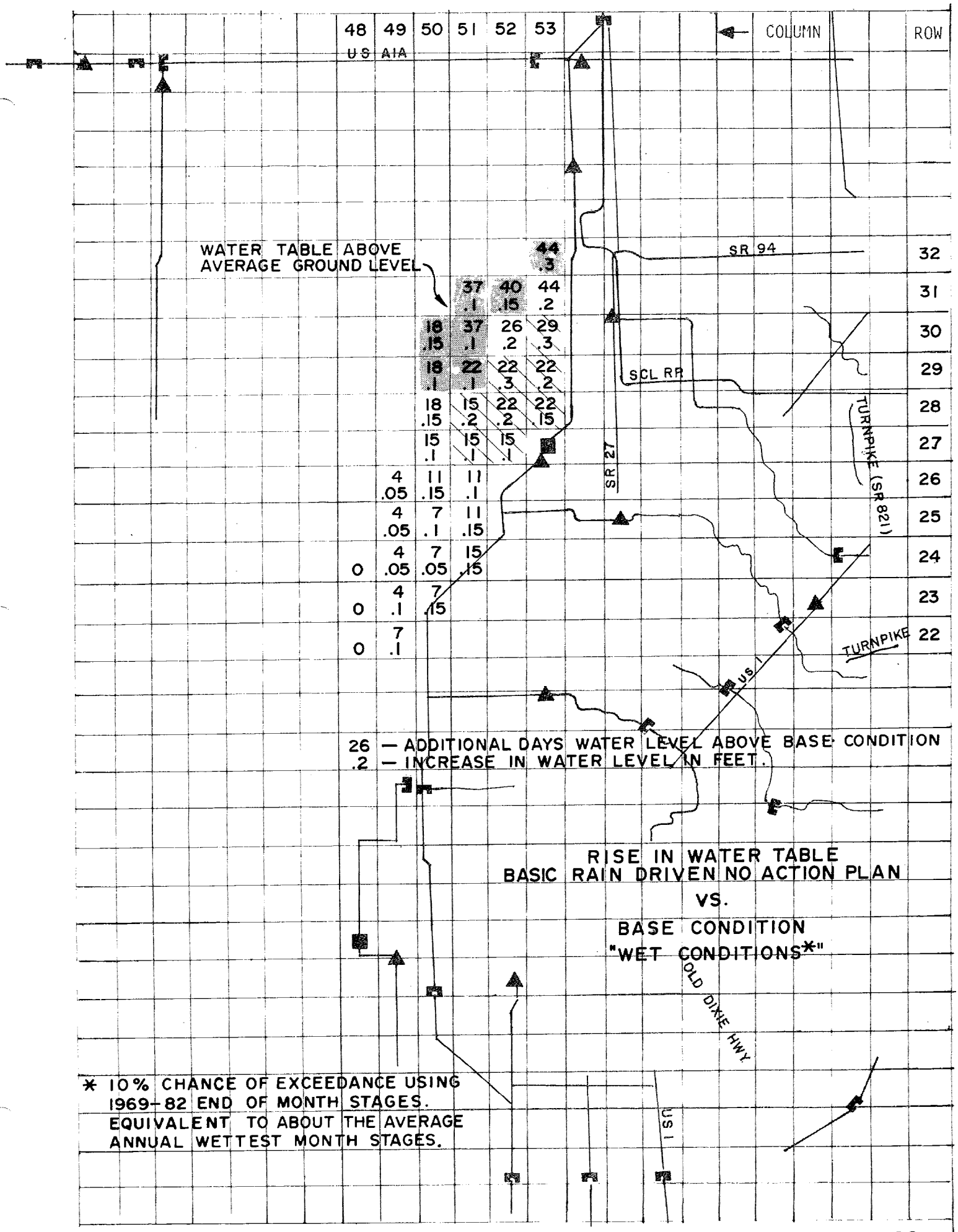

48 49 50 51 52 53
COLUMN
ROW
US AIA
WATER TABLE ABOVE AVERAGE GROUND LEVEL
44 .3
37 .1
40 .15
44 .2
18 .15
37 .1
26 .2
29 .3
18 .1
22 .1
22 .3
22 .2
18 .15
15 .2
22 .2
22 .15
15 .1
15 .1
15 .1
4 .05
11 .15
11 .1
4 .05
7 .1
11 .15
4 .05
7 .05
15 .15
0
4 .1
7 .15
0
7 .1
0
32
31
30
29
28
27
26
25
24
23
22
SR 94
SCL RR
SR 27
TURNPIKE (SR 821)
TURNPIKE
US 1
26 — ADDITIONAL DAYS WATER LEVEL ABOVE BASE CONDITION
.2 — INCREASE IN WATER LEVEL IN FEET.
RISE IN WATER TABLE
BASIC RAIN DRIVEN NO ACTION PLAN
VS.
BASE CONDITION
"WET CONDITIONS*"
OLD DIXIE HWY
US 1
* 10% CHANCE OF EXCEEDANCE USING 1969-82 END OF MONTH STAGES. EQUIVALENT TO ABOUT THE AVERAGE ANNUAL WETTEST MONTH STAGES.

FIGURE D-20

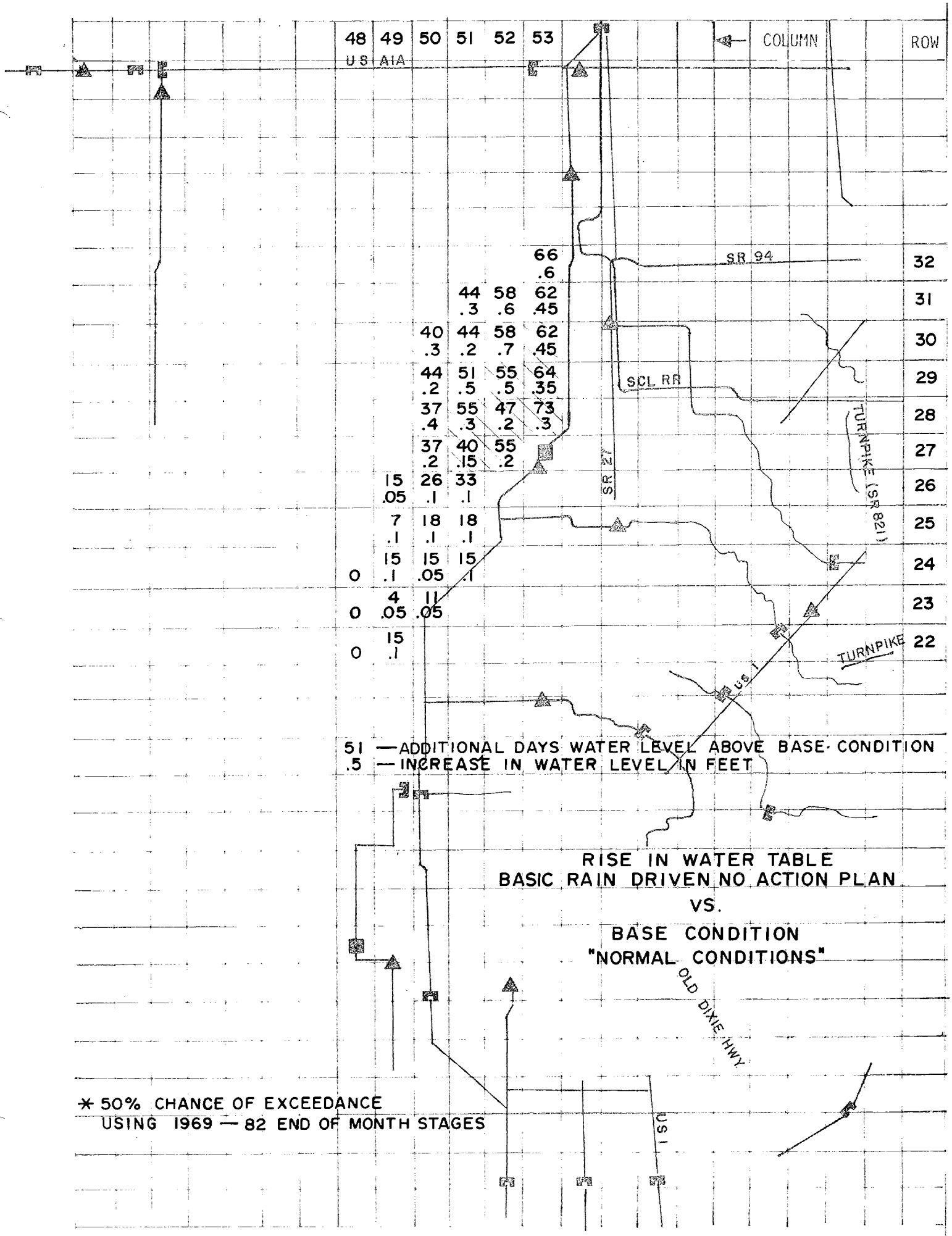

48 49 50 51 52 53
COLUMN
ROW
US AIA
66 .6
44 .3
58 .6
62 .45
40 .3
44 .2
58 .7
62 .45
44 .2
51 .5
55 .5
64 .35
37 .4
55 .3
47 .2
73 .3
37 .2
40 .15
55 .2
15 .05
26 .1
33 .1
7 .1
18 .1
18 .1
15 .1
15 .05
15 .1
0
4 .05
11 .05
0
15 .1
0
32
31
30
29
28
27
26
25
24
23
22
SR 94
SCL RR
SR 27
TURNPIKE (SR 821)
TURNPIKE
US 1
OLD DIXIE HWY
US 1
51 — ADDITIONAL DAYS WATER LEVEL ABOVE BASE CONDITION
.5 — INCREASE IN WATER LEVEL IN FEET
RISE IN WATER TABLE
BASIC RAIN DRIVEN NO ACTION PLAN
VS.
BASE CONDITION
"NORMAL CONDITIONS"
* 50% CHANCE OF EXCEEDANCE
USING 1969 — 82 END OF MONTH STAGES

FIGURE D-21

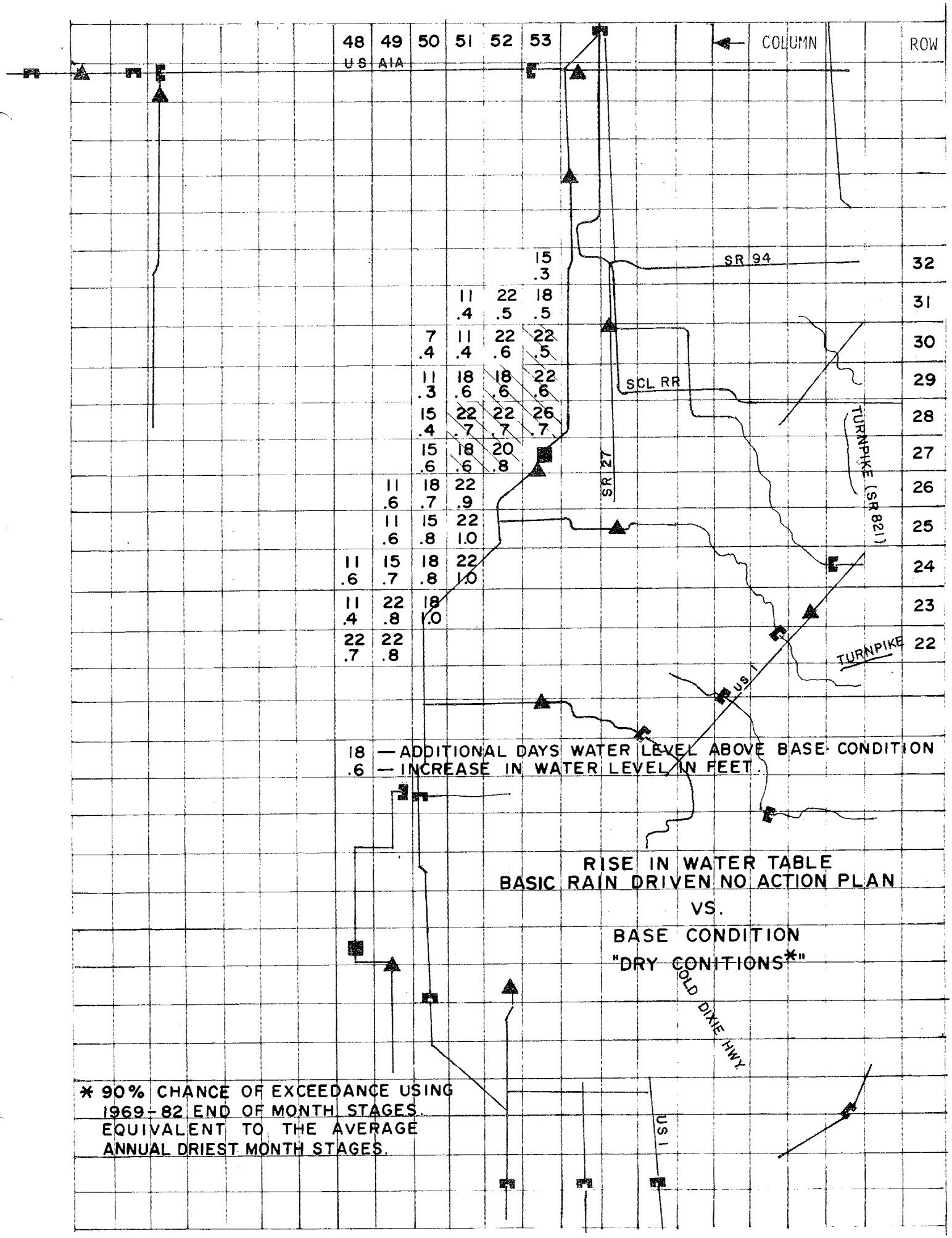

48 49 50 51 52 53
COLUMN
ROW
US AIA
SR 94
SCL RR
SR 27
TURNPIKE (SR 821)
TURNPIKE
US 1
COLD DIXIE HWY.
32 31 30 29 28 27 26 25 24 23 22
18 — ADDITIONAL DAYS WATER LEVEL ABOVE BASE CONDITION
.6 — INCREASE IN WATER LEVEL IN FEET.
RISE IN WATER TABLE
BASIC RAIN DRIVEN NO ACTION PLAN
VS.
BASE CONDITION
"DRY CONITIONS*"
* 90% CHANCE OF EXCEEDANCE USING
1969-82 END OF MONTH STAGES.
EQUIVALENT TO THE AVERAGE
ANNUAL DRIEST MONTH STAGES.

FIGURE D-22

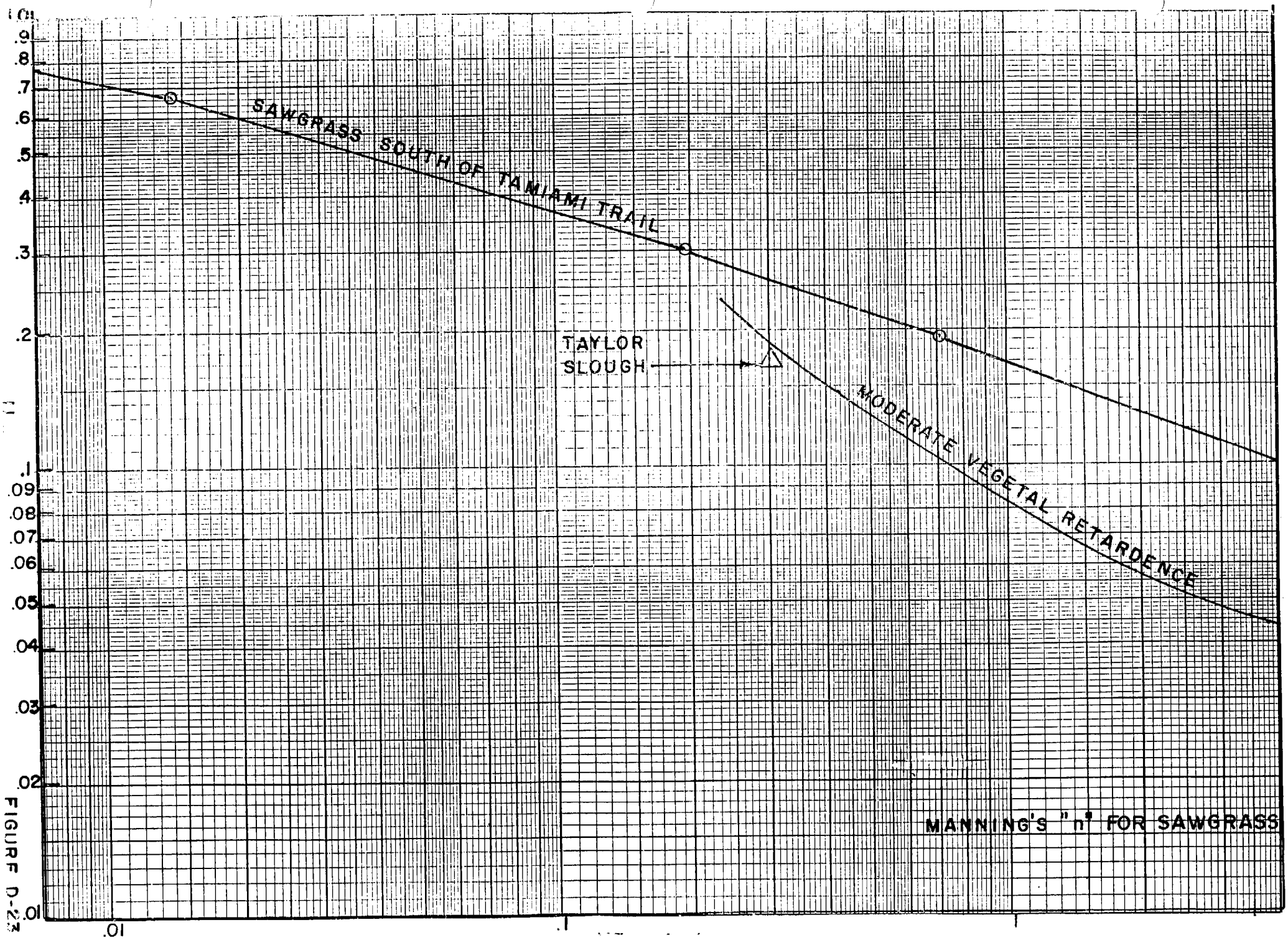

1.0
.9
.8
.7
.6
.5
.4
.3
.2
.1
.09
.08
.07
.06
.05
.04
.03
.02
.01
.01
1
SAWGRASS SOUTH OF TAMIAMI TRAIL
TAYLOR SLOUGH
MODERATE VEGETAL RETARDENCE
MANNING'S "n" FOR SAWGRASS

FIGURE D-23

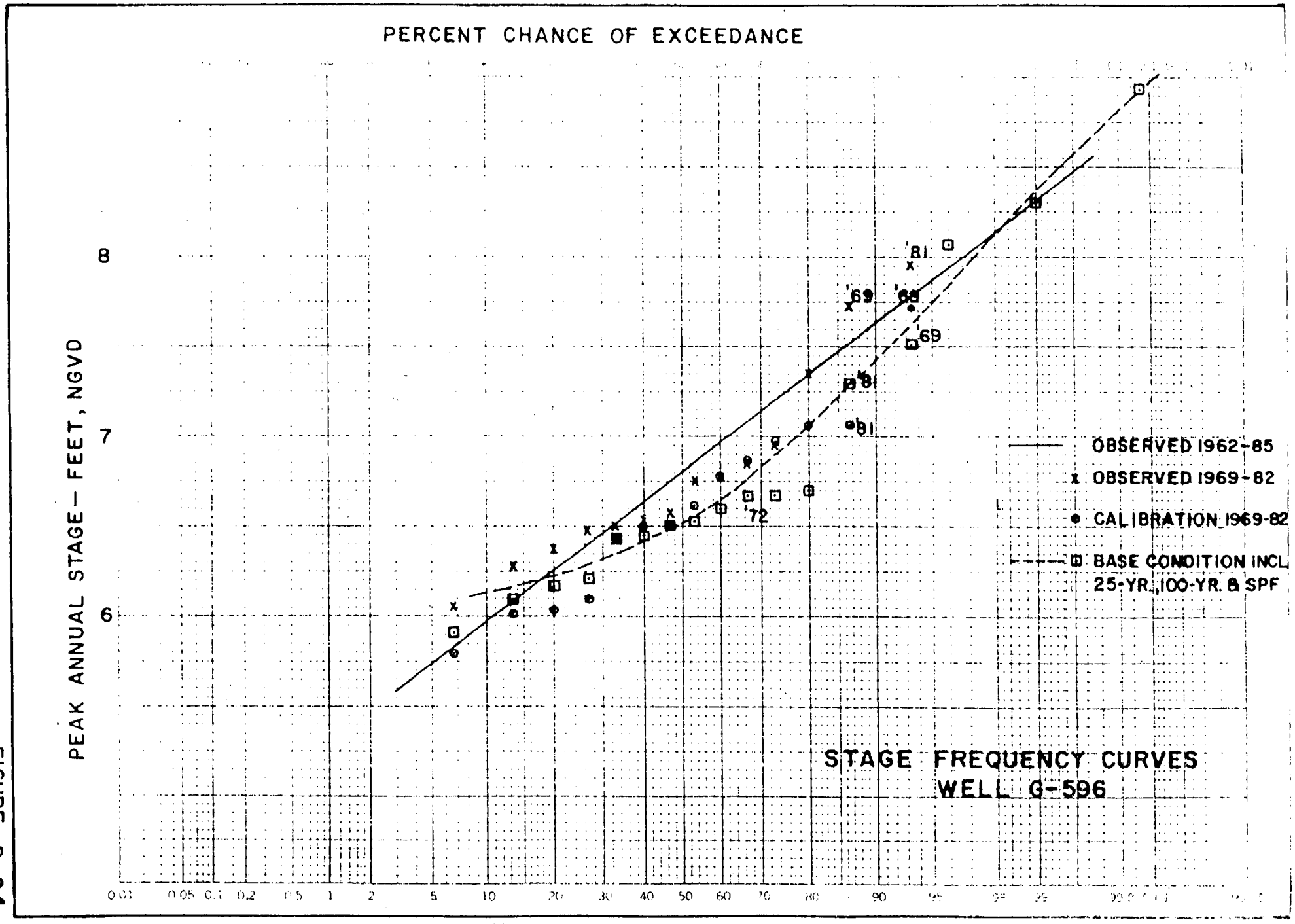

PERCENT CHANCE OF EXCEEDANCE
PEAK ANNUAL STAGE— FEET, NGVD
8
7
6
0.03
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.8
'81
'69
'69
'69
'81
'72
OBSERVED 1962-85
OBSERVED 1969-82
CALIBRATION 1969-82
BASE CONDITION INCL
25-YR., 100-YR. & SPF
STAGE FREQUENCY CURVES
WELL G-596

FIGURE D-24

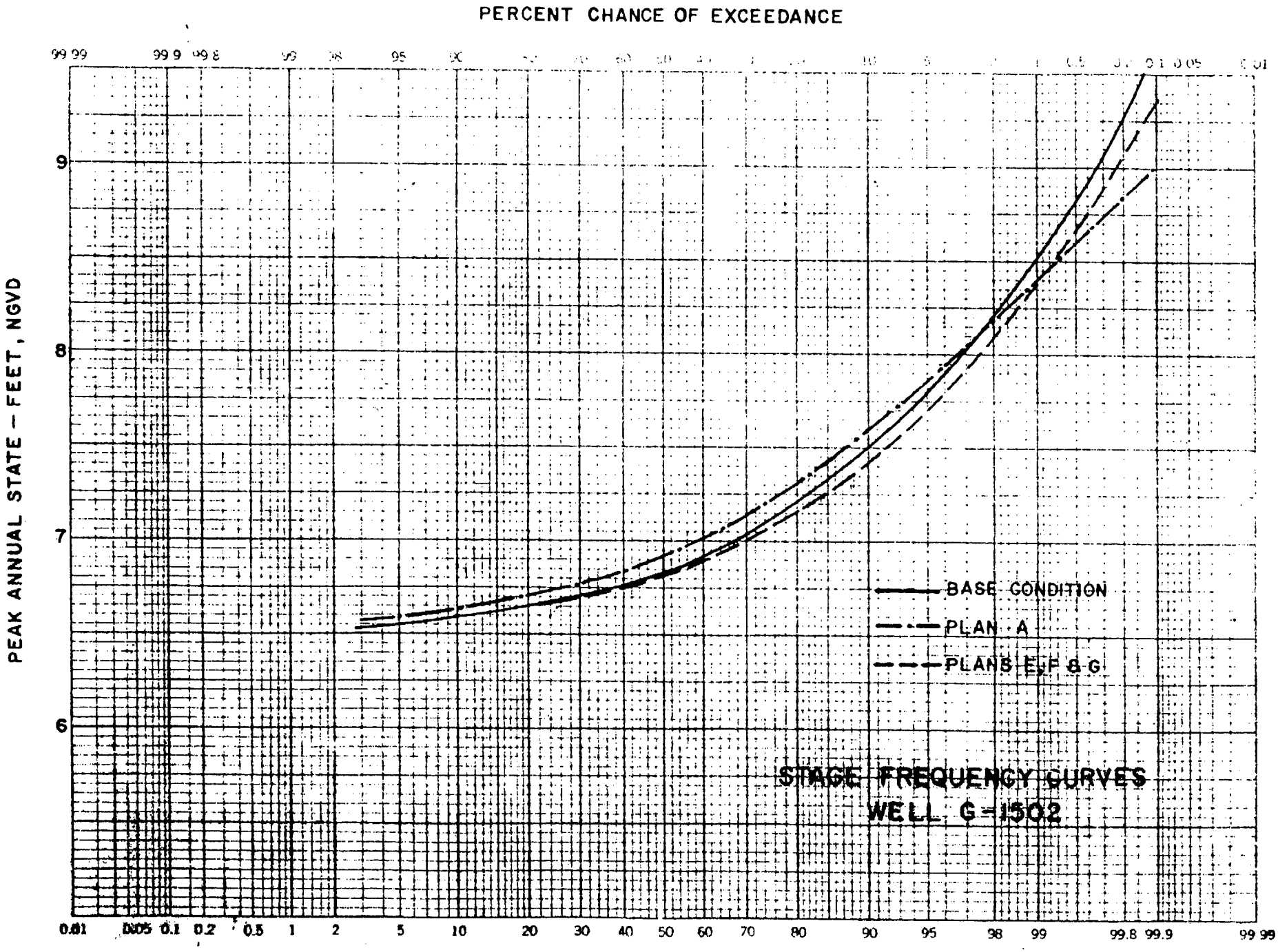
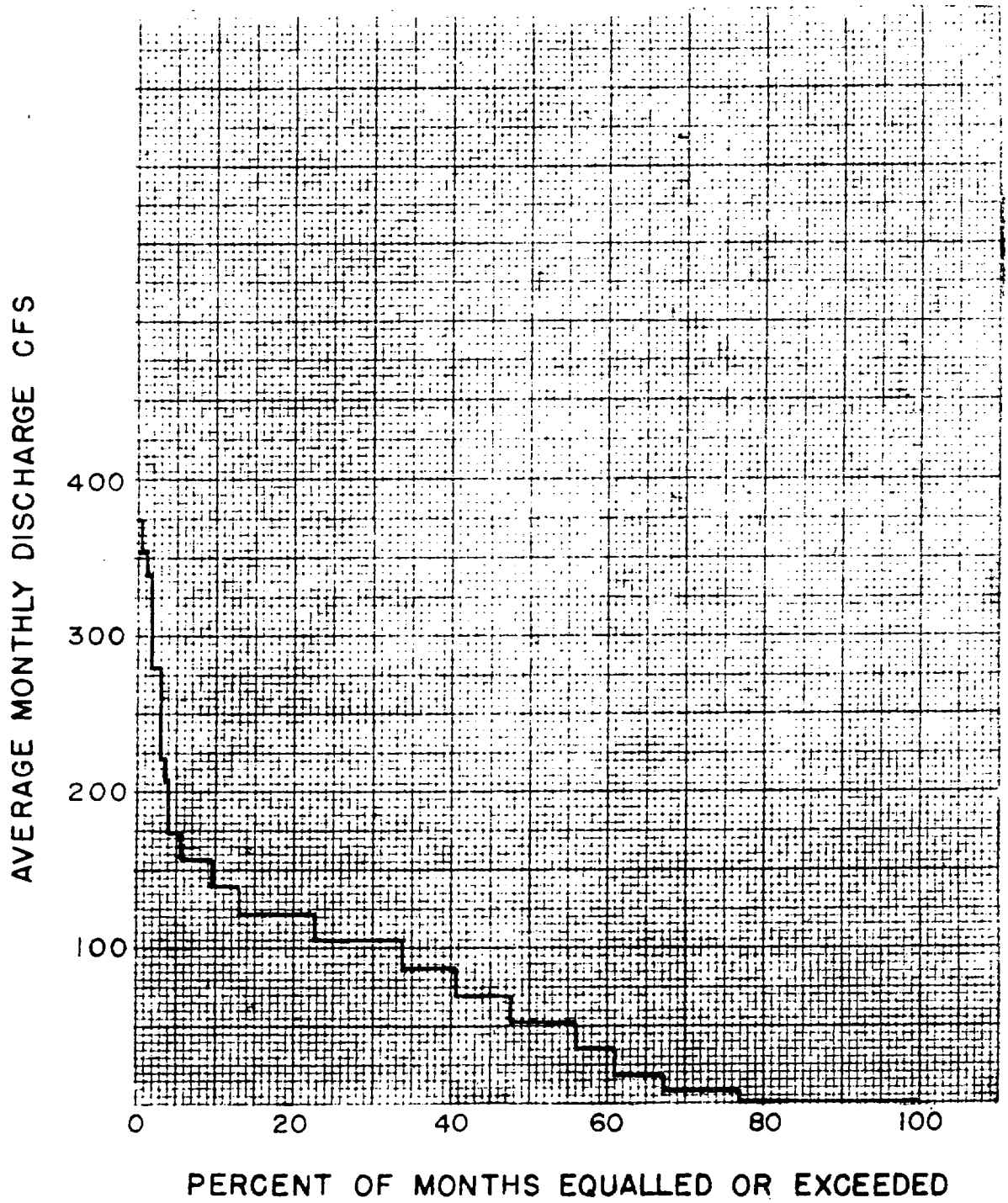
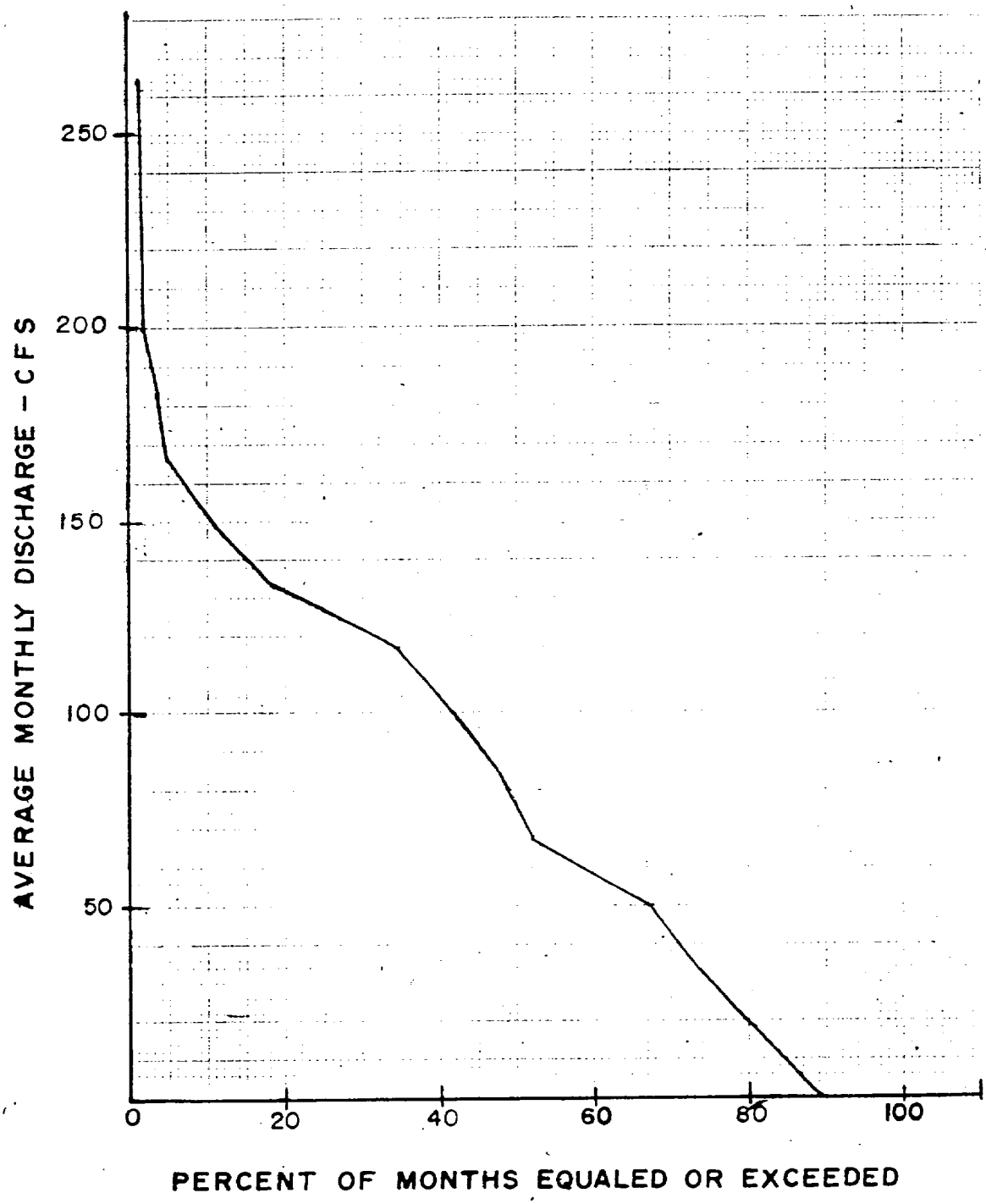

PERCENT CHANCE OF EXCEEDANCE
PEAK ANNUAL STATE – FEET, NGVD
9
8
7
6
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.8
99.9
99.99
BASE CONDITION
PLAN A
PLANS E, F & G
STAGE FREQUENCY CURVES
WELL G-1502

FIGURE D-25



DISCHARGE DURATION CURVE
AT S-357
(PLAN G-533 CFS PUMP)

FIGURE D-26



DISCHARGE DURATION CURVE

AT S-356

(950 CFS PUMP STATION - PLANS E, F & G)

FIGURE D-27

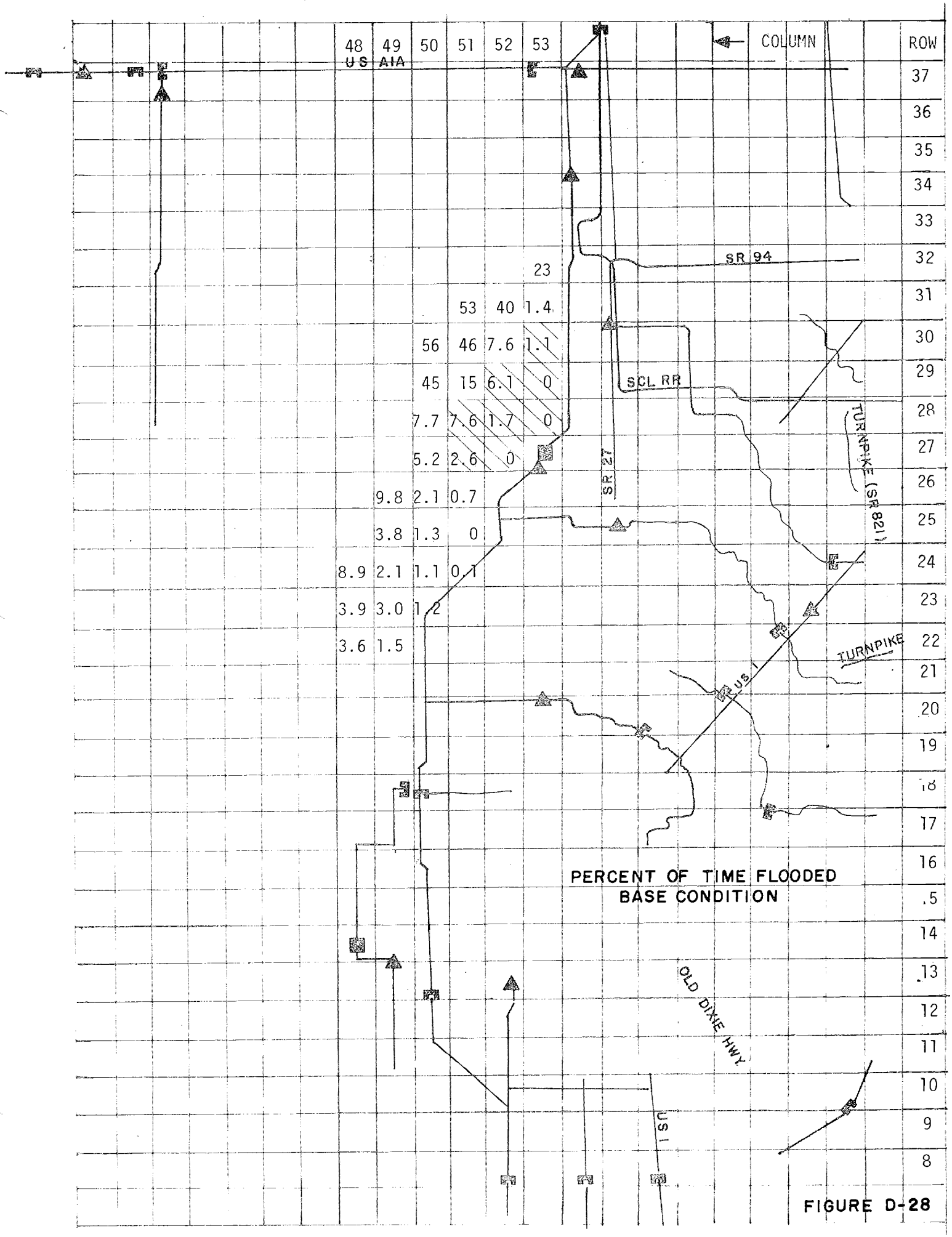

COLUMN
ROW
48 49 50 51 52 53
US AIA
SR 94
23
53 40 1.4
56 46 7.6 1.1
45 15 6.1 0
7.7 7.6 1.7 0
5.2 2.6 0
9.8 2.1 0.7
3.8 1.3 0
8.9 2.1 1.1 0.1
3.9 3.0 1.2
3.6 1.5
SCL RR
SR 27
TURNPIKE (SR 821)
TURNPIKE
US 1
OLD DIXIE HWY
US 1
PERCENT OF TIME FLOODED
BASE CONDITION

FIGURE D-28

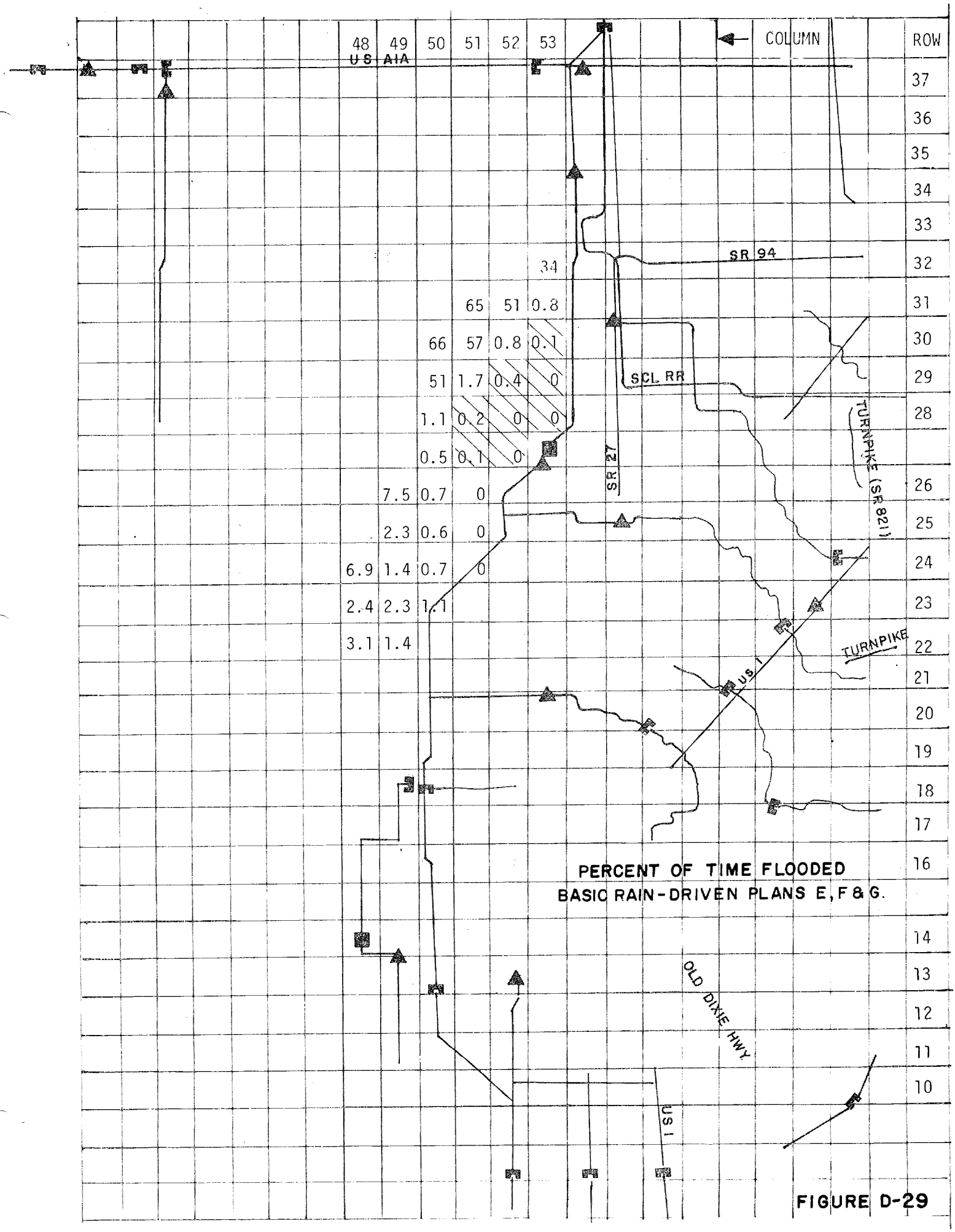
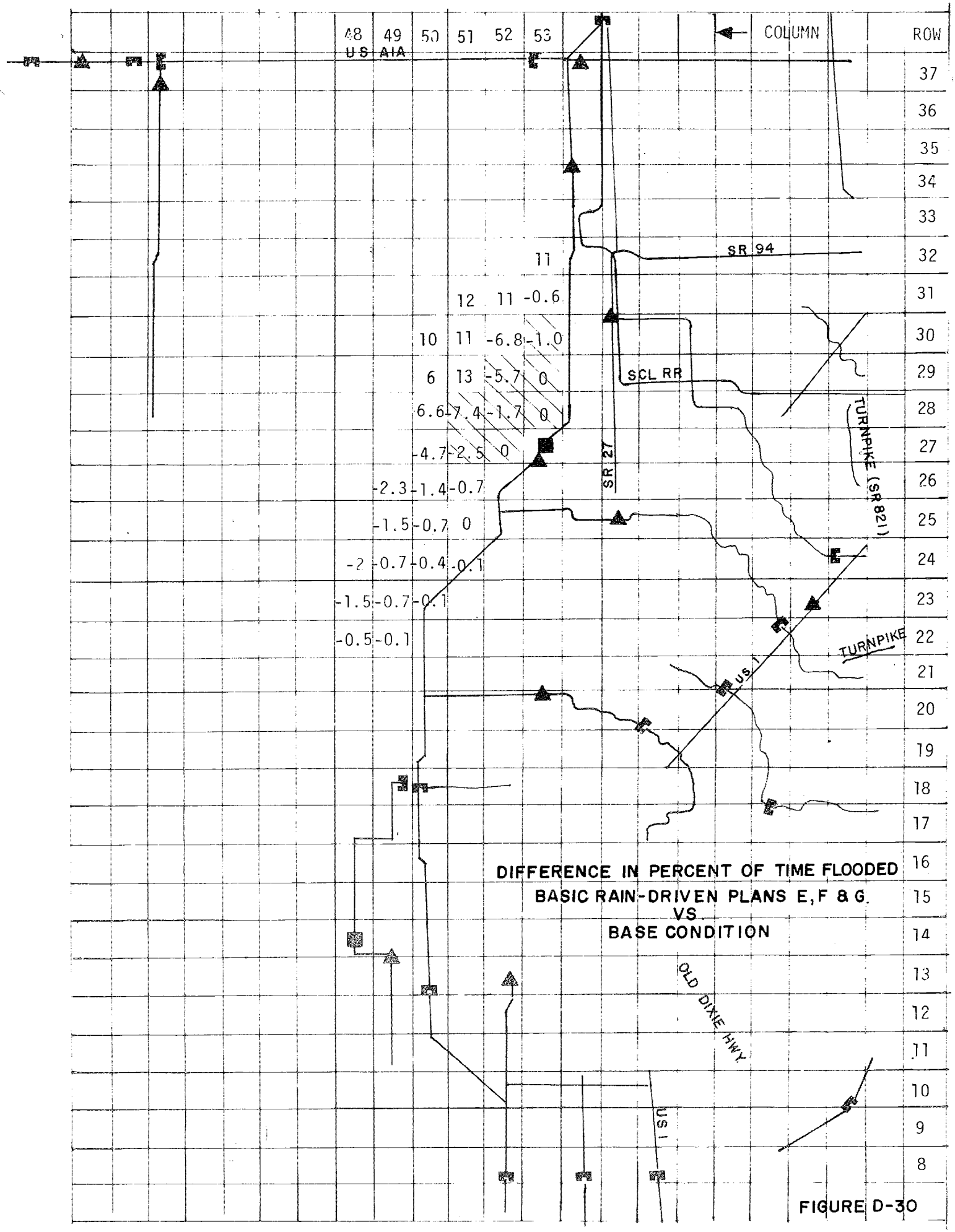

COLUMN
ROW
48
49
50
51
52
53
US AIA
SR 94
SCL RR
SR 27
TURNPIKE (SR 821)
TURNPIKE
US 1
OLD DIXIE HWY
US 1
34
65 51 0.8
66 57 0.8 0.1
51 1.7 0.4 0
1.1 0.2 0 0
0.5 0.1 0
7.5 0.7 0
2.3 0.6 0
6.9 1.4 0.7 0
2.4 2.3 1.1
3.1 1.4
PERCENT OF TIME FLOODED
BASIC RAIN-DRIVEN PLANS E, F & G.

FIGURE D-29



DIFFERENCE IN PERCENT OF TIME FLOODED
BASIC RAIN-DRIVEN PLANS E, F & G.
VS.
BASE CONDITION

FIGURE D-30

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX E

HYDROLOGIC ANALYSIS OF ALTERNATIVE PLANS

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

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APPENDIX E

HYDROLOGIC ANALYSIS OF ALTERNATIVE PLANS

A. MODIFIED MINIMUM DELIVERY vs. BASE CONDITION

1. General. This analysis is based on a comparison of results from the South Florida Water Management Model for the Modified Minimum Delivery alternative vs. the Base Condition.

2. Lake Okeechobee.

a. Water Budget. The Lake Okeechobee/EAA hydrologic relationship is not significantly impacted by the Modified Minimum Delivery alternative. Water supply and regulatory releases from Lake Okeechobee are not greatly affected by the Modified Minimum Delivery alternative.

b. Stage and Storage. Because the total inflows and outflows to and from Lake Okeechobee are not affected by this alternative, the stages and storages are also not affected. Figure E-1 shows a storage hydrograph for the 1969 through 1982 simulation period. Figures E-2 and E-3 show the stage hydrograph and stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

3. Everglades Agricultural Area Water Supply Runoff. Neither water supply or runoff requirements for the EAA are affected by the Modified Minimum Delivery alternative. A typical stage hydrograph and stage-duration curve for the Modified Minimum Delivery vs. Base Condition are shown on Figures E-4 and E-5. The ground elevation in the area is approximately 7.8 ft., NGVD. The stage hydrograph and stage-duration curve shows little differences under these two plans. Water was only above ground during 1969, an extremely wet year.

4. Water Conservation Areas No. 1 and 2. The Modified Minimum Delivery alternative does not include any structural or operational changes in WCA's No. 1 or 2. Because this alternative causes essentially no impacts in the EAA and Lake Okeechobee, there are also essentially no impacts to WCA No. 1 and 2. Figures E-6 through E-8 are the storage hydrographs for the WCA's.

5. Water Conservation Area No. 3.

a. Inflows. Average total annual inflows to WCA No. 3A from the Modified Minimum Delivery alternative are slightly below the same annual inflows of the Base Condition.

b. Outflows. Average annual total outflows from WCA No. 3A are essentially the same for the Modified Minimum Delivery alternative. Table E-1 summarizes average annual outflows from WCA No. 3A.

TABLE E-1

AVERAGE ANNUAL OUTFLOWS FROM WCA NO. 3A
(1,000 AF)

	BASE	MODIFIED MINIMUM DELIVERY
S-12	460	469
S-151	266	257
S-333	37	30
TOTAL	763	756

(1) S-12. Average annual S-12 discharges for the Modified Minimum Delivery alternative are about the same as those for the Base Condition.

(2) S-151. Average annual S-151 discharges for the Modified Minimum Delivery alternative are slightly less than those for the Base Condition. For both alternatives, S-151 discharges for water supply to the ENP - South Dade County Conveyance System are proportionately cut-back when there is not adequate capacity at S-12 to fully meet the minimum monthly water delivery requirement.

(3) S-333. For both alternatives, S-333 discharges only for water supply deliveries to ENP and south Dade County.

c. Stages and Storages. Total inflows and total outflows for the Modified Minimum Delivery alternative remain essentially the same as those for the Base Condition. The minimum and maximum storages that occur during the year are also not substantially different. Changes in stage caused by the Modified Minimum Delivery alternative vary with location in the WCA No. 3.

(1) Stages. Stages at gage 3-2, located in the northwestern portion of WCA No. 3A, are not affected by the Modified Minimum Delivery alternative. Water levels at gage 3-3, in the northeast portion of WCA No. 3A, are generally slightly higher for the Modified Minimum Delivery alternative during the dry season. Figures E-9 and E-10 are the gage 3-2 stage hydrograph and stage-duration curve and Figures E-11 and E-12 are the gage 3-3 stage hydrograph and stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

Water levels at gage 3-4 in central WCA No. 3A, are increased slightly during the dry season as a result of the Modified Minimum Delivery alternative. Figure E-13 is the gage 3-4 stage hydrograph and Figure E-14 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

Water levels at gage 3-28, in southern WCA No. 3A, are also generally increased during the dry season water level recession as a result of the Modified Minimum Delivery alternative. Water levels during the driest months of the dry season fall slightly lower for the Modified Minimum Delivery alternative than for the Base Condition. During the wet season, water levels

for the Modified Minimum Delivery alternative rise similar to the Base Condition. Figure E-15 is the gage 3-28 stage hydrograph and Figure E-16 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

In WCA No. 3B, at gage 3-29, water levels for the Modified Minimum Delivery alternative are generally slightly higher than those for the Base Condition. Figure E-17 is the gage 3-29 stage hydrograph and Figure E-18 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

(2) Storage. The seasonal minimum and maximum storage volumes in WCA No. 3A with the Modified Minimum Delivery alternative are generally similar to those of the Base Condition. Figure E-19 is the WCA No. 3A storage hydrograph for the Modified Minimum Delivery vs. Base Condition.

In WCA No. 3B, inflows and outflows with the Modified Minimum Delivery alternative have a net effect of storages that are usually close to those of the Base Condition. The years of 1978 and 1980 are exceptions because storages for the Modified Minimum Delivery are greater than the Base Condition during those years. Figure E-20 is the WCA No. 3B storage hydrograph for the Modified Minimum Delivery vs. Base Condition.

6. ENP - South Dade County Conveyance System.

a. Inflows. Total inflows to the Conveyance System are less for the Modified Minimum Delivery alternative than the Base Condition and the differences increase in the future. Table E-2 includes the average annual inflows to the Conveyance System.

TABLE E-2

AVERAGE ANNUAL INFLOWS TO THE ENP
SOUTH DADE COUNTY CONVEYANCE SYSTEM
(1,000 AF)

	Base	Modified Minimum Delivery
S-334	22	17
S-335	54	50
TOTAL	76	67

Lower inflows for the Modified Minimum Delivery alternative are caused, to a great extent, by increased seepage from NESRS into the L-31N borrow canal. As a result, demands are reduced.

b. Stages. Higher stages in NESRS with the Modified Minimum Delivery alternative cause greater seepage into the L-29 and L-31N borrow canals and, therefore, higher canal stages. This affect is seen mostly upstream of S-331 and diminishes to the south.

(1) L-29 Borrow Canal. Minimum and maximum stages in the L-29 borrow canal are higher with the Modified Minimum Delivery. With the removal of L-67 Extension, discharges through the S-12's would slightly affect canal levels. Figures E-21 through E-26 show minimum, maximum, and average stage hydrographs and stage-duration curves in the L-29 borrow canal.

(2) L-31N Borrow Canal Above S-331 (L31N). Minimum canal stages above S-331 are higher as a result of the Modified Minimum Delivery alternative. Discharging water into NESRS and raising water levels with the Modified Minimum Delivery alternative causes an increase in seepage into the L-31N borrow canal and maintains higher canal stages. Maximum canal stages are also increased by the Modified Minimum Delivery alternative. Figures E-27 and E-28 are L-31N borrow canal minimum and maximum stages for the Modified Minimum Delivery vs. Base Condition.

(3) L-31N Borrow Canal Above S-176 (L-31). Minimum canal stages above S-176 are about the same as the Base Condition. The differences between the Modified Minimum Delivery and Base Condition minimum stages are not as great in this canal reach as in the upstream reach. Maximum wet season canal stages for the Modified Minimum Delivery alternative are higher than those for the Base Condition in this canal reach. Figures E-29 and E-30 are minimum and maximum L-31 borrow canal stages above S-176 for the Modified Minimum Delivery vs. Base Condition.

(4) C-111 Above S-177 (C111). Minimum canal stages in this reach are higher during the dry season as a result of the Modified Minimum Delivery alternative. Maximum wet season canal stages are not substantially affected by the Modified Minimum Delivery alternative. Figures E-31 and E-32 are C-111 minimum and maximum canal stages for the Modified Minimum Delivery vs. Base Condition.

(5) C-111 Above S-18C (C111E). Minimum and maximum canal stages above S-18C are slightly higher for the Modified Minimum Delivery alternative than those for the Base Condition. Figures E-33 and E-34 are C-111E minimum and maximum canal stages for the Modified Minimum Delivery vs. Base Condition.

(6) C-111 Above S-197 (S-18C). Canal stages in this reach are not significantly impacted by the Modified Minimum Delivery alternative. Figures E-35 and E-36 are the minimum and maximum stages for the Modified Minimum Delivery vs. Base Condition.

(7) L-31W Borrow Canal. Canal stages in the L-31W borrow canal are not affected by the Modified Minimum Delivery alternative. Figures E-37 and E-38 are the minimum and maximum stages for the Modified Minimum Delivery vs. Base Condition.

7. Everglades National Park.

a. Inflows. Water deliveries to Taylor Slough and to the eastern panhandle via C-111 are not affected by the Modified Minimum Delivery alternative. For the Base Condition, no water deliveries are made into NESRS. Table E-3 shows the distribution of water deliveries to Shark River Slough

(SRS) between S-12 and NESRS for the Base Condition and the Modified Minimum Delivery alternative. It can be seen that, with the Modified Minimum Delivery alternative, the average annual water delivery to NESRS is about 24 percent of the total.

TABLE E-3

AVERAGE ANNUAL FLOW DISTRIBUTION AT S-12 AND TO NESRS

BASE	MODIFIED MINIMUM DELIVERY			
S-12 (1000AF)	S-12 (1000AF)	NESRS* (1000AF)	TOTAL (1000AF)	% OF TOTAL DISCHARGED TO NESRS
460	469	150	619	24

b. Stages. The Modified Minimum Delivery Water Delivery alternative substantially alters water levels in the upper portions of the Park and NESRS. Removal of L-67 Ext. allows full hydrologic interaction and a continuous water level profile across the width of the slough. Water deliveries to Taylor Slough and to the eastern panhandle area from C-111 do not change as a result of the Modified Minimum Delivery alternative. There are no significant changes in water levels in these parts of the Park.

(1) NP-201. Water levels at gage NP-201, south of S-12C, are increased somewhat by the Modified Minimum Delivery alternative, and the average hydroperiod would only be slightly increased. Water levels generally recede in the dry season to slightly higher levels and rise at a faster rate early in the wet season. Figure E-39 is the gage NP-201 stage hydrograph and Figure E-40 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

(2) NP-205. Gage NP-205 is located on the western edge of SRS below S-12A. The average hydroperiod at NP-205 is also the same as the Modified Minimum Delivery alternative. Figure E-41 is the NP-205 stage hydrograph and Figure E-42 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

(3) P-33. Removal of the L-67 Ext. borrow canal as a part of the Modified Minimum Delivery alternative reduces the efficiency of conveyance of water deliveries to P-33. The net affect of the Modified Minimum Delivery alternative on water levels at P-33 is a slight reduction much of the time, especially during the dry season and early wet season. The average hydroperiod is reduced by less than a month as a result of the Modified Minimum Delivery alternative. Figures E-43 and E-44 are the stage hydrograph and stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

(4) NP-206. South of gage P-33, at gage NP-206, water levels are impacted only slightly by the Modified Minimum Delivery alternative. Dry

season stages during February and March are generally slightly lower with the Modified Minimum Delivery alternative while during the late wet season, water levels are only slightly higher during August and September. Figure E-45 is the gage NP-206 stage hydrograph and Figure E-46 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

8. East Everglades. The largest impacts of the Modified Minimum Delivery alternative occur in the East Everglades. Hydroperiods and water depths are substantially increased due to the removal of L-67 Ext. and the introduction of water flows into NESRS.

a. Stages.

(1) NE-2. This gage is located in the center of upper NESRS. Average water levels at gage NE-2 are raised by about 0.6 ft. and the average hydroperiod is increased from about 9 months to about 11 months as a result of the Modified Minimum Delivery alternative. Figures E-47 and E-48 are gage NE-2 stage hydrographs and stage duration curves for the Modified Minimum Delivery vs. Base Condition.

(2) NE-1. Gage NE-1 is located in the center of NESRS downstream of NE-2. Average stages are increased by about 0.5 ft. and the average hydroperiod is increased from about just less than 10 months to about 11 months as a result of the Modified Minimum Delivery alternative. Figures E-49 and E-50 are the gage NE-1 stage hydrographs and stage duration curves for the Modified Minimum Delivery vs. Base Condition.

(3) G-596. Gage G-596 is located in the developed portion of the East Everglades about a mile west of L-31N. It is generally considered to be representative of what conditions are in the residential area. Water levels are increased at G-596 by 0.2 to 0.3 ft. in the dry season and decreased in the wet season by the Modified Minimum Delivery alternative. Stages during the extremely wet conditions of 1969 are increased by just over 0.5 ft. Figures E-51 and E-52 are the G-596 stage hydrograph and stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

(4) G-1502. Gage G-1502 is located near Grossman's Hammock (Chekika State Park). With the Modified Minimum Delivery alternative, water levels at G-1502 are 0.1 to 0.8 feet higher than Base Condition stages except during very wet periods when there are no differences. Water levels remain below ground surface with the Base Condition and the Modified Minimum Delivery alternative. Figure E-53 is the gage G-1502 stage hydrograph and Figure E-54 is the stage-duration curve for the Modified Minimum Delivery vs. Base Condition.

9. Total System Storage. Total system storage for the Modified Minimum Delivery alternative is generally slightly greater than that for the Base Condition. However, the differences are very small. The largest differences

in storage occur during the dry season months (January through March) when more water is stored in the system for the Modified Minimum Delivery alternative. This is shown in Figure E-55.

10. Conclusions.

a. WCA No. 3.

(1) Outflows. As a result of the Modified Minimum Delivery alternative, average annual outflows from WCA No. 3A are the same. Average S-12 discharges are the same discharges at S-333 are made for water deliveries to NESRS. The timing of WCA No. 3A outflows also changes slightly.

(2) Water Levels. Water levels in the northern portion of WCA No. 3A are not greatly affected by the Modified Minimum Delivery alternative. The largest impact is in the southern part of the area at gage 3-28. Water levels are affected by changes in the timing of outflows from WCA No. 3A with the Modified Minimum Delivery alternative. Relative to the Base Condition, water levels are higher during dry season and rise at a slightly slower rate during the early wet season with the Modified Minimum Delivery alternative.

(3) Storage. Storage in WCA No. 3A is generally slightly higher as a result of the Modified Minimum Delivery alternative. The largest differences occur during the dry season when more water is held in storage with the Modified Minimum Delivery alternative.

b. Northwest Wellfield. Additional storage in WCA No. 3 and increased eastward seepage from the area with the Modified Minimum Delivery alternative enable water levels to be held at the desired level more of the time.

c. ENP - South Dade County Conveyance System. Higher water levels in NESRS with the Modified Minimum Delivery alternative cause an increase in seepage into the L-31N borrow canal, especially in the northern reach. Canal stages are held at optimum levels for longer periods without the need for inflows from WCA No. 3A. Consequently, average annual releases from WCA No. 3A to the Conveyance System are reduced and canal stages do not fall below desirable levels as frequently. The increased seepage causes maximum canal stages to increase in the L-31N borrow canal above S-331. The affects of the Modified Minimum Delivery alternative diminish with distance southward but can be seen as far south as C-111 above S-18C.

d. Everglades National Park.

(1) Water Deliveries. Taylor Slough and the eastern panhandle water deliveries are not changed by the Modified Minimum Delivery alternative. Average annual total water deliveries to SRS remain the same.

(2) Water Levels. Water levels on the west edge of SRS are reduced somewhat and water levels in the eastern portion of SRS, in NESRS, are substantially increased by the Modified Minimum Delivery alternative.

Hydroperiods would be lengthened and water depths increased in NESRS. At gage P-33, the net affect of increased water deliveries and reduced conveyance efficiency to the gage result in slightly lower water levels much of the time.

e. East Everglades. Water levels in NESRS are increased substantially by the Modified Minimum Delivery alternative. In the developed area, water levels are also increased. At gage G-596, water levels never rise above ground under the Base Condition but with the Modified Minimum Delivery alternative, water levels do rise above ground during the extremely wet conditions in 1969.

f. Total System Storage. Total system storage is increased a small amount by the Modified Minimum Delivery alternative especially during dry season months.

B. BASIC RAIN-DRIVEN PLAN vs. BASE CONDITION

11. General. This analysis is based on a comparison of results from the South Florida Water Management Model for the Basic Rain-Driven vs. Base Condition. The operating criteria for this alternative is discussed in the main text and in Appendix F.

12. Lake Okeechobee.

a. Water Budget. The Lake Okeechobee/EAA hydrologic relationship is slightly modified by the Basic Rain-Driven Plan. Total water supply releases from Lake Okeechobee to the EEA and WCA's increase about 3 percent. Regulatory releases from Lake Okeechobee are slightly decreased by the Basic Rain-Driven Plan.

b. Stage and Storage. Because the total inflows and outflows to and from Lake Okeechobee are slightly affected by this alternative, the stages and storages are also minimally affected. The storage and stage hydrographs, and stage-duration curves for the Basic Rain-Driven vs. Base Condition are shown on Figures E-56 through E-58.

13. Everglades Agricultural Area Water Supply Runoff. Water supply and runoff requirements for the EAA are affected by the Basic Rain-Driven plan. Water supply to the EAA increase about 7 percent. Runoff from the area is not significantly affected (less than 1 percent increase). A typical stage hydrograph and stage-duration curve for the EAA area are included as Figures E-59 and E-60. There is little difference between the Base Condition and the Basic Rain-Driven Plan. Water was above ground elevation (7.8 ft., NGVD) only during the year 1969, which was extremely wet.

14. Water Conservation Areas No. 1 and 2. The Basic Rain-Driven Plan does not include any structural or operational changes in WCA's No. 1 or 2. Because the Basic Rain-Driven Plan causes essentially no impacts in the EAA and Lake Okeechobee, there are also essentially no impacts to WCA No. 1 and 2. The storage hydrographs are found on Figures E-61 through E-63.

15. Water Conservation Area No. 3.

a. Inflows. Average total annual inflows to WCA No. 3A from the Basic Rain-Driven Plan are slightly below the same annual inflows of the Base Condition.

b. Outflows. Average annual total outflows from WCA No. 3A are substantially increased for the Basic Rain-Driven Plan. Total outflows for the Basic Rain-Driven Plan are about 21 percent higher than those for the Base Condition. Table E-4 summarizes average annual outflows from WCA No. 3A. Another major difference between the Basic Rain-Driven Plan and the Base Condition is the timing of outflows. Larger releases are generally initiated earlier in the wet season for the Basic Rain-Driven Plan and releases during

the early dry season are reduced relative to the Base Condition. Also, with the Basic Rain-Driven Plan, some periods of zero flow releases would be expected during droughts or relatively dry years.

(1) S-12. Average annual S-12 discharges for the Basic Rain-Driven Plan are about 56 percent of those for the Base Condition. The operational target for S-12 discharges is only 45 percent of the total flow to Shark River Slough for the Basic Rain-Driven Plan while 100 percent of these flows are discharged at S-12 for the Base Condition.

TABLE E-4

AVERAGE ANNUAL OUTFLOWS FROM WCA NO. 3A
(1,000 AF)

	BASE	RAIN-DRIVEN
S-12	460	246
S-151	266	255
S-333	38	162
S-345	---	258
TOTAL	764	921

(2) S-151. Average annual S-151 discharges for the Basic Rain-Driven Plan are slightly greater than those for the Base Condition. For the Base Condition, S-151 discharges for water supply to the ENP - South Dade County Conveyance System are proportionately cut-back when there is not adequate capacity at S-12 to fully meet the minimum monthly water delivery requirement.

However, for the Basic Rain-Driven Plan, there are no cut-backs and S-151 discharges are made for water supply deliveries based upon available upstream storage and hydraulic capacity.

The Northwest Wellfield is a major water user of S-151 discharges. Minimum stages in canals adjacent to the wellfield (DBLEV) can be prevented from dropping below the minimum desirable level as often with the Basic Rain-Driven Plan. Maximum canal stages (in DBLEV) for the Basic Rain-Driven Plan may rise to slightly higher peaks such as occurred in 1969 and 1979 under 1990 condition.

(3) S-333. For the Base Condition, S-333 discharges only for water supply deliveries to ENP and south Dade County. For the Basic Rain-Driven Plan, however, S-333 discharges water deliveries to NESRS and water supply releases to ENP and south Dade County if additional capacity is available. As a result, Basic Rain-Driven S-333 discharges are about 440 percent of the Base Condition discharges. Increasing water supply demands in the future cause a reduction in the differences between future S-333 discharges for the Base Condition and the Basic Rain-Driven Plan.

(4) S-345. This is a new structure added as a component of the Basic Rain-Driven Water Delivery Plan. The structure starts discharging from WCA No. 3A to 3B when water levels in WCA No. 3A are 0.5 feet below Zone B or C of the regulation schedule and there is a positive head. The average annual discharges are 258,000 AF under these conditions. This structure passes about 28 percent of the total outflow from WCA No. 3A.

c. Stages and Storages. Although total inflows for the Basic Rain-Driven Plan remain essentially the same as those for the Base Condition, total outflows are greater. Because the timing of outflows for the Basic Rain-Driven Plan is different than the Base Condition (more water is discharged in the early wet season and less water released during the early dry season), the pattern of seasonal storage changes. However, the maximum and minimum storages that occur during the year are not substantially different. Changes in stage caused by the Basic Rain-Driven Plan vary with location in the WCA.

(1) Stages. Stages at gage 3-2, located in the northwestern portion of WCA No. 3A, are not affected by the Basic Rain-Driven Plan. Water levels at gage 3-3, in the northeast portion of WCA No. 3A, are generally slightly higher for the Basic Rain-Driven Plan during the dry season. However, during the dry season of 1974, Basic Rain-Driven water levels at 3-3 are slightly lower than the Base Condition. Figures E-64 and E-65 contain the stage hydrograph and stage-duration curve for the Basic Rain-Driven vs. Base Condition for gage 3-2. Figures E-66 and E-67 are the stage hydrograph and stage-duration curve for gage 3-3.

Water levels at gage 3-4 in central WCA No. 3A, are increased slightly during the dry season as a result of the Basic Rain-Driven Plan. Rain-driven water levels are slightly lower throughout the period of record during the peak of the wet season. Figure E-68 is the stage hydrograph and Figure E-69 is the stage-duration curve for the Basic Rain-Driven vs. Base Condition.

Water levels at gage 3-28 in southern WCA No. 3A, are also generally increased during the dry season water level recession as a result of the Basic Rain-Driven Plan. Water levels during the driest months of the dry season fall lower for the Basic Rain-Driven Plan than for the Base Condition. During the wet season, water levels for the Basic Rain-Driven Plan rise at a slower rate than for the Base Condition. Figure E-70 is the gage 3-28 stage hydrograph and Figure E-71 is the stage-duration curve for the Basic Rain-Driven vs. Base Condition.

In WCA No. 3B, at gage 3-29, water levels for the Basic Rain-Driven Plan are generally slightly higher than those for the Base Condition. Figure E-72 is the stage hydrograph and Figure E-73 is the stage-duration curve for Basic Rain-Driven vs. Base Condition for gage 3-29.

(2) Storage. The seasonal maximum and minimum storage volumes in WCA No. 3A with the Basic Rain-Driven Plan are generally similar to those of the Base Condition. Storages are usually higher during early dry season months and are usually lower during early wet season months. Figure E-74 is the storage hydrograph for WCA No. 3A for Basic Rain-Driven vs. Base Condition.

Increases in WCA No. 3B inflows and outflows with the Basic Rain-Driven Plan have a net affect of storages that are usually lower to those of the Base Condition. The years of 1978 and 1980 are exceptions because storages for the Basic Rain-Driven are much greater than the Base Condition during those years. During these years, S-345 discharges (into WCA No. 3B) greatly exceeded S-355 discharges (out of WCA No. 3B). Figure E-75 is the storage hydrograph for the Basic Rain-Driven vs. Base Condition in WCA No. 3B.

16. ENP - South Dade County Conveyance System.

a. Inflows. Total inflows to the Conveyance System are slightly increased for the Basic Rain-Driven Plan than the Base Condition. Table E-5 includes the average annual inflows to the Conveyance System.

The difference in inflows for the Basic Rain-Driven Plan are caused, to a great extent, by increased seepage from NESRS into the L-31N borrow canal. As a result, demands are increased due to closer canal level fluctuations resulting from the pumping action of S-356.

TABLE E-5

AVERAGE ANNUAL INFLOWS TO THE ENP
SOUTH DADE COUNTY CONVEYANCE SYSTEM
(1,000 AF)

	Base	Rain-Driven
S-334	22	22
S-335	54	58
TOTAL	76	80

b. Stages. With the Basic Rain-Driven Plan, 55 percent of water supply to ENP would be delivered to the NESRS area. This increase in flows to the area would also increase stages in the L-29 borrow canal and south of Tamiami Trail with the Basic Rain-Driven Plan. This affect is seen mostly downstream of S-335 through S-331, and greatly diminishes to the south.

(1) L-29 Borrow Canal. Minimum and maximum canal stages in the L-29 borrow canal are considerably higher with the Basic Rain-Driven Plan. With the removal of L-67 Extension discharges through the S-12's would fully interact with the NESRS area. Figures E-76 through E-81 show minimum, maximum, and average stage hydrographs and stage duration curves in the L-29 borrow canal.

(2) L-31N Borrow Canal Above S-331 (L31N). Minimum canal stages above S-331 are higher as a result of the Basic Rain-Driven Plan. Discharging water into NESRS and raising water levels as a part of the Basic Rain-Driven Plan causes an increase in seepage into the L-31N borrow canal and maintains higher canal stages. Maximum canal stages are also increased by the Basic Rain-Driven Plan. Figures E-82 and E-83 are minimum and maximum canal stages for the Basic Rain-Driven vs. Base Condition.

(3) L-31N Borrow Canal Above S-176 (L-31). Higher minimum canal stages above S-176 are also caused by the Basic Rain-Driven Plan, primarily during dry season months. The differences between the Basic Rain-Driven and Base Condition minimum stages are not as great in this canal reach as in the upstream reach. Maximum wet season canal stages for the Basic Rain-Driven Plan are similar to those for the Base Condition in this canal reach. Figures E-84 and E-85 are the minimum and maximum L-31 borrow canal stages above S-176 for the Basic Rain-Driven vs. Base Condition.

(4) C-111 Above S-177 (C111). Minimum canal stages in this reach are higher during the dry season as a result of the Basic Rain-Driven Plan. Maximum wet season canal stages are not substantially affected by the Basic Rain-Driven Plan. Figures E-86 and E-87 are the minimum and maximum canal stages for the Basic Rain-Driven vs. Base Condition.

(5) C-111 Above S-18C (C111E). Minimum and maximum canal stages above S-18C are slightly higher for the Basic Rain-Driven Plan than those for the Base Condition. Figures E-88 and E-89 are the minimum and maximum canal stages for the Basic Rain-Driven vs. Base Condition.

(6) C-111 Above S-197 (S-18C). Canal stages in this reach are not significantly impacted by the Basic Rain-Driven Plan. Figures E-90 and E-91 are the minimum and maximum canal stages for the Basic Rain-Driven vs. Base Condition.

(7) L-31W Borrow Canal. Canal stages in the L-31W borrow canal are not affected by the Basic Rain-Driven Plan.

17. Everglades National Park.

a. Inflows. Water deliveries to Taylor Slough and to the eastern panhandle via C-111 are not affected by the Basic Rain-Driven Plan. Water deliveries to Shark River Slough are substantially modified. As part of the Basic Rain-Driven Plan, there is an operational goal of providing 55 percent of the total computed volume of water delivery into NESRS. For the Base Condition, no water deliveries are made into NESRS. Table E-6 shows the distribution of water deliveries to Shark River Slough (SRS) between S-12 and NESRS for the Base Condition and the Basic Rain-Driven Plan. It can be seen that, with the Basic Rain-Driven Plan, the average annual water delivery to NESRS is about 58 percent of the total which is slightly more than the target. The total average annual water deliveries to SRS for the Basic Rain-Driven Plan are much greater than those for the Base Condition. S-333 discharges are made when there is not adequate capacity at S-355 to pass the full computed flow to NESRS. Average annual discharges at S-333 (minus S-334 discharges) and S-355 are roughly equal for the Basic Rain-Driven Plan.

TABLE E-6

AVERAGE ANNUAL FLOW DISTRIBUTION AT S-12 AND TO NESRS

BASE	RAIN-DRIVEN WATER DELIVERIES TO ENP			
S-12 (1000AF)	S-12 (1000AF)	NESRS* (1000AF)	TOTAL (1000AF)	% OF TOTAL DISCHARGED TO NESRS
460	246	283	529	58

* Water deliveries to NESRS (Northeast Shark River Slough) are computed as the sum of S-333 and S-355 discharges minus S-334 discharges.

b. Stages. The Basic Rain-Driven Plan substantially alters water levels in the upper portions of the Park and NESRS. Removal of L-67 Ext. allows full hydrologic interaction and a continuous water level profile across the width of the slough. Discharges into NESRS via S-333 and S-355 also have a substantial impact on downstream water levels. Because water deliveries to Taylor Slough and to the eastern panhandle area from C-111 do not change as a result of the Basic Rain-Driven Plan, there are no significant changes in water levels in these parts of the Park.

(1) NP-201. Water levels at gage NP-201, south of S-12C, are reduced somewhat by the Basic Rain-Driven Plan, although the average hydroperiod would only be slightly reduced. Water levels generally recede earlier in the dry season to slightly lower levels and rise at a slower rate early in the wet season. Figure E-92 is the NP-201 stage hydrograph and Figure E-93 is the stage-duration curve for the Basic Rain-Driven vs. Base Condition.

(2) NP-205. Gage NP-205 is located on the western edge of SRS below S-12A. Because water deliveries to SRS are shifted eastward with the Basic Rain-Driven Plan and S-12 discharges are reduced, water levels are reduced at gage NP-205. The average hydroperiod at NP-205 is about one month shorter with the Basic Rain-Driven Plan. Similar to the pattern at NP-201, water levels at NP-205 for the Basic Rain-Driven Plan recede earlier in the dry season to lower levels and rise at a faster rate early in the wet season. Figure E-94 is the stage hydrograph and Figure E-95 is the stage-duration curve for the Basic Rain-Driven vs. Base Condition.

(3) P-33. Water levels at P-33 are affected by two components of the Basic Rain-Driven Plan that have conflicting impacts. Increased total flows to SRS for the Basic Rain-Driven Plan should cause higher stages at P-33. However, removal of the L-67 Ext. borrow canal as a part of the Basic Rain-Driven Plan reduces the efficiency of conveyance of water deliveries to P-33. The net affect of the Basic Rain-Driven Plan on water levels at P-33 is a slight reduction much of the time, especially during the dry season and early wet season. The average hydroperiod is reduced by less than a month as a

result of the Basic Rain-Driven Plan. Figures E-96 and E-97 are the stage hydrograph and stage-duration curve for the Basic Rain-Driven vs. Base Condition.

(4) NP-206. South of gage P-33, at gage NP-206, water levels are impacted only slightly by the Basic Rain-Driven Plan. Dry season stages during February and March are generally slightly lower with the Basic Rain-Driven Plan while during the late wet season, water levels are only slightly higher during August and September. Figure E-98 is the stage hydrograph and E-99 is the stage-duration curve for the Basic Rain-Driven vs. Base Condition.

18. East Everglades. The largest impacts of the Basic Rain-Driven Plan occur in the East Everglades. Hydroperiods and water depths are substantially increased due to the removal of L-67 Ext. and the introduction of water flows into NESRS.

a. Stages.

(1) NE-2. This gage is located in the center of upper NESRS. Average water levels at gage NE-2 are raised by about 0.8 ft. and the average hydroperiod is increased from about nine months to about 11 months as a result of the Basic Rain-Driven Plan. Figures E-100 and E-101 are the stage hydrograph and stage duration curve for the Basic Rain-Driven vs. Base Condition.

(2) NE-1. Gage NE-1 is located in the center of NESRS downstream of NE-2. Average stages are increased by about 0.5 ft. and the average hydroperiod is increased from about just less than 10 months to about 11 months as a result of the Basic Rain-Driven Plan. Figures E-102 and E-103 are the stage hydrograph and stage duration curve for Basic Rain-Driven vs. Base Condition.

(3) G-596. Gage G-596 is located in the developed portion of the East Everglades about a mile west of L-31N. It is generally considered to be representative of what conditions are in the residential area. Water levels are increased at G-596 by 0.2 to 0.3 ft. under most conditions by the Basic Rain-Driven Plan. Stages during the extremely wet conditions of 1969 are increased by just over 1.0 ft. Water levels do not rise above ground under the Base Condition but do rise slightly above ground with the Basic Rain-Driven Plan. Figures E-104 and E-105 are the stage hydrograph and stage-duration curve for the Basic Rain-Driven vs. Base Condition.

(4) G-1502. Gage G-1502 is located near Grossman's Hammock (Chekika State Park). With the Basic Rain-Driven Plan, water levels at G-1502 are 0.1 to 0.4 feet higher than Base Condition stages except during very wet periods when there are no differences. Water levels remain below ground surface with the Basic Rain-Driven vs. Base Condition. Figure E-106 is the stage hydrograph and Figure E-107 is the stage-duration curve for the Basic Rain-Driven vs. Base Condition.

19. Total System Storage. Total system storage for the Basic Rain-Driven Plan is generally slightly greater than that for the Base Condition. However, the differences are very small. The largest differences in storage occur during the dry season months (January through March) when more water is stored in the system for the Basic Rain-Driven Plan. Figure E-108 is the Total System Storage for the Basic Rain-Driven vs. Base Condition.

20. Conclusions.

a. WCA No. 3.

(1) Outflows. As a result of the Basic Rain-Driven Plan, average annual outflows from WCA No. 3A are increased 21 percent. Although average S-12 discharges are reduced by 47 percent, increased discharges at S-333 for water deliveries to NESRS and discharges at a new structure, S-345, cause the total outflow to increase for the Basic Rain-Driven Plan. The timing of WCA No. 3A outflows also changes. With the Basic Rain-Driven Plan, larger releases are made earlier in the wet season and smaller releases are made during the dry season, relative to the Base Condition.

(2) Water Levels. Water levels in the northern portion of WCA No. 3A are not greatly affected by the Basic Rain-Driven Plan. The largest impact is in the southern part of the area at gage 3-28. Water levels are affected by changes in the timing of outflows from WCA No. 3A with the Basic Rain-Driven Plan. Relative to the Base Condition, water levels are higher during dry season and rise at a slightly slower rate during the early wet season with the Basic Rain-Driven Plan.

(3) Storage. Storage in WCA No. 3A is generally slightly higher as a result of the Basic Rain-Driven Plan. The largest differences occur during the dry season when more water is held in storage with the Basic Rain-Driven Plan.

b. Northwest Wellfield. Additional storage in WCA No. 3 and increased eastward seepage from the area with the Basic Rain-Driven Plan enable water levels to be held at the desired level more of the time.

c. South Dade County Conveyance System. Higher water levels in NESRS with the Basic Rain-Driven Plan cause an increase in seepage into the L-31N borrow canal, especially in the northern reach. Canal stages are held at optimum levels for longer periods without the need for inflows from WCA No. 3A. Consequently, average annual releases from WCA No. 3A to the Conveyance System are reduced and canal stages do not fall below desirable levels as frequently. The increased seepage causes maximum canal stages to increase in the L-31N borrow canal above S-331. The affects of the Basic Rain-Driven Plan diminish with distance southward but can be seen as far south as C-111 above S-18C.

d. Everglades National Park.

(1) Water Deliveries. Taylor Slough and the eastern panhandle water deliveries are not changed by the Basic Rain-Driven Plan. Average annual total water deliveries to SRS are increased by 126,000 AF as a result of the Basic Rain-Driven Plan. About 58 percent of the total Basic Rain-Driven water deliveries are discharged to NESRS. S-333 and S-355 average discharges to NESRS are about equal with the Basic Rain-Driven Plan. The timing of water deliveries is shifted by the Basic Rain-Driven Plan so that more water is delivered early in the wet season and deliveries in the dry season are reduced slightly.

(2) Water Levels. Water levels on the west edge of SRS are reduced somewhat and water levels in the eastern portion of SRS, in NESRS, are substantially increased by the Basic Rain-Driven Plan. Hydroperiods would be lengthened and water depths increased in NESRS. At gage P-33, the net affect of increased water deliveries and reduced conveyance efficiency to the gage result in slightly lower water levels much of the time.

e. East Everglades. Water levels in NESRS are increased substantially by the Basic Rain-Driven Plan. In the developed area, water levels are also increased. At gage G-596, water levels never rise above ground under the Base Condition but with the Basic Rain-Driven Plan, water levels do rise above ground during the extremely wet conditions as those that occurred in 1969.

f. Total System Storage. Total system storage is increased a small amount by the Basic Rain-Driven Plan especially during dry season months.

C. MODIFIED RAIN-DRIVEN PLAN vs. BASE CONDITION

21. General. This analysis is based on a comparison of results from the South Florida Water Management Model for the Modified Rain-Driven Plan vs. Base Condition.

22. Lake Okeechobee.

a. Water Budget. The Lake Okeechobee/EAA hydrologic relationship is slightly modified by the Modified Rain-Driven alternative. Total water supply releases from Lake Okeechobee to the EEA and WCA's increase about 12 percent. Regulatory releases from Lake Okeechobee are decreased by the Modified Rain-Driven alternative.

b. Stage and Storage. Because the total inflows and outflows to and from Lake Okeechobee are slightly affected by this alternative, the stages and storages are also minimally affected. The storage and stage hydrographs, and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition are shown on Figures E-109 through E-111.

23. Everglades Agricultural Area Water Supply Runoff. Water supply and runoff requirements for the EAA are not significantly altered by the Modified Rain-Driven alternative. Water supply to the EAA increase about 3 percent. Runoff from the area changed very little in comparison with the Base Condition. A typical stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition are included in Figures E-112 and E-113. Water reaches above ground elevation (7.8 ft., NGVD) only during the wet year of 1969.

24. Water Conservation Areas No. 1 and 2. The Modified Rain-Driven alternative does not include any structural or operational changes in WCA's No. 1 or 2. Because the Modified Rain-Driven alternative causes essentially no impacts in the EAA and Lake Okeechobee, there are also essentially no impacts to WCA No. 1 and 2. Figures E-114 through E-116 are the storage hydrographs for the WCA's.

25. Water Conservation Area No. 3.

a. Inflows. Average total annual inflows to WCA No. 3A for the Modified Rain-Driven alternative are 10% greater than the Base Condition.

b. Outflows. Average annual total outflows from WCA No. 3A are substantially increased for the Modified Rain-Driven alternative. Total outflows for the Modified Rain-Driven alternative are 26% greater than those for the Base Condition. Table E-7 summarizes average annual outflows from WCA No. 3A. A major difference between the Modified Rain-Driven alternative and the Base Condition is the timing of outflows. Larger releases are generally initiated earlier in the wet season for the Modified Rain-Driven alternative and releases during the early dry season are reduced relative to the Base Condition.

TABLE E-7

AVERAGE ANNUAL OUTFLOWS FROM WCA NO. 3A
(1,000 AF)

	BASE	MODIFIED RAIN-DRIVEN
S-12	460	297
S-151	266	268
S-333	38	208
S-345		189
TOTAL	764	962

(1) S-12. Average annual S-12 discharges for the Modified Rain-Driven alternative are about 55% of those for the Base Condition. The operational target for S-12 discharges is only 45% of the total flow to Shark River Slough for the Modified Rain-Driven alternative while 100% of these flows are discharged at S-12 for the Base Condition.

(2) S-151. Average annual S-151 discharges for the Modified Rain-Driven alternative are 22% less than those for the Base Condition. For the Base Condition, S-151 discharges for water supply to the ENP - South Dade County Conveyance System are proportionately cut-back when there is not adequate capacity at S-12 to fully meet the minimum monthly water delivery requirement.

However, for the Modified Rain-Driven alternative, there are no cut-backs and S-151 discharges are made for water supply deliveries based upon available upstream storage and hydraulic capacity.

The Northwest Wellfield is a major water user of S-151 discharges. Minimum stages in canals adjacent to the wellfield (DBLEV) can be prevented from dropping below the minimum desirable level as often with the Modified Rain-Driven alternative. Maximum canal stages (in DBLEV) for the Modified Rain-Driven alternative may rise to slightly higher peaks such as occurred in 1969 and 1979 under 1990 conditions.

(3) S-333. For the Base Condition, S-333 discharges only for water supply deliveries to ENP and south Dade County. For the Modified Rain-Driven alternative, however, S-333 discharges water deliveries to NESRS and water supply releases to ENP and south Dade County if additional capacity is available. As a result, Modified Rain-Driven S-333 discharges are about 7 times greater than the Base Condition discharges.

(4) S-345. This is a new structure added as a component of the Modified Rain-Driven Water Delivery alternative. The structure starts discharging from WCA No. 3A to 3B when water levels in WCA No. 3A are 0.5 feet below Zone B or C of the regulation schedule and there is a positive head. For the Modified Rain-Driven alternative, S-345 annual discharges range between zero and just over 189,000 AF.

c. Stages and Storages. Total inflows for the Modified Rain-Driven alternative and total outflows are greater than those for the Base Conditions. Because the timing of outflows for the Modified Rain-Driven alternative is different than the Base Condition (more water is discharged in the early wet season and less water released during the early dry season), the pattern of seasonal storage changes. However, the maximum and minimum storages that occur during the year are not substantially different. Changes in stage caused by the Modified Rain-Driven alternative vary with location in the WCA.

(1) Stages. Stages at gage 3-2, located in the northwestern portion of WCA No. 3A, are not affected by the Modified Rain-Driven alternative. Water levels at gage 3-3, in the northeast portion of WCA No. 3A, are generally slightly higher for the Modified Rain-Driven alternative during the dry season. However, during the dry season of 1974, Modified Rain-Driven water levels at 3-3 are slightly lower than the Base Condition. Figures E-117 and E-118 contain the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition for gage 3-2. Figures E-119 and E-120 contain the stage hydrograph and stage-duration curve for gage 3-3.

Water levels at gage 3-4 in central WCA No. 3A, are increased slightly during the dry season as a result of the Modified Rain-Driven alternative, with the exception of the 1974 dry season. Modified Rain-Driven water levels are lower throughout the 1974 dry season. Figure E-121 is the stage hydrograph for gage 3-4 and Figure E-122 is the stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

Water levels at gage 3-28, in southern WCA No. 3A, are also generally increased during the dry season water level recession as a result of the Modified Rain-Driven alternative. Water levels during the driest months of the dry season fall slightly lower for the Modified Rain-Driven alternative than for the Base Condition. During the wet season, water levels for the Modified Rain-Driven alternative rise at a slower rate although peak stages during the year are similar to the Base Condition. Figure E-123 is the stage hydrograph and Figure E-124 is the stage-duration curve for gage 3-28 for the Modified Rain-Driven Plan vs. Base Condition.

In WCA No. 3B, at gage 3-29, water levels for the Modified Rain-Driven alternative are generally slightly higher than those for the Base Condition. Figures E-125 and E-126 contain the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

(2) Storage. The seasonal maximum and minimum storage volumes in WCA No. 3A with the Modified Rain-Driven alternative are generally similar to those of the Base Condition. Storages are usually higher during early dry season months and are usually lower during early wet season months. The storage hydrograph for WCA No. 3A for the Modified Rain-Driven Plan vs. Base Condition is found on Figure E-127.

Increases in WCA No. 3B inflows and outflows with the Modified Rain-Driven alternative have a net affect of storages that are usually close to those of the Base Condition. The years of 1978 and 1980 are exceptions

because storages for the Modified Rain-Driven Plan are much greater than the Base Condition during those years. During these years, S-345 discharges (into WCA No. 3B) greatly exceeded S-355 discharges (out of WCA No. 3B). The storage hydrograph for WCA No. 3B for the Modified Rain-Driven Plan vs. Base Condition is found on Figure E-128.

26. ENP - South Dade County Conveyance System.

a. Inflows. Total inflows to the Conveyance System are greater for the Modified Rain-Driven alternative than the Base Condition. Table E-8 includes the average annual inflows to the Conveyance System.

The difference in inflows for the Modified Rain-Driven alternative are caused, to a great extent, by increased seepage from NESRS into the L-31N borrow canal. As a result, demands are increased due to closer canal level fluctuations resulting from the pumping action of S-356.

TABLE E-8

AVERAGE ANNUAL INFLOWS TO THE ENP
SOUTH DADE COUNTY CONVEYANCE SYSTEM
(1,000 AF)

	Base	Modified Rain-Driven
S-334	22	22
S-335	54	60
TOTAL	76	82

b. Stages. With the Modified Rain-Driven alternative, 55 percent of water supply to ENP would be delivered to the NESRS area. This increase in flows to the area would also increase stages in the L-29 borrow canal and south of Tamiami Trail. This affect is seen mostly downstream of S-335 through S-331, and greatly diminishes to the south.

(1) L-29 Borrow Canal. Minimum and maximum canal stages in the L-29 borrow canal are considerably higher with the Basic Rain-Driven alternative. With the removal of L-67 Extension, discharges through the S-12's would fully interact with the NESRS area. Figures E-129 through E-134 show minimum, maximum, and average stage hydrographs and stage-duration curves in the L-29 borrow canal.

(2) L-31N Borrow Canal Above S-331 (L31N). Minimum canal stages above S-331 are higher as a result of the Modified Rain-Driven alternative. Discharging water into NESRS and raising water levels as a part of the Modified Rain-Driven alternative causes an increase in seepage into the L-31N borrow canal and maintains higher canal stages. Maximum canal stages are also increased by the Modified Rain-Driven alternative. Figures E-135 and E-136 are the minimum and maximum canal stages for the Modified Rain-Driven Plan vs. Base Condition.

(3) L-31N Borrow Canal Above S-176 (L-31). Higher minimum canal stages above S-176 are also caused by the Modified Rain-Driven alternative, primarily during dry season months. The differences between the Modified Rain-Driven and Base Condition minimum stages are not as great in this canal reach as in the upstream reach. Maximum wet season canal stages for the Modified Rain-Driven alternative are similar to those for the Base Condition in this canal reach. Figures E-137 and E-138 are the minimum and maximum canal stages for the Modified Rain-Driven Plan vs. Base Condition.

(4) C-111 Above S-177 (C111). Minimum canal stages in this reach are higher during the dry season as a result of the Modified Rain-Driven alternative. Maximum wet season canal stages are not substantially affected by the Modified Rain-Driven alternative. Figures E-139 and E-140 are the minimum and maximum canal stages for the Modified Rain-Driven Plan vs. Base Condition.

(5) C-111 Above S-18C (C111E). Minimum and maximum stages above S-18C are slightly higher for the Modified Rain-Driven alternative than those for the Base Condition. Figures E-141 and E-142 are the minimum and maximum canal stages for the Modified Rain-Driven Plan vs. Base Condition.

(6) C-111 Above S-197 (S-18C). Canal stages in this reach are not significantly impacted by the Modified Rain-Driven alternative. Figures E-143 and E-144 are the minimum and maximum canal stages for the Modified Rain-Driven Plan vs. Base Condition.

(7) L-31W Borrow Canal. Canal stages in the L-31W borrow canal are not affected by the Modified Rain-Driven alternative. Figures E-145 and E-146 are the minimum and maximum canal stages for the Modified Rain-Driven Plan vs. Base Condition.

27. Everglades National Park.

a. Inflows. Water deliveries to Taylor Slough and to the eastern panhandle via C-111 are not affected by the Modified Rain-Driven alternative. Water deliveries to Shark River Slough are substantially modified. As part of the Modified Rain-Driven alternative, there is an operational goal of providing 55% of the total computed volume of water delivery into NESRS. For the Base Condition, no water deliveries are made into NESRS. Table E-9 shows the distribution of water deliveries to Shark River Slough (SRS) between S-12 and NESRS for the Base Condition and the Modified Rain-Driven alternative. It can be seen that, with the Modified Rain-Driven alternative, the average annual water delivery to NESRS is about 59% of the total which is slightly more than the target. The total average annual water deliveries to SRS for the Modified Rain-Driven alternative are much greater than those for the Base Condition. S-333 discharges are made when there is not adequate capacity at S-355 to pass the full computed flow to NESRS. Average annual discharges at S-333 (minus S-334 discharges) and S-355 are roughly equal for the Modified Rain-Driven alternative.

TABLE E-9

AVERAGE ANNUAL FLOW DISTRIBUTION AT S-12 AND TO NESRS

BASE	MODIFIED RAIN-DRIVEN WATER DELIVERIES TO ENP			
S-12 (1000AF)	S-12 (1000AF)	NESRS* (1000AF)	TOTAL (1000AF)	% OF TOTAL DISCHARGED TO NESRS
460	297	330	627	53

* Water deliveries to NESRS (Northeast Shark River Slough) are computed as the sum of S-333 and S-355 discharges minus S-334 discharges.

b. Stages. The Modified Rain-Driven Water Delivery alternative substantially alters water levels in the upper portions of the Park and NESRS. Removal of L-67 Ext. allows full hydrologic interaction and a continuous water level profile across the width of the slough. Discharges into NESRS via S-333 and S-355 also have a substantial impact on downstream water levels. Because water deliveries to Taylor Slough and to the eastern panhandle area from C-111 do not change as a result of the Modified Rain-Driven alternative, there are no significant changes in water levels in these parts of the Park.

(1) NP-201. Water levels at gage NP-201, south of S-12C, are reduced somewhat by the Modified Rain-Driven alternative, although the average hydroperiod would only be slightly reduced. Water levels generally recede earlier in the dry season to slightly lower levels and rise at a faster rate early in the wet season. Figure E-147 is the stage hydrograph and Figure E-148 is the stage-duration curve for NP-201 for the Modified Rain-Driven Plan vs. Base Condition.

(2) NP-205. Gage NP-205 is located on the western edge of SRS below S-12A. Because water deliveries to SRS are shifted eastward with the Modified Rain-Driven alternative and S-12 discharges are reduced, water levels are reduced at gage NP-205. The average hydroperiod at NP-205 is about one month shorter with the Modified Rain-Driven alternative. Similar to the pattern at NP-201, water levels at NP-205 for the Modified Rain-Driven alternative recede earlier in the dry season to lower levels and rise at a faster rate early in the wet season. Figure E-149 is the stage hydrograph and Figure E-150 is the stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

(3) P-33. Water levels at P-33 are affected by two components of the Modified Rain-Driven alternative that have conflicting impacts. Increased total flows to SRS for the Modified Rain-Driven alternative should cause higher stages at P-33. However, removal of the L-67 Ext borrow canal as a part of the Modified Rain-Driven alternative reduces the efficiency of conveyance of water deliveries to P-33. The net affect of the Modified Rain-Driven alternative on water levels at P-33 is a slight reduction much of the time, especially during the dry season and early wet season. The average

hydroperiod is reduced by less than a month as a result of the Modified Rain-Driven alternative. Figures E-151 and E-152 are the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

(4) NP-206. South of gage P-33, at gage NP-206, water levels are impacted only slightly by the Modified Rain-Driven alternative. Dry season stages during February and March are generally slightly lower with the Modified Rain-Driven alternative while during the late wet season, water levels are only slightly higher during August and September. Figures E-153 and E-154 are the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

28. East Everglades. The largest impacts of the Modified Rain-Driven alternative occur in the East Everglades. Hydroperiods and water depths are substantially increased due to the removal of L-67 Ext. and the introduction of water flows into NESRS.

a. Stages.

(1) NE-2. This gage is located in the center of upper NESRS. Average water levels at gage NE-2 are raised by about 0.8 ft. and the average hydroperiod is increased from about nine months to about 11 months as a result of the Modified Rain-Driven alternative. Figures E-155 and E-156 are the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

(2) NE-1. Gage NE-1 is located in the center of NESRS downstream of NE-2. Average stages are increased by about 0.7 ft. and the average hydroperiod is increased from about just less than 10 months to about 11 months as a result of the Modified Rain-Driven alternative. Figures E-157 and E-158 are the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

(3) G-596. Gage G-596 is located in the developed portion of the East Everglades about a mile west of L-31N. It is generally considered to be representative of what conditions are in the residential area. Water levels are decreased at G-596 by 0.2 to 0.3 ft. under most conditions by the Modified Rain-Driven alternative. Figures E-159 and E-160 are the stage hydrograph and stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

(4) G-1502. Gage G-1502 is located near Grossman's Hammock (Chekika State Park). With the Modified Rain-Driven alternative, water levels at G-1502 are 0.1 to 0.4 feet higher than Base Condition stages except during very wet periods when there are no differences. Water levels remain below ground surface with the Base Condition and the Modified Rain-Driven alternative. Figure E-161 is the stage hydrograph and Figure E-162 is the stage-duration curve for the Modified Rain-Driven Plan vs. Base Condition.

29. Total System Storage. Total system storage for the Modified Rain-Driven alternative is generally slightly greater than that for the Base Condition. However, the differences are very small. The largest differences in storage occur during the dry season months (January through March) when more water is stored in the system for the Modified Rain-Driven alternative. Figure E-163 is the Total System Storage Hydrograph.

30. Conclusions.

a. WCA No. 3.

(1) Outflows. As a result of the Modified Rain-Driven alternative average annual outflows from WCA No. 3A are increased. Although average S-12 discharges are reduced by more than 50%, increased discharges at S-333 for water deliveries to NESRS and discharges at a new structure, S-345, cause the total outflow to increase for the Modified Rain-Driven alternative. The timing of WCA No. 3A outflows also changes. With the Modified Rain-Driven alternative, larger releases are made earlier in the wet season and smaller releases are made during the dry season, relative to the Base Condition.

(2) Water Levels. Water levels in the northern portion of WCA No. 3A are not greatly affected by the Modified Rain-Driven alternative. The largest impact is in the southern part of the area at gage 3-28. Water levels are affected by changes in the timing of outflows from WCA No. 3A with the Modified Rain-Driven alternative. Relative to the Base Condition, water levels are higher during dry season and rise at a slightly slower rate during the early wet season with the Modified Rain-Driven alternative.

(3) Storage. Storage in WCA No. 3A is generally slightly higher as a result of the Modified Rain-Driven alternative. The largest differences occur during the dry season when more water is held in storage with the Modified Rain-Driven alternative.

b. Northwest Wellfield. Additional storage in WCA No. 3 and increased eastward seepage from the area with the Modified Rain-Driven alternative enable water levels to be held at the desired level more of the time.

c. ENP - South Dade County Conveyance System. Higher water levels in NESRS with the Modified Rain-Driven alternative cause an increase in seepage into the L-31N borrow canal, especially in the northern reach. Canal stages are held at optimum levels for longer periods without the need for inflows from WCA No. 3A. Consequently, average annual releases from WCA No. 3A to the Conveyance System are reduced and canal stages do not fall below desirable levels as frequently. The increased seepage causes maximum canal stages to increase in the L-31N borrow canal above S-331. The affects of the Modified Rain-Driven alternative diminish with distance southward but can be seen as far south as C-111 above S-18C.

d. Everglades National Park.

(1) Water Deliveries. Taylor Slough and the eastern panhandle water deliveries are not changed by the Modified Rain-Driven alternative. Average annual total water deliveries to SRS are increased by 273,000 AF as a result of the Modified Rain-Driven alternative. About 59% of the total Modified Rain-Driven water deliveries are discharged to NESRS. S-333 and S-355 average discharges to NESRS are about equal with the Modified Rain-Driven alternative. The timing of water deliveries is shifted by the Modified Rain-Driven alternative so that more water is delivered early in the wet season and deliveries in the dry season are reduced slightly.

(2) Water Levels. Water levels on the west edge of SRS are reduced somewhat and water levels in the eastern portion of SRS, in NESRS, are substantially increased by the Modified Rain-Driven alternative. Hydroperiods would be lengthened and water depths increased in NESRS. At gage P-33, the net affect of increased water deliveries and reduced conveyance efficiency to the gage result in slightly lower water levels much of the time.

e. East Everglades. Water levels in NESRS are increased substantially by the Modified Rain-Driven alternative. In the developed area, water levels are also increased. At gage G-596, water levels never rise above ground under the Base Condition but with the Modified Rain-Driven alternative, water levels do rise above ground during the extremely wet conditions.

f. Total System Storage. Total system storage is increased a small amount by the Modified Rain-Driven alternative, especially during dry season months.

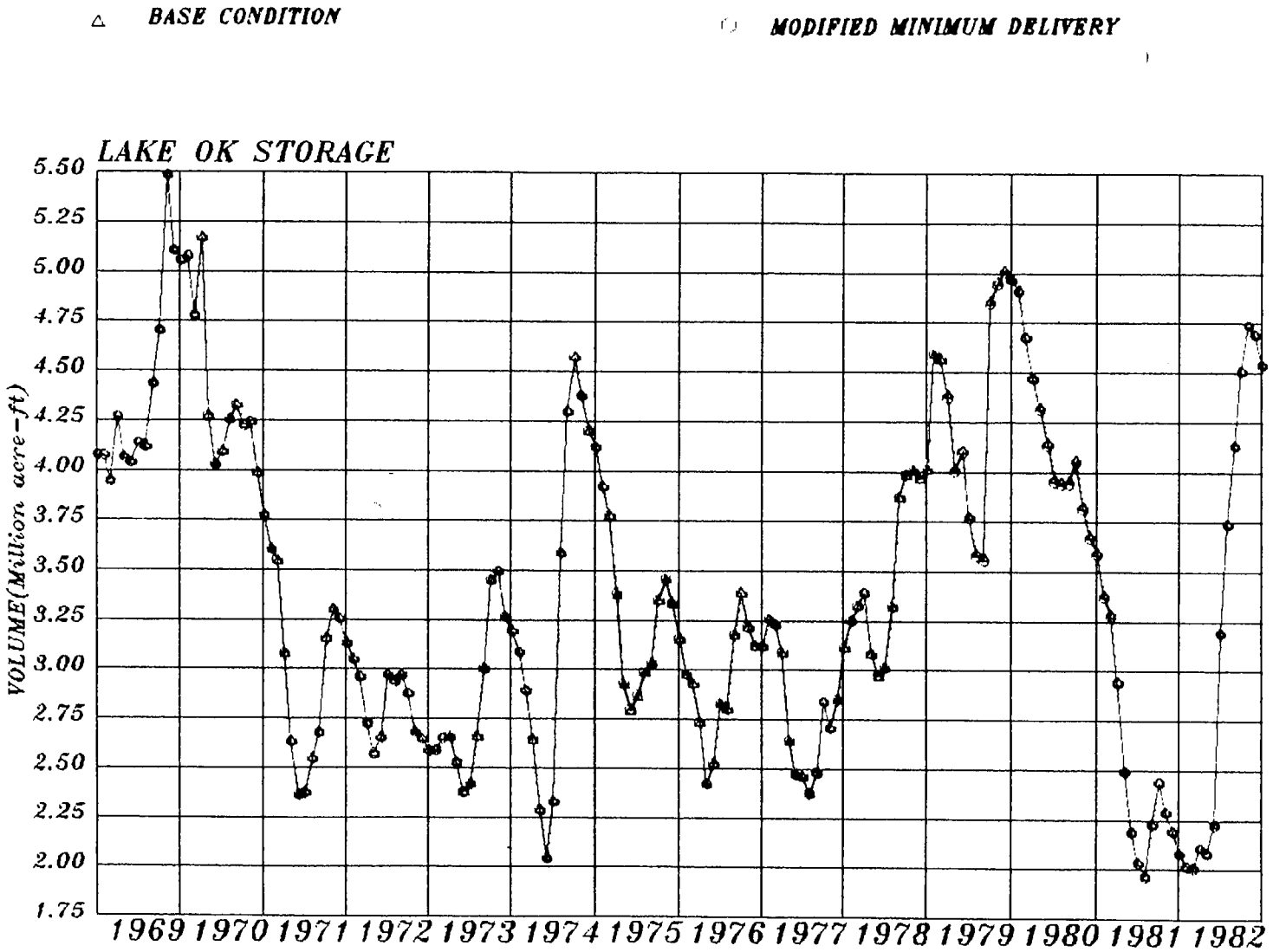
BASE CONDITION
MODIFIED MINIMUM DELIVERY
LAKE OK STORAGE
VOLUME(Million acre-ft)
5.50
5.25
5.00
4.75
4.50
4.25
4.00
3.75
3.50
3.25
3.00
2.75
2.50
2.25
2.00
1.75
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-1

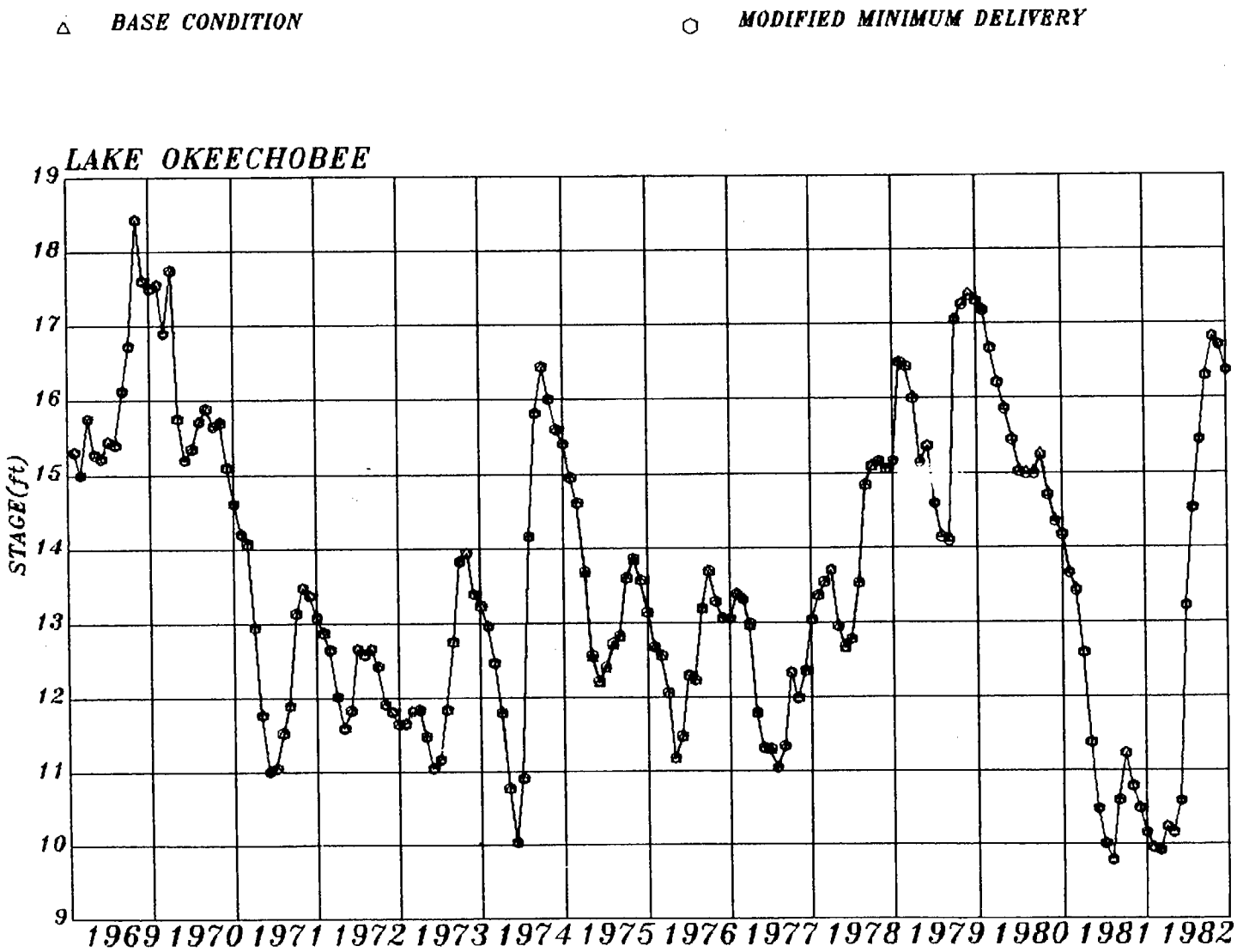
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
LAKE OKEECHOBEE
STAGE(ft)
19
18
17
16
15
14
13
12
11
10
9
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-2

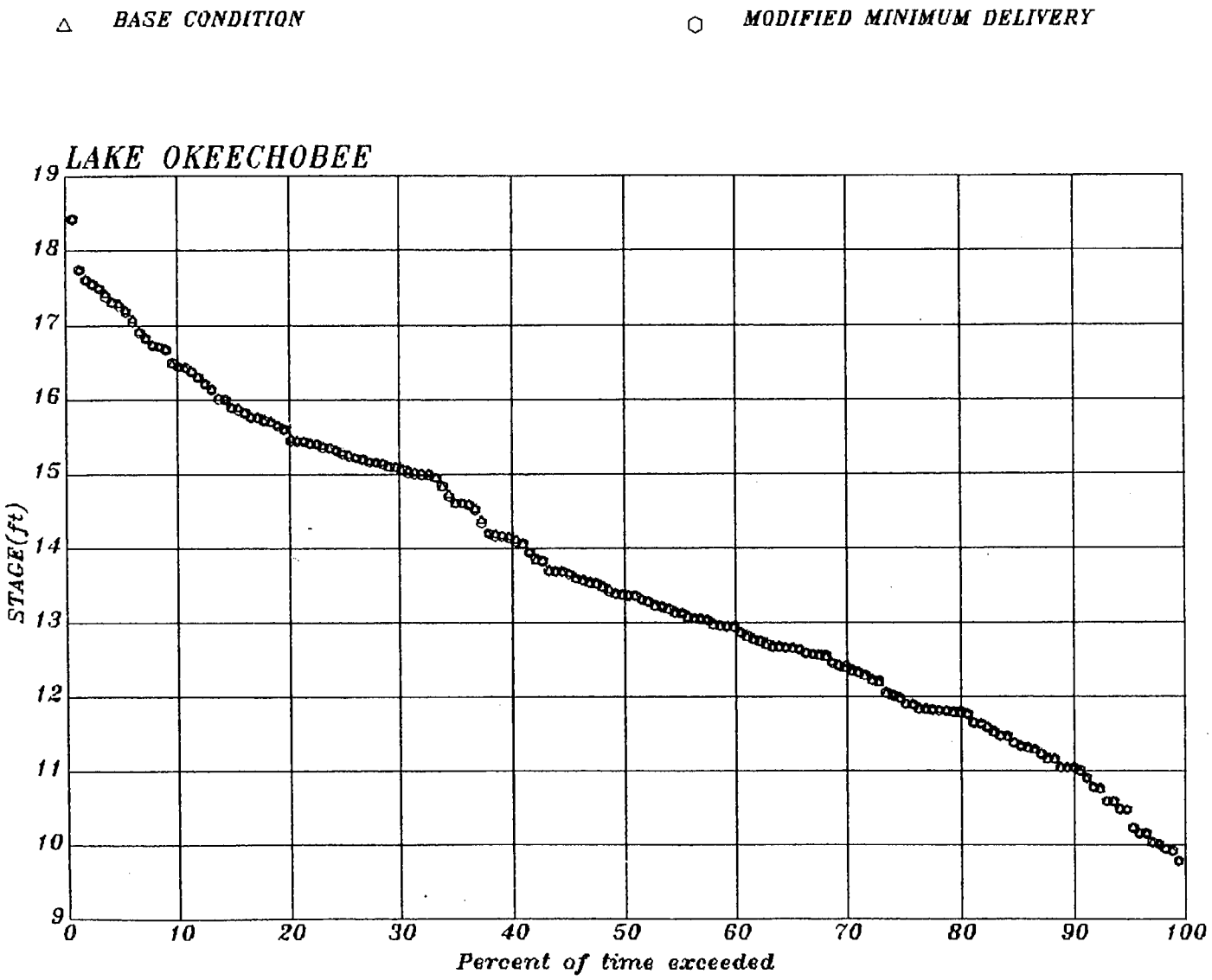
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
LAKE OKEECHOBEE
STAGE(ft)
19
18
17
16
15
14
13
12
11
10
9
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-3

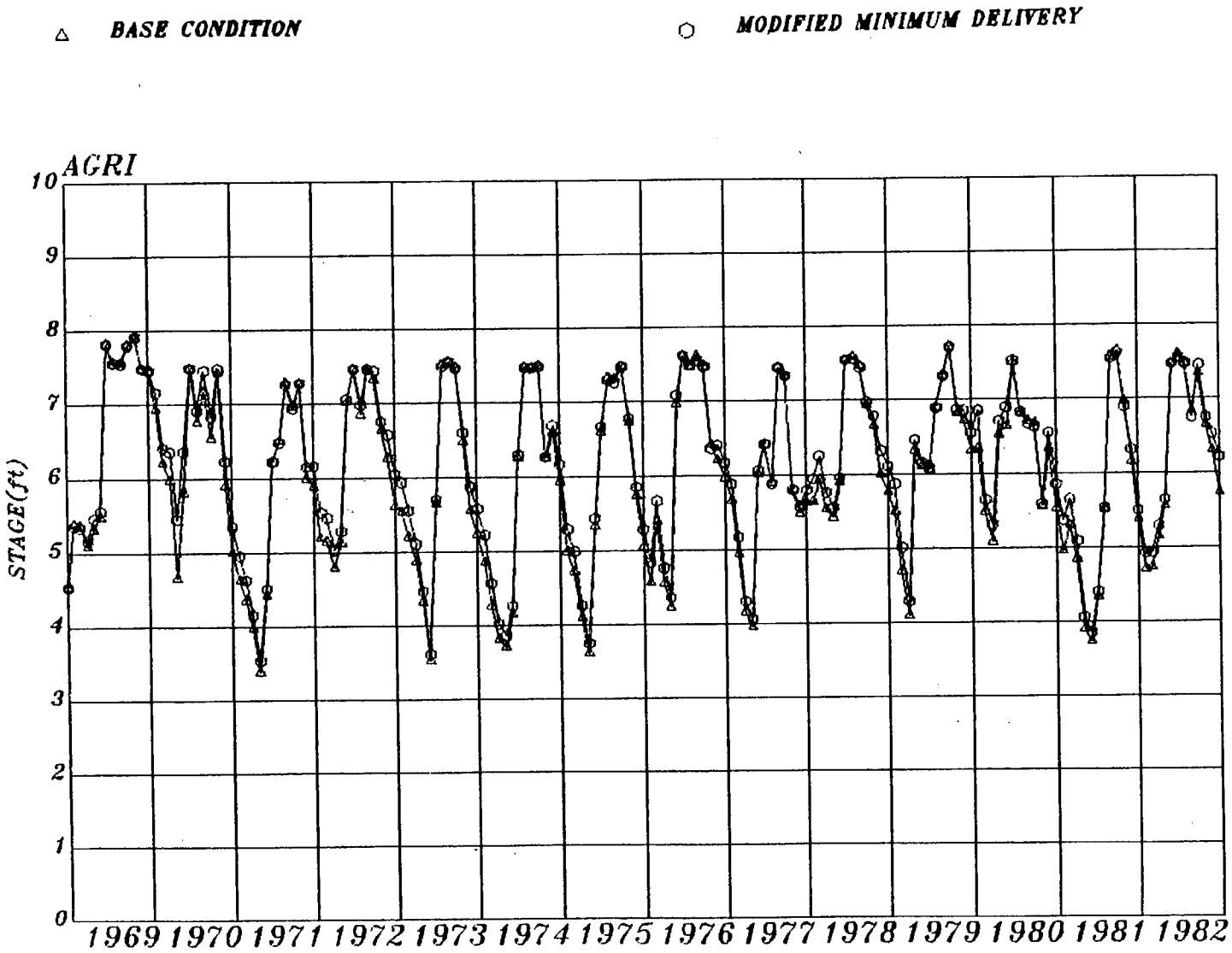
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
AGRI
STAGE(ft)
10
9
8
7
6
5
4
3
2
1
0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-4

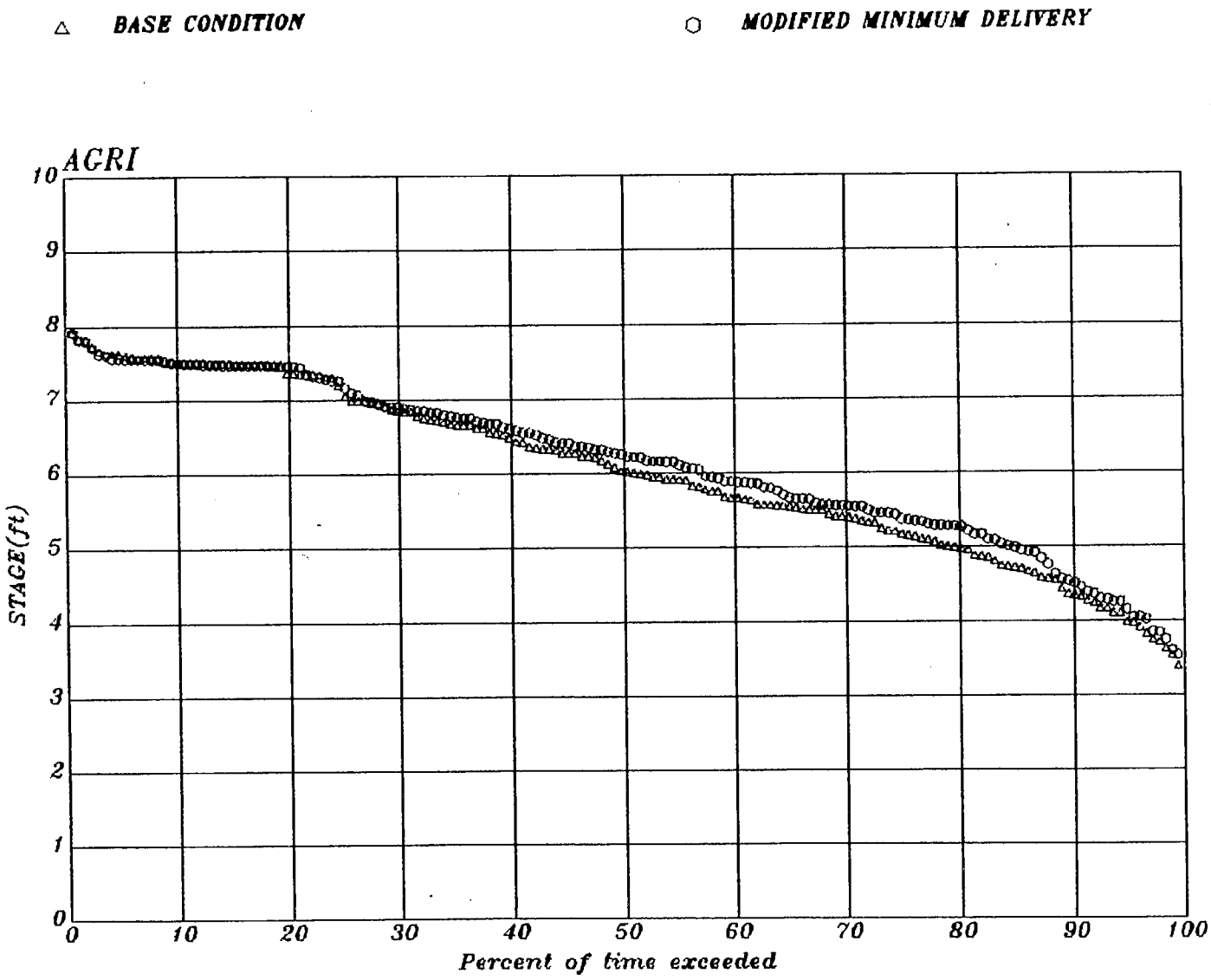
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
AGRI
STAGE(ft)
0
1
2
3
4
5
6
7
8
9
10
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-5

