APPENDIX G

DRAFT ENGINEERING ANALYSIS AND NUMERICAL MODELING STUDY

EXECUTIVE SUMMARY

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SOUTHERN PALM BEACH ISLAND COMPREHENSIVE SHORELINE STABILIZATION PROJECT DRAFT ENGINEERING ANALYSIS AND NUMERICAL MODELING STUDY EXECUTIVE SUMMARY

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1.0 INTRODUCTION

The Town of Palm Beach and Palm Beach County (County) have each proposed shoreline stabilization projects that are adjacent to one another. These projects will require Department of the Army (DA) permits authorizing the discharge of dredge or fill material into waters of the United States (US), under Section 404 of the Clean Water Act (CWA). Accordingly, the United States Army Corps of Engineers (USACE) is evaluating the anticipated combined direct and indirect effects of both projects together through the preparation of an Environmental Impact Statement (EIS). After review of the data and previous work, the USACE has determined that numerical modeling and engineering analysis is required to obtain necessary data that is not currently available.

The Project Area for the Southern Palm Beach Island Comprehensive Shoreline Stabilization Project (the Project) comprises approximately 2.07 miles of shoreline and nearshore environment. The northern and southern limits are defined by Florida Department of Environmental Protection (FDEP) range monuments (R-monuments) R-129-210 (south end of Lake Worth Municipal Beach) and R-138+551 (south of the Eau Palm Beach Resort and Spa in Manalapan), respectively (Figure 1-1). For the purposes of the report, the Town of Palm Beach portion of the Project Area extends from R-129-210 to R-134+135. The Palm Beach County portion extends from R-134+135 to R-138+551 and is referenced to as the "County."

The Project Area's beaches provide storm protection to residential and public infrastructure, and serve as nesting areas for marine turtles. The area is characterized by a narrow beach with seawalls and dunes along its landward boundary and by ephemeral hardbottom formations in the nearshore. The active hurricane and tropical storm activity that occurred between 2004 and 2008 has resulted in a narrow, low profile beach along the majority of Project Area's shoreline. Over the past 8 years, the annual shoreline change has averaged a loss of 2.25 feet per year (CPE, 2013). Previous attempts to rebuild dunes in the Project Area have not resulted in a stable beach and dune system. The coastline within the Project Area and to the south has been designated by FDEP as "critically eroded" (FDEP, 2014). The applicant's proposed

Project under evaluation in the Environmental Impact Statement (EIS) intends to stabilize and widen the beach, thereby extending the construction interval between projects.

The following was assessed to obtain the additional data:

- Storm Protection: The SBEACH model was utilized to analyze the level of storm protection. The IH2VOF model was utilized to evaluate the amount of dune/seawall overtopping during storm events.

- Potential Hardbottom Impacts: The DELFT3D model was utilized to simulate the movement of sand within the littoral system in the vicinity of ephemeral hardbottom. The equilibrium toe of fill due to cross-shore spreading was evaluated based on analytic engineering analysis.

- Surfability: The BOUSS2D model was utilized to assess wave breaking and associated surfing conditions within and adjacent to the Project Area.

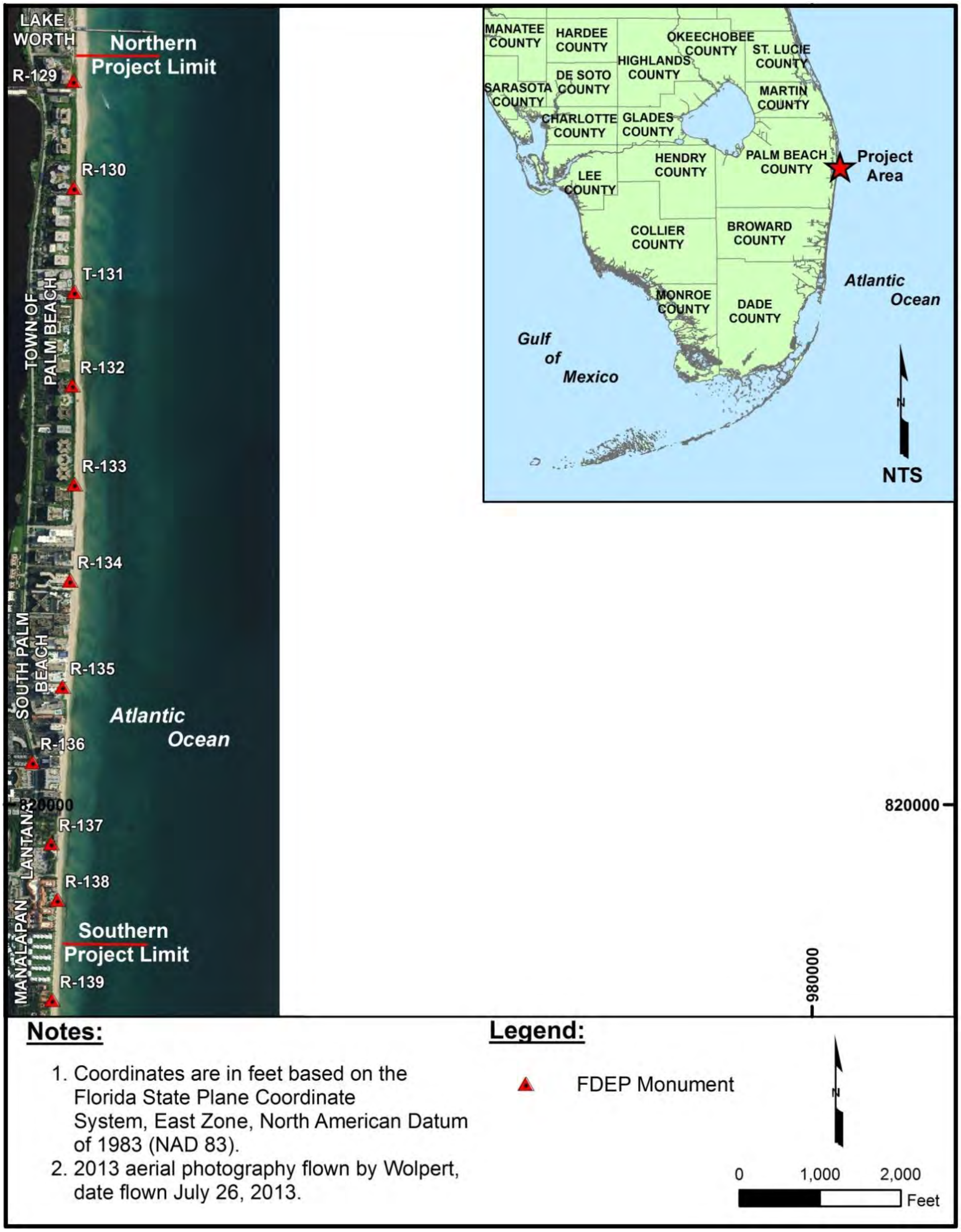

Figure 1-1. Southern Palm Beach Island Comprehensive Shoreline Stabilization Project Location.

2.0 DESCRIPTION OF ALTERNATIVES

Preliminary screening of alternatives was performed to identify a range of reasonable and practical alternatives to be considered for further evaluation. The screening process resulted in 6 alternatives to be considered. A seventh alternative, proposed by residents within the Town of Palm Beach, was also considered within the modeling effort.

- Alternative 1 is the No Action (Status Quo) alternative where the Applicants would continue the measures presently being implemented in the Project Area (R-129-210 to R-138+551) without any additional actions. No sand placement would occur below the mean high water and seasonal high tide line, nor would groins be constructed. However, the dunes may continue to be enhanced periodically through placement of small volumes of sand in portions of the Project Area. For the analysis, Alternative 1 was assumed to be the existing conditions, which were represented by the beach profile surveys collected the winter of 2011/2012. This alternative serves as the basis for which all other alternatives are compared to.

- Alternative 2 is the Applicants' Preferred Alternative (Proposed Action): Beach and Dune Fill with Shoreline Protection Structures. From north to south, the project would include placing sand to enhance the dune from R-129-210 to R-129+150, dune and beach berm from R-129+150 to R-131, dune from R-131 to R-134+135 (Town of Palm Beach southern limit), and beach berm from R-134+135 to R-138+551 (Figure 2-1). South of the Town of Palm Beach seven (7) low-profile groins were included from R-134+135 to R-138+551. The volume of fill required to fill the construction template within the Town of Palm Beach (R-129-210 to R-134+135) was estimated at 75,000 cy based on September 2009 beach profiles surveys (CSI, 2011). The fill volume required to fill the construction template within the County (R-134+135 to R-138+551) was estimated at 75,000 cy based on December 2008 beach profile surveys (CPE, 2011). This equated to a total fill volume of 150,000 cy. While maintaining the seaward berm crest location of the fill template, the volume of sand required to fill the template based on the winter 2011/2012 conditions was estimated at 117,300 cy (Table 2-1). The fill volume was further delineated above the high tide line (HTL = +2.6 feet, NAVD), between mean high water (MHW = +0.4 feet, NAVD) and the HTL, and below MHW. A schematic of the delineations is shown in Figure 2-1.

The footprint of the construction template was estimated at 24.3 acres based on the winter 2011/2012 beach profiles surveys (Table 2). Similar to the volume estimates, the acreages were further delineated landward of HTL, MHW to HTL, and seaward of MHW.

Table 2-1. Construction Template Fill Volumes – Alternative 2

	Template Volume (CY)					
	Estimated based on Winter 2011/2012 Survey				Prior Estimate	
Alternative 2	Above HTL[1]	MHW[2] to HTL[1]	Below MHW[2]	Total	Total	Survey Date
Town	34,500	9,300	10,000	53,800	75,000	Sept. 2009
County	33,200	10,800	19,500	63,500	75,000	Dec. 2008
Total	67,700	20,100	29,500	117,300	150,000	

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

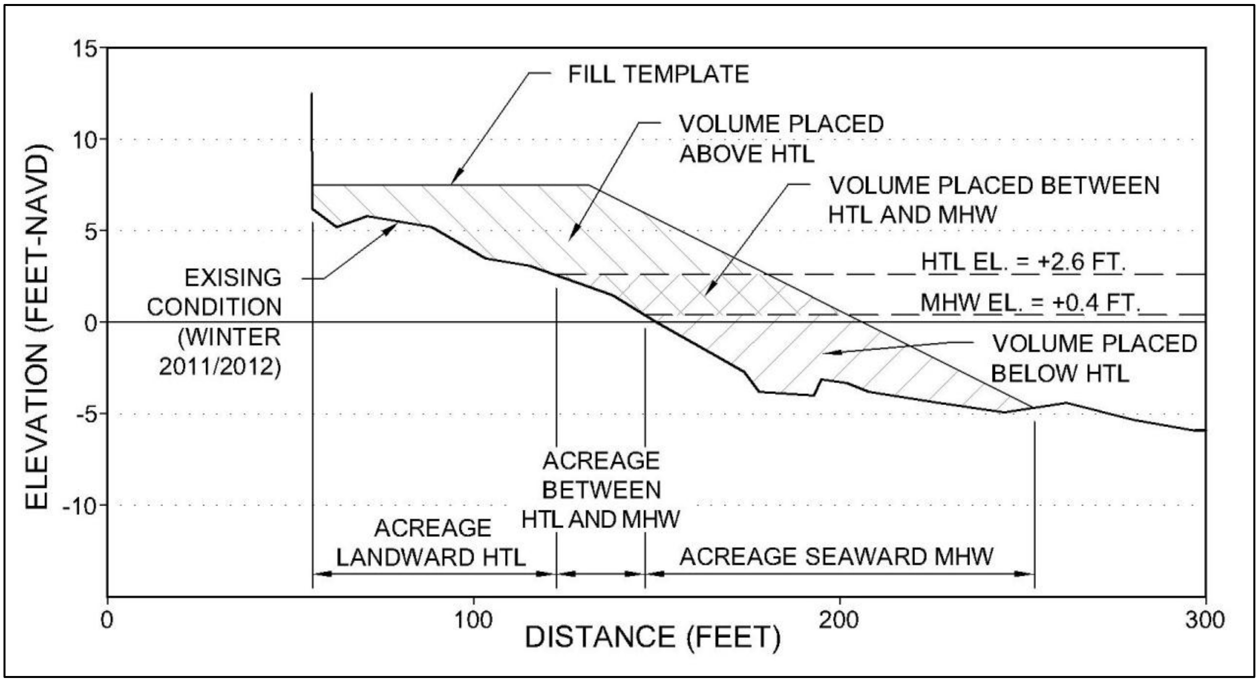

Figure 2-1. Schematic of Construction Volume and Acreage Estimates.

Table 2-2. Construction Template Acreages – Alternative 2

	Template Acerage (acres)			
Alternative 2	Landward HTL[1]	MHW[2] to HTL[1]	Seaward MHW[2]	Total
Town	7.2	1.7	3.1	12.0
County	3.9	2.4	6.0	12.3
Total	11.1	4.1	9.1	24.3

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

- Alternative 3 is the Applicants' Preferred Project (Proposed Action) without Shoreline Protection Structures. The template fill volumes and acreages are the same as those shown in Table 2-1 and Table 2.

- Alternative 4 is the Town of Palm Beach Preferred Project and County Increased Sand Volume Project without Shoreline Protection Structures. The alongshore extents of the fill defined by Alternative 2 are maintained. The sand volume within the County was increased by advancing the beach berm on average 50 feet seaward as compared to Alternative 2. The shoreline protection structures (groins) from Alternative 2 are not included. A breakdown of the construction template fill volumes and acreages are shown in Table 2-3 and Table 2-4, respectively.

Table 2-3. Construction Template Fill Volumes – Alternative 4

	Template Volume (CY)			
	Estimated based on Winter 2011/2012 Survey			
Alternative 4	Above HTL[1]	MHW[2] to HTL[1]	Below MHW[2]	Total
Town	34,500	9,300	10,000	53,800
County	78,700	27,600	65,800	172,100
Total	113,200	36,900	75,800	225,900

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

Table 2-4. Construction Template Acreages – Alternative 4

	Template Acerage (acres)			
Alternative 4	Landward HTL[1]	MHW[2] to HTL[1]	Seaward MHW[2]	Total
Town	7.2	1.7	3.1	12.0
County	3.9	2.6	12.5	19.0
Total	11.1	4.3	15.6	31.0

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

- Alternative 5 is the Town of Palm Beach Increased Sand Volume and County Preferred Project. The alongshore extents of the fill defined by Alternative 2 are maintained. The sand volume within the Town of Palm Beach was increased by advancing the dune and beach berm on average 10 feet seaward from R-129-210 to R-131 and the dune on average 50 feet seaward from R-131 to R-134+135 (Town of Palm Beach southern limit) as compared to Alternative 2. The shoreline protection structures (groins) from Alternative 2 are included. A breakdown of the construction template fill volumes and acreages are shown in Table 2-5 and Table 2-6, respectively.

Table 2-5. Construction Template Fill Volumes – Alternative 5

	Template Volume (CY)			
	Estimated based on Winter 2011/2012 Survey			
Alternative 5	Above HTL[1]	MHW[2] to HTL[1]	Below MHW[2]	Total
Town	75,100	11,700	14,100	100,900
County	33,200	10,800	19,500	63,500
Total	108,300	22,500	33,600	164,400

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

Table 2-6. Construction Template Acreages – Alternative 5

	Template Acerage (acres)			
Alternative 5	Landward HTL[1]	MHW[2] to HTL[1]	Seaward MHW[2]	Total
Town	9.5	3.1	0.3	12.9
County	3.9	2.4	6.0	12.3
Total	13.4	5.5	6.3	25.2

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

- Alternative 6 is the Town of Palm Beach Increased Sand Volume and County Increased Sand Volume without Shoreline Protection Structures. The alongshore extents of the fill defined by Alternative 2 are maintained. The volume was increased by advancing the dune and beach berm on average 10 feet seaward from R-129-210 to R-131, the dune on average 50 feet seaward from R-131 to R-134+135 (Town of Palm Beach southern limit), and the beach berm on average 50 feet seaward from R-134+135 to R-138+551 as compared to Alternative 2. The shoreline protection structures (groins) from Alternative 2 are not included. A breakdown of the construction template fill volumes and acreages are shown in Table 2-7 and Table 2-8, respectively.

Table 2-7. Construction Template Fill Volumes – Alternative 6

	Template Volume (CY)			
	Estimated based on Winter 2011/2012 Survey			
Alternative 6	Above HTL[1]	MHW[2] to HTL[1]	Below MHW[2]	Total
Town	75,100	11,700	14,100	100,900
County	78,700	27,600	65,800	172,100
Total	153,800	39,300	79,900	273,000

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

Table 2-8. Construction Template Acreages – Alternative 6

	Template Acerage (acres)			
Alternative 6	Landward HTL[1]	MHW[2] to HTL[1]	Seaward MHW[2]	Total
Town	9.5	3.1	0.3	12.9
County	3.9	2.6	12.5	19.0
Total	13.4	5.7	12.8	31.9

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

- Alternative 7 was based on a plan presented by The Coalition to Save Our Shoreline, Inc. (SOS). It consists of placement of sand within the Town of Palm Beach and shoreline protection structures (T-head groins). Two T-head groins were included between R-132 and R-134. The sand fill volumes required for the SOS plan are greater than the volumes for Alternative 6 within the Town of Palm Beach. The sand volume within the Town of Palm Beach was increased by advancing the dune on average 30 feet from R-129-210 to R-131, advancing the beach berm on average 70 feet seaward from R-129-210 to R-131, and including a beach berm with an average width of 135 feet from R-130 to R-134 as compared to Alternative 2. For the purpose of modeling, Alternative 7 was defined as the SOS plan north of R-134+135 and Alternative 2 to the south. The shoreline protection structures (groins) from Alternative 2 are included.

Table 2-9. Construction Template Fill Volumes – Alternative 7

	Template Volume (CY)			
	Estimated based on Winter 2011/2012 Survey			
Alternative 7	Above HTL[1]	MHW[2] to HTL[1]	Below MHW[2]	Total
Town	153,900	49,500	134,700	338,100
County	33,200	10,800	19,500	63,500
Total	187,100	60,300	154,200	401,600

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

Table 2-10. Construction Template Acreages – Alternative 7

	Template Acerage (acres)			
Alternative 7	Landward HTL[1]	MHW[2] to HTL[1]	Seaward MHW[2]	Total
Town	9.3	3.0	22.5	34.8
County	3.9	2.4	6.0	12.3
Total	13.2	5.4	28.5	47.1

[1]High tide line (HTL) defined at +2.6 feet, NAVD.

[2]Mean high water (MHW) defined at +0.4 feet, NAVD.

3.0 STORM PROTECTION

The coastline within the Project Area provides storm protection to upland property. The width and elevation of the beach and dune system and the presence of seawalls are factors that contribute to the storm protection afforded by the coastline.

3.1. SBEACH

The level of storm protection was analyzed using the Storm Induced Beach Change Model (SBEACH) (Larson and Kraus, 1989). The model results are detailed in Appendix G-1. The objectives of analysis were as follows:

- To verify the need for a project along all sections of the Project Area
- Determine the level of storm protection provided by the existing conditions
- Evaluate the range of storm protection associated with proposed fill alternatives

SBEACH simulates changes to beach (and dune) profile due to storm-driven erosion. Inputs to the model include the initial profile, the time histories of the waves and water levels during each storm, and a set of model calibration parameters. Changes to the beach and dune profiles were simulated for storms event with return periods of 5, 15, 25, 50, and 100 years. The level of storm protection afforded was defined by the storm return period that causes a 0.5 foot vertical loss at the landward limit of the beach. The impacts from the return period storm events were modeled for each of the following sceneries:

- Existing conditions (Alternative 1): The existing conditions were modeled to provide a baseline if no action was taken. The seawalls that existed at R-monuments were included in the model and assumed to not fail.

- Seawall failure: The existing conditions were modeled, but the existing seawalls were omitted to simulate the impacts associated with seawall failure during the storm events.

- Future scenarios without Project: The profiles from the existing conditions were translated landward based on background erosion rates to forecast future scenarios after 10 and 50 years. These scenarios assumed that no periodic sand placement would occur.

- Alternative 3 and Alternative 6: Alternatives 3 (Alternative 2 without shoreline protection structures) and 6 bracketed the level of protection that could be achieved as they included the smallest and greatest fill volumes, respectively.

Alternatives 2, 4, and 5 were not modeled as they were various combinations of fill volumes were bracketed by Alternatives 3 and 6. Alternative 7 was also not included as it was not being considered at the time the modeling was conducted. Groins were not included since SBEACH is a cross-shore model and shoreline protection structures (groins) oriented perpendicular to shore are not applicable.

The simulated conditions were identified to represent extreme storm events, but there is considerable variability among events that may be encountered. The following are the primary findings based on the results of the SBEACH modeling analysis:

- The critical return interval storm resulting in property damage under existing conditions is between a 15-year and 25-year storm. On average, 7.3 to 7.7 cubic yards per foot was simulated to erode from the beach above mean low water during a 15-year and 25-year storm, respectively. This volumetric loss coincides with a steepening of the dune face, shoreline retreat and lowering of the beach profile elevation. Based on 2011/2012 conditions, erosion and wave impacts were simulated to extend landward damaging infrastructure and maintained (landscaped) property areas at FDEP R-monuments R-130, R-133, R-135 and R-137. These locations lack seawalls or have seawalls located further landward on the property.
- Seawalls prevent erosion into the upland property until wall failure. Scouring at the toe of exposed seawalls increases their likelihood of failure. Based on the 2011/2012 conditions response to a storm event, the berm elevation adjacent to exposed seawalls will lower increasing the likelihood of seawall failure during storms. If seawall failure is assumed to occur along the Project Area, infrastructure would be impacted from R-130 through R-138. A detailed analysis of the structural stability of the individual seawalls along the Project Area would be necessary to truly assess the vulnerability of this critical component of storm protection infrastructure.
- Alternative 1 was the No Action (Status Quo) alternative, in which fill placement would occur periodically to enhance the dunes. This alternative was assumed to be represented by the existing conditions (winter 2011/2012). Based on erosion during the modeled storm events (above MLW) and background erosion rates, Alternative 1 is not sufficient to sustain the existing conditions. The majority of the placed fill would be lost during a single 15-year storm event or after 2 to 5 years of average wave climate period without major storms.

- Two future scenarios were simulated to represent the beach conditions after 10 and 50 years of erosion assuming that no periodic sand placement would occur. For both scenarios, all remaining storm protection provided by the dune between R-130 and R-134 would be lost after a single 15-year storm event. Seawalls that were buried within the dune would become exposed and subjected to wave action. The seawalls between R-136 and R-138 would possibly fail due to toe scour depending on the depth of the wall, allowing erosion of upland property and damage to infrastructure.

3.2. IH2VOF

The SBEACH model was utilized to analyze the level of storm protection that the existing conditions and alternatives provide to upland property. While erosion of the beach profile during these return period storm events is anticipated, the elevated water levels and large waves can cause additional damage if the dune and seawalls are overtopped. Overtopping water can cause flooding, erosion on landward (back) slopes, and seawall failure. The IH2VOF model was used to evaluate the amount of overtopping during the 15, 25, and 50 year return period storm events. The model results are detailed in Appendix G-2.

The IH2VOF model uses the "volume of fluids" approach and was run at two beach profile locations. Based on the winter 2011/2012 beach conditions, one location was characterized with a dune and no seawall (R-131), and the other with a seawall that was partially buried by a dune (R-137). The profiles at each location used in the model were the storm profiles generated during the SBEACH analysis. The profiles after being exposed to wave action and elevated water levels at the peak of the storm events were exported from the SBEACH model and imported into the IH2VOF model to provide a better representation of the overtopping that could be anticipated. At each location, the following were simulated.

- Existing conditions (Alternative 1)
- Alternative 2 (or Alternative 3)
- Alternative 6

Similar to the SBEACH model, the IH2VOF model is a cross-shore model. The storm erosion profiles from SBEACH are used as the input profile for IH2VOF. The shoreline protection structures (groins) proposed in the various alternatives were oriented perpendicular to shore and are not applicable in the model. The fill templates for Alternative 3 and Alternative 2 were the same, but Alternative 2 included structures. Alternatives 2 and 3 would require the same model inputs yielding the same model output. Alternatives 4 and 5 were not modeled as they were various combinations of fill volumes bracketed by Alternatives 2 and 6. Alternative 7 was not modeled as it was not being considered at the time of the SBEACH modeling.

The simulated conditions were identified to represent extreme storm events, but there is considerable variability among events that may be encountered. The following are the primary findings based on the results of the IH2VOF modeling analysis:

- The existing beach conditions are susceptible to wave overtopping during 15, 25 and 50 year return period storms. Overtopping increases as wave and water level conditions increase. This is attributed to the reduction in dry beach width and the dune crest (or seawall) height above the waves and water level.
- For the return period storms, the alternatives provide a reduction in overtopping and consequently an increase in storm protection as compared to the existing conditions.
    - At R-131, the overtopping during the 15 year storm was reduced up to 67% for the alternatives as compared to the existing conditions. Similarly, the overtopping during 25 and 50 year storms were reduced up to 75% and 58%, respectively.
    - At R-137, the larger fill volume associated with Alternative 6 provided greater storm protection by reducing overtopping as compared to Alternative 2. The incremental benefit of Alternative 6 was 50% less overtopping for the 15 year return period storm, 22% less for the 25 year storm, and 8% less for the 50 year storm as compared to the existing

conditions. Similarly, Alternative 2 provided 25% less overtopping for the 15 year storm, 0% for the 25 year storm, and 8% for the 50 year storm.

- Given the existing conditions, seawalls are subject to wave attack during storm events. These wave forces to which the seawalls are exposed increase with the intensity of the storm events. The exposure of seawalls to waves can cause damage thereby reducing the designed level of protection and/or increasing the frequency and need for structural repairs in order to maintain their integrity. Sand fill placed in front of the seawalls may offer additional protection.
- According to the USACE safety criteria, the mean overtopping discharge during the storm events is expected to cause some level of damage to the dune (or seawall) and create unsafe, dangerous situations for vehicles and pedestrians at the point of overtopping. Overtopping was not eliminated by having the alternatives in place. However, the alternatives did reduce overtopping, which would in turn reduce damage and unsafe, dangerous situations during storm events.

4.0 POTENTIAL HARDBOTTOM IMPACTS

The SBEACH and IH2VOF modeling analyzed the level of protection and evaluated the overtopping during storm events in order to identify the anticipated benefits of the additional fill volumes associated with the alternatives. The additional fill introduced into the littoral system will be transported offshore and alongshore over time as the sand is reworked by wave action. While the additional sand will create a wider beach increasing storm protection and benefiting nesting marine sea turtles, the reworked sand may be deposited offshore causing adverse impacts to ephemeral, nearshore hardbottoms.

The Project is proposed along a 2.07-mile segment of the Atlantic Ocean shoreline in the Towns of Palm Beach, South Palm Beach, Lantana, and Manalapan, in eastern Palm Beach County, Florida. The Project is located between Florida Department of Environmental Protection (FDEP) Range Monuments R-129-210 and R-138+551. The

Study Area is located between R-127 and R-141+586 and is characterized as a dynamic coastal marine system with a supra-littoral dune, beach, and intertidal beach with discontinuous nearshore hardbottom resources. The hardbottom is subject to periodic burial and exposure. Based on the most recent aerial photographs from March 2013, hardbottom was detected up to 700 feet from the shoreline.

The area has been the subject of more than 10-years of hardbottom mapping and analysis. Over the years, the data has been compiled and analyzed to differentiate the areas of ephemeral and persistent hardbottom exposure. Described here are the Delft3D coastal process model and analytical assessments using equilibrium profile theory that have been applied to assess potential impacts on hardbottom resources resulting from the proposed construction and equilibration of the fill.

4.1. DELFT3D

As part of a previous study conducted for Palm Beach County, a Delft3D numerical model (CPE, 2013) was developed, calibrated and applied to evaluate Project alternatives along the shoreline of South Palm Beach, Lantana and Manalapan. This setup was focused on the County project area and was expanded in order to evaluate the combined project area, with the Town of Palm Beach. The existing model was updated and recalibrated for use in evaluating the proposed actions and alternatives in the EIS and quantifying the estimating potential hardbottom coverage. The model results are detailed in Appendix G-3.

Hardbottom was incorporated into the Delft3D model by spatially varying the erodible sediment depth and sediment thickness based on physical measurements, survey data and aerial delineations. Erodible sediment depth is defined by an elevation fixed in time demarking the surface of the hardbottom such that erosion of sand cannot occur below this depth in the model.

The Delft3D model was then utilized to simulate the movement of sand within the littoral system and the results were used to quantify the potential impacts to the ephemeral hardbottom. Seven “combined” alternatives, six “separated” alternatives were modeled for a total of 13 simulations. The “combined” alternatives are defined in Section 2.0 and included both the Town of Palm Beach and County. The "separated" alternatives (2T, 2C, 3C, 6T, 6C, and 7T) were modeled individually to evaluate the effects/impacts attributable to the individual projects within the Town of Palm Beach and County. In the following Alternatives "C" refers to the County-only project and "T" refers to the Town of Palm Beach-only project:

- Alternative 1
- Alternative 2
    - Alternative 2T (The portion of Alternative 2 within the Town of Palm Beach)
    - Alternative 2C (The portion of Alternative 2 within the County)
- Alternative 3
    - Alternative 3C (The portion of Alternative 3 within the County)
- Alternative 4
- Alternative 5
- Alternative 6
    - Alternative 6T (The portion of Alternative 6 within the Town of Palm Beach)
    - Alternative 6C (The portion of Alternative 6 within the County)
- Alternative 7
    - Alternative 7T (The portion of Alternative 7 within the Town of Palm Beach)

Not all of the "combined" alternatives required "separated" alternatives as the fill volumes were captured by the combinations of the other "separated" alternatives.

The Delft3D morphological model from previous studies of Southern Palm Beach Island was recalibrated (updated) based on more recent erosion patterns and available data. The performance and impact of each alternative over a 3 year project life was then assessed using the updated calibrated model on an expanded model grid. The performance and impacts were assessed in terms of volume changes and erosion/sedimentation patterns at 1 year increments during simulation period. The following are the primary findings based on the Delft3D model results.

- Greater fill volumes result in increased sedimentation areas and net hardbottom coverage as the fill is redistributed cross shore and transported alongshore.

- Groins retain a portion of the sand that otherwise would be transported downdrift to adjacent beaches. The model indicated that with the same fill volumes the groins within the County for Alternative 2 (and Alternative 2C) resulted in greater sedimentation offshore of the groin field as compared to Alternative 3 (and Alternative 3C), but with less downdrift sedimentation. This is attributed to a greater volume of sand being retained within the groin field and being redistributed cross shore as opposed to alongshore in the absence of the groins. The net hardbottom coverage was less for Alternative 2 (and Alternative 2C) as compared to Alternative 3C.

- When comparing the “combined” and “separated” alternatives for Alternative 6, the fill placed south of R-134+135 within the County spreads north resulting in increased sedimentation within the Town of Palm Beach.

- When comparing the “combined” and “separated” alternatives for Alternative 7, the fill placed north of R-134+135 within the Town of Palm Beach is transported south resulting in increased sedimentation within the County.

- Alternative 2 resulted in the least area of sedimentation and net hardbottom coverage as compared to the other combined alternatives.

4.2. Analytical Equilibrium Toe of Fill

The volumes of fill required to construct the alternatives were estimated based on the condition of the beach as surveyed in 2011/2012. The beach conditions were represented by profile surveys spaced approximately 1,000 feet alongshore at the FDEP R-Monuments within the Project Area (R-129 through R-137). After fill placement, it is anticipated that the constructed profile would equilibrate due to natural coastal processes adjusting back to the shape of the pre-construction profile. However, the cross-shore extent of this equilibration process is limited by the low density fill placements and strong alongshore current that exists in the Project Area.

The resulting equilibrium profiles were developed by translating the pre-construction profiles using the method described in the Coastal Engineering Manual, Part 5, Chapter 4. The profile translation theory conserves volume by redistributing the fill cross-shore to an estimated depth of closure (DOC). Sediment transport beyond the annual DOC is assumed to be insignificant. The DOC for the Project Area was defined to be -19.9 feet, NGVD, consistent with previous studies (FDEP, 2012). Application of this method results in the equilibrium toe of fill (ETOF) coinciding with the DOC in all locations. Consequently, the ETOF would encompass a vast majority of the ephemeral hardbottom, independent of the volume of fill placed at a given profile.

Considering the relatively low density of fill proposed, the analysis was further evaluated to account for the alongshore variability of the fill placement in determining an adjusted ETOF. At each FDEP R-Monument, the equilibrium profile was compared to the pre-construction profile. To determine the cross-shore location beyond which the profile variability could be considered insignificant, the vertical change between existing and translated profiles was evaluated.

Each profile was divided into 100-foot cross-shore increments to a point 2,300 feet seaward of the monument for vertical change assessment. The volumetric change within each increment was estimated and the average vertical difference between the profiles was determined. The equilibrium profile was determined to close with the preconstruction profile at the cross-shore location where the profiles varied by ≤0.25

feet. The tolerance of 0.25 feet was selected in the analysis as a fraction of typical survey error, which is on the order of +/-0.4 feet. Each equilibrium profile was then adjusted to ensure that the fill volume was conserved resulting in the final ETOF.

It is noted that cross-shore fill equilibration is not instantaneous as the theory suggests because sand migrates alongshore due to background erosion and littoral transport. Therefore, the reasonably anticipated extent of hardbottom impacts account for the analytical estimation of the ETOF and the Delft3D model results described above.

4.3. Calculation of Hardbottom Coverage

Hardbottom exposure along the Project Area and the adjacent beaches varies widely over time with hardbottom being covered and uncovered due to natural processes (CPE 2007a, pp. 44-45). The natural variability in hardbottom exposure is one of the primary reasons that Palm Beach County and the Town of Palm Beach compile hardbottom mapping information on a frequent basis.

Potential coverage of hardbottom can be readily assessed based on the erosion and deposition patterns that occur in the model. By comparing the output of each alternative to the baseline (No Action) condition, a resulting sediment thickness greater than 0.2 feet was selected as threshold to define areas of sedimentation based on reasonable model capabilities. Overlaying these areas on mapped hardbottom locations provides an estimate of the potential impact for each alternative.

The simulated areas of hardbottom coverage comparing the No Action scenario to the alternatives are well within the natural variability of hardbottom observed in the last 10-15 years. Additionally, the simulated sediment erosion and deposition patterns compare well to the various hardbottom delineations demonstrating the model's ability to represent the ephemeral nature of the hardbottom. The results indicate that the model reasonably simulates the movement of sand in the vicinity of hardbottom features and provides a consistent method for assessing impacts.

Impacts are quantified spatially as acres of potential impacted hardbottom estimated by comparing results of the different scenarios. The model output is processed as polygons of sedimentation that are then overlaid on various hardbottom delineations in a GIS platform. This allows the sedimentation results to be analyzed with several years of hardbottom exposure, which are then used to determine annual and time-averaged impacts. This assessment method includes the area of hardbottom coverage within the Project Area and areas north and south of the Project fill limits quantified in acres of time averaged exposure.

Based on the methodology presented in the EIS, acres of hardbottom impact will be quantified as the time-averaged exposed hardbottom over the last 10 years (2003-2013). The delineations of hardbottom will be compared to the model sedimentation results and analytical ETOF to estimate the direct and temporarily impacted acreage for the Project Area and the adjacent areas. These estimates will be used to develop the inputs for the UMAM assessments that will determine the potential mitigation requirements for the Project alternatives. Please refer to Appendix H for the details of this analysis.

5.0 SURFABILITY

Concern regarding potential impacts to surfing has been expressed in the public scoping meeting for the proposed Project. In order to evaluate Project-related effects on surfing, the BOUSS-2D model was used in this study to simulate breaking waves within the Project Area. BOUSS-2D model was developed by the U.S. Army Corps of Engineers (Nwogu and Demirbilek, 2001) and utilized through the Surface Water Modeling System (SMS) interface (Aquaveo, 2008). The model results are detailed in Appendix G-4.

To assess the potential impacts on surfability within the Study Area (R-127 to R-141+586), resulting bathymetries from the 3 year simulation period with the Delft3D model were exported and imported into the BOUSS2D model. These bathymetries were the basis for evaluating the impacts to surfability within the Project Area and adjacent areas. The alternatives that were considered in the analysis included:

- Alternative 1
- Alternative 2
- Alternative 6
- Alternative 7

The remainder of the alternatives (Alternatives 3, 4, and 5) were not included in the analysis. They consisted of various combinations of the sand fill volumes and shoreline protection structures comprising Alternatives 2 and 6.

In particular, the surfability was evaluated at two popular southern Palm Beach surf spots, Lantana Park and the Lake Worth Pier. Three wave conditions: (i) southeast, (ii) cold front and (iii) hurricane (pre-landfall), were used to replicate the range of surfing conditions experienced at the two locations. The significant wave height for existing conditions (Alternative 1) was analyzed as well as the relative differences (%) between existing conditions and the other alternatives. In addition, the main parameters to assess surfability (Iribarren number ξ_b, peel angle, velocity of wave, peel rate and velocity of surfer) were compared to evaluate the quality of wave for surfing. The following are the primary findings based on the model results:

- The minimum skill level required of surfers to surf at the two locations was rated at 5 (out of 10), representing an intermediate skill level.
- Differences of significant wave heights (Hs) between existing and alternatives scenarios were more noticeable for alternatives with higher amount of sediment placement.
- A decrease of wave height was observed near the beach for all alternatives. This decrease would not impact surfing directly since it happened after wave breaking and was landward of optimal surfing areas.
- The wave condition that showed more impact from the alternatives was the southeast waves (smaller waves with smaller periods as compared to the other

wave conditions). Under the southeast wave condition, the waves would break close to the beach where the differences in bathymetry (between alternatives and existing) are higher. For hurricane and cold front wave conditions (higher waves with higher periods) the waves would break offshore where the bathymetry presents little or no differences between existing and alternatives.

- An increase of wave height before wave breaking is observed for southeast waves conditions in Alternative 7. This wave height increase is noticed due to the combination of the wave condition used in the model and the bathymetry of Alternative 7. The southeast wave condition represents the smallest simulated wave height and period and Alternative 7 presents the highest amount of sediment placed.

- Although there were small variations in the Iribarren number, there were no changes in the breaker wave type for the alternatives.

- In general, a small variation in peel angle, peel rate and velocity of surfer was observed in the wave conditions for the different alternatives. The changes in the surfability at the two locations due to the alternatives were also small.

6.0 LITERATURE CITED

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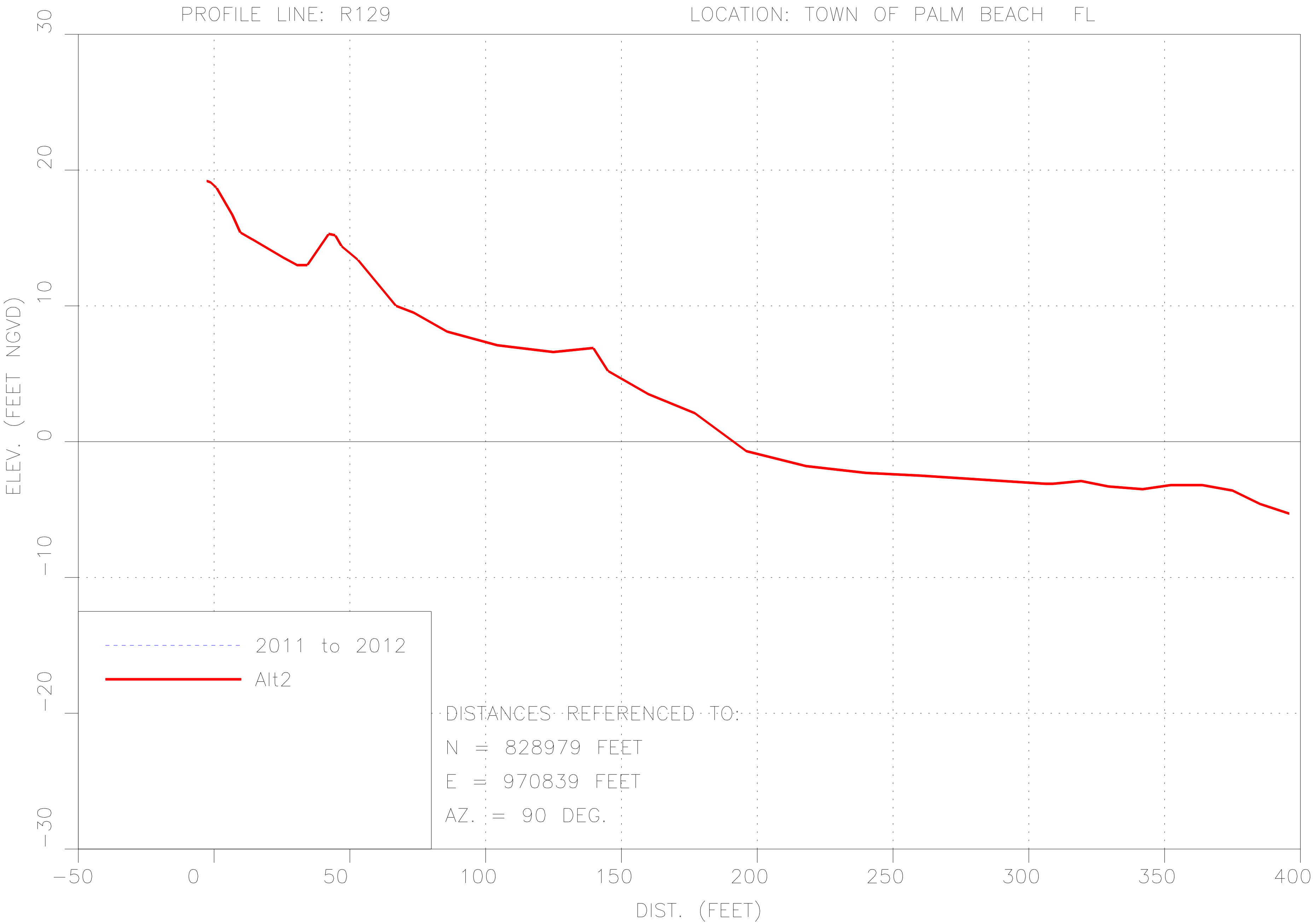
Fowler, J.E., 1993. Coastal Scour Problems and Methods for Prediction of Maximum Scour. Technical Report CHL 93-8, U.S. Army Corps of Engineers Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

SUB-APPENDIX G-1

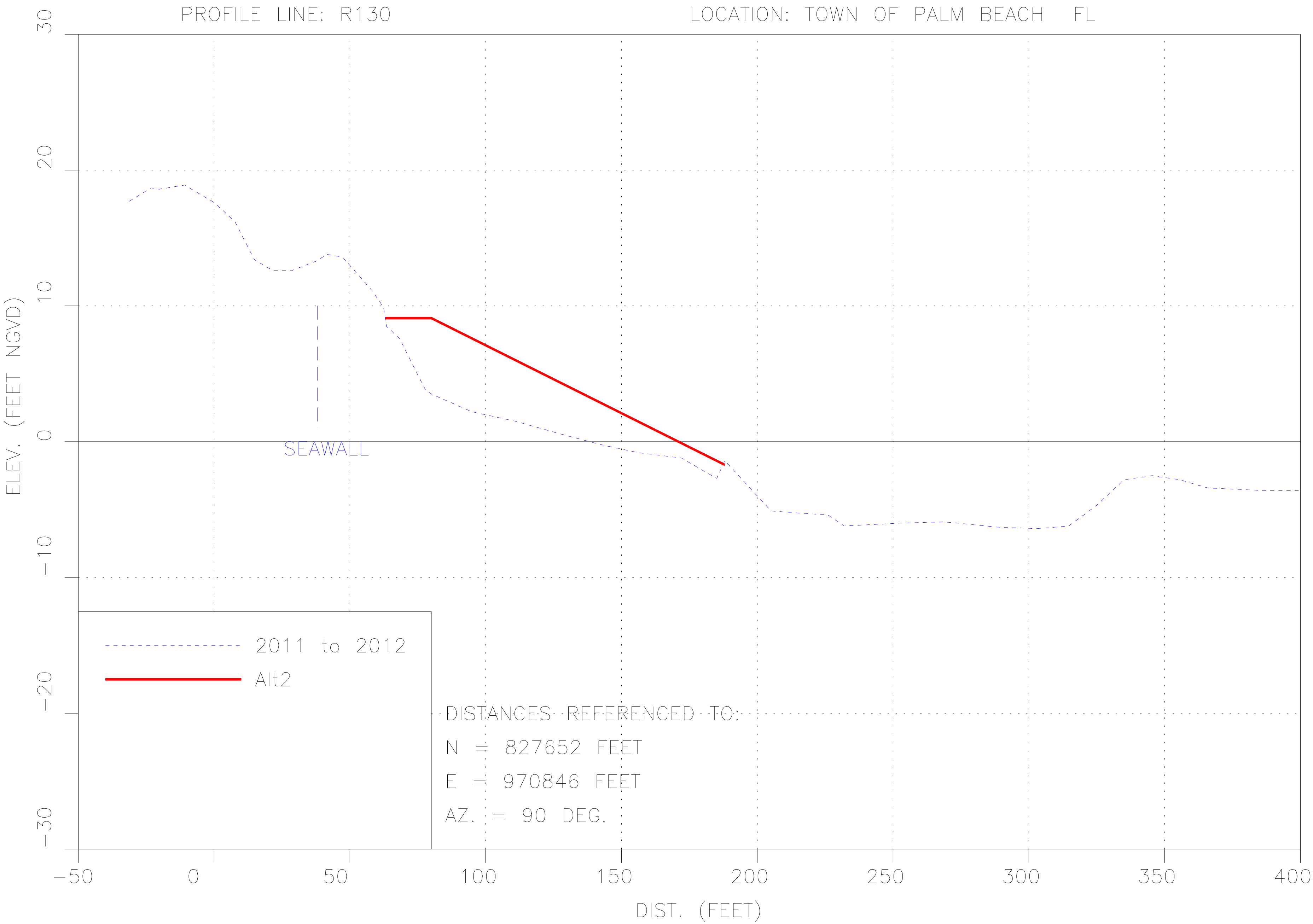
SBEACH ANALYSIS REPORT

ATTACHMENT A

PROSPECTIVE DESIGN PROFILES



PROFILE LINE: R129
LOCATION: TOWN OF PALM BEACH FL
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
N = 828979 FEET
E = 970839 FEET
AZ. = 90 DEG.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LOCATION: TOWN OF PALM BEACH FL
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20
10
0
-10
-20
-30
ELEV. (FEET NGVD)
SEAWALL
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
N = 827652 FEET
E = 970846 FEET
AZ. = 90 DEG.
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100
150
200
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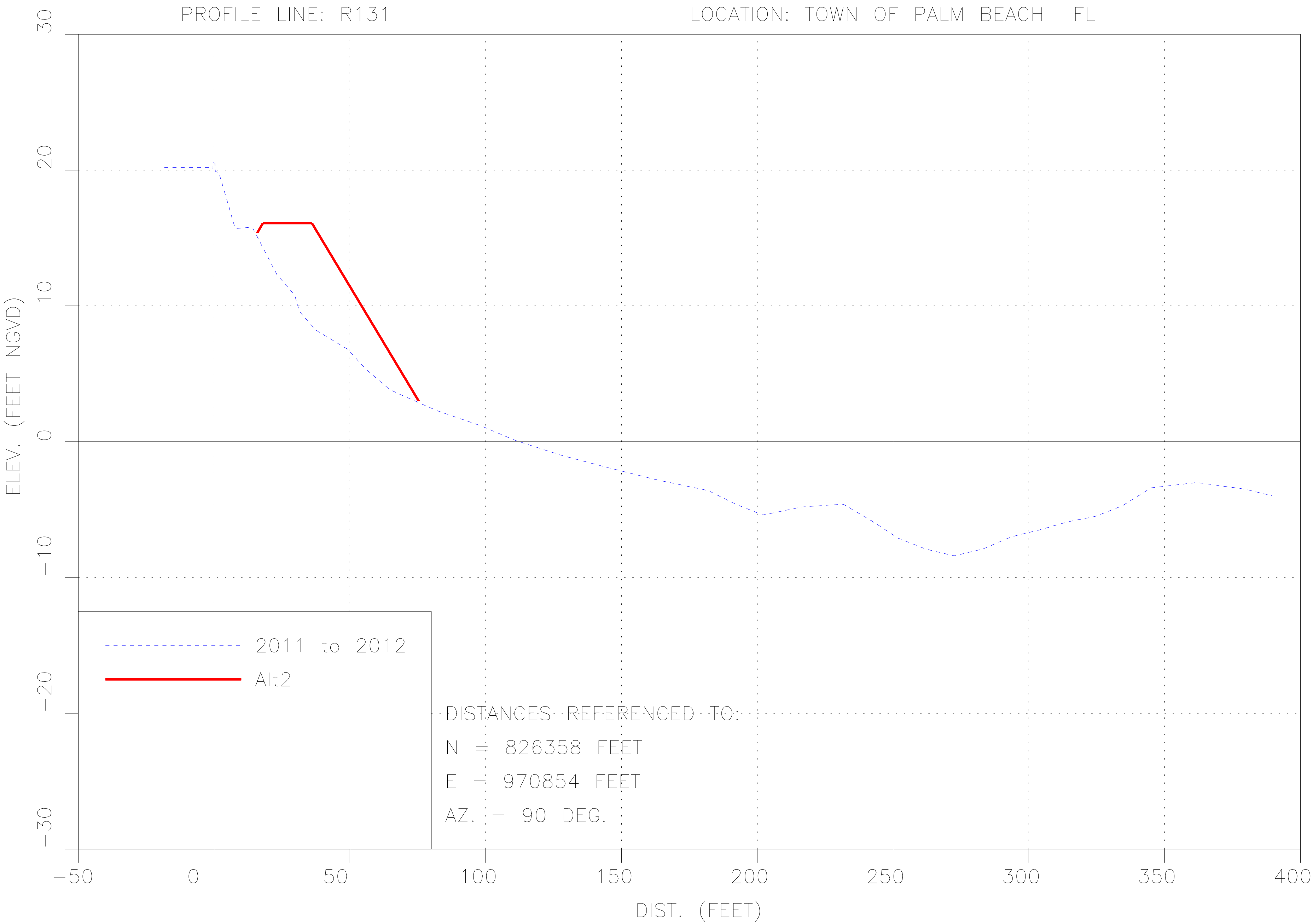
300
350
400
DIST. (FEET)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ELEV. (FEET NGVD)
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20
10
0
−10
−20
−30
DIST. (FEET)
−50
0
50
100
150
200
250
300
350
400
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
N = 826358 FEET
E = 970854 FEET
AZ. = 90 DEG.


PROFILE LINE: R132
LOCATION: TOWN OF PALM BEACH FL
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
SEAWALL
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
N = 825200 FEET
E = 970827 FEET
AZ. = 95 DEG.
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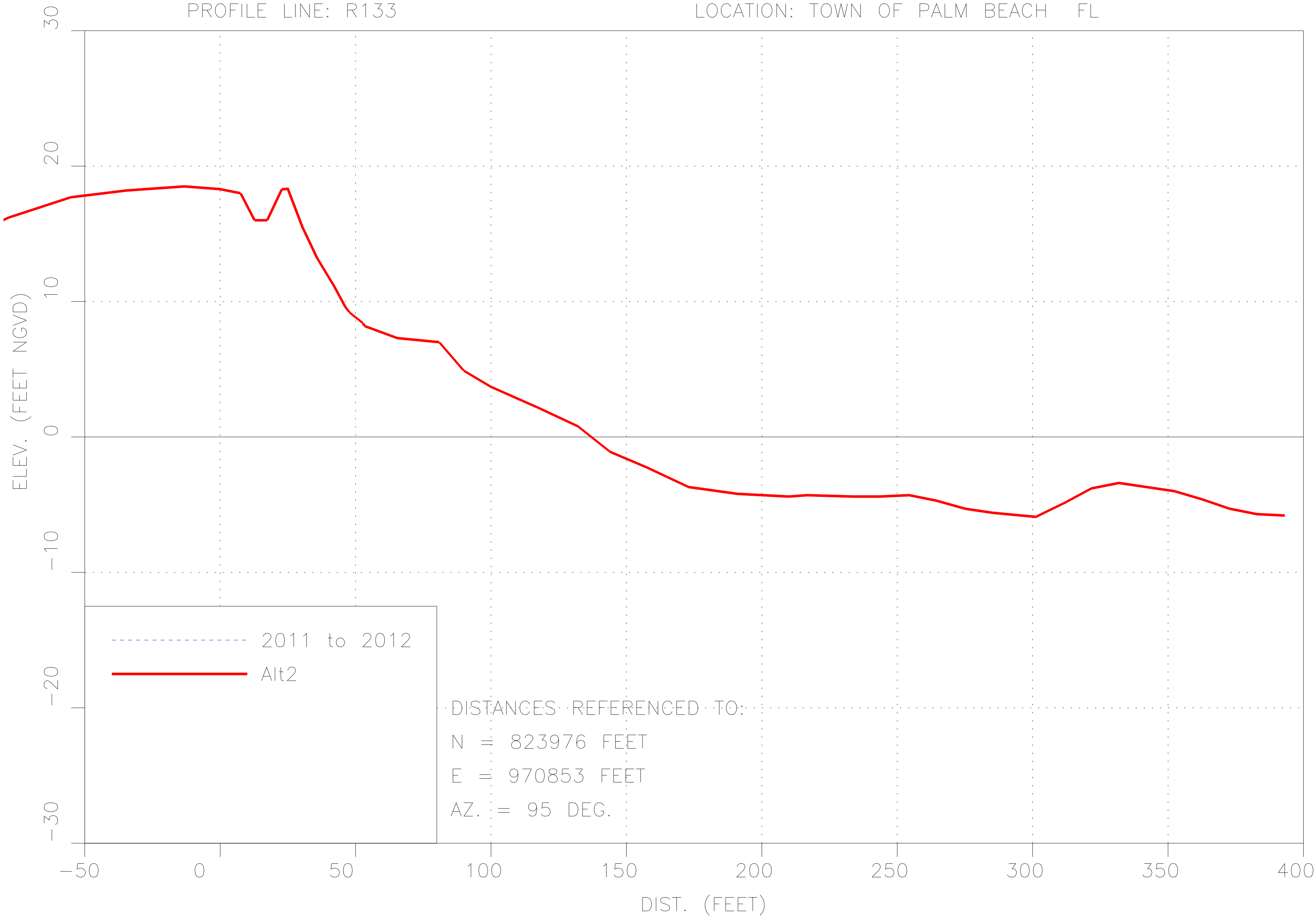
0
50
100
150
200
250
300
350
400
DIST. (FEET)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ELEV. (FEET NGVD)
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20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
2011 to 2012
Alt2
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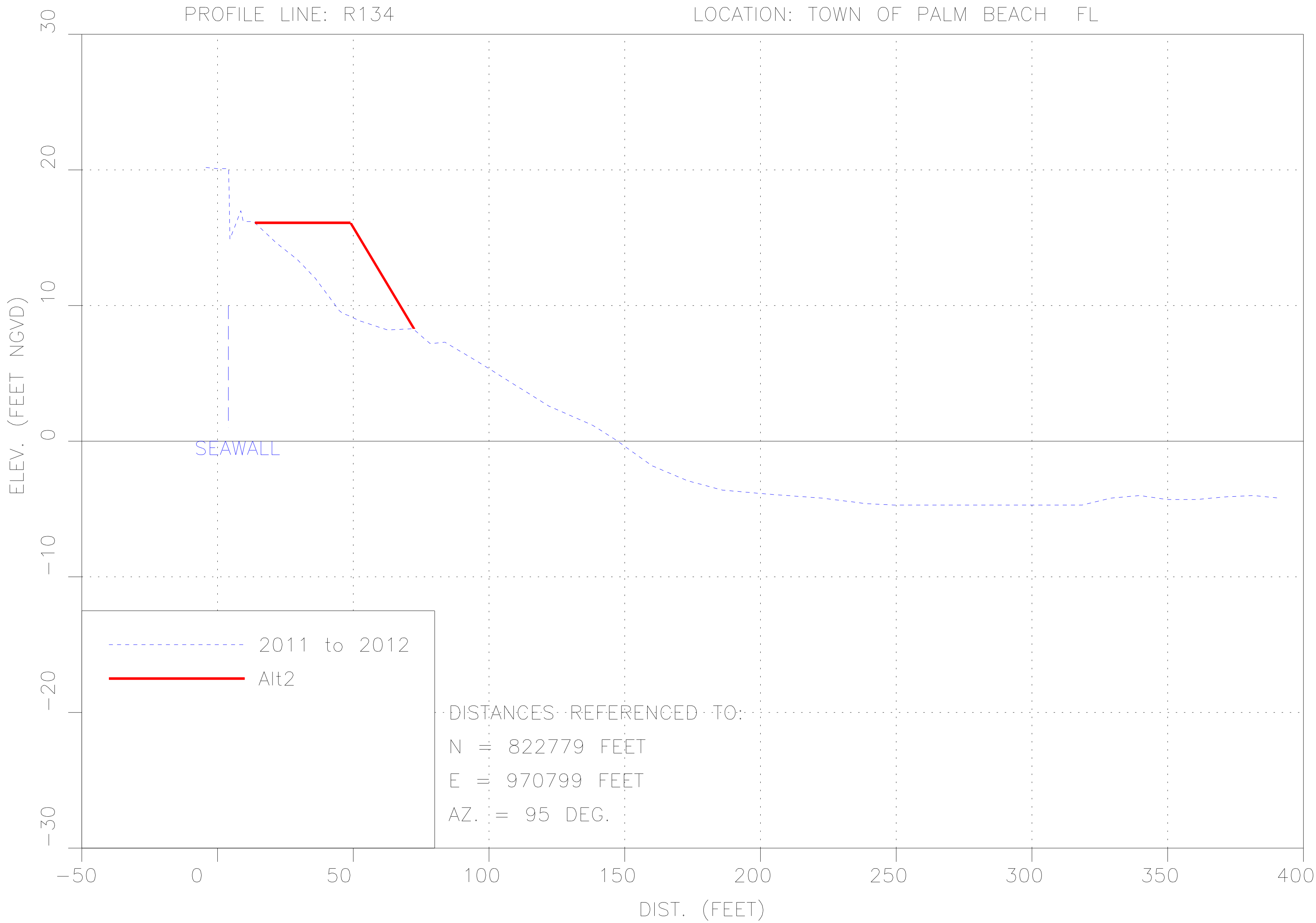
AZ. = 95 DEG.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LOCATION: TOWN OF PALM BEACH FL
ELEV. (FEET NGVD)
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20
10
0
−10
−20
−30
SEAWALL
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
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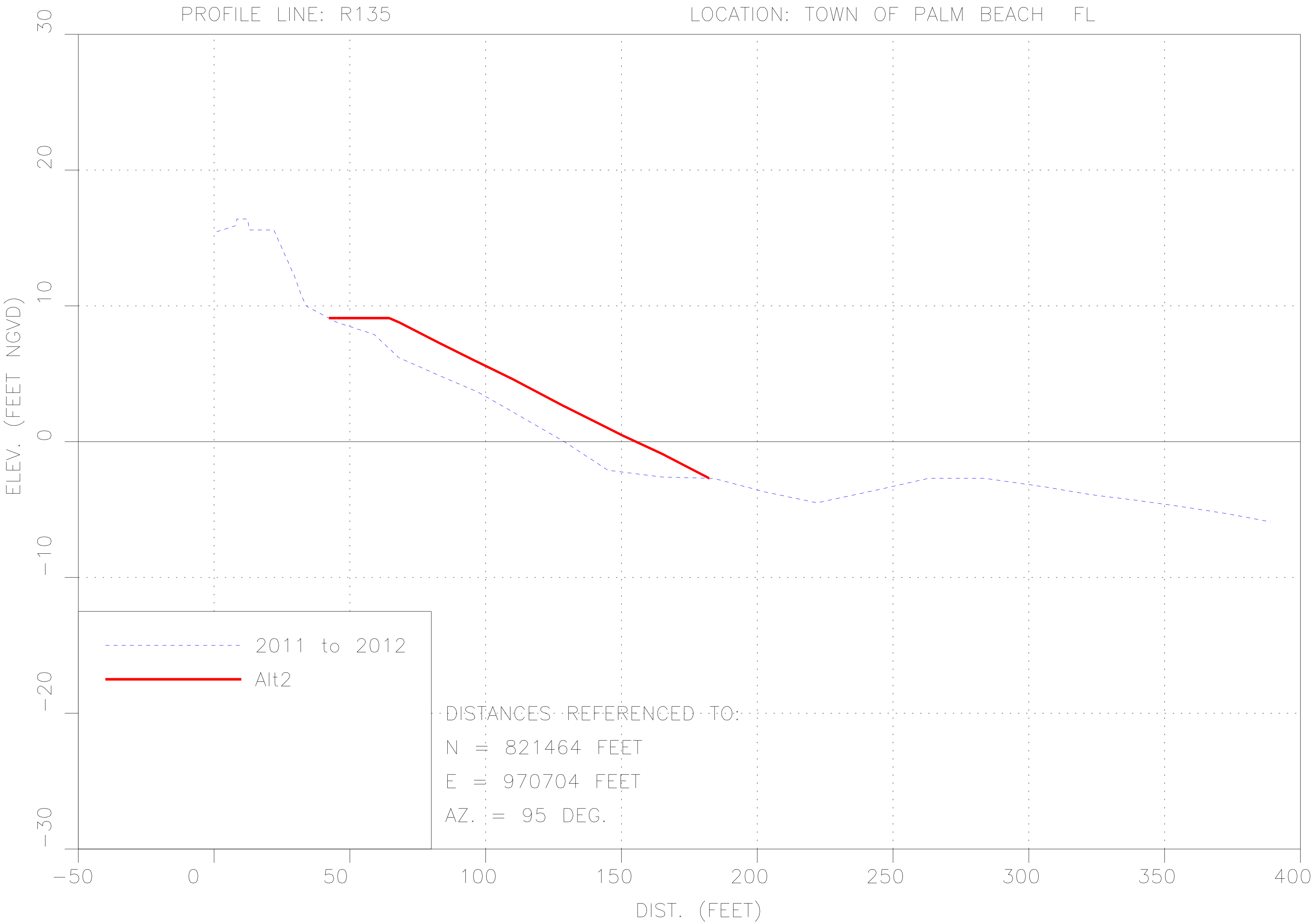
50
100
150
200
250
300
350
400
DIST. (FEET)



