APPENDIX B
ENGINEERING DESIGN AND COST ESTIMATES
BROWARD COUNTY, FLORIDA
SHORE PROTECTION PROJECT
GENERAL REEVALUATION REPORT

SEGMENT III

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BACKGROUND

B-1. The Broward County, Florida Shore Protection Project was authorized by Section 301 of Public Law 89-298, passed on 27 October 1965. The project was authorized in accordance with the report of the Chief of Engineers dated 15 June 1964 and is described in House Document 91, 89th Congress. The project was to be constructed in three separable segments. These three segments are: I) the north county line to Hillsboro Inlet, II) Hillsboro Inlet to Port Everglades, and III) Port Everglades Inlet to the south county line. This appendix is concerned with Segment III of the authorized project. Since the Broward County Shore Protection Project was authorized, two reaches of Segment III have been constructed. These are (1) the northern section of the John U. Lloyd Beach State Park shoreline and (R-86 to R-94) and (2) the Hollywood/Hallandale shoreline (R-101 to R-128). The location and extent of these reaches is summarized in Figure B-1.

B-2. The authorization for the Segment III shoreline provided for the restoration of 8.1 miles of shoreline and periodic nourishment for a period of 10 years following initial construction of the project. Following a 1991 Reevaluation Report Section 934 Study, Federal participation in the authorized project was extended to 50 years after initial construction. Additionally, Section 506 of the Water Resources Development Act of 1996 (P.L. 104-303) extended the authorization to 50 years from initial construction.

B-3. Initial construction of the John U. Lloyd portion of Segment III occurred in late 1976 and early 1977. That project extended along about 1.52 miles of shoreline between FDEP monuments R-86 and R-94. This project's first renourishment occurred in 1989.

B-4. The Hollywood and Hallandale project reach was originally constructed in 1979. This project included about 5.25 miles of shoreline between R-101 and R-128. The 1978 G&DDM concerning Segment III (BCEPD, 1978) altered project features for the Hollywood and Hallandale beaches from those prescribed in HD91/89 to reflect changed site conditions and Federal criteria. An evaluation of the 1979 project's performance and recommendations for the project dimension modifications were included in the 1990 General Design Memorandum Addendum for the Hollywood and Hallandale shorelines.

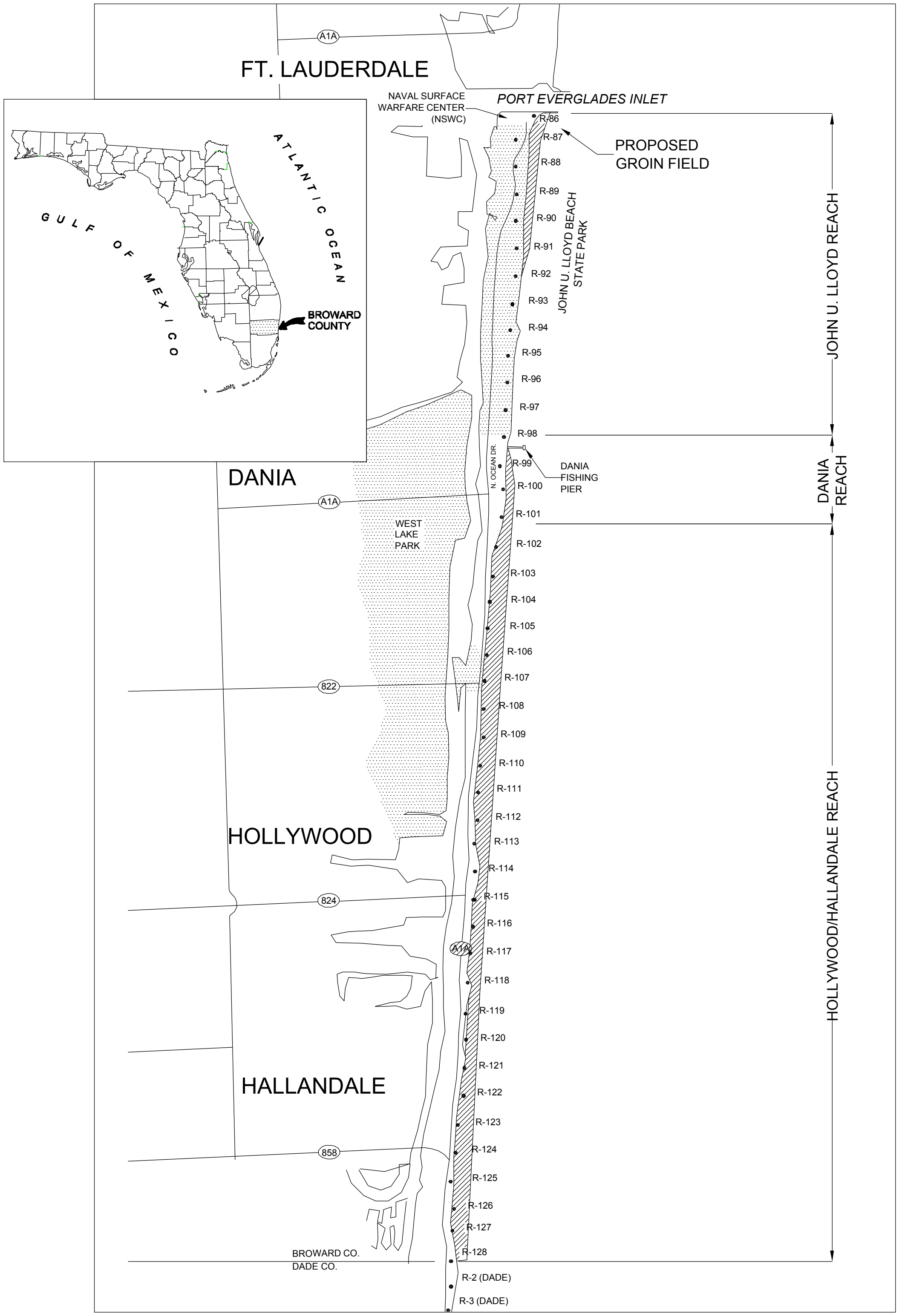

Figure B-1: Location and extent of Segment III reaches.

B-5. The objective of this appendix is to quantify the historical shoreline erosion problem along the Broward County Segment III shoreline, to evaluate the performance of previously constructed portions of the authorized project, and to investigate alternatives to reduce the total cost of the shore protection project. The analyses include an evaluation of historical shoreline and beach volume changes, an estimate of the impact of Port Everglades to the Segment III shoreline, and evaluation of the typical longshore sand transport rates and the magnitude of cross-shore sand transport and beach recession due to storm events.

NATURAL FORCES

B-6. Many factors influence the coastal processes along the Broward County shoreline. These include winds, tides, currents, waves, storm effects, coastal structures, and the nearshore reef system. The role of each of these factors and their contribution to beach erosion in Broward County is described in the following paragraphs.

Winds

B-7. Winds, and the waves they generate, are the primary mechanisms of sand transport along the Segment III shoreline at the project site. Typical prevailing winds are from the northeast through the southeast with easterly winds occurring most often. During winter months (December through March), winds are often out of the northeast and north. Winter storms include nor'easters that can cause extensive beach erosion and shorefront damage. The summer months (June to September) are characterized by tropical weather systems traveling east to west in the lower latitudes. These tropical cyclones can develop into tropical storms and hurricanes, which can generate devastating winds, waves and storm surge. Southeast trade winds make up the typical summer winds.

Tides and Currents

B-8. Astronomical tides along the Broward County coast are semi-diurnal. The mean and spring tide ranges at Port Everglades are 2.5 feet and 3.1 feet, respectively. On a regional scale, tidal ranges decrease from a mean range of 2.4 feet at the north county line to a range of 2.1 feet at the south county line (NOAA, 1997). All elevations presented in this appendix are referenced to National Geodetic Vertical Datum of 1929 (NGVD), unless stated otherwise. For survey purposes in Broward County coastal areas, the U.S. Army Corps of Engineers, Jacksonville District (CESAJ) has established an invariant construction datum, equivalent to mean low water (MLW) which is 0.78 feet below NGVD and 2.58 feet below mean high water (MHW). Tidal measurements at NOAA's gage 872-2951 indicate that the highest and lowest observed water levels were +3.65 feet NGVD, on 25 Oct 1973, and –2.04 feet, NGVD, on 26 Apr 1971, respectively[1].

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[1]Statistics obtained at the following website: http://www.opsd.nos.noaa.gov/bench/

B-9. Currents affecting the beaches of Broward County include littoral currents and inlet-related tidal currents. Littoral currents may be classified as longshore or cross-shore currents. Longshore currents are caused by waves breaking at an angle relative to the shoreline, and they generally determine the long-term direction and magnitude of littoral sand transport. The most influential cross-shore currents are typically generated during storm events that may be characterized by short-term extreme wave and/or water level conditions. Storm-induced cross-shore currents often result in the offshore transport of beach material, in some cases to locations seaward of the local closure depth. In other cases, the transported beach material remains in the zone of active transport, and may be redistributed back onto the beach during periods of onshore transport. More detailed discussions of longshore and cross-shore sediment transport will be presented in subsequent sections of this appendix.

Waves

B-10. The principal forcing mechanism that causes beach erosion is the dissipation of wave energy (and corresponding transport of sand particles) as waves enter the nearshore zone and break. Wave height and period, along with magnitude and phasing of the tide level and in some cases, storm surge, are the most important factors influencing the project shoreline. Since the 1980's, the U.S. Army Engineer, Waterways Experiment Station's Coastal Engineering Research Center (CERC) has executed a series of wave hindcast studies for sites in the Gulf of Mexico, Atlantic and Pacific Oceans. The revised Atlantic wave data time series resulted from the Wave Information Study (WIS) Phase II hindcast for the 20-year period 1956-1975, as documented in WIS Report 30 (Hubertz et al., 1993). This study excluded any waves generated by tropical cyclones and swell propagating from the South Atlantic; extratropical storms, however, are included in the data set. CERC has also made available an updated Atlantic hindcast covering the 20-year period 1976-1995 (Brooks and Brandon, 1995). The updated hindcast included wave information for both extratropical storms and tropical cyclones.

B-11. The wave statistics used for this analysis were obtained from WIS Station A2009 that is located at latitude 26.00 degrees north and 80.00 degrees west (Figure B-2). Water depth at this station is 220 meters (722 feet).

B-12. Tables B-1 to B-4 summarize the 1976-1995 hindcast wave results for Station 9. Table B-1 contains estimated wave heights for various return periods. Table B-2 is a summary of the mean and largest significant wave by month and year for the 20-year period. This table is useful in showing the range distribution of wave height throughout the year. The percent occurrence of wave height and period for all directions is shown in Table B-3.

B-13. The hindcasts provide time histories of wave height, period and direction, listed at three-hour intervals over the 20-year study periods. The significant wave height (H_{mo}) represents a combination of sea and swell. The wave period (T_p) and direction reflect characteristics of the dominant wave. Wave direction (D_p) is measured clockwise in degrees from true north.

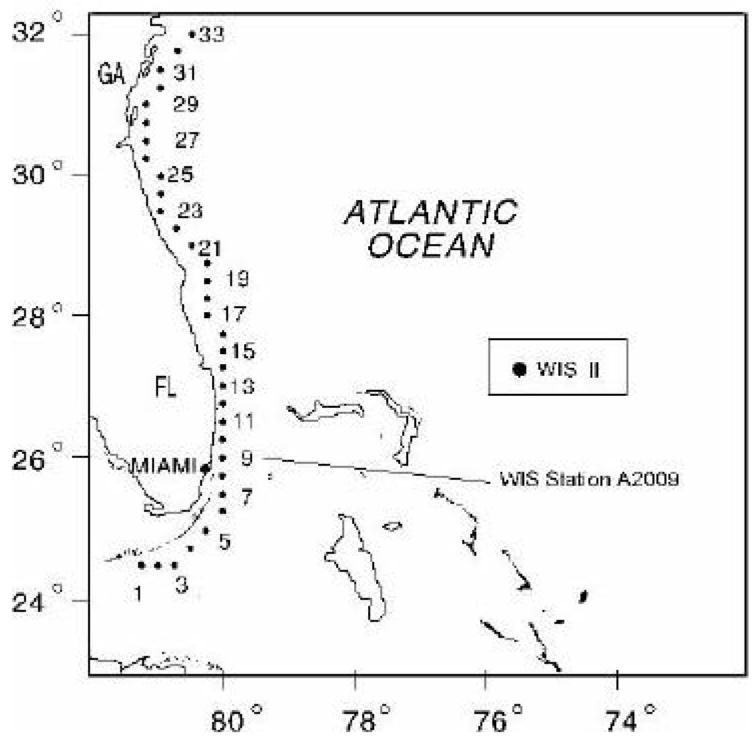

Figure B-2: WIS Station A2009 Location Map

Table B-1: Wave heights and return periods - WIS Station 9 (1976-1995).

Return Period (Years)	Significant Wave Height (meters)	Significant Wave Height (feet)
2	5.3	17.3
5	6.0	19.7
10	6.4	21.0
20	6.9	22.6
25	7.0	23.0
50	7.5	24.6

Table B-2: Mean and maximum wave heights (1976-1995).

MEAN WAVE HEIGHT (IN METERS) BY MONTH AND YEAR

STATION: A2009 (26.00N/ 80.00W / 220.0M)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1976	1.3	1.1	1	1	0.9	0.7	0.4	0.7	0.4	1.3	1.3	1.5	1
1977	1.1	1	1.1	1.5	1	0.4	0.5	0.9	0.6	0.9	1.4	1.2	1
1978	1.2	1.2	1	0.9	0.7	0.6	0.6	0.6	0.8	1.4	1.3	1.4	1
1979	1.7	1.3	1.3	1.3	0.9	0.8	0.6	0.5	1.3	0.9	1.6	1.3	1.1
1980	1.1	1.4	1.2	0.9	0.7	0.5	0.4	0.8	0.6	0.8	1.3	1.3	0.9
1981	1	1.7	1.4	1.1	0.7	0.7	0.5	0.8	0.8	1.1	1.2	1	1
1982	1	0.9	1	0.8	0.9	0.7	0.5	0.5	0.6	1.1	1.2	1.3	0.9
1983	0.9	1.4	1.2	1.1	0.9	0.6	0.5	0.5	0.9	1.1	1	1.4	1
1984	1.6	1.2	1.2	0.9	1.1	0.7	0.6	0.5	1.2	1.5	1.7	1.3	1.1
1985	1	1.4	1.1	1.1	0.5	0.5	0.5	0.7	1.3	1	1.3	1.3	1
1986	1.2	1	1.5	0.9	1.1	0.6	0.4	0.8	0.9	1.1	1.2	1.3	1
1987	1.2	1.1	1.7	0.9	0.9	0.7	0.6	0.5	0.5	1.3	1.4	1	1
1988	1.4	1.1	1	0.9	0.8	0.8	0.6	0.5	1	1	0.9	0.9	0.9
1989	0.9	1	1	0.7	0.6	0.6	0.4	0.4	0.8	0.9	0.7	0.8	0.7
1990	0.9	1.3	1.3	1	0.9	0.6	0.6	0.4	0.7	1	1.1	1.1	0.9
1991	0.9	1	1	1	0.9	0.6	0.4	0.5	0.6	1.1	1.1	1	0.8
1992	1	0.9	0.9	1	0.8	0.6	0.6	0.6	0.7	1	1.4	1.1	0.9
1993	1.3	1.1	1.2	1.1	1	0.7	0.4	0.5	0.7	0.8	1.2	1.1	0.9
1994	1.4	1.2	1	1.1	0.8	0.6	0.7	0.7	0.8	0.9	1.3	1.2	1
1995	1	0.9	1.3	0.9	0.7	0.7	0.6	0.9	0.8	1.3	1.1	1.1	1
MEAN	1.2	1.2	1.2	1	0.8	0.6	0.5	0.6	0.8	1.1	1.2	1.2	

Table B-2: Mean and maximum wave heights (1976-1995). (cont'd)

LARGEST WAVE HEIGHT (IN METERS) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1976	3.9	3.7	3.8	2.9	3.2	1.2	0.8	1.9	0.8	3.5	3.3	4
1977	2.4	2.4	3.3	4.1	3	1.2	1.1	2.5	2.1	2.7	4.3	3.1
1978	3.1	3.7	3.1	2.7	2.4	3.2	1.9	1.4	1.7	3.9	4	5.2
1979	5.7	2.8	4.3	4.2	2.7	2.9	2.9	1.4	6.6	2.7	4.1	3.8
1980	2.6	4.9	3	3	2.3	1.9	1.4	4.8	1.4	2	3.3	3
1981	3.6	5.1	2.9	3.5	1.8	1.7	1.5	4.4	1.7	3.2	3.4	2.9
1982	3.2	3	4.4	2.2	2.1	2.8	1.7	1.1	1.1	2.4	3.4	3.1
1983	2.4	4.2	5	3.2	2.2	1.2	1.9	2.4	2.9	3.9	3.4	5.2
1984	6.7	3.5	3.8	2	2.8	1.8	1.4	2.3	5.1	3	5.2	3.7
1985	3	3.9	4.1	4	1.2	2	3.6	2.3	3.6	2.4	6.6	3.5
1986	4.7	3.2	4.1	2	2.5	1.6	1.4	2	1.9	3.6	3	4.1
1987	4.5	2.8	5.6	2.3	3.7	2.7	1.6	1.5	0.8	3.7	3.7	3.4
1988	4.3	2.7	2.4	2.5	2.4	2.9	1.4	2	3.5	2.5	2.1	2.1
1989	2.1	2.2	3.7	1.4	1.7	1.1	1	1.8	2.1	2.3	1.6	1.9
1990	2.3	3.2	3.5	2.7	1.9	1.3	1.5	0.9	1.1	2.3	3.5	3.6
1991	2.8	2	3.4	2.7	3.6	1.6	1	1.2	1.6	2.8	2.2	4.1
1992	2.5	2	2.6	2.7	1.6	1.2	1.2	6.9	1.4	3.2	3	2.1
1993	3.5	2.5	5	2.5	2	1.9	1	1.4	2.1	2.8	2.3	3.5
1994	3.2	3.9	3.7	2.1	2.6	1.2	1.5	1.5	1.6	2.8	5.9	3.6
1995	3.2	1.7	2.8	1.9	1.3	2	2.6	2.5	1.3	2.7	1.9	2.5

20-YEAR STATISTICS

MEAN SPECTRAL WAVE HEIGHT	(METERS) 0.9
MEAN PEAK WAVE PERIOD	(SECONDS) 7.3
MOST FREQUENT 22.5 DEGREE(CENTER)DIRECTION BAND	(DEGREES) 45
STANDARD DEVIATION OF WAVE Hmo	(METERS) 0.6
STANDARD DEVIATION OF WAVE TP	(SECONDS) 3.5
LARGEST WAVE Hmo.	(METERS) 6.9
WAVE TP ASSOCIATED WITH LARGEST WAVE Hmo.	(SECONDS) 10
PEAK DIRECTION ASSOCIATED WITH LARGEST WAVE HS	(DEGREES) 54
DATE LARGEST Hmo OCCURRED	(YRMODYHR) 92082409

Table B-3: Occurrence of wave height and period for all directions (1976-1995).