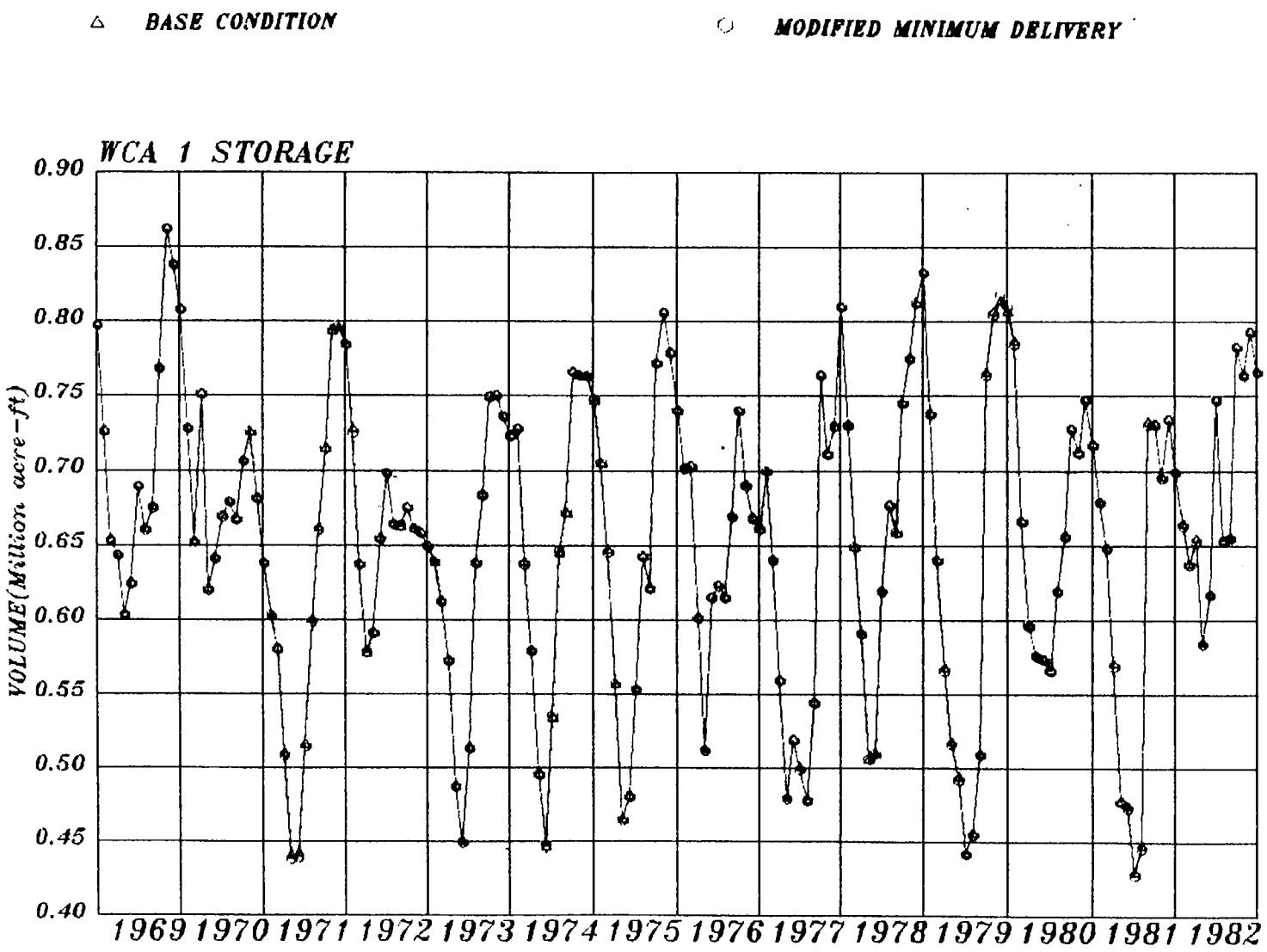
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
WCA 1 STORAGE
VOLUME(Million acre-ft)
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-6

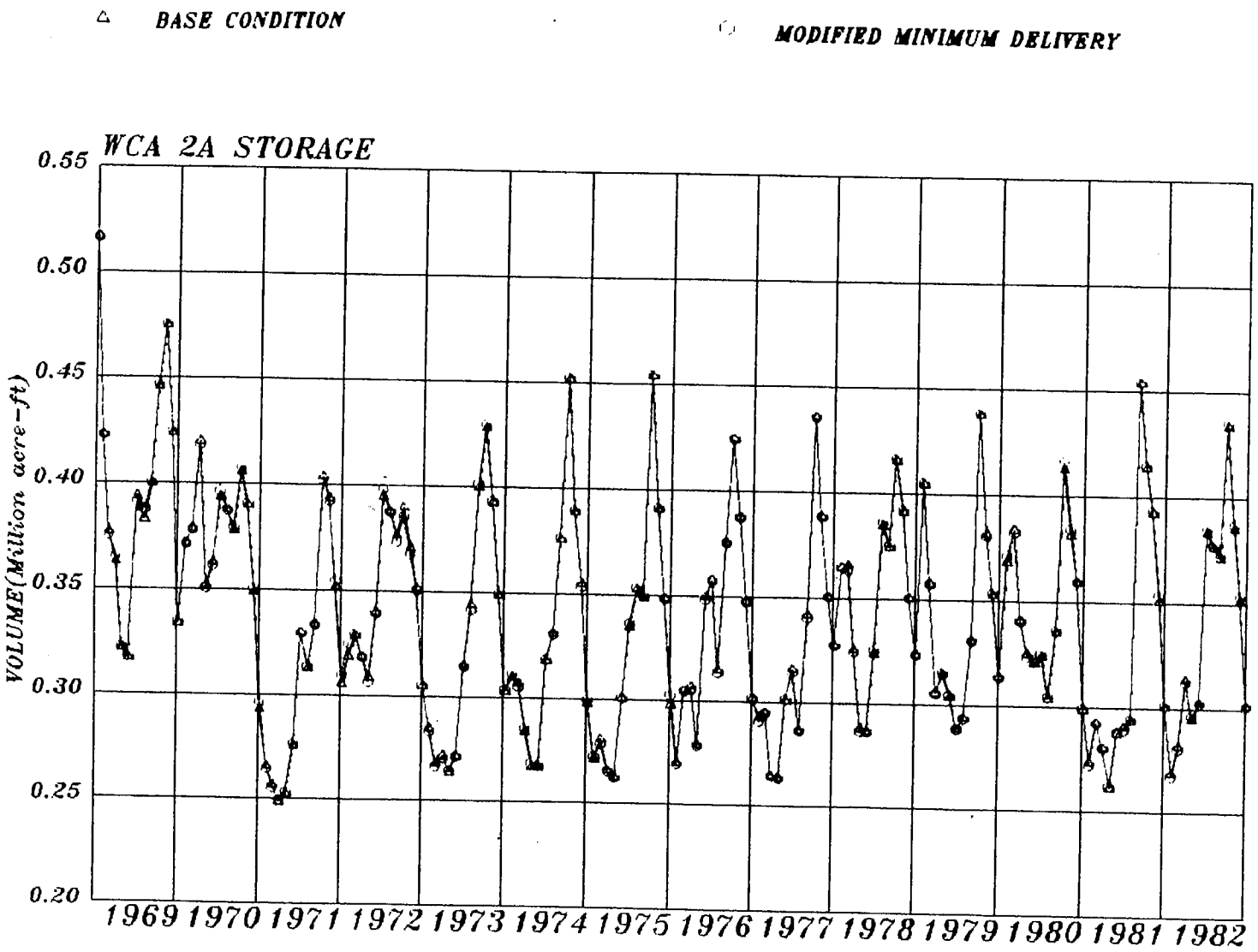
BASE CONDITION
MODIFIED MINIMUM DELIVERY
WCA 2A STORAGE
VOLUME(Million acre-ft)
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-7

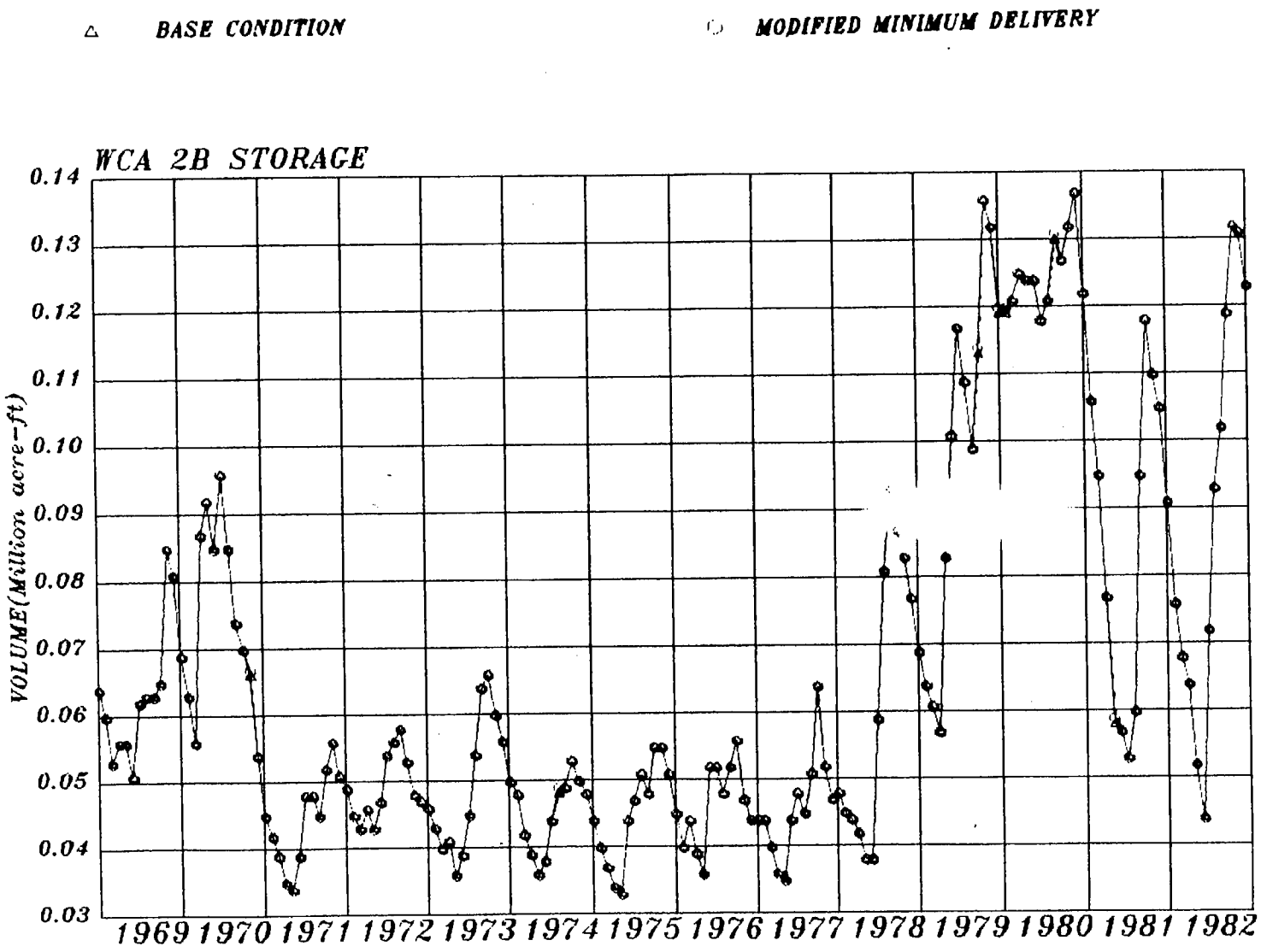
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
WCA 2B STORAGE
VOLUME(Million acre-ft)
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-8

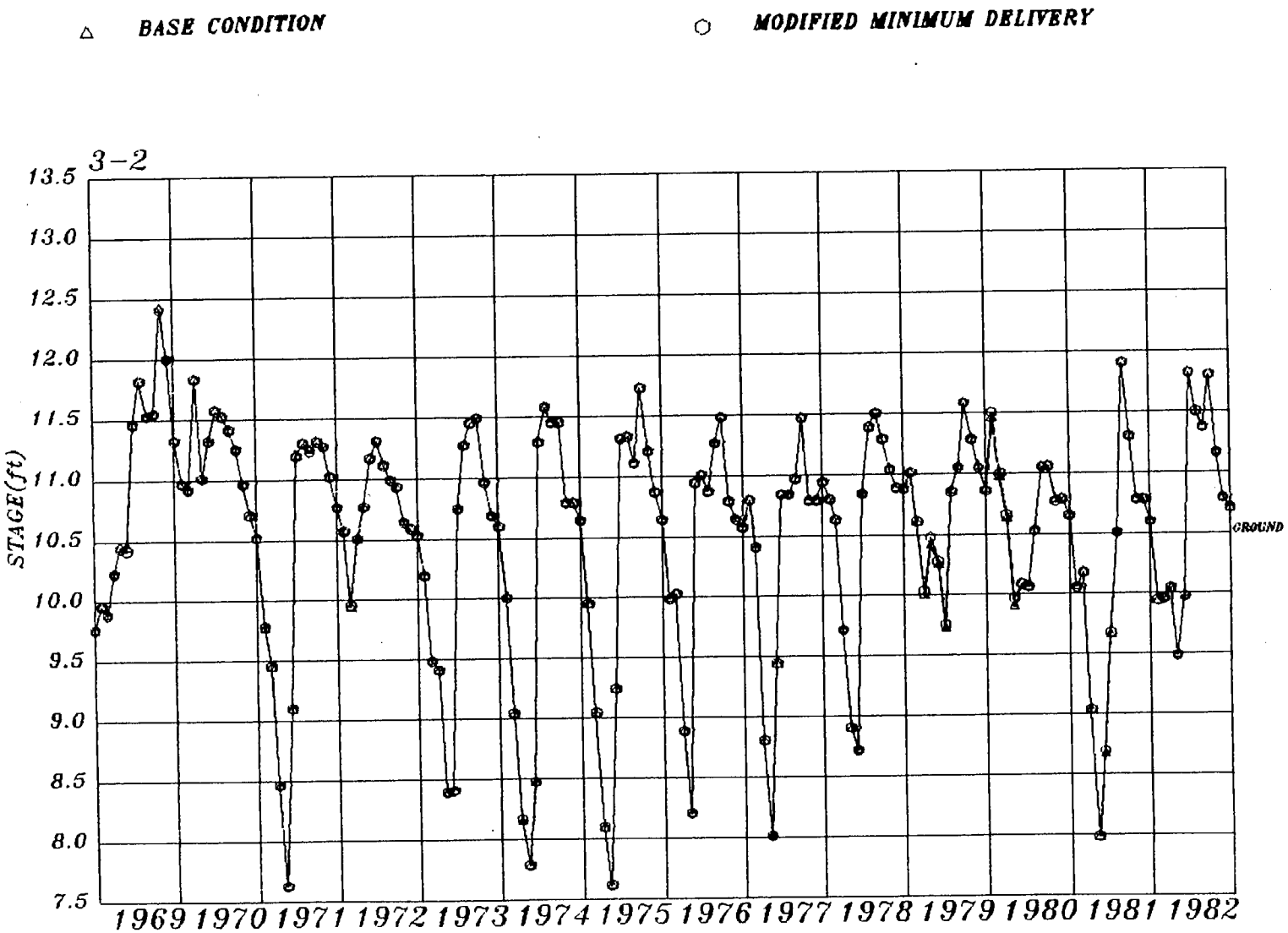
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
3-2
STAGE(ft)
13.5
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
GROUND

Figure E-9

△ BASE CONDITION ○ MODIFIED MINIMUM DELIVERY

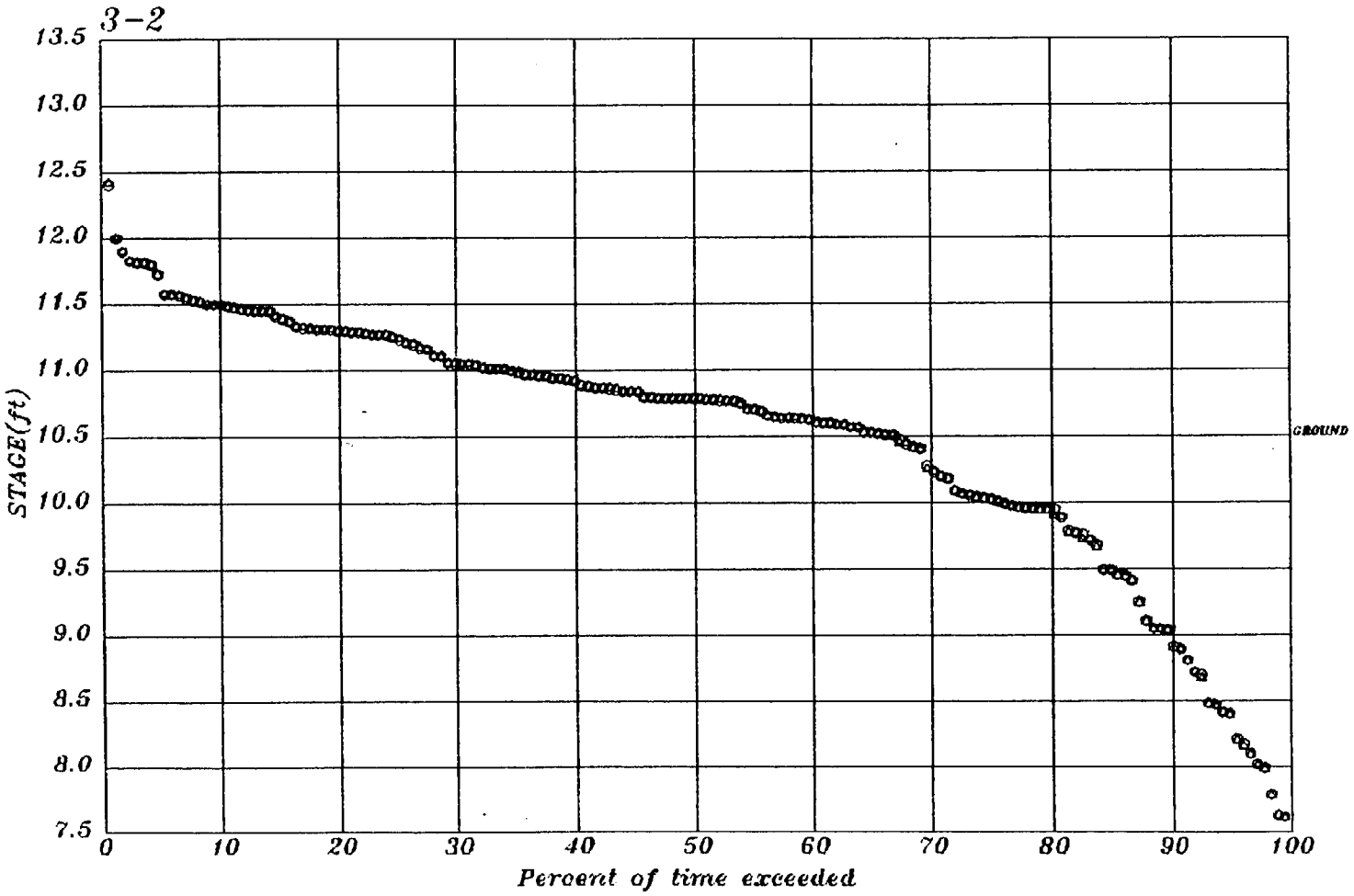

Figure E-10

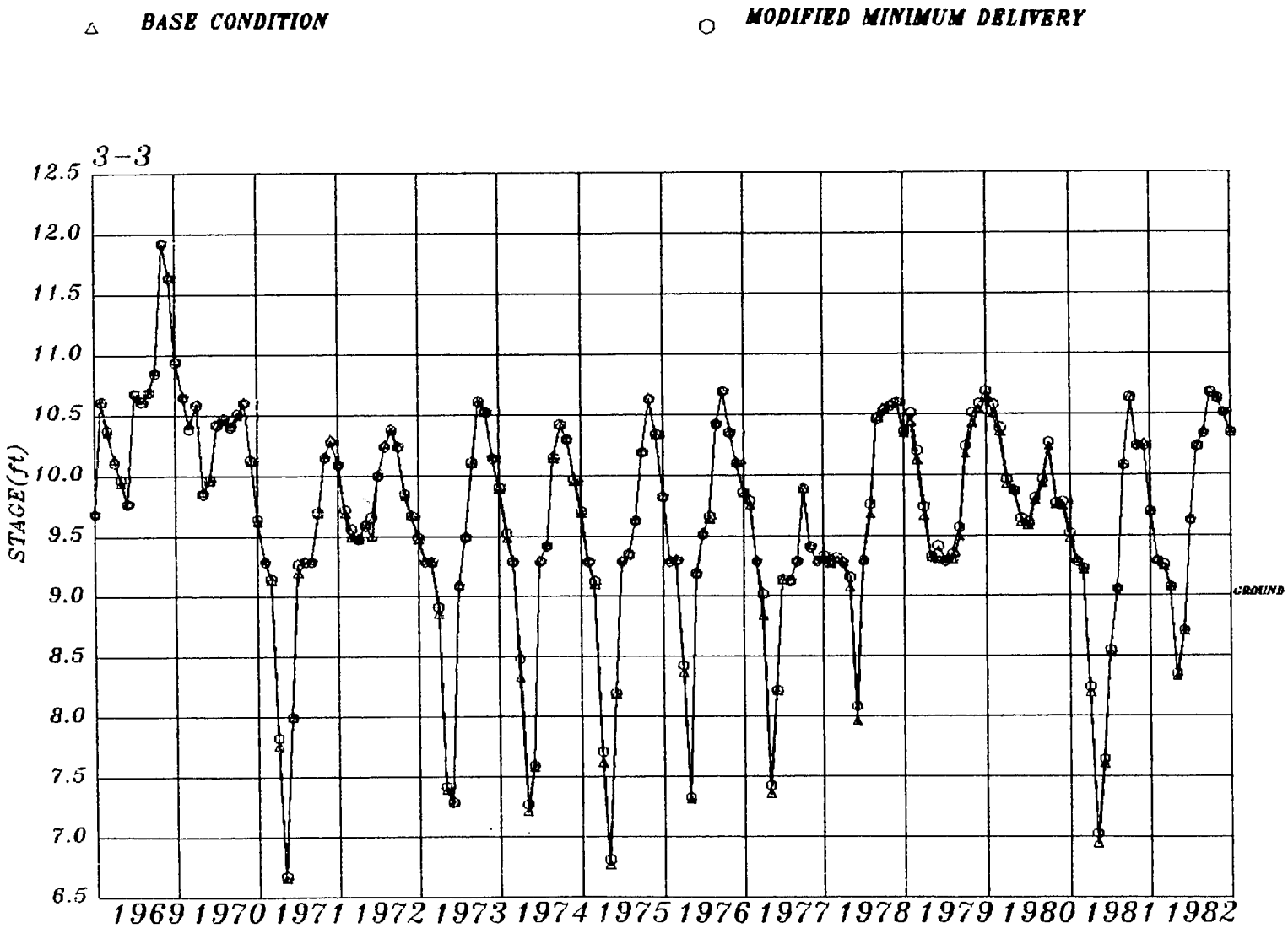
BASE CONDITION
MODIFIED MINIMUM DELIVERY
3-3
STAGE(ft)
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
GROUND

Figure E-11

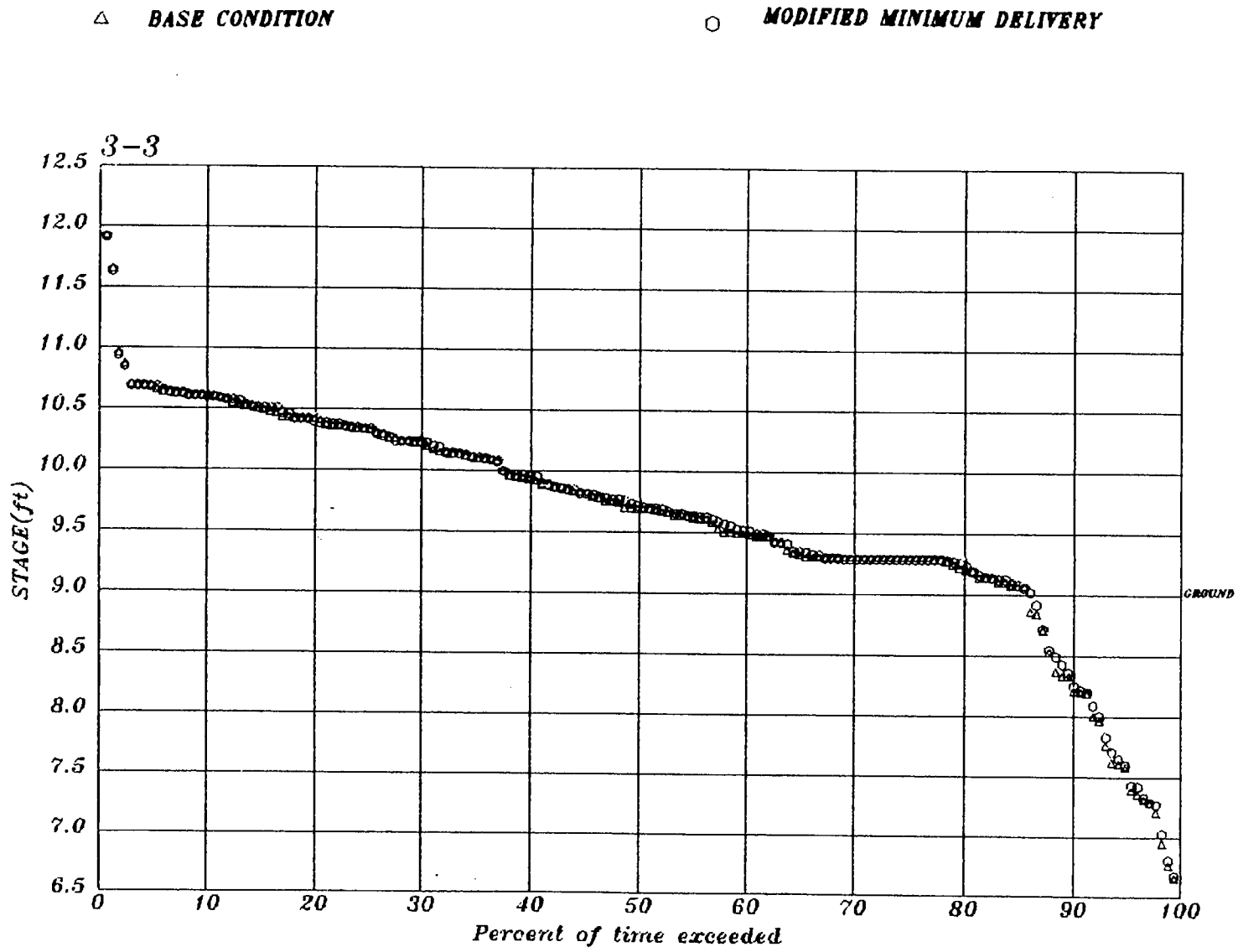
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
3-3
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-12

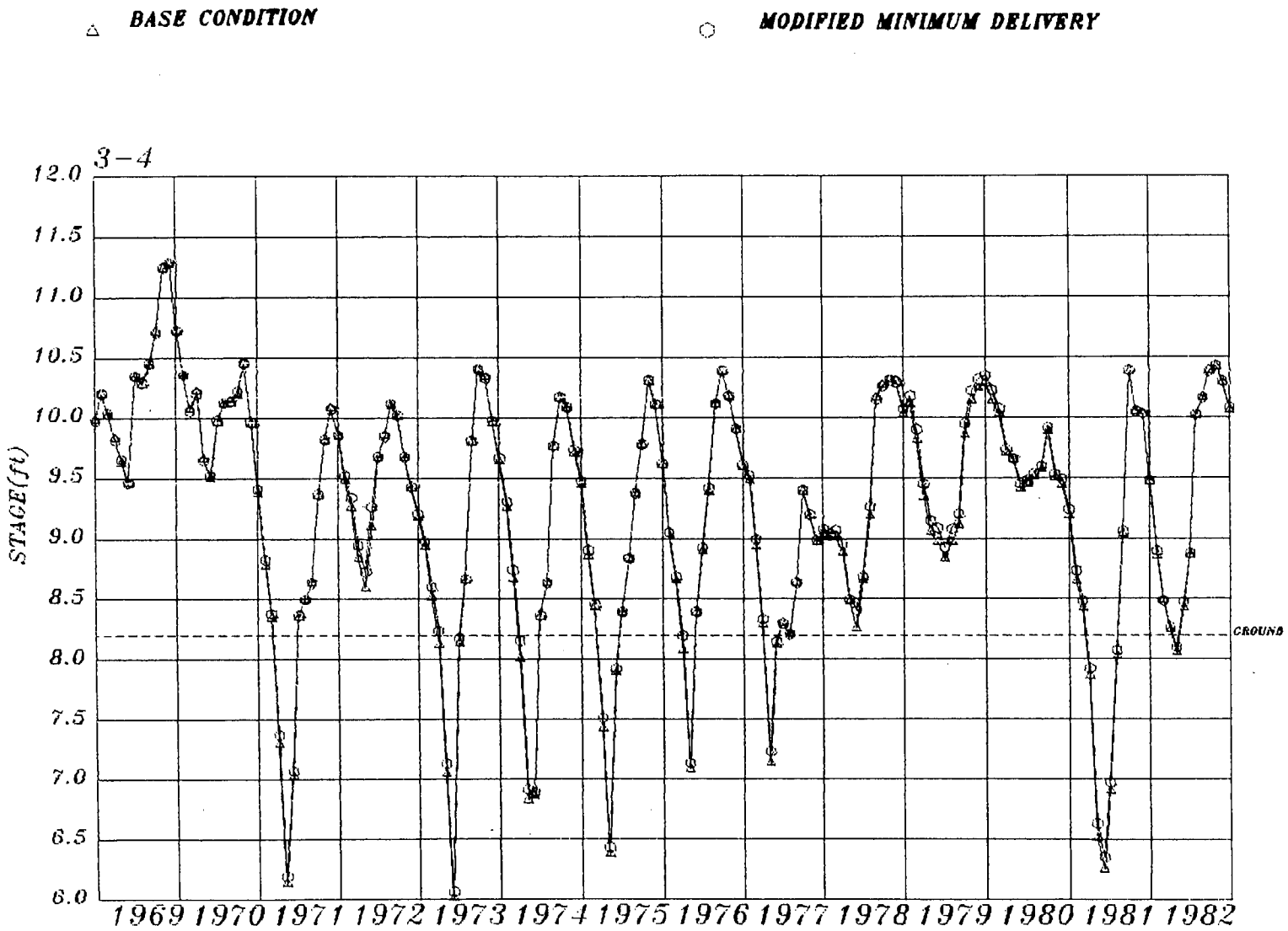
BASE CONDITION
MODIFIED MINIMUM DELIVERY
3-4
STAGE(ft)
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-13

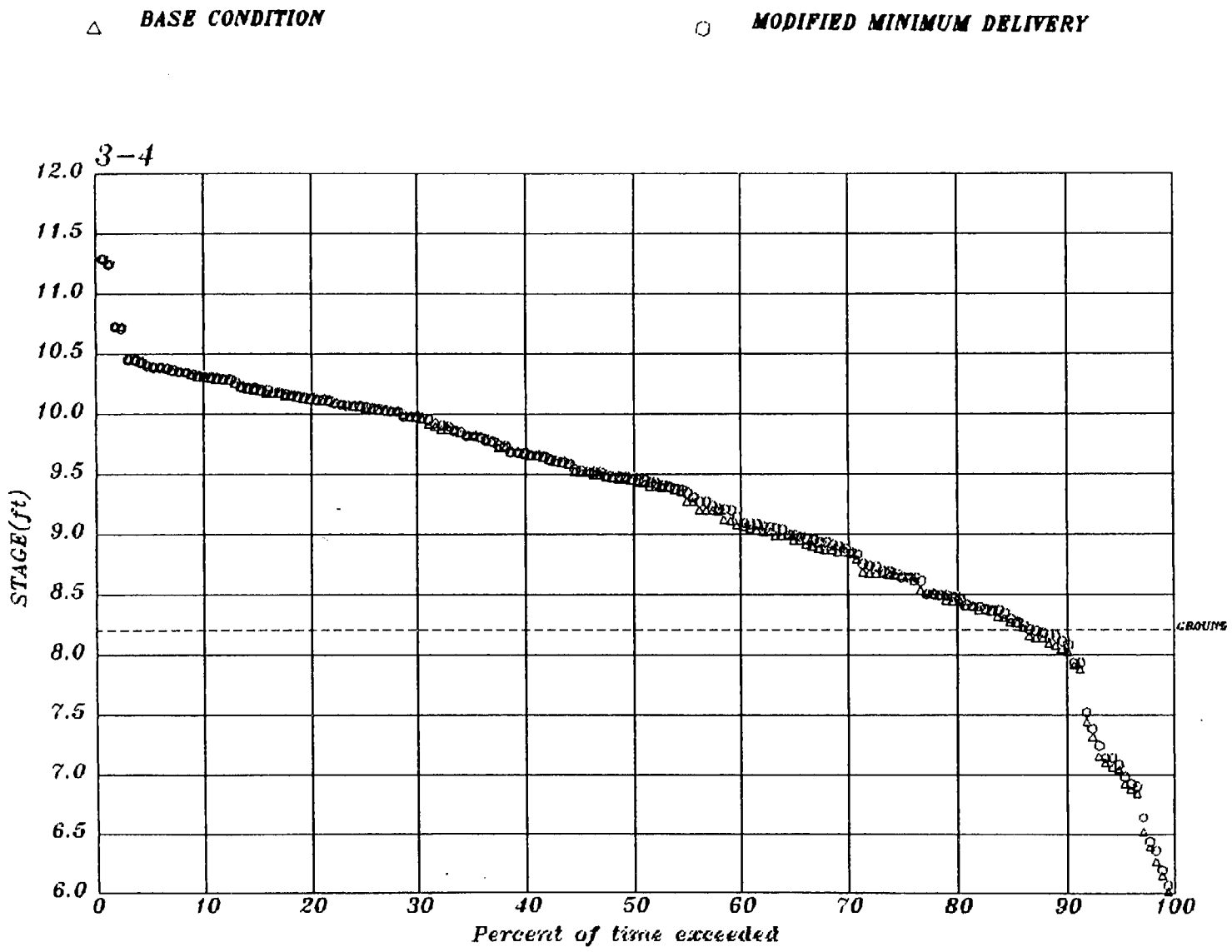
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
3-4
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-14

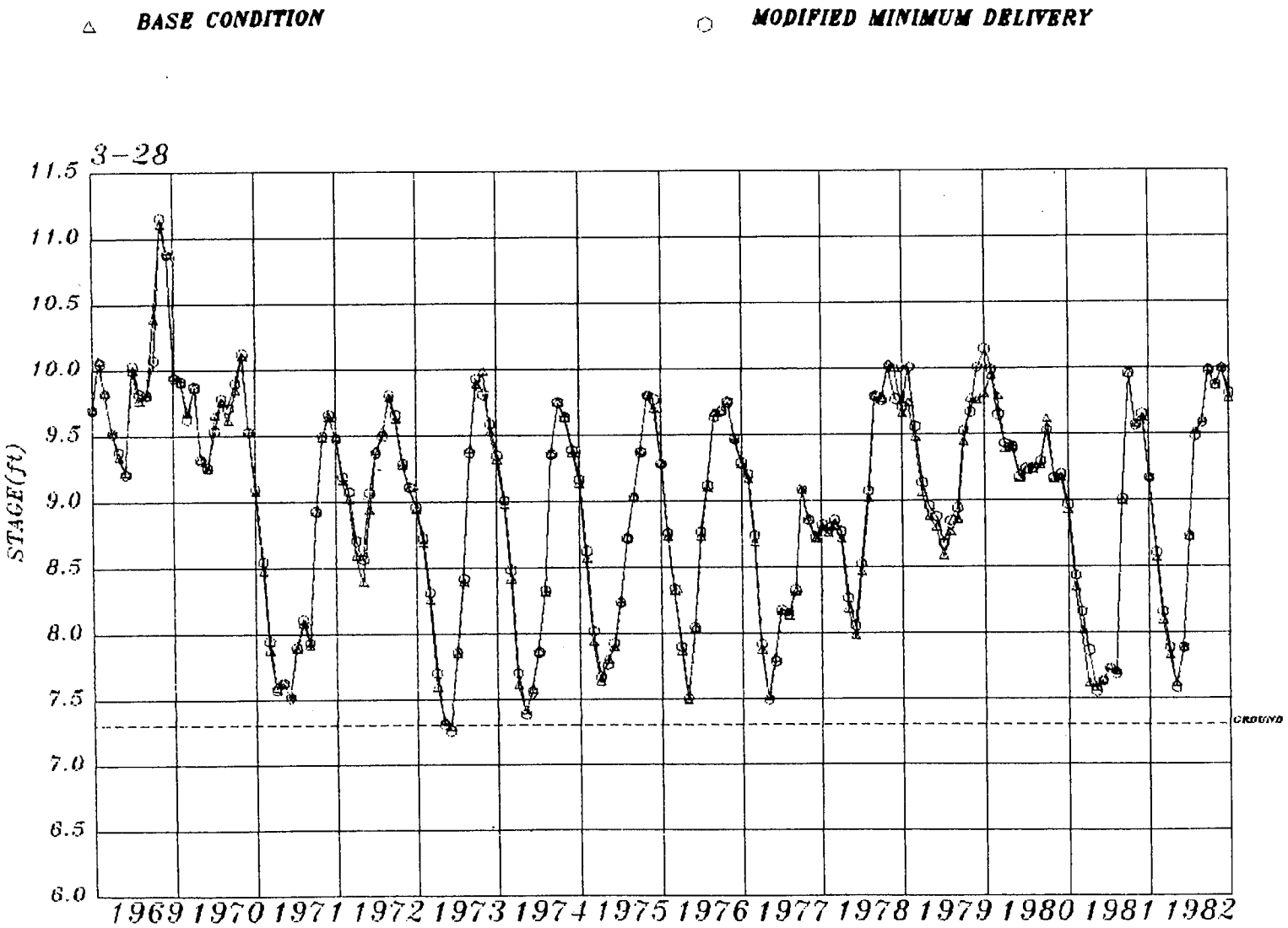
BASE CONDITION
MODIFIED MINIMUM DELIVERY
3-28
STAGE(ft)
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-15

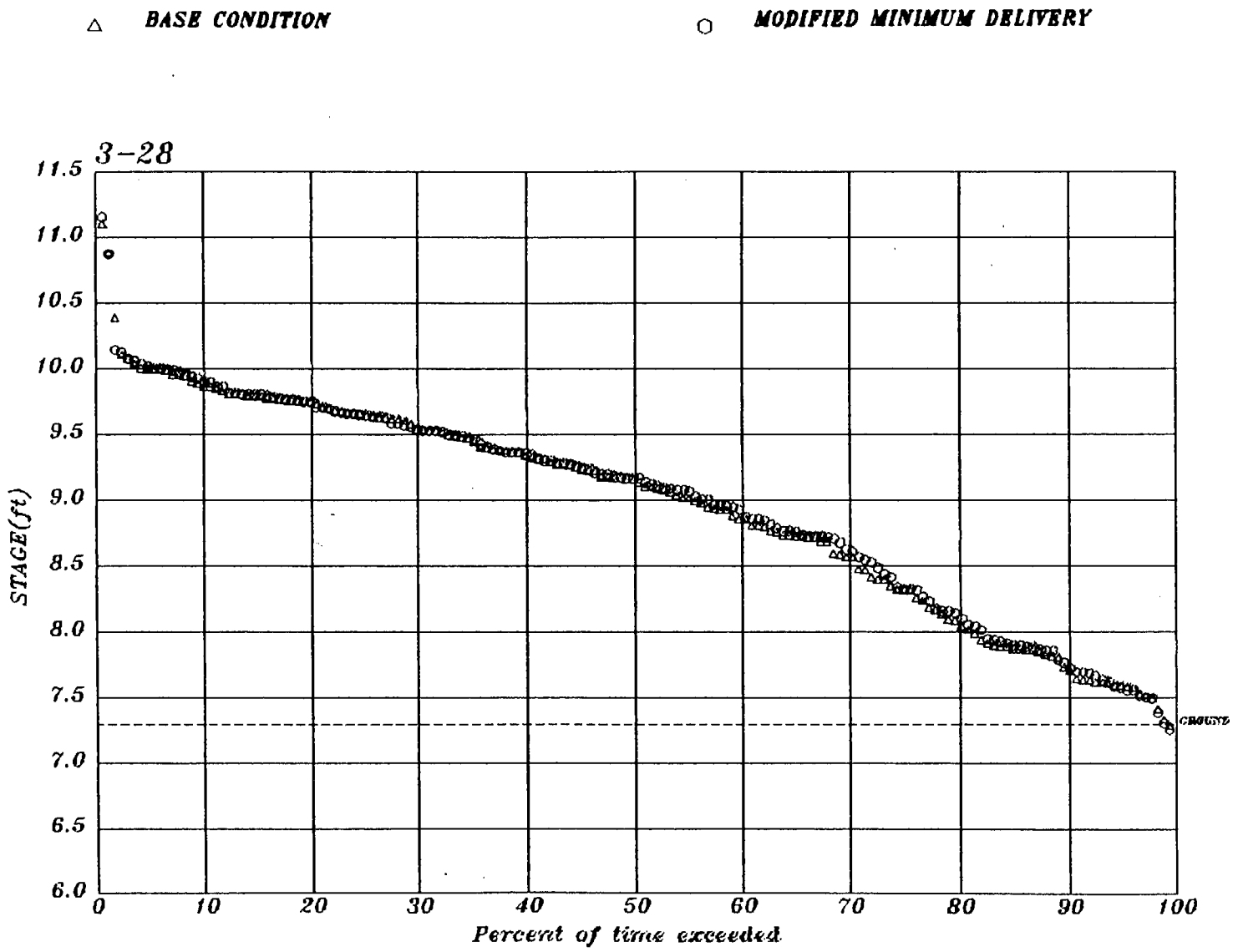
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
3-28
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-16

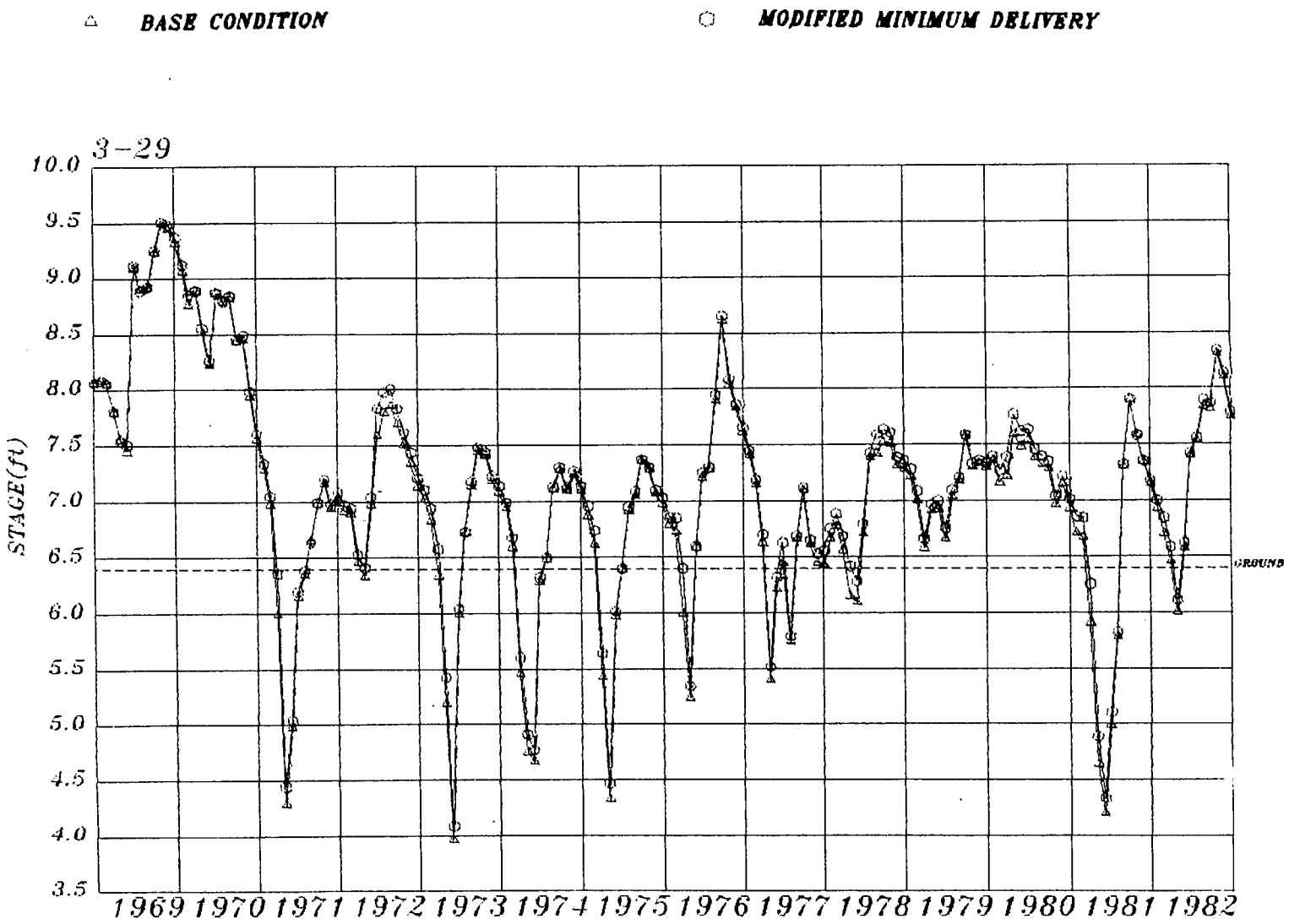
BASE CONDITION
MODIFIED MINIMUM DELIVERY
3-29
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-17

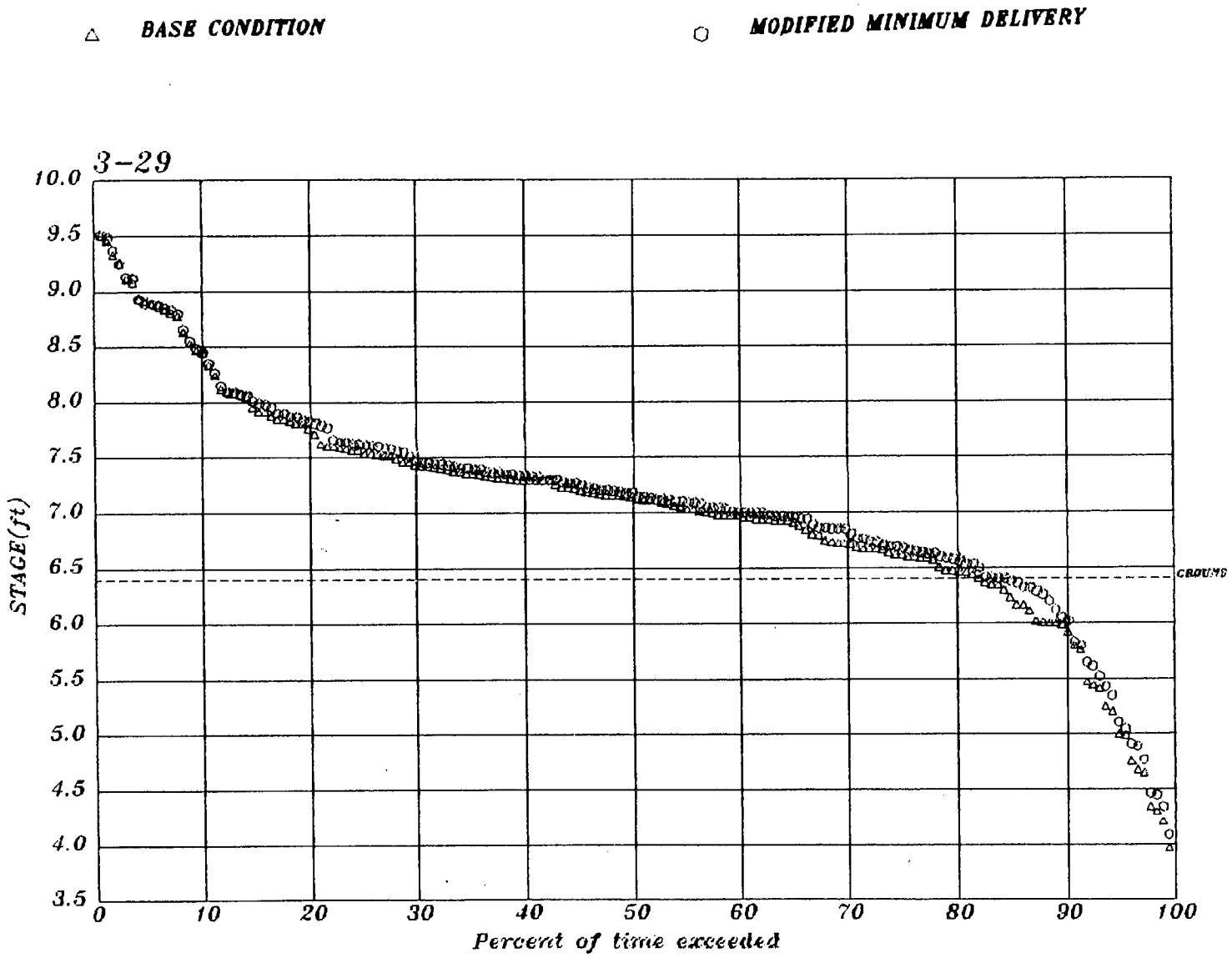
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
3-29
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-18

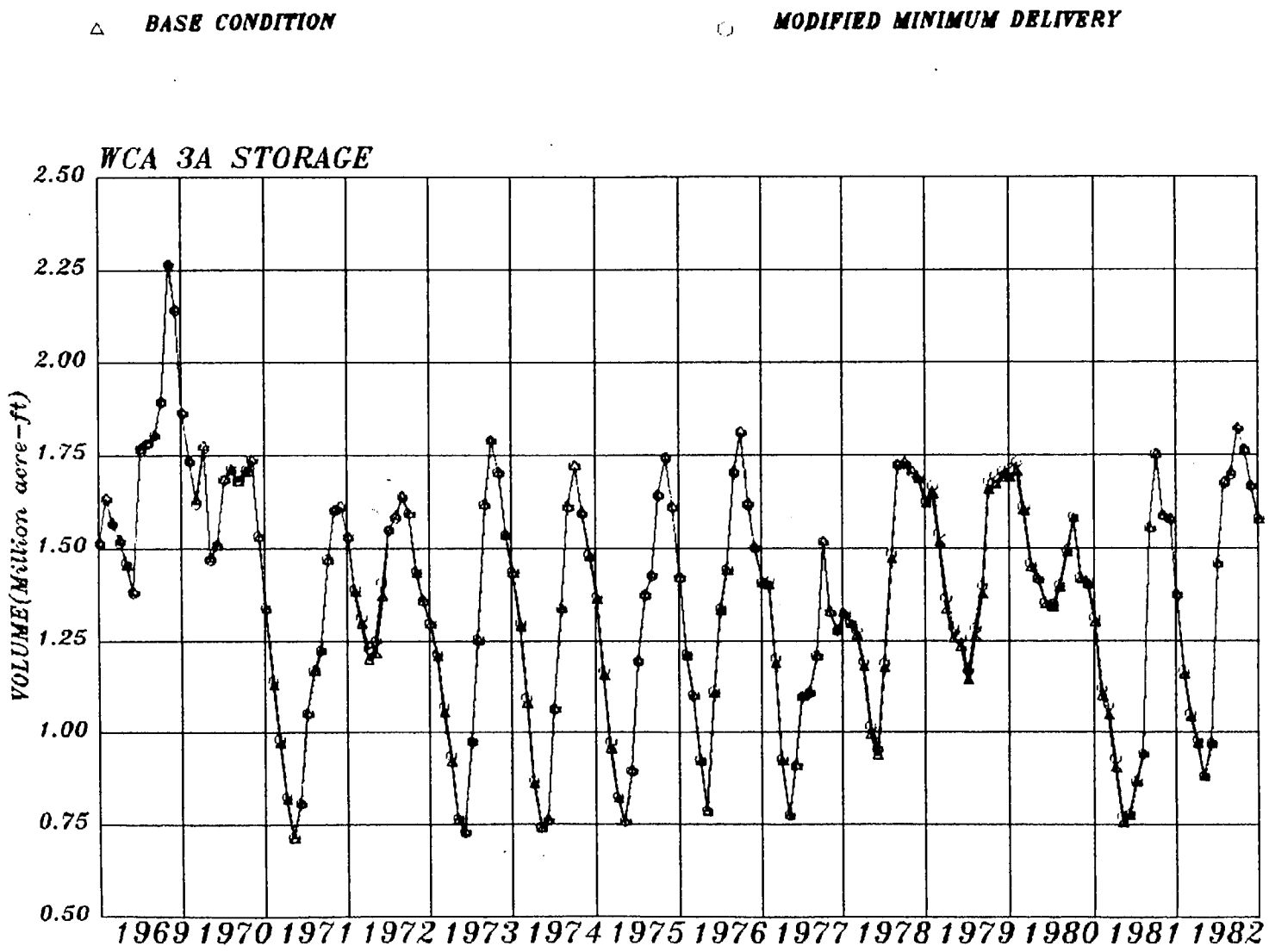
BASE CONDITION
MODIFIED MINIMUM DELIVERY
WCA 3A STORAGE
VOLUME(Million acre-ft)
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-19

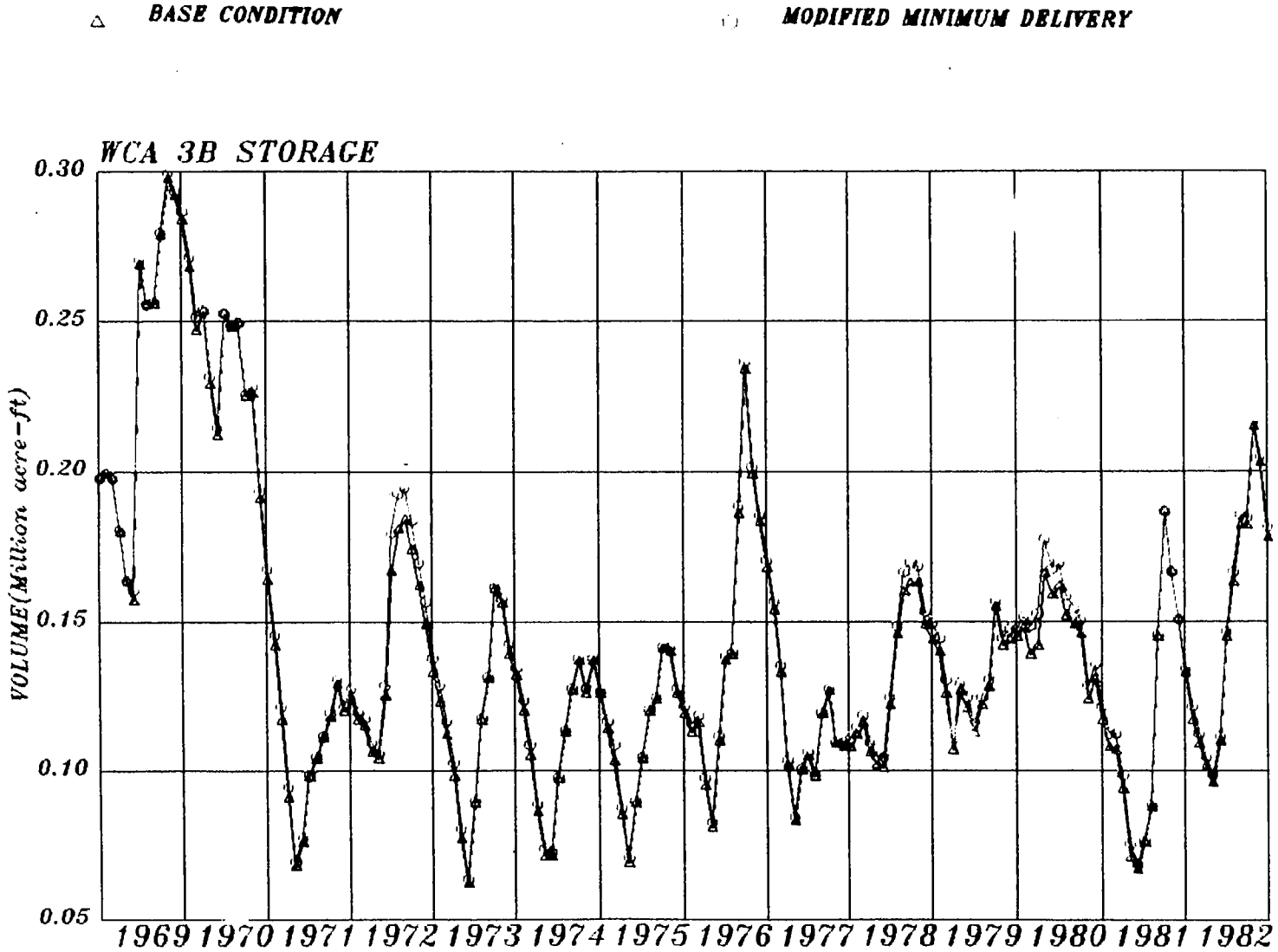
BASE CONDITION
MODIFIED MINIMUM DELIVERY
WCA 3B STORAGE
VOLUME(Million acre-ft)
0.30
0.25
0.20
0.15
0.10
0.05
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-20

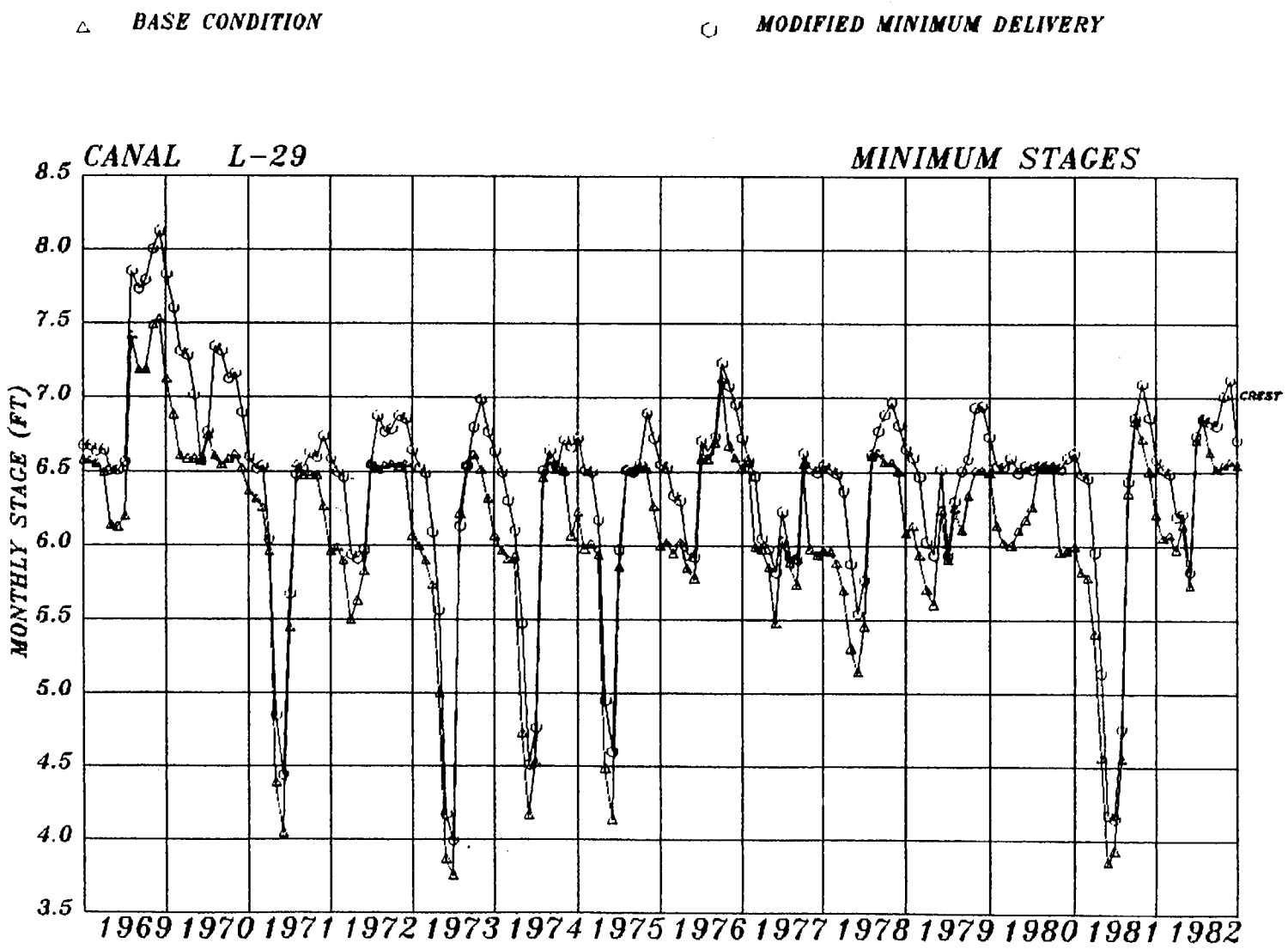
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L-29
MINIMUM STAGES
MONTHLY STAGE (FT)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-21

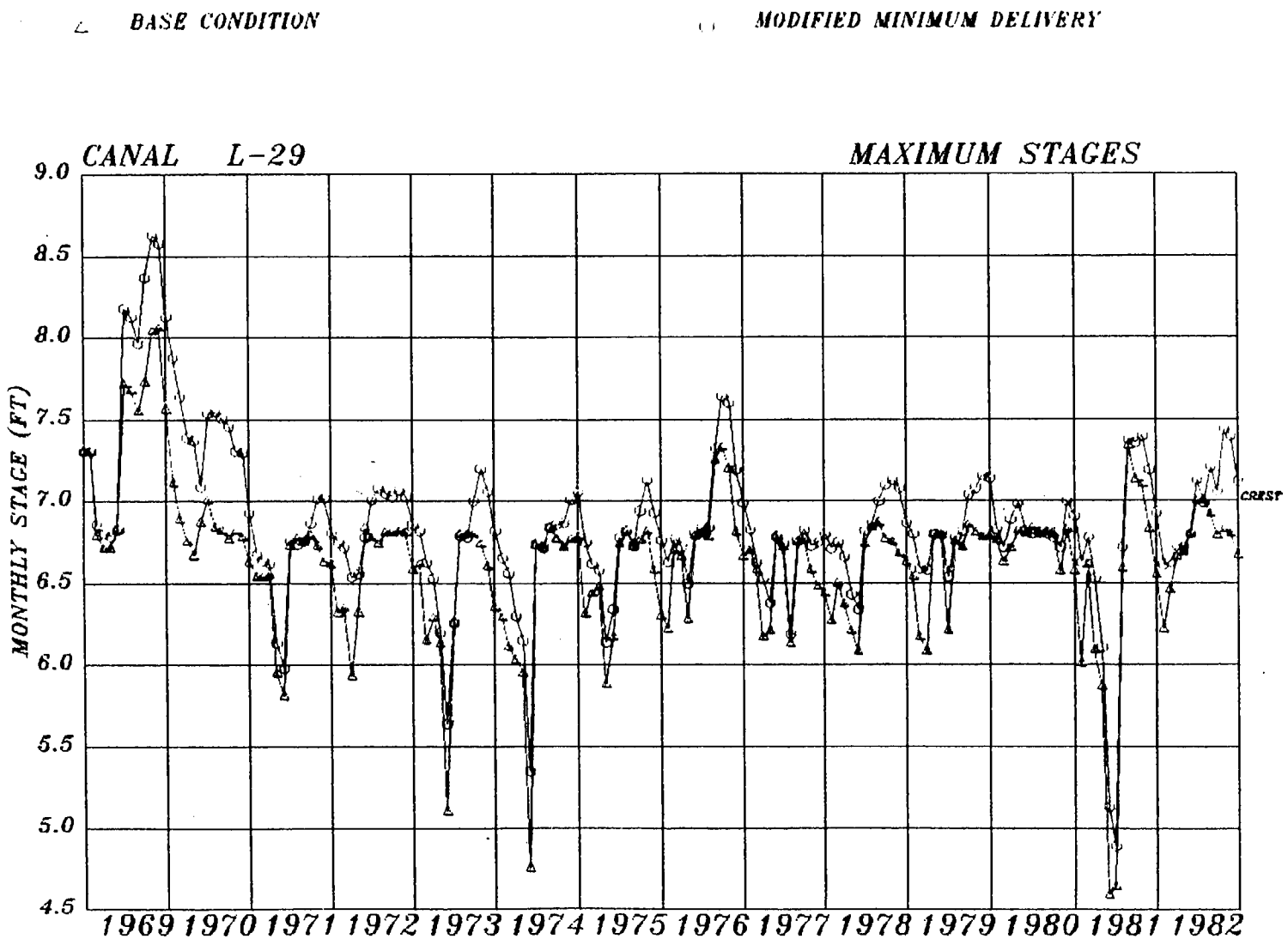
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L-29
MAXIMUM STAGES
MONTHLY STAGE (FT)
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CRRST

Figure E-23

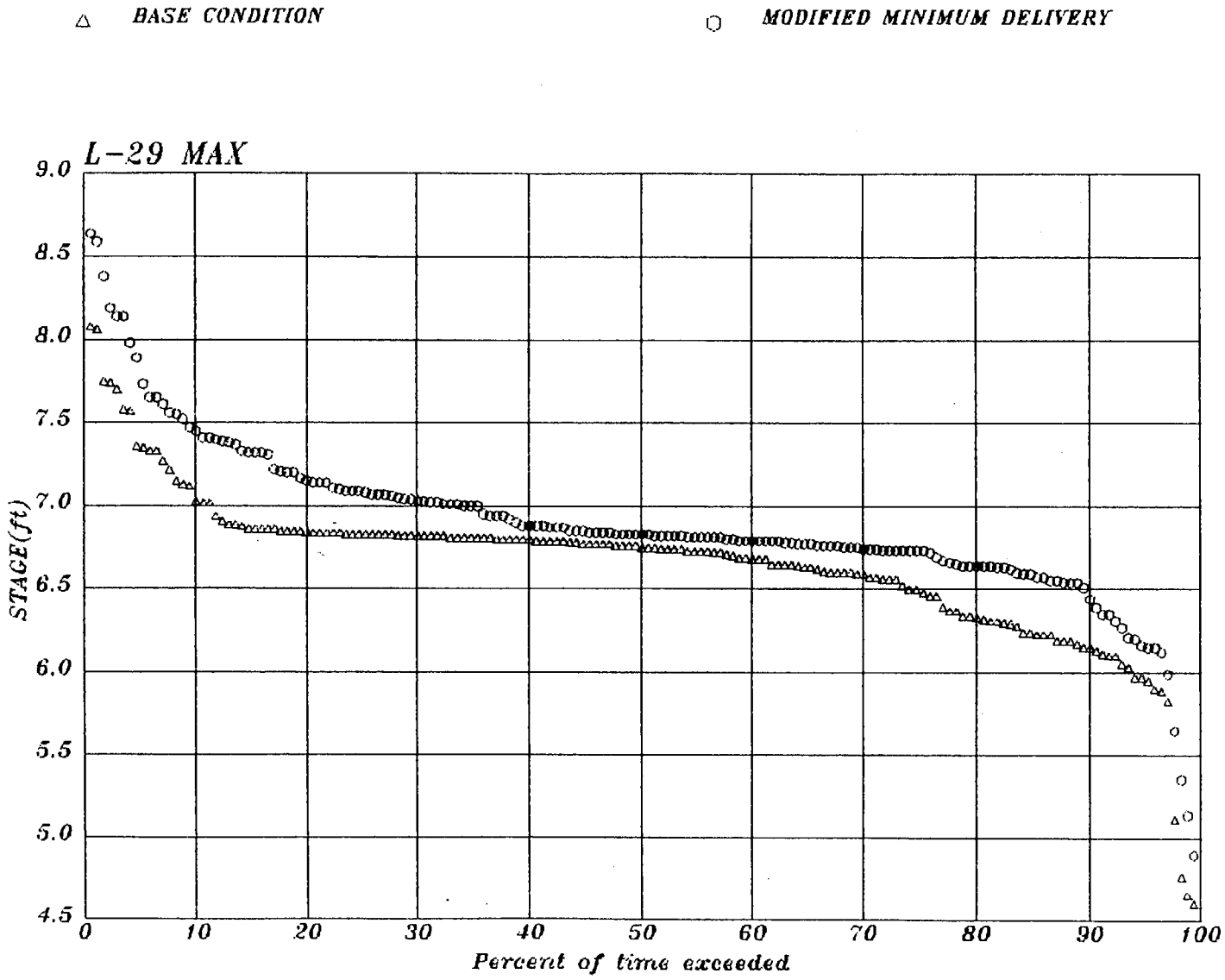
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
L-29 MAX
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-24

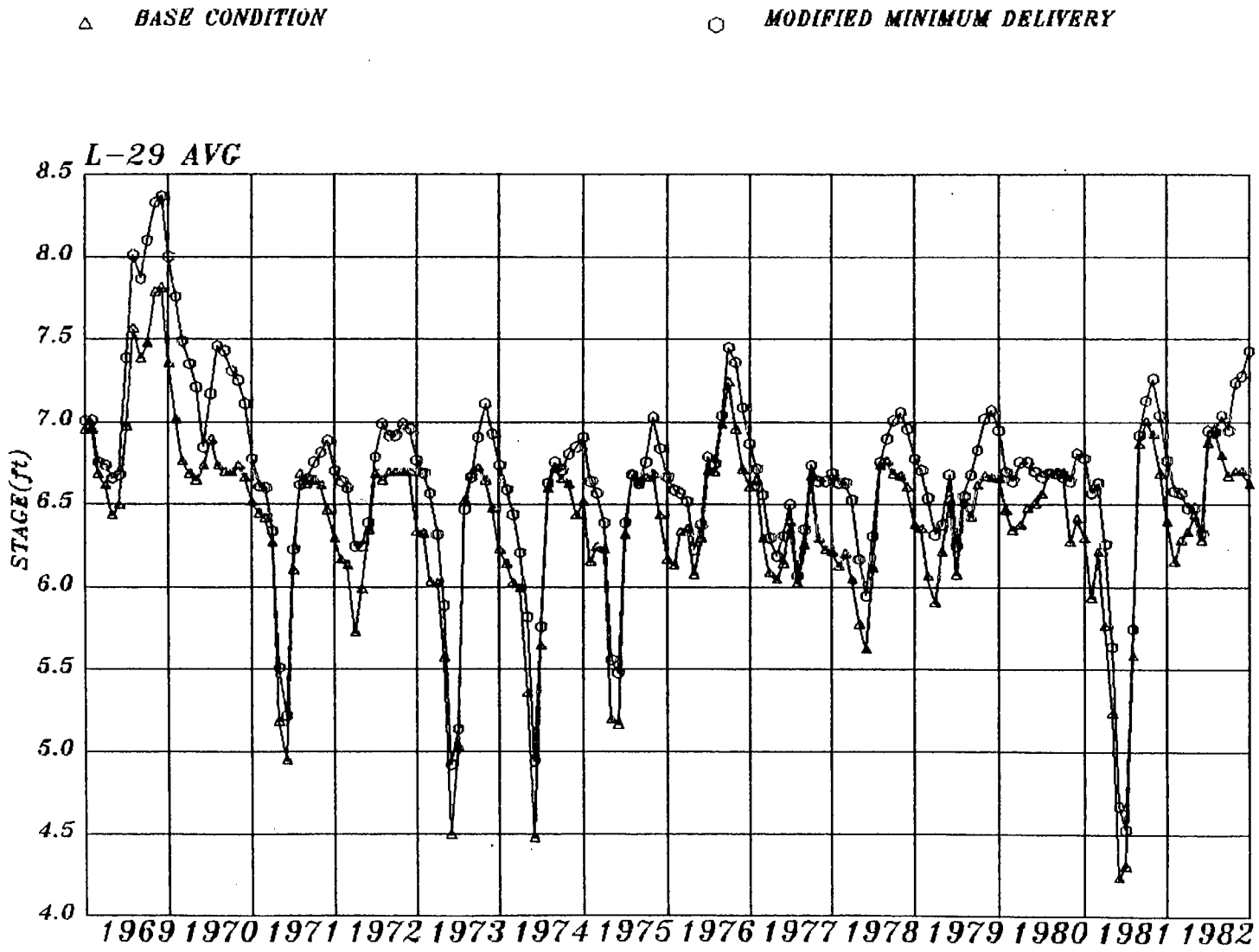
BASE CONDITION
MODIFIED MINIMUM DELIVERY
L-29 AVG
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-25

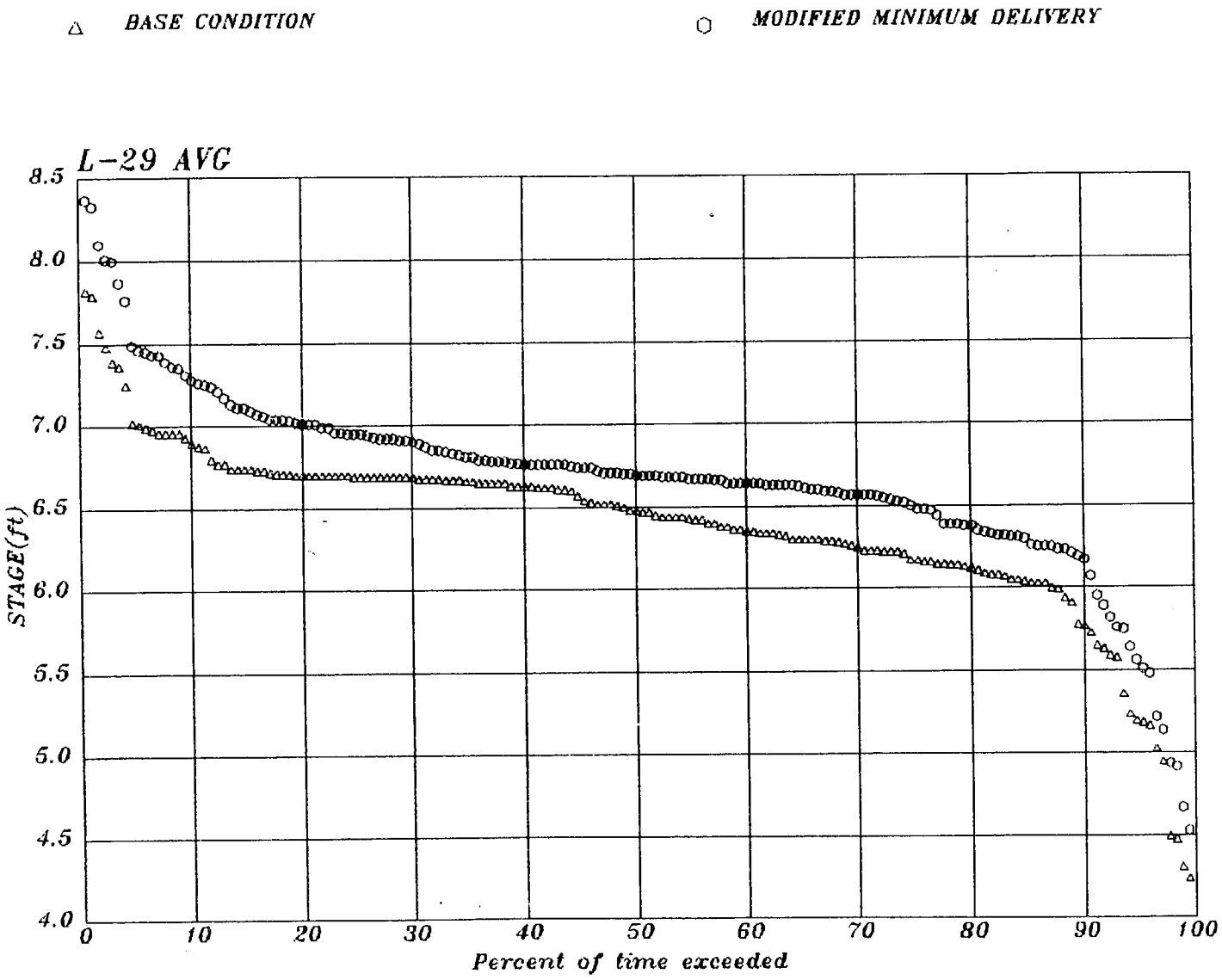
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
L-29 AVG
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-26

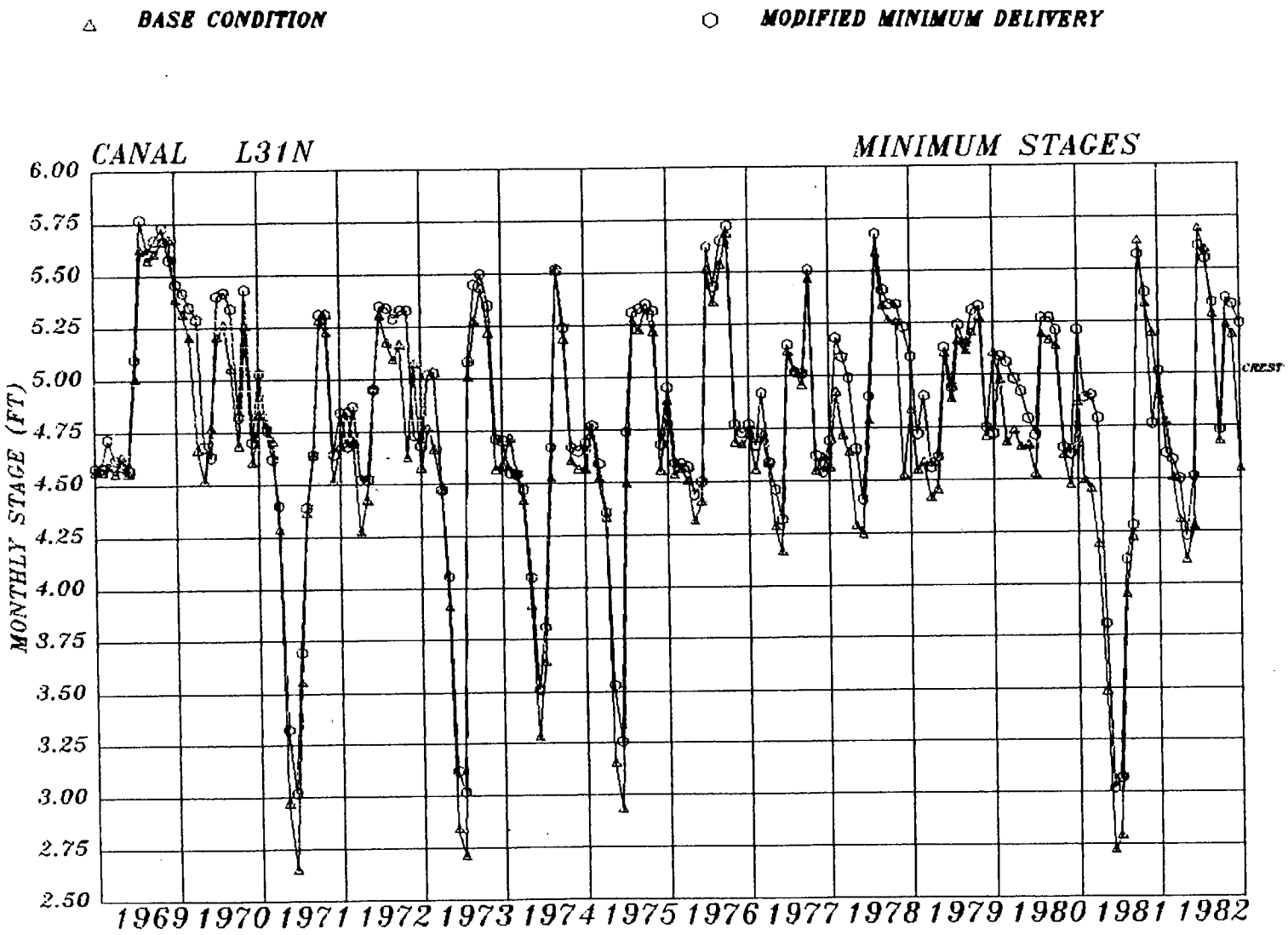
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L31N
MINIMUM STAGES
MONTHLY STAGE (FT)
6.00
5.75
5.50
5.25
5.00
4.75
4.50
4.25
4.00
3.75
3.50
3.25
3.00
2.75
2.50
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-27

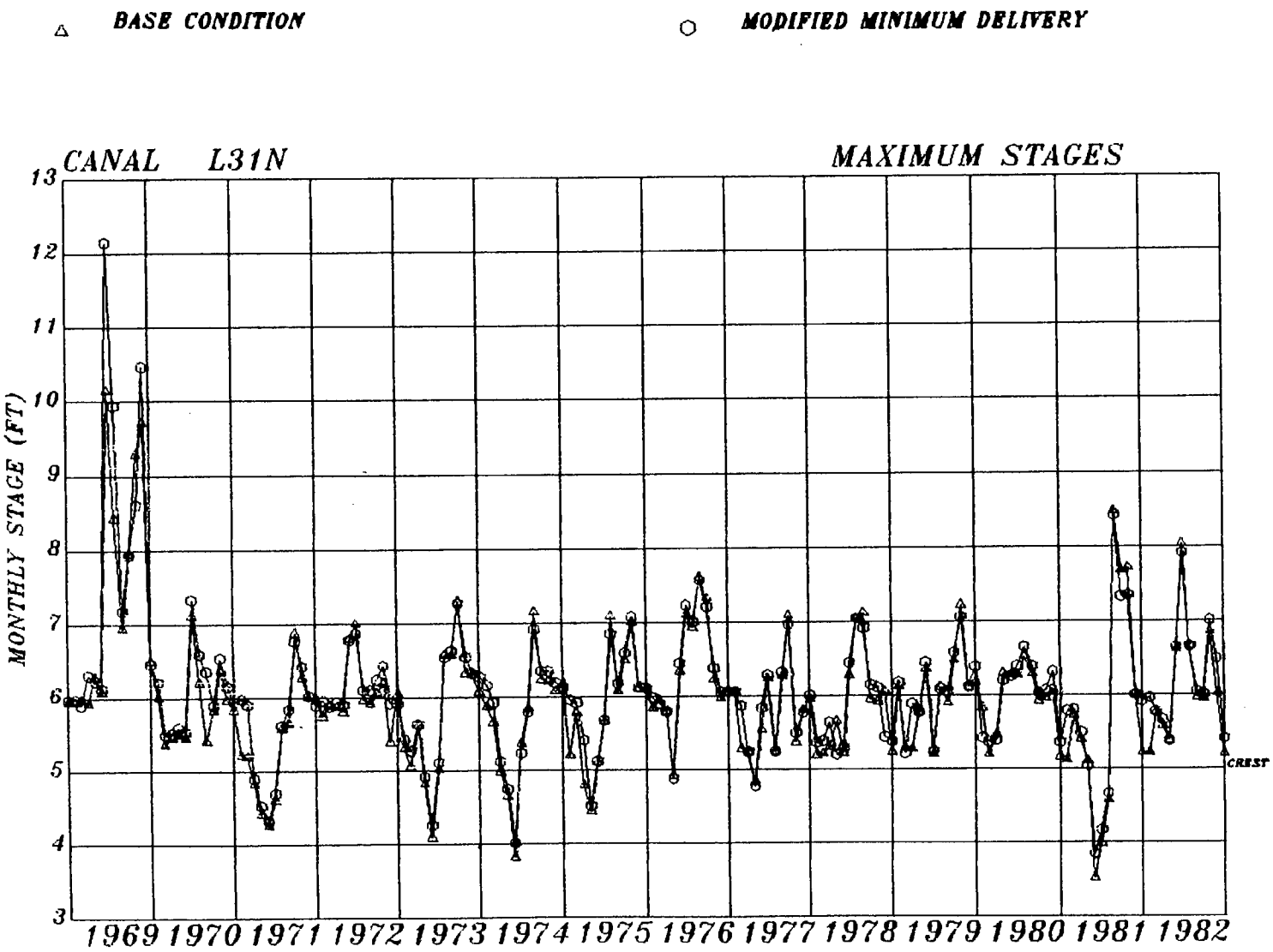
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L31N
MAXIMUM STAGES
MONTHLY STAGE (FT)
13
12
11
10
9
8
7
6
5
4
3
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-28

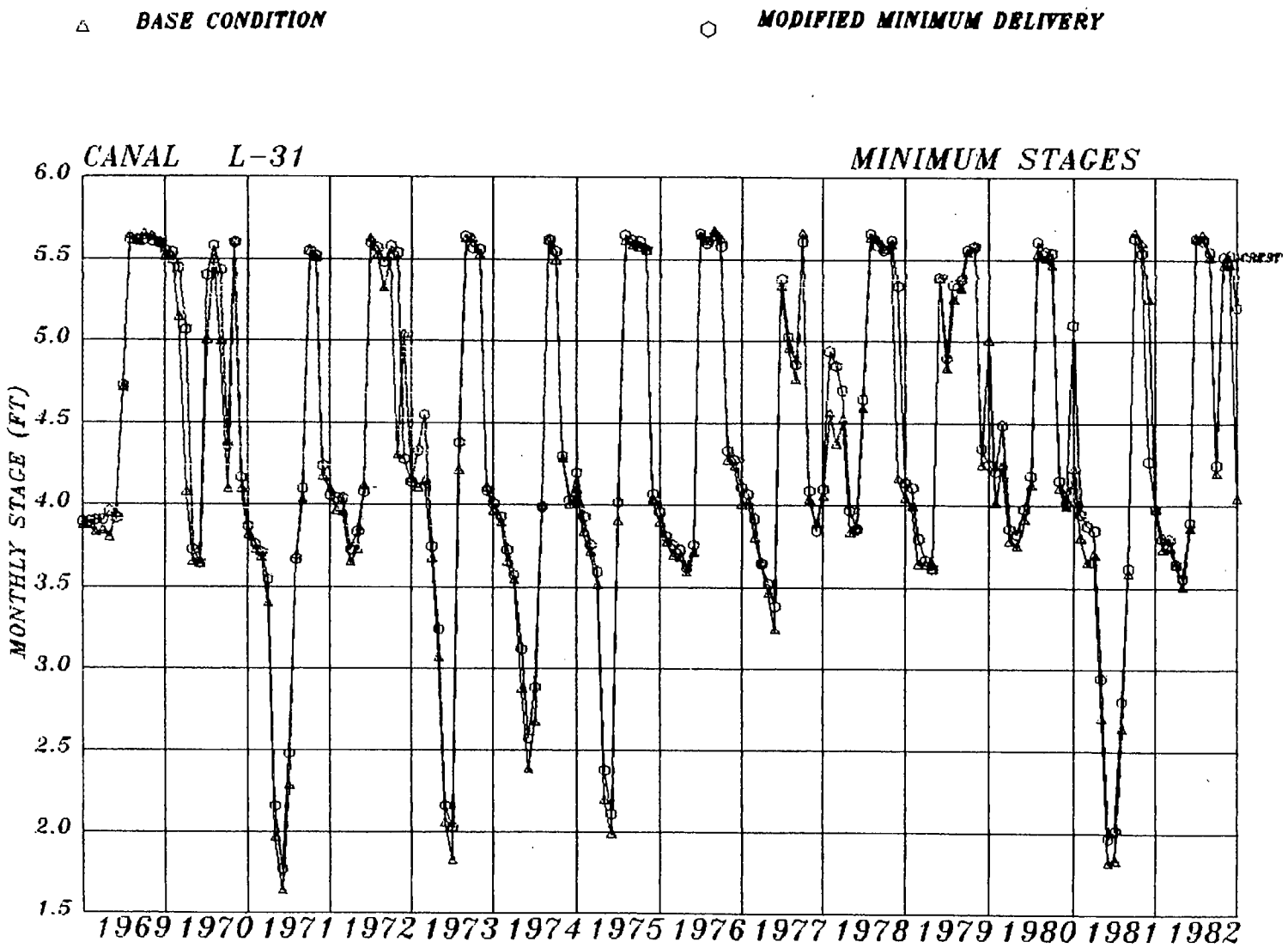
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L-31
MINIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-29

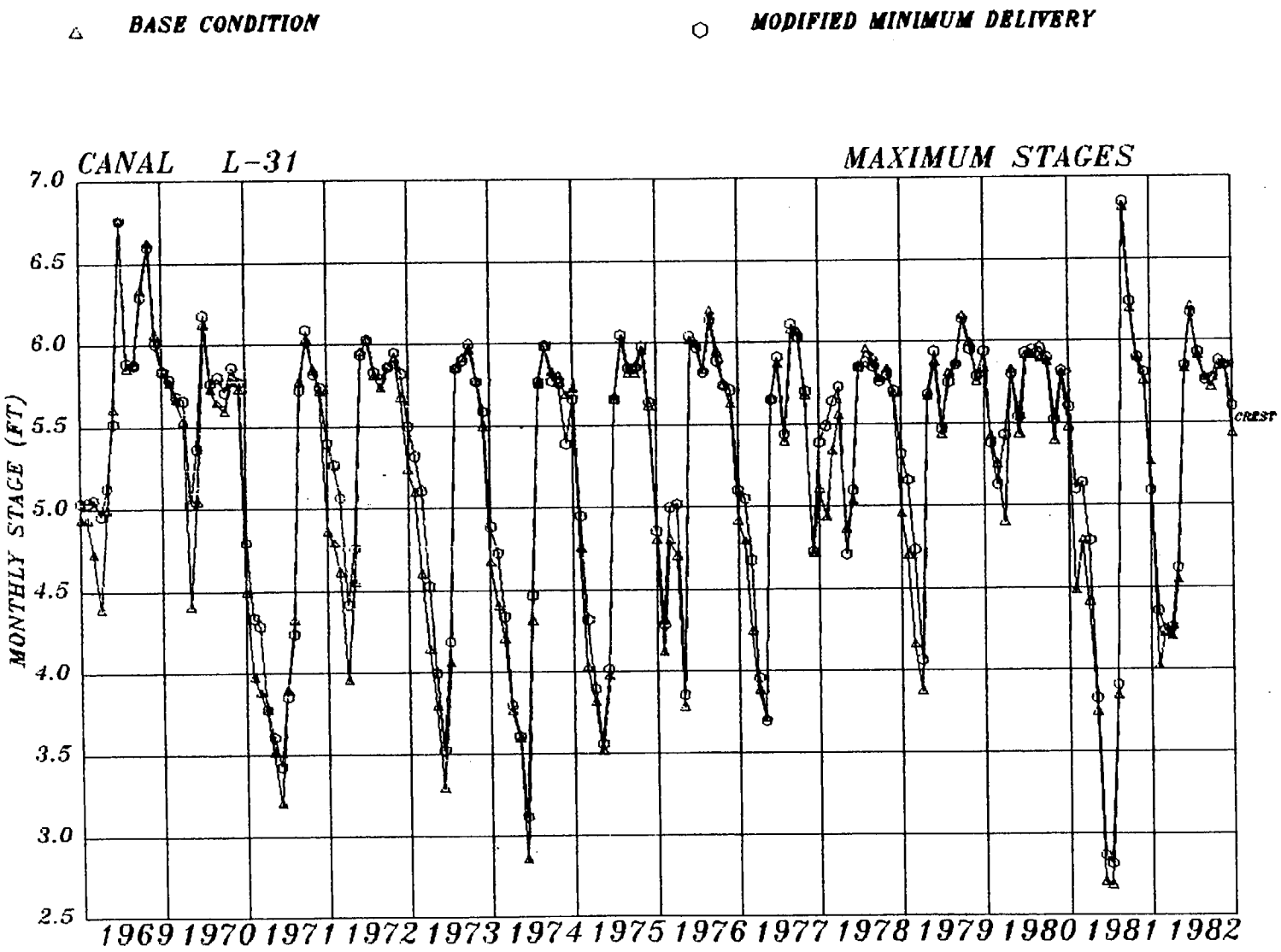
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L-31
MAXIMUM STAGES
MONTHLY STAGE (FT)
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-30

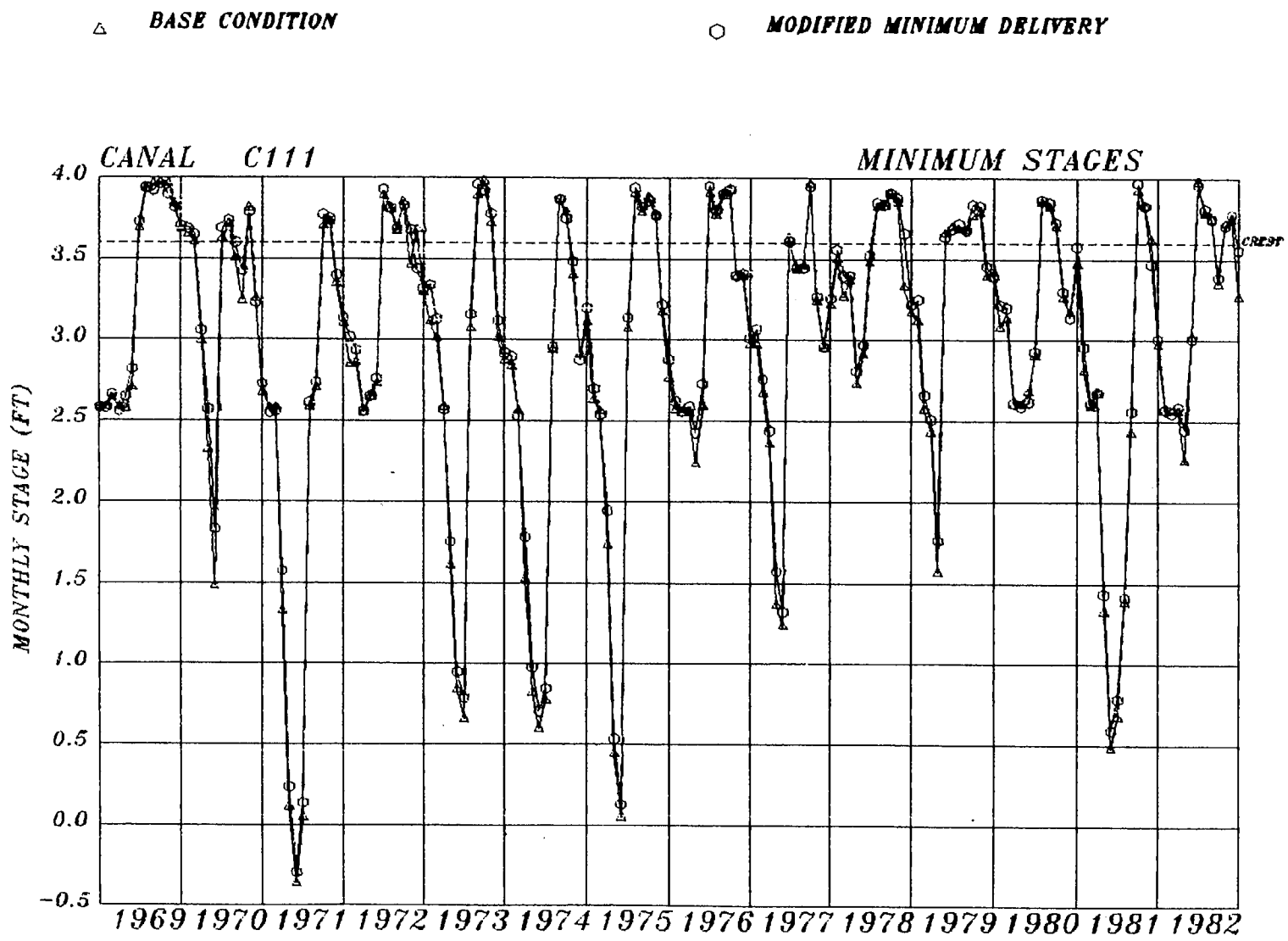
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL C111
MINIMUM STAGES
CREST
MONTHLY STAGE (FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-31

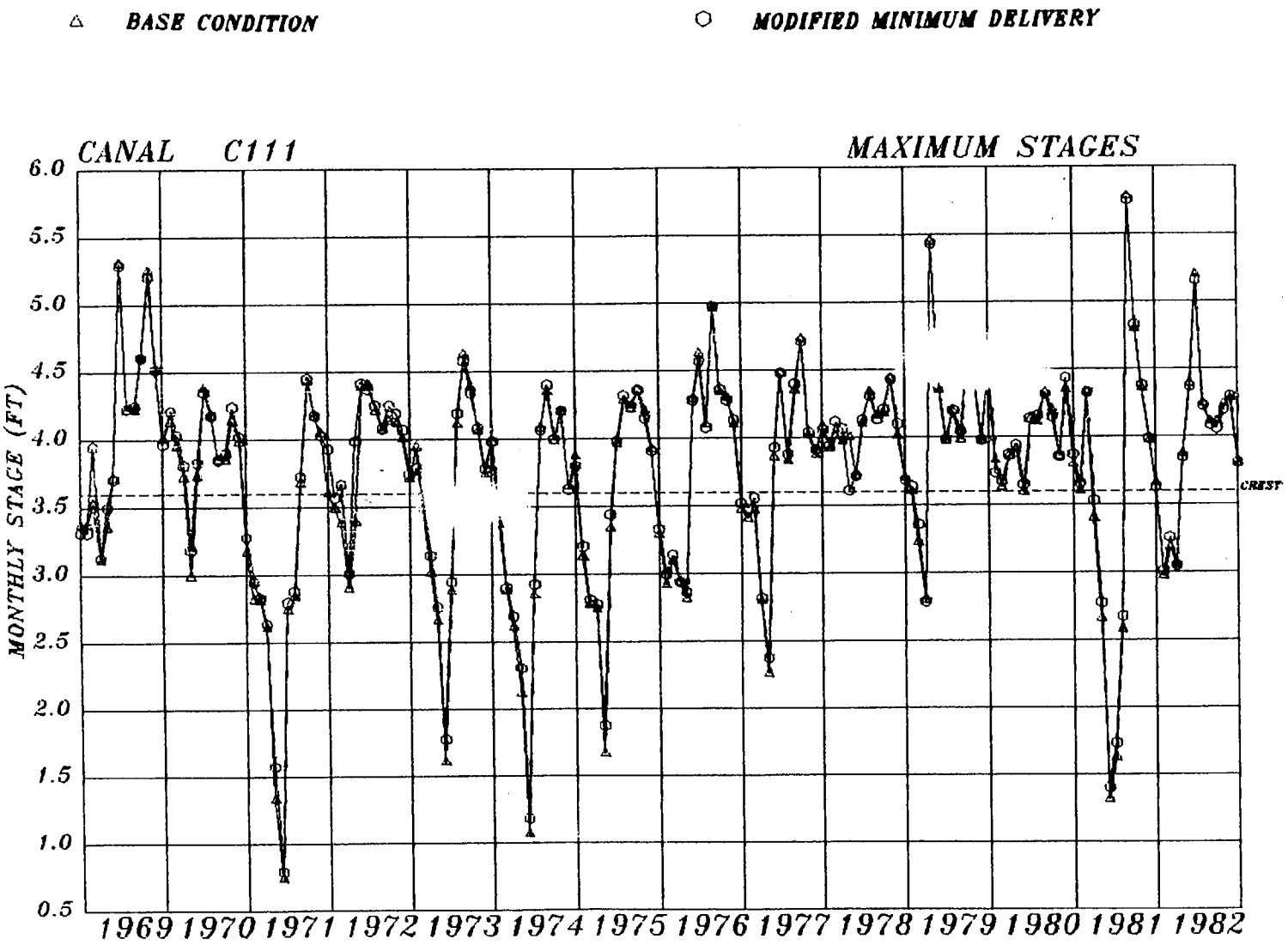
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
CANAL C111
MAXIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-32

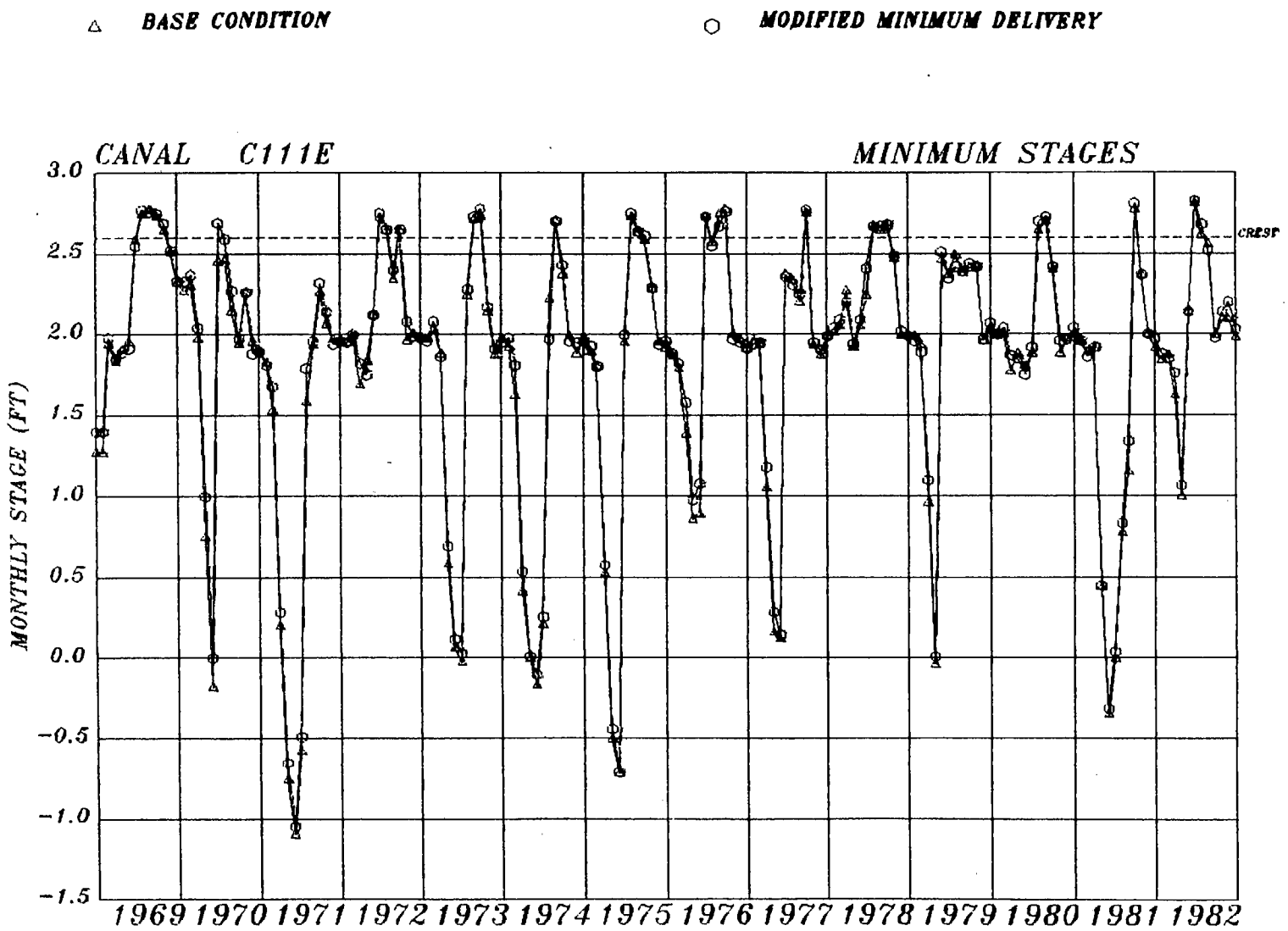
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL C111E
MINIMUM STAGES
MONTHLY STAGE (FT)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
CRESF
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-33

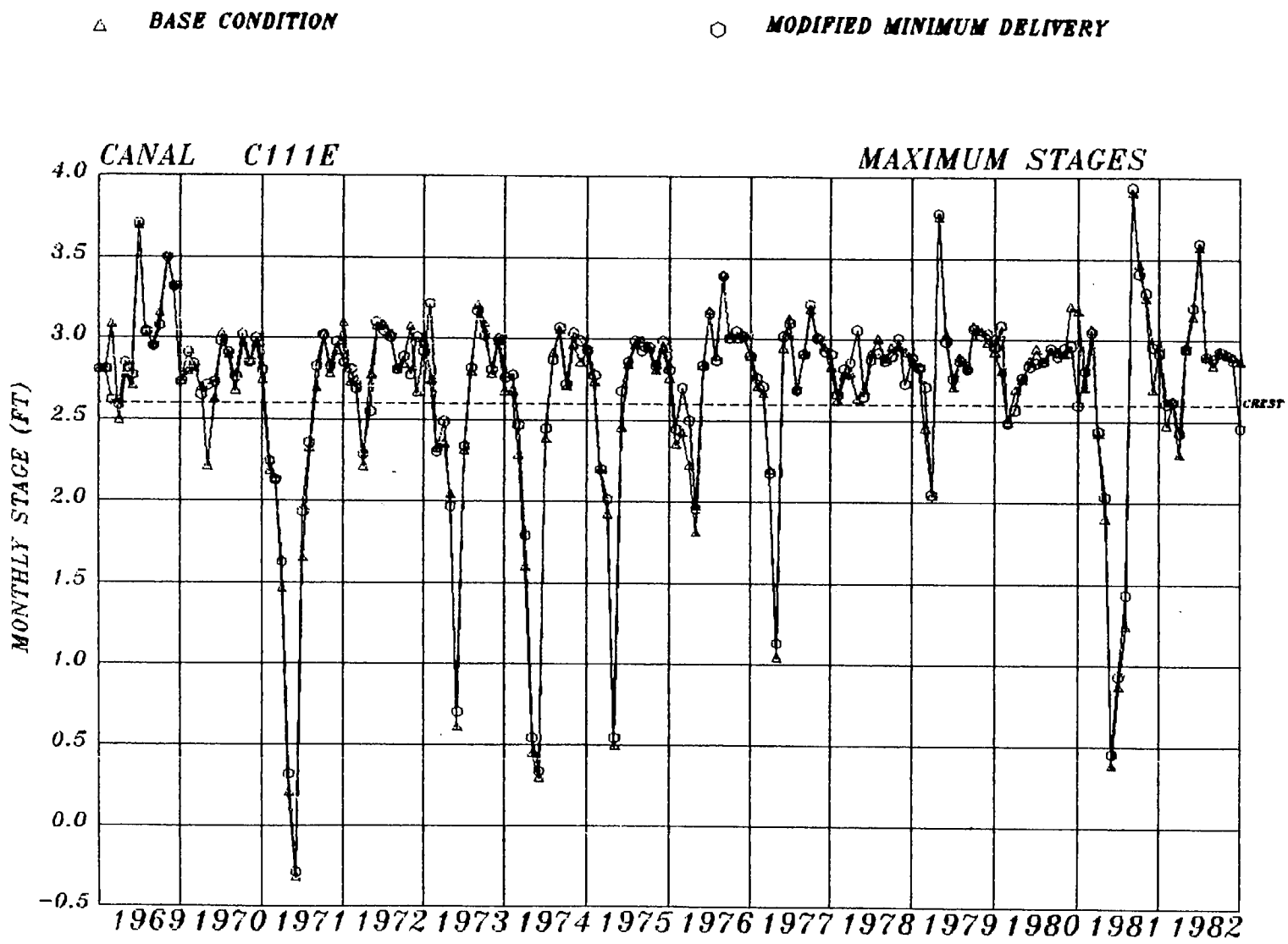
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
CANAL C111E
MAXIMUM STAGES
MONTHLY STAGE (FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-34

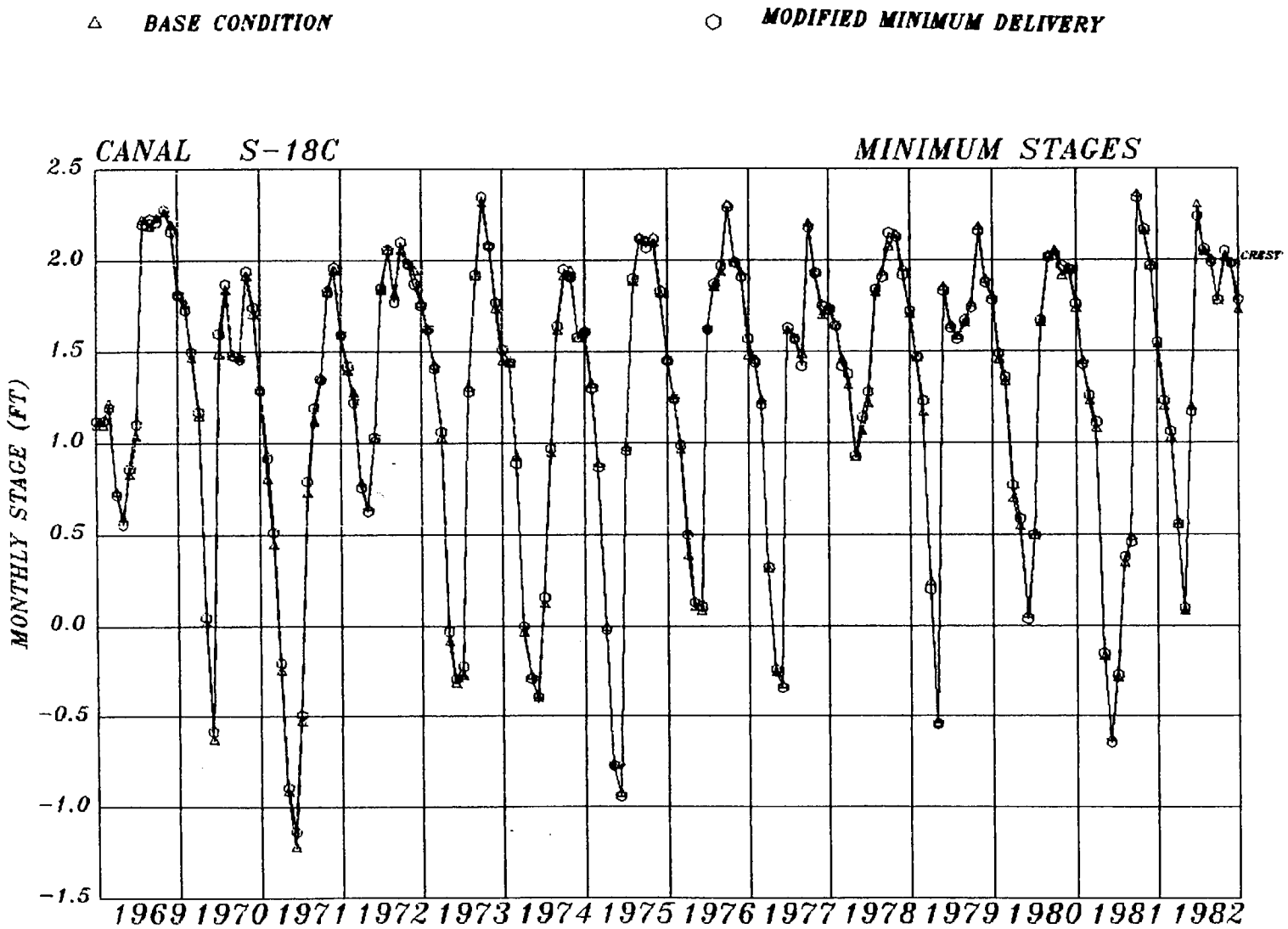
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
CANAL S-18C
MINIMUM STAGES
MONTHLY STAGE (FT)
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-35

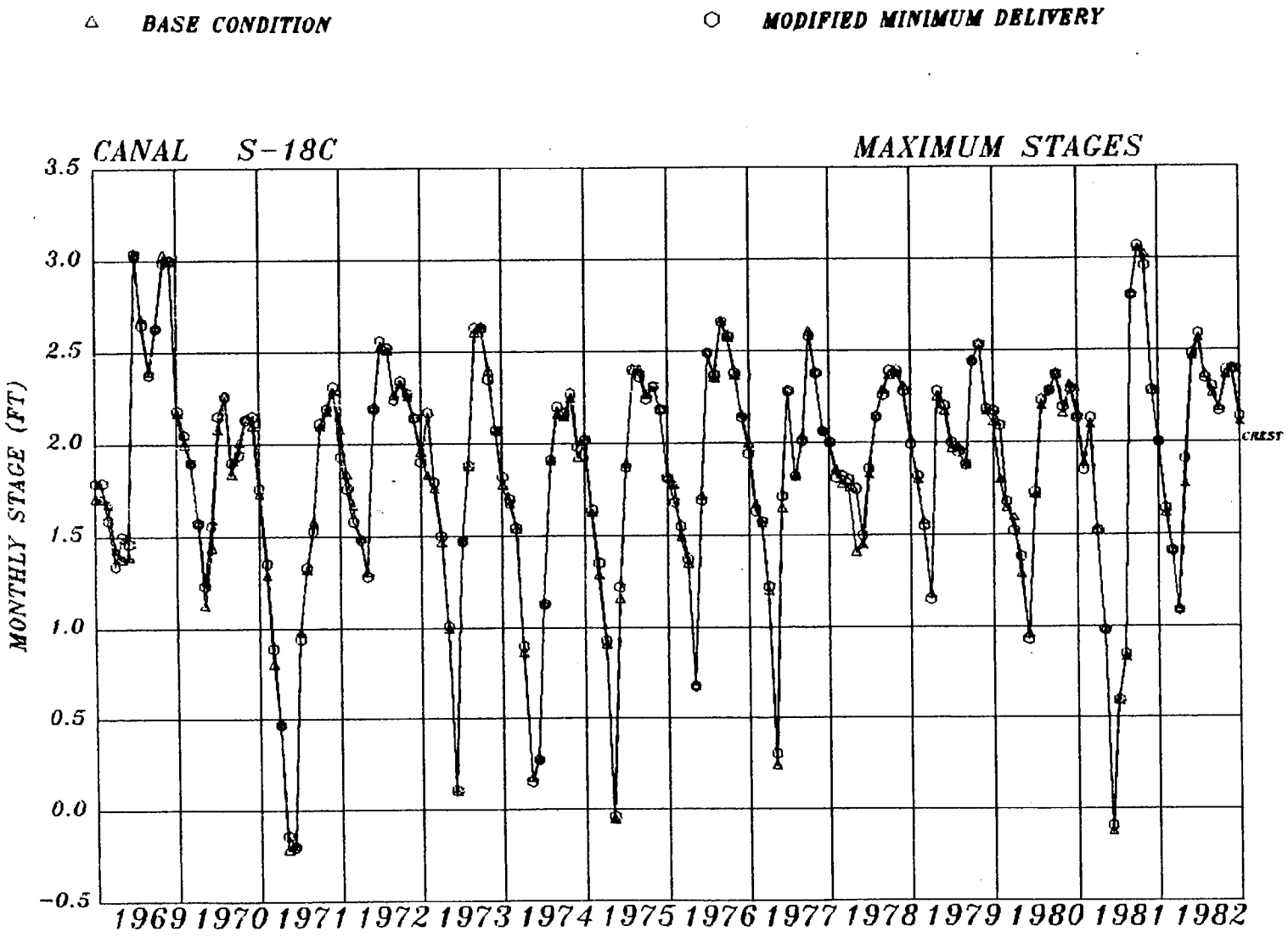
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
CANAL S-18C
MAXIMUM STAGES
MONTHLY STAGE (FT)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-36

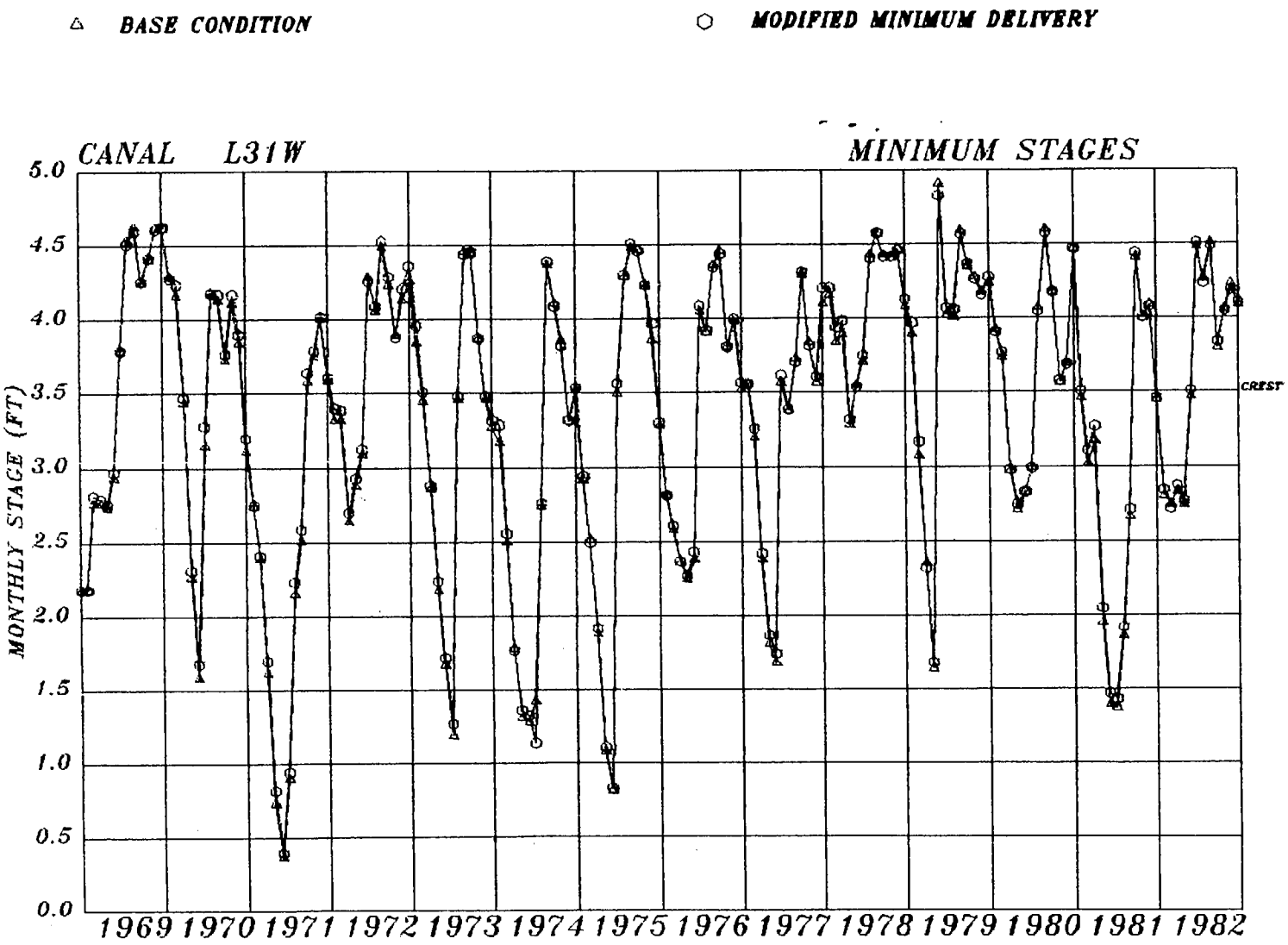
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L31W
MINIMUM STAGES
MONTHLY STAGE (FT)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-37

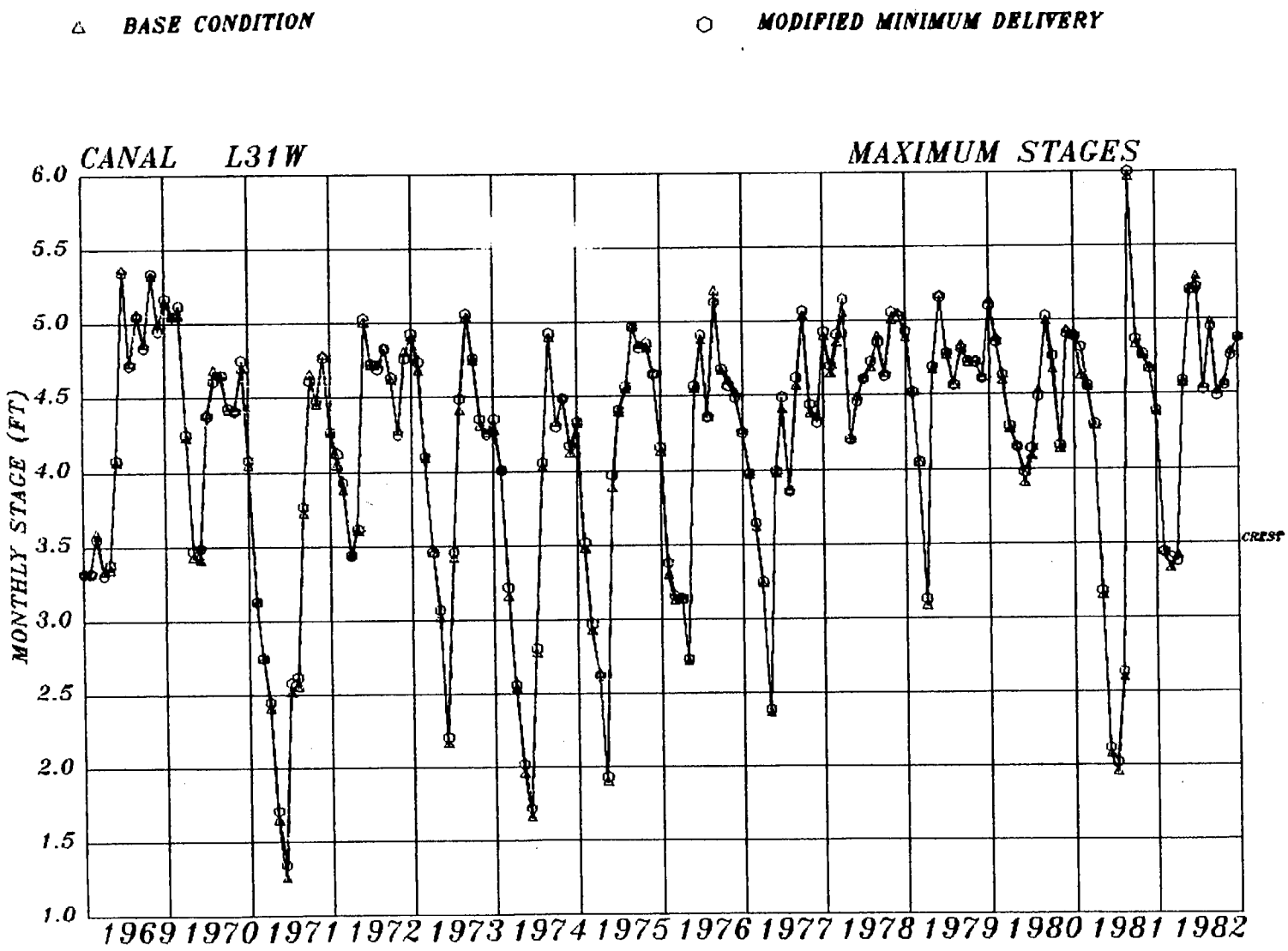
BASE CONDITION
MODIFIED MINIMUM DELIVERY
CANAL L31W
MAXIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-38