PROFILE LINE: R135
LOCATION: TOWN OF PALM BEACH FL
ELEV. (FEET NGVD)
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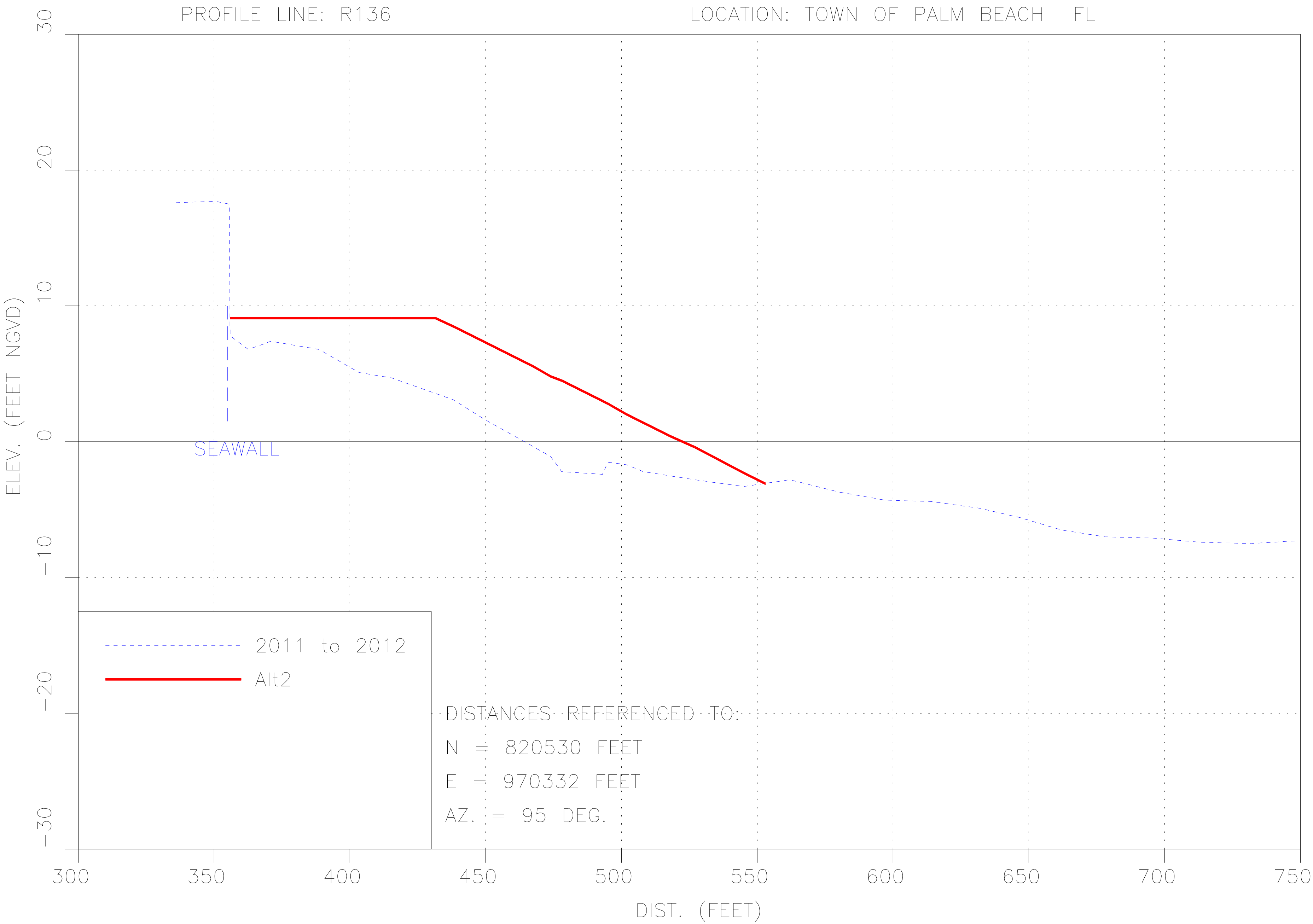
20
10
0
−10
−20
−30
DIST. (FEET)
−50
0
50
100
150
200
250
300
350
400
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
N = 821464 FEET
E = 970704 FEET
AZ. = 95 DEG.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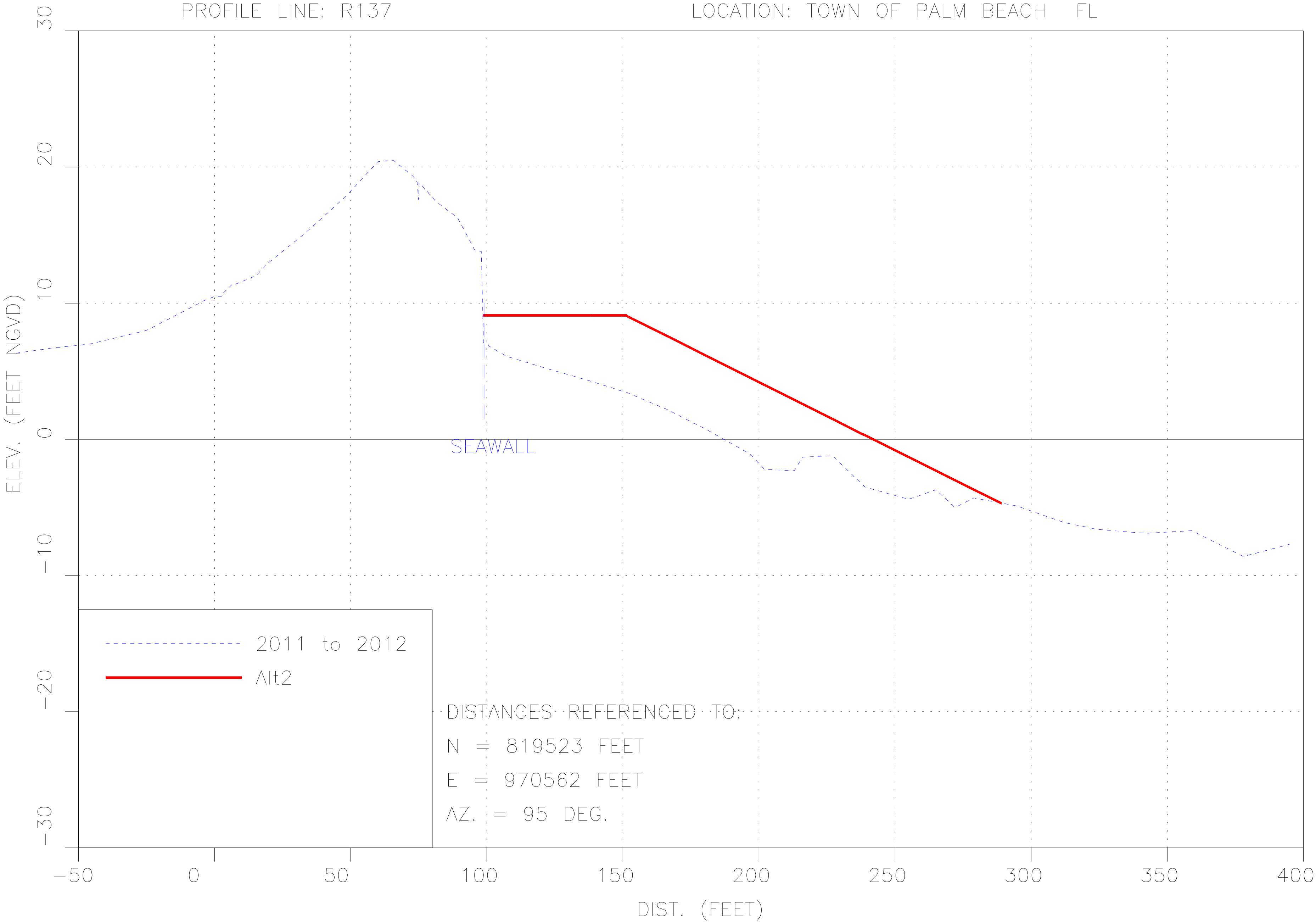
ELEV. (FEET NGVD)
30
20
10
0
-10
-20
-30
SEAWALL
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
N = 820530 FEET
E = 970332 FEET
AZ. = 95 DEG.
300
350
400
450
500
550
600
650
700
750
DIST. (FEET)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ELEV. (FEET NGVD)
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20
10
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−10
−20
−30
SEAWALL
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
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0
50
100
150
200
250
300
350
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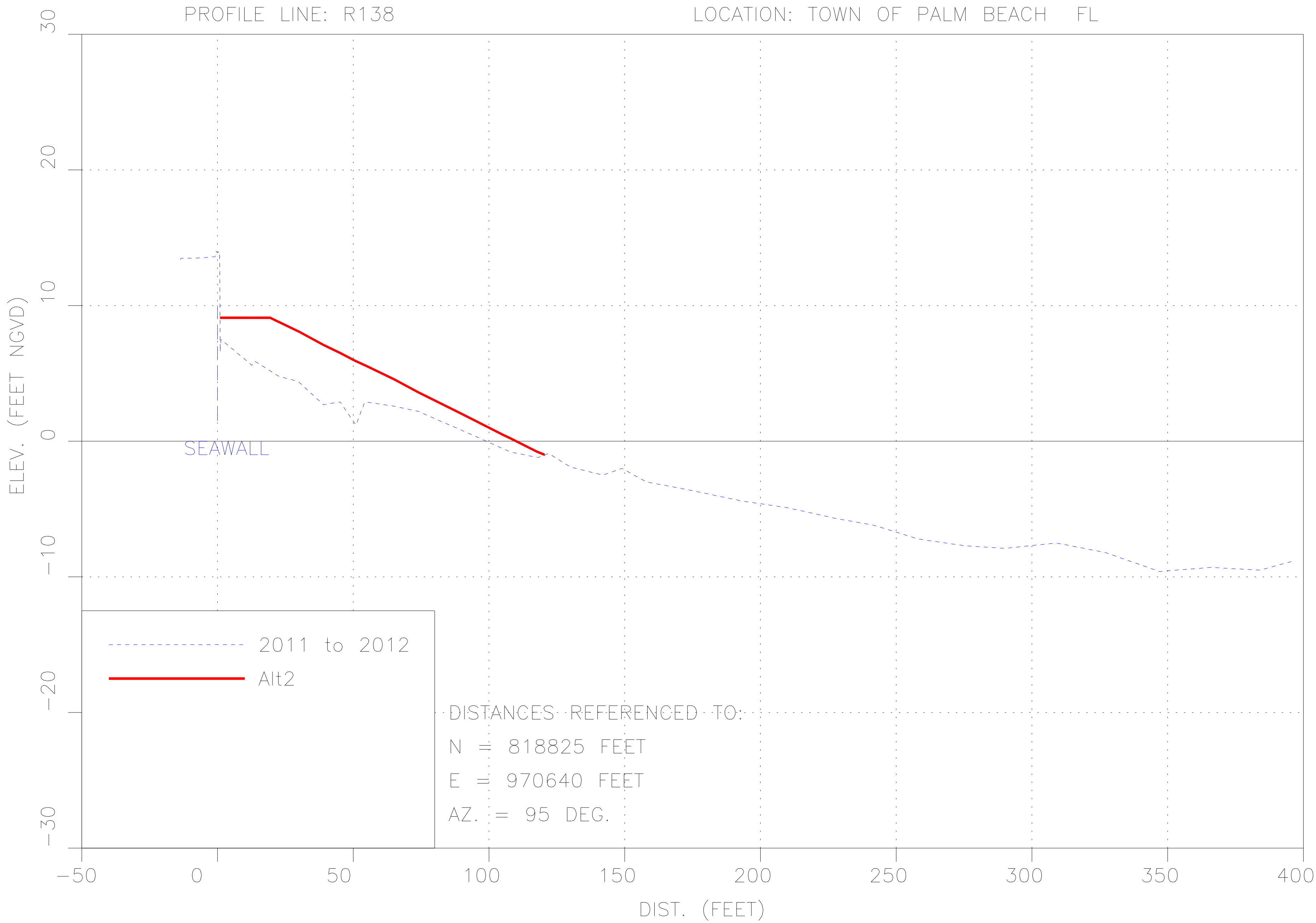
DIST. (FEET)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-10
-20
-30
ELEV. (FEET NGVD)
SEAWALL
2011 to 2012
Alt2
DISTANCES REFERENCED TO:
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150
200
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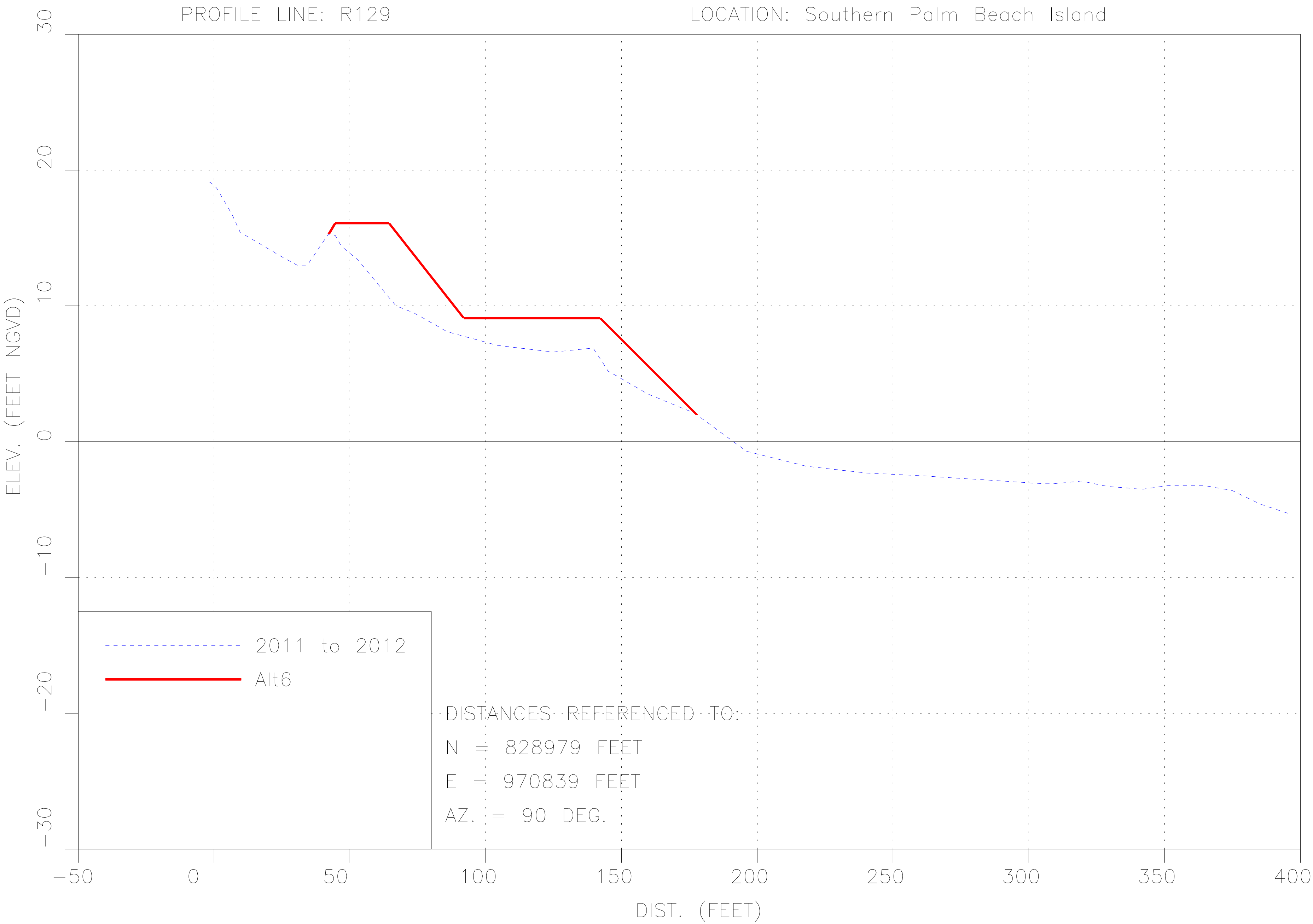
DIST. (FEET)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ELEV. (FEET NGVD)
DIST. (FEET)
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20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 828979 FEET
E = 970839 FEET
AZ. = 90 DEG.


PROFILE LINE: R130
LOCATION: Southern Palm Beach Island
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
SEAWALL
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 827652 FEET
E = 970846 FEET
AZ. = 90 DEG.
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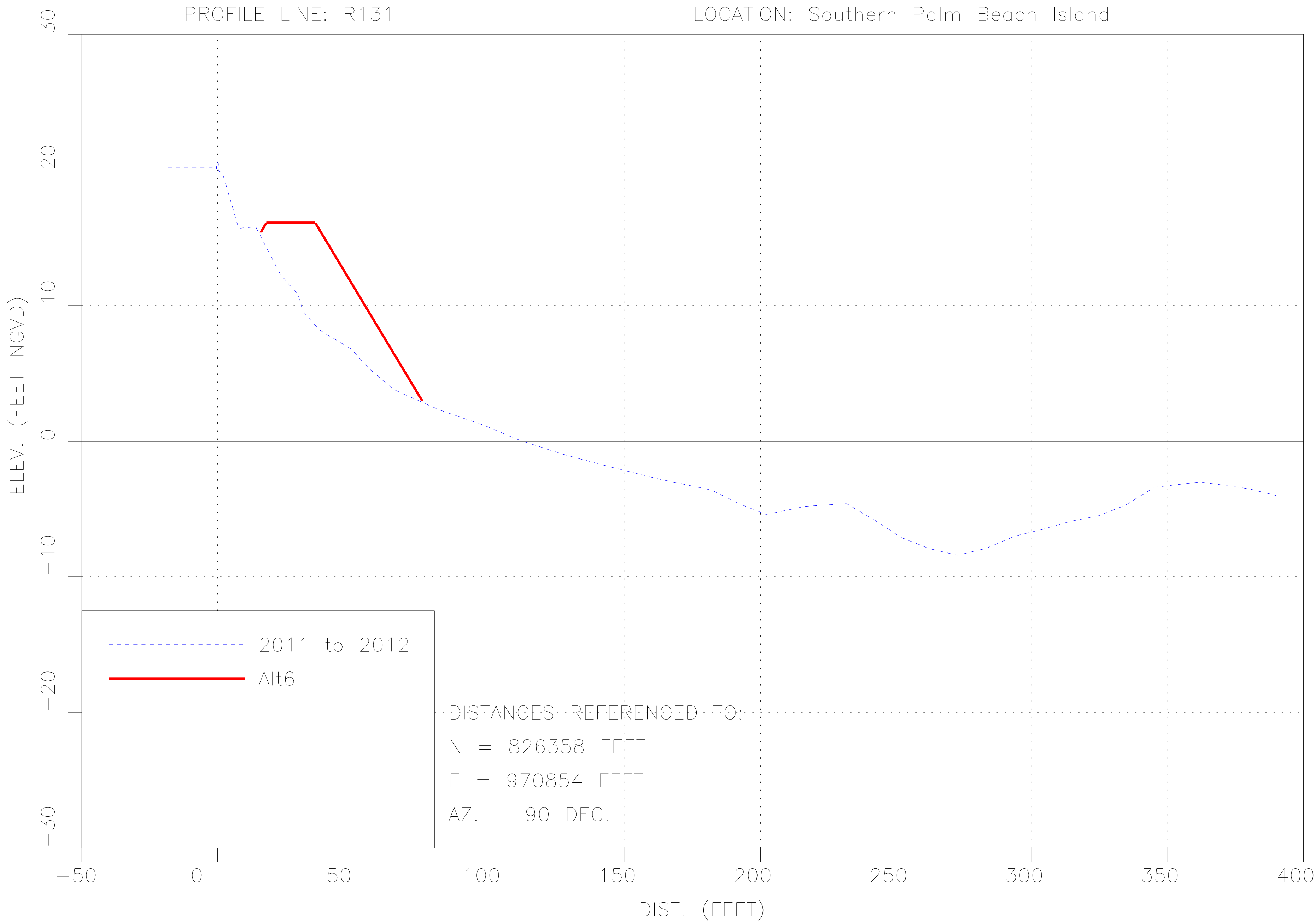
0
50
100
150
200
250
300
350
400
DIST. (FEET)



PROFILE LINE: R131
LOCATION: Southern Palm Beach Island
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20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 826358 FEET
E = 970854 FEET
AZ. = 90 DEG.

PROFILE LINE: R132
LOCATION: Southern Palm Beach Island
ELEV. (FEET NGVD)
30
20
10
0
-10
-20
-30
SEAWALL
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 825200 FEET
E = 970827 FEET
AZ. = 95 DEG.
-50
0
50
100
150
200
250
300
350
400
DIST. (FEET)

PROFILE LINE: R133
LOCATION: Southern Palm Beach Island
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 823976 FEET
E = 970853 FEET
AZ. = 95 DEG.

PROFILE LINE: R134
LOCATION: Southern Palm Beach Island
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
SEAWALL
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 822779 FEET
E = 970799 FEET
AZ. = 95 DEG.
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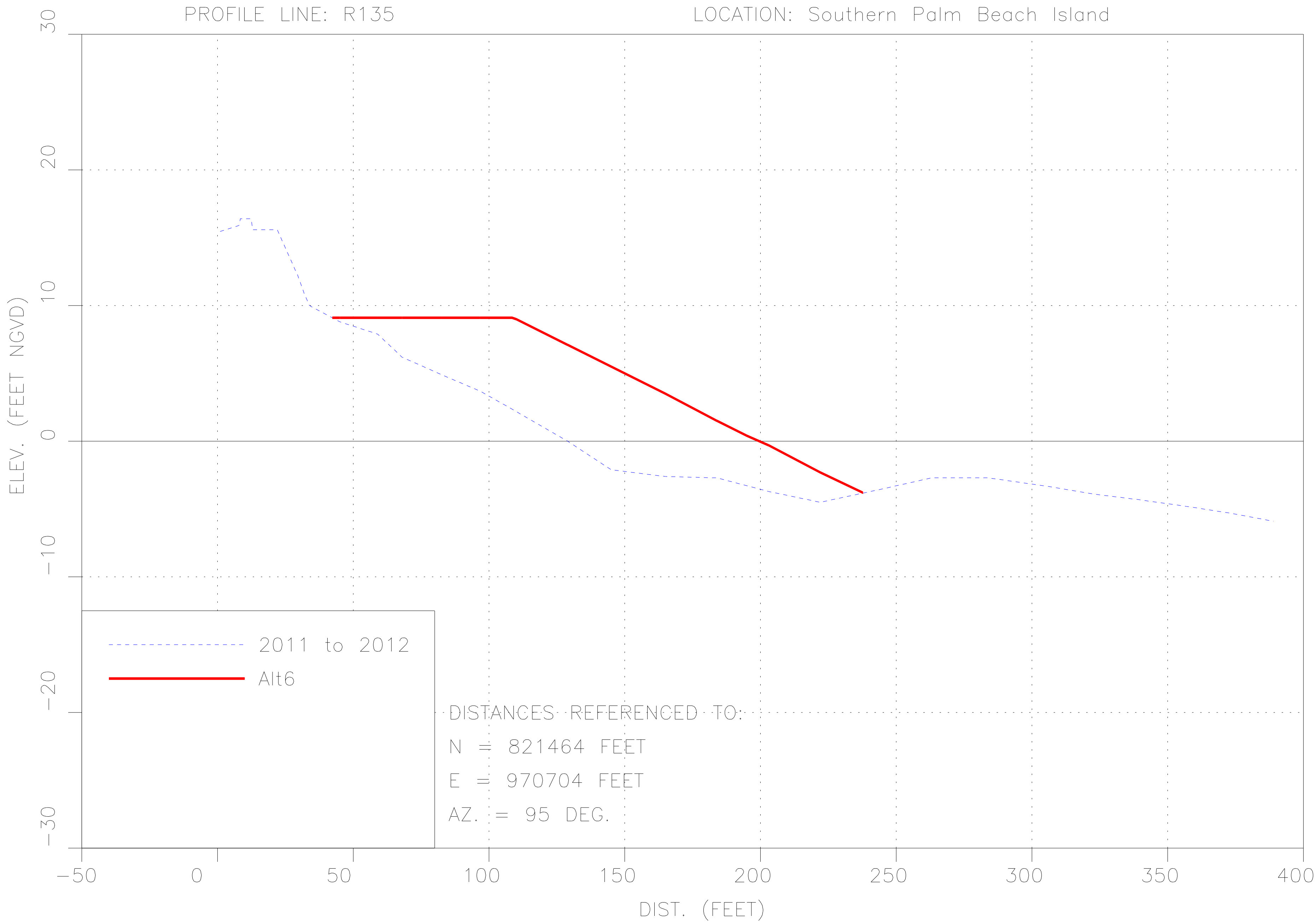
DIST. (FEET)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−30
ELEV. (FEET NGVD)
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0
50
100
150
200
250
300
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DIST. (FEET)
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
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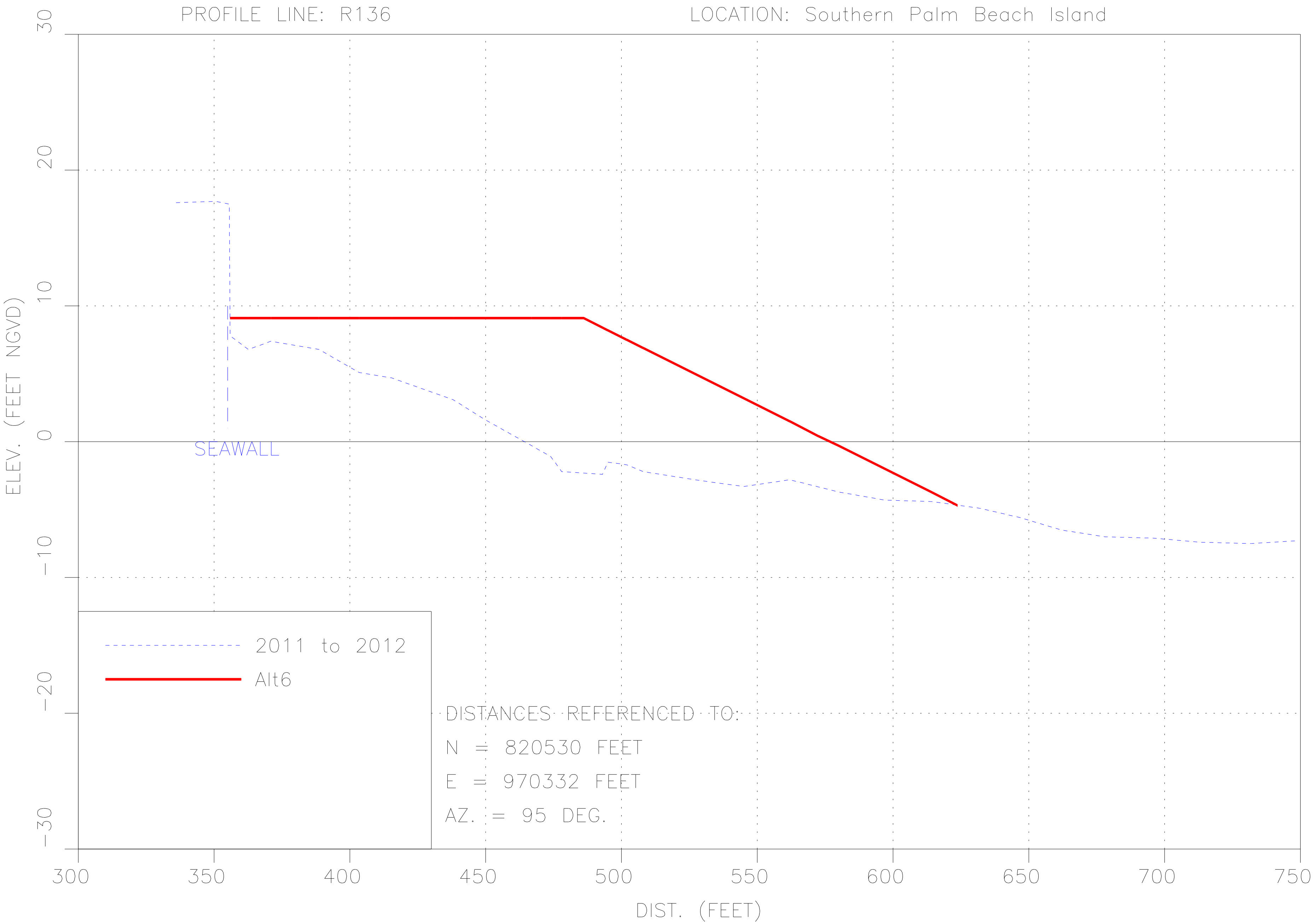
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ELEV. (FEET NGVD)
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-30
SEAWALL
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 820530 FEET
E = 970332 FEET
AZ. = 95 DEG.
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550
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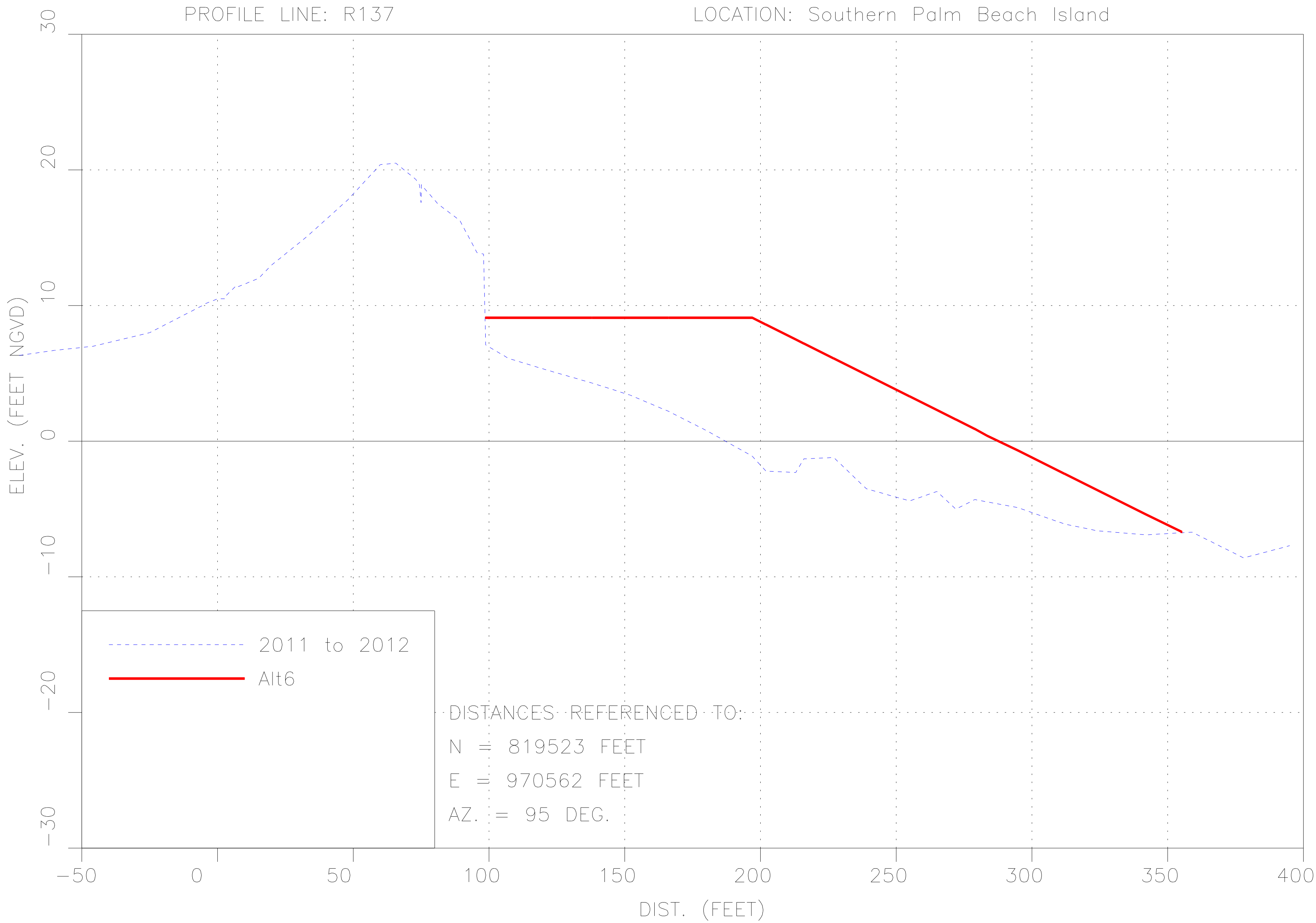
DIST. (FEET)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0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
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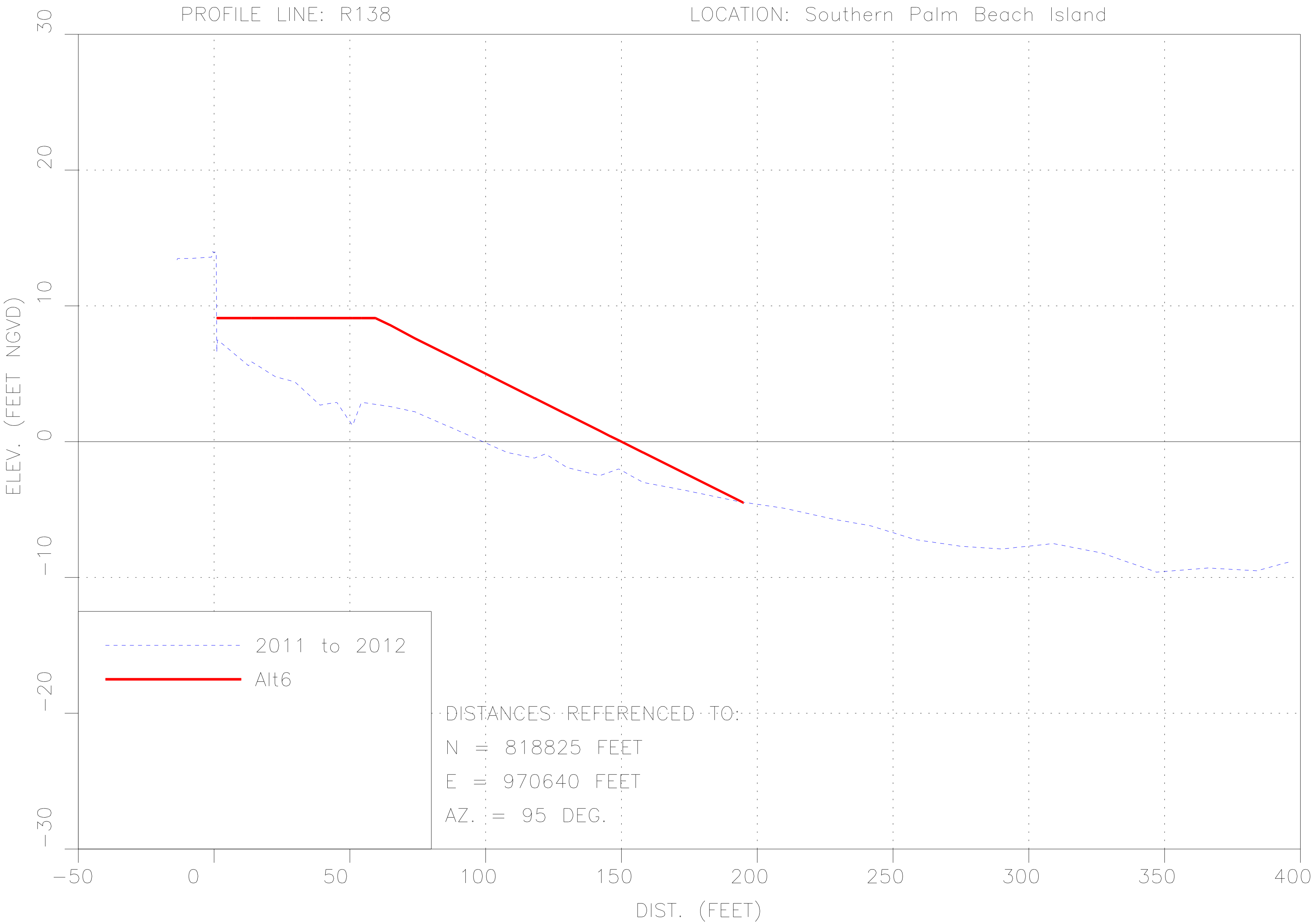
E = 970562 FEET
AZ. = 95 DEG.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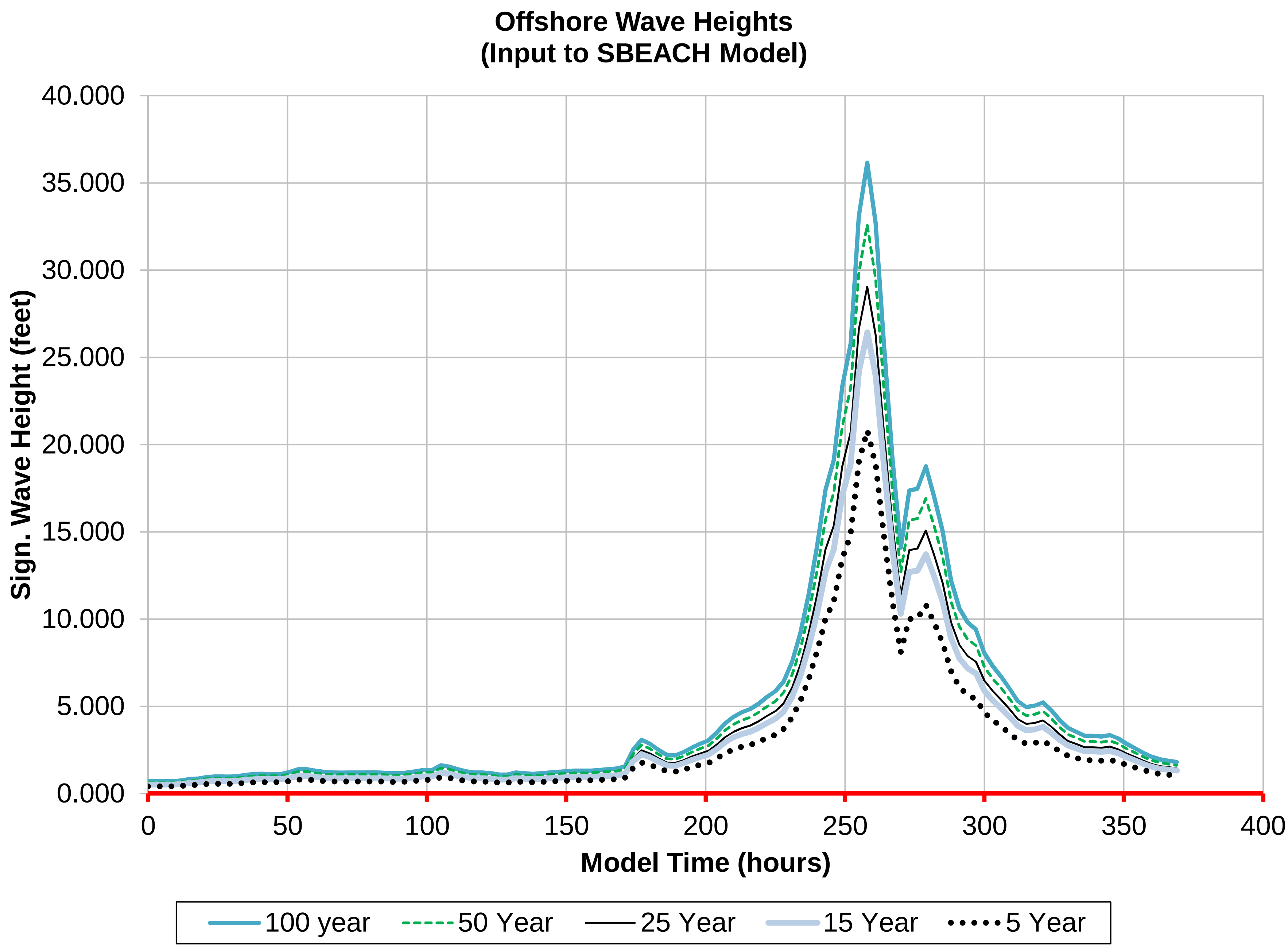
LOCATION: Southern Palm Beach Island
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
2011 to 2012
Alt6
DISTANCES REFERENCED TO:
N = 818825 FEET
E = 970640 FEET
AZ. = 95 DEG.

SUB-APPENDIX G-1

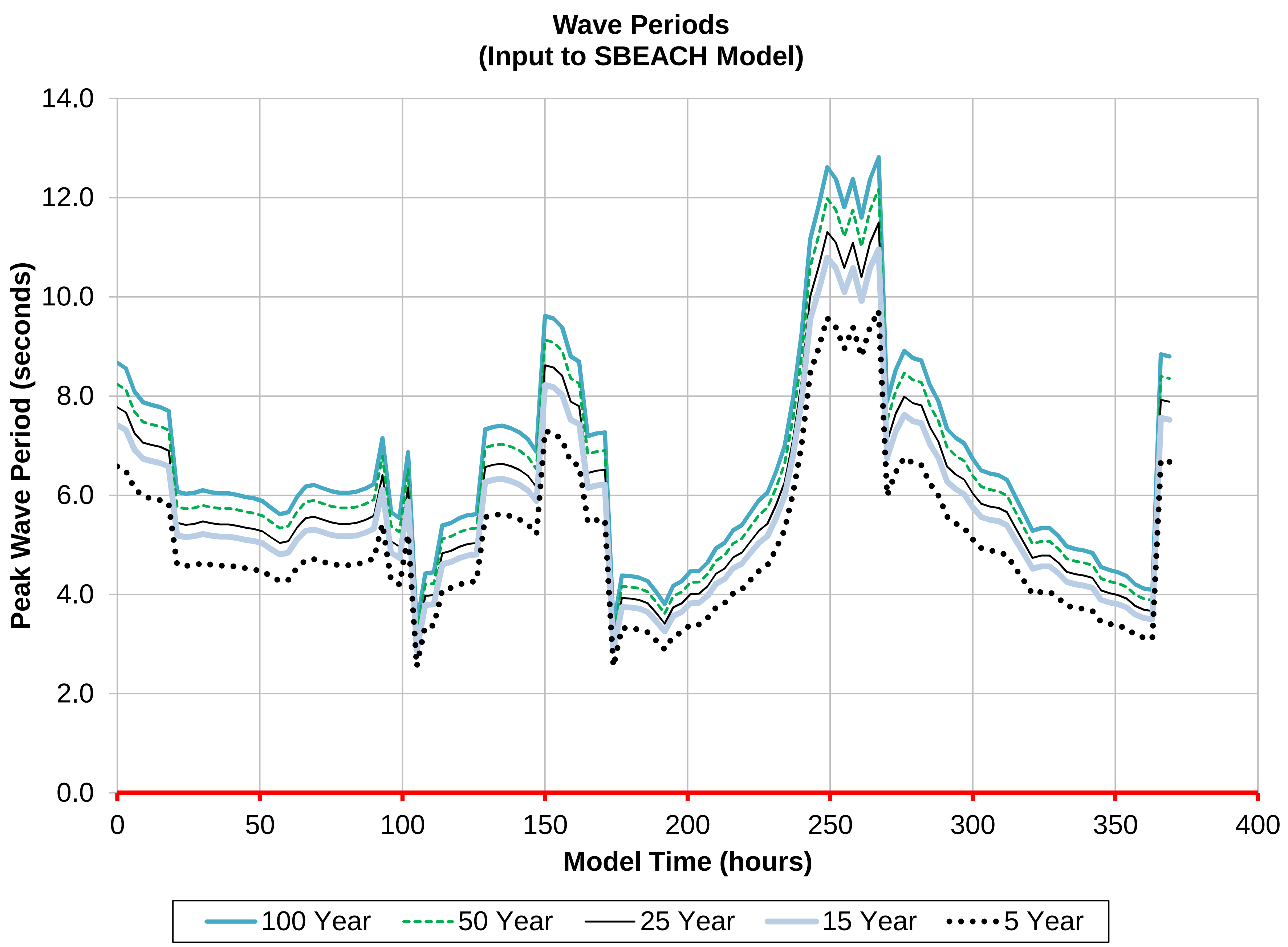
SBEACH ANALYSIS REPORT

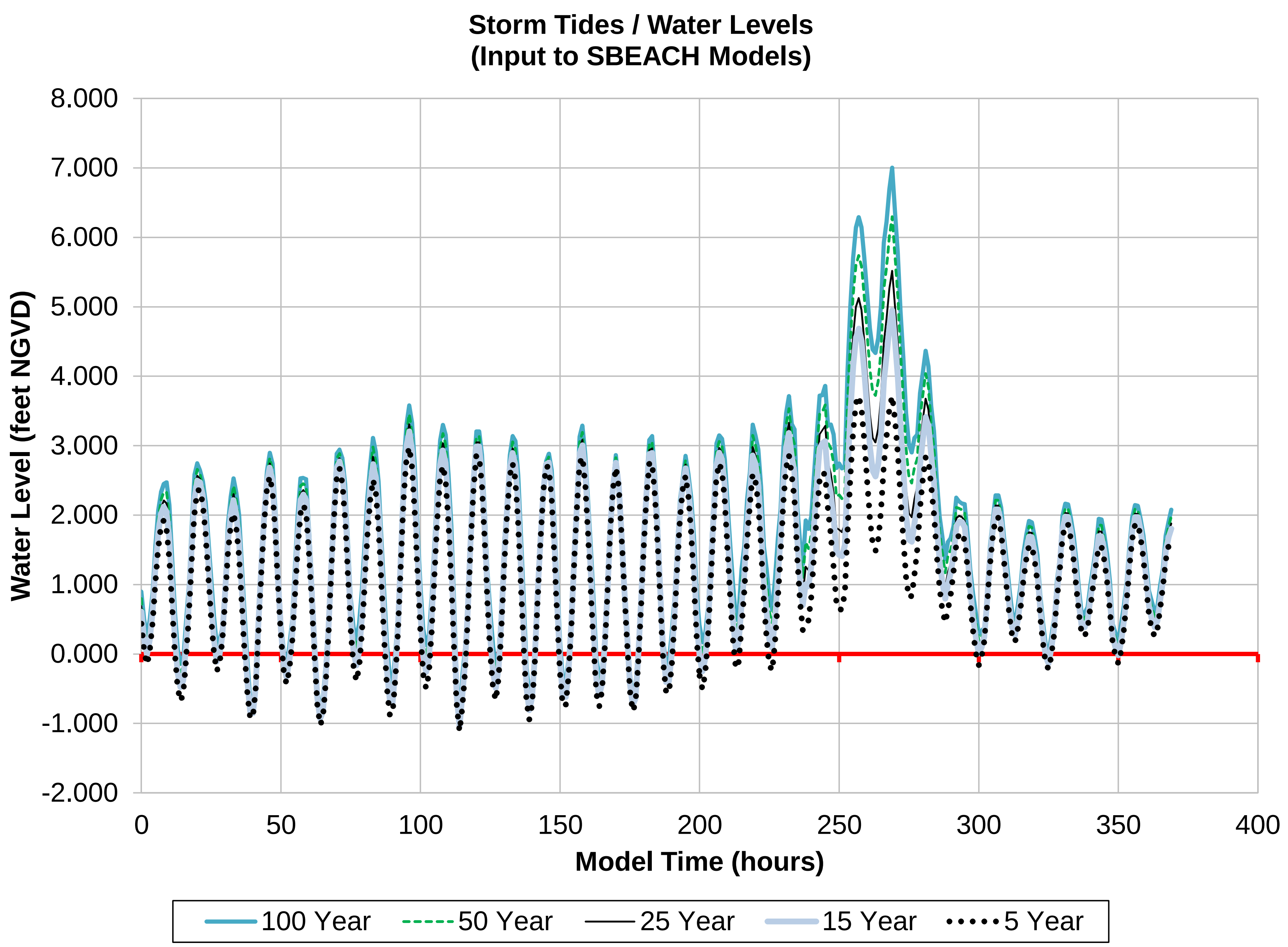
ATTACHMENT B

INPUT STORMS FOR SBEACH MODEL

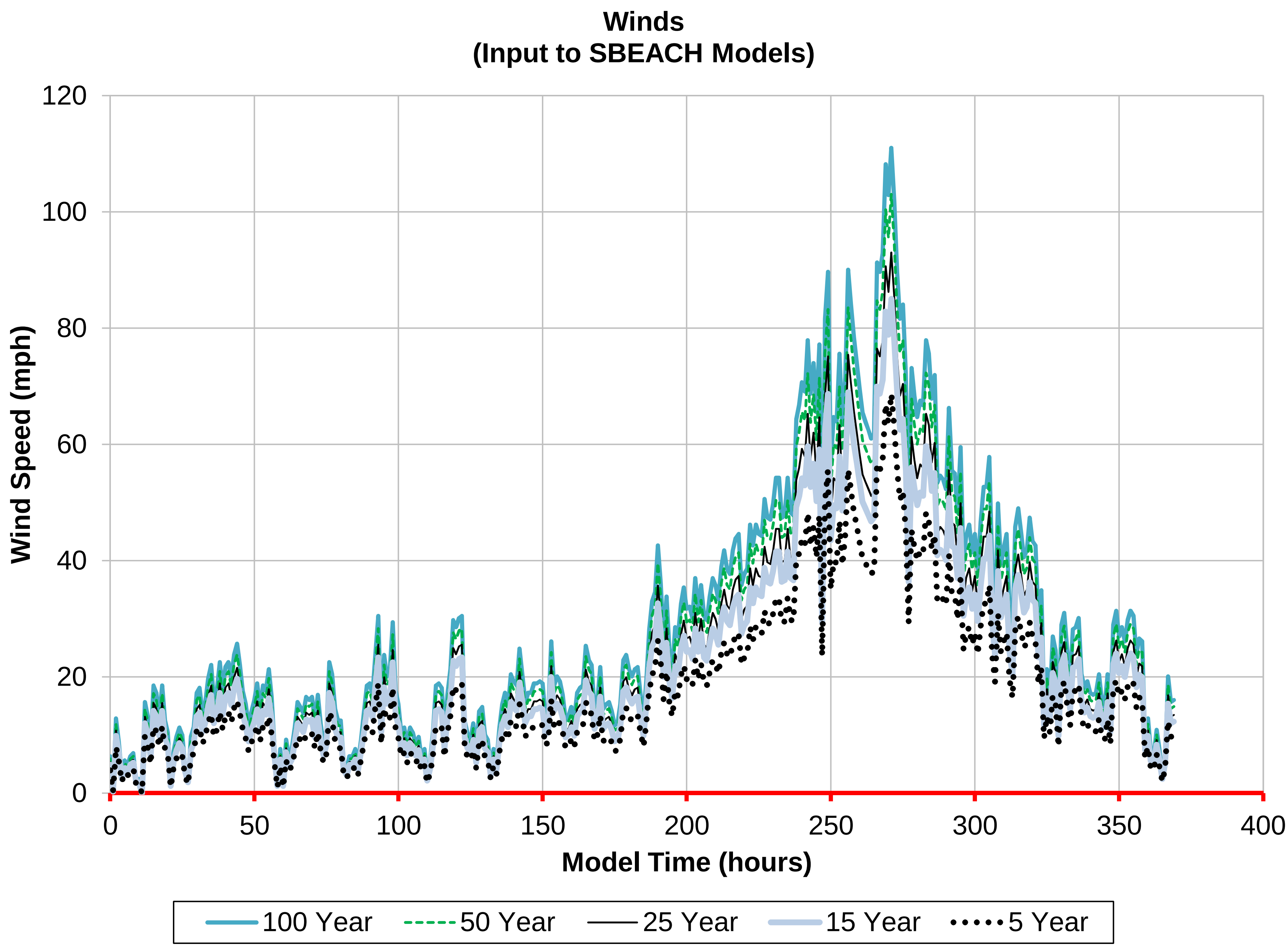


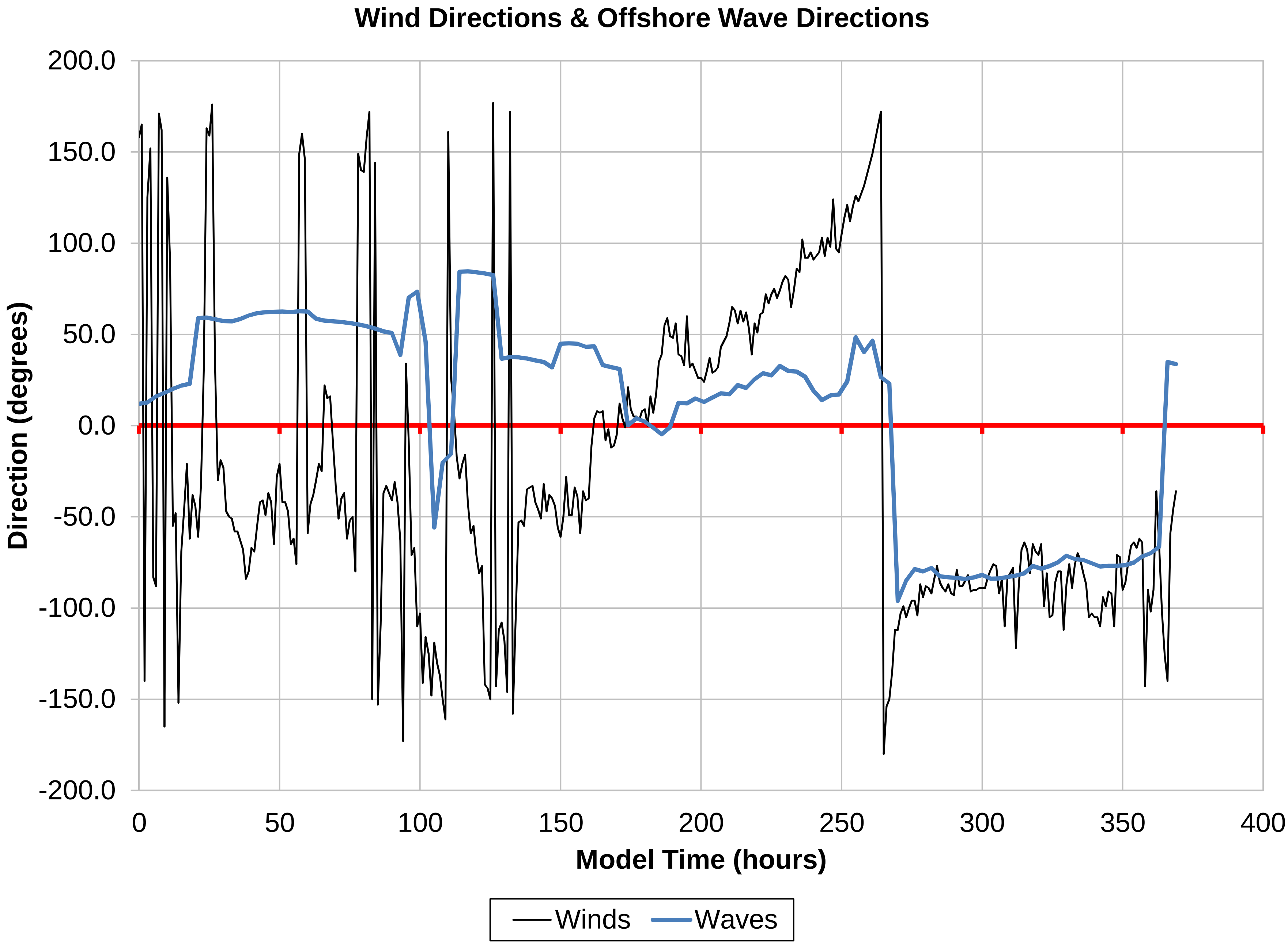
Offshore Wave Heights
(Input to SBEACH Model)
Sign. Wave Height (feet)
40.000
35.000
30.000
25.000
20.000
15.000
10.000
5.000
0.000
0
50
100
150
200
250
300
350
400
Model Time (hours)
100 year
50 Year
25 Year
15 Year
5 Year


Wave Periods
(Input to SBEACH Model)
Peak Wave Period (seconds)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
50
100
150
200
250
300
350
400
Model Time (hours)
100 Year
50 Year
25 Year
15 Year
5 Year



Storm Tides / Water Levels
(Input to SBEACH Models)
Water Level (feet NGVD)
8.000
7.000
6.000
5.000
4.000
3.000
2.000
1.000
0.000
-1.000
-2.000
0
50
100
150
200
250
300
350
400
Model Time (hours)
100 Year
50 Year
25 Year
15 Year
5 Year


Winds
(Input to SBEACH Models)
Wind Speed (mph)
0
20
40
60
80
100
120
Model Time (hours)
0
50
100
150
200
250
300
350
400
100 Year
50 Year
25 Year
15 Year
5 Year



Wind Directions & Offshore Wave Directions
200.0
150.0
100.0
50.0
0.0
-50.0
-100.0
-150.0
-200.0
Direction (degrees)
0
50
100
150
200
250
300
350
400
Model Time (hours)
Winds
Waves

SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT C

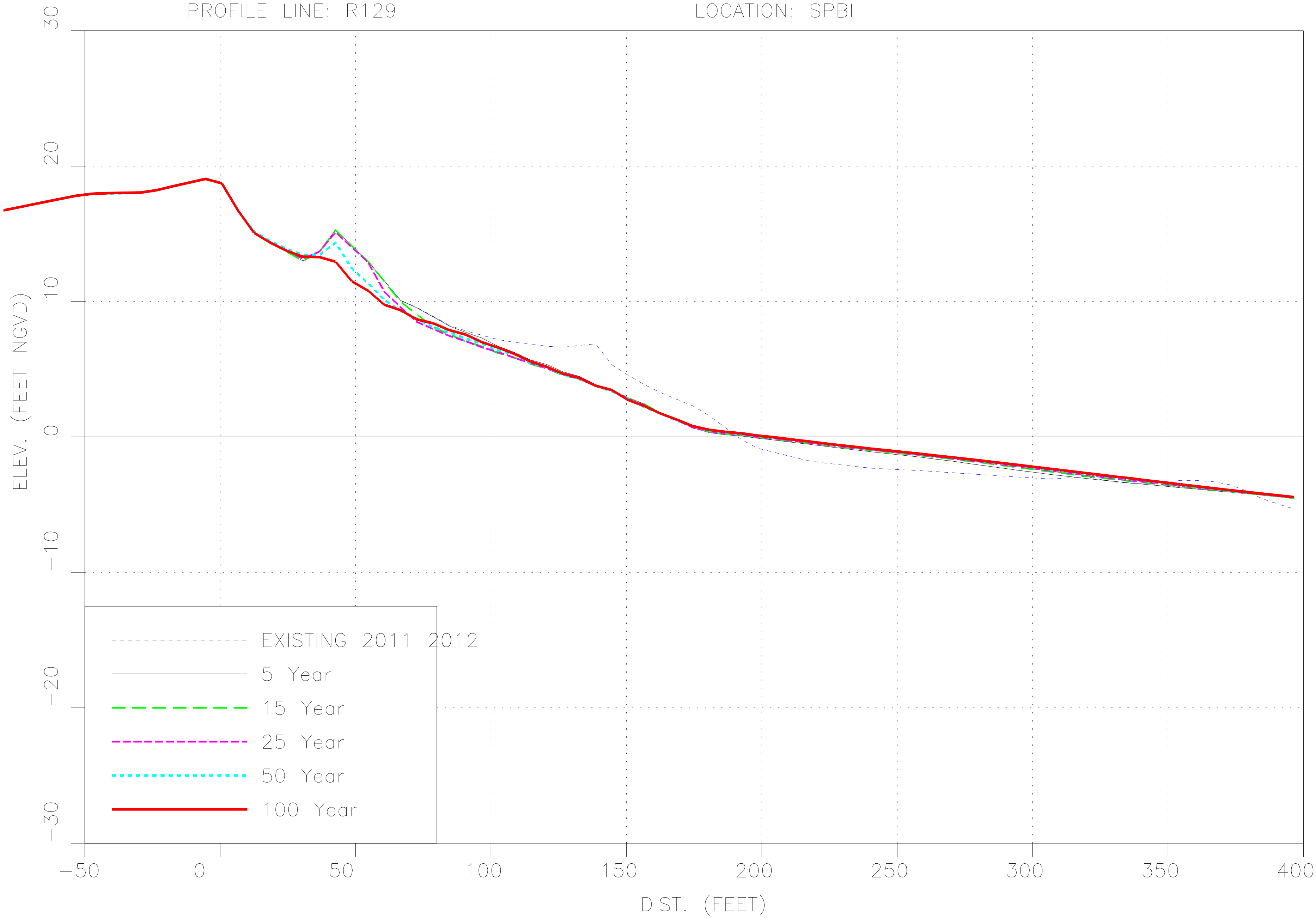
SBEACH MODEL RESULTS

SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT C-1

EXISTING CONDITIONS (2011/2012 SURVEY)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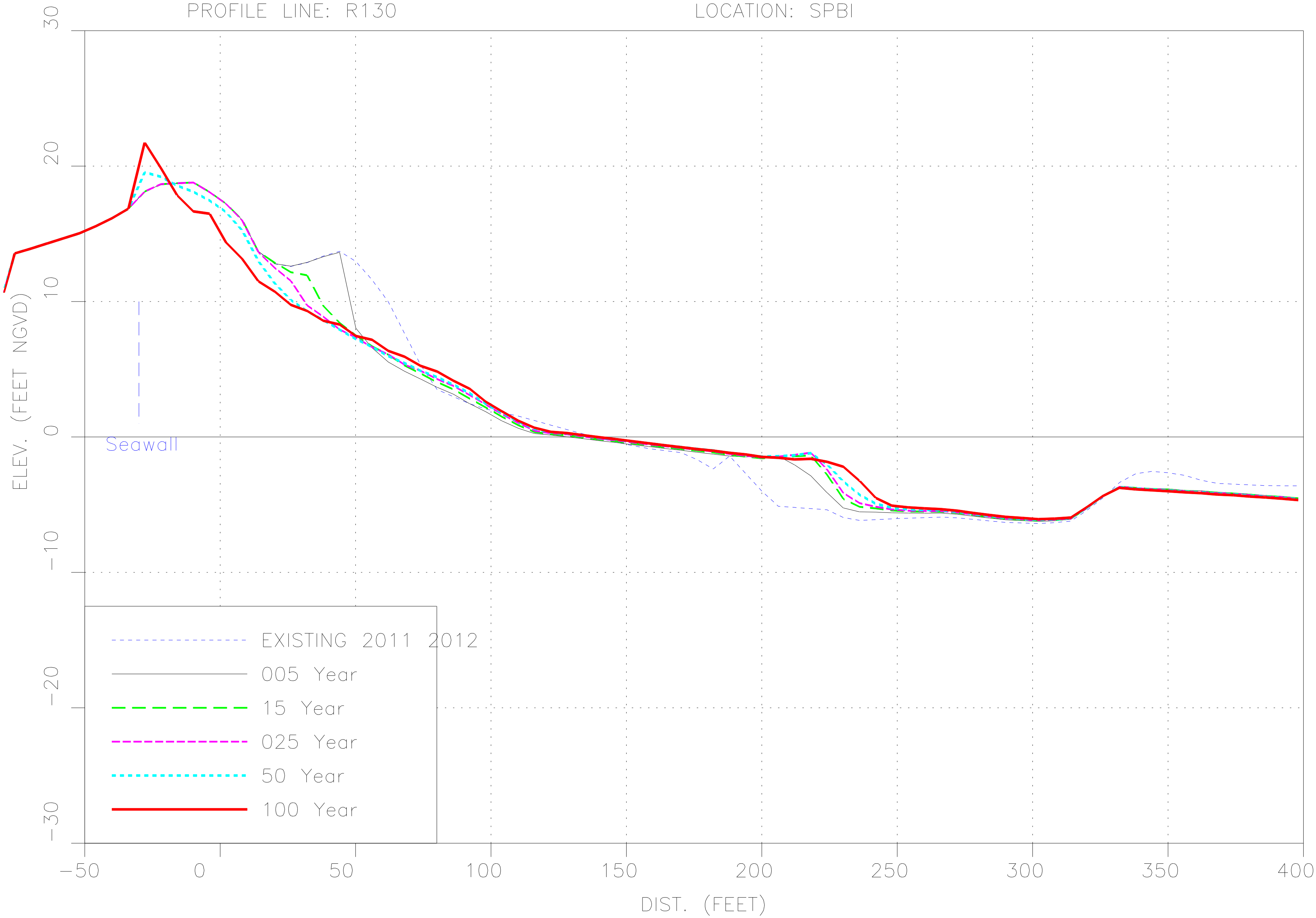
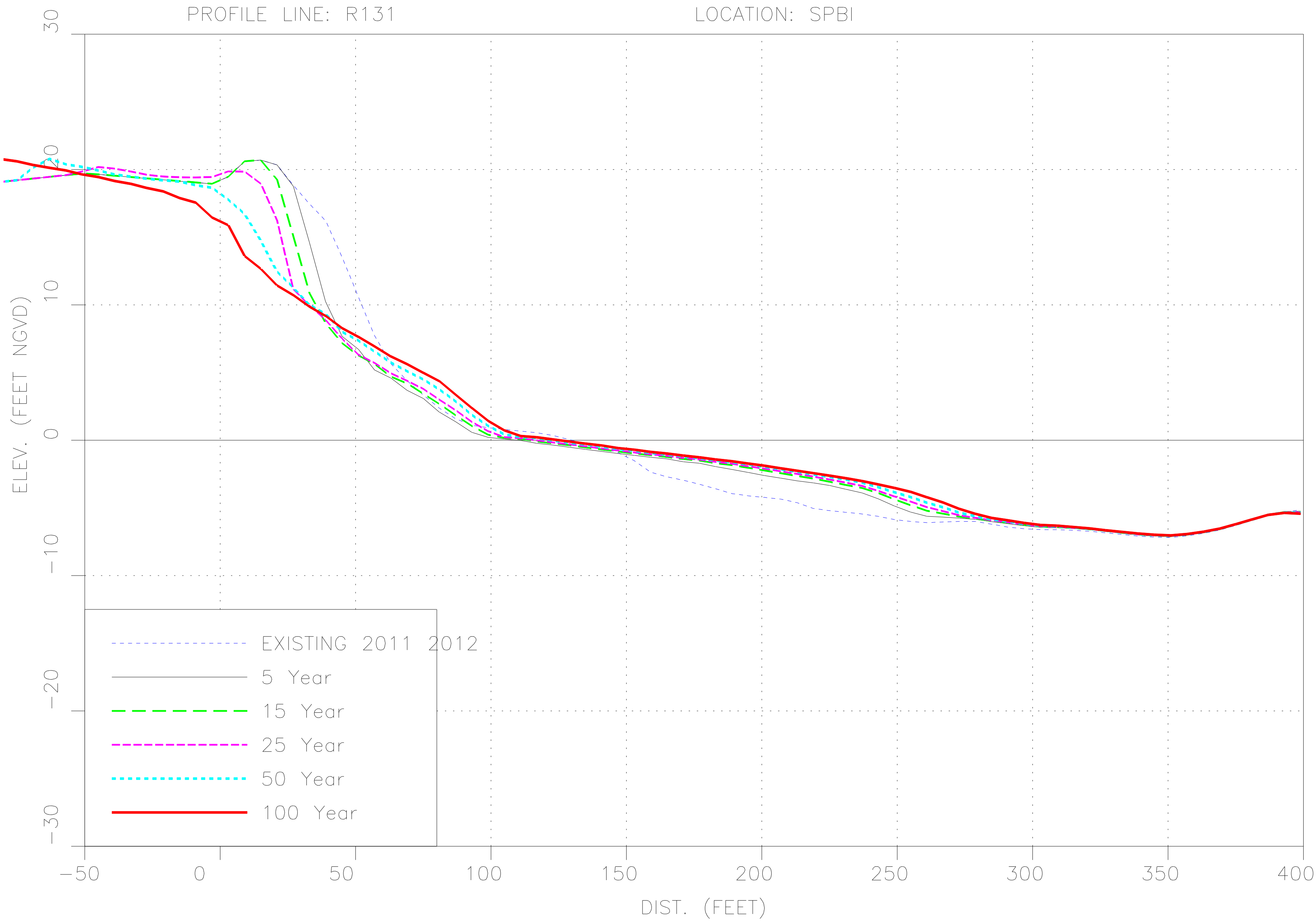
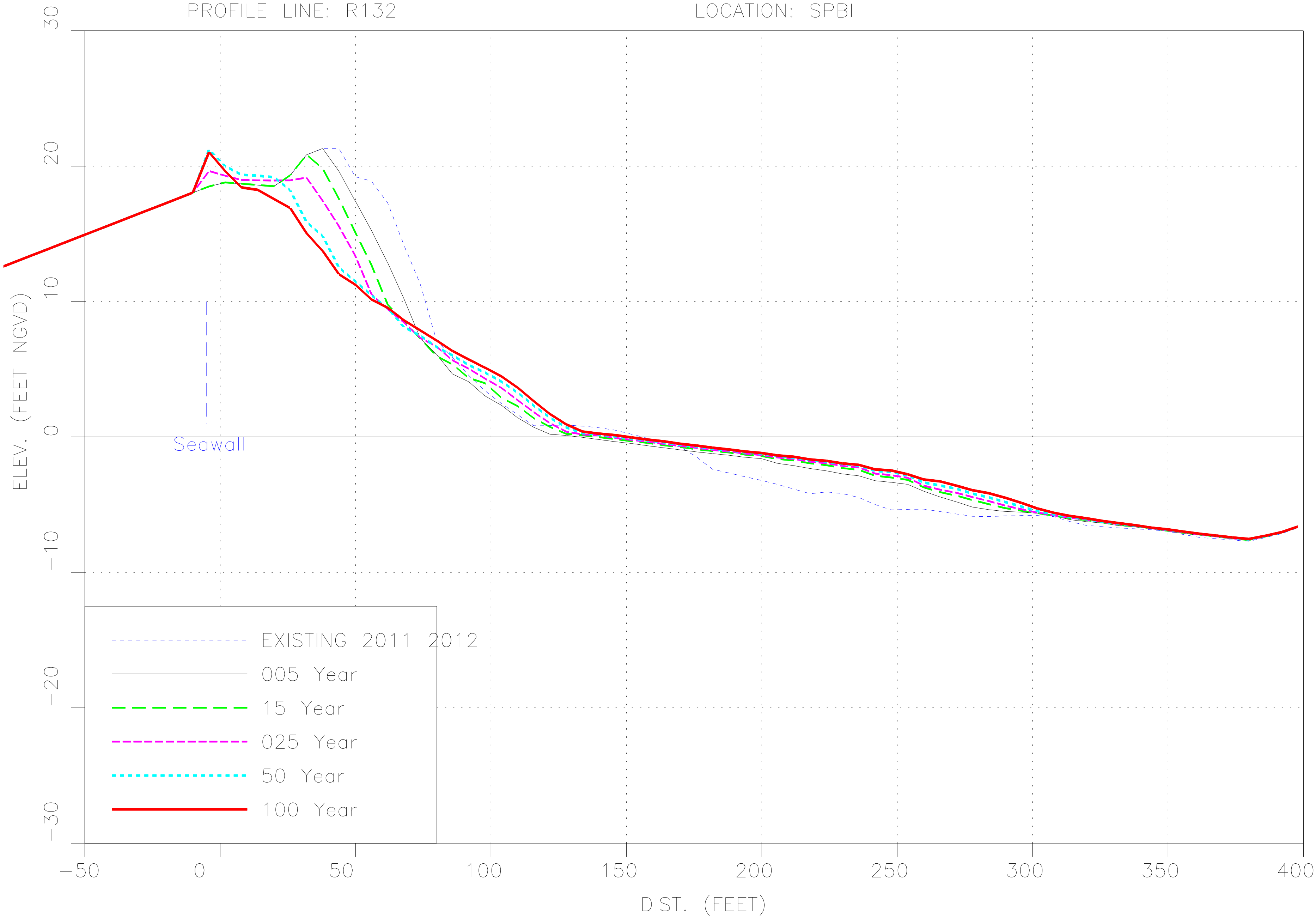
LOCATION: SPBI
ELEV. (FEET NGVD)
DIST. (FEET)
30
20
10
0
-10
-20
-30
-50
0
50
100
150
200
250
300
350
400
EXISTING 2011 2012
5 Year
15 Year
25 Year
50 Year
100 Year