PERCENT OCCURRENCE (X1000) OF HEIGHT AND PERIOD
FOR ALL DIRECTIONS

STATION: A2009 (26.0N, 80.0W / 220.0M) NO. CASES: 58440
% OF TOTAL: 100.0

HEIGHT IN METERS	PEAK PERIOD (IN SECONDS) <4.0	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-11.9	12.0-LONGER	TOTAL
.00- .99	7245	13112	7402	5292	4618	4269	3480	3013	2936	10752	62119
1.00-1.99	.	1658	8453	9662	2703	1468	1517	924	636	3692	30713
2.00-2.99	.	.	23	467	2802	1620	174	176	123	361	5746
3.00-3.99	.	.	.	.	68	407	542	44	6	41	1108
4.00-4.99	.	.	.	.	.	18	106	95	10	.	229
5.00-5.99	.	.	.	.	.	.	10	15	22	6	53
6.00-6.99	.	.	.	.	.	.	.	5	3	5	13
7.00-7.99	.	.	.	.	.	.	.	.	.	.	0
8.00-8.99	.	.	.	.	.	.	.	.	.	.	0
9.00-9.99	.	.	.	.	.	.	.	.	.	.	0
10.00+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	7245	14770	15878	15421	10191	7782	5829	4272	3736	14857	

MEAN Hmo(M) = .9 LARGEST Hmo(M) = 6.9 MEAN TP(SEC) = 7.3

Storm Surge

B-14. Storm surge is generally defined as an increase in water level that results from forcing by atmospheric weather systems. Surges occur primarily as a result of atmospheric pressure gradients and surface stresses created by wind blowing over a water surface. When the water's momentum carries it beyond the position of static equilibrium, a long-wave phenomenon results in which the water surface increases downwind and decreases upwind. In addition to wind speed, direction and duration, the surge is also influenced by water depth, length of fetch, and frictional characteristics of the nearshore sea bottom. An estimate of these water level changes is required for storm modeling and the design of beach fill crest elevations.

B-15. The Federal Emergency Management Agency (FEMA) has performed investigations to determine hurricane surge elevations in the Flood Insurance Studies (FIS) for Broward County. Wave heights were computed along transects located along the shoreline, considering the combined effects of changes in ground elevation, vegetation and physical features.

B-16. Higher frequency storms and storm surge elevations for other meteorologically induced water level anomalies (i.e., nor'easter type storms) were obtained from WIS Report 7 (Ebersole, 1982). Hindcasting of the nor'easter storm surges was performed utilizing historical wind and pressure fields.

B-17. Figure B-3 provides storm frequency versus return period curves for Broward County. The FEMA hurricane surge curve is based on data points for the 10, 50, 100, and 500-year recurrence interval points. The WIS northeaster surge curve is based on data points for the 2, 5, 10, 20 and 50-year recurrence interval at Miami Beach, Florida. The WIS northeaster surge data does not include tide, therefore, since the normal duration of a northeaster is several days (i.e. several tide cycles), a curve which provides the WIS northeaster surge height with a spring tide, a worst case scenario, is included on Figure B-3.

B-18. The cross shore sediment transport analysis, discussed more thoroughly in paragraphs B-52 through B-79, involved the modeling of beach profile changes in response to specific historical storms; therefore, storm surge hydrographs characteristic of those specific storms were required as input. Those surge hydrographs were obtained from a database of storm information (Scheffer et al., 1994) that was generated by CERC as a product of the Dredging Research Program (DRP). Tasks undertaken to generate this database included: 1) selection of historic storm events (of both tropical and extratropical origin), 2) estimation of descriptive storm parameters to be used as input to a planetary boundary layer wind field model, 3) execution of that model to generate temporal and spatial storm-induced wind and pressure fields, and 4) use of that wind and pressure data as input to the large scale hydrodynamic model, ADCIRC, which computes spatial and temporal distributions of storm surge elevations and currents. The resulting DRP database includes storm surge and current data for 486 discrete locations, located throughout the Atlantic Ocean, Gulf of Mexico, and Caribbean Sea. DRP Station 442, at latitude 25.994 degrees north and 80.084 degrees west, was selected for this Broward County application. This selection was based on the proximity of DRP Station 442 to the source of corresponding wave data, WIS Station A2009. More detailed information on the character and use of this storm surge data is provided in the discussion of cross-shore (storm-induced) sediment transport analyses.

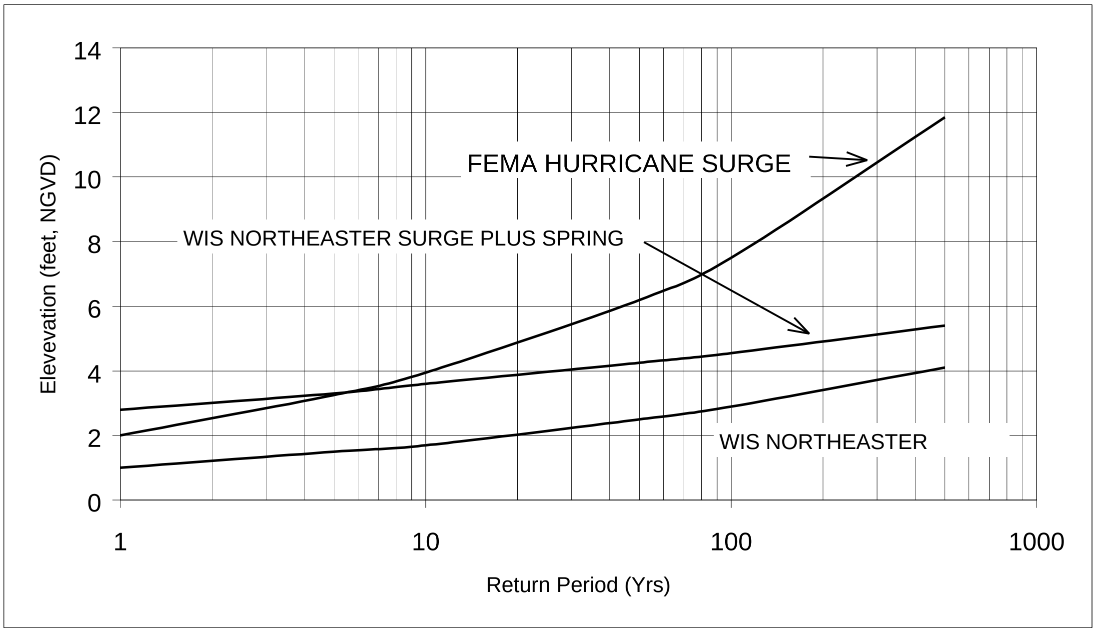

Figure B-3: FEMA and WIS storm surge frequency relationships for Broward County.

Yearly Depth Limit

B-19. For natural sand beaches, a useful coastal processes parameter is the yearly depth limit of the active nearshore beach profile. This is also referred to as the depth of closure (DOC). Beyond this depth one negligible sand movement is expected under average annual conditions. Hallermeier (1978) developed a procedure for estimating the depth of closure, d_c. This depth is based upon the approximate extreme wave condition for nearshore significant waves, and may be calculated by:

$$d_c = 2.28\ H_e - 68.5\ (H_e^2/gT_e^2)$$

where:

H_e = nearshore extreme significant wave height (in meters)
T_e = nearshore extreme significant wave period (in meters)
g = acceleration of gravity constant, 9.81 m/sec^2.

The extreme nearshore significant wave height, H_e, is defined as the "effective" wave height, which has a 0.137% probability of occurring. This wave height is related to the deepwater mean wave height as follows (Dean and Dalrymple, 1996):

$$H_e = H_{mean} + 5.6\mathbf{s}$$

where $\mathbf{s}$ is the standard deviation of the annual wave height (in meters).

B-20. The mean wave height, from the WIS hindcast data (Table B-2), is 0.9 meter and the standard deviation is 0.6 meter. The nearshore extreme significant wave period used is the wave period associates with the largest wave, which is 10.0 sec (Table B-2). Using the above values and equations, the predicted depth of closure is 27.7 feet.

B-21. The theoretical depth of closure was also calculated using the Birkemeier equation (Birkemeier, 1985). This approach typically provides a more reasonable estimate, compared to Hallermeier's approach, which usually over-predicts the depth of closure. The Birkemeier equation is as follows:

$$d_c = 1.75\ H_e - 57.9\ (H_e^2/gT_e^2)$$

This approach yields a depth of closure of 20.9 feet, which is a more reasonable estimated than Hallermeier's, but still deeper than the inner reef.

B-22. Both of the aforementioned methods do not consider the energy dissipation associated with the reef systems offshore of Broward County. These reefs reduce the wave energy that eventually reaches the beach along the County's shoreline. Therefore, it is expected that the limit of active sand transport would be much shallower than predicted with these methods.

B-23. Review of historical beach profiles collected along the Segment III shoreline indicates that the actual depth of closure along the shoreline varies between 5.5 and 16 feet. The variations in the elevations are related to the highly variable offshore reef conditions that regulate the amount of wave energy that reaches any particular area of shoreline. It is also due to the highly irregular nature of the nearshore reef system and the associated perching effects. Irregularities in the latter would produce localized shallow and deep areas at the toe of the beach.

B-24. The depth of closure, as indicated for the historical beach profile data, was estimated for the beach at each R-monument location along Segment III. The Segment was divided into two sub-reaches that include (1) John U. Lloyd Beach State Park (R-86 to R-95) and (2) the cities of Hollywood and Hallandale (R-99 to R-128). The estimated DOC's for each profile location are summarized in Table B-4.

B-25. For the John U. Lloyd reach, surveys associated with the pre- and post-construction of the 1989 beach restoration project are compared with surveys taken on the following dates: November 1990, August 1991, October 1993, August 1998, and August 1999. DOC for the Hollywood/Hallandale reach was estimated using pre- and post-construction surveys of that area's 1991 beach fill along with previously mentioned October 1993, August 1998 and August 1999 surveys. Figure B-4 details profile lines and the DOC estimate at monument R-89 in John U. Lloyd Beach State Park. Here, the depth of closure is estimated at 6.0 feet NGVD. Figure B-5 depicts a DOC of 14.0 feet NGVD at monument R-114 in Hollywood/Hallandale.

Table B-4: Estimated depth of closure in Segment III.

John U. Lloyd		Hollywood/Hallandale	
Monument	DOC (-ft-NGVD)	Monument	DOC (-ft-NGVD)
86	5.5	99	12.0
87	9.0	100	14.0
88	6.5	101	16.0
89	6.0	102	15.0
90	7.0	103	15.0
91	7.0	104	15.0
92	7.5	105	10.0
93	8.5	106	10.0
94	13.0	107	10.0
95	13.0	108	10.0
		109	14.0
Average	8.3	110	10.0
		111	8.0
		112	9.0
		113	12.0
		114	13.0
		115	12.0
		116	14.0
		117	12.0
		118	14.0
		119	13.0
		120	12.0
		121	13.0
		122	12.0
		123	14.0
		124	14.0
		125	14.0
		126	14.0
		127	13.0
		128	13.0
		Average	12.6
Overall Average		11.5	

B-26. The overall average DOC for John U. Lloyd and Hollywood/Hallandale combined is 11.5 feet NGVD. The average DOC along the John U. Lloyd Beach State Park is 8.3 feet NGVD while the DOC along Hollywood/Hallandale averages 12.6 feet NGVD. The depth of closure along the John U. Lloyd reach is much shallower than that for Hollywood the perching effects of a rock shelf along the northern areas of Segment III.

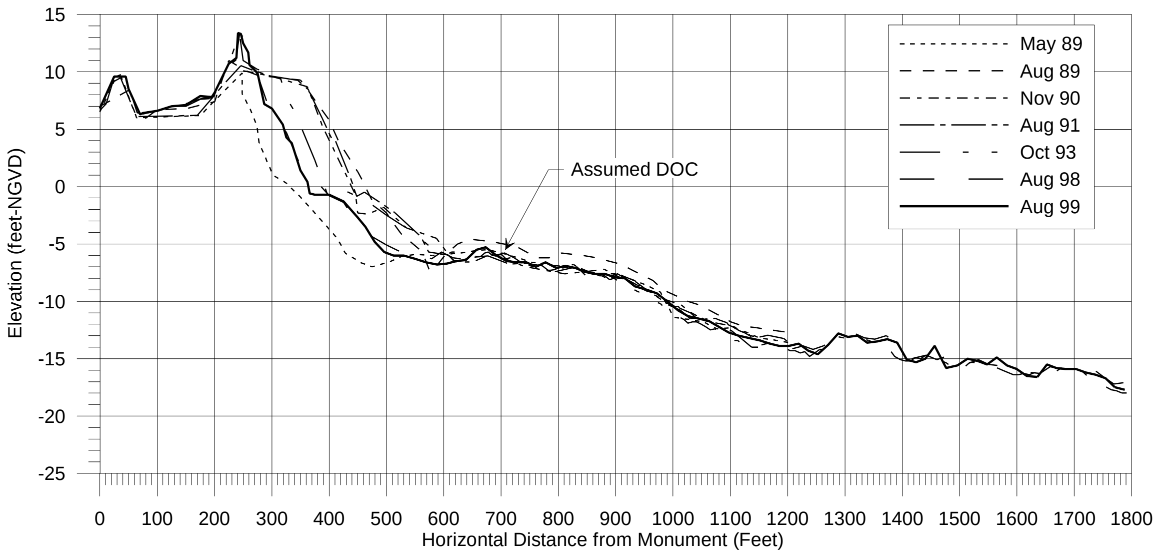

Figure B-4: Depth of closure assumption at R-89 in John U. Lloyd Beach State Park.