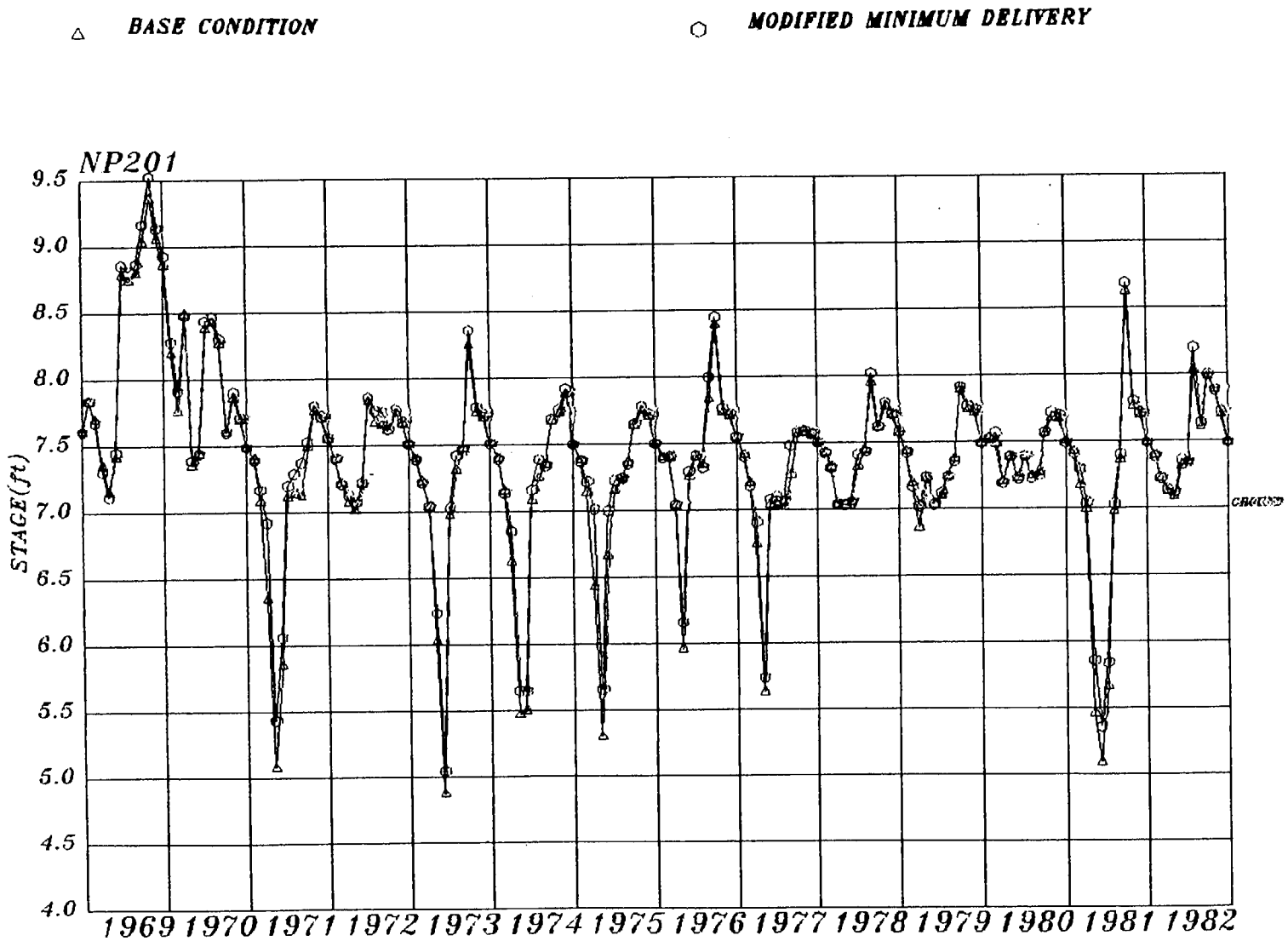
BASE CONDITION
MODIFIED MINIMUM DELIVERY
NP201
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-39

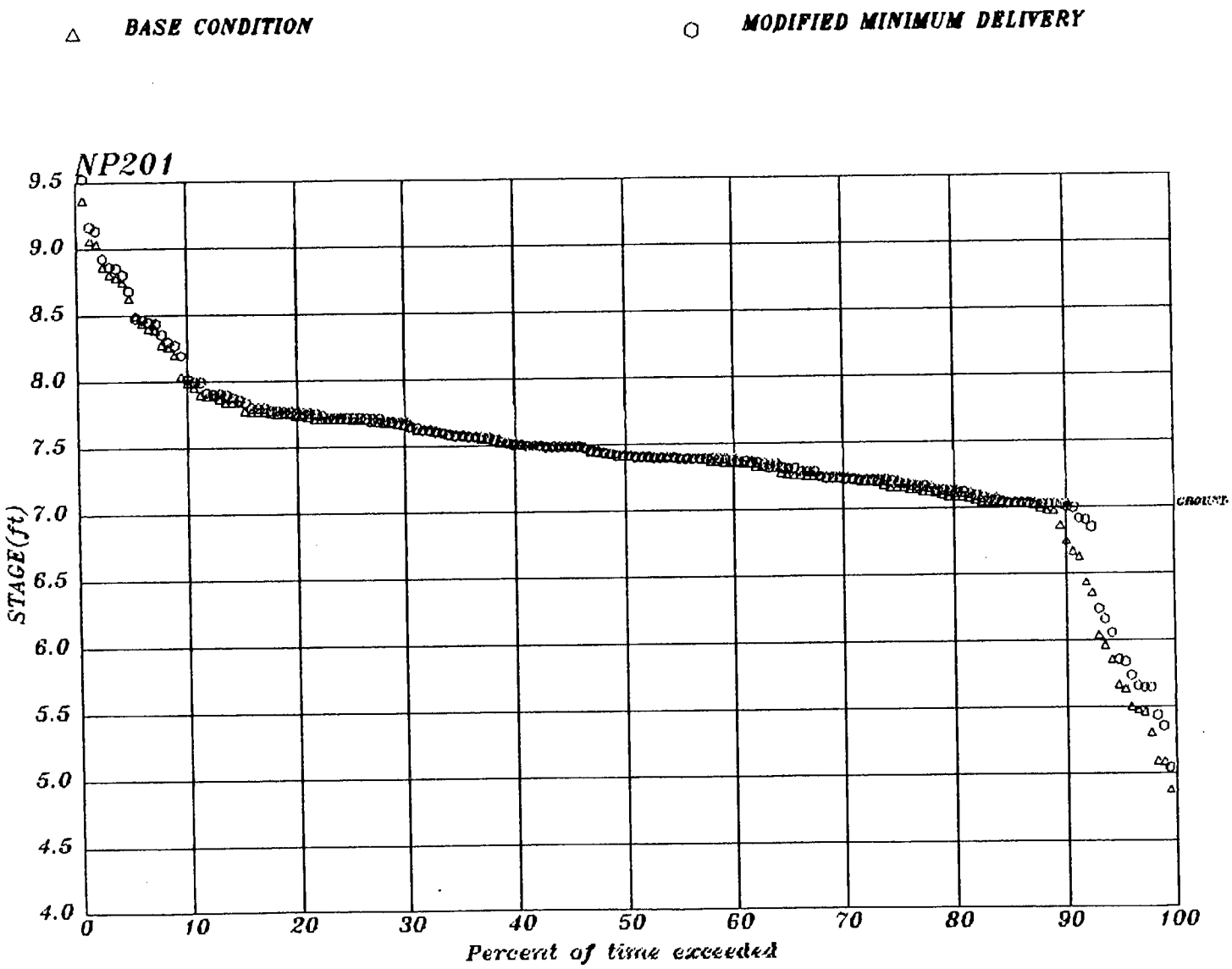
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NP201
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-40

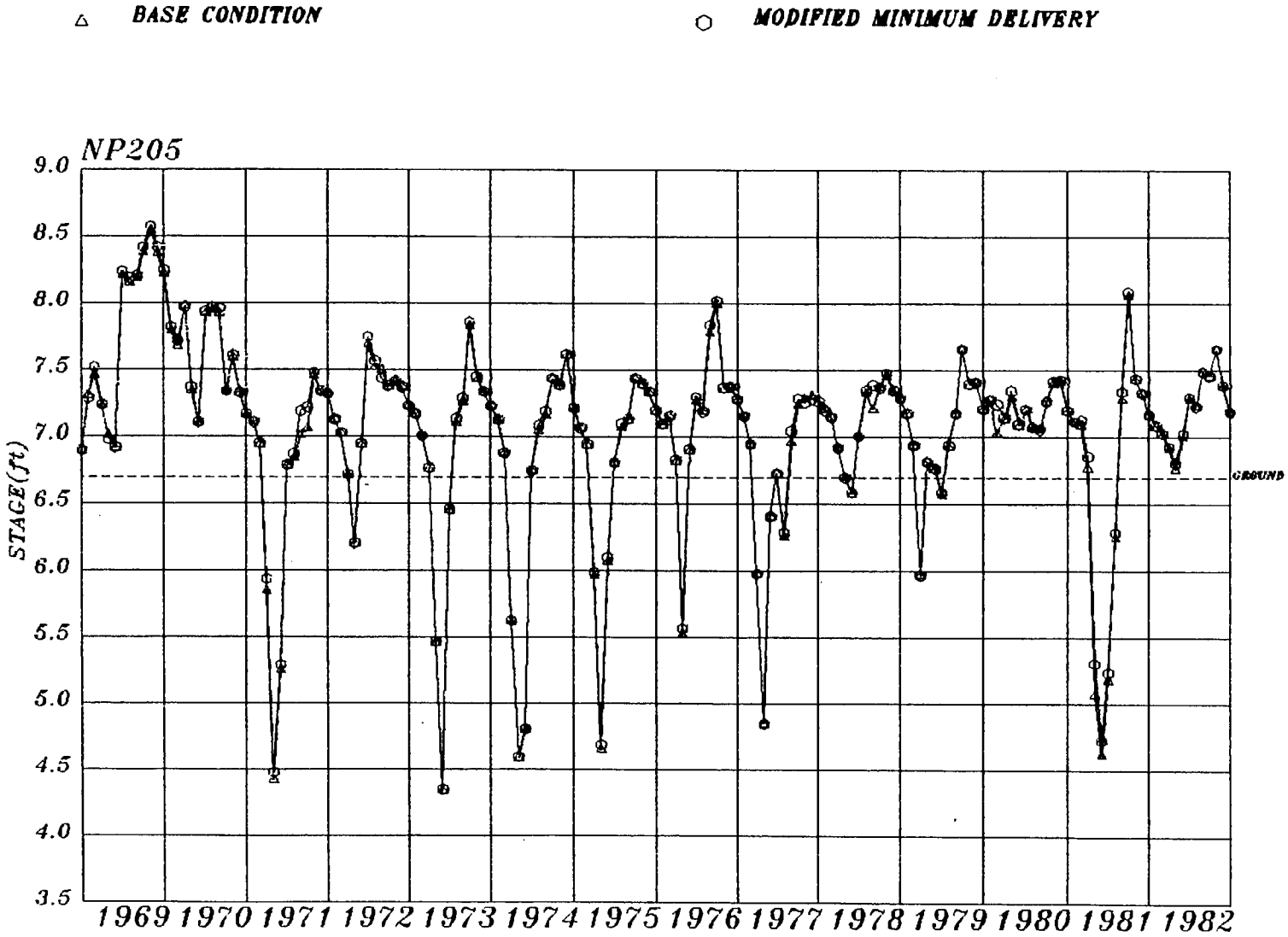
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NP205
STAGE(ft)
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-41

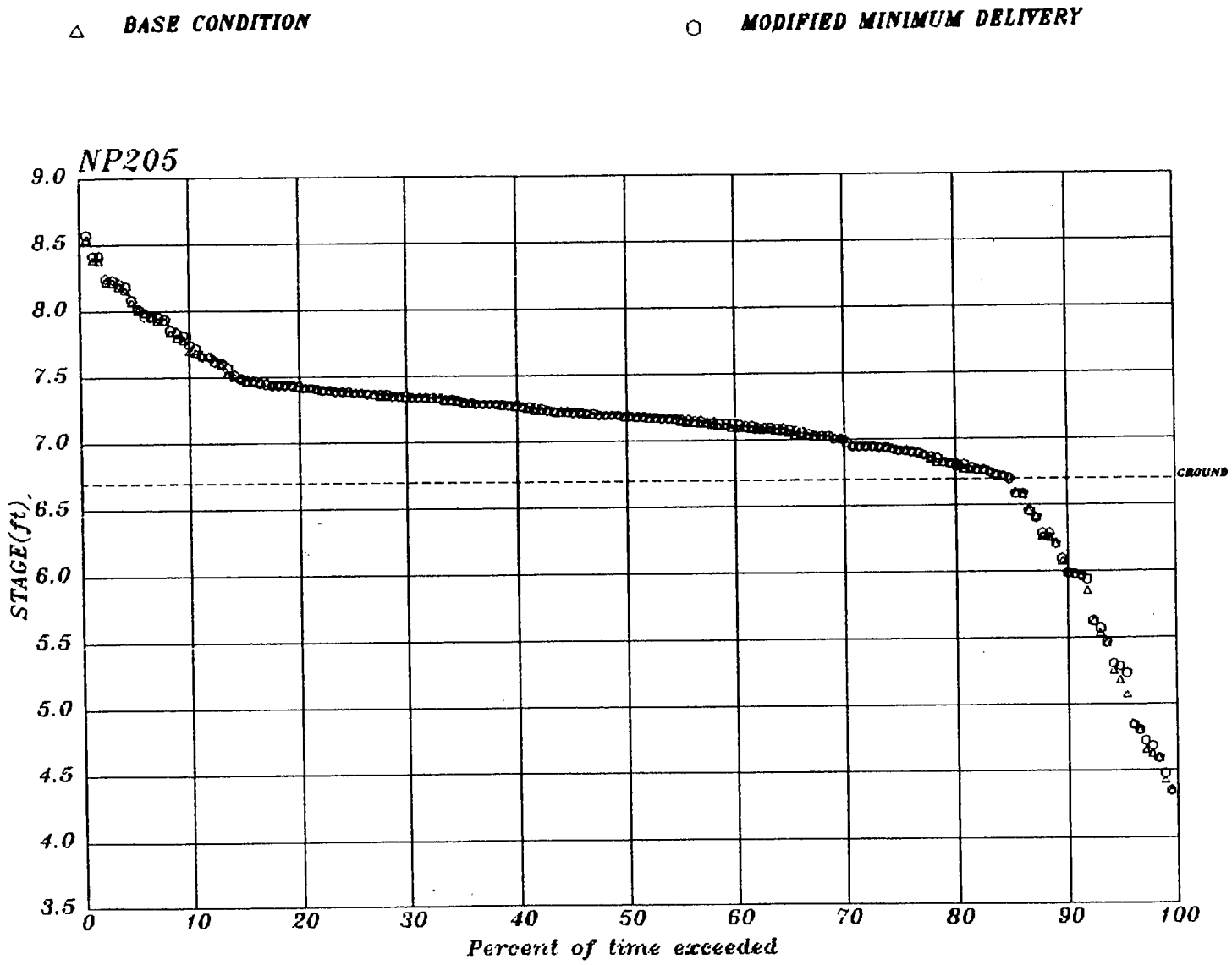
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NP205
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-42

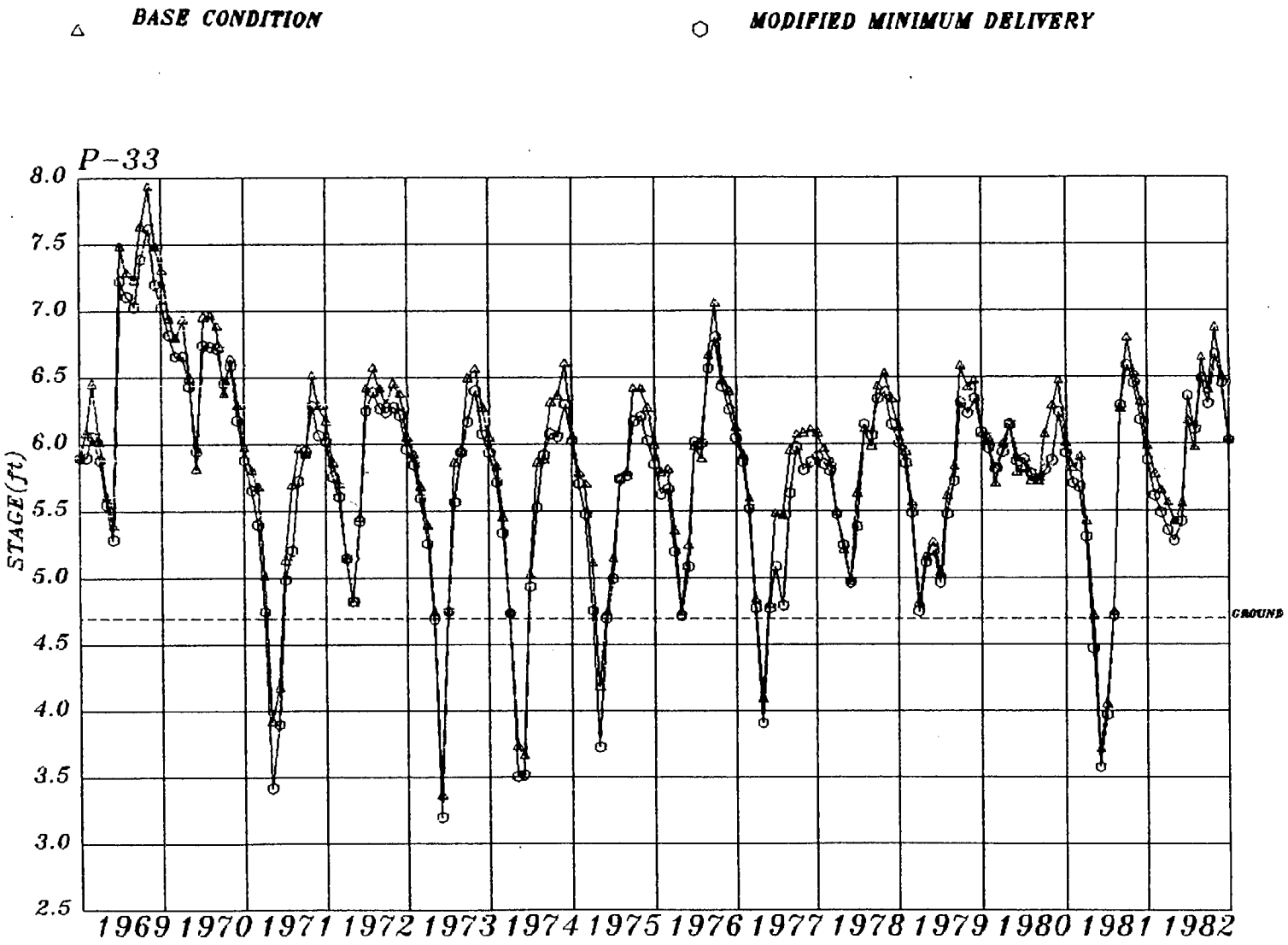
BASE CONDITION
MODIFIED MINIMUM DELIVERY
P-33
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-43

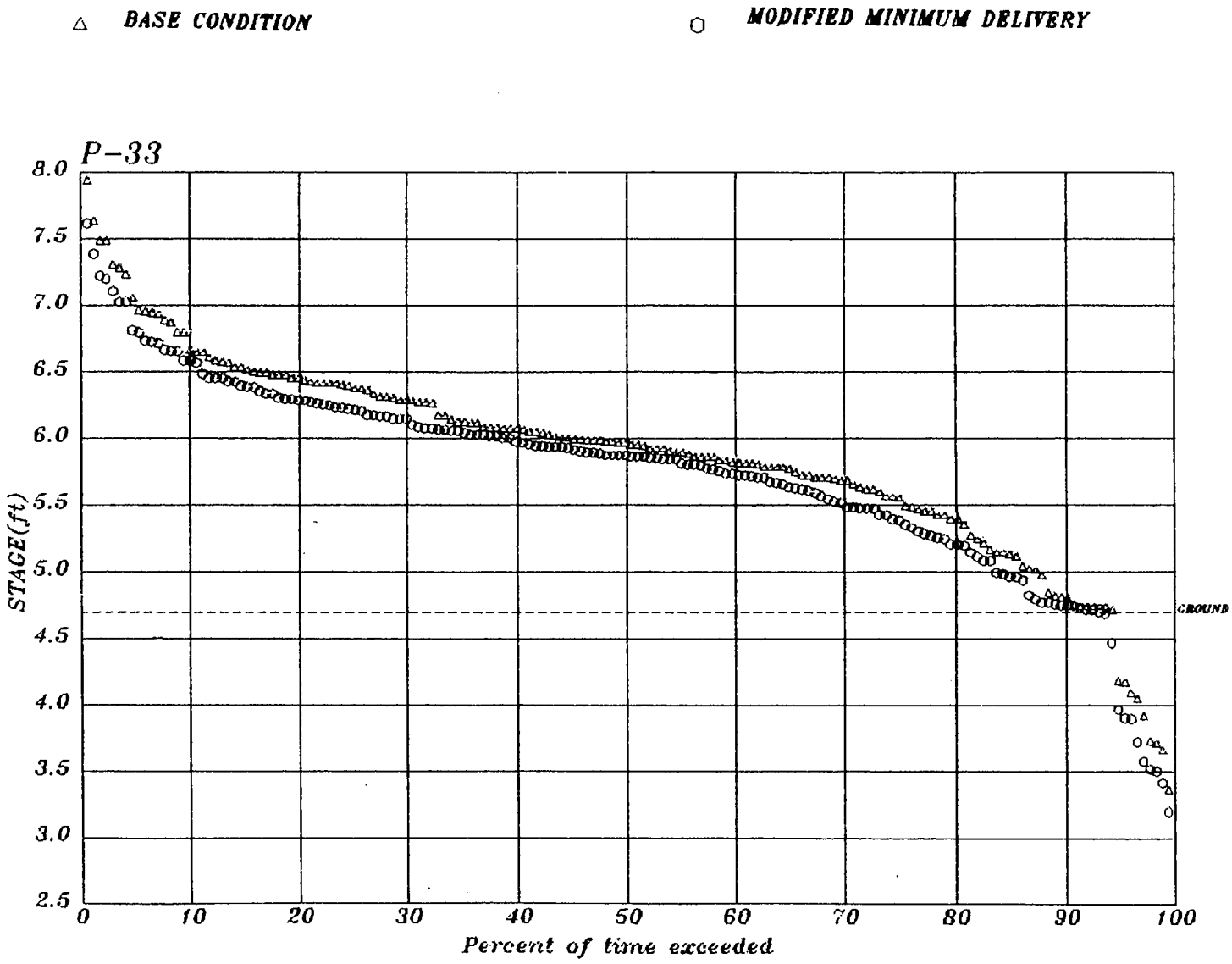
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
P-33
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-44

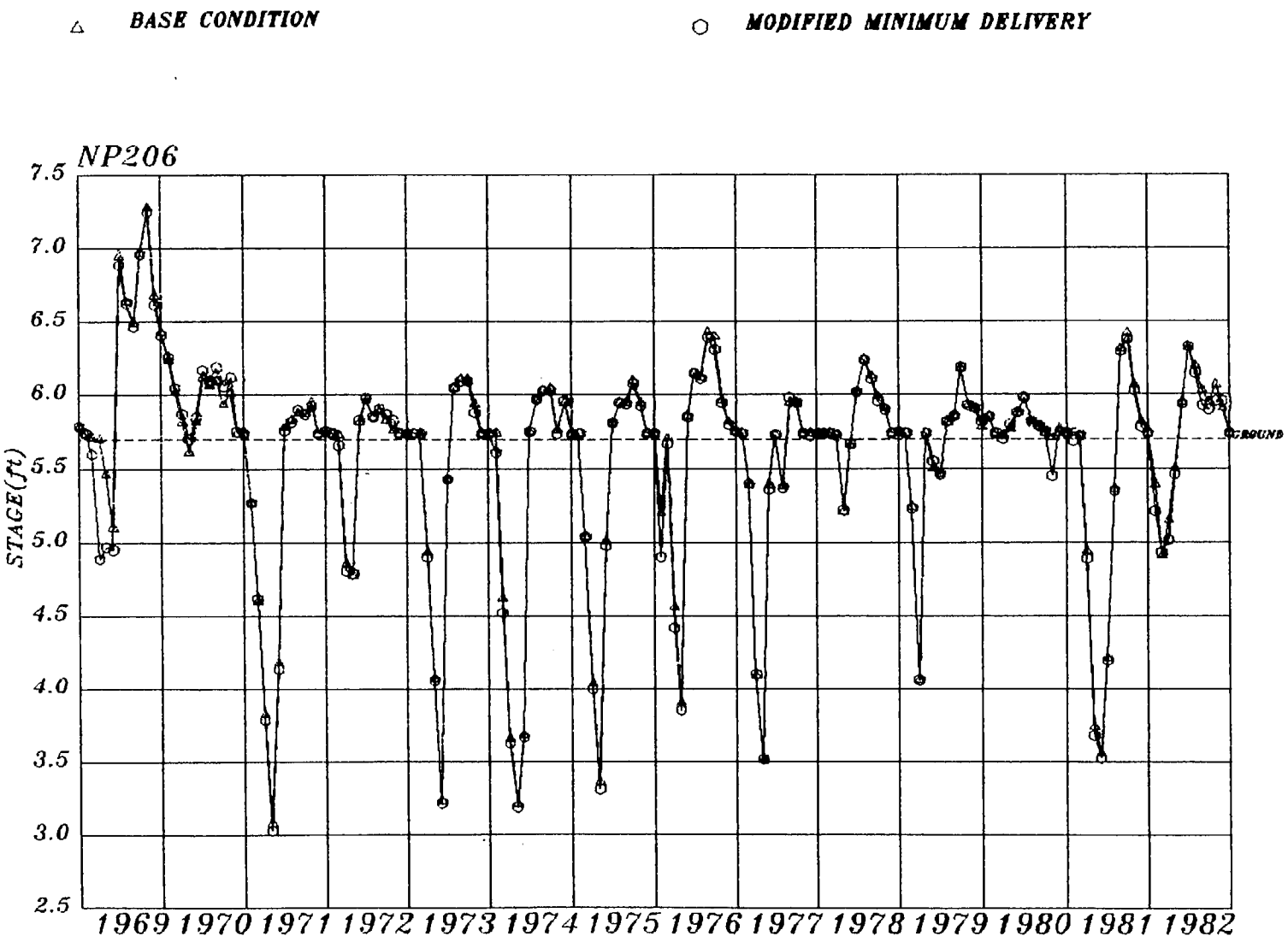
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NP206
STAGE(ft)
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-45

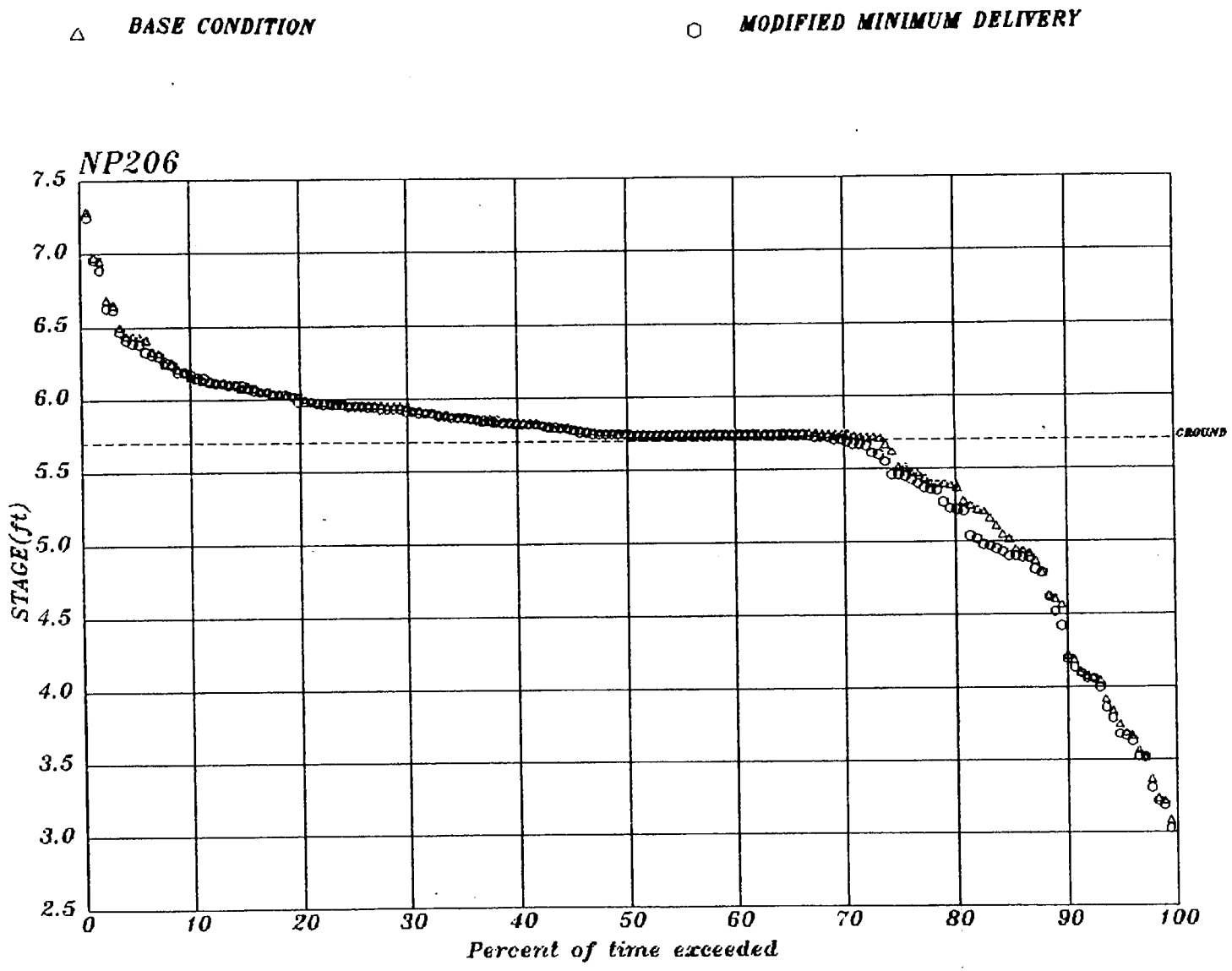
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NP206
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-46

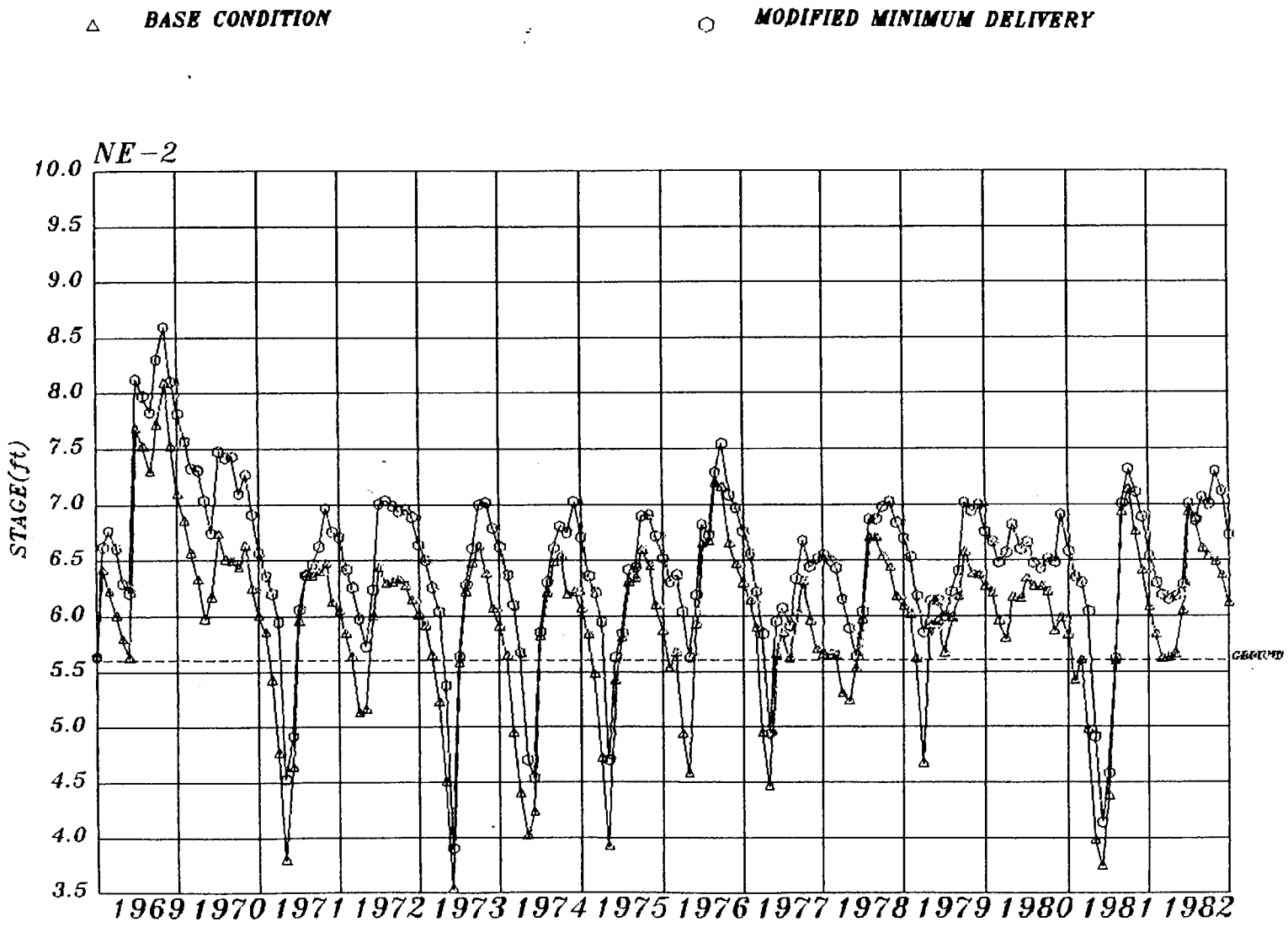
BASE CONDITION
MODIFIED MINIMUM DELIVERY
NE-2
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-47

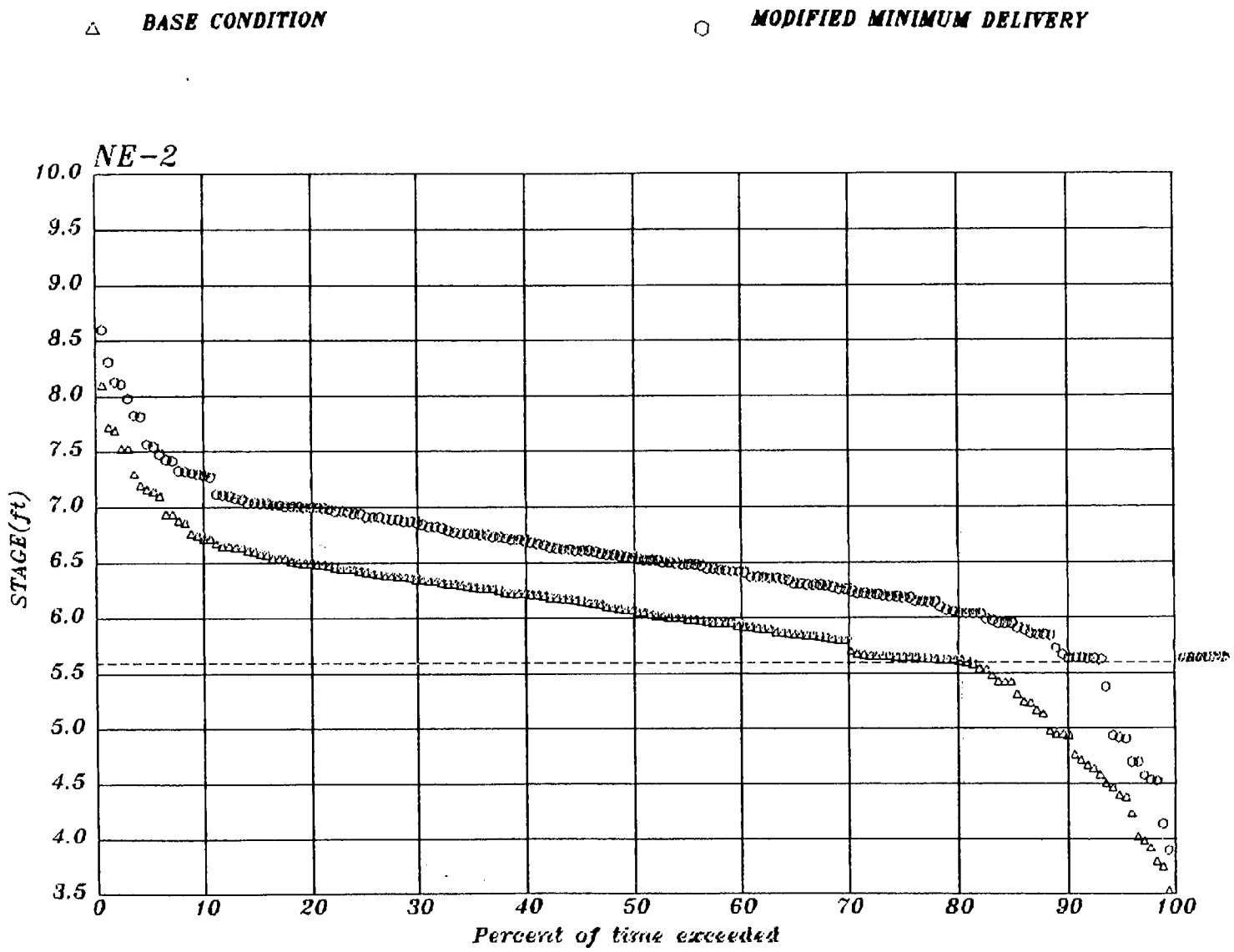
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NE-2
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-48

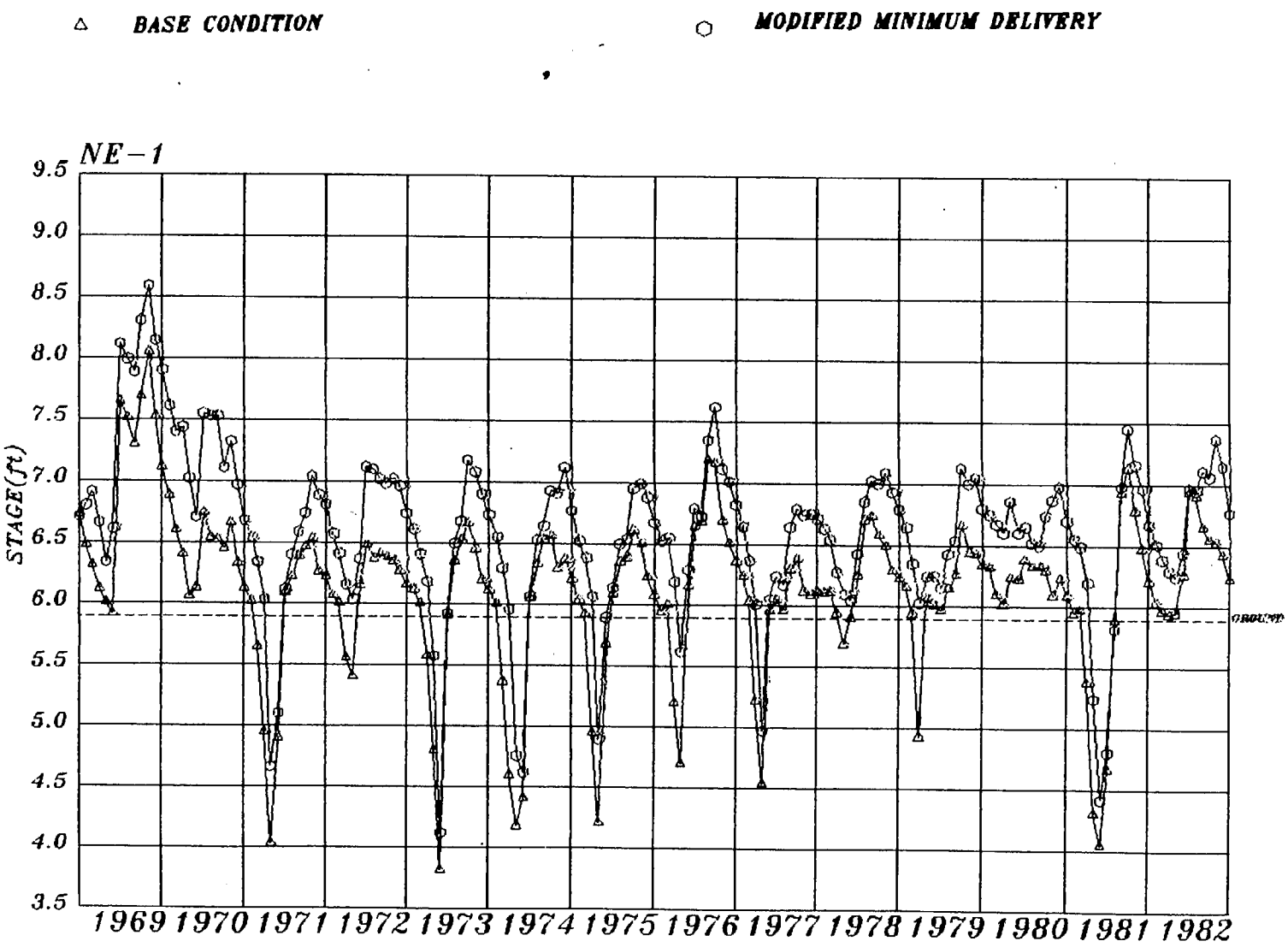
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NE-1
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-49

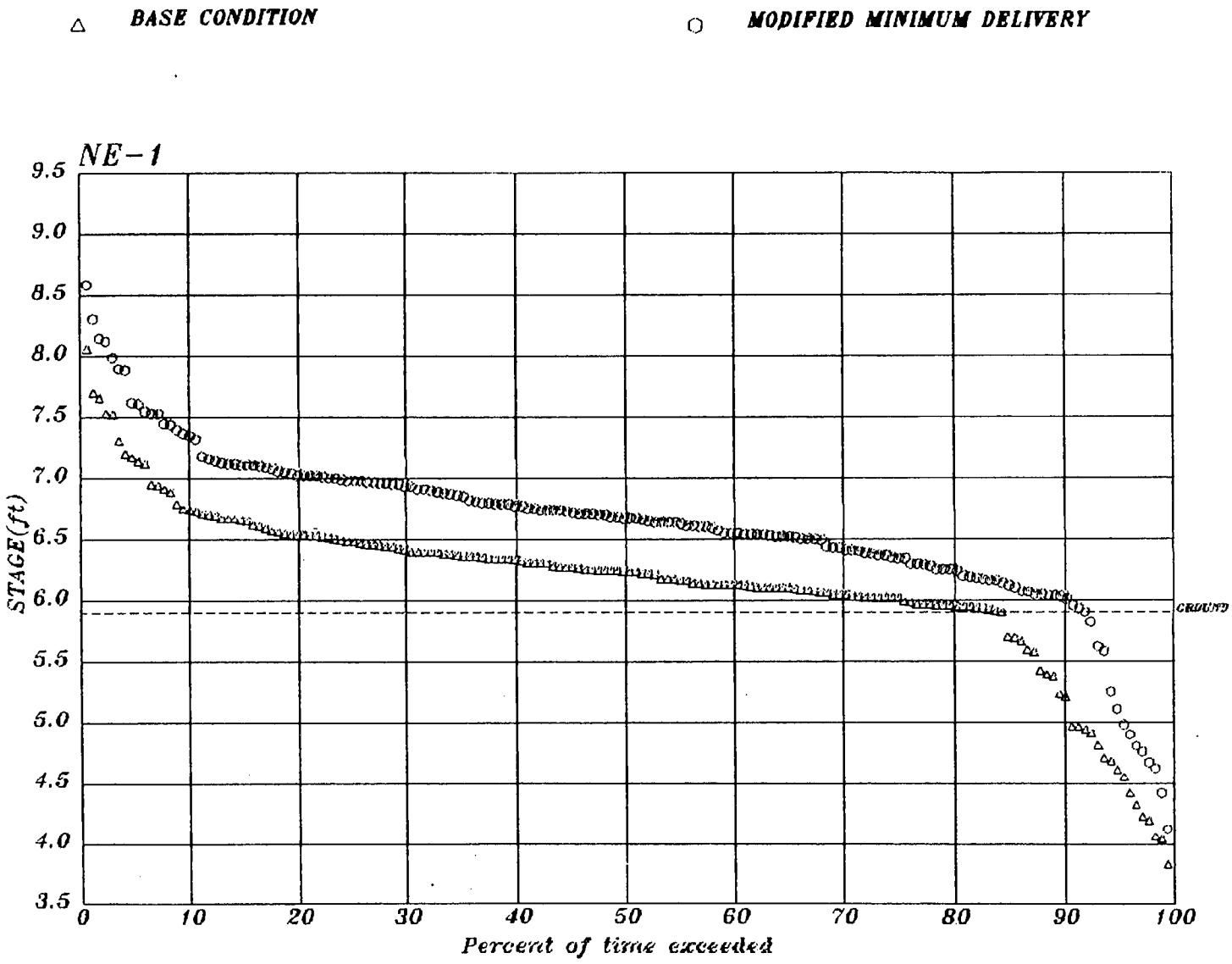
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
NE-1
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-50

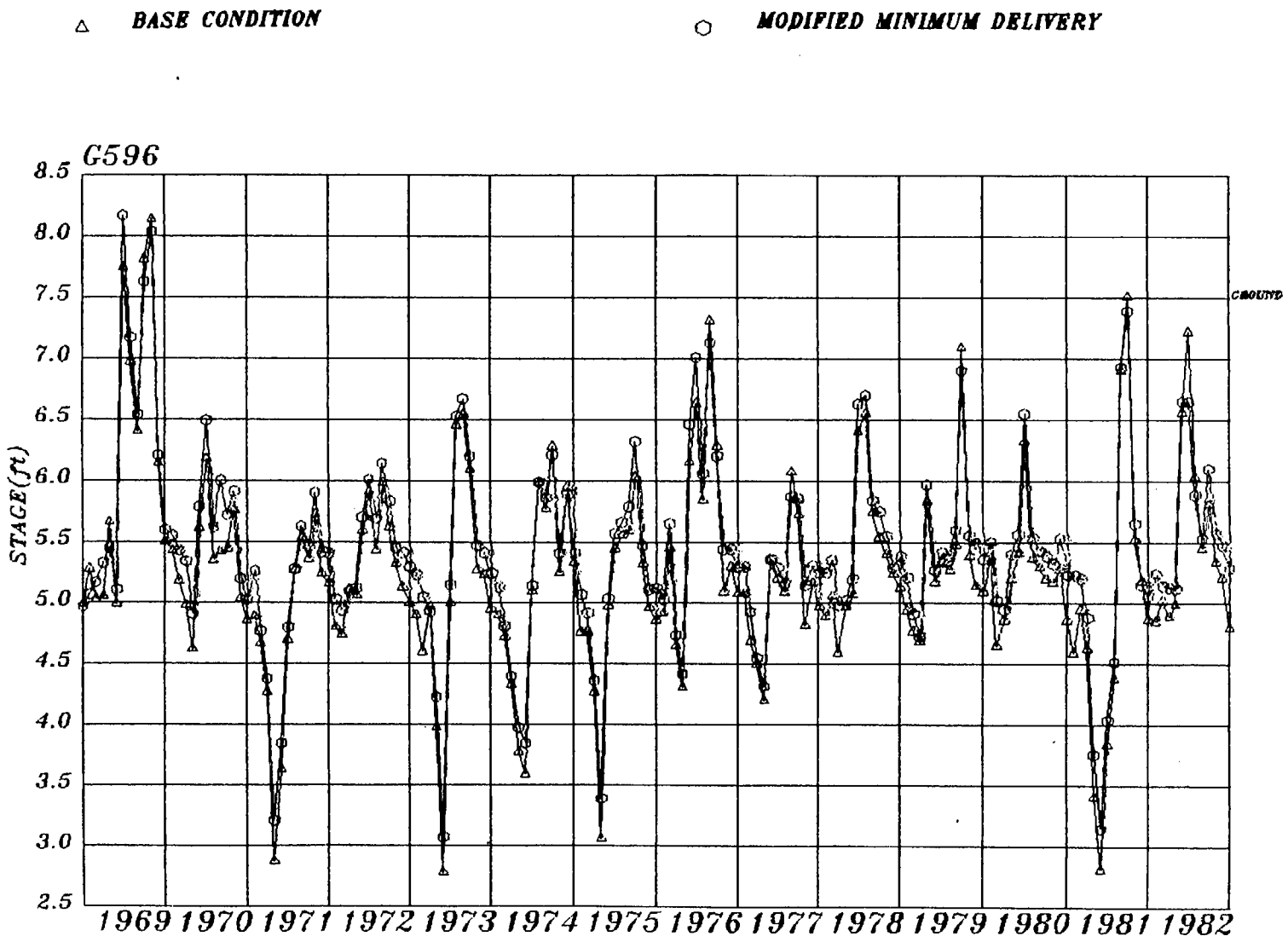
BASE CONDITION
MODIFIED MINIMUM DELIVERY
G596
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-51

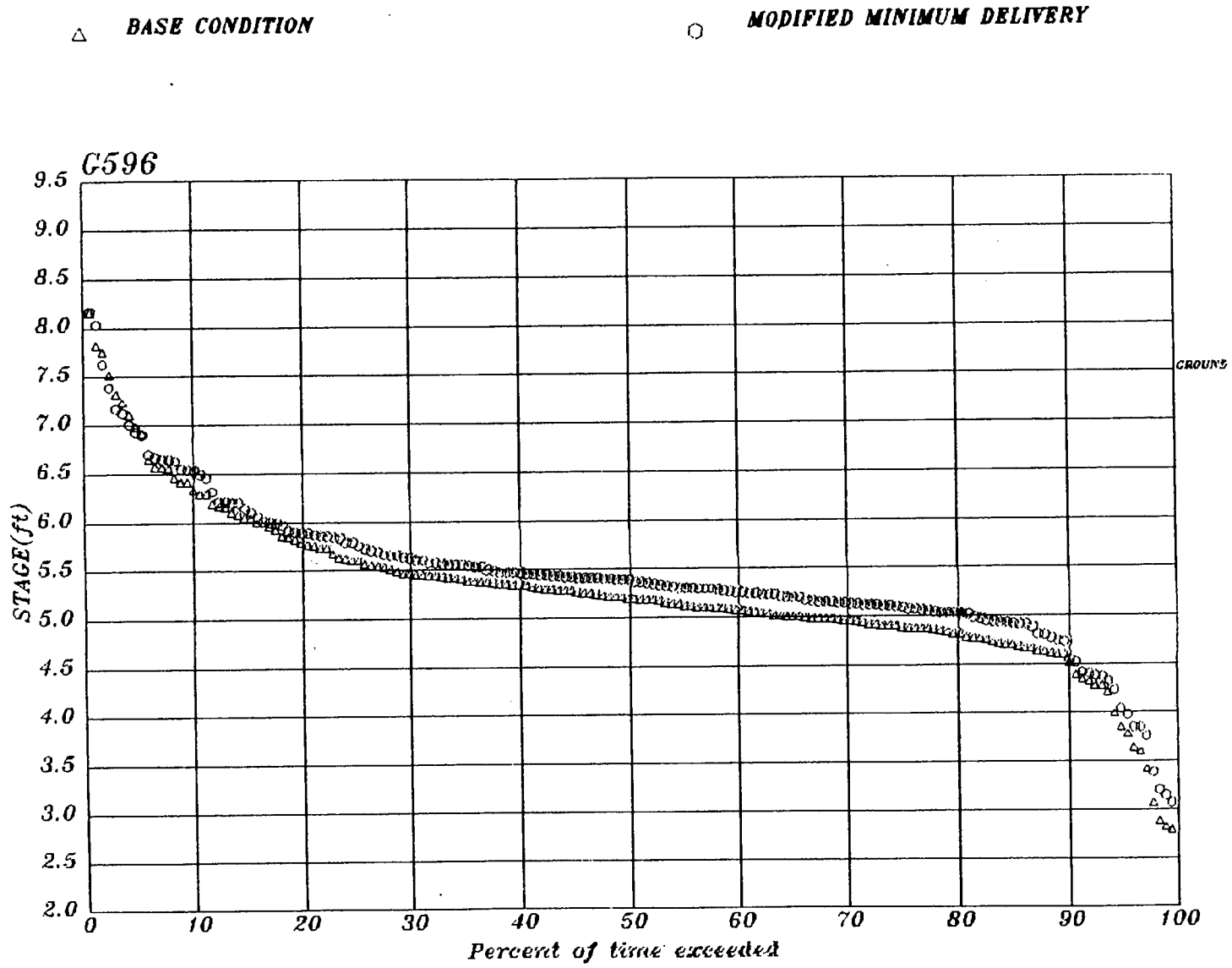
△ BASE CONDITION
○ MODIFIED MINIMUM DELIVERY
G596
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-52

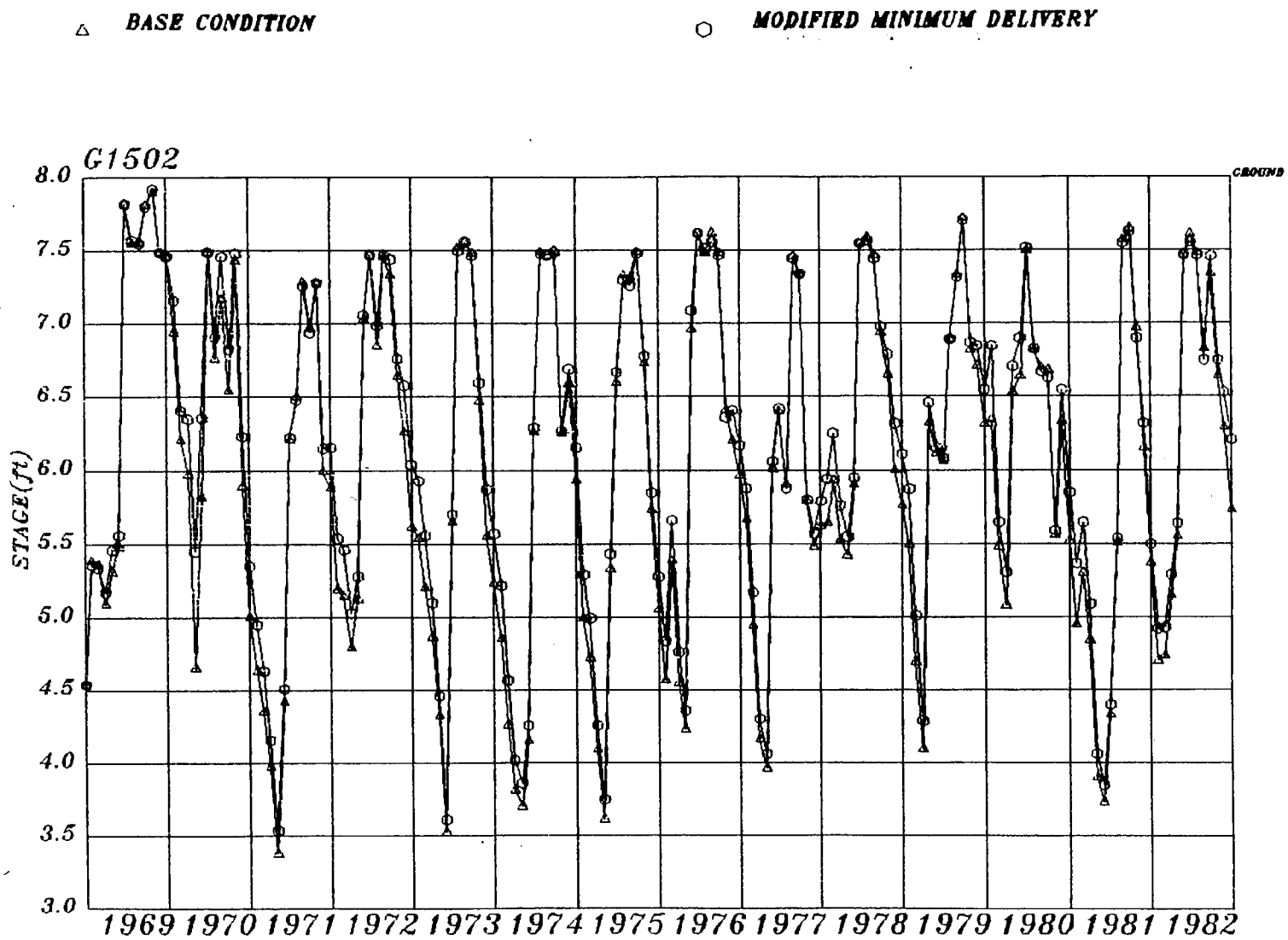
BASE CONDITION
MODIFIED MINIMUM DELIVERY
G1502
GROUND
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-53

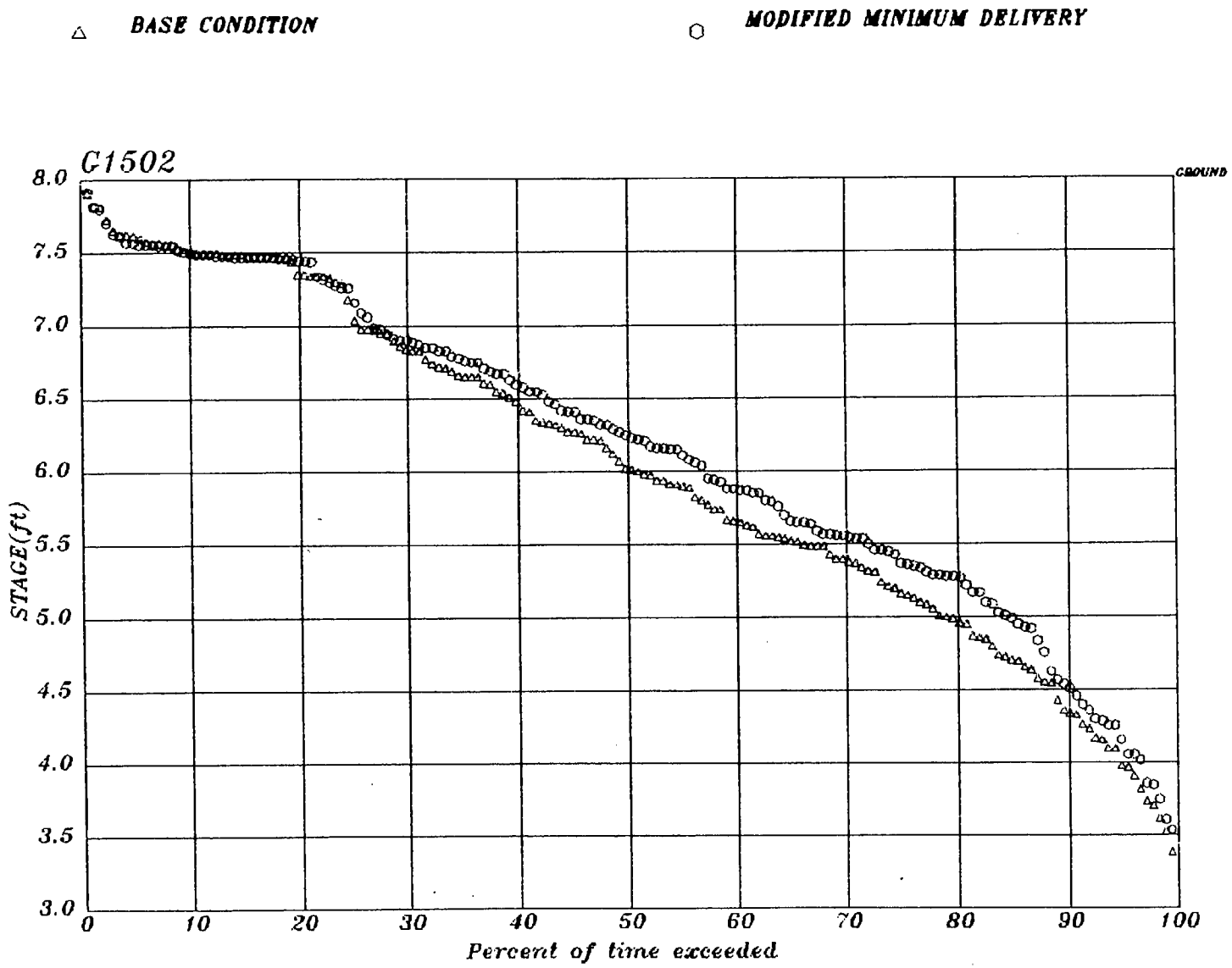
BASE CONDITION
MODIFIED MINIMUM DELIVERY
G1502
GROUND
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-54

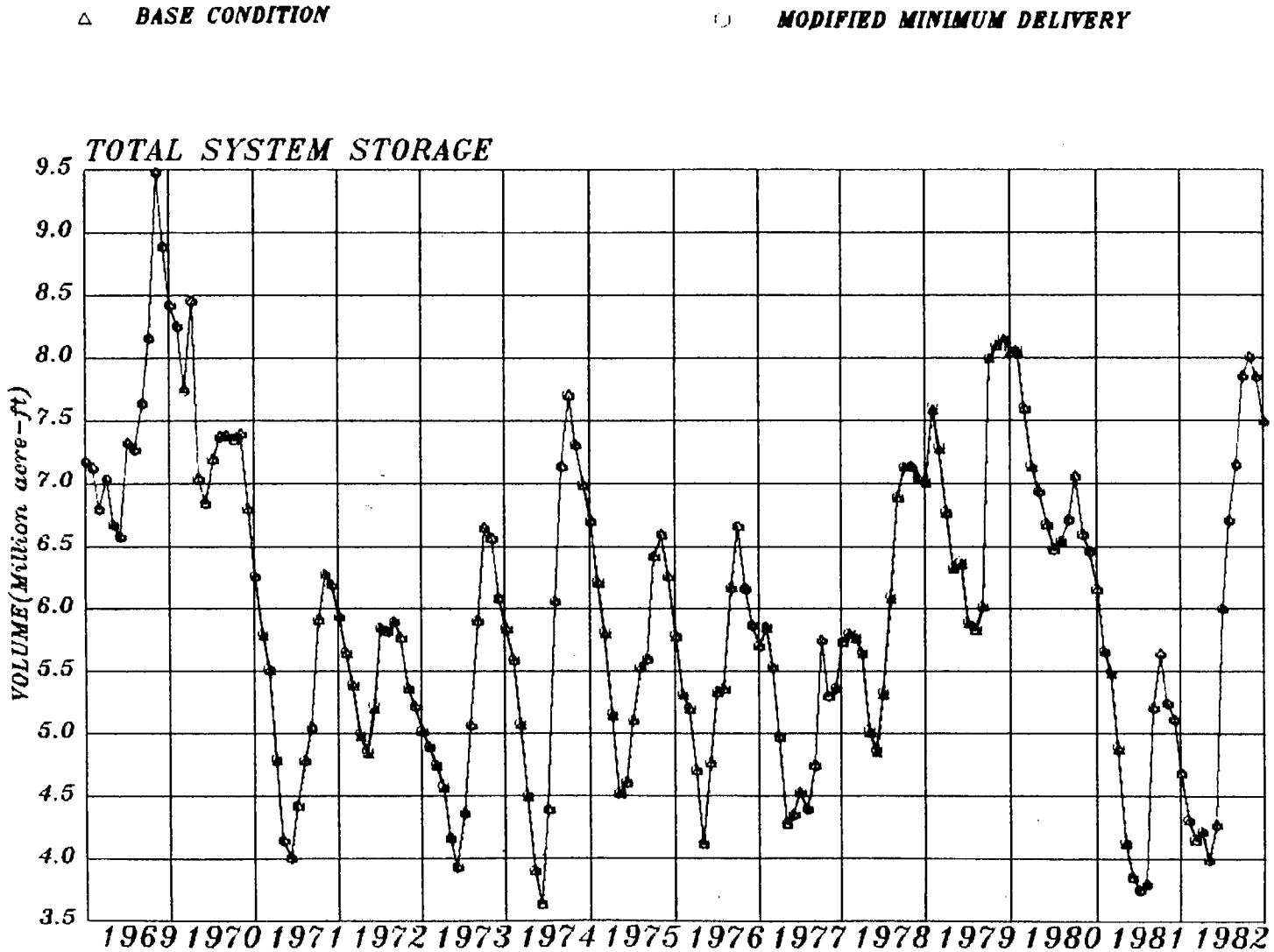
BASE CONDITION
MODIFIED MINIMUM DELIVERY
TOTAL SYSTEM STORAGE
VOLUME(Million acre-ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-55

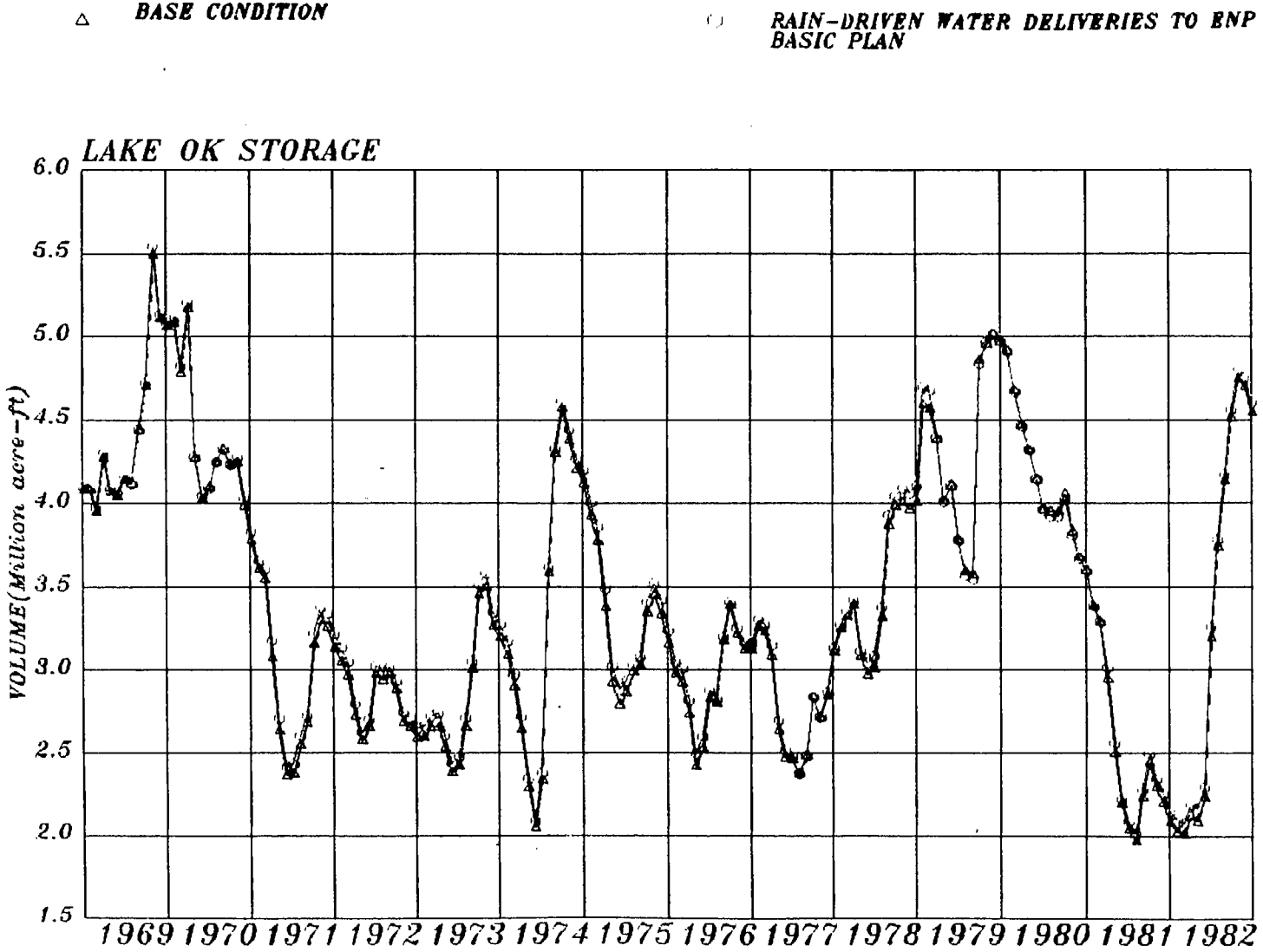
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
LAKE OK STORAGE
VOLUME(Million acre-ft)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-56

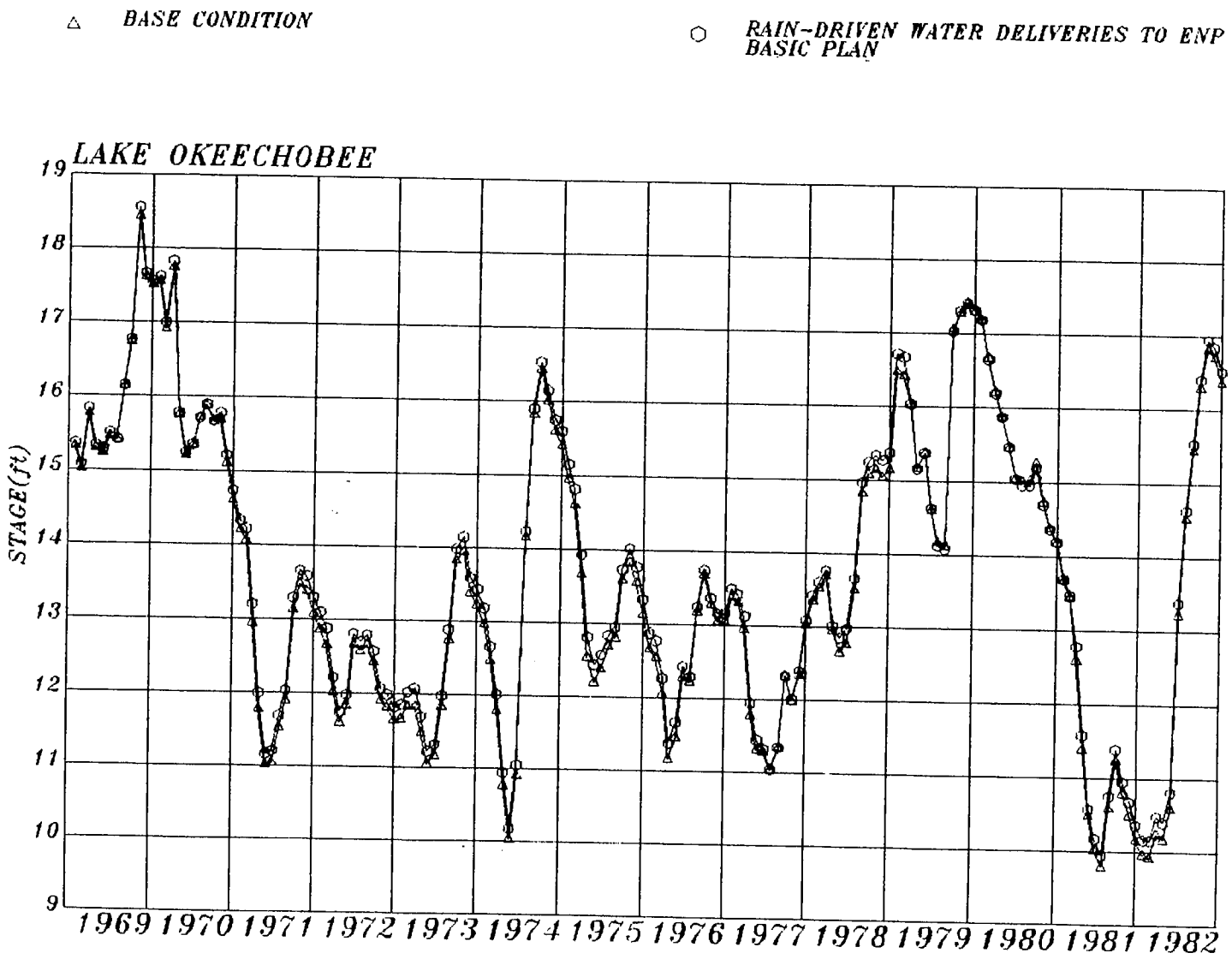
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
LAKE OKEECHOBEE
STAGE(ft)
19
18
17
16
15
14
13
12
11
10
9
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-57

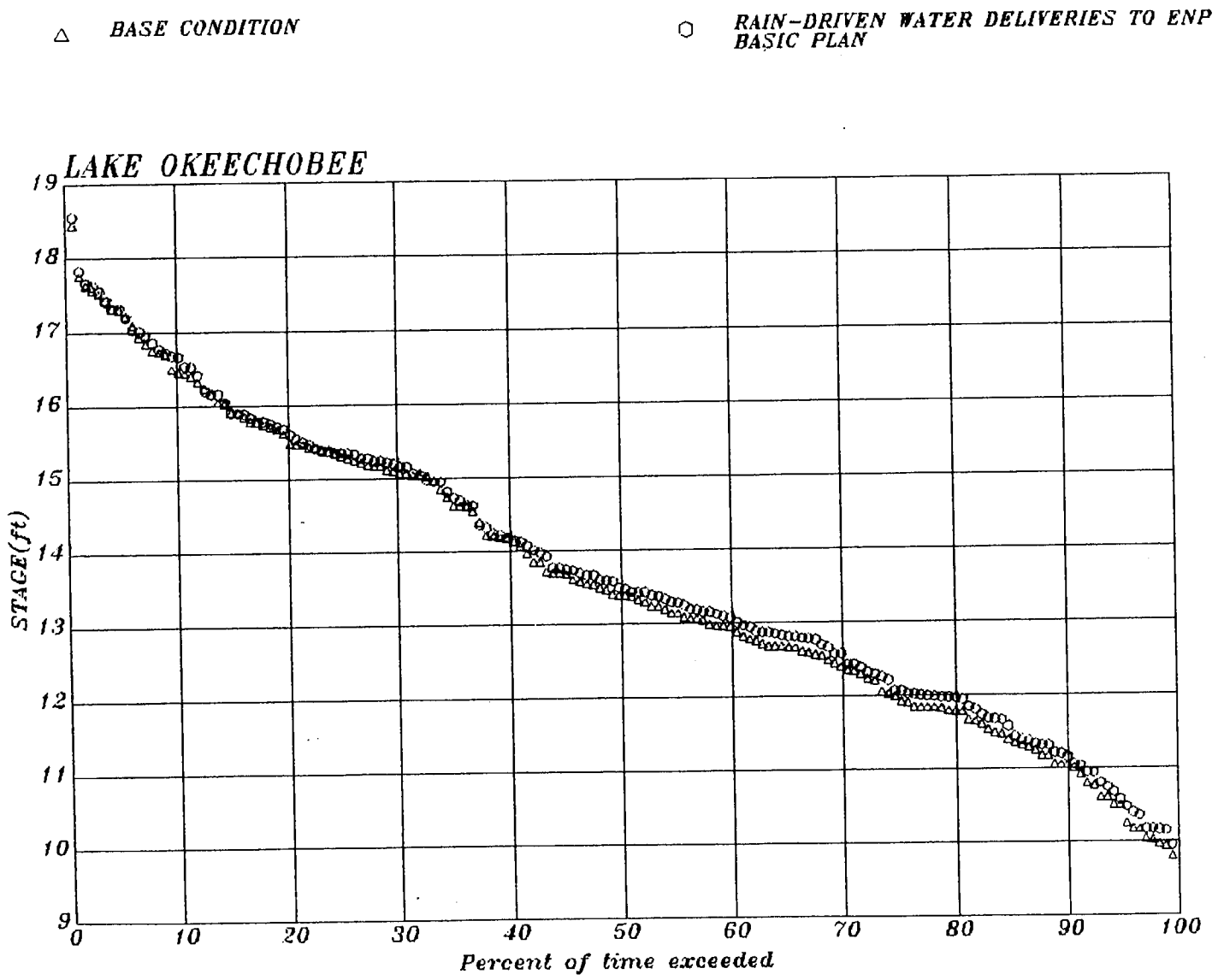
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
LAKE OKEECHOBEE
STAGE(ft)
19
18
17
16
15
14
13
12
11
10
9
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-58

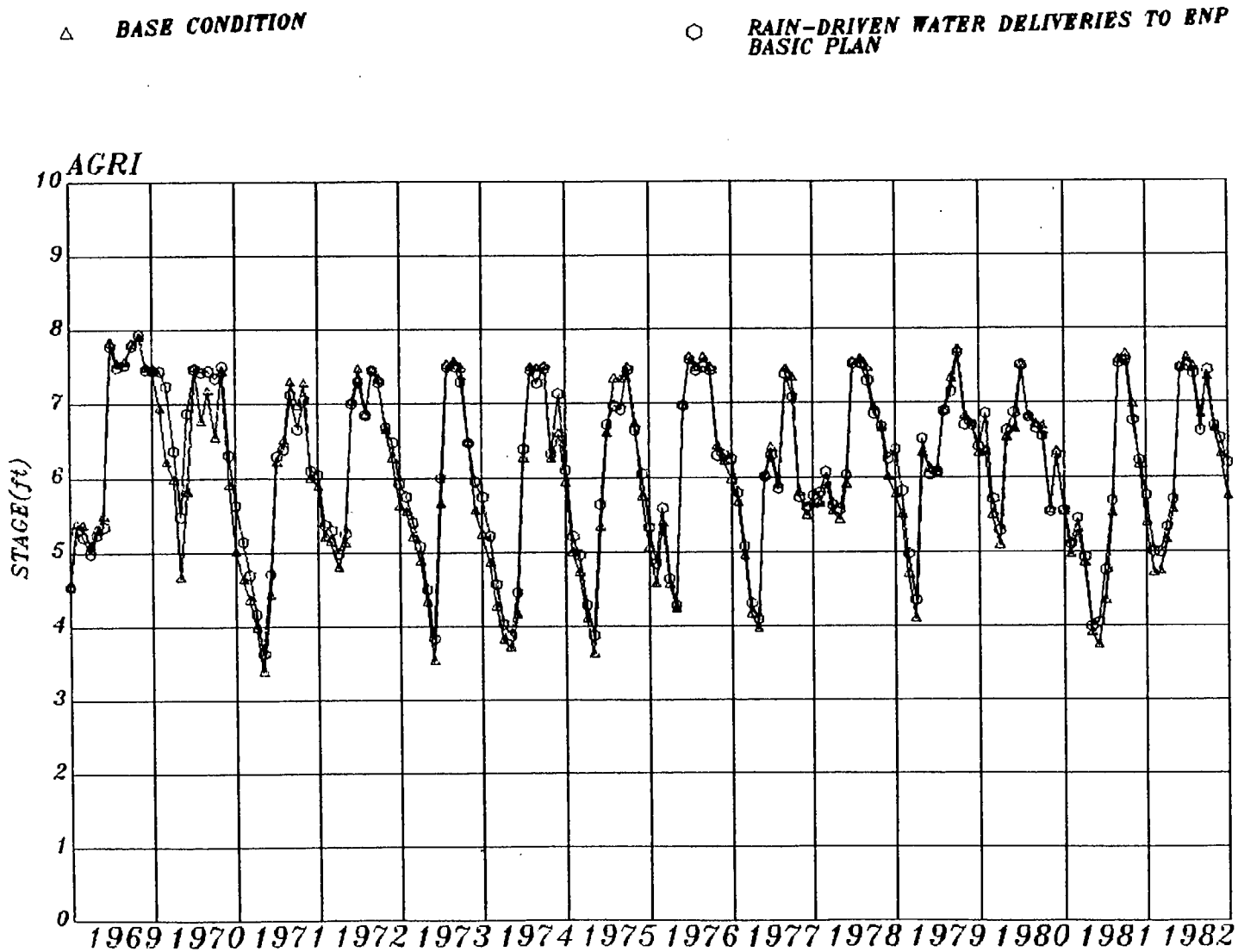
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
AGRI
STAGE(ft)
10
9
8
7
6
5
4
3
2
1
0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-59

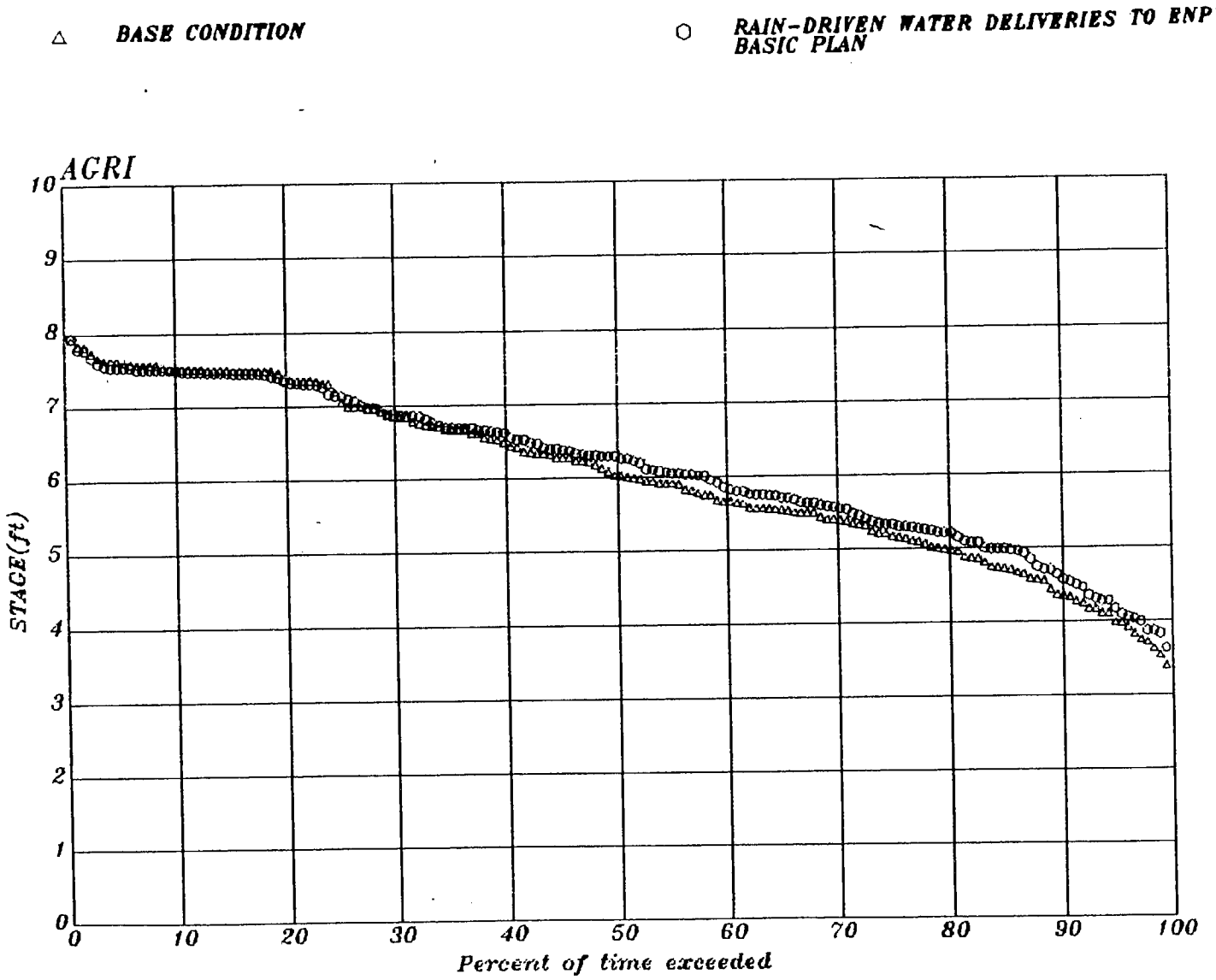
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
AGRI
STAGE(ft)
0
1
2
3
4
5
6
7
8
9
10
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-60

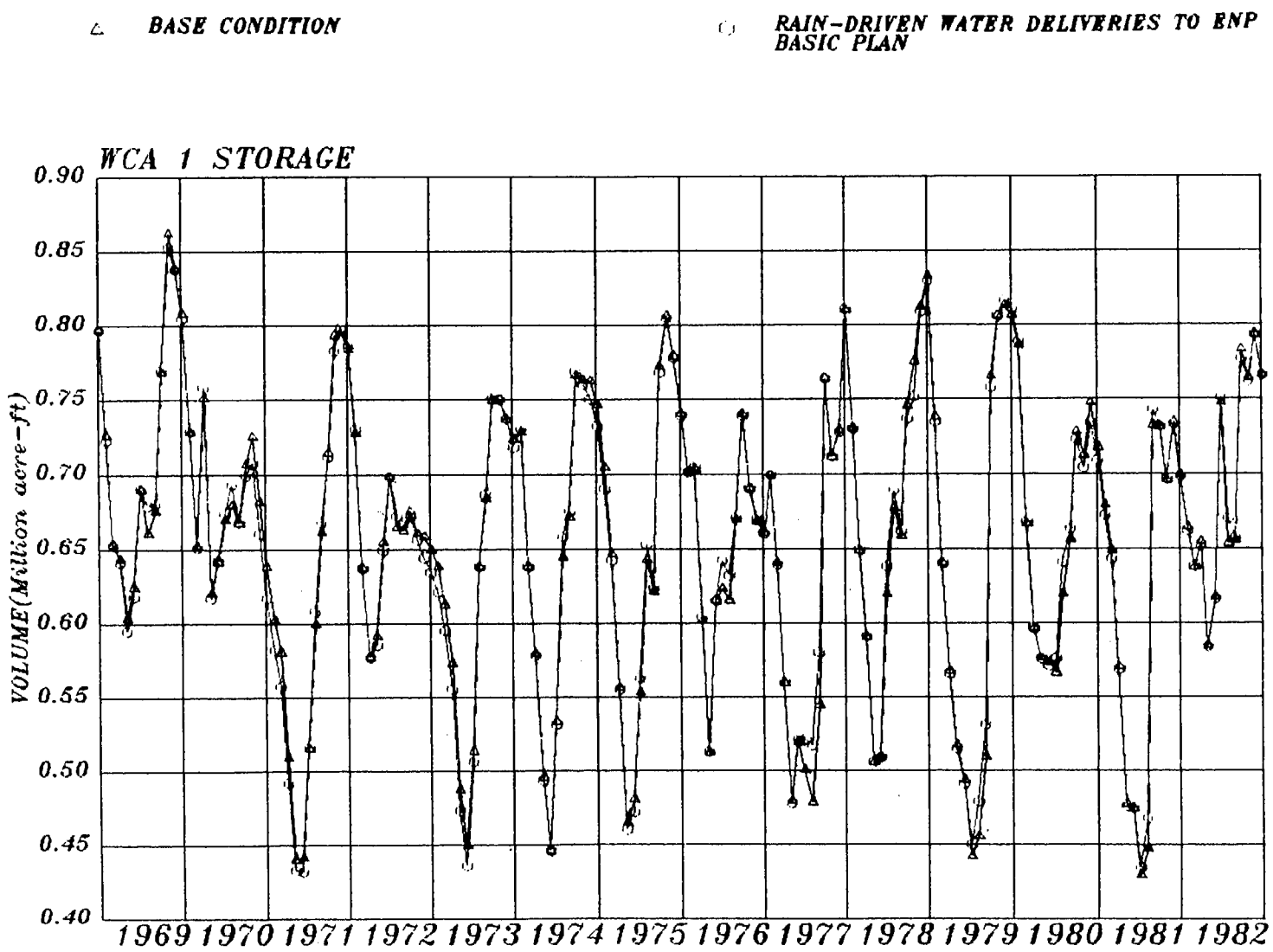
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
WCA 1 STORAGE
VOLUME(Million acre-ft)
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-61

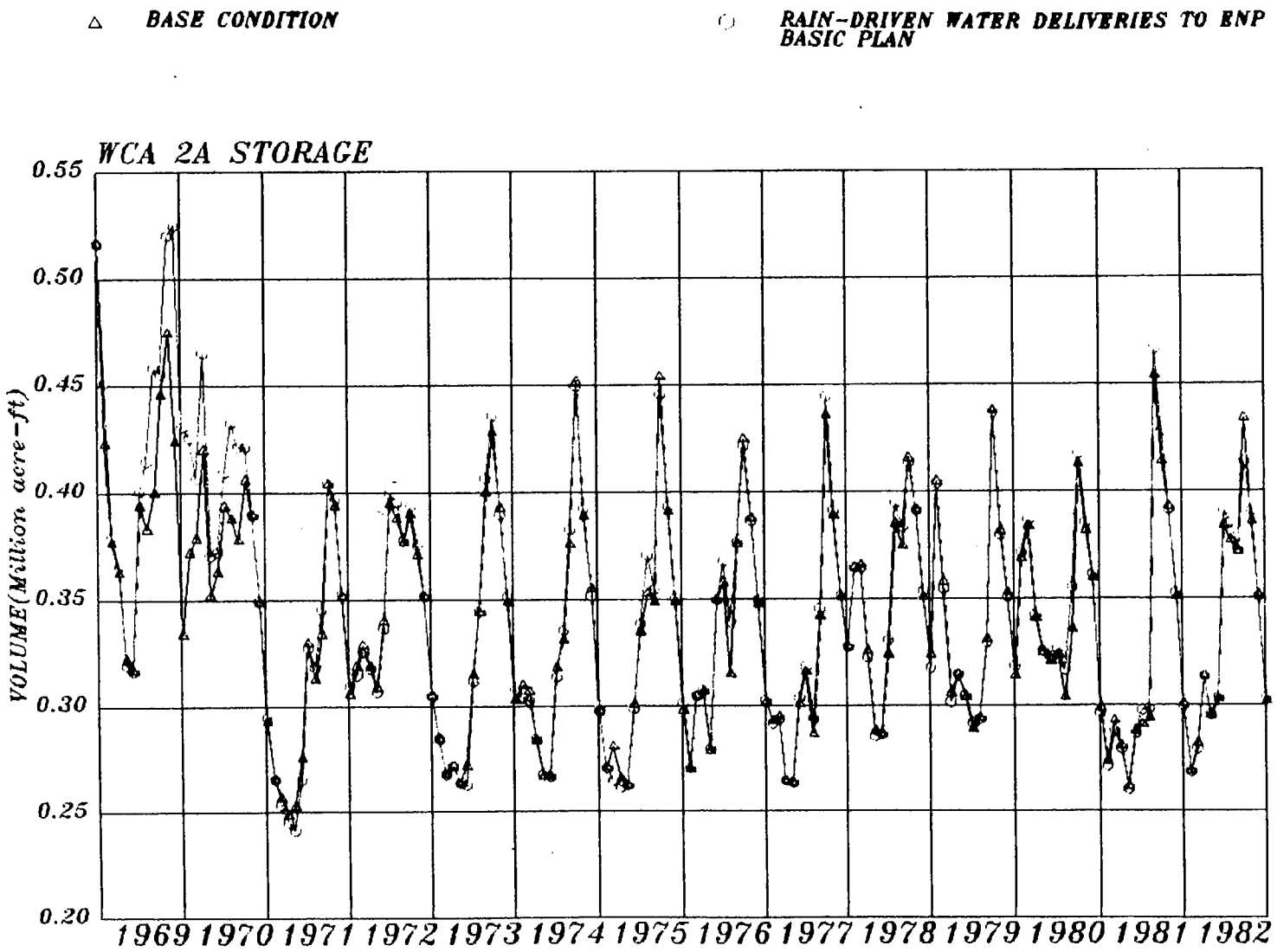
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
WCA 2A STORAGE
VOLUME(Million acre-ft)
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-62

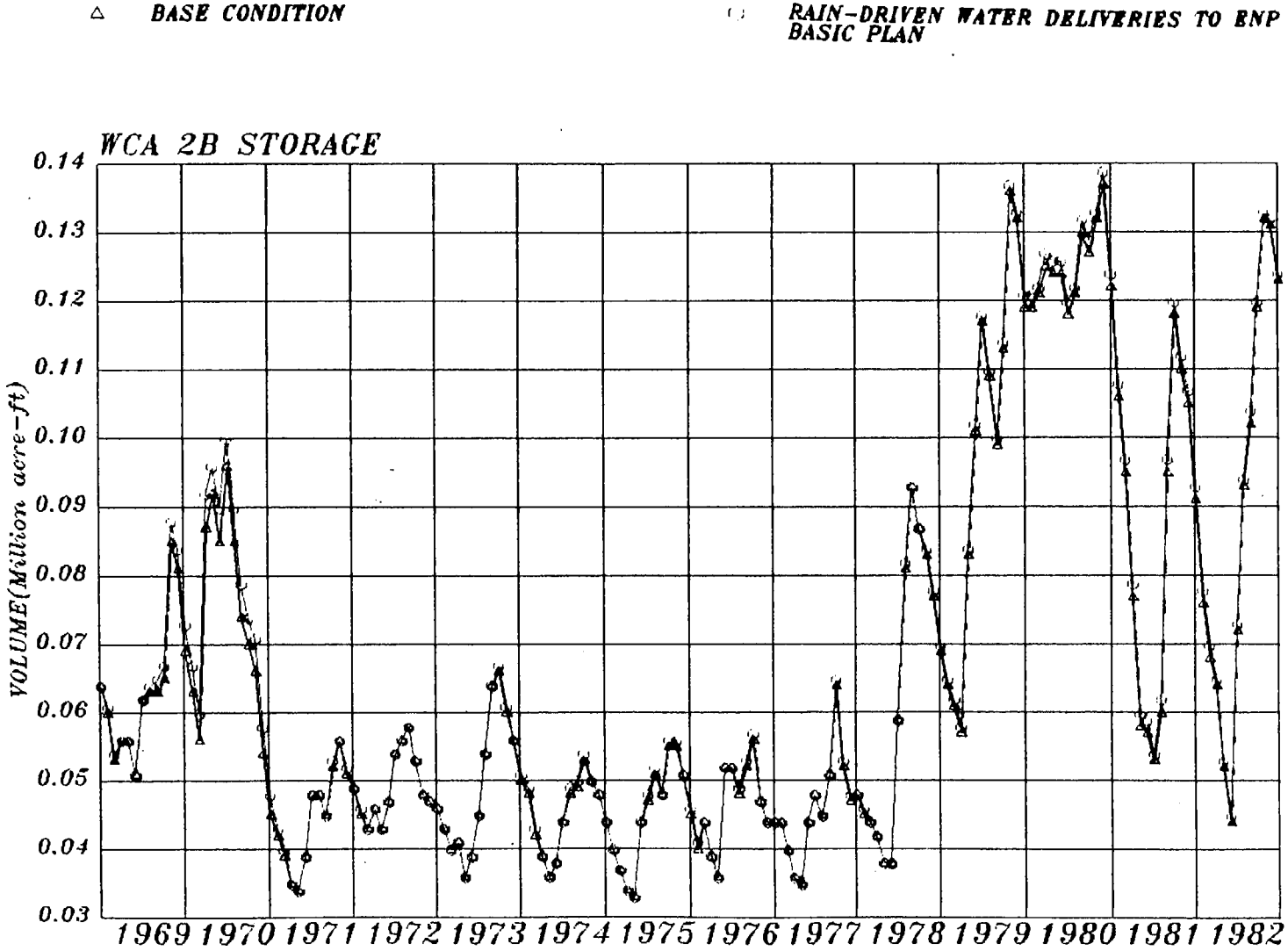
△ BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
WCA 2B STORAGE
VOLUME(Million acre-ft)
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-63

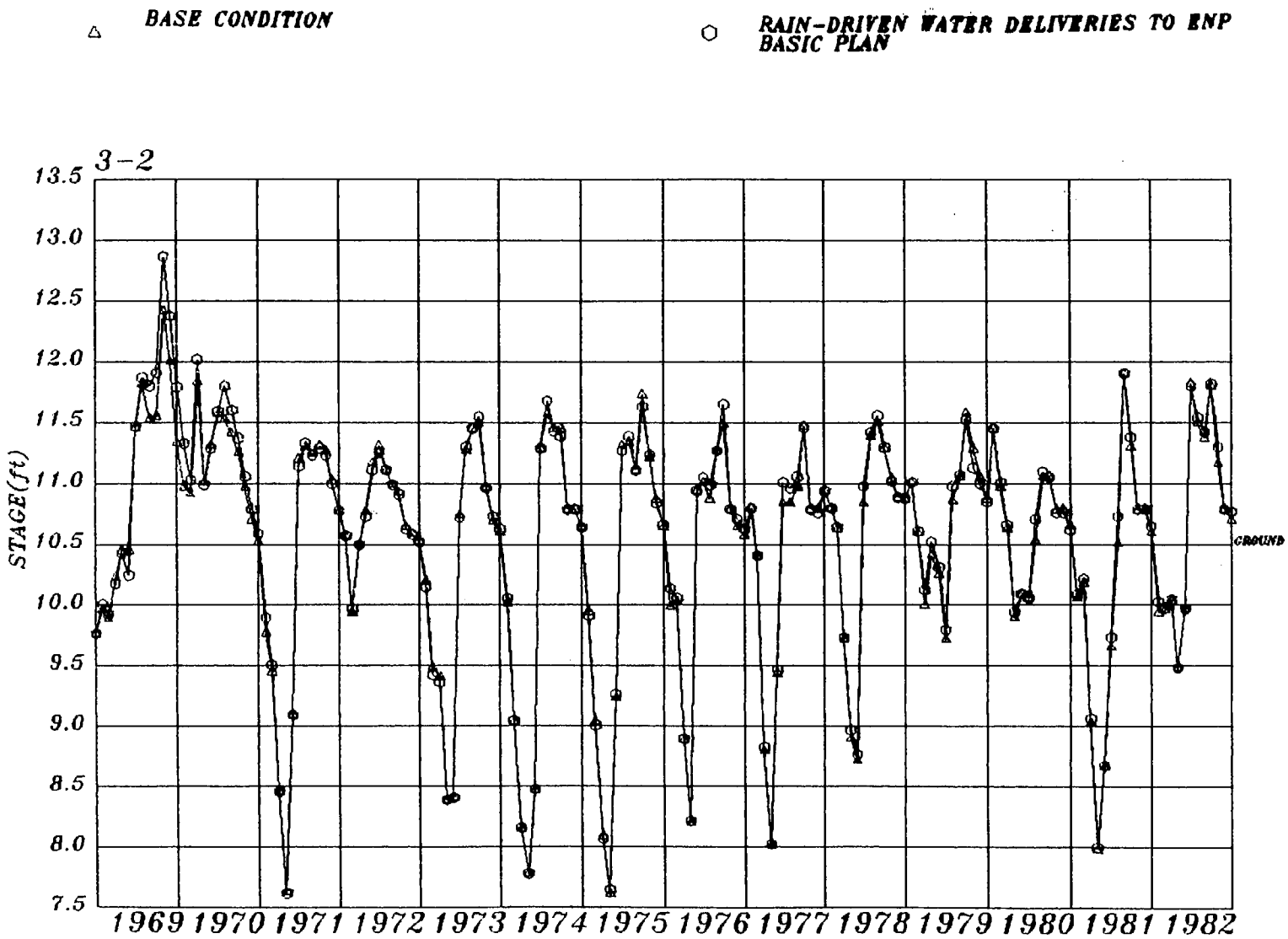
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
3-2
STAGE(ft)
13.5
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
GROUND

Figure E-64

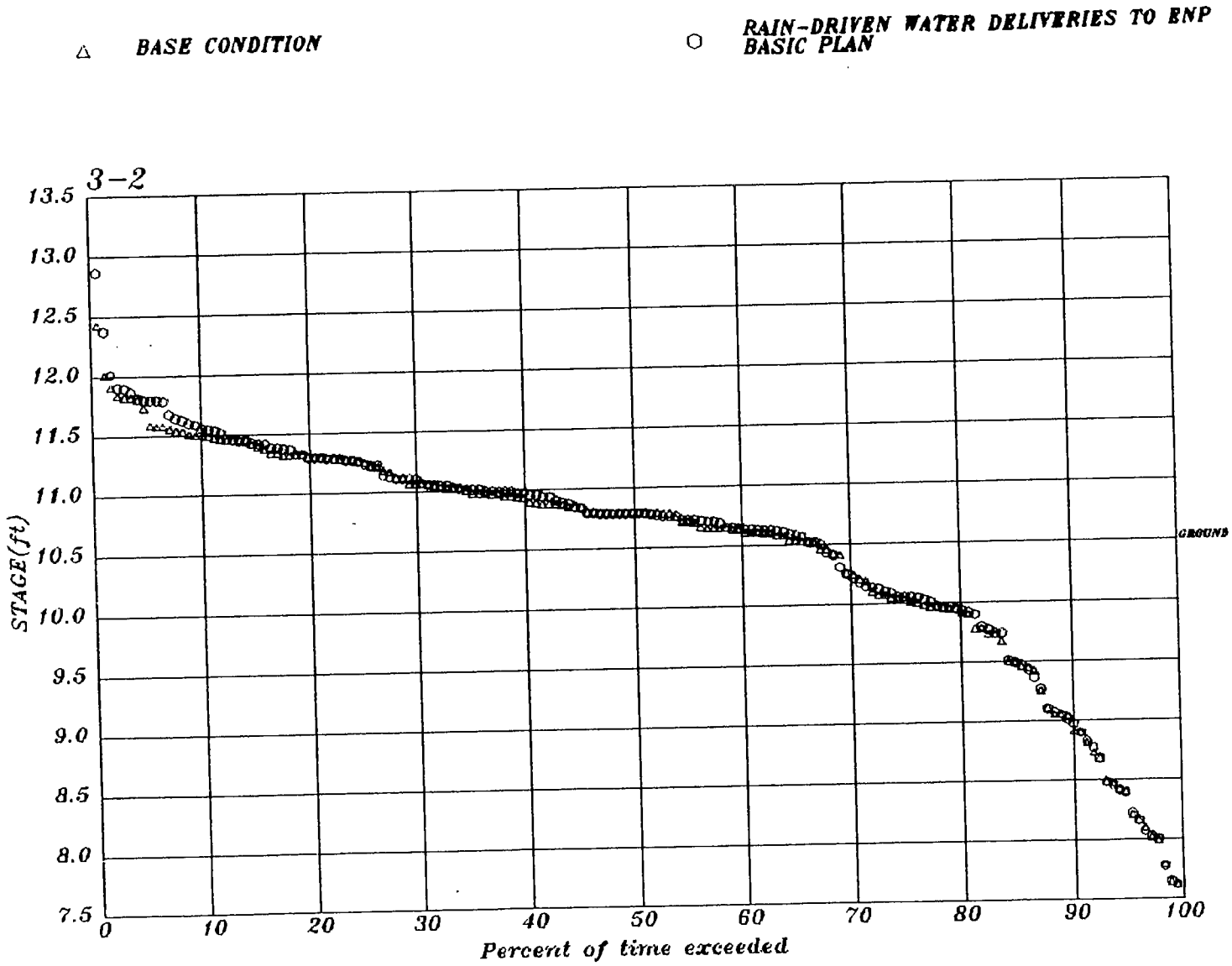
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-2
13.5
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-65

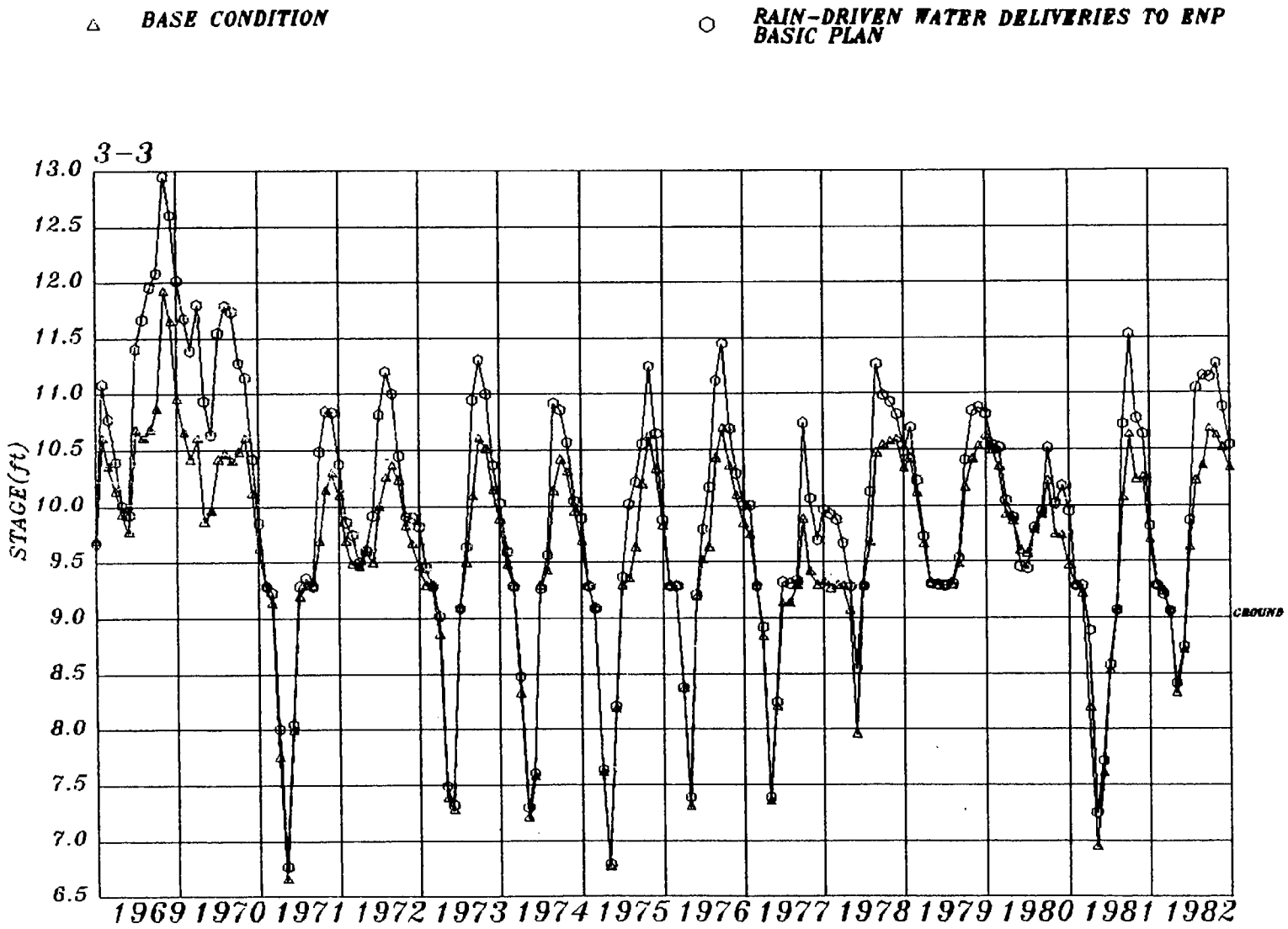
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-3
STAGE(ft)
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
GROUND

Figure E-66

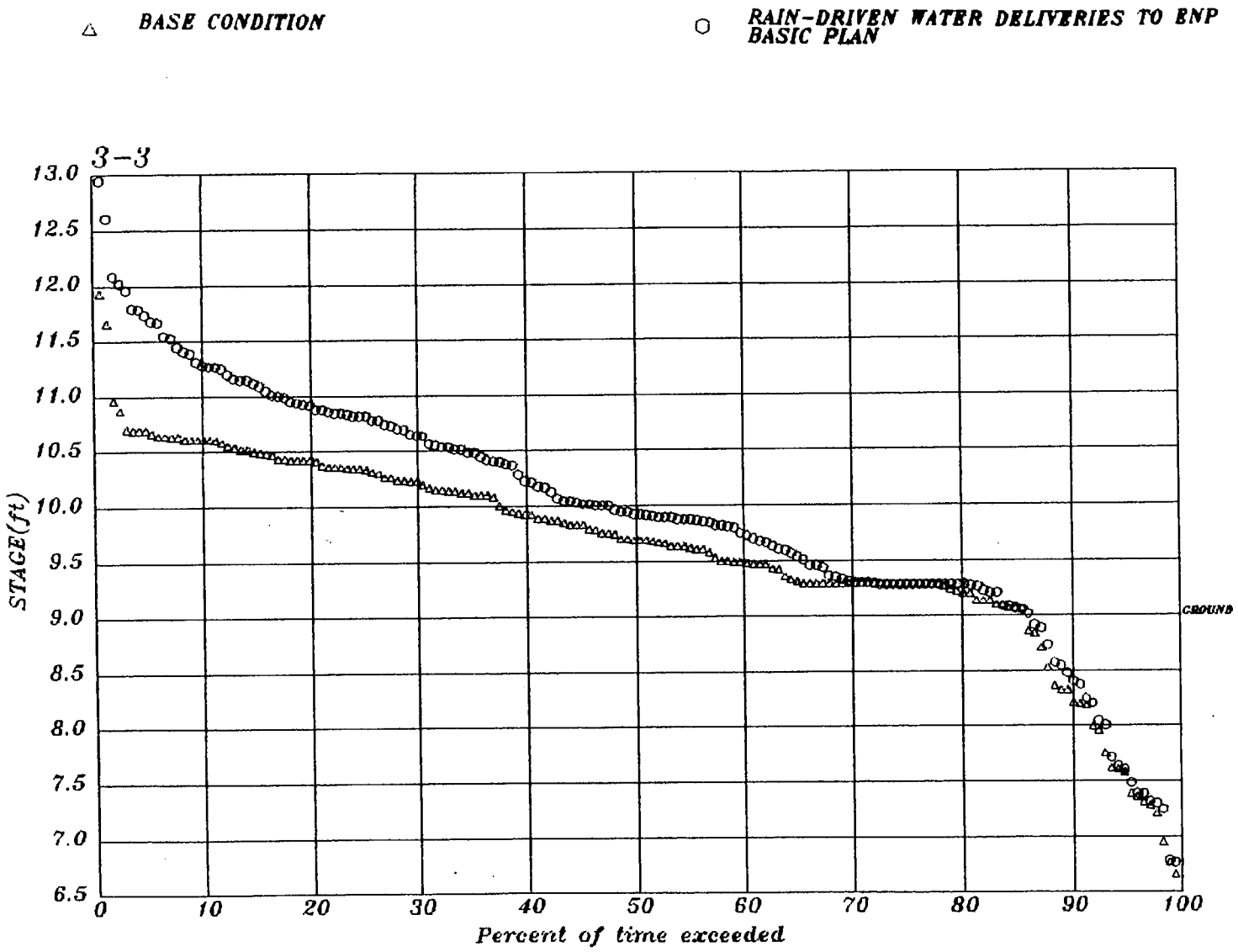
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-3
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-67

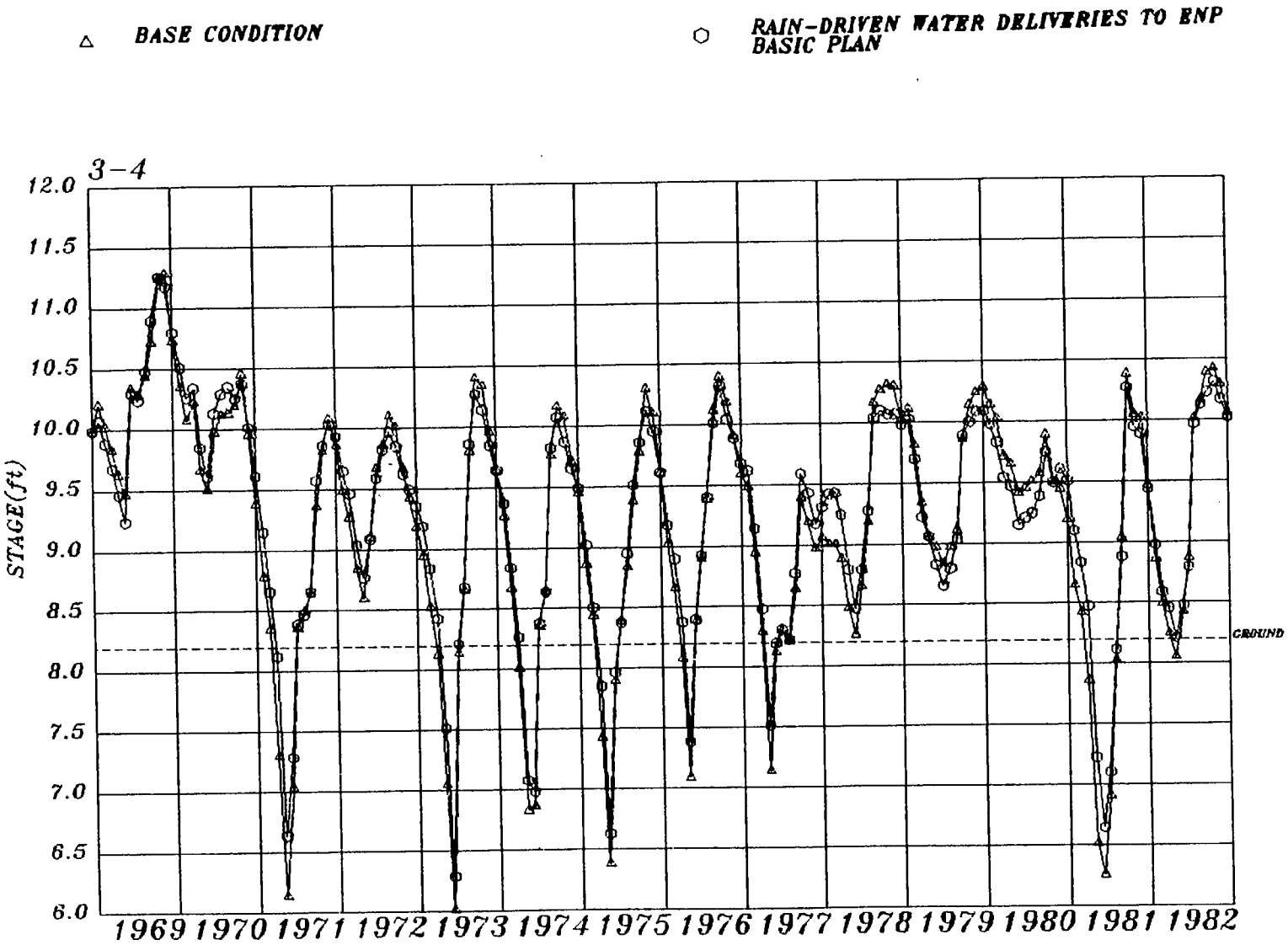
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-4
STAGE(ft)
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-68

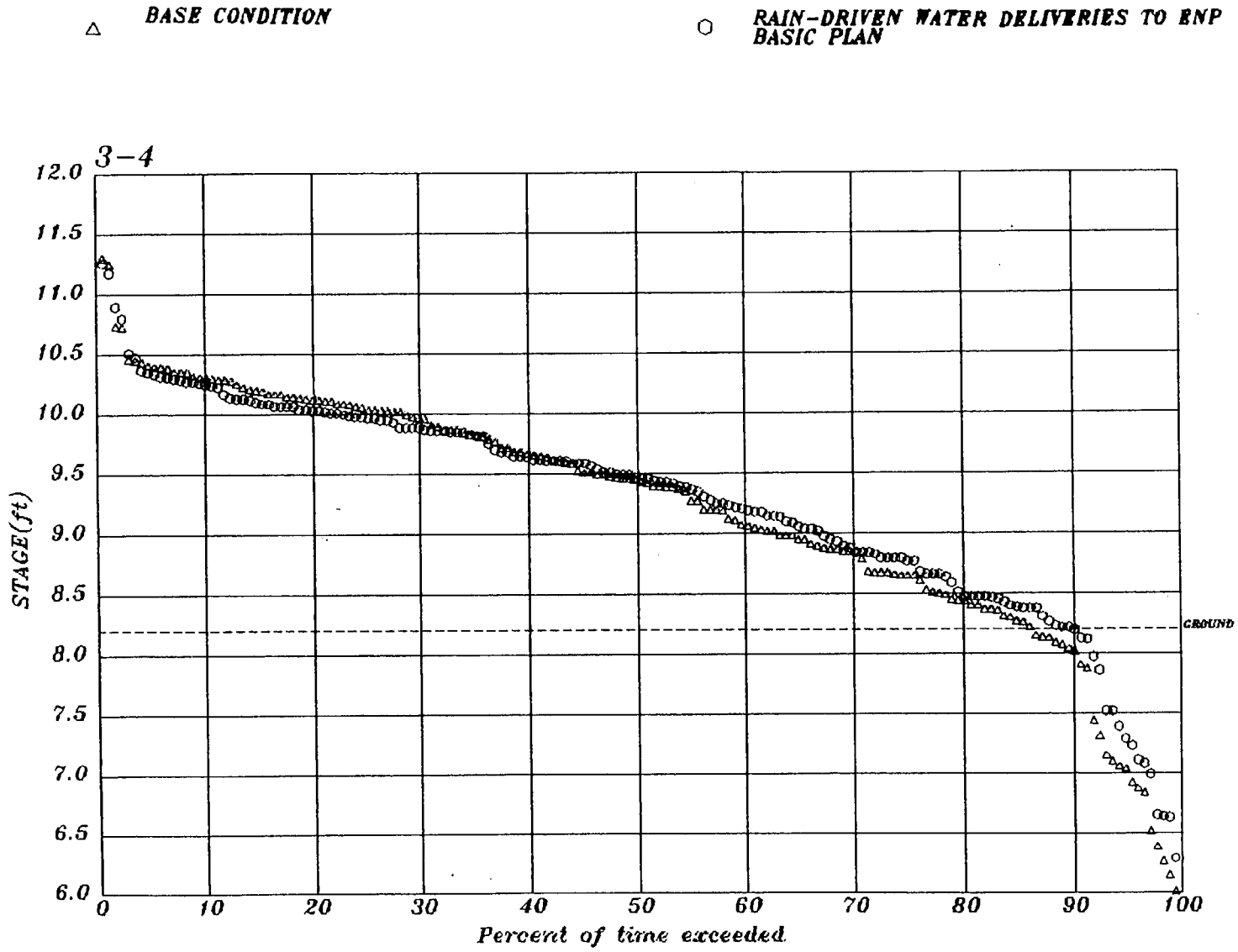
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
3-4
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-69