PROFILE LINE: R130
LOCATION: SPBI
ELEV. (FEET NGVD)
DIST. (FEET)
Seawall
EXISTING 2011 2012
005 Year
15 Year
025 Year
50 Year
100 Year
30
20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400



PROFILE LINE: R131
LOCATION: SPBI
ELEV. (FEET NGVD)
30
20
10
0
−10
−20
−30
−50
0
50
100
150
200
250
300
350
400
DIST. (FEET)
EXISTING 2011 2012
5 Year
15 Year
25 Year
50 Year
100 Year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DIST. (FEET)
Seawall
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15 Year
025 Year
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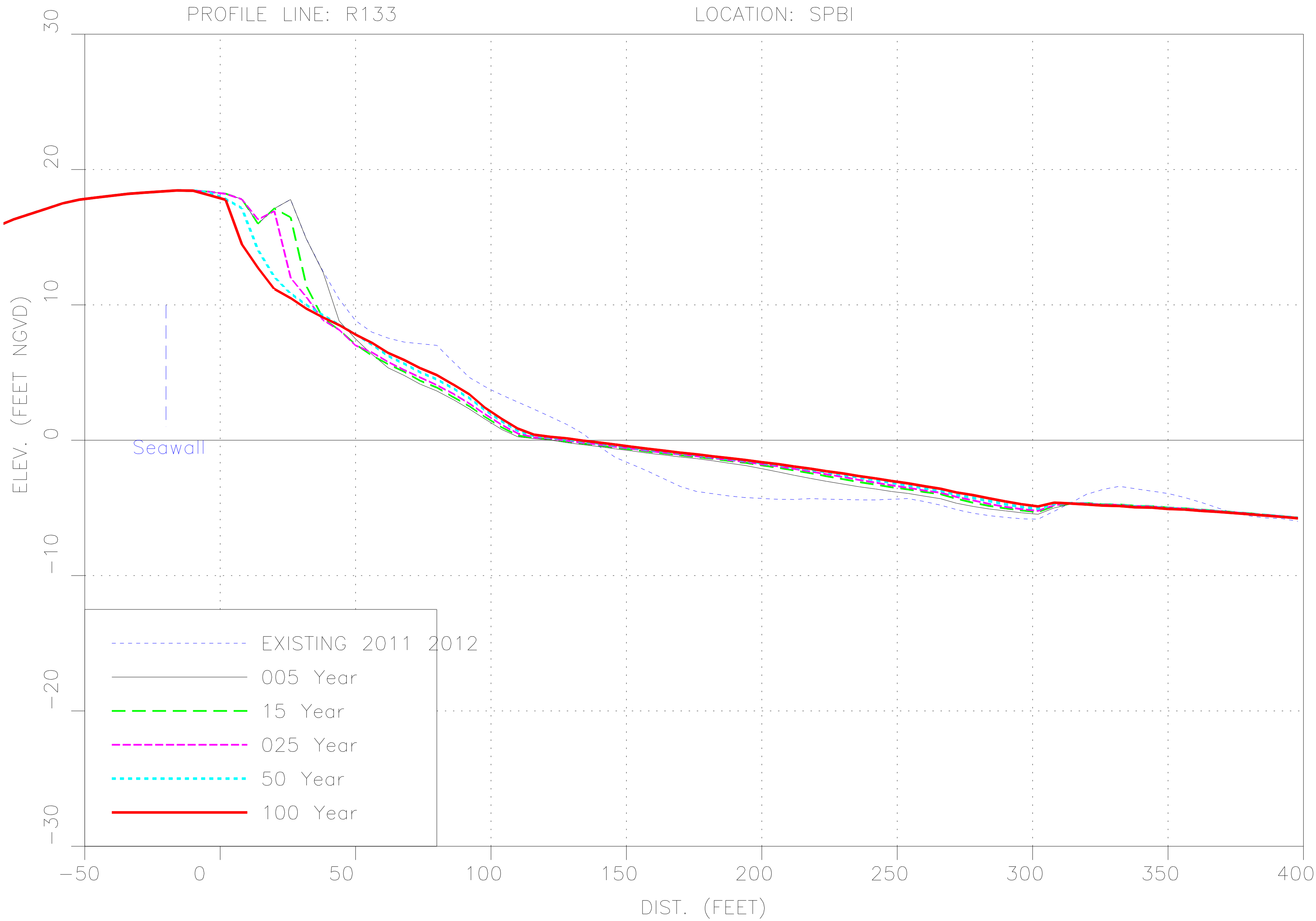

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DIST. (FEET)
Seawall
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15 Year
025 Year
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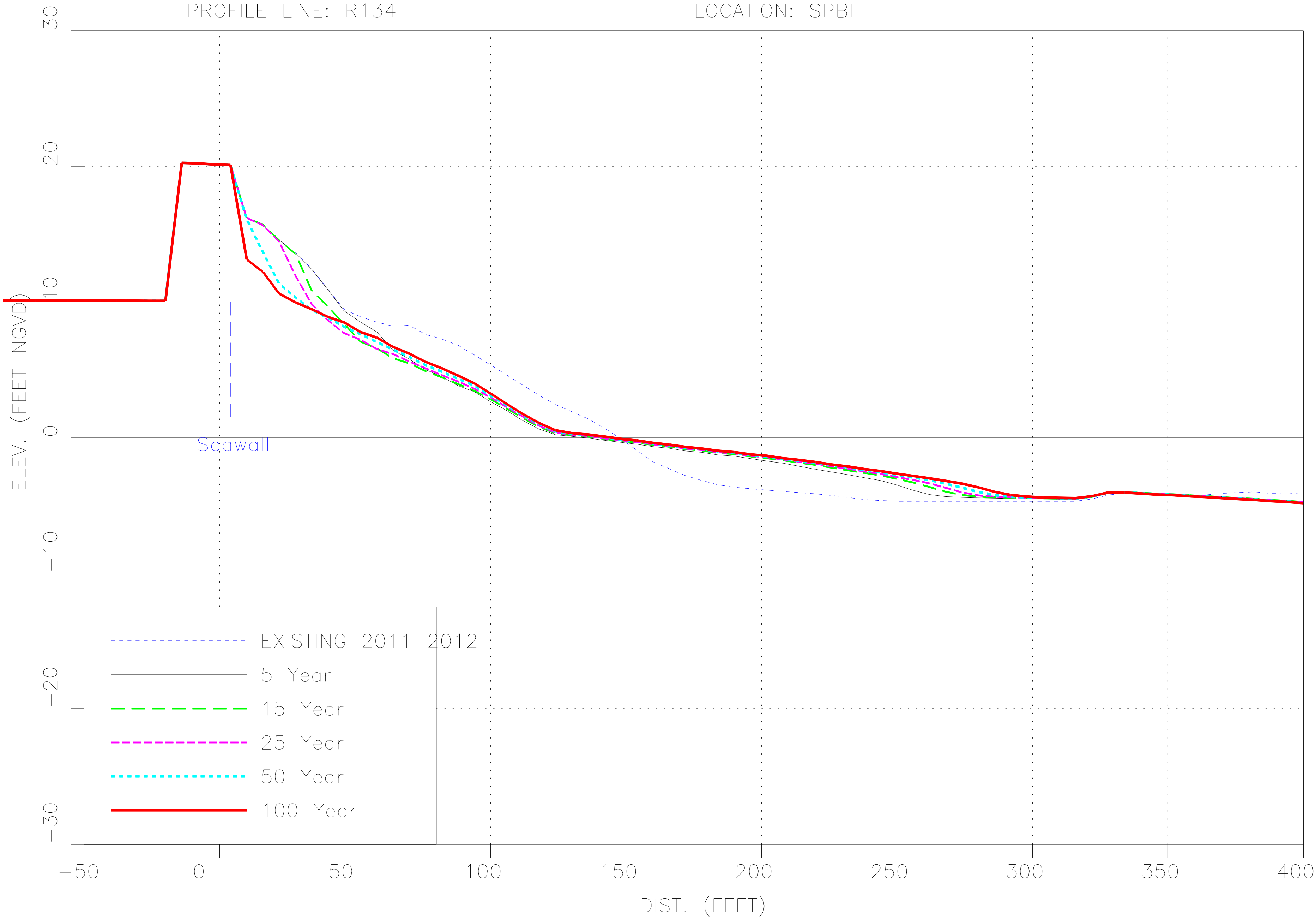

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EXISTING 2011 2012
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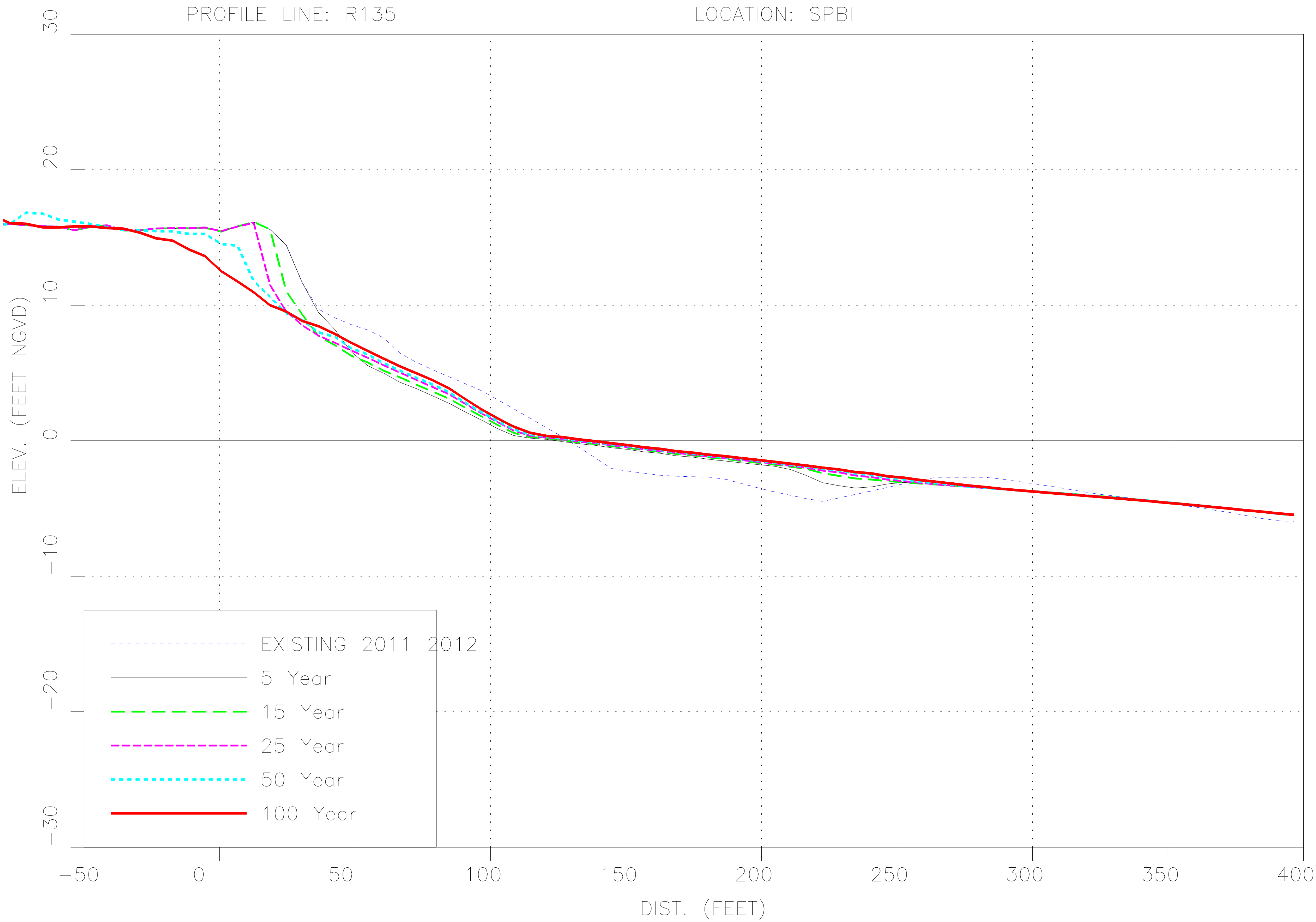

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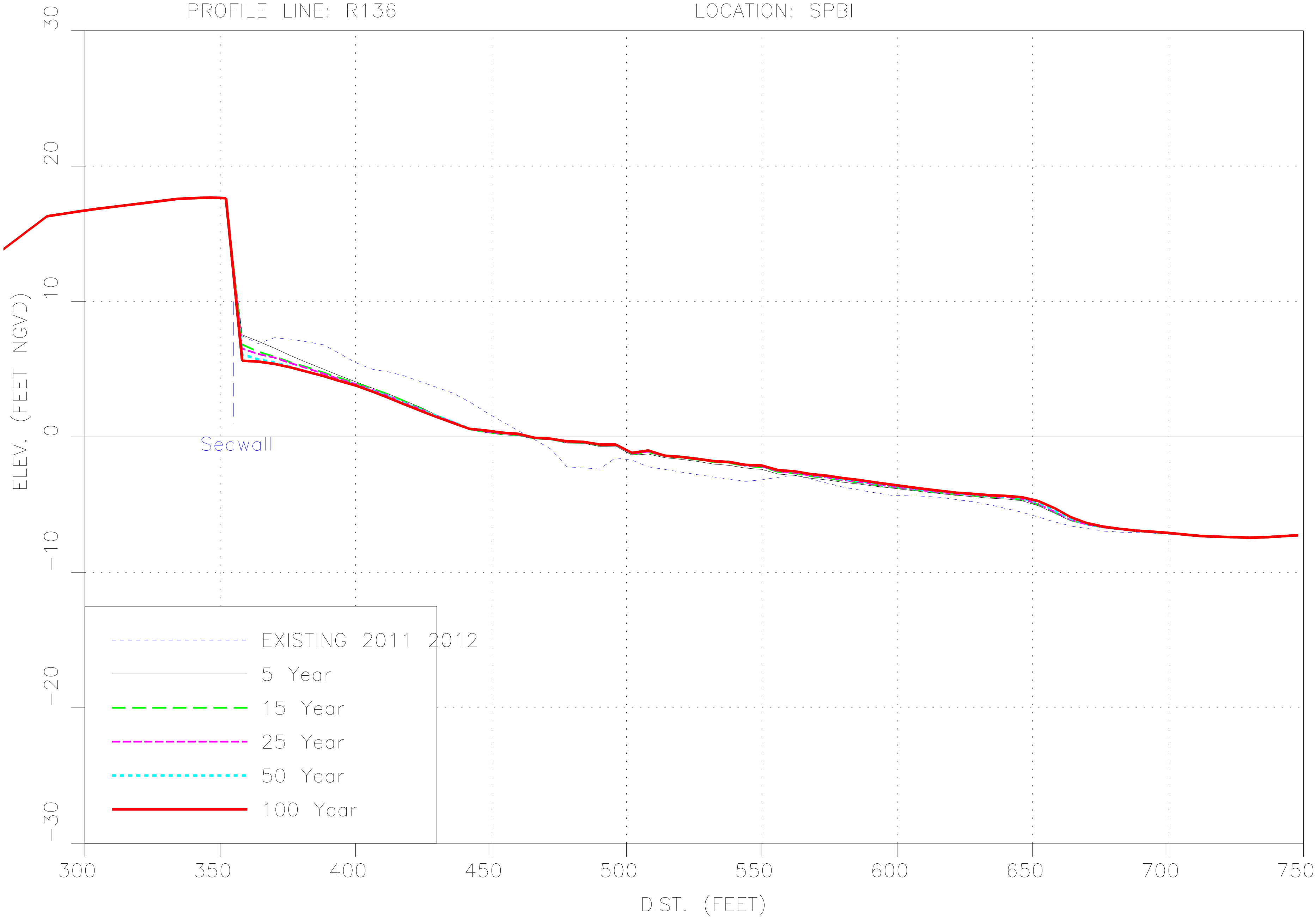
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Seawall
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100 Year
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DIST. (FEET)

PROFILE LINE: R137
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ELEV. (FEET NGVD)
DIST. (FEET)
Seawall
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15 Year
25 Year
50 Year
100 Year
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PROFILE LINE: R138
LOCATION: SPBI
ELEV. (FEET NGVD)
DIST. (FEET)
Seawall
EXISTING 2011 2012
5 Year
15 Year
25 Year
50 Year
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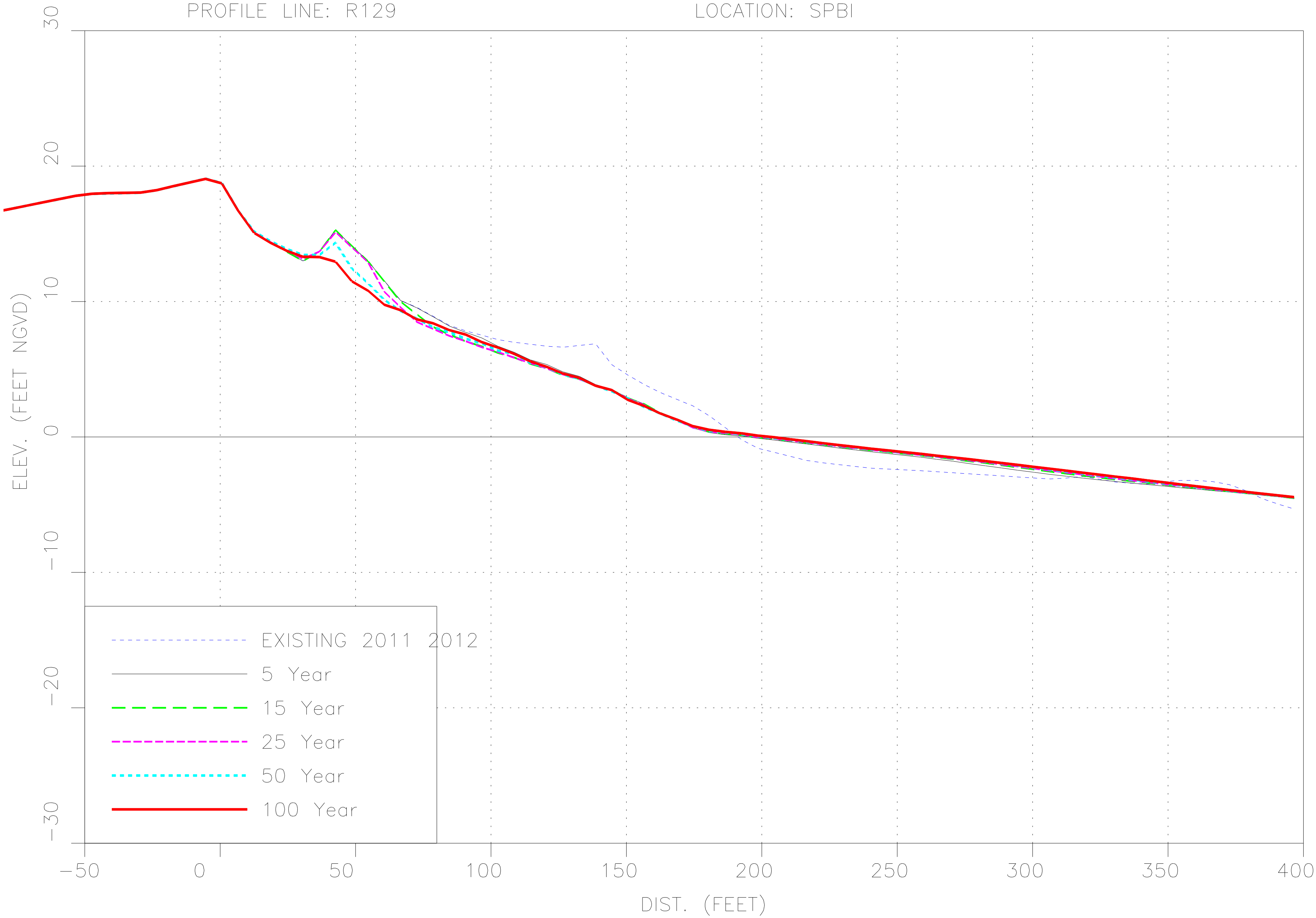
SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT C-2

EXISTING CONDITIONS (2011/2012 SURVEY)

NO SEAWALL/SEAWALL FAILURE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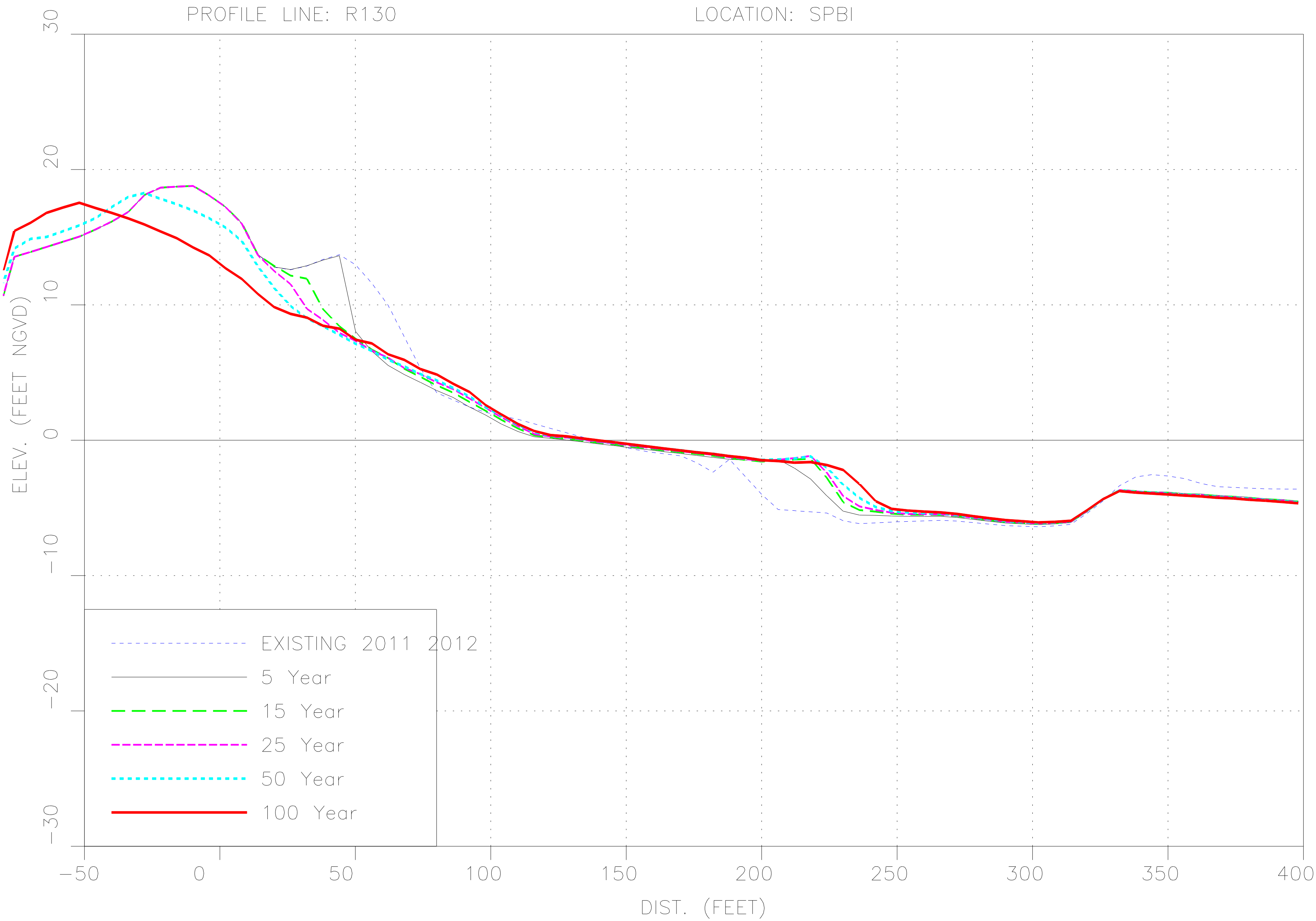

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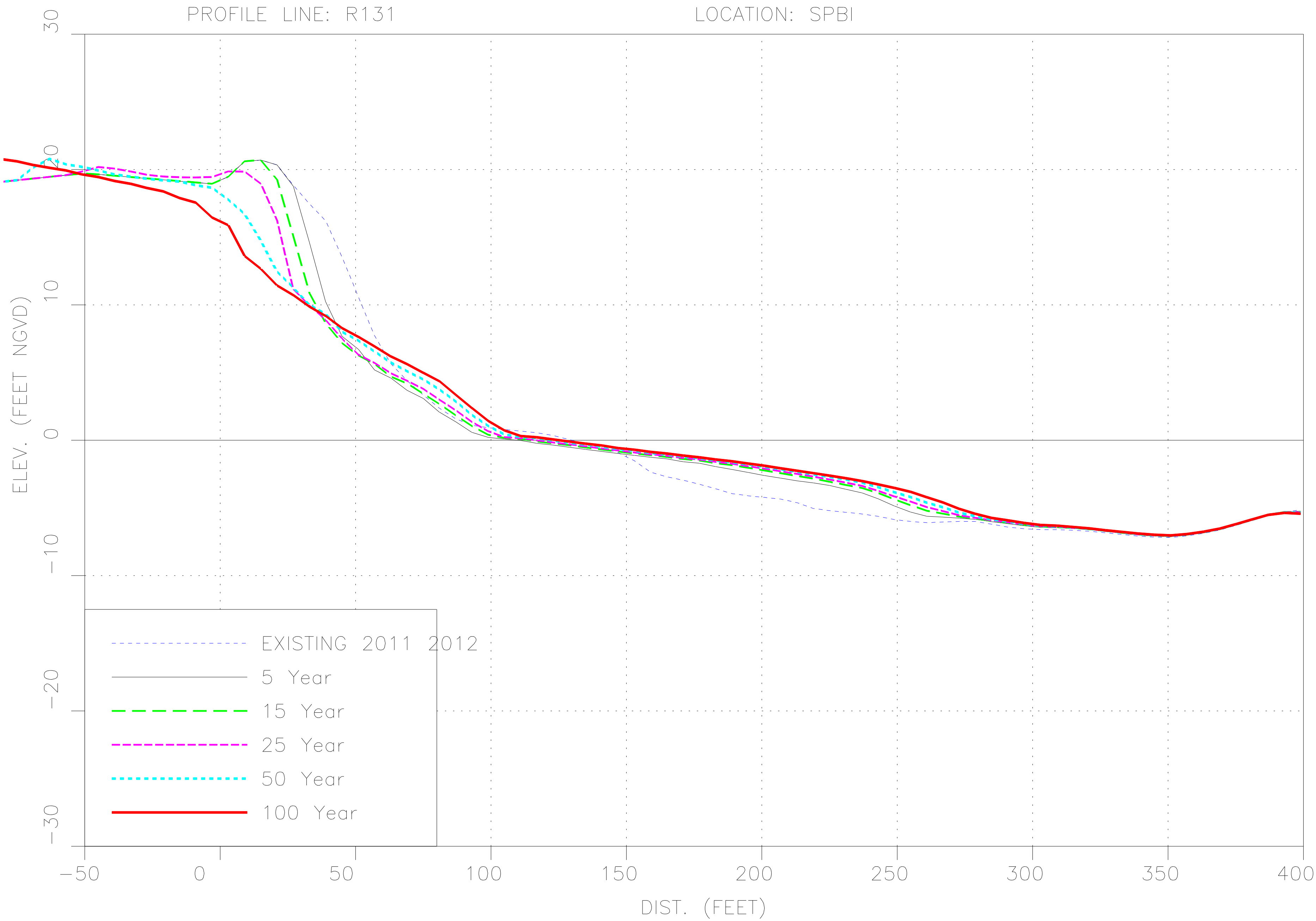
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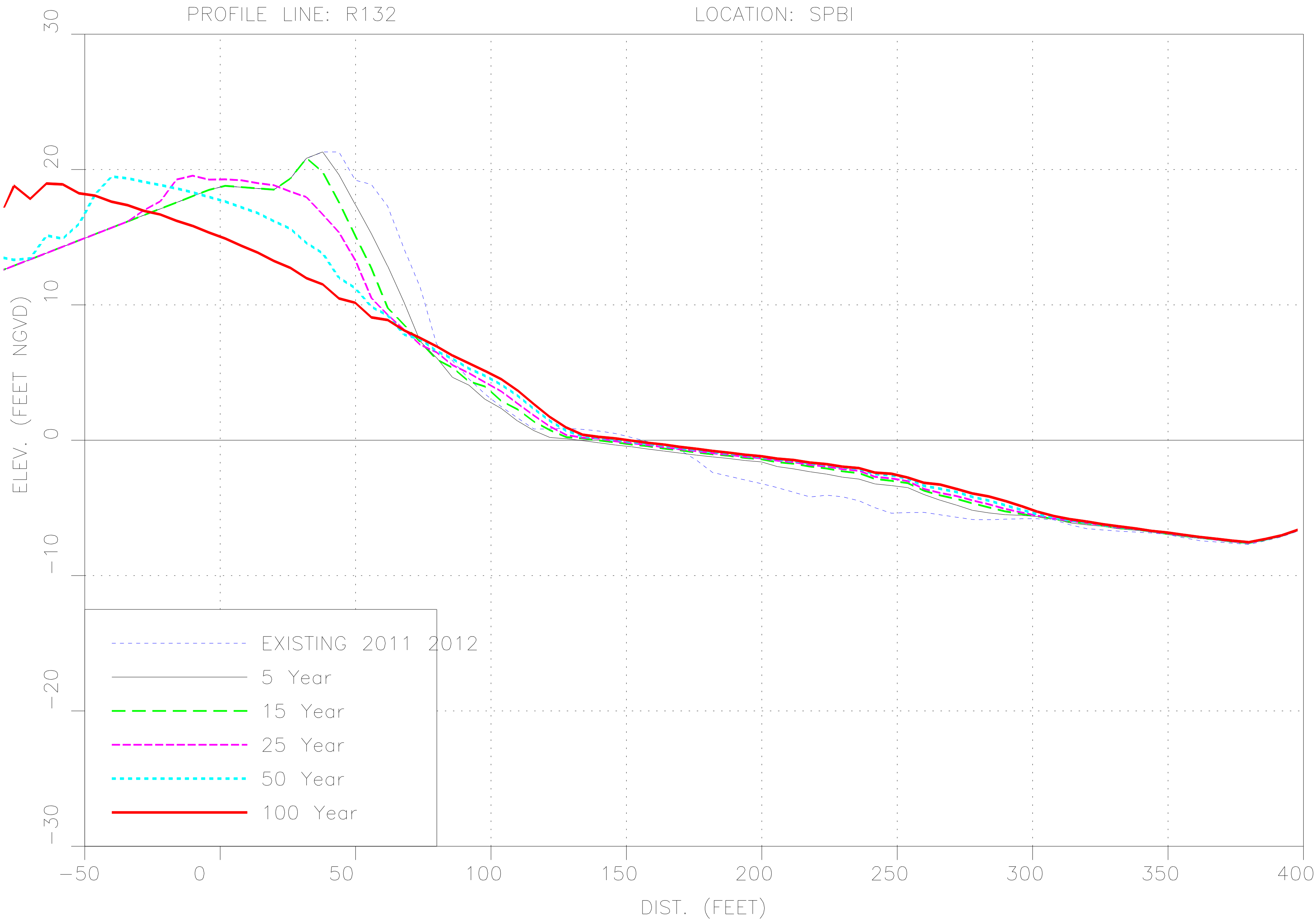
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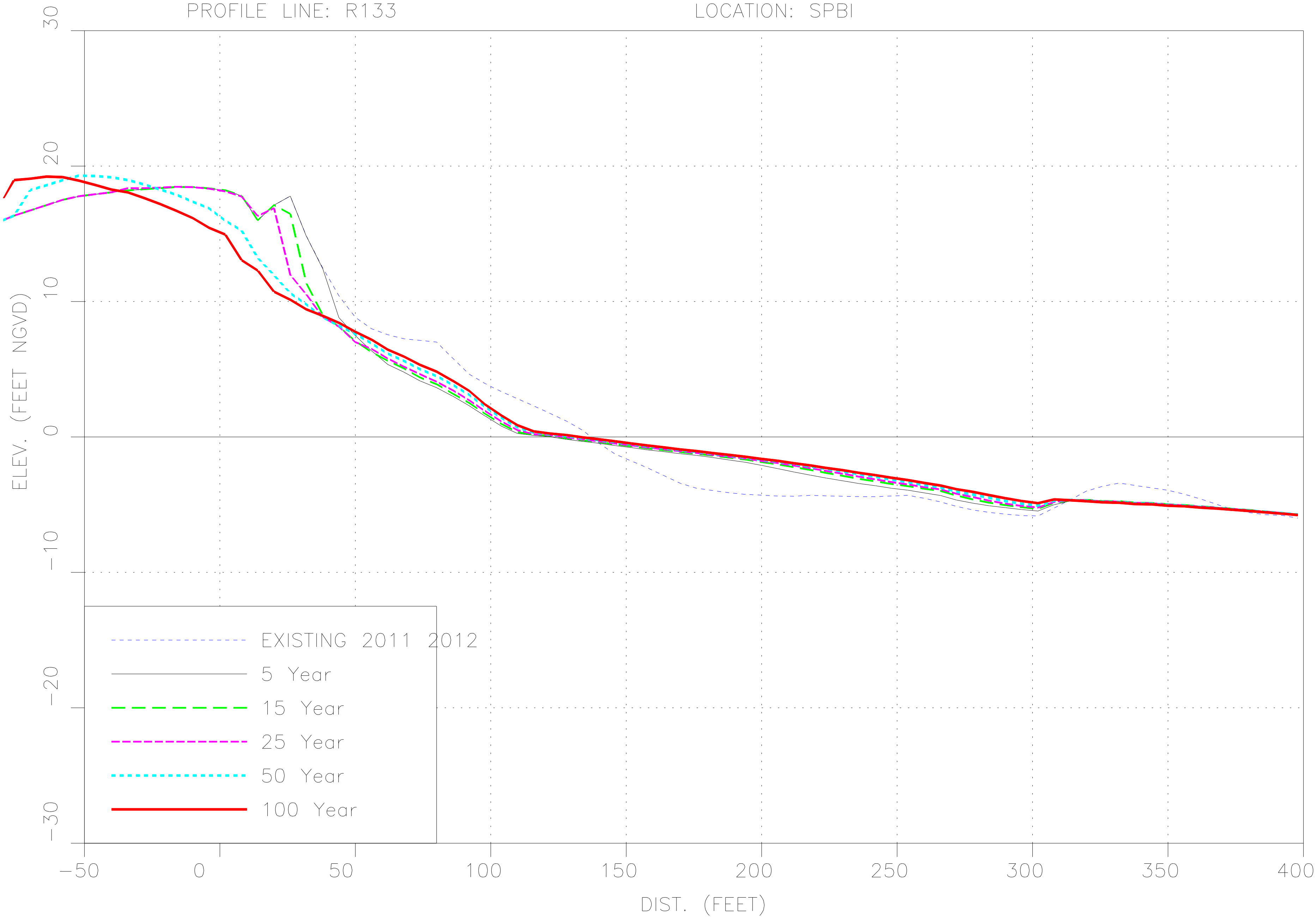
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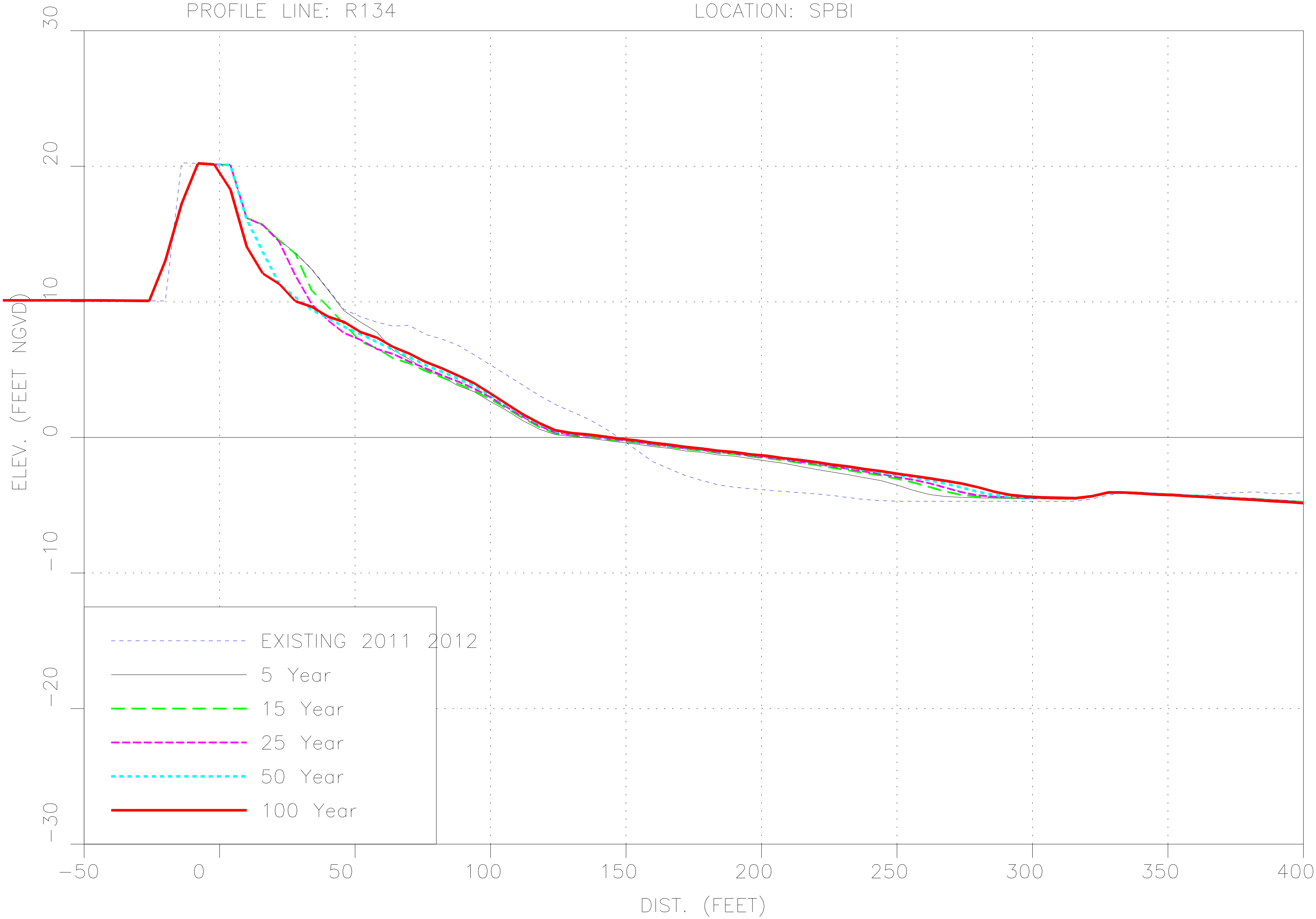
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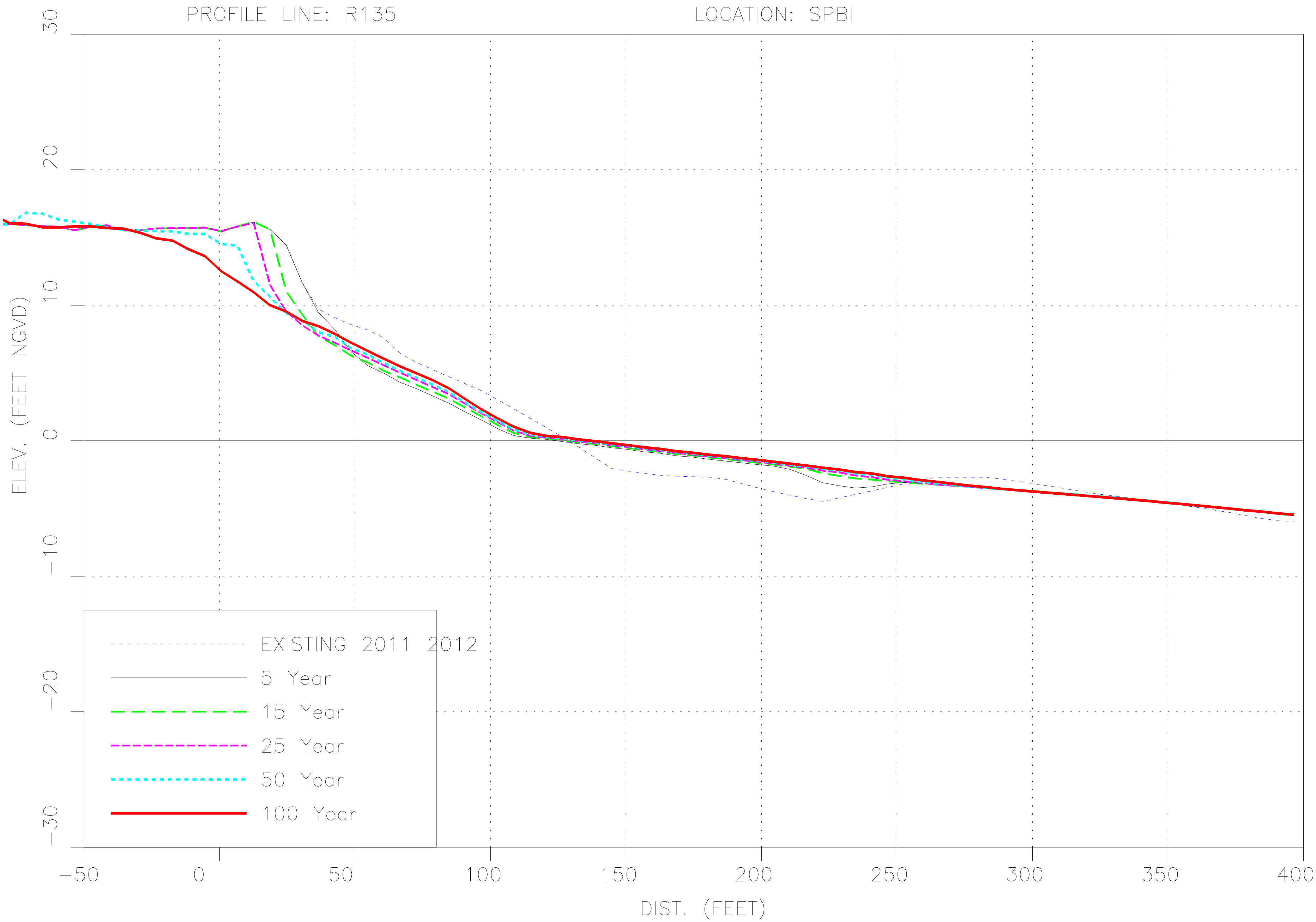
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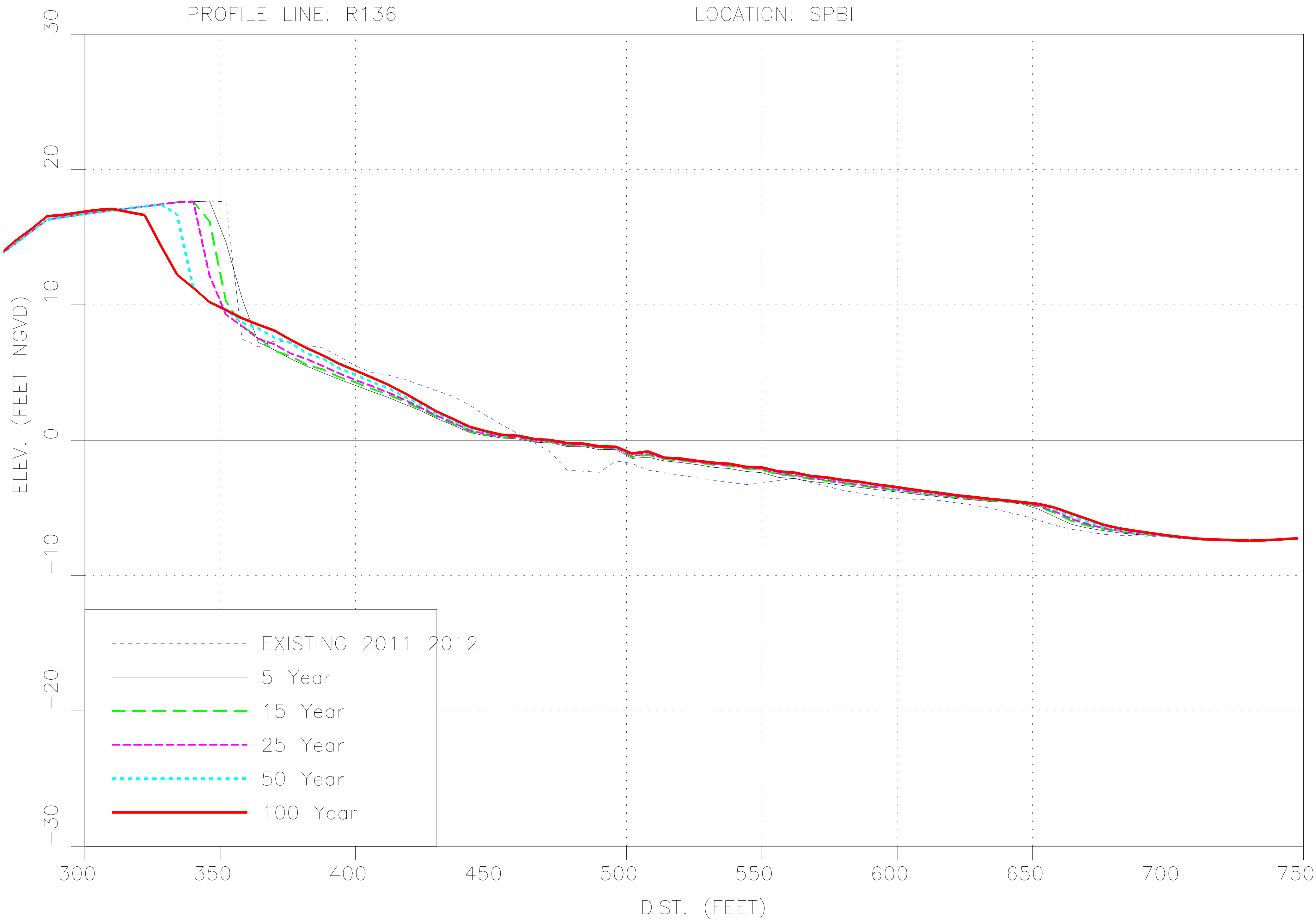
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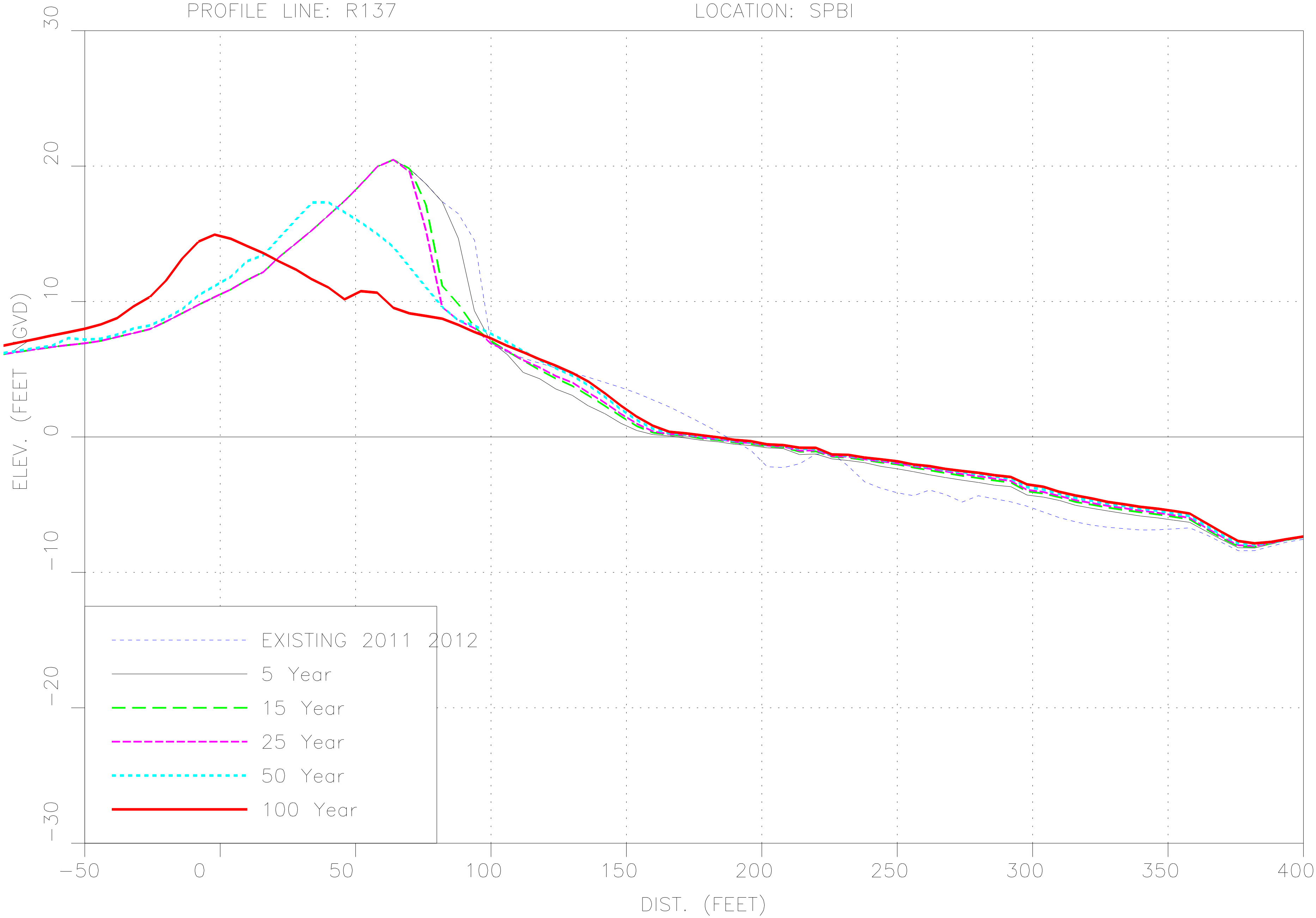
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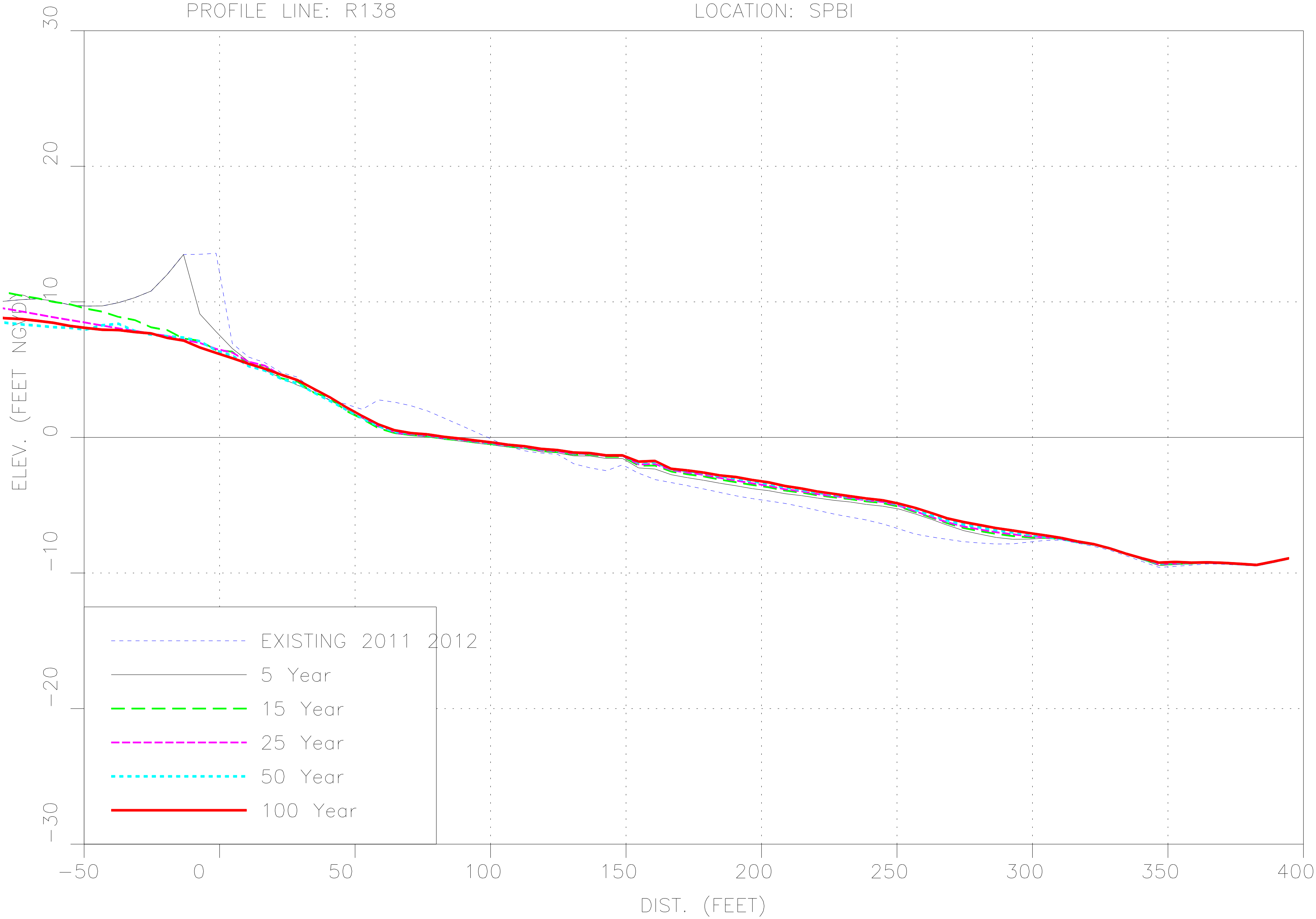
100 Year



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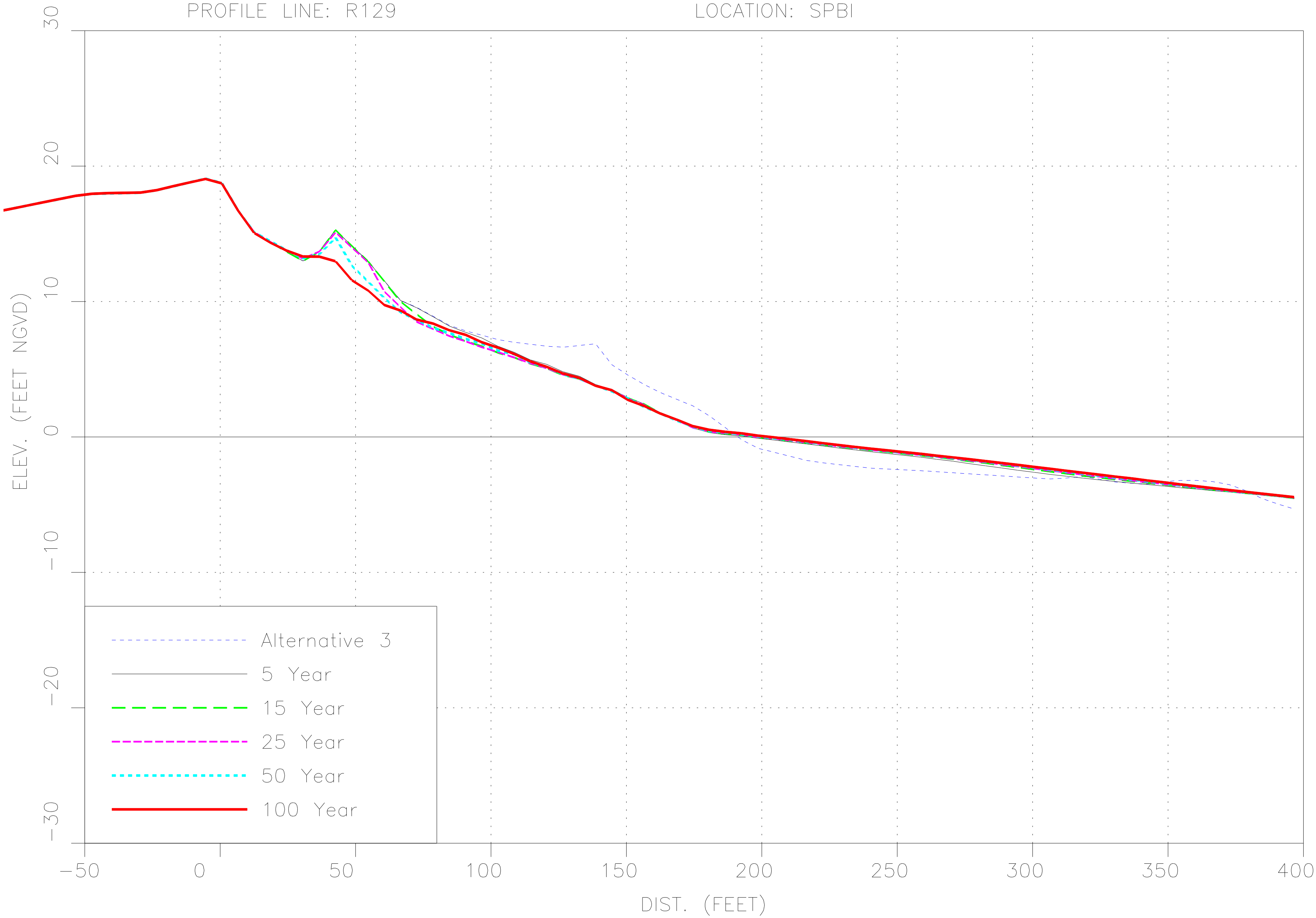
EXISTING 2011 2012
5 Year
15 Year
25 Year
50 Year
100 Year

SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT C-3

ALTERNATIVE 3 (APPLICANTS' PREFERRED)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25 Year
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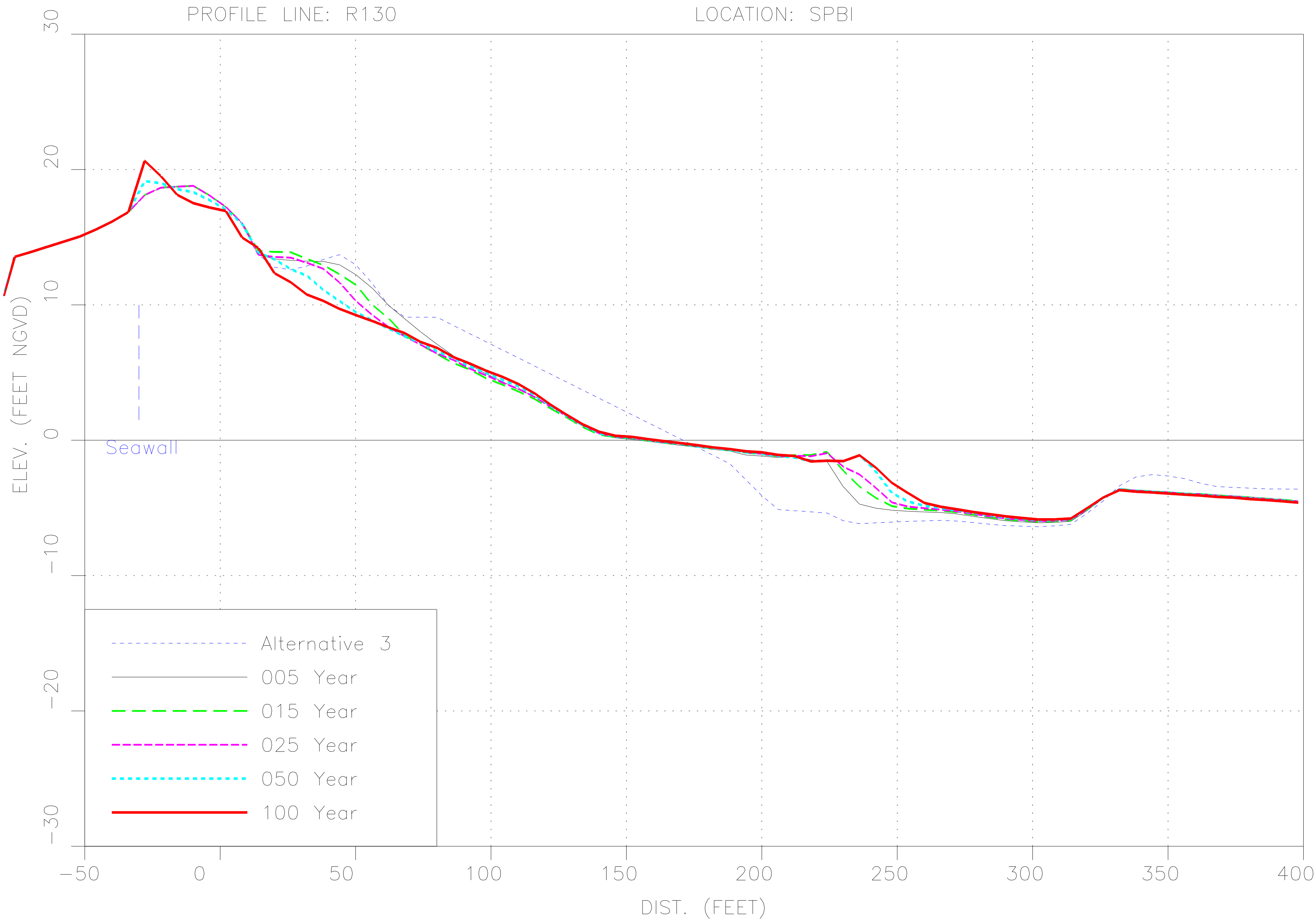
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015 Year
025 Year
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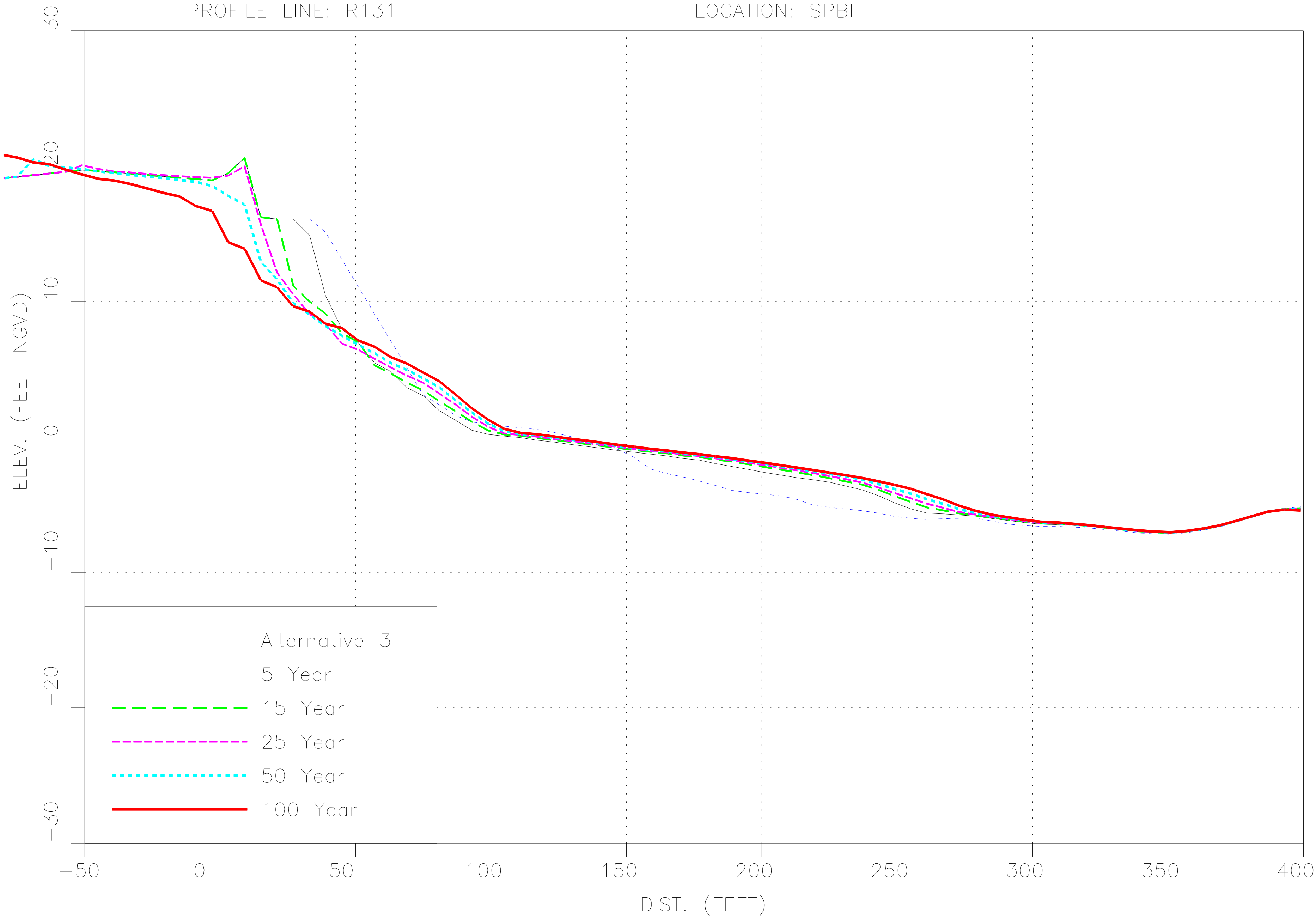

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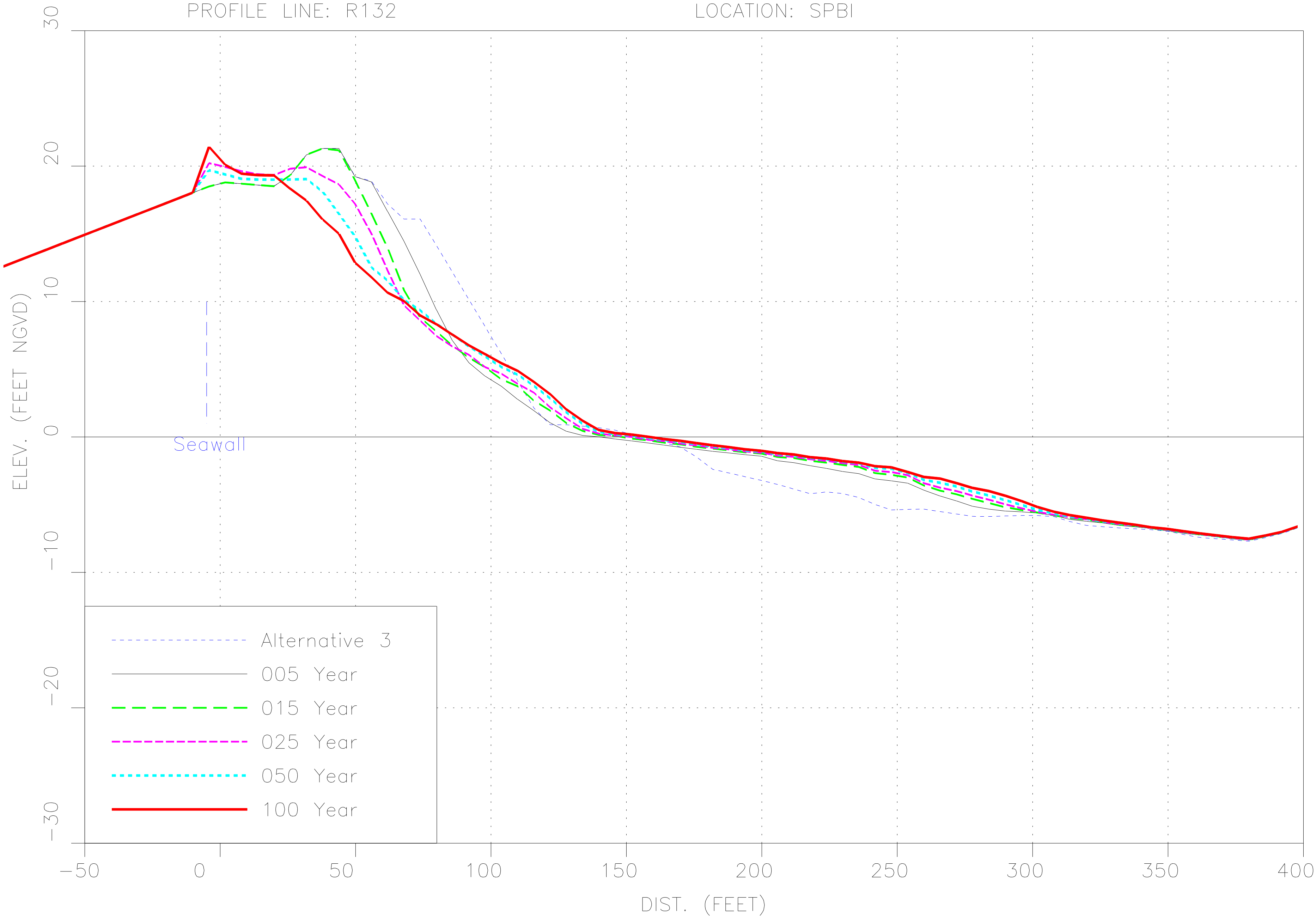
100 Year



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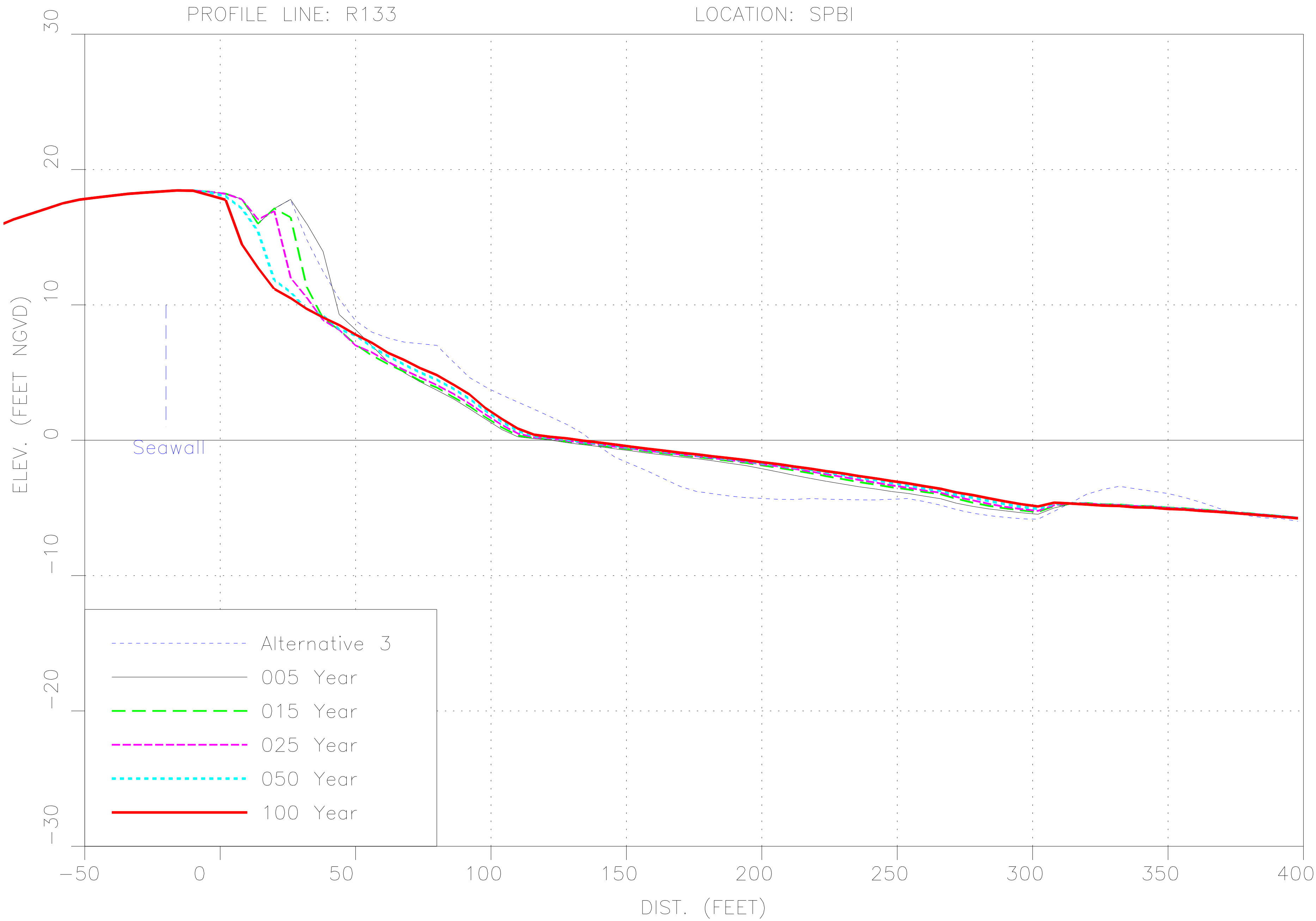
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Seawall
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025 Year
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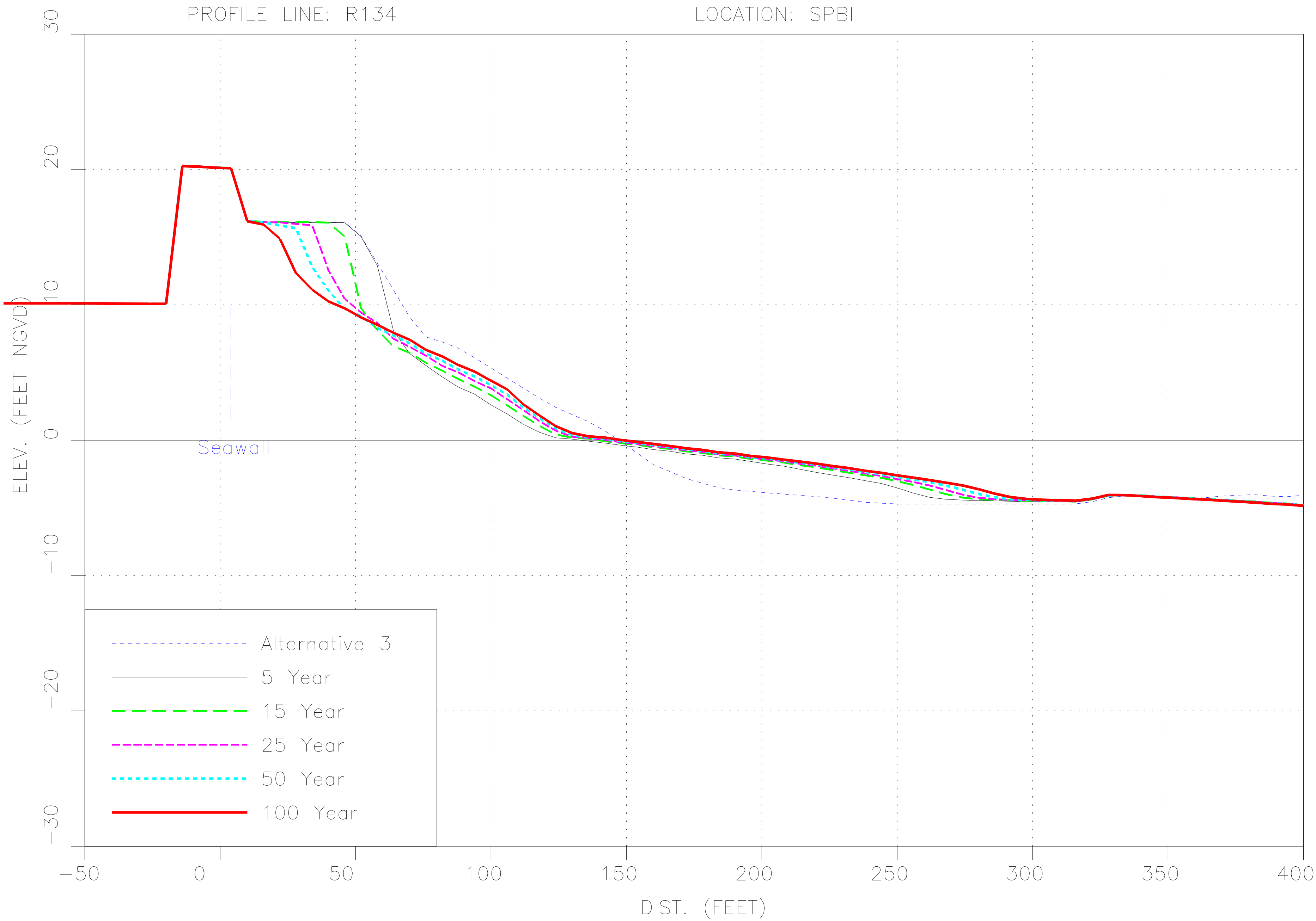
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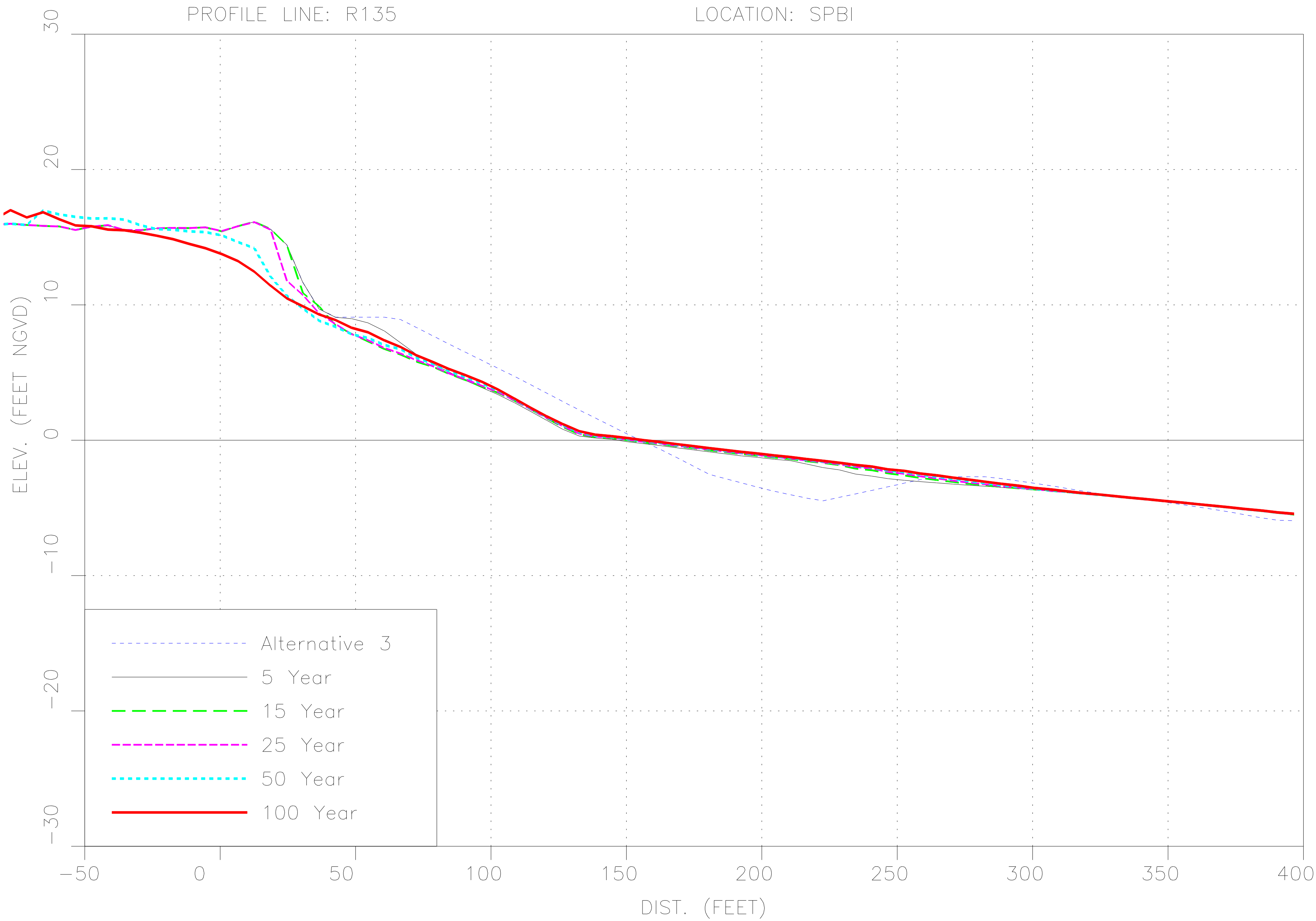
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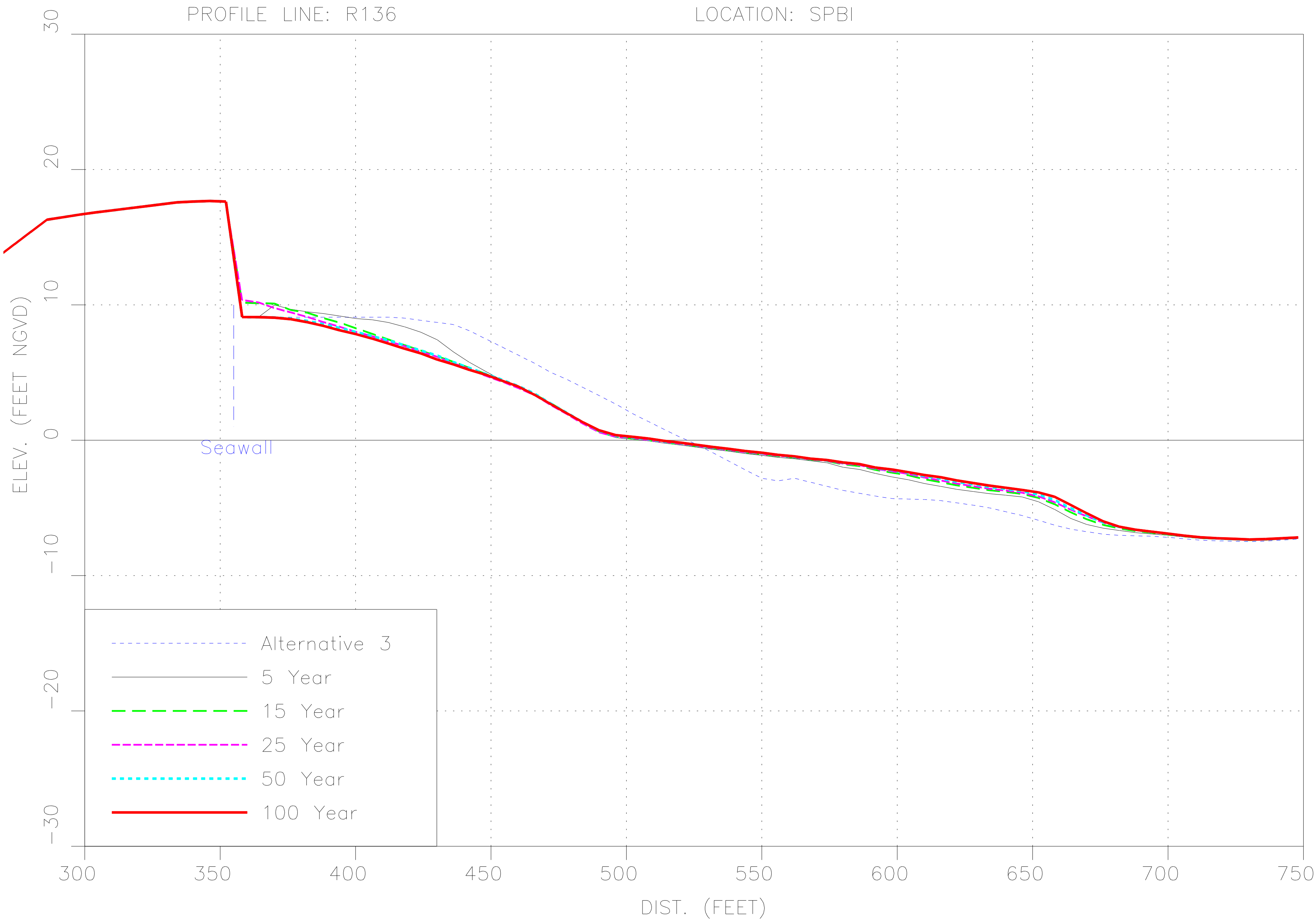

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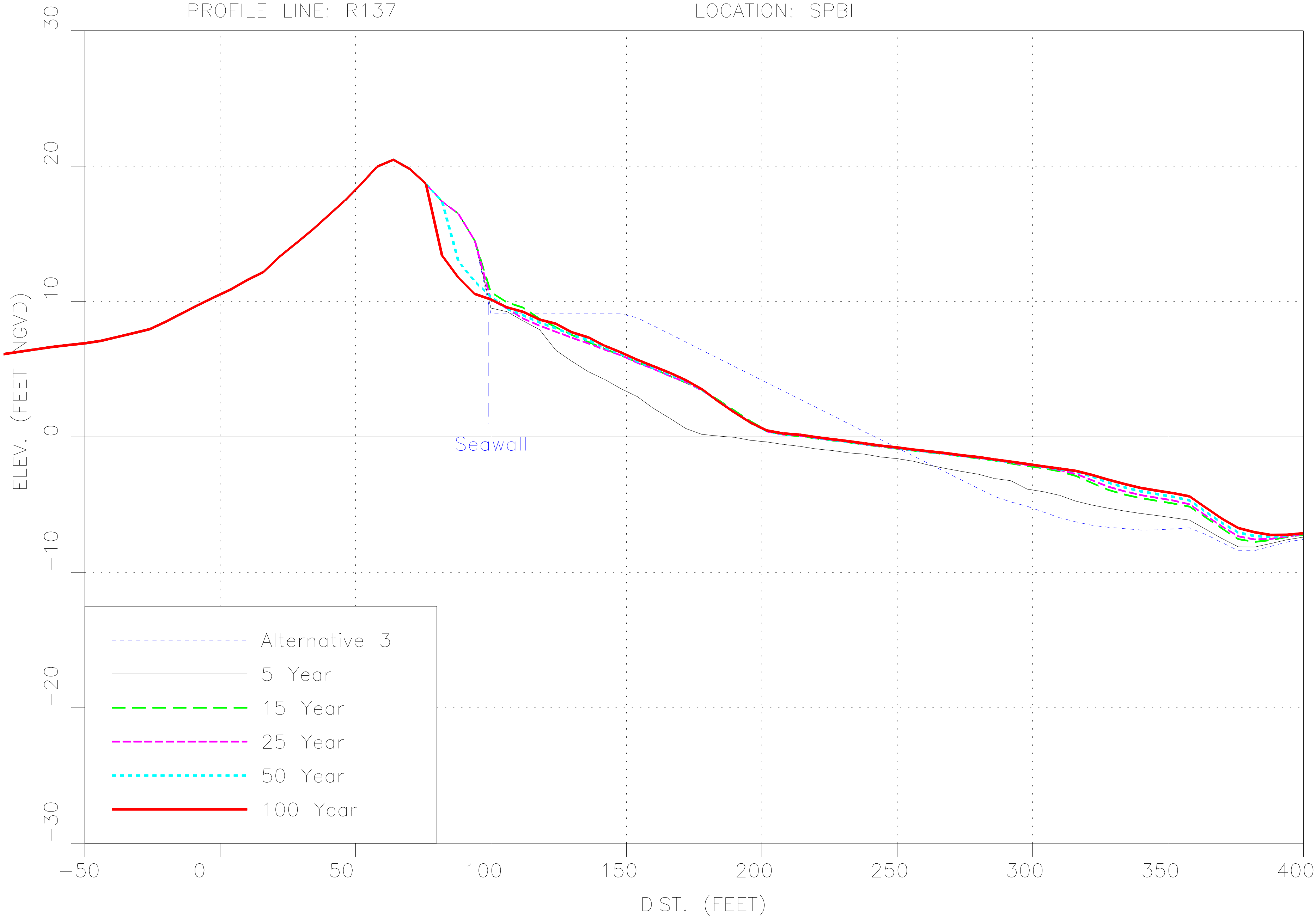
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Alternative 3
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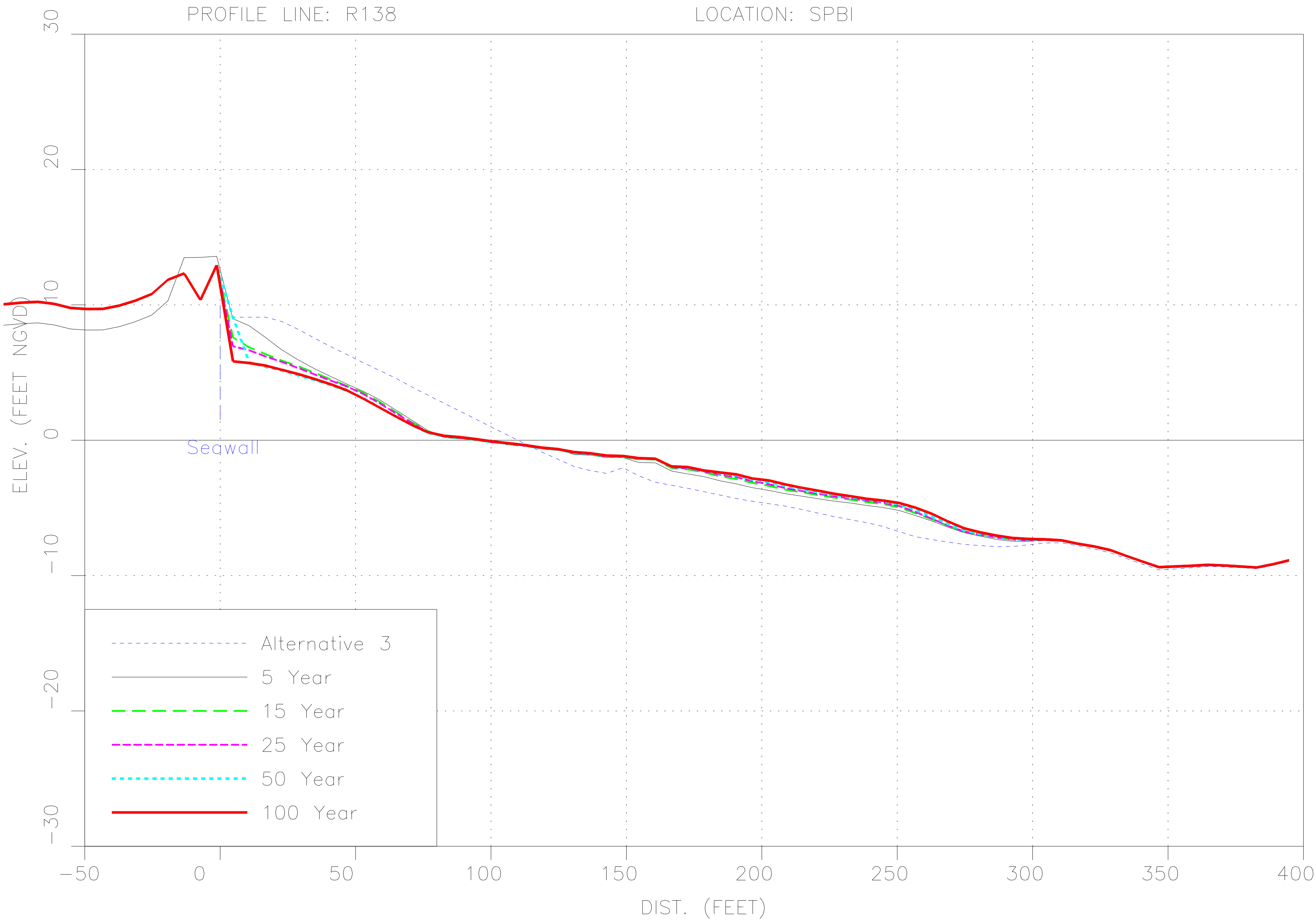

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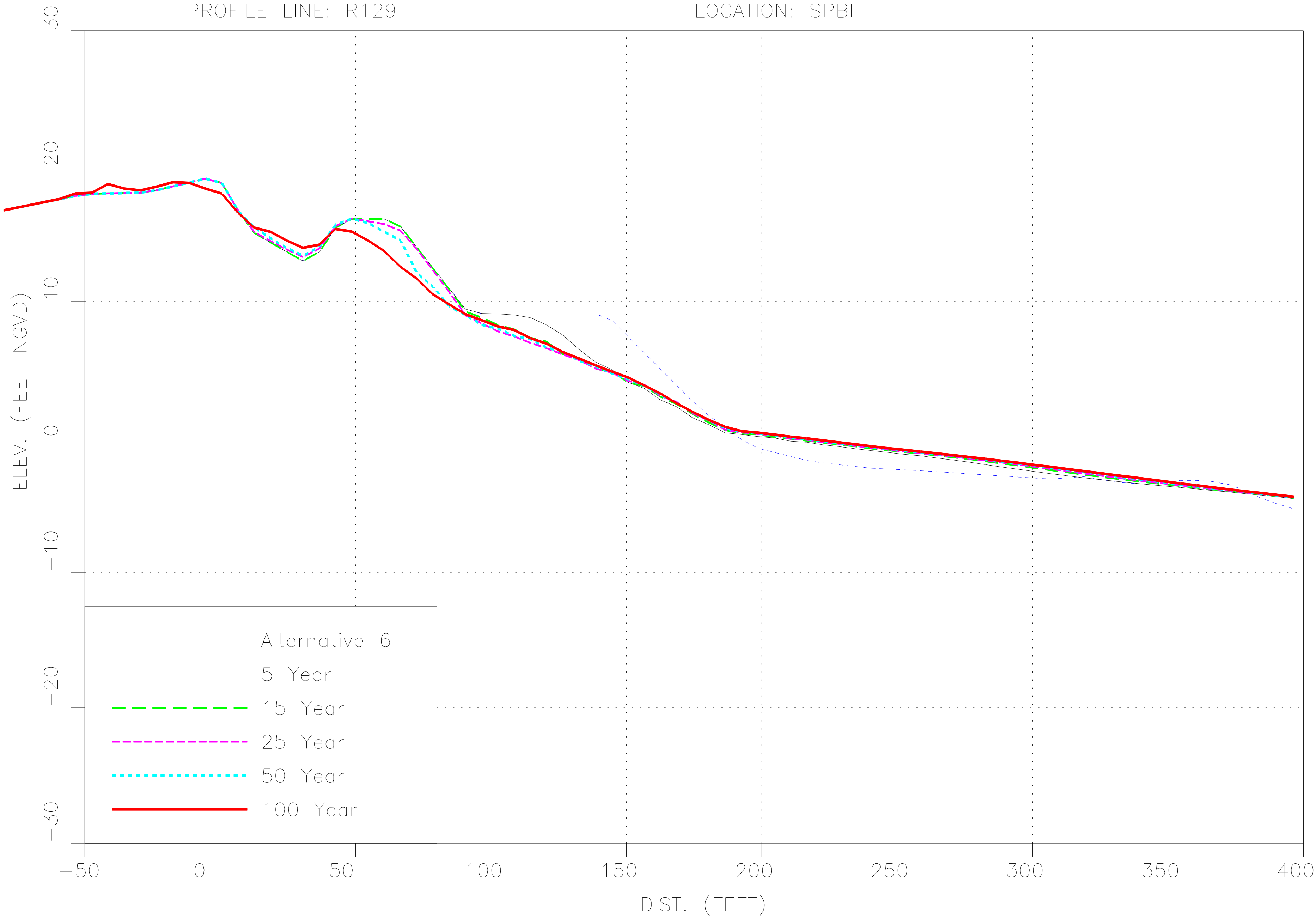
SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT C-4

ALTERNATIVE 6 (LARGER FILL DESIGN)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25 Year
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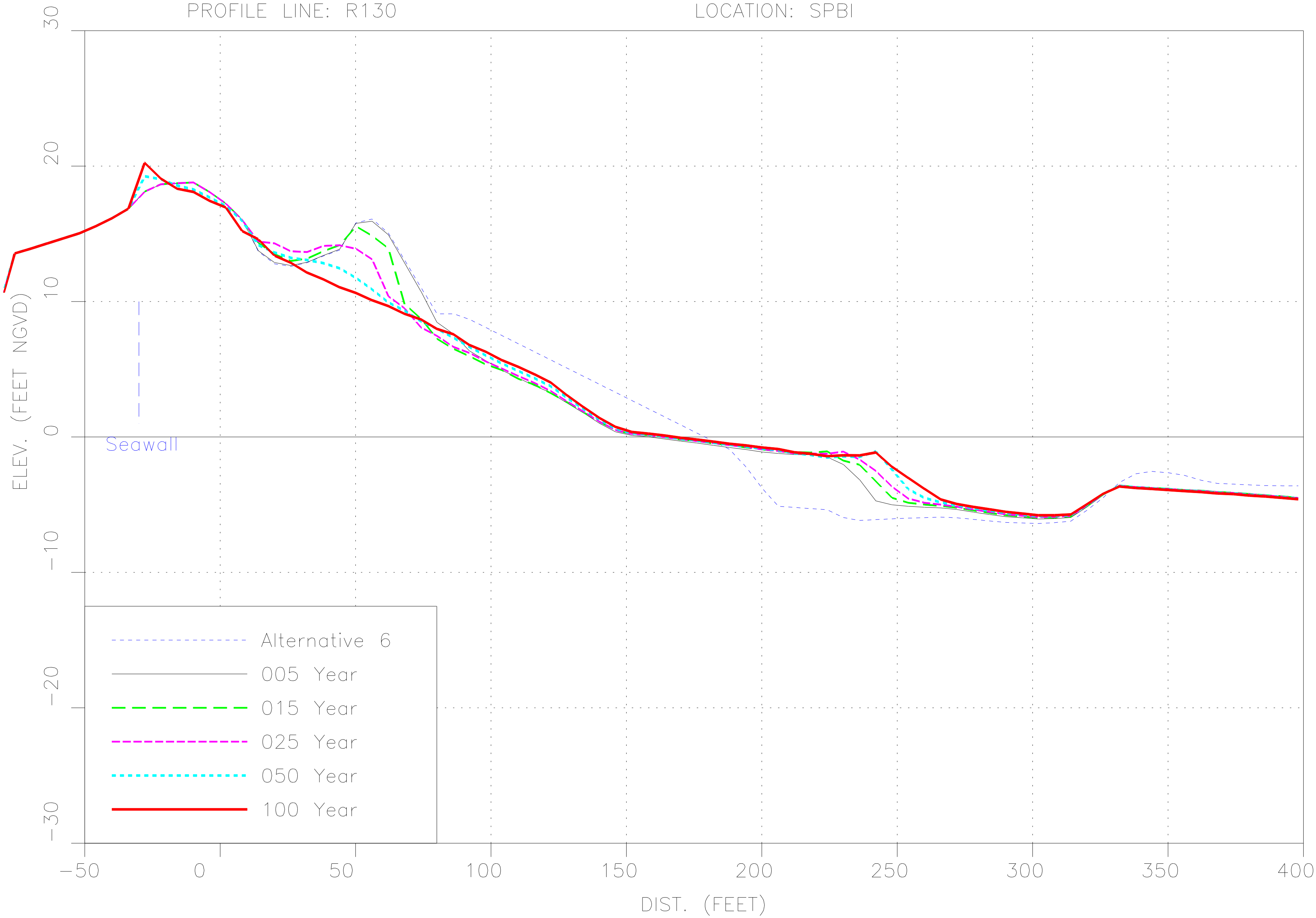
100 Year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Seawall
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015 Year
025 Year
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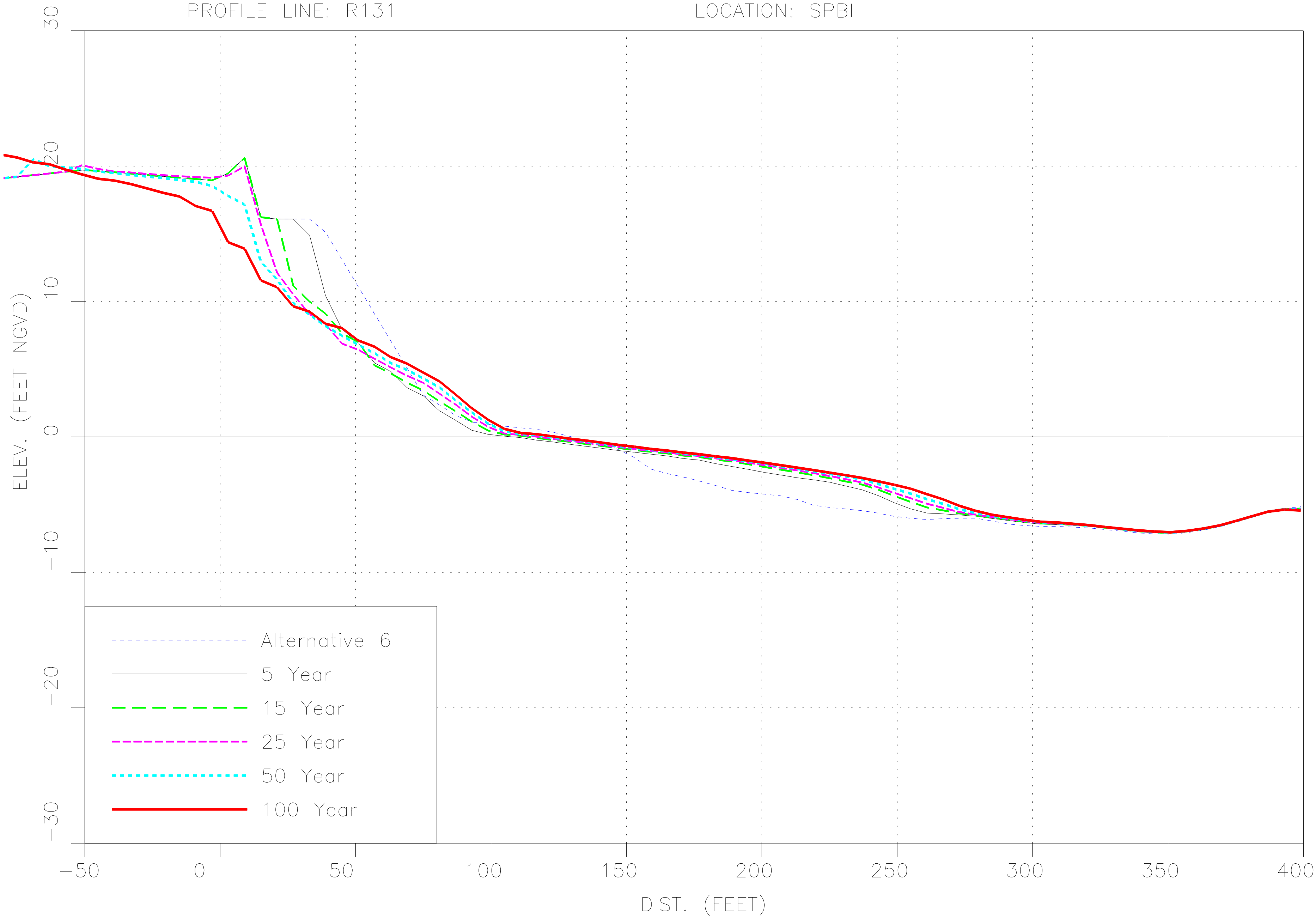
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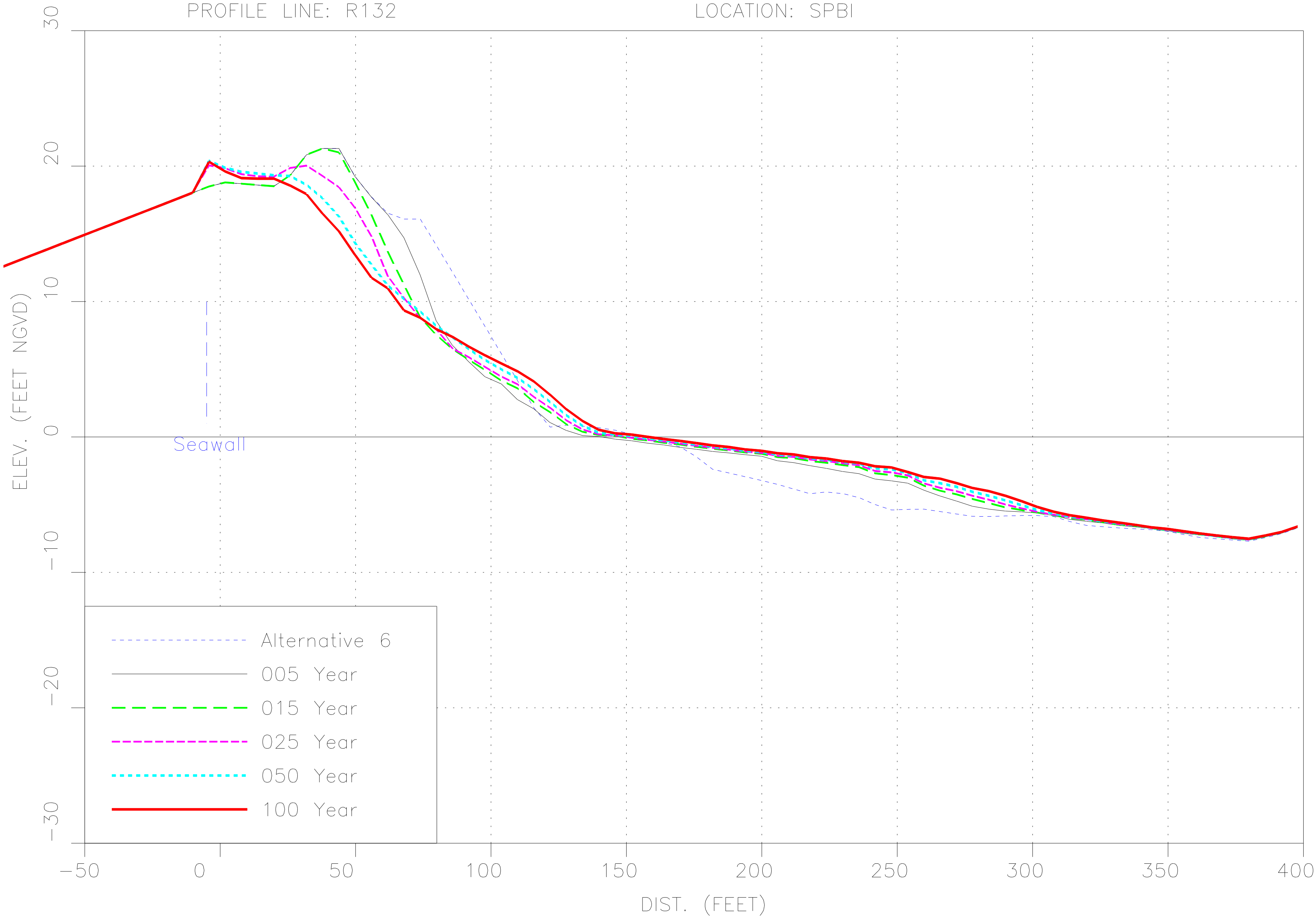
100 Year



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Alternative 6
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025 Year
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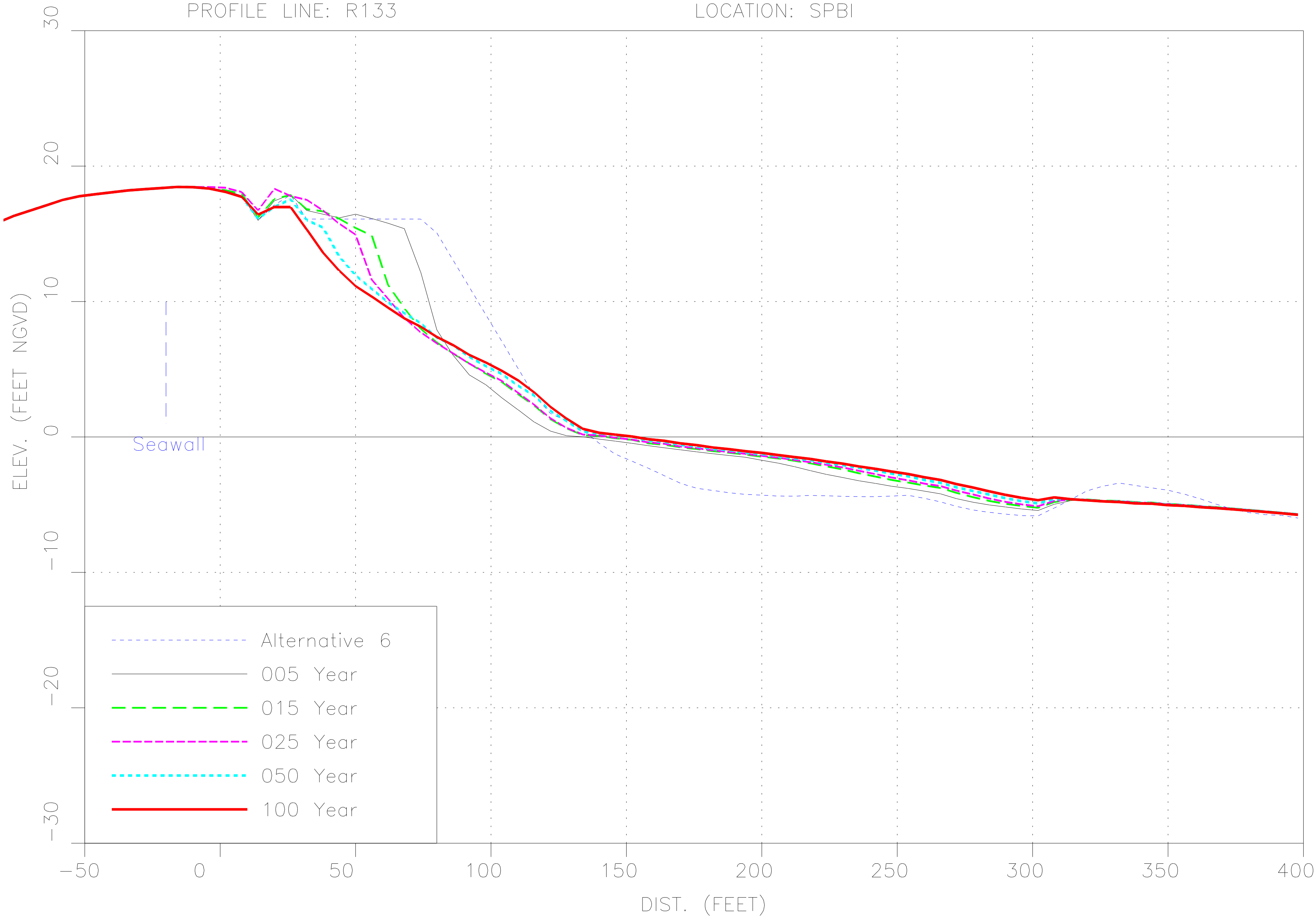
100 Year



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DIST. (FEET)
Alternative 6
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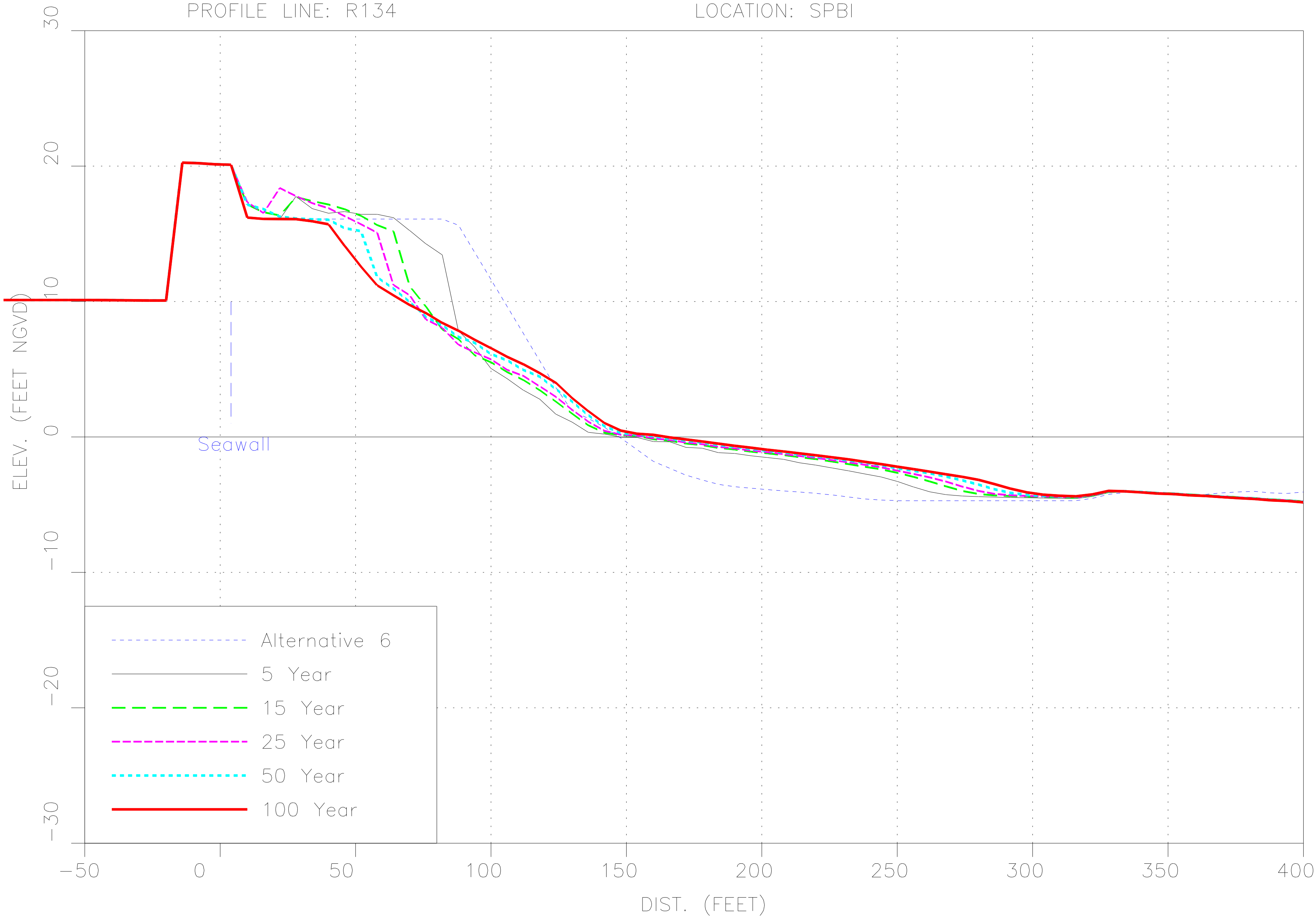
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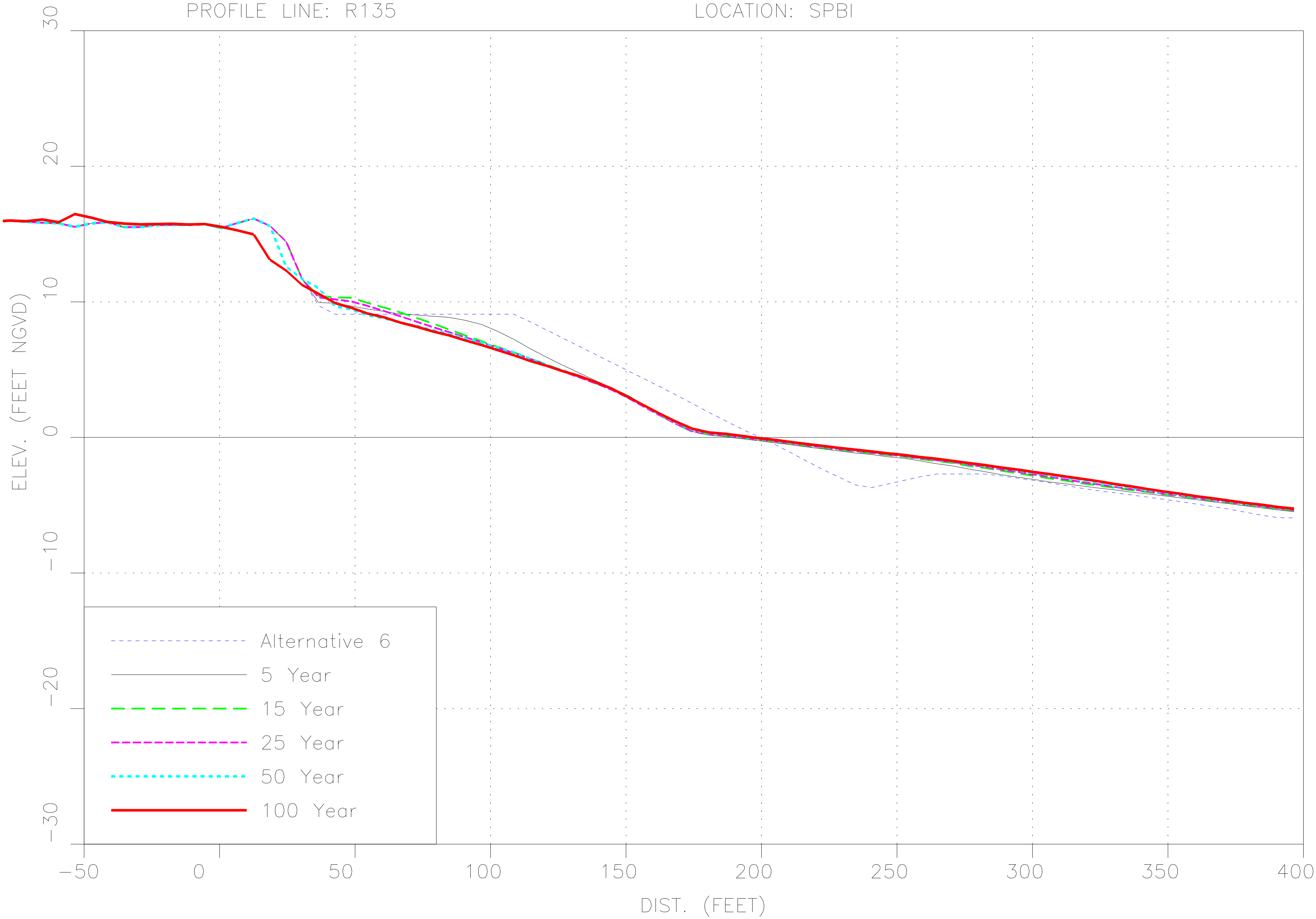