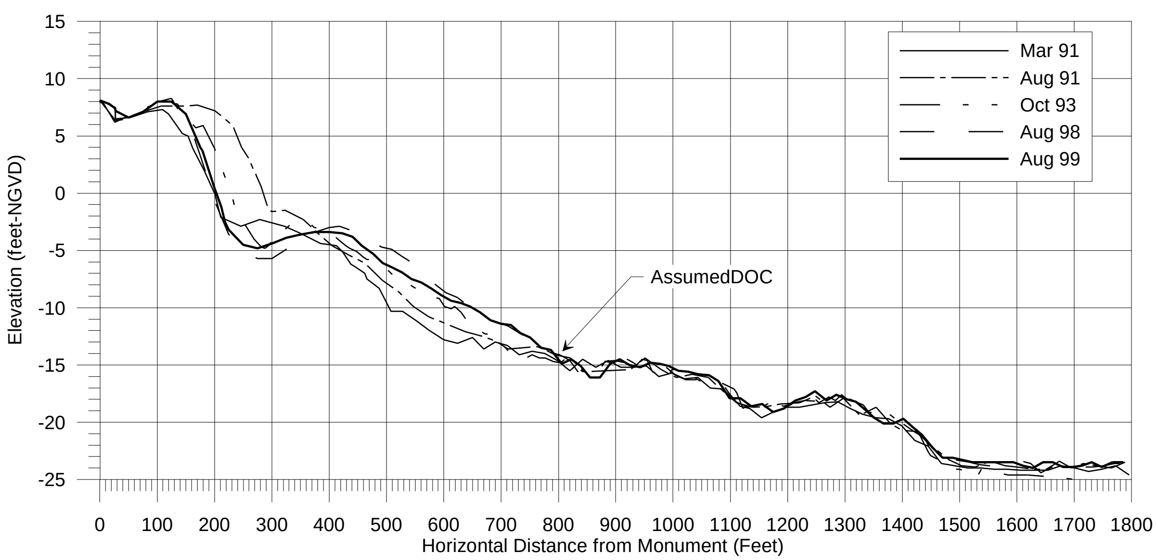

Figure B-5: Depth of closure assumption at R-114 in Hollywood/Hallandale.

Sea Level Rise

B-27. The geologic record of historical sea level variations indicates that both increases and decreases in global sea level have occurred. Some authorities claim that evidence indicates our planet may be entering a new ice age, which would result in a lower sea level. Others argue that increasing atmospheric concentrations of carbon dioxide and other gases are causing the Earth to warm, contributing to a sea level rise. Nevertheless, global cooling and warming both contribute to absolute global sea level change, or eustatic sea level change. Total relative sea level change has been estimated to be 2.3mm per year based on data at Miami Beach (Lyles et al., 1988). This trend suggests that during the 30 years of remaining project life (2001-2030), the sea level will rise about 69mm (0.23ft) along Segment III.

B-28. *Shoreline Recession-Sea Level Rise.* As sea level rises, the shoreline will be subjected to flooding, profile recession, and possibly, erosion. Per Bruun (1962) proposed a formula for estimating the rate of shoreline recession based on the local rate of sea level rise. This methodology also includes consideration of local topography and bathymetry. Bruun's approach assumes that with a rise in sea level, the beach profile will attempt to reestablish the same bottom depths relative to the surface of the sea that existed before the sea level rise. If the longshore littoral transport in and out of a given shoreline area is equal, then the quantity of material required to reestablish the nearshore slope must be derived from erosion of the shore. Shoreline recession resulting from sea level rise can be estimated using Bruun's Rule, as defined below:

$x = ab/(h+d)$
where,

x = shoreline recession (in feet) attributable to sea level rise.
h = average elevation of shoreline above mean high water (+8.0 ft, NGVD).
d = MLW depth contour beyond which there is no significant
sediment motion (-11.5 ft, NGVD).
b = horizontal distance (700 feet averaged) from the beach profile
berm elevation to the depth contour d.
a = specified relative sea level rise (ft) for time period t (0.23 ft.).

As mentioned above, the mean estimated sea level rise for the year 2030 along Broward County shores is 0.23 feet. Shoreline recession corresponding to this estimate is 8.3 feet, or 0.28 feet per year.

B-29. The Bruun procedure is applicable to long straight sandy beaches having an uninterrupted supply of sand. Little is known about the rate at which profiles respond to changes in water level; therefore, this procedure should only be used for estimating long-term changes. The procedure is not a substitute for the analysis of historical shoreline and profile change. If little or no historical data is available, then historical analysis may be supplemented by this method to provide an estimate of long-term erosion rates attributable to sea level rise. The offshore contours in the project area are not entirely straight and parallel. Also, the presence of offshore rock formations in Broward County can affect the shoreline in a manner that might be inconsistent with this rule. However, Bruun's Rule can provide an estimate of the potential shoreline changes within the project area attributable to a projected rise in sea level.

B-30. *Shoreline Erosion-Sea Level Rise.* For this discussion, it is assumed that as an unarmored beach erodes, it maintains approximately the same profile above the seaward limit of significant transport; therefore, the volume of eroded material per foot of shoreline equals the vertical distance from the berm crest (+8.0 feet) to the depth of the seaward limit of the active profile (-11.5 feet), multiplied by the horizontal recession of the profile, x. Using the most likely estimate of shoreline recession due to sea level rise (i.e., x = 8.3 feet), the potential erosion volume for the period 2001-2030 would be 0.2 cubic yards per foot of shoreline per year.

HISTORICAL SHORELINE CHANGES

Pre-Project Erosion Rates

B-31. Pre-project Segment III shoreline and beach volume change rates were evaluated as part of a reconnaissance report for Port Everglades to the south county line (USACE, 1963). These rates, which were used to formulate the authorized project, are also reported in House Document 91, 89th Congress. The shoreline change rates were evaluated for the period 1929-1961 along three reaches of the Segment III shoreline. The reaches included the first two miles south of Port Everglades (approximately R-86 to R-97), along with R-98 to R-100, and R-101 to R-128. The reported pre-project shoreline and beach volume change rates for these reaches are summarized in Table B-5. These rates are assumed to represent pre-project conditions for the purposes of this reevaluation report.

Table B-5: Pre-project shoreline and beach volume change rates.

Location	Reach (ft)	Monuments	Volume Change (cy/yr)	Shoreline Change (ft/yr)
JUL	8,000	R86 - R94	-54,606	-5.0
SJUL/Dania	7,300	R95 - R100	-19,091	-2.5
Hollywood/ Hallandale	27,500	R101 - R128	-84,364	-1.0
Total	42,800	R86 – R128	-158,061	-2.0

Post-Project Erosion Rates

B-32. Two reaches of the Segment III shoreline have been constructed following authorization of the project segment. These include the northern 8,000 feet of the John U. Lloyd Beach State Park shoreline and approximately 28,800 feet of shoreline along Hollywood/Hallandale.

B-33. The 8,000-ft (approx.) shoreline south of Port Everglades Entrance -- from about R-86 through R-94 -- has been nourished twice: first in 1977 (1.09 Mcy) and most recently in 1989 (over 0.6 Mcy). The physical performance of the 1977 project was assessed in 1988 as part of the planning for the project's first renourishment in 1989 (BCEPD, 1987).

B-34. Survey data collected following completion of the 1978 JUL project suggest severe shoreline recession along the first 3,000 to 3,500 ft south of the inlet, decreasing at 5,000 to 6,000-ft south thereof. It was estimated that the shoreline change rate along the northern reach of JUL was approximately 31,000 cubic yards per year following the 1978 project (USACE, 1990). This estimated rate was developed through comparison of a 1978 and 1985 beach profile surveys.

B-35. The 27,500-ft shoreline from the northern end of Hollywood to the south County line -- from about R-101 through R-128 -- has also been nourished twice: first in 1979 (1.98 Mcy) and most recently in 1991 (over 1.11 Mcy).

B-36. The performance of the 1979 Hollywood/Hallandale shoreline was also evaluated for purposes of formulation of the first renourishment (USACE, 1990). In general, 1979 project suffered from planform equilibration due to irregular sand volume placement. This resulted in areas of high erosion and accretion shortly after the project's completion. The nominal shoreline recession during the six-year period after the 1979 fill was about 75 feet (or, about 12.5 ft/yr, on average). It is estimated that the average-annual sand loss rate for the project was about 54,000 cubic yards per year. This estimated rate was developed through comparison of a 1979 and 1988 beach surveys.

B-37. The results of the physical performance assessment of both the 1977 John U. Lloyd and 1979 Hollywood Hallandale beach fill projects suggest that the average annual sand volume loss rate was lower than estimated in the pre-authorizing documents. It is noted, however, that the performance of the 1977 John U. Lloyd and 1979 Hollywood/Hallandale beach fill projects was evaluated with only limited survey data.

B-38. Beach profile surveys associated with the construction and monitoring of the 1989 John U. Lloyd and 1991 Hollywood first renourishment projects were collected more frequently. Comprehensive surveys of the Segment III shoreline were collected in October 1993 and August 1998. Along the northern reach of the John U. Lloyd shoreline, additional beach profile surveys were collected in August 1978, May 1989, August 1989, and November 1990. Also, along the Hollywood and Hallandale shorelines, additional beach surveys were performed in March 1991, August 1991, February 1992, and August 1992.

B-39. Due to large amount of beach profile survey data available for the 1989 John U. Lloyd and 1991 Hollywood/Hallandale projects, the measured performance of these projects is considered to represent proto-type conditions for beach fills along the proposed project shoreline. Both short-term process such as equilibration and long-term processes such as annual alongshore change can be evaluated with beach profile data for these projects. Therefore, the expected short- and long-term performance of the future projects is expected to be similar to the 1989 and 1991 projects.

B-40. John U. Lloyd North Shoreline (R-86 to R-94). The most recent beach nourishment along the northern half of John U. Lloyd Beach State Park, downdrift of Port Everglades Entrance, included approximately 0.69 Mcy placed in 1989. The shoreline position over the approximately ten years following construction is depicted relative to the pre-project shoreline in Figure B-6. Inspection of the figure indicates that rapid and localized retreat characterized the northern 1,500 to 2,500 ft of the project (i.e., immediately downdrift of the inlet's south jetty). Further south, between about 2,500 and 5,500 ft from the jetty, the fill appears to have receded in a more uniform -- though rapid -- manner. The southernmost 1,500-ft of the fill (i.e., from about 5,500 to 7,000 ft south of the inlet appears to have exhibited some additional end-effect retreat.

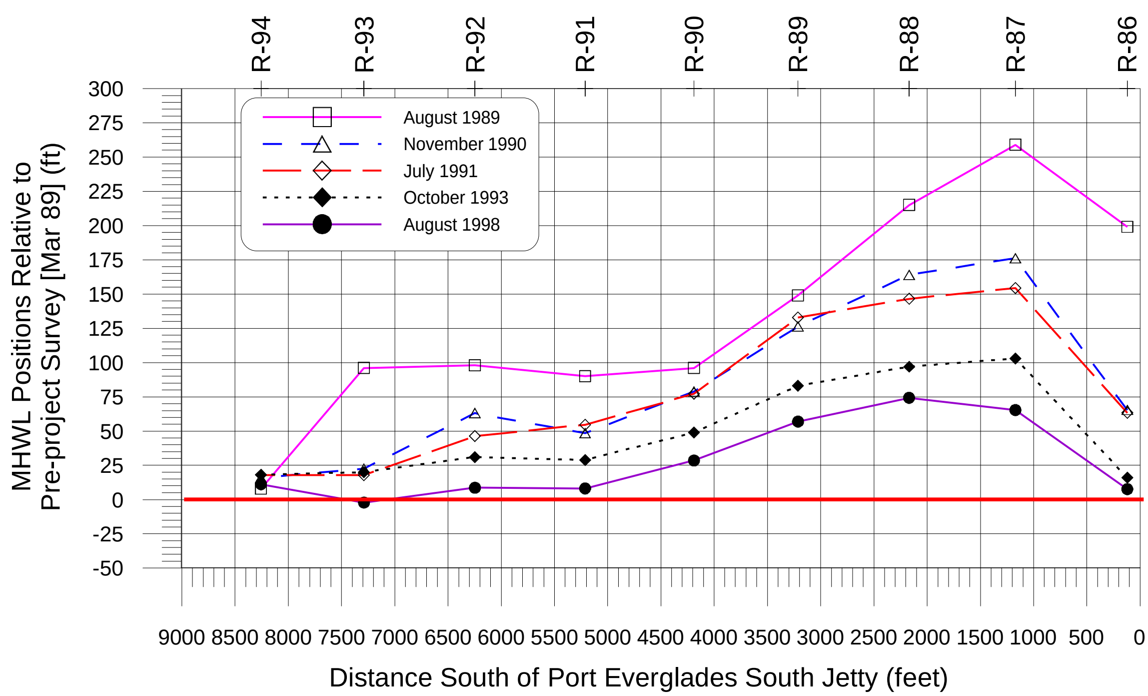

Figure B-6: MHW Shoreline Position pursuant to 1989 Beach Nourishment Project.

B-41. More specifically, the mean high water shoreline along the 1989 John U. Lloyd project retreated at a nominal, average rate of about 16 ft/yr over period from August 1989 to August 1998. The average retreat rate nearest the south jetty exceeded 35 ft/yr while reaches further south, along the center portions of the fill receded at about 9 to 11 ft/yr, on average. The highest rates of recession occurred between the inlet and R-89 during the project's first two years. These rates, which include equilibrium effects, were as high as 35 to 55 ft/yr. The average shoreline change rates as computed with available beach profile survey data are summarized in Figure B-7.

B-42. It is noted that the shoreline recession rate continually decreased over the life of the project. This is most likely due to the continual loss of sandy littoral material from the beach fill project. As the beach fill eroded, the amount of sand material available for transport decreased thus the apparent shoreline change rate as measured with beach profile survey data also decreased. Planform equilibration of the beach fill may also be a contributor to the observed reduction in sand loss rates as the beach fill matured.

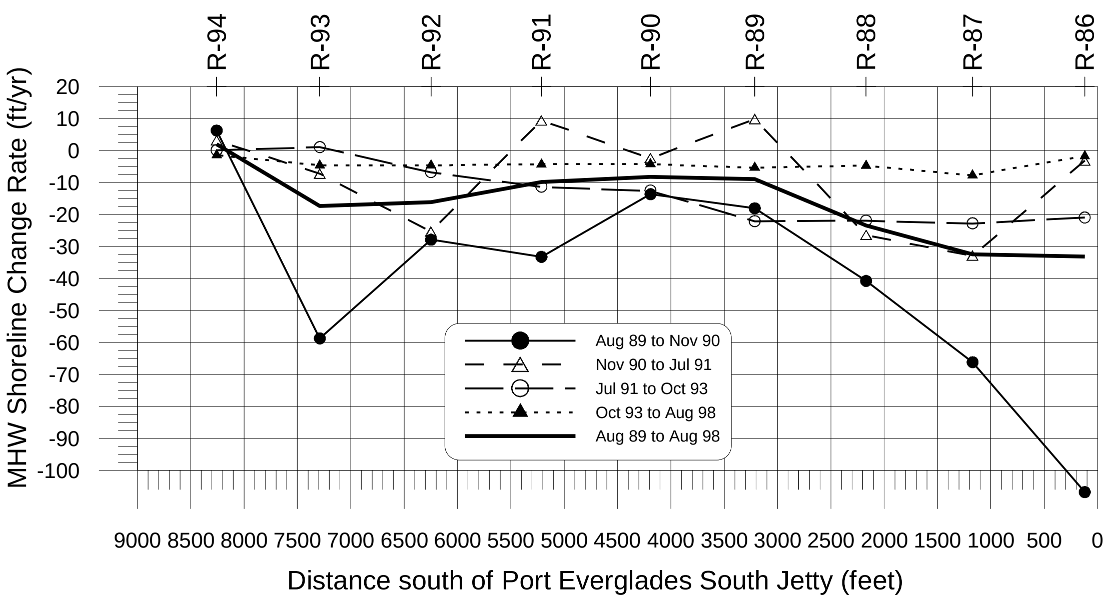

Figure B-7: Shoreline change rates following 1989 beach nourishment.

B-43. Volume changes in John U. Lloyd from May 1989 are illustrated in Figure B-8 with the 1989 record representing the August fill of approximately 0.69 Mcy. Volume changes between depth contours have been considered in an attempt to recognize an equilibrium response of the beach.

B-44. Figure B-8 suggests that during the first year of the project large amounts of sediment were removed from the local system at all depths out to –16 feet (NGVD). First year losses between R-86 and R-93 were approximately 0.2 Mcy. Prior to October 1993, volume losses can be seen across the entire profile indicating little sediment transport offshore. Post 1993 calculations suggest accretion below the –6 foot contour with volume reduction continuing, now at a slower rate, above the same contour. As of August 1998, only about one-third of the original fill volume remains in place from the August 1989 John U. Lloyd beach nourishment, approximately 230,000 cy.