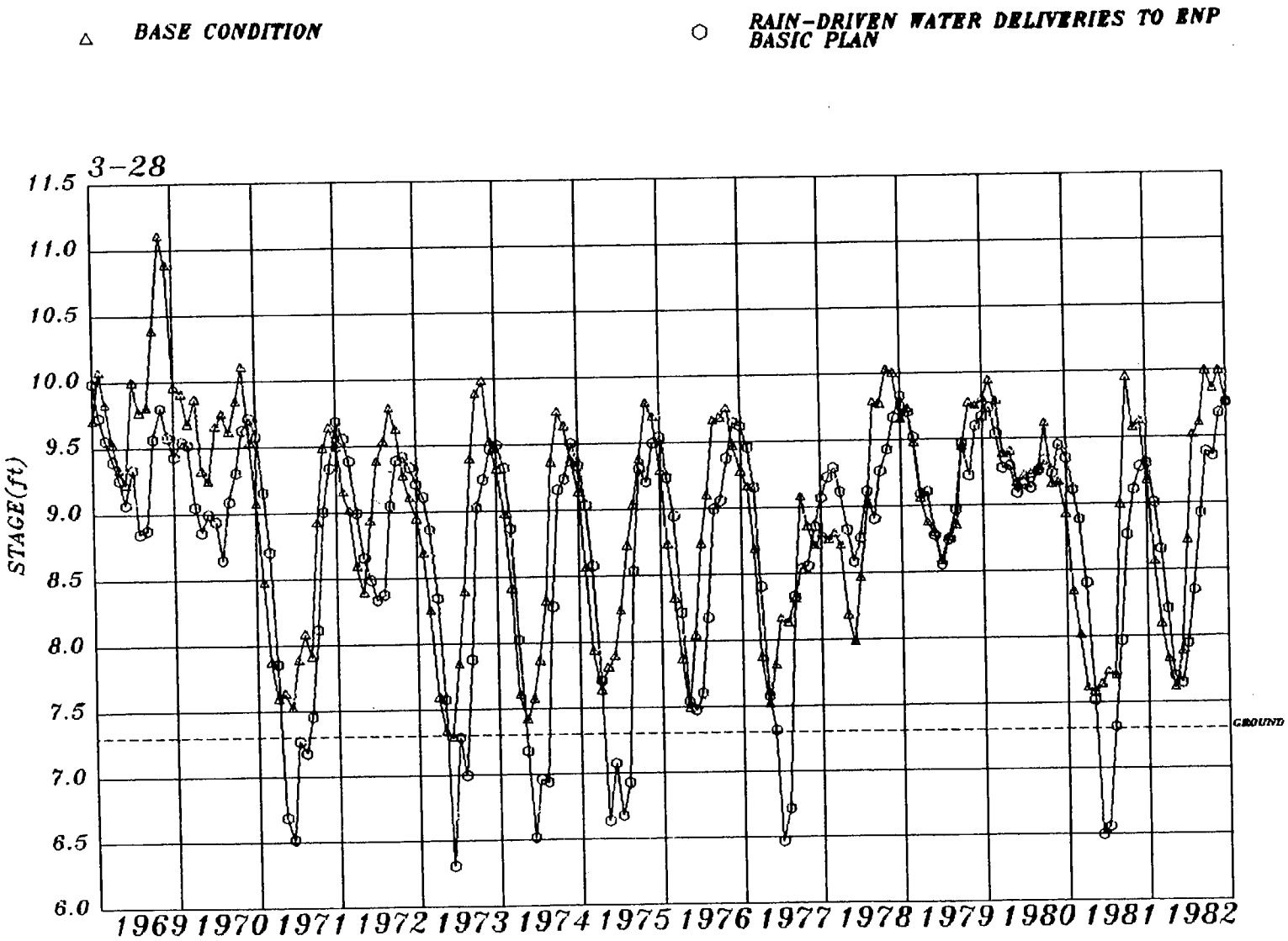
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-28
STAGE(ft)
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-70

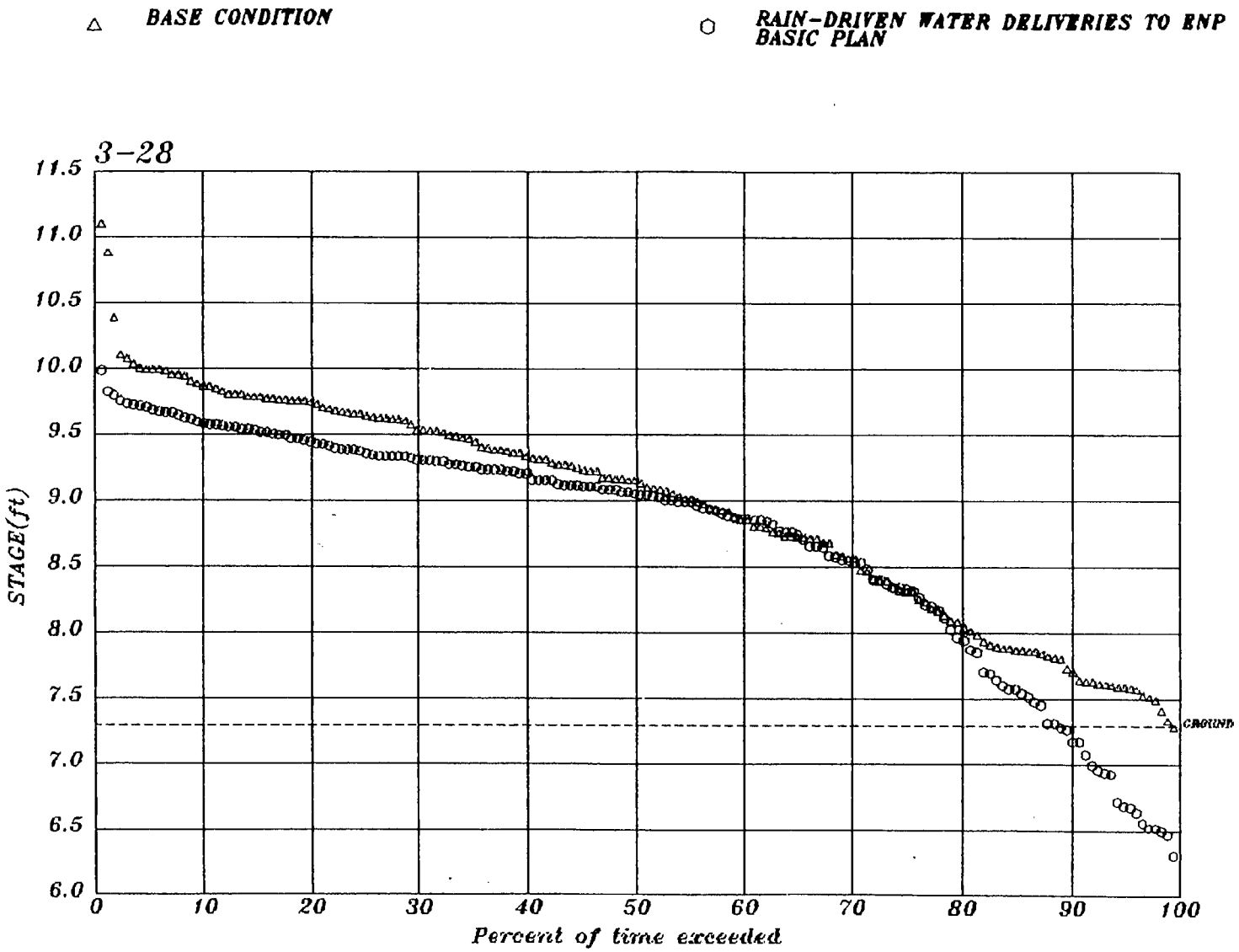
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-28
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-71

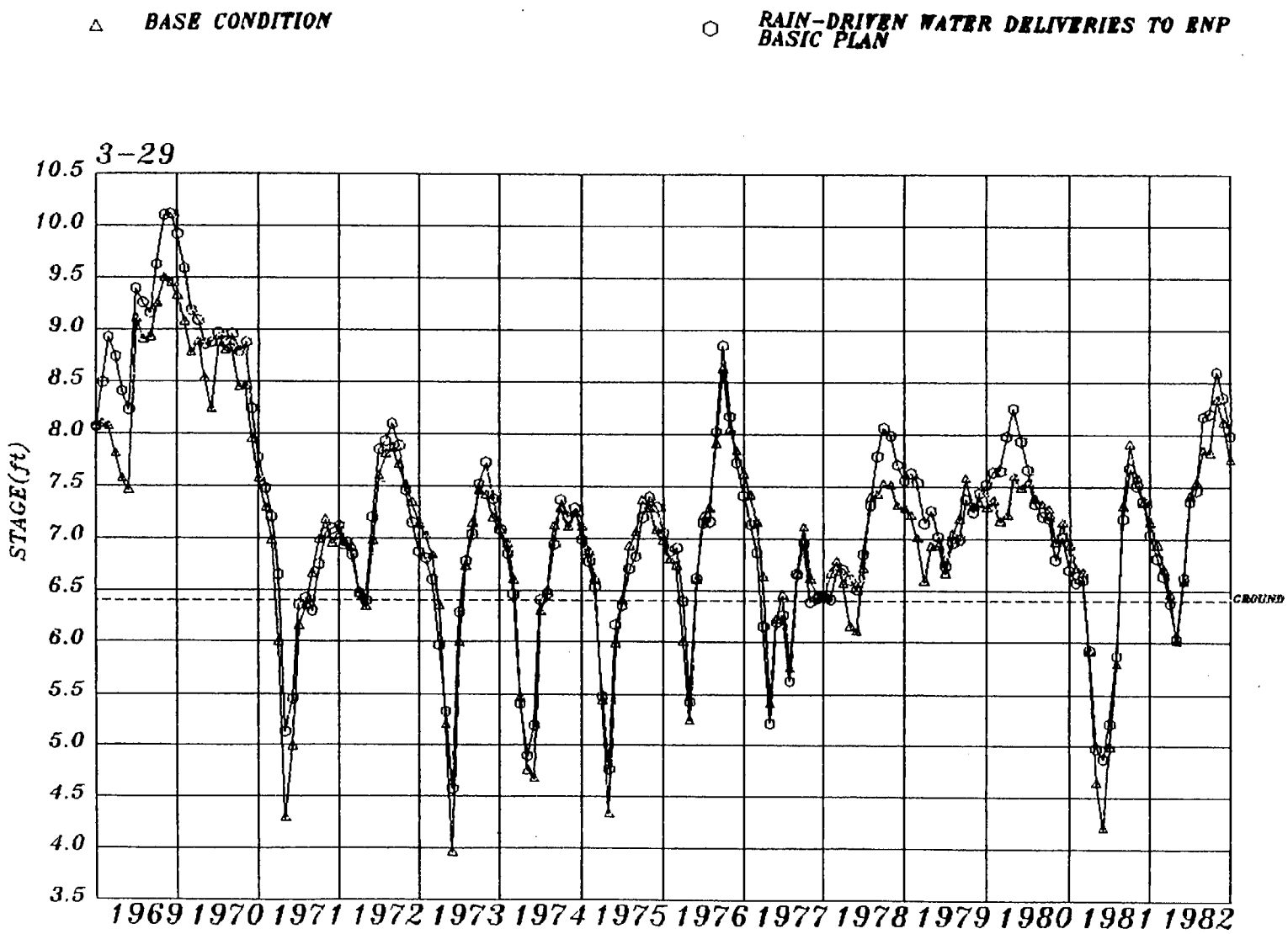
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-29
STAGE(ft)
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-72

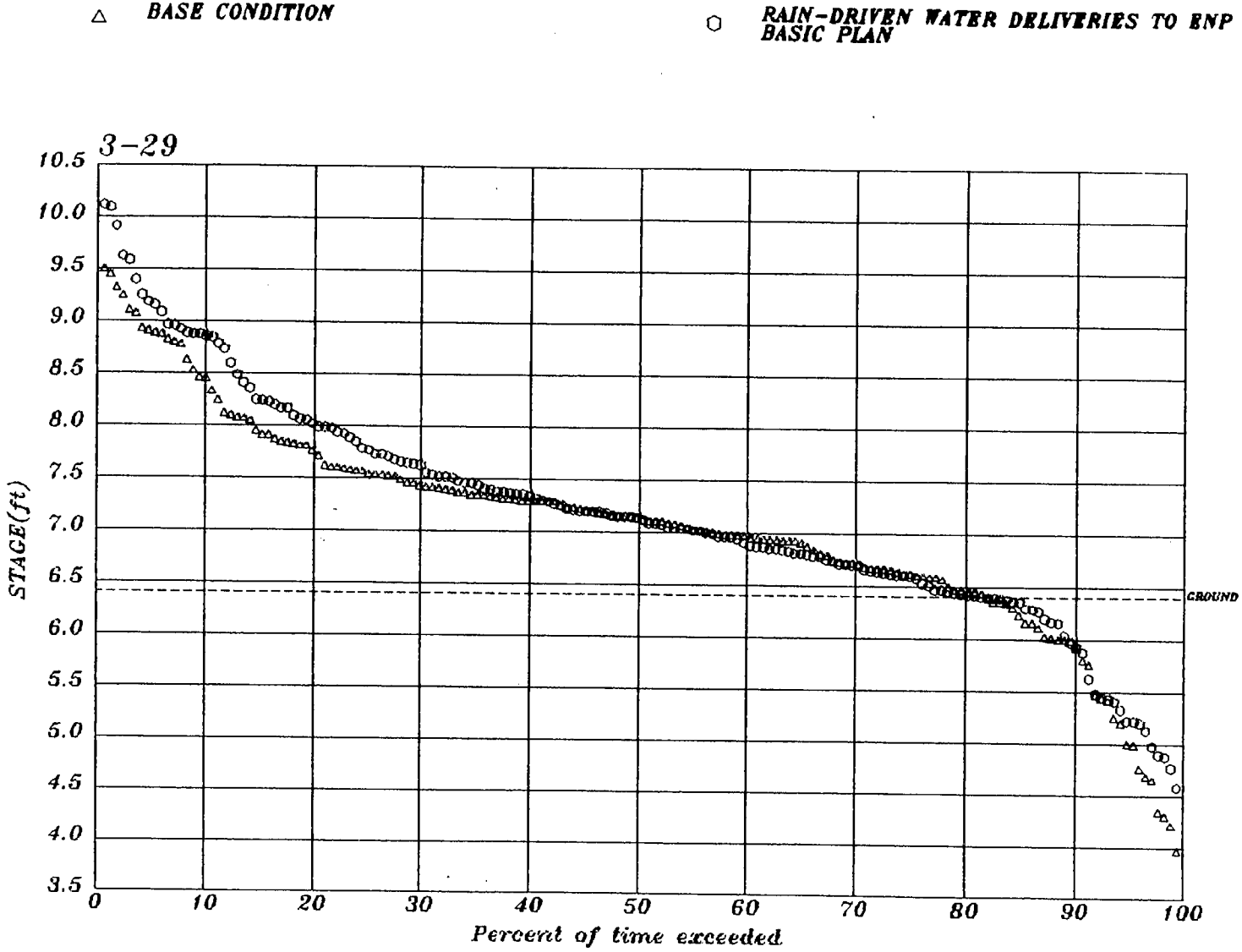
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
3-29
STAGE(ft)
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-73

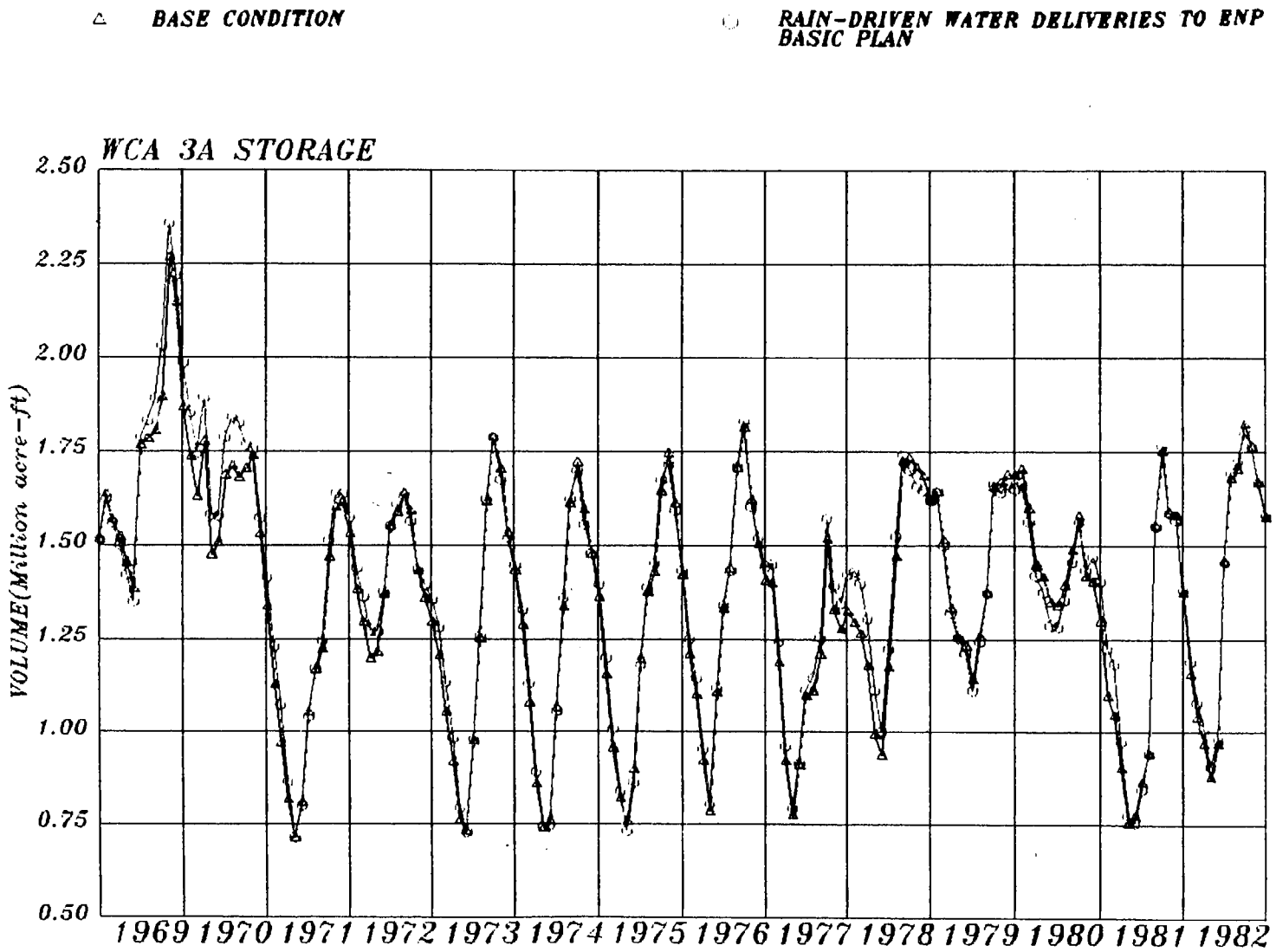
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
WCA 3A STORAGE
VOLUME(Million acre-ft)
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-74

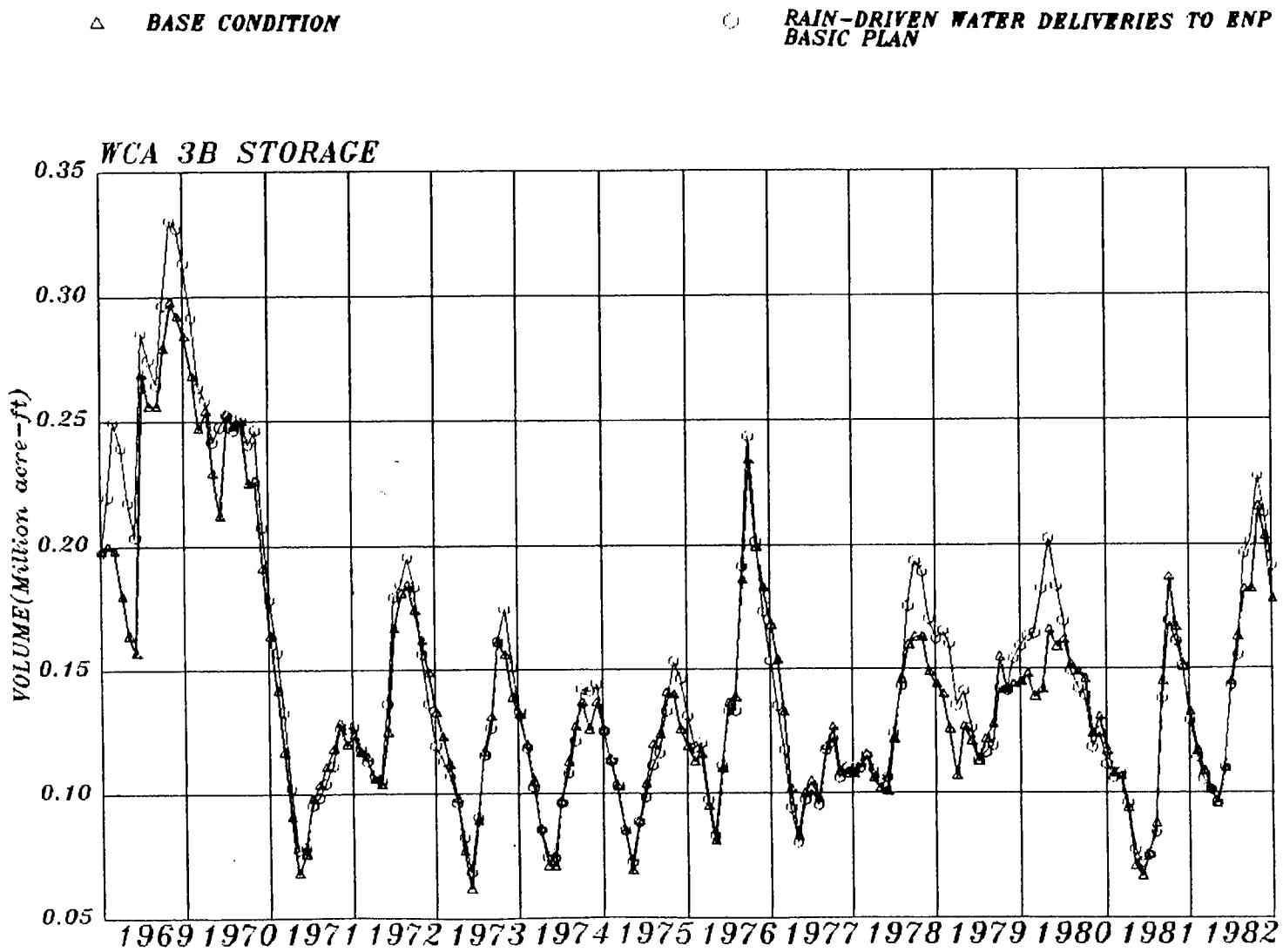
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
WCA 3B STORAGE
VOLUME(Million acre-ft)
0.35
0.30
0.25
0.20
0.15
0.10
0.05
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-75

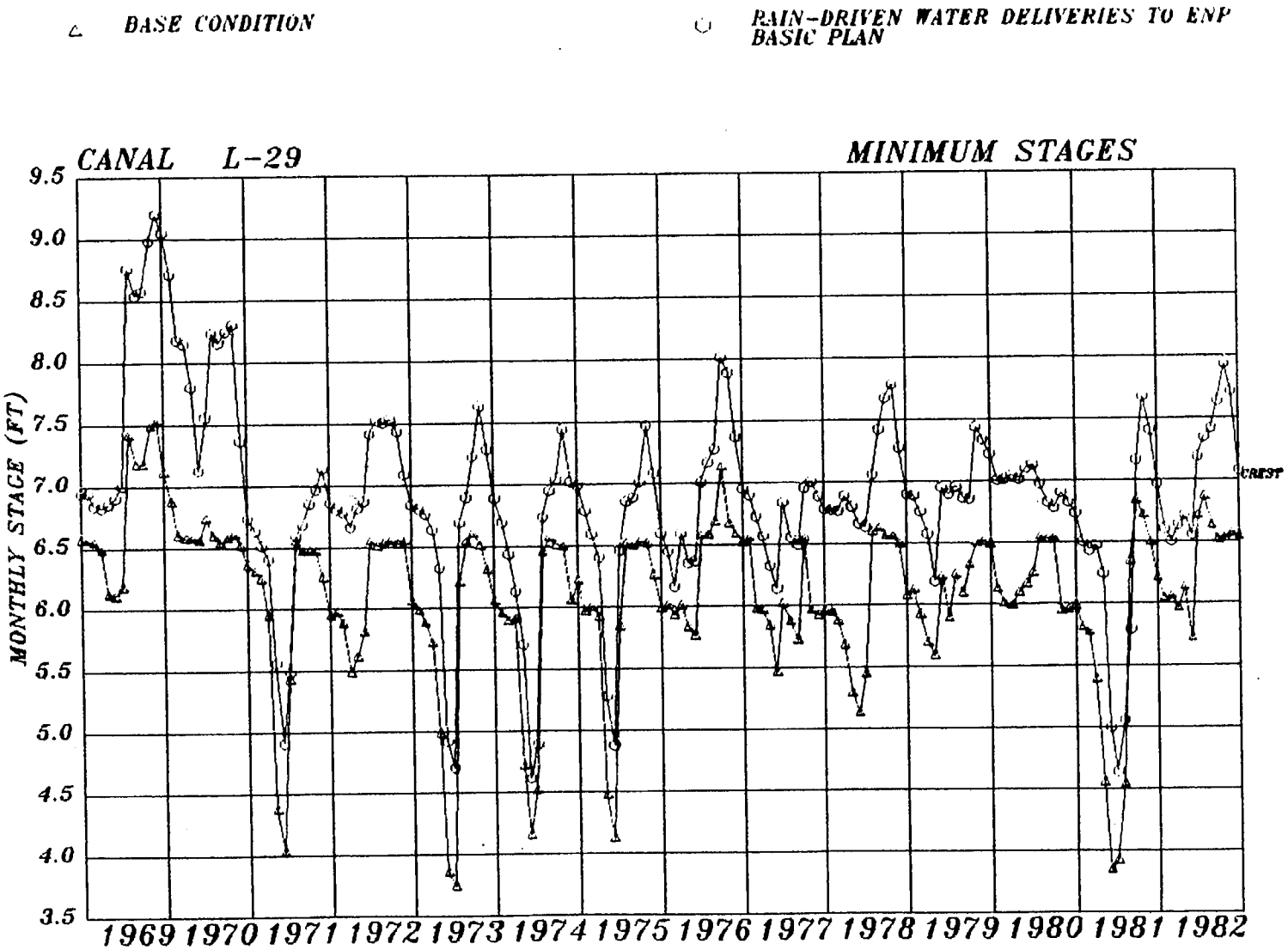
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
CANAL L-29
MINIMUM STAGES
MONTHLY STAGE (FT)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-76

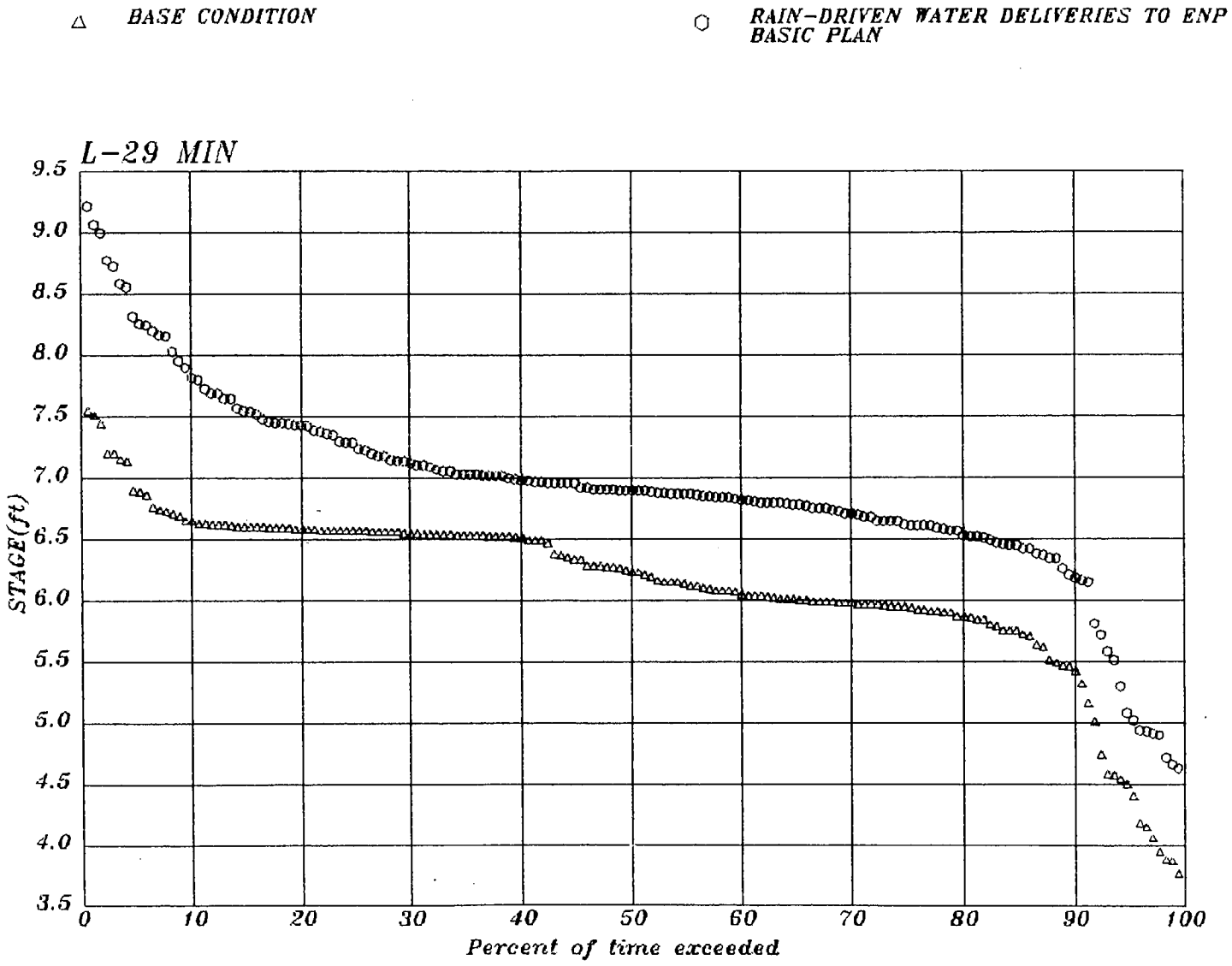
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
L-29 MIN
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-77

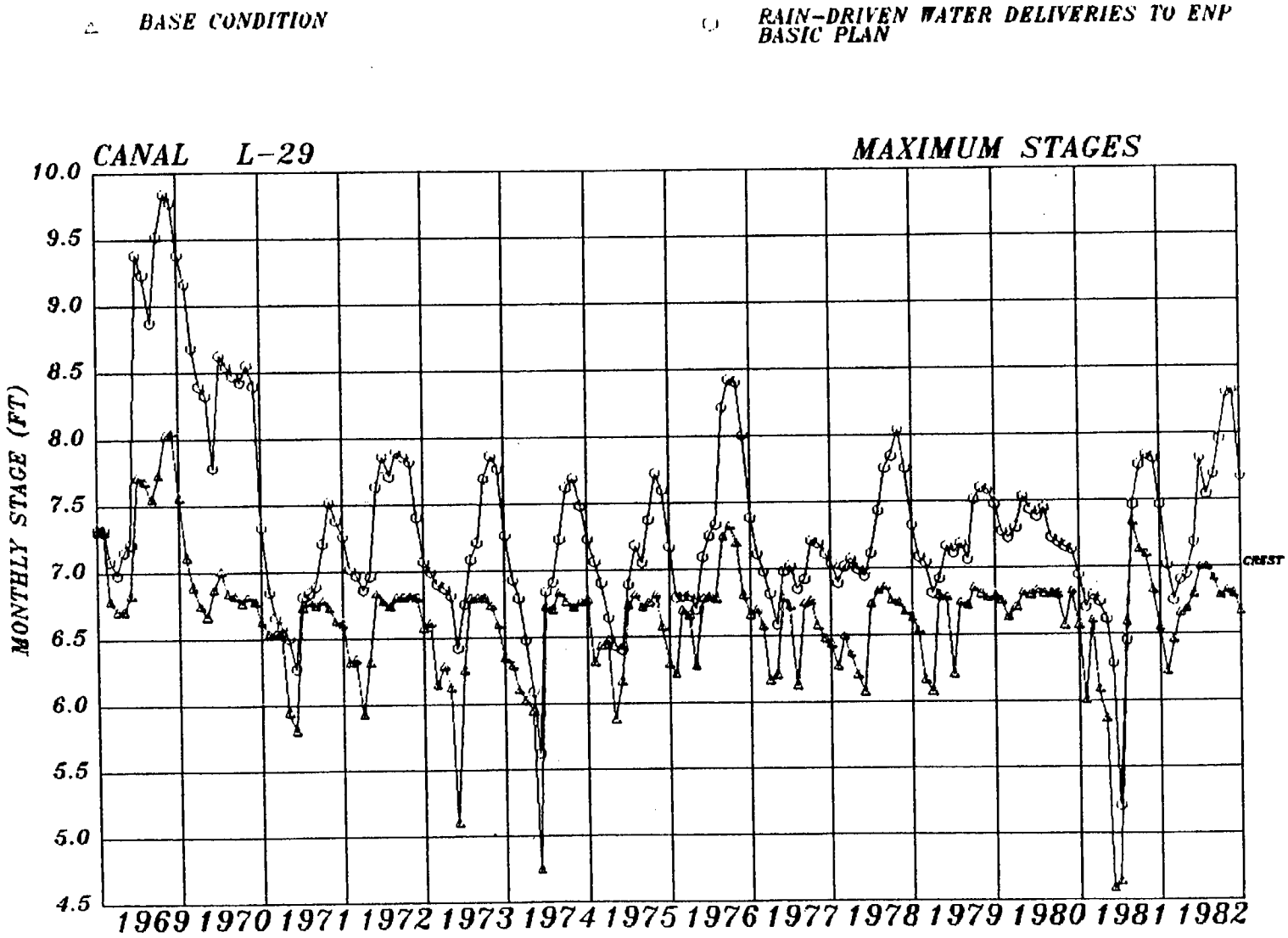
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
CANAL L-29
MAXIMUM STAGES
MONTHLY STAGE (FT)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-78

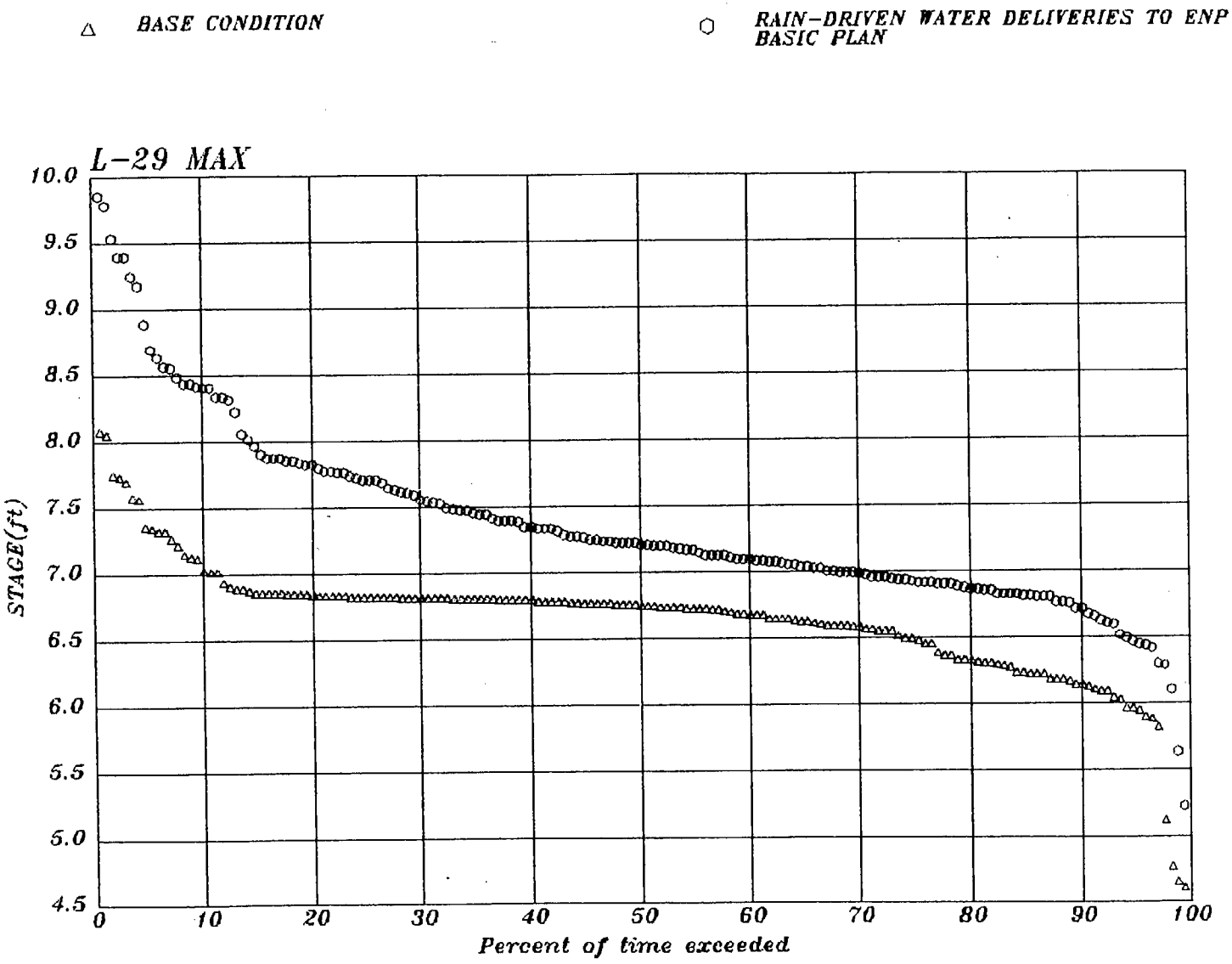
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
L-29 MAX
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-79

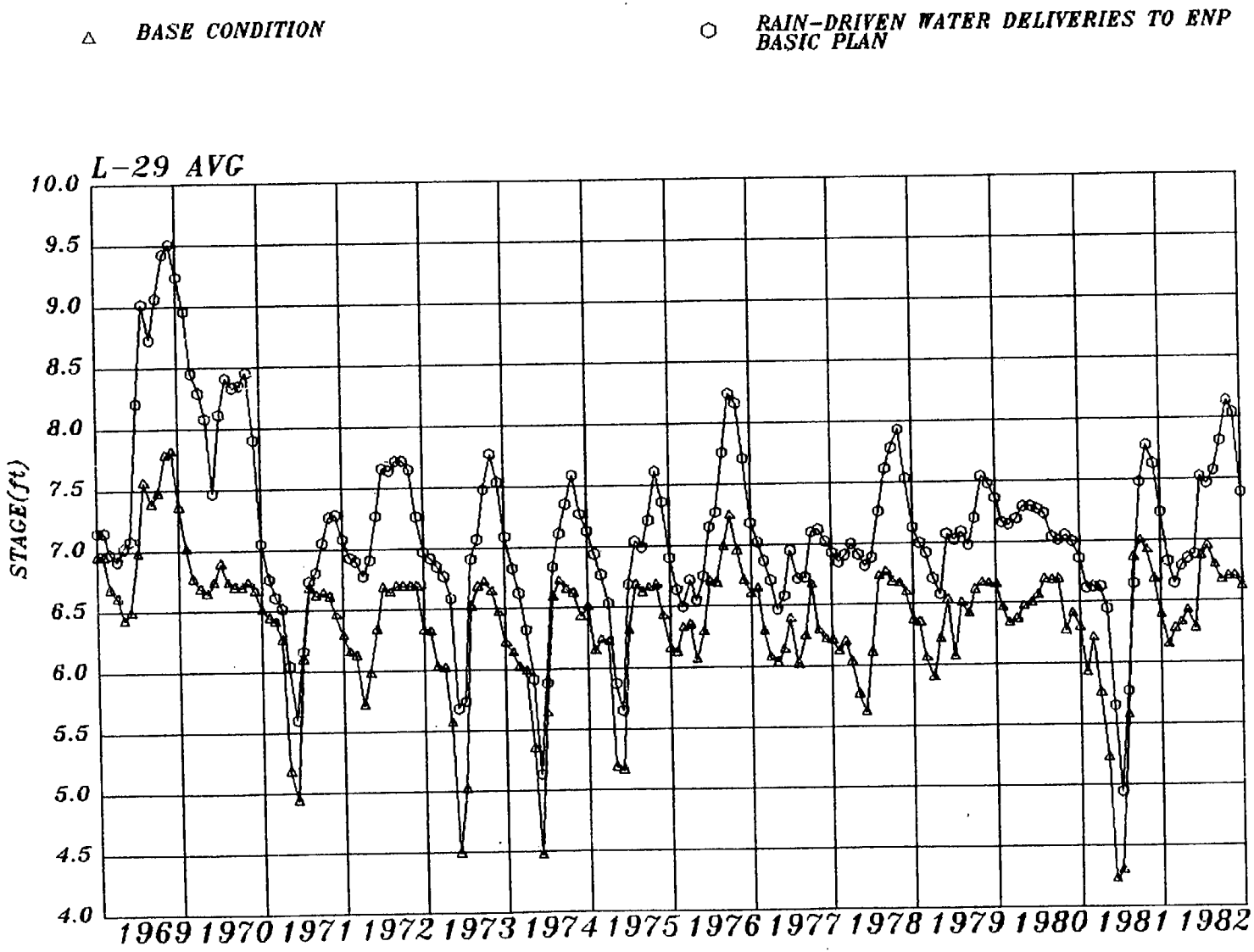
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
L-29 AVG
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-80

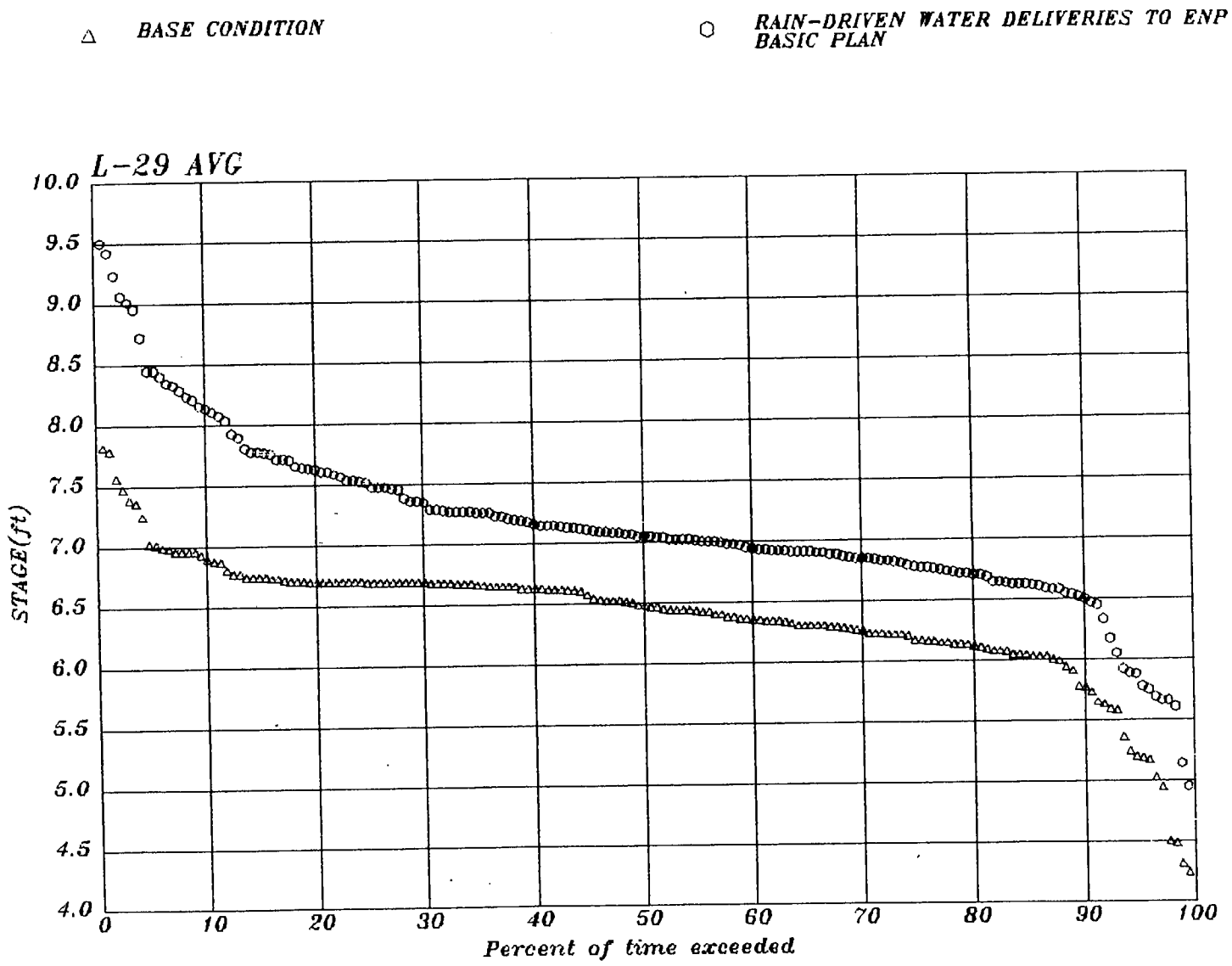
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
L-29 AVG
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-81

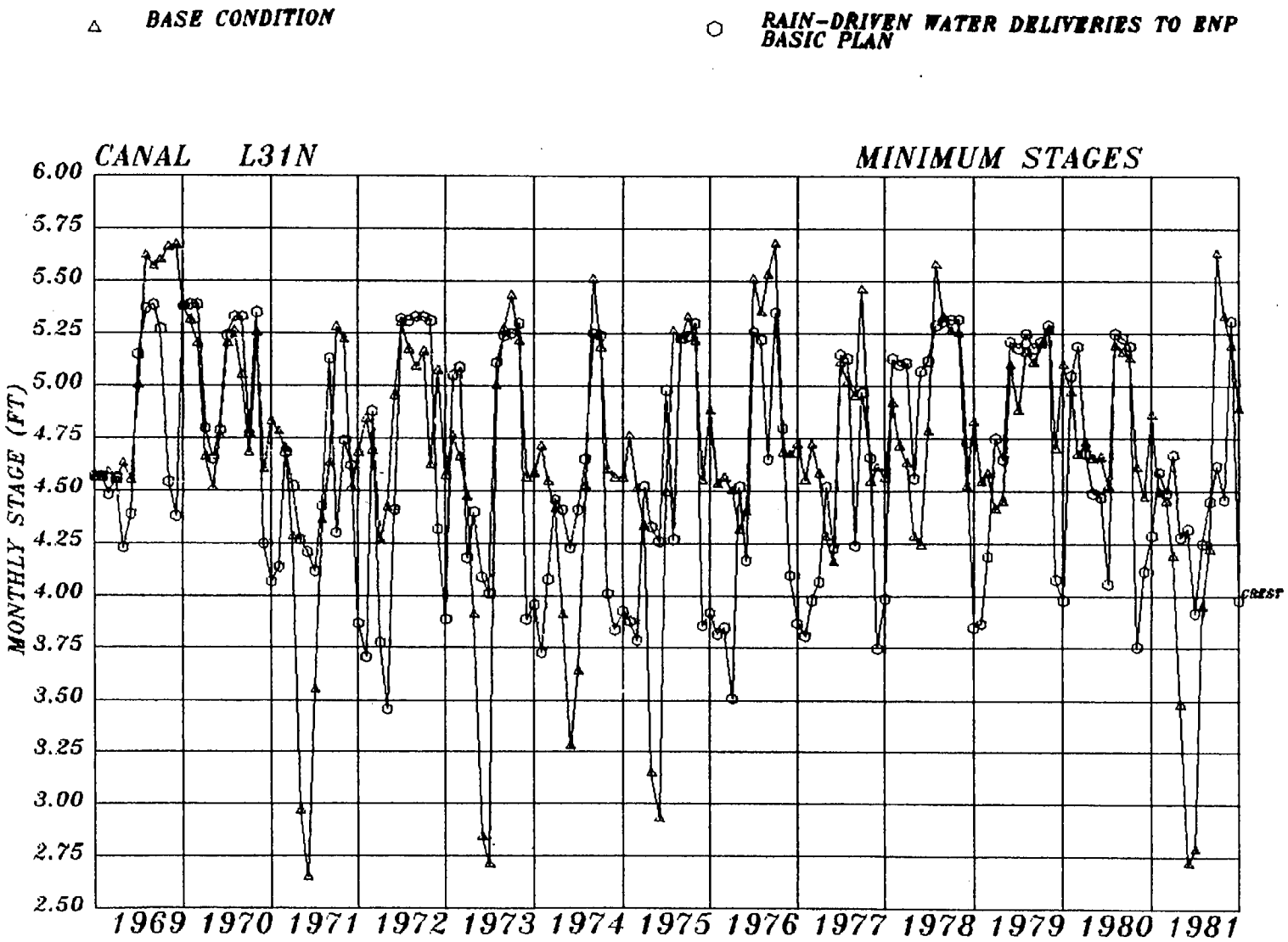
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
CANAL L31N
MINIMUM STAGES
MONTHLY STAGE (FT)
6.00
5.75
5.50
5.25
5.00
4.75
4.50
4.25
4.00
3.75
3.50
3.25
3.00
2.75
2.50
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981
CREST

Figure E-82

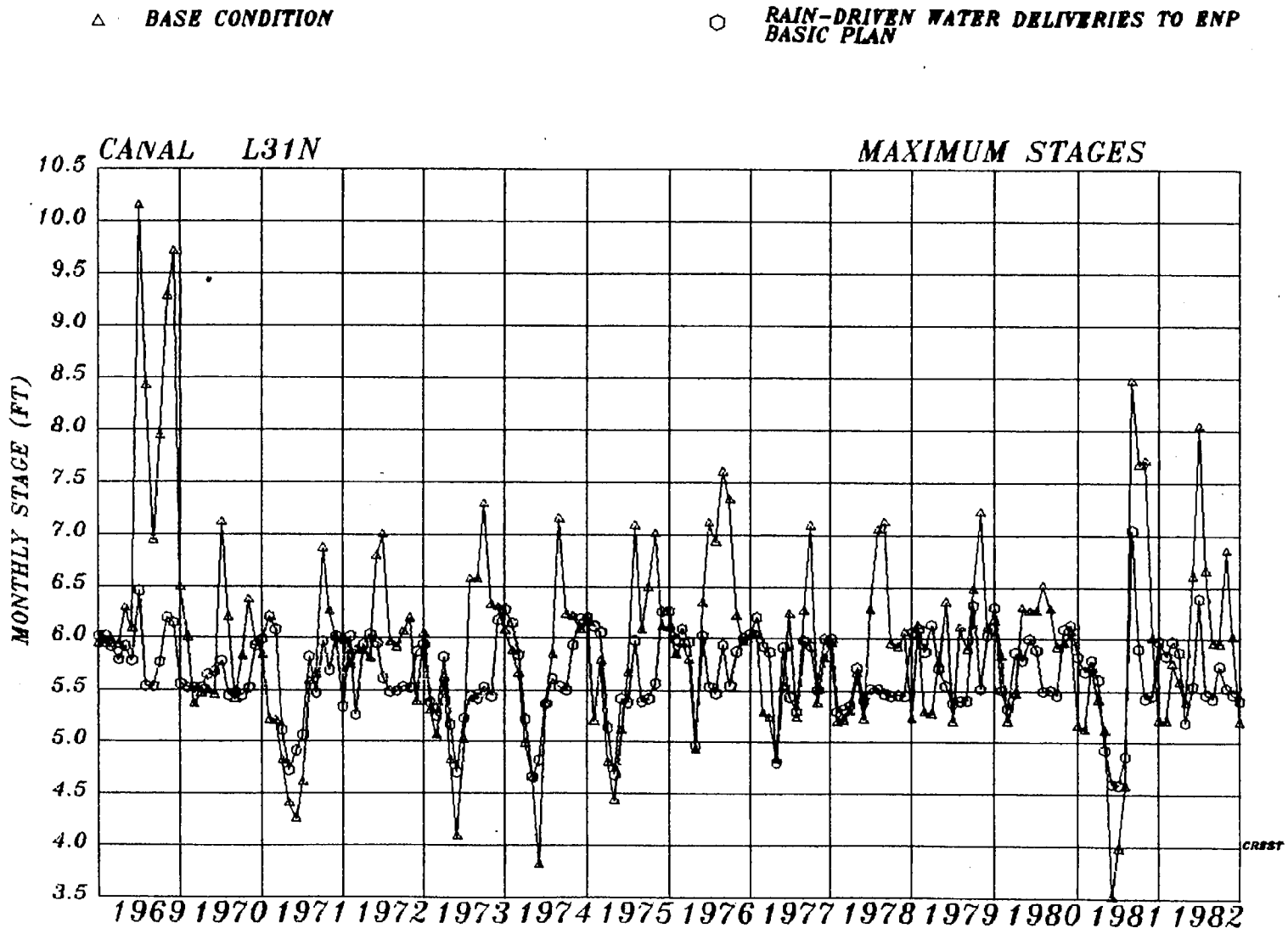
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
CANAL L31N
MAXIMUM STAGES
MONTHLY STAGE (FT)
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-83

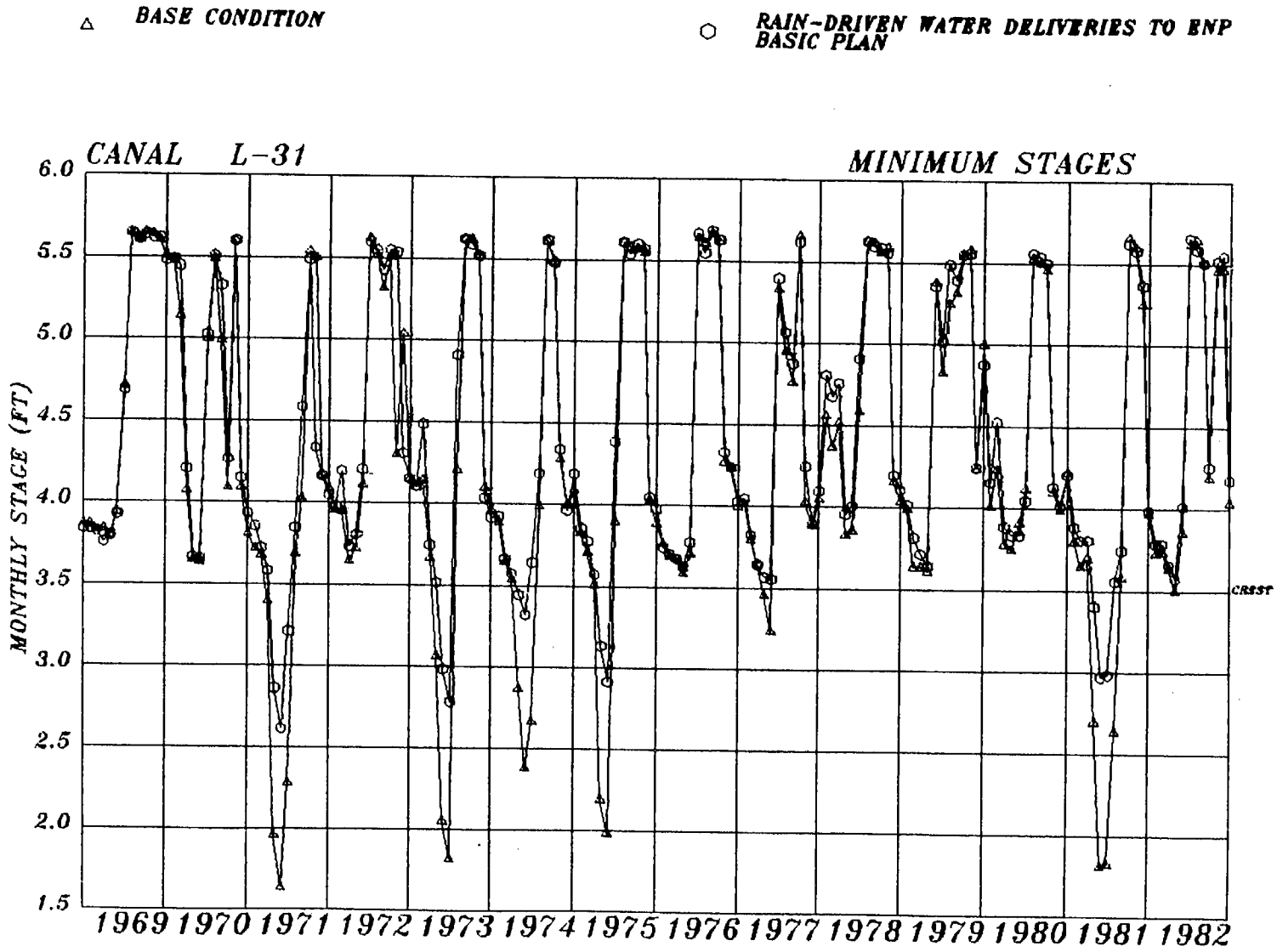
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
CANAL L-31
MINIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
CRSST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-84

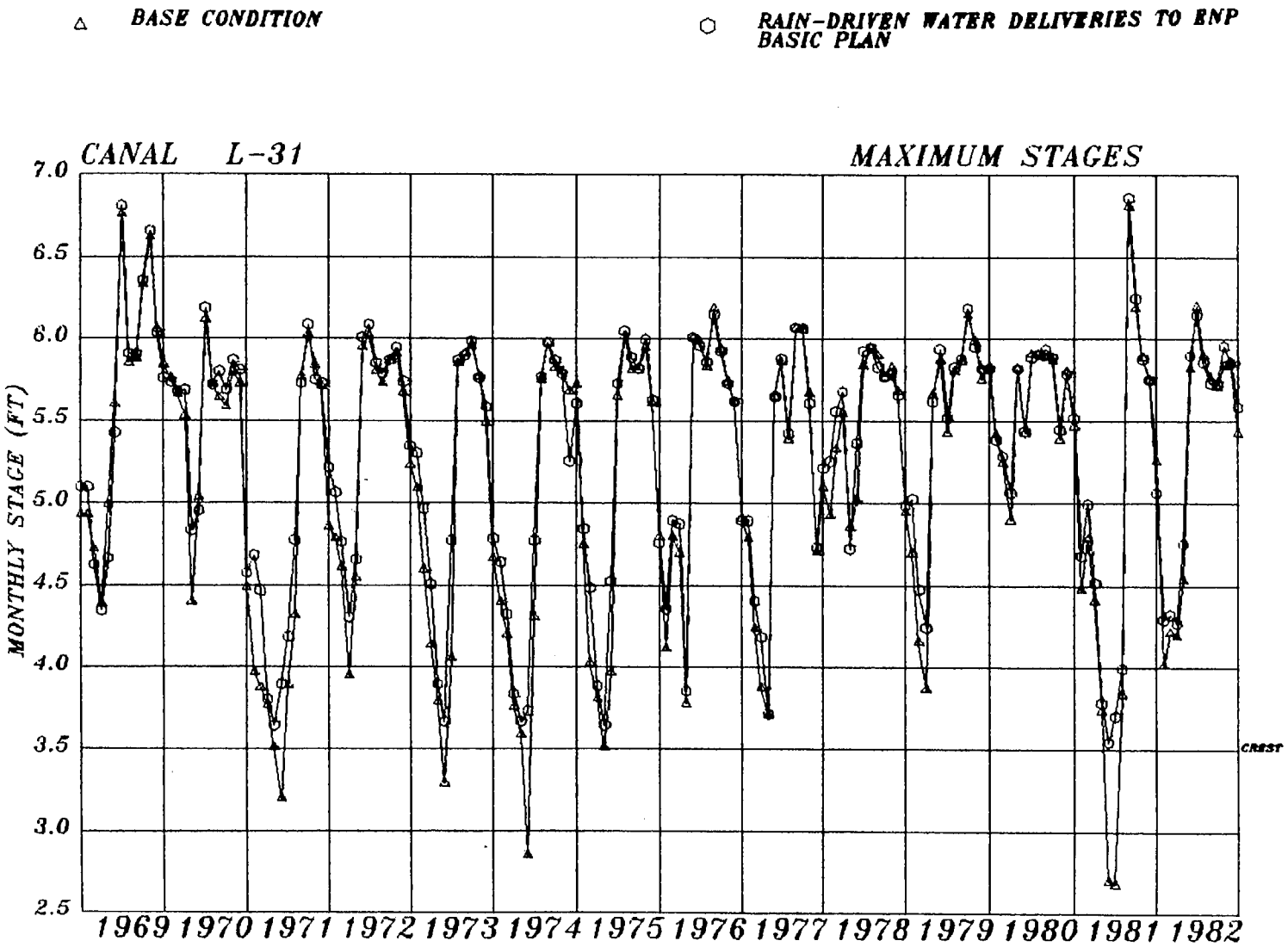
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
CANAL L-31
MAXIMUM STAGES
MONTHLY STAGE (FT)
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-85

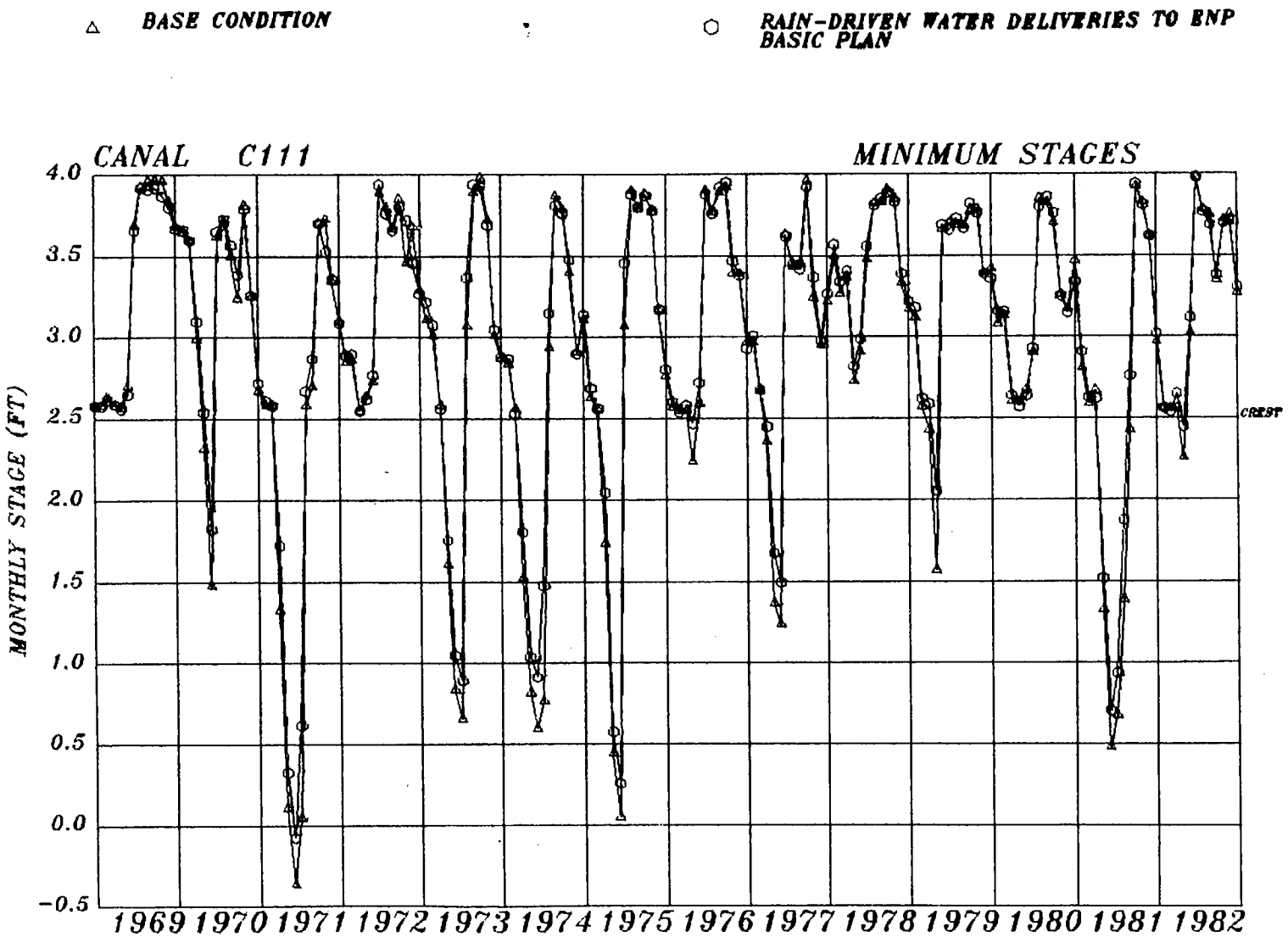
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
CANAL C111
MINIMUM STAGES
MONTHLY STAGE (FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-86

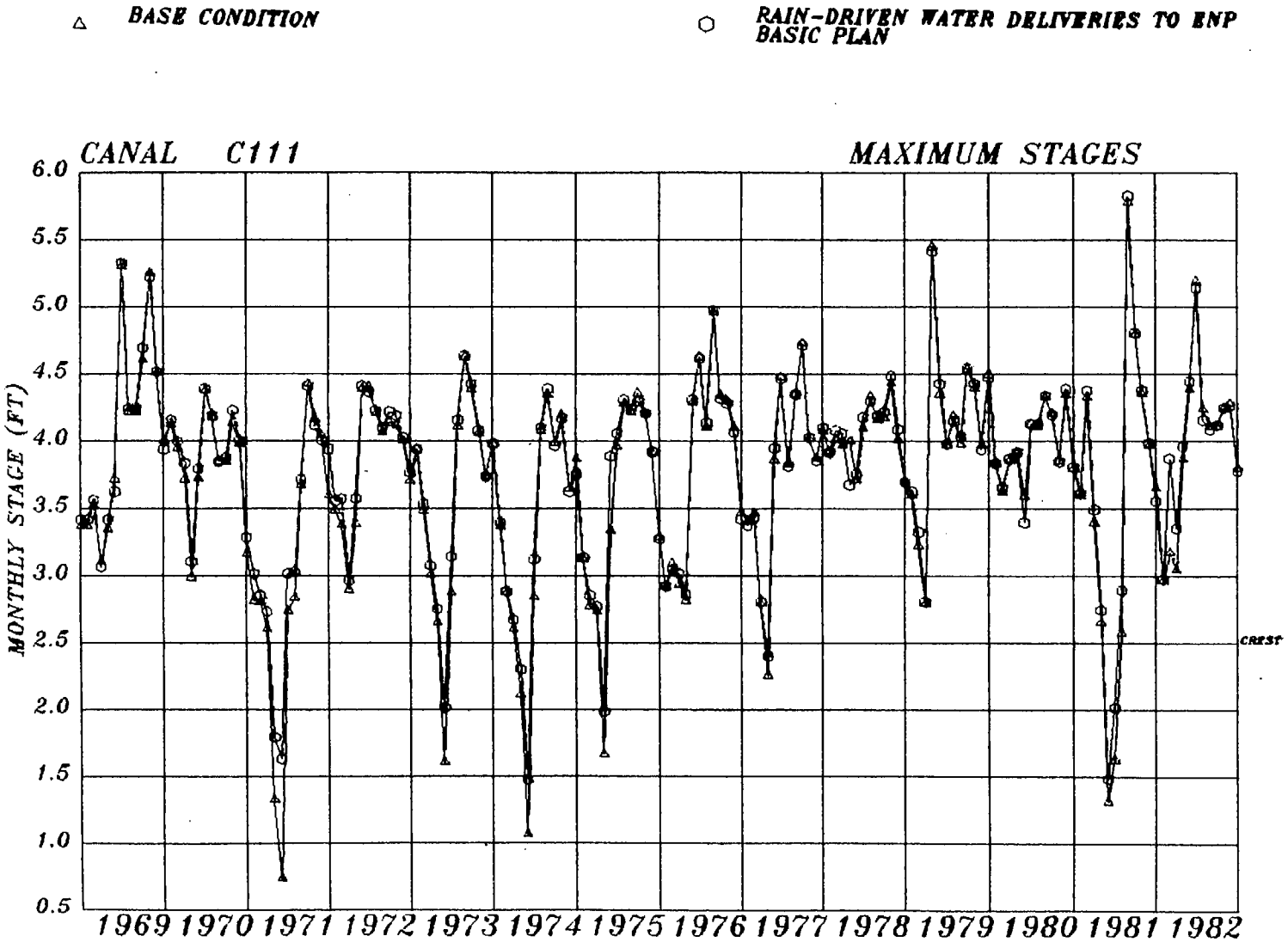
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
CANAL C111
MAXIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-87

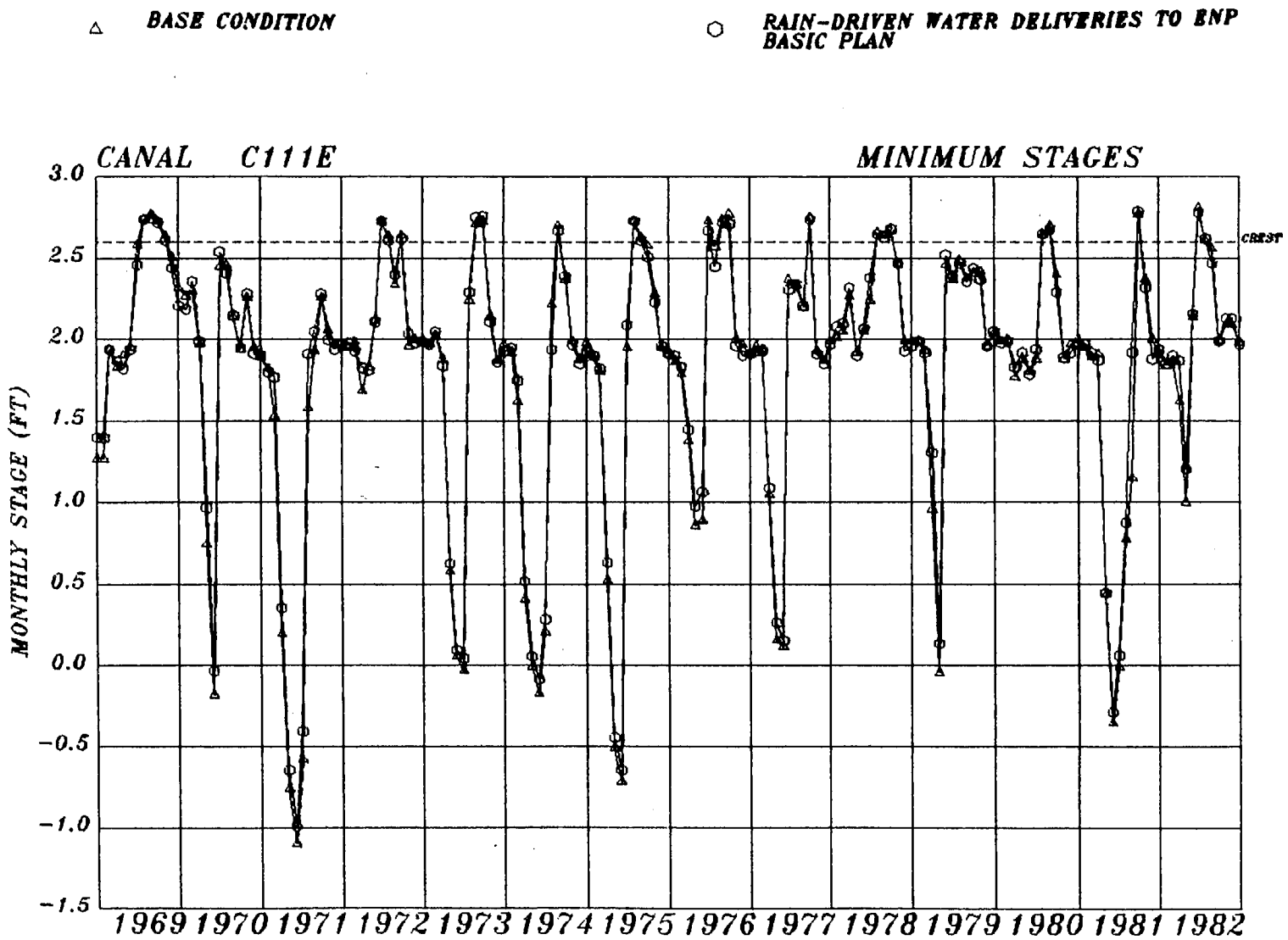
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO BNP BASIC PLAN
CANAL C111E
MINIMUM STAGES
CREST
MONTHLY STAGE (FT)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-88

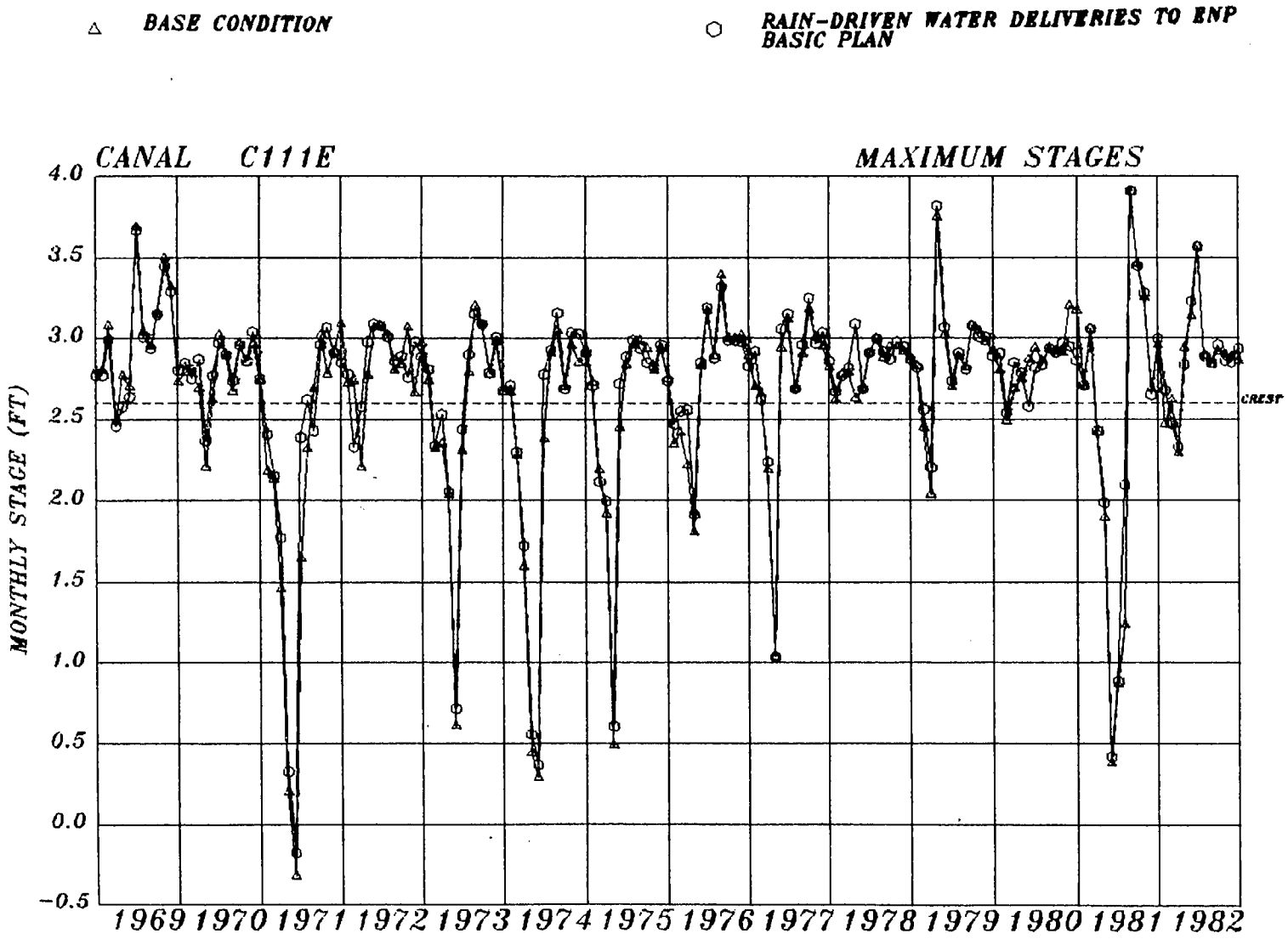
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
CANAL C111E
MAXIMUM STAGES
MONTHLY STAGE (FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-89

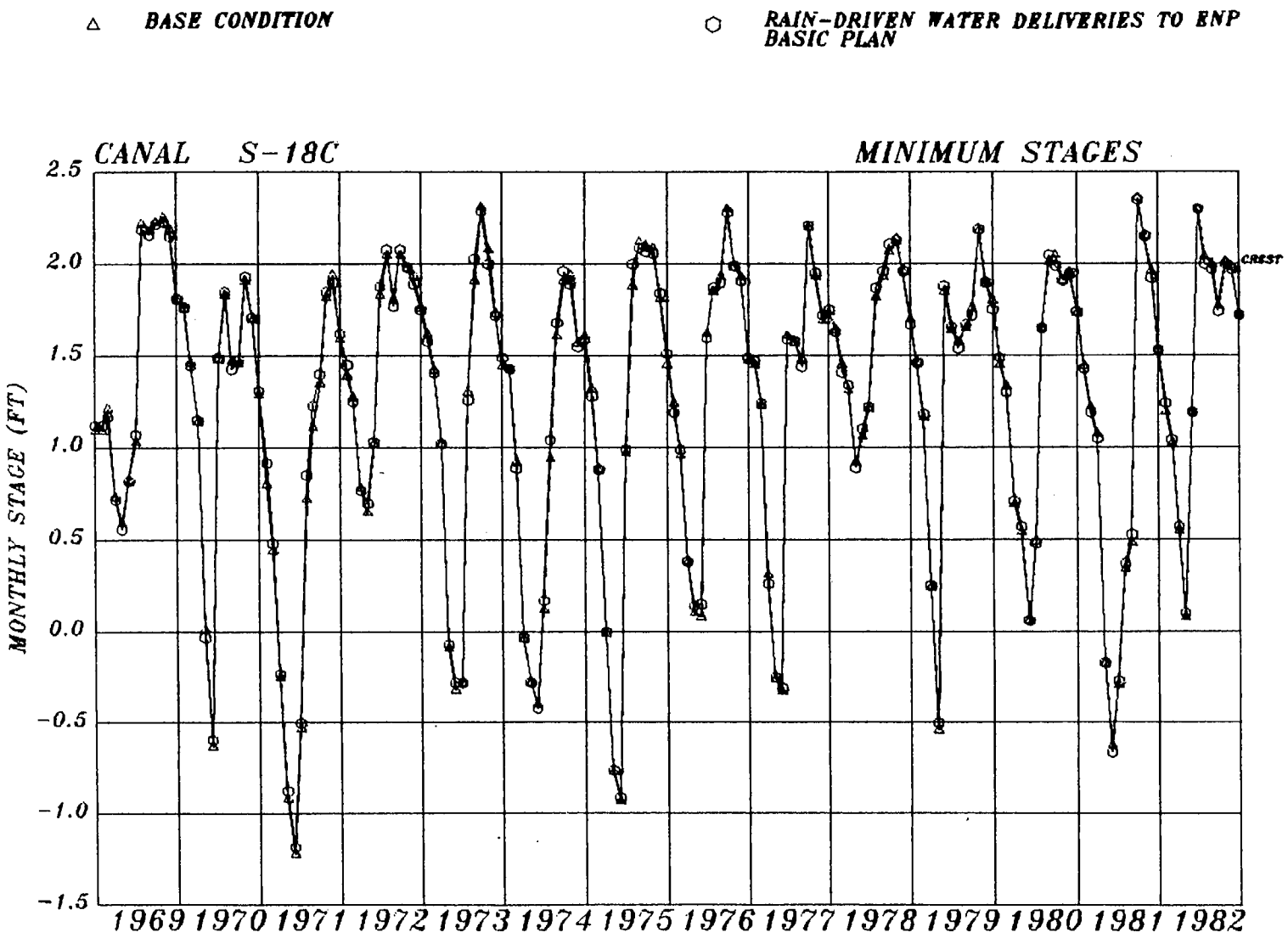
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
CANAL S-18C
MINIMUM STAGES
MONTHLY STAGE (FT)
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-90

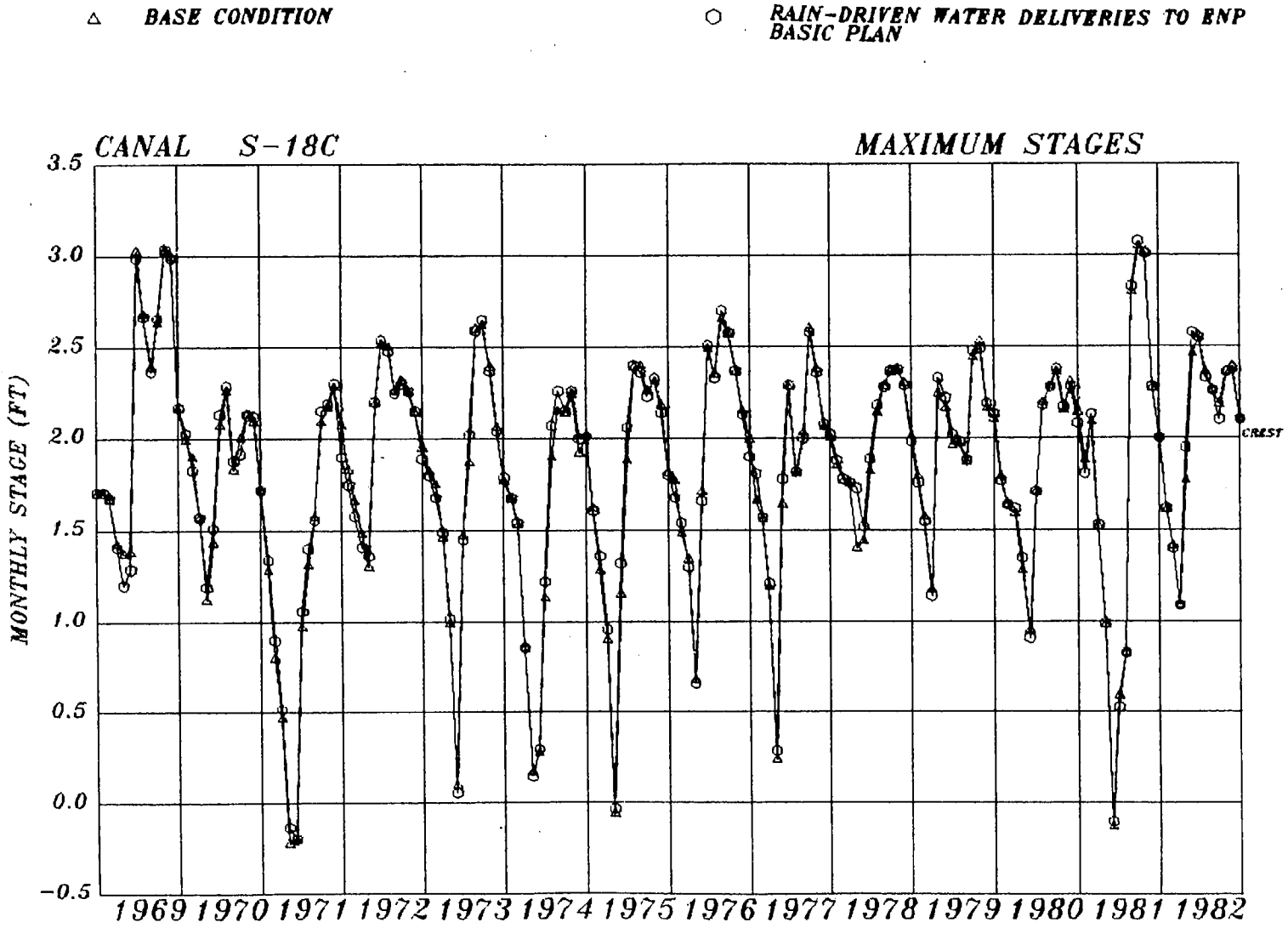
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
CANAL S-18C
MAXIMUM STAGES
MONTHLY STAGE (FT)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-91

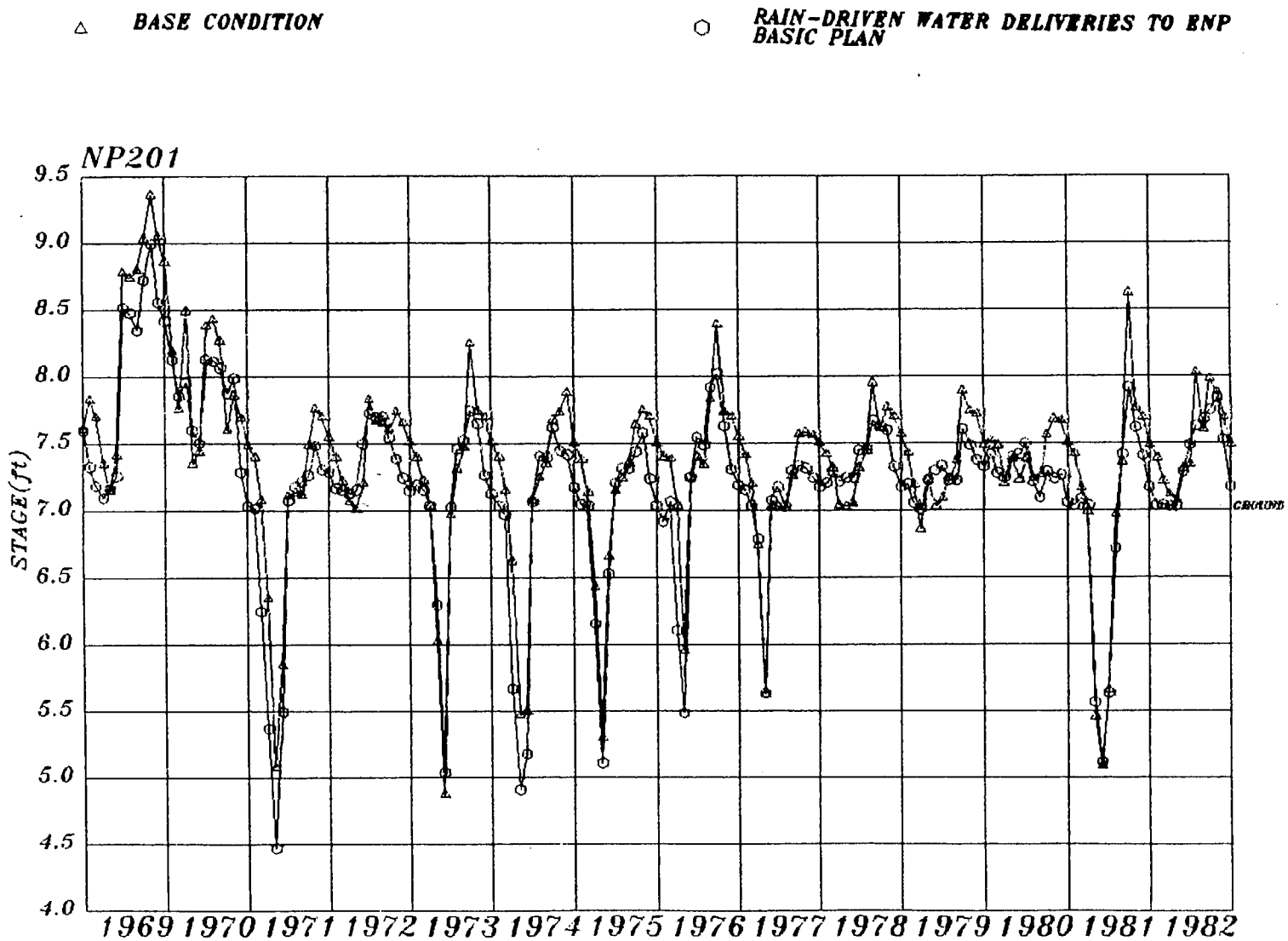
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO BNP BASIC PLAN
NP201
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-92

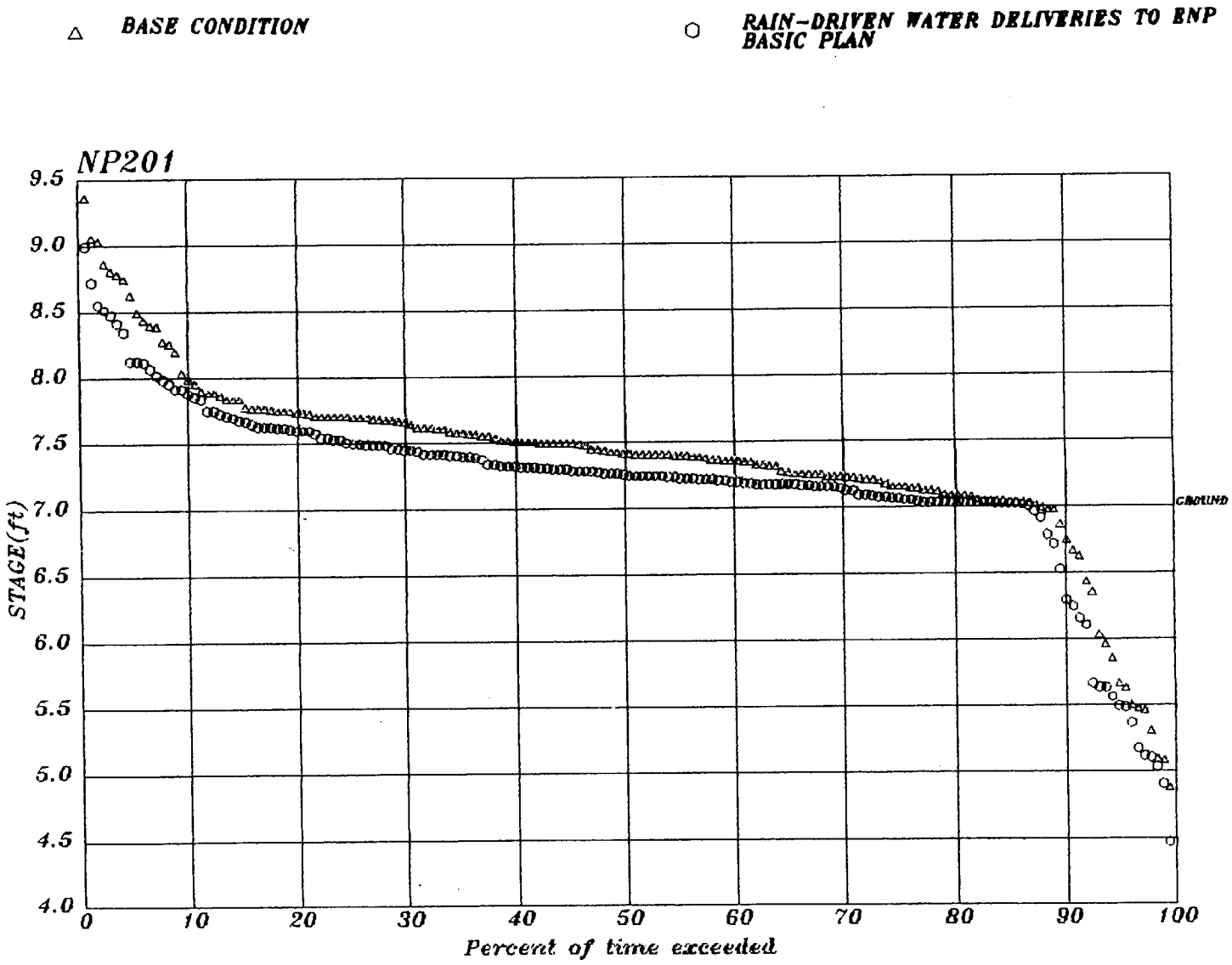
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NP201
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-93

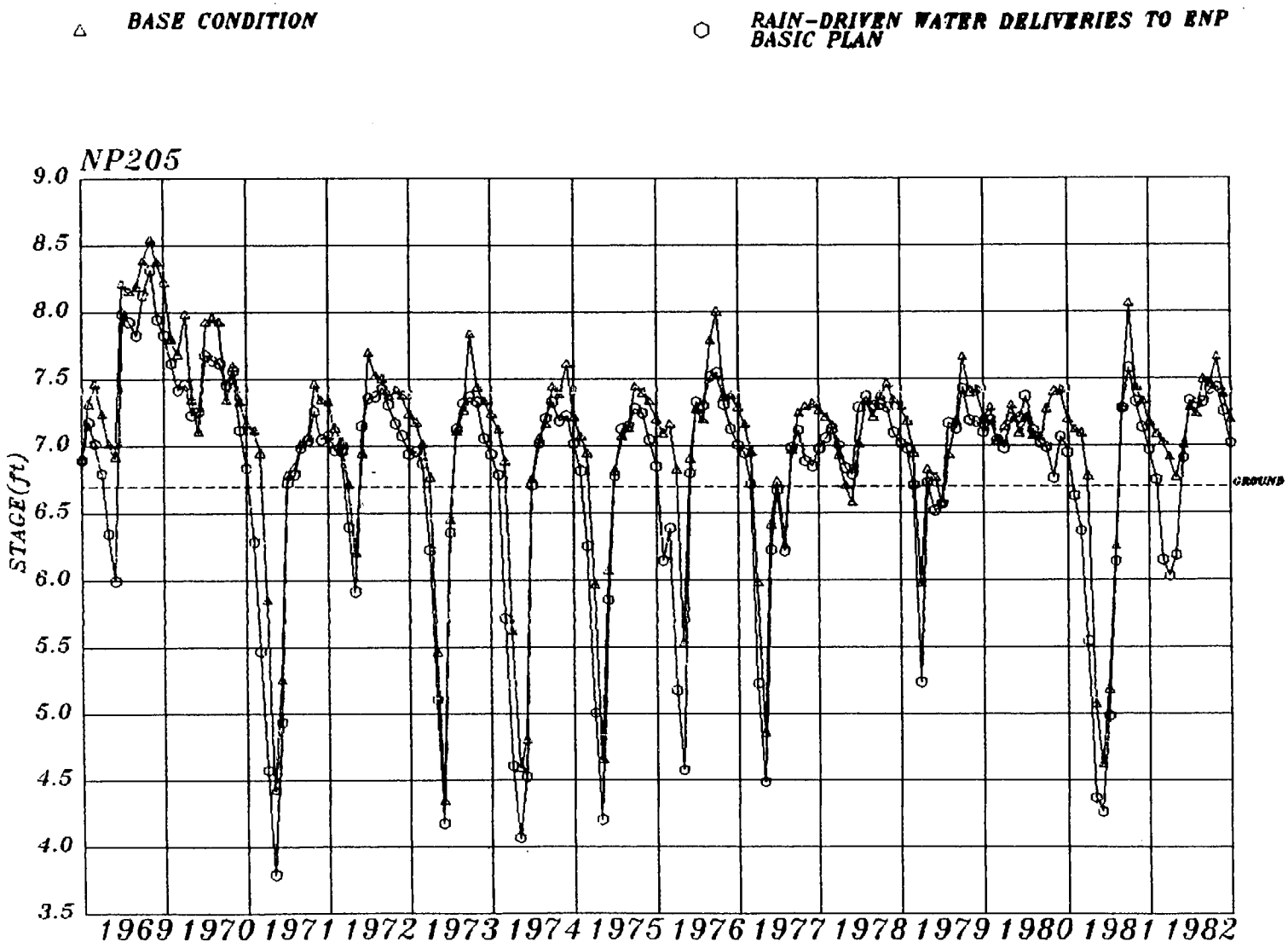
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NP205
STAGE(ft)
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-94

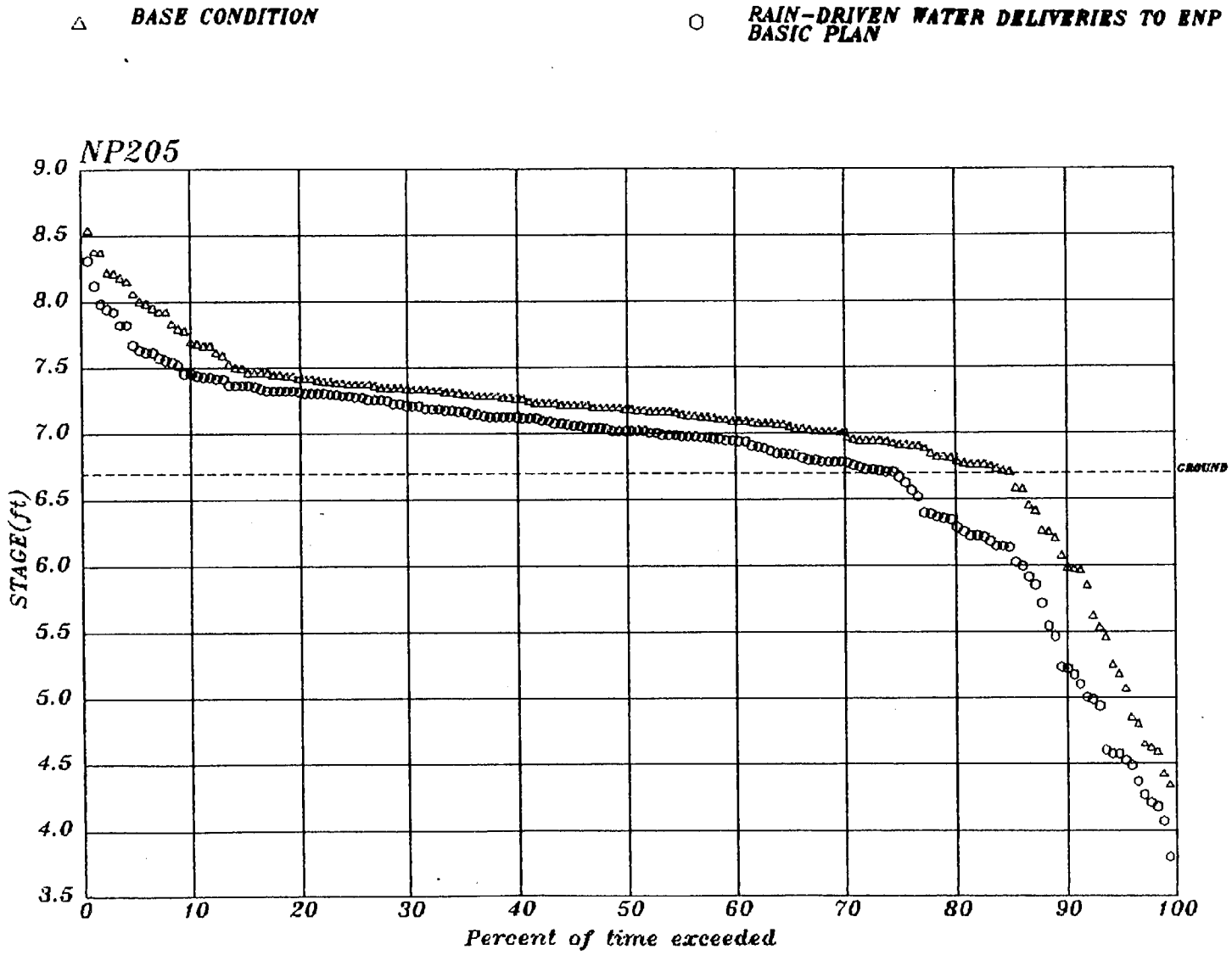
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NP205
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-95

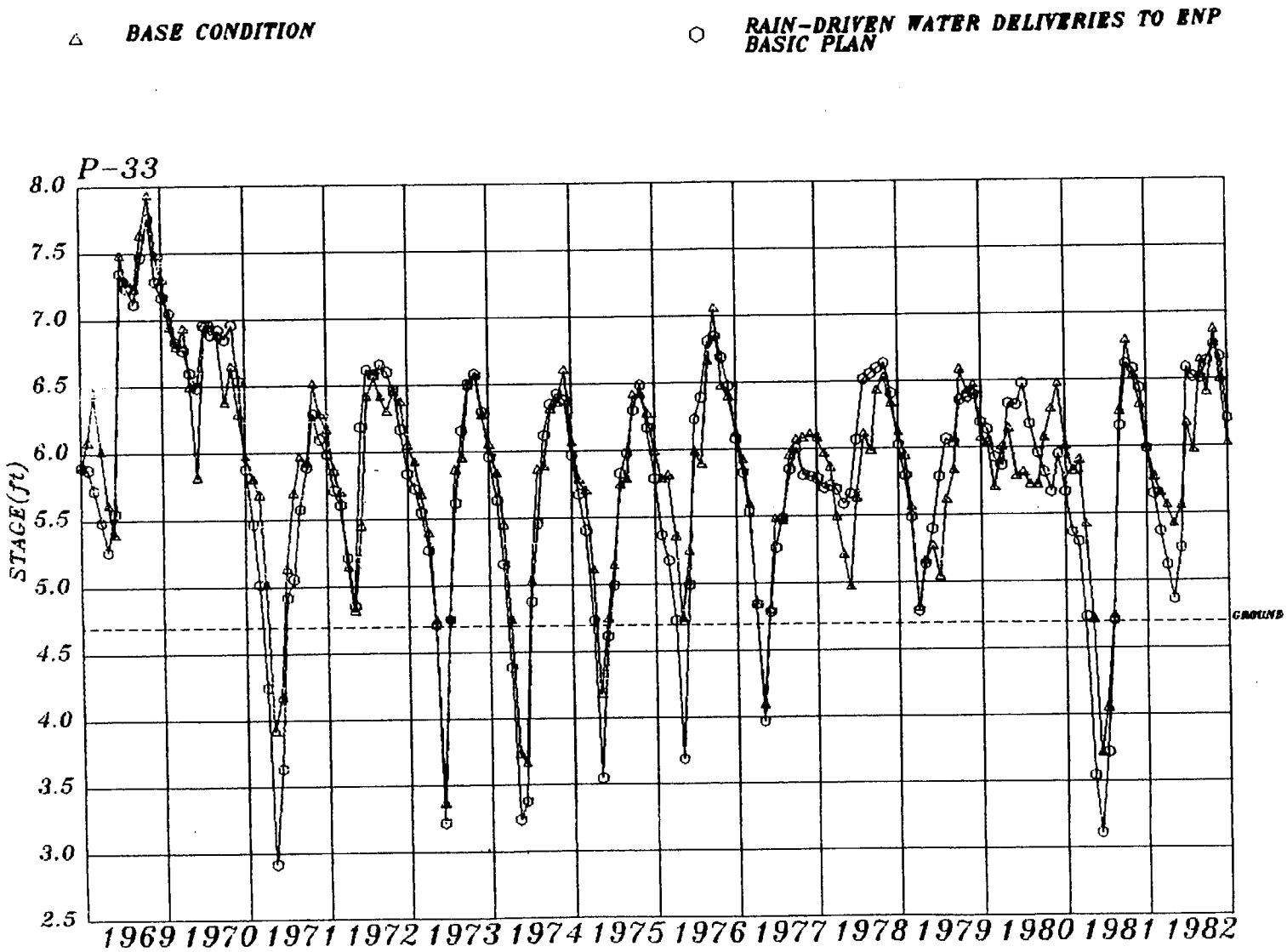
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
P-33
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-96

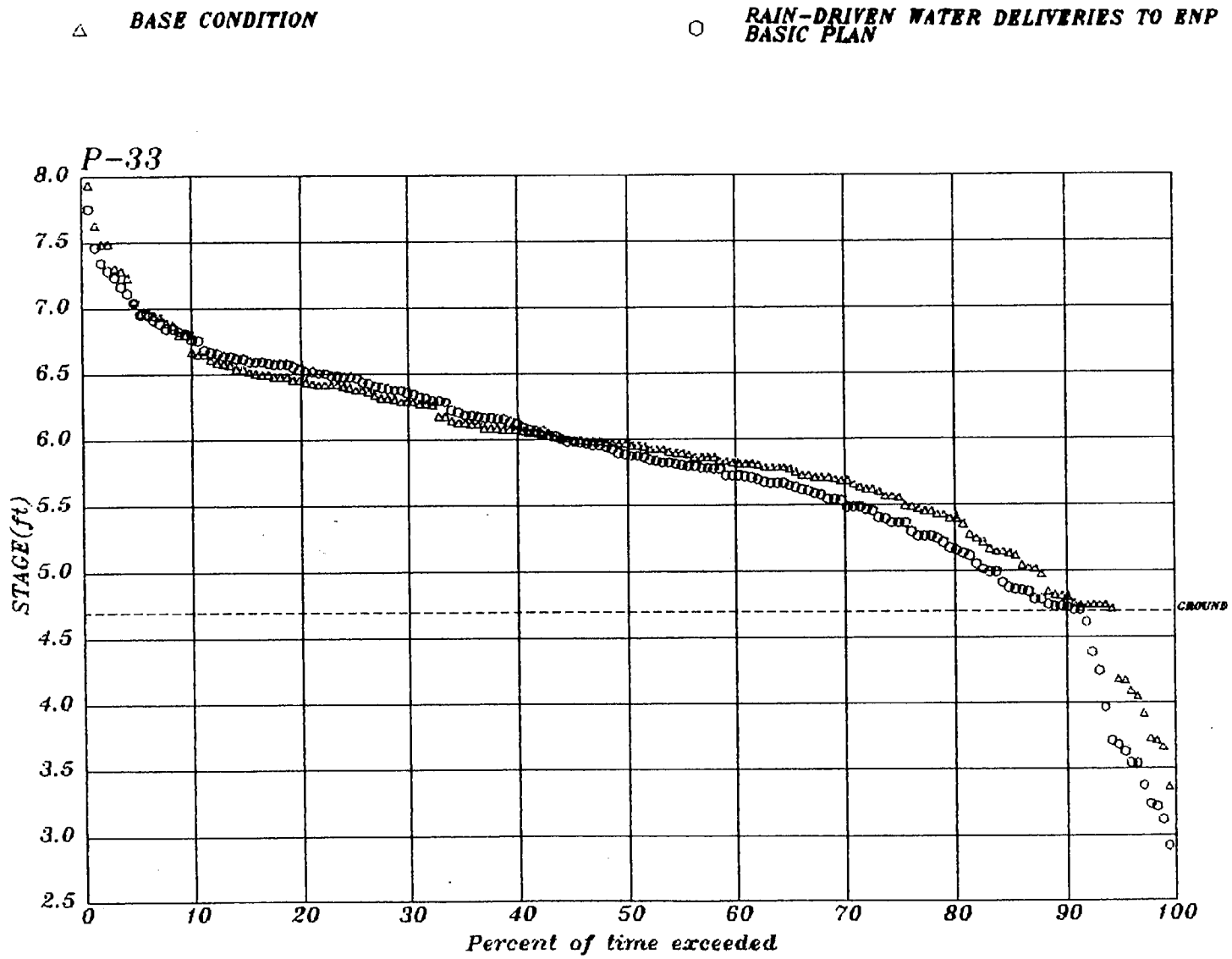
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
P-33
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-97

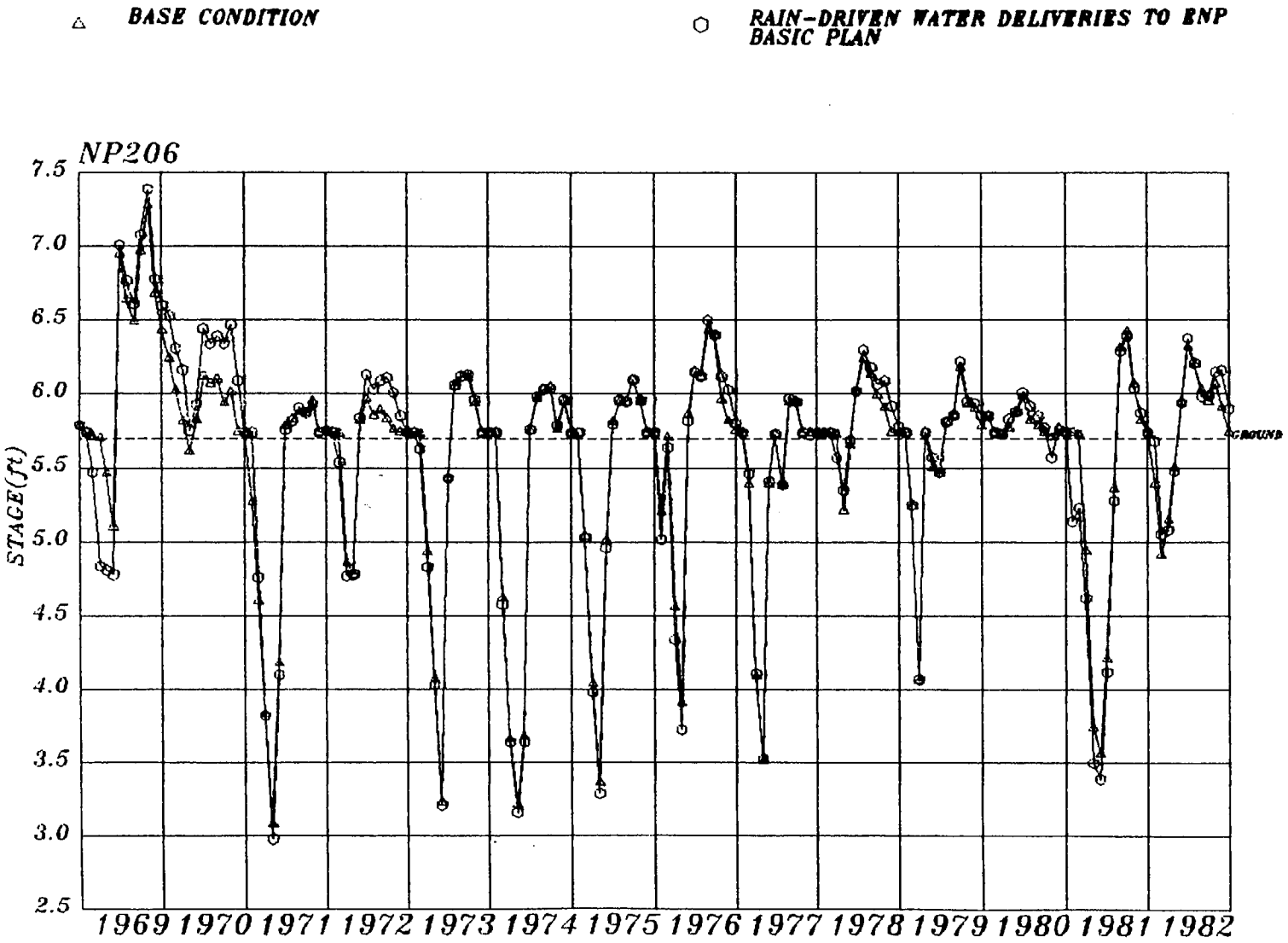
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NP206
STAGE(ft)
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-98

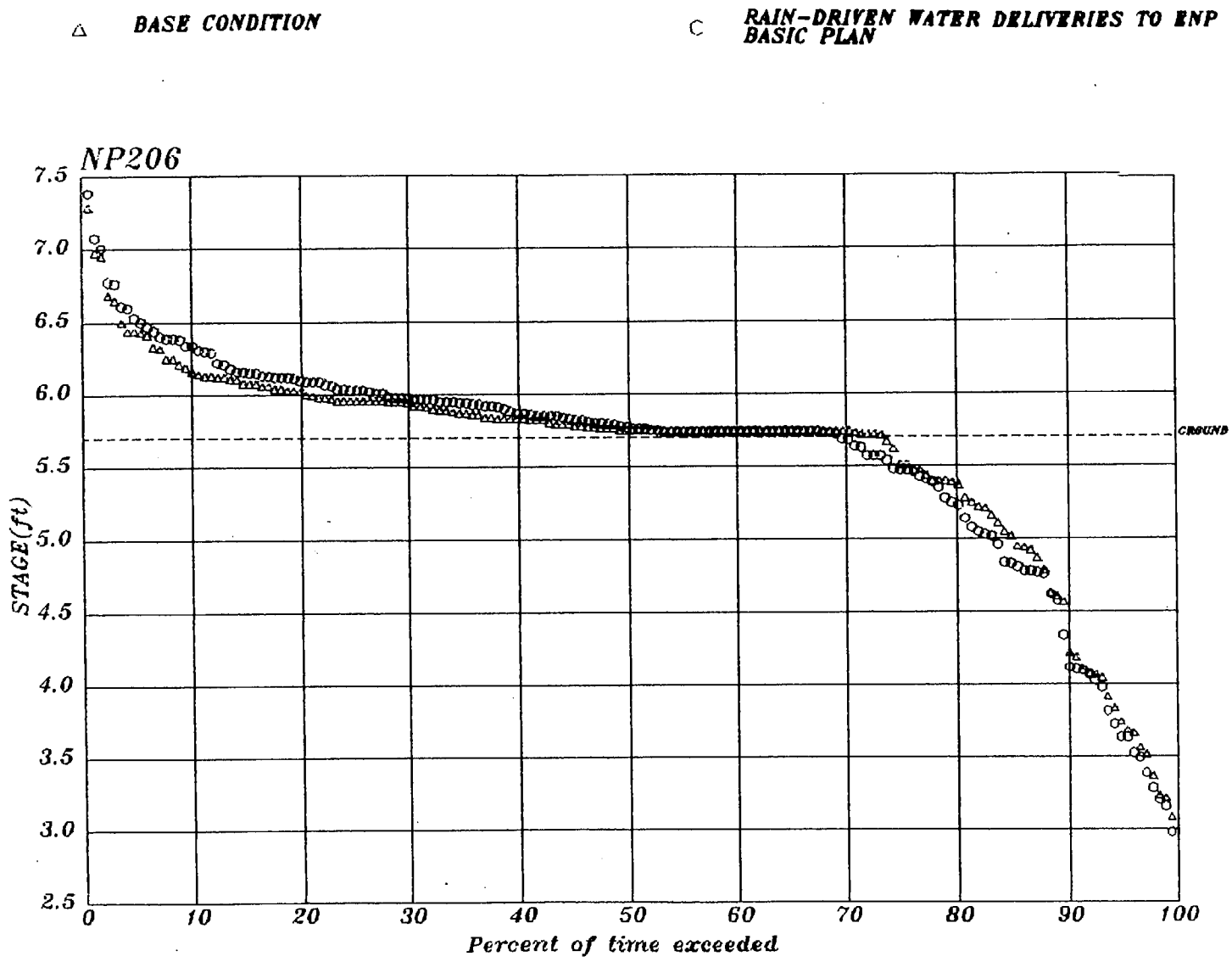
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NP206
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-99

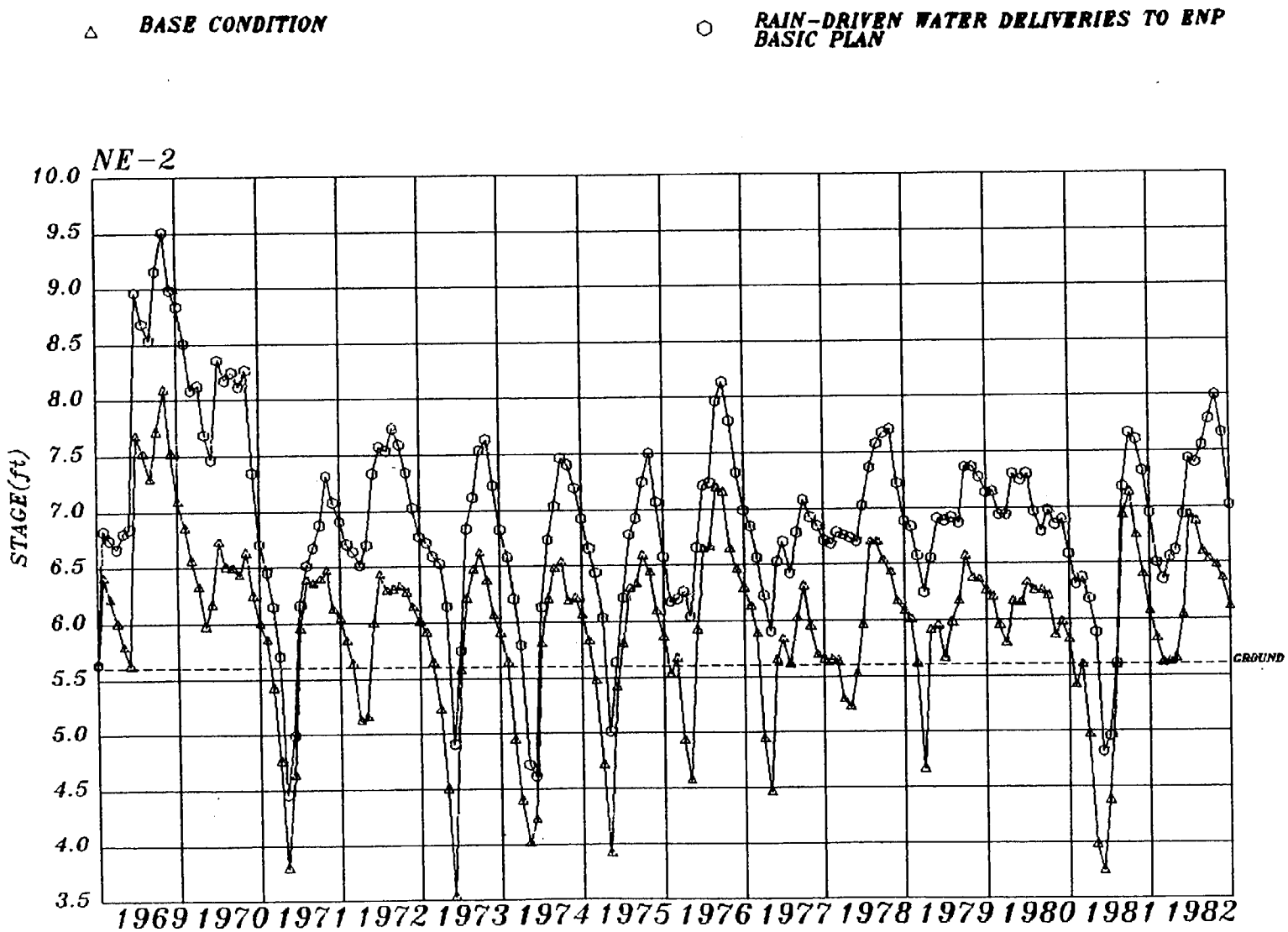
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NE-2
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-100

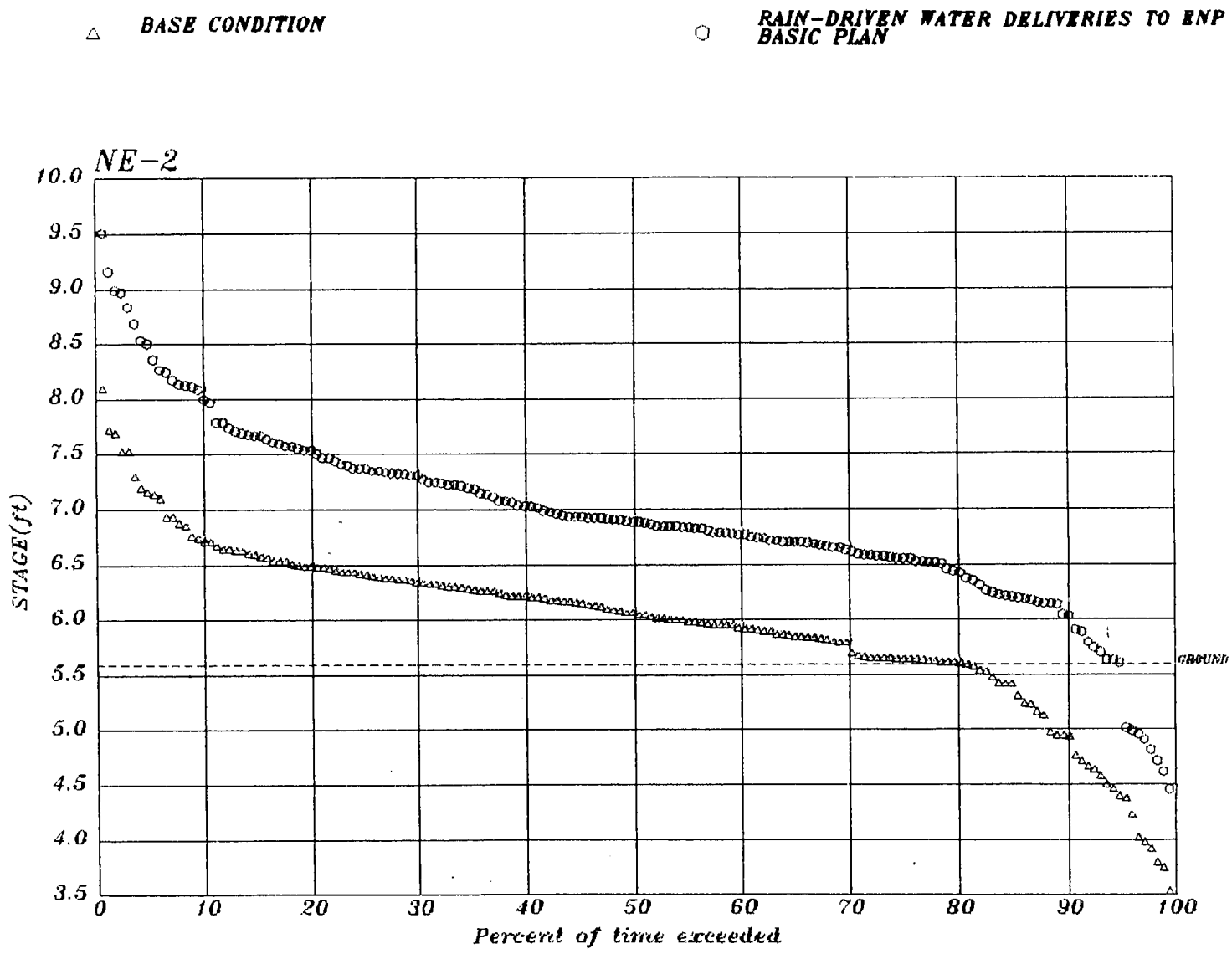
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
NE-2
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-101