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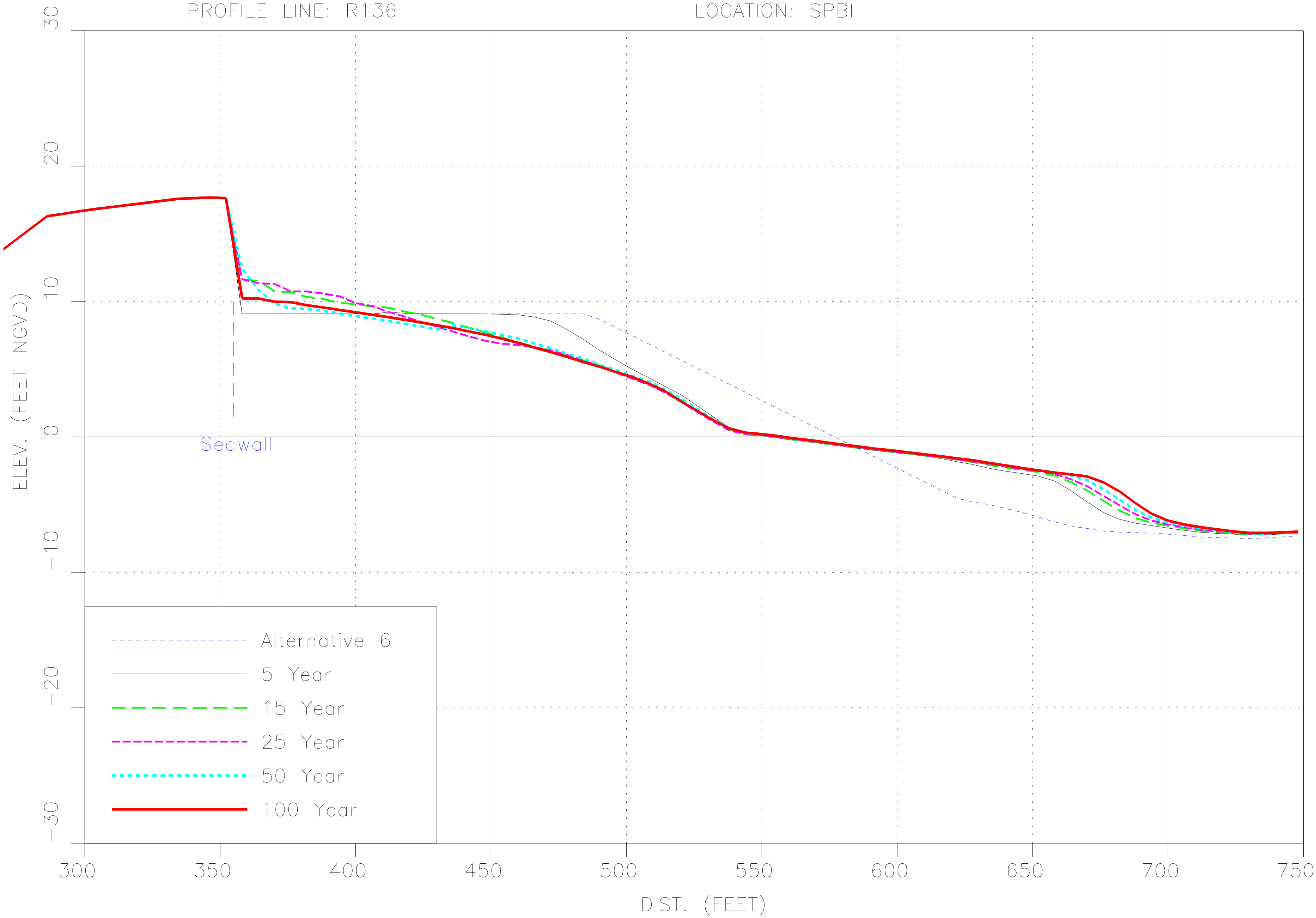
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Seawall
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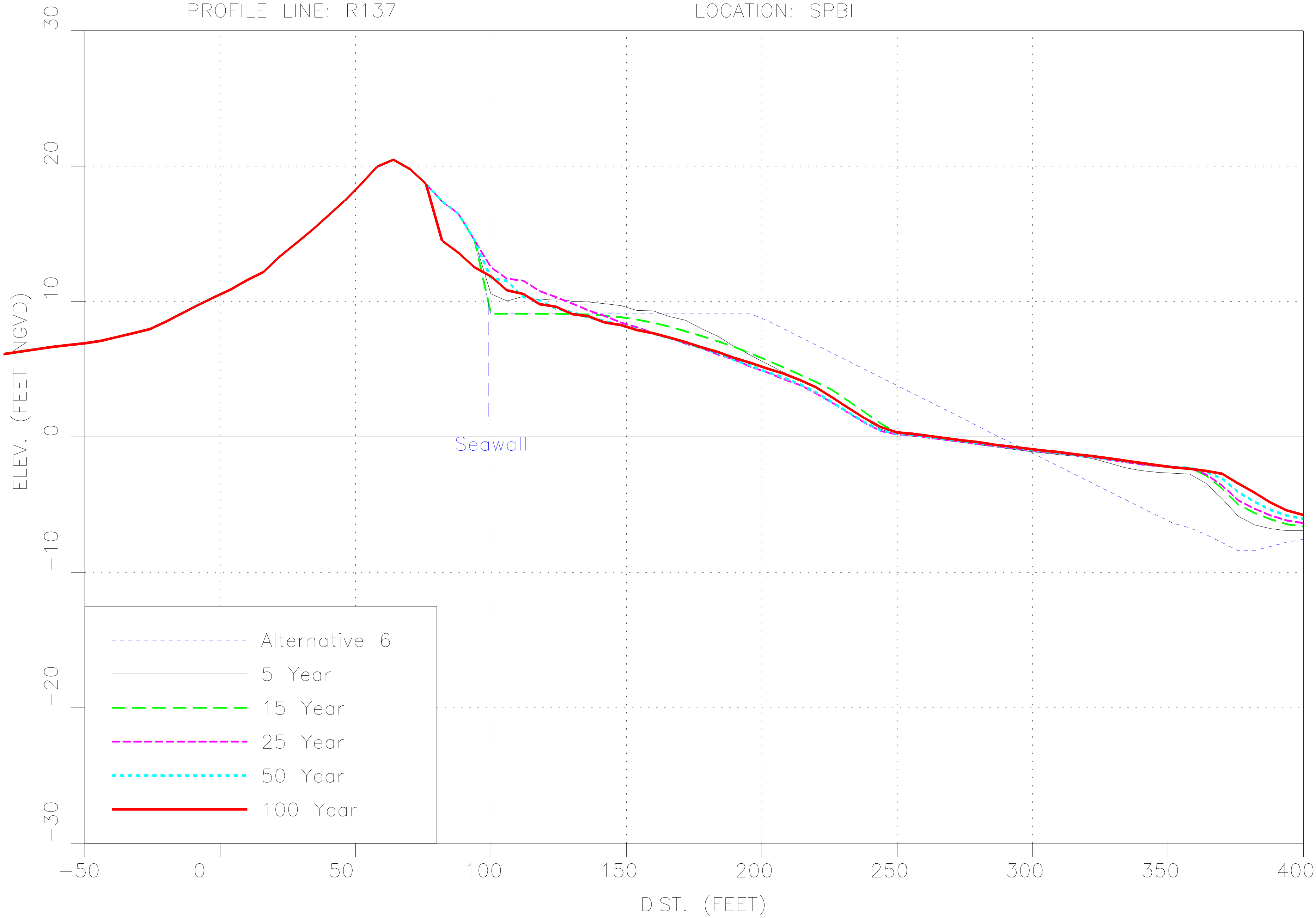
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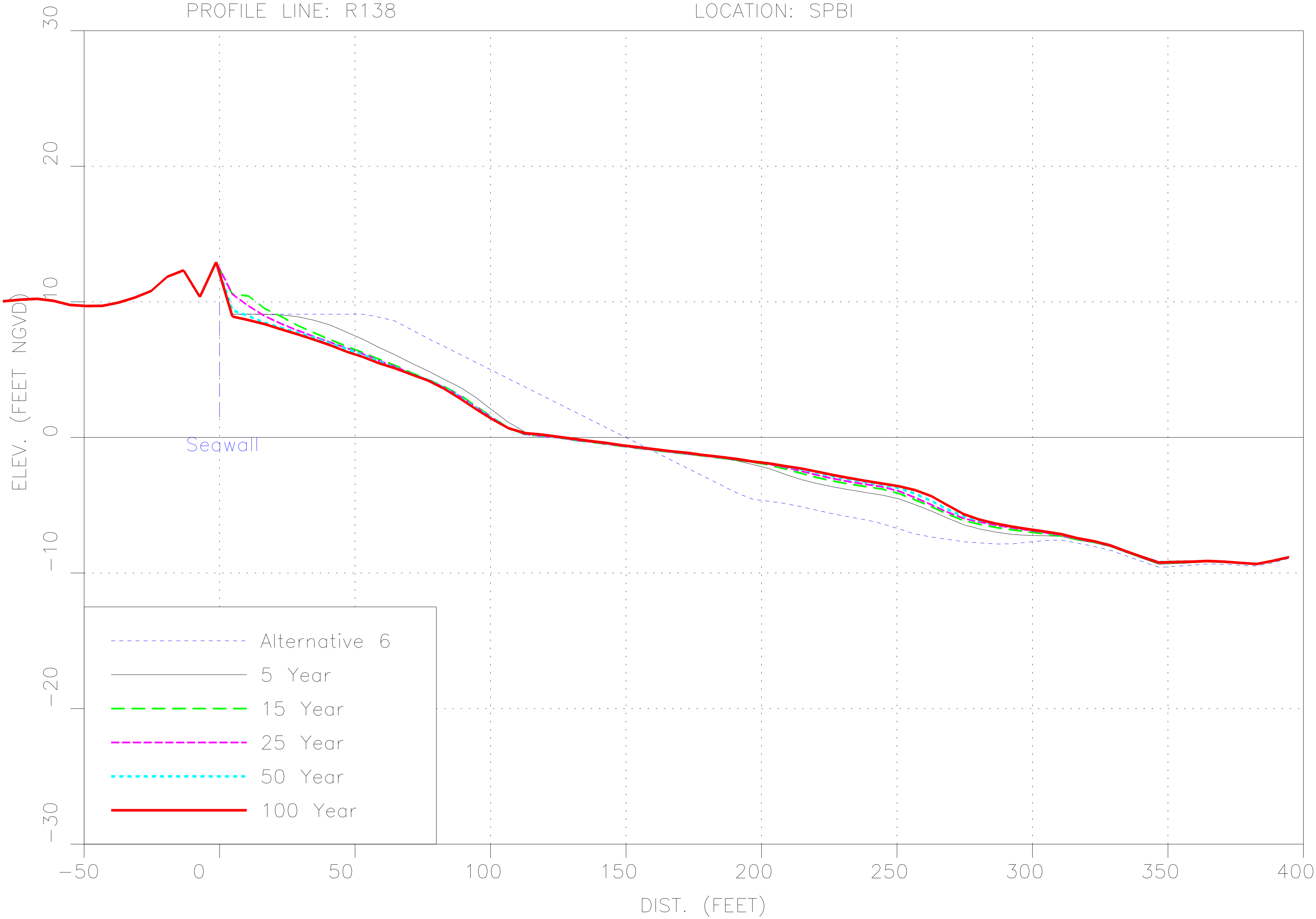

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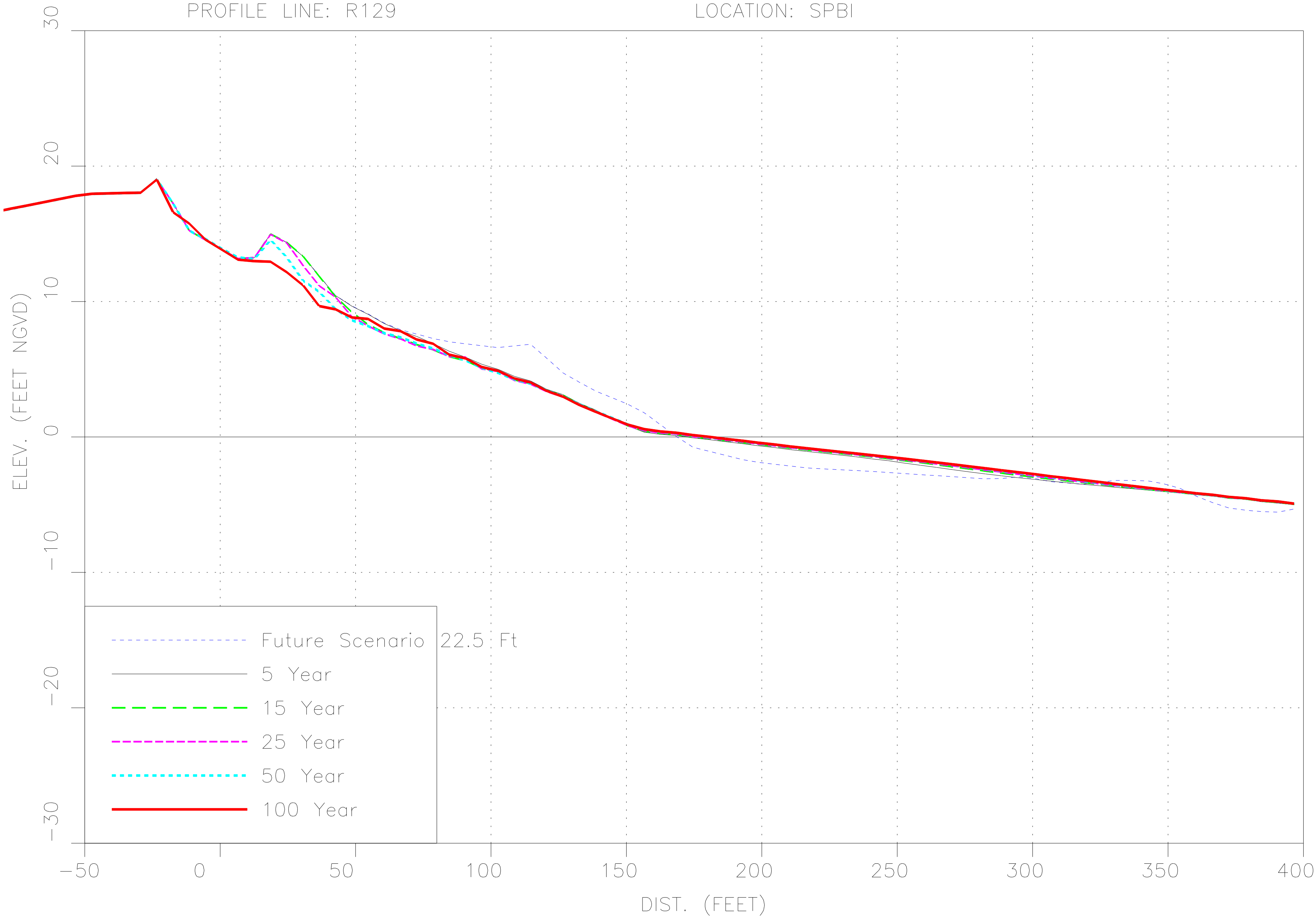
SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT C-5

FUTURE SCENARIO (WITHOUT PROJECT CONDITIONS)



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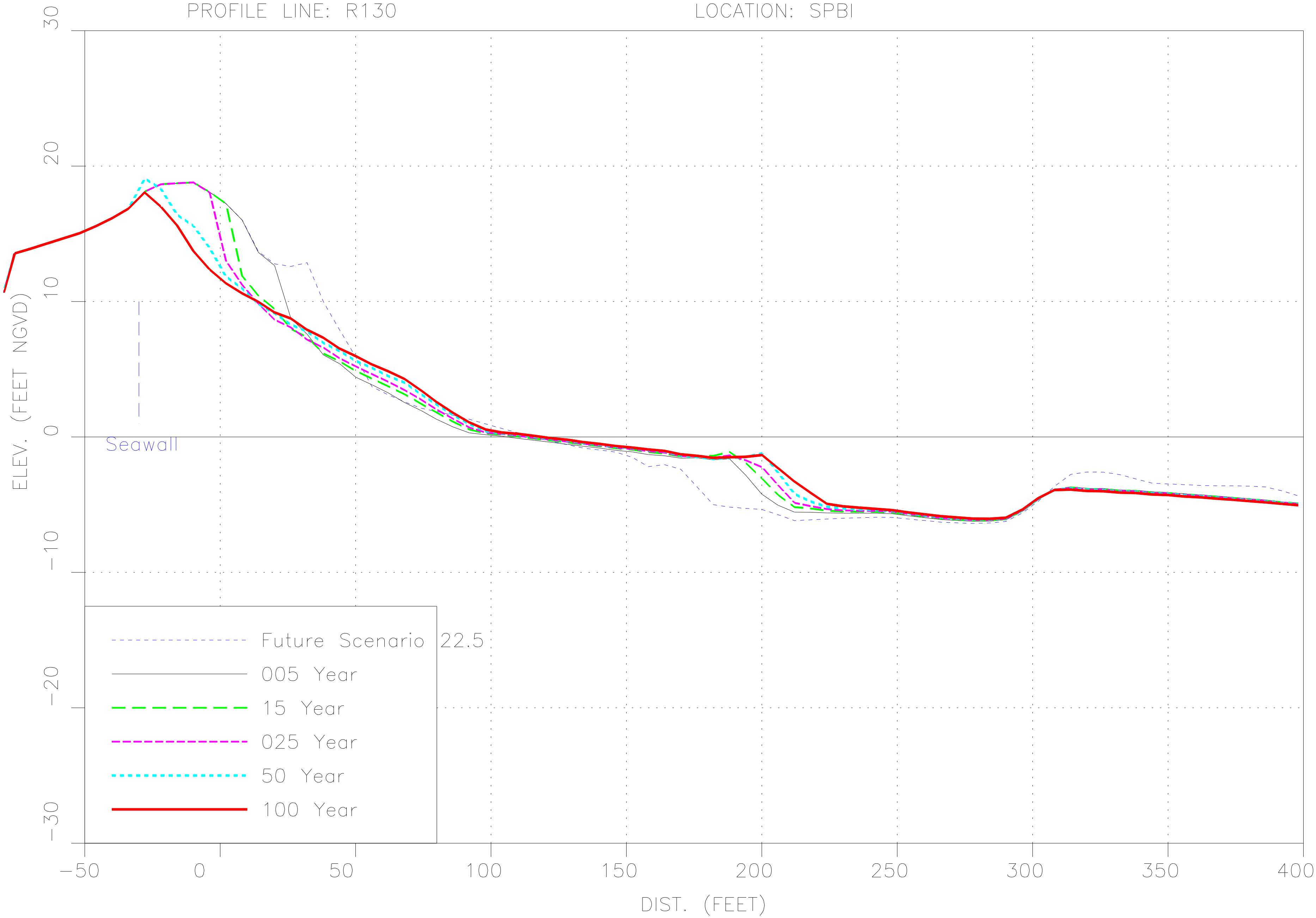
5 Year
15 Year
25 Year
50 Year
100 Year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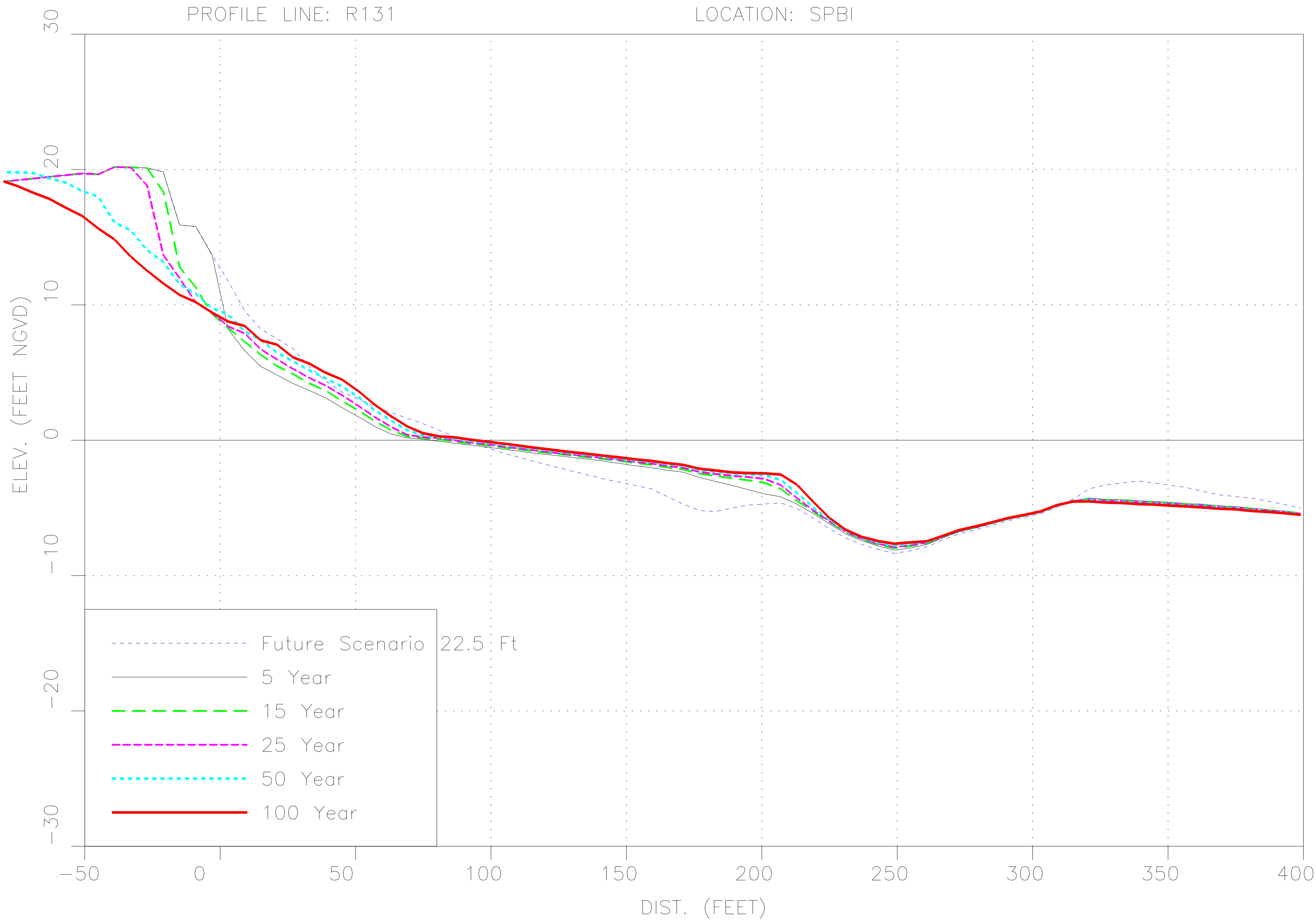
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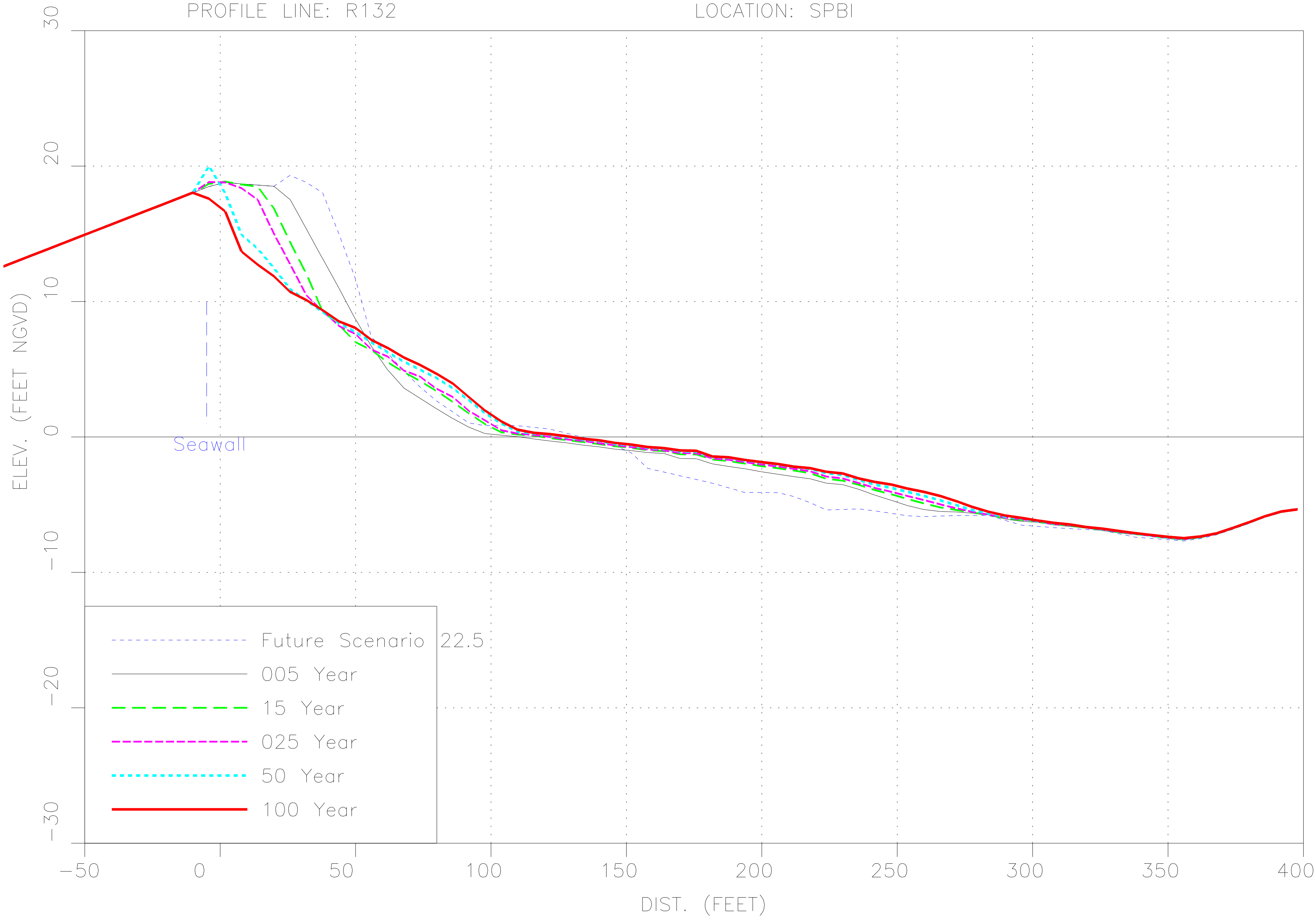
25 Year
50 Year
100 Year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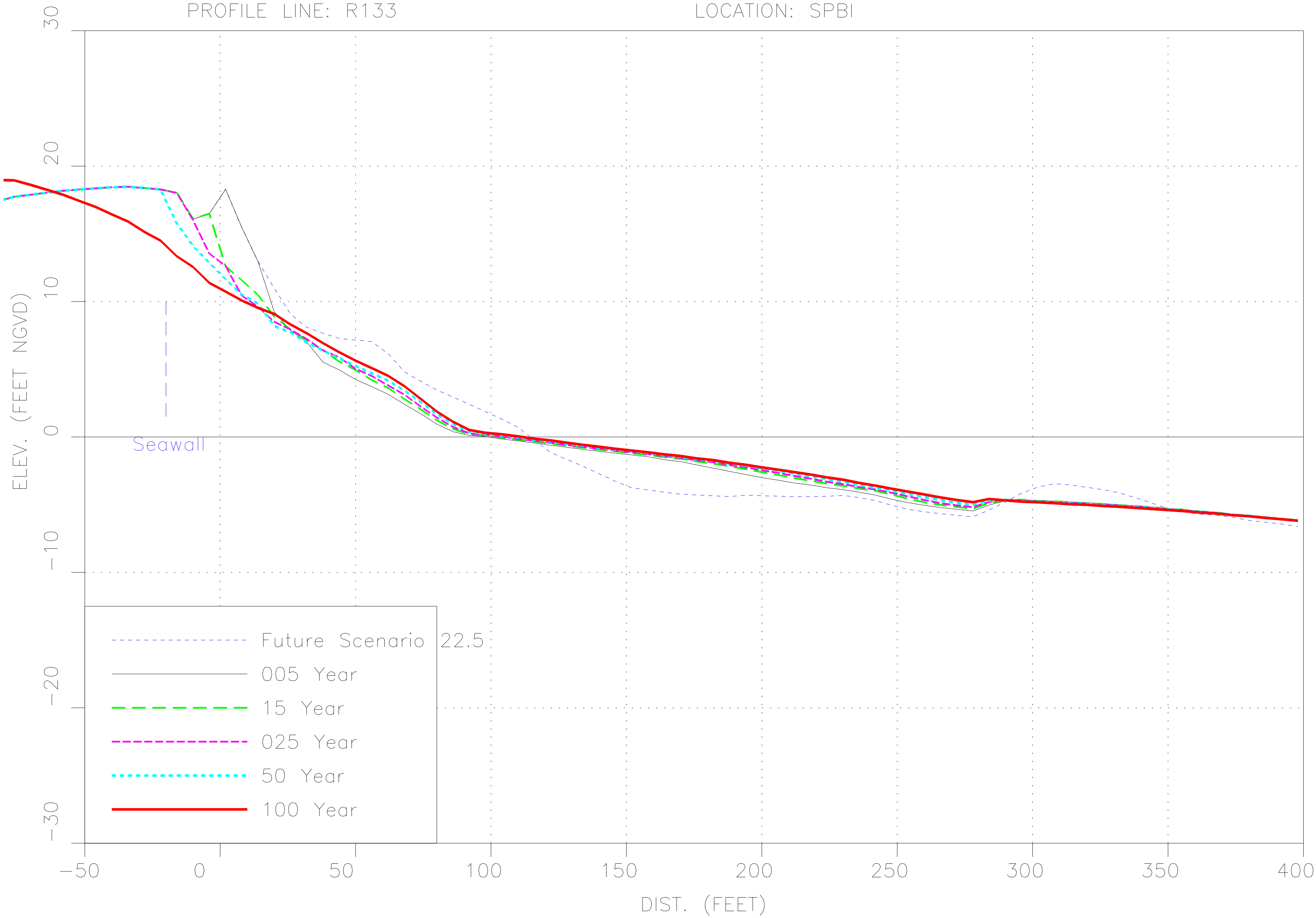

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Seawall
Future Scenario 22.5
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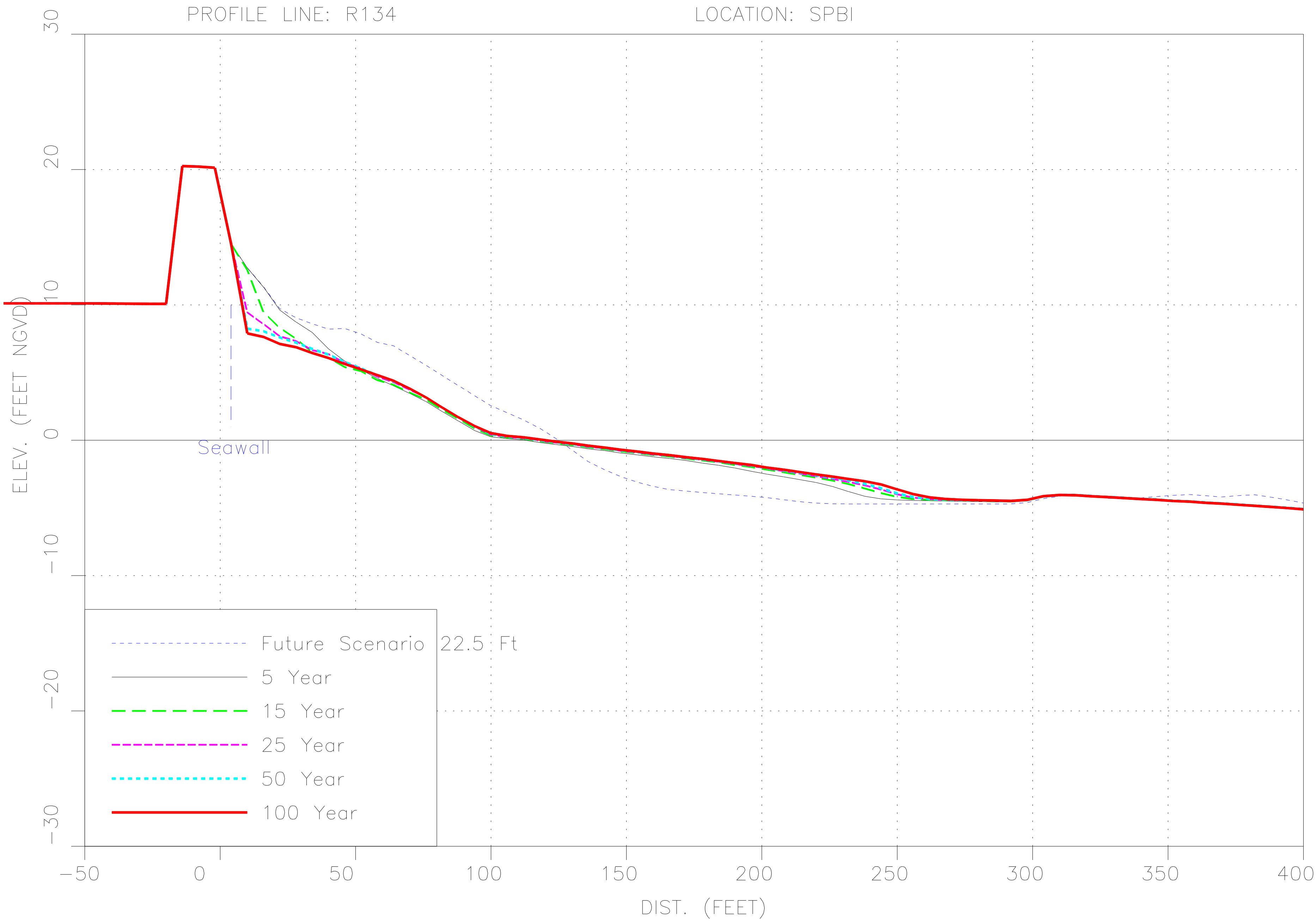
15 Year
025 Year
50 Year
100 Year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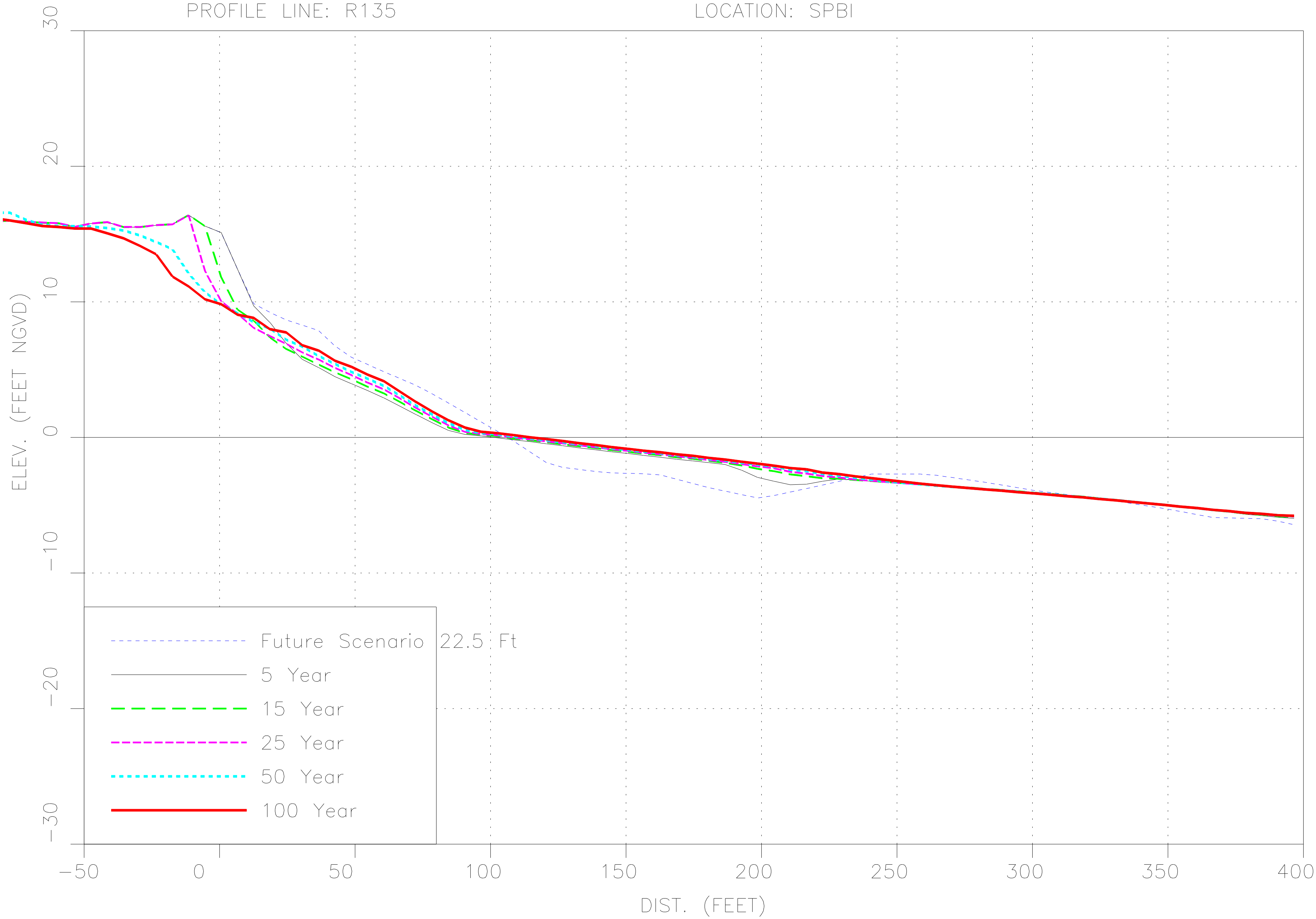

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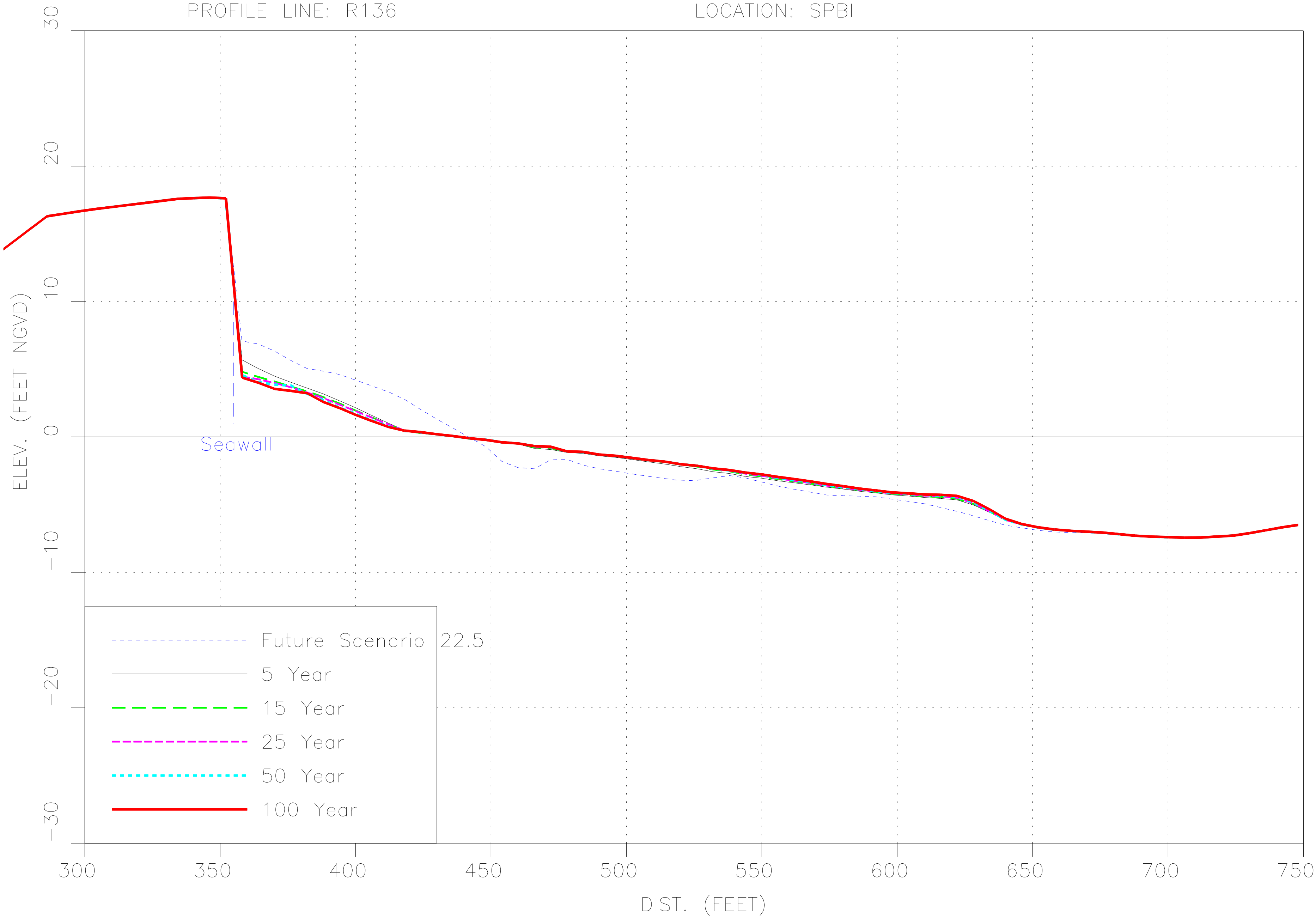
25 Year
50 Year
100 Year



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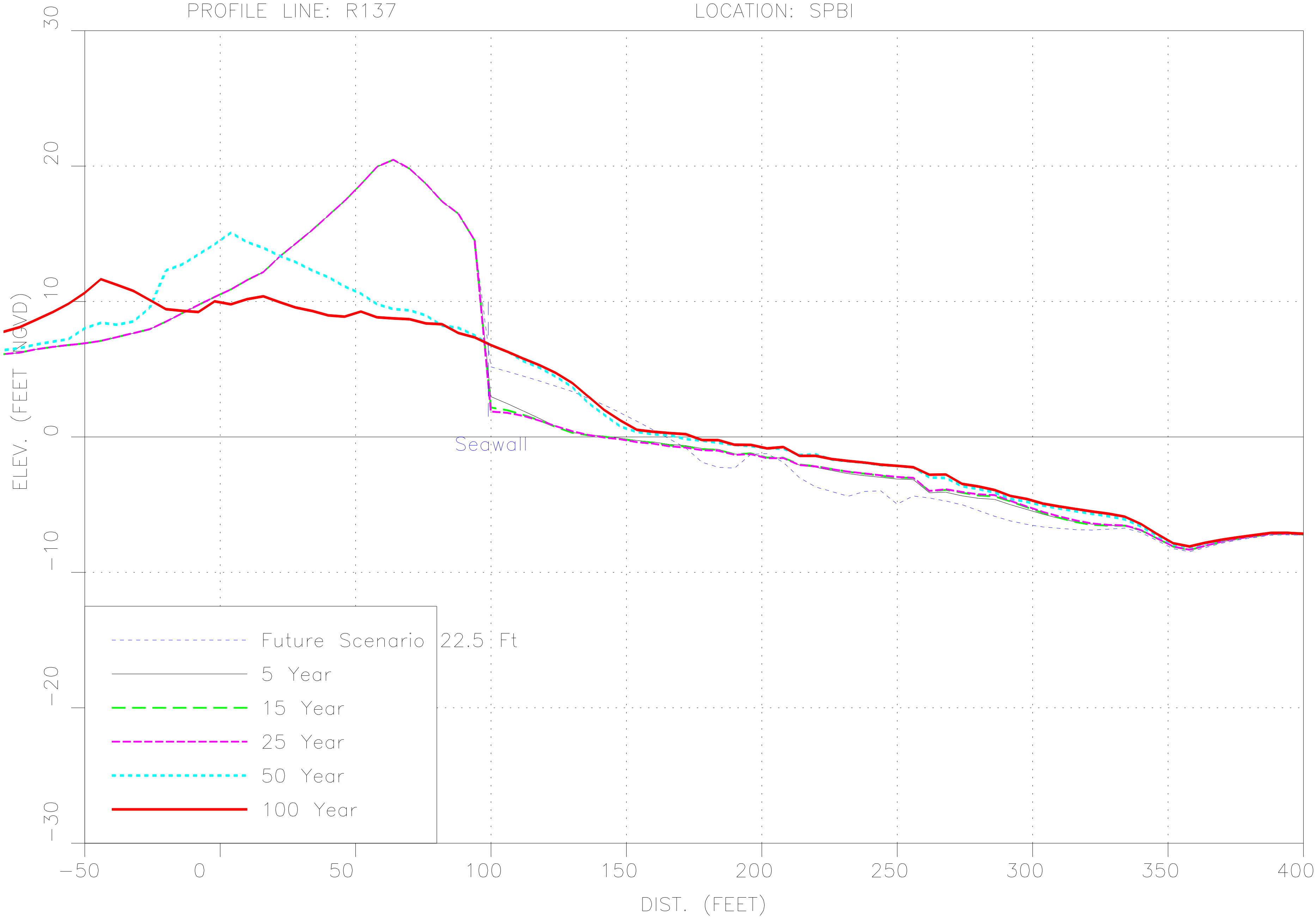
Seawall
Future Scenario 22.5
5 Year
15 Year
25 Year
50 Year
100 Year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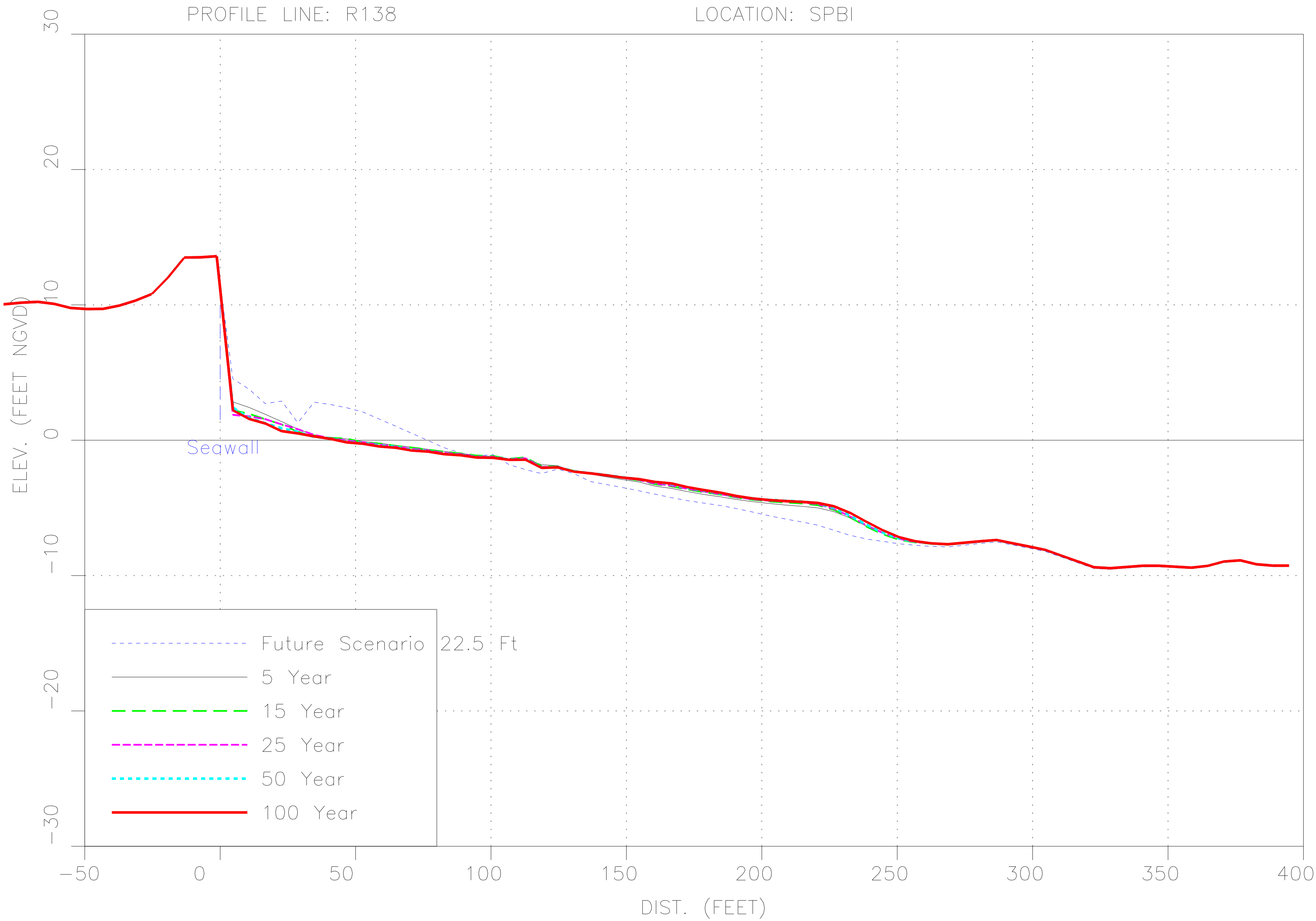

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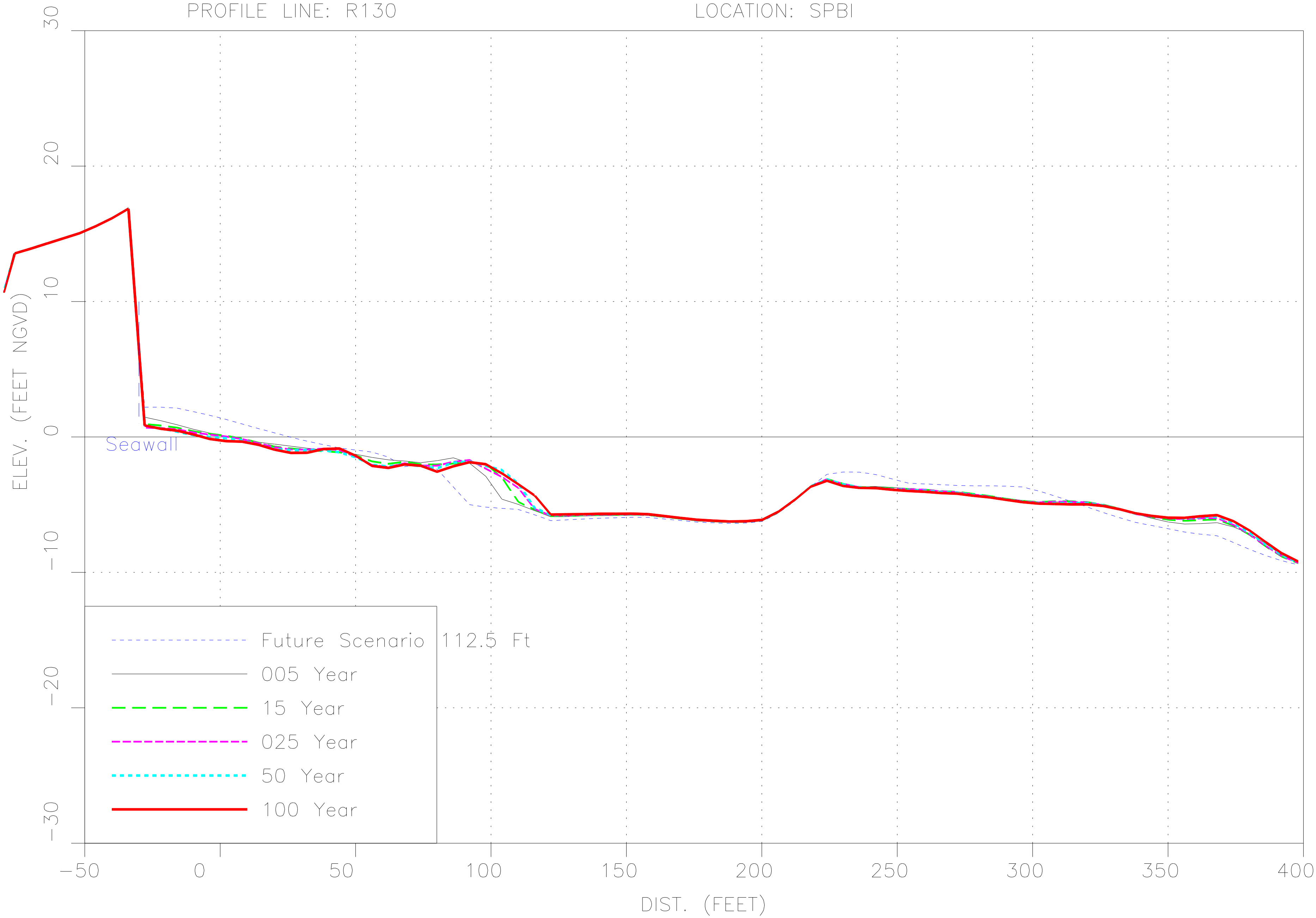
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005 Year
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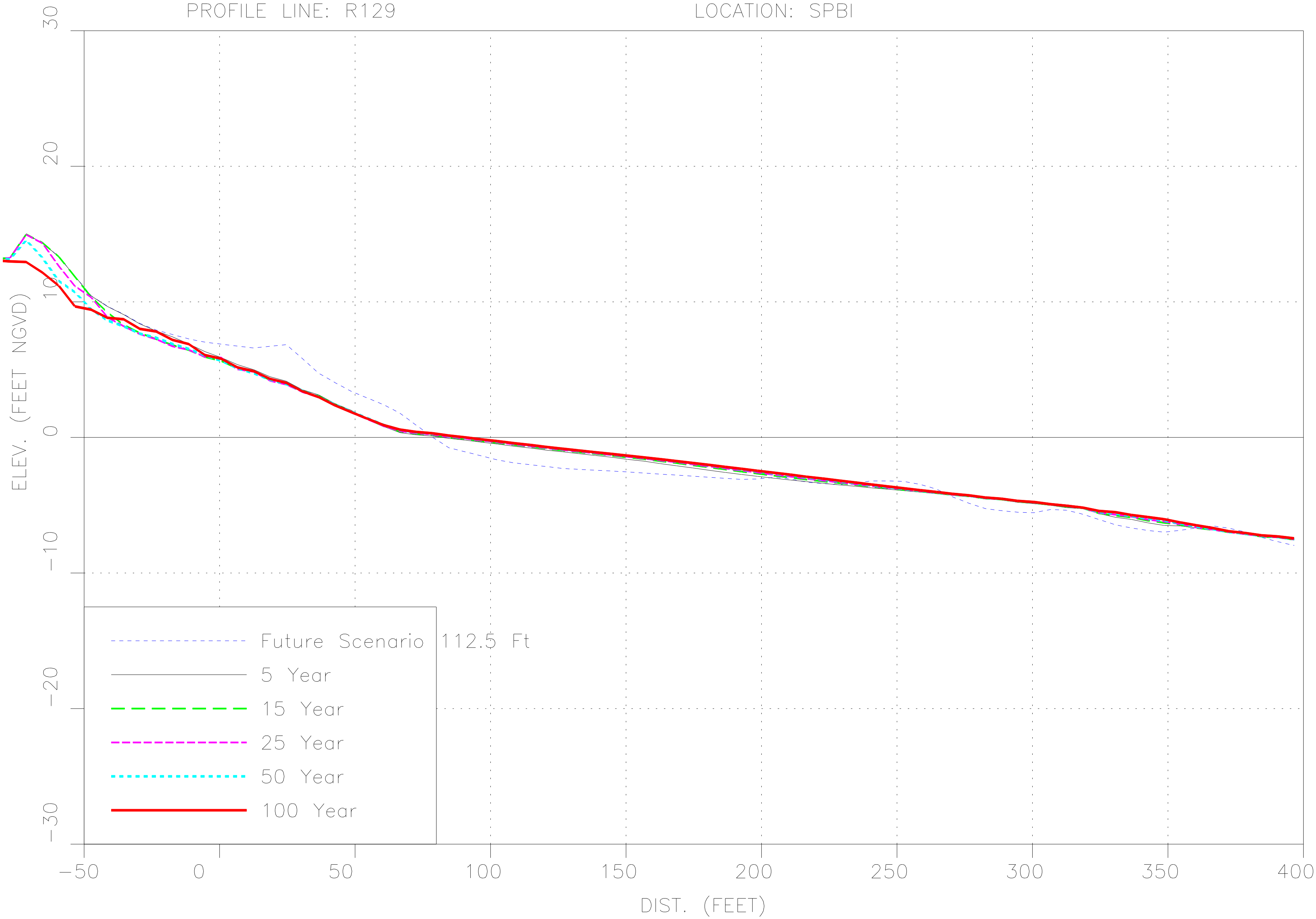
025 Year
50 Year
100 Year



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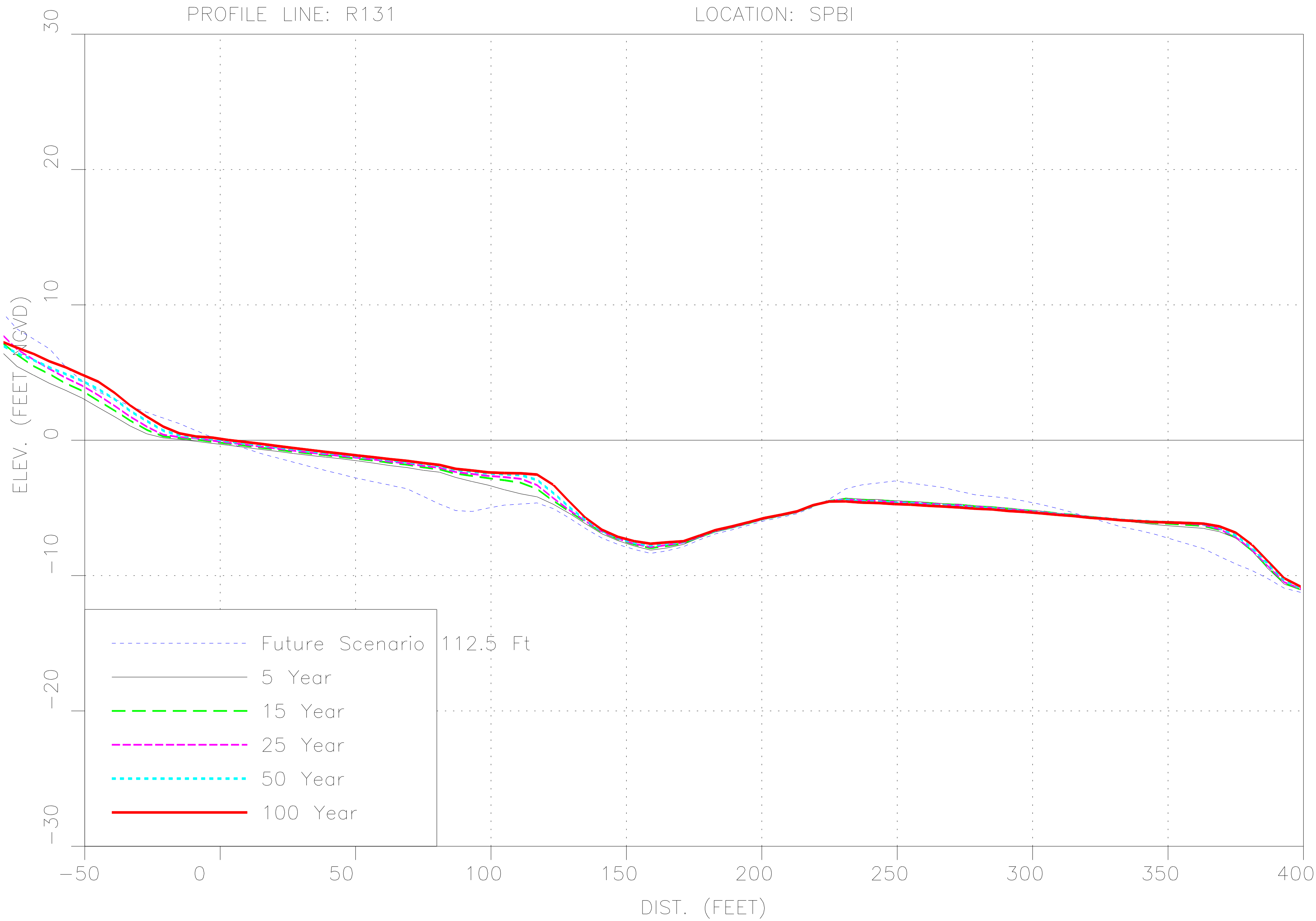

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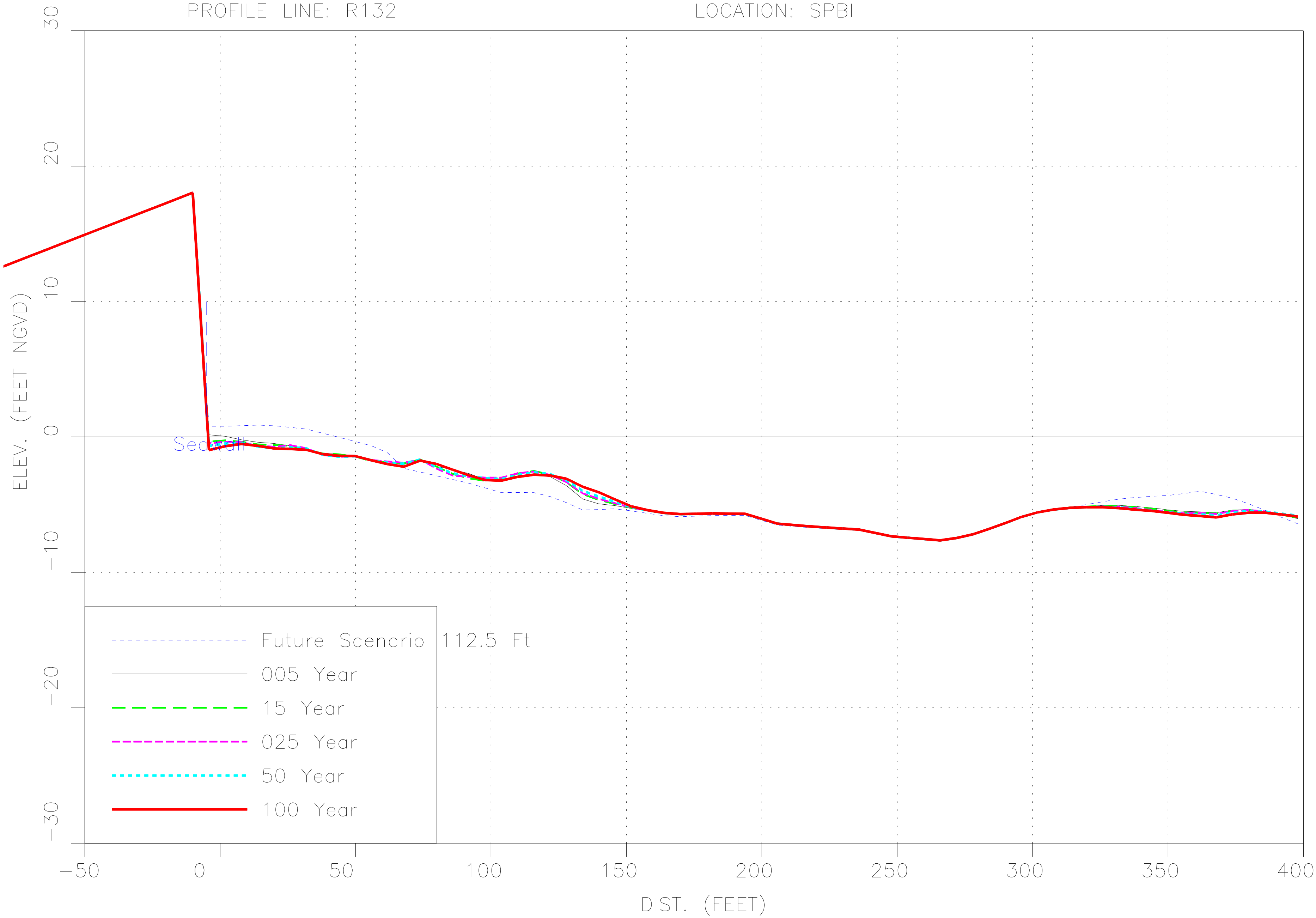
100 Year



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Seawall
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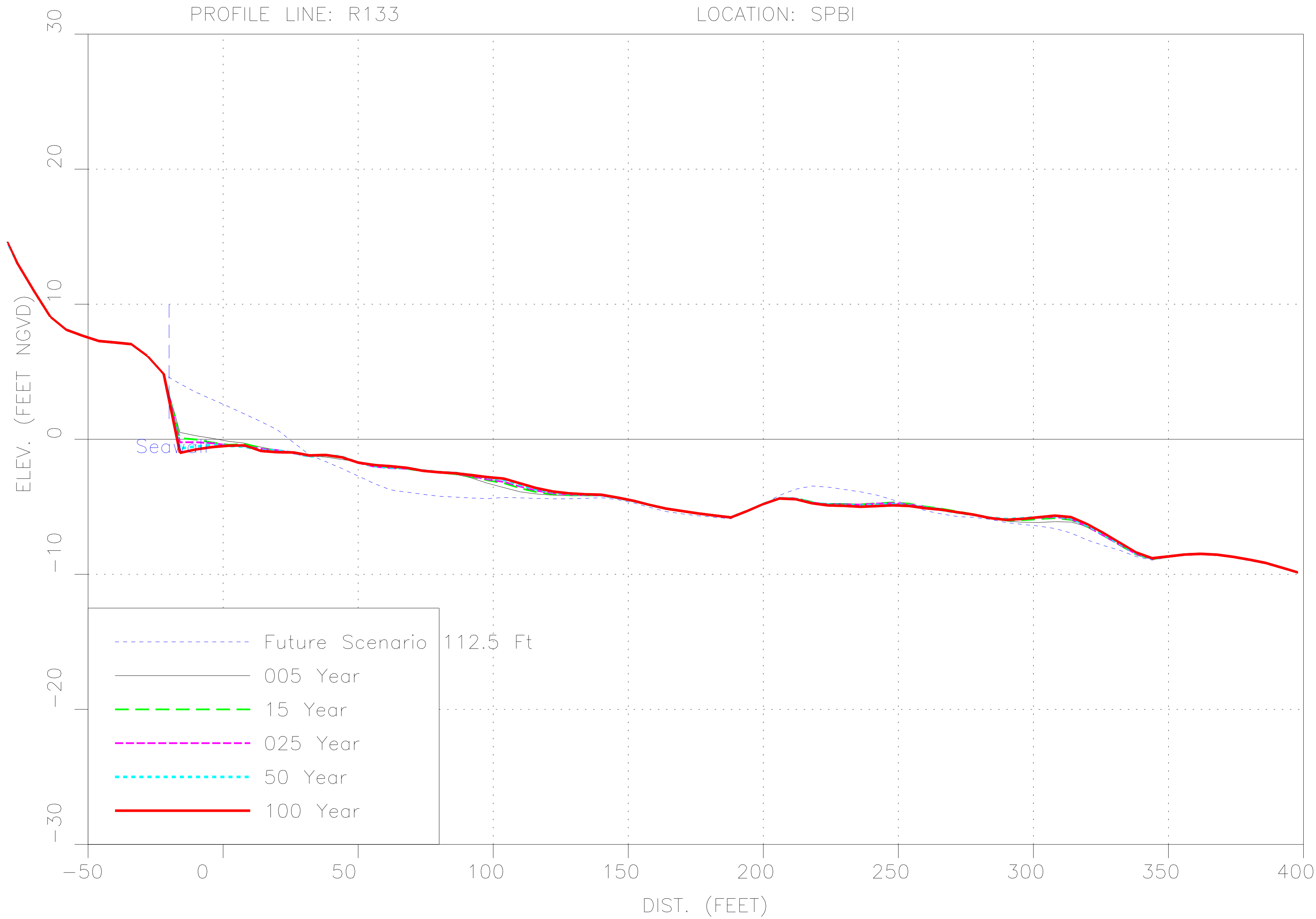
025 Year
50 Year
100 Year