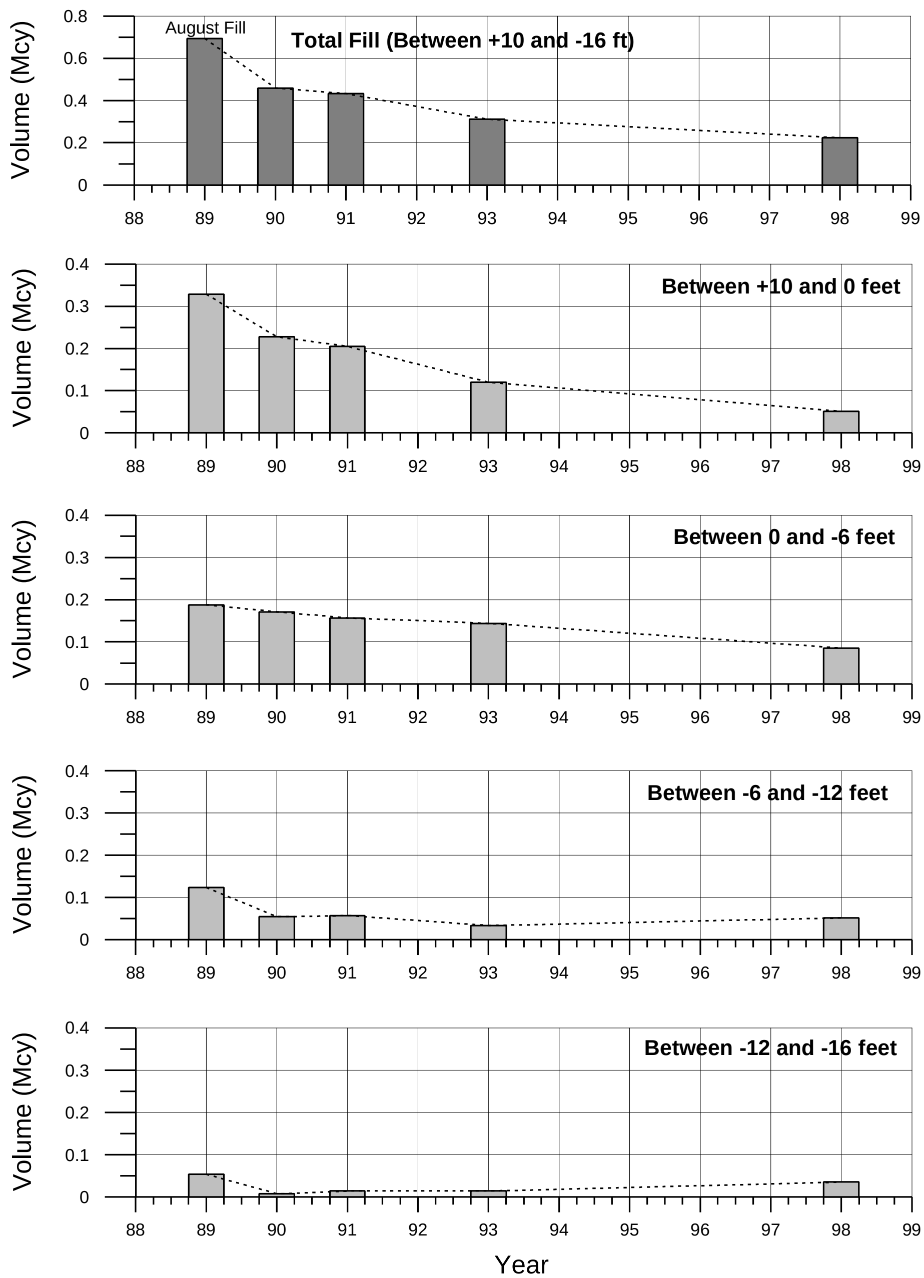

Figure B-8: Volumetric change along John U. Lloyd shoreline since May 1989.

B-45. John U. Lloyd South Shoreline and Dania (R-94 to R-101). The shoreline along the southern end of John U. Lloyd Beach State Park and Dania (R-94 and R-101) has never been nourished with a beach fill. As a result, only limited beach profile survey data are available for this section of shoreline. Typically, profiles R-94 to R-98 have been surveyed independently of R-98 to R-101. For purposes of discussion, these profiles have been referenced together (as R-94 to R-101) because they share a lack of prior beach fill placement. Inconsistent survey data make graphical comparisons of the two sub-reaches impractical. Therefore, only historical shoreline locations between R-94 and R-98 are presented in Figure B-9. Maintaining survey consistency, shoreline positions from R-98 to R-101 are presented in the following section of this report.

B-46. Judging from available measurements, there appear to be few significant long-term trends in shoreline position. The shoreline along this portion of Segment III is considered to be relatively stable. Figure B-10 depicts the annual rate of MHW shoreline change since 1979. It can be seen that the shoreline change rate between R-94 and R-98 is fairly close to zero and is currently eroding at a rate of less than six inches per year. The shoreline from R-98 to R-100 has historically behaved in a manner consistent with aforementioned sections of Segment III, as shown in the following section of this report.

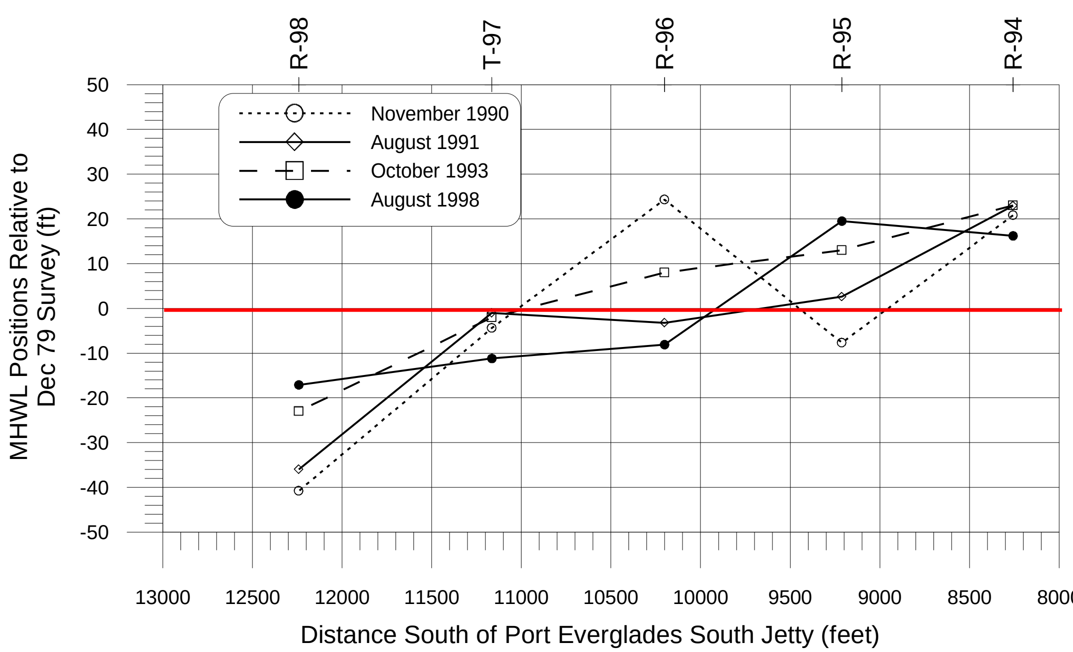

Figure B-9: The MHW shoreline location between R-94 and R-98.

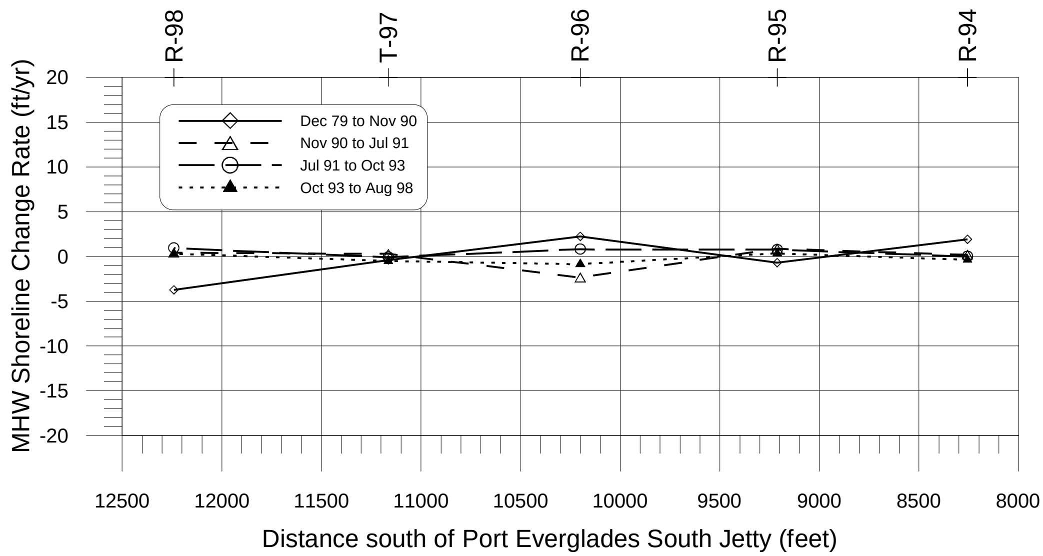

Figure B-10: MHW shoreline change rate between R-94 and R-98.

B-47. Hollywood/Hallandale Shoreline (R-101 to R-128). The most recent beach renourishment project along Hollywood/Hallandale was constructed between March and August 1991. This project included the placement of about 1.16 million cubic yards of sand along about 5.2 miles of shoreline. Figure B-11 illustrates the changes in shoreline positions subsequent to the construction of this project.

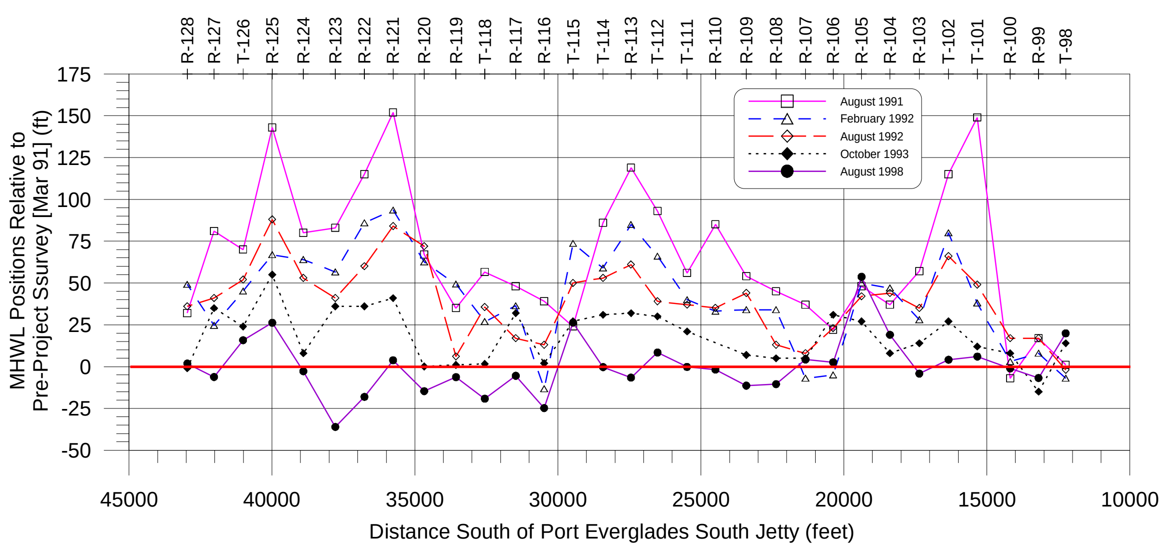

Figure B-11: Hollywood/Hallandale shoreline positions following 1991 nourishment.

B-48. Much like the previously discussed 1979 beach fill, the initial 1991 fill width along the project shoreline was not uniform; thus, the project experienced significant planform equilibration during the first 12 months following construction. It was not until about August 1992 that the project began to recede more or less uniformly along the entire reach. To demonstrate this, shoreline positions following project construction are shown in Figure B-12. Tracking the shoreline positions through time indicates extreme fluctuations immediately following project construction. Changes clearly appear less erratic in October 1993 where the average rate of recession is approximately 1-3 ft/yr with a fairly low deviation. With a limited number of exceptions, the MHW shoreline has currently eroded near or landward of its pre-construction position throughout this reach with heavy areas of sand loss occurring around R-101 and R-123.

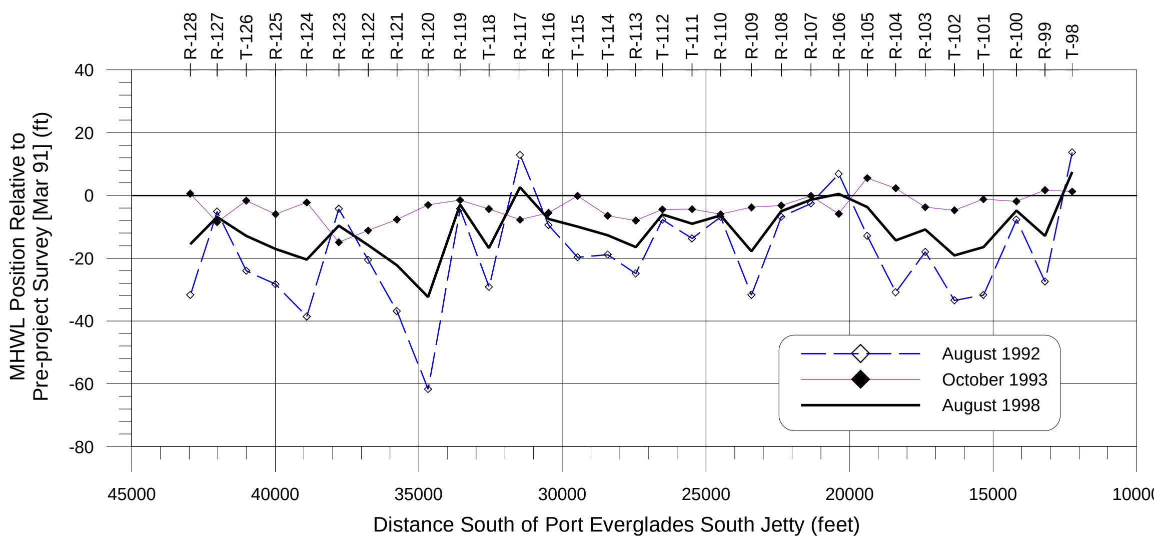

Figure B-12: Shoreline positions following 1991 Hollywood/Hallandale nourishment.

B-49. Remaining fill volume calculated after the 1991 fill is shown in Figure B-13. Again, volumes have been presented between specific elevation contours out to a depth of -16 feet (NGVD). Data show sediment immediately accreting offshore in depths between -6 and -12 feet. As of August 1998, 49 percent of the total original fill volume remains above the –16 ft contour in Hollywood/Hallandale. This represents an estimated 568,400 cubic yards of sediment. As previously discussed, many areas of this reach have eroded to or are now landward of the pre-project MHW shoreline. This becomes more apparent in Figure B-13 where, on average, nearly all of the volume between +10 and 0 feet has been lost and over 150,000 cy of pre-construction beach have been eroded between 0 and –6 feet. Estimates also indicate offshore accretion of about 360,000 cy between the –12 and –16 foot contours since 1991.

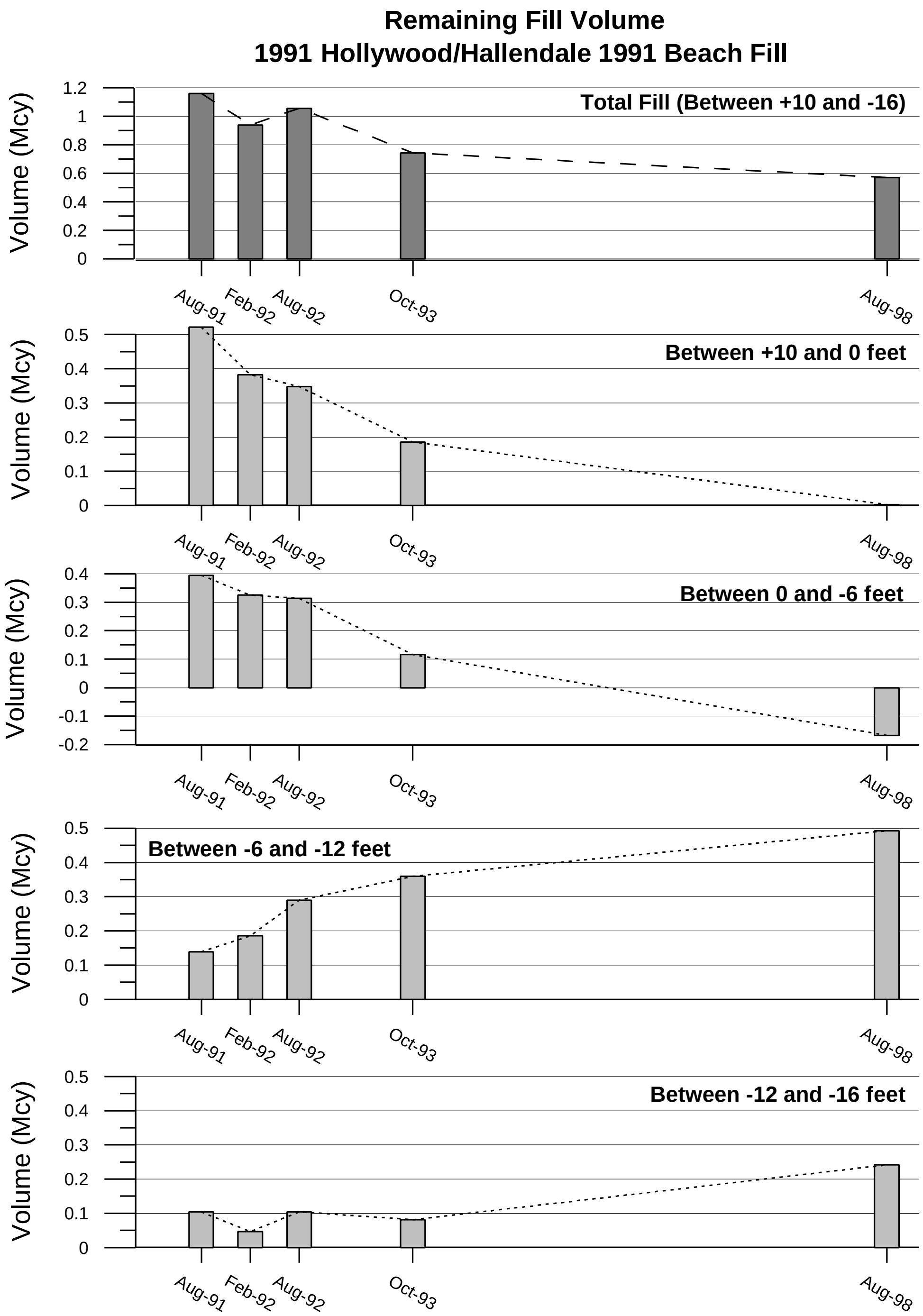

Figure B-13: Performance of 1991 Hollywood/Hallendale Beach Nourishment Project.