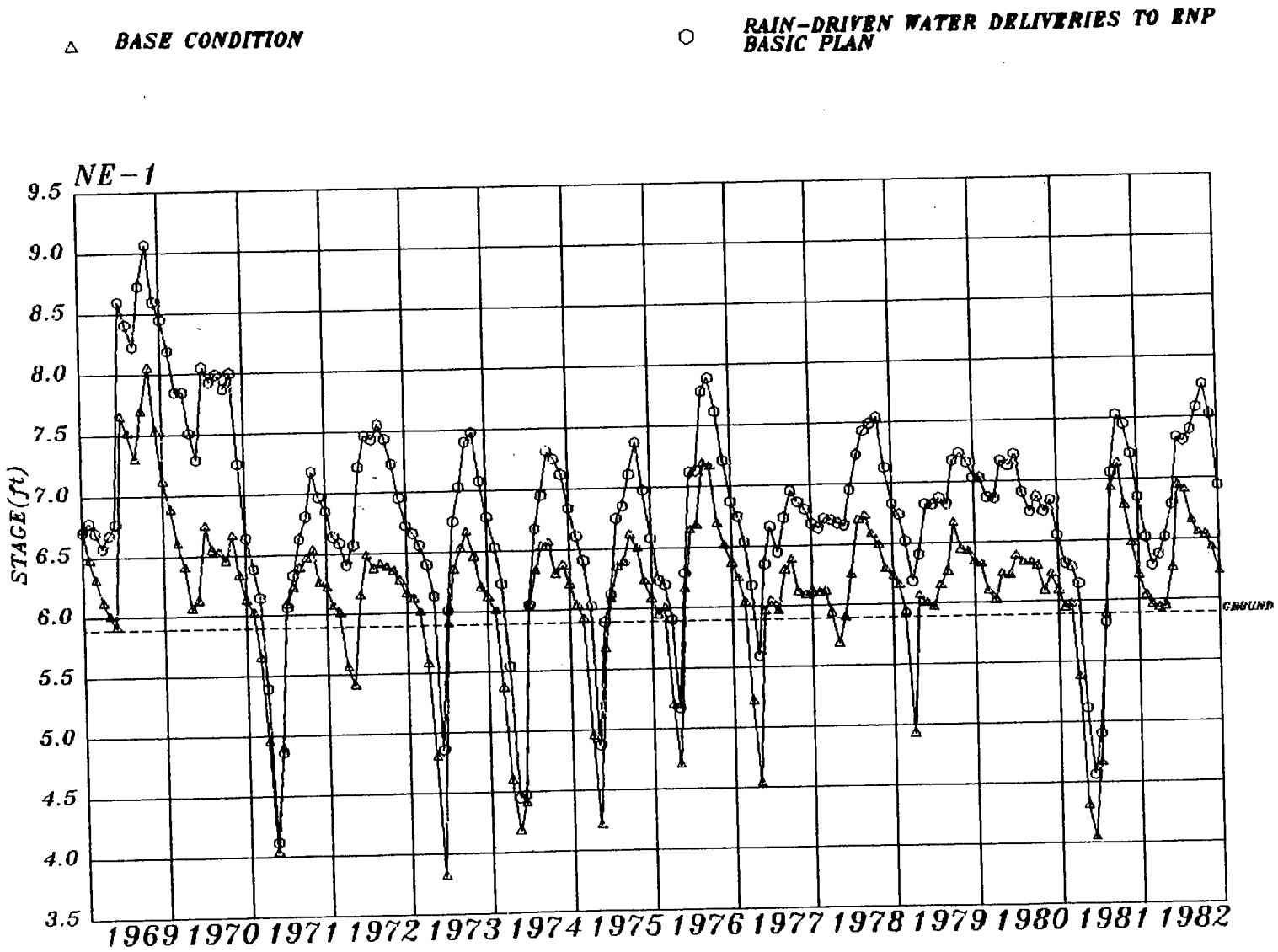
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO BNP BASIC PLAN
NE-1
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-102

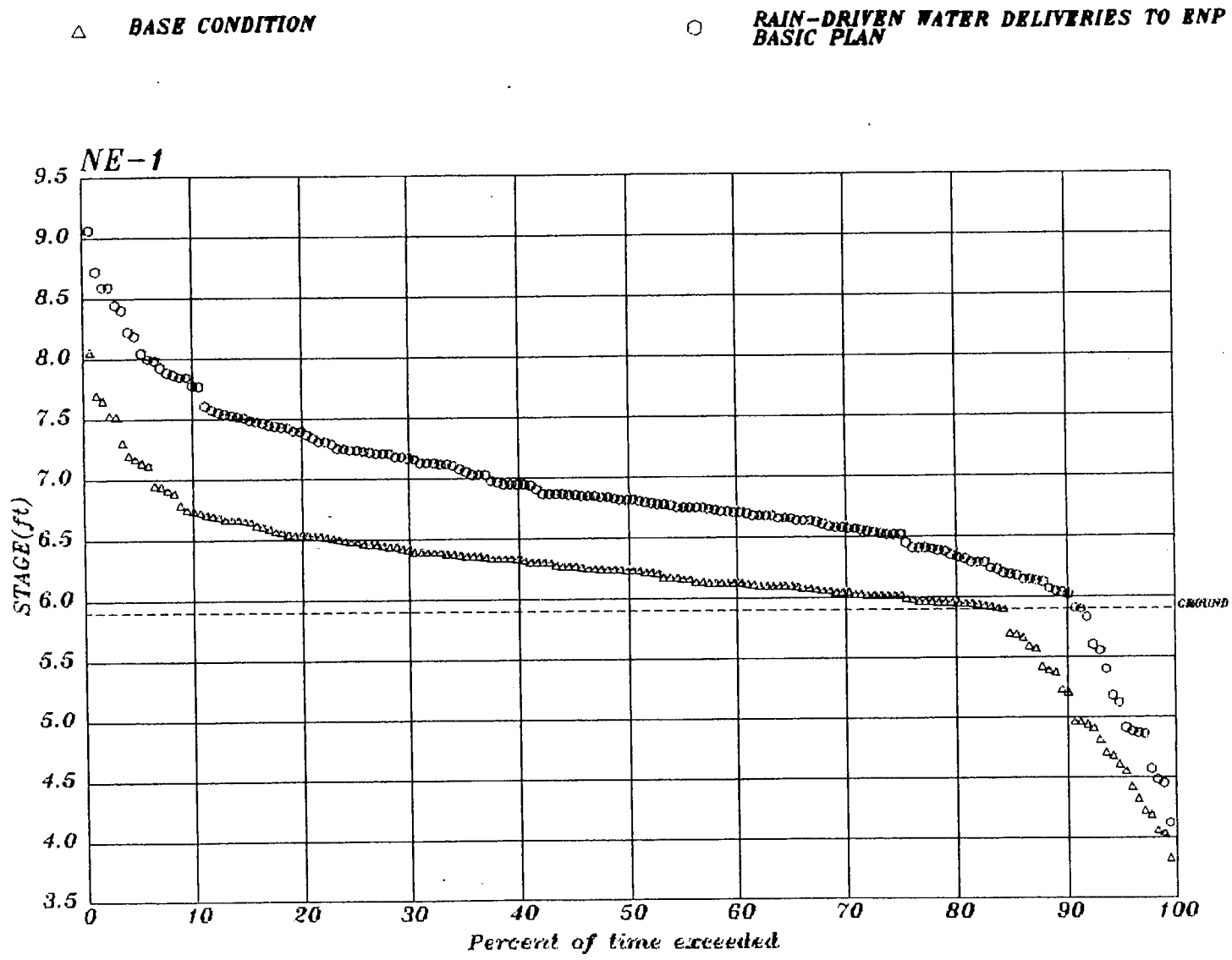
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
NE-1
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-103

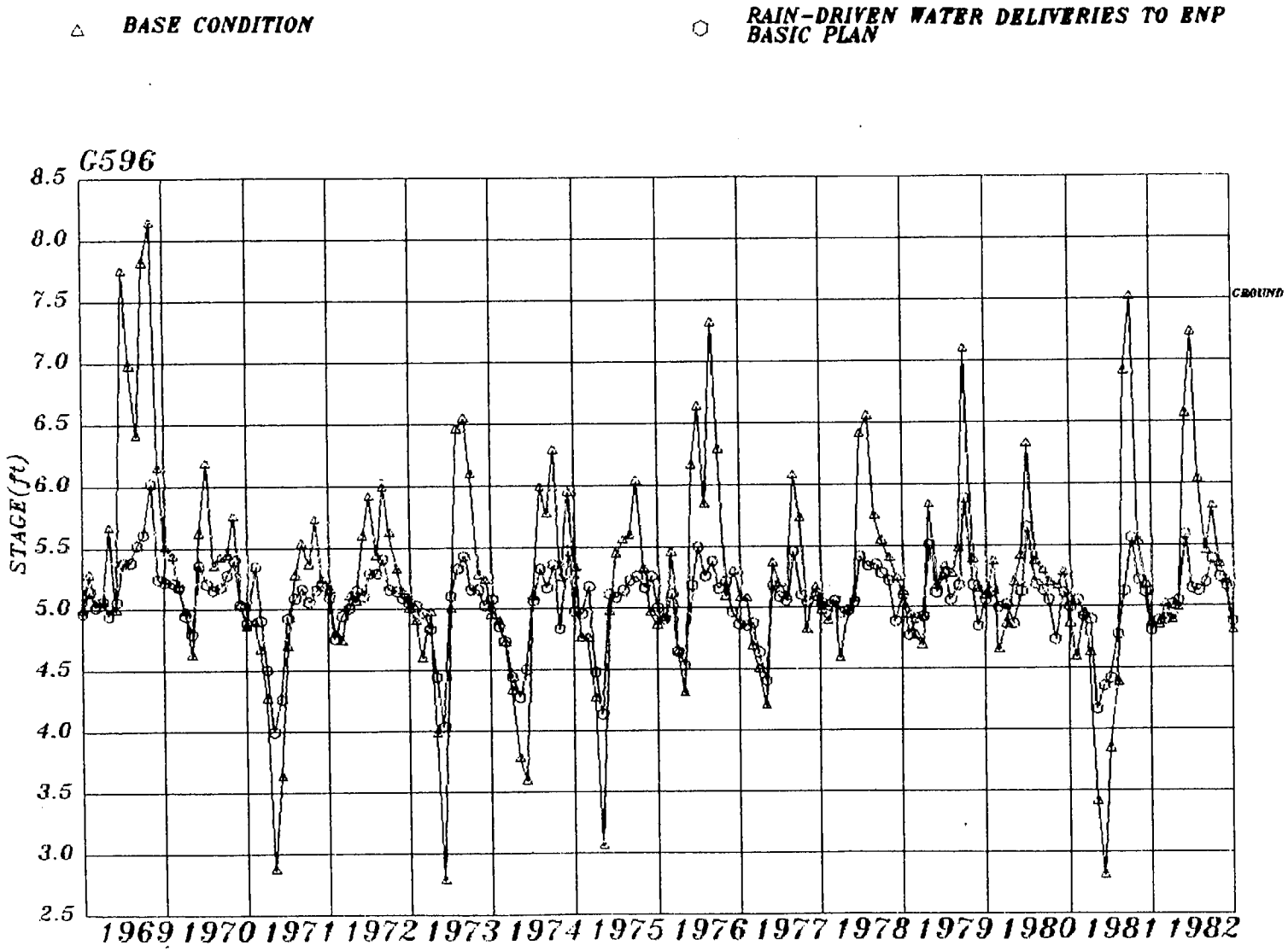
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
G596
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-104

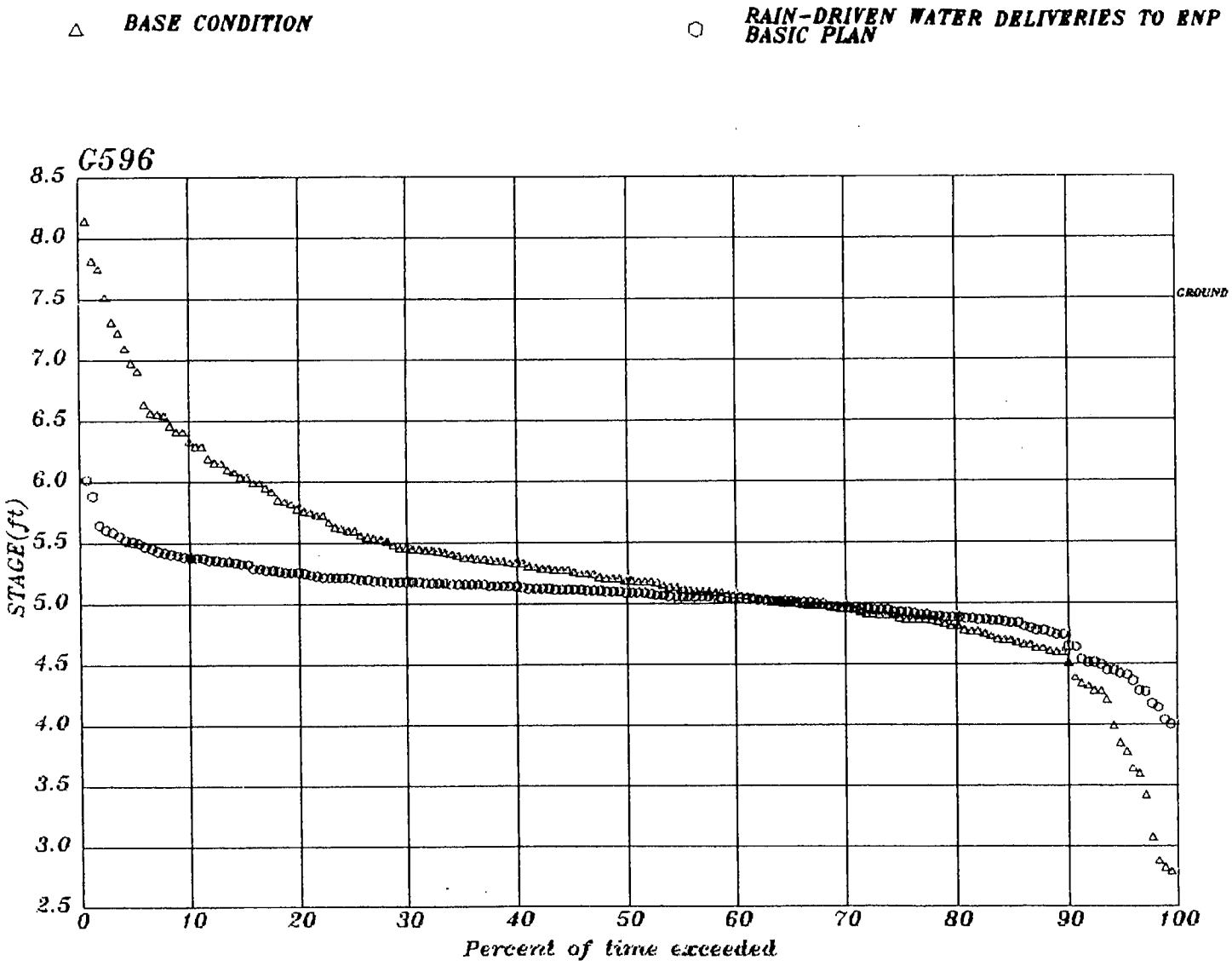
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
G596
GROUND
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-105

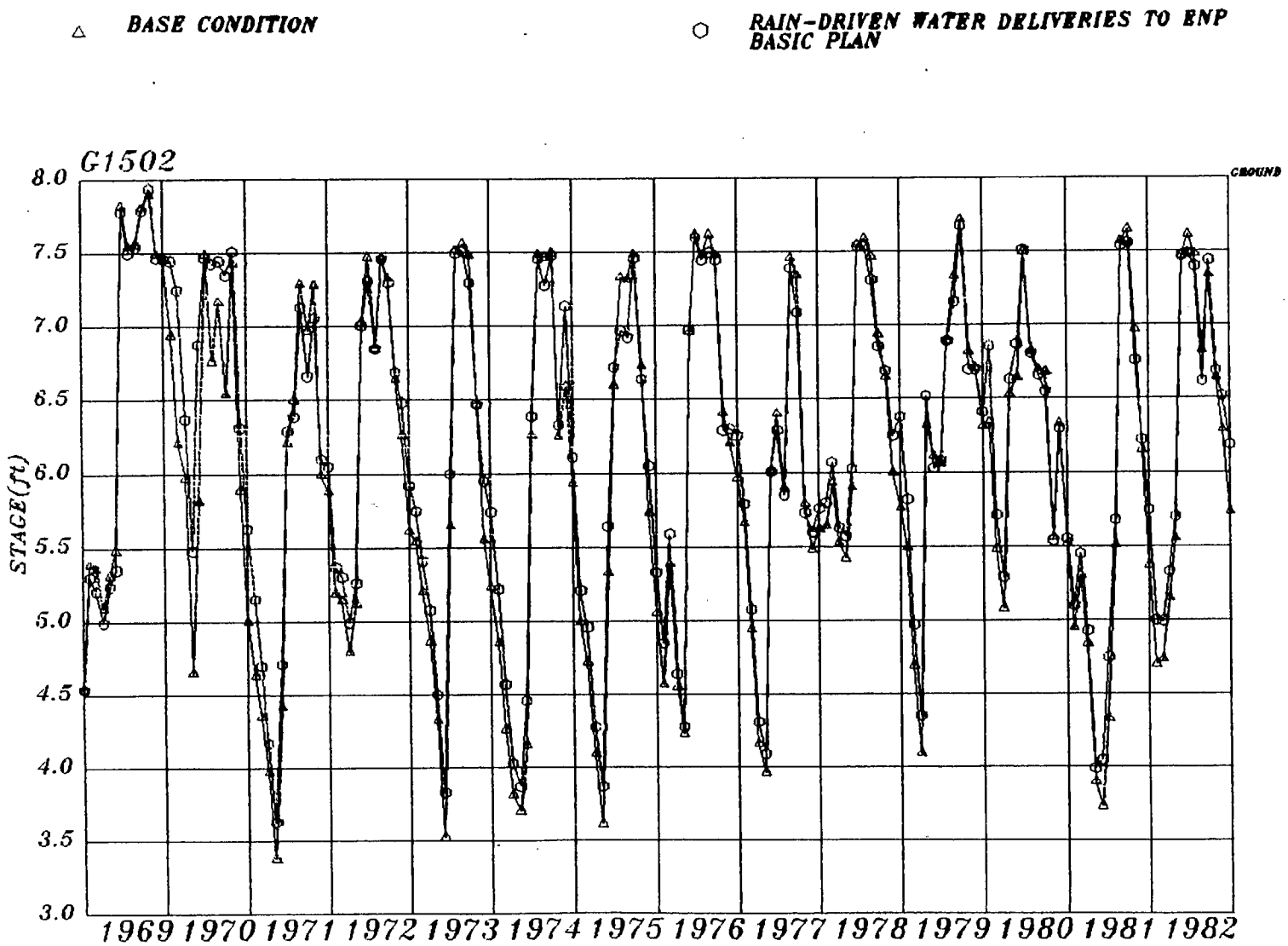
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
G1502
GROUND
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-106

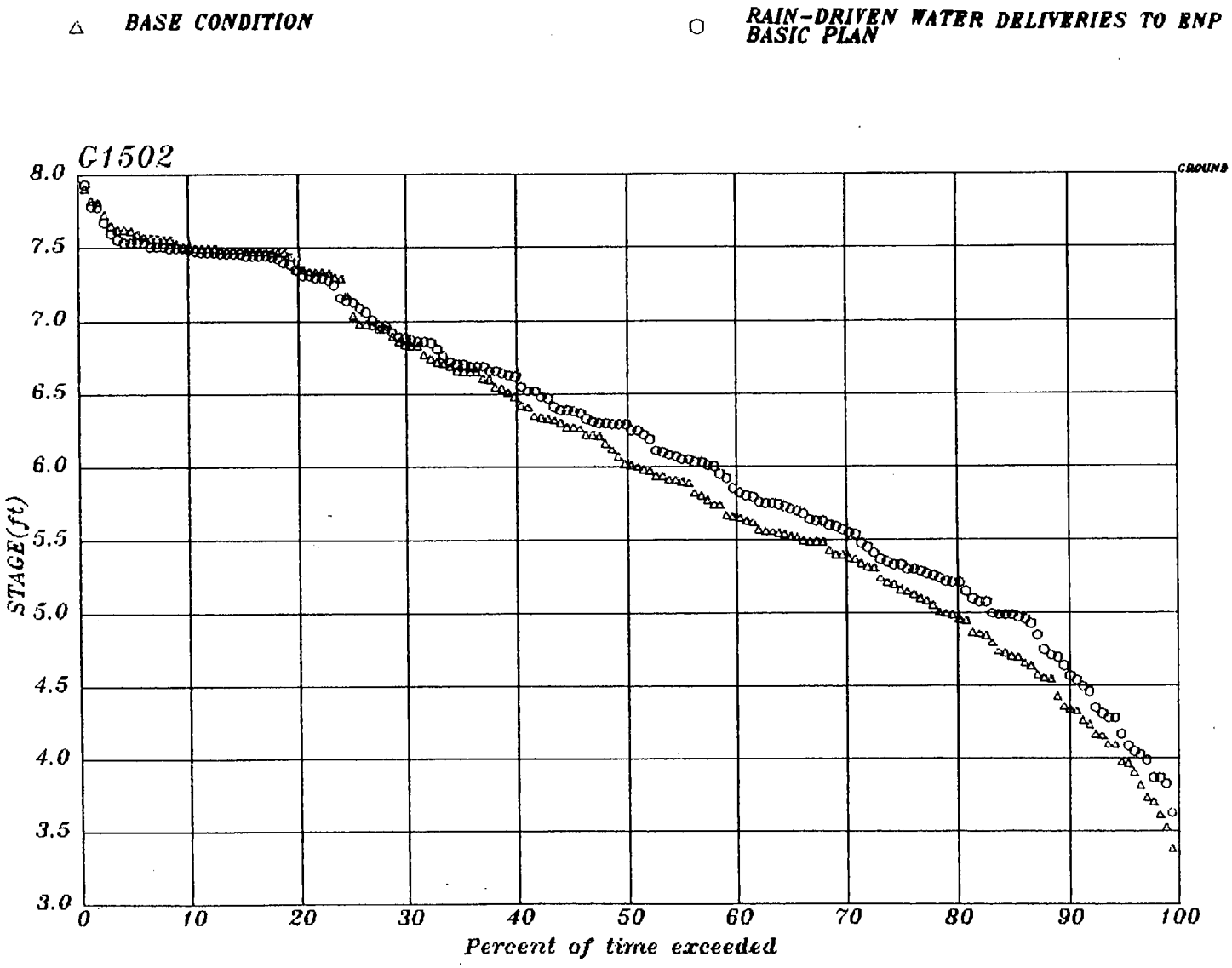
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP BASIC PLAN
G1502
GROUND
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-107

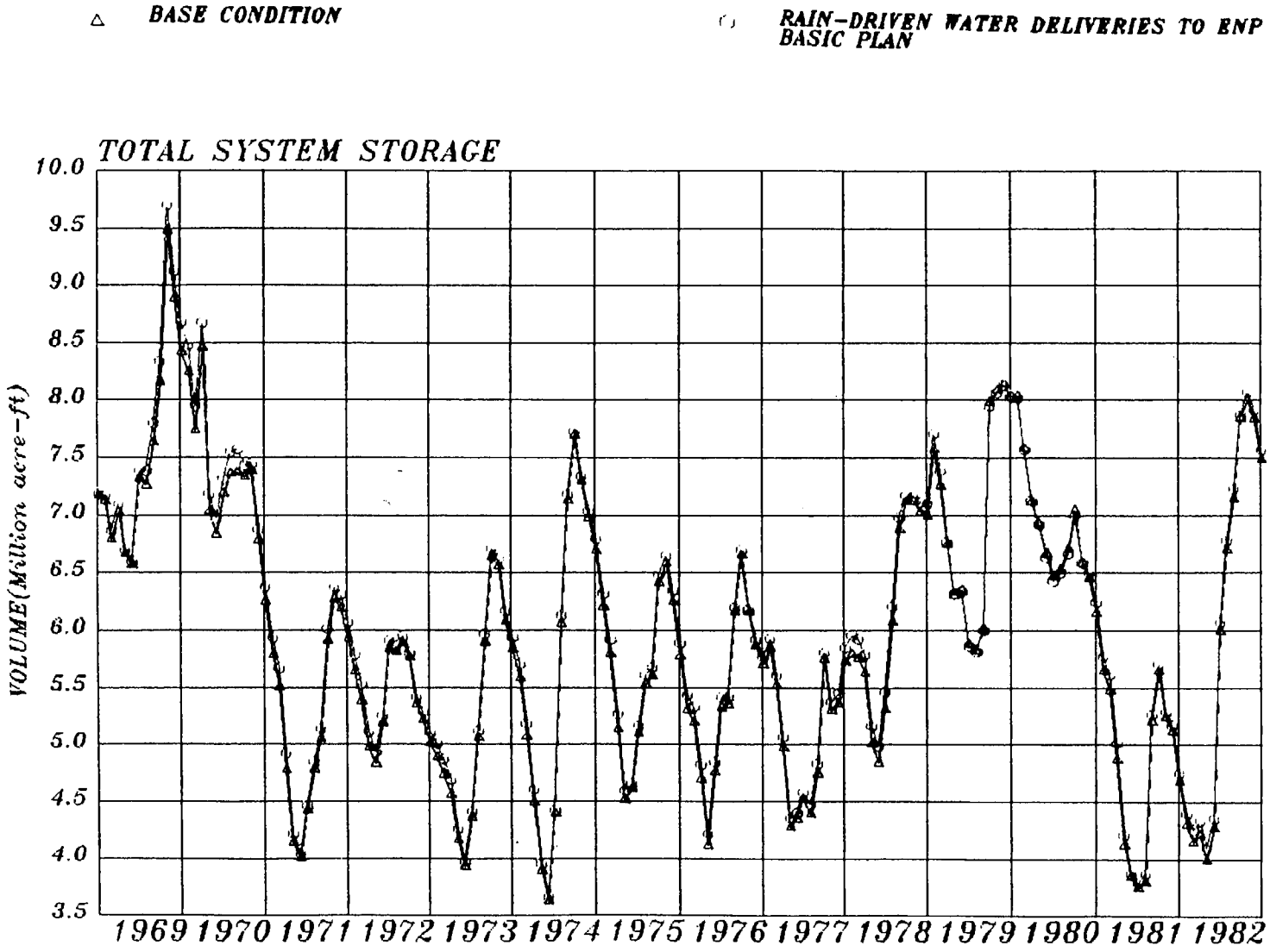
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
BASIC PLAN
TOTAL SYSTEM STORAGE
VOLUME(Million acre-ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-108

△ BASE CONDITION

RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN

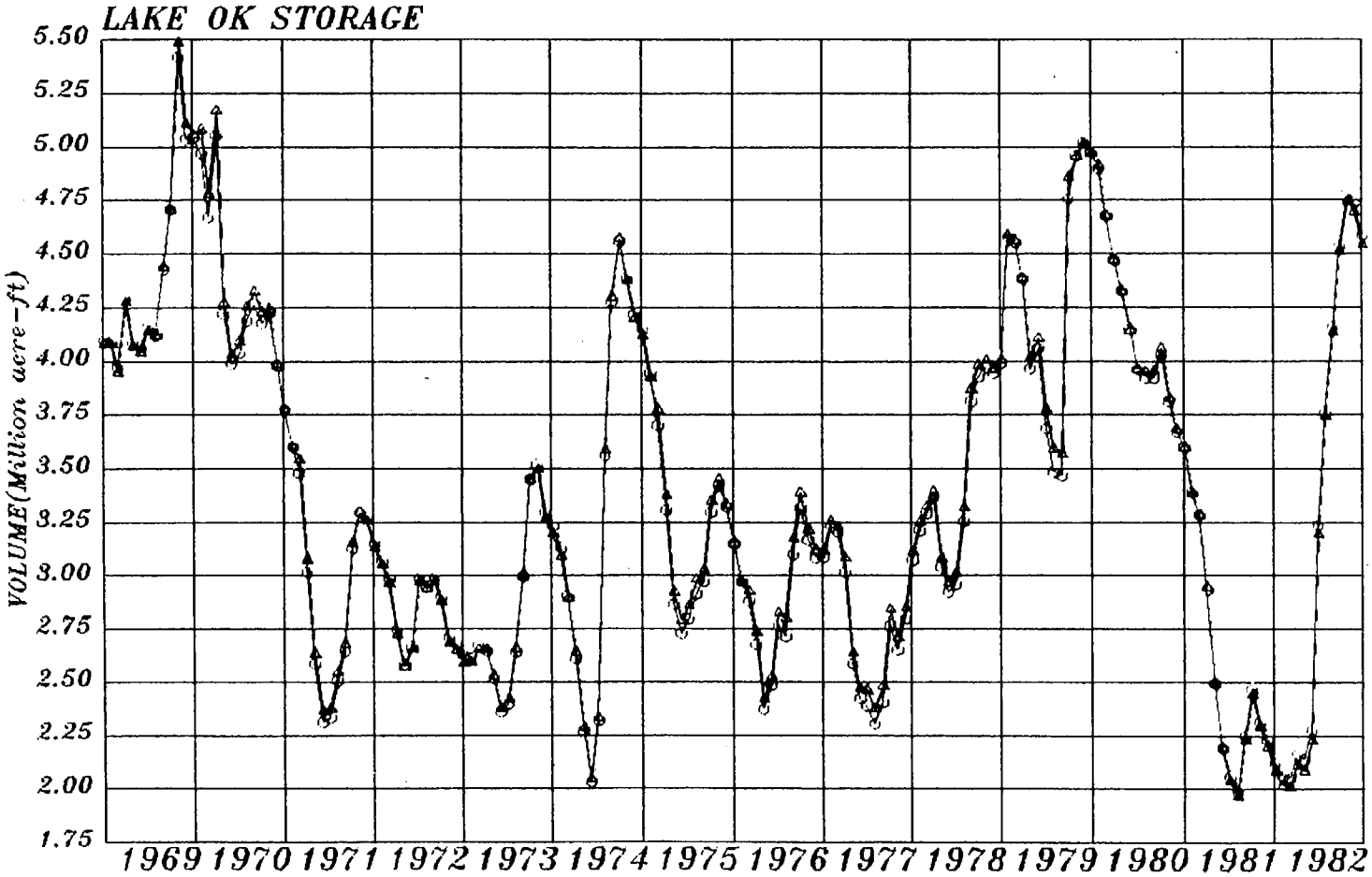

Figure E-109

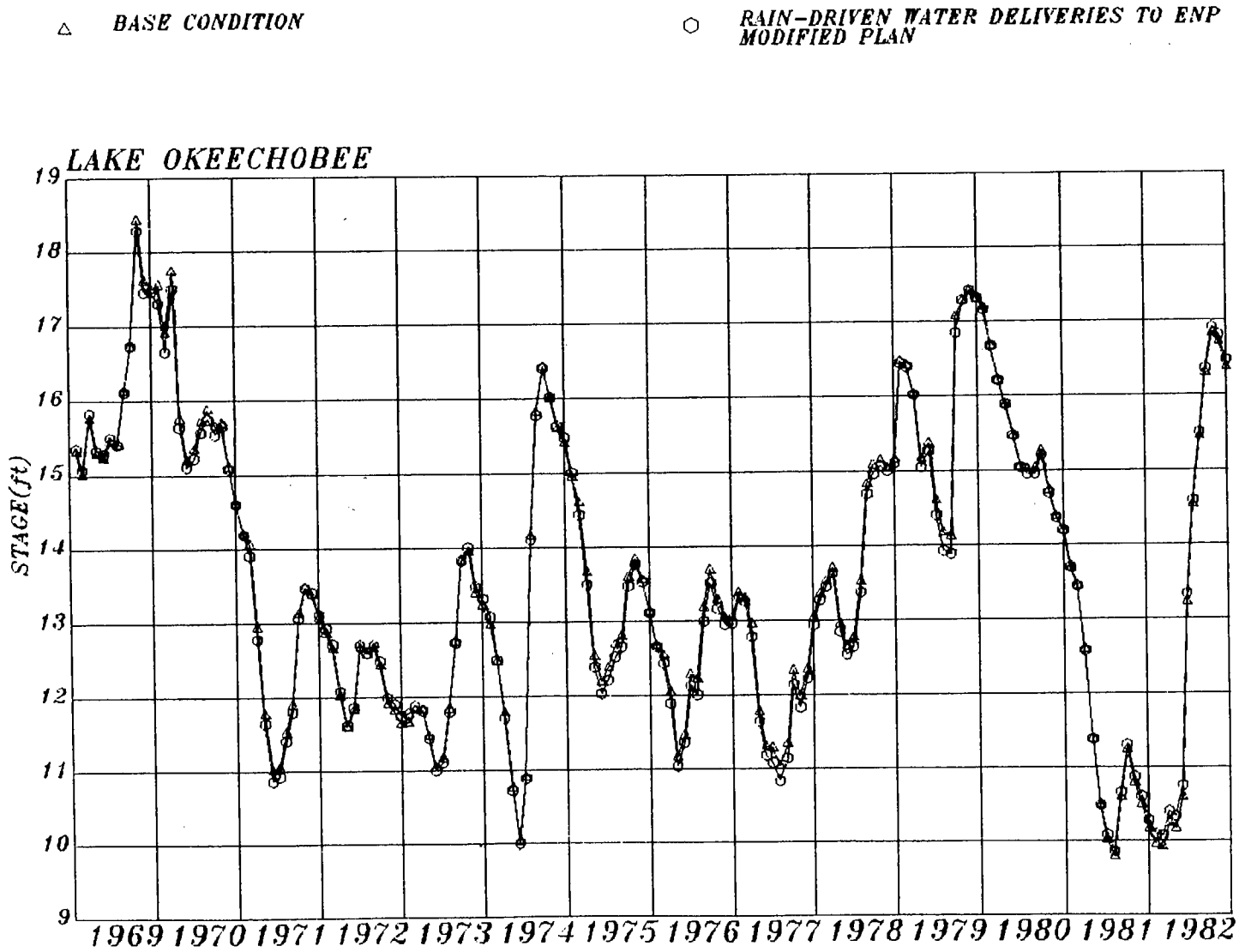
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
LAKE OKEECHOBEE
STAGE(ft)
19
18
17
16
15
14
13
12
11
10
9
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-110

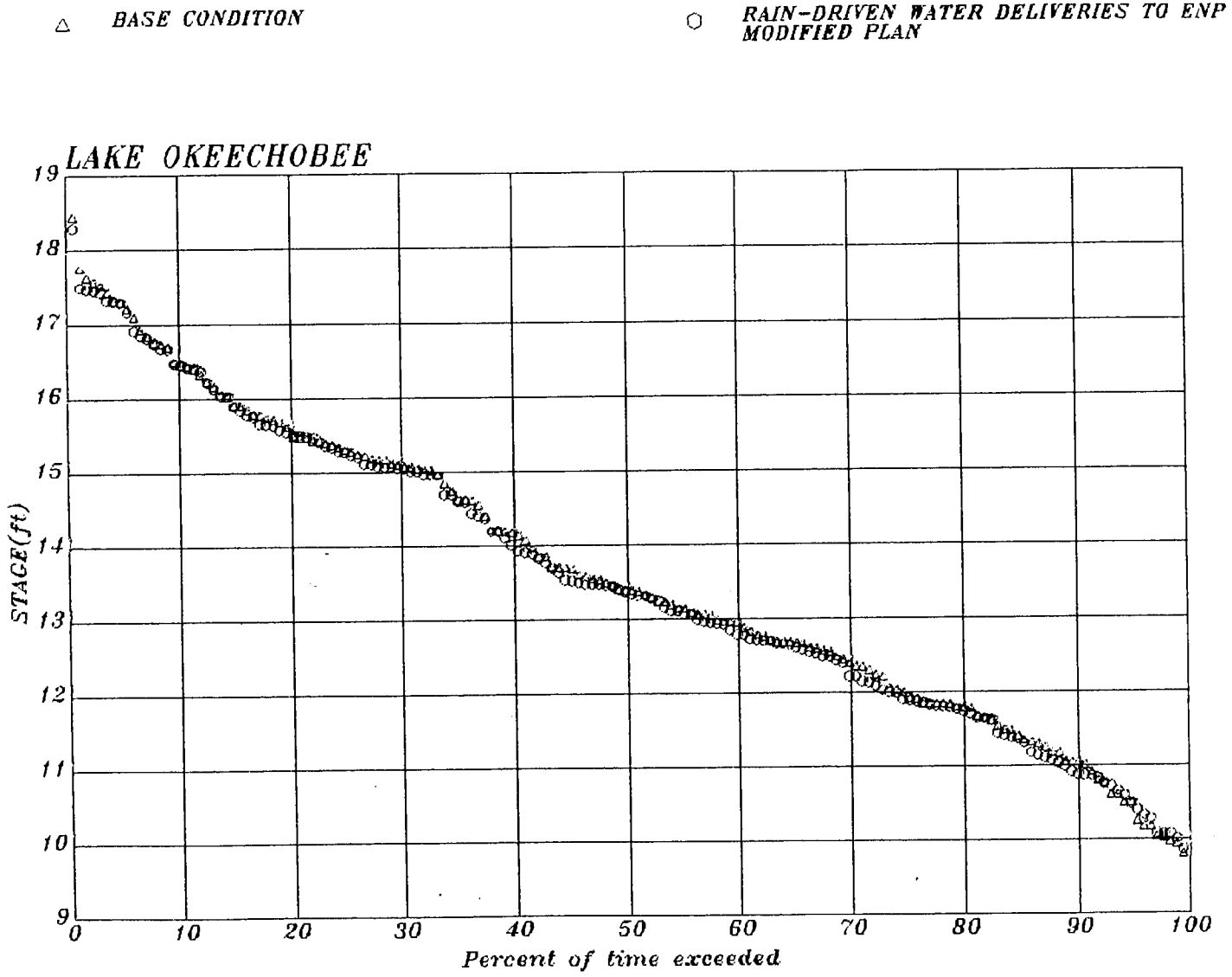
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
LAKE OKEECHOBEE
STAGE(ft)
19
18
17
16
15
14
13
12
11
10
9
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-111

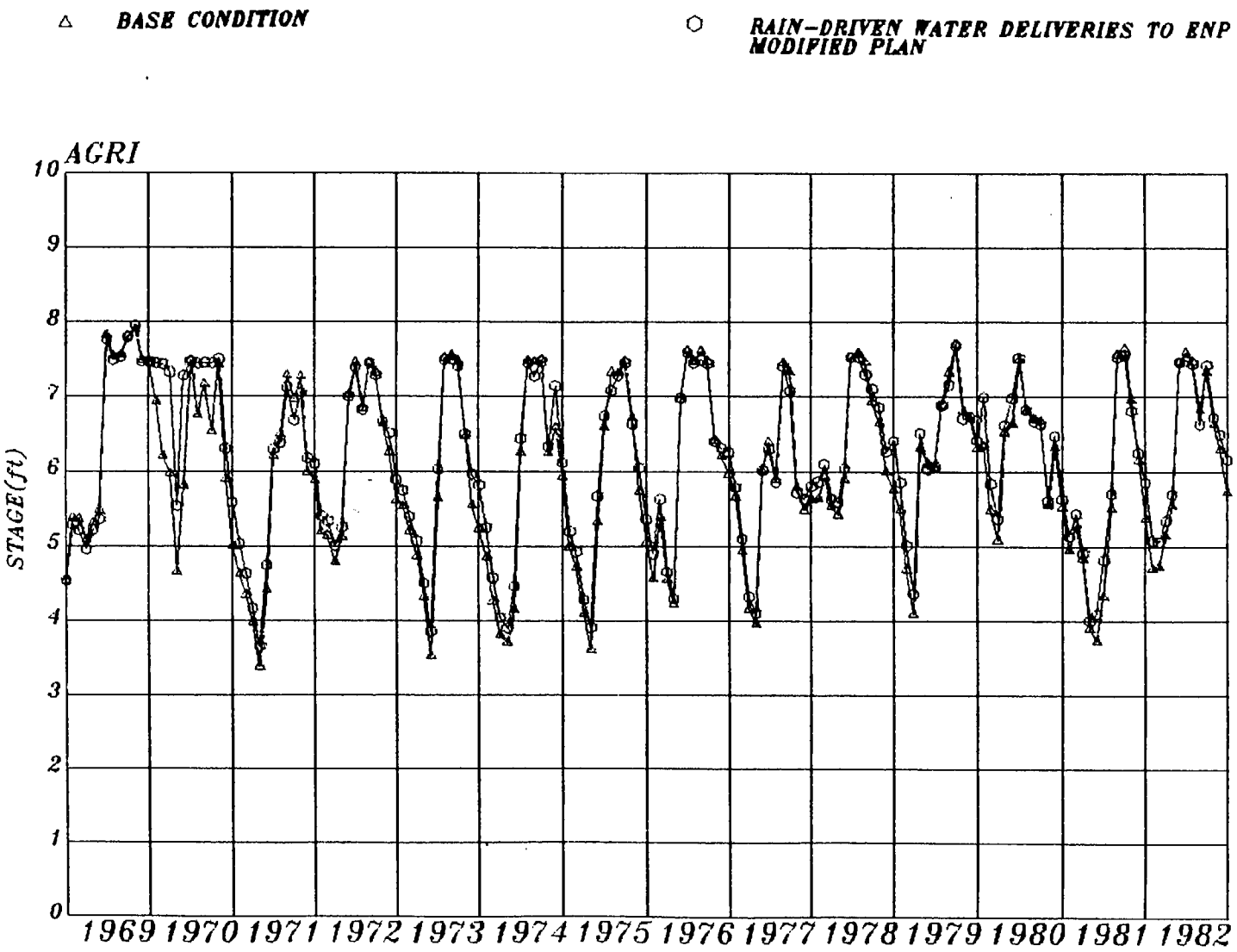
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
AGRI
STAGE(ft)
10
9
8
7
6
5
4
3
2
1
0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-112

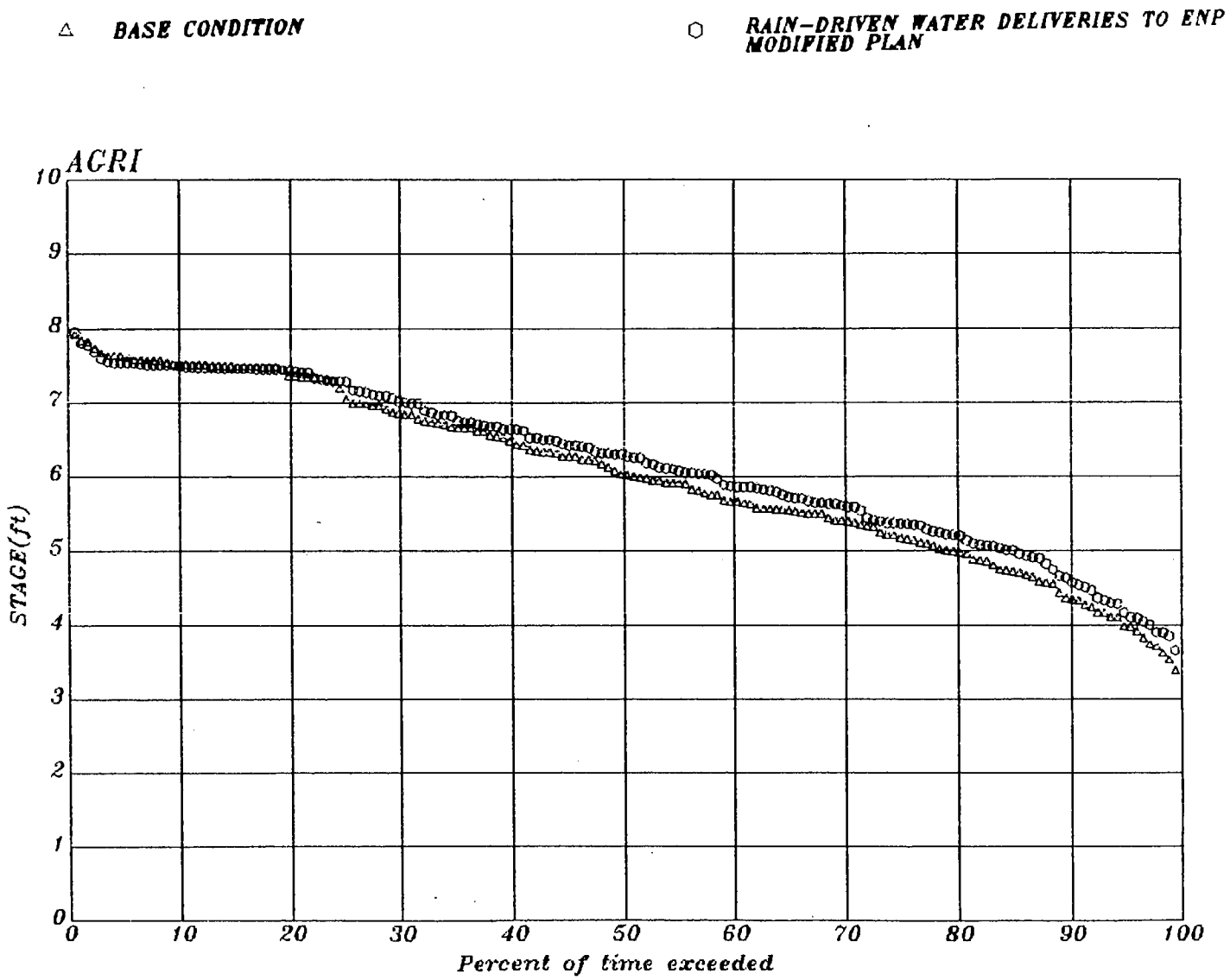
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
ACRI
STAGE(ft)
10
9
8
7
6
5
4
3
2
1
0
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-113

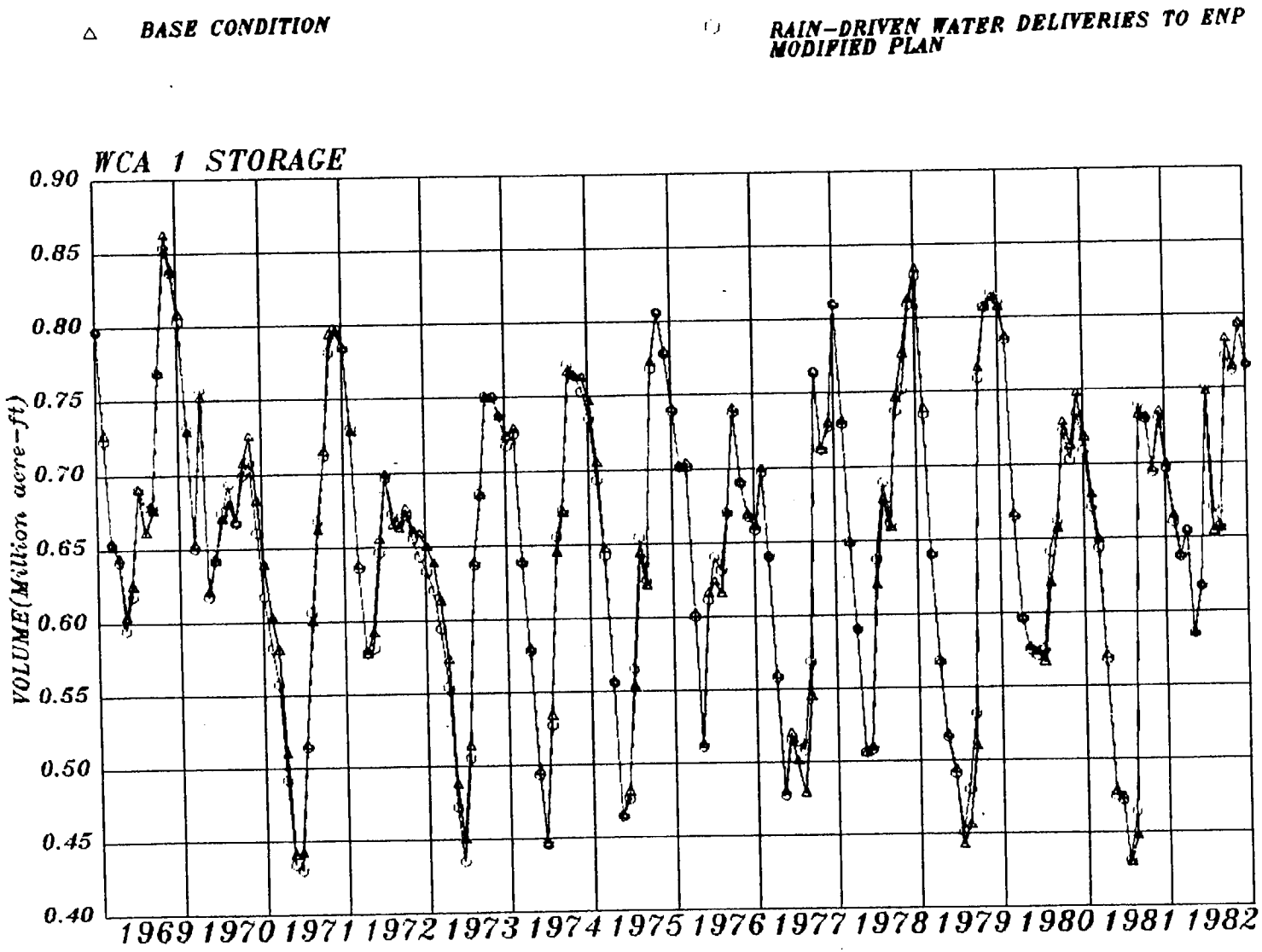
△ BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
WCA 1 STORAGE
VOLUME(Million acre-ft)
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-114

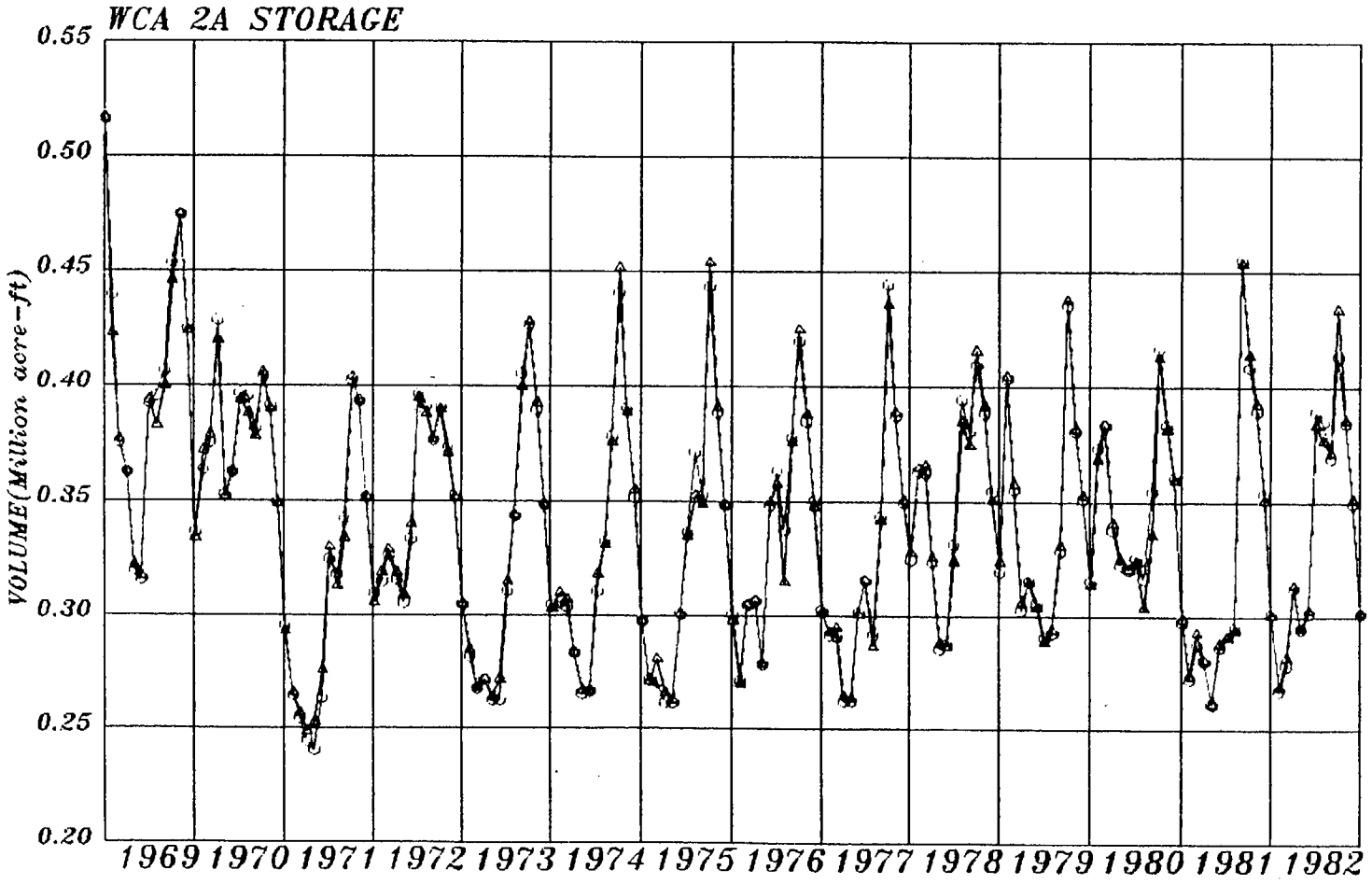
△ BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
WCA 2A STORAGE
VOLUME(Million acre-ft)
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-115

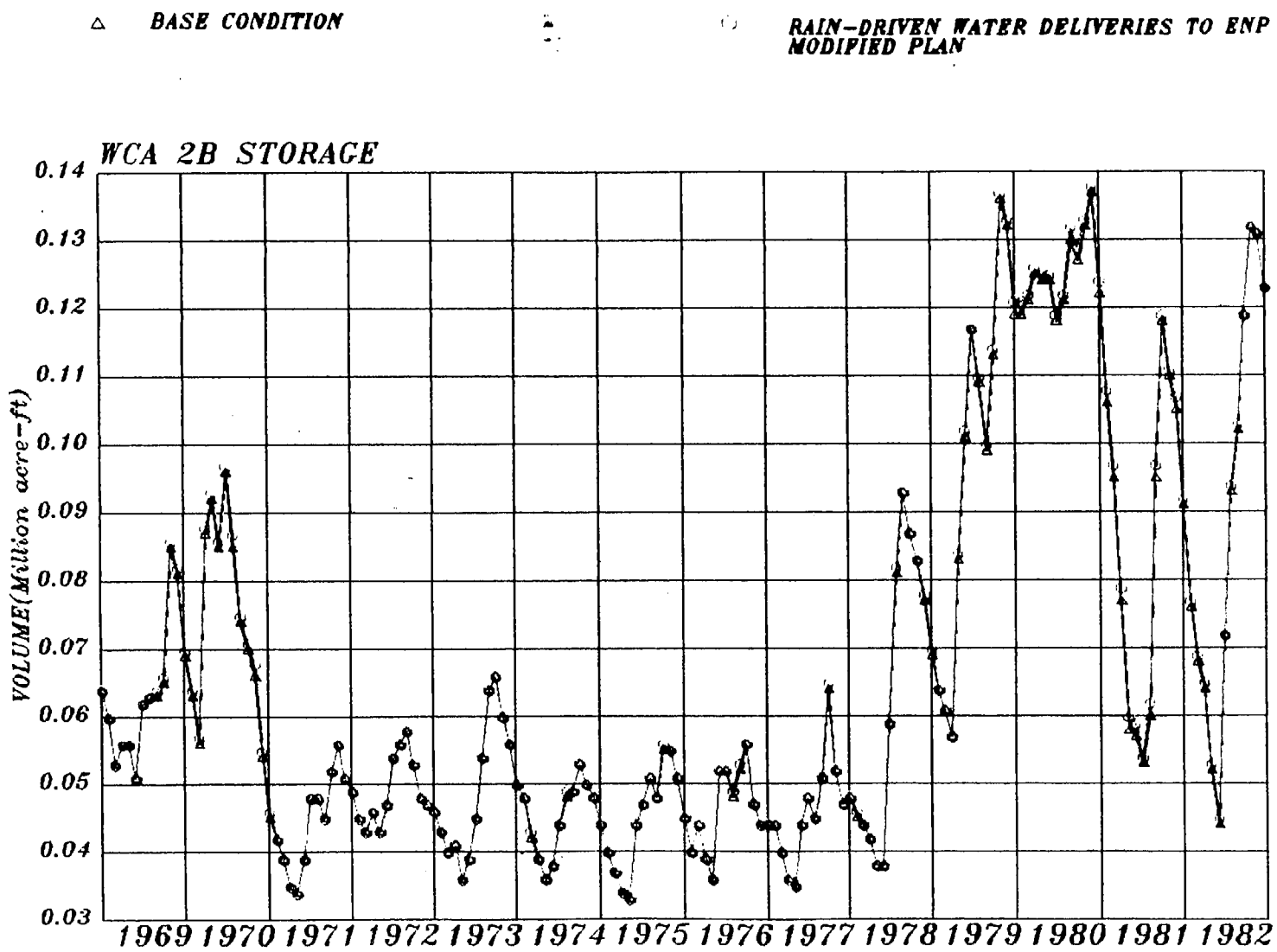
△ BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN
WCA 2B STORAGE
VOLUME(Million acre-ft)
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-116

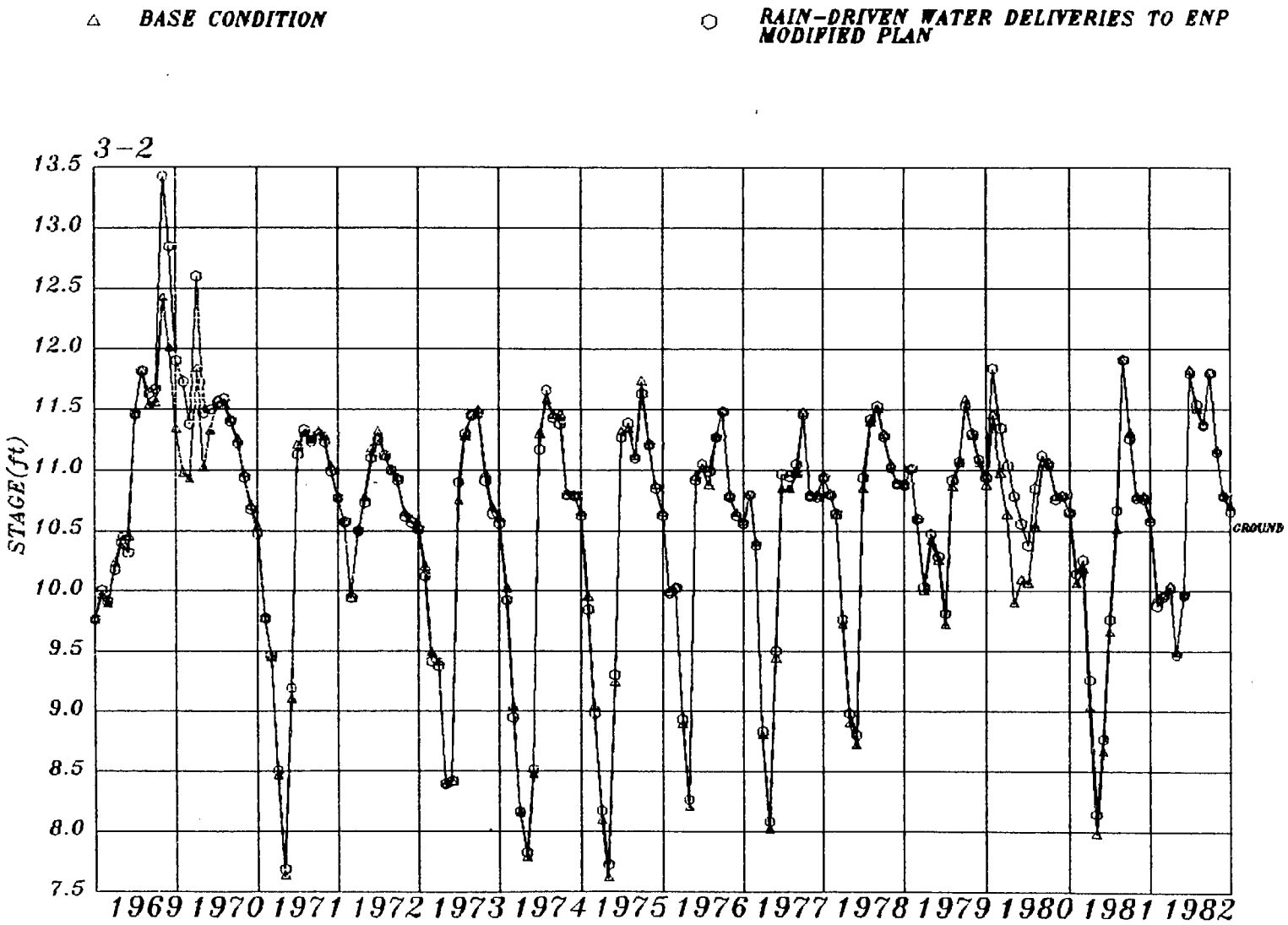
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-2
STAGE(ft)
13.5
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-117

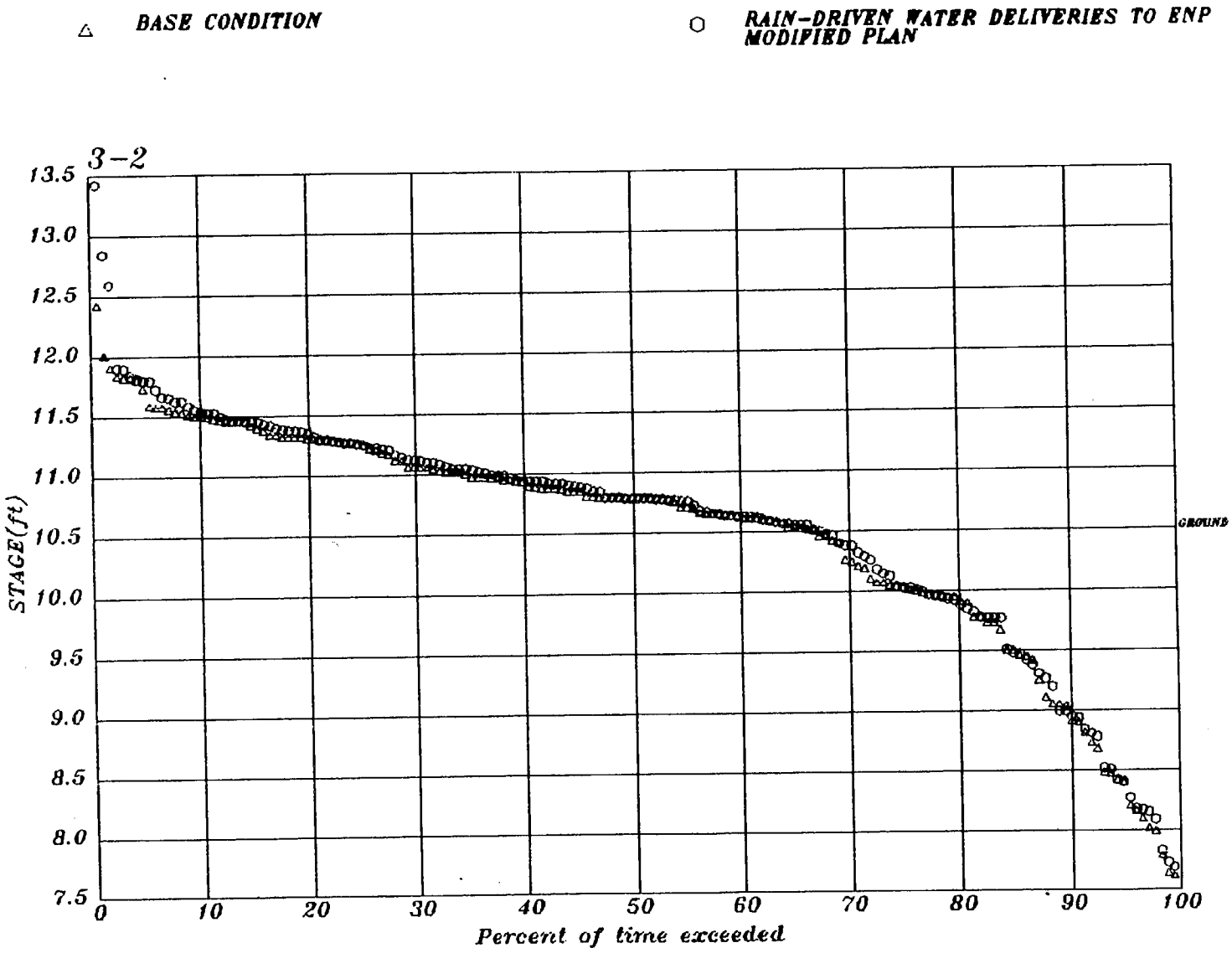
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-2
13.5
13.0
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-118

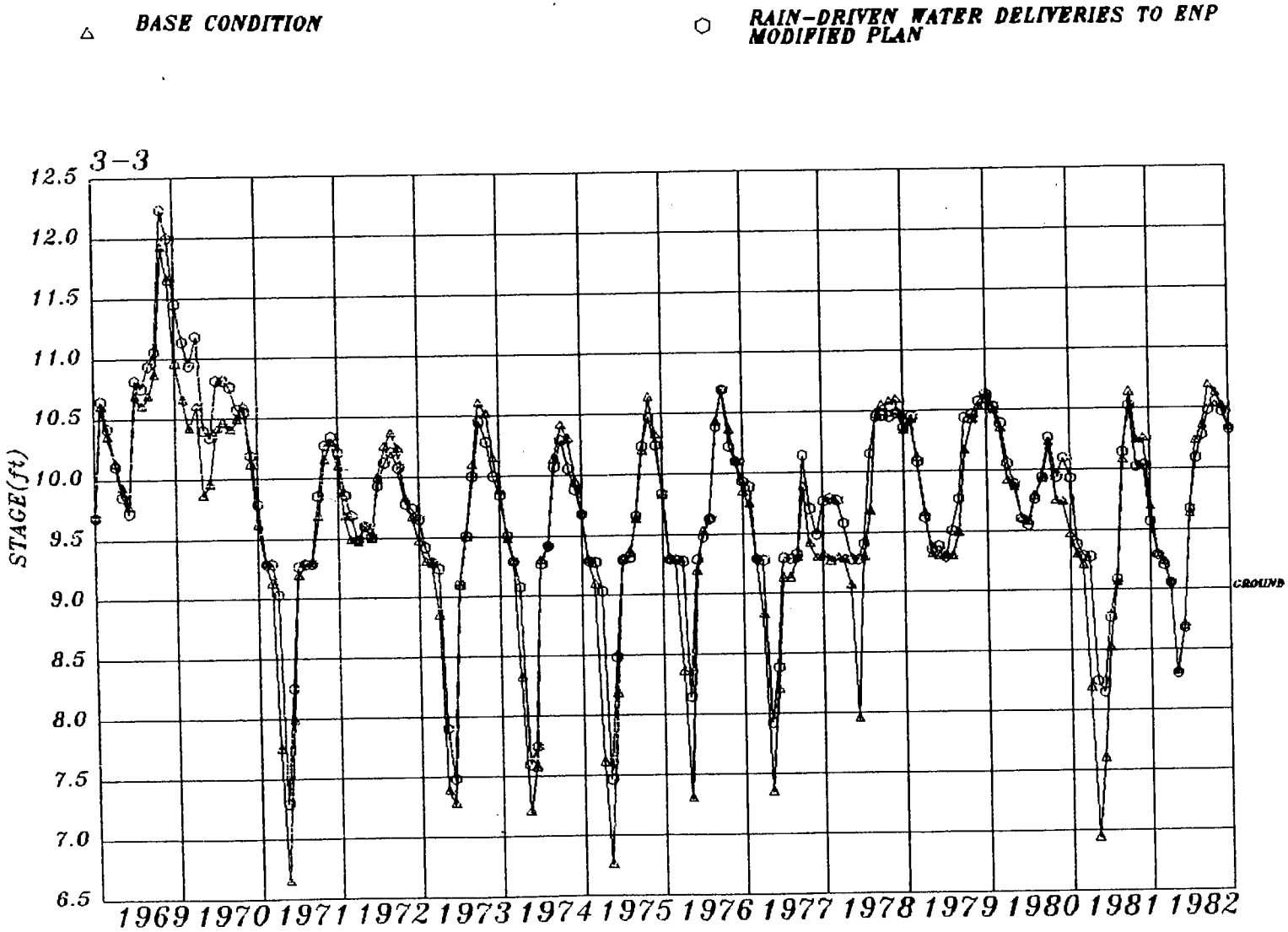
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-3
STAGE(ft)
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
GROUND

Figure E-119

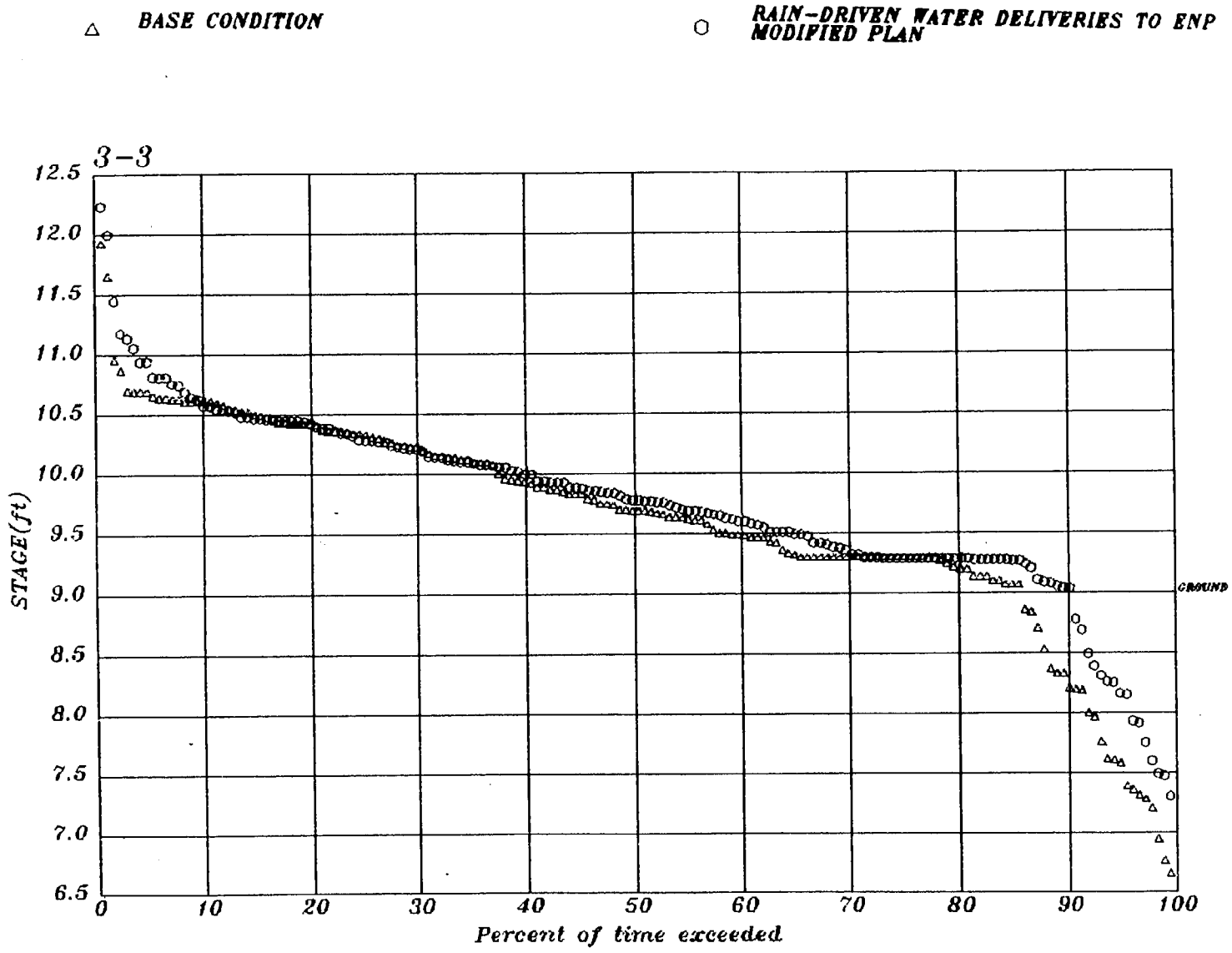
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-3
12.5
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-120

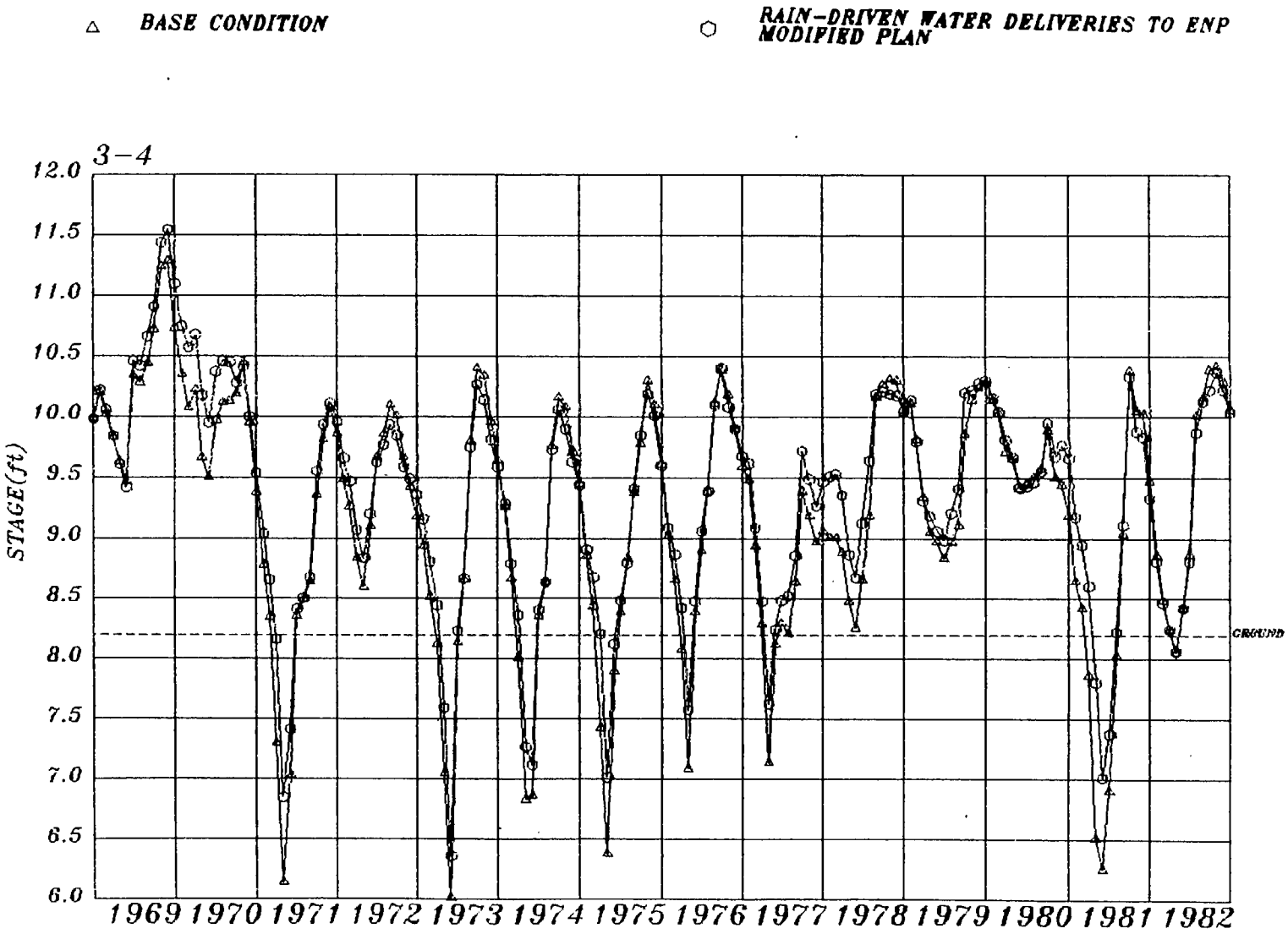
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-4
STAGE(ft)
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-121

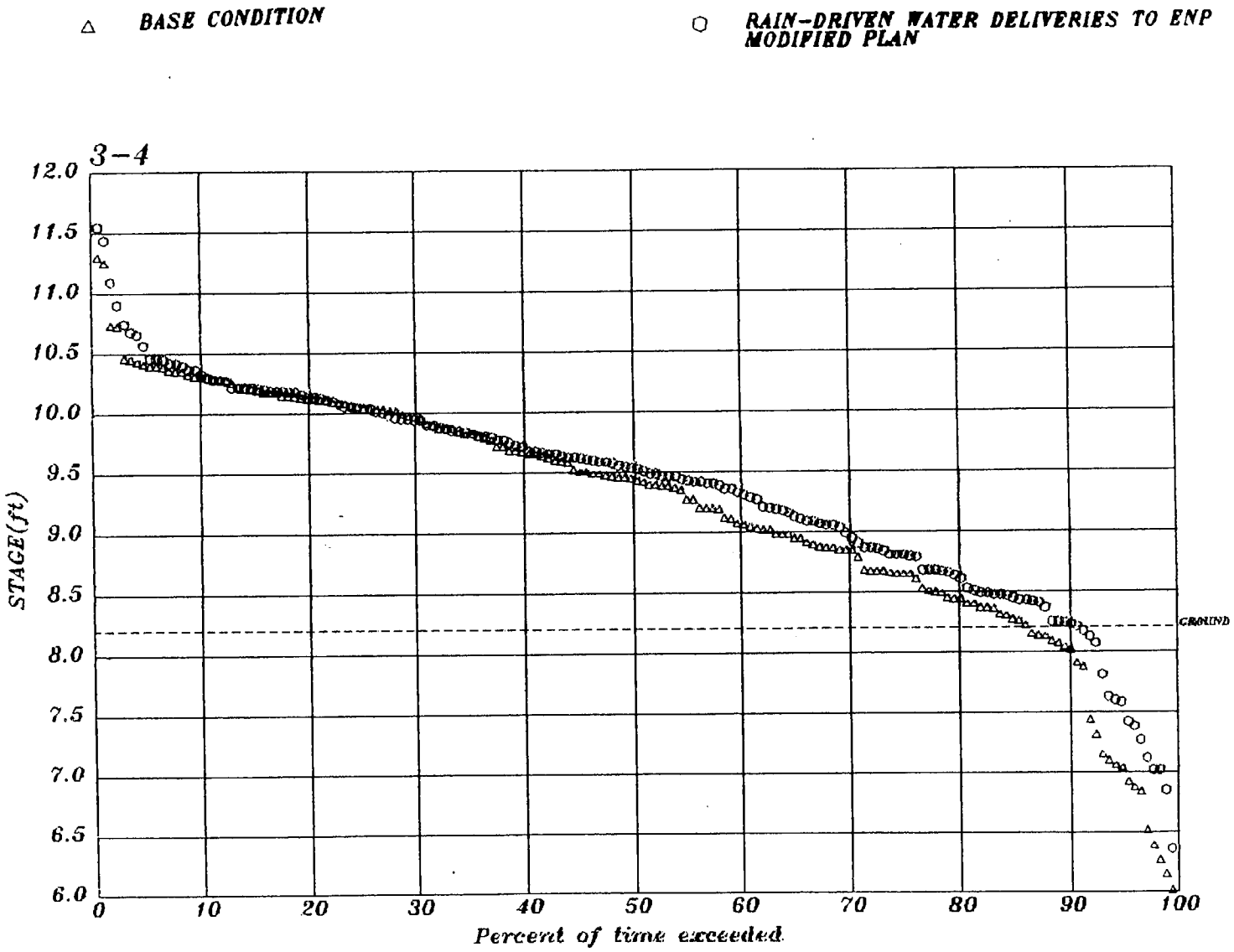
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-4
12.0
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-122

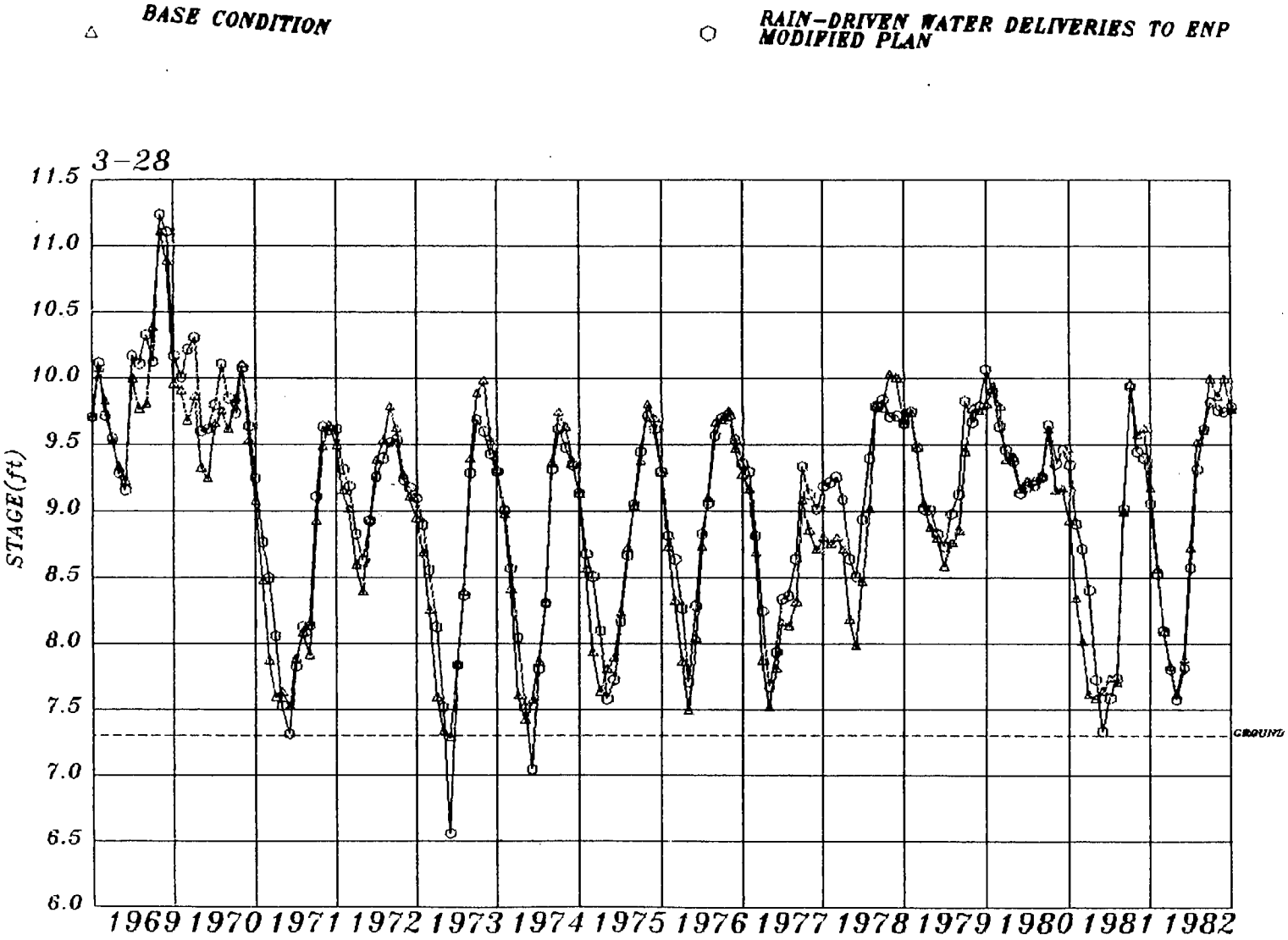
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-28
STAGE(ft)
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-123

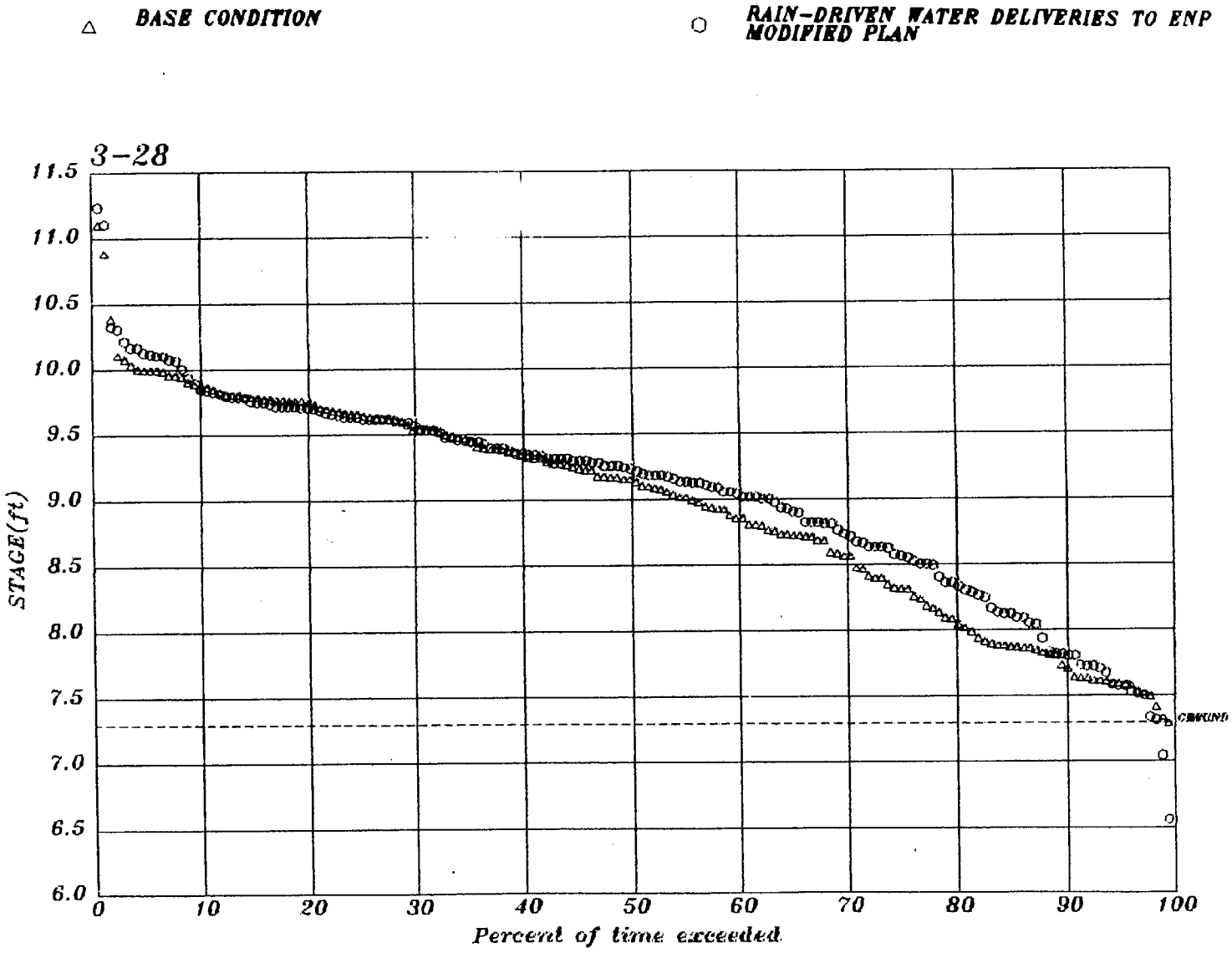
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-28
11.5
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-124

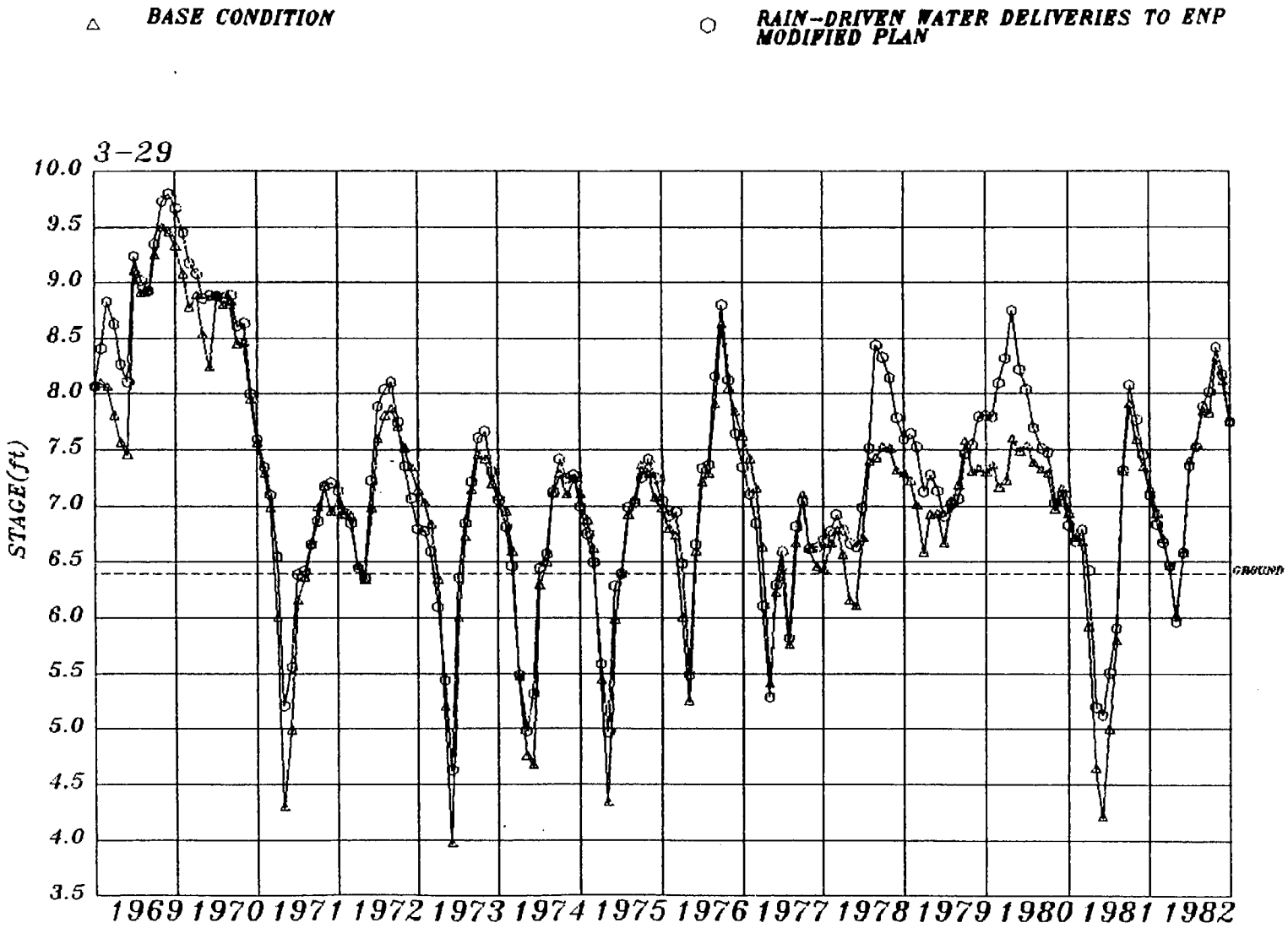
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
3-29
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-125

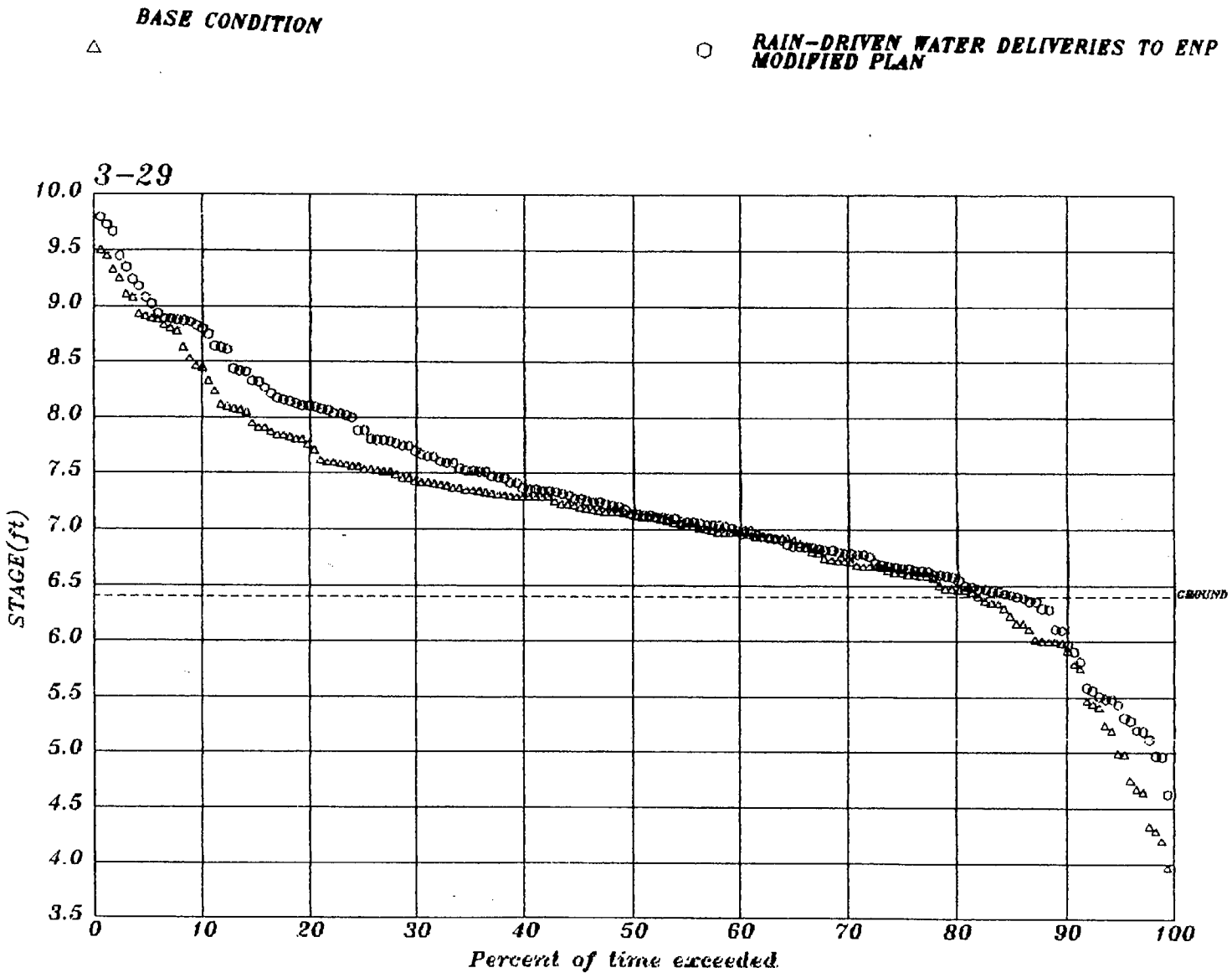
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN
3-29
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-126

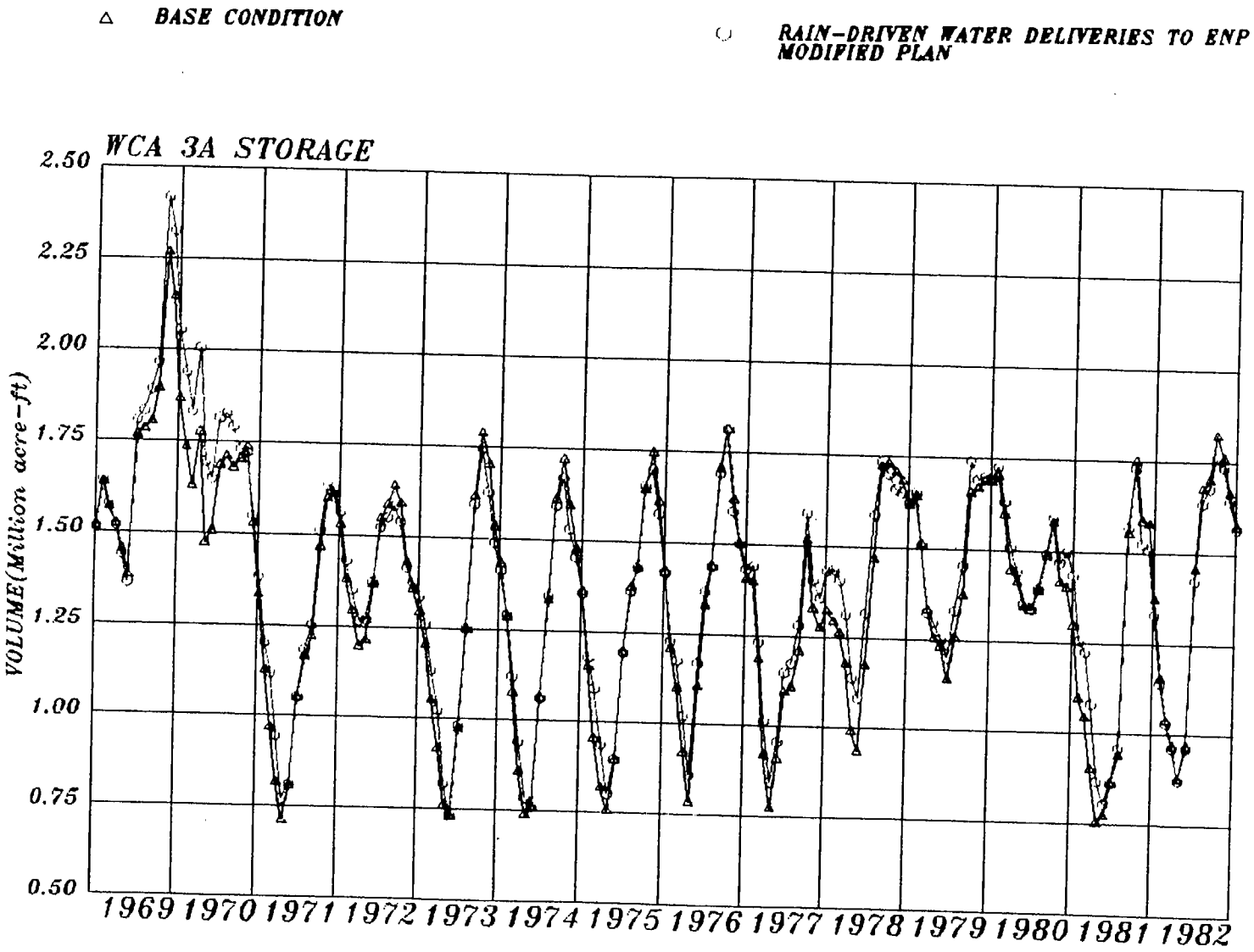
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
WCA 3A STORAGE
VOLUME(Million acre-ft)
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-127

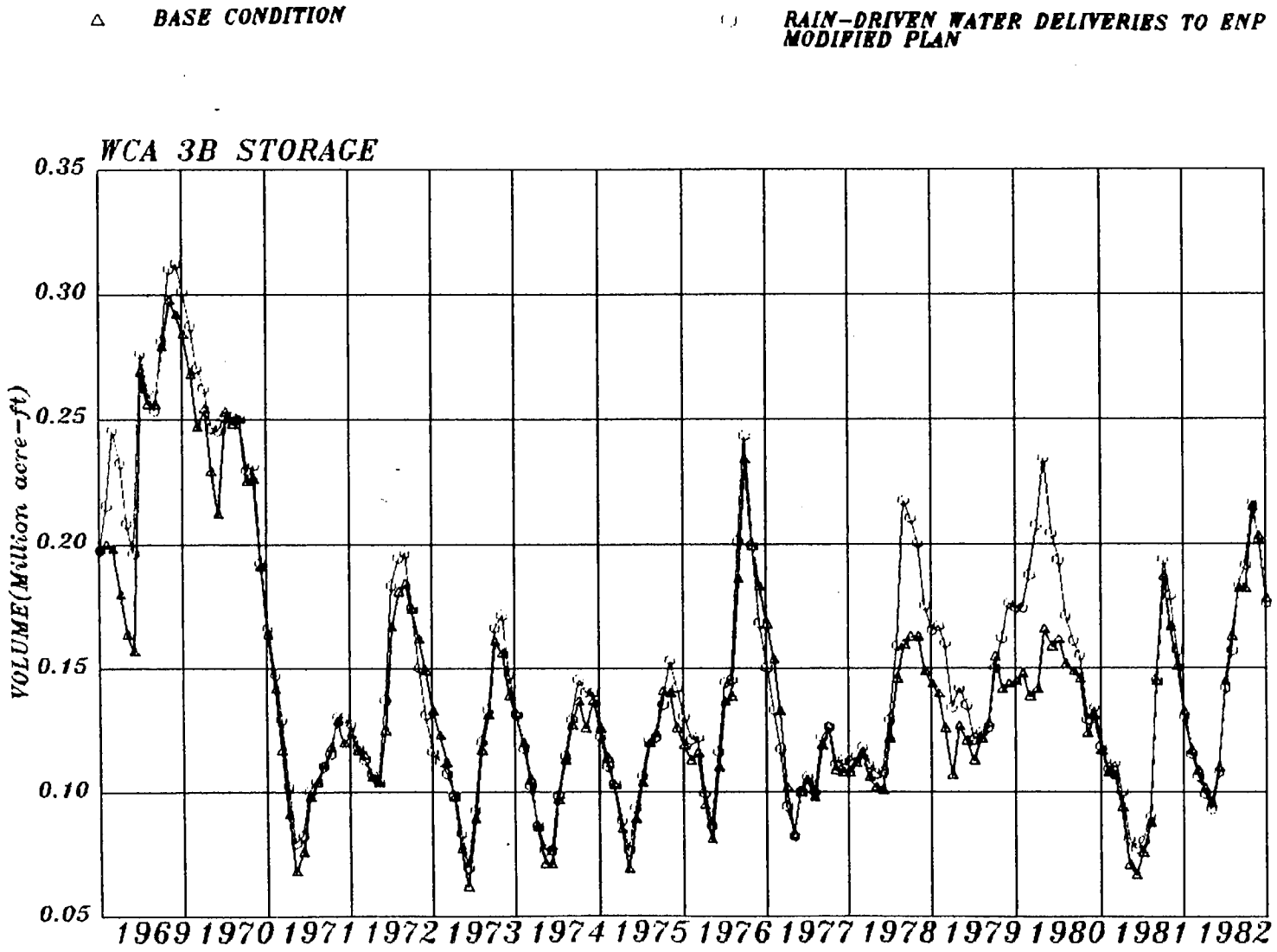
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
WCA 3B STORAGE
VOLUME(Million acre-ft)
0.35
0.30
0.25
0.20
0.15
0.10
0.05
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-128

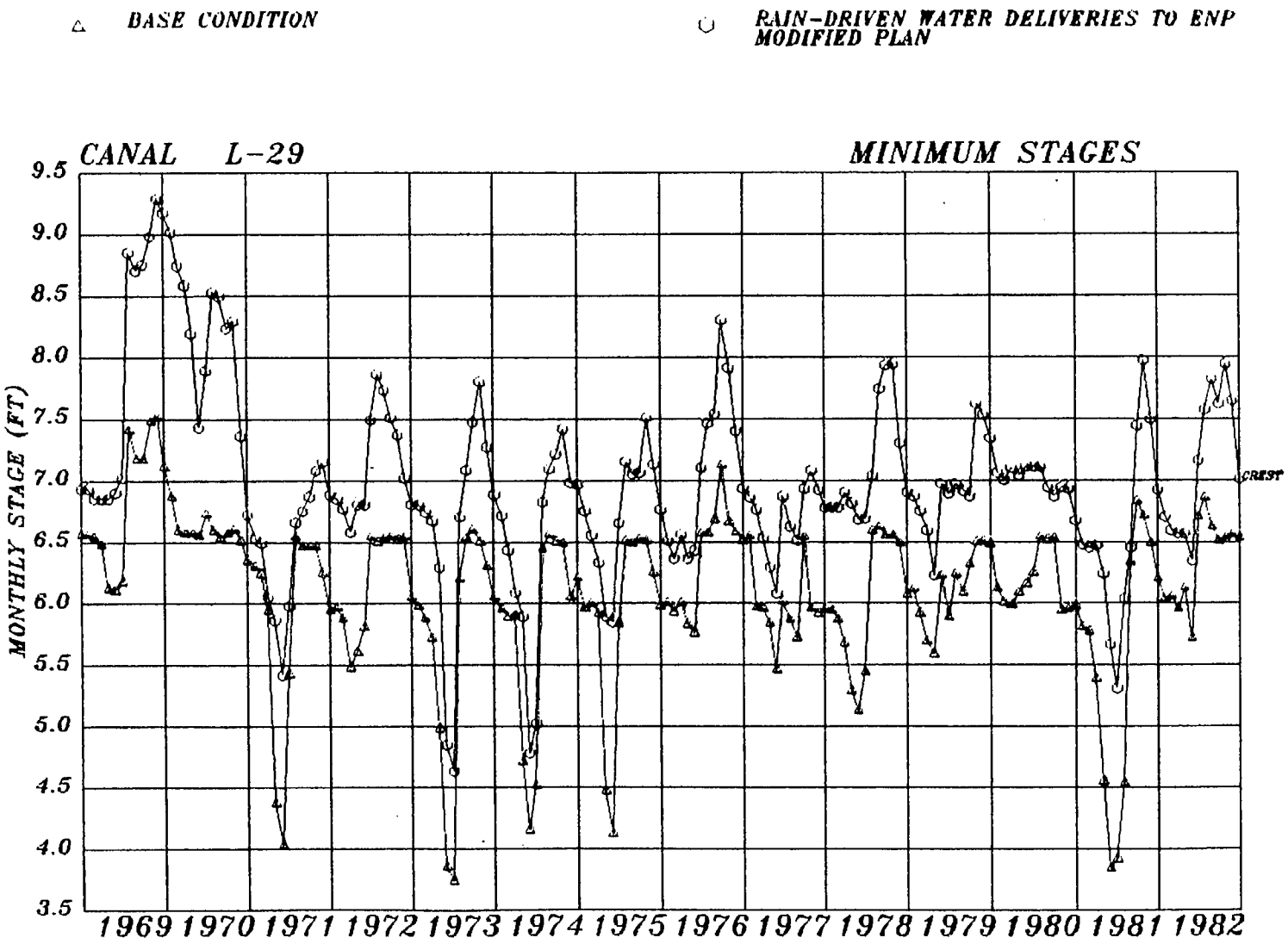
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN
CANAL L-29
MINIMUM STAGES
MONTHLY STAGE (FT)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-129

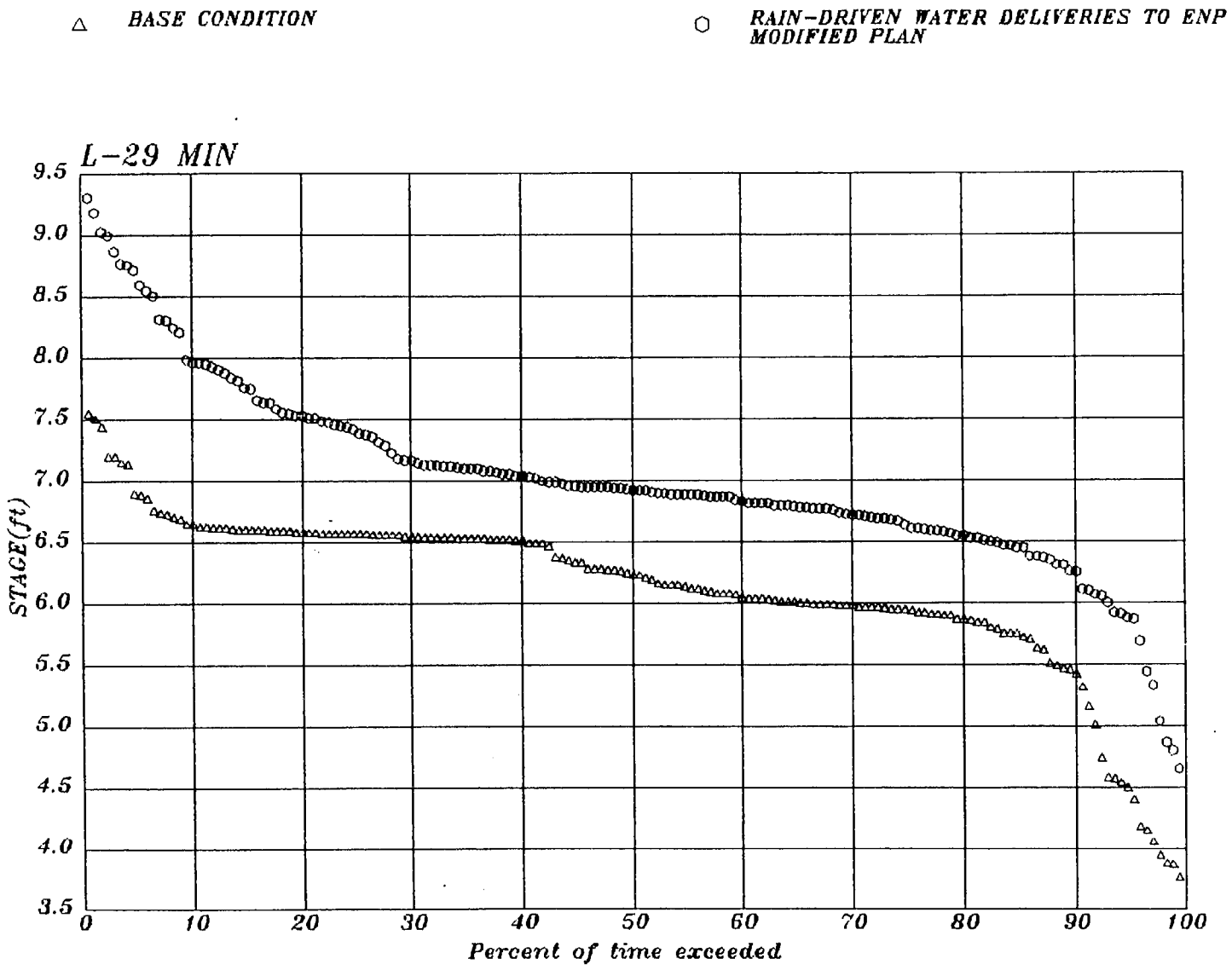
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
L-29 MIN
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-130

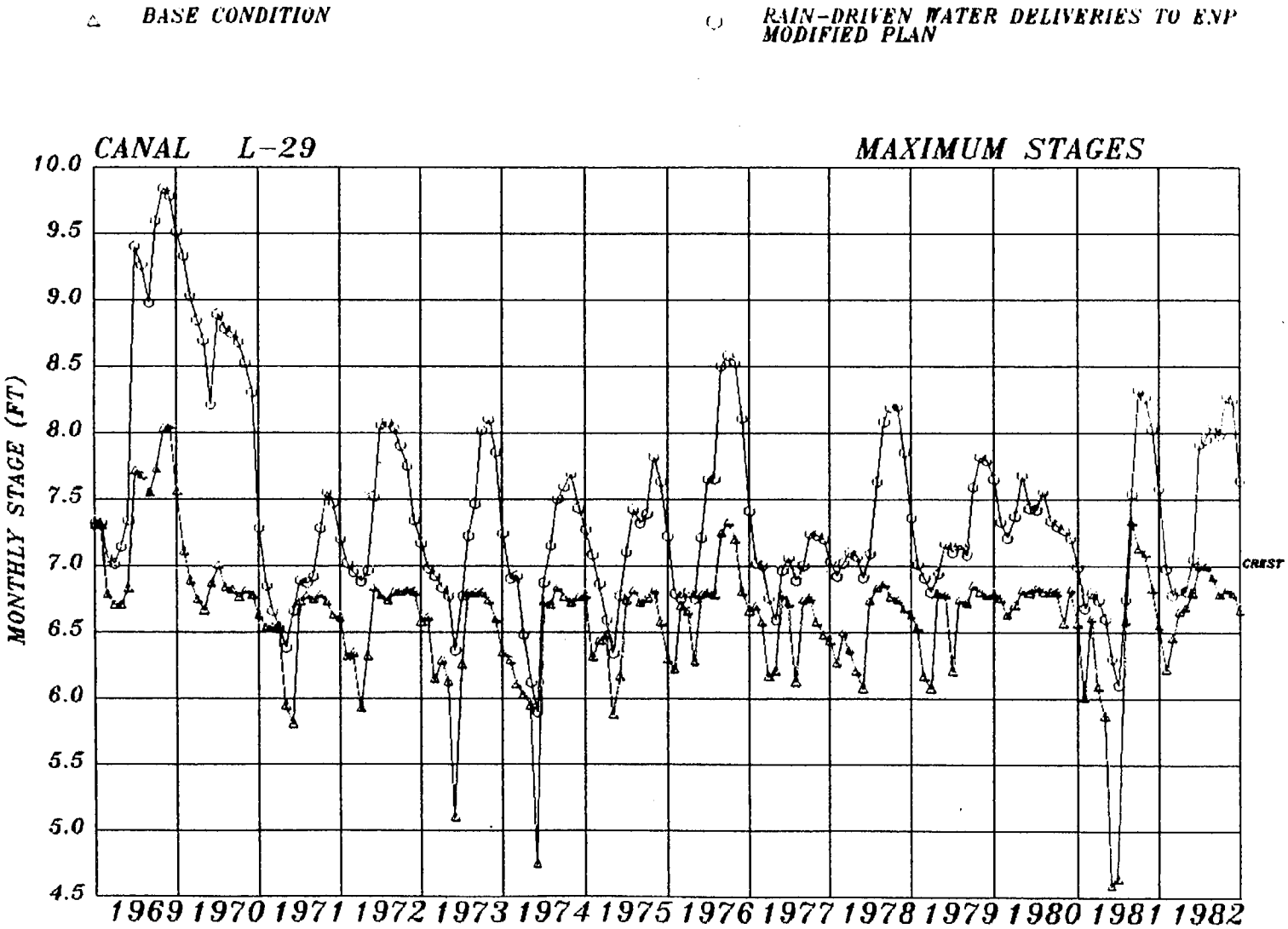
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L-29
MAXIMUM STAGES
MONTHLY STAGE (FT)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-131

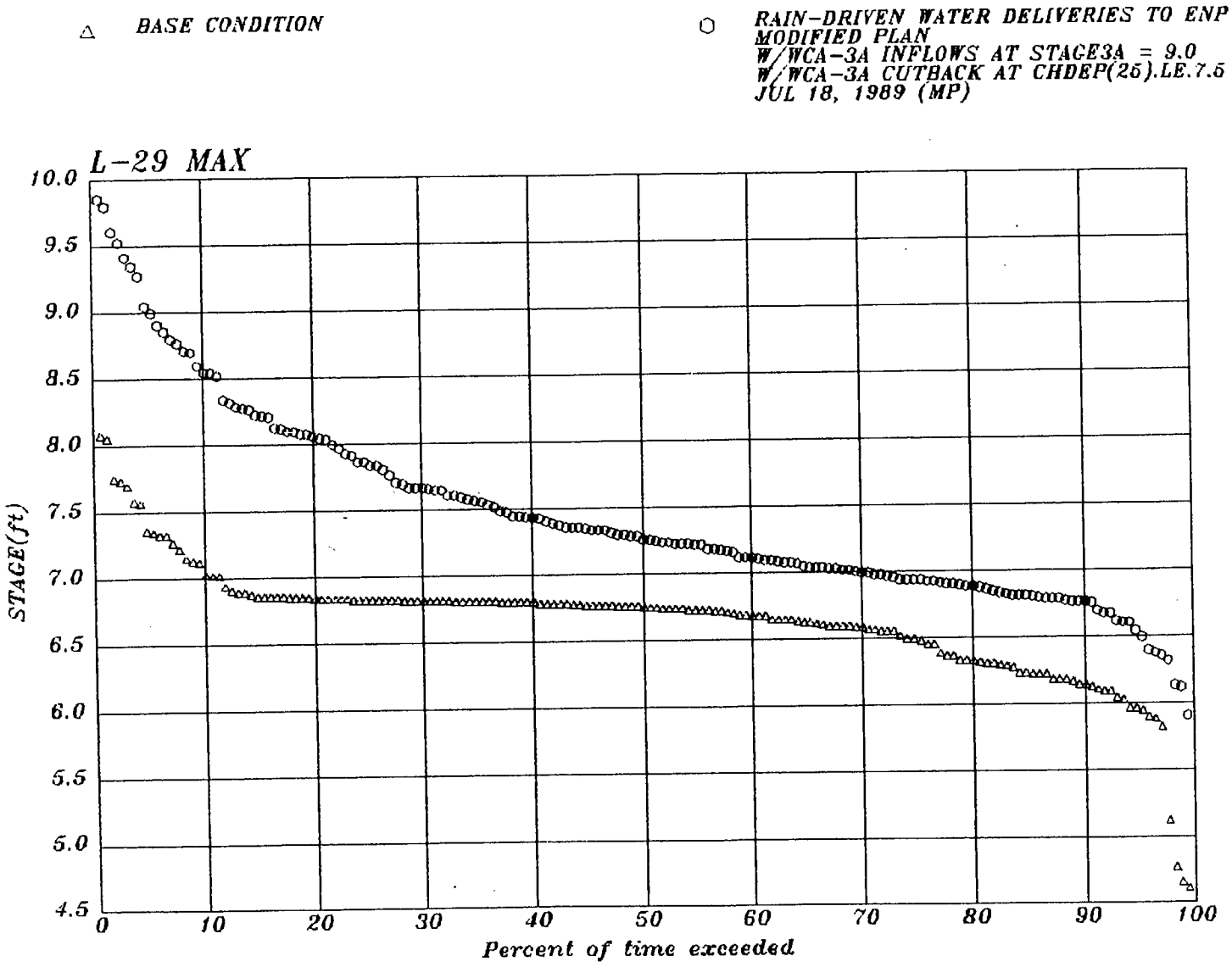
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN
W/WCA-3A INFLOWS AT STAGE3A = 9.0
W/WCA-3A CUTBACK AT CHDEP(25).LE.7.5
JUL 18, 1989 (MP)
L-29 MAX
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-132

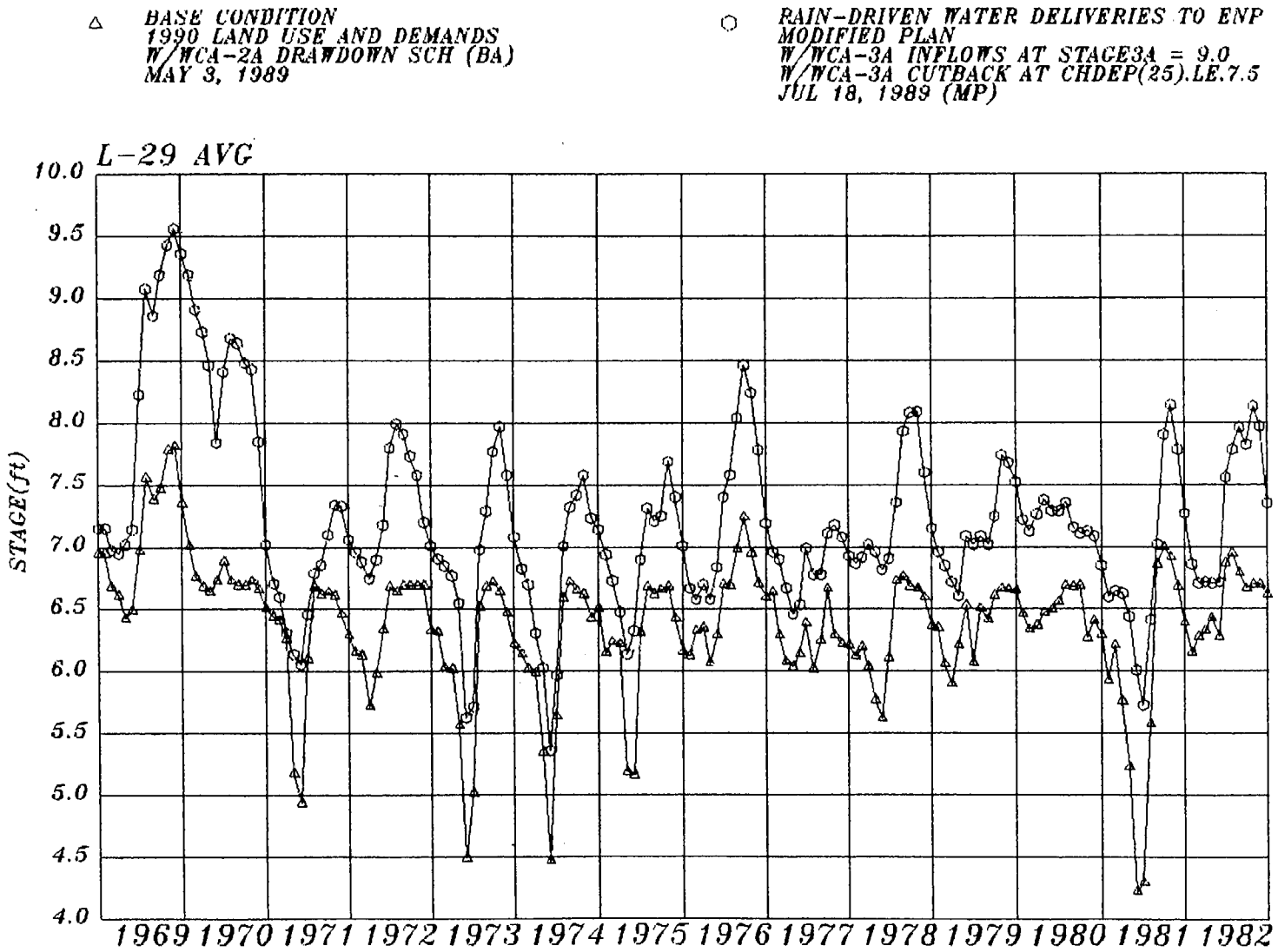
△ BASE CONDITION
1990 LAND USE AND DEMANDS
W/WCA-2A DRAWDOWN SCH (BA)
MAY 3, 1989
○ RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN
W/WCA-3A INFLOWS AT STAGE3A = 9.0
W/WCA-3A CUTBACK AT CHDEP(25).LE.7.5
JUL 18, 1989 (MP)
L-29 AVG
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-133