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ELEV. (FEET NGVD)
DIST. (FEET)
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15 Year
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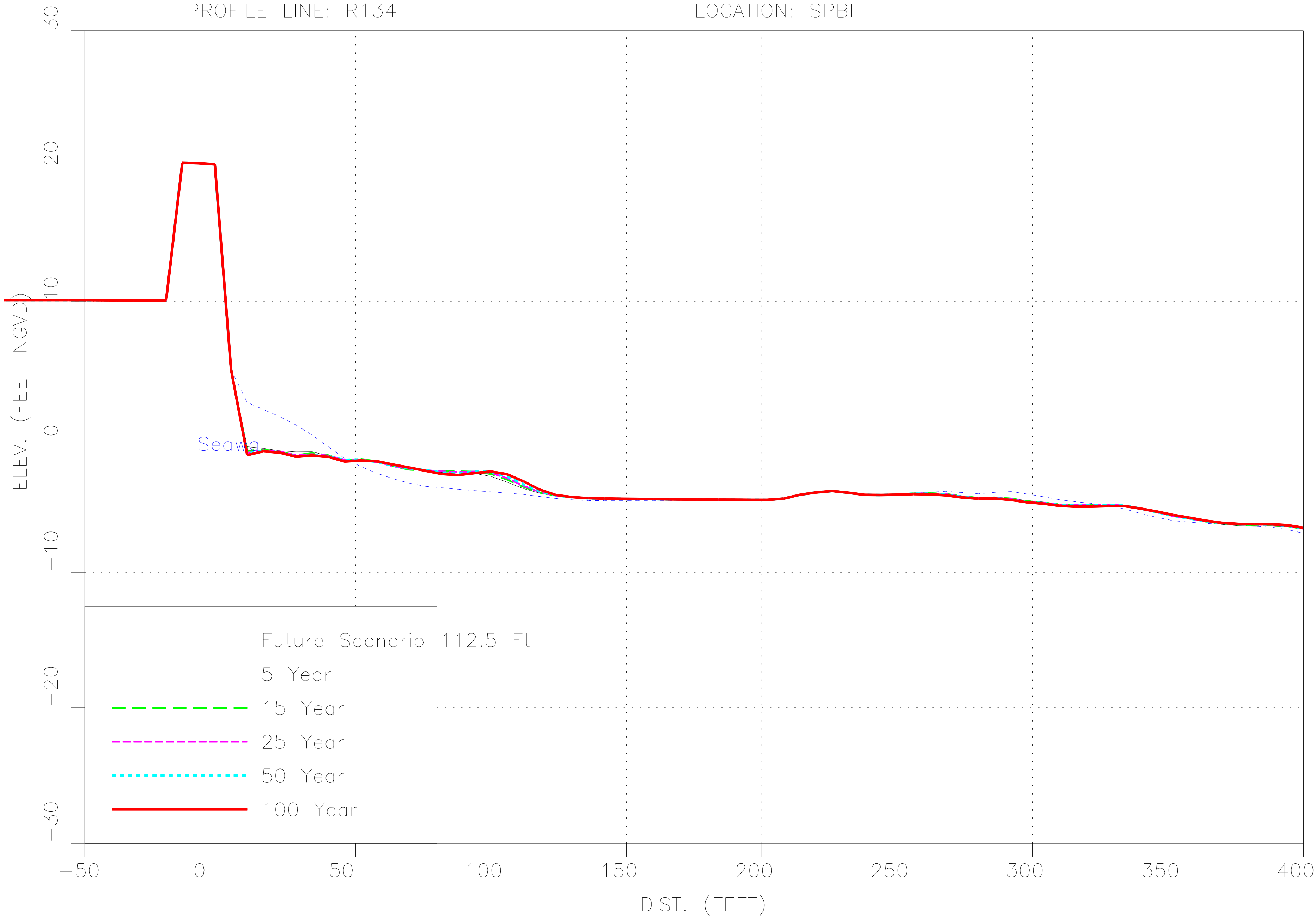

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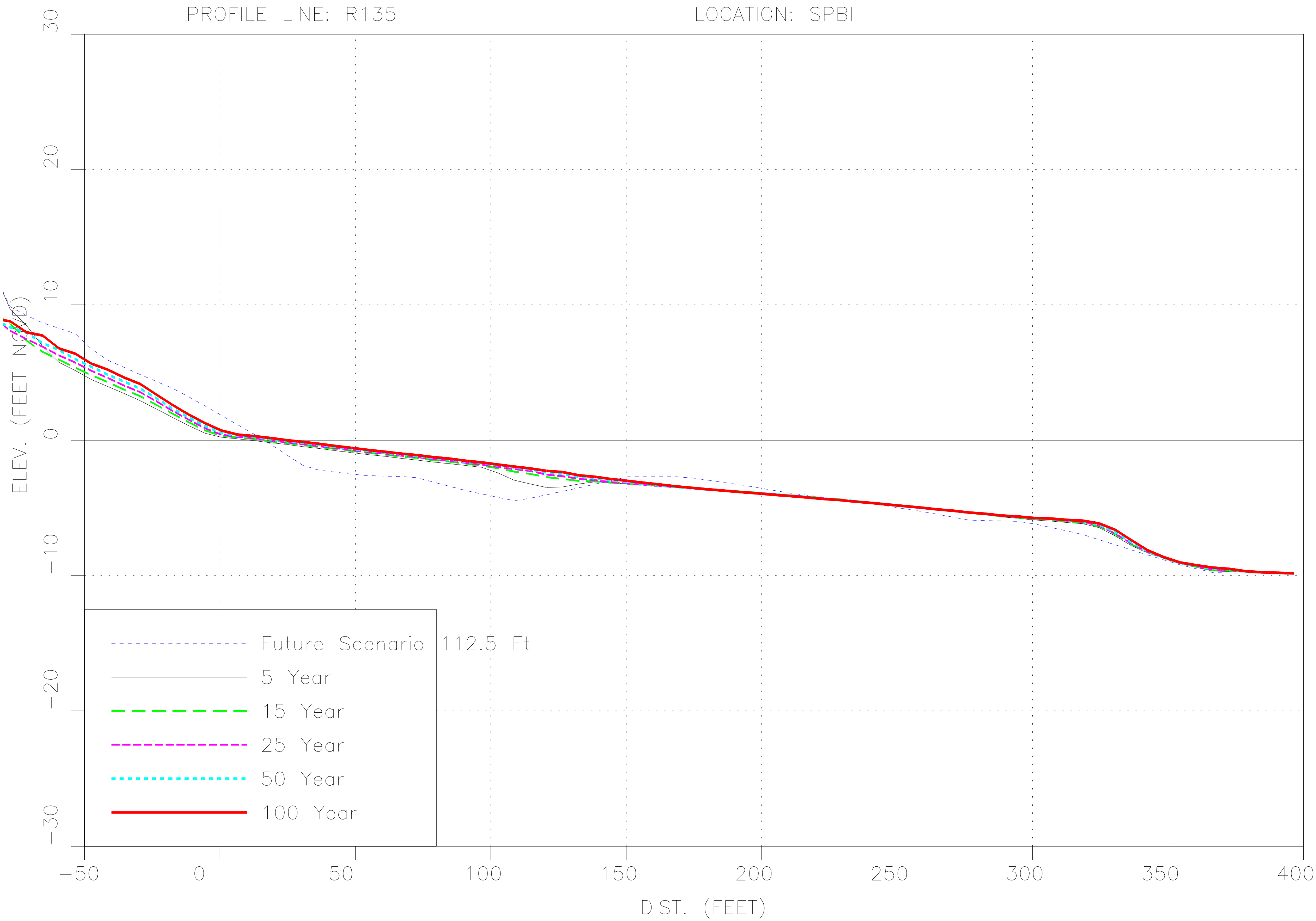

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Future Scenario 112.5 Ft
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50 Year
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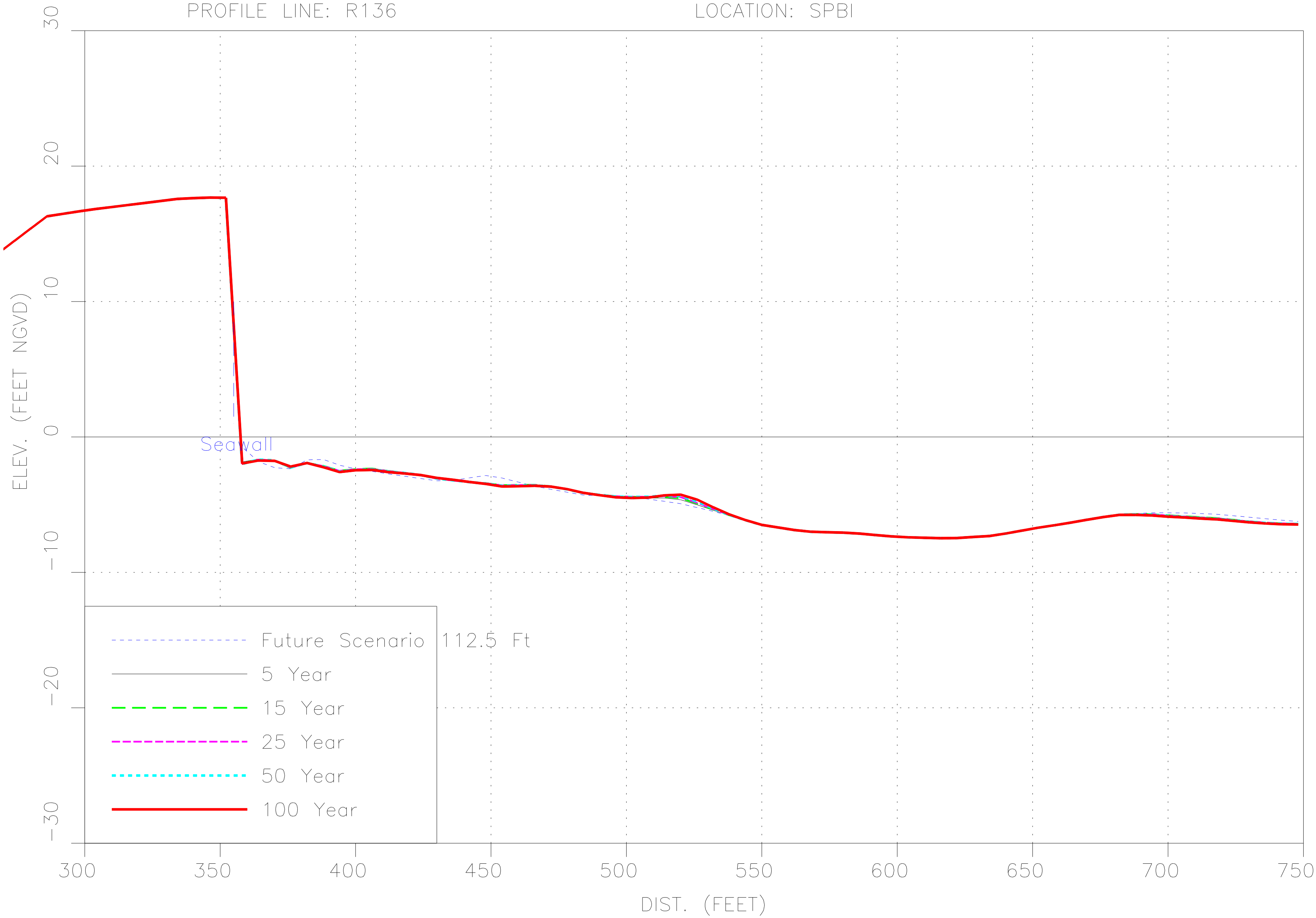

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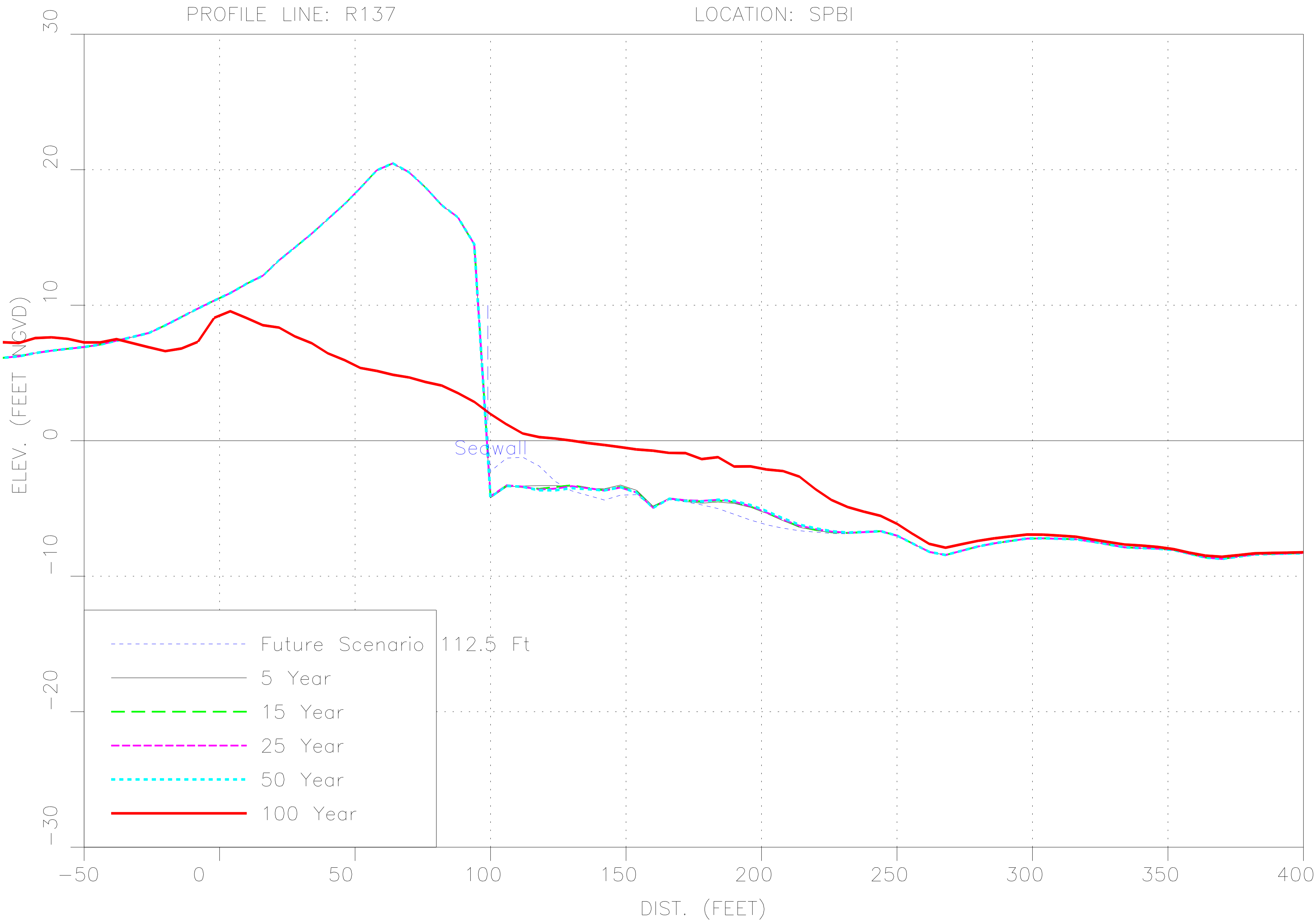
Future Scenario 112.5 Ft
5 Year
15 Year
25 Year
50 Year
100 Year



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Seawall
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25 Year
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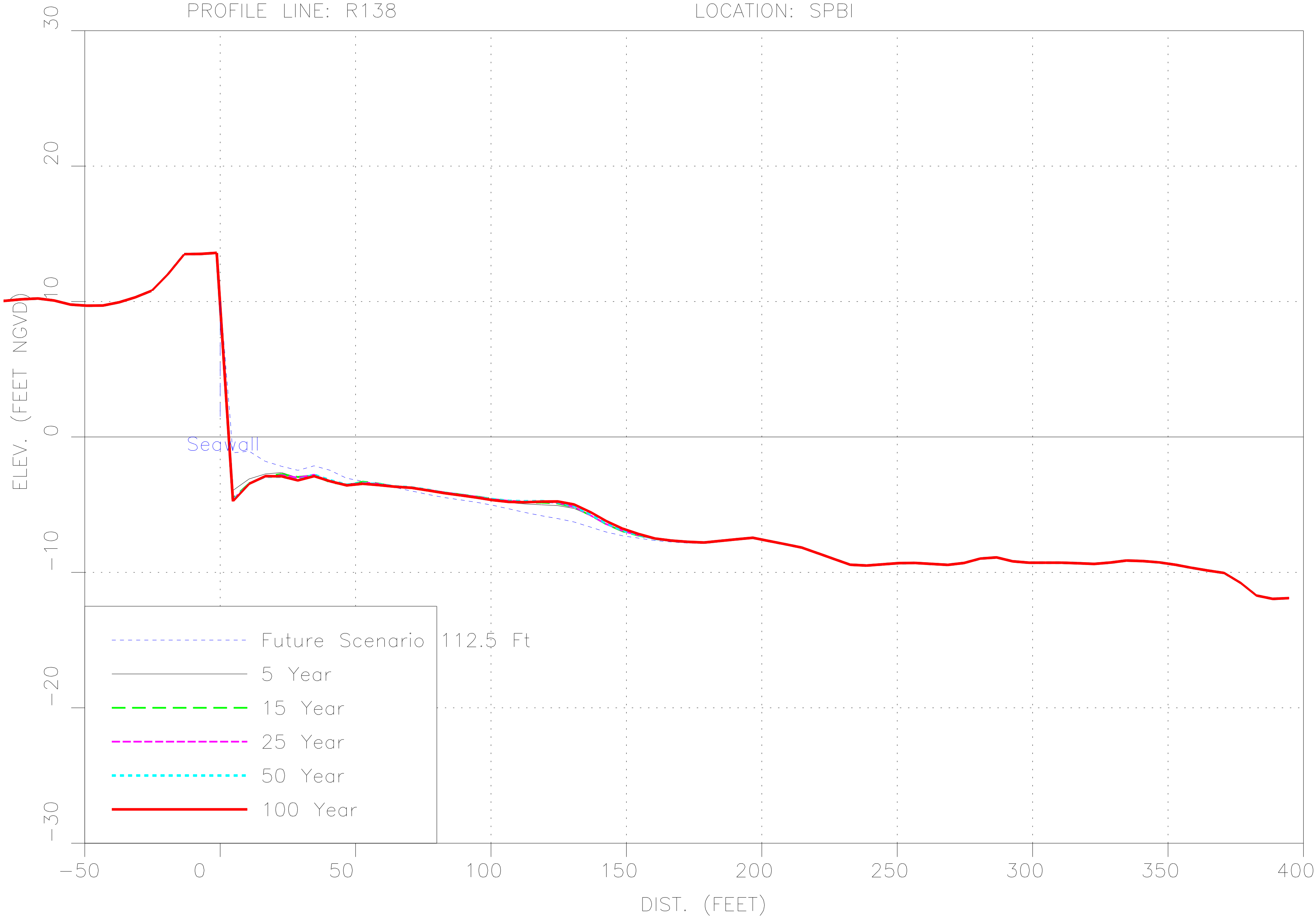
100 Year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50 Year
100 Year
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400

SUB-APPENDIX G-1

SBEACH ANALYSIS REPORT

ATTACHMENT D

LANDWARD LIMIT OF RECESSION BY RETURN PERIOD STORM BASED ON SBEACH RESULTS

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Notes:

1. Coordinates are in feet based on the Florida State Plane Coordinate System, East Zone, North American Datum of 1983 (NAD 83).
2. Aerial photography provided by theTown of Palm Beach, date flown March 30, 2012.

Legend:

- Year 5
- Year 15
- Year 25
- Year 50
- Year 100
- Monuments

EC - Existing Conditions (2011/2012 survey)
SF - Existing Conditions with Seawall Falilure
3- Alternative 3
6 - Alternative 6

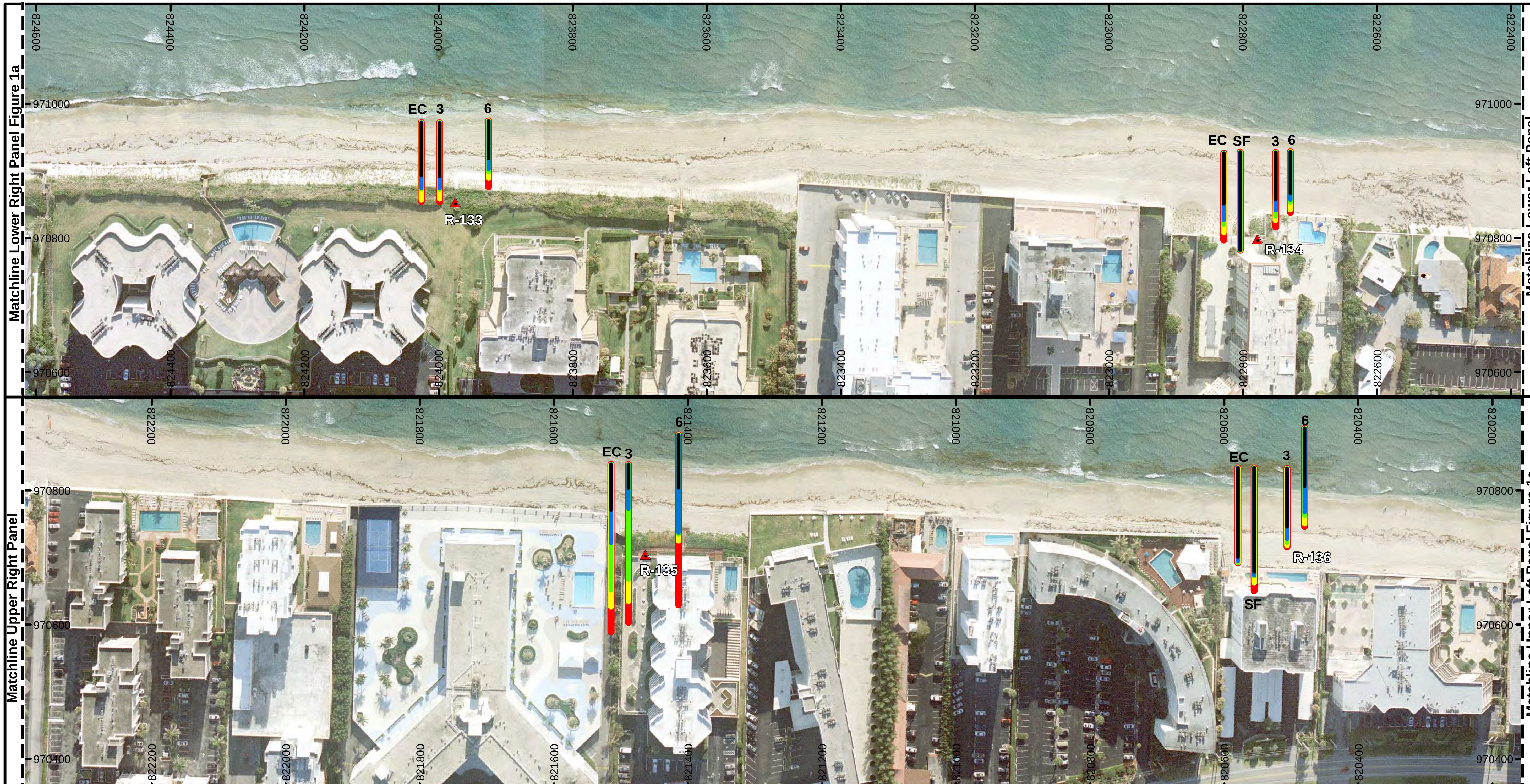
N

Feet
0 75 150
1 inch = 150 feet

TITLE:
Southern Palm Beach Island Shoreline Stabilization Project EIS Landward Limit of Recession by Return Period Storm Based on SBEACH Model Results

COASTAL PLANNING & ENGINEERING, INC.
A CB&I COMPANY
2481 N. W. BOCA RATON BOULEVARD
BOCA RATON, FL 33431
PH. (561) 391-8102 FAX (561) 391-9116

Date: 03/05/14	By: HMV	Comm No. : 150814	**Figure No.: 1a**



Notes:

1. Coordinates are in feet based on the Florida State Plane Coordinate System, East Zone, North American Datum of 1983 (NAD 83).
2. Aerial photography provided by theTown of Palm Beach, date flown March 30, 2012.

Legend:

Year 5
Year 15
Year 25
Year 50
Year 100
Monuments

EC - Existing Conditions (2011/2012 survey)
SF - Existing Conditions with Seawall Falilure
3- Alternative 3
6 - Alternative 6

N

Feet
0 75 150

1 inch = 150 feet

TITLE:

Southern Palm Beach Island
Shoreline Stabilization Project EIS
Landward Limit of Recession
by Return Period Storm
Based on SBEACH Model Results

COASTAL PLANNING & ENGINEERING, INC.
A CB&I COMPANY
2481 N. W. BOCA RATON BOULEVARD
BOCA RATON, FL 33431
PH. (561) 391-8102 FAX (561) 391-9116

Date: 03/05/14	By: HMV	Comm No. : 150814	**Figure No.: 1b**

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Notes:

1. Coordinates are in feet based on the Florida State Plane Coordinate System, East Zone, North American Datum of 1983 (NAD 83).
2. Aerial photography provided by theTown of Palm Beach, date flown March 30, 2012.

Legend:

- Year 5
- Year 15
- Year 25
- Year 50
- Year 100
- Monuments

EC - Existing Conditions (2011/2012 survey)
SF - Existing Conditions with Seawall Falilure
3- Alternative 3
6 - Alternative 6

N

Feet
0 75 150
1 inch = 150 feet

TITLE: **Southern Palm Beach Island Shoreline Stabilization Project EIS Landward Limit of Recession by Return Period Storm Based on SBEACH Model Results**

COASTAL PLANNING & ENGINEERING, INC.
A CB&I COMPANY
2481 N. W. BOCA RATON BOULEVARD
BOCA RATON, FL 33431
PH. (561) 391-8102 FAX (561) 391-9116

Date: 03/05/14	By: HMV	Comm No. : 150814	**Figure No.: 1c**

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SUB-APPENDIX G-1

DRAFT SBEACH ANALYSIS REPORT

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SOUTHERN PALM BEACH ISLAND COMPREHENSIVE SHORELINE STABILIZATION PROJECT DRAFT SBEACH ANALYSIS REPORT

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APPENDICES

1.0 INTRODUCTION

Under direction of the U.S. Army Corps of Engineers (USACE), CB&I Coastal Planning & Engineering, Inc. (CB&I) assisted in the development of the Southern Palm Beach Island Comprehensive Shoreline Stabilization Project Environmental Impact Statement (EIS). The initial tasks associated with the effort included public scoping and agency coordination to determine what data was necessary to develop the EIS. After review of the data and previous work, the USACE has determined that the level of storm protection needed to be analyzed using the Storm Induced Beach Change Model (SBEACH).

The Project Area for the Southern Palm Beach Island Comprehensive Shoreline Stabilization Project (the Project) comprises approximately 2.07 miles of shoreline and nearshore environment. The north and south limits are Florida Department of Environmental Protection (FDEP) range monuments (R-monuments) R-129-210 (south end of Lake Worth Municipal Beach) and R-138+551 (south of the Eau Palm Beach Resort and Spa in Manalapan), respectively (Figure 2-1). The Project Area's beaches provide storm protection to residential and public infrastructure and serve as nesting areas for marine turtles. The Project Area has been designated as "critically eroded" (FDEP, 2014). The active hurricane tropical storm activity that occurred between 2004 and 2008 has resulted in a narrow, low profile beach along the majority of its shoreline. Over the past 8 years, the annual shoreline change has averaged a loss of 2.25 feet per year (CPE, 2013). Previous attempts to rebuild dunes in the Project Area have not resulted in a stable dune system or a stable beach. The Applicants' Proposed Project under evaluation in the Environmental Impact Statement (EIS) intends to address the current erosion rates by stabilizing and widening the shoreline, thereby extending the construction interval between projects.

2.0 OBJECTIVE OF SBEACH MODEL STUDY

The objectives of this beach profile storm response study using the SBEACH model are as follows:

- To verify the need for a project along all sections of the Project Area
- Determine the level of storm protection provided by the existing conditions
- Preliminarily evaluate the storm protection benefits of two proposed fill alternatives

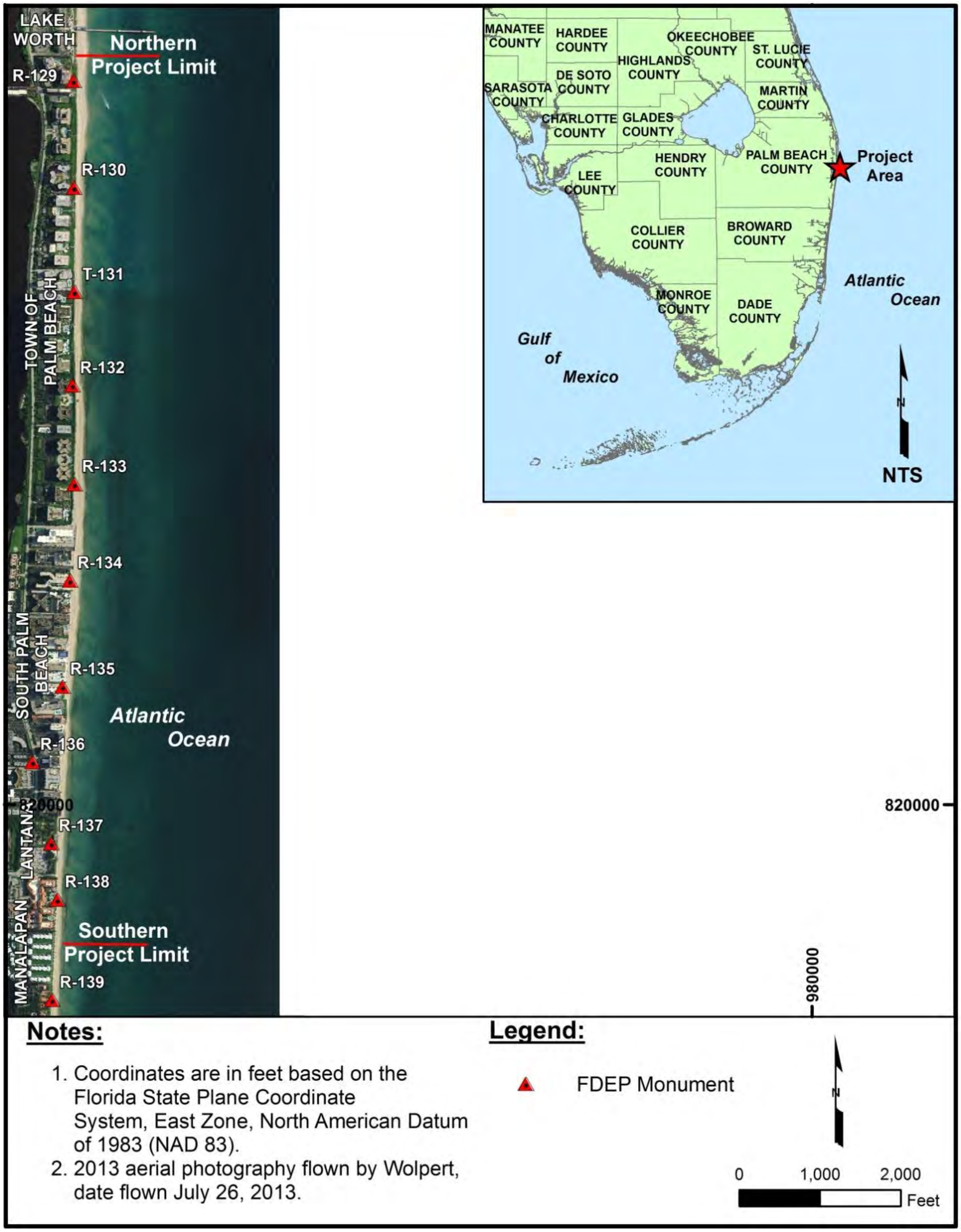

Figure 2-1. Southern Palm Beach Island Comprehensive Shoreline Stabilization Project Location.

3.0 METHODOLOGY

Cross-shore storm impact evaluations for the Project Area were conducted using the Storm Induced Beach Change Model (SBEACH) (Larson and Kraus, 1989). SBEACH is a numerical model that simulates changes to beach and dune profiles due to storm-driven erosion. Inputs to the SBEACH model include the initial profile, the time histories of the waves and water levels during each storm, and a set of model calibration parameters. Changes to the beach and dune profiles were simulated for storms return periods of 5, 15, 25, 50, and 100 years. The level of storm protection afforded by the existing beach and by the design beach fill and dune is defined by the return period of the storm event that causes a 0.5 foot vertical loss at the landward limit of the beach.

4.0 SBEACH MODEL SETUP

4.1. Model Background

SBEACH Version 4.03 (Larson et al., 2004) was used to model the cross-shore response of the design cross-section to the 5, 15, 25, 50 and 100 year storms. SBEACH is a one-dimensional model that simulates beach profile changes resulting from varying storm waves and water levels. These profile changes include the formation and movement of morphological features such as longshore bars, troughs, berms, and dunes. SBEACH evaluates storm impacts through simulated profile changes produced by cross-shore processes.

SBEACH is an empirically based numerical model, formulated using both field data and the results of large-scale physical model tests. Input data required by SBEACH includes the beach cross-section, the median sediment grain size, several calibration parameters, and the waves, wind velocities, and water surface elevations over the duration of the storm. SBEACH calculates the cross-shore variation in wave height and wave setup at discrete points along the profile from the offshore zone to the landward survey limit.

The following basic assumptions underlie the SBEACH model:

- Breaking waves and variations in water level are the major causes of sand transport and profile change.
- The influence of structures blocking longshore transport is small, and the shoreline is straight (i.e., longshore effects are negligible during the term of simulation).
- Linear wave theory is applicable everywhere along the beach profile.

4.2. Model Calibration

The model calibration was conducted using Hurricanes Frances (Category 2) and Jeanne (Category 3) because of the availability of beach profile survey data before and after the storms. These storms made landfall approximately 54 miles north of the Project Area near Hutchinson Island between August 25, 2004 and September 30, 2004.

The following wave, water level, and wind data collected during Hurricanes Frances and Jeanne was used in the SBEACH model setup:

- Waves were primarily based on the NOAA WAVEWATCH hindcast for the Western North Atlantic for the period from August 25, 2004 through September 30, 2004. Wave heights, wave periods, and wave directions at 3 hour intervals were taken from an observation point 12 miles northeast from the project site (Palm Beach Country Club, 26°45'N, 80°W) at a depth of -126.76 feet NGVD.
- Water levels were based on hourly measurements collected during the storms at the Lake Worth Pier tide gauge (NOAA Station ID LKWF1- 8722670), located immediately north of the project site.
- Wind data from NOAA Buoy LKWF1, Lake Worth was also used for calibration. Wind speed and direction was recorded hourly throughout the storm. There were two instances in the record when the station went offline for 3 to 9 hours. The

wind statistics were linearly interpolated during these periods to generate a continuous record.

The following beach profile surveys were used for the SBEACH model setup and calibration:

- Pre-storm beach profile survey conducted by Morgan & Eklund dated August 20, 2004.

- Post-storm LIDAR survey conducted by the NOAA Coastal Services Center Coastal Remote Sensing Program between November 22, 2004 and December 3, 2004.

- Post-storm beach profile survey including R-137 conducted by Palm Beach County dated October 4, 2004

The following LIDAR surveys were used to extend the SBEACH profiles landward where necessary:

- US Army Corps of Engineers (USACE) Joint Airborne Lidar Bathymetry Technical Center of Expertise (JALBTCX) survey data collected by the Compact Hydrographic Airborne Rapid Total Survey (CHARTS) system along the coast of Florida from August 31 - October 3, 2009.

- Airborne Topographic Mapper LIDAR data collected in partnership with the National Oceanic and Atmospheric Administration (NOAA) Coastal Services Center along the coast of Florida in 1990.

4.3. Model Parameters

The observed changes due to Hurricanes Frances and Jeanne were used as the basis for determining the calibration settings. The initial calibration run utilized the default parameters. In the following runs, a range of values for each calibration parameter were considered until the settings with the best agreement between observed and simulated conditions were identified. Varying calibration parameters to correct the agreement at a specific profile resulted in greater discrepancies at other profiles; therefore, the final calibration parameters were selected based on the agreement across the Project Area as a whole.

The final calibration parameters used in the production runs were as follows:

- The transport rate coefficient, which was equal to the ratio between the cross-shore transport rate and the wave energy dissipation rate was set to $K = 2.5 \times 10^{-7}$ m^4/N.

- The slope dependent coefficient, which governed the influence of the profile slope on the cross-shore transport, was set to $\varepsilon = 0.001$ m^2/s.

- The transport rate decay coefficient, which governed the reduction in the wave height over the beach profile due to wave breaking, was set to $\lambda = 0.5$.

- The assumed depth at landward end of the surf zone was set to Dfs = 1 foot.

In addition to the parameters above, the following assumptions were made for parameters required in the most recent version of SBEACH (4.03):

- A median grain size of 0.3 mm for the existing conditions. Samples collected in 2006 confirm the native grain size to be 0.3 mm (CPE, 2007). As an additional note, dune nourishments constructed in 2011 placed a small amount of coarser sand along the dune measuring 0.45 mm from an upland sand source (ATM, 2012).

- A grain size of 0.3 mm for the beach and dune fill. The grain size of sand in the borrow areas included in the Beach Management Agreement range from 0.25 to 0.29 mm with a compliance range of 0.25 mm to 0.6 mm for the region containing the Project Area (FDEP, 2013). Additionally, using the same grain size sediment for the various alternatives during production runs as was used in calibration allows the results to be comparable and eliminates a potential source of error.

- Average water temperature of 28.5°C (83°F) (NOAA, 2013).

- A default avalanche slope of 45°.
- The beach profiles were represented in the model with grid cell spacing of 6 feet.
- The time step used in simulations was 1 minute.
- An overwash coefficient of 0.008. The overwash coefficient is a relatively recent addition to the SBEACH model (see Larson, et al, 2004). The default value of this parameter is 0.005 for an unreinforced dune. No significant difference is noticed between simulations with varying overwash parameters for the 5, 15, 25, and 50 year storms. During the 100 year storm, the profiles are sensitive to the overwash coefficient and the magnitude of overwash increases as the coefficient increases (Figure 4-1).

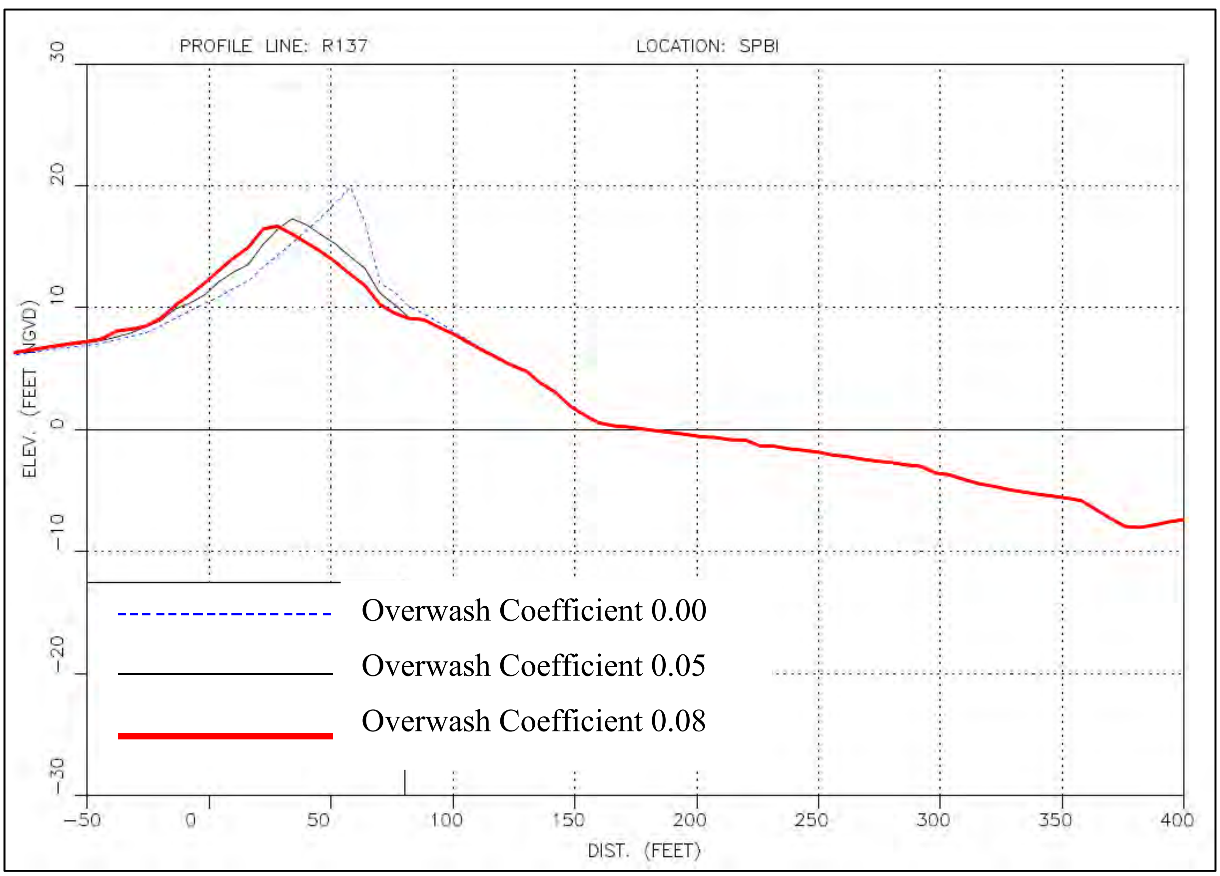

Figure 4-1. Sensitivity of overwash coefficient for R-137 profile, 100 year storm.

4.3.1. Final Calibration Results

The simulated beach profile responses with the final calibration settings agree well with the observed conditions within the Project Area. A comparison of the observed and calibrated shoreline changes, volume changes and landward limits of erosion is presented in Table 4-1. The average difference between the observed and calibrated shoreline changes was 6 feet. The average difference between the observed and calibrated volume change above mean low water (MLW) was 4 cubic yards per foot (cy/ft). The average difference between the observed and calibrated landward limit of storm recession, where at least 0.5 feet of elevation was lost, was 5 feet. On average, the calibration slightly overpredicted the erosion resulting from Hurricanes Frances and Jeanne along most profiles. This overprediction rather than underprediction of erosion is expected to positively affect the reliability of the results of the production runs. Unlike the calibration storms, the storms used in the production runs will be assumed to make landfall at the Project Area. As a result, the erosion simulated in production during an equivalent return period storm as Hurricanes Frances is expected to be more severe than what was observed in calibration.

Table 4-1. Observed vs. calibration run results for volume and shoreline changes.

Profile[1]	Shoreline Change (feet)		Volume Change above Mean Low Water (-0.73 feet NGVD) (cy/ft)		Landward Limit of Storm Erosion[2] (feet from R-monument)	
	Observed	Calibrated	Observed	Calibrated	Observed	Calibrated
R-129	-49	-53	-13	-15	60	60
R-130	-50	-45	-16	-15	37	53
R-131	-71	-46	-22	-14	27	35
R-132	-16	-21	-8	-13	33	N/A
R-133	9	-21	-5	-13	3	11
R-134	-19	-6	-13	-13	0	0
R-135	-13	-35	-8	-19	15	N/A
R-136	-20	-32	-7	-17	11	N/A
R-137	-27	-48	-4	-19	80	80
Average[3]	-28	-34	-11	-15	35	40
Difference		-6		-4		-5

[1]Survey data was not available at R-138
[2] Survey data near the landward limit of the active profile was not available at profiles R-132, R-135 and R-136.
[3]Averages only include profiles where data was available.

4.4. Seawalls

Seawalls are present along 78% of the Project Area (CPE, 2007) and serve as an important component of storm protection for upland properties. The seawalls are non-homogeneous in that the quality and age of construction materials used and design criteria utilized varies by property. The information available about these seawalls is limited to the elevation of the top of the wall. Despite the limited information available, including seawalls in SBEACH is critical for simulating the beach profile response to storms.

In SBEACH, location and seawall failure criteria can be included in the model setup. The locations of the seawalls as included in the model are shown on the figures in Appendix A. The SBEACH model has three modes of failure 1) scour at the toe of the structure, 2) direct wave attack and 3) inundation. The seawall is assumed to fail and erosion occurs landward of the seawall if one or more of these criteria are met during a time step. Detailed information about the construction and stability of each seawall within the Project Area was not available. The following assumptions were made to incorporate seawalls into the SBEACH model setup. These assumptions were intended to conservatively represent the conditions of the seawalls.

- Toe scour failure was assumed to occur when the beach profile elevation at the seawall lowered to -3 feet NGVD. Based on an average seawall height of +17 feet NGVD, the depths of the seawalls were anticipated to extend to at least -3 feet NGVD.

- The wave height at the seawall which causes failure was computed for each design storm based on the maximum water level that occurred during each storm and the overtopping failure criteria of 0.015 cubic meters per second per meter (Allsop et al, 2005; USACE, 2000).

- The water level at the seawall which was expected to cause inundation failure was assumed to be equal to the top elevation of the seawall.

Recent storms have provided evidence of the likelihood of seawall failure along the Project Area. Along the southern portion of the Project Area, many of the seawalls are exposed directly to wave action during storms (Figure 4-2 and Figure 4-3). The seawalls along the Project Area vary in age, stability and degree of exposure, leaving them more or less vulnerable to the modes of failure discussed previously. As an example, wave impacts and scouring that occurred during Hurricane Sandy led to failure and undermining of walls less than one mile south of the project site resulting in significant property damage and loss (Figure 4-4). Examining the likelihood and magnitude of toe scour using SBEACH will assist in understanding the risk of seawall failure along the Project Area and determining the overall need for the project.

Figure 4-2. Impacts of Hurricane Sandy near R-136, Town of South Palm Beach (October 26, 2012).

Figure 4-3. Impacts of Hurricane Sandy near R-137, Town of South Palm Beach (October 26, 2012).

Figure 4-4. Failure of seawall in Manalapan after Hurricane Sandy (1 mile south of the Project Area, R-143.5) (Coastal Star, 2013).

4.4.1. Seawall Replacement Cost

The estimated cost per mile to replace a seawall in Palm Beach County is approximately $30.6 million based on the 2009 seawall construction that occurred near R136. Therefore, the cost to replace all of the seawalls (78% of shoreline) along the 2.07-mile long Project Area after catastrophic failure would be approximately $49.4 million.

4.5. Representative Profiles

Ten beach profiles were modeled using SBEACH (R-129 to R-138). To represent the most recent conditions, profile survey data collected between 2011 and 2012 was utilized. The datum used during the surveys were the Florida State Plane Coordinate System, North American Datum of 1983. The surveys were converted to the National Geodetic Vertical Datum using Corpscon (ver. 6.x) for consistency of datums throughout the calibration and production runs. The beach profile cross sections were extended landward for modeling purposes using the 1990 Survey for R-129 to R-137 and the 2009 Light Detection and Ranging (LIDAR) Survey for R-138.

The most recent survey of the Project Area which is being used for analysis and model setup was collected in November 2011 along the Town of Palm Beach (R-129-R-134) and in January 2012 for the County shoreline (R-135-R-138). Table 4-2 lists the most recent dune nourishments within the Project Area. The dune nourishments occurred approximately 9 months to 3 years prior to the survey dates for the Town of Palm Beach and County, respectively. Based on the information reviewed, neither of the surveys was an as-built survey. No major hurricanes have made a direct impact to the Project Area since the nourishments; however, storms have occurred and likely contributed to the background erosion rate.

Table 4-2. Most recent dune nourishments.

Date	Project	Project Extents	Volume (cy)	Sand Source
2009	South Palm Beach/Lantana Dune Restoration	R-135+460 to R-137+410	10,000	Upland
December 2010 – February 2011	Phipps Ocean Park Beach and Dune Restoration	Dune R-129 to R-133	56,000	Upland

4.5.1. Design Cross-Sections

The Southern Palm Beach Island Comprehensive Shoreline Stabilization Project currently evaluates seven alternatives:

1) No Action Alternative (Status Quo)

1a) No Action Alternative (Status Quo) (includes dune nourishment)

2) The Applicants' Preferred Alternative (Proposed Action): Beach and Dune Fill with Shoreline Protection Structures

3) The Applicants' Preferred Project without Shoreline Protection Structures

4) The Town of Palm Beach Preferred Project and County Increased Sand Volume Project without Shoreline Protection Structures (3 years fill)

5) The Town of Palm Beach Increased Sand Volume Project (modified Erickson alternative) and County Preferred Project

6) The Town of Palm Beach Increased Sand Volume Project and County Increased Sand Volume without Shoreline Protection Structures Project

SBEACH modeling was conducted for Alternatives 1, 3 and 6 (Table 4-3). Alternative 2 was not modeled since the fill design is the same as Alternative 3. SBEACH is a cross-shore transport model and does not include the option of including groins as present in Alternative 2.

- Alternative 1 utilized the 2011/2012 surveys without modification to represent the existing conditions or No Action (Status Quo) Alternative. No Action Alternative includes dune nourishments with fill volume placements of approximately 11 cubic yards per foot from R-129 to R-133 and 5 cubic yards per foot from R-135-460 to R-137+410 every 1 to 5 years.

- Alternative 2 was not simulated in SBEACH. The results from Alternative 3 are applicable to Alternative 2. Alternative 2 has 7 low-profile pile and panel groins as part of the design. SBEACH cannot consider the effects of groins in simulating the cross-shore storm response of beach profiles.

- Alternative 3 utilized the Applicants' Preferred fill design which consisted of dune fill only from R-129-210 to R-129+150, dune and beach fill from R-129+150 to R-131, dune fill only from R-131 to R-134+135 (Town of Palm Beach southern limit), and beach fill from R-134+135 to R-138+551 (Towns of South Palm Beach, Lantana and Manalapan). This alternative was originally designed to require approximately 150,000 cubic yards of fill for the entire project based on 2009 surveyed profiles along the Town of Palm Beach and 2011 surveyed profiles for the remainder of the Project Area. The design of the Town of Palm Beach section (R-129-210 to R-134+135) was updated based on the available 2012 profiles for use in the SBEACH model setup. The seaward crests of the dune and berm from the original design remained at the same range and elevation in the updated design with two exceptions 1) if the 2012 dune was located seaward of the original design, no fill was added to the dune and 2) no fill was placed landward of the edge of vegetation as shown in the 2011/2012 aerials.

- Alternative 4 utilized the same Applicants' Preferred design as Alternative 2 for the Town of Palm Beach portion of the project area (R-129-210 to R-134+135) and a larger design along the County portion (R-134+135 to R-138+551). The fill volume from R-134+135 to R-138+551 was increased from 75,000 cubic yards to 160,600 cubic yards.

- Alternative 5 utilized a modified design for the Town of Palm Beach portion (R-129-210 to R-134+135) and the Applicants' Preferred design along the County portion of the project area (R-134+135 to R-138+551). The modified design consisted of placing additional fill on the dry beach (R-129-210 to R-134+135) where feasible, totaling 96,000 cubic yards.

- Alternative 6 utilized the same design as Alternative 5 which placed more fill along the dry beach of Town of Palm Beach (R-129-210 to R-134+135; 96,000 cubic yards) and the same larger design used in Alternative 4 (~160,000 cy) along the County portion (R-134+135 to R-138+551).

Table 4-3. Cross-sections simulated in the SBEACH model.

Profile	Dune/Berm Width (feet)	Dune/Berm Slope	Seawall Included
Alternatives 2 & 3			
R129	No fill added		No
R130	17	1V:10H	Yes
R131	18	1V:3H	No
R132	10	1V:3H	Yes
R133	No Fill added		No
R134	35.2	1V:3H	Yes
R135	22.2	1V:10H	No
R136	75.6	1V:10H	Yes
R137	52.7	1V:10H	Yes
R138	18.5	1V:10H	Yes
Alternative 4			
R129	No fill added		No
R130	17	1V:10H	Yes
R131	18	1V:3H	No
R132	10	1V:3H	Yes
R133	No Fill added		No
R134	35.2	1V:3H	Yes
R135	66.2	1V:10H	No
R136	130.2	1V:10H	Yes
R137	98.4	1V:10H	Yes
R138	58.5	1V:10H	Yes

Table 4-3 (cont.). Cross-sections simulated in the SBEACH model.

Profile	Dune/Berm Width (feet)	Dune/Berm Slope	Seawall Included
	Alternative 5		
R129	65.2	1V:5H	No
R130	17	1V:10H	Yes
R131	18	1V:3H	No
R132	0	1V:3H	Yes
R133	46.9	1V:3H	No
R134	72.8	1V:3H	Yes
R135	22.2	1V:10H	No
R136	75.6	1V:10H	Yes
R137	52.7	1V:10H	Yes
R138	18.5	1V:10H	Yes
Alternative 6			
R129	65.2	1V:5H	No
R130	17	1V:10H	Yes
R131	18	1V:3H	No
R132	10	1V:3H	Yes
R133	46.9	1V:3H	No
R134	72.8	1V:3H	Yes
R135	66.2	1V:10H	No
R136	130.2	1V:10H	Yes
R137	98.4	1V:10H	Yes
R138	58.5	1V:10H	Yes

4.6. Storm Data

Five specific return interval storm events were used in the SBEACH cross-shore analyses, 5 year, 15 year, 25 year, 50 year and 100 year. Wind, water level and wave data from Hurricane Frances observed during the time period from August 25, 2004 to September 9, 2004 was used as the basis for the design of the return interval storms.

The Hurricane Frances data was scaled accordingly to match the maximum values listed in Table 4-4 for each storm. Maximum wave heights, wave periods, and water levels during each storm appear in Table 4-4. Plots of the wave height, wave period, and water level versus time appear in Appendix B.

Table 4-4. Design Storm Summary

	Maximum Values			
Return Period (years)	**Offshore Significant Wave Height[1] (feet)**	**Peak Wave Period (seconds)**	**Water Level [2] (feet NGVD)**	**Wind Speed [3] (mph)**
5	20.8	9.7	*3.7*	*69*
15	26.4	11.0	*5.0*	*85*
25	29.1	11.5	*5.5*	93
50	32.6	12.2	6.3	103
100	36.2	12.8	7.0	111

NOTES: 1. Wave heights are given at a depth of 356 meters (USACE, 2012).
2. Values in italics are interpolated or extrapolated from FEMA (1982). These values do not include wave setup as it is calculated and included by SBEACH during the simulations.
3. Values in italics are interpolated or extrapolated from USACE (1985).

FEMA return period water level accounts for tidal effects. FEMA used a numerical hydrodynamic model of the region to simulate the coastal surge generated by different return period storms. The astronomical tide for the region was statistically combined with the computed storm tide to yield recurrence intervals of total water level shown in the published water levels (FEMA, 1982).

5.0 MODEL RESULTS

5.1. General

SBEACH model results appear in Appendices C and D and include the post-storm profiles for all design storms in Table 4-4.

5.2. Existing Conditions (2011/2012 Beach Profiles) / No Action Status Quo Scenario

The existing conditions along the Project Area shoreline consist of eroded dunes, exposed seawalls and steep gradient berms. Along the Town of Palm Beach, there is a continuous dune feature and line of vegetation separating the beach from the residential infrastructure. There are several buried seawalls along this section of shoreline (R-129-210 to R-134+135). Along the Towns of South Palm Beach, Lantana and Manalapan, there is no dune feature and the majority of the beach profiles consist of partially exposed seawalls.

The degree of erosion during a storm will vary spatially due to the characteristics of the beach profiles (Table 5-1; Appendix C). Profiles R-131 through R-134 will experience the most erosion. Profile R-131 is not protected by seawalls. This profile also has the steepest existing beach face which leads to higher breaking waves in the surf zone and increases the potential for runup and erosion. Profiles R-132, R-134 and R-137 will experience similar erosion. The exposed seawalls present on these profiles leads to scouring and volume loss at the base of the wall. The other profiles have similar but slightly lower erosion rates. The average volume change above mean low water during a 5, 10, 25, 50 and 100-year return interval storm along the Project Area was -6.0 cy/ft, -7.3 cy/ft, -7.7 cy/ft, -8.4 cy/ft and -9.1 cy/ft, respectively (Table 5-2).

Under existing conditions, the seawalls and revetments at monuments R-130, R-132, R-136 are exposed. Scouring at the toe of the seawalls occurs at these locations in all of the simulated return interval storms (Appendix C). Scouring increases incrementally with magnitude of storm. No seawall failures were observed during the simulations.

The landward limit of erosion was quantified to determine the potential impacts to infrastructure and property landward of the Project Area (Table 5-3). The landward limit of erosion was defined as the landward position where at least 0.5 feet of elevation was lost as a result of the storm. The values in Table 5-3 are referenced to the FDEP R-monuments since the monuments are at a fixed location. As the profiles erodes landward towards the R-monuments, the values in the table decrease until they retreat landward of the monument and then the values are negative. The table values in red signify that recession landward of the improved or maintained property has occurred. Maintained property refers to landscaped areas or paved/ gravel areas. While a seawall is operational, the landward limit of recession is the same for different return interval storms because the seawalls prevent further landward recession as shown in the table at R-130 for the 15, 25, 50, 100-year storms. In general, profiles without seawalls, R-131 and R-135 are certainly at risk of damage during the occurrence of a 25-year return interval storm or stronger storm. Damage is possible adjacent to profile R-133 as a result of a 50-year return interval or stronger storm. The critical storm return interval for damage to property to occur is between a 15-year and 25-year storm.

Table 5-1. SBEACH shoreline retreat and erosion under existing conditions (2011/2012) and a 15 year storm.

Profile	MLW Change (feet)	Volume Change above MLW (cy/foot)
R-129	-17	-5.6
R-130	0	-6.4
R-131	2	-8.1
R-132	4	-8.1
R-133	-23	-9.2
R-134	-22	-8.7
R-135	-17	-7.1
R-136	-22	-5.8
R-137	-24	-7.4
R-138	-40	-6.1

NOTE: Mean Low Water (MLW) = -0.73’ NGVD.

Table 5-2. SBEACH shoreline retreat and erosion, existing conditions (2011/2012).

	5 Year Storm	15 Year Storm	25 Year Storm	50 Year Storm	100 Year Storm
Profile	Volume Change above MLW (cy/ft)	Volume Change above MLW (cy/ft)	Volume Change above MLW (cy/ft)	Volume Change above MLW (cy/ft)	Volume Change above MLW (cy/ft)
R-129	-4.6	-5.6	-6.0	-6.6	-7.1
R-130	-5	-6.4	-6.8	-7.4	-8
R-131	-6.5	-8.1	-8.8	-9.9	-10.7
R-132	-6.3	-8.1	-8.8	-9.8	-10.6
R-133	-7.6	-9.2	-9.9	-10.7	-11.4
R-134	-7.4	-8.7	-9.3	-10	-10.5
R-135	-5.9	-7.1	-7.6	-8.2	-8.7
R-136	-5.0	-5.8	-6.0	-6.5	-6.7
R-137	-6.5	-7.4	-7.5	-8	-10.3
R-138	-5.3	-6.1	-6.3	-6.7	-7.2

Table 5-3. SBEACH landward limit of storm erosion.

FDEP R-Monument[1]	Simulation ID	Landward Limit of Storm Erosion[2] (feet from seaward edge of maintained property) Given Return Period in Years:				
		5	15	25	50	100
R-129	Existing Conditions	97	66	52	33	31
	Alternative 3	97	66	52	36	31
	Alternative 6	111	93	85	50	31
R-130	Existing Conditions	55	37	32	**-2**	**-7**
	Seawall Failure[3]	55	37	32	**-14**	**-24**
	Alternative 3	80	49	47	40	**-6**
	Alternative 6	88	61	59	56	49
R-131	Existing Conditions	19	9	**-1**	**-13**	**-42**
	Alternative 3	21	13	**-2**	**-12**	**-56**
	Alternative 6	21	13	**-2**	**-11**	**-56**
R-132	Existing Conditions	24	18	16	11	10
	Seawall Failure[3]	24	18	8	**-20**	**-38**
	Alternative 3	45	34	23	18	16
	Alternative 6	48	34	23	18	16
R-133	Existing Conditions	30	12	10	**-6**	**-8**
	Alternative 3	29	12	10	**-4**	**-8**
	Alternative 6	55	39	35	26	13
R-134	Existing Conditions	54	30	23	11	0
	Seawall Failure[3]	**-17**	**-17**	**-17**	**-17**	**-17**
	Alternative 3	59	43	34	28	18
	Alternative 6	68	59	55	44	40
R-135	Existing Conditions	48	**-1**	**-71**	**-96**	**-133**
	Alternative 3	81	50	**-55**	**-88**	**-119**
	Alternative 6	81	14	12	2	**-93**

Table 5-3. (cont.). SBEACH landward limit of storm erosion.

FDEP R-Monument[1]	Simulation ID	Landward Limit of Storm Erosion[2] (feet from seaward edge of maintained property) Given Return Period in Years:				
		5	**15**	**25**	**50**	**100**
R-136	Existing Conditions	8	2	0	0	0
	Seawall Failure[3]	**-14**	**-19**	**-20**	**-30**	**-42**
	Alternative 3	54	36	31	26	24
	Alternative 6	110	71	66	54	50
R-137	Existing Conditions	**-15**	**-27**	**-29**	**-29**	**-29**
	Seawall Failure[3]	**-15**	**-27**	**-29**	**-54**	**-77**
	Alternative 3	13	22	**-10**	**-16**	**-22**
	Alternative 6	73	61	47	43	**-16**
R-138	Existing Conditions	0	0	0	0	0
	Seawall Failure[3]	**-21**	**-51**	**-88**	**-144**	**-142**
	Alternative 3	3	0	0	0	0
	Alternative 6	28	18	13	8	1

[1]Profiles R-129, R-131 and R-135 do not have a seawall.
[2]Values bolded in red represent erosion landward of the edge of maintained or improved property or infrastructure. Cells shaded yellow represent exposed seawalls.
[3]Simulations run assuming seawall had failed.

5.3. Future scenario without project conditions

Evaluating the existing conditions alone does not provide a complete perspective of the beach response to storms without a project. Based on the erosional trend along the Project Area, the beach profile is likely to continue recessing and lowering in elevation. To represent future scenarios without a project, 10-year and 50-year projections of beach profiles were developed and simulated with SBEACH. The existing condition profiles were translated landward based on the background erosion rate of 2.25 feet per year (CPE, 2013). Seawalls were included in the future scenarios as they were in the existing conditions simulations.

The landward limits of erosion for the future scenarios are presented in Table 5-4. Based on the future scenario simulations, all storm protection provided by the dune between R-130 and R-134 is lost. Seawalls that were buried within the dune have become exposed and are subject to wave action. The seawalls along the shoreline between R-136 and R-138 fail due to toe scour, allowing erosion of upland property and damage to infrastructure (Figure 5-1).

Table 5-4. SBEACH landward limit of storm erosion future scenario.

FDEP R-Monument[1]	Future Scenario	Landward Limit of Storm Erosion[2] (feet from seaward edge of maintained property) Given Return Period in Years:				
		5	15	25	50	100
R-129	10 years into future	91	59	39	29	-9
	50 years into future	1	**-31**	**-51**	**-61**	**-99**
R-130	10 years into future	13	5	**-2**	**-11**	**-16**
	50 years into future	**-21**	**-32**	**-36**	**-43**	**-43**
R-131	10 years into future	**-11**	**-34**	**-40**	**-67**	**-83**
	50 years into future	**-101**	**-124**	**-130**	**-180**	**-188**
R-132	10 years into future	29	16	**-7**	**-15**	**-34**
	50 years into future	**-25**	**-26**	**-26**	**-26**	**-26**
R-133[3]	10 years into future	0	**-19.5**	**-25**	**-56**	**-72**
	50 years into future	**-37**	**-37**	**-37**	**-37**	**-37**
R-134	10 years into future	21	1	**-5**	**-5**	**-5**
	50 years into future	**-5**	**-5**	**-5**	**-5**	**-5**
R-135	10 years into future	**-246**	**-246**	**-246**	**-246**	**-246**
	50 years into future	**-236**	**-236**	**-236**	**-236**	**-236**
R-136	10 years into future	354	354	353	353	353
	50 years into future	354	354	354	354	355
R-137	10 years into future	95	95	95	24	**-8**
	50 years into future	77	77	77	77	**-51**
R-138	10 years into future	**-11**	**-11**	**-11**	**-11**	**-11**
	50 years into future	**-11**	**-11**	**-11**	**-11**	**-11**

[1]Profiles R-129, R-131 and R-135 do not have a seawall.
[2]Values bolded in red represent erosion landward of the edge of maintained or improved property or infrastructure.