B-50. Figure B-14 summarizes volumetric changes along the entire Segment III shoreline. The broken line on the graph represents all available data following 1989 and 1991 beach construction projects. In an attempt to isolate equilibrium effects during the first 12-months following construction, a composite of volumetric change rates excluding those computed immediately after construction was developed and shown as a solid line below. Also, only data that were collected during similar annual seasons are presented to minimize the effects of seasonal variations in shoreline recession computations. This compilation more adequately identifies long-term performance trends of constructed nourishment projects and provides a foundation for the design of future works. In northern John U. Lloyd, actual recession is likely much higher than noted in Figure B-14 due to the limited sand volume currently available for transport. In considering the present sand deficit along northern John U. Lloyd, the most recent volume change data was not included during final recession estimates.

B-51. Overall, the average annual shoreline change rates measured from the 1989 JUL and 1991 Hollywood/Hallandale beach fill project suggest that the northern 8,100 feet of the John U. Lloyd shoreline losses about 6.5 cy/ft or 53,000 cy of sand lost each year. Considering the typical berm and depth of closure elevations along this reach of shoreline, the associated annual shoreline retreat rate is approximately –9.0 feet per year. Along the southern 4,000 feet of the John U. Lloyd shoreline, the area is generally accretional with an annual net gain of about 7,600 cubic yards. The Dania shoreline is only mildly erosional, losing about 600 cubic yards per year. Hollywood/Hallandale on the other hand continues to be erosional with an average alongshore sand loss rate of about 2.8 cy/ft per year. This is equivalent to an overall sand loss rate of 77,000 cy per year along the 27,600 feet of Hollywood/Hallandale shoreline. The shoreline recession rate associated with these sand loss estimates in Hollywood/Hallandale averages about -4 ft/yr.

B-52. The beach monitoring data collected as part of the 1989 and 1991 Segment III beach fill projects represent shoreline change associated with healthy beach conditions where a sufficient supply of sand was available for natural rates to be realized. It is argued that these rates more appropriately represent natural shoreline change conditions than those reported in the authorizing documents. Those latter rates were formulated from information collected during a period when the beach was in a highly eroded condition and armored with walls. The rates computed with the most recent shoreline change data are more consistent with those reported from the Segment II shoreline than those presented in the authorizing documents.

B-53. In all, the beach change data for the period between 1989 and 1998 suggests that the Segment III shoreline losses about 123,000 cubic yards of sand per year (see Table B-6). The reaches of the Segment III shoreline along which beach fill projects have been previously constructed lose approximately 130,000 cy/yr of sand each year.

Table B-6: Beach volume change rates for the Segment III shoreline 1989-1998.

Reach	Length of Reach (ft)	Volume Change Rate (cy/ft/yr)	Volume Change (cy/yr)	Shoreline Change (ft/yr)
John U. Lloyd - North	8,100	-6.5	-53,000	-9.0
John U. Lloyd - South	4,000	+1.9	7,600	+2.5
Dania	3,200	-0.2	-600	-0.5
Hollywood / Hallandale	27,500	-2.8	-77,000	-4.0
TOTAL	42,800		**-123,000**	

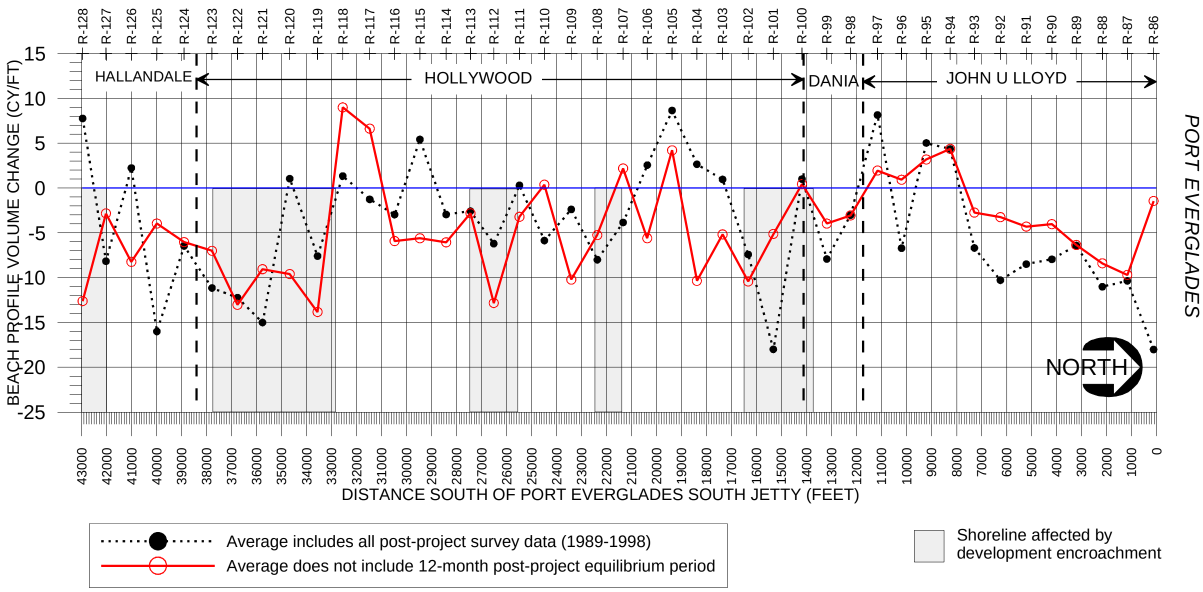

Figure B-14: Summary of volumetric change rates for Segment III shoreline.

B-54. In summary, previously constructed projects with renourishment have been successful in maintaining a wide protective and recreational beach along sections of the Segment III shoreline. There have been several areas along the Segment III shoreline, however, that have continued to experience heavily erosive conditions. These areas include the portion of shoreline extending about 3,000 feet South of the Port Everglades jetty (R-86 to R-89), the northern end of Hollywood (R-101 to R-102), and a localized area in southern northern Hollywood in the vicinity of the Diplomat Hotel (R-121 to R-124). Unique problems afflicting the aforementioned reaches present difficulties in developing specialized, effective engineering solutions.

PORT EVERGLADES IMPACTS

B-55. Port Everglades Entrance appears to act as a complete littoral sediment sink. That means that it not only prevents the *net* transport of sediment southward across the inlet, but it also captures northerly transported sand *from* Segment III. The inlet's littoral impact is primarily manifest as shoreline recession south of the inlet.

B-56. It is conservatively assumed that approximately 58,000 to 73,000 cubic yards of sand per year approach Port Everglades along the southern reaches of Segment II (Olsen Associates, Inc. and Coastal Planning and Engineering, Inc., 1998). Instead, the existing influx of sand to Segment III is generally thought to be zero. That is, the 58,000 to 73,000 cy/yr of sand that would normally be expected to reach Segment III is diverted to updrift impoundment, offshore, and into Port Everglades. At least for the period 1979 to 1993, it appears that about half of the material is diverted offshore and/or to the seabed, and half is diverted to impoundment.

B-57. The inlet does not only interrupt net drift from the north; it also acts as a sink to sand that is transported from the downdrift beach toward the inlet. There are insufficient survey data to determine this quantity directly; however, a reasonable value is inferred from the results of the refraction/diffraction and GENESIS analyses. The refraction/diffraction and sand transport potential analysis demonstrates that the *potential* for northerly transport into Port Everglades was approximately 10 percent of the southerly-directed net potential sand transport immediately south of Port Everglades. The latter is about 50,000 cy/yr; therefore, the net northerly drift potential directed toward Port Everglades from the south is about 5,000 cy/yr. That is, the presence of Port Everglades has created the *potential* for the inlet to sink 5,000 cy/yr of transport from the Segment III beaches during transport reversals.

B-58. The annual impact from Port Everglades Entrance is the sum of the inlet's interruption of net southerly transport and the sink effect upon the reversal transport from the south; i.e.,

58,000 to 73,000 cy/yr (interruption of net southerly drift to the downdrift beach)
+ 5,000 cy/yr (sink effect to transport from the downdrift beach)
63,000 to 78,000 cy/yr (net inlet impact)

That is, the inlet's potential total impact to the littoral system is between about 63,000 and 78,000 cy/yr. The magnitude of total inlet impact is expected to be the same as existing conditions at the time of the 2001 project construction. No significant changes would be expected in the absence of engineering sand bypassing at the inlet.

CROSS-SHORE SEDIMENT TRANSPORT

B-59. Cross-shore sediment transport characteristics for Broward County beaches were estimated using the Storm Induced BEAch CHange model, SBEACH (Larson and Kraus, 1989). SBEACH simulates beach profile changes that result from varying storm waves and water levels. These beach profile changes include the formation and movement of major morphological features such as longshore bars, troughs, and berms. SBEACH is an empirically-based numerical model, which was formulated using both field and the results of large-scale physical model tests. Input data required by SBEACH describes (1) the storm being simulated, and (2) the beach of interest. Basic requirements include time histories of wave height, wave period, and water surface elevation, as well as beach surveys and median sediment grain size.

B-60. SBEACH calculates the cross-shore variation in wave height and wave- and wind-induced setup at discrete points along the profile from the seaward boundary to the shoreline. The model calculates the limit of wave run-up in order to define the landward boundary of profile change. Profile changes are calculated at each model time step by solving for conservation of mass. An explicit finite-difference scheme is used for this solution.

B-61. The extent of beach erosion is often quantified in terms of beach recession. Throughout this discussion, recession is defined as the horizontal distance from the mean high water mark on the pre-storm profile to the landward most point where the vertical difference in pre- and post-storm profiles equals 0.5 feet. This definition is presented graphically in Figure B-15.

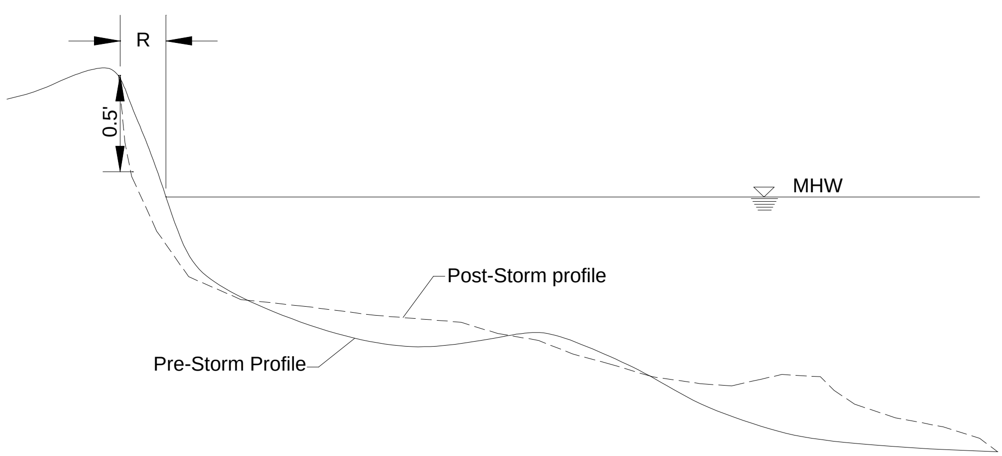

Figure B-15: Beach recession, R, definition sketch.

B-62. Basic assumptions underlying SBEACH simulations are that (1) breaking waves and variations in water level are the major causes of sand transport and profile change, (2) cross shore sand transport takes place primarily in the surf zone, (3) conservation of mass dictates that the amount of material eroded must equal the amount deposited, (4) median sediment diameter on the profile is reasonably uniform across shore, (5) influence of structures blocking longshore transport is small, and the shoreline is straight (i.e., longshore effects are negligible during the term of simulation), and (6) linear wave theory is applicable everywhere along the profile without shallow-water wave approximations.

B-63. SBEACH has significant capabilities that make it useful for quantitative studies of beach profile response to storms. It accepts as input a pre-storm beach profile (either idealized or surveyed), time series of water level as produced by storm surge and tide, time series of wave height and period, a representative sediment grain size, three transport parameters and two characteristic slope parameters. The model allows for variable cross shore grid spacing, wave refraction by specifying wave direction, randomization of input wave to better represent forcing conditions in the field, and water level setup due to input wind parameters. Output data consists of a final calculated profile at the end of the simulation, simulated profiles at intermediate time steps, intermediate and maximum wave heights, intermediate and maximum total water elevations plus setup, maximum water depth, volume change and a record of various coastal processes that may occur at any time-step during the simulation (accretion, erosion, overwash, boundary-limited runup, and/or inundation).

B-64. SBEACH requires the calibration of three empirical parameters: (1) the transport rate coefficient (K), (2) the transport rate slope dependence (ε), and (3) the transport rate decay factor (λ). Calibration of these parameters requires measurement of pre- and post-storm profiles at the site where the model is used.

B-65. Site specific pre- and post-storm beach profile data for the Broward County Segment III shoreline are not available. However, previous efforts have produced accepted calibration coefficients for other areas of the Eastern Florida coast. These shorelines are located in Martin County, Brevard County, and the Ponce de Leon Inlet area in Volusia County. Of the three, only the Martin County study was calibrated using measured pre- and post-storm profile data. Default calibration coefficients used in SBEACH were developed with water level, wave, and beach change data collected at Duck, North Carolina. In this study, it is not assumed that storm-induced beach change at Duck, North Carolina is representative of that in South Florida. Instead of relying solely upon the default values for this study, however, a sensitivity analysis comparing previous calibration efforts with the default values was conducted.

B-66. During the sensitivity analysis, only the coefficients, K, λ, and ε were varied. These coefficients were varied as indicated in Table B-7. Each set of calibration coefficients were run and compared using one extratropical and two tropical storm simulations, herein named extratropical storm number 6, HURDAT storm number 194 and HURDAT storm 353 respectively. Extratropical storm number 6 occurred on November 23, 1980 and was modeled with a tidal phase of 270 degrees. Tropical storm 194 occurred on October 9, 1909 and was also modeled with a tidal phase of 270 degrees. Tropical storm 353 made landfall on August 29, 1935 and was modeled using a tidal phase of 180 degrees. Each storm was modeled impacting the three composite profiles developed for the Segment III study area. Reach 1 represents the shoreline from R-086 to R-099. Reach 2 represents the shoreline from R-100 to R-104, and Reach 3 represents the shoreline from R-105 to R-128. The development of these profiles is discussed later in this report.

B-67. The results of the sensitivity analysis including the corresponding recession distances are shown in Table B-7. The location at which the recession distances were measured is the +1.64 ft NGVD elevation. This elevation is considered the natural mean high water line along the study area shoreline.

B-68. Inspection of Table B-7 indicates little sensitivity of MHW recession to the various calibration coefficients used in this analysis. The deviation about the average recession averages 4.6 feet. The default calibration coefficients produced the greatest amount of MHW recession, while the martin county coefficients produced the least. The conservative nature of the Martin County coefficients combined with the fact that they were calibrated using pre- and post-storm profiles make them the best choice for the purposes of project justification.

Table B-7: Sensitivity analysis for SBEACH transport coefficients.