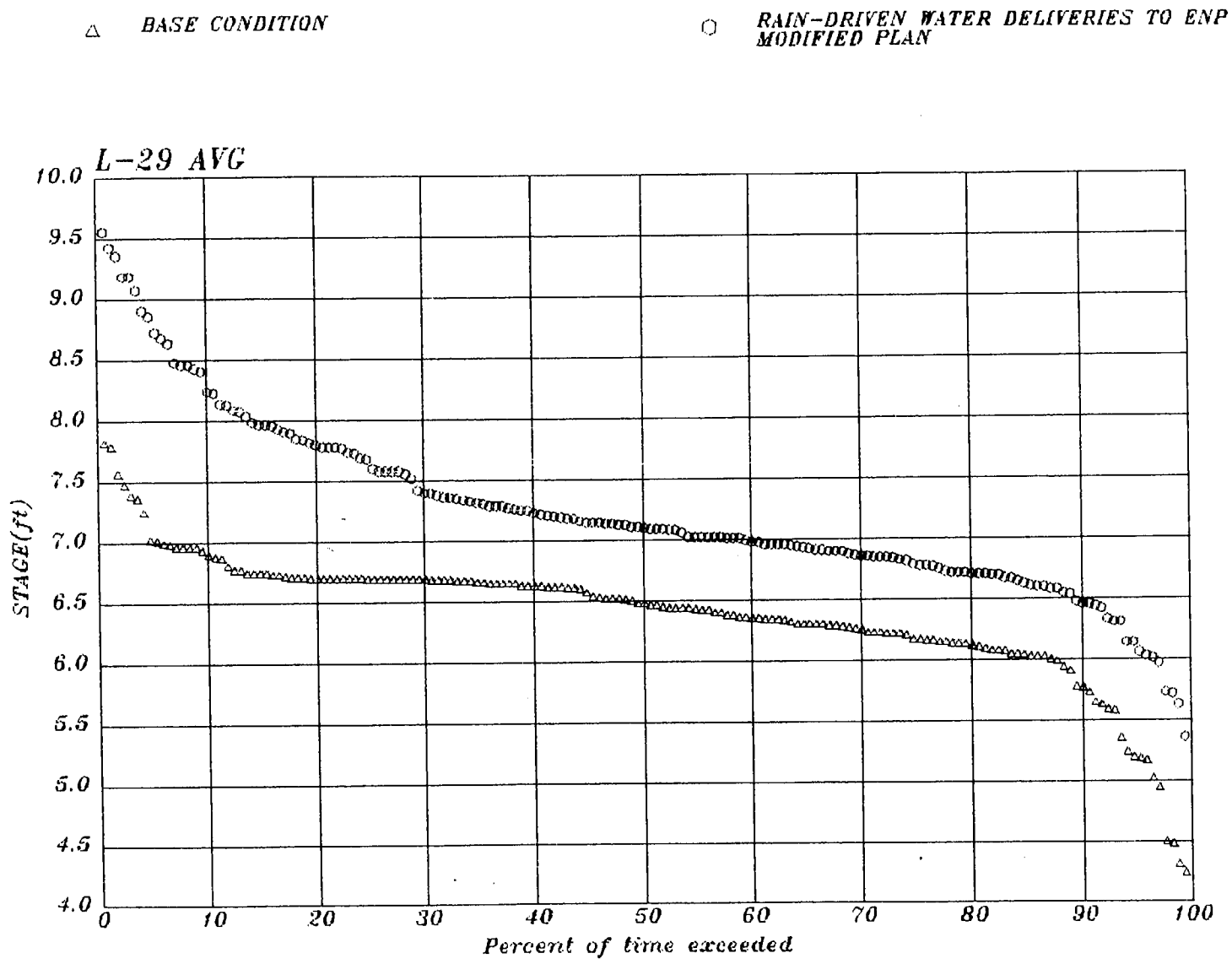
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
L-29 AVG
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-134

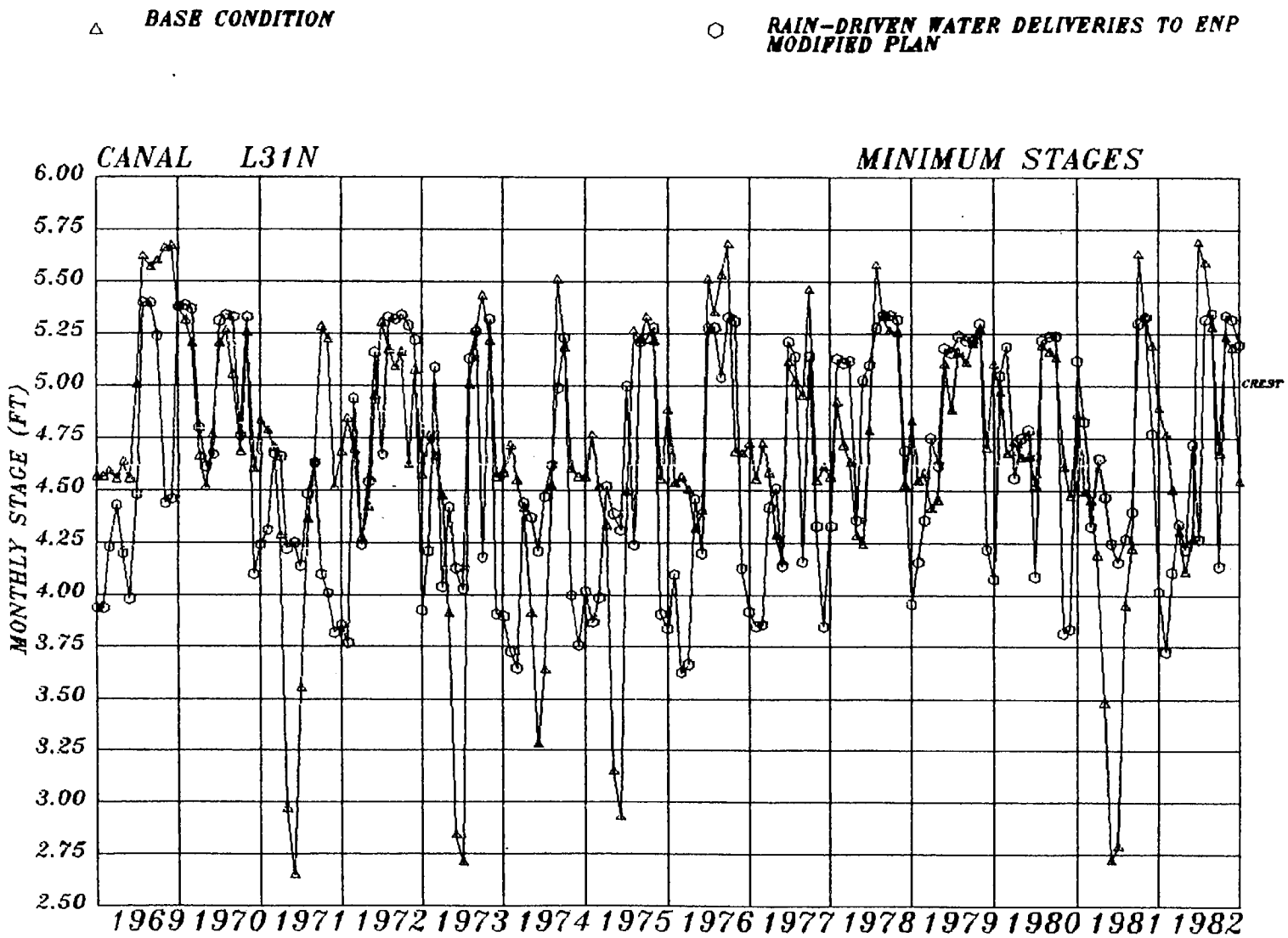
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L31N
MINIMUM STAGES
MONTHLY STAGE (FT)
6.00
5.75
5.50
5.25
5.00
4.75
4.50
4.25
4.00
3.75
3.50
3.25
3.00
2.75
2.50
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-135

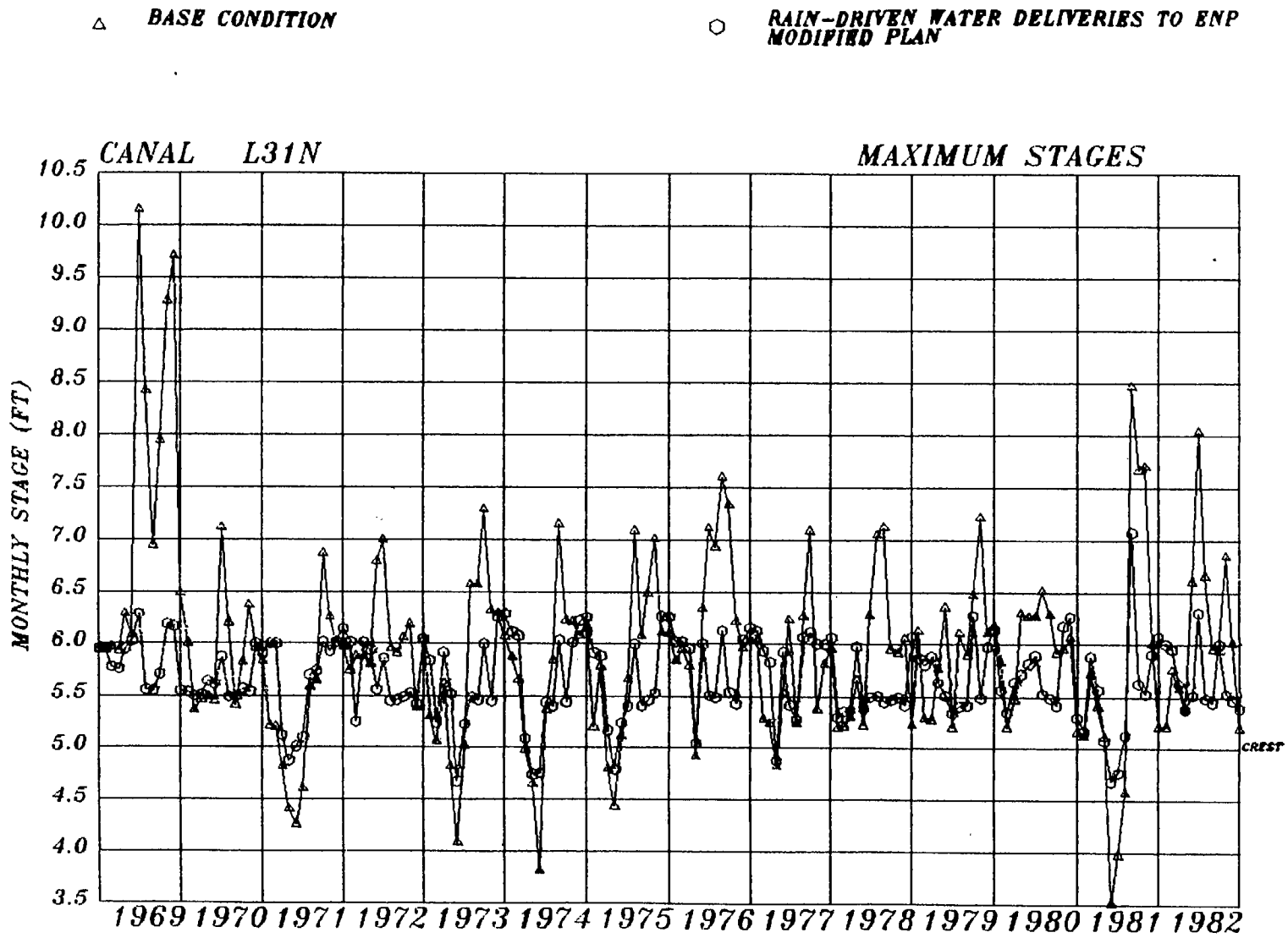
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L31N
MAXIMUM STAGES
MONTHLY STAGE (FT)
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-136

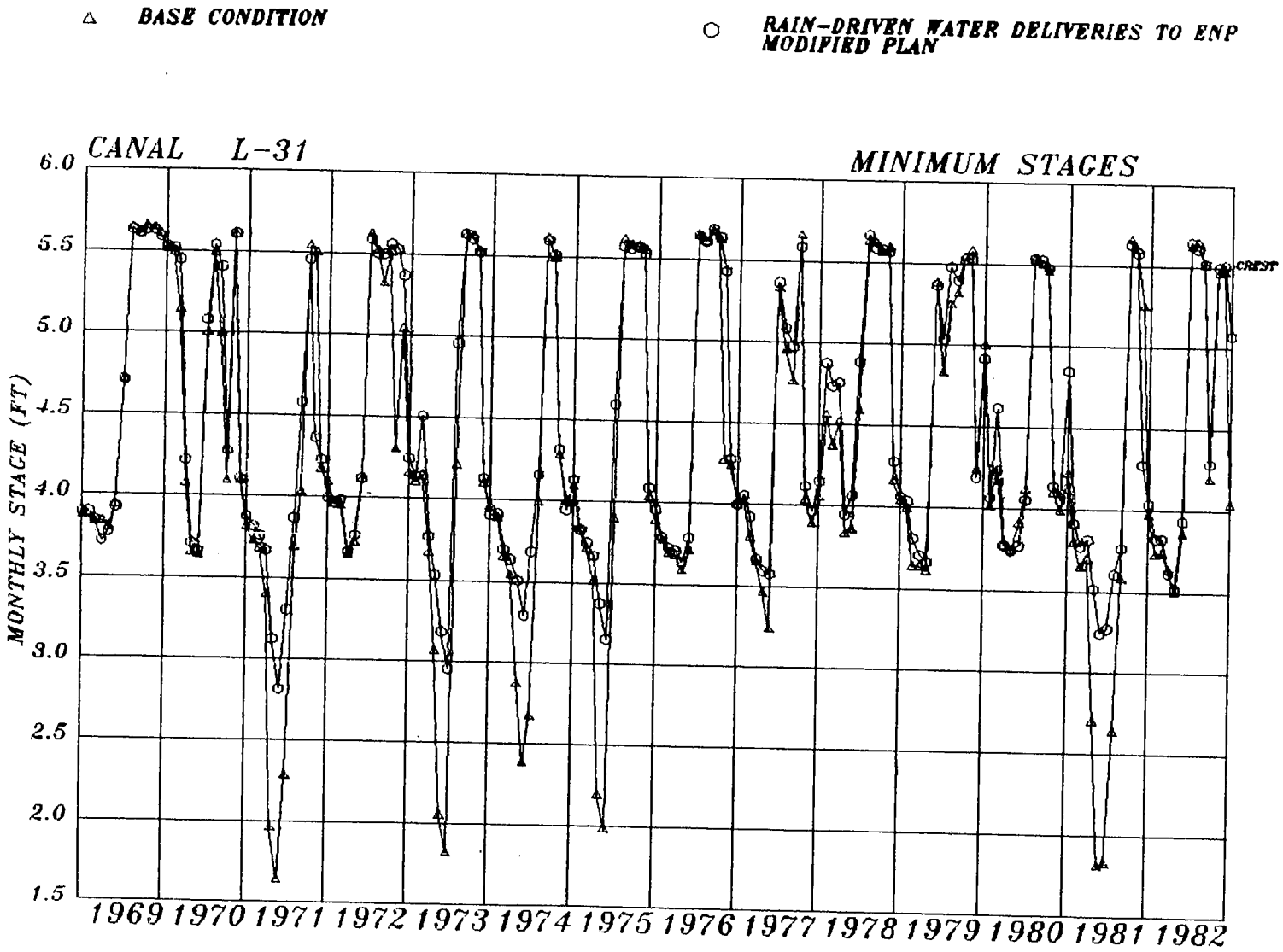
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L-31
MINIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-137

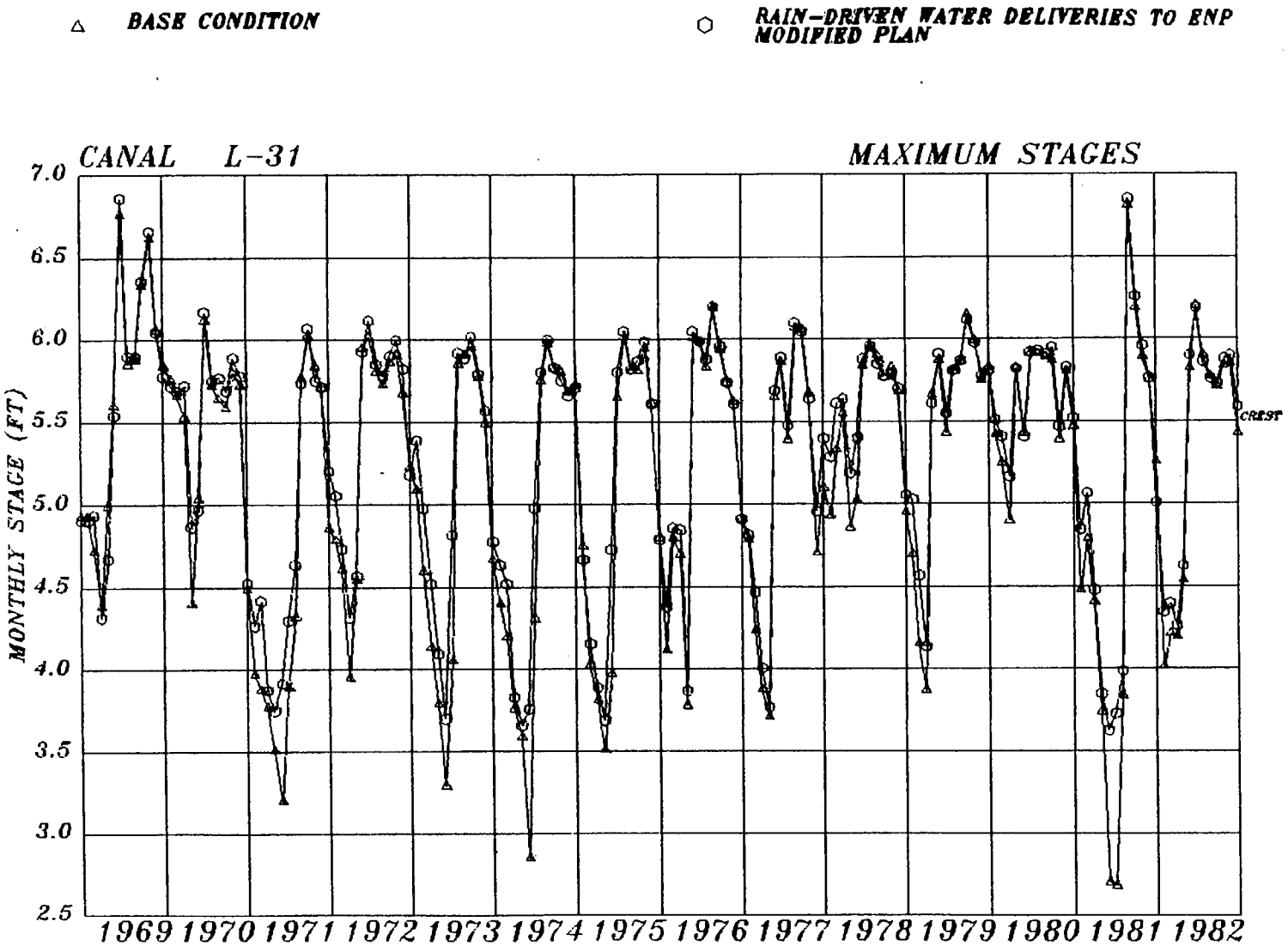
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L-31
MAXIMUM STAGES
MONTHLY STAGE (FT)
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-138

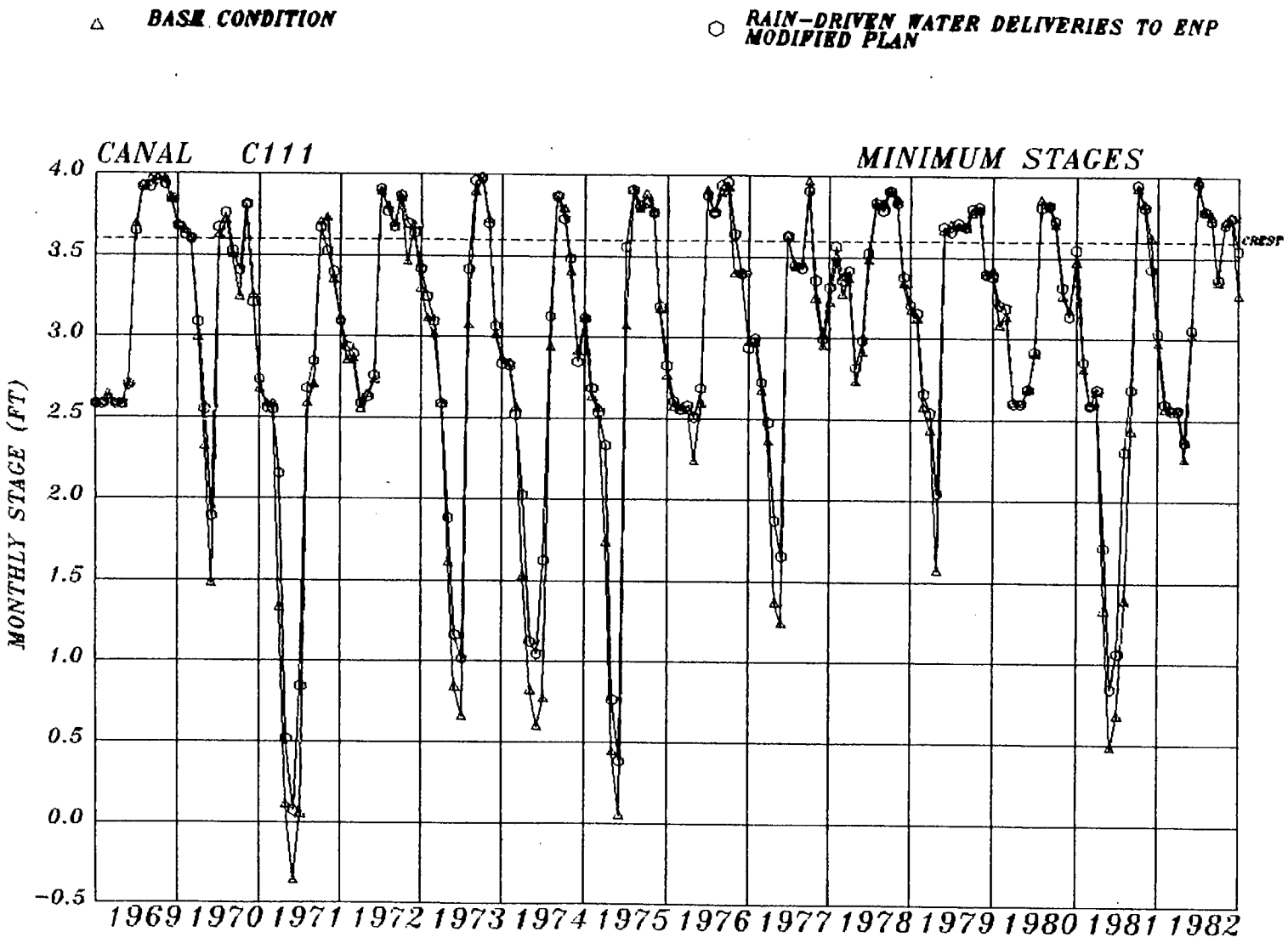
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL C111
MINIMUM STAGES
MONTHLY STAGE (FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-139

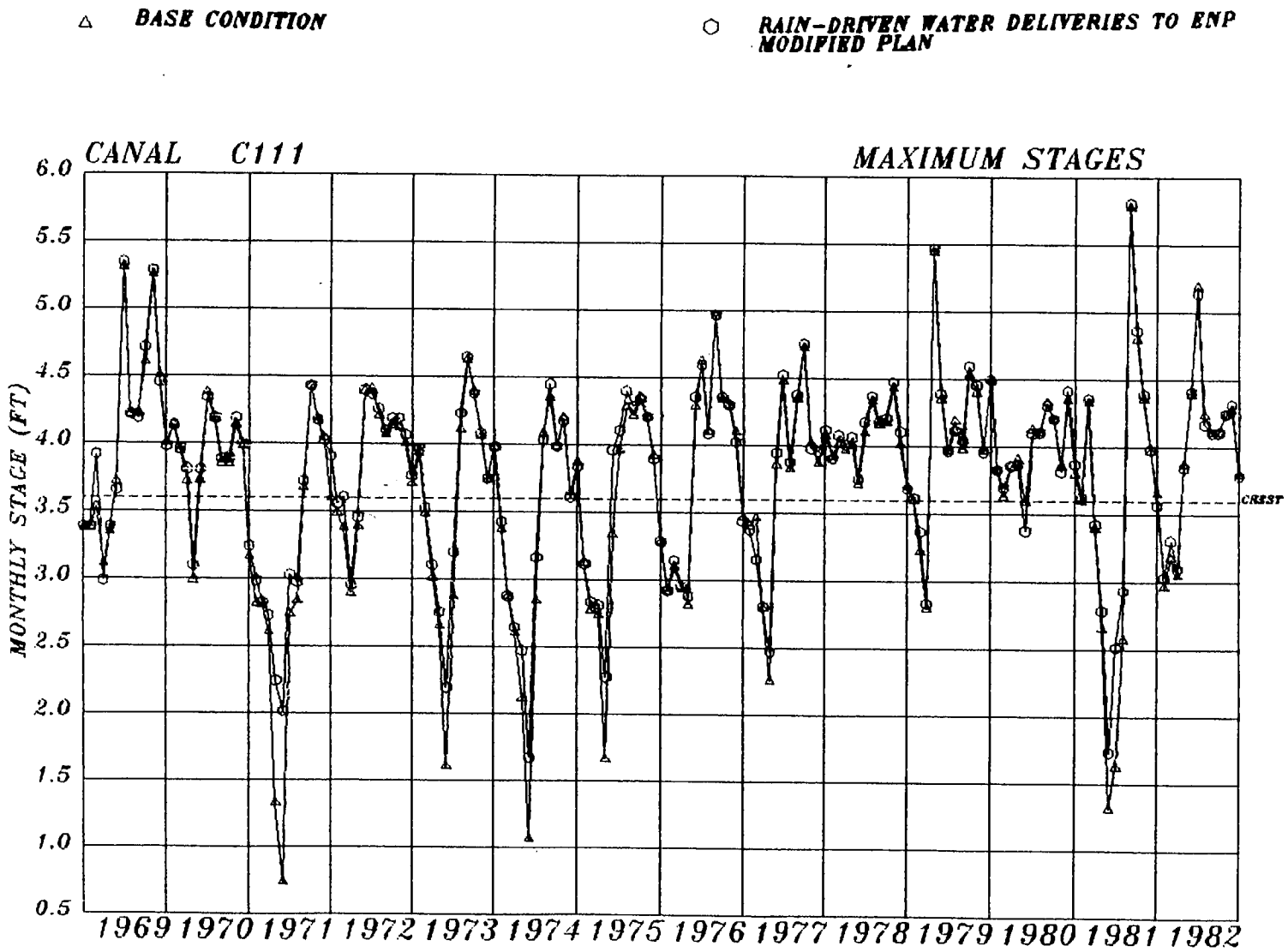
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL C111
MAXIMUM STAGES
MONTHLY STAGE (FT)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-140

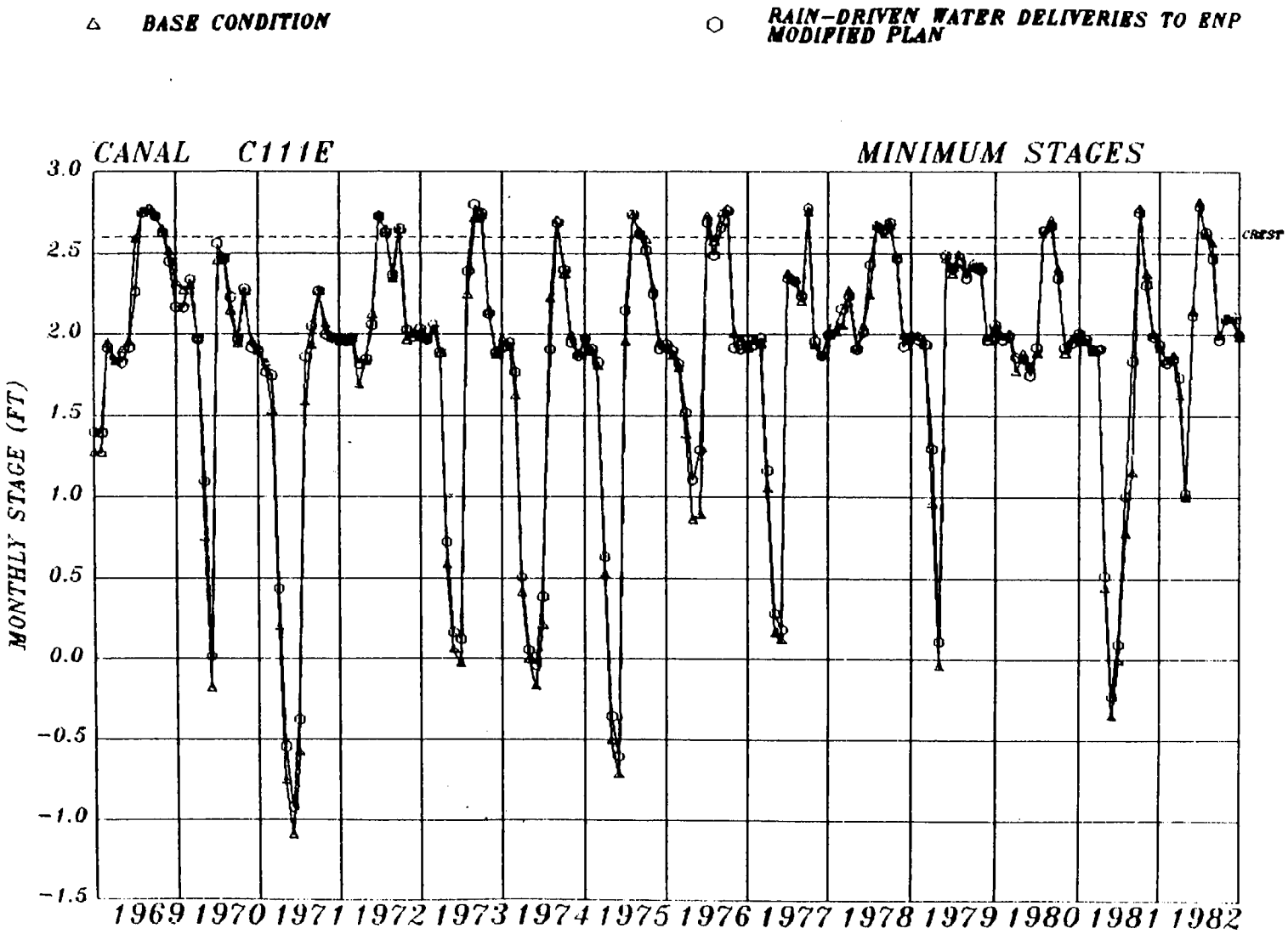
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL C111E
MINIMUM STAGES
MONTHLY STAGE (FT)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-141

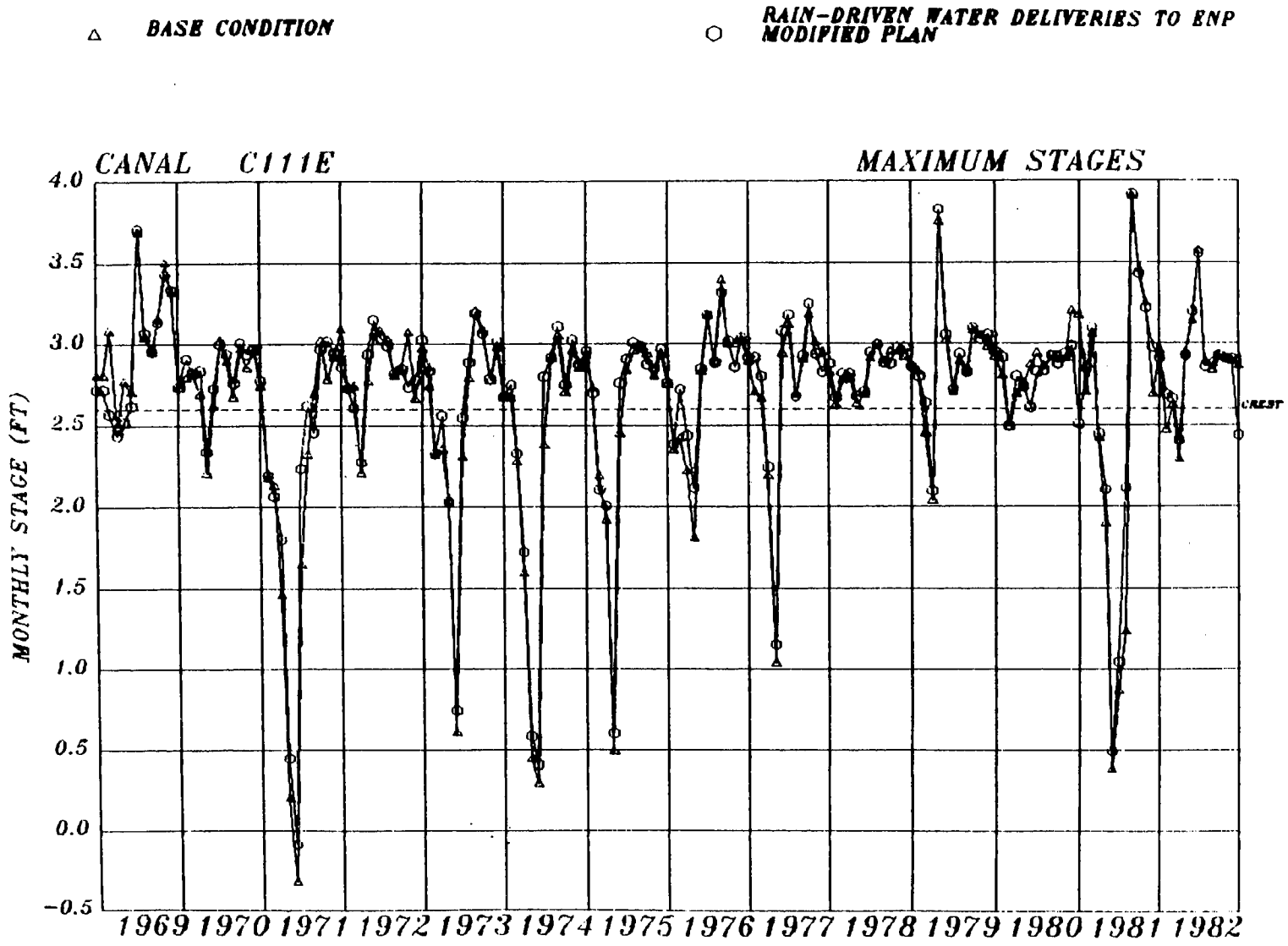
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL C111E
MAXIMUM STAGES
MONTHLY STAGE (FT)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-142

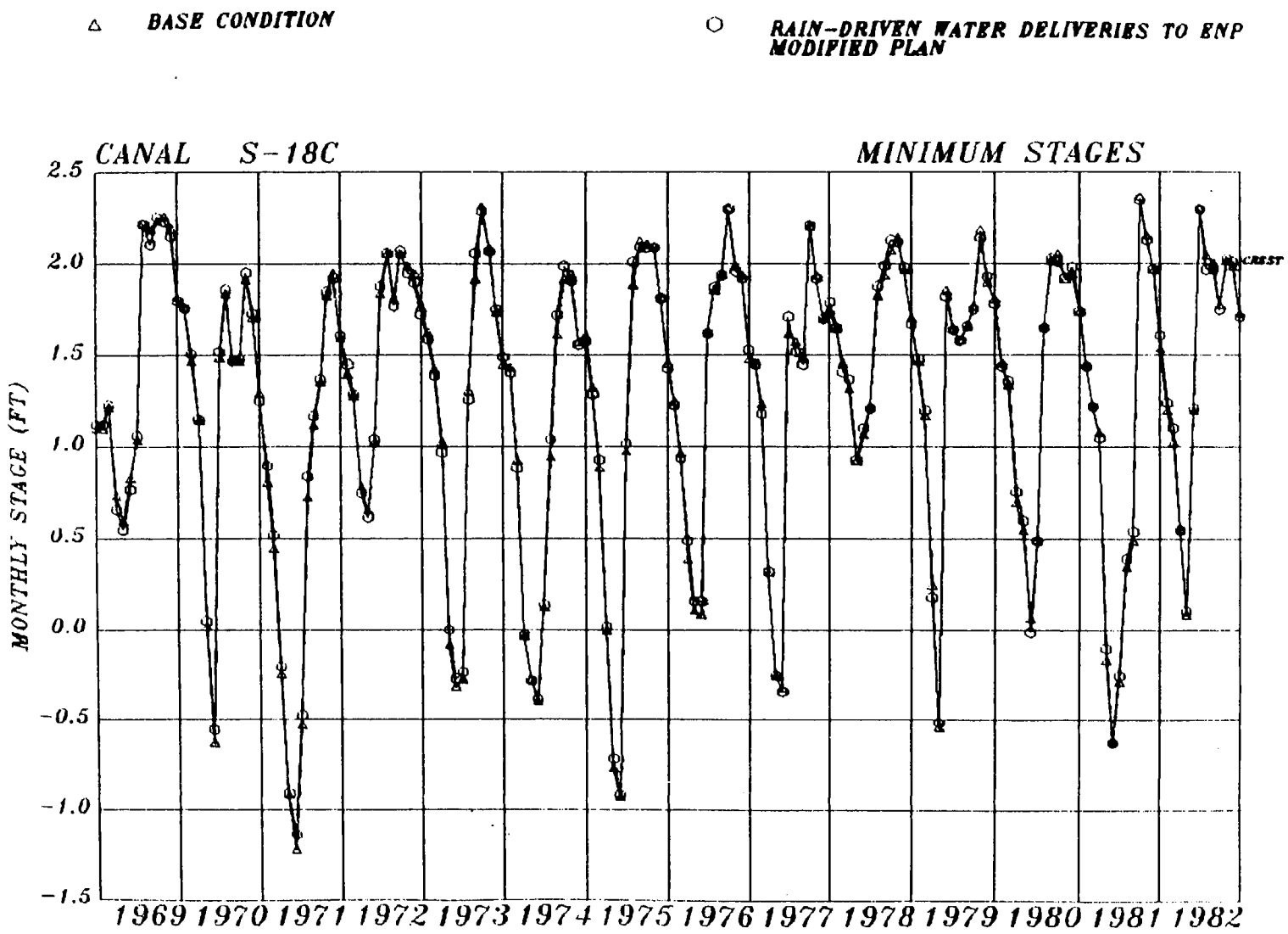
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL S-18C
MINIMUM STAGES
MONTHLY STAGE (FT)
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
CREST

Figure E-143

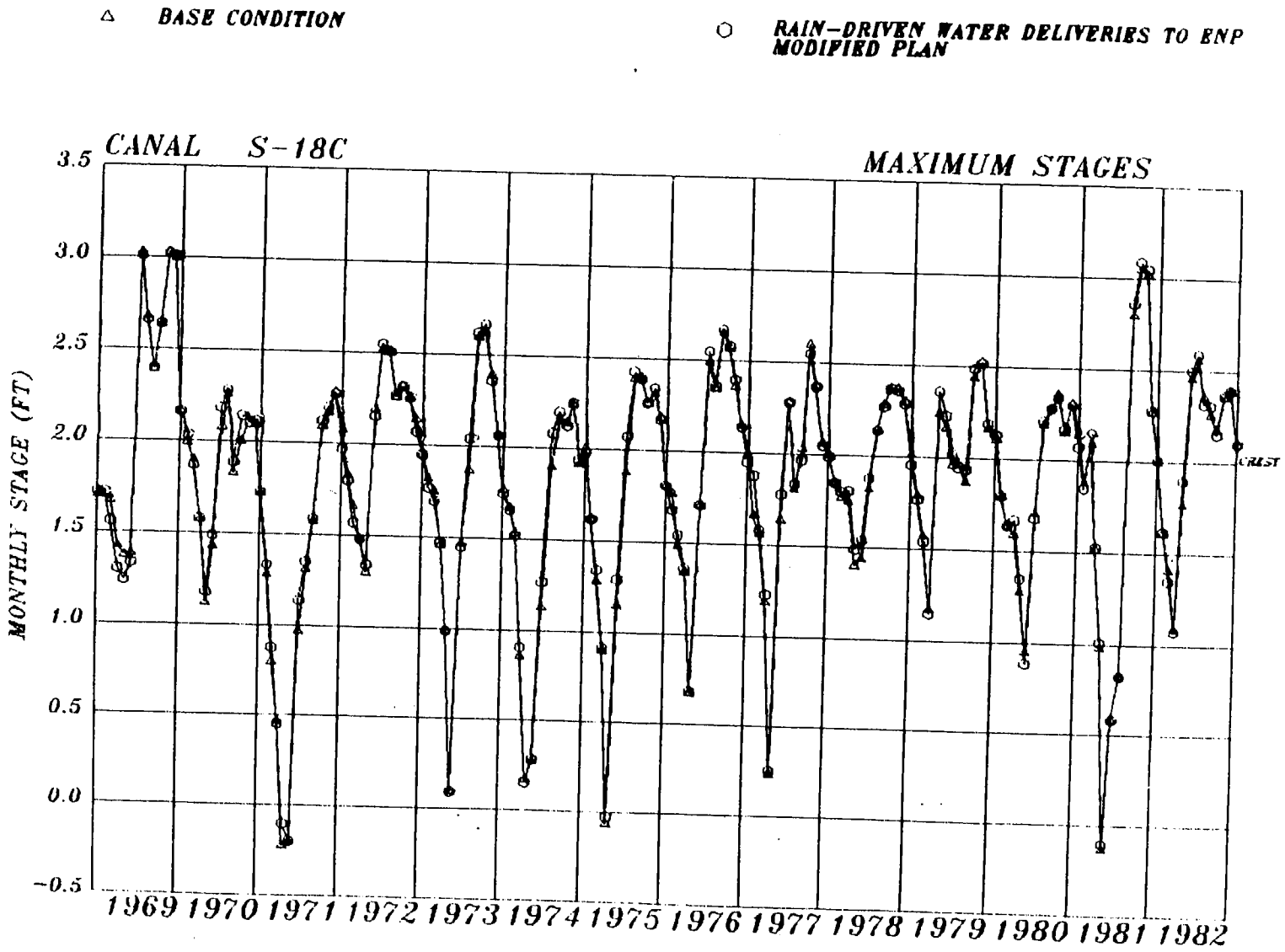
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL S-18C
MAXIMUM STAGES
MONTHLY STAGE (FT)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-144

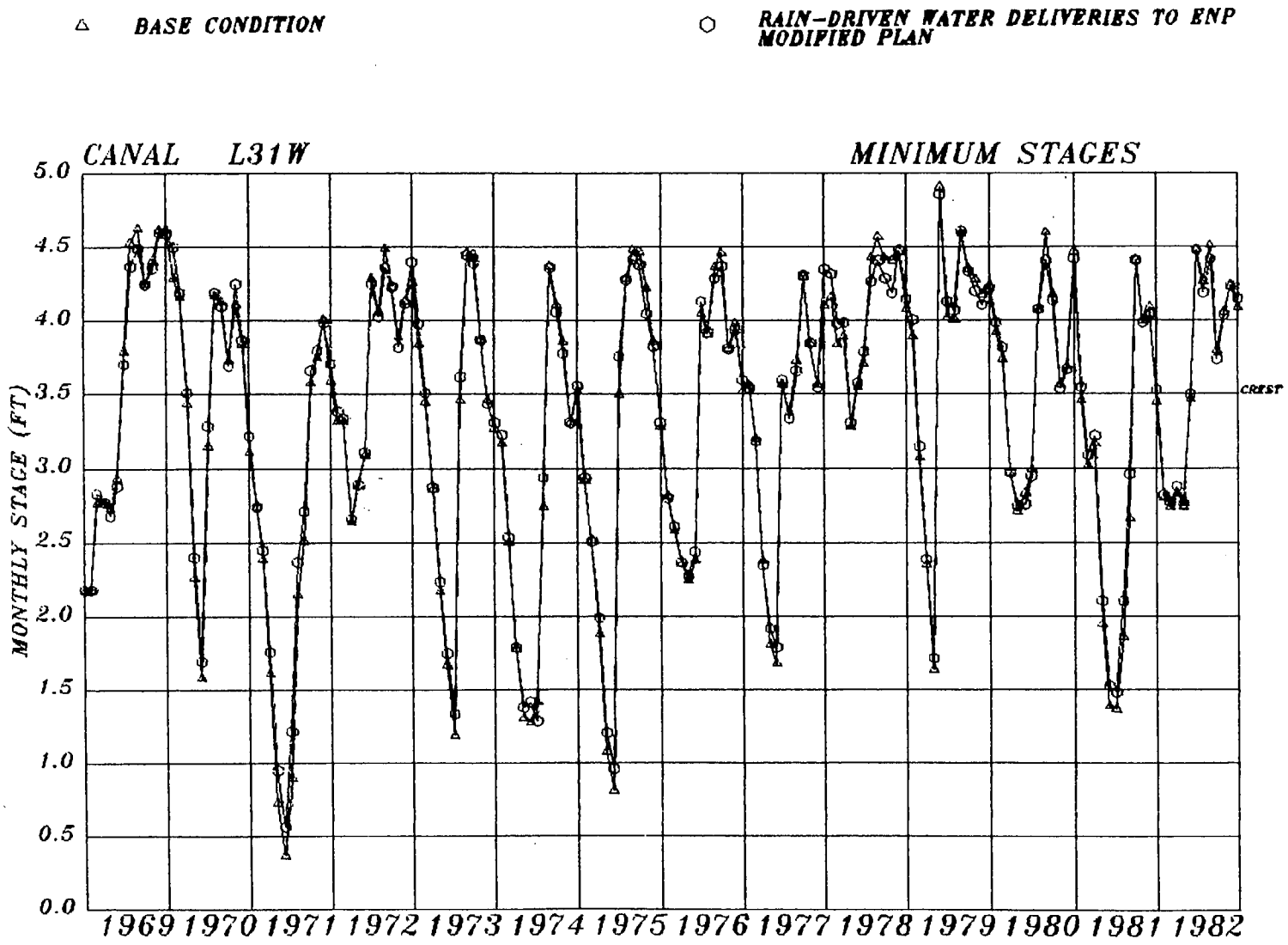
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L31W
MINIMUM STAGES
MONTHLY STAGE (FT)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-145

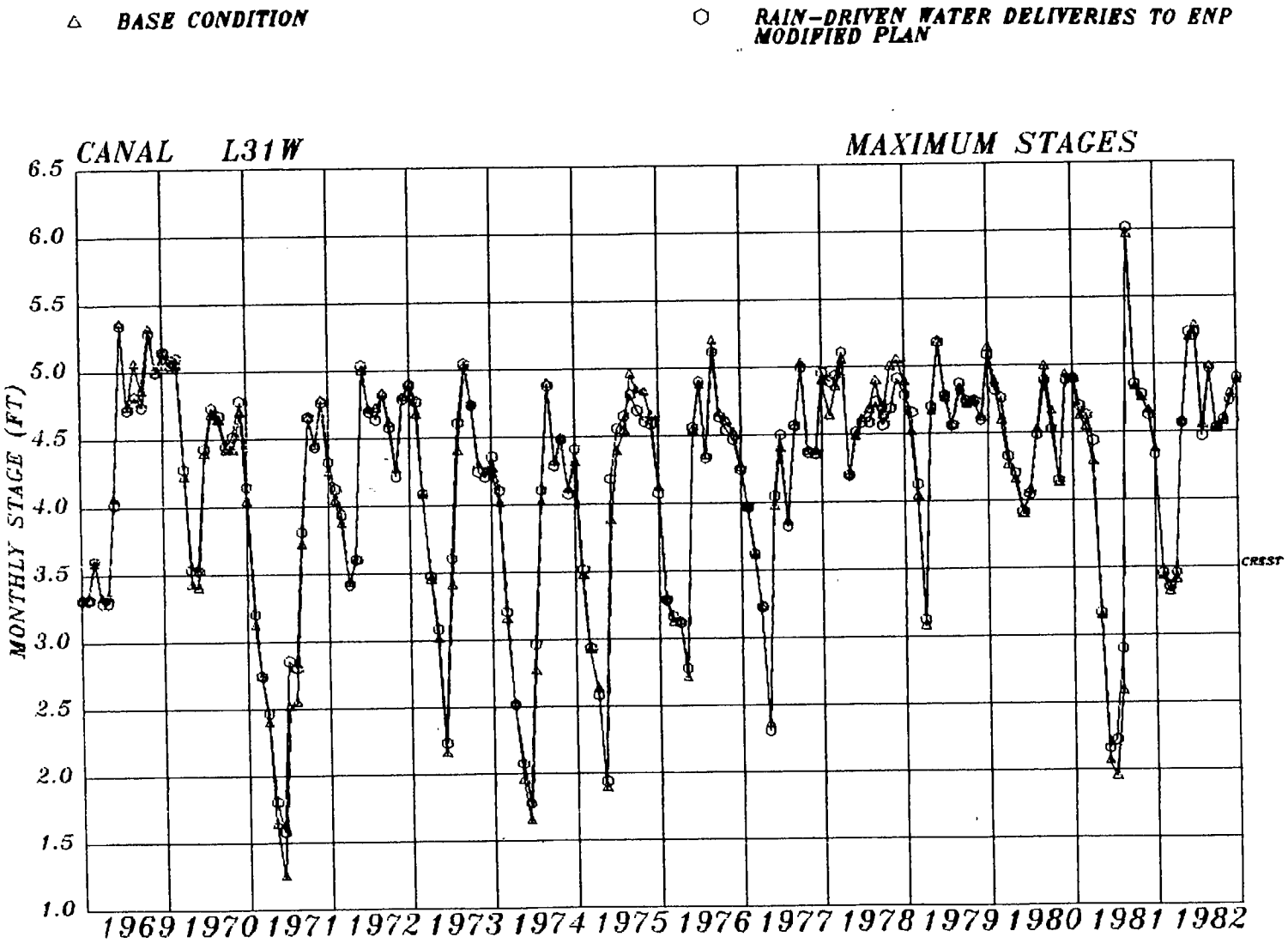
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
CANAL L31W
MAXIMUM STAGES
MONTHLY STAGE (FT)
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
CREST
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-146

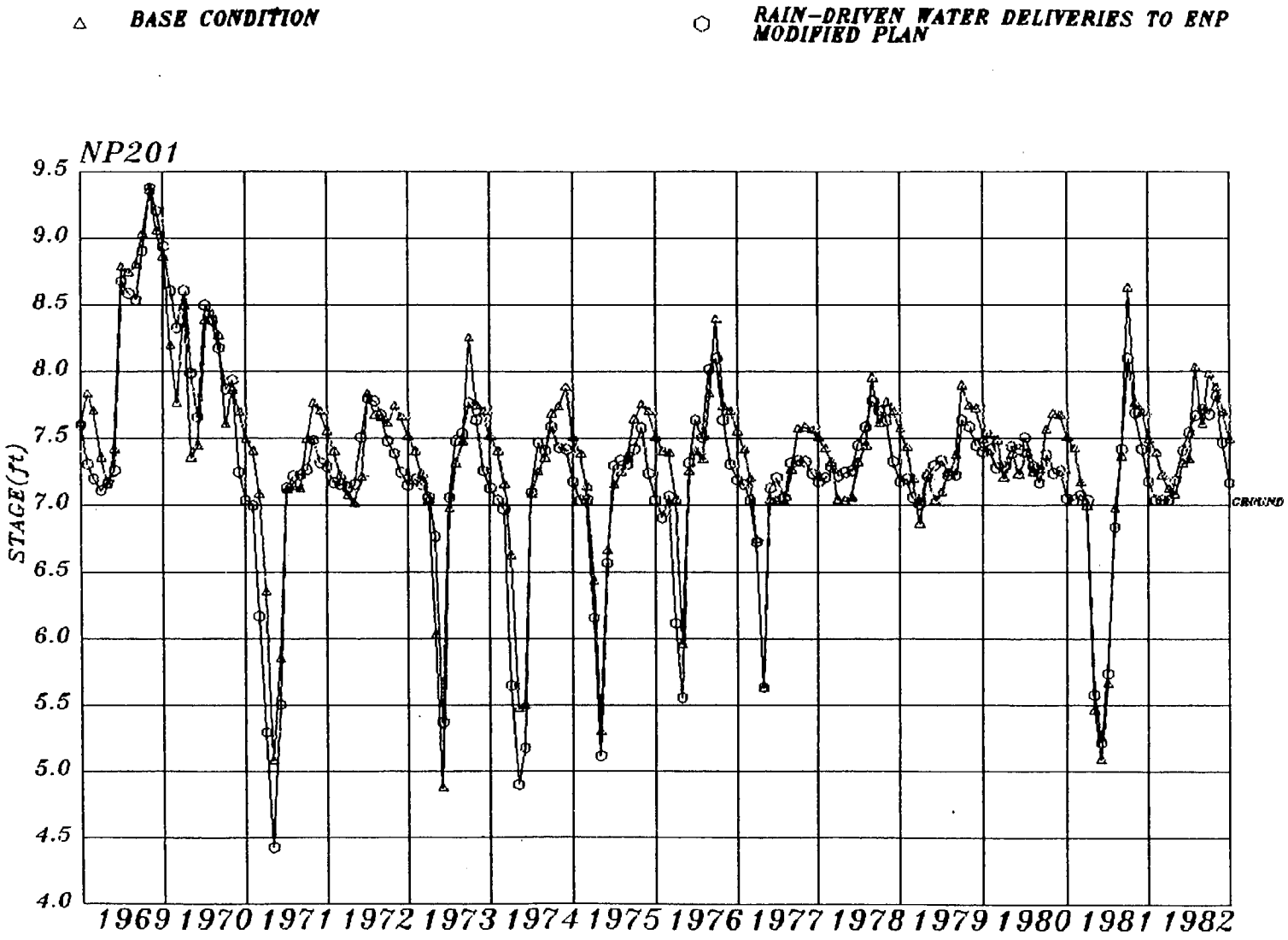
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NP201
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982
GROUND

Figure E-147

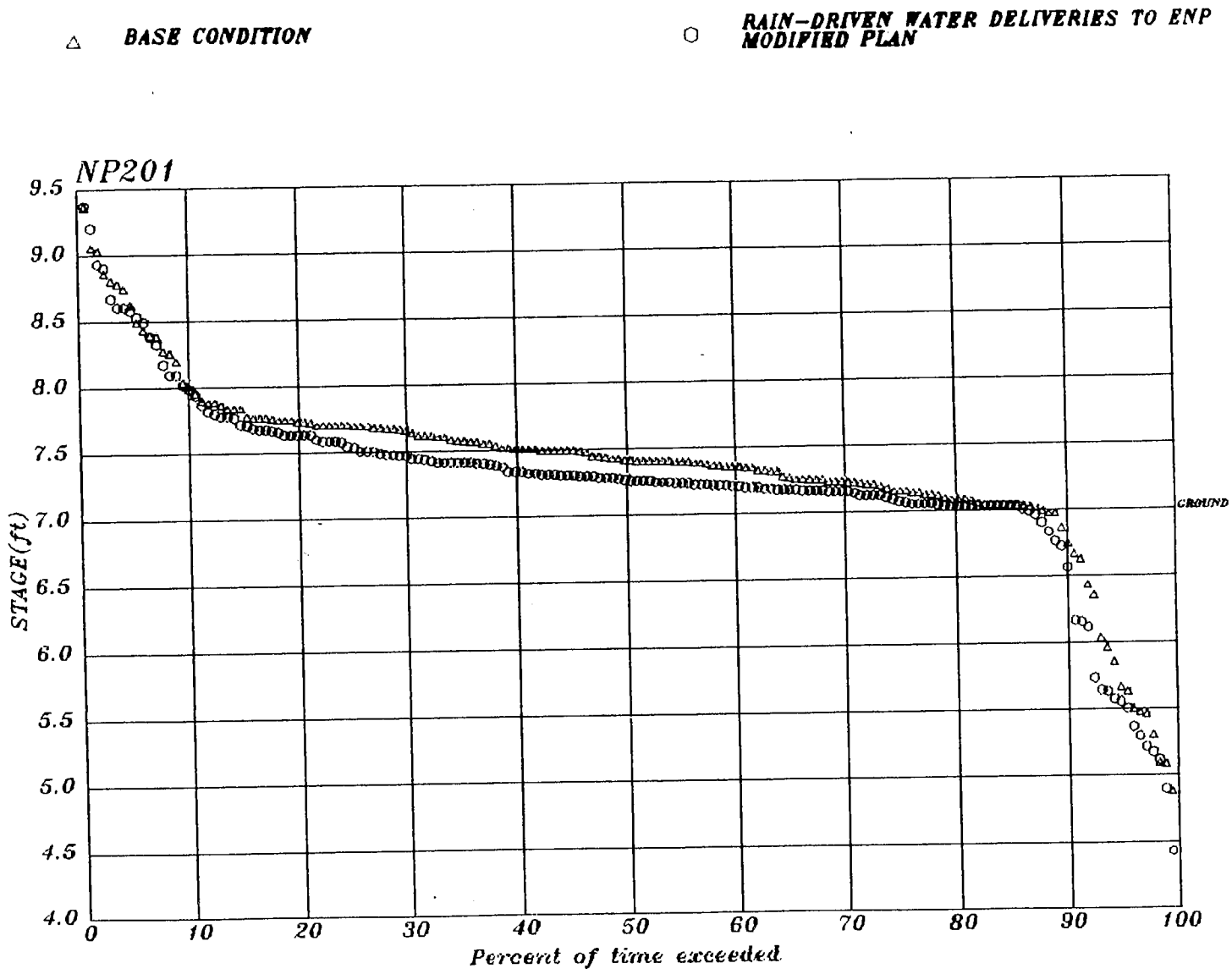
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NP201
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-148

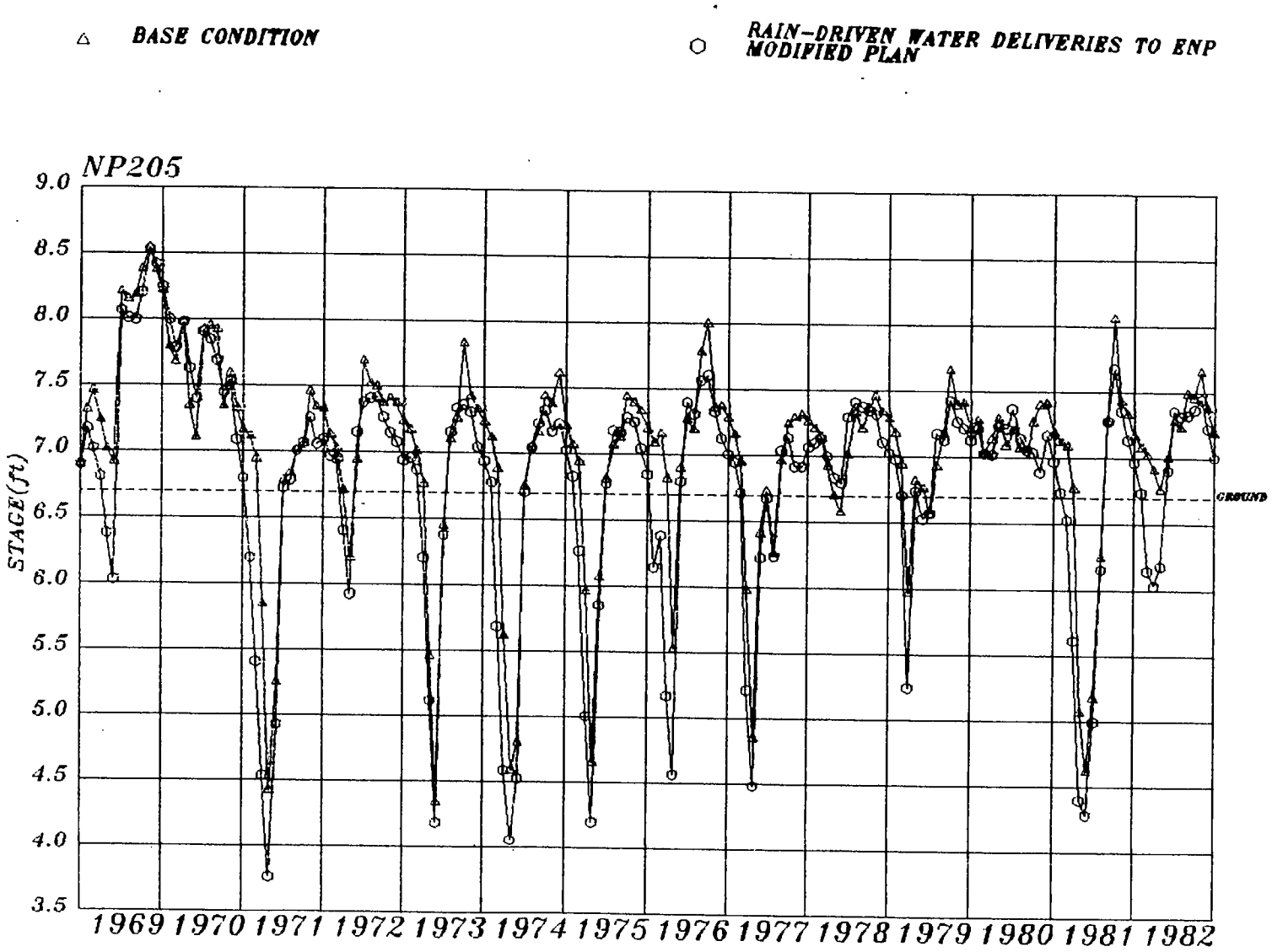
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NP205
STAGE(ft)
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-149

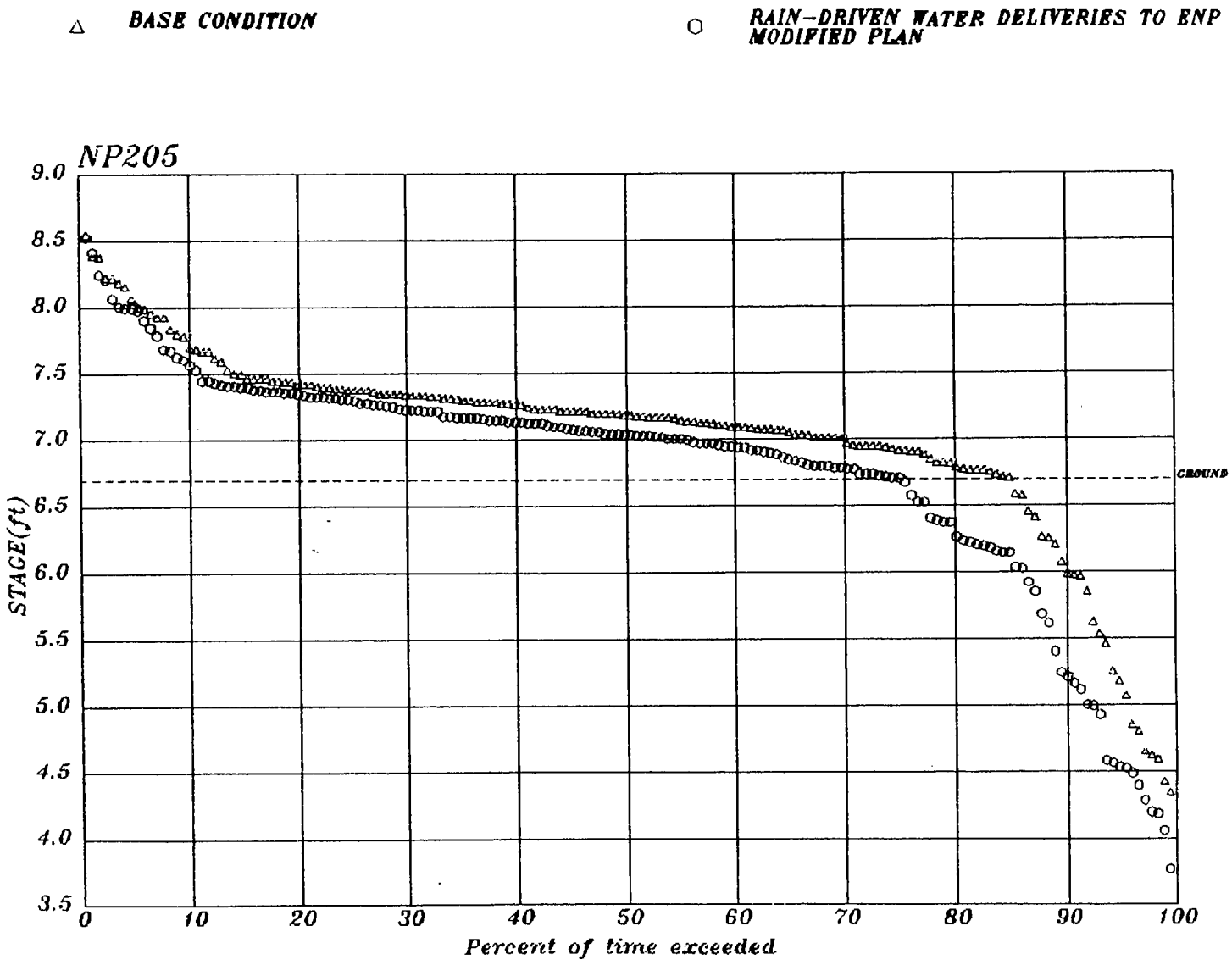
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NP205
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-150

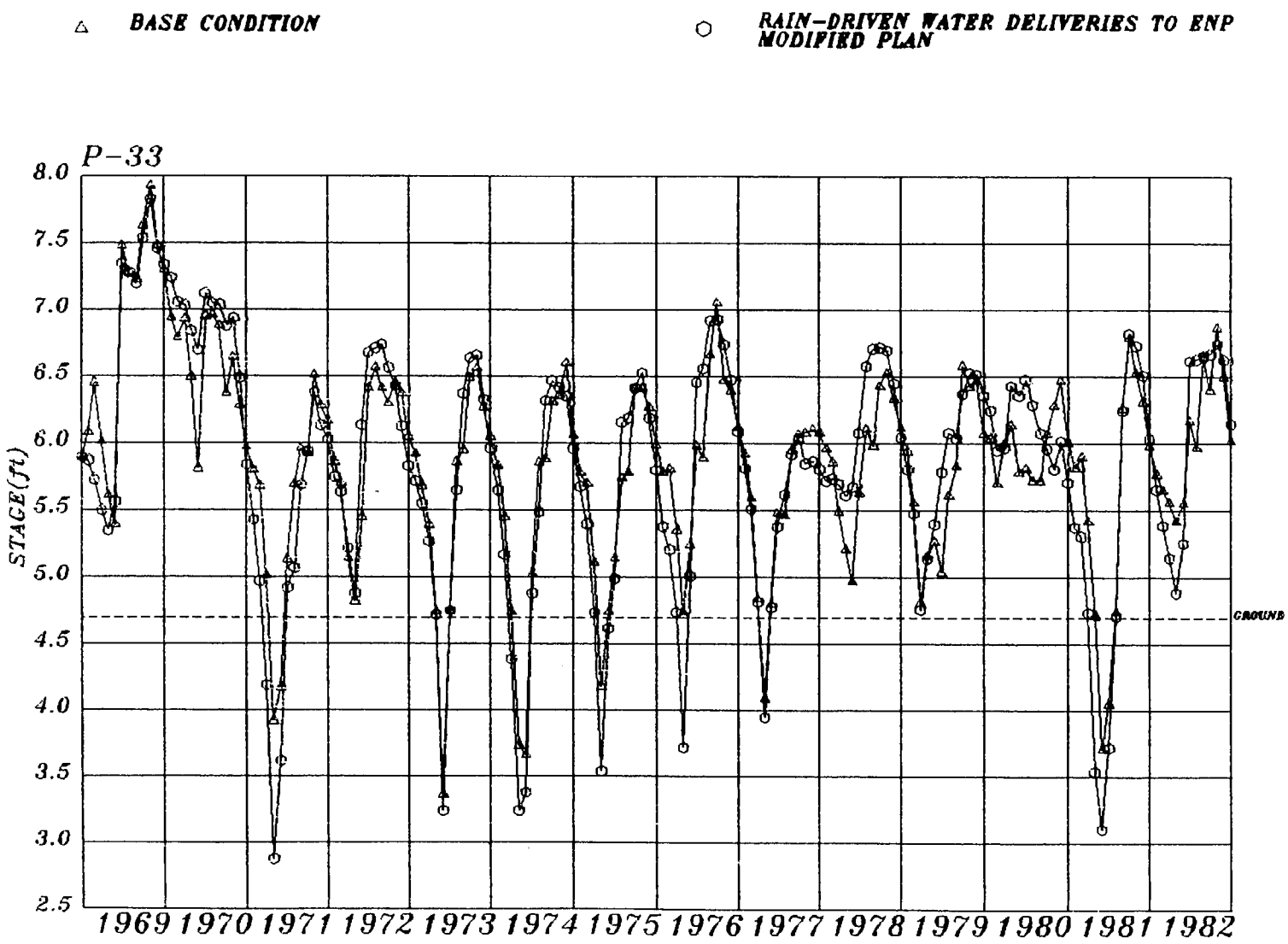
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
P-33
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-151

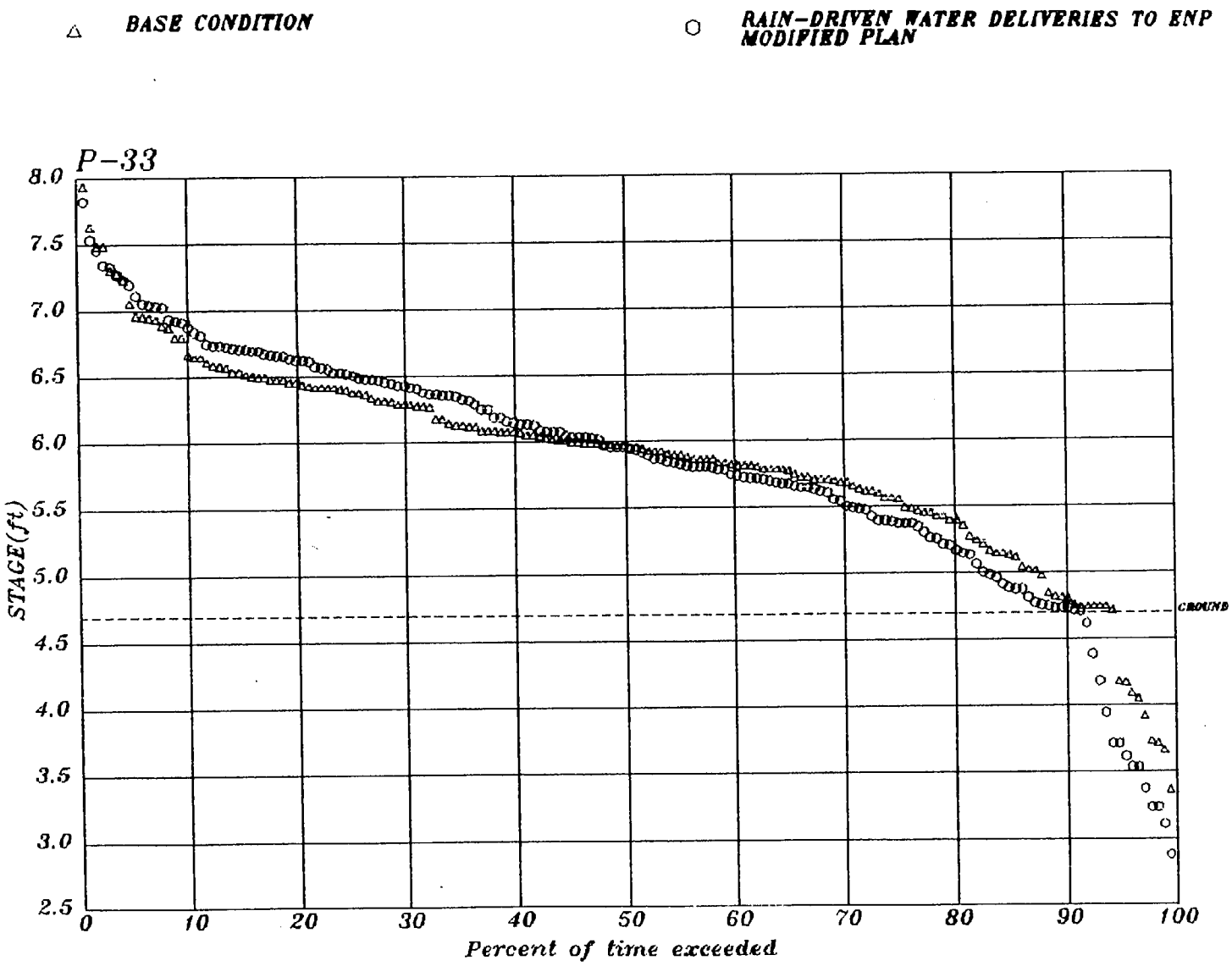
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
P-33
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
STAGE(ft)
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-152

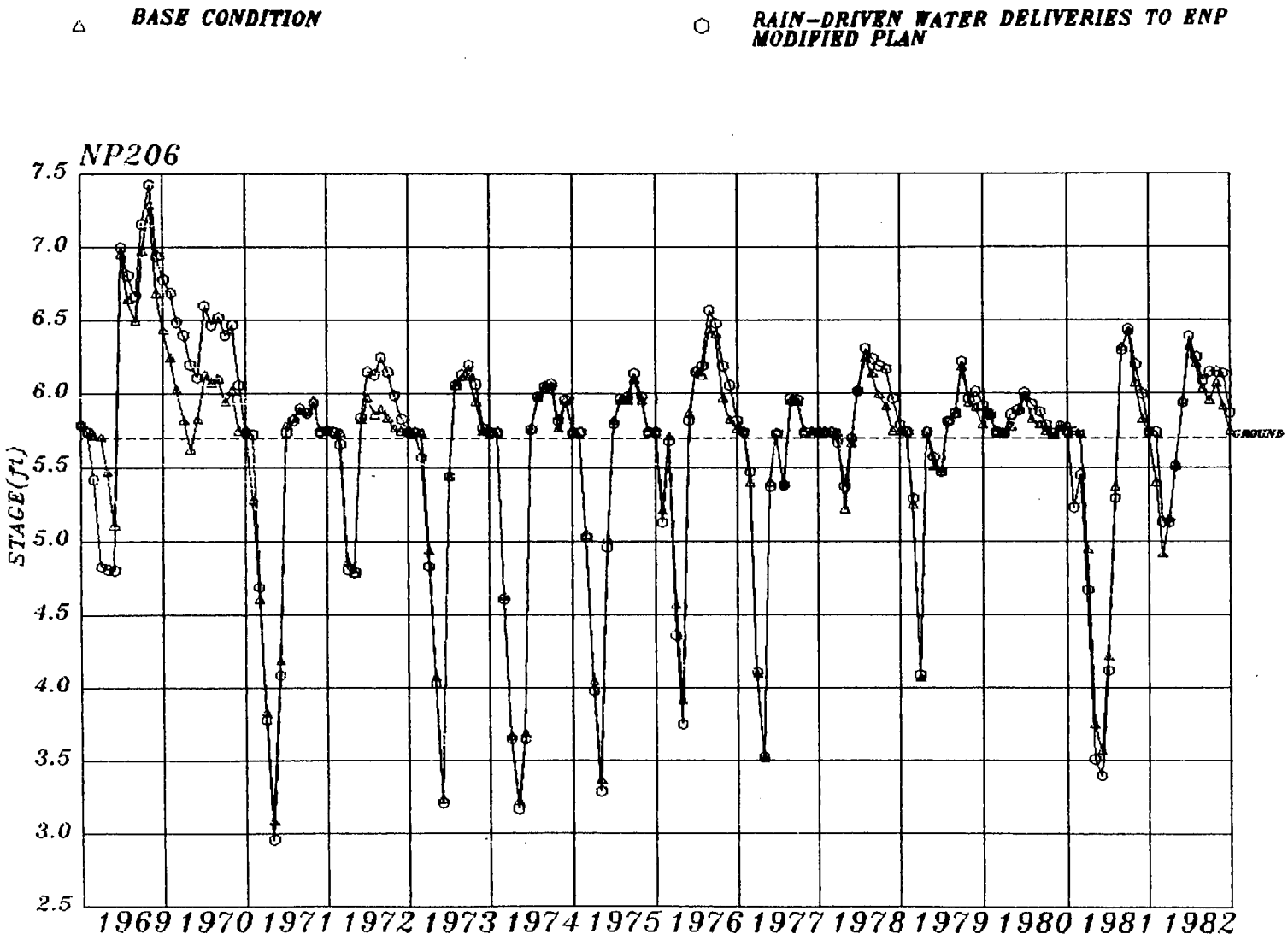
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NP206
STAGE(ft)
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-153

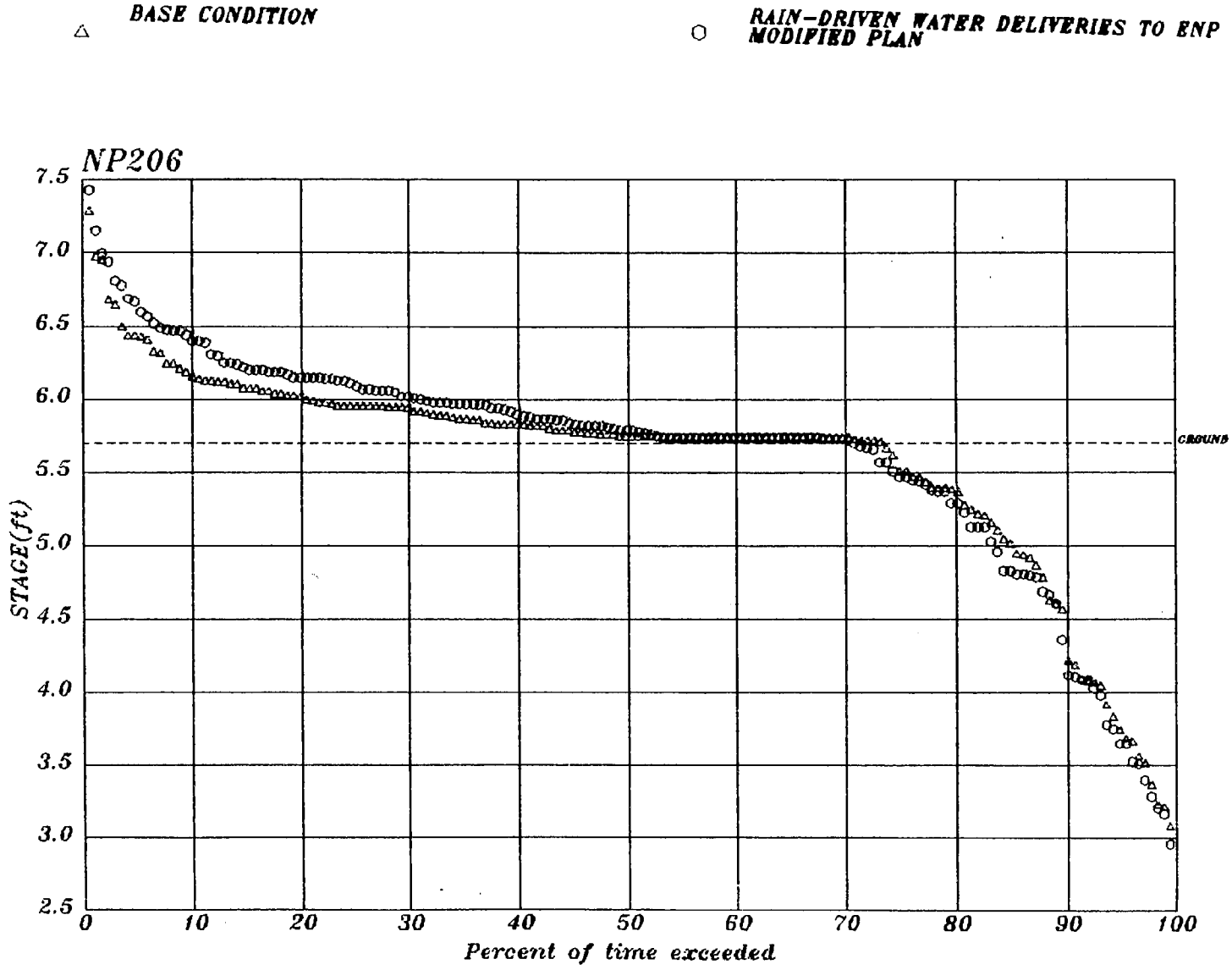
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NP206
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-154

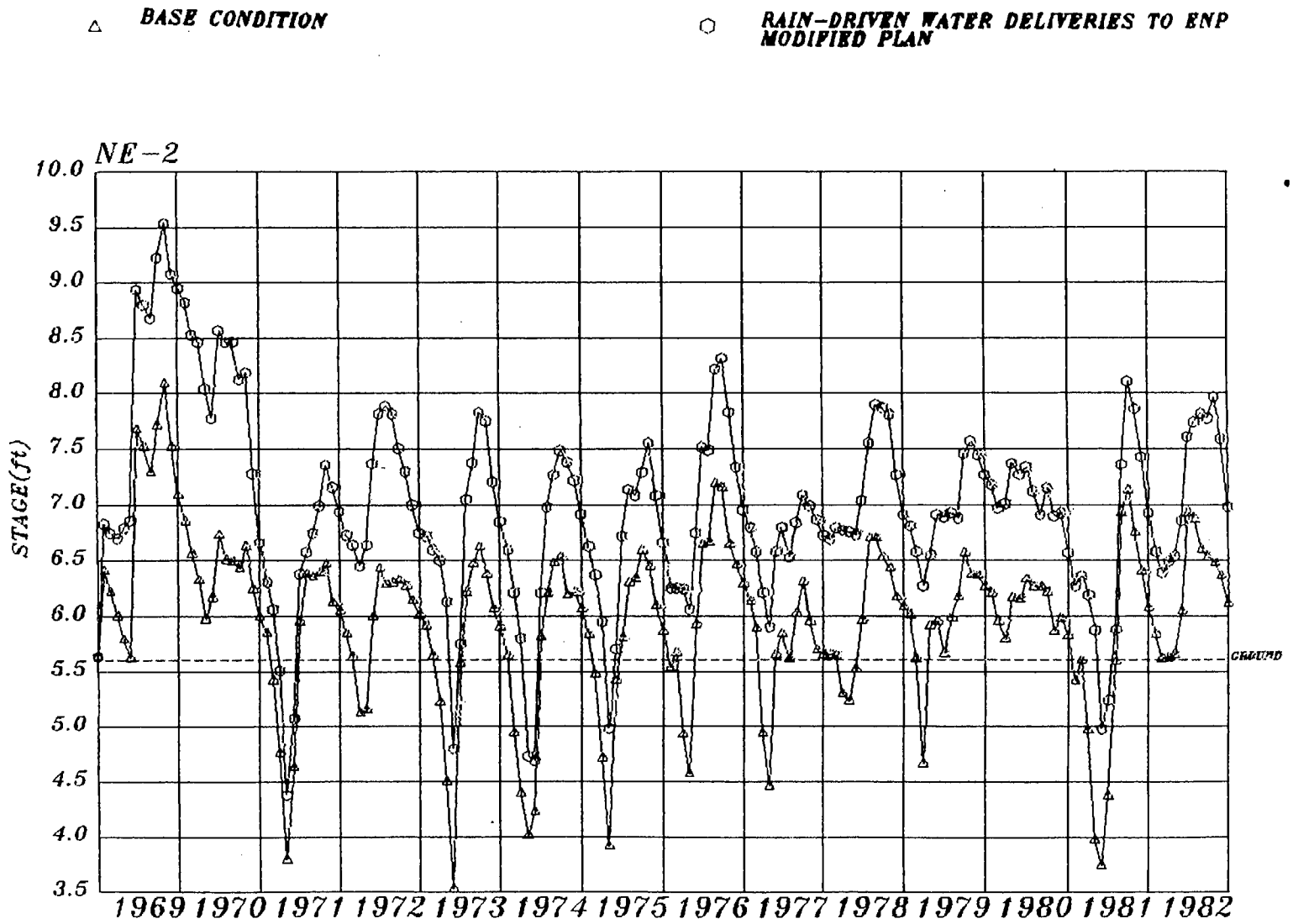
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NE-2
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
GROUND
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-155

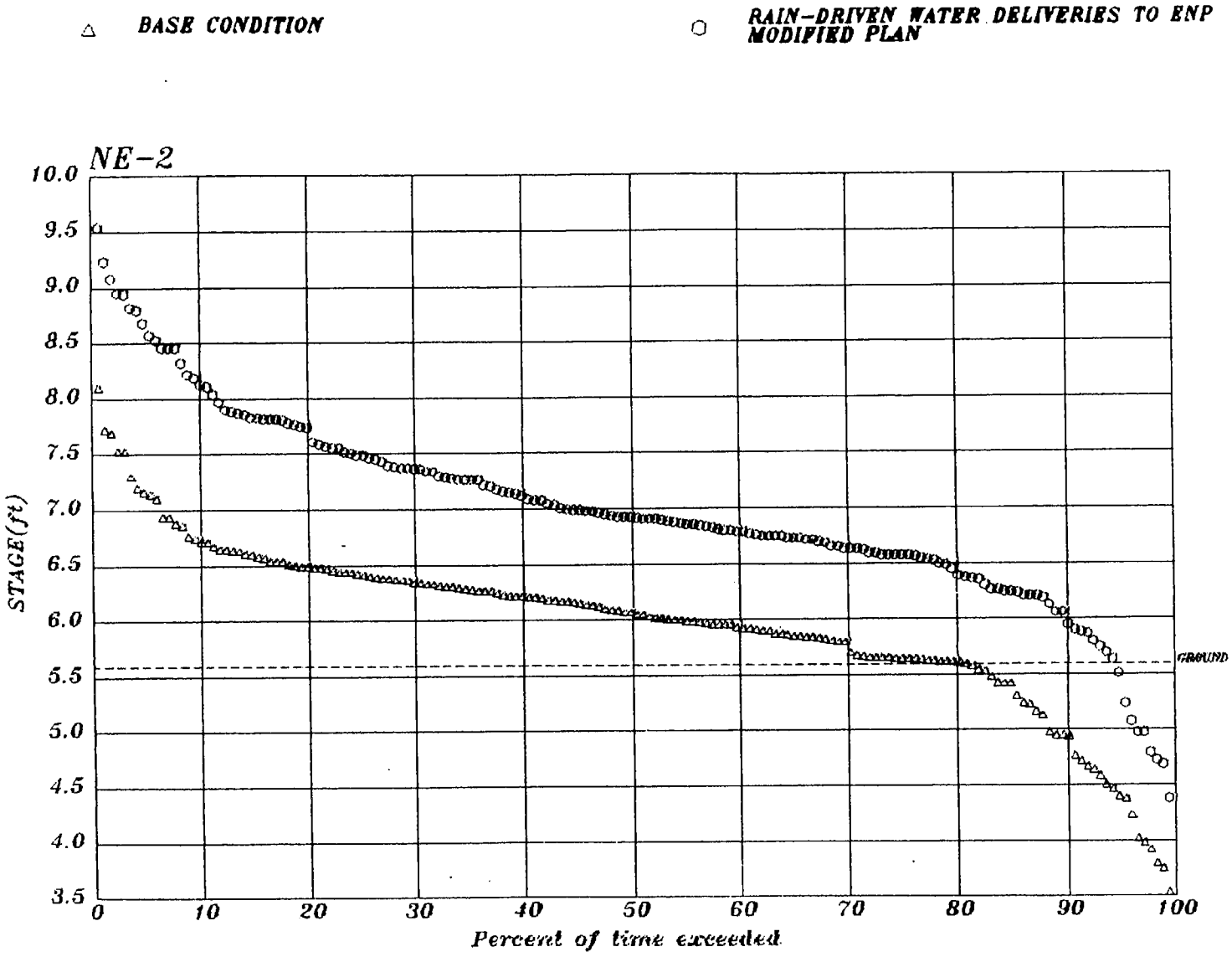
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NE-2
STAGE(ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded
GROUND

Figure E-156

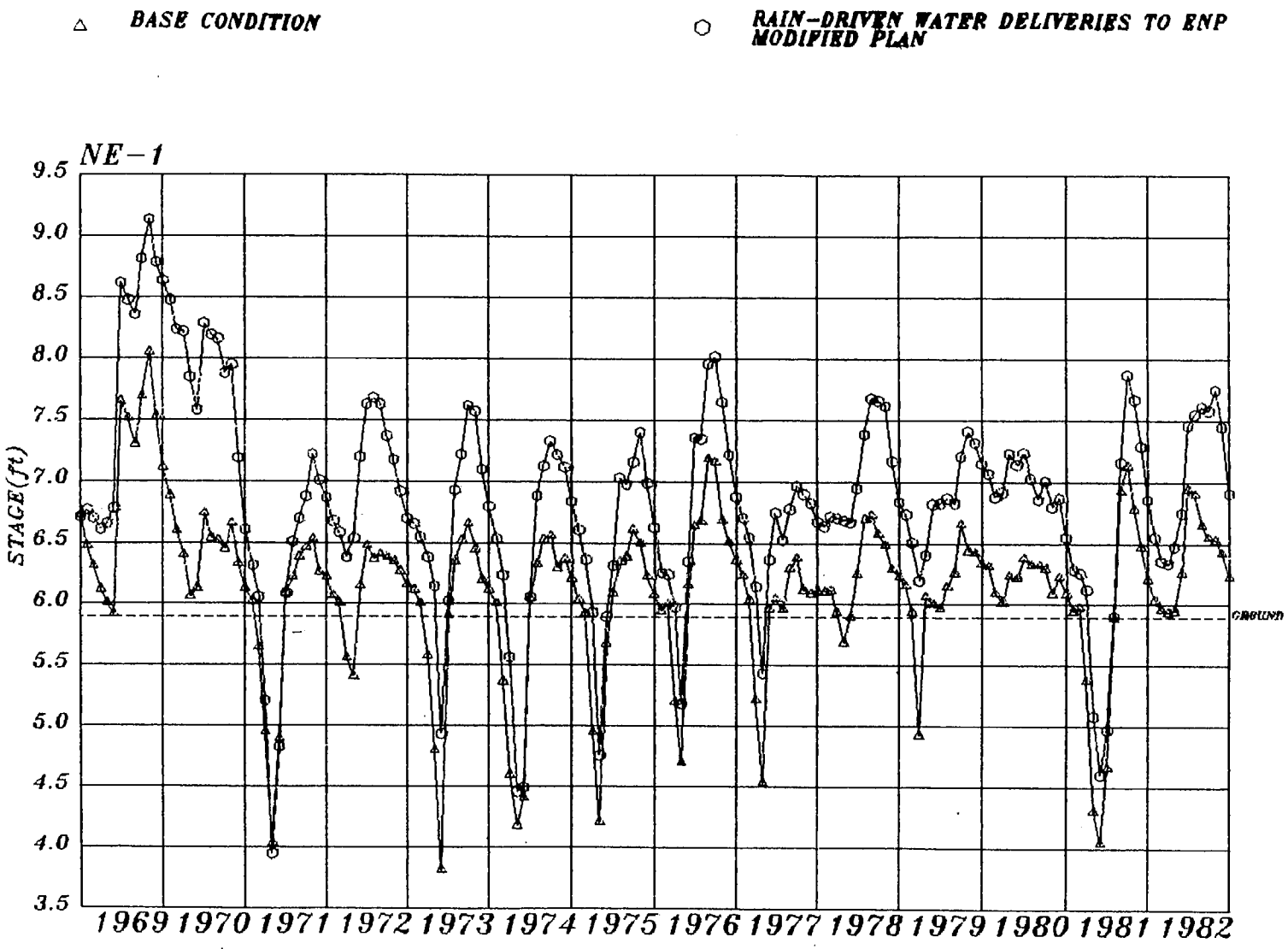
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
NE-1
STAGE(ft)
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-157

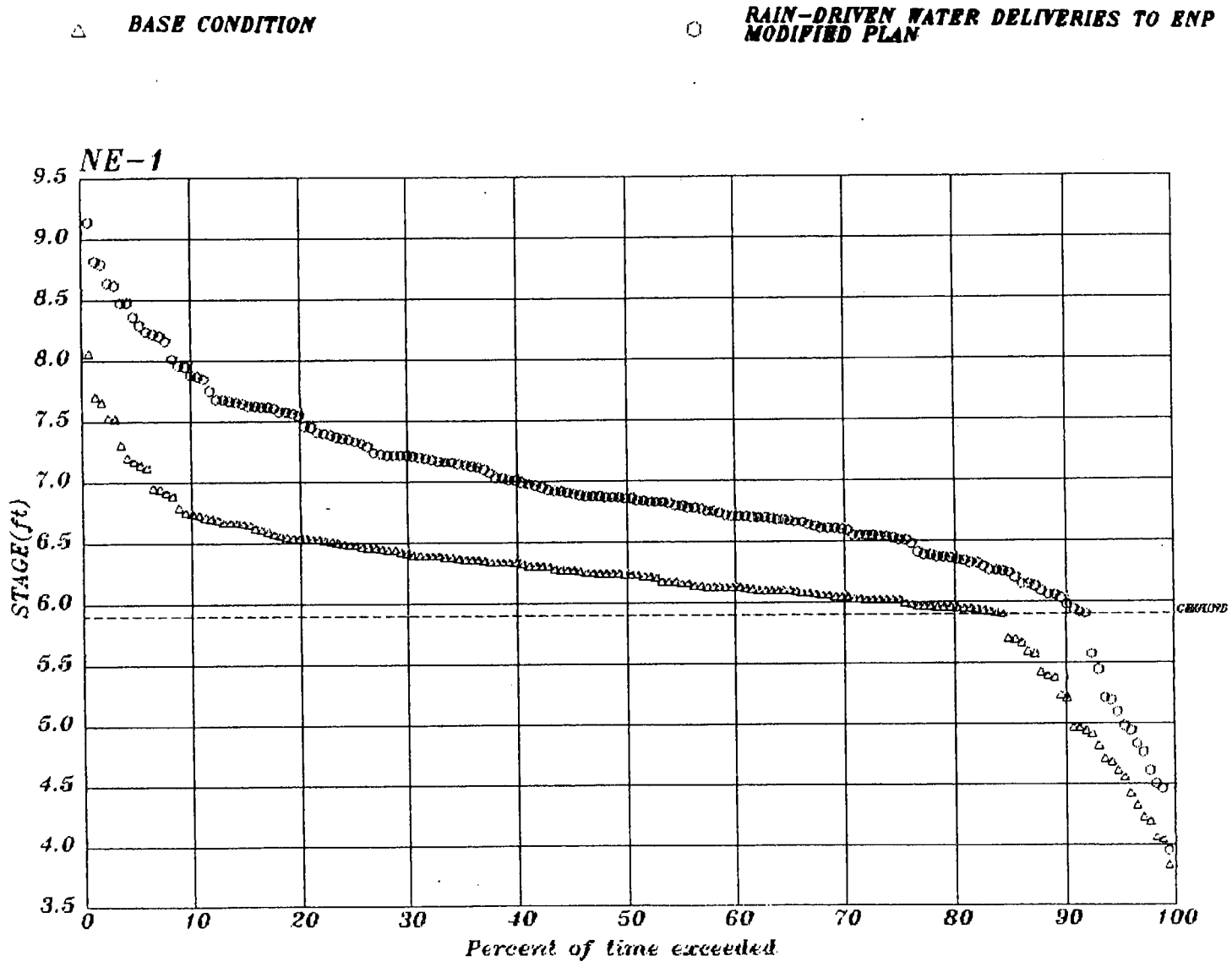
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP
MODIFIED PLAN
NE-1
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
STAGE(ft)
GROUND
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-158

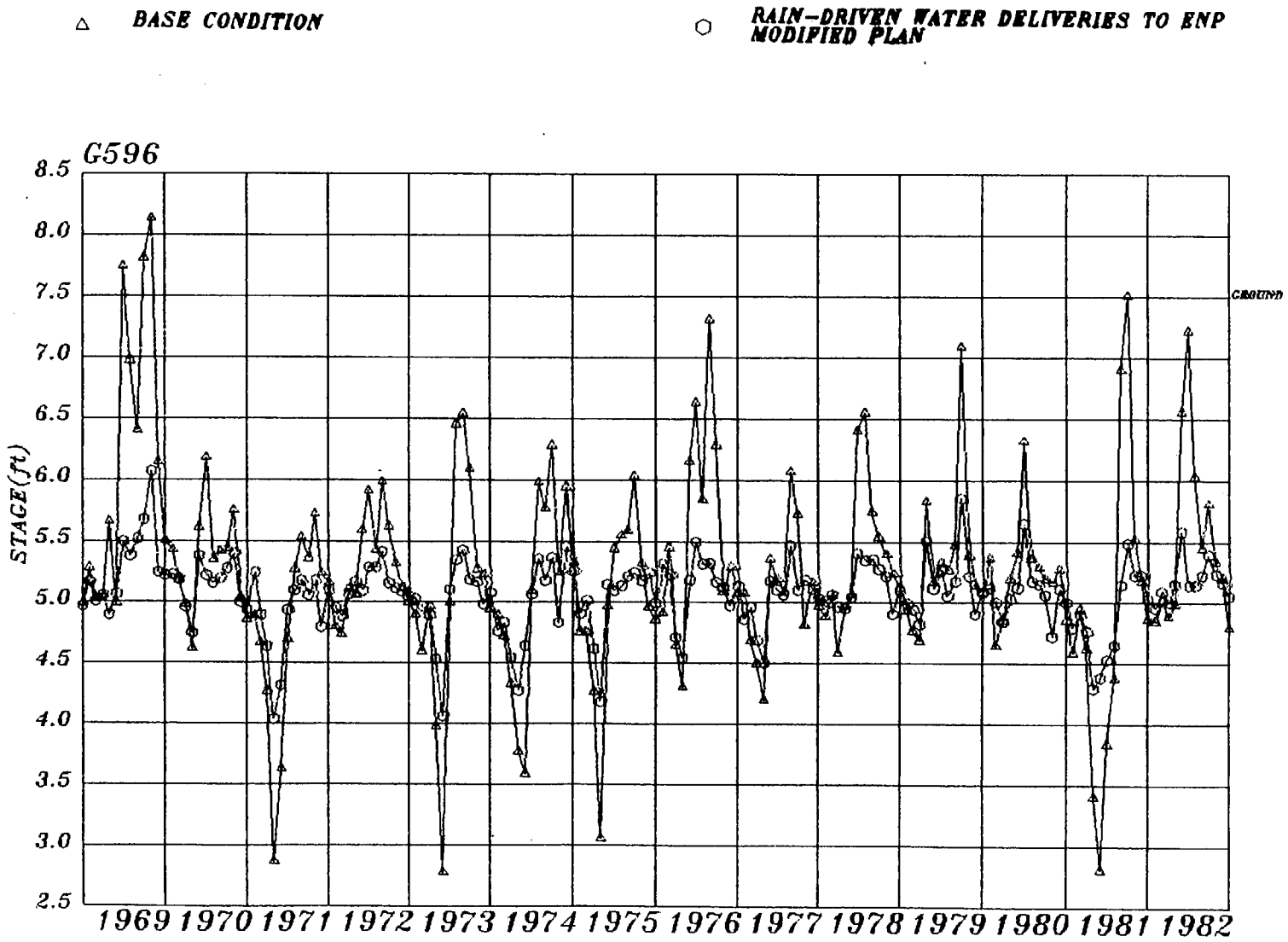
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
G596
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-159

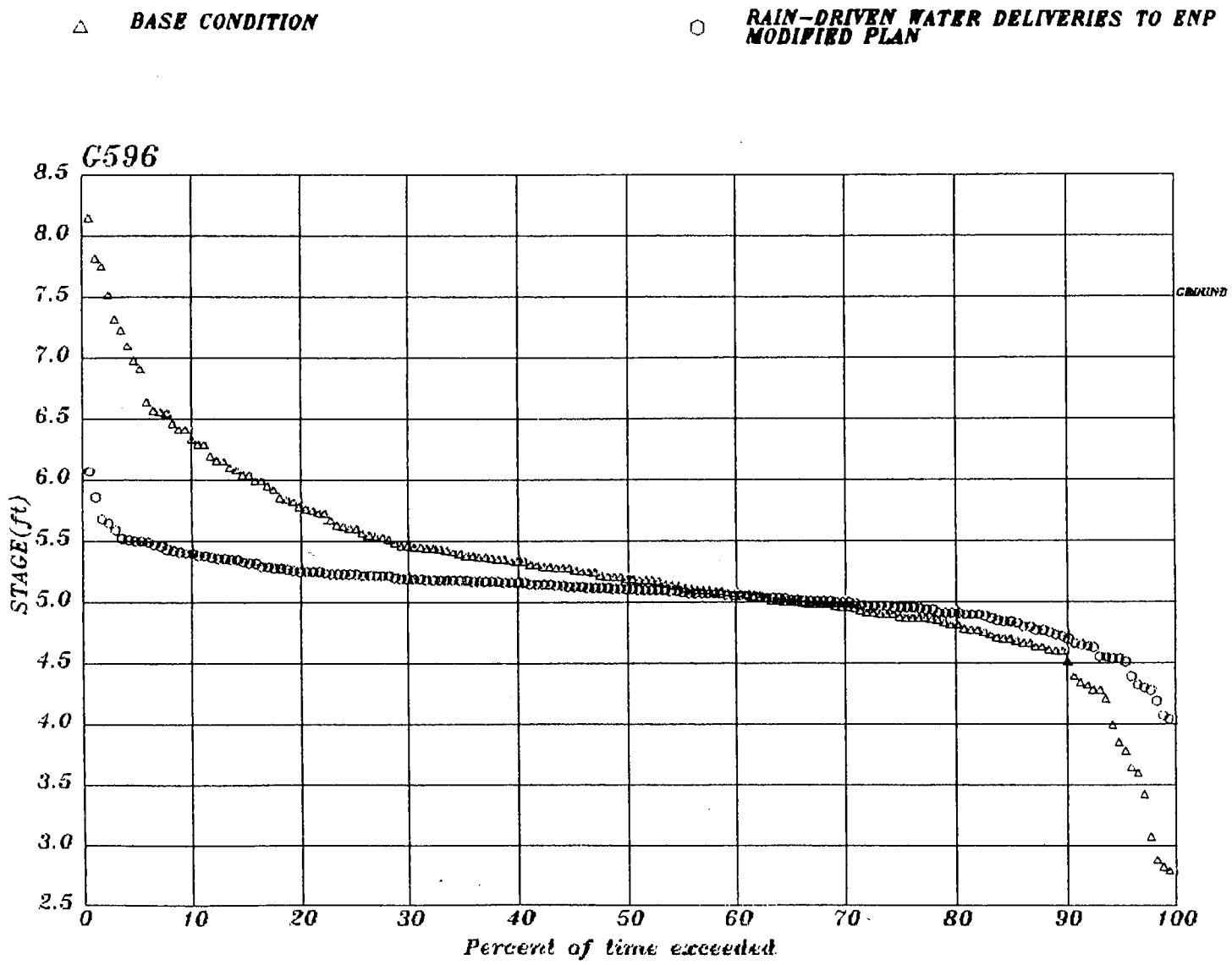
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
G596
GROUND
STAGE(ft)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-160

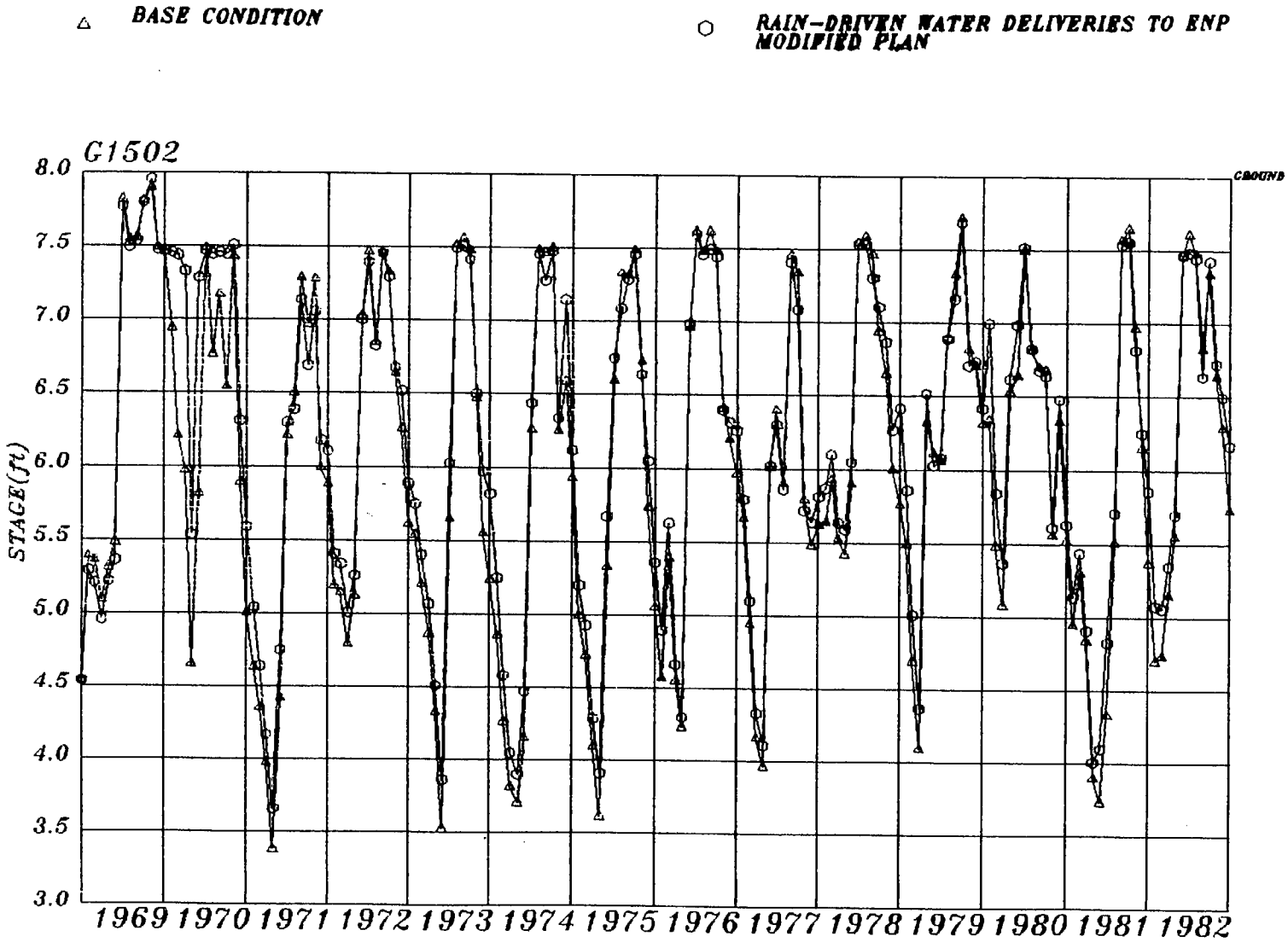
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
G1502
GROUND
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-161

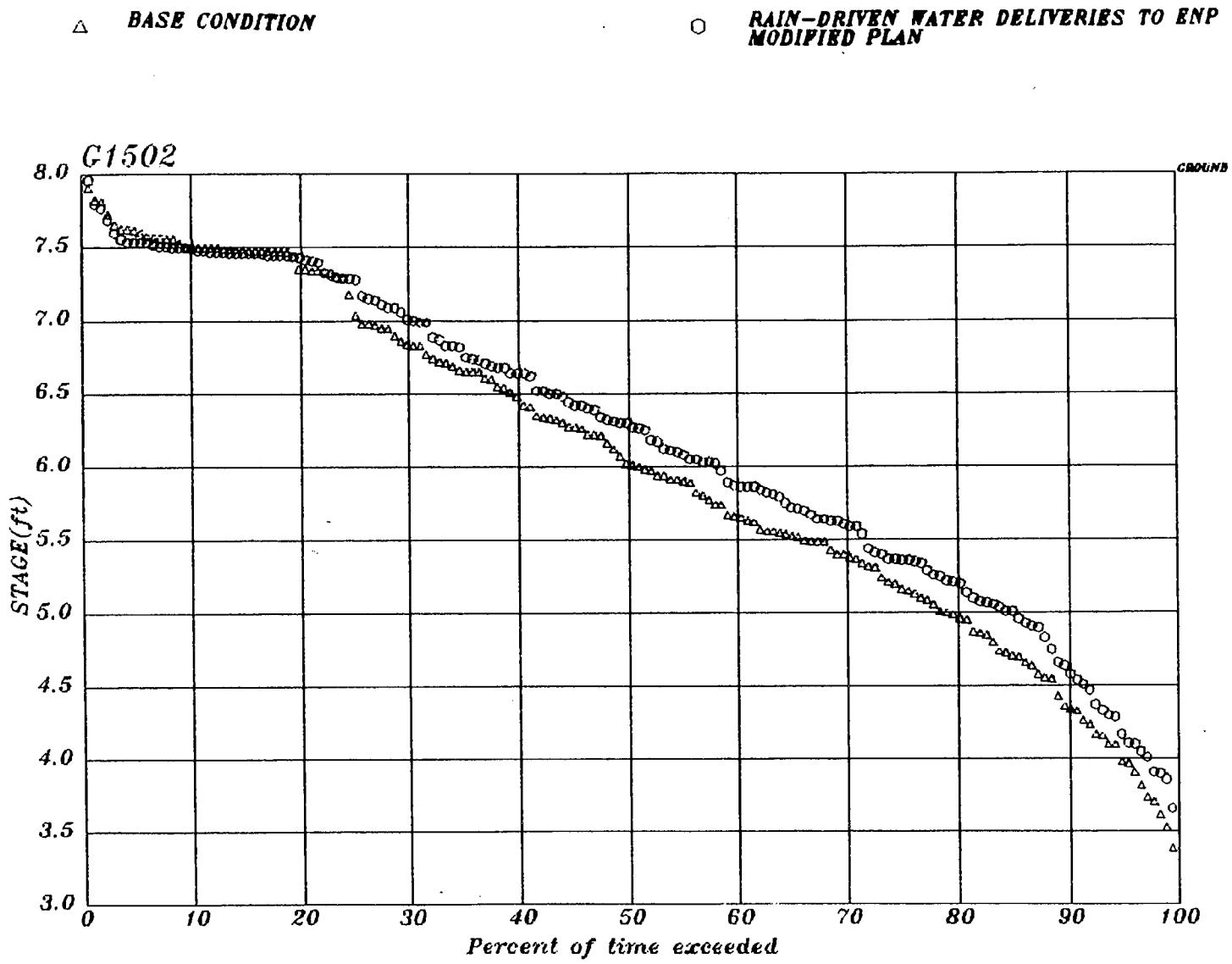
△ BASE CONDITION
○ RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
G1502
GROUND
STAGE(ft)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
0
10
20
30
40
50
60
70
80
90
100
Percent of time exceeded

Figure E-162

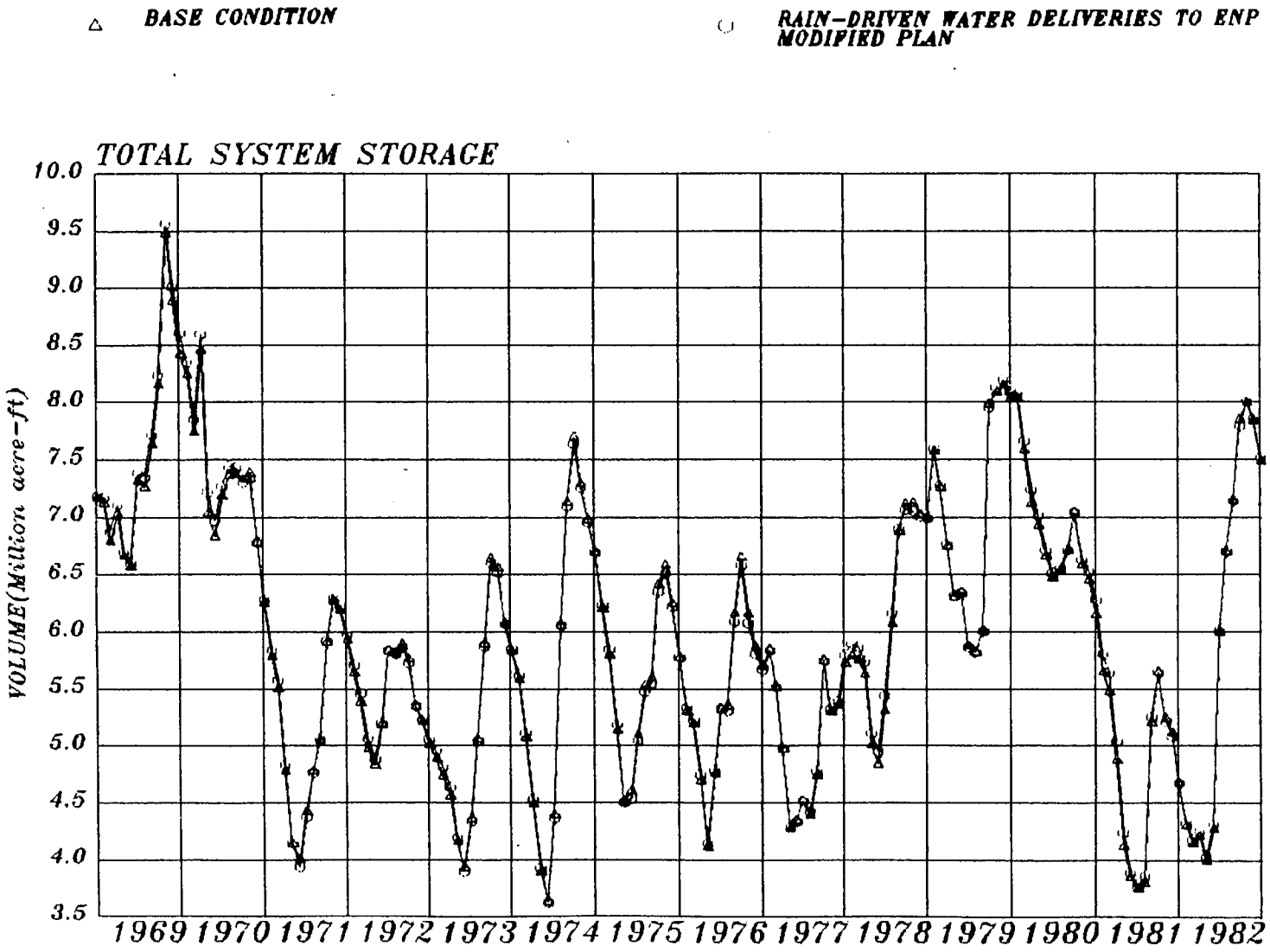
BASE CONDITION
RAIN-DRIVEN WATER DELIVERIES TO ENP MODIFIED PLAN
TOTAL SYSTEM STORAGE
VOLUME(Million acre-ft)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
1969 1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982

Figure E-163

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX F

STRUCTURE OPERATIONS OF ALTERNATIVE PLANS

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX F

STRUCTURE OPERATIONS OF ALTERNATIVE PLANS

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APPENDIX F

STRUCTURES OPERATION OF ALTERNATIVE PLANS

A. BASE CONDITION

1. General. This appendix provides a detailed description of how each alternative has been modeled in the South Florida Water Management Model (SFWMM). Variations between alternative plans are primarily made in the ROUTE subroutine of the model. This subroutine provides coding for operational rules at each structure. The remainder of the model is documented in Technical Publication 84-3, South Florida Water Mangement Model Documentation Report, South Florida Water Management District.

2. Lake Okeechobee.

a. General. Inflows to Lake Okeechobee are historical rainfall and historical inflows from the Kissimmee River, Taylor Creek-Nubbin Slough; and the northwest and northeast shores and runoff from the EAA as computed in the model. Outflows are computed for lake regulation and water supply to the EAA and the WCA's. The current regulation schedule (15.5 - 17.5 ft, NGVD) is used for Lake Okeechobee (Plate 5), with the Interim Action Plan in affect.

b. S-77. The following discharges are used:

45 cfs when Lake Okeechobee is in Zone C.
4500 cfs when Lake Okeechobee is in Zone B.
9300 cfs when Lake Okeechobee is in Zone A.

c. S-308. Since Zone B discharges through the St. Lucie Canal are limited at S-80 to 2500 cfs, the S-308 discharge was reduced by 12% to account for local inflow to the canal between S-308 and S-80. The following discharges are used:

45 cfs when Lake Okeechobee is in Zone C.
.88*(2500) cfs when Lake Okeechobee is in Zone B.
4000 cfs when Lake Okeechobee is in Zone A.

d. Releases to the Water Conservation Areas. Water supply transfers are made from Lake Okeechobee to the WCA's when the following conditions are satisfied:

(1) the Lake Okeechobee stage is above 9.5 feet,
(2) the WCA stage is below its specified level, and
(3) the Lake stage is greater than the WCA stage.

Water deliveries are called for when the WCA No. 3A stage is at, or below 8.5 feet, when the WCA No. 2A stage is at, or below 10.0 feet, or when

the WCA No. 1 stage is at, or below 12.5 feet. If the three conditions described above are satisfied, water deliveries to the WCA's are limited to the available capacities of the pump station at the southern end of the EAA. When pumping for flood control in the EAA is being performed up to capacity at the southern pump station, no water supply releases are made from the lake. When the flood control pumping is less than capacity, water supply releases from the lake are added to increase pumping up to capacity. When no flood control pumping is necessary, water supply deliveries from the lake are made up to capacity.

Water supply releases from the lake to WCA No. 3A are based on the available capacity at S-8. The total capacity at S-8 is computed as follows:

```
S8CAP = 1543.17 * STAGELO ** 3 - 1587.5 * STAGE 3A ** 3
      - 25,220 * (STAGELO - STAGE3A) ** 1.5 - 3,810,000/STAGELO
      + 399,227
```

Where S8CAP = S-8 total discharge capacity in cfs.
STAGELO = Lake Okeechobee stage in feet.
STAGE3A = WCA No. 3A stage in feet.

Water supply releases from the lake to WCA No. 2A are based on the available capacity at S-7. The total S-7 capacity is computed as follows:

```
S7CAP = 1423.07 * STAGELO ** 3 - 1331.59 * STAGE2A ** 3
      + 179,454.9 * STAGE2A - 1,551,100 * STAGELO ** 0.5
      + 3,069,177
```

Where S7CAP = S-7 total discharge capacity.
STAGELO = Lake Okeechobee stage in feet.
STAGE2A = WCA No. 2A stage in feet.

Water supply releases from the lake to WCA No. 1 are based on available capacities at S-5A and S-6. The total capacities are computed as follows:

```
S6CAP = 600 when STAGELO > 13.0.
S6CAP = 600.0 * (STAGELO - 12.0) when 13 > STAGELO > 12.0.
S6CAP = 0.0 when STAGE < 12.0.
```

Where S6CAP = S-6 total capacity in cfs.
STAGELO = Lake Okeechobee stage in feet.

```
S5CAP = 8.75 * (STAGELO - 4.0) * (0.073 * STAGELO - 0.168)
      * (STAGELO - STAGA1) ** 0.5
```

Where S5CAP = S-5 total capacity in cfs.
STAGELO = Lake Okeechobee stage in feet.
STAGE1 = WCA No. 1 stage in feet.

3. Everglades Agricultural Area (EAA). The EAA is divided into four basins which are served by S-8, S-3, and HGS-3; S-7, S-2, and HGS-4; S-6; and S-5A and HGS-5. The water level in each of these basins is maintained between 1.0

and 2.0 feet below the ground level, if possible. When the groundwater level is within 1.0 foot of the ground surface, runoff is removed from the basin. When the groundwater level is more than 2.0 feet below the ground surface, water supply is delivered to the basin. The total volume of water released from, or pumped into Lake Okeechobee for water supply or flood control, respectively, for the four basins is increased by 48%. This adjustment is made to account for agricultural lands served by the lake that are outside the model area.

The volumes of water removed from or added to the agricultural basins are computed into two steps. First, the total volumes of water that should be added to or removed from the basins to maintain stage between one and two feet below ground are computed in the AGAREA subroutine. Then, these volumes are adjusted in the ROUTE subroutine if there is not adequate structural capacity for water supply.

a. Water Supply Cutbacks. Water supply releases from Lake Okeechobee to the EAA are incrementally reduced when the lake stage falls below defined levels. The cutback schedule is as follows:

TABLE F-1

Lake Okeechobee Cutback Schedule
for Water Supply to EAA

Lake Okeechobee Stage	Cutback of EAA Demand (%)
≤ 9.8 ft. NGVD	90
≤ 5 feet below Zone B	75
≤ 4 feet below Zone B	25
> 4 feet below Zone B	No Cutback

The physical capability of the system is checked after the above cutback water supply deliveries are computed and the actual deliveries are adjusted accordingly, if necessary.

b. Interim Action Plan. The Interim Action Plan is used in this model. Accordingly, flood runoff from the EAA basins is first pumped into the WCA's until the capacity of the southern pumps is exceeded. Then pumping runoff from the EAA into Lake Okeechobee is initiated.

c. S-8, S-3, and HGS-3 Basin. The following structure capacities are used:

S-3 Capacity = 3,000 cfs.
S-8 Capacity = 4,200 cfs.
HGS-3 Capacity = 4,000 cfs.

It should be noted that the capacity used for HGS-3 is based on the assumption that the structure will be rehabilitated to its current design capacity. Water supply releases from Lake Okeechobee into this basin do not

exceed the HGS-3 capacity even if the demands have been reduced by the cut-back schedule.

For flood control in the EAA, the following equation is used to determine if pumping at S-3 is required; in addition to S-8 pumping:

RLOKN8= 0.002759* Q** 1.5 - 17.525* QLOG+17.6

Where
RLOKN8 = total discharge (in cfs) required at S-3, in addition to S-8 pumping, to prevent water levels in the S-8 basin from exceeding the upper desirable limit.
QLOG = LOG10 (total flood control discharge (in cfs) required for the S-8 basin).

If RLOKN8, as computed above, is greater than zero, pumping at S-3 is required. However, S-3 pumping is limited by the defined capacity. If no S-3 pumping for flood control is required, S-8 pumps the necessary runoff into WCA No. 3A, up to its design capacity.

d. S-7, S-2, and HGS-4 Basin. The following structure capacities are used:

S-2 Capacity = 2,000 cfs.
HGS-4 Capacity = 2,000 cfs.
S-7 Capacity = 4,000 cfs.