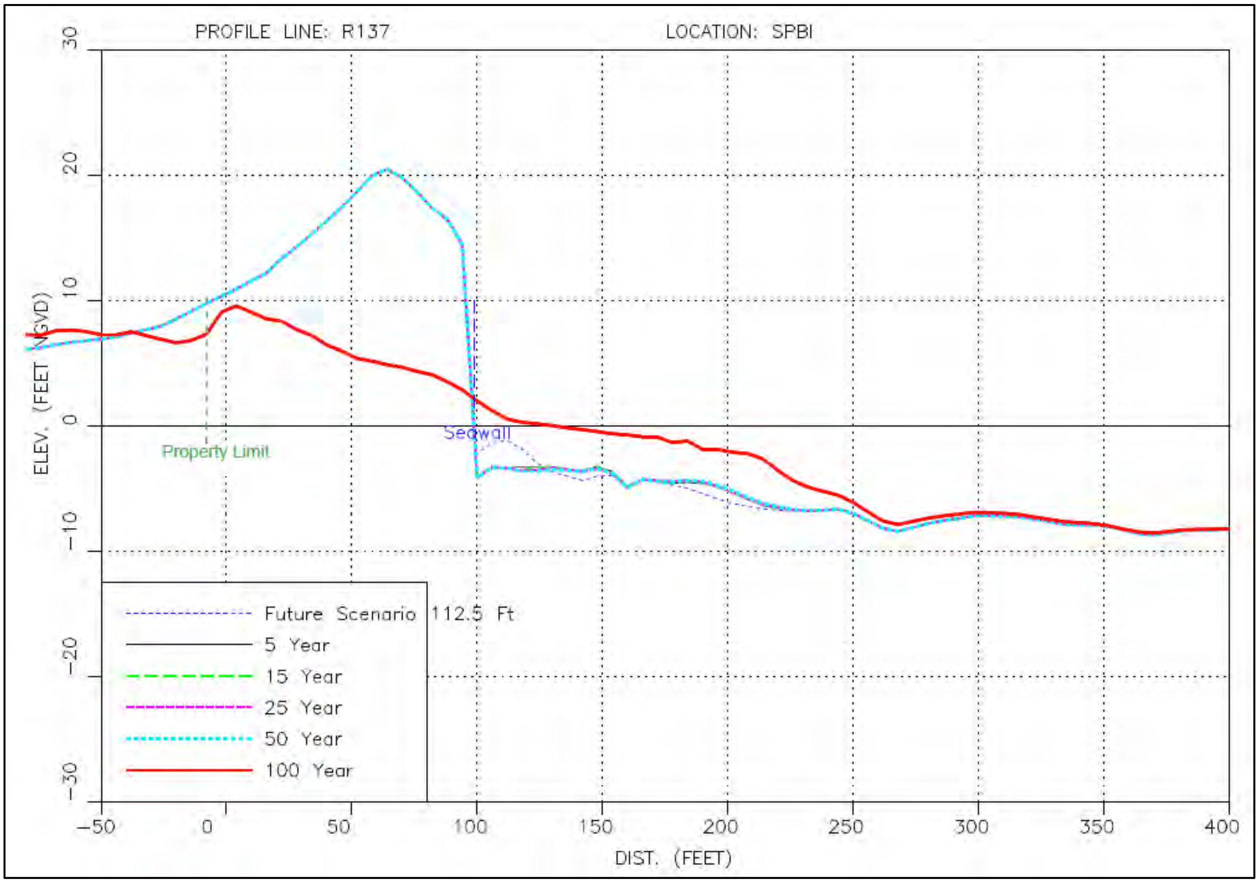

Figure 5-1. Seawall failure profile R-137 Future Scenario (50 years into the future)

5.4. Alternative 3: Applicants' Preferred Project Without Shoreline Protection Structures

The Applicants' Preferred Alternative fill design consists of dune only and dune and berm fill from R-129-210 to R-134+135 (75,000 cy) and berm fill only from R-134+135 through R-138+551 (75,000 cy). No fill was simulated at R-129 since the existing conditions met the design criteria for the seaward dune extent. The placement of berm fill only from R-134+135 to R-138+551 allows the seawalls to remain partially exposed.

The project prevents scouring at the toe of the seawalls at all locations simulated except R-136 and R-138 (Appendix C). At these two locations, scouring increases incrementally with magnitude of storm. Furthermore, none of the buried seawalls were exposed as a result of the return interval storms. No seawall failures were observed during the simulations.

In general, the project provides storm protection against a 15-year storm with little to no impact to the pre-construction profile (Table 5-3). Under the occurrence of a 5, 15 and 25-year storm, the frontal dunes present at profiles R-129 through R-133 retained their shape but lost volume. Recession into the pre-construction profile increases with increasing magnitude of return interval storm. The berm profile remains at a 2 to 3-foot higher elevation than the pre-construction profile even after a 100-year storm.

5.5. Alternative 6: The Town of Palm Beach Increased Sand Volume Project and County Increased Sand Volume Project Without Shoreline Protection Structures

Alternative 6 consists of a wider dune fill at profiles R-129-210 through R-134+135 (96,000 cubic yards) and a wider berm fill at profiles R-134+135 through R-138+551 than the Applicants' Preferred Alternative (approximately 160,000 cubic yards). Berm widths range from approximately 17 to 130 feet from the pre-construction profile (Table 4-3).

The project prevents scouring at the toe of the seawalls at all locations (Appendix C). None of the buried seawalls were exposed as a result of the return interval storms. No seawall failures were observed during the simulations.

In general, the project provides storm protection against a 15-year storm with little to no impact to the pre-construction profile from profiles R-129 to R-134 and 50-year return interval storm protection to the pre-construction profiles from R-135 through R-138. Under the occurrence of a 5, 15 and 25-year storm, the frontal dunes present at profiles R-129 through R-133 retained their shape but receded and lost volume. Recession into the pre-construction profile increases with increasing magnitude of return interval storm. The berm profile remains at a 2 to 5-foot higher elevation than the pre-construction profile even after a 100-year storm.

Based on the landward limit of erosion calculation, damage to property is possible adjacent to profile R-131 as a result of a 25-year return interval or stronger storm (Table

5-3). Property along profiles R-135 and R-137 are at risk of damage during the occurrence of a 100-year return interval storm or stronger storm.

6.0 CONCLUSIONS AND RECOMMENDATIONS

To determine the level of storm protection provided by existing and potential dunes and berms along the Project Area, the SBEACH model was applied and storm erosion given the existing (Winter 2011/2012) conditions and two alternatives of beach and dune fill cross-sections was analyzed. The following conclusions were made based on the results of the model study:

- The critical return interval storm resulting in property damage under existing conditions is between a 15-year and 25-year storm. On average, 7.3 to 7.7 cubic yards per foot was simulated to erode from the beach above mean low water during a 15-year and 25-year storm, respectively. This volumetric loss coincides with a steepening of the dune face, shoreline retreat and lowering of the beach profile elevation. Based on 2011/2012 conditions, erosion and wave impacts were simulated to extend landward damaging infrastructure and maintained (landscaped) property areas at FDEP R-monuments R-130, R-133, R-135 and R-137. These locations lack seawalls or have seawalls located further landward on the property.

- Seawalls prevent erosion into the upland property until wall failure. Scouring at the toe of exposed seawalls increases their likelihood of failure. Based on the 2011/2012 conditions response to a storm event, the berm elevation adjacent to exposed seawalls will lower increasing the likelihood of seawall failure during storms. If seawall failure is assumed to occur along the Project Area, infrastructure would be impacted from R-130 through R-138. A detailed analysis of the structural stability of the individual seawalls along the Project Area would be necessary to truly assess the vulnerability of this critical component of storm protection infrastructure.

- Based on the SBEACH simulations and background erosion rates, the status quo dune nourishments alone are not sufficient to sustain the existing conditions. The No Action Status Quo conditions for the Project Area include dune nourishments of 5 to 11 cubic yards per foot of fill between R-135+460 to R-137+410 and R-129 to R-133, respectively, placed every 1 to 5 years. This conclusion is made based on the storm response simulation of the 2011/2012 conditions which are representative of the No Action Status Quo Scenario. The 2011/2012 conditions represent the beach 9 months to 3 years after a dune nourishment and without the impacts of a major storm. The majority if not all of this placed volume would be lost during a 15-year storm or after 2 to 5 years of average wave climate period without major storms.

- Based on the simulation of two forecasted No Action scenarios 10 and 50 years from the present (not Status Quo, no dune nourishments included in simulation setup), all remaining storm protection provided by the dune between R-130 and R-134 would be lost after one major storm event. Seawalls that were buried within the dune would become exposed and subjected to wave action. The seawalls between R-136 and R-138 would possibly fail due to toe scour depending on the depth of the wall, allowing erosion of upland property and damage to infrastructure.

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SUB-APPENDIX G-2

DRAFT IH2VOF MODELING REPORT

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SOUTHERN PALM BEACH ISLAND COMPREHENSIVE SHORELINE STABILIZATION PROJECT DRAFT IH2VOF MODELING REPORT

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1.0 INTRODUCTION

Under direction of the U.S. Army Corps of Engineers (USACE), CB&I Coastal Planning & Engineering, Inc. (CB&I) assisted in the development of the Southern Palm Beach Island Comprehensive Shoreline Stabilization Project Environmental Impact Statement (EIS). The initial tasks associated with the effort included public scoping and agency coordination to determine what data was necessary to develop the EIS. After review of the data and previous work, the USACE determined that numerical modeling of seawall overtopping and wave forces was required to obtain necessary data that is not currently available.

The upland property along Southern Palm Beach Island is at risk of flooding if seawalls fail or are overtopped by waves. It was necessary to assess the potential for seawall overtopping as well as the wave generated forces on it. To do that, the IH2VOF model was applied. The model solves the Reynolds-Averaged Navier-Stokes equation using a Volume-of-Fluid (VOF) Approach. The storm profiles generated during the SBEACH analysis of alternatives (CB&I, 2014) was used to analyze the potential wave overtopping and to provide visual and numerical results. The model was also used to evaluate wave forces on seawalls.

Two locations were simulated in IH2VOF, one without a seawall (R131) and one with a seawall (R137). At each location, the existing condition (SBEACH storm profile) and two alternatives were simulated and compared. The alternatives that were considered in the analysis included:

- Alternative 2 – Applicants' Preferred Project (Proposed Action): Beach and Dune Fill with Shoreline Protection Structures.
- Alternative 6 – The Town of Palm Beach Increased Sand Volume and County Increased Sand Volume without Shoreline Protection Structures

The remainder of the alternatives (Alternatives 3, 4, and 5) did not need to be in the analysis. Alternative 3 included the same fill volume as Alternative 2, but without shore protection structures. Alternatives 4 and 5 were combinations of Alternatives 2 and 6. Alternative 1 was considered the No Action Alternative (Status Quo) and was represented by the comparisons with the existing conditions.

2.0 IH2VOF

IH2VOF is a two-dimensional (vertical) wave model developed by IH Cantabria. The model can be applied to a wide range of cases including coastal, ocean, offshore and hydraulic engineering.

2.1. Governing Equations

IH2VOF solves the two-dimensional wave flow for hybrid domain based on coupled Navier-Stokes-type equations. The hybrid domain contains two parts: the clear fluid region and the porous media region. At the clear fluid region, the coupled Reynolds Averaged Navier–Stokes (RANS) equations system is implemented. The Volume-Averaged Reynolds Averaged Navier–Stokes (VARANS) equations are used inside the porous media regions. IH2VOF simulates both mean flow and turbulence with the κ-ε equations for the turbulent kinetic energy κ, and the dissipation rate ε. It permits the modeling of wave flow against any kind of coastal structure (e.g. rubble mound, vertical or mixed breakwaters). The free surface movement is tracked by the volume of fluid (VOF) method.

The RANS equations (clear fluid region) are redefined as follows:

$$\frac{\partial \bar{u}_i}{\partial x_i} = 0$$

$$\frac{\partial \bar{u}_i}{\partial t} + \bar{u}_j \frac{\partial \bar{u}_i}{\partial x_j} = -\frac{1}{\rho}\frac{\partial \bar{p}}{\partial x_i} + g_i + \frac{1}{\rho}\frac{\partial \bar{\tau}_{ij}}{\partial x_j} - \frac{\partial \overline{(u_i{'} u_j{'})}}{\partial x_j}$$

where p is the density of the fluid, g, is the i_{th} component of the gravitational acceleration and τ_{ij} is the mean viscous stress tensor.

The flow inside the porous media is modeled by solving the VARANS equations, first presented by Hsu et al. (2002). These equations are derived by integrating the RANS equations over a control volume, and their final form is presented below:

$$\frac{\partial \langle \overline{u_\iota} \rangle}{\partial x_i} = 0$$

$$\frac{\partial \langle \overline{u_\iota} \rangle}{\partial t} + \frac{\langle \overline{u_J} \rangle}{1 + C_A} \frac{\partial \langle \overline{u_\iota} \rangle}{\partial x_j} = \frac{1}{\rho (1 + C_A)} \left[-\frac{\partial \langle \bar{P} \rangle}{\partial x_i} - \frac{\partial \rho \langle \overline{u_\iota{}' u_J{}'} \rangle}{\partial x_j} + \frac{\partial \langle \overline{\tau_{\iota J}} \rangle}{\partial x_j} + \rho g_i \right] - \frac{1}{1 + C_A} \left[\alpha v \frac{(1-n)^2}{n^2 D_{50}^2} \langle \overline{u_\iota} \rangle + \frac{\beta (1-n)}{n D_{50}^2} \sqrt{\langle \overline{u_1} \rangle^2 \langle \overline{u_2} \rangle^2} \langle \overline{u_\iota} \rangle \right]$$

In the free fluid region, i.e, with $n = 1$ and $C_A = 0$, the VARANS equations is synonymous with the original RANS equations.

2.2. Wave Maker

Wave generation is a key factor for numerical models devoted to coastal engineering, as the generated waves have to resemble observation in the field and laboratory. Several wave generation methods are implemented in IH2VOF in order to compare their abilities to reproduce realistic waves. The mechanisms of wave generation include internal wave maker, static wave paddle (Direchlet boundary condition) and dynamic wave paddle (virtual force method). In this study, the static wave paddle was used. The theory of static wave paddle gives analytical expressions for free surface and the velocity distribution throughout the water column. It is the simplest and most commonly used wave maker in wave models. The static wave paddle can also be used to replicate the behavior of any laboratory wave paddle such as a piston-type wave generator.

2.3. Volume of Fluid Method

In the IH2VOF model, the free surface is tracked using the Volume of Fluid (VOF) method presented by Hirt and Nichols (1981). Instead of pursuing the exact location of the free surface, this method identifies the free surface by tracking the density change in each grid cell. The model identifies three cell types: empty (E), surface (S) and interior (I) cells depending on the value of the VOF function F defined as follows:

$$F = \frac{\rho}{\rho_f}$$

Where

$$\rho = \frac{\rho_f V_f}{V_f + V_a}$$

being ρ the fluid density, V the volume of fluid in the cell, and V_a the volume of air in the cell.

Empty cell is defined as $F = 0$, which contains only air. Interior cell is defined as $F = 1$, which contains pure fluid. Surface cell is defined as $1 > F > 0$, which contains both fluid and air. The introduction of the VOF function in the equation of mass conservation yields the transport equation for $F(x, y, t)$:

$$\rho(x, y, t) = F(x, y, t)\rho_f$$

$$\frac{\partial F}{\partial t} + \frac{\partial}{\partial x}(\bar{u}F) + \frac{\partial}{\partial y}(\bar{v}F) = 0$$

2.4. Overtopping Damage Criteria

According to Peng and Zou (2011), overtopping is defined as the volume of water that passes over the crest of a structure per one unit of length per one unit of time. The mean discharge is expressed in m³/m/s. EurOtop (2007) describes the overtopping discharge over a structure crest as a random process over time and volume due to wave nonlinearity. Larger waves will overtop greater quantities of water in a short time period (less than a wave period), smaller waves may not produce any overtopping.

Several important factors contributing to the overtopping process due to waves have been identified including wave height and period, the structure (or dune) elevation, structure (or beach) slope, water thickness and current velocity at the top of the structure (Schüttrumpf and Oumeraci, 2005; Lykke Andersen et. al., 2006; Van der

Meer et. al., 2009). EutOtop (2007) classified the mean overtopping discharge (q) according to the impact factor:

$q < 0.1$ liters/s/m: Insignificant with respect to strength of crest and rear of structure;

$q = 1$ liters/s/m: On crest and inner slopes grass and/or clay may start to erode;

$q = 10$ liters/s/m: Significant overtopping for dikes and embankments. Some overtopping for rubble mound breakwaters;

$q = 100$ liters/s/m: Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated.

The Coastal Engineering Manual (USACE, 2006) compiled information from a series of studies in a table that presents the critical values of mean overtopping discharge (Figure 2-1). The values presented should be considered as guidelines because for a given amount of instantaneous overtopping, the damage caused by the overtopped water largely depends on the geometry of structure (or beach profile) and the distance from the structure. Maximum intensity may be locally two orders of magnitude greater than the mean overtopping discharge. The acceptable condition is a matter that varies depending on the location and the objective of each project.

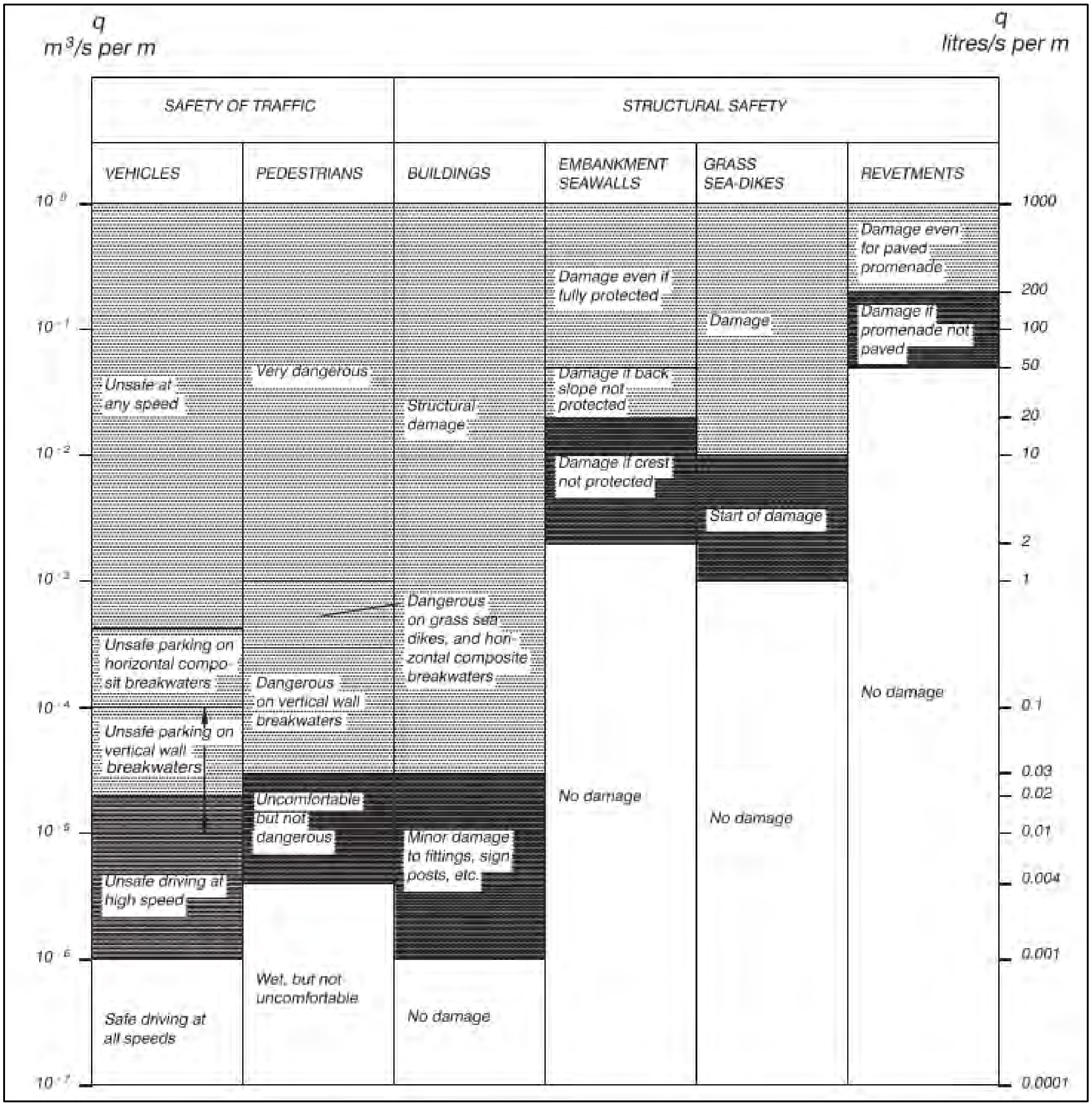

Figure 2-1. Critical overtopping discharge according to USACE (2006).

2.5. Input Data and Simulated Scenarios

Input data for the IH2VOF model consists of beach profile, water level, and wave conditions. Two locations were simulated: one without a seawall (R131) and one with a seawall (R137). Alternative 2 represented the smallest beach fill quantity, while Alternative 6 represented the largest volume. The beach fills' response to storm events were first modeled in SBEACH. The storm profiles were taken from SBEACH simulation (CB&I, 2014) at the event peak for each alternative. At R131, both alternatives (2 and 6) are identical. Figure 2-2 shows the input profiles at R131 and R137 with elevation in meters referred to mean sea level.

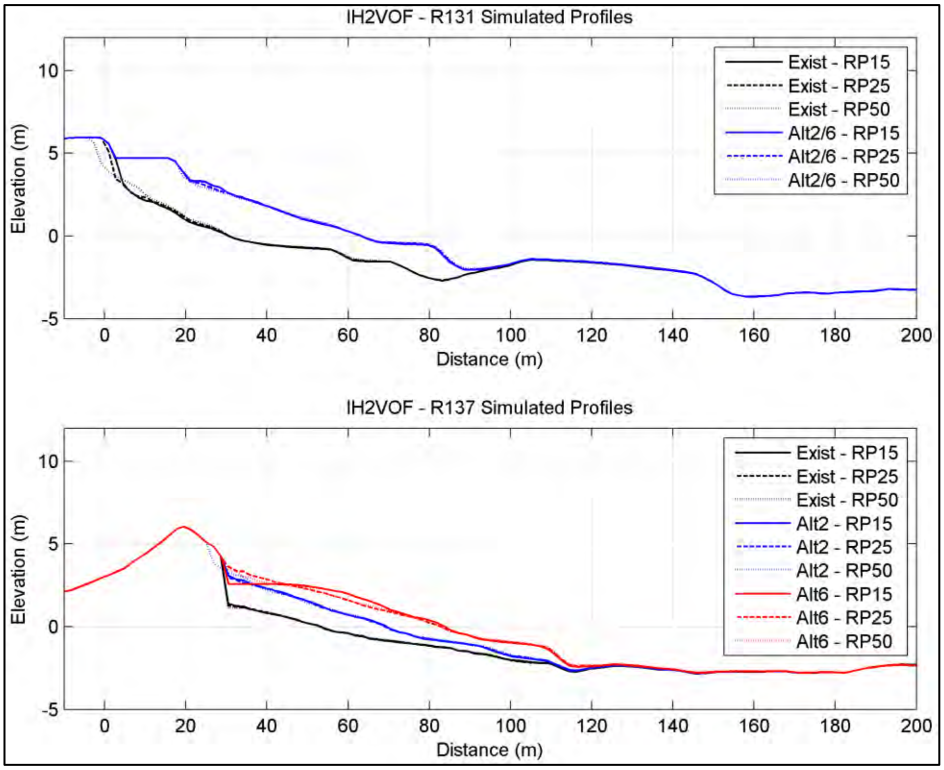

Figure 2-2. IH2VOF input profiles taken from SBEACH at event peak for existing conditions and for alternative 2/6. R131 (top) and R137 (bottom).

The water level used in IH2VOF was also taken from the SBEACH simulations (CB&I, 2014) and was defined as a constant elevation above mean sea level. For return periods (RP) storms of 15, 25 and 50 years, the water level considered was 1.52, 1.68 and 1.92 m, respectively.

Wave conditions were also obtained from the SBEACH simulation (CB&I, 2014). At event peak, the wave profile was analyzed and the values were obtained at a distance coincident to the IH2VOF offshore boundary, which was approximately 500 m seaward of the monuments. Three extreme wave conditions were simulated (15, 25 and 50 years return period) for each alternative and the existing profile (Figure 2-3).

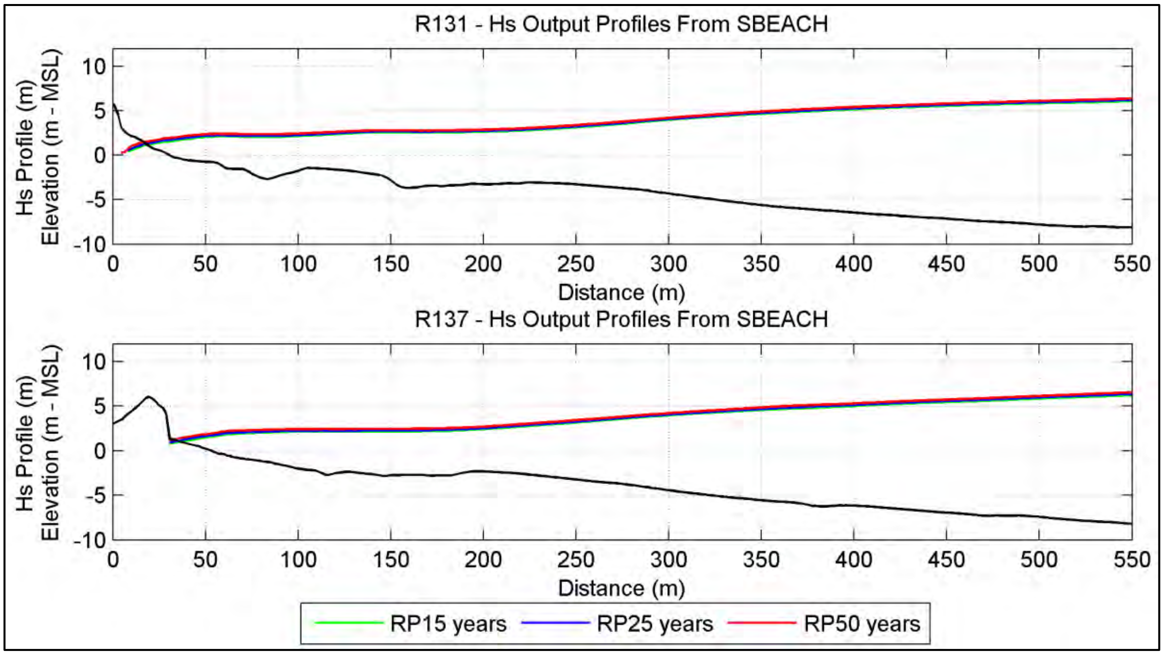

Figure 2-3. Wave profiles at R131 and R137 for return period waves of 15 (green), 25 (blue) and 50 (red) years.

Differences between wave conditions for both profiles as well as between alternatives are presented in Table 2-1.

Regardless of the alternatives or existing conditions, it can be seen that the significant wave heights vary less than 0.01, 0.02 and 0.04 m for the 15, 25 and 50 year return period waves, respectively. This represented less than 0.1% difference between the wave height for all alternatives, so the difference was considered negligible and the same wave condition for each return period wave (5.86 m for 15 year, 5.99 m for 25 year, 6.16 m for 50 year) was imposed at the IH2VOF boundary.

Table 2-1. Offshore significant wave height SBEACH output at IH2VOF boundary.

Return period / Case (year)	R131 (m)		R137 (m)		
	Existing	Alternative 2/6	Existing	Alternative 2	Alternative 6
15	5.85	5.85	5.86	5.86	5.86
25	5.97	5.97	5.99	5.99	5.99
50	6.12	6.12	6.16	6.14	6.14

Three wave conditions were simulated for each return period storm. The wave parameters presented above were used in IH2VOF wave maker, which generated irregular wave timeseries. Based on the wave timeseries, the horizontal (U) and vertical (V) current velocity field were also imposed at model boundary for continuity. The timeseries, histograms, spectrum and Hs x Tp and U and V field plots are presented below for 15 (Figure 2-4), 25 (Figure 2-5) and 50 (Figure 2-6) years return period waves.

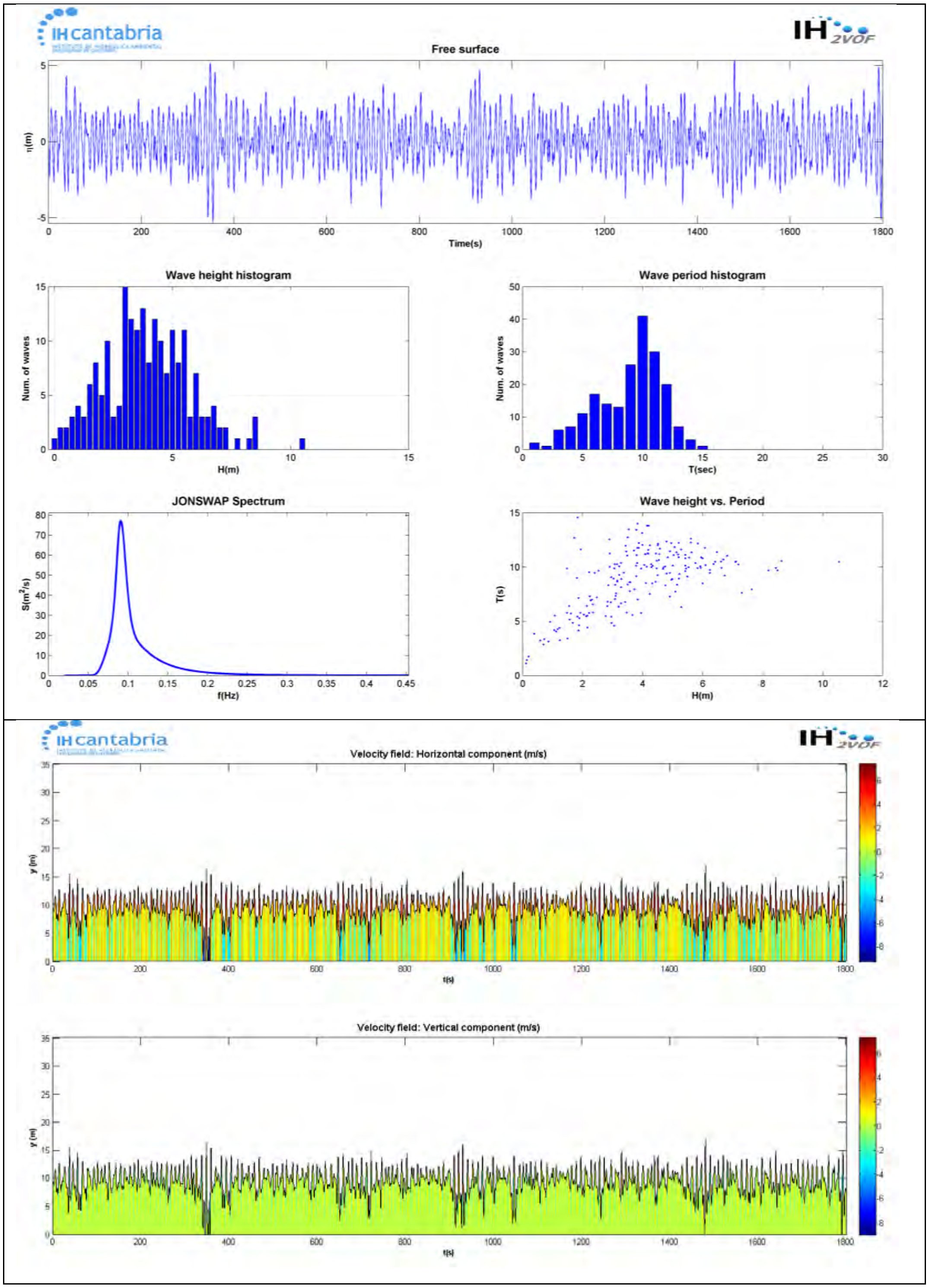

Figure 2-4. 15 years return period waves – η timeseries, wave height and period histograms, JONSWAP spectrum, Hs x Tp and U and V velocity fields.

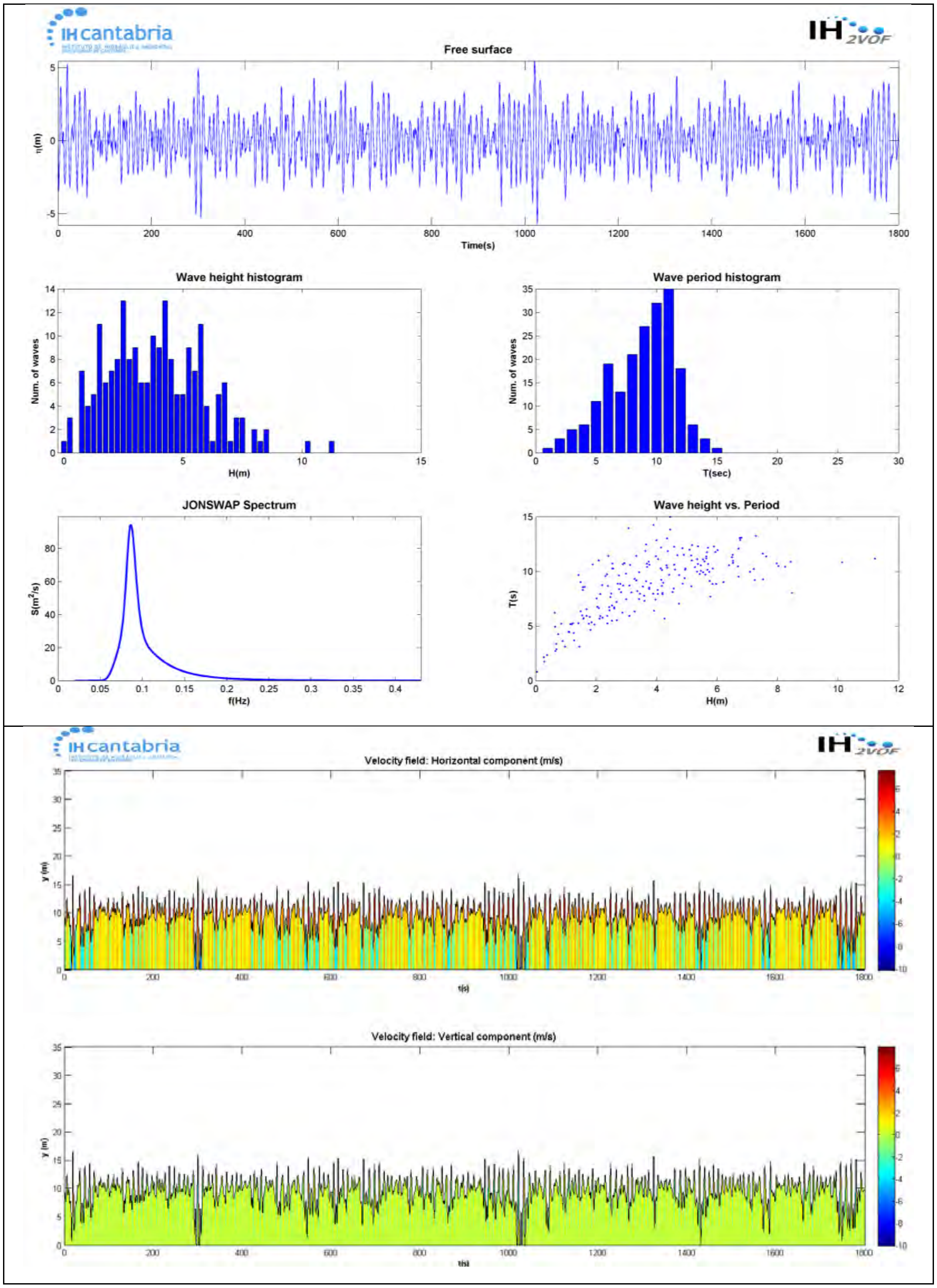

Figure 2-5. 25 years return period waves – η timeseries, wave height and period histograms, JONSWAP spectrum, Hs x Tp and U and V velocity fields.

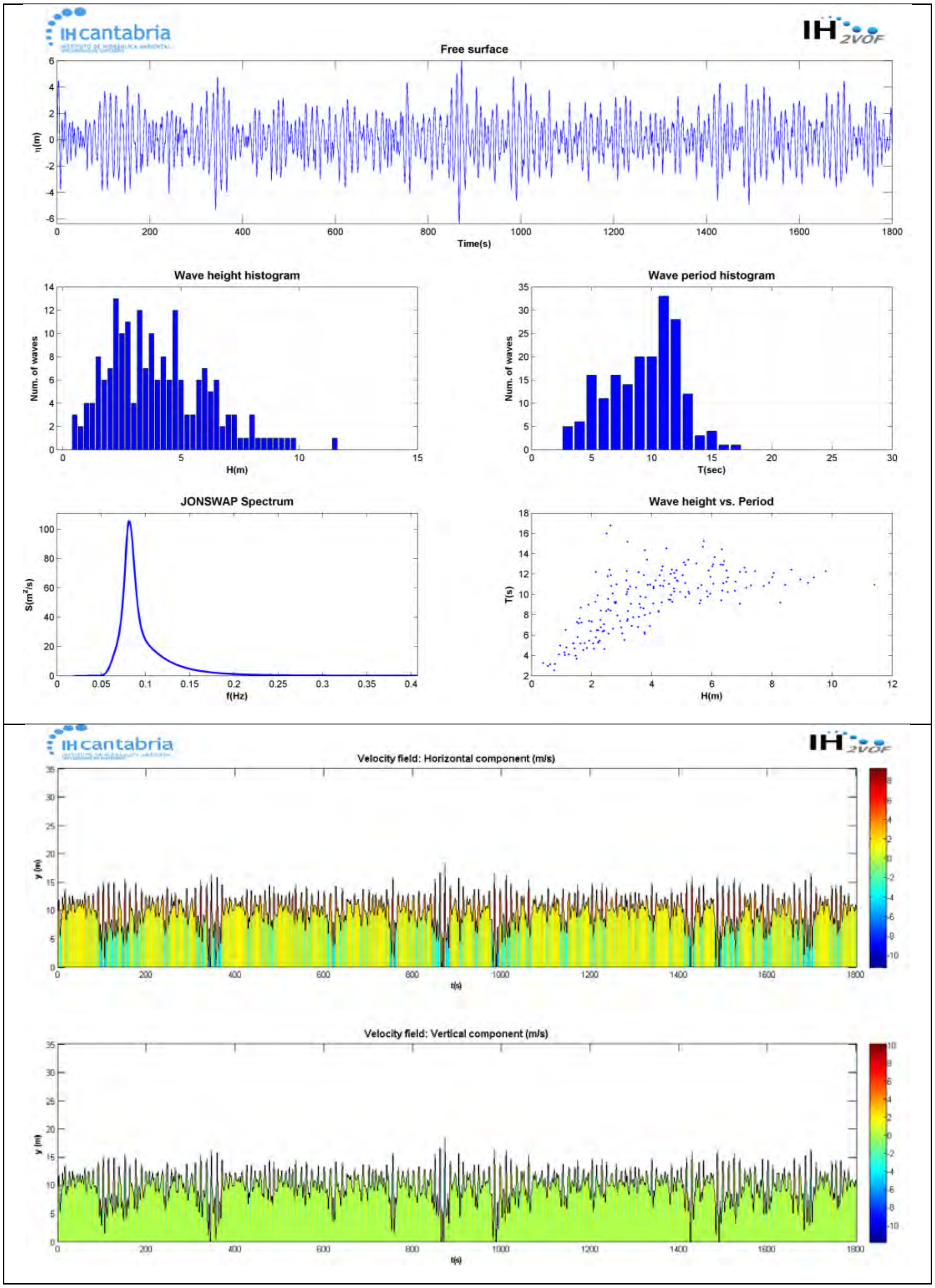

Figure 2-6. 50 years return period waves – η timeseries, wave height and period histograms, JONSWAP spectrum, Hs x Tp and U and V velocity fields.

The regular numerical grid used in IH2VOF consists of a 2DV grid of 501 meter x 35.1 meter (1670 x 234 numerical elements) with horizontal resolution of 0.3 meter and vertical resolution of 0.15 meter (Figure 2-7). The recommended relation of $Dx < 2.5\,Dy$ was maintained to avoid wave false breaking. The grids and outputs from IH2VOF model are oriented with landward being to the right and offshore to the left. For reference, this is opposite of the orientation of the SBEACH profiles shown in Figure 2-2 and Figure 2-3.

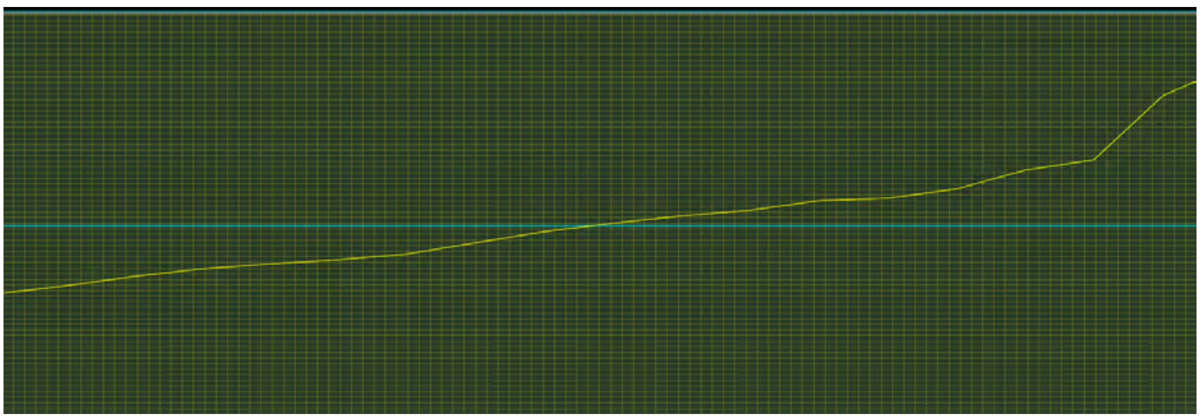

Figure 2-7. IH2VOF numerical grid and a zoom-in view at swash zone: beach profile (yellow) and water level (cyan).

A total of 15 simulations were conducted at R131 and R137. The return period conditions of 15, 25 and 50 years were considered for the existing condition and alternatives. A summary of simulated scenarios are presented in Table 2-2. All the simulations started with a constant water level condition (cold start). Thus, the first 5 minutes were considered as the "spinup" time and were not used in the analysis presented below. Following the "spinup" time, the total analysis time was 25 minutes (1500 seconds). It is important to highlight that the swash zone is a very active zone with nonlinear wave interactions. The wave randomness and nonlinear wave interactions may cause a single overtopping wave to occur higher in a wider beach than existing condition, while the frequency and cumulative overtopping are less. To avoid misinterpretation, a period of 25 minutes was chosen, so the stochastic events are minimized and the mean condition is considered.

Table 2-2. Offshore wave and water level conditions simulated on IH2VOF.

Profile	Condition	Return Period (years)	Hs (m)	Tp (s)	Water Level (m)
R131	Existing	15	5.86	11	1.52
	Existing	25	5.99	11.5	1.68
	Existing	50	6.16	12.2	1.92
R131	Alternative 2/6	15	5.86	11	1.52
	Alternative 2/6	25	5.99	11.5	1.68
	Alternative 2/6	50	6.16	12.2	1.92
R137	Existing	15	5.86	11	1.52
	Existing	25	5.99	11.5	1.68
	Existing	50	6.16	12.2	1.92
R137	Alternative 2	15	5.86	11	1.52
	Alternative 2	25	5.99	11.5	1.68
	Alternative 2	50	6.16	12.2	1.92
R137	Alternative 6	15	5.86	11	1.52
	Alternative 6	25	5.99	11.5	1.68
	Alternative 6	50	6.16	12.2	1.92

3.0 RESULTS

IH2VOF results presented hereafter for both R131 and R137 include:

- Screenshots at specific times comparing the existing condition with the alternatives
- Water level, η, timeseries are presented and the existing condition is compared to the alternatives
- Overtopping results are presented and discussed.

3.1. R131

Three screenshots are presented below representing the peak of the 15 (Figure 3-1), 25 (Figure 3-2) and 50 year (Figure 3-3) return period storms for R131.

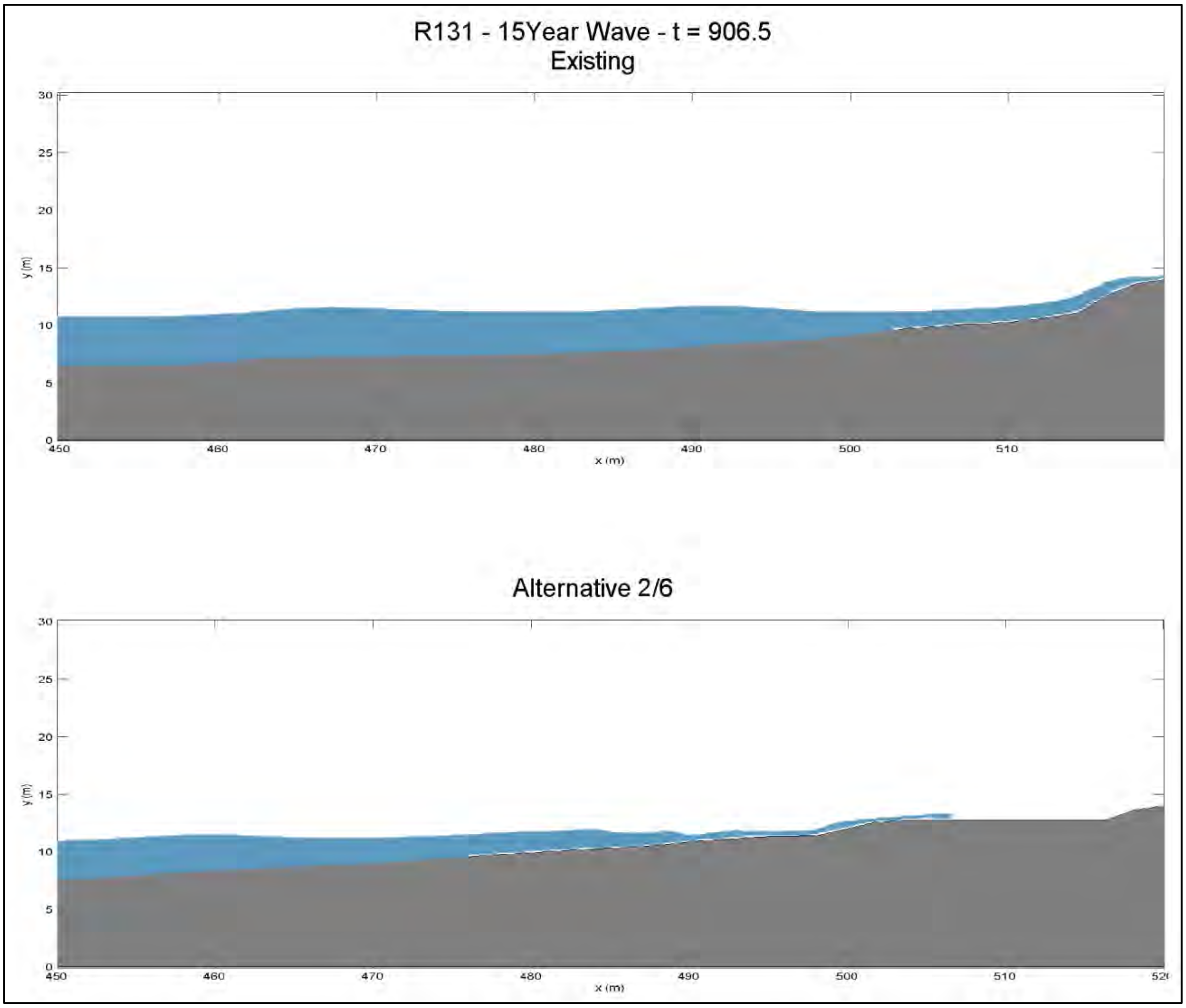

Figure 3-1. R131 - 15 year return period wave at 906.5 s of simulation for existing (top) and Alternative 2/6 (bottom).

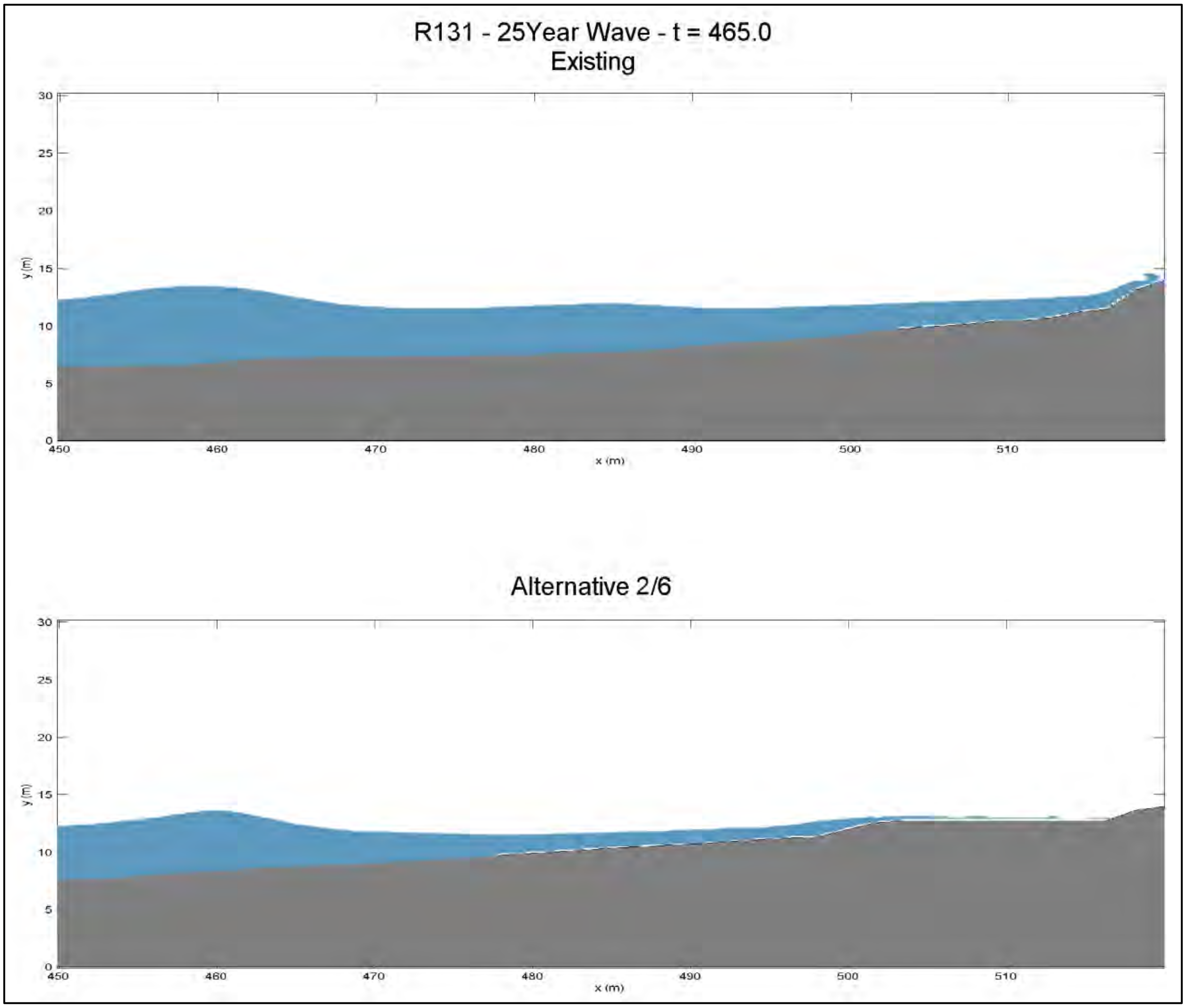

Figure 3-2. R131 - 25 year return period wave at 465 s of simulation for existing (top) and Alternative 2/6 (bottom).

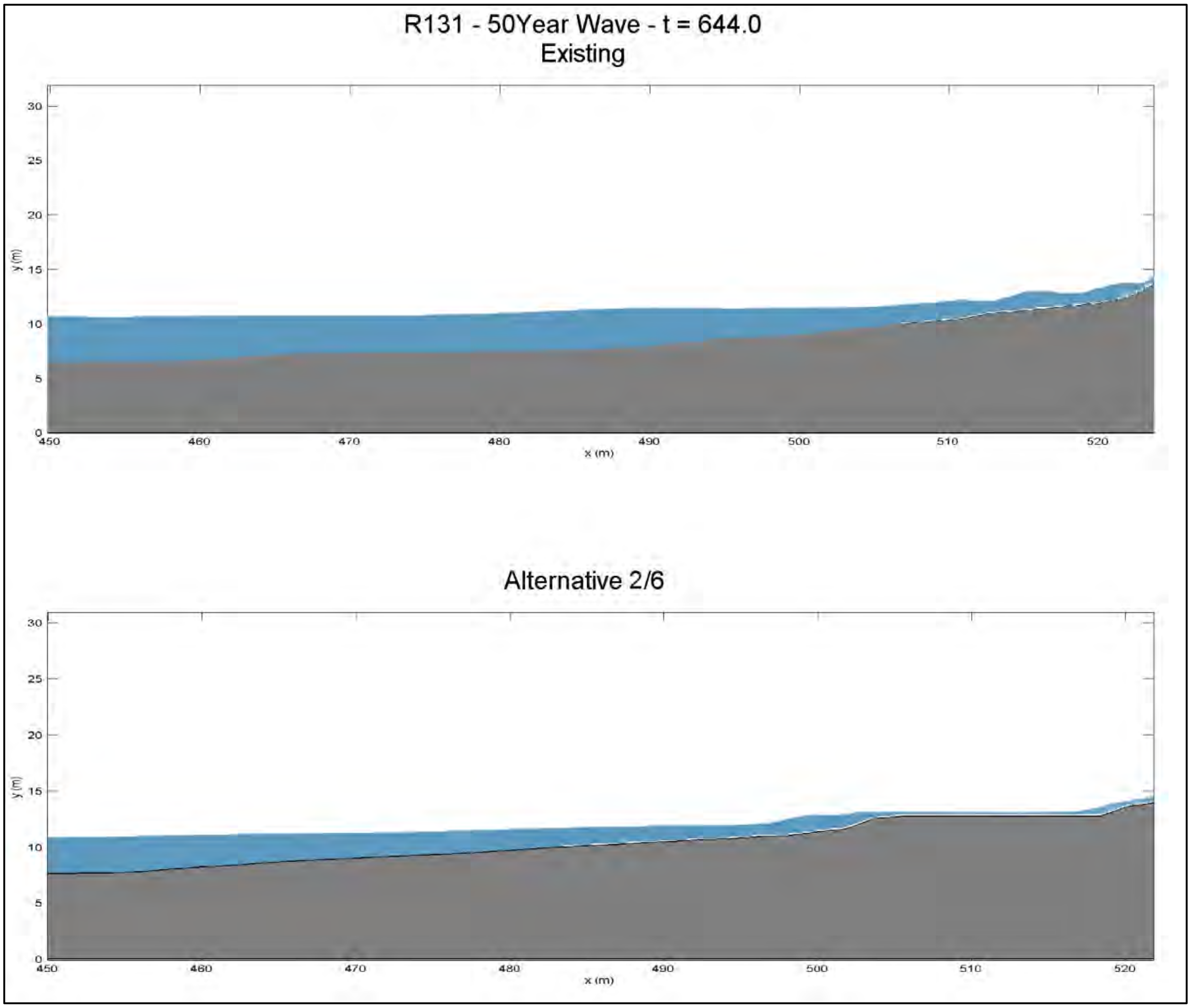

Figure 3-3: R131 - 50 year return period wave at 644 s of simulation for existing (top) and Alternative 2/6 (bottom).

- 15 Year Return Period Storm: Under existing conditions, it is possible to see the wave overtopping at the dune crest during the 15 year storm. The same wave condition does not reach the dune for alternative 2/6. This is attributed to the rapid loss of wave energy by depth-induced breaking as waves propagate across the filled beach profile.

- 25 Year Return Period Storm: For 25 year return period, the waves also overtop the dune crest under the existing condition. The same wave condition for alternative 2/6 only reaches the toe of the dune.

- 50 Year Return Period Storm: For both the existing conditions and alternative 2/6, the dune crest is overtopped by waves during the 50 year storm. However, the overtopping volume is much less for the alternative 2/6.

The IH2VOF models allows for "probes" to be defined in order to extract information at a specific points across the beach profile. Seven probes were placed across the profile to analyze water surface, η, timeseries as the wave propagates toward the beach. The results for 15 (Figure 3-4), 25 (Figure 3-5) and 50 (Figure 3-6) years return period waves are presented below for the existing conditions and the alternatives. The figures suggest that the wave height decreases across the profile from probe 1 to probe 7.

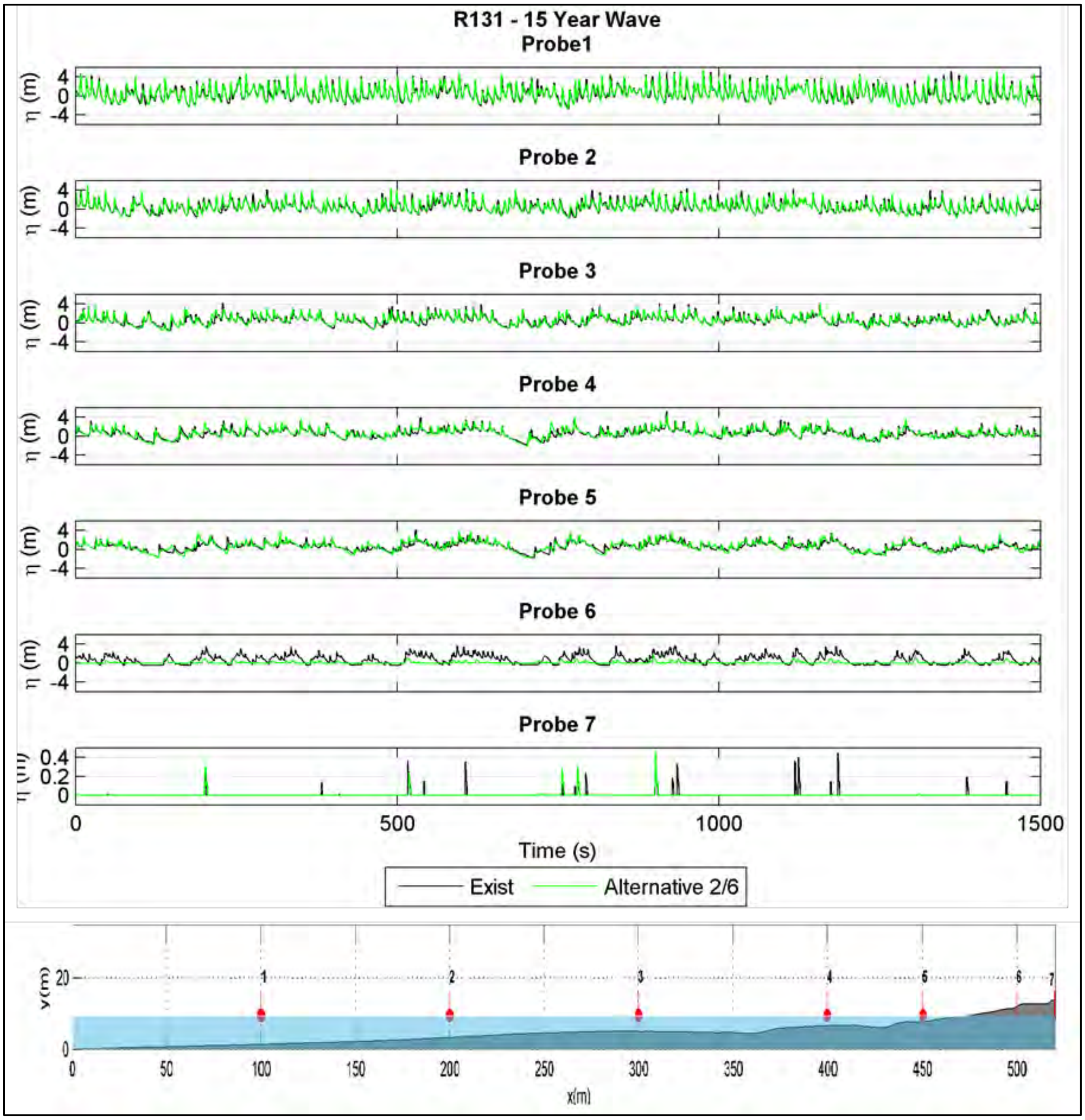

Figure 3-4. R131 – 15 year return period wave η at probes 1 to 7.

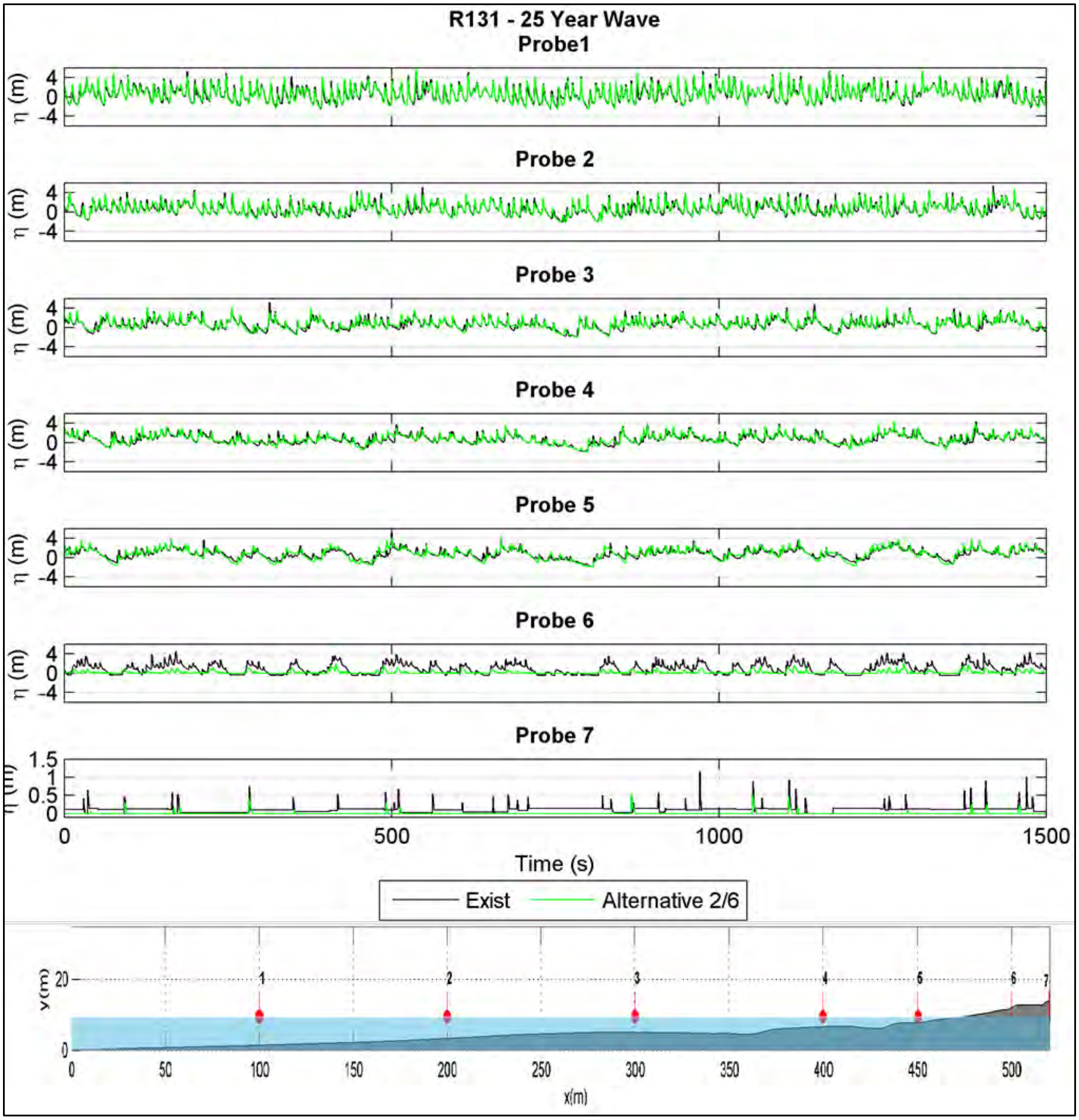

Figure 3-5. R131 – 25 year return period wave η at probes 1 to 7.