		Distance from pre-storm MHW to landward limit of 0.5 foot erosion. (feet)					
Reach	**Storm**	**Default**	**Ponce**	**Brevard**	**Martin**	**AVG (ft)**	**SD (ft)**
1	#6	0	0	0	0	0	0
	194	162.9	174.7	167.7	152.3	164.4	9.4
	353	169.9	166.0	163.2	163.8	165.7	3.0
2	#6	43.3	41.0	41.5	41.4	41.8	1.1
	194	188.3	186.0	186.9	185.9	186.8	1.1
	353	224.9	214.6	206.4	214.2	215.0	7.6
3	#6	41.7	39.1	38.8	38.5	39.5	1.5
	194	144.0	159.9	159.1	133.8	149.2	12.6
	353	142.2	136.0	130.0	138.4	136.6	5.1

Adjusted Calibration Coefficients				
Project	**Default**	**Ponce**	**Brevard**	**Martin**
K (m^4/N)	1.75E-06	1.75E-06	1.70E-06	1.50E-06
EPS (m^2/s)	0.002	0.001	0.001	0.0015
LAMM (m^{-1})	0.4	0.5	0.5	0.4

B-69. *Production Model Runs*. The cross-shore sediment transport analysis procedure involved the use of the SBEACH model to perform multiple simulations of historical tropical and extratropical storms that have influenced the project shoreline. Recent Broward county beach profile surveys (August 1998) were used to represent pre-storm conditions. The study area was divided into three reaches, based on morphological dissimilarities. Representative beach profiles, R86, R100, and R105, were generated to represent pre-storm conditions along each reach. Simulations of all historical storms were then executed for each composite profile. This resulted in a comprehensive database of site-specific tropical and extratropical storm recession information. This database was then used to generate beach recession versus frequency of occurrence relationships, which are discussed in the following paragraphs.

B-70. J*oint-Probability Analysis of Storm-induced Beach Recession*. Proposed shore protection measures must be subjected to a benefit-cost analysis in order to assess whether Federal participation in the project is appropriate. Primary benefits are typically quantified in terms of the reduction of storm-induced damages to existing property and/or structures. In order to quantify those benefits, one must estimate a) the damage potential which exists without the proposed protection measures (i.e., for existing conditions), and b) the damage potential which exists with shore protection measures in place. Benefits are expressed as the reduction in storm-induced damages resulting from the presence of the shore protection measures. In order to account for risks and uncertainties inherent to the analysis procedure, methods were required in the form of recession versus frequency of occurrence relationships. The Empirical Simulation Technique (EST) (Borgman et al., 1992), was selected as the joint-probability analysis tool used to establish those relationships. The beach recession analysis procedure can be described by applying the following major tasks:

1. Identify storm events that have impacted the study area.
2. Construct or obtain the water surface elevation and wave field hydrographs characteristic of each of the identified storms while in the vicinity of the study site.
3. Apply the numerical model, SBEACH, to estimate the beach recession associated with each of the storm events.
4. Construct EST input data files using descriptive storm parameters and calculated recession values.
5. Use the EST to generate multiple repetitions of multi-year scenarios of storm events and their corresponding beach erosion confidence limits.
6. Apply the resulting recession-frequency curves as input to an appropriate economics based model for computation of damages, costs, and benefits.

B-71. The initial step in any storm-induced recession/frequency analysis is identification of all historical storms that have impacted the area of interest. For Atlantic coast sites, such as Broward county, the shoreline is subjected to both tropical cyclones (tropical depressions, tropical storms, and hurricanes) and extratropical storms (northeasters). While tropical storms are often characterized by very high wind, wave, and surge conditions, the longer duration of extratropical storms can result in beach erosion of equal or greater magnitude than the erosion caused by storms of tropical origin. Once the historical storms of interest are identified, corresponding storm surge hydrographs and wave condition time series must be extracted from appropriate data sources. For this application, those data sources consisted of the DRP storm surge database and the WIS hindcast wave database.

B-72. Selection of tropical cyclones to be simulated begins with identification of the DRP station that lies nearest to the site in question. As explained previously, DRP Station 442 was chosen. The tropical surge database indicated that 12 tropical cyclones have significantly impacted the area represented by station 442. For this application, a significant influence implies the storm resulted in a surge of at least 0.5 meters at the study site. The 12 storms identified for the Broward county area are listed in Table B-8. Individual storm tracks and maximum surge elevations at all nearshore stations are available in the tropical cyclone database summary report (Scheffner et al., 1994). An estimate of the frequency of occurrence of tropical cyclones which impact the project shoreline can be computed as: 12 events/104 seasons = 0.12 events per year. This can be expressed as a recurrence frequency of roughly one tropical cyclone every eight years.

B-73. The DRP extratropical storm database contains 16 winter seasons of storm surge and current hydrographs from 1977 to 1993. Extratropical storms were identified by visual inspection of each season's storm surge hydrographs at DRP station 442. These hydrograph inspections, combined with a general estimation of the frequency of extratropical storms along the east coast of Florida, and knowledge concerning the more prominent storms, resulted in a 0.085-meter threshold magnitude of the storm surge. In other words, individual extratropical storms were identified as those events characterized by deepwater surge magnitudes that equaled or exceeded 0.085 meters. Analysis of all 16 extratropical storm seasons resulted in a compilation of the storms listed in Table B-9. It also identifies the approximate date of occurrence and magnitude of the peak storm surge elevation, relative to mean sea level (msl). An estimate of the frequency of occurrence of extratropical storms which impact the project site can be computed as: 16 events/15 seasons = 1.07 events per year.

B-74. In summary, the selection of storm events from the available databases resulted in the identification of 12 tropical cyclones and 16 extratropical storms that have influenced Broward county beaches. The tropical storm database encompasses those storms that occurred during the 104-year period from 1886 through 1989. The extratropical storm database includes 15 years of data, from 1977 through 1993. Estimated frequencies of occurrence for tropical cyclones and extratropical storms that impact the project shoreline are 0.12 and 1.07 storms per year, respectively.

Table B-8: Tropical storms with influence on Broward County.

HURDAT STORM NUMBER	DATE	STORM NAME
112	3-Aug-1889	
127	4-Aug-01	
189	6-Oct-09	
194	9-Oct-10	
276	11-Sep-26	#6
292	6-Sep-28	#4
296	22-Sep-29	
353	29-Aug-35	
357	30-Oct-35	
473	18-Sep-48	#7
597	29-Aug-60	DONNA
629	20-Aug-64	CLEO

Table B-9: Extratropical storms with influence on Broward County.

STORM NUMBER	STORM SEASON	DATE	MAXIMUM SURGE HEIGHT (m)
	1977-1978	NO STORMS	
1	1978-1979	29-Dec	0.087
2		17-Feb	0.094
3	1979-1980	20-Jan	0.091
4		8-Feb	0.091
5		4-Mar	0.096
6	1980-1981	23-Nov	0.121
7		13-Feb	0.099
	1981-1982	NO STORMS	
	1982-1983	NO STORMS	
8	1983-1984	25-Dec	0.087
9		1-Jan	0.13
10		22-Feb	0.105
11	1984-1985	8-Nov	0.088
12		24-Nov	0.111
	1985-1986	NO STORMS	
	1986-1987	NO STORMS	
	1987-1988	NO STORMS	
13	1988-1989	10-Mar	0.139
	1990-1991	NO STORMS	
14	1991-1992	30-Oct	0.094
15	1992-1993	16-Dec	0.09
16		19-Mar	0.104

B-75. *Storm Surge and Wave Hydrograph Development*. The second major step of the EST procedure is construction of the appropriate storm surge and wave field hydrographs. The total storm-induced surge elevation (prior to inclusion of wave and wind setup) can be divided into two major components, storm surge and astronomical tide. The tropical and extratropical simulations that generate the storm surge characteristics contained in the DRP database did not include consideration of tides at the time of the storm event. Storm surge modeling was performed with respect to mean sea level. Total surge elevation and corresponding beach recession estimates can be significantly influenced by the magnitude and phasing of the tidal component. Tidal influence was accounted for by assuming that each storm event had an equal probability of occurring during the tidal cycle. For this analysis, that assumption was simplified by allowing the onset of the storm conditions to coincide with four individual tidal phases. Those phases were designated as 0 degrees (high tide), 90 degrees (msl during peak flood), 180 degrees (low tide), and 270 degrees (msl during peak ebb). Tidal components characteristic of the project site were obtained from the DRP database for computation of tidal elevations. The result of combining storm surge and tidal components of the total surge elevation is a four-fold increase in the number of individual storms in the tropical and extratropical databases. For example, each individual storm in the original 12-storm database was represented by four storms that differ solely with respect to tidal phasing. Therefore, the tropical cyclone database was expanded from 12 storms to 48 storms, and the extratropical database grew from 16 storms to 64 storms.

B-76. It should be noted that the time histories of the storms in question were limited in duration to the periods in which the storms were influencing the project beaches. The appropriate hydrograph duration for tropical and extratropical storms was determined to be 43 hours and 147 hours respectively. Extratropical hydrographs were generated with a 3-hour time-step to accomplish compatibility with the hindcasted wave data. Tropical storm hydrographs were generated using a 1-hour timestep.

B-77. Wave conditions corresponding to each of the extratropical storms were obtained from the WIS hindcast database. Those wave height and period hydrographs represented deepwater wave conditions at WIS Station A2009. Wave conditions characteristic of tropical cyclones were computed in accordance with procedures specified in the Shore Protection Manual (USACE, 1984). Storm track direction, and minimum height and period values were specified based on information from the WIS summary tables (Hubertz et al., 1993) for Station A2009.

B-78. *Application of SBEACH Model*. The third step in the EST procedure is the application of the cross-shore sediment transport model to compute storm-induced erosion. For each storm simulation, wave transformation was computed with algorithms included in SBEACH. For this application, profiles extended approximately 10,000 feet offshore where depths ranged from about 140 to 15 feet. Wave transformations were performed using methods described for random waves impinging upon a non-monotonic profile (Larson and Kraus, 1989).

B-79. A comparative analysis of beach profile surveys indicated that the project shoreline could be divided into three reaches. SBEACH simulations of the 48 tropical and 64 extratropical cyclones were then performed for each reach. The estimated beach recession corresponding to each of these storms was archived for input into the EST joint probability analysis.

B-80. *EST Input Development*. The fourth step in the empirical simulation procedure involves preparation of the EST input files. These files contain input vectors, response vectors, and frequency of storm occurrence parameters. The values of the input parameters reflect the storm intensity. The response vector, in this application, quantifies the beach recession resulting from a given storm; and the storm frequency parameters are used to dictate the occurrence of extratropical and tropical storms throughout the multi-year life cycle analysis.

B-81. The characteristics of individual tropical storms were defined as: (a) tidal phase, (b) closest distance from the eye to the project site, (c) direction of propagation at time of closest proximity, (d) central pressure deficit, (e) forward velocity of the eye, (f) maximum wind speed, and (g) radius to maximum winds. As noted, the response to each storm was defined as the beach recession modeled by SBEACH. The frequency of occurrence of tropical events that impact the project beaches was previously estimated at 0.12 events per year. This corresponds to one event every 8.3 years.

B-82. Input vectors describing extratropical storms were defined as: (a) tidal phase, (b) storm duration, (c) maximum surge elevation, (d) wave height, and (e) wave period. The response vector was, of course, beach recession; and the frequency of occurrence of extratropical storms was previously estimated at 1.07 events per year.

B-83. *EST Execution*. The fifth step of the EST is the execution of empirical simulation procedures to generate multiple repetitions of multi-year scenarios in which storm events may occur. For this application, 100 repetitive simulations of a 200-year period of storm activity were performed. Simulations of extratropical and tropical storm histories were performed separately. For each simulation, a 200-year tabulation was generated to include the number of storms that occurred during each year and the corresponding beach recession. This information provides the basis for calculation of return periods associated with various degrees of beach recession.

B-84. The final step in the EST procedure is analysis of results and presentation of those results in a format suitable for subsequent probabilistic analyses. In this case, the EST results were used as input for an economic evaluation of the impacts of beach recession. The economic model estimates damage and repair costs (related to storm-induced beach recession) that would be incurred over a multi-year period if no project improvements were constructed. The economic model makes no distinction between extratropical and tropical storms; therefore, the tropical and extratropical EST results were combined to generate a single storm-induced recession versus frequency of occurrence relationship.

The following algorithm was used to accomplish this combination of extratropical and tropical results:

For a given recession value: $T_c = (1/T_t + 1/T_e)^{-1}$

Where: T_c denotes return period corresponding to the chosen recession.
T_t represents the tropical storm return period corresponding to the chosen recession.
T_e equals the extratropical storm return period corresponding to the chosen recession.

B-85. As expected, due to their greater frequency of occurrence, the extratropical storms dominate the results corresponding to lower return periods. The greatest recession values were characteristic of the most severe tropical cyclones (i.e., hurricanes). Return periods associated with levels of combined tropical and extratropical storm-induced beach recession are provided in Table B-10.

Table B-10: Recession vs. frequency of occurrence results.

Return Period (yr)	REACH	
	R-86 to R-94	R-101 to R-128
200	187	177
100	171	160.5
50	148	129
20	103	90
10	65	80
5	52	71
2	41	58.5
1	26.5	33

B-86. *Summary of Cross-Shore Transport Analysis*. The preceding information was provided to summarize how EST procedures were applied to this probabilistic analysis of cross-shore sediment transport in Broward County. This application generated frequency of occurrence relationships for storm-induced beach recession along Segment III of the Broward County shoreline, as tabulated above. The beach recession-frequency relationships were subsequently utilized as input to economic model for quantification of recession related damages to shorefront properties.

LONGSHORE SEDIMENT TRANSPORT

B-87. Longshore sand transport along the Segment III shoreline is the dominant mechanism for shoreline change. Longshore sand transport rates are highly variable due to the presence of the Port Everglades Entrance jetties, irregularities in the elevation of the nearshore reef structure and the orientation of the shoreline. Additional variabilities in the longshore sand transport rates have been due to end effects at the terminus of past beach fills. At those locations, specifically at the south end of the John U. Lloyd and northern end of the Hollywood/Hallandale projects, beach fill performance has been poor due to high alongshore sand loss rates.

B-88. For purposes of formulating project modifications necessary to improve beach fill performance in Segment III, the Generalized Model for Simulating Shoreline Change (GENESIS) model (Hanson and Kraus, 1989) is used to predict shoreline changes and sediment transport quantities, with and without project modifications. The GENESIS model provides a numerical method for determining long-term shoreline change on an open coast in response to spatial and temporal variations in longshore sediment transport. The model can be calibrated to site-specific conditions which are defined by shoreline surveys, sediment budget analyses, wave conditions, offshore bathymetry, and the presence of coastal armoring, beach fills, groins, offshore breakwaters, and inlet sand bypassing operations. Locations of the shoreline, coastal structures, and beach fills are referenced to a baseline that defines the orientation of the modeling grid. Longshore transport rates are calculated at the cell boundaries utilizing methodology described in the Shore Protection Manual. Site-specific wave data (period, wave height, and direction) are used in concert with the longshore transport equation (USACE, 1984) at incremental time steps to simulate shoreline changes due to the addition or removal of sand from a discrete section of shoreline. The discrete shoreline sections are represented by model grid cells. The computed rate of longshore sand transport and shoreline change is calibrated to the input wave data and historical shoreline change through two calibration coefficients (K_1 and K_2).

Shoreline Change Model (GENESIS)

B-89. Overview. The purpose of the modeling exercise is to evaluate the potential for alongshore shoreline changes along the Segment III shoreline and simulate the effects of proposed project modifications. The proposed modifications include beach fill tapers at the southern end of the John U. Lloyd and the northern end of the Hollywood/Hallandale design beach section and the a groin field at the northern end of the John U. Lloyd Beach State Park shoreline. Additionally, the potential benefits of mechanical sand bypassing at Port Everglades to the Segment III sediment budget is evaluated with the calibrated GENESIS model.

B-90. To accomplish these modeling tasks with a version of GENESIS that is limited to 200 grid cells, two separate GENESIS domains were developed. The first model was formulated to represent the entire John U. Lloyd Beach State Park shoreline. This model was intended to accurately simulate shoreline change along the groin field shoreline and along the shoreline immediately downdrift of the groin field. The second model represents the entire Hollywood/Hallandale shoreline plus about 5,000 feet north and south of that area. This model consists of larger grid cell widths. The wider grid cells allow for the entire Segment III shoreline to be represented with the 200-grid cell model. The input wave data were common for both model domains. Detailed model calibration and verification simulations were performed with the John U. Lloyd model. The calibration results were modified slightly for the Hollywood/Hallandale model during an independent verification of that model.