Note: The S-2 and HGS-4 capacities are limited to 50% of the actual capacities and the remaining 50% of the structure capacities are reserved for the S-6 basin.

It should be noted that the capacity used for HGS-4 assumes the structure has been rehabilitated to its design capacity. Water supply releases to the S-7 basin do not exceed the capacity at HGS-4, even if demands have been reduced by the cutback schedule.

For flood control in the EAA, the following equation is used to determine if S-2 pumping is necessary; in addition to S-6 pumping.

*RLOKS7= (0.003042 * (TOTQ**1.5) + 10.86* (TOTQ ** 0.5)
-144.12* QLOG + 24.19)* PCTS7

Where
RLOKS7 = total discharge (in cfs) required at S-2, in addition to pumping at S-7, to prevent water levels in the S-7 basin from exceeding the maximum desirable level.

TOTQ = the sum of flood control discharge from the S-7 and S-6 basins (in cfs) required to prevent water levels in either basin from rising above the upper desirable limit.

QLOG = LOG10 (TOTQ).
PCTS7 = ratio of the total flood control discharge required from the S-7 basin to TOTQ.

If RLOKS7 is greater than zero, S-2 and S-7 pump up to capacity to maintain desirable water levels in the S-7 basin. If RLOKS7 is zero, or less, only S-7 pumps as necessary for flood control.

e. S-6 Basin. The following structure capacities are used:

S-2 Capacity = 2,000 cfs.
HGS-4 Capacity = 2,000 cfs.
S-6 Capacity = 4,000 cfs.

Note: The capacities at S-2 and HGS-4 are reduced by 50% of the actual structural capacities and the remaining capacity at each structure is reserved for the S-7 basin.

Water supply releases to the S-6 basin do not exceed the defined HGS-4 capacity even if the demands have been reduced by the cutback schedule. For flood control, the following equation is used to determine if S-2 pumping is necessary in addition to S-6 pumping:

```
RLOKS6 = (0.003042*TOTQ*1.5 + 10.86* (TOTQ)**0.5
         -144.72* QLOG + 24.19)* PCTS6
```

Where
RLOKS6 = total discharge (in cfs) required at S-2, in addition to pumping at S-6, to prevent water levels in the S-6 basin from exceeding maximum desirable levels.
TOTQ = the total discharge (in cfs) required from the S-6 and S-7 basins to prevent water levels from exceeding the maximum desirable levels.
QLOG = LOG10 (TOTQ).
PCTS6 = ratio of the discharge required from the S-6 basin to TOTQ.

If RLOKS6 is greater than zero, S-2 and S-6 pump flood water runoff from the S-6 basin up to capacity. If RLOKS6 is zero, or less, S-6 pumps flood water runoff from the S-6 basin up to its capacity, as necessary.

f. S-5A, HGS-5, and BEL5A: The following structure capacities are used:

S-5A capacity = 4,000 cfs.
HGS-5 capacity = 3,000 cfs.

It should be noted that the HGS-5 capacity used assumes that the structure is rehabilitated to it's design capacity. Water supply releases from Lake Okeechobee are limited to the HGS-5 capacity, even if the demands are reduced by the cutback schedule. Flood control pumping at S-5A is

conducted to prevent water levels in the S-5A basin from exceeding the maximum desirable level. Pumping is conducted as necessary up to capacity.

4. Water Conservation Area (WCA) No. 1. WCA No. 1 is operated according to the current (14.0 - 17.0 ft, NGVD) regulation schedule. Gage 1-8T stages are used in the regulation of WCA No. 1. Regulatory releases are made through S-10 and water supply releases are made through S-39. When the WCA No. 1 stage is above 12.5 ft, NGVD, water supply releases are made from storage. When the WCA No. 1 stage is 12.5 ft, NGVD or less, water supply releases at S-39 do not exceed the combined inflow from S-6 and S-5A.

a. S-10. Regulatory releases are made as follows:

When the WCA No. 1 stage > 15.5 ft, NGVD
Q = 1746.7 * (WCA No. 1 stage - 15.5) + 703.0
When 15.5 ≥ WCA No. 1 > 14.0
Q = 468.7 * (WCA No. 1 stage - 14.0)

No S-10 discharges are made when WCA No. 1 is below schedule.

b. S-39. Water supply releases are made at S-39 when the Hillsboro Canal stage falls below 6.5 ft, NGVD. However, the water supply releases do not exceed the structure discharge capacity based on the following criteria:

If (WCA No. 1 stage - 10.5) > 0.0
Q (Capacity)= 953.0 * (WCA No. 1 stage - 10.5) ** .523

If (WCA No. 1 stage - 10.5) ≤ 0.0
Q (Capacity)= 0.0

Where WCA No. 1 stage = WCA No. 1 stage in ft, NGVD.

Water supply requirements for the Hillsboro Canal are computed as follows:

If HLSB ≤ 5.5
Q (Water Supply)= Q (Capacity)
If 5.5 < HLSB < 6.5
Q (Water Supply) = Q (Capacity)*(6.5 - HLSB)
If HLSB ≥ 6.5
Q (Water Supply) = 0.0

Where HLSB = stage in the Hillsboro Canal in ft, NGVD.

When the WCA No. 1 stage falls below 12.5 ft, NGVD, the S-39 discharge does not exceed to total inflow to WCA No. 1 through S-5A and S-6.

c. Pumpage from Hillsboro Canal to Agricultural Area in Northern Broward County. Water supply deliveries are made from the Hillsboro Canal to two local canals when the canal stages fall below 14.5 ft, NGVD. Water supply releases are made into a local east-west canal south of the Hillsboro Canal when its stage falls below 14.5 ft, NGVD up to a maximum of 98 cfs. These

releases satisfy demands in the Deerfield Agricultural District and are computed as follows:

Q (Water Supply) = 65.3*(14.5-CHDEP)

Where CHDEP = local canal's stage in ft, NGVD.

Water supply deliveries are also made to one of three Lake Worth Drainage District canals. All three of these canals receive water supply directly from WCA No. 1 but LWD 2 is supplemented by water from the Hillsboro Canal. These supplemented water deliveries are initiated when the LWD 2 canal stage falls below 14.5 ft, NGVD and are made up to a maximum ratio of 130 cfs according to the following equation:

Q (Water Supply) = 260. * (14.5 - LWD2)

Where LWD2 = LWD2 canal stage in ft, NGVD.

All water supply deliveries from the Hillsboro Canal are discontinued when the canal stage falls below 6.0 ft, NGVD.

d. Lake Worth Drainage District Pumps. Water supply to recharge three canals in the Lake Worth Drainage District is pumped from WCA No. 1. If water levels in any one of three canals, LWD1, LWD2, or LWD3, falls to 14.0 ft, NGVD or less, pumping starts at 220 cfs and continues until the canal stage(s) rise to 15.0 ft, NGVD. No water supply is pumped if the WCA No. 1 stage falls to 14.0 ft, NGVD or less.

5. Water Conservation Area (WCA) No. 2. WCA No. 2A is operated according to the current authorized (11.0 - 13.5 ft, NGVD) regulation schedule (Plate 6). The Gage 2-17 stage is used for comparison with the area's regulation schedule. Regulatory releases are made through S-11 and water supply releases are made through S-34 and S-38. When the WCA No. 2 stage is above 10.0 ft, NGVD, water supply releases are made from storage. When the WCA No. 2A stage is 10.0 ft, NGVD or less, water supply releases do not exceed the inflow at S-7.

a. S-11. Discharges are made through S-11 only when the WCA No. 2A stage is above schedule and are computed as follows:

If DEL2A3A > 1.04 ft,
    Q = 999.0 * (DEL2A3A - 1.04) + 3214.0
If 1.04 ≥ DEL2A3A > 0.38 ft,
    Q = 4870.0 * (DEL2A3A - 0.38)
If DEL2A3A ≤ 0.38
    Q = 0.0

Where DEL2A3A is the head across S-11 and is computed as the upstream water level minus the downstream water level at the structures. When the discharge computed above is greater than 2,000 cfs, the following adjustment is made:

$$Q = 2000.0 + (Q\ (Computed) - 2000.0) * 0.65$$

Where Q (Computed) = the computed S-11 discharge in cfs based on head cross the structure.

b. S-34. This structure is used to satisfy water supply requirements in the North New River Canal (L-35 borrow canal) service area up to the structure's capacity. Water supply deliveries are made when the stage at gage G617 is below 3.5 ft, NGVD to satisfy water supply requirements based on the following equation:

$$Q\ (Water\ Supply) = -400.0 * (G617 - 2.5) + 400.0$$

Where G617 = stage at gage G617 in ft, NGVD.

Water supply releases do not exceed the structure capacity based on the upstream water level and no releases are made if the upstream stage is 8.0 ft, NGVD or less. The structure capacity is computed using the following equation:

$$Q\ (Capacity) = 147.0 * (L38 - 8.0) ** 0.5$$

Where L38 = stage in L38 borrow canal in ft, NGVD.

When the WCA No. 2A stage is 10.0 ft, NGVD or less, the S-34 discharge used is the minimum of the required supply discharge, the structure capacity, or the inflow at S-7 minus the outflow at S-38.

c. S-38. This structure satisfies water supply requirements of the C-14 service area based on water levels in C-14 up to the structure's capacity. Water supply deliveries are made when the canal stage is below 6.5 ft, NGVD when possible and are computed by the following equation.

If the C-14 stage $\leq$ 6.0
    Q (Water Supply) = structure capacity
If the C-14 stage < 6.5
    Q (Water Supply) = (Structure Capacity)*(6.5-STG C-14)
If C-14 stage is greater than or equal to 6.5
    Q (Water Supply) = 0.0

Where STG C-14 = stage in C-14 in ft, NGVD.

Water supply releases do not exceed the structure's capacity based on upstream water levels and no releases are made if the upstream stage is 10.0 ft, NGVD, or less. The structure's capacity is computed using the following equation:

$$Q\ (Capacity) = 151.0 * ((L38 - 10.0)**0.5)$$

Where L38 = stage in the L38 borrow canal in ft, NGVD.

When the WCA No. 2A stage is 10.0 ft, NGVD or less, the S-38 discharge is the minimum of either the required water delivery, the structure's capacity, or the inflow at S-7.

6. Water Conservation Area (WCA) No. 3. WCA No. 3A is operated according to the current (9.5 - 10.5 ft, NGVD) regulation schedule. The average of water levels at gages 3-3, 3-4, and 3-28 are used for comparison with the regulation schedule. Regulatory releases are made through S-12, S-343A and B, S-344, and S-151. Water supply deliveries to Everglades National Park are made through S-12 in accordance with the requirements of Public Law 91-282. Other water supply requirements are satisfied by discharges through S-151 and S-32. S-339 and S-340 are operated to relieve high water conditions in northwestern WCA No. 3A under wet conditions and to facilitate water supply deliveries in the Miami Canal under dry conditions. S-346 is operated to facilitate water supply deliveries to ENP under dry conditions.

a. S-12. Regulatory releases are made through S-12 when the WCA No. 3A stage is above schedule and when the gage 3-28 stage is above 9.0 ft, NGVD. When the gage 3-28 stage is 9.0 ft, NGVD or less, no regulatory releases are made through S-12. Regulatory releases are computed as follows:

Q (Regulatory) = 1912.0 * ((3-28) - 9.0) + 1600.0

When WCA No. 3A stages are below schedule and greater than 6.9 ft, NGVD, the minimum monthly water deliveries required by PL 91-282 are provided, if there is adequate capacity based on head across S-12. Minimum water deliveries are computed as follows:

Q (Minimum Delivery) = VOLENP (Month)/86400.0
Q (Capacity) = 167.0 * (WCA No. 3A - 6.9) ** 2.5
Q (Water Delivery) = minimum of Q (Minimum delivery) or Q (Capacity)

Where: WCA No. 3A = three station average stage for WCA No. 3A in ft, NGVD.
VOLENP (Month) = the monthly minimum water delivery specified in PL 91-282 in acre-feet, as shown in Table F-2.

Table F-2
Minimum Monthly Water Deliveries to
ENP at Shark River Slough (VOLENP)

Minimum Water Delivery at S-12 (acre-feet)

January	22,000	July	7,400
February	9,000	August	12,200
March	4,000	September	39,000
April	1,700	October	67,000
May	1,700	November	59,000
June	5,000	December	32,000

b. S-346. Under low water conditions, when gage 3-28 stages are less than 8.5 ft, NGVD, S-346 is opened and passes the S-12 discharge down the L-67 Ext. borrow canal. The structure remains closed at all other times.

c. S-151 (with S-337 and S-335). The operation of S-151 is coordinated with S-32 operation for regulatory releases from WCA No. 3A based on the three station average. Water supply deliveries are also made through S-151 to satisfy demands in the Miami Canal (C-6) area, and the L-31N borrow canal and C-111 areas, the Northwest Wellfield, and the C-2 area.

The discharge capacity of S-151 based on the upstream and downstream water levels is computed according to the following equation:

If STG3A4 > 9.5
Q (S-151 Capacity) = 120. * (STG3A4 - 9.5) + 1000.
If 9.5 $\geq$ STG3A4 > 8.0
Q (S-151 Capacity) = 667. * (STG3A4 - 8.0)
If STG3A4 $\leq$ 8.0
Q (S-151 Capacity) = 0.0

Where STG3A4 = stage at gage 3-4 in ft, NGVD.

The required discharges for both flood control and water supply are always compared with the structure capacity and do not exceed the S-151 capacity as above. However, when the S-12 discharge capacity is not adequate to pass the full minimum water delivery requirement to ENP (VOLENP), then the S-151 capacity is reduced by the same percentage that the S-12 capacity is below the delivery rate.

(1) Regulatory Releases. Regulatory releases are made at capacity through S-151 when WCA No. 3A is above its regulation schedule, unless the stage in WCA No. 3B is high during the wet season. No S-151 regulatory discharges are made during the wet season if the WCA No. 3B stage is 9.0 ft, NGVD or higher. S-151 regulatory releases are linearly varied from 0.0 cfs when the WCA No. 3B stage is 9.0 ft, NGVD to 50% of the discharge capacity when the WCA No. 3B stage is 8.8 ft, NGVD. When the WCA No. 3B is below 8.8 ft, NGVD, S-151 regulatory releases are made at capacity.

(2) Water Supply. S-151 releases water supply deliveries from WCA No. 3A when demands are identified at any of the service areas and if there is adequate water available and structure capacity to pass it. When WCA No. 3A is below 6.9 ft, NGVD, water deliveries at S-151 do not exceed the S-340 discharge. Under this condition:

Q (S-151 Capacity) = Q (S-340)

Where

Q (S-151 Capacity) = S-151 discharge capacity in cfs.

Q (S-340) = S-340 discharge in cfs.

There is an additional limitation imposed when the minimum water deliveries to ENP cannot be fully released at S-12. Under this condition, the maximum capacity discharge at S-151 is computed as follows:

Q (S-151 Max) = Q (S-151 Capacity) * ((S-12 Capacity)/(VOLENP))

Where

Q (S-151 Capacity) = S-151 discharge capacity in cfs based on head across the structure.

S-12 Capacity = S-12 discharge capacity in cfs based on head across the structure.

VOLENP = Minimum water delivery in cfs required for Shark River Slough by PL 91-282 (see Table F-2).

(a) Northwest Wellfield. Water supply releases through S-151 are called for when the stages in the Dade-Broward Levee borrow canal fall below 2.75 ft, NGVD. These releases are no longer called for when the stage rises above 3.0 ft, NGVD. S-151 releases to the Northwest Wellfield are passed through S-337 into the L-30 borrow canal and then through a new structure into a new canal running east-west connecting the Dade-Broward Levee and a canal located adjacent to the Florida Turnpike. The new structure has a capacity of 500 cfs when the stage in the Dade-Broward Levee borrow canal is at or below 2.75 ft, NGVD. As the canal stage rises, the structure discharge is linearly reduced to 0.0 cfs when the canal stage is at, or above 3.0 ft, NGVD.

(b) C-6. Water supply deliveries are called for at S-151 to satisfy demands in the C-6 basin when the stage at gage S-19 falls below 0.2 ft, NGVD. These deliveries are no longer called for when the gage S-19 stage rises above 0.4 ft, NGVD. S-151 releases for C-6 water supply are passed through S-32. The S-151 discharges called for to satisfy the C-6 demands are computed as follows:

DEMAND (C-6) = -400. * (STAGE 19) + 500.
Where STAGE 19 = stage in ft, NGVD at gage S-19.
DEMAND (C-6) = required S-151 water supply discharge in cfs.

(c) C-2. Water supply deliveries are called for at S-151 to satisfy demands in the C-2 basin when water levels in the canal fall below 2.5 ft, NGVD. S-151 releases for C-2 water supply are passed through S-337 and S-335. If there is not a positive head across S-335 (i.e. from the L-30 borrow canal to the L-31N borrow canal), no water supply deliveries are called for the C-2 basin. If there is head across S-335, the required C-2 water supply deliveries are computed as follows:

DEMAND (C-2) = 900 * (L-30 - L-31)** .5

Where L-30 = L-30 borrow canal stage in ft, NGVD.
L-31N = L-31N borrow canal stage in ft, NGVD.
DEMAND (C-2) = water supply requirements for C-2.

(d) ENP - South Dade County Conveyance System. Water supply deliveries are called for at S-151 when Conveyance System canal stages fall below prescribed levels. If any of the following conditions are satisfied, deliveries to the Conveyance System are called for:

L-31 borrow canal stage above S-331 < 4.0 ft, NGVD
L-31 borrow canal stage below S-331 < 3.5 ft, NGVD
C-111 stage above S-177 < 2.5 ft, NGVD
C-111 stage above S-18C < 2.0 ft, NGVD
C-1 stage above S-21 < 1.9 ft, NGVD
S-151 water supply releases to the Conveyance System are passed through S-337 and S-335.

(e) Cutbacks. When there is not adequate discharge capacity at S-151 to satisfy all demands from the Northwest Wellfield, C-6, C-2, and the ENP - South Dade County Conveyance System, the water supply deliveries are allocated so the deficits in each demand area are equal percentages of each area's demand. Additionally, when there is not adequate capacity at S-337 to meet all the demands at the Northwest Wellfield, C-2, and the Conveyance System, the S-337 water deliveries are allocated to the demand areas so there are proportionate deficits in each area. When there is not adequate capacity at S-335 to pass all demands of C-2 and the Conveyance System, total capacity at S-335 is spilt equally between the C-2 demands and the Conveyance System demands.

d. S-337. This structure provides water supply to the Northwest Wellfield, the ENP-South Dade County Conveyance System and to the C-2 area. Discharges through the structure are made to satisfy demands as described above and/or not exceed the following structure capacity.

Q (Capacity)= 1125.06* (HW - TW)** 0.5

Where HW = headwater and TW = tailwater stages at S-337 in NGVD.

This capacity is reduced if there is not adequate capacity at S-151 to satisfy all demands, as described above.

e. S-31. The operation of S-31 is coordinated with S-151 for water supply and regulatory discharges. Under most conditions, discharges at S-31 are computed as follows:

Q (BASE) = 45.0 * (STAG3B - 3.0) + 117.0

Where STAG3B = stage in WCA No. 3B measured at gage 3-29 in ft, NGVD.

There are two conditions when S-31 discharges are not computed with the above equation; under a regulatory condition when the WCA No. 3B stage is above 8.9 ft, NGVD during the dry season and under a water supply condition when S-151 water supply discharges are being made to the Miami Canal area (when the stage at gage S-19 falls below 0.2 ft).

When WCA No. 3A is above schedule; the WCA No. 3B stage is above 8.9 ft, NGVD; and it is the dry season, the S-31 discharge is set equal to the S-151 discharge. When S-151 is making water supply discharges to the Miami Canal area, the S-31 discharge is increased as follows:

Q (Water Supply) = Q (BASE) + Q (S-151 Miami Canal Water Supply)

f. S-339 and S-340. The operation of these structures is coordinated to relieve high water conditions in northern WCA No. 3A when the stage at gage 3-2 rises above its regulation schedule or to pass water supply deliveries required at S-151 and S-32. Under all other conditions, the structures remain closed. Discharges through the structures under high water conditions or to provide water supply, do not exceed the structures' capacity. The capacity of the structures is computed using the following equation:

Q (Capacity) = 2600.0 * HEAD** 0.5

Where HEAD = upstream stage-downstream stage in ft., NGVD.

Under high water conditions, when the gage 3-2 stage is above its regulation schedule, S-339 and S-340 discharges are made if conditions permit. The S-339 discharge used is the smaller value of either the S-339 structure capacity or the downstream canal capacity which is computed as follows:

If DEL $\leq$ 0.13', Q (Canal Capacity) = 0.0
If 0.13 < DEL $\leq$ 0.8',
Q (Canal Capacity) = 2553.0 * (DEL - .13)
If DEL > 0.8', Q (Canal Capacity) = 0.40 * S-8Q

Where DEL = S-339 tailwater - S-340 headwater in ft.
Q (Canal Capacity) = canal capacity between S-339 and S-340 in cfs.
S-8Q = S-8 discharge in cfs.

Under high water conditions, the S-340 discharge used is the smaller value of either the S-339 discharge or the S-340 structure capacity.

Water supply discharges, when gage 3-2 is below schedule, can be made through S-339 and S-340 if all the following conditions are satisfied.

- There is structural capacity (i.e. Q (Capacity) greater than 0.0);
- S-8 discharges are being made from Lake Okeechobee to WCA No. 3A;
- The WCA No. 3A stage is less than 8.5 ft, NGVD;
- Water supply deliveries are being made at S-151;
- The Lake Okeechobee stage is greater than the stage in C-123 upstream of S-339.

If one of these conditions is not met, S-339 and S-340 remain closed.

If all of the above conditions are met, the S-8 discharge is computed by the following equation:

```
Q (S-8) = (1,543.17 * LOK ** 3 - 1,587.5 * S-339HW ** 3
          - 25,220.0 * (LOK - S-339HW) ** 1.5
          - 3,810,000 / LOK + 399,027.0) ** .5
```

Where LOK = Lake Okeechobee stage in ft, NGVD.
S-339HW = headwater at S-339 in ft, NGVD.

The water supply discharge used at S-339 is the minimum of the S-8 discharge or the structure capacity. The water supply discharge at S-340 is the minimum the S-339 discharge or the structure capacity.

g. Gap in Miami Canal Levee. When the stage in the Miami Canal, between S-340 and S-151 is above the adjacent marsh water level (one node to the south), flow through the gap in the canal levee into the southward marsh is computed as follows:

```
Q (GAP) = 3.09 * 25.0 * (DELT ** 1.5)
```

Where DELT = Canal stage - marsh stage one node to the south in ft.

h. S-9. Pump station S-9 pumps into WCA No. 3A for flood protection of the South New River Canal west of S-13A. Pumping is started when the canal stage climbs above 5.2 ft, NGVD and pumping is discontinued when the canal stage falls below 4.0 ft, NGVD. When the South New River Canal stage is above 4.8 ft, NGVD, S-9 pumping is computed as follows:

```
Q (S-9) = 895.0 * (STAGE - 4.8) + 300.0
```

Where STAGE = stage in the South New River Canal between S-9 and S-13A in ft, NGVD.

When 4.0 < STAGE ≤ 4.8
Q (S-9) = 130.4 * (STAGE - 2.5)

No pumping is done when the canal stage is 4.0 ft, NGVD or less.

i. S-343 A and B. These culverts discharge from WCA No. 3A into the eastern portion of Big Cypress Preserve in the vicinity of 40 Mile Bend on Tamiami Trail when WCA No. 3A is above regulation schedule and there is a positive head across the structure. No discharges are made when the area is below schedule. The following equation is used to compute total regulatory releases through the culverts:

If 3-28 > 9.8, Q (S-343 A & B) = 400.0
If 7.8 < 3-28 ≤ 9.8,
    Q (S-343 A & B) = 201.0 * ((3-28) - 7.8)
If 3-28 ≤ 7.8, Q = 0.0

Where 3-28 = stage at gage 3-28 in ft, NGVD.

j. S-344. This culvert discharges from WCA No. 3A into eastern Big Cypress Preserve when WCA No. 3A is above its regulation schedule. When the area is below schedule, the structure remains closed. Regulatory releases are made at S-344, if there is a positive head across the structure, according to the following procedures:

If S-344HW > 9.8, Q (S-344) = 300.0
If 8.5 < S-344HW ≤ 9.8,
    Q (S-344) = 238.7 * (S-344HW - 8.5)
If S-344HW ≤ 8.5, Q (S-344) = 0.0

Where S-344HW = headwater stage at S-344 in ft, NGVD.

7. East Coast Structures. A series of structures on canals between the WCA's and the east coast are operated independently to maintain optimum water levels for prevention of salt water intrusion and to provide flood protection.

a. S-26. S-26 discharges for flood control of the Miami Canal basin between S-26 and S-32. Discharges are made when the canal stages are above 2.8 ft, NGVD and are computed as follows:

If STAGE > 4.0, Q (S-26) = 1,000.0
If 4 ≥ STAGE > 2.8,
    Q (S-26) = 800.0 * (STAGE - 2.8) + 40.0
If STAGE ≤ 2.8, Q (S-26) = 0.0

Where STAGE = headwater stage at S-26 in ft, NGVD.

b. S-13. Pump station S-13 pumps to tidewater for flood control protection of the South New River Canal east of S-13A. The structure pumps only when the canal stage is above 1.5 ft, NGVD. The following equation is used to compute the pump rate:

Q (S-13) = 230.0 * (STAGE -1.5)

Where STAGE = S-13 headwater stage in ft, NGVD.

c. Sewell Lock. This structure is operated to provide flood control for the North New River Canal basin below S-34. Flood control discharges are made when the canal stage is above 3.0 ft, NGVD and are computed as follows:

If STAGE > 4.0 Q(SEWELL)=1250.0*(STAGE-4.0) + 250.0
If 4.0 ≥ STAGE > 3.0,
    Q (SEWELL) = 250.0 * (STAGE - 3.0)
If STAGE ≤ 3.0, Q (SEWELL) = 0.0

d. S-37A. This structure is operated to provide flood control to the C-14 basin downstream of S-38. Flood control discharges are made when the canal stage is above 5.0 ft, NGVD and are computed as follows:

If STAGE > 6.5, Q(S-37A)=1200.0*(STAGE-6.5) + 100.0
If 6.5 ≥ STAGE > 5.0,

$$Q\ (S\text{-}37A) = 66.7 * (STAGE - 5.0)$$
$$\text{If } STAGE \leq 5.0,\ Q\ (S\text{-}37A) = 0.0$$

Where STAGE = headwater stage at S-37A in ft, NGVD.

8. Everglades National Park - South Dade County Conveyance System. Water supply discharges are made from WCA No. 3A via S-333 and S-334 to satisfy the requirements of ENP at S-332 and C-111 and to maintain canal stages in south Dade County.

a. S-334. This structure makes water supply releases when (1) the stage in the L-31N borrow canal upstream of S-331 is below 5.0 ft, NGVD; (2) the stage upstream of S-176 is less than 3.5 ft, NGVD; (3) the stage in C-111 upstream of S-177 is less than 2.5 ft, NGVD; or (4) the stage upstream of S-18C is less than 2.0 ft, NGVD. When the canal stage is 5.0 ft, NGVD or higher, the structure remains closed. Water supply discharges increase up to a maximum of 1,000.0 cfs when the canal stage is below 4.0 ft, NGVD. The S-334 discharge is computed as follows:

$$\text{If } STAGE < 4.0,\ Q\ (S\text{-}334) = 1{,}000.0 \text{ cfs}$$
$$\text{If } 5.0 > STAGE \geq 4.0,$$
$$Q\ (S\text{-}334) = -\ 800.0 * (STAGE - 4.0) + 1000.0$$
$$\text{If } STAGE \geq 5.0,\ Q\ (S\text{-}334) = 0.0$$

However, the S-334 discharge computed also will not exceed what can be conveyed from S-333. Consequently, if S-333 capacity is not great enough to pass the required discharge at S-334, in addition to 125.0 cfs loss between structures, the S-334 discharge is reduced appropriately, as follows:

$$Q\ (S\text{-}334) = Q\ (S\text{-}333\ (Capacity)) - 125.0$$

Where Q (S-333 (Capacity)) = the discharge capacity at S-333 based on upstream and downstream stages.

b. S-333. This structure discharges water supply deliveries from WCA No. 3A to be introduced into the L-31N borrow canal via S-334. S-333 discharges are made only when S-334 is discharging. The S-333 discharge is 125.0 cfs greater than the S-334 discharge to account for losses from the L-29 borrow canal between structures. The S-333 discharge capacity is computed based on upstream stage at gage 3-28 and downstream the water level as follows:

$$\text{If } STAGE > 9.5,\ Q\ (Capacity) = 1000.0$$
$$\text{If } 9.5 \geq STAGE > 8.0,$$
$$Q\ (Capacity) = 533.3 * (STAGE - 8.0) + 200.0$$
$$\text{If } 8.0 \geq STAGE > 6.5,$$
$$Q\ (Capacity) = 133.3 * (STAGE - 6.5)$$
$$\text{If } 6.5 \geq STAGE,\ Q\ (Capacity) = 0.0$$

Where STAGE = stage at gage 3-28 in ft, NGVD.

The discharge used at S-333 is the minimum of either the S-333 discharge capacity or the S-334 discharge plus 125.0 cfs. If the S-333 discharge capacity is the smaller of these two values, the S-334 discharge is reduced

accordingly. If the S-12 capacity is not adequate to provide the minimum water delivery to ENP, the S-33 discharge is reduced by the same percentage that S-12 capacity is deficient to meet the minimum requirements.

c. S-331. This structure is not operated in the ROUTE subroutine as are the other structures described in this section. S-331 discharges are computed using the weir equation in the CHNFL subroutine. In this subroutine, the outflow from the L-31N borrow canal is computed and used to adjust the net inflow to the L-31 borrow canal. Discharges are computed as follows:

Q (L-31N OUTFLOW) = 7.22 * GWDTH * ((DEPTH - CREST)** 1.5)

Where GWDTH = S-331 gate width = 50.0 ft.
DEPTH = stage upstream of S-331 in ft, NGVD.
CREST = crest elevation (varies) in ft, NGVD.

The crest elevation used in the above elevation is varied depending on downstream canal stages. When all the downstream canal stages are at, or above, the defined levels, the crest elevation is set to 5.0 ft, NGVD. However, if the S-21 headwater stage falls to 1.9 ft, NGVD or less, or if the S-176 headwater stage falls to 3.5 ft, NGVD or less or if the S-177 headwater stage falls to 2.5 ft, NGVD or less or if the S-18C headwater falls to 2.0 or less, the crest elevation is lowered to 4.0 ft, NGVD.

d. S-148. This structure is modeled similar to S-133 in that the discharges are computed with a weir equation in the CHNFL subroutine. The crest elevation is changed from 5.0 to 3.0 ft, NGVD in the ROUTE subroutine if the S-21 headwater stage falls to 1.9 ft, NGVD or less. The weir equation for S-148 follows:

Q (S-148) = 1.72 * GWDTH * ((DEPTH - CREST) ** 1.5)

Where GWDTH = 60 ft = gate width.
DEPTH = canal stage in ft, NGVD.
CREST = crest elevation in ft, NGVD.

e. S-176. This structure is modeled similar to S-331 in that the discharges are computed with a weir equation in the CHNFL subroutine. The crest elevation used in the weir equation is lowered from 5.5 to 3.5 ft, NGVD in the ROUTE subroutine if the S-177 headwater stage falls below 2.5 ft, NGVD or if the S-18C headwater stage falls to 2.0 ft, NGVD or less. The weir equation for S-176 follows:

Q (S-176) = 9.22 * GWDTH * ((DEPTH - CREST) ** 1.5)

Where GWDTH = 55 ft = gate width.
DEPTH = S-176 headwater stage in ft, NGVD.
CREST = crest elevation in ft, NGVD.

f. S-194. This structure is modeled similar to S-331 in that discharges are computed with a weir equation in the CHNFL subroutine. The crest elevation used in the weir equation is lowered from 5.5 to 3.5 when the S-177

headwater stage falls to 2.5 ft, NGVD or less, or when the S-18C headwater stage falls to 2.0 ft, NGVD or less. The weir equation for S-194 follows:

Q (S-194) = 9.22 * GWDTH * ((DEPTH - CREST) ** 1.5)

Where GWDTH = 20 ft = gate width.
DEPTH = S-194 headwater stage in ft, NGVD.
CREST = crest elevation in ft, NGVD.

g. S-177. The structure is modeled similar to S-331 in that the discharges are computed with a weir equation in the CHNFL subroutine. The crest elevation used in the weir equation is changed in the ROUTE subroutine from 3.7 to 2.5 ft, NGVD if the S-18C headwater stage falls to 2.0 ft, NGVD or less. The weir equation for S-177 follows:

Q (S-177) = 15.22 * GDWTH * ((DEPTH - CREST) ** 1.5)

Where GWDTH = 30 ft = gate width.
DEPTH = S-177 headwater stage in ft, NGVD.
CREST = crest elevation in ft, NGVD.

h. S-332. S-332 transfers water from the L-31W borrow canal into Taylor Slough to satisfy the minimum monthly water deliveries required by PL 91-282. The following water deliveries are made until the stage in the L-31W borrow canal falls to 1.5 ft, NGVD or less at which time deliveries are discontinued.

TABLE F-3

MONTHLY WATER DELIVERIES TO TAYLOR SLOUGH
(cfs)

January	12	July	120
February	6.7	August	48
March	3.0	September	100
April	3.1	October	126
May	6.0	November	62
June	112	December	12

i. S-18C. Minimum monthly water deliveries to ENP's eastern panhandle area required by PL 91-282 are provided at S-18C. When the canal stage upstream of S-18C falls to 2.0 ft, NGVD or less, water deliveries through S-18C are discontinued. When the canal stage upstream of S-18C is above 2.0 ft, NGVD, the following water deliveries are made through S-18C.

TABLE F-4

ENP EASTERN PANHANDLE WATER DELIVERIES VIA S-18C
(cfs)

January	25	July	8
February	11	August	14
March	5	September	45
April	2	October	75
May	2	November	68
June	6	December	36

The structure capacity based on upstream and downstream stages is checked and the water deliveries to ENP are limited to this capacity. The S-18C discharge capacity is computed as follows:

Q (S-18C CAP) = (3069. * S18CHW) - 6038

Where S18CHW = S-18C headwater stage in ft, NGVD.

Discharges at S-18C are also computed with a weir equation in the CHNFL subroutine. Under flood conditions, these are added to the minimum water deliveries through S-18C. The discharges are computed with the following weir equation:

Q (S-18C WEIR) = 15.22 * GWDTH * ((DEPT - CREST) ** 1.5)

Where GWDTH = 42 ft = gate width.
DEPTH = S-18C headwater stage in ft, NGVD.
CREST = crest elevation in ft, NGVD.

B. BASE CONDITION WITHOUT THE LAKE OKEECHOBEE OPERATING PERMIT (LOOP) OPERATING CRITERIA

9. General. In July 1979, the Interim Action Plan (IAP) was implemented on an experimental basis as a component of the Temporary Operating Permit (TOP). In 1986, the operating criteria prescribed by the IAP were permanently adopted by SFWMD. These criteria call for pumping as much of the EAA runoff as possible to the WCA's. Runoff is pumped to Lake Okeechobee only when necessary to avoid flooding. The goal of the LOOP is to reduce nutrient loading of Lake Okeechobee by reducing the inflow of runoff from agricultural lands.

Prior to 1979, the operational strategy being used was the reverse of the LOOP. As much of the EAA runoff as possible was pumped into Lake Okeechobee. EAA runoff was pumped to the WCA's only when necessary to avoid flooding. The goal was to store water as "high" in the system as possible to provide greater flexibility in providing water supply deliveries where needed. This is the operational strategy assumed for this alternative.

10. Physical Modifications. This alternative consists of operational modifications only. The physical system would be identical to that of the Base Condition.

11. Operations. With the exception of structure operations described below, this alternative would be identical to the Base Condition.

a. S-8, S-3, and HGS-3. Water supply deliveries would be provided to the EAA as necessary, via HGS-3 up to its capacity of 4,000 cfs. Operational criteria for water supply discharge at HGS-3 would be identical to those used in the Base Condition.

When it is necessary to remove runoff from the S-8 basin, S-3 would pump the runoff into Lake Okeechobee as necessary or up to its capacity of 3,000 cfs. If the runoff exceeds the capacity of S-3, then S-8 would pump into WCA No. 3A as necessary or up to its capacity of 4,200 cfs. If the combined capacity of S-3 and S-8 is not adequate to remove the runoff, flooding occurs in the EAA.

b. S-6, S-7, S-2, and HGS-4. Water supply deliveries would be provided to the EAA as necessary via HGS-4 up to its capacity of 4,000 cfs. Since the S-6 and S-7 basins are both served by HGS-4, 50% of the HGS-4 capacity is dedicated to each basin. Operational criteria for water supply discharges at HGS-4 would be identical to those used in the Base Condition.

S-2 pumps runoff from the EAA to Lake Okeechobee from the S-6 and S-7 basins. Therefore, 50% of the S-2 pump capacity is reserved for each basin. When it is necessary to remove runoff from the S-6 or the S-7 basins, S-2 pumps as necessary up to its allotted 50% capacity for each basin. However, if during the same day, water supply deliveries are required in one basin (via HGS-4) while removal of runoff is required in the other, S-2 would not pump. In this

case, either S-6 or S-7 would pump the necessary runoff from its basin and HGS-4 would provide water supply flows to the other basin. When the runoff from either basin exceeds the allotted capacity at S-2, then S-6 and/or S-7 pumps as necessary up to capacity. For example, if runoff in the S-6 basin exceeds 50% of the S-2 capacity, then S-2 pumps at 50% capacity (plus whatever is required for the S-7 basin) and S-6 pumps as necessary up to its capacity. If the combined S-2 and S-6 and/or S-7 capacities are not adequate to remove the runoff, flooding occurs.

C. BASE CONDITION WITH WATER CONSERVATION AREA NO. 2A DRAWDOWN

12. General. In the Base Condition, WCA No. 2A would be operated in accordance with the currently authorized (13.0 - 14.5 ft, NGVD) regulation schedule for WCA No. 2A. However, between October 1980 and January 1987, WCA No. 2A was operated in accordance with a (9.5 - 11.5 foot, NGVD) drawdown schedule although the drawdown was temporarily abandoned for short periods during 1981, 1983, and 1986.

In January 1987, a revised drawdown schedule was implemented. The revised schedule varies between 11.0 and 13.0 ft, NGVD and is assumed to be in use for the alternative. All other structural and operational components of this alternative are identical to the Base Condition.

13. WCA No. 2A Regulation Schedule. The (11.0 - 13.0 ft, NGVD) WCA No. 2A regulation schedule that would be used is shown in Plate F-1. From January through June, the S-11B headwater stage would be used for regulation; from July through September, gage 2-17 would be used; and from October through December, gage 2-17 would be used until the stage recedes to 11.0 ft, NGVD when the S-11B would be used.

14. S-11. Regulatory releases are made from WCA No. 2A when all of the following conditions are satisfied.

a. The WCA No. 2A stage is above the regulation schedule.

b. The S-11 headwater is higher than the tailwater:

c. The L-38 borrow canal stage is above 11.0 ft, NGVD.

If any of the above conditions are not satisfied, S-11 does not discharge.

When S-11 is discharging the flows are computed by the following equation:

$$S\text{-}11Q = 999^* (HEAD - 1.04) + 3214 \quad \text{when } HEAD > 1.04$$
$$S\text{-}11Q = 4850^* (HEAD - .38) \quad \text{when } HEAD \leq 1.04$$

Where

S-11Q = S-11 discharge in cfs.
HEAD = S-11 headwater stage minus the tailwater stage in feet.

D. FLOW-THROUGH WATER DELIVERIES TO EVERGLADES NATIONAL PARK

15. General. This alternative plan for water deliveries to the Everglades National Park (ENP) was proposed for consideration by ENP research staff by letter dated October 17, 1984. The plan calls for a more natural distribution of flows through WCA No.'s 3A and 3B into ENP. To create a natural type of flow, structural modifications would be made in WCA No.'s 3A and 3B. The major water control structures through which water would be delivered to the Park would remain fully open, with the goal of allowing water deliveries that vary with respect to timing and volume in response to local hydrometerologic conditions. Consequently, the regulation schedule for WCA No. 3A would be abandoned.

In addition to operational changes, this plan includes several structural modifications to the existing water management system. Three gated water control structures would be placed in L-67A borrow to increase the natural sheet flow of water through the marsh and canal to prevent the rapid conveyance of water from the northern portion of WCA No. 3A directly to S-12 under wet conditions. Under most conditions, these structures would remain closed. However, when water supply deliveries are required to the ENP-South Dade County Conveyance System, the structures would be opened to convey the required flows.

Three sets of gated culverts (S-345) would be constructed in L-67A, located just upstream of each of the three structures to be added to the borrow canal. Under most conditions, these culverts would remain fully open to allow free flow of water from WCA No. 3A into WCA No. 3B. When water supply is required for the Conveyance System, the culverts would be closed to allow conveyance of the required flows through the L-67A borrow canal structures.

In order to enable discharges from WCA No. 3B to the Shark River Slough, two gated control structures (S-355) would be constructed in L-29 between L-67 and L-31N. S-355 would remain fully open to discharge water from WCA No. 3B into Northeast Shark River Slough. The structure would be closed when the borrow canal stage exceeds the stage in WCA No. 3B to avoid reverse flow. The L-67 Extension would be degraded and pushed into the borrow canal. These modifications would create a hydrologic connection between the WCA's No. 3A and 3B, ENP, and NESRS.

A seepage levee in the southeast corner of WCA No. 3B would be added as a part of this alternative. It would connect L-30 just north of S-335 and L-29 just west of S-334. The purpose of this levee would be to reduce seepage from WCA No. 3B to the L-30 borrow canal. Also, a seepage levee would be constructed west of and parallel to L-31N from Tamiami Trail southward to the developed portion of the Everglades. Since the water levels will be higher in NESRS, this levee would reduce seepage inflow to the L-31N borrow canal.

With the exception of the structural and operational modifications to WCA No. 3 and water deliveries to the ENP described in this section, all other

aspects of this alternative plan would be identical to the base condition. A description of each component of this alternative plan that differs from the Base Conditions is provided below. For descriptions of components of this plan that are not provided below, refer to the Base Condition description.

16. Structural Operations. The operating criteria for all new structures required for this alternative and revised operation criteria for existing structures are described below. All other operations remain the same as under the Base Conditions.

a. S-12. All gates at S-12 would remain clear of the water as a part of this alternative and discharges are computed as follows:

Q = 2667. * (STG 3A28 - 9.5) + 1600.	when STG 3A28 $\geq$ 9.5
Q = 1100. * (STG 3A28 - 8.5) + 500.	when 9.5 $\geq$ STG 3A28 > 8.5
Q = 500 * (STG 3A28 - 7.5)	when 8.5 $\geq$ STG 3A28 > 7.5
Q = 0	when STG 3A28 $\leq$ 7.5

Where

Q = S-12 discharge in cfs.
SJG 3A28 = water level at gage 3-28 in feet, NGVD.

b. S-333. All gates at S-333 would remain fully open and the discharge would be computed as follows:

QS333 = 1000.	when STG 3A28 > 9.5
QS333 = 533.3 * (STG 3A28 - 8.0) + 200.0	when 9.5 $\geq$ STG 3A28 > 8.0
QS333 = 133.3 * (STG 3A28 - 6.5)	when 8.0 $\geq$ STG 3A28 > 6.5
QS333 = 0	when STG 3A28 $\leq$ 6.5

Where

QS333 = discharge at S-333 in cfs.

c. S-334. As in the Base Condition, S-334 would discharge water supply deliveries to the conveyance system when downstream canal stages drop below specified levels and when there is discharge capacity available at the structure. It is assumed that 125 cfs would be lost between S-333 and S-334. Consequently, when there are downstream demands, S-334 discharges would be computed as follows:

QS334 = S334CAP	when S334 demand $\leq$ QS333-125
QS334 = QS333-125	when S334 demand > QS333-125

Where

QS334 = S-334 discharge in cfs.
QS334CAP = discharge required at S-334 to satisfy the demand, in cfs.
QS333 = S-333 discharge in cfs = S-333 discharge capacity.

d. S-343A, 343B, and S-344. These L-28 culverts would continuously discharge water from WCA No. 3 westward to Big Cypress at capacity. Discharges would be computed as follows:

QS343 = 400.	when STG 3A28 > 9.8
QS343 = 201.1 * (STG 3A28 - 7.8)	when 9.8 ≥ STG 3A28 > 7.8
QS343 = 0	when STG 3A28 ≤ 7.8

Where

QS343 = total discharge at S-343A and 343B in cfs.
QS344 = total discharge through S-343A and S-343B in cfs.

QS344 = 300.	when STG S344 > 9.5
QS344 = 238.7 * (STG S344 - 8.5)	when 9.8 ≥ STG S344 > 8.5
QS344 = 0	when STG S344 ≤ 8.5

Where

QS344 = total discharge through S-344 in cfs.
STG S344 = headwater elevation at S-344 in feet.

e. S-151. S-151 would continuously discharge at capacity from WCA No. 3A into 3B and the discharge would be computed as follows:

QS151 = 1200.	when SA151 > 11.0
QS151 = 180. * (SA151 - 9.5) + 930.	when 11 ≥ SA151 9.5
QS151 = 547.1 * (SA151 - 7.8)	when 9.5 ≥ SA151 > 7.8
QS151 = 0	when SA151 ≤ 7.8

Where

QS151 = S-151 discharge in cfs.
SA151 = S-151 headwater elevation in feet, NGVD.

f. S-345. This structure would discharge water continuously from WCA No. 3A into WCA No. 3B according to the following equations:

QS345 = 1500.	when HEAD ≥ 2.0
QS345 = 750.0* HEAD	when 0 < HEAD < 2.0
QS345 = 0.0	when HEAD ≤ 0

Where

QS345 = S-345 discharge in cfs.
HEAD = S345 headwater elevation - the WCA No. 3B stage in feet.

g. S-355. This structure would discharge water from WCA No. 3B into the L-29 canal when there is a positive head and the stage in WCA No. 3B is greater than 6.5 feet, NGVD (approximately ground elevation). Discharges would be computed as follows:

QS355 = 2000.0	when HEAD $\geq$ 0.3
QS355 = 6666.67* HEAD	when 0 < HEAD < 0.3
QS355 = 0.0	when HEAD $\leq$ 0

E. RAIN-DRIVEN WATER DELIVERIES TO EVERGLADES NATIONAL PARK

17. General. This alternative plan consists of modifying the timing, location, and volume of water deliveries to Shark River Slough in Everglades National Park (ENP). Water deliveries to the park's Taylor Slough and eastern panhandle area are provided in accordance with the current requirement of PL 91-282. Based on the authority provided in PL 98-181 for the Experimental Program, the South Florida Water Management District is currently reviewing the delivery schedule for Taylor Slough and proposing modifications. If implemented, the new schedule would deliver water to the slough according to a rainfall-based model similar to the water delivery model used for Northeast Shark River Slough. Structural and operational modifications to WCA No. 3 are required for implementation of this alternative plan. Structural modifications are shown in Figure 1. The timing and volume (or flow rate) of water deliveries is computed as the sum of two components; a rain-driven component and a Water Conservation Area No. 3A regulatory component. New zones have been added to the regulation schedule to enable a smooth transition from non-regulatory to regulatory operation (see Plate 9). Water deliveries are adjusted on a weekly basis to match the computed rates.

The location of water deliveries to Shark River Slough in this plan is across a section of Tamiami Trail from near 40-mile bend on the west to L-31N on the east. This section is approximately twice the section length across which flows are provided under the Base Condition. As described in the main text, the total computed flow rate is split between the S-12 and the Tamiami Trail. The goal would be for 45 percent of the total to be discharged at S-12 and 55 percent of the delivery discharged across the section of Tamiami Trail between L-67 Extension and L-31N.

With the addition of new structures, WCA No. 3B would be fully incorporated into the Shark River Slough system to allow inflow from WCA No. 3A and to allow outflow into Northeast Shark River Slough. The L-67 Extension will be removed and its borrow canal is filled. This will allow complete hydrologic interaction between Shark River Slough (within ENP) and Northeast Shark River Slough (within the East Everglades).

With the exception of the structural and operational modifications to WCA No. 3 and water deliveries to ENP described in this section, all other aspects of this alternative plan are identical to the Base Condition. A description of each component of this alternative plan that differs from the Base Condition is described below. For descriptions of components of this plan that are not provided below, refer to the Base Condition description in Appendix F.

18. Timing and Volume of Water Deliveries to Shark River Slough. The objective of this method is to provide water deliveries to Shark River Slough that vary in response to hydro-meteorologic conditions in WCA No. 3A. Total rain-driven water deliveries from Water Conservation Area No. 3A are the sum of a rain-driven component and a regulatory component. Deliveries are computed and adjusted on a weekly basis. Description of how the delivery components are computed is provided below.

a. Rain-Driven Water Delivery Component. This component is based on a regression equation that was developed using historical rainfall over WCA No. 3A to compute flow across Tamiami Trail into Shark River Slough. The regression equation was developed using historical rainfall and evaporation data for the period from 1941 through 1952, the earliest period for which the necessary data were available. However, some man-made modifications to the natural Everglades had been constructed at that time, including Tamiami Trail. In an attempt to account for the affects of structural changes that existed at that time, coefficients in the regression equation are adjusted. Weekly water deliveries are computed based on comparisons of the previous ten weeks rainfall and evaporation with average historical rainfall and evaporation for the corresponding period. The first step in this computation is to compare the historical mean weekly rainfall and evaporation totals with the actual values for the ten preceeding weeks. The differences in rainfall excesses between actual and mean historical values are then computed for three periods, as follows:

$$RL1 = (RF-RFM)1\text{-}2 - 0.8 * (EVP - EVPM)1\text{-}2$$
$$RL2 = (RF-RFM)3\text{-}6 - 0.8 * (EVP - EVPM)3\text{-}6$$
$$RL3 = (RF-RFM)7\text{-}10 - 0.8 * (EVP - EVPM)7\text{-}10$$

WHERE

(RF- RFM)1-2 = actual rainfall minus average mean rainfall for the prior two weeks.

(RF- RFM)3-6 = same as above for third through sixth prior weeks.

(RF- RFM)7-10 = same as above for seventh through tenth prior weeks.

(EVP - EVPM)1-2 = actual evaporation minus mean historical evaporation for prior two weeks.

(EVP - EVPM)3-6 = same as above for prior third through sixth week.

(EVP - EVPM)7-10 = same as above for prior seventh through tenth week.

The weekly rainfall delivery is then computed as follows:

$$FRF = QM + CQ*QL + CR1*RL1 + CR2*RL2 + CR3*RL3$$

WHERE

FRF = Computed rainfall response component of the rain-driven water delivery in cfs.

RL1, RL2, RL3 = Actual minus mean historical weekly rainfall excess terms computed as described above.

QM = average historic discharge for present week in cfs.

QL = previous weeks computed discharge minus previous week's average historical discharge in cfs.

CQ = 0.907 (lagged flow coefficient).

CR1, CR2, CR3 = lagged rainfall coefficients.

The lagged rainfall coefficients were estimated using 1941 through 1952 historic weekly data. The upper and lower 95 percent confidence values are as follows:

95% Confidence Levels

	Lower	Upper
CR1	52.22	70.58
CR2	-4.24	9.49
CR3	-3.41	9.18

Analysis of available hydrologic records, and structural changes prior to 1952, indicate that the hydrology of Shark River Slough had been affected by drainage works adjacent to it. The most important effects were noticed in the dry season when the artificial drainage near the slough dropped the water table to lower levels, and for longer periods, than would have been the case naturally. Also, some reductions in the peak flow rates may have occurred.

To compensate for this, the final formula calls for some adjustment of the coefficients within the 95 percent confidence interval. The high estimate (.907) of the discharge coefficient (CQ) is used throughout. The rainfall coefficients are varied depending on whether the rainfall terms are positive or negative. If positive, the high estimates are used. If negative, CR1 is set to its low estimate and CR2 and CR3 are set to zero. The low estimates of CR2 and CR3 are negative and are not used since to do so would be contrary to the natural process. The result of these adjustments is to strengthen the response to rainfall excess and to change the timing slightly by beginning the flow sooner and by prolonging the recession of the hydrograph.

b. WCA No. 3A Regulatory Component. When the stage in WCA No. 3A is in Zone E (see Plate 9), the regulatory component of water deliveries is zero. When the stage rises above Zone E, a supplemental regulatory component is added to the rainfall response component. The regulatory component is calculated by multiplying the distance (in feet) the WCA No. 3A stage is above Zone E by 2,500 cfs per foot. However, when the stage in WCA No.3A enters Zone A, regulatory releases are made at capacity through all outlets. The purpose of this flow component is to maintain the authorized flood protection function of WCA No. 3A.

19. Location of Water Deliveries to Shark River Slough. One goal of this alternative is to provide water deliveries to Shark River Slough across Tamiami Trail similar to a natural distribution. James Hartwell, USGS, determined that the flow distribution varied with flow rates, approximately as follows:

TABLE F-4

Flow Distribution to Shark River Slough
Across Tamiami Trail

Total Flow	% of Total West of L-67	% of Total Between L-67 and L-31N
Flow > 1,000 cfs	47	53
Flow < 1,000 cfs	40	60
Flow < 1,000 cfs and increasing	45	55

A distribution goal of 45 percent of the total flow passing west of L-67 and the remaining 55 percent passing between L-67 and L-31N was chosen for this plan. Therefore, 45 percent of the total rain-driven delivery is discharged through S-12 and 55 percent is discharged through S-355, the proposed new structure in L-29, if possible. If the new L-29 structure does not have adequate capacity to pass the computed flow, S-333 discharges will make up the difference.

20. Regulation of Water Conservation Area No. 3. The operation of WCA No. 3 is modified in conjunction with this alternative plan to accomplish two general objectives; to allow a smoother transition between non-regulatory and regulatory releases, and to increase the hydrologic interaction of WCA No. 3A, WCA No. 3B, and northeast Shark River Slough.

21. Structure Operations. The operating criteria for all new structures required for this alternative and revised operation criteria for existing structures are described below. All other operations remain the same as under the Base Condition.

a. S-345. New structure(s) are added in L-67A for the purpose of discharging water from WCA No. 3A into 3B. The structure(s) start discharging when the stage in WCA No. 3A is 0.5 foot below Zone C or when it enters Zone D. The discharges linearly increase up to the structure's capacity of 1,500 cfs when the stage reaches Zone C or B and remains at capacity until the stage receeds and the discharge decreases linearly back to zero in the same manner. A check is made to insure there is adequate head across the levees between WCA No. 3A and 3B. Two feet of head is required to discharge the 1,500 cfs capacity.

b. S-355. A new structure, S-355, is added in L-29 to discharge water from WCA No. 3B into Northeast Shark River Slough. This structure is the first choice for discharging 55 percent of the total computed rain-driven delivery requirement. Discharges are made up to capacity or up to the delivery target, whichever is less. The structure capacity is 2,000 cfs with 0.3 foot of head across L-29.

c. S-12. The structure's capacity is computed as follows:

When WCA No. 3A (3 station average) stage is in Zone A:

S12CAP = 2667. * (3A28 - 9.5) + 1600. when 3A28 > 9.5
S12CAP = 1100. * (3A28 - 8.5) + 500. when 9.5 ≥ 3A28 > 8.5
S12CAP = 500. * (3A28 - 7.5) when 8.5 ≥ 3A28 > 7.5
S12CAP = 0.0 when 3A28 ≤ 7.5

When WCA No. 3A (3 Station average) stage is below Zone A

S12CAP = .50416 * 167.0 * (AV3A -6.9) ** 2.5 when AV3A > 6.9
S12CAP = 0.0 when AV3A ≤ 6.9

WHERE S12CAP = S-12 discharge capacity in cfs.
3A28 = Stage at gage 3-28 in feet.
AV3A = Three station average stage at gages 3-3, 3-4, 3-28.

S-12 discharges 45 percent of the computed total rain-driven water delivery or the structure capacity, whichever is less.

d. S-333. The structure capacity is computed as follows:

S333CAP = 1,000 when 3A28 > 9.5
S333CAP = 533.3 * (3A28 - 8.0) + 200.0 when 9.5 ≥ 3A28 > 8.0
S333CAP = 133.3 * (3A28 - 6.5) when 8.0 ≥ 3A28 > 6.5
S333CAP = 0.0 when 3A28 ≤ 6.5

WHERE S333CAP = S-333 discharge capacity in cfs.
3A28 = stage at gage 3-28 in feet.

This capacity is the same as that used for the Base Condition.

Structure 333 discharges at its capacity when the average stage in WCA No. 3A is in Zone A. When WCA No. 3A is below Zone A, and the discharge capacity at S-355 is not adequate to pass the full 55 percent of the total rain-driven delivery, S-333 will discharge enough to make up the difference, if there is adequate capacity. S-333 discharges are increased for water supply transfers to south Dade County and ENP (Taylor Slough and C-111) when there is excess capacity at S-333 above that required for water deliveries to ENP. Such discharges are then passed through S-334. Similar to the Base Condition, a loss of 125 cfs between S-333 and S-334 is assumed.

e. S-343A, 343B, and 344. These structures are operated to make regulatory releases from WCA No. 3A westward to Big Cypress. If WCA No. 3A is in Zone A, B or C, these culverts discharge water to Big Cypress as follows:

QS343 = 400. when 3A29> 9.8
QS343 = 201. * (3A28 - 7.8) when 9.8 ≥ 3A28 > 7.8
QS343 = 0.0 when 3A28 ≤ 7.8

WHERE QS343 = total discharge through S-343A and 343B in cfs.
3A28 = stage at gage 3-28 in feet.

QS344 = 300. when HW > 9.8
QS344 = 238.7 * (HW - 8.5) when 9.8 ≥ HW > 8.5
QS-344 = 0.0 when HW ≤ 8.5

WHERE QS344 = discharge through S-344 in cfs.
HW = stage upstream of S-344 in feet.

f. S-151. The operation of S-151 to discharge water supply for the Miami Canal, L-31N borrow canal southward to C-111, the northwest wellfield, and the C-2 service area remains the same as the Base Condition. The operations of S-337 and S-335 to pass these water supply discharges also remains the same. Only the regulatory operations at S-151 (and S-31) change for this alternative.

The structure discharge capacity is the same as for the Base Condition and is described below:

S151CAP = 120. * (3A4 - 9.5) + 1000. when 3A4 > 9.5

S151CAP = 667. * (3A4 - 8.0) when 9.5 ≥ 3A4 > 8.0

S151CAP = 0.0 when 3A4 ≤ 8.0

WHERE S151CAP = S-151 discharge capacity in cfs.

3A4 = stage at gage 3-4 in feet.

When the WCA No. 3A stage is in Zone A, B, or C in the dry season, S-151 discharges at capacity. During these times, S-31 passes the S-151 discharges into the Miami Canal if the WCA No. 3B stage is above 8.9 feet. Otherwise, S-31 passes a base flow and a portion of the S-151 discharges are stored in WCA No. 3B.

When the WCA No. 3A stage is in Zone A, B, or C in the wet season, S-31 passes a base flow and S-151 discharges are made based on stage in WCA No. 3B as follows:

QS151 = S151CAP when WCA 3B < 8.8
QA151 = S151CAP * (9.0 - WCA 3B) * 2.5 when 9.0 > WCA 3B > 8.8
QS151 = 0.0 when WCA 3B > 9.0

WHERE QS151 = discharge through S-151 in cfs
S151CAP = S-151 discharge capacity in cfs
WCA 3B = stage in WCA No. 3B

When the WCA No. 3A stage is below Zone C, S-151 discharges for water supply only.

F. RAIN-DRIVEN WATER DELIVERIES WITH S-349

22. General. Except for the project features described below, all other components of this alternative are identical to the Basic Rain-Driven alternative described in Section A.

23. S-349A, B, and C. The structures are not modeled in the ROUTE subroutine. Instead, the section of L-67A borrow canal between S-151 and Tamiami Trail is divided into three equal but separate canals. There is no discharge from one canal to another except through overland flow. Changes are reflected in the CNDTA22 and CANAL22 subroutines of the South Florida Water Management Model (SFWMM).

G. RAIN-DRIVEN WATER DELIVERIES WITH S-349 AND S-356

24. General. Except for the project features described below, all components of this alternative are identical to the Rain-Driven Water Delivery alternative with S-349.

25. S-356. S-356 is a pump station included in this alternative in lieu of the WCA No. 3B and L-31N seepage levees included in prior alternatives. The purpose of S-356 would be to control water levels in the L-31N borrow canal by pumping excess seepage into the L-29 borrow canal. The pump station would be located adjacent to S-334.

The capacity of S-356 would be 500 cfs. Pumping would begin when water levels in the L-31N borrow canal rise above 5.5 ft, NGVD and would be linearly increased up to capacity when water levels rose to 6.0 ft, NGVD. However, there is no pumping at S-356 when there are any demands for water supply in downstream canals.

26. S-331. In order to avoid increased flows in canals downstream of L-31N borrow canal, gravity flows from L-31N are allowed only under a narrow range of conditions. Southward gravity flow begins when L-31N borrow canal stages rise above 5.5 ft, NGVD and there is a positive downstream head. Such discharges are discontinued when downstream stages exceed 6.0 ft, NGVD.

H. RAIN-DRIVEN WATER DELIVERIES WITH S-349, S-356 AND L-29 DEGRADED

27. General. Except for the project features described below, all other components of this alternative are identical to the basic Rain-Driven alternative with S-349 and S-356 described in Section C.

28. S-355. This structure is not included in this alternative.

29. L-29 Modifications. Instead, a 6-mile section of L-29 (from L-30 westward) is removed. The STATDTA of the SFWMM file is modified to remove L-29 and to change the basin numbers for grid cells in WCA No. 3B to match those in ENP and NESRS. With the same basin numbers, overland flow from one grid cell to another is allowed. Only the eastern section of L-29 is degraded to encourage more southward flow when the S-333 discharges are being made and the tailwater stage in the western canal is high. The intent is to limit northward flow from L-29 borrow canal into WCA No. 3B immediately downstream of S-333 under these conditions.

I. RAIN-DRIVEN WATER DELIVERIES WITH S-349, S-356, AND RESIDENTIAL AREA FLOOD PROTECTION (PUMPED TO NORTHEAST SHARK RIVER SLOUGH)

30. General. With the exception of the project features described below, all components of this alternative are identical to the Rain-Driven alternative with S-349 as described in section F.

31. S-356. S-356 is a pump station included in this alternative in lieu of the WCA No. 3B and L-31N seepage levees. It is located in L-31N just north of S-331. S-356 would pump excees seepage water from the L-31N borrow canal westward into a new canal constructed through the flood protected residential area.

The capacity of S-356 is 500 cfs. Pumping would begin when L-31N canal stages exceeded 5.5 ft, NGVD and are linearly increased up to capacity when the canal stages reach 6.0 ft, NGVD or higher. However, there is no pumping at S-356 when the water supply discharges are made into L-31N borrow canal at S-334 or S-335.

32. Residential Area Flood Control Works. Detailed designs of levees, canals, and structures required for flood protection of the resiential area were developed independent of the SFWMM. The flood protection is simulated in the model however. One continuous canal collects runoff and seepage for flood control of the area, in addition to excess seepage water pumped from L-31N borrow canal via S-356. This water is then pumped from the flood control canal directly into NESRS via S-357. Overland flow from outside grid cells is prevented from entering the protected area by assigning different basin numbers to the interior grid in the Models STATDTA subroutine.

$$Q = 600.00 * (STGRES - 5.0)$$

Where

Q = S-357 pumpage in cfs.

STGRES = average groundwater elevation in the flood protected area.

When the average groundwater elevation in the protected area is 6.5 ft, NGVD or higher, S-357 pumps at its capacity of 900 cfs. Once pumping is initiated at S-357, it is continued until the average groundwater level drops to or below 5.0 ft, NGVD. S-357 does not operate if the flood control canal stage is 4.5 ft, NGVD or lower.

J. RAIN-DRIVEN WATER DELIVERIES WITH S-349, S-356, AND RESIDENTIAL AREA FLOOD PROTECTION (PUMPED TO L-31N)

33. General. Except for the project works described below, this alternative is identical to the Rain-Driven Water Delivery alternative with S-349 and S-356 as described in section C.

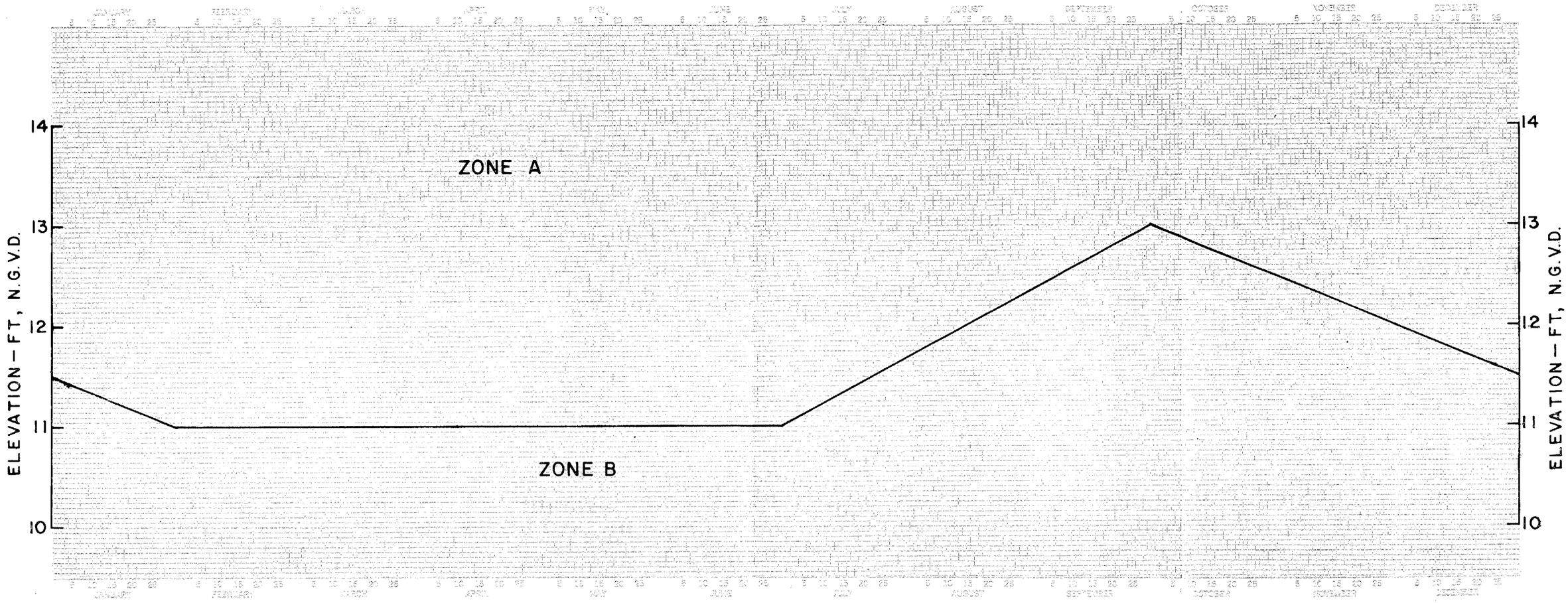
34. Residential Area Flood Protection. Detailed designs of levees, canals, and structures required for flood protection of the developed area were developed independent of the SFWMM. The flood protection system is simu-lated in this model however. One continuous canal collects seepage and overland runoff from the protected area. Water is pumped by S-357 from this canal into the L-31N borrow canal. S-357 pumps when the groundwater elevation rises above 5.5 ft, NGVD according to the following criteria:

$$Q = 266.67 * (STGRES - 5.0)$$

Where

Q = S-357 pumpage in cfs.
STGRES = average groundwater (or surface water) elevation in three grid cells within the protected area.

When the average groundwater elevation is 6.5 ft, NGVD or higher, S-357 pumps at its capacity of 400 cfs. Once pumping is initiated, it continues until the average groundwater elevation falls to 5.0 ft, NGVD. However, pumping is discontinued if the flood control canal stage within the protected area falls to, or below, 4.5 ft, NGVD.



ZONE	RELEASES THROUGH OUTLETS AS INDICATED
A	UP TO MAXIMUM CAPACITY AT S-11; MAXIMUM CAPACITY AT S-144, S-145 AND S-146; MAXIMUM PRACTICABLE AT S-143 AND S-38 WHEN REQUESTED BY THE CORPS OF ENGINEERS, BUT NOT TO EXCEED 11.0 FEET, NGVD, IN POOL 2B. L-35B AND L38 BORROW CANALS SHOULD NOT BE DRAWDOWN BELOW 10.5 FEET, NGVD.
B	WATER SUPPLY. L-35B AND L-38 BORROW CANAL SHOULD NOT BE DRAWDOWN BELOW 10.5 FEET, NGVD, UNLESS WATER SUPPLIED FORM ANOTHER SOURCE.

INDICATOR GAGES FOR REGULATION WILL BE AS FOLLOWS:

DATE	USE GAGE	CONDITIONS
1 JAN - 31 JAN	2-17	IF 2-17 STAGE RECEDES TO 11.5 FEET, NGVD SWITCH TO S-11B HEADWATER GAGE.
1 FEB - 30 JUN	S-11B	ALL
1 JUL - 31 DEC	2-17	ALL

CENTRAL AND SOUTHERN FLORIDA
INTERIM REGULATION SCHEDULE
WATER CONSERVATION AREA NO. 2A
DEPARTMENT OF THE ARMY, JACKSONVILLE DISTRICT
CORPS OF ENGINEERS, JACKSONVILLE FLORIDA

PROPOSED 3/28/88
COORDINATED 6/21/89

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK, FLORIDA

WATER CONSERVATION
AREA NO. 2A

REGULATION SCHEDULE
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMORANDUM
PART I, SUPPLEMENT 54, DATED: MARCH 1990
D.O. FILE NO. 400-3E GEN

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX G

GEOTECHNICAL INVESTIGATIONS

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX G

GEOTECHNICAL INVESTIGATIONS

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APPENDIX G

GEOTECHNICAL INVESTIGATIONS

A. GEOTECHNICAL AND MATERIALS

1. Structures S-345A, S-345B, & S-345C, and Structures S-349A, S-349B, & S-349C.

a. Investigations Performed. Core borings were taken at approximately one mile intervals along L-67(A) in 1960. No other subsurface investigations were performed for this report. Core borings and material tests at each structure site will be performed and included in the FDM. The structures will be constructed in and adjacent to existing L-67(A).

b. Materials Encountered. The foundation materials are anticipated to be similar to those encountered beneath L-67(A). The materials at the structure sites consist of 2 - 4 feet of peat underlain by a thin layer (one foot or less) of silt. Beneath the peat and silt is solution riddled, often cavernous, highly permeable, medium hard to hard limestone. The groundwater table exists near or above the ground surface. A geologic cross-section of L-67(A) with approximate locations of where the structures will be constructed is shown on Plate G-1.

c. Foundation Conditions. The ground surface elevation is approximately 6.5 feet, NGVD. The structures will be founded at approximately elevation -1.0 foot, NGVD (S-345 structures) and elevation -24.0 feet, NGVD (S-349 structures) in medium hard to hard limestone. These materials will provide adequate structural support.

d. Excavation. The maximum depth of excavation will be approximately 7.5 feet at the S-345 structures and approximately 30.5 feet at the S-349 structures. Unconsolidated surficial materials can be excavated by conventional means. Excavation of the underlying hard limestone materials will require blasting.

e. Structural Backfill. Backfill material will be obtained from required structure and levee breaching excavations. Select backfill consisting of material with no more than 10 percent passing a No. 200 sieve will be placed adjacent to structures. Backfill will be compacted to 95 percent of the standard proctor density. No compaction will be required for backfill placed below water surfaces at the S-345 structures. Backfill placed below water will be predominantly clean sand obtained from adjacent excavations.

f. Dewatering.

(1) S-345A, B, and C. These culvert structures are 72 inch corrugated metal pipes on gravel bedding and are proposed to be constructed in the wet. L-67(A) will be breached during culvert construction. Therefore, an

upstream cofferdam will be required to prevent the loss of water from Water Conservation Area 3A during construction.

(2) S-349A, B, and C. Dewatering will be accomplished using a sheetpile cofferdam, a tremie concrete seal, and sump pumps.

g. Stone Protection. Riprap, placed upstream and downstream of the structures, will provide slope protection for soil slopes in the immediate vicinity of the structures.

h. Access Channels Around Spur Levees. Shallow channels will be excavated around the spur levees adjacent to the S-349 structures. These channels will provide boat access around the structures. No borings have been taken along the channel alignments, but the materials are expected to be similar to those along L-67(A). During future investigations, borings will be taken along the channel alignments to better define the geologic conditions. Excavation of the access channels will be as described in paragraph 1d above.

2. S-345A, S-345B, and S-345C Discharge Canals and Containment Levees.

a. Investigations Performed. Core borings were taken along L-67(A) in 1960 and along L-67(C) in 1965. No other subsurface investigations were performed for this report. Core borings and material tests will be performed and included in the FDM report. The canals and containment levee%wwill be constructed between L-67(A) and L-67(C).

b. Materials Encountered. The foundation materials are anticipated to be similar to those encountered beneath L-67(A) and L-67(C). The general geology of the area consists of medium hard to hard, porous, solution riddled, permeable limestone overlain by 3 - 6 feet of peat. The groundwater table is located near or above the ground surface.

c. Discharge Canal Excavation. The maximum depth of excavation will be approximately 7 feet. Unconsolidated surficial materials can be excavated by conventional means. Excavation of the hard underlying limestone materials will require blasting. Proposed excavation side slopes are 1 vertical on 3 horizontal. Excavated materials will be placed in the adjacent containment levees. The surficial peat materials will be stripped, stockpiled, and used for levee slope plating. Since canal excavation can be performed in the wet, no dewatering will be required.

d. Containment Levees.

(1) Foundation Conditions. No levee foundation preparation will be required. Levee fill material will be placed on the existing ground. The 3 - 6 foot surficial layer of peat should displace and consolidate during levee construction. The underlying limestone will provide adequate levee support. No dewatering of the levee foundations will be required.

(2) Levee Settlement. The total construction displacement, consolidation, and post-construction settlement is 3.5 feet. This number is based on existing borings in the general area of the levees. A detailed

settlement analysis will be performed after subsurface information in the containment levee area is obtained from future borings.

(3) Containment Levee Fill. Levee fill will be obtained from the adjacent required discharge canal excavation and commercial borrow sites near Miami. Fill materials will mainly consist of limestone. All boulders produced by blasting the hard limestone will be placed in the interior portions of the levees, covered with finer materials, and dressed to the required grade. Proposed levee fill side slopes are 1 vertical on 2 horizontal. Levee fill placed above water surfaces will be compacted to 90 percent of the standard proctor density.

3. Structures S-355A and S-355B.

a. Investigations Performed. Core borings were drilled at about 2000 - 3000 foot intervals along L-29 during 1950, 1951, and 1960. In 1973, ten additional core borings were drilled along the north berm of the borrow canal and in the canal itself. No other subsurface investigations were performed for this report. The structures will be constructed in L-29. Core borings and material tests at each structure site will be performed and included in the FDM.

b. Materials Encountered. The foundation materials are anticipated to be similar to those encountered beneath L-29. These materials consist of 3 - 6 feet of peat at the ground surface. Underlying the peat is interbedded hard and medium hard limestone. The limestone is porous, solution riddled, and permeable. The groundwater table is located near or above the ground surface. A geologic cross-section of L-29 with the approximate locations of where S-355A and S-355B will be constructed is shown on Plate G-2.

c. Foundation Conditions. The ground surface elevation at the structure sites is approximately 7.0 feet, NGVD. The structures will be founded at approximately elevation -14.0 feet, NGVD in medium hard to hard limestone. These materials will provide adequate structural support.

d. Excavation. The maximum depth of excavation will be approximately 21 feet. Unconsolidated surficial materials can be excavated by conventional means. Excavation of the hard underlying limestone materials will require blasting.

e. Structural Backfill. Backfill material will be obtained from the required structure and levee breaching excavations. Select backfill consisting of material with no more than 10 percent passing a No. 200 sieve will be placed adjacent to the structures. Backfill will be compacted to 95 percent of the standard proctor density.

f. Dewatering. Dewatering will be accomplished using a sheetpile cofferdam, a tremie concrete seal, and sump pumps.

g. Stone Protection. Riprap, placed upstream and downstream in the immediate vicinity of the structures, will provide slope protection for any soil slopes.

4. Residential Area Flood Mitigation.

a. Pump Stations S-356 and S-357.

(1) Investigations Performed. Analyses for this report were performed using subsurface information obtained for the construction of S-334, L-29, and L-31(N). No other subsurface investigations were performed for this report. Core borings and material tests will be performed at the structure sites and included in the FDM. Structure S-356 will be constructed near existing S-334 where L-29 and L-31(N) intersect. Structure S-357 will be constructed in or adjacent to L-31(N).

(2) Materials Encountered. The foundation materials are anticipated to be similar to those encountered beneath L-29 and L-31(N). The structure sites are characterized by thin deposits of peat and sandy, organic silts and clays that contain limestone fragments. This layer of soft overburden varies in thickness from 0 - 6 feet. The underlying rock consists predominantly of solution riddled, highly permeable, interbedded medium hard and hard limestone. Geologic cross-sections of L-29 and L-31(N), with an approximate location of where S-356 will be constructed, are shown on Plates G-2 and G-3. A geologic cross-section of L-31(N), with an approximate location of where S-357 will be constructed, is shown on Plate G-3.

(3) Foundation Conditions. The structure founding elevations have not been determined. However, the area is underlain predominantly by medium hard to hard limestone that will provide adequate structural support.

(4) Excavation. Unconsolidated surficial materials can be excavated by conventional means. Excavation of the underlying hard limestone materials will require blasting.

(5) Structural Backfill. Backfill material will be obtained from the required structure and levee breaching excavations. Select backfill consisting of material with no more than 10 percent passing a No. 200 sieve will be placed adjacent to the structures. Backfill will be compacted to 95 percent of the standard proctor density.

(6) Dewatering. The groundwater table is near the ground surface. Dewatering will be accomplished using a sheetpile cofferdam, a tremie concrete seal, and sump pumps.

(7) Stone Protection. Levee and canal slope protection will be provided for soil slopes in the immediate vicinity of the structures by riprap placed upstream and downstream of the structures.

b. Residential Area Flood Mitigation Levees.

(1) Investigations Performed. A pump test was conducted in 1972 near the L-31(N) borrow canal. This test was approximately 0.7 miles northeast of pump station S-331. The USGS conducted an additional pump test about 1.5 miles north of S-331 near Howard Drive. Two additional pump tests were performed in 1986 in the proposed mitigation area. One of these tests was

located at the northwest corner of the proposed levees. Section B (Seepage Analysis) contains further information about the pump tests. Plate G-4 shows the locations of the existing and planned pump test sites in this area.

Additional core borings, pump tests, and material tests will be performed and included in the FDM. The proposed levees will tie into and be constructed in the area of L-31(N). Analyses for this report were performed using subsurface information available from the pump tests and construction of L-31(N). Proposed levee alignments with previous and proposed pump test locations are shown on Plate G-4.

(2) Materials Encountered. The foundation materials are anticipated to be similar to those encountered beneath L-31(N). The project site is characterized by thin deposits of calcareous silt and peat. The silt is underlain by solution riddled, highly permeable, interbedded medium hard and hard limestone.

(3) Foundation Conditions. No levee foundation preparation will be required. The levees will be founded on the thin layer of silt and peat that should displace and consolidate during levee construction. The underlying medium hard and hard limestone will provide adequate levee support. No dewatering of the levee foundation will be required.

(4) Levee Settlement. Levee settlement is not anticipated to be a problem because surficial silt and peat deposits are thin and underlain by limestone. The total construction displacement, consolidation, and post-construction settlement for the proposed levee is 1.3 feet. The consolidation settlement in the peat layer was calculated using strengths obtained for peats in the Upper St. Johns project area in central Florida. A detailed levee consolidation settlement analysis will be performed when subsurface information along the levee alignment is available.

(5) Levee Fill Materials. Approximately 35 percent of the required levee fill material will be obtained from the adjacent collector canal excavation consisting predominantly of limestone. The remainder of the fill material will be obtained from commercial sources or from an existing spoil bank just east of the L-31(N) borrow canal.

(6) Levee Design Slopes. Original plans for the flood mitigation levee included a southern levee along Richmond Drive with hurricane runup protection. The proposed levee was approximately 12.5 feet high. Computer models predicted a maximum head of approximately 9.5 feet under SPF and hurricane runup conditions. A slope stability analysis was done for this critical section using the UTEXAS2 computer program.

For levee side slopes of 1 vertical on 3 horizontal, the End of Construction factor of safety is 2.02 and the Steady-State Seepage factor of safety is 1.25. Although EM 1110-2-1913 (Design and Construction of Levees, 1978) requires a factor of safety of 1.4 for this condition, the levee would not be subjected to the maximum predicted head long enough to allow steady seepage to develop. Therefore, a factor of safety of 1.25 is adequate for this condition.

After this analysis was done, however, the southern reach of the levee and the hurricane protection was dropped from the recommended plan. Although the analyzed section is no longer proposed, it is still the critical case. A more detailed stability analysis will be done in the FDM design phase when more water level information is available. Plates G-5 and G-6 show the results of the slope stability analyses done for this report.

(7) Seepage Collector Canal Excavation and Design Slopes. The maximum depth of excavation will be approximately 12 feet. Excavated materials will be placed in the adjacent flood mitigation levees. Unconsolidated surficial materials can be excavated by conventional means. The surficial materials (silt and peat) will be stripped, stockpiled, and used for levee slope plating. Excavation of the underlying hard limestone materials will require blasting. Since the canal side slopes will be predominately cut in limestone materials, the proposed canal side slopes are 1 vertical on 1 horizontal. Since canal excavation can be performed in the wet, no dewatering will be required.

(8) Trash Removal. In several areas along the proposed levee alignment and on the lands that will be acquired by Everglades National Park, there are areas of illegally dumped trash. A reconnaissance trip to the area revealed that most of the trash consisted of old automobiles, household appliances, and discarded automobile tires. In an effort to determine if Hazardous or Toxic Waste (HTW) was present at any of the sites, the trash and trash areas were examined for burns or acidic discolorations, strong odors, empty or leaking storage drums, car batteries, paint cans, and other obvious signs of possible HTW. From all indications, the trash in these areas is unwanted municipal waste that has been illegally dumped. There is no reason to believe that any of these areas are HTW sites.

During construction, the trash will be removed by the contractor and either salvaged or disposed of in municipal landfills in accordance with applicable laws and ordinances. If the contractor finds a potential HTW problem during the removal of the trash, he will be required to stop work and contact the District Office immediately. At that point, a risk assessment will be made and appropriate actions will be taken to handle the problem.

B. SEEPAGE ANALYSIS

5. General. The Biscayne aquifer dominates the geology in the study area. The aquifer is very porous with the transmissivity, the rate of groundwater flow through a unit width of the aquifer, increasing rapidly frow west to east. There is limited pump test data available and the variability of the limestone makes it difficult to define the aquifer characteristics over the large study area. Therefore, transmissivities in the study area were obtained by adjusting values estimated by Appel (Appel, 1973) to reflect the results of the pump tests available for this study. Figure G-1 shows the resulting transmissivity map.

6. Subsurface Investigations.

a. Core Borings. Core borings in the East Everglades Residential Area include the existing borings along L-31(N) and the borings drilled during the two 1986 pump tests. Based on these borings, the geology of the area consists of 0 - 4 feet of organic topsoil underlain by approximately 50 feet of solution riddled, cavernous, medium hard to hard limestone. The groundwater table in this area is approximately 2 feet beneath the ground surface.

b. Pump Tests.

(1) Pump Tests No. 1, 3, and 4. These tests were conducted in 1951 and 1952 near Tamiami Canal and Levee 30, 12.5 miles north of S-335, and near Tamiami Canal about 8 miles west of Levee 30, respectively. The geology at these sites consists of 2 - 4 feet of peat and organic, calcareous silts and clays underlain by Miami Oolite and Fort Thompson limestone. Detailed geologic sections, as well as the mathematical theory and test data for each site, are presented in C&SF Part I, Supplement 7. The table below shows the transmissivities calculated from these tests:

Pump Test No.	Transmissivity (CFS/FT)
1	33.0
3	7.4
4	10.3

(2) Pump Test near S-331. Early in 1972, a pump test was conducted near the L-31(N) borrow canal about 0.7 miles northeast of pump station S-331. C&SF Part V, Supplement 52 describes the test and the methods used to evaluate the data. The transmissivity (T) for this site is 38 CFS/FT with an effective porosity (n) of 0.06. The test did not reveal any extensive layers of low permeability material. Although the transmissivity is very high, it does not greatly exceed the transmissivity at the site of Pump Test No. 1. Additionally, the USGS obtained a similar transmissivity (T = 41.6 CFS/FT) at a pump test site located about 1.5 miles north of S-331.

(3) Pump Test PT-EE-2. In 1986, a pump test was conducted in the East Everglades Residential Area at the intersection of SW 120th Street and SW 212th Avenue. The geology at this site consists of a 1 foot layer of slightly organic silt that is underlain by approximately 52 feet of highly porous limestone. Below the aquifer is a clay layer of the relatively impermeable Tamiami Formation. Figures G-2 and G-3 show the boring log for the pump well.

The 12 inch diameter pump well was drilled through the aquifer to a depth of 53.5 feet. To permit observation of the aquifer drawdown during testing, eight piezometers were installed in lines running east and south of the pump well. A vertical velocity meter measured the distribution of flow into the pump well during the test. The well was pumped at discharge rates of 1,010 and 1,325 gallons per minute.

The transmissivity of the aquifer was computed for each rate of pump and the results from both pump rates were averaged. The table below shows the transmissivities computed from the test results:

Pump Test Date	Drawdown Data	Recovery Data
29 Jan 1986	$T = \frac{Q}{2.73s} = \frac{(1325\ GPM)/449}{2.73(0.193)} = 5.6\ CFS/FT$	$T = \frac{Q}{2.73s} = \frac{(1325\ GPM)/449}{2.73(0.194)} = 5.6\ CFS/FT$
30 Jan 1986	$T = \frac{Q}{2.73s} = \frac{(1010\ GPM)/449}{2.73(0.132)} = 6.2\ CFS/FT$	$T = \frac{Q}{2.73s} = \frac{(1010\ GPM)/449}{2.73(0.130)} = 6.3\ CFS/FT$

(4) Pump Test PT-EE-3. An additional pump test was conducted in 1986 in the agricultural area south of the East Everglades Residential Area. This test was located near SW 23rd Avenue and SW 200th Street. The aquifer at this location consists of 44 feet of limestone. Below the limestone is a 10 foot deep cavity filled with sand that is underlain by dense sand. Figures G-4 and G-5 show the boring log for the pump well.

After drilling the 6 inch diameter pump well through the underlying aquifer, eight piezometers were installed to measure the aquifer drawdown. A vertical velocity meter measured the distribution of flow into the pump well during the test. The well was pumped at discharge rates of 594 and 954 gallons per minute.

The table below shows the transmissivities computed from the test results:

Pump Test Date	Drawdown Data	Recovery Data
06 Mar 1986	$T = \frac{Q}{2.73s} = \frac{(954\ GPM)/449}{2.73(0.087)} = 9.0\ CFS/FT$	$T = \frac{Q}{2.73s} = \frac{(954\ GPM)/449}{2.73(0.086)} = 9.0\ CFS/FT$
07 Mar 1986	$T = \frac{Q}{2.73s} = \frac{(594\ GPM)/449}{2.73(0.037)} = 6.2\ CFS/FT$	$T = \frac{Q}{2.73s} = \frac{(594\ GPM)/449}{2.73(0.037)} = 13.1\ CFS/FT$

7. Design Assumptions.

a. Aquifer Characteristics. Based on the results from the existing pump tests, the following assumptions for permeability were used in the seepage analysis:

Levee north of Howard Drive:	k = 0.114 FEET/SECOND
Levee south of Howard Drive:	k = 0.241 FEET/SECOND
L-31(N) and Adjacent Area:	k = 0.741 FEET/SECOND
L-31(N) south of Tamiami Trail:	k = 0.400 FEET/SECOND

The permeabilities calculated from PT-EE-2 and PT-EE-3 were used for the levee sections north and south of Howard Drive. Similarly, the S-331 and USGS pump tests along L-31(N) provided the permeability along the east edge of the residential area. Pump Tests 1, 3, and 4 provided the permeability south of Tamiami Trail. Additionally, regional aquifer data from the USGS and the transmissvity contours shown in Figure G-1 were also considered in the determination of aquifer characteristics for the study area.

A clear understanding of the aquifer flow characteristics is essential to the design of both the levees and the pump stations. Research into other aquifer tests in the area did not provide any additional useful information. The possibility of doing more pump tests using local irrigation wells was not feasible due to the pump limitations and well spacings available. Right of entry problems precluded the possibility of performing additional pump tests in the time available to complete this report.

During the detailed design phase of the project, however, more pump tests will be done to get a better understanding of the aquifer transmissivity. Additionally, several recharge tests will be done while the levee core borings are being drilled.

b. Geologic Stratigraphy. In much of the study area, there is a layer of peat and silty clay overlying the limestone aquifer. This stratum can be as thick as 4 feet. Around the residential area, however, field investigations and site visits indicate that this layer is not thick enough or continuous enough to reduce the amount of seepage flowing into East Everglades. Additionally, no information is currently available on the permeability of this layer. Therefore, the seepage analysis done for this report assumes the limestone aquifer to be at the ground surface.

North of the East Everglades Residential Area, however, along the L-31(N) alignment, the organic layer is thicker and more continuous. Therefore, for this analysis, the top layer was modeled as a relatively impermeable blanket. This assumption is based on previous work done in the area and recent site visits. By assuming a large transmissivity for the limestone aquifer and a continuous, relatively impermeable surficial blanket, the seepage rate into L-31(N) during flood stages is effectively reduced.

During future subsurface investigations, a detailed evaluation of the top stratum will be made. Test pits will be dug and percolation tests will be performed to determine the continuity and permeability of this layer. If the

layer is continuous and impermeable enough to impede the seepage flow in the residential area, it will be accounted for in the FDM seepage analysis. Similarly, if future testing indicates that the layer does not provide sufficient coverage to reduce seepage along the L-31(N) alignment, then the hydraulic models and seepage analyses will be adjusted.

c. Levee Construction. In the seepage analysis, flow through the levee is considered to be negligible compared with the flow under the levee. Therefore, the levee must be relatively impermeable. This will require some special requirements for the levee fill used in construction or the use of some type of impermeable barrier such as a geomembrane. A clay blanket is not practical for this design because of the amount of clay required and the scarcity of clay borrow areas in South Florida.

8. Seepage Analysis.

a. Computer Model. In order to compute water levels and pump capacities, a computer program, described in Appendix D, was developed. In order to compute the seepage under the levees, the equation Q = §kH was used where

Q = seepage in CFS/FT of levee

$$\S = \frac{1}{\left[0.86 + \frac{W}{d}\right]} \qquad \text{for } W/d > 1$$

$$\S = -\tfrac{1}{2} \log\left[\frac{\cosh(W/8d-1)}{3\cosh(W/8d-1)}\right] \qquad \text{for } W/d < 1$$

W = base width of levee in feet
d = depth of aquifer in feet
k = permeability of aquifer in ft/day
H = head loss across levee in feet

The equations for § were derived by Forchheimer in 1917 for laminar flow. Although turbulent flow was encountered near the pump in nearly all pump tests, seepage flow beneath the levees was assumed to be laminar. Turbulent flow in the aquifer was only considered near the pump stations.

b. Levee Underseepage Analysis. Three different types of analyses were done to compute the flow under the two levee system. These analysis techniques are a traditional flow net analysis, a Method of Fragments analysis described in EM 1110-2-1913 (Design and Construction of Levees, 1978), and a finite element computer analysis. All the analysis techniques provided approximately the same flow per foot of head.

Seepage under the proposed levee ranged from 260 CFS/MILE/FOOT OF HEAD to 860 CFS/MILE/FOOT OF HEAD depending on the permeability and levee base width used. Seepage beneath L-31(N) ranged from 1275 CFS/MILE/FOOT OF HEAD to 2000 CFS/MILE/FOOT OF

HEAD in the residential area. Seepage beneath L-31(N) south of Tamiami Trail was approximately 140 CFS/MILE/FOOT OF HEAD due to the influence of the surficial blanket. Plate G-4 shows a method of fragments analysis done for a 1 foot head differential in the residential area.

Using the above the seepage rates, the seepage under the levee system was computed for the pool levels provided by the computer model for the SPF and 1969 10 year storms. Two storm events were analyzed to ensure that the increased flooding would be mitigated for all storm conditions. Results from the analyses revealed that the 1969 storm had higher head differentials and would control the pump station design. The total inflow beneath both the double levee and L-31(N) was approximately the same as the 500 cfs used in the computer model to size the proposed pump station S-357. Further refinements of the computer model and more detailed investigations into the transmissivities in the project area will provide better data to use in designing the pump station capacity in the FDM.

Although the proposed plan includes two seepage levees, initial seepage calculations indicate that a single levee with a long seepage path can provide the same reduction of seepage. During the FDM design, a single flood mitigation levee, as well as other alternatives such as flood walls and slurry trenches, will be investigated. The most economical alternative that accomplishes the project's goals will be selected as the final plan.

9. Further Investigations. During the FDM design phase, at least two additional pump tests will be done in East Everglades. Also, core borings will be drilled along the levee alignment and recharge tests will be performed in several of them. Test pits and percolation tests along the proposed levee alignment and along L-31(N) will provide more information about the continuity of the top layer and its ability to reduce seepage.

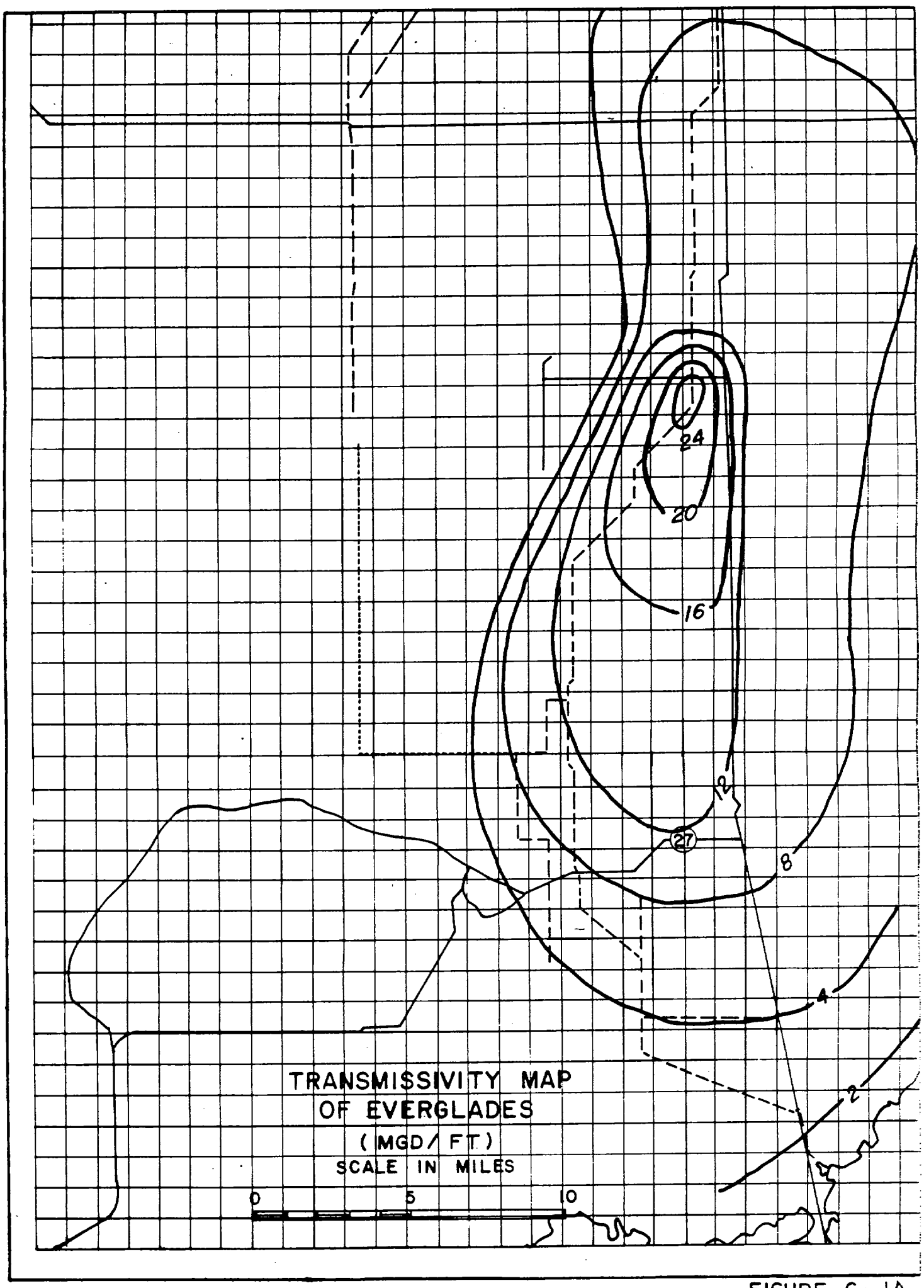

24
20
16
12
27
8
4
2
TRANSMISSIVITY MAP
OF EVERGLADES
(MGD/FT)
SCALE IN MILES
0
5
10

FIGURE G-1A

P.T. EE - 2

DRILLING LOG	DIVISION South Atlantic	INSTALLATION Jacksonville District	SHEET 1 OF 2 SHEETS

1. PROJECT: Pump Tests in Ea. Everglades
2. LOCATION (Coordinates or Station):
3. DRILLING AGENCY: Corps of Engineers
4. HOLE NO. (As shown on drawing title and file number): P.T. EE - 2
5. NAME OF DRILLER: C. Mason
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN:
8. DEPTH DRILLED INTO ROCK:
9. TOTAL DEPTH OF HOLE: 58.0'
10. SIZE AND TYPE OF BIT:
11. DATUM FOR ELEVATION SHOWN (TBM or MSL): M.S.L.
12. MANUFACTURER'S DESIGNATION OF DRILL: Failing 1500
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED: ; UNDISTURBED:
14. TOTAL NUMBER CORE BOXES: 6
15. ELEVATION GROUND WATER: +7.0
16. DATE HOLE: STARTED 11/26/85; COMPLETED 12/4/85
17. ELEVATION TOP OF HOLE: +7.6
18. TOTAL CORE RECOVERY FOR BORING: 78 %
19. SIGNATURE OF INSPECTOR: GEOLOGIST J. Hand

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BLOWS PER 6 IN. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
+7.6	0.0					Bit or Barrel +7.6 SLS/FT
+6.6	1.0		SILT, calcareous, slightly organic, brown (ML)	10	1	6" Rock Bit
			LIMESTONE, permeable, many voids approximately 1/6" diameter, many solution channels ½"/1" diameter, some with secondary calcite deposits, brown to gray	96	-	+3.6 Diamond 4"x3½" DT 60 min HP 50 psi -1.4
				60		" DT 55 min HP 50 psi -5.4
-5.4	13.9					
			LIMESTONE, hard, dense, a vertical solution channel, lined with secondary calcite, light gray	110	-	-5.9 DT [illegible]
-8.2	15.8			90	-	" DT 72 min HP 100 psi -10.9
			LIMESTONE, medium hard, very porous, permeable, solution channels with secondary calcite, light gray Nodules of hard limestone from 20.5' to 22.0' [illegible]	25	-	" DT [illegible] min HP 75 psi -13.9
-14.4	22.8					
			LIMESTONE, soft, porous, permeable, slightly fossiliferous, white, layers of medium hard limestone	80	-	" DT [illegible] min HP 75 psi -16.4
				85		" DT 11 min HP 100 psi -18.9
-19.6	27.2					
			LIMESTONE, hard, dense, white, a few small solution channels from 27.2' to 28.5'	55	-	-20.4 DT 55 min HP 75 psi
-22.1	29.7			98		" DT 5 min HP [illegible]
-22.9	30.5		LIMESTONE, soft, very porous, permeable, white			-24.4
				92		" -29.4

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT)

PROJECT: EVERGLADES P.T. EE-2

FIGURE G-1

DRILLING LOG (Cont Sheet) ELEVATION TOP OF HOLE +7.6

Hole No. PT EE-2

PROJECT: Pump Tests in East Everglades

INSTALLATION: Jacksonville District

SHEET ___ OF ___

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
						-23.4
-31.9	39.5		LIMESTONE, hard, dense, slightly porous, to porous, permeable fossiliferous. layers of medium hardness, white	56	-	Diamond 4 x 5½" DT 26 min HP 75 psi
-34.4	42.0		LIMESTONE, soft, white			-34.4
-35.3	43.5		LIMESTONE, medium hard, fossiliferous, very porous, permeable, many solution channels, white	72	-	" DT 45 min HP 75 ps
-37.2	44.0		LIMESTONE, hard, dense, slightly porous, solution channels, white			-3[illegible].4
			LIMESTONE, medium hard, fossiliferous, very porous, many solution channels, permeable, secondary calcite deposits and shell replacement, light gray soft layers	88	-	" DT 22 min HP 50 psi -42.4
-45.3	53.5			96	-	" DT 30 min HP 75 psi
			CLAY, green, layers of medium quartz sand (CL)			-47.1
-5[illegible]	58.0			--	--	" Settled -5[illegible]

NOTE: 1. Approx. El top of hole +7.6 as determined from USGS Quadrangle map.
2. Level of water table remained constant before, during, and after drilling operations; there was no water return.

FORM 1836-A (ER 1110-1-1801) GPO 1969 OF-[illegible]

PROJECT PUMP TESTS [illegible]

PT. EE-2

FIGURE G-2

Hole No. PT-EE-3

DRILLING LOG	DIVISION South Atlantic	INSTALLATION Jacksonville District	SHEET 1 OF 2 SHEETS

1. PROJECT: Pump Test in East Everglades
2. LOCATION (Coordinates or Station):
3. DRILLING AGENCY: Corps of Engineers
4. HOLE NO. (As shown on drawing title and file number): PT-EE-3
5. NAME OF DRILLER:
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN:
8. DEPTH DRILLED INTO ROCK:
9. TOTAL DEPTH OF HOLE: 57.0'
10. SIZE AND TYPE OF BIT: see remarks
11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL
12. MANUFACTURER'S DESIGNATION OF DRILL: Failing 1500
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED: | UNDISTURBED:
14. TOTAL NUMBER CORE BOXES: 2
15. ~~TOTAL NO. OF CORES~~ GROUND WATER: DEPTH 2.4
16. DATE HOLE: STARTED 3 FEB 86 | COMPLETED 4 FEB 86
17. ELEVATION TOP OF HOLE:
18. TOTAL CORE RECOVERY FOR BORING: 69 %
19. SIGNATURE OF INSPECTOR: GEOLOGIST J. Hand

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			RECHARGE TESTS-GPM/FT. HEAD			Bit or Barrel
6.6'	0.0					6.6'
			LIMESTONE, hard, fossiliferous, porous, permeable, solution channels, brown to gray		--	5" Roll Bit
						1.6'
				80	--	Diamond Ni HP 100 psi DT 40 min
						-3.4'
				70	34	" HP 100 psi DT 60 min
						-8.4'
				30	24	" HP 100 psi DT 40 min
			white, dense, very silty, porous, very slightly permeable, 22.7 to 25.5			-13.4'
				55	34	" HP 100 psi DT 55 min
						-18.4'
-20.9'	27.5			71	41	" HP 100 psi DT 15 min
			LIMESTONE, medium hard, porous, permeable, fossiliferous, white			-23.4'
-25.4'	32.0			82	36	" HP 100 psi DT 50
			LIMESTONE, hard, dense, fossiliferous, very porous, permeable, gray (white, dense, slightly porous 34.2 to 34.9 cavity 35.0 to 35.7, very porous, very permeable, very fossiliferous 37.0 to 44.0			
-29.1'						-34.1'

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PROJECT

PT-EE-3

FIGURE G-3

DRILLING LOG (Cont Sheet) ELEVATION TOP OF HOLE

Hole No. PT-EE-3

PROJECT: Pump Test in East Everglades

INSTALLATION: Jacksonville District

SHEET 2 OF 2 SHEETS

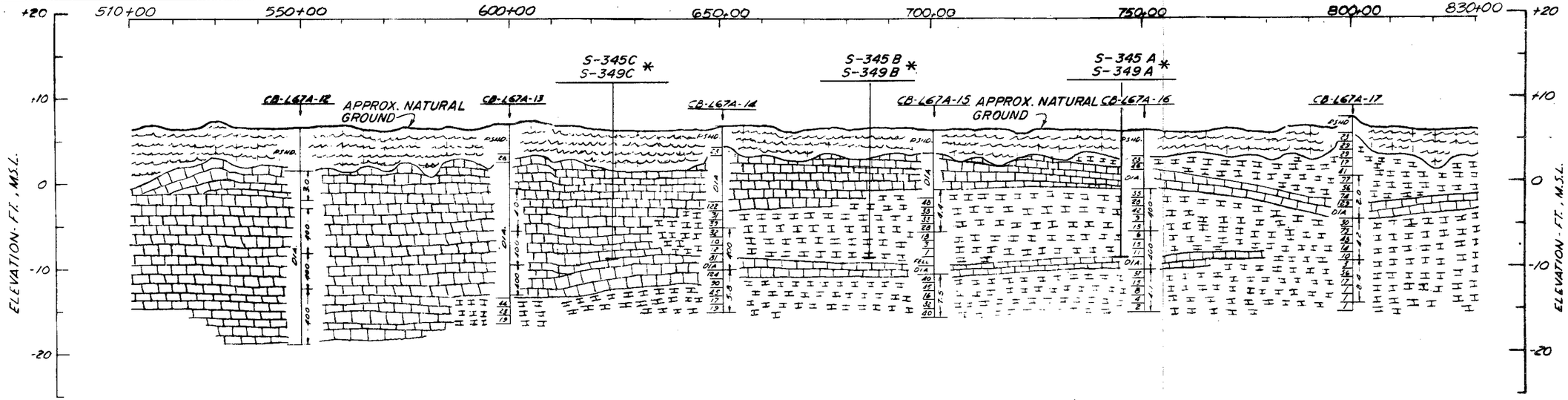
ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			RECHARGE TESTS-GPM/FT. HEAD			-29.1'
				100	41	Diamond M HP 100 psi DT 15 min -33.4'
-37.4'	44.0			40	34	Diamond M HP 100 psi DT 14 min -37.4'
-47.4'	54.0		Cavity Filled with sand	-	--	2" Sampler Settled -47.4'
-49.1'	55.7		SAND, medium, quartz, white (SP)	100	--	2" Sampler
-50.4'	57.0		SAND, fine, quartz, silty, green (SM)			-50.4'

ENG FORM 1836-A (ER 1110-1-1801)

PUMP TEST

Fig. G-4

PT. EE. 2



LEGEND

- LIMESTONE HARD
- LIMESTONE MEDIUM HARD
- SAND
- SILTY SAND
- SILT - LOW PLASTICITY
- PEAT
- CB-L67A-13 — CORE BORING DESIGNATION AND LOCATION - PROFILE VIEW

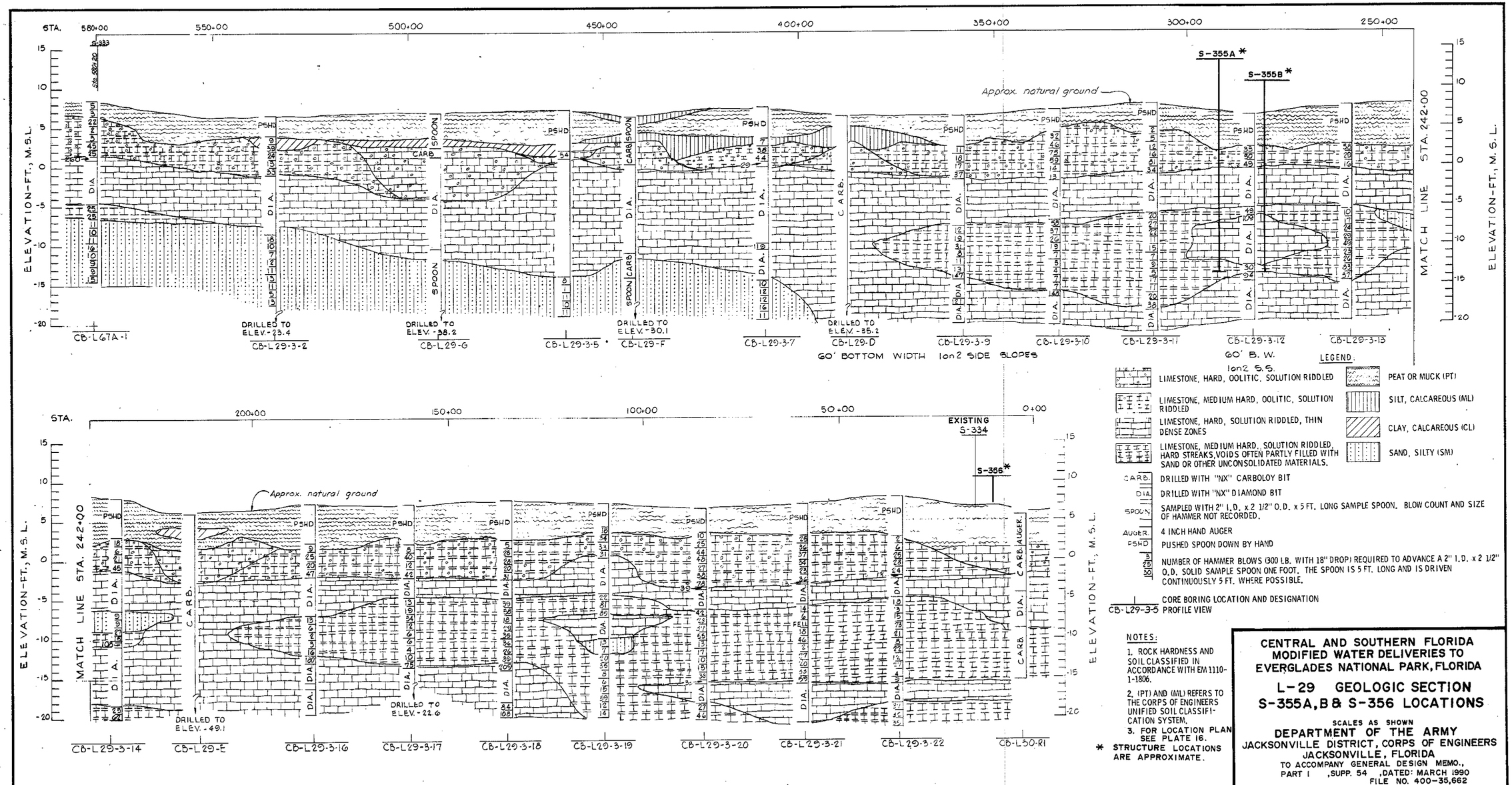
- 7 / 3 / 7 — Number Of Hammer Blows (450 ft. lbs energy) Required to Advance a 2 inch I.D. Sample Spoon 1-Ft.
- PSHD. — Pushed Spoon Down Without Aid Of Hammer
- FELL — Spoon Suddenly Drops As Entering A Cavity
- CASE. — Drilled with "NX" casing
- DIA. — Drilled with "NX" diamond bit
- CARBOLOY — Drilled with "NX" carboloy bit
- 37 — Indicates Recharge Test in GPM/Ft. of head

* STRUCTURE LOCATIONS ARE APPROXIMATE.

NOTE:

1. FOR LOCATION PLAN, SEE PLATE 16.

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK, FLORIDA
L-67A GEOLOGIC SECTION
S-345A,B,C & S-349A,B,C
LOCATIONS
SCALES AS SHOWN
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO., PART I , SUPP. 64 , DATED: MARCH 1990
FILE NO. 400-35,662



NOTES:

1. ROCK HARDNESS AND SOIL CLASSIFIED IN ACCORDANCE WITH EM 1110-1-1806.
2. (PT) AND (ML) REFERS TO THE CORPS OF ENGINEERS UNIFIED SOIL CLASSIFICATION SYSTEM.
3. FOR LOCATION PLAN SEE PLATE 16.

* STRUCTURE LOCATIONS ARE APPROXIMATE.

CENTRAL AND SOUTHERN FLORIDA MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK, FLORIDA

L-29 GEOLOGIC SECTION S-355A, B & S-356 LOCATIONS

SCALES AS SHOWN

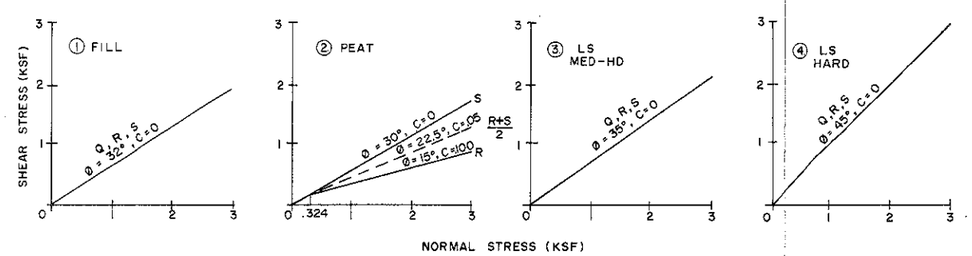
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

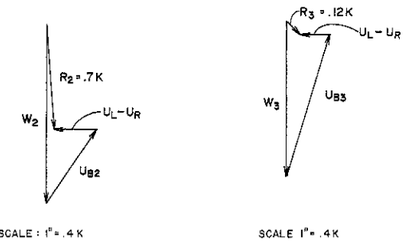
TO ACCOMPANY GENERAL DESIGN MEMO., PART I ,SUPP. 54 ,DATED: MARCH 1990
FILE NO. 400-35,662

MTL.	SLICE	HORIZ. WIDTH (FT.)	SLICE HT. (FT.) LT. SIDE	RT. SIDE	AVERAGE	AREA OF SLICE (FT.²)	WEIGHT (KIPS) MOIST	SUBMERGED	TOTAL	BASE LENGTH OF SLICE, ΔL (FT.)	CΔL (KIPS)	$C_D = \frac{CΔL}{F.S.}$ (KIPS)*	$\phi_D = \frac{\phi}{F.S.}$ (DEGREES)*	WT. (EQ. FACTOR)** (KIPS)	PZ HT AT BASE OF SLICE (FT.) LT.	RT.	WATER PRESS. ON BASE (KSF) LT.	RT.	U_L (K)	U_R (K)	$U_L - U_R$ (K)	UPLIFT U_B (K)
1	1	1.8	0	2.0	1.0	1.8	.2	—	.2	3.4	0	0	25.6	0	—	—	—	—	—	—	—	—
1	2	5.0	2.0	1.0	1.5	7.5	.8	—	1.2	—	—	—	—	0	0	2.8	0	.2	0	.3	.3	.6
1	2	5.0	0	3.0	1.5	7.5	—	.4		6.2	0	0	25.6	0								
1	3	4.9	1.0	0	.5	2.5	.3	—		—	—	—	—	0	2.8	3.8	.17	.23	.23	.43	.2	1.0
1	3'	4.9	3.0	2.4	2.7	13.2	—	.7	1.04	—	—	—	—	0								
2	3"	4.9	0	1.4	.7	3.4	—	.04		5.0	.25	.2	S 24	0	—	—	—	—	—	—	—	—
1	4	3.3	2.4	1.4	1.9	6.3	—	.3		—	—	—	—	0								
2	4'	3.3	1.4	1.8	1.6	5.3	—	.07	.31	3.4	.17	.14	S 24	0								
1	5	4.0	1.4	0	.7	2.7	—	.14		—	—	—	—	0								
2	5'	4.0	1.8	1.2	1.5	6.0	—	.08	.22	4	.2	.16	S 24	0								
2	6	3.6	1.2	0	.6	2.2	—	.03	.03	3.8	.19	.15	S 24	0								

* F.S. = 1.25

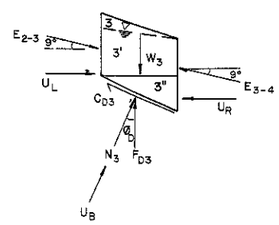
** EQ. COEFFICIENT = 0

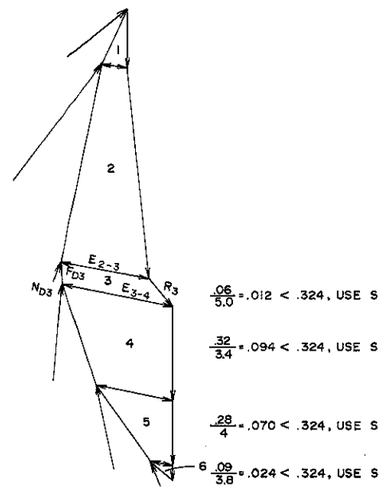




RESULTANTS OF WEIGHT AND WATER FORCES ON SLICES 2 & 3

FORCES ON TYPICAL SLICE
(N.T.S.)



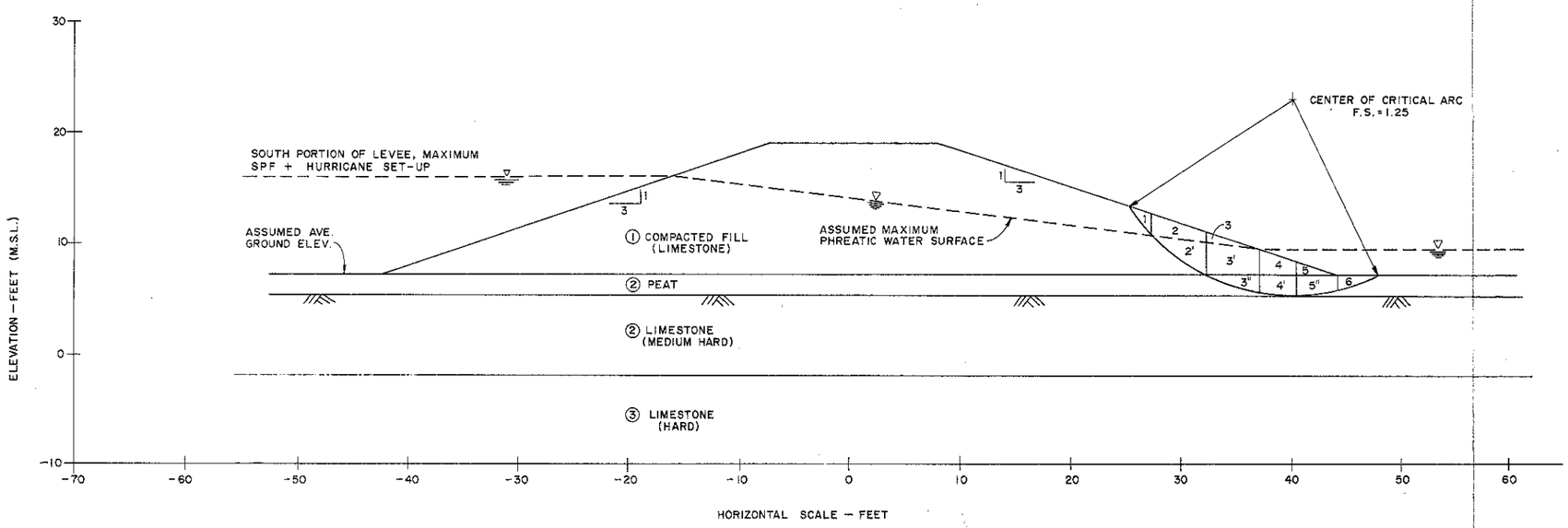


COMPOSITE FORCE POLYGON

F.S. = 1.25

SCALE: 1" = .2 KIPS

ASSUMED DESIGN DATA									
MTL. NO	MTL.	UNIT γ_M	WT. (PCF) γ_{SAT}	Q C(PSF)	Q ø°	R C(PSF)	R ø°	S C(PSF)	S ø°
①	FILL	110	115	0	32	0	32	0	32
②	PEAT	70	75	300	0	100	15	0	30
③	LS MED-HD	120	125	0	35	0	35	0	35
④	LS HARD	140	145	0	45	0	45	0	45



NOTES:

1. SLOPE STABILITY ANALYSES PERFORMED USING UTEXAS 2 (SPENCER METHOD) COMPUTER PROGRAM. VARIOUS ARC AND WEDGE FAILURE SURFACE LOCATIONS WERE TRIED. FAILURE SURFACE WHICH PRODUCED THE LOWEST F.S. IS SHOWN.