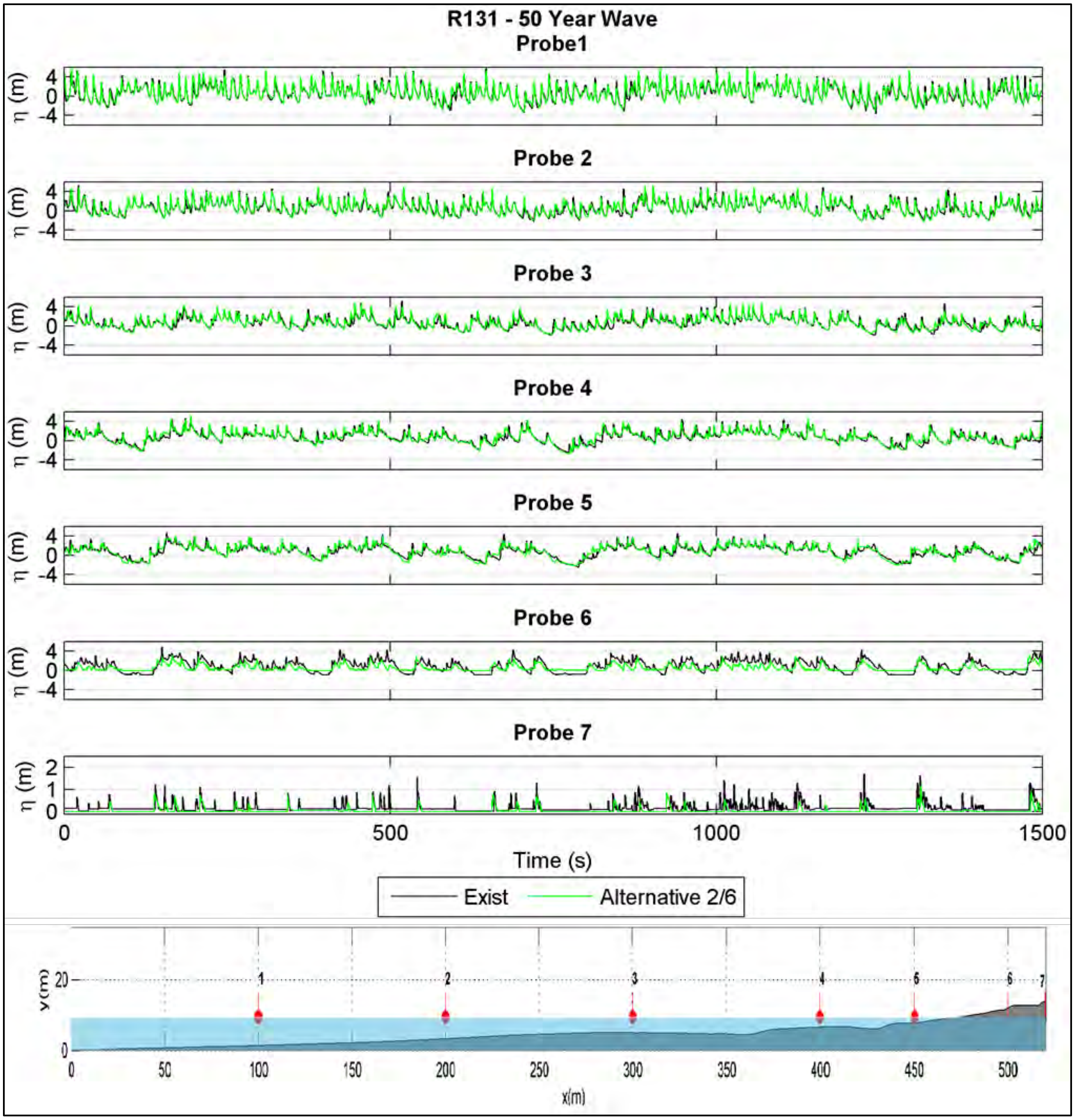

Figure 3-6. R131 – 50 year return period wave η at probes 1 to 7.

- <u>15 Year Return Period Storm:</u> From probe 3 to probe 5, it is possible to observe the transfer of the energy from higher to lower frequencies due to depth-induced wave breaking. At probe 6, the presence of the beach fill from Alternatives 2/6 reduces wave energy. Overtopping of the dune crest is shown by probe 7. The thickness of water during overtopping was less than 0.5 m.

- <u>25 Year Return Period Storm:</u> The 25 return period wave is more energetic and also presents a higher water level reducing the differences between existing conditions and alternatives. The overtopping thickness at probe 7 increased to 1.0 m compared to 0.5 m for the 15 year storm.

- <u>50 Year Return Period Storm:</u> The water level and wave height increase for 50 year storm, further reducing the differences between existing and alternative conditions. It should be noted that the observed water level at probe 6 is sometimes negative for existing condition. This is because the beach elevation at probe 6 is submerged for existing condition and above water for alternatives (Figure 3-6. R131 – 50 year return period wave η at probes 1 to 7.). The situation also happens in R137 (Figure 3-15. R137 – 50 year return period wave η at probes 1 to 7.). The overtopping thickness at probe 7 increased to 1.5 m compared to 1.0 m for the 25 year storm.

The dune overtopping results are shown below for 15 (Figure 3-7), 25 (Figure 3-8) and 50 (Figure 3-9) year return period waves. The alternatives reduce wave energy as they propagate over the beach profile and consequently, yield less overtopping than the existing conditions. Due to the randomness of generated waves, the instantaneous overtopping values for the alternatives such as maximum overtopping volume can be larger than the existing condition. However, the cumulative or mean overtopping values are less with the alternatives.

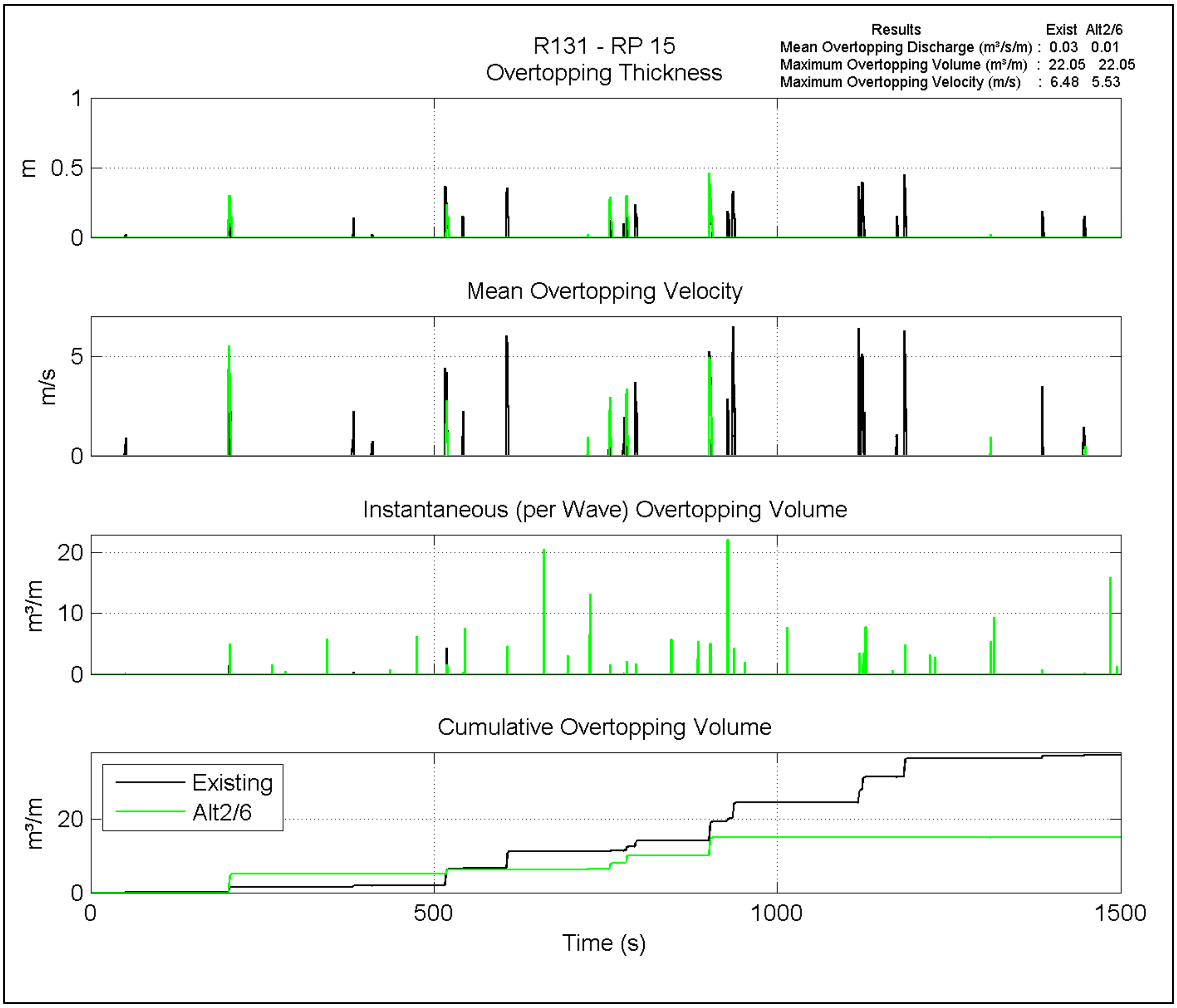

Figure 3-7. Overtopping results - R131 - 15 year return period for existing conditions (black) and with alternative (green).

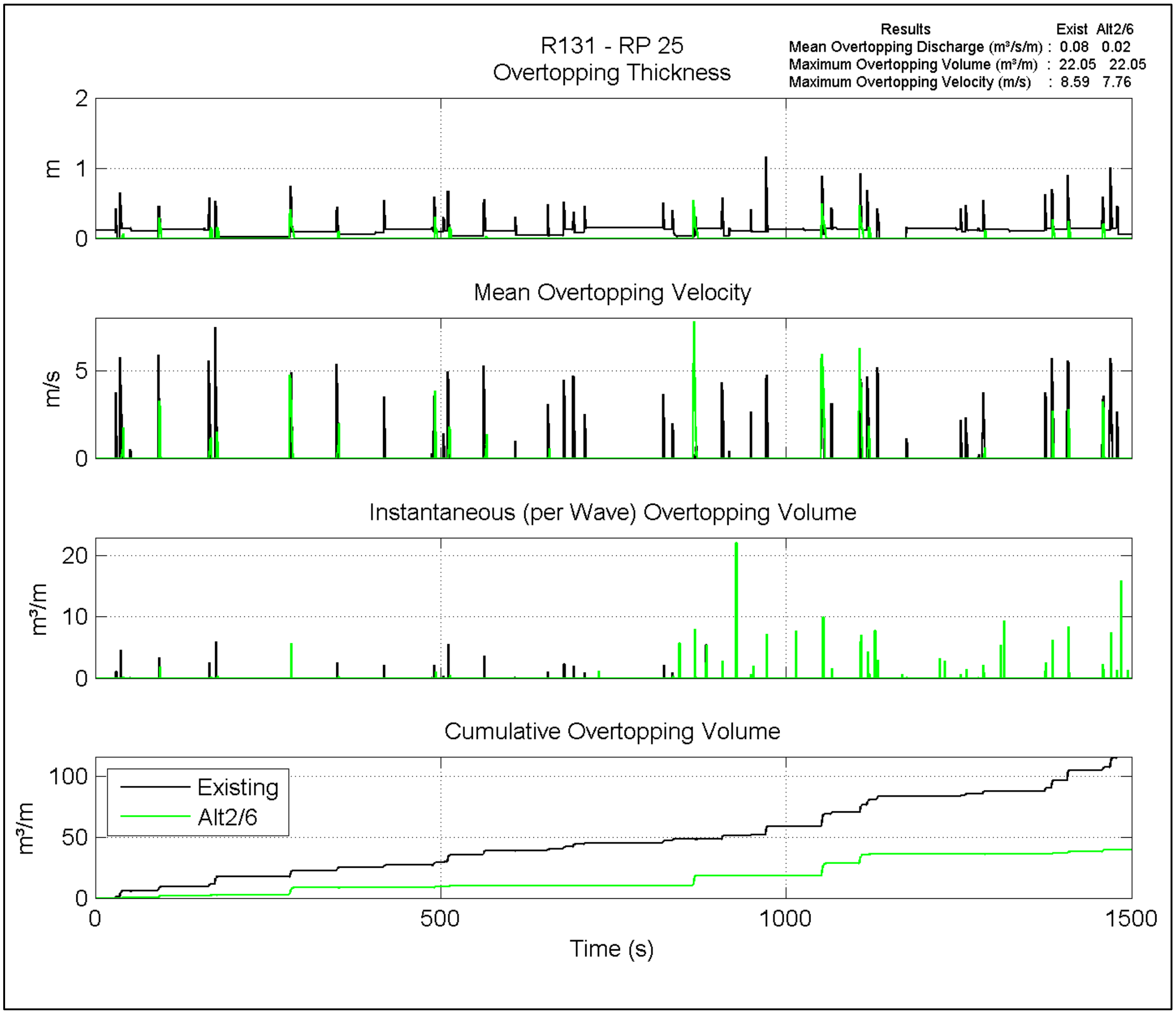

Figure 3-8. Overtopping results - R131 - 25 year return period for existing conditions (black) and with alternative (green).

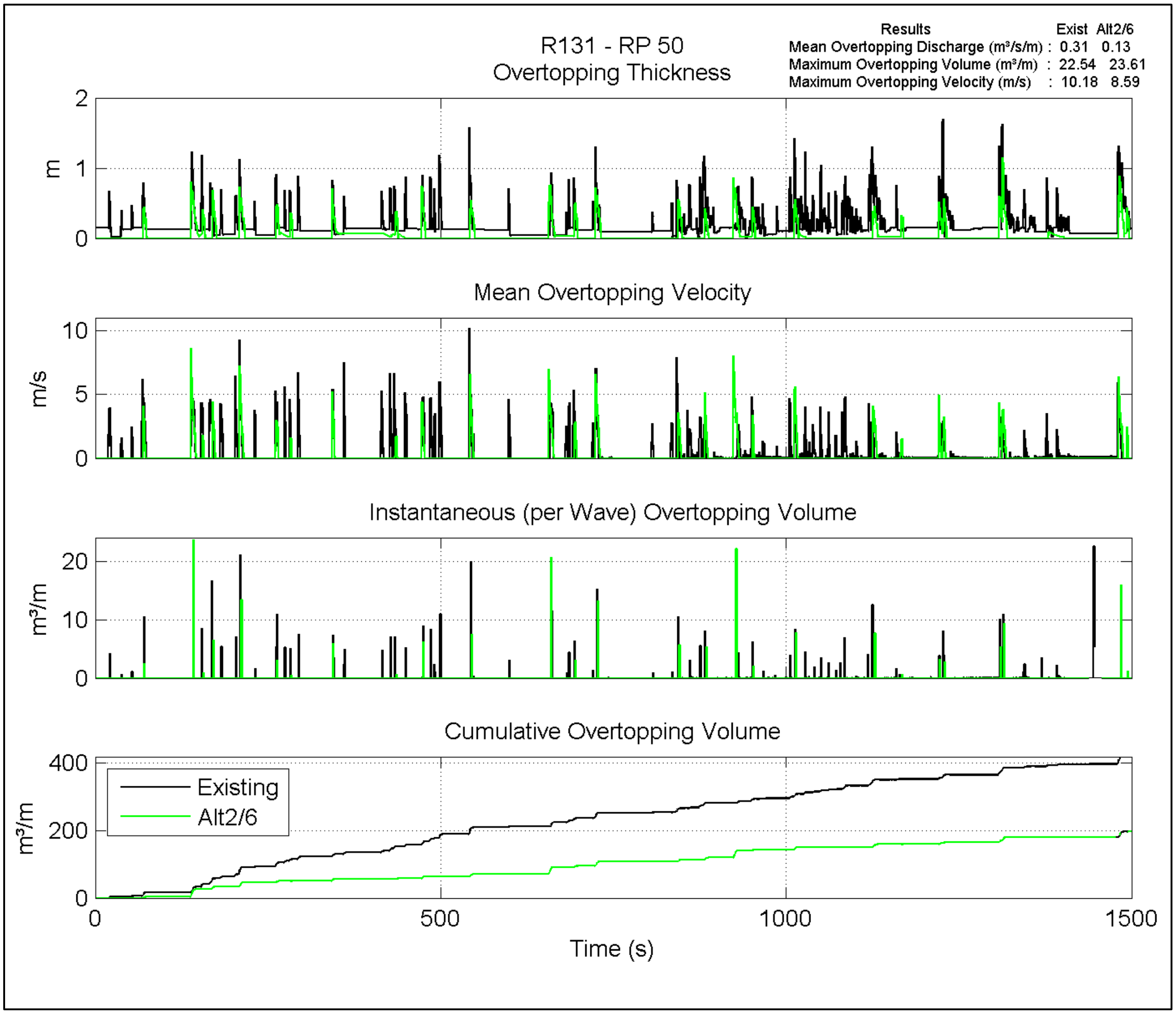

Figure 3-9. Overtopping results - R131 - 50 year return period for existing conditions (black) and with alternative (green).

- <u>15 Year Return Period Storm:</u> The overtopping thickness was lower than 0.5 m. The mean overtopping velocity can reach 6.48 and 5.53 m/s for existing conditions and alternatives, respectively. The instantaneous overtopping volume (volume overtopped by one wave) can reach 5 m^3/m for the existing conditions and alternatives. The mean overtopping discharge for existing and alternative conditions was 0.03 and 0.01 $m^3/s/m$, respectively. This equates to a 67% reduction in overtopping with the alternatives. It should be noted that during the first 500 seconds of the simulation (Figure 3-7) the cumulative overtopping volume for the alternatives exceeded the existing condition, which was attributed to the randomness of generated waves. Over the entire simulation period, the cumulative overtopping volume for the alternatives was less than the existing condition.

- <u>25 Year Return Period Storm:</u> The results showed an overtopping thickness with an order of magnitude of 1 m. The mean overtopping discharge was reduced by 75% from 0.08 $m^3/m/s$ under existing conditions to 0.02 $m^3/m/s$ for the alternatives. The maximum overtopping volumes were similar ranging from 9.71 to 9.99 m^3/m for the existing conditions and the alternatives. The maximum overtopping velocities were reduced from 8.59 m/s to 7.76 m/s for the existing conditions and the alternatives, respectively.

- <u>50 Year Return Period Storm:</u> The results showed an overtopping thickness with an order of magnitude of 1.5 m. There was a 58% reduction of mean overtopping discharge from 0.31 to 0.13 $m^3/s/m$ for the existing conditions and alternatives, respectively. The maximum overtopping volume were similar increasing from 22.54 m^3/m to 23.61 m^3/m and the maximum overtopping velocity was reduced from 10.18 m/s to 8.59 m/s for the existing conditions and alternatives, respectively.

3.2. R137

The variations in water levels and overtopping for the existing conditions, Alternative 2, and Alternative 6 for which a seawall is in place are presented below. Unlike the beach fill at R131, the fill volume at R137 for Alternative 6 was greater than the volume for Alternative 2. Screenshots of simulations at R137 during the peak of the storms are shown in Figure 3-10 through Figure 3-12.

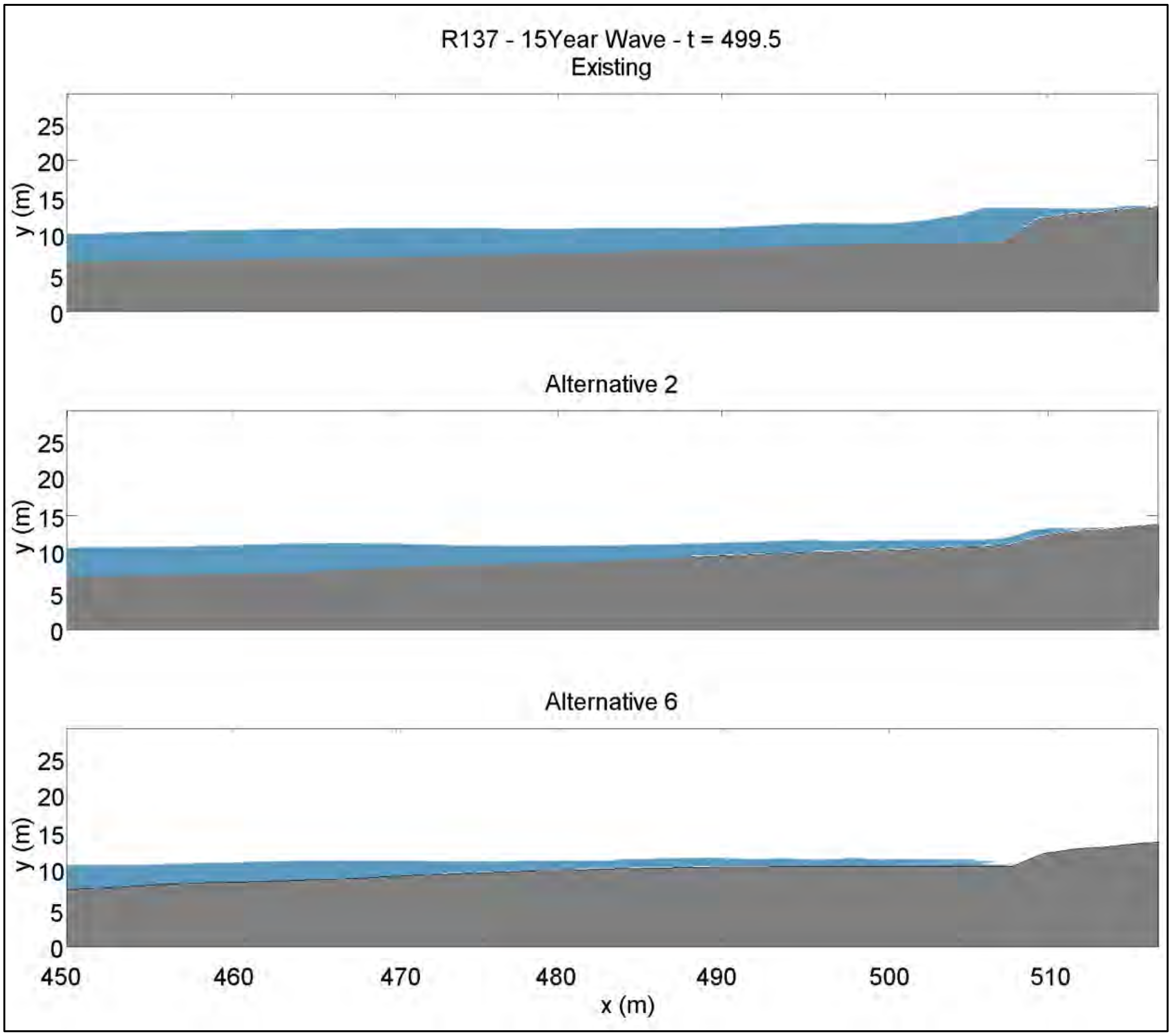

Figure 3-10. R137 - 15 year return period wave at 499.4 s of simulation for existing (top), Alternative 2 (centre) and Alternative 6 (bottom).

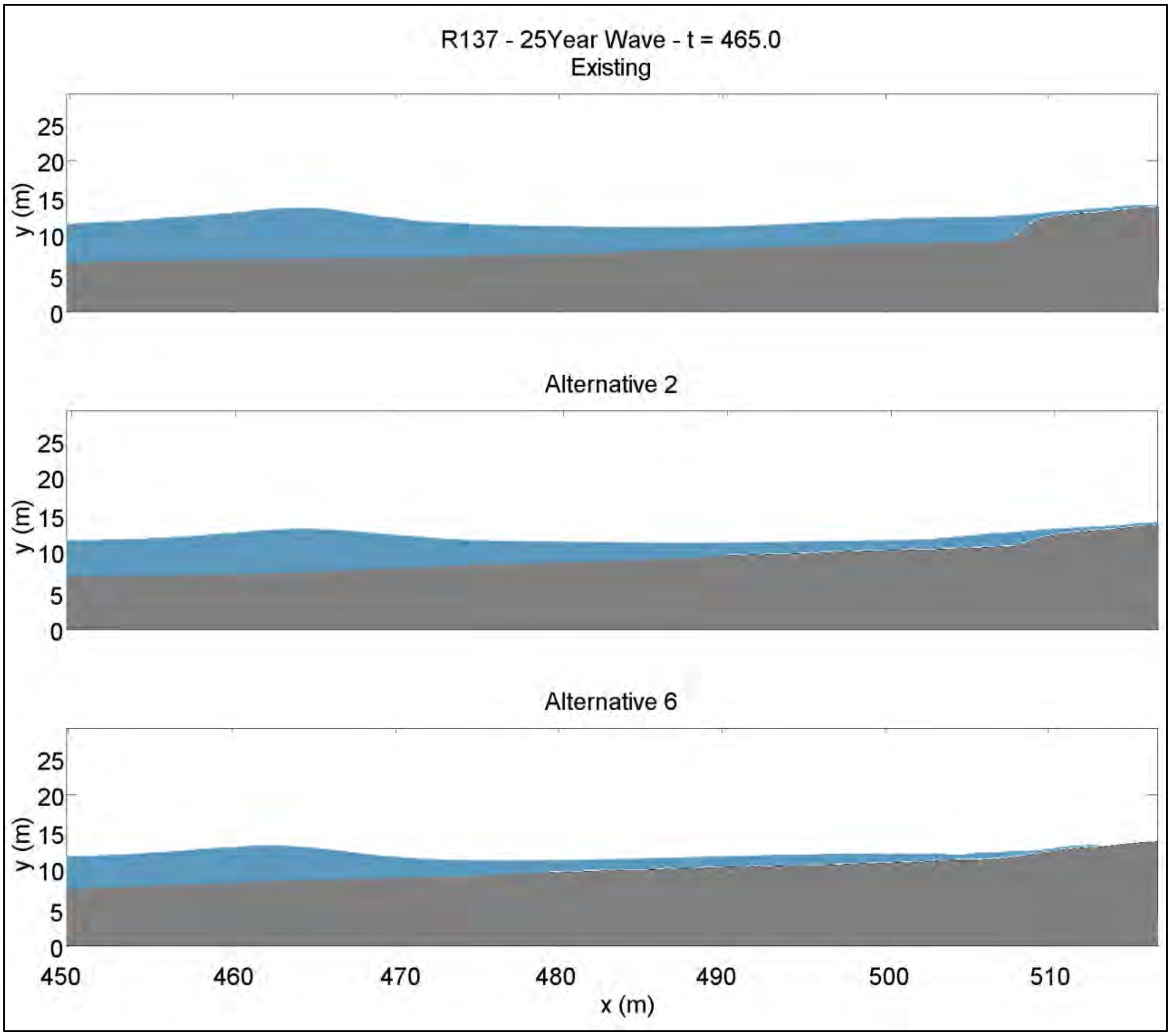

Figure 3-11. R137 - 25 year return period wave at 465 s of simulation for existing (top), Alternative 2 (centre) and Alternative 6 (bottom).

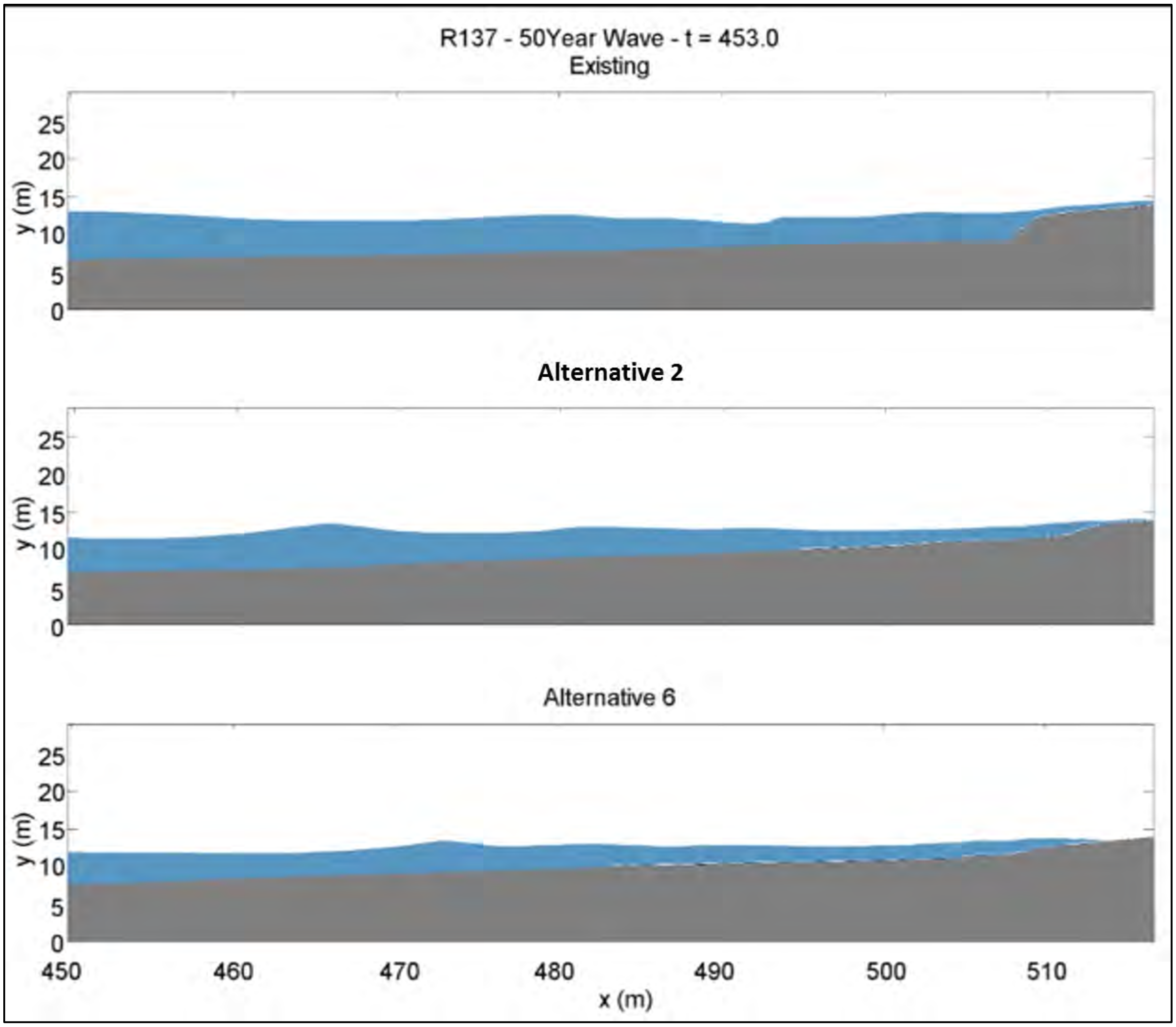

Figure 3-12. R137 - 50 year return period wave at 440 s of simulation for existing (top), Alternative 2 (centre) and Alternative 6 (bottom).

- 15 Year Return Period Storm: Waves overtopped the dune crest for existing conditions. Waves reached but did not overtop the dune crest for Alternative 2, while the waves did not reach the dune for Alternative 6.

- 25 Year Return Period Storm: Waves overtopped the dune crest for the existing conditions and Alternative 2, while overtopping was not evident with Alternative 6.

- 50 Year Return Period Storm: The dune crest was overtopped for all scenarios, but overtopping was less with the alternatives.

Similar to the analysis for R131, seven probes were placed along the beach profile in order to analyze water surface, η, timeseries as the wave propagates toward the beach. The results for 15 (Figure 3-13), 25 (Figure 3-14) and 50 (Figure 3-15) years return period waves are presented below to compare the existing conditions, Alternative 2 and Alternative 6. The figures indicate a reduction in wave height as the waves propagate toward the coast. From probe 4 to probe 6, the wave energy transfer from higher to lower frequencies due to wave breaking, similar to that observed at R131. At probe 6, the beach nourishment on wave propagation as the wave height is reduced with the alternatives as compared to the existing conditions.

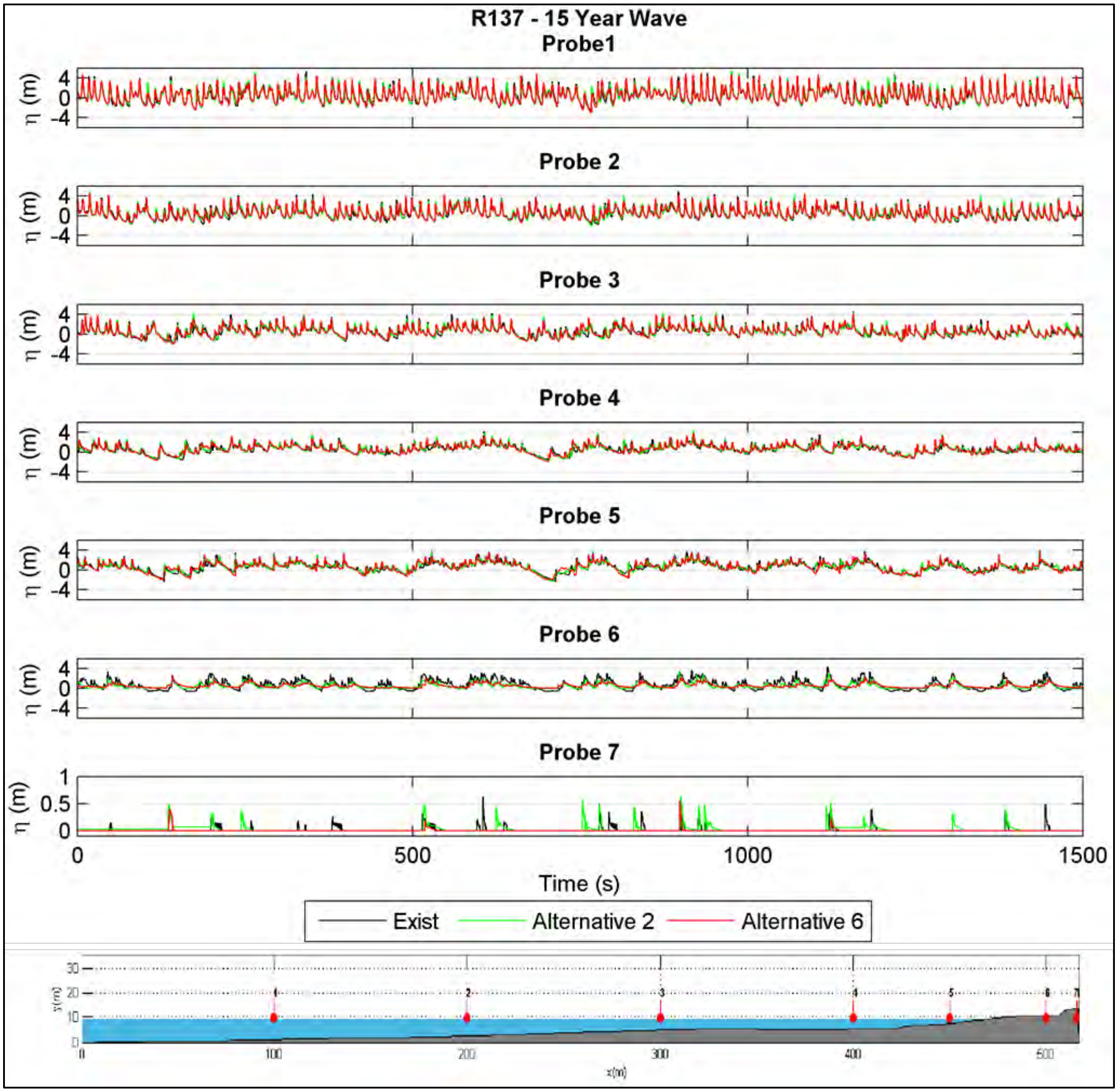

Figure 3-13. R137 – 15 year return period wave η at probes 1 to 7.

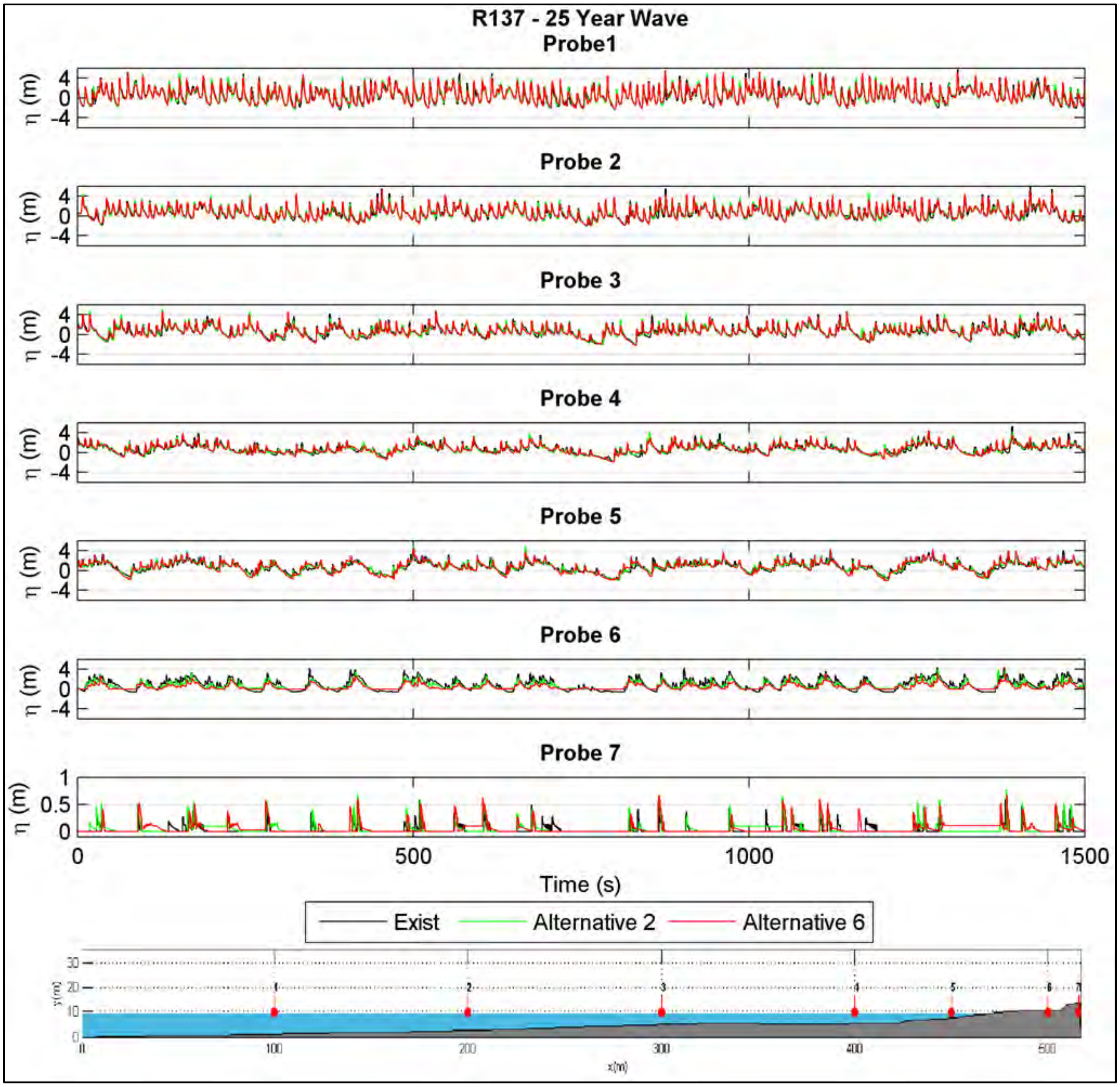

Figure 3-14. R137 – 25 year return period wave η at probes 1 to 7.

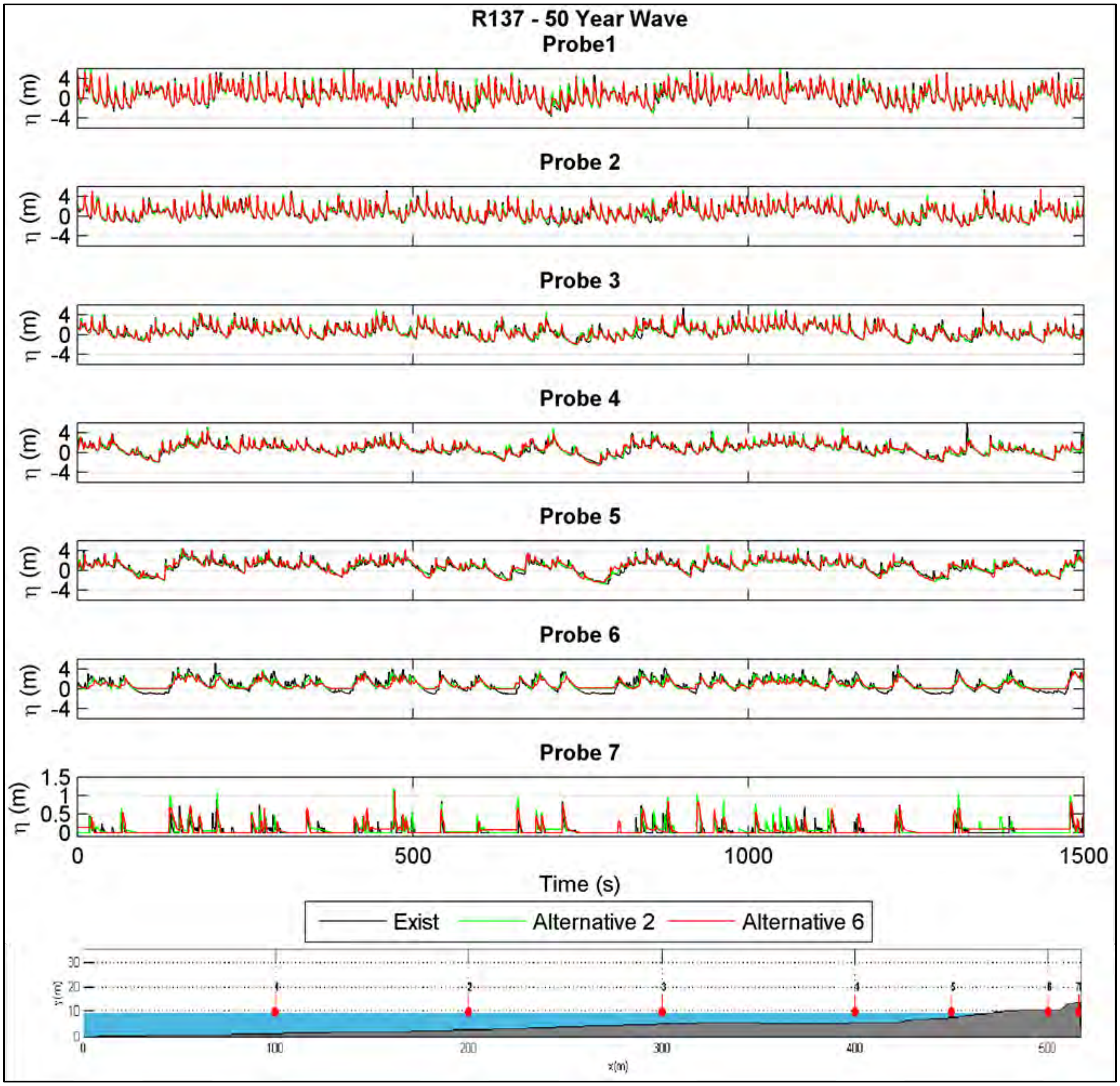

Figure 3-15. R137 – 50 year return period wave η at probes 1 to 7.

- <u>15 Year Return Period Storm:</u> The η timeseries for existing conditions presented much higher range than alternative condition, especially for 15 year return period wave. Probe 7 indicates the overtopping thickness for 15 year return period was 0.6 m.

- <u>25 Year Return Period Storm:</u> The 25 year wave is more energetic and also presents a higher water level, which reduces the differences between existing and alternative conditions. Probe 7 indicates the overtopping thickness for 25 year return period was 0.8 m.

- <u>50 Year Return Period Storm:</u> For 50 year return period waves, the water level is even higher, and waves reached probe 7 more frequently and the differences between existing and alternative conditions are lower. From probe 7, it is possible to see the thickness of water overtopping the dune crest. The overtopping thickness for 50 year return period was 1.2 m. For the same wave condition at probe 6 it is possible to observe that for existing condition the wave runs down and can be below sea level several times during the simulation. On the other hand, for Alternative 6, the probe is located at dry beach and it will not present values lower than zero.

The dune/seawall overtopping results are shown below for 15 (Figure 3-16), 25 (Figure 3-17) and 50 (Figure 3-18) year return period waves. All the results showed that presence of the alternatives reduced the wave energy propagating across the beach profile and consequently the overtopping the dune crest.

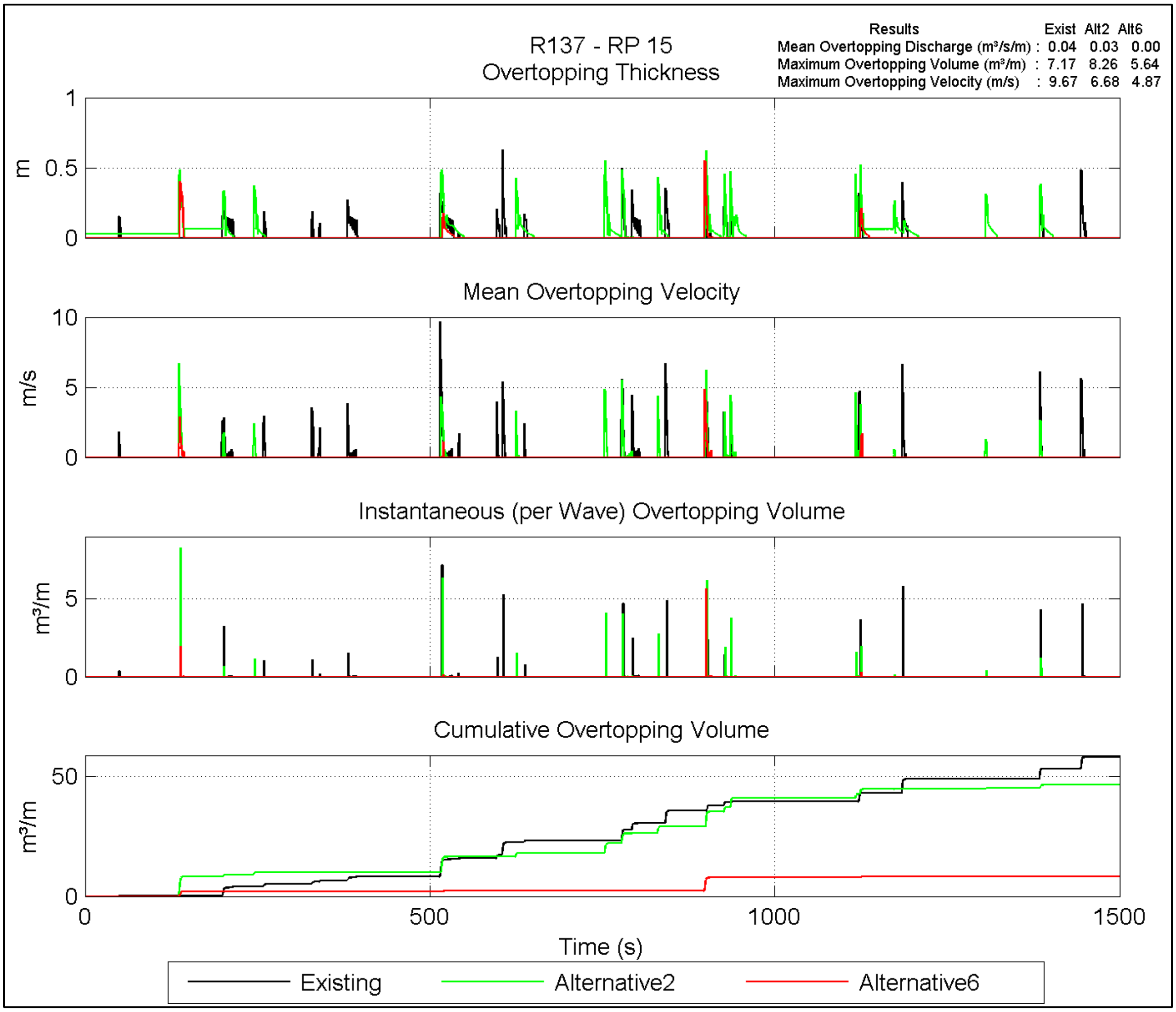

Figure 3-16. Overtopping results - R137 - 15 year return period for existing conditions (black), Alternative 2 (green), Alternative 6 (red).

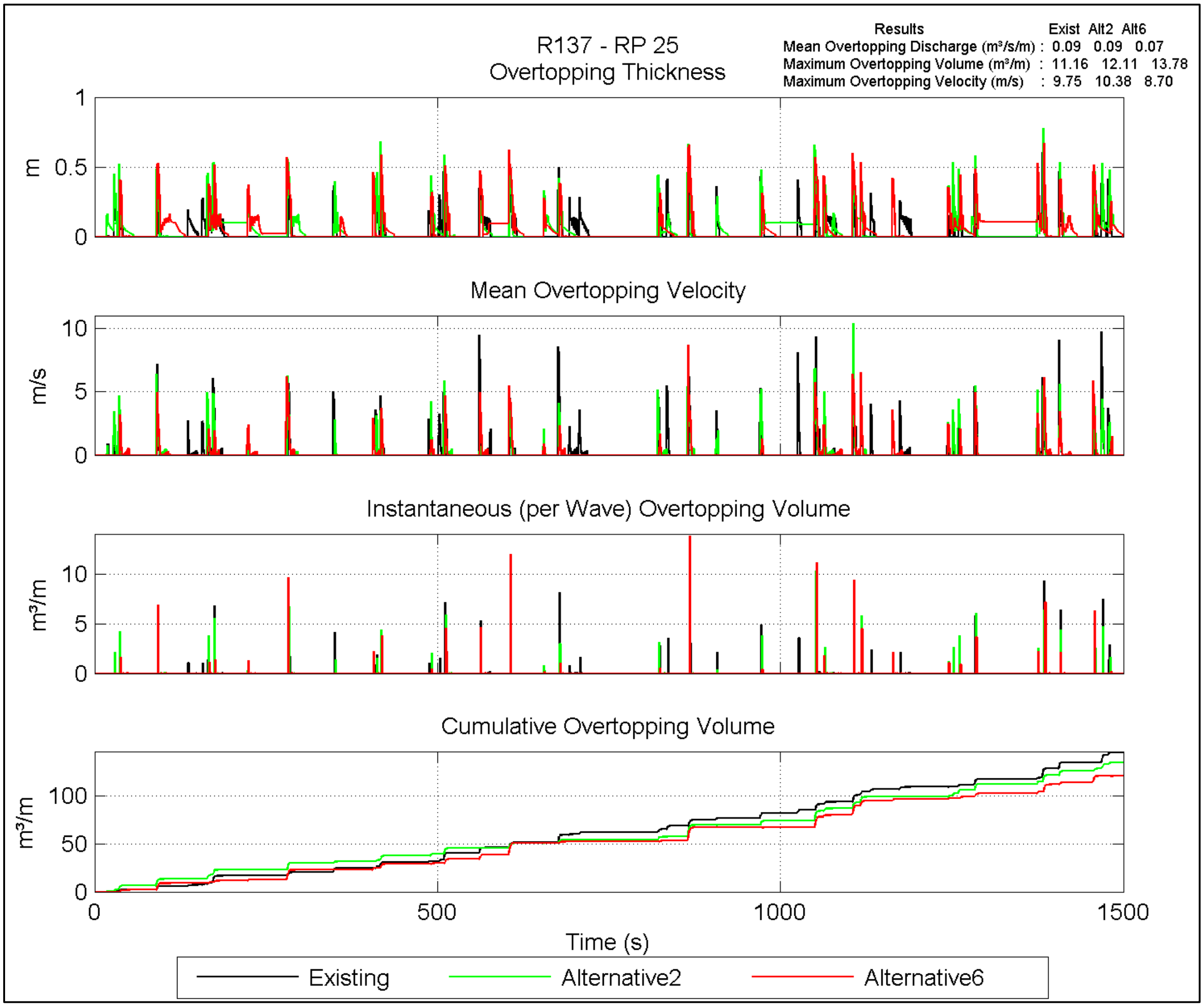

Figure 3-17. Overtopping results - R137 - 25 year return period for existing conditions (black), Alternative 2 (green), Alternative 6 (red).

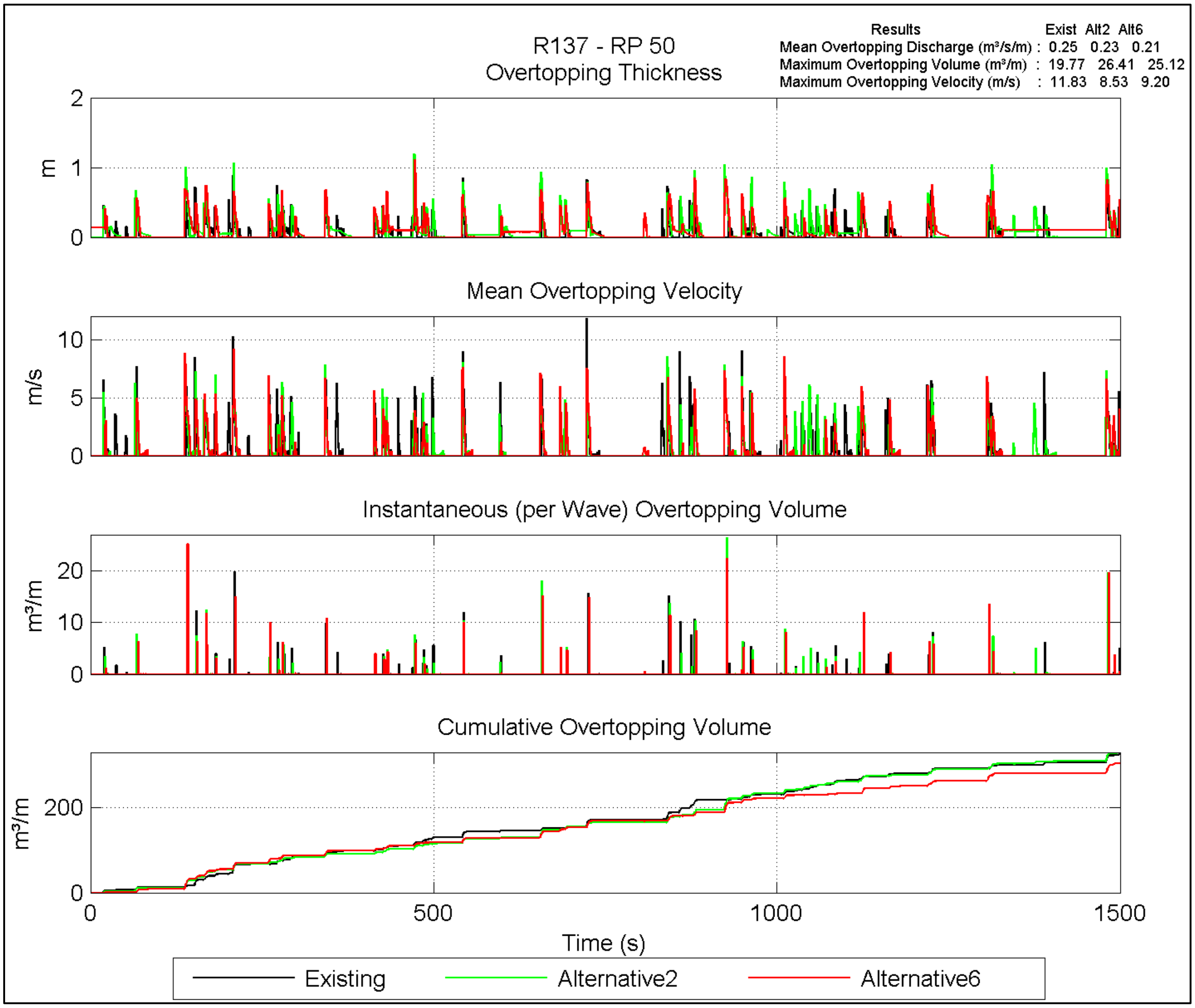

Figure 3-18. Overtopping results - R137 - 50 year return period for existing conditions (black), Alternative 2 (green), Alternative 6 (red).

- <u>15 Year Return Period Storm</u>: The overtopping thickness is about 0.6 m for the existing conditions. Alternative 6 was considerably more effective to avoid overtopping than existing conditions and Alternative 2. The mean overtopping velocity can reach 9.67 (existing), 6.68 (Alternative 2) and 4.87 m/s (Alternative 6). The mean overtopping discharge for existing conditions, Alternative 2, and Alternative 6 was 0.04, and 0.03 and <0.01 $m^3/s/m$, respectively. That represents a reduction of 25% for Alternative 2 and over 88% for Alternative 6 as compared to the existing conditions. The cumulative overtopping volume for Alternative 6 is less than the existing conditions indicating a reduction in the frequency that the dune crest is overtopped. The cumulative overtopping volume for Alternative 2 does not reduce the cumulative overtopping volume significantly from existing condition.

- <u>25 Year Return Period Storm:</u> The results showed an overtopping thickness of about 0.8 m for the exiting conditions. A mean overtopping discharge of 0.09 (existing conditions), 0.09 (Alternative 2) and 0.07 (Alternative 6) $m^3/m/s$ was observed. Alternative 2 showed no improvement, while Alternative 6 resulted in a 22% reduction in the overtopping discharge as compared to the existing conditions.

- <u>50 Year Return Period Storm:</u> The results showed an overtopping thickness of over 1.5 m. For this case, the mean overtopping discharge calculated was 0.25, 0.23 and 0.21 for the existing conditions, Alternative 2 and Alternative 6, respectively. This represented a reduction of 8% for Alternative 2 and 16% for Alternative 6.

Table 3-1. Mean overtopping discharge summary for all cases.

Profile	Condition	Return Period (years)	Mean Overtopping Discharge ($m^3/s/m$)	Reduction in Overtopping
R131	Existing Conditions	15	0.03	-
		25	0.08	
		50	0.31	
	Alternative 2/6	15	0.01	67 %
		25	0.02	75 %
		50	0.13	58 %
R137	Existing Conditions	15	0.04	-
		25	0.09	
		50	0.25	
	Alternative 2	15	0.03	25 %
		25	0.09	0 %
		50	0.23	8 %
	Alternative 6	15	<0.01	>75 %
		25	0.07	22 %
		50	0.21	16 %

The mean overtopping discharges for the existing and alternatives are listed in Table 3-1. The safety guide presented in the Coastal Engineering Manual (USACE, 2006) identifies the mean overtopping discharge as an important parameter to consider for the traffic and structural safety criteria during the storm events (Figure 2-1). The safety criteria are specific at the point of overtopping (i.e. the dune crest and seawall) and do not indicate the safety criteria further landward.

- Safety of Traffic – Once the mean overtopping discharge exceeds 0.00005 $m^3/s/m$ the criteria become "unsafe" for vehicles and "dangerous" for pedestrians. The IH2VOF model quantifies overtopping discharge to an accuracy of 0.01 $m^3/s/m$. Thus, based on the overtopping simulated by the model, the vehicle safety criteria is expected to be “unsafe at any speed” and the pedestrian safety criteria is expected to be “very dangerous.”

- Structural Safety
    - A seawall exists at R137. For the existing conditions, the top 3-4 feet of the seawall is exposed. In this circumstance, the structural safety of the seawall is most closely characterized by the “embankment seawalls” category. During the 15 year return period storm, the overtopping discharge was estimated at 0.04 m^3/s/m, which would result in “damage if back slope is not protected.” During the 25 and 50 return period storms, the overtopping discharge increases and would result in “damage even if fully protected.”
    - The seawall at R137 is buried for the alternatives resulting in a dune similar to the situation at R131. In these circumstances at R131 and R137, the structural safety of the dune is most closely characterized by the “grass sea-dikes” category. According to USACE the “start of damage” is expected once the overtopping discharge exceeds 0.001 m^3/s/m. “Damage” is expected once an overtopping discharge of 0.01 m^3/s/m is exceeded. Based on the model results, the overtopping discharge exceeded 0.01 m^3/s/m indicating “damage” for all the return period storms under the existing conditions and the alternatives. The exception being during the 15 year storm event for the alternatives. At R131 for Alternative 2/6, the discharge was at the threshold between the “start of damage” and “damage.” At R137 for Alternative 6, the discharge was less than 0.01 m^3/s/m within the range of “start of damage.”

4.0 WAVE FORCES ON SEAWALL

Under existing conditions, the upland seawalls are exposed to wave attack. At R137, the top 2-3 feet of the seawall is exposed with the existing conditions. The IH2VOF model was used to estimate the dynamic load at the seawall as shown in Figure 4-1. For existing conditions, all the return period storms simulated impacted the seawall. The maximum horizontal force for a return period waves of 15, 25, and 50 years were 41.8, 53.3 and 69.1 kN/m, respectively. The maximum horizontal momentum calculated by IH2VOF for existing condition with return period waves of 15, 25 and 50 years was 25.6, 37.2 and 52.2 kN/m, respectively. Since the condition, age and structural integrity of the seawalls are unknown, it is not clear how these wave forces could impact their ability to protect the upland areas. Adding sand fill in front of the seawalls may provide additional protection by buffering the wave attack.

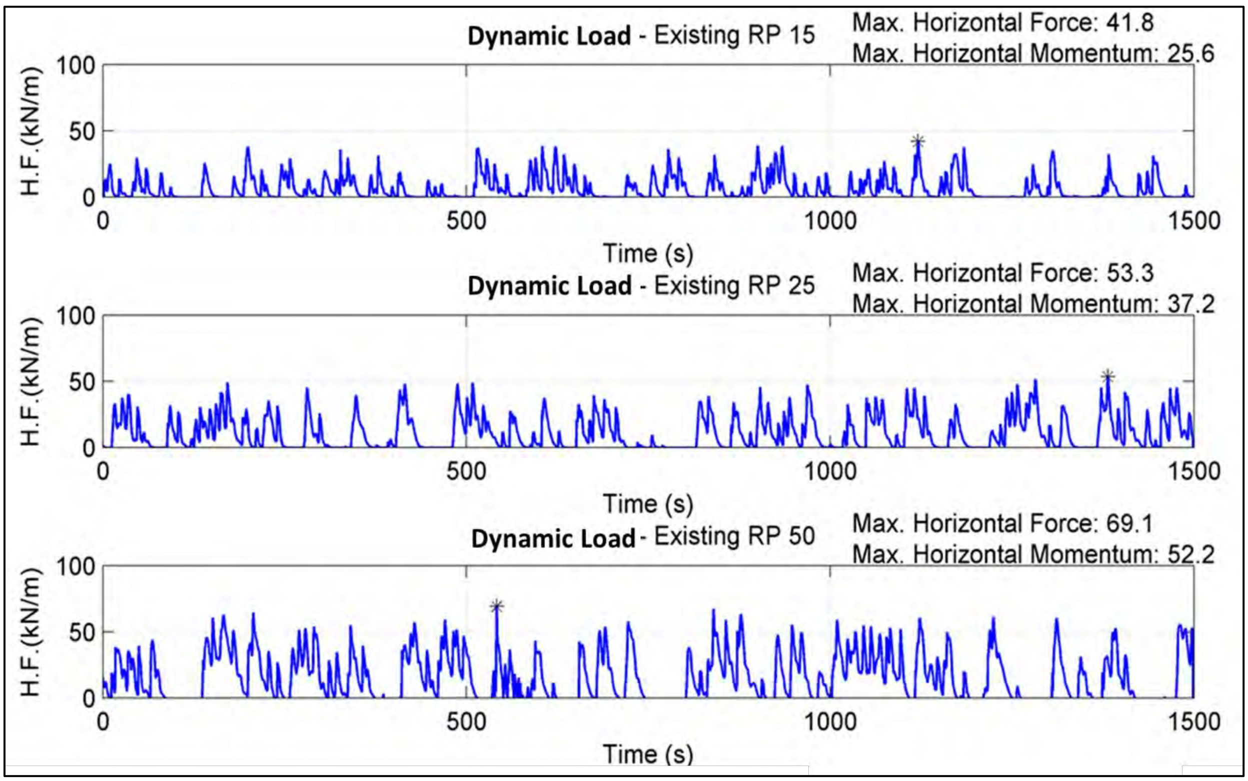

Figure 4-1. Dynamic load at seawall: top to bottom: Existing condition RP15, Existing condition RP25, Existing condition RP50.