Offshore Wave Data

B-91. Offshore wave data used to represent typical wave conditions at the project site were derived from WIS hindcast wave data. Hindcast data from WIS Station A2009 were used to represent local wave conditions. These data, which are available from the CEDRS database were prepared by the U.S. Army Corps of Engineers Coastal Engineering Research Center (CERC) (Hubertz et al., 1993). These hindcast wave data represent wave conditions offshore of the Broward County Segment II shoreline for the period between 1956 and 1995. It is noted that the wave hindcast data for the period between 1956 and 1975 do not include tropical weather systems. This database comprises 40 years of hindcast wave data from atmospheric pressure and wind speed records over that time period.

B-92. The two 20-year times series were processed using wave analysis utilities included in the Shoreline Modeling System (SMS) that accompanies the GENESIS model. The time series were converted from their reported offshore depth and orientation (720 ft, 0 degrees true north; Phase II) to a nearshore depth of 145 ft and a shoreline orientation of 2 degrees E of N (Phase III). This procedure, which was accomplished with the SMS utility, WAVETRAN, aligned the wave data with the subject shoreline and the subsequent nearshore wave refraction grid. The resultant time series were then processed using the utility, RCRIT, to eliminate wave events in the time series that either were traveling away from the shoreline or were too small to generate significant longshore sand transport. The criteria used to eliminate wave events from the time series follows the method of Hanson and Kraus (1991). Both the primary and secondary components of the wave time series were retained throughout the analyses.

B-93. The conditioned offshore wave time series were analyzed to determine the potential longshore sediment transport rates for each of the forty years or record. This procedure included the use of the SEDTRAN utility that estimates the annual northerly, southerly, net and gross sediment transport potentials at a local project site. The results of this analysis are summarized in Table B-11.

Table B-11: Uncalibrated longshore sand transport rates 1976-1995 (cy/yr).

YEAR	NORTH	SOUTH	NET	GROSS
1976	1,400,000	190,000	1,210,000	1,590,000
1977	1,100,000	190,000	910,000	1,290,000
1978	1,300,000	180,000	1,120,000	1,480,000
1979	1,600,000	710,000	890,000	2,310,000
1980	940,000	260,000	680,000	1,200,000
1981	1,200,000	280,000	920,000	1,480,000
1982	600,000	340,000	260,000	940,000
1983	860,000	410,000	450,000	1,270,000
1984	1,700,000	250,000	1,450,000	1,950,000
1985	1,200,000	260,000	940,000	1,460,000
1986	1,100,000	260,000	840,000	1,360,000
1987	870,000	370,000	500,000	1,240,000
1988	610,000	280,000	330,000	890,000
1989	420,000	120,000	300,000	540,000
1990	690,000	320,000	370,000	1,010,000
1991	550,000	320,000	230,000	870,000
1992	1,000,000	470,000	530,000	1,470,000
1993	730,000	290,000	440,000	1,020,000
1994	900,000	450,000	450,000	1,350,000
1995	570,000	410,000	160,000	980,000
AVERAGE (CY/YR)	967,000	318,000	649,000	1,285,000
LOW	609,139	187,471	284,339	888,482
HIGH	1,324,861	448,529	1,013,661	1,681,518

Nearshore Wave Data

B-94. Overview. Gradients in the longshore sand transport potential are related to alongshore variations in nearshore wave conditions. Nearshore wave conditions in the GENESIS model are represented at each time step by normalized refraction and shoaling coefficients created from the results of a grid-based refraction model. There are four steps required to formulate the nearshore wave conditions. The first includes the determination of representative wave conditions to be simulated in the refraction model. The second consists of compiling hydrographic data collected in the vicinity of the study site and developing the model computational grid. The third involves the execution of the refraction model for each representative wave condition. The final step requires review and selection of the appropriate computed breaking wave conditions along the entire refraction grid domain and creating the input nearshore wave file to be used by GENESIS. The details of each of these steps are briefly described below.

B-95. Representative Wave Conditions. To minimize the number of required wave refraction/diffraction simulations, the 20-yr Phase III wave time series (1976-1995) was processed using the WHEREWAV utility in the SMS package. This procedure sorts the wave data into direction and period bins which then serve to "represent" each individual wave event in the time series. The individual wave conditions within each bin were compiled to determine the average wave height, period and direction for each bin. Table B-12 presents the resultant wave conditions (23 cases) used in the wave refraction modeling. The average wave heights were used in the following refraction/diffraction analysis, rather than unit wave heights as described in the GENESIS Workbook and System User's Manual (Gravens and Kraus, 1991), because it was necessary to determine actual wave heights at breaking. The resultant wave heights used in the preparation of the nearshore wave transformation file were then normalized using the average wave height in each bin to accommodate the GENESIS format.

B-96. Bathymetric Data and Grid Preparation. The refraction/diffraction analysis requires a computational grid that represents the offshore bathymetry. The bathymetric grid was developed from several hydrographic data sets that have been collected along various portions of the study area. These include an August 1998 beach and nearshore survey by Broward County, a 1997 LIDAR and NOS (National Ocean Survey) survey of the offshore area immediate to Port Everglades, a 1993 hydrographic survey of the area south of Port Everglades conducted as part of the Coast of Florida Study, and a hydrographic survey of the area from Port Everglades to the Dade County Line conducted as part of the current investigation. Portions of each of these data sets were combined to formulate a representative hydrographic data set for the entire area of interest.

Table B-12: Summary of nearshore wave events by angle and period band.

Wave Condition Number	Angle Band	Period Band	NSWAV Key #	Number of Events	Percent Occurrence	Average Wave Angle (deg)	Average Wave Height (ft)	Average Wave Period (sec)
1	3	1	131	2364	2.0	47.2	2.0	3.8
2	3	2	132	5518	4.7	49.2	3.1	5.5
3	3	3	133	9441	8.1	51.7	3.1	7.6
4	3	4	134	8762	7.5	49.9	2.5	9.5
5	3	5	135	4591	3.9	44.6	2.1	11.4
6	3	6	136	1060	0.9	39.3	2.0	13.3
7	4	1	141	4161	3.6	23.6	1.9	3.9
8	4	2	142	5343	4.6	24.3	3.9	5.5
9	4	3	143	1953	1.7	25.0	6.7	7.3
10	4	4	144	328	0.3	29.5	6.9	9.2
11	4	5	145	448	0.4	34.5	1.3	11.5
12	4	6	146	1976	1.7	33.9	1.6	13.5
13	4	7	147	1146	1.0	30.1	1.5	15.4
14	4	8	148	576	0.5	26.9	1.8	18.0
15	5	1	151	5826	5.0	2.1	2.0	3.9
16	5	2	152	7525	6.4	2.7	3.6	5.4
17	5	3	153	1515	1.3	4.2	6.6	7.2
18	6	1	161	4920	4.2	-21.1	1.9	3.8
19	6	2	162	3969	3.4	-20.3	3.4	5.3
20	6	3	163	548	0.5	-20.1	5.6	7.2
21	7	1	171	3490	3.0	-42.5	1.8	3.8
22	7	2	172	3089	2.6	-42.6	3.1	5.4
23	7	3	173	552	0.5	-42.3	5.2	7.3

B-97. The final refraction/diffraction grid consisted of 113 onshore rows and 598 alongshore columns. The grid spacing was 100 ft alongshore and 100 ft onshore. This grid represents an area that is 59,800 feet long in the north/south (alongshore) direction and 11,300 feet in the east/west (cross-shore) direction. The offshore boundary of the model grid was located seaward of the third reef system offshore of Broward County in 145 feet of water. For the purposes of this investigation, it was assumed that the bottom contours seaward of that depth were straight and parallel and that wave conditions in 145 feet of water are more or less uniform along the entire Segment III shoreline.

B-98. Wave Refraction/Diffraction Analysis. The wave refraction/diffraction model used in the analysis was REFDIF-1 (Version 2.5) developed by Kirby and Dalrymple (1992). Simulations were performed for the 23 representative offshore wave conditions summarized in Table B-12. In each case, the nearshore pattern of each representative wave condition was computed across the entire computational grid. Figure B-14 presents the wave refraction/diffraction results in the vicinity of Port Everglades for the most frequently occurring condition (Case 3). The length and orientation of the arrows in the vector plot indicate the wave direction and height, respectively, as the waves are transformed across the irregular bathymetry of the study area.

B-99. Refraction/Diffraction Modeling Observations. Several features of the bathymetry alongshore directly influence the shape and behavior of the subject shoreline. Most notably, Port Everglades Entrance itself controls the location of the shoreline immediately adjacent to the inlet. The shape of the shoals and jetties associated with Port Everglades modulates the approaching wave field in a manner that results in a focusing of wave energy immediately downdrift of Port Everglades. This wave focusing, combined with sheltering from the inlet jetties, causes a large gradient in breaking wave heights and directions along the downdrift shoreline. This phenomenon is most notable within 3,000 feet of the south jetty.

B-100. GENESIS Nearshore Wave File. The GENESIS model employs a nearshore wave transformation file to transform waves from the offshore time series to the shoreline to calculate breaking conditions. The method involves determining wave height and angle conditions at a pre-determined “nearshore reference depth.” This depth is chosen such that very few (if any) of the waves in the offshore time series will break at this depth, so as to avoid the truncation of any wave energy in the offshore time series. From this nearshore reference depth, the input wave heights and angles from the refraction analysis are assumed to propagate onshore to breaking over straight and locally parallel contours, consistent with linear wave theory.

B-101. The difficulty in the assumption of locally straight and parallel contours between the reference depth (typically 20 ft or deeper) and the shoreline is the omission of any bathymetric features that lie in between. Inspection of Figure B-16 illustrates that along the shorelines adjacent to Port Everglades, very significant bathymetric features lie in water depths of 15 ft or less. Omission of the effects of these features on the wave field would essentially invalidate any shoreline change modeling or longshore transport analysis.

B-102. Bodge et al., (1996), present a method by which input wave data for shoreline change models may be improved by accounting for nearshore bathymetric features up to the breaking point. This method, termed “backward refraction[2],” involves computing the breaking wave height and angle alongshore from the wave refraction analysis, then computing via linear theory the corresponding wave height and angle at the chosen nearshore reference depth. In this method, any depth can be chosen as a reference depth, thus allowing the modeler to assure that no wave energy would be truncated in the offshore time series. The “backwards refracted” wave data are ultimately converted to GENESIS compatible input files using the SMS software utility WTNSWAV.

[2] The process is termed “backward refraction” since most refraction calculations involve transforming a wave of given properties from a deeper water condition to a shallower depth, whereas with this analysis the shallower water wave is transformed “backward” offshore.

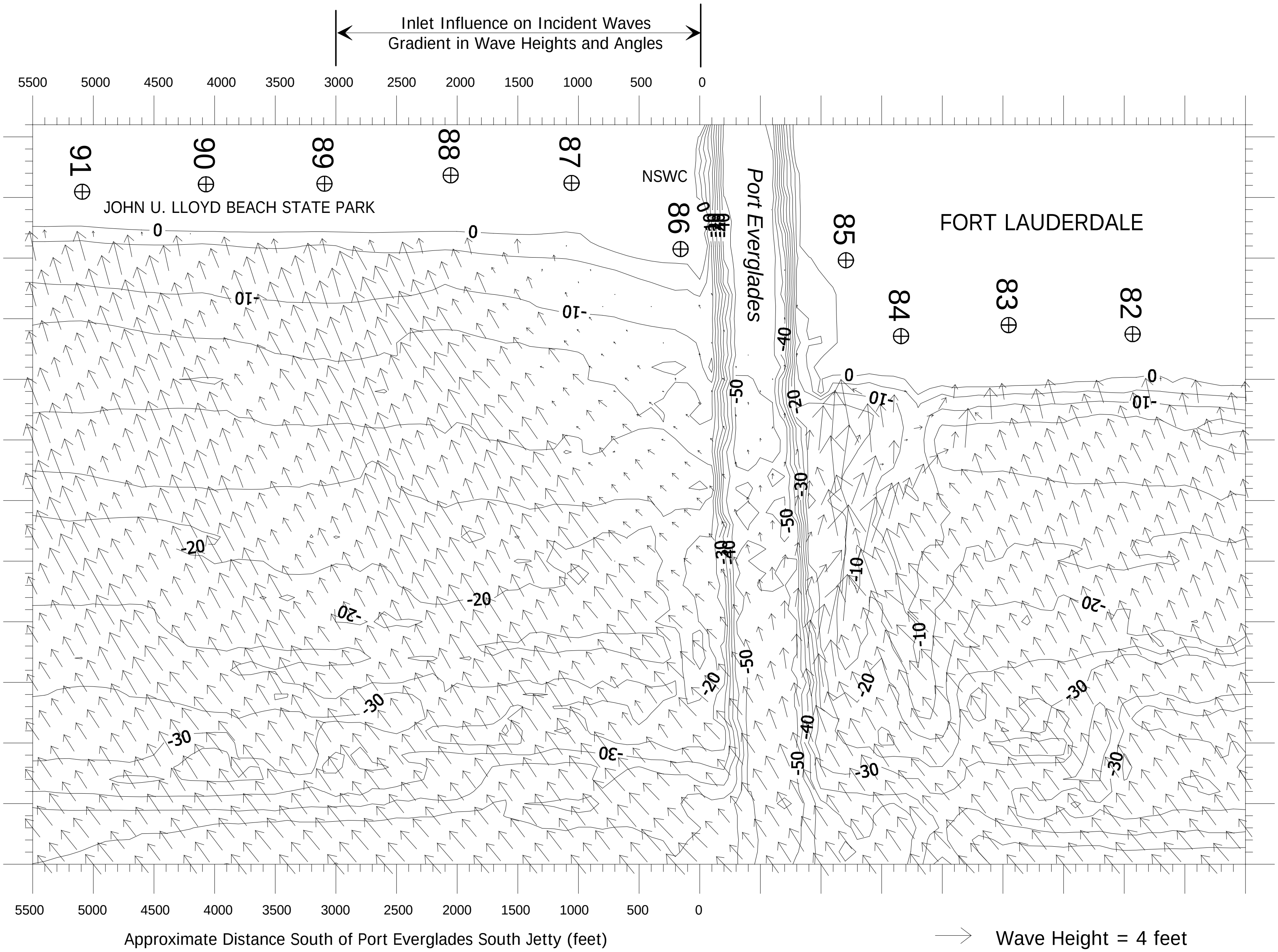

Figure B-16: Example of refraction/diffraction model results at Port Everglades.

Calibration/Verification

B-103. General. Of specific interest to the GENESIS shoreline change model study are the potential effects of proposed project modifications along the northernmost reach of the Segment III shoreline. Therefore, for purposes of this investigation, the GENESIS shoreline change model was calibrated and verified with measured shoreline change along the northernmost reach of the Segment III shoreline. Specifically, measured shoreline data for the 1989 John U. Lloyd Beach State Park beach fill was used.

B-104. Model Domain. The GENESIS model is a one-dimensional shoreline change model that requires one-dimensional grids for the simulations. Grid cell spacing for the John U. Lloyd model was set at 60 feet. This allowed for the minimum of three grid cells between simulated groin locations. (It is noted that the proposed groins are spaced between 270 and 300 feet apart). Considering the 200-grid cell capacity of the GENESIS model used in this investigation, only 12,000 feet of shoreline were modeled. The Dania Gap and adjacent shorelines are between 12,000 and 16,000 feet south of Port Everglades.

B-105. The northern boundary of the GENESIS grid corresponds approximately to the Port Everglades south jetty. The southern boundary was set in the vicinity of FDEP monument R-98. The resulting model (N-S) distance was about 12,000 feet, or 200 cells. The grid was generated with a 2 degrees east-of-north rotation angle, which is approximately the study area shoreline orientation.

B-106. Physical Input Data. Physical input data for the model was taken from recent beach survey and geotechnical data. The berm elevation was set at +10 feet, NGVD and the depth of closure was assumed to be –8.3 feet, NGVD, on average, along the entire study shoreline (see Table B-4, p. B-11). The median grain size of the beach sediments was assumed to be 0.33 mm.

B-107. Input Shorelines. The shoreline surveys used as input to the model were acquired from the available shoreline position database. The November 1990, October 1993, and August 1998 beach profile surveys were used for model calibration and verification. The model calibration period was November 1990 to October 1993. The verification simulation was for the period between October 1993 and August 1998. The Erosion Control Line (ECL) is assumed to represent pre-project initial conditions for all simulations. The ECL is the assumed pre-project shoreline for the project formulation in this analysis.

B-108. Calibration. The GENESIS model was calibrated for the period between November 1990 and October 1993. The project was completed in August 1989. Therefore, it is assumed that the November 1990 survey represents equilibrated conditions. Furthermore, during the calibration period a sufficient supply of sand was in the littoral system to realize the areas sand transport and shoreline change potential.