2. BORING CB-L31N-W11 WAS USED FOR SECTION SUBSURFACE INFORMATION.

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK, FLORIDA

RESIDENTIAL AREA LEVEE
SLOPE STABILITY
STEADY SEEPAGE

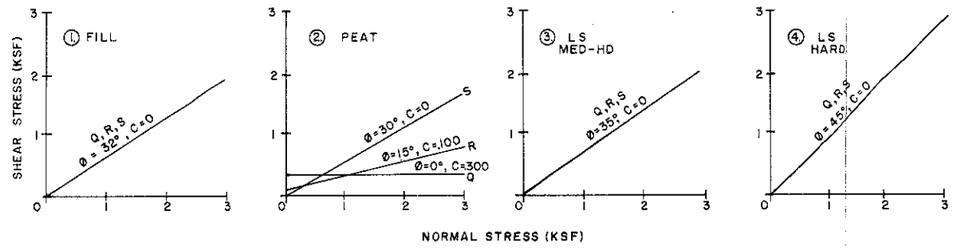
SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.
PART I , SUPP. 54 , DATED: MARCH 1990
FILE NO. 400-35.662

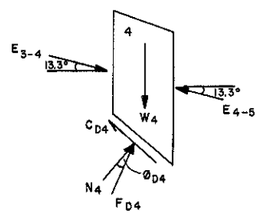
MTL.	SLICE	HORIZ. WIDTH (FT.)	SLICE HT. (FT.) LT. SIDE	RT. SIDE	AVERAGE	AREA OF SLICE (FT.^{2})	WEIGHT (KIPS) MOIST	SUBMERGED	TOTAL	BASE LENGTH OF SLICE, ΔL (FT.)	$C\Delta L$ (KIPS)	$C_D = \frac{C\Delta L}{F.S.}$ (KIPS)*	$\phi_D = \frac{\phi}{F.S.}$ (DEGREES)*	WT. (EQ. FACTOR)** (KIPS)
1	1	4.5	0	4	2	9	.99	—	.99	6	0	0	16	—
1	2	2.6	4	6.4	5.2	13.5	1.5	—	1.5	3.5	0	0	16	—
1	3	3.9	6.4	8.5	7.5	29	3.2	—	3.2	5	0	0	16	—
1	4	4	8.5	9.4	9	36	3.9	—		3.5	0	0	16	—
2	4'	1.2	0	1	.5	.6	—	.01	3.9	1.6	.4	.2	0	—
1	5	4.4	9.4	8	8.7	38.3	4.2	—		4.4	0	0	16	—
2	5'	4.4	1	1	1	4.4	—	.06	4.3	4.4	1.3	.7	0	—
1	6	5	8	6	7	35	3.9	—		5	0	0	16	—
2	6'	5	1	1	1	5	—	.06	4	5	1.5	.8	0	—
1	7	5	6	4.5	5.3	26.5	3	—		5	0	0	16	—
2	7'	5	1	1	1	5	—	.06	3.1	5	1.5	.8	0	—
1	8	5.5	4.5	2.7	3.6	19.8	2.2	—		5.5	0	0	16	—
2	8'	5.5	1	0	.5	2.8	—	.03	2.5	5.6	1.7	.8	0	—
1	9	5.5	2.7	0	1.4	7.4	.82	—	.82	5.6	0	0	16	—

* F.S = 2.02

** EQ. COEFFICIENT = 0

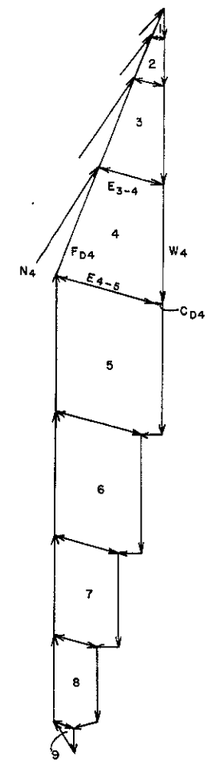


ASSUMED DESIGN DATA									
MTL. NO	MTL.	UNIT γ_M	WT. (PCF) γ_{SAT}	Q C(PSF)	Q Ø°	R C(PSF)	R Ø°	S C(PSF)	S Ø°
①	FILL	110	115	0	32	0	32	0	32
②	PEAT	70	75	300	0	100	15	0	30
③	LS MED-HD	120	125	0	35	0	35	0	35
④	LS HARD	140	145	0	45	0	45	0	45



FORCES ON TYPICAL SLICE

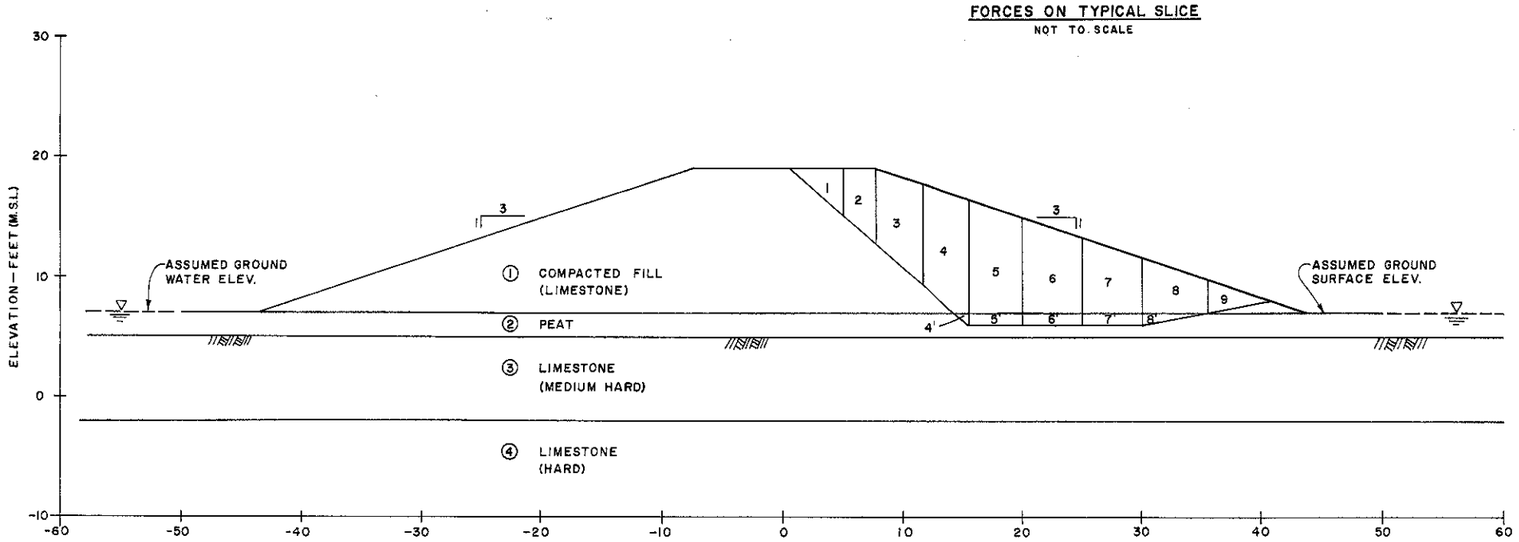
NOT TO SCALE



COMPOSITE FORCE POLYGON

F.S. = 2.02

SCALE: 1" = 2K



NOTES:

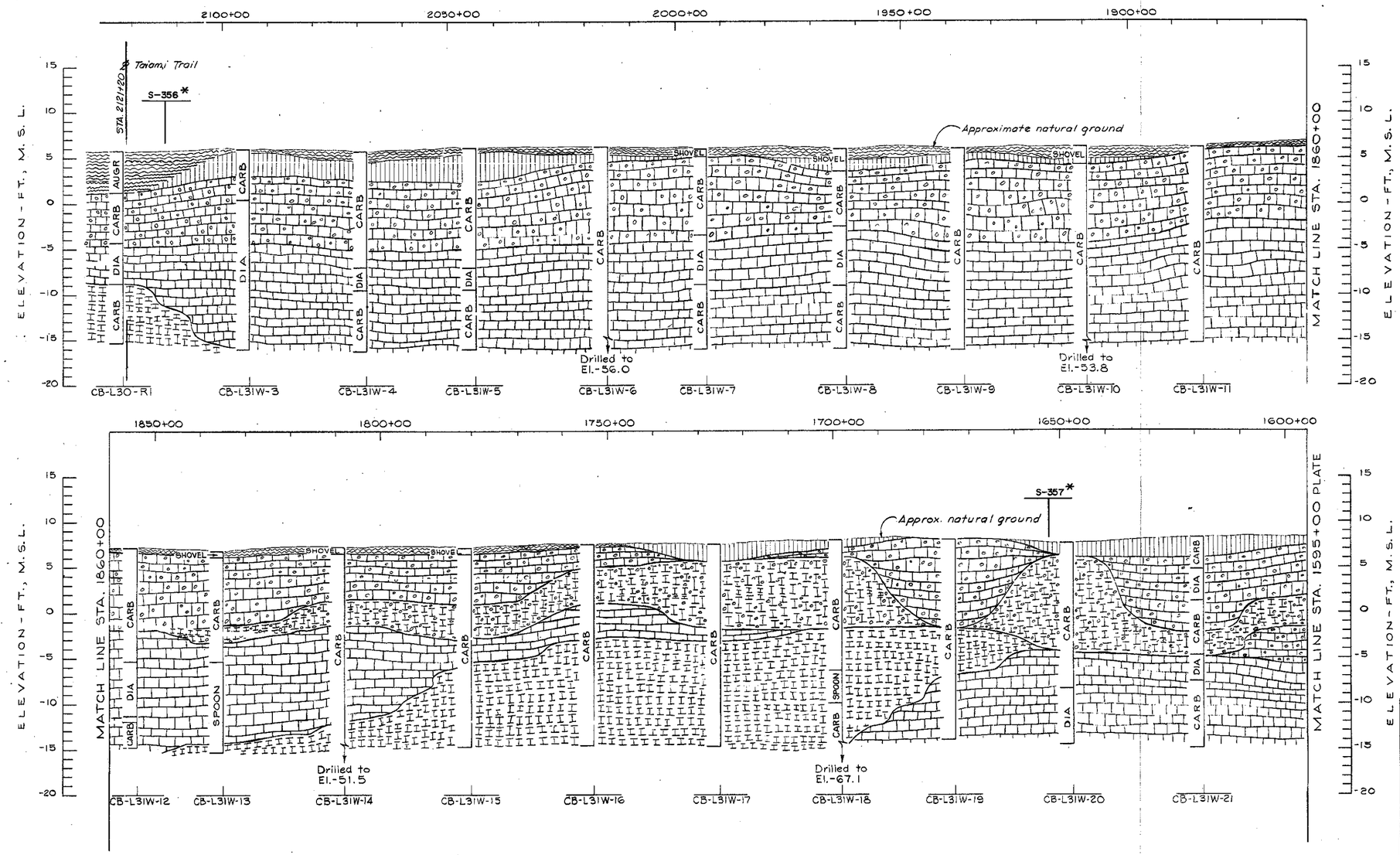
1. SLOPE STABILITY ANALYSES PERFORMED USING UTEXAS 2 (SPENCER METHOD) COMPUTER PROGRAM. VARIOUS ARC AND AND WEDGE FAILURE SURFACE LOCATIONS WERE TRIED. SOME SHALLOW ARC FAILURE SURFACES PRODUCED A F.S. = 1.88. DEEP FAILURE SURFACE WHICH PRODUCED THE LOWEST F.S. IS SHOWN.
2. BORING CB-L3IN-W11 WAS USED FOR SECTION SUBSURFACE INFORMATION.

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK, FLORIDA

RESIDENTIAL AREA LEVEE
SLOPE STABILITY
END OF CONSTRUCTION

SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO., PART I , SUPP. 54 , DATED: MARCH 1990
FILE NO. 400-35.662



LEGEND:

- PEAT OR MUCK (PT)
- SILT, CALCAREOUS (MARL) (ML)
- LIMESTONE, HARD, OOLITIC, SOLUTION RIDDLED
- LIMESTONE, MEDIUM HARD, OOLITIC, SOLUTION RIDDLED
- LIMESTONE, HARD, SOLUTION RIDDLED, CONTAINS THIN DENSE ZONES
- LIMESTONE, MEDIUM HARD, SOLUTION RIDDLED, THIN HARD STREAKS, VOIDS OFTEN PARTLY FILLED WITH SAND OR OTHER UNCONSOLIDATED MATERIALS.
- CARB — DRILLED WITH "NX" CARBOLOY BIT
- DIA — DRILLED WITH "NX" DIAMOND BIT
- SPOON — SAMPLED WITH 2 INCH I.D. x 2-1/2 INCH O.D. x 5 FT. LONG SAMPLE SPOON. BLOW COUNT AND SIZE OF HAMMER NOT RECORDED
- AUGR — 4-INCH HAND AUGER
- 100/5 — NUMBER OF HAMMER BLOWS (300 LB. WITH 18-INCH DROP) REQUIRED TO ADVANCE A 2 INCH I.D. x 2-1/2 INCH O.D. SOLID SAMPLE SPOON ONE FOOT. THE SPOON IS 5 FT. LONG AND IS DRIVEN CONTINUOUSLY 5 FT. WHERE POSSIBLE.
- CB-L31W-7 — CORE BORING LOCATION AND DESIGNATION PROFILE VIEW

NOTES:

1. ROCK HARDNESS AND SOIL CLASSIFIED IN ACCORDANCE WITH EM 1110-1-1806.
2. (PT) AND (ML) REFERS TO THE CORPS OF ENGINEERS UNIFIED SOIL CLASSIFICATION SYSTEM
3. FOR LOCATION PLAN SEE PLATE 16.

* STRUCTURE LOCATIONS ARE APPROXIMATE.

CENTRAL AND SOUTHERN FLORIDA MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK, FLORIDA

L-31(N) GEOLOGIC SECTION S-356 & S-357 LOCATIONS

SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO., PART I, SUPP. 54, DATED: MARCH 1990
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LEVEE UNDERSEEPAGE ANALYSIS

- DIVIDE LEVEE SYSTEM INTO NORTH-SOUTH SECTIONS (SEE PUMP TEST LOCATIONS-PLAN)
- TO COMPUTE UNDERSEEPAGE (Q) USE EM 1110-2-1913 (1978) APPENDIX B, PARAGRAPH B-5, PAGE B-12, EQTN. (B-11) AND EQTN. (B-13).

(B-11) $Q = \$ k H$ — Q = UNDERSEEPAGE (CFS); $ = SHAPE FACTOR; k = ACQUIFER PERMEABILITY (FPS); H = HEAD (FT)

(B-13) $\$ \quad \frac{d}{L + 0.86\ d}$ — d = ACQUIFER DEPTH; L = LEVEE BASE LENGTH

NEW LEVEE NORTH SECTION - SEE TYPICAL SECTION BELOW

- COMPUTE Q OF OUTER LEVEE FOR 1.0' HEAD.
- USE PT-EE-2 PUMP TEST RESULTS - d = 52.5'
  - k = 0.114 FPS

$$\$ = \frac{52.5}{79.6 + 0.86\ (52.5)} = 0.421$$

Q = (0.421) (0.114) (1.0) = 0.048 CFS/FOOT OF LEVEE/FOOT OF HEAD

- COMPUTE HEAD ON SUBLEVEE FOR 1.0' OF HEAD FLOW UNDER OUTER LEVEE.

$$\text{SUBLEVEE } \$ = \frac{52.5}{28 + 0.86\ (52.5)} = 0.718$$

$$H = \frac{Q}{\$ k} = \frac{0.048}{(0.718)\ (0.114)} = 0.6'$$

- COMUTE Q UNDER LEVEE SYSTEM FOR MILE OF LEVEE FOR FOOT OF HEAD

Q = (0.048) (5280') = 253 CFS/MILE/FOOT OF HEAD

NEW-LEVEE SOUTH SECTION - SEE TYPICAL SECTION BELOW

- COMPUTE Q OF OUTER LEVEE FOR 1.0' HEAD
- USE PT-EE-3 PUMP TEST RESULTS - d = 44.0'
  - k = 0.241 FPS

$$\$ = \frac{44.0}{79.6 \pm 0.86\ (44.0)} = 0.375$$

Q = (0.375) (0.241) (1.0) = 0.090 CFS/FOOT OF LEVEE/FOOT OF HEAD

- COMPUTE HEAD ON SUBLEVEE FOR 1.0' OF HEAD UNDER OUTER LEVEE.

$$\text{SUBLEVEE } \$ = \frac{44.0}{28 + 0.86\ (44.0)} = 0.668$$

$$H = \frac{Q}{\$ k} = \frac{0.090}{(0.668)\ (0.241)} = 0.6'$$

- COMPUTE Q UNDER LEVEE SYSTEM FOR MILE OF LEVEE FOR FOOT OF HEAD.

Q = (0.090) (5280') = 475 CFS/MILE/FOOT OF HEAD

L-31(N) EXISTING LEVEE NORTH SECTION

- COMPUTE Q FOR 1.0' HEAD USING DEMENSION OF L = 130.0' FOR EXISTING LEVEE BASE WIDTH.
- USE PT-USGS PUMP TEST RESULTS - k = 0.81 FPS
  - d = 52.0' (FROM L-31(N) BORINGS)

$$\$ = \frac{52.0}{130 + 0.86\ (52.0)} = 0.298$$

Q = (0.298) (0.81) (1.0) = 0.241 CFS/FOOT OF LEVEE/FOOT OF HEAD

- COMPUTE Q UNDER LEVEE FOR MILE OF LEVEE.

Q = (0.241) (5280') = 1273 CFS/MILE/FOOT OF HEAD

L-31 (N) EXISTING LEVEE SOUTH SECTION

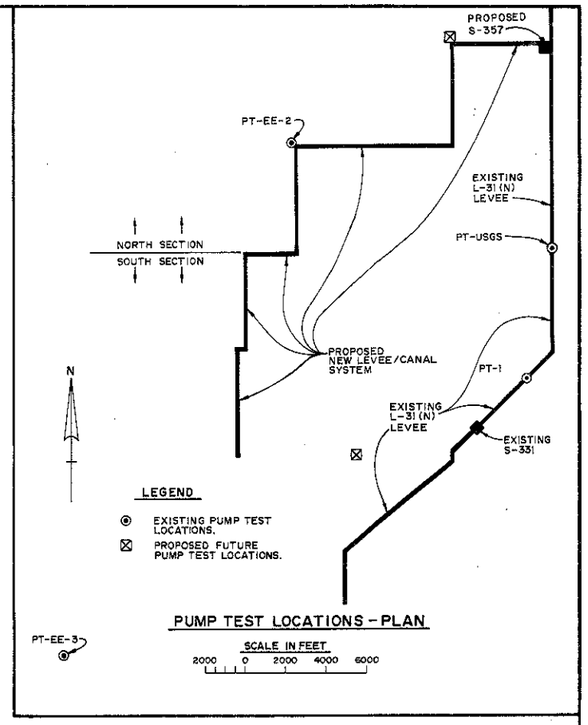
- COMPUTE Q FOR 1.0' HEAD USING DIMENSION OF L = 130.0' FOR EXISTING LEVEE BASE WIDTH.
- USE PT-1 PUMP TEST RESULTS - k = 0.67 FPS
  - d = 60.0' (FROM L-31 (N) BORINGS)

$$\$ = \frac{60.0}{130 + 0.86\ (60.0)} = 0.330$$

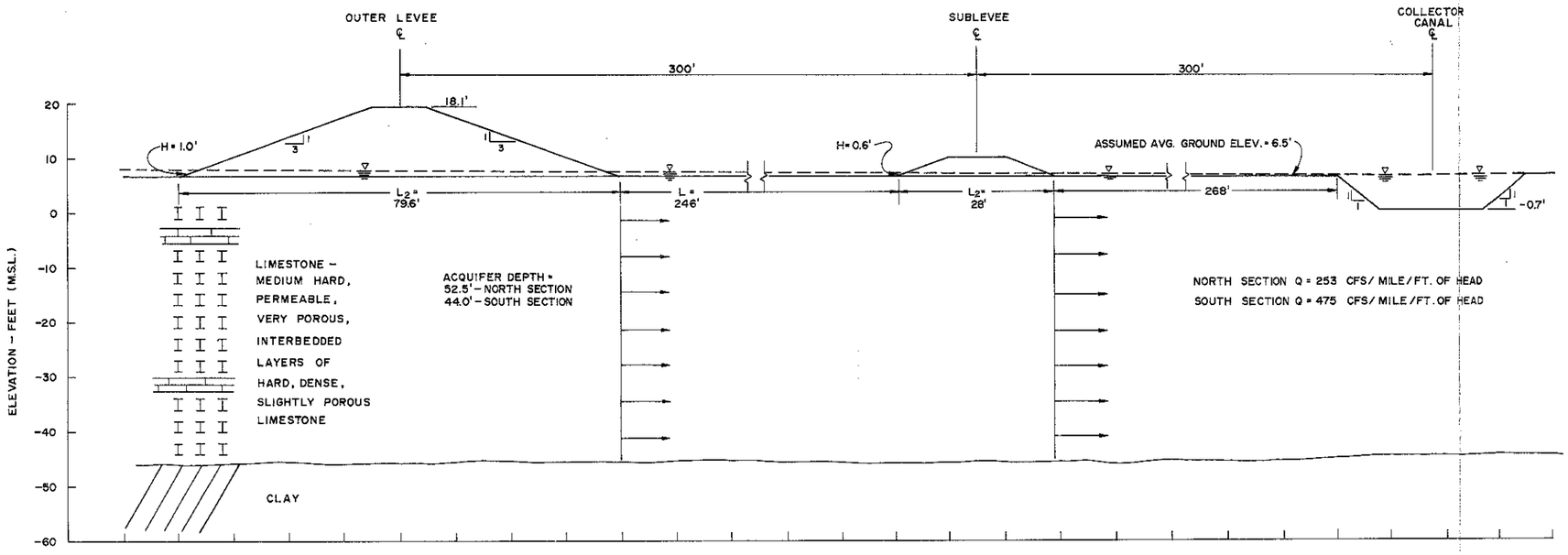
Q = (0.330) (0.67)(1.0) = 0.221 CFS/FOOT OF LEVEE/FOOT OF HEAD

- COMPUTE Q UNDER LEVEE FOR MILE OF LEVEE.

Q = (0.221) (5280') = 1169 CFS/MILE/FOOT OF HEAD



PUMP TEST LOCATIONS - PLAN



HORIZONTAL SCALE: 1" = 10' OR AS SHOWN

TYPICAL SECTION - NEW (RESIDENTIAL) LEVEE

CENTRAL AND SOUTHERN FLORIDA
MODIFIED WATER DELIVERIES TO
EVERGLADES NATIONAL PARK, FLORIDA

RESIDENTIAL AREA LEVEE

UNDERSEEPAGE

SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA
TO ACCOMPANY GENERAL DESIGN MEMO.,
PART I ,SUPP. 54 ,DATED: MARCH 1990
FILE NO. 400-35,662

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX H

ECONOMIC ANALYSIS

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX H

ECONOMIC ANALYSIS AND BENEFIT EVALUATION

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APPENDIX H

ECONOMIC ANALYSIS AND BENEFIT EVALUATION

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APPENDIX H

ECONOMIC ANALYSIS AND BENEFIT EVALUATION

SECTION I. ECONOMIC DEVELOPMENT

A. General. An economic analysis was performed to determine if the proposed modifications would induce flood damages and, thus, require construction of mitigation measures as directed by PL 101-229, Section 104. The analysis was performed in accordance with the procedures outlined in ER 1105-2-40. This portion of the appendix will describe the economic environment surrounding the study area and provide economic and demographic information useful in analyzing the water and land resources within the study boundary.

The specific objectives to be addressed in this segment include the following:

- a description of the study area along with a State and Regional overview.
- a discussion of the study area in a regional context.
- an overview of the County area.
- the economic base of the study area.
- the existing conditions and existing land use with land use maps.
- future conditions without and the with project.
- an analysis of benefits and methodology of preparation.
- a discussion of existing condition damages and with project damages.

There are some general assumptions made during the analysis. These assumptions are limited to the following:

(1) During the project life there will be no major economic recessions which will seriously affect the long-term growth patterns of the Nation's economy.

(2) There will be continued upward trends in population, employment, and production, accompanied by upward trends in total volume of consumption. International political tensions will remain at approximately the present level and there will be no widespread outbreak of hostilities.

B. Location and Description of the Study Area. The study area is located on the southern tip of the State of Florida in western Dade County. It contains approximately 191 square miles of area and is bordered on the north by the Tamiami Trail, the east by Levee 31N and on the west and south by the Everglades National Park. A map of the area is shown on Plate 1. It is located within the State Planning Area 11 which includes Broward, Dade and Monroe Counties and is associated with the Miami-Hialeah Standard Metropolitan Statistical Area (SMSA). This SMSA includes the whole of Dade County. The study area is largely undeveloped with only approximately 8% of the land area being employed in agricultural production and sparse residential development. The State of Florida owns about 54 square miles in the southern portion of the area. The remaining 72% of the area is in private ownership. Little economic statistical data is

readily available about the study area itself; therefore, the economic setting will be discussed in terms of the State, the planning area, and the county perspective.

C. State and Regional Overview. Population growth and economic activity in the study area are affected and influenced by external socio-economic characteristics and trends. This section outlines conditions in a Statewide and regional context.

1. Florida in the National Context.

a. Population. The State began showing tremendous population growth after World War II. The number of people has more than quadrupled between 1950 and 1985 as shown in Table H-1 primarily because of in-migration. During this 35-year period of growth, the State share of total United States' population increased steadily from 1.8 percent in 1950 to nearly 5 percent in 1985. Annual net in-migration exceeds 200,000 with about one-third of these persons of retirement age. Future projections for population show approximately 19 million persons by the year 2035 with in-migration accounting for more than 90 percent of future growth. Florida's share of the National population will then be over 6 percent.

TABLE H-1

POPULATION IN FLORIDA

Year	Florida Population (in thousand)	Percent Change	Percent Share of U.S. Population
1950	2,771.3	-	1.8
1960	4,951.6	78.7	2.7
1970	6,791.4	47.9	3.3
1980	9,747.0	43.5	4.3
1985	11,287.9	16.8	4.9

b. Economic Base. Florida's economy is largely dependent on the trade, services and government sectors to generate income within the State. Much of this activity is supported by the large number of tourists who visit the State each year and the large number of people from other areas in the United States who select Florida as a retirement location. This increase in resident population requires a large service-oriented labor force which expands the job opportunities for existing residents and new in-migrants. Construction activity is also supported by the demands of these consumers. In past years, these sectors of economic activity were affected by forces which influenced recreation and relocation attitudes. Tourism and the construction industry suffered heavily during the gas crisis of the mid-1970's. The larger resident population base which has been built up during the past thirty years has tended to dampen the effects of these crises. Agriculture is another important economic sector in the State. Florida is the national leader in citrus fruit growing and the manufacture of processed citrus products, accounting for over 70 percent of the nation's citrus production. Sugarcane, live stock, vegetables, and ornamental

horticulture also represent substantial portions of the State's agricultural output. Manufacturing is primarily resource-oriented, utilizing the State's agricultural produce and minerals in the production activity. There is an emerging high technology manufacturing sector. Much of this is supported by military spending. This economic activity in Florida is in contrast to the national economy which is more dependent upon manufacturing. A comparison of the economic structure of Florida and the United States is shown in Table H-2.

TABLE H-2

PERCENT DISTRIBUTION OF EMPLOYMENT AND EARNED INCOME
(1983)

	U.S.		Florida	
Industry Sector	Employment	Income	Employment	Income
Agriculture	4.4	1.8	3.8	2.7
Mining	0.9	1.7	0.2	0.3
Construction	4.8	5.3	7.0	7.4
Manufacturing	17.6	23.7	10.2	13.1
Transportation, Communications, & Public Utilities	5.0	7.8	5.4	8.7
Wholesale & Retail Trade	21.4	16.2	24.5	19.4
Finance, Insurance, & Real Estate	5.8	6.4	7.2	7.3
Services	22.6	20.1	25.3	23.9
Government	17.5	17.0	16.4	17.1

D. The Study Area in a Regional Context.

1. Planning Regions. As the Florida economy has expanded, individual areas have tended to become diverse and take on their own distinct economic characteristics. In order to better describe each area, Florida has been divided into eleven planning areas. The study area is within planning area number eleven which includes Broward, Dade, and Monroe Counties. In terms of numbers, this area is the most populated in the State. In 1985, it had an estimated population of 2.9 million people and represented 26.2 percent of the State's population. Although in the same planning area, each of these counties maintains its own individual economic temperament. Dade County is the largest metropolitan area in Florida. This county has become less dependent upon the regional market as a support for its economy than other neighboring counties. With a mature and urban business base, it has become a financial and banking center for international business, especially Central and South America. Growth in the local economy is more centered upon the formation of numerous small businesses. Dade County has more corporations than any other county in the State. Miami, with its large Hispanic population, has injected a Latin ambience into the area and many of the new businesses have been formed to serve its ethnic population.A large segment of the southwest part of the county is devoted to production of winter vegetables, fruits and other agricultural products.

To the south of Dade County, Monroe County's economy has changed very little in the past three decades. The major foundation of its economy is in tourism and fishing. Consisting of a string of islands, its growth is extremely sensitive to limitations of land area and the necessity to maintain a balance between development and the environment.

In contrast, the economy of Broward County, the northernmost county of this triad, is more nearly represented by white collar employment in larger corporate structures. The county has become one of the high technology centers of Florida requiring a highly trained and highly skilled workforce. These activities support a well paid population which generates a large demand for the local services sector of the economy. Broward County has taken the dominant role in the health of this region. Many business activities which in former years were drawn to the Miami area are now locating in Broward County.

2. The Dade County Area. This section discusses in general the current economic activity in Dade County. The area under study is a small portion of Dade County. Most of the published information available relates to the county and will provide the major basis of the description of the economics and social activity associated with the proposals being examined in this study. This section will be divided into several parts including a general examination of the geography, demographics, employment and income characteristics of the county. The displacement of agricultural lands by urban lands, one of the major concerns in the area, will be reviewed.

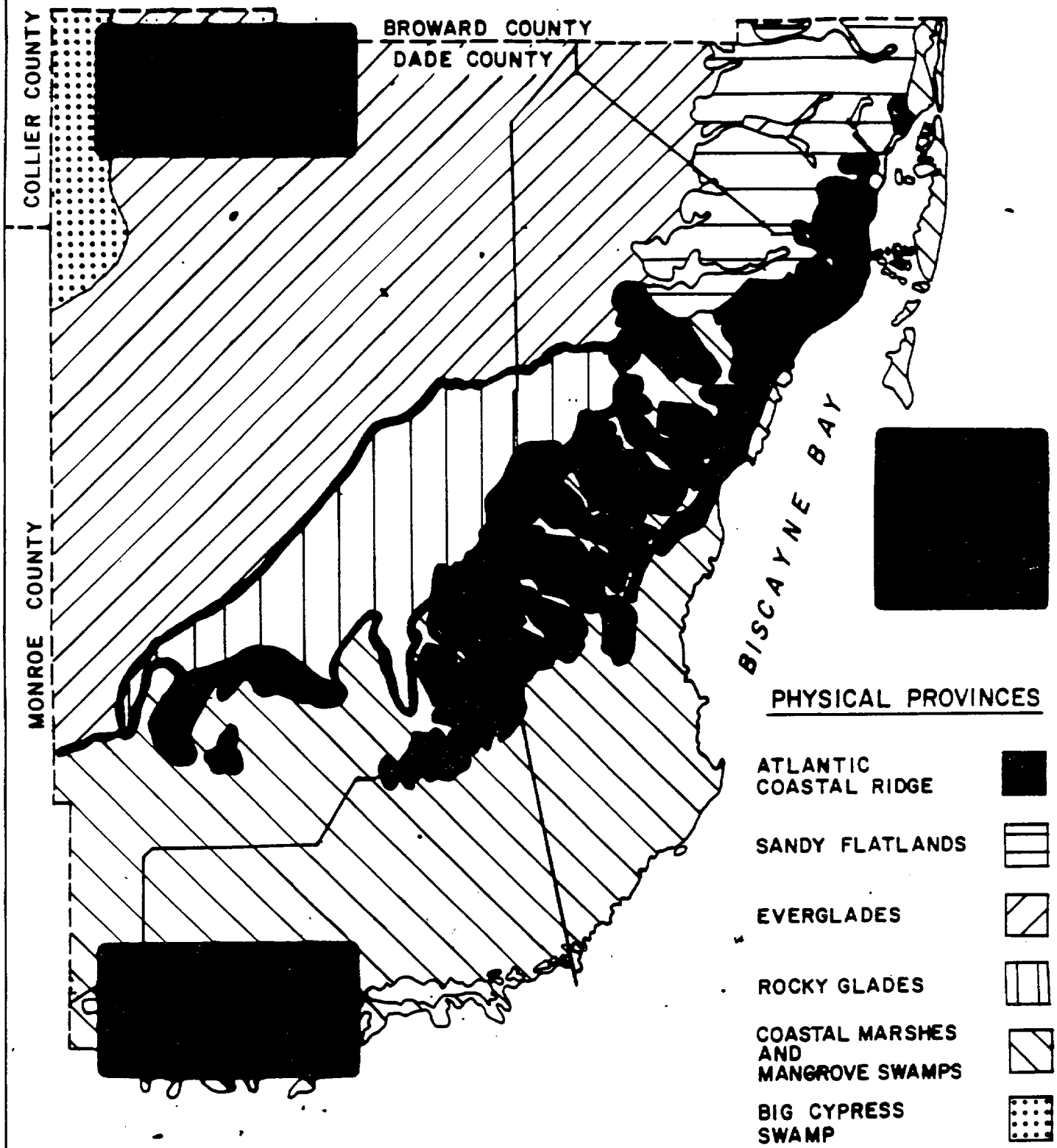
a. Geography. Dade County covers about 2,000 square miles of land surface and approximately 355 square miles of ocean and bay waters. It is the third largest county in land area in the State[1]. The land use within this area may be divided into six broad categories[2-3]. These six categories are:

Land Use	% County Area
1. Conservation Area 3A	15
2. Everglades National Park	28
3. Ocean and Bay Areas	15
4. Existing Developed Areas	13
5. Developable Areas	13
6. Not Readily Developable Areas	16

The physical features of the land in terms of elevation, drainage, soil cover and vegetation are separated into groups with individual characteristics and occupy six different land areas. These areas are[3]:

(1) The Atlantic Coastal Ridge
(2) The Sandy Flatlands
(3) The Everglades
(4) The Rocky Glades
(5) The Coastal Marshes and Mangrove Swamps and
(6) The Big Cypress Swamp.

The general outline of these areas can be seen on Figure 1.



SOURCE: METROPOLITAN MIAMI AREA STUDY
HUNTER MOSS AND COMPANY

SCALE IN MILES
0 5 10 15

MAJOR PHYSICAL CLASSIFICATIONS OF LAND AREA IN DADE COUNTY

FIGURE 1

The Atlantic Coastal Ridge has an average height of 10 to 15 feet above sea level with a maximum elevation of 25 feet mean sea level. It is composed of Miami oolite. The soils range from shallow to moderately deep sands with good to excessive drainage. The Sandy Flatlands have an average elevation from 5 to 10 feet above mean sea level. The soils are deep fine sands of the Pamlico Formation. There is little surface drainage and precipitation drains into the porous limestone or evaporates. The Everglades is a sea of tall grass with small islands rising six to 18 inches above the surrounding ground. The Rocky Glades are a transition area between the Everglades and the Atlantic Coastal Ridge. The land is higher than the Everglades and the surface is covered with outcroppings of Miami oolite limestone almost devoid of soil. The Coastal Marshes sand Mangrove Swamps lie along the coastal areas and are usually associated with brackish water. They contain marl soils which are moderately deep and poorly or very poorly drained. The Big Cypress Swamp is a poorly drained area slightly higher than the Everglades. The soils are poorly drained marls underlain with Tamiami Formation and Miami oolite.

The limerock underlying these areas is a major mineral resource in the county. It is used as base materials in road construction, as fill material to raise the elevation of land areas to be used as building sites and in the manufacture of cement. The quarries are easily mined and when they have occurred in expanding urban areas the land surrounding them has demanded premium waterfront sales values. These quarry lakes range in size from 10 to 375 acres and are about 40 feet in depth. About 2400 acres are presently being mined and permits have been issued for the mining of an additional nine to ten thousand acres.5

It is evident from this brief description of land usage and topography that land suitable and available for development in the county is quite limited relative to the total county area.

One of the major reasons people choose Dade County as a place to visit and live is its outstanding weather. Overall, the climate is subtropical with warm days, rain showers and gentle breezes. There is a wet and dry season with approximately 75% of the rainfall occurring between May and October.
The average yearly rainfall is 60 inches. Sun-shiny days occur throughout the year and during the summer localized rain storms lasting an hour or two occur during the days and nights. Winter rainy weather can result in a few dreary days with light misty rainfall and overcast skies. There is about a 20-degree average temperature range during the year with the temperature averaging in the mid-60's during the winter and the mid-80's during the summer[6]. Although there are occasional droughts and tropical storms, the climate is ideally suited to the production of tropical fruits and other agricultural activities.

b. Population. Dade County is the most populous county within the State with an estimated 1.8 million residents in 1985. It ranked third for population density with 892 persons per square mile. From 1960 to 1985, the population increased at a 2.6 percent average annual rate almost doubling the number of persons in the county in this 25-year period. This growth was due primarily to the net migration of persons attracted by the employment opportunities, the appeal of the area as a retirement location and the influx of refugees from Cuba and Haiti during 1980-81. During this period, the county would have to be considered one of the fastest growing areas in Florida. By the

year 2035 the county is estimated to have a population of about 2,250,000 or an increase of 25 percent. This would represent a significant decrease in growth rate in future years. A comparison of the historical and estimated future population of the county and the State of Florida is shown in Table H-3.

The racial and ethnic composition of the population of the county has undergone a dramatic change in the last four decades. It has moved from a society composed of mostly non-Hispanic Whites with minor Hispanic and Black population to a society of about equal non-Hispanic Whites and Hispanics with a minor Black population. This shift in composition is shown in Table H-4.

TABLE H-3

Population Comparison

Year	Number of Persons: Florida	Number of Persons: Dade County
1960	4,951,600	935,100
1970	6,791,418	1,267,792
1980	9,747,197	1,625,724
1985	11,287,932	1,759,251
1990	12,527,778	1,765,401
2000	14,627,733	1,876,698
2015	16,862,538	2,067,347
2035	18,996,043	2,249,365

TABLE H-4

Dade County
Population by Race and Ethnic Group
(1,000)

Year	Total	Hispanic	Black	Non-Hispanic Whites
1950	495	20	65	410
1960	935	50	137	748
1970	1,268	299	190	779
1980	1,610	617	233	760
1985	1,736	750	250	736
		Percentages		
1950	100	4	13	83
1960	100	5	15	80
1970	100	23	15	62
1980	100	38	15	47
1985	100	43	15	42

The housing boom in the late 1960's and early 1970's added more than 20,000 units annually to the Dade County areas. An estimated two-fifths of the county's housing units have been built in the last 15 years. There were 665,382 housing units in 1980. Of that total, 662,000 were classified as year-round units of which 332,527 were occupied by owners and 277,303 were occupied by renters. The median house value in dollars was $54,700 which over a 10-year span had tripled in value. The median house value in dollars in 1970 was $19,000. The median contract rent was $122 in 1970 and $213 in 1980.

During the 1970-1980 period, 182,000 new households were formed and by 1982 reached a total of 632,700. New households include those formed by young single persons moving into the area, divorced individuals, and others setting up separate households. The growth of single and two-person households contributed to the decline from 3.0 persons in 1970 to 2.6 residents per housing unit by 1982.

c. Employment and Income. Dade County is dependent upon the services, trade and government sectors of the economy to generate the major portions of the jobs necessary to provide employment for its citizens.
As shown in Table H-5, these three sectors provide for 66 percent of the employment in 1983[7]. They are expected to maintain this relative share into future years. Much of this type employment is associated with providing for the demands of a mature urban economy heavily influenced by a large retirement community. In the manufacturing sector, which contributed 11 percent of the employment, the largest employers are in apparel, printing and publishing, fabricated metal products, and rubber and plastics products groups. Food products and non-electrical machinery firms are also large employers. High technology companies that produce computer hardware, electronic components, communications equipment and instruments account for about 10 percent of manufacturing employment. The transportation, communications and public utilities sector demands higher skills and offer a higher wage. Although a small contributor to earnings and employment in the county, agriculture plays an important role due to its demands for scarce land resources. With the dynamic population growth in the county, urban pressure for land area for development is continuously encroaching on the remaining suitable agricultural lands in the county. The results of this pressure and the search for additional lands suitable for cultivation are an integral part of the problems being examined in the study area. Because of this a discussion of agriculture in the county is appropriate. Table H-6 shows the percent earnings by industry.

TABLE H-5

PERCENT EARNINGS BY INDUSTRY

	1983	2000	2035
Agricultural	0.8	0.7	0.7
Mining	0.3	0.3	0.3
Construction	5.7	5.2	4.7
Manufacturing	10.6	10.6	10.4
Transportation, Communications, & Public Utilities	13.9	14.2	14.7
Wholesale and Retail Trade	20.5	19.9	19.6
Finance, Insurance, & Real Estate	8.3	9.4	9.6
Services	26.4	27.4	27.7
Government	13.5	12.3	12.6

TABLE H-6

PERCENT EMPLOYMENT BY INDUSTRY

	1983	2000	2035
Agricultural	1.4	1.7	1.8
Mining	0.2	0.2	0.1
Construction	5.2	4.6	4.2
Manufacturing	11.0	10.2	9.5
Transportation, Communications, & Public Utilities	8.6	8.4	8.6
Wholesale and Retail Trade	25.1	25.9	26.4
Finance, Insurance, & Real Estate	8.0	8.6	8.7
Services	28.0	29.1	29.4
Government	12.5	11.3	11.3

d. Agriculture. The Dade County agricultural industry is rather unique in the State both in the types of commodities produced and the method of cultivation. Almost all of the crops grown are high value crops consisting of fresh vegetables, ornamentals and fresh fruits. The per-acre value of the crop production is among the highest in the nation and the acreage under cultivation is intensively farmed. Over 50 percent of the agricultural acreage is allocated to vegetable crops, and is grown to meet the demands of the winter market for fresh vegetables in the eastern United States. The most active growing season is between September and May. Because of the wet and dry rainy seasons in the area, the farmers planting times are controlled by the elevation of the ground water and generally follow the water table in planting. Higher ground is put under cultivation first and as the rainfall decreases and the water table is lowered progressively lower elevation lands are brought into cultivation.

The three major land areas used for agriculture are located in the southeast portion of the county and are known as the Redlands, the East Everglades and the East Glade. Their location is shown on the generalized land use map Figure 2. The area is rural in nature with sparse residential development scattered throughout. The type of vegetable and fruit commodities grown in the area are shown in Table H-7. In addition to these, there are numerous ornamental nurseries, several livestock and grazing operations and cultivation of lands in field crops such as seed corn and sorghum. The total agricultural acreage under cultivation in 1980 is shown in Table H-8.

The soils in these agricultural areas are rocky soils and marl soils. The finer texture of the marl soils make them more suitable for tuber crops such as potatoes and ornamental plants grown in the ground and requiring a root ball when harvested. The rocky soils, including rockdale and rockland, require a preparation process which gives this type farming a unique character. It is necessary to break the hard limestone outcroppings into smaller particles by scarifying or rock plowing before cultivation can take place. When the material is sufficiently pulverized, the fields are prepared in row mounds to gain added protection from high water tables. Extensive fertilizer is used in both marls and rockland soil farming. Because of the high cost of preparation, the equipment required and labor necessary for this type farming, individually operated farms are usually relatively large to ensure the efficiency of the equipment and labor. For vegetable production they will be about 200 acres. Fruit tree cultivation does not require the same intensity of yearly maintenance. Groves can be efficiently cultivated on a smaller area such as 20 acres[8].

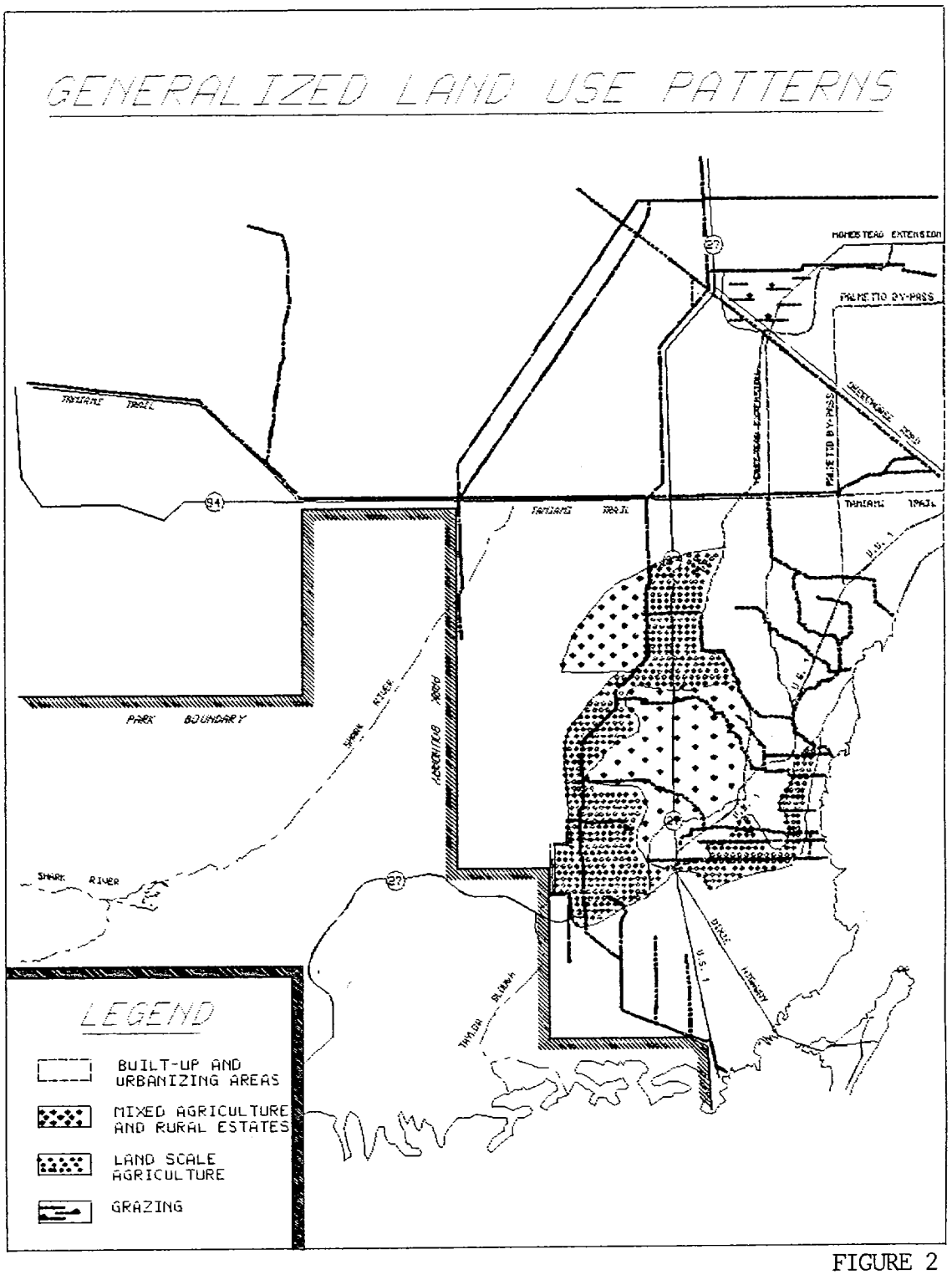

GENERALIZED LAND USE PATTERNS
HOMESTEAD EXTENSION
PALMETTO BY-PASS
TAMIAMI TRAIL
PARK BOUNDARY
SHARK RIVER
ALLIGATOR ROAD
U.S. 1
TAYLOR SLOUGH
LEGEND
BUILT-UP AND URBANIZING AREAS
MIXED AGRICULTURE AND RURAL ESTATES
LAND SCALE AGRICULTURE
GRAZING

FIGURE 2

TABLE H-7

DADE COUNTY AGRICULTURAL PRODUCTION
1979 - 1980 SEASON

CROP	ACREAGE (Acres)
Vegetables	
Tomatoes	11,400
Potatoes	6,750
Boniatos	5,500
Bush Beans	5,000
Malanga	4,100
Sweet Corn	3,900
Squash	3,600
Pole Beans	2,750
Okra	2,300
Cucumbers	1,200
Calabaza	1,100
Cabbage	200
Yuca	200
Eggplant	120
Strawberries	100
Peppers	100
Southern Peas	100
Vegetable Total	48,420
Fruit	
Avocados	9,338
Limes	5,641
Mangos	1,449
Papayas	600
Plantains	300
	17,328

Source: Agricultural Land Use Plan for Metropolitan Dade County, Florida; March 1983.

TABLE H-8

1980 AGRICULTURAL LAND USE IN DADE COUNTY

	Acreage	
Row crops: vegetables, field crops	45,132	
Fruit Groves	19,311	
Plant Nurseries	2,798	
Total Active Cropland		67,241
Fallow	14,201	
Total Cropland		81,442
Livestock and Grazing	8,878	
Misc. Agricultural Use	588	
Total Agricultural Use		90,908

Source: Agricultural Land Use Plan for Metropolitan Dade County, Florida; March 1983.

With the increase in population in the county, large demands have been created for the lands necessary to support the residential communities associated with an urban society. This type development places a very high unit value on lands and other types of uses find it economically impossible to compete in the market for the use of the land. This is especially evident in the displacement of agricultural lands by urban lands in the county. The remaining supply of lands suitable for agriculture production is becoming scarce and in order to take advantage of the areas climatic advantages producers have been moving into lands of lower elevations. Because of the great value of the produce a greater risk of crop loss can be assumed.
The utilization of the lands in the study area is an example of this event. There is a limit to agricultural displacement and the county is studying the problems and searching for solutions providing for a compromise in future development which will include agriculture as a part of the economy.

e. The Study Area. The location of the study area was described in paragraph B. It contains approximately 183 square miles of area and is known as the East Everglades area. Virtually all of the acreage in commercial agricultural productive use exists along a 2 - 4-mile wide stretch of land along the west side of L-31. The primary crops under cultivation are winter vegetables, tropical vegetables, tropical fruits, citrus and ornamental plants. Some lands are used for raising horses, cows, chickens, and other animals on a small scale usually associated with family usage. The method of agricultural production is similar to that described for rockland cultivation.9 Areas farmed are smaller in area than those farmed east of L-31. A listing of the crops grown in the area in 1985 and their acreage is shown in Table H-9.

Mostly, the residential development in the area is concentrated in the Howard and Richmond Drive area. About 400 to 450 persons make their homes in this vicinity. The settlement is rural in character with houses often associated with small agriculture activity. The average household is about 3.5 to 4.0 persons. The

residential area has only four paved roads. The basic infrastructure serving the area includes electric and telephone service; however, water supply is provided by individual private wells and waste disposal systems are individual septic systems.

There is a State maintained recreation park, Chekika State Recreation Area, located in the central part of the area. Nature type activities such as camping, swimming, and natural habitat appreciation are available. Hunting and fishing in the area are generally associated with off-the-road vehicles such as airboats and glade buggies.

As stated previously, the commercialization of the East Everglades Area is due to the inability of agriculture to compete with urban development for land area. The farm ventures in this area are much more subject to the risks of high water tables and high probability of loss from root zone inundation by water.

TABLE H-9

EXISTING LAND USE

ACREAGE BY TYPE OF CROPS

Grids Squares	Citrus Crops	Tree Crops	Veg. Crops	Urban Land	Undev. Land	Ornamental	Totals
1	63.27	0	205.38	0	0	0	268.65
2	38.27	0	36.63	0	23.46	0	98.36
6	19.45	0	87.03	0	34.56	0	141.04
7	35.90	0	249.20	0	336.81	0	621.91
10	0	23.31	15.91	0	493.28	0	532.50
13	0	5.35	0	0	125.97	0	131.32
14	0	0	304.74	0	308.49	0	613.23
16	206.30	10.30	161.65	0	97.01	0	475.26
18	105.22	34.13	267.53	0	85.99	0	492.87
19	0	0	30.54	0	318.93	0	349.47
20	63.56	0	358.46	0	180.60	0	602.62
21	40.19	7.15	222.98	0	92.87	0	363.19
22	15.27	10.74	33.19	14.53	261.34	7.79	342.86
24	1.32	1.70	9.94	10.16	123.22	0	146.34
25	139.57	22.52	110.03	43.85	310.04	0	626.01
26	4.90	4.88	0	12.86	226.14	4.35	253.13
29	17.72	42.93	65.44	33.02	311.28	5.38	475.77
30	4.60	0	9.16	0	7.73	0	21.49
31	7.13	22.80	181.62	16.26	52.12	22.68	302.61
32	27.47	46.94	337.79	56.39	101.36	37.44	607.39
33	40.46	40.40	276.04	15.33	65.22	2.24	439.69
34	0	0	6.15	0	6.37	0	12.52
35	19.95	38.01	140.67	12.16	213.10	0	423.89
36	17.92	29.07	42.18	54.55	182.24	0	325.96
37	62.81	40.55	174.67	119.18	223.72	20.18	641.11
38	8.65	4.55	142.28	210.66	31.79	27.56	425.49
39	0	9.79	4.90	0	37.65	3.10	55.44
40	3.38	6.72	46.69	5.14	81.68	3.14	146.75
TOTALS	943.31	401.84	3,520.80	604.09	4,332.97	133.86	9,936.87

SECTION II. PROJECT EVALUATION

A. General. The main objective of this evaluation was to describe what effects the proposed modifications to the water deliveries to the Everglades National Park would have on the agricultural and urban development in the study area. The areas of concern were the selection of criteria and methodology for describing the impact of changes in water elevation on agricultural cultivation and urban developments, the data requirements of this methodology, the data control system, the data display systems, and the actual analysis. A description of each item is contained in the following paragraphs.

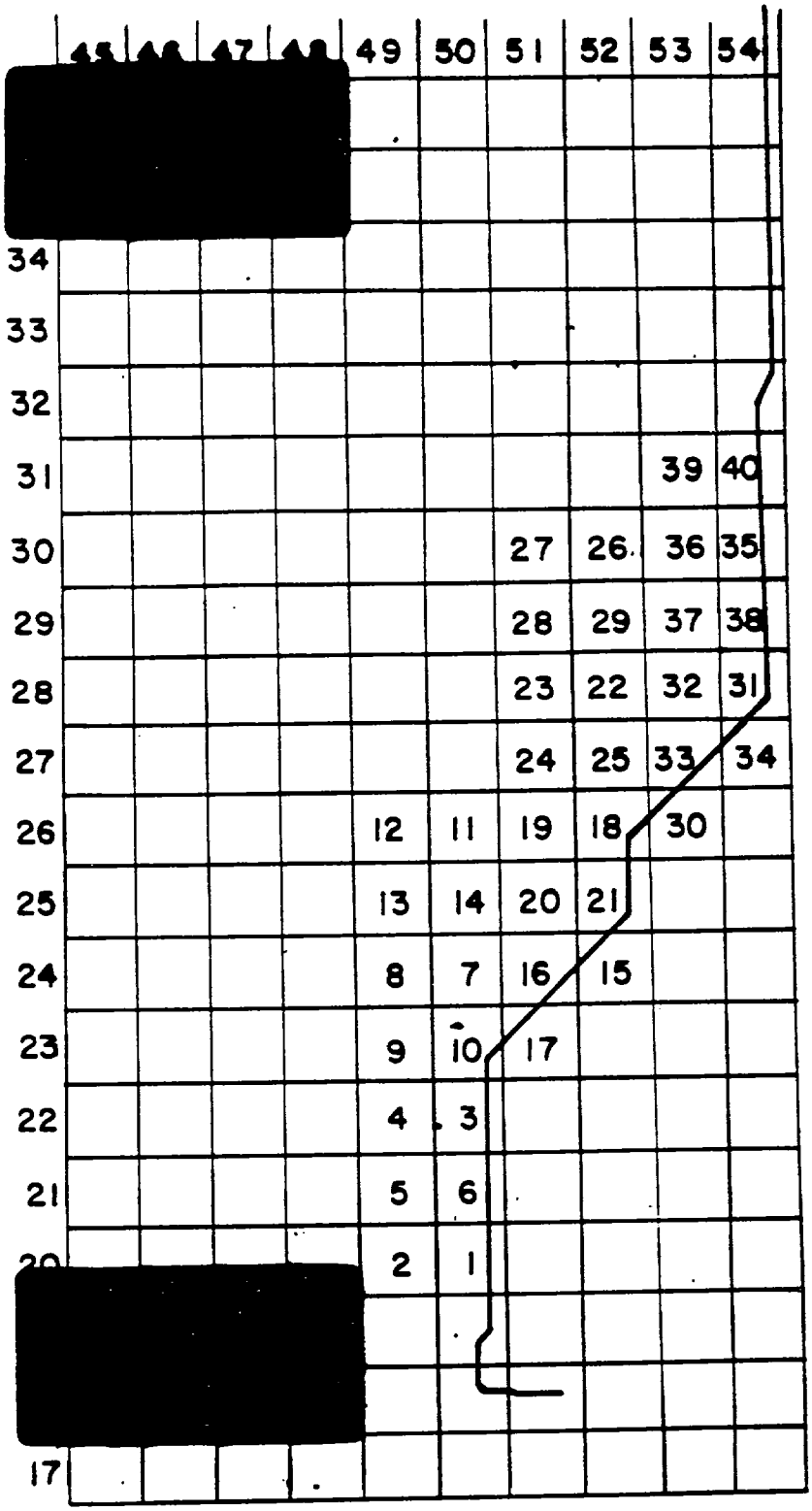
B. Criteria and Methodology. There are two basic land uses in the study area which can be readily described in economic terms. They are residential and agricultural usage. For this study they have been placed in separate categories and each will be evaluated as independent items.

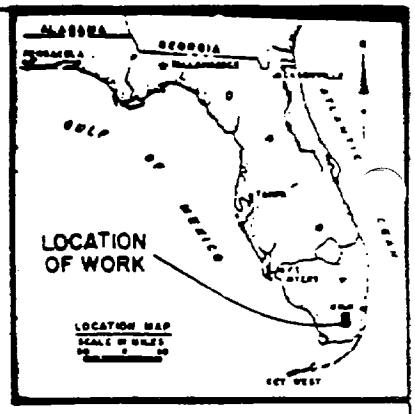
In urban development, the factors most prominently affected by the proposed changes in the water deliveries would be those impacted by the amount of water inside the structure and the duration of water surrounding the structure. To determine this it was necessary to develop a house inventory by grid cell location site, establish the ground elevation at the structure site, the elevation of the structure first floor, a value of the structure, and an estimate of the landscaping associated with the structure. A description of the access, i.e., roadway, to each structure was also required.

For agriculture, the plant physiologic factors most prominently affected by the proposed changes in the water deliveries would be those impacted by the plants root system exposure to a saturated groundwater table in the root zone. To determine this, it was necessary to develop a specifically located land use inventory, establish the ground elevations at the cultivation site, determine the root depths of the products grown at each site and their tolerance to water, measure the effects of the changes in water levels caused by the various project proposals, and translate these determinations to an economic description of the impact.

1. Data Control System. A mathematical model was employed in the hydrologic examination. This model used a 1-mile by 1-mile grid control system. To make the data collected in the effects evaluation interface with the hydrologic data, this same grid system was used. The system was based on the state plain coordinate control system. The matrix is not coincident with the Township, Range, and Section matrix on the U.S.G.S. quad sheets. A schematic of a portion of the system is shown on figure 3. Each square has been given a numeric identification and all data associated with that square has been identified with that number.

2. Land Use. Development in the study area has taken place in the eastern edge in a 2 to 4-mile-wide stretch along the west side of L-31. Only the lands in this area are to be examined in the project effects evaluation in this section of the report. The area of each individual cultivated plot and the commodity produced on it was determined and recorded.





GRID SQUARE LOCATION MAP

FIGURE 3

Data were tabulated according to grid cell location. The commodities grown in the area are similar to those listed in Table H-7. A broad category listing of the acreage in each grid cell is shown in Table H-9. Urban development in the area was based on data collected by the Jacksonville District Real Estate Division. The information was obtained from the Dade County Tax Assessors office and included structure location, size, and assessment evaluation. These data were tabulated according to grid cell location. A broad category listing is shown in Table H-10.

The land use in 1987 was used as the data base of the resources for the calculation of the project effects evaluation. Future additional use in the area will be determined by political institutions reacting to pressure created by interest groups whose concerns include among other things agricultural production, the environment, home ownership, and the Everglades National Park. Zoning regulations and building permit controls presently in effect severely restrict any changes in the land usage in the area. For this study, it is assumed the existing and future land use will be the same under the without project conditions.

3. Evaluation Methodology. Two different criteria are used when evaluating project effects on agriculture and on urban land usages. In the agricultural evaluation, the groundwater depth and duration are the most significant variables. For urban evaluations, the water depth above ground and its duration are the most important. Each will be discussed separately.

a. Agricultural Evaluation Methodology. The impact of change in groundwater environment on agricultural production is directly related to the groundwater level in the root zone of the plant, the time the water is in this zone, the tolerance of the plant to this water, and the loss to production caused by the root inundation. Root depths, inundation periods to plant mortality or fruit losses, production costs, harvest periods, and growing periods were obtained from the Institute of Food and Agricultural Sciences in Homestead, Florida. The data for vegetable crops is shown in Table H-11 and for tree crops in Table H-12. By interfacing the root depth information contained in these tables with the crop location ground elevation, mean sea level, the bottom depth of the root zone was determined.

The loss per acre column in Tables H-11 and H-12 shows the total damageable value per acre for each crop. This value includes both the direct production crop loss and the income loss associated with each crop, derived from the production cost information obtained from the University of Florida. An example of the production cost information for tomatoes for 1987-88 is shown in Table H-13. The operating costs shown in the table correspond to direct production crop losses and the fixed costs correspond to income losses. The sum of operating and fixed costs, shown in the table as Total Preharvest Cost is the total damageable value. Those values were averaged for a four year period, from 1986 to 1989, resulting in the $3,659 loss per acre for tomatoes as shown in Table H-11. The same method was used to calculate the losses per acre for other crops displayed in Table H-11 and H-12.

From these tables, it was determined that the critical durations of root zone inundation were 1-day, 5-days, 10-days and 15-days. For this evaluation, it

was assumed that (i.e., all fields are under cultivation but no crops have yet been harvested when flooding occurs) any ground water in the root zone of sufficient duration to cause damage would result in the losses per acre shown in the tables. For tree crops, losses for both fruit and growing stock were claimed in cases where the duration of groundwater inundation of the root zone was sufficient to cause plant mortality.

The depth of the groundwater table, mean sea level, and the time the water is at that depth was furnished by the Engineering Division of the Jacksonville District for each grid cell and for each condition being examined. These values were supplied for the 2, 5, 10, 20, 50, 100 and SPF frequencies. They also furnished a one-half foot interval contour map of the area. A spread sheet was prepared for each grid cell which tabulated the crop type, the area in acres, the bottom of the root zone depth mean sea level, the groundwater depth mean sea level, and the total damageable value for the crop type. This spread sheet made possible a comparison of the effects of the various water management schemes being proposed on the agricultural development in the area.

The methodology described above yields an estimate of the maximum agricultural net income loss associated with each alternative. Actual agricultural income losses would probably be less than the maximum, as it is unlikely in reality that all floods (root zone inundation) would occur when all fields were under cultivation and had not yet been harvested. The average annual damage estimates for crop losses should be viewed as an upper limit on potential losses for each alternative. This represents the worst case condition for agricultural damage. As such, those estimates are useful for demonstrating that the basic rain fall driven without mitigation plan has the potential to result in increased damages and for showing the potential reduction in damages for plans incorporating mitigation.

b. Urban Evaluation Methodology. The measure of the impact of changes in groundwater environment in the urban area is directly related to depth of water level inside the structure and the duration the water remains in the area. The types of damage associated with this category of flooding includes damage to structures, damage to the contents within structures, damage to residential landscaping such as lawns, shrubs, and trees; damage to access roads, damage to well fields and septic fields, and emergency costs incurred when dwelling units are uninhabitable due to inaccessibility from high water or structural damage.

An inventory of the structures in the study area was obtained from the Jacksonville District Real Estate Division. This listing included the structure size, structure value, lot value and their location. Of the 600 to 700 structures recorded on the tax rolls, 218 are considered to be residences. Those buildings with valuations of less than $15,000 are considered to be outbuildings and barns. Each residential structure has been located to a grid square address. A listing of the number of residential structures and the high and low values of the structure in each grid square is shown in Table H-10. Grid square ground elevations were available from the 1/2 foot contour map of the study area. The Jacksonville District Engineering Division developed depths and durations of water in the grid areas for seven frequency storm events. Tables H-14, 15, 16, 17, 18, and 19 display the average ground

elevation, flood elevation and duration of flooding in each grid square for each flood frequency examined under the six conditions analyzed.

Residential building in the study area has recognized the high water hazard and potential for structural flooding in the area. Building sites have been elevated above the existing ground level. Zoning regulations also require septic tanks and distribution fields to be constructed on raised sites. The precautions against flooding have prevented damages caused by flooding to structures in the area and their contents and landscaping; however, wells and septic tank facilities do not function during high water periods. Access roadways are inundated by water and residences become isolated islands in the surrounding flood waters. A summary comparison of the relative flood durations for the 5-, 20-, and 50-year frequencies for the six conditions analyzed is shown in Table H-20.

The main concern associated with the flooding in the urban areas of the study area is the inaccessibility of the homes because of access roadway flooding, inoperative wells, inoperative septic systems and the undesirable characteristics associated with having a residence surrounded by flood waters for an extended period of time. Because of this problem, residents of the study area are force to relocate temporarily during periods when flood waters render their homes inaccessible. The cost of temporary relocation has been selected as a measure of the worth of the inaccessibility to homes and the undesirability of having a home surrounded by floodwater. The most readily available alternate shelter would be motel accommodations. They are valued at $75.00 per day per 3 persons. Each household in the area has an average of three to four persons per unit. Interfacing this data with the values in Tables H-14, 15, 16, 17, 18, and 19, an average temporary relocation cost may be computed for the seven frequencies and six conditions examined. There are 7.38 miles of paved roadway and 34.39 miles of unpaved roadway in the study area. The most damage from high water in the area is sustained by the unpaved roadways. The cost of maintenance on this type of roadway is estimated to be $1,000 per mile. The length of roadway in each grid square is shown in Table H-21.

C. Evaluation of Effects. The effects evaluation of agriculture and urban development will be discussed separately.

(1) Agriculture. The effects on agricultural production of changes in the water delivery schedules to the Everglades National Park are founded on the premises that high water frequency occurs during the period of the year when agricultural cultivation is taking place, that the commodity planted could not survive with water in the root zone and the measure of the effects is total damageable value (maximum agricultural net income loss) associated with each crop for each flood condition. Tables H-22 through H-36 show those values for the commodities in each grid square subject to groundwater root inundation for the various water delivery schemes. Tables H-34 and 35 are a summary tabulation of the effects of the various plans.

(2) Urban. The effects on urban development of changes in the water delivery schedules to the Everglades National Park are founded on the premise that structures are constructed above the flood elevations and the

costs of high water in the area are due to the inability to have access to dwelling units during flooding periods, requiring residents to incur costs for temporary location. Table H-37 is a summary tabulation of the temporary relocation costs associated with the various plans. Table H-38 is a summary tabulation of the effects of the various plans associated with road maintenance in the area.

D. Conclusion. In addition to the without project condition, five variations on the basic rain driven plan were evaluated. All but one of these alternatives included measures to mitigate for increased flood durations (and associated damages) which would otherwise result from the change in water delivery plans.

Evaluations focused on the agricultural and urban flood damages associated with each plan. Damages were calculated for seven different flood events for each of 28 one-square-mile grid cells for each alternative plan. The results of those calculations were compared against the without project condition to determine whether any of the alternatives would cause an increase in potential damages. Examination of the calculated damages for each flood event and grid cell revealed that only one alternative, the without mitigation plan, would result in increased damage in any part of the study area. All of the other alternatives would result in damages the same as or less than the without project damage at every grid cell location. The increased damages associated with the without mitigation plan are fully avoided with any of the other plans considered.

TABLE H-10

LOCATION AND RANGE OF SIZE AND VALUE OF RESIDENTIAL STRUCTURES

Grid Square	Number of Houses	INSIDE AREA OF INDIVIDUAL HOUSE		VALUE OF INDIVIDUAL HOUSE	
		High (Sq. Ft.)	Low (Sq. Ft.)	High ($)	Low ($)
22	36	3,773	600	212,616	20,872
23	10	4,037	817	194,265	36,610
24	12	2,822	460	151,039	46,266
25	20	3,392	512	180,424	27,177
26	5	1,540	954	104,812	48,797
27	5	2,418	1,558	125,182	94,779
29	6	6,802	1,536	518,306	35,072
31	9	19,680	360	1,357,176	31,228
32	31	13,860	240	214,307	28,442
33	20	3,880	450	871,522	39,576
35	1	1,734	-	92,319	-
36	11	5,213	1,196	512,044	36,249
37	37	8,625	401	273,175	38,859
38	15	2,498	309	194,360	42,797

TABLE H-11

Vegetable Crop Loss Per Acre

Crops	Inundation Period To Plant Mortality	Root Depth	*Loss Per Acre	Harvest Period	Growing Period
Tomatoes	12 hours	6-8"	$3,659	Dec-Apr	60 days
Potatoes	24 hours	6-8"	1,656	Dec-May	60-90 days
Squash	12 hours	6-8"	1,067	Dec-Mar	45 days
Beans,Snap	12 hours	6-8"	788	Nov-Feb Apr-May	45 days
Beans,Pole	12 hours	6-8"	1,550	Mar-Apr	60-70 days
Cabbage	12 hours	6-8"	940	Dec-Feb	70-90 days
Corn	12 hours	6-8"	1,160	Dec-May	65-90 days
LeafCrops**	12 hours	-	1,510	-	-
Peppers	12 hours	6-8"	3,180	Mar-Apr Oct-Dec	65-90 days
Sweet Pot.	24 hours	8"	1,178	Weekly- Year Round	120-160days
Malanga	12 hours	8"	1,532	Oct-Jan	Year Round
Calabaza	12 hours	8"	737	Year Round	120-150days
Okra	12 hours	6-8"	986	Jun-Jul Oct-Dec	70-80 days
Yuca	12 hours	6-8"	1,061	Oct-Jan	8-12 Mths.
Mixed Veg.	12 hours	-	1,003		-
Cucumbers	12 hours	6-8"	1,500	Apr-Nov	40-55 days
Sugar Cane***	-	6"	-	-	-

* Includes preharvest operation costs and any fixed cost (rent, depreciation, licenses, and insurance). Per acre. Loss assumes all costs incurred by grower up to harvest Period.
** Lettuce, Endive, and Romaine
*** No data available.

Source: Institute of Food and Agriculture Sciences (IFAS), University of Florida, Gainesville; Dade County Extension Service (IFAS), Homestead, Florida.

Information displayed in this table was provided by the Dade County Farm Bureau in the form of written correspondence initiated by the firm of Kelly, Black, Black & Earle and subsequent telephone conversation held with Mr. James of the Farm Bureau.

TABLE H-12

FRUIT CROP LOSS PER ACRE

Crops	Root Depth	Fruit Loss After --	Loss Per Acre	Tree Loss After --	Loss Per Acre	Growing Periods	Harvest Periods
Limes	18"	2 days	$1,524	7-14 days	$6,095	Year Round	Every 120 days
Mangoes	18"	10 days	2,550	6 weeks	7,111	Year Round	June-Sept
Avocadoes	18"	6 hours	1,524	48-72 hrs	6,603	Year Round	Feb-May
Papaya	12"	6 hours	2,235	12 hours	2,235	45 days	Fall& Winter
Oranges	18"	2 days	1,100	7-14 hours	5,100	Year Rd.	Jan-Feb
Carambola	18"	13 hours	1,500	2 days	4,500	90 days	Aug-Sept Dec-Jan
Bananas**	12"	Water Tolerant		-	Bloom Fr. 5-4 Mos. Plant Fr. 14 Mos.	Year Round	
Guava**	6-12"	Water Tolerant		-	-	-	-
Lychee	12-18"	2 days	-	7 days	-	Mar-Jun	Jun-Jul

*Tree Loss include operating and fixed costs during tree maturity period to fruit-bearing, break-even year.

**Highly water tolerant. No data available on water-resistance period or known damages in previous flooding situations.

Source: Dade County Extension Service (IFAS), University of Florida Tropical Research Station and Florida Lime & Avocado Committee, Homestead, Florida.

TABLE H-13

EXAMPLE OF CROP PRODUCTION COST
GROUND TOMATOES
DADE COUNTY AREA, 1987-88

CATEGORY	AVERAGE PER ACRE
YIELD (25 LB.CARTONS)	1,000
OPERATING COSTS	----DOLLARS----
Seed	200.55
Fertilizer	298.75
Bactericide	73.78
Fumigant	202.50
Fungicide	118.40
Herbicide	45.34
Insecticide	201.39
Surfactant	36.86
Labor	610.65
Machinery	154.08
Interest	127.92
Miscellaneous	
Crop insurance	235.00
Custom work	115.03
Plastic	183.00
Wells	23.60
Total operating cost	2,626.85
FIXED COSTS	
Land rent	325.00
Machinery	151.99
Overhead	208.31
Total fixed cost	685.30
TOTAL PREHARVEST COST	3,312.15
HARVEST AND MARKETING COSTS	
Harvesting	630.00
Hauling	70.00
Packing	1,740.00
Containers	580.00
Selling	150.00
Total harvest and marketing cost	3,170.00
TOTAL COST	6,482.15

TABLE H-14

BASE CONDITION

DEPTH AND DURATION OF FLOODING

GRID SQUARE	NUMBER OF STRUCTURES	GROUND ELEVATION (MSL)	FLOOD FREQUENCY													
			2 YEAR		5 YEAR		10 YEAR		20 YEAR		50 YEAR		100 YEAR		SPF	
			ELEV. (MSL)	DURA. (DAYS)	ELEV. (MSL)	DURA. (DAYS)	ELEV. (MSL)	DURA. (DAYS)	ELEV. (MSL)	DURA. (DAYS)	ELEV. (MSL)	DURA. (DAYS)	ELEV. (MSL)	DURA. (DAYS)	ELEV. (MSL)	DURA. (DAYS)
22	36	6.9	6.8	0	7.2	5	7.6	15	7.9	36	8.2	117	8.4	240	8.9	365
23	10	6.6	6.8	6	7.2	27	7.5	80	7.6	210	8.1	365	8.4	365	8.9	365
24	12	6.7	6.6	0	7.1	10	7.4	24	7.7	70	8.0	260	8.3	365	8.8	365
25	20	7.4	6.4	0	7.0	0	7.3	0	7.6	5	7.9	14	8.2	24	8.5	74
26	5	6.7	6.9	5	7.2	17	7.4	52	7.7	130	8.0	350	8.3	365	8.9	365
27	5	6.5	6.9	5	7.2	17	7.4	52	7.7	130	8.0	350	8.3	365	8.9	365
29	6	6.6	6.9	5	7.3	19	7.6	50	7.8	160	8.2	365	8.4	365	8.9	365
31	9	7.5	6.5	0	7.1	0	7.4	0	7.6	13	8.1	20	8.4	37	8.8	60
32	31	7.5	6.5	0	7.1	0	7.4	0	7.6	13	8.1	20	8.4	37	8.8	60
33	20	7.5	6.4	0	7.1	0	7.4	0	7.6	13	8.1	20	8.4	37	8.8	60
35	1	7.0	6.8	0	7.1	8	7.4	23	7.7	49	8.1	155	8.4	170	8.9	240
36	11	7.0	6.8	0	7.1	8	7.4	23	7.7	49	8.1	155	8.4	170	8.9	240
37	37	7.4	6.6	0	7.1	0	7.4	0	7.6	13	8.1	20	8.4	37	8.8	60
38	15	7.4	6.6	0	7.1	0	7.4	0	7.6	13	8.1	20	8.4	37	8.8	60

TABLE H-15

WITHOUT MITIGATION PLAN

DEPTH AND DURATION OF FLOODING

GRID SQUARE	NUMBER OF STRUCTURES	GROUND ELEVATION	FLOOD FREQUENCY 2 YEAR		5 YEAR		10 YEAR		20 YEAR		50 YEAR		100 YEAR		SPF	
			ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.
		(MSL)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)
22	36	6.9	6.8	0	7.2	9	7.2	23	7.9	56	8.3	160	8.5	290	9.1	340
23	10	6.6	6.9	6	7.3	27	7.3	71	7.8	260	8.2	365	8.5	365	9.1	365
24	12	6.7	6.6	1	7.1	13	7.1	34	7.8	90	8.2	265	8.4	365	9.0	365
25	20	7.4	6.4	0	7.0	0	7.3	0	7.6	10	8.0	23	8.3	40	8.8	76
26	5	6.7	6.9	8	7.3	36	7.6	96	7.9	365	8.2	365	8.5	365	9.3	365
27	5	6.8	6.9	8	7.3	36	7.6	96	7.9	365	8.3	365	8.6	365	9.3	365
29	6	6.6	6.9	7	7.3	33	7.6	88	7.9	365	8.3	365	8.6	365	9.1	365
31	9	7.8	6.6	0	7.1	0	7.5	0	7.9	9	8.4	22	8.7	37	9.1	70
32	31	7.5	6.6	0	7.1	0	7.5	0	7.9	9	8.4	22	8.7	37	9.1	70
33	20	7.6	6.6	0	7.0	0	7.4	0	7.8	9	8.3	22	8.6	37	9.0	70
35	1	7.0	6.8	0	7.3	12	7.6	33	7.9	85	8.3	240	8.6	365	9.3	365
36	11	7.0	6.8	0	7.3	12	7.6	33	7.9	85	8.3	240	8.6	365	9.3	365
37	37	7.4	6.6	0	7.1	0	7.5	6	7.9	16	8.4	39	8.7	70	9.4	120
38	15	7.4	6.6	0	7.1	0	7.5	6	7.9	16	8.4	39	8.7	70	9.4	120

TABLE H-16

841 CFS - PLAN A

DEPTH AND DURATION OF FLOODING

GRID SQUARE	NUMBER OF STRUCTURES	GROUND ELEVATION	FLOOD FREQUENCY													
			2 YEAR		5 YEAR		10 YEAR		20 YEAR		50 YEAR		100 YEAR		SPF	
			ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.
		(MSL)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)
22	36	6.9	5.9	0	6.6	0	7.0	1	7.5	3	8.0	10	8.3	19	8.9	77
23	10	6.6	6.2	0	6.7	1	7.1	5	7.4	12	7.9	27	8.3	45	9.1	130
24	12	6.7	5.9	0	6.5	0	7.0	3	7.4	10	8.0	20	8.5	35	9.5	100
25	20	7.4	5.8	0	6.4	0	6.9	0	7.4	1	8.0	4	8.4	10	9.2	48
26	5	6.7	6.4	0	7.0	3	7.4	12	7.8	23	8.2	44	8.6	70	9.2	160
27	5	6.8	6.4	0	7.0	3	7.4	12	7.8	23	8.2	44	8.6	70	9.2	160
29	6	6.6	5.9	0	6.6	0	7.0	2	7.4	7	7.9	21	8.3	38	9.1	140
31	9	7.8	6.0	0	6.7	0	7.1	0	7.5	2	8.1	8	8.4	16	9.2	58
32	31	7.5	6.0	0	6.7	0	7.1	0	7.5	2	8.1	8	8.4	16	9.2	58
33	20	7.6	5.8	0	6.4	0	6.9	0	7.4	2	8.0	8	8.4	16	9.0	58
35	1	7.0	6.0	0	6.7	0	7.4	1	7.6	5	8.2	20	8.6	33	9.4	90
36	11	7.0	6.0	0	6.7	0	7.4	1	7.6	5	8.2	20	8.6	33	9.4	90
37	37	7.4	6.3	0	6.7	0	7.1	0	7.5	2	8.1	8	8.4	16	9.2	58
38	15	7.4	6.3	0	6.7	0	7.1	0	7.5	2	8.1	8	8.4	16	9.2	58

TABLE H-17

841 + 404 CFS - PLAN B

DEPTH AND DURATION OF FLOODING

GRID SQUARE	NUMBER OF STRUCTURES	GROUND ELEVATION	FLOOD FREQUENCY													
			2 YEAR		5 YEAR		10 YEAR		20 YEAR		50 YEAR		100 YEAR		SPF	
			ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.
		(MSL)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)
22	36	6.9	5.9	0	6.5	0	7.0	1	7.5	3	8.0	10	8.4	19	9.2	78
23	10	6.6	6.2	0	6.7	1	7.0	4	7.4	12	7.9	27	8.4	45	9.3	130
24	12	6.7	5.9	0	6.5	0	7.0	2	7.4	7	8.0	18	8.4	36	9.4	120
25	20	7.4	5.7	0	6.4	0	6.9	0	7.4	1	8.0	4	8.3	9	9.1	44
26	5	6.7	6.4	0	7.0	3	7.4	13	7.6	24	8.2	46	8.5	69	9.1	160
27	5	6.8	6.4	0	7.0	3	7.4	13	7.6	24	8.2	46	8.5	69	9.1	160
29	6	6.6	5.9	0	6.6	0	7.0	2	7.4	7	8.0	20	8.3	36	9.1	150
31	9	7.8	6.0	0	6.7	0	7.1	0	7.5	1	8.1	5	8.4	15	9.2	54
32	31	7.5	6.0	0	6.7	0	7.1	0	7.5	1	8.1	5	8.4	15	9.2	54
33	20	7.6	6.0	0	6.7	0	7.1	0	7.5	1	8.1	5	8.4	15	9.2	54
35	1	7.0	6.0	0	6.7	0	7.1	1	7.6	5	8.1	19	8.5	32	9.3	94
36	11	7.0	6.0	0	6.7	0	7.1	1	7.6	5	8.1	19	8.5	32	9.3	94
37	37	7.4	6.0	0	6.7	0	7.1	0	7.5	1	8.1	5	8.4	15	9.2	54
38	15	7.4	6.0	0	6.7	0	7.1	0	7.5	1	8.1	5	8.4	15	9.2	54

TABLE H-18

1338 CFS - PLAN C

DEPTH AND DURATION OF FLOODING

GRID SQUARE	NUMBER OF STRUCTURES	GROUND ELEVATION	FLOOD FREQUENCY													
			2 YEAR		5 YEAR		10 YEAR		20 YEAR		50 YEAR		100 YEAR		SPF	
			ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.
		(MSL)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)	(MSL)	(DAYS)
22	36	6.9	5.8	0	6.4	0	6.9	0	7.3	2	7.9	5	8.3	14	9.2	56
23	10	6.6	6.1	0	6.5	0	6.9	2	7.3	5	7.7	20	8.1	31	8.9	63
24	12	6.7	5.6	0	6.3	0	6.8	1	7.2	3	7.9	14	8.4	29	9.7	64
25	20	7.4	5.7	0	6.3	0	6.7	0	7.2	0	7.7	2	8.2	7	9.2	41
26	5	6.7	6.9	0	7.3	1	7.5	4	7.8	18	8.2	34	8.5	45	9.3	72
27	5	6.8	6.9	0	7.3	1	7.5	4	7.8	18	8.2	34	8.5	45	9.3	72
29	6	6.6	5.8	0	6.4	0	6.9	1	7.3	3	7.8	14	8.2	28	9.1	74
31	9	7.8	5.9	0	6.5	0	6.9	0	7.3	0	7.9	6	8.3	12	9.2	40
32	31	7.5	5.9	0	6.5	0	6.9	0	7.3	0	7.9	6	8.3	12	9.2	40
33	20	7.6	5.9	0	6.5	0	6.9	0	7.3	0	7.9	6	8.3	12	9.2	40
35	1	7.0	6.0	0	6.6	0	7.1	1	7.5	3	8.1	15	8.6	26	9.5	63
36	11	7.0	6.0	0	6.6	0	7.1	1	7.5	3	8.1	15	8.6	26	9.5	63
37	37	7.4	5.9	0	6.5	0	6.9	0	7.3	0	7.9	6	8.3	12	9.2	40
38	15	7.4	5.9	0	6.5	0	6.9	0	7.3	0	7.9	6	8.3	12	9.2	40

TABLE H-19

470+900 - PLAN E

DEPTH AND DURATION OF FLOODING

GRID SQUARE	NUMBER OF STRUCTURES	GROUND ELEVATION	FLOOD FREQUENCY													
			2 YEAR		5 YEAR		10 YEAR		20 YEAR		50 YEAR		100 YEAR		SPF	
			ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.	ELEV.	DURA.
22	36	6.9	5.6	0	6.4	0	6.9	1	7.3	4	7.8	11	8.1	24	8.7	96
23	10	6.6	6.2	0	6.6	1	7.0	5	7.4	12	7.9	31	8.3	52	9.1	180
24	12	6.7	5.9	0	6.6	0	7.0	3	7.4	8	8.0	22	8.4	42	9.2	190
25	20	7.4	5.8	0	6.6	0	7.1	0	7.6	2	8.1	6	8.4	12	9.0	48
26	5	7.0	6.6	0	7.0	0	7.3	23	7.5	36	7.9	58	8.1	87	8.6	240
27	5	6.7	6.9	0	7.2	10	7.4	60	7.7	36	8.0	58	8.3	87	9.0	240
29	6	6.6	5.9	0	6.6	10	7.0	4	7.4	15	7.9	36	8.2	61	8.8	210
31	9	7.4	6.0	0	6.7	0	7.2	0	7.8	1	8.1	5	8.4	13	8.6	60
32	31	7.4	6.0	0	6.7	0	7.2	0	7.8	1	8.1	5	8.4	13	8.6	60
33	20	7.6	6.5	0	7.1	0	7.4	1	7.6	1	8.1	5	8.4	13	8.8	60
35	1	7.0	6.1	0	6.6	0	7.0	1	7.4	5	7.9	15	8.3	27	9.1	93
36	11	7.0	6.1	0	6.6	0	7.0	1	7.4	5	7.9	15	8.3	27	9.1	93
37	37	7.4	5.7	0	6.4	0	6.9	0	7.4	1	8.0	5	8.3	13	9.0	60
38	15	7.4	5.7	0	6.4	0	6.9	0	7.4	1	8.0	5	8.3	13	9.0	60

TABLE H-20

HOUSES AND DURATION OF FLOODING

	5 Year Freq.		20 Year Freq		50 Year Freq.	
Plans	No. of Houses	Duration	No. of Houses	Duration	No. of Houses	Duration
Base Condition	20 66	27-30 days 5-19 days	38 180	70-160 days 7-49 days	86 132	117-365 days 14-78 days
Without Mitigation	26 59	27-36 days 9-13 days	26 60 132	190-365 days 6-50 days 9-16 days	50 80	240-365 days 22-160 days
1338 CFS-C	10	1 day	86	2-18 days	20 198	20-34 days 2-15 days
841+404 CFS-B	20	1-3 days	16 202	12-24 days 1-17 days	20 198	27-46 days 5-19 days
841 CFS-A	20	1-3 days	20 198	12-23 days 1-10 days	20 198	27-44 days 8-21 days
470+900 CFS-E	5 5	1-10 days 16-30 days	165 48 5	1-5 days 6-12 days 36 days	180 28 10	5-15 days 16-31 days 58 days

TABLE H-21

MILES PER GRID SQUARE

Grid Square	Total Road (miles)	Paved Road (miles)	Unpaved Road (miles)
22	5.5	0.00	5.50
23	1.24	.96	.28
24	1.00	.57	.43
25	4.27	1.00	3.27
26	1.55	0.00	1.55
27	.63	.37	.26
28	1.42	.99	.43
29	4.46	.99	3.47
31	1.99	0.00	1.99
32	6.36	0.00	6.36
33	4.19	.90	3.29
35	3.02	0.00	3.02
36	1.98	0.00	1.98
37	.97	.97	0.00
38	3.19	.63	2.56
TOTAL	41.77	7.38	34.39

TABLE H-22

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
BASE CONDITION

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
22	163.2	165.6	165.6	165.6	165.6	165.6	165.6
24	21.4	21.5	21.5	21.5	21.5	21.5	21.5
25	891.1	1,065.7	1,069.8	1,069.8	1,069.8	1,069.8	1,069.8
26	62.1	62.1	62.1	62.1	62.1	62.1	62.1
29	250.3	250.3	250.3	250.3	250.3	250.3	250.3
31	126.3	332.7	406.9	412.1	412.1	412.1	412.1
32	582.0	837.9	841.3	846.1	846.3	846.3	846.3
33	266.3	606.9	783.7	794.9	794.9	794.9	794.9
34	.0	.0	4.8	4.8	4.8	4.8	4.8
35	279.7	304.2	304.2	304.2	304.2	304.2	304.2
36	146.5	168.6	168.6	168.6	168.6	168.6	168.6
37	610.5	715.9	733.0	761.2	761.3	761.3	761.3
38	67.6	178.7	182.6	189.3	189.4	189.4	189.4
39	11.6	11.6	11.6	11.6	11.6	11.6	11.6
40	82.2	82.2	82.2	82.2	82.2	82.2	82.2
TOTAL	3,560.8	4,803.9	5,088.2	5,144.3	5,144.7	5,144.7	5,144.7

TABLE H-23

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
BASE CONDITION

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
1	778.5	778.5	778.5	778.5	778.5	778.5	778.5
2	289.4	289.4	289.4	289.4	289.4	289.4	289.4
6	437.0	437.0	437.0	437.0	437.0	437.0	437.0
7	506.7	555.7	555.7	555.8	555.8	555.8	555.8
10	78.4	78.4	78.4	78.4	78.4	78.4	78.4
13	4.2	4.2	4.2	4.2	4.2	4.2	4.2
14	261.6	277.7	278.5	278.5	278.5	278.5	278.5
16	391.2	1,253.9	1,423.2	1,423.3	1,423.3	1,423.2	1,423.2
18	693.5	1,052.3	1,058.2	1,065.6	1,065.6	1,065.6	1,065.6
19	40.4	40.4	40.4	40.4	40.4	40.4	40.4
20	99.3	675.2	771.6	771.6	771.6	771.6	771.6
21	128.2	459.4	469.3	469.6	469.7	469.7	469.7
30	28.0	35.2	35.2	35.3	35.3	35.3	35.3
TOTAL	3,736.4	5,937.3	6,219.6	6,227.6	6,227.7	6,227.6	6,227.6

TABLE H-24

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
WITHOUT MITIGATION PLAN

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
22	163.1	165.6	165.6	165.6	165.6	165.6	165.6
24	21.5	21.5	21.5	21.5	21.5	21.5	21.5
25	974.9	1,065.8	1,069.8	1,069.8	1,069.8	1,069.8	1,069.8
26	62.1	62.1	62.1	62.1	62.1	62.1	62.1
29	250.4	250.4	250.4	250.4	250.4	250.4	250.4
31	126.4	332.8	412.2	412.2	412.2	412.2	412.2
32	623.8	841.5	846.3	846.3	846.3	846.3	846.3
33	339.5	630.3	783.8	794.9	794.9	794.9	794.9
34	.0	.0	4.8	4.8	4.8	4.8	4.8
35	278.9	282.6	304.2	304.2	304.2	304.2	304.2
36	168.6	168.6	168.6	168.6	168.6	168.6	168.6
37	648.8	733.5	761.4	761.4	761.4	761.4	761.4
38	80.5	178.7	189.4	189.4	189.4	189.4	189.4
39	11.6	11.6	11.6	11.6	11.6	11.6	11.6
40	82.1	82.1	82.1	82.1	82.1	82.1	82.1
TOTAL	3,832.2	4,827.1	5,133.8	5,144.9	5,144.9	5,144.9	5,144.9

TABLE H-25

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
WITHOUT MITIGATION PLAN

FREQUENCY

GRID SQUARE NUMBER	2-YR VALUE OF GROSS RETURNS	5-YR VALUE OF GROSS RETURNS	10-YR VALUE OF GROSS RETURNS	20-YR VALUE OF GROSS RETURNS	50-YR VALUE OF GROSS RETURNS	100-YR VALUE OF GROSS RETURNS	SPF VALUE OF GROSS RETURNS
1	778.5	778.5	778.5	778.5	778.5	778.5	778.5
2	289.4	289.4	289.4	289.4	289.4	289.4	289.4
6	437.0	437.0	437.0	437.0	437.0	437.0	437.0
7	515.1	555.8	555.8	555.8	555.8	555.8	555.8
10	78.4	78.4	78.4	78.4	78.4	78.4	78.4
13	4.2	4.2	4.2	4.2	4.2	4.2	4.2
14	268.6	277.8	278.5	278.5	278.5	278.5	278.5
16	586.3	1,376.0	1,423.3	1,423.3	1,423.3	1,423.3	1,423.3
18	915.1	1,052.4	1,065.7	1,065.7	1,065.7	1,065.7	1,065.7
19	40.3	40.3	40.3	40.3	40.3	40.3	40.3
20	237.4	675.2	771.6	771.6	771.6	771.6	771.6
21	241.3	459.4	470.0	470.6	470.6	470.6	470.6
30	28.0	35.3	35.3	35.3	35.3	35.3	35.3
TOTAL	4,419.6	6,059.7	6,228.0	6,228.6	6,228.6	6,228.6	6,228.6

TABLE H-26

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
PLAN A

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
22	54.9	122.2	157.8	165.5	165.6	165.6	165.6
24	9.4	21.4	21.5	21.5	21.5	21.5	21.5
25	854.0	1,057.2	1,065.6	1,069.7	1,069.7	1,069.8	1,069.8
26	62.1	62.1	62.1	62.1	62.1	62.1	62.1
29	129.1	250.3	250.3	250.3	250.4	250.4	250.4
31	104.8	139.2	332.7	412.1	412.2	412.1	412.1
32	291.5	578.2	841.8	846.8	846.2	846.3	846.3
33	2.2	286.4	426.8	783.3	784.9	794.9	794.9
34	.0	.0	.0	4.8	4.8	4.8	4.8
35	143.9	194.9	304.1	304.1	304.1	304.1	304.1
36	76.0	168.7	168.7	168.6	168.6	168.6	168.6
37	337.5	635.8	733.0	739.4	761.2	761.3	761.3
38	43.2	67.6	178.7	189.4	189.3	190.4	190.4
39	11.6	11.6	11.6	11.6	11.6	11.6	11.6
40	50.5	82.0	82.1	82.1	82.1	82.1	82.1
TOTAL	2,170.7	3,677.6	4,636.8	5,111.3	5,134.3	5,145.6	5,145.6

TABLE H-27

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
PLAN A

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
1	778.5	778.5	778.5	778.5	778.5	778.5	778.5
2	289.4	289.4	289.4	289.4	289.4	289.4	289.4
6	437.0	437.0	437.0	437.0	437.0	437.0	437.0
7	506.4	555.9	555.9	555.9	555.9	555.9	555.9
10	78.4	78.4	78.4	78.4	78.4	78.4	78.4
13	4.2	4.2	4.2	4.2	4.2	4.2	4.2
14	249.4	278.0	278.5	278.5	278.5	278.5	278.5
16	540.3	1,375.9	1,423.3	1,423.3	1,423.3	1,423.3	1,423.3
18	269.5	952.7	1,052.3	1,065.6	1,065.6	1,065.6	1,065.6
19	40.4	40.4	40.4	40.4	40.4	40.4	40.4
20	99.6	121.9	767.2	771.5	771.6	771.6	771.6
21	110.0	459.5	469.8	469.8	469.9	469.9	469.9
30	7.0	28.0	35.2	35.2	35.2	35.2	35.2
TOTAL	3,410.1	5,399.8	6,210.1	6,227.7	6,227.9	6,227.9	6,227.9

TABLE H-28

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
PLAN B

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
22	53.9	122.2	165.6	165.6	165.6	165.6	165.6
24	9.4	321.4	321.4	321.4	321.4	321.4	321.4
25	843.0	945.9	1,038.0	1,069.7	1,069.7	1,069.7	1,069.7
26	62.1	62.1	62.1	62.1	62.1	62.1	62.1
29	129.1	250.3	250.3	250.3	250.4	250.4	250.4
31	104.8	139.3	332.8	412.7	412.3	412.1	412.1
32	176.3	846.6	842.6	846.8	846.9	846.9	846.9
33	2.2	286.4	469.3	783.7	794.9	794.9	794.9
34	.0	.0	.0	4.8	4.8	4.8	4.8
35	141.9	194.9	304.1	304.2	304.2	304.2	304.2
36	76.0	168.6	168.6	168.6	168.6	168.6	168.6
37	337.7	645.9	733.1	739.7	761.5	761.5	761.5
38	43.2	67.6	178.7	189.3	189.4	189.4	189.4
39	11.6	11.6	11.6	11.6	11.6	11.6	11.6
40	50.6	82.2	82.2	82.2	82.2	82.2	82.2
TOTAL	2,041.8	4,145.0	4,960.4	5,412.7	5,445.6	5,445.4	5,445.4

TABLE H-29

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
PLAN B

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
1	778.5	778.5	778.5	778.5	778.5	778.5	778.5
2	289.4	289.4	289.4	289.4	289.4	289.4	289.4
6	437.0	437.0	437.0	437.0	437.0	437.0	437.0
7	492.8	535.8	555.8	555.8	555.8	555.8	555.8
10	78.4	78.4	78.4	78.4	78.4	78.4	78.4
13	4.2	4.2	4.2	4.2	4.2	4.2	4.2
14	148.1	247.0	270.0	273.2	278.5	278.5	278.5
16	351.2	824.4	1,285.3	1,423.3	1,423.3	1,423.3	1,423.3
18	272.6	952.5	1,050.5	1,051.0	1,065.6	1,065.6	1,065.6
19	40.4	40.4	40.4	40.4	40.4	40.4	40.4
20	99.2	375.9	675.8	667.2	771.6	771.6	771.6
21	61.3	324.6	469.6	469.6	469.6	469.6	469.6
30	7.1	10.4	35.3	35.2	35.2	35.3	35.3
TOTAL	3,060.2	4,898.5	5,970.2	6,103.2	6,227.5	6,227.6	6,227.6

TABLE H-30

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
PLAN C

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
22	46.6	90.3	124.5	159.0	165.6	165.6	165.6
24	9.4	21.4	21.4	21.5	21.5	21.5	21.5
25	27.7	122.3	207.1	1,069.8	1,069.8	1,069.8	1,069.8
26	62.0	62.0	62.0	62.0	62.0	62.0	62.0
29	87.7	250.4	250.4	250.4	250.4	250.4	250.4
31	16.2	126.4	328.2	406.9	412.2	412.2	412.2
32	213.6	547.1	803.5	841.4	846.2	846.2	846.2
33	2.2	259.5	311.3	677.1	692.4	794.8	794.9
34	.0	.0	.0	.0	4.8	4.8	4.8
35	110.5	194.8	304.1	304.1	304.2	304.2	304.2
36	46.8	160.1	168.6	168.6	168.6	168.6	168.6
37	262.8	578.6	687.6	733.1	761.3	761.4	761.4
38	8.1	43.2	176.3	182.5	189.4	189.4	189.4
39	11.6	11.6	11.6	11.6	11.6	11.6	11.6
40	32.8	82.1	82.1	82.1	82.1	82.1	82.1
TOTAL	938.0	2,549.8	3,538.7	4,970.1	5,042.1	5,144.6	5,144.7

TABLE H-31

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
PLAN C

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
1	778.5	778.5	778.5	778.5	778.5	778.5	778.5
2	289.4	289.4	289.4	289.4	289.4	289.4	289.4
6	437.0	437.0	437.0	437.0	437.0	437.0	437.0
7	506.4	555.8	555.8	555.8	555.8	555.8	555.8
10	78.4	78.4	78.4	78.4	78.4	78.4	78.4
13	4.2	4.2	4.2	4.2	4.2	4.2	4.2
14	233.1	273.3	278.5	278.5	278.5	278.5	278.5
16	391.1	1,380.3	1,423.2	1,423.2	1,423.2	1,423.2	1,423.2
18	272.6	955.6	1,051.0	1,058.3	1,065.7	1,065.7	1,065.7
19	40.3	40.3	40.3	40.3	40.3	40.3	40.3
20	99.3	666.7	771.4	771.6	771.6	771.6	771.6
21	61.2	232.5	469.7	469.8	469.9	469.9	469.9
30	7.0	28.0	35.2	35.2	35.2	35.2	35.2
TOTAL	3,198.5	5,720.0	6,212.6	6,220.2	6,227.7	6,227.7	6,227.7

TABLE H-32

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
PLANS E AND F

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
22	56.3	122.3	165.7	165.7	165.7	165.7	165.7
24	9.4	20.4	21.5	21.5	21.5	21.5	21.5
25	854.0	1,033.1	1,066.9	1,066.9	1,066.9	1,066.9	1,066.9
26	62.1	62.1	62.1	62.1	62.1	62.1	62.1
29	163.9	250.3	250.3	250.4	250.4	250.4	250.4
31	16.2	126.4	161.7	412.2	412.2	412.2	412.2
32	160.3	517.3	713.8	846.3	846.3	846.3	846.3
33	47.3	281.1	658.3	786.8	794.9	794.9	794.9
34	.0	.0	.0	4.8	4.8	4.8	4.8
35	146.9	194.9	299.6	304.2	304.2	304.2	304.2
36	80.5	168.6	168.6	168.6	168.6	168.6	168.6
37	238.0	578.7	687.6	739.4	761.4	761.4	761.4
38	.0	43.3	178.7	189.4	189.4	189.4	189.4
39	11.6	11.6	11.6	11.6	11.6	11.6	11.6
40	82.1	82.1	82.1	82.1	82.1	82.1	82.1
TOTAL	1,928.6	3,492.2	4,528.5	5,112.0	5,142.1	5,142.1	5,142.1

TABLE H-33

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
PLANS E AND F

FREQUENCY

GRID SQUARE NUMBER	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES
1	778.5	778.5	778.5	778.5	778.5	778.5	778.5
2	289.4	289.4	289.4	289.4	289.4	289.4	289.4
6	437.0	437.0	437.0	437.0	437.0	437.0	437.0
7	506.7	555.8	555.8	555.8	555.8	555.8	555.8
10	78.4	78.4	78.4	78.4	78.4	78.4	78.4
13	4.2	4.2	4.2	4.2	4.2	4.2	4.2
14	207.6	276.4	278.5	278.5	278.5	278.5	278.5
16	391.2	1,423.3	1,423.3	1,423.3	1,423.3	1,423.3	1,423.3
18	295.5	952.6	1,052.3	1,065.6	1,065.7	1,065.7	1,065.7
19	40.4	40.4	40.4	40.4	40.4	40.4	40.4
20	99.3	375.8	694.3	771.6	771.7	771.7	771.7
21	61.2	430.1	469.9	469.8	469.8	469.8	469.8
30	7.0	28.0	35.2	35.2	35.3	35.3	35.3
TOTAL	3,196.4	5,669.9	6,137.2	6,227.7	6,228.0	6,228.0	6,228.0

TABLE H-34

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
RESIDENTIAL AREA
(thousands of dollars)

FREQUENCY

PLAN	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES	AVG ANNL MAXIMUM VALUE OF LOSSES
Base	3,560.8	4,803.9	5,088.2	5,144.3	5,144.7	5,144.7	5,144.7	3,152.6
No Action	3,832.2	4,827.1	5,133.8	5,144.9	5,144.9	5,144.9	5,144.9	3,269.2
Plan A	2,170.7	3,677.6	4,636.8	5,111.3	5,134.3	5,145.6	5,145.6	2,335.9
Plan B	2,041.8	4,145.0	4,960.4	5,412.7	5,445.6	5,445.4	5,445.4	2,424.9
Plan C	938.0	2,549.8	3,538.7	4,970.1	5,042.1	5,144.6	5,144.7	1,527.4
Plan E(F&G)	1,928.6	3,492.2	4,528.5	5,112.0	5,142.1	5,142.1	5,142.1	2,194.0

TABLE H-35

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SOUTHERN AGRICULTURAL AREA
(thousands of dollars)

FREQUENCY

PLAN	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES	AVG ANNL MAXIMUM VALUE OF LOSSES
Base	3,736.4	5,937.3	6,219.6	6,227.6	6,227.7	6,227.6	6,227.6	3,615.6
No Action	4,419.6	6,059.7	6,228.0	6,228.6	6,228.6	6,228.6	6,228.6	3,914.0
Plan A	3,410.1	5,399.8	6,210.1	6,227.7	6,227.9	6,227.9	6,227.9	3,376.8
Plan B	3,060.2	4,898.5	5,970.2	6,103.2	6,227.5	6,227.6	6,227.6	3,113.6
Plan C	3,198.5	5,720.0	6,212.6	6,220.2	6,227.7	6,227.7	6,227.7	3,356.1
Plan E(F&G)	3,196.4	5,669.9	6,137.2	6,227.7	6,228.0	6,228.0	6,228.0	3,339.9

TABLE H-36

DAMAGEABLE VALUE OF CROPS SUBJECT TO GROUND WATER ROOT INUNDATION
SUM OF RESIDENTIAL AND SOUTHERN AGRICULTURAL AREAS
(thousands of dollars)

FREQUENCY

PLAN	2-YR MAXIMUM VALUE OF LOSSES	5-YR MAXIMUM VALUE OF LOSSES	10-YR MAXIMUM VALUE OF LOSSES	20-YR MAXIMUM VALUE OF LOSSES	50-YR MAXIMUM VALUE OF LOSSES	100-YR MAXIMUM VALUE OF LOSSES	SPF MAXIMUM VALUE OF LOSSES	AVG ANNL MAXIMUM VALUE OF LOSSES
Base	7,297.2	10,741.2	11,307.8	11,371.9	11,372.4	11,372.3	11,372.3	6,768.1
No Action	8,251.8	10,886.8	11,361.8	11,373.5	11,373.5	11,373.5	11,373.5	7,183.2
Plan A	5,580.8	9,077.4	10,846.9	11,339.0	11,362.2	11,373.5	11,373.5	5,712.7
Plan B	5,102.0	9,043.5	10,930.6	11,515.9	11,673.1	11,673.0	11,673.0	5,538.5
Plan C	4,136.5	8,269.8	9,751.3	11,190.3	11,269.8	11,372.3	11,372.4	4,883.5
Plan E(F&G)	5,125.0	9,162.1	10,665.7	11,339.7	11,370.1	11,370.1	11,370.1	5,533.9

TABLE H-37

TEMPORARY RELOCATION
($1,000)

PLAN	2-YR	5-YR	10-YR	20-YR	50-YR	100-YR	SPF	AVG.ANN.
BASE	27.0	314.1	859.2	2,514.0	6,423.2	8,758.2	11,559.0	482.1
NO ACTION	39.1	465.6	928.2	3,485.4	6,440.4	10,604.1	11,850.0	582.8
PLAN A	0.0	22.2	79.8	284.7	876.6	1,461.0	4,874.4	69.5
PLAN B	0.0	12.0	83.4	253.8	717.0	1,459.5	4,903.2	65.0
PLAN C	0.0	4.8	30.6	137.4	589.8	1,098.0	3,183.0	42.4
PLAN E(F&G)	28.1	52.5	164.4	327.6	840.3	1,585.2	5,656.2	87.4

TABLE H-38

FLOOD DAMAGE TO ROADWAYS
(1986 Dollars)

PLAN	2-YR	5-YR	10-YR	20-YR	50-YR	100-YR	SPF	AVG. ANN
BASE	2,000	5,700	9,200	20,900	21,900	27,900	36,000	4.1
NO ACTION	3,578	6,830	12,766	23,882	27,669	32,880	41,300	5.6
PLAN A	0	0	1,000	2,600	10,400	16,200	28,800	0.6
PLAN B	0	0	1,000	2,600	10,400	16,200	28,800	0.6
PLAN C	0	0	0	2,000	4,000	12,300	26,000	0.4
PLAN E(F&G)	0	468	6,051	10,563	11,988	15,515	31,960	1.4

NOTES

1. Hunter Moss and Company, Metropolitan Miami, undated, p.17.

2. Metropolitan Dade County Planning Department, Comprehensive Development Master Plan for Metropolitan Dade County, (Metro. Dade County Planning Dept., Miami, Florida), July 1979, p.99.

3. Hunter Moss and Company, Metropolitan Miami, undated, p.12.

4. Hunter Moss and Company, Metropolitan Miami, undated, p.13-16.

5. Metropolitan Dade County Planning Department, Comprehensive Development Master Plan for Metropolitan Dade County, (Metro. Dade County Planning Dept., Miami, Florida), July 1979, p.34.

6. U.S. Department of Interior, Resource and Land Information for South Dade County, Florida, (U.S. Government Printing Office, Washington, D.C.), 1973, p.2.

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CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES

PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
MODIFIED WATER DELIVERIES TO EVERGLADES NATIONAL PARK

APPENDIX I

OTHER SOCIAL EFFECTS

CENTRAL AND SOUTHERN FLORIDA PROJECT
FOR FLOOD CONTROL AND OTHER PURPOSES
PART I

AGRICULTURAL AND CONSERVATION AREAS
SUPPLEMENT 54 - GENERAL DESIGN MEMORANDUM
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APPENDIX I

OTHER SOCIAL EFFECTS

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APPENDIX I

OTHER SOCIAL EFFECTS

1. Study Scope. The Other Social Effects (OSE) account is one of four types of evaluations used in water resource planning. This OSE assessment examines the existing conditions and the effects of the water delivery system and flood protection/acquisition alternatives considered by the Corps of Engineers. The categories of effects in the OSE account include urban and community impacts; life, health, and safety factors; displacement; long-term productivity; and energy requirements and conservation.

2. Study Purpose. The Supplemental Appropriations Act of 1984 (PL 98-181) authorizes development and implementation of a new water delivery schedule for Everglades National Park (ENP). This project involves structural and operational modifications of the Central and Southern Florida (C&SF) project, and requires protection or acquisition of agricultural and residential areas potentially affected by resulting increased flooding. The modifications for water delivery and the features for flood protection will create social effects.

3. Background. Congress authorized creation of the Everglades National Park in 1934; the park was actually established in 1947. ENP serves to preserve a unique environmental system and encompasses approximately 1.4 million acres of subtropical wetland. The natural environment of the park has been altered by water control activities affecting the flow entering the Everglades.

Adjacent to the park and integrally linked to its ecological balance is an area designated as the East Everglades, which encompasses the Northeast Shark River Slough (NESRS). The area bordering the park on the east is privately owned and is inhabited/utilized by individual homeowners and agriculturalists. Over 90% of this area is undeveloped, and it serves as a buffer to protect the park from encroaching growth and development. County and State activities have historically been designed to continue development in this area in order to decrease harm to the environment and the Park. This social analysis focuses on the East Everglades.

4. Description of Existing Conditions. The East Everglades includes an area of approximately 183 square miles, including areas designated wetlands, and areas zoned for residential and agricultural use. The residential and agricultural areas are concentrated along the eastern edge of L-31N (see Map #1).

5. Residential. Development in the East Everglades began in earnest during the 1960's due to pressure from South Florida's growing population. Many of the existing roads in this area were constructed between 1965 and 1967, and subdivided land was sold in 1.25 and 2.5 acre tracts. The majority of these lots were purchased by a "contract for deed arrangement" in which the deed was transferred pending fulfillment of an installment contract (scheduled for 8-10

years). Landowners were from outside of the state, for the most part, and often from outside of the country.

The original zoning regulations specified one unit per 1.25 acres. In 1974, zoning was upgraded to one unit per five (5) acres, but previous nonconforming uses were "grandfathered in", and property owners with lots of record purchased and platted prior to 1974 were also allowed to build on substandard lots as small as 1.25 acres. Ninety-two percent of registered properties are five acres or less.

A similar process designed to increase lot size requirements and decrease development was implemented again in 1981 with the passage of the Zoning Overlay Ordinance (No. 81-121) requiring 40 acres per unit (with some exceptions for land with "an established residential character", and dwelling units adjacent to agricultural lands, e.g., with less stringent requirements of 1/5 and 1/20). Unlike its predecessor, the 1981 regulation does not contain a "grandfather clause" for substandard lots in the East Everglades. It does provide the possibility, however, for reversion to 1/5 zoning in the residential area if 1/10 year flood protection is available. The Overlay Ordinance also prohibits construction of new roads in sections of the existing residential and agricultural areas, and extending northwest into the wetlands.

These changes have been designed to retard development in the East Everglades and, as a result, protect Everglades National Park. The most recent zoning requirements (1981) have been challenged in court by homeowners who argue that 1/40 requirement is so limited that it precludes development for most landowners and, therefore, constitutes an illegal "taking" by the County.

To offset some of the effects of the 1981 zoning ordinance, Dade County also adopted a General Use Rights Ordinance to protect the Park from the impacts of development but without depriving landowners of their expected development benefits (No. 81-122).
The population of this area has been estimated to be 418 persons occupying 82 single family dwellings and 62 mobile homes (1978, East Everglades Resources Planning Project Residential Land Use, Dade County Planning Department), or more generally 400-450 persons (1980, East Everglades Resources Planning Project, Proposed Management for the East Everglades).

The residents of the East Everglades live without many county-provided amenities (e.g., garbage pickup). The few commercial enterprises in the vicinity include airboat operations along the Tamiami Trail, several nurseries, a kennel, and a few restaurants and gas stations. The majority of the residents commute outside of the East Everglades for employment; some work on local farms.

In addition to this concentrated residential area are two Miccosukee Indian villages, Tigertail Camp and Osceola Camp, which will be affected by elevated water levels from the project. The Tigertail camp is located along the north side of the Tamiami Trail, between L-67 and L-30. The Indians are affiliated with the Miccosukee Tribe, but the property they inhabit is not Tribal land. This group is comprised of one or more extended families (ca. 20 individuals) living in chickees and conventional wood frame structures. The Indians

operate airboat businesses and crafts stores, and work in nearby gas stations and farms. Some also hunt frogs for sale to nearby restaurants. The remainder (and majority) of the tribe lives elsewhere along the Tamiami Trail or on the State Indian Reservation south of Lake Okeechobee.

To mitigate the effects of increased flooding to the Tigertail camp, a plan was developed to salvage and/or demolish the existing camp structures, relocate the villagers to temporary on-site housing, elevate the village site with 3 to 4 feet of fill, and rebuild the village with structures comparable and in placement similar to those existing now. However, the reconstruction of the camp would be in accordance with minimum property standards and would, in general, constitute an improvement over existing conditions while attempting to maintain cultural continuity.

6. Agriculture. Agriculture is critical to Dade County's economy and to the East Everglades in particular. Approximately 3-5 percent of the study area is currently devoted to agriculture, predominantly for the production of tropical fruits, winter vegetables, citrus and ornamentals).

The pressure on agricultural land from urban growth in Dade County is severe. Between 1966 and 1976, 78 square miles of farmland were lost (East Everglades Resources Planning Project-Agricultural Use, Metropolitan Dade County Planning Department, 1978). To help mitigate continued losses, Dade County adopted a Comprehensive Development Master Plan (CDMP) to protect agricultural land and discourage urban extension into these areas.

Much of the nation's exotic fruits and vegetables are grown in Dade County. Historically, the mix of crops and location have shifted in response to population/land pressures and changing crop markets/profitability. Lands recently planted to help offset these losses includes land in the East Everglades adjacent to Levee 31N. The land further west of the levee is wetlands and is considered marginal for agricultural use. However, the profit from production could justify expansion into this area as available land continues to become scarce.

7. Wetlands. The wetlands of the study area are inundated for much of the year and are considered generally uninhabitable and unsuitable for farming. Only a small portion of the area by the Tamiami Trail has potential for development. Nonetheless, there are a few scattered houses constructed on stilts.

The property in the wetlands is owned by over 7,000 individuals, many of whom live outside of Florida. Foreign land speculators own some of the lands. The State owns and maintains approximately 35,200 acres including the Aerojet Wildlife Management Area and a State Park.

8. Miccosukee Indians. The Miccosukee Tribe of Indians has been granted certain rights within Conservation Area 3A as part of the Florida Indian Land Claims Settlement Act of 1982. These rights are defined for the "Leased Area" within Conservation Area 3A; the Federal Indian Reservations, which partially overlap this area; and the Use Permit Area, south of Conservation Area 3A.

The Miccosukee are entitled to hunt and fish and take frogs for subsistence and commercial purposes in the Leased Area. Moreover, they may receive compensation for public hunting and fishing in this area. They also are allowed to engage in subsistence agriculture, reside in the Leased Area in "traditional homes", and use it for religious purposes. The Tribe agrees that these rights "shall not interfere with the rights, duties and obligations of the SFWMD or the United States Army Corps of Engineers, pursuant to the requirements of the ... federally authorized project, conveyances, easements, grants, rules, statutes, or any other present or future lawful authority to manage, regulate, raise or lower the water levels within the Leased Area or Water Conservation Area 3..." (Miccosukee Indian Tribe Position Paper, 29 October 1990, page 6).

The Broward County Federal Reservation includes lands within Conservation Area 3A. This land is held in Federal trust for the Miccosukee Indian Tribe.

The Tribe also occupies a five mile by 500 foot wide corridor just south of the northern boundary of the Everglades National Park, referred to as the "Use Permit Area". The Tribe is permitted to "live, make and sell handicrafts within the permit area and to construct administrative, operational, educational, medical and community facilities" (Position Paper, page 7).

Implementation of the modified delivery system may mean insufficient water for airboat access to concessions and agricultural areas in Conservation Area 3A during the dry season. This limitation to Indian activities in this area is only anticipated during dry seasons and droughts. The Tribal rights specified in the Miccosukee's 1990 Position Paper should not be affected for the majority of the year.

9. Impact Evaluation

a. Without Project

The "Without Project" evaluation examines projected conditions of the study area if no plan were implemented. While Congressional mandate precludes that possibility, this description will provide a necessary base line for assessing the impacts of proposed alternatives.

Development and population growth within the study area is not expected to be extensive. The lack of services and amenities makes this area undesirable for development.

Ownership by foreigners suggests plans for investment rather than development (Draft Feasibility Report and Draft Environmental Impact Statement on Central and Southern Florida Project-Shark River Slough, U.S. Army Corps of Engineers, September 1982). More importantly, development could be constrained by zoning ordinances. Some construction could occur, of course, on parcels and tracts meeting or exceeding the 40-acre minimum. If one in ten-year flood protection is obtained, the County could allow a return to the 1974 one unit per five acres regulation. This would allow greater construction. The prohibition on new roads would discourage extending residential development into agricultural or wetland areas, as would the one unit per 40 acres zoning requirements.

There is no provision for returning to 1974 zoning requirements in these areas.

These restrictions, coupled with the decreasing availability of desirable agricultural land would also suggest that future large scale residential development of agricultural lands in the East Everglades is unlikely.

The abnormal dryness during the 1970's provided a false sense of security for prospective homeowners in the East Everglades and may have inappropriately encouraged development in floodable areas. This "hydrologic delusion" was destroyed in 1981 by Tropical Storm Dennis.

The unignorable reality of flooding could contribute to a slower development growth as well. Finally, plans being considered by the County to construct a wellfield in the East Everglades would discourage future growth in keeping with a general policy to protect the Park (ENP/EE Resource Planning and Management, 1985) Landowners in the residential and agricultural zones covered by the 1981 SUR ordinance could transfer their development rights to other unincorporated areas in Dade County.

If historical agricultural trends from the 1970's continue, crops should be expected to shift in relation to market demands. Similarly, land value could increase as pressure for replacement agricultural acreage increases (as noted above), or pressure for development continues. The East Everglades agricultural area is bounded by the levee on the east and marginal land on the west. If the value of crops remains high and the amount of available land continues to shrink, farmers may become increasingly willing to expand into more marginal lands and assume the risks. As existing conditions indicate, it may be environmental concerns and political decisions that determine the future of this land in the East Everglades.

The conditions in the wetlands would continue to make the vast majority of the East Everglades marginal for cultivation and unusable for residential development. Only a small section of land by the Tamiami Trail could be developed for homes.

The land presently occupied by the Miccosukee on the north side of the Tamiami Trail would be subject to periodic flooding. This periodic flooding would continue to affect the quality of human life. Without the project, the potential exists for loss of life and property to the Indian group and displacement of the people and their concessions.

b. Alternative Plans. All of the plans considered for a modified water delivery system to Everglades National Park entail providing flood protection to residential areas. Flood protection will include construction of levees, canals, culverts structures and a pump station.

The residential areas needed for constuction of the flood protection structures would be acquired in fee. Easements would most likely be obtained for the wetlands.

The property owners would be compensated for their land and improvements. Residents requiring assistance to relocate to "decent, safe and sanitary" housing would received up to an additional $22,500, in accordance with Public Law 91-646. If the agricultural land were acquired in fee and removed from agricultural use, it would cause a new loss in the available acreage in Dade County. The farmers here own additional agricultural land elsewhere which would increase in value as a result.

The most significant potential impact of flood protecting the residential area would be on zoning and development. With flood protection, property owners could request that the County consider reverting to a one unit per five acre zoning requirement (as allowed in the 1981 Overlay Zoning Ordinance). This zoning change, if implemented, would allow and, combined with the related flood protection, could encourage greater development in this area.

Flowage easement through the wetlands and implementation of the project would not significantly affect that area or its unsuitability for development or agriculture, with some exceptions. The one area that could be developed-i.e., near Tamiami Trail-would be rendered equally as unusable for habitation as the surrounding wetlands.

Changes in the size/capacity of the pump stations could provide additional protection for agricultural areas, causing a shift in viable crops.

Relocation of the Miccosukee is not acceptable to the Indians and is not being considered as an alternative because of Individual Aboriginal Title. With all project alternatives, the lands presently occupied by the Indians would be built up in elevation to a point where the property was flood protected. All structures located on the Indian property would be demolished and replaced. Residents will be given the option of receiving traditional (chickees) or conventional replacement structures. Residents will be temporarily relocated while the earth work and structural work is completed. Relocation will be to an area as nearly adjacent as possible to avoid disruption to the Indians quality of life. With the project, the potential for loss of life and property caused from flooding will be greatly diminished. Maintenance and enhancement of productivity of resources for use by future generations will be improved by the project. Interference with access to Conservation Area 3A and the traditional Indian activities allowed the Miccosukee Tribe is considered limited. Inadequate water will only be a problem during the dry season and droughts.