B-109. Transport coefficients K_1 and K_2 were set at 0.03. The calibrated *net* transport rate along the John U. Lloyd Beach State Park shoreline averaged about 42,000 cubic yards per year. A maximum *net* transport rate of about 45,000 cubic yards per year was realized about 2,500 feet south of the inlet. These rates fall within the accepted transport rates in the vicinity of Port Everglades. The Port Everglades Inlet Management Plan reports the *net* rate to be on the order of 44,000 cubic yards per year. The calibrated model also computed the northerly-directed transport through the inlet south jetty to be about 11,000 cubic yards per year. This is consistent with previously reported losses to the Port Everglades Inlet (OAI and CPE, 1998).

B-110. Initial shoreline position results from the calibration simulation are presented in Figure B-17. Both the predicted and actual shoreline locations are depicted in the upper figure. Also in the lower figure, the measured and predicted shoreline change from the initial position is shown. It is noted that the calibrated model predicted the highly erosional area within 3,000 feet of the inlet quite well. Between about 3,000 and 6,000 feet of the inlet, however, the model predicts the shoreline to be stable to accretional where measured shoreline data suggest erosion. The GENESIS shoreline change model is unable to strictly simulate offshore sand losses.

B-111. <u>Offshore Sand Losses.</u> The difference between the measured and predicted shorelines may be explained by the potential for offshore sand transport along this localized section of shoreline. Considering the agreement between the model results immediately north and south of this area and the configuration of the nearshore rock structure, it is believed that considerable sand losses may occur to the offshore area.

B-112. The mechanism for the offshore losses is suspected to be venting of sand through low areas, or gaps, in the nearshore rock structure. As with any irregular structure in the surf zone, return flow from the wave breaking induced run-up is concentrated through low areas in the surf zone bathymetry. Along an open coast, sandy shoreline, these low areas usually exist as run-outs through the nearshore bar and migrate along the coastline. At John U. Lloyd, the run-outs are fixed in the nearshore rock structure. The offshore-directed flow through these low areas jet beach sands to offshore areas reducing nourishing benefits to the downdrift shoreline.

B-113. The existence of the sand venting low areas along the John U. Lloyd shoreline is demonstrated graphically with a detailed contour model of the nearshore area. Of benefit to this exercise is a comprehensive LIDAR dataset of the nearshore data south of Port Everglades that was collected in 1997. This survey provides a high-resolution representation of bathymetric conditions in the area with individual elevation data points centered on about a one-foot spacing. The contoured LIDAR data are depicted in the lower portion of Figure B-18 along with the initial GENESIS calibration results in the upper portion of the figure. Inspection of this figure reveals a highly irregular bathymetric condition along the 6,000 feet of shoreline downdrift of the inlet. Several low areas in the rock structure are clearly evident in the figure between R-87 and R-91.

The most prominent of these features is situated in the vicinity of R-89. Of particular interest is the correlation between the location of the gaps in the rock and the area of disagreement between measured and predicted GENESIS calibration results. The GENESIS results do not consider offshore losses so it would be expected that if offshore losses actually occur, the model would predict less recession than that measured.

B-114. To quantify the amount of sand that may be lost to offshore venting, the sand-bypassing feature of the GENESIS model was used to remove sand from the model domain over the simulation period. The extent and rate of sand removal was determined by the magnitude of disagreement between measured and predicted results.

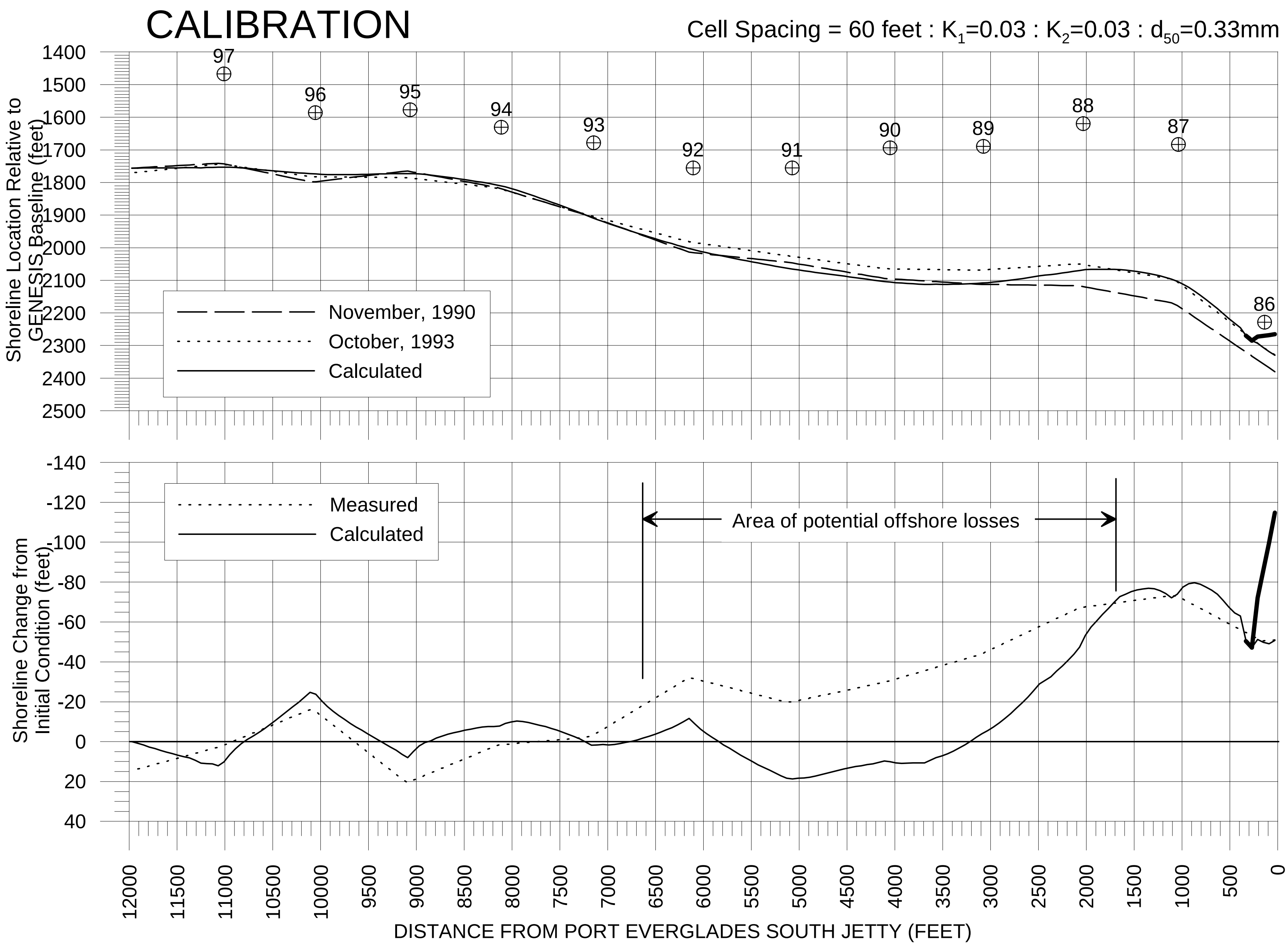

Figure B-17: John U. Lloyd GENESIS model calibration - initial results.

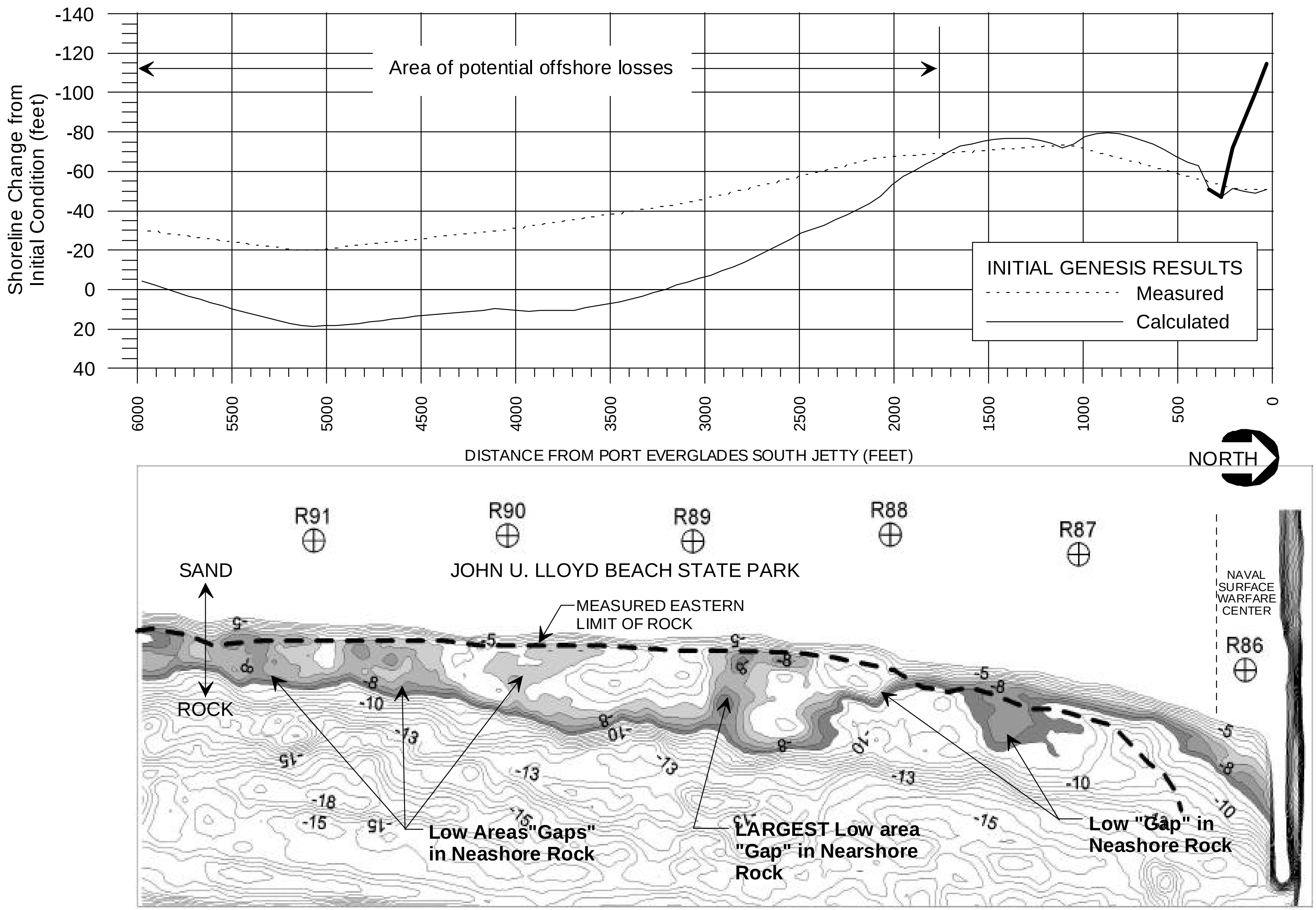

Figure B-18: Location of low areas in nearshore rock relative to potential offshore sand loss areas.

B-115. For the calibration period, it is estimated that approximately 25,000 cubic yards per year of sand are lost from the project shoreline to the offshore area. The predicted shoreline position using this technique is shown relative to measured condition in Figure B-19. The location of the sand removal area is also shown in the figure. When offshore sand losses are considered, the calibration results are greatly improved. It is noted that this *adhoc* modification to the model provides for the apparent sand transport potential along the entire project reach to be realized. Therefore, the model will not falsely predict accretion along a known erosional shoreline where proposed project modifications may have an influence. The use of this additional calibration procedure resulted in a calibration/verification factor of just under 5.0 feet for the entire 12,000 feet of shoreline. The total computed net volume change along the model reach averaged about 32,800 cubic yards per year.

B-116. <u>Verification.</u> A verification simulation was performed to test the model calibration. The verification simulation was for the period between October 1993 and August 1998. The results of the verification are present in Figure B-20. The agreement from the verification period is poor compared to the calibration results. Adjustments to the calibration coefficients, however, are not made due to the verification results.

B-117. The poor agreement between the measured and predicted shorelines is a product of the GENESIS model's inability to model various sediment sizes during a simulation. The input sediment size must be constant throughout the entire simulation. The model assumes there is an unlimited amount of sand of a given size available for unlimited transport if there are no seawalls present.

B-118. Interestingly, the shoreline along the northern reach of John U. Lloyd Beach State Park between 1993 and 1998 was in a highly eroded condition. Only a limited amount of sand was available along the northernmost reach. The sediment matrix along the northern end of John U. Lloyd consisted mostly of larger sands and shells and gravel to cobble sized stones that are not transported as easily by the normal wave climate as more typical beach sands. This material is rubble excavated from the offshore borrow areas during construction of the initial beach fill project that was not removed from the fill material prior to placement upon the beach. This rubble essentially armors the shoreline thus resulting in a lower than normal measured shoreline recession rate. Therefore, the over-prediction of shoreline recession by the GENESIS model is not surprising. Based upon the agreement achieved for the calibration period, where there was a sufficient supply of sand in the littoral system, it is assumed that the calibrated model accurately represents the shoreline change potential of beach fill along the shoreline south of Port Everglades.

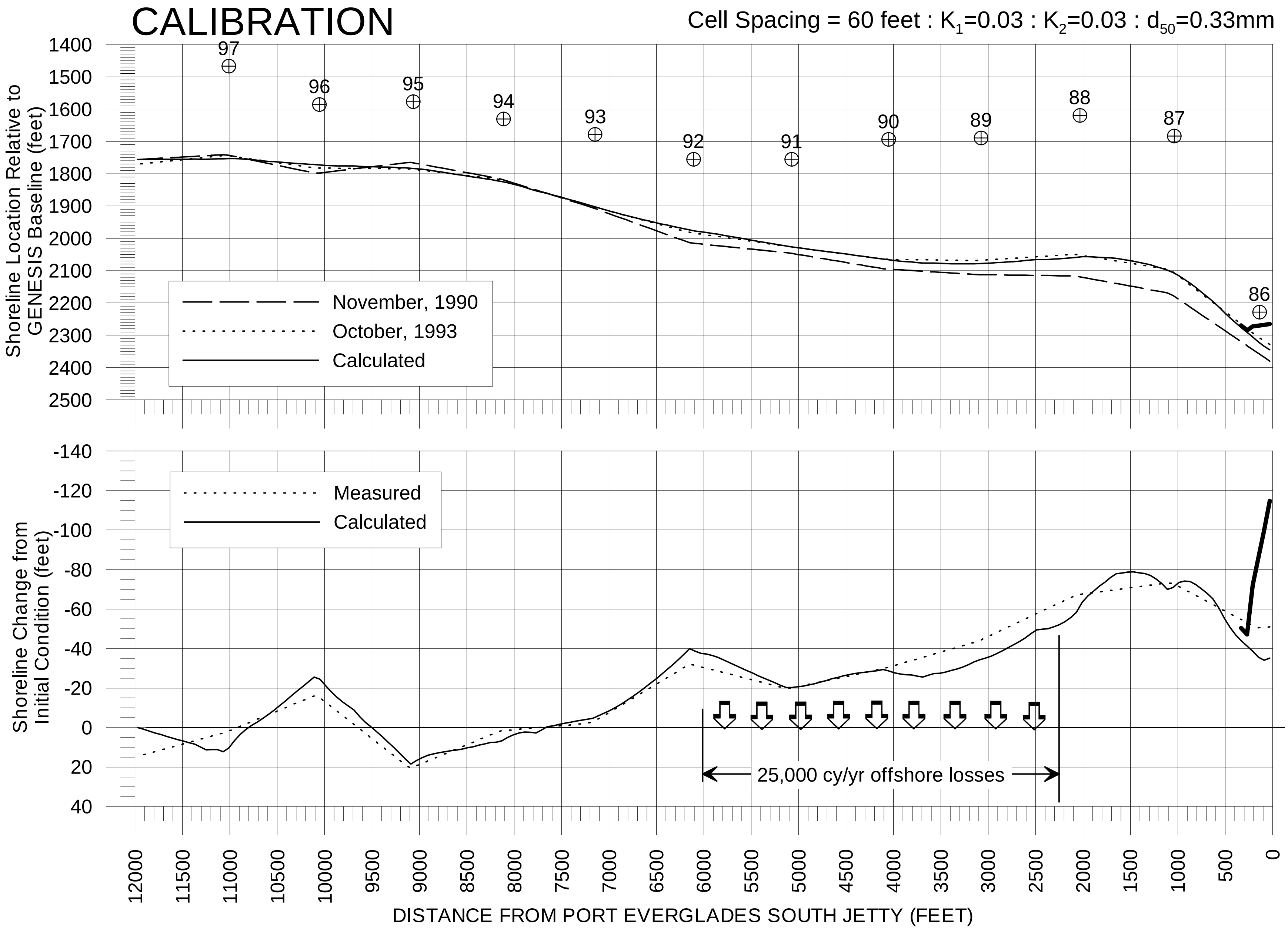

Figure B-19: John U. Lloyd GENESIS model calibration with consideration of offshore sand losses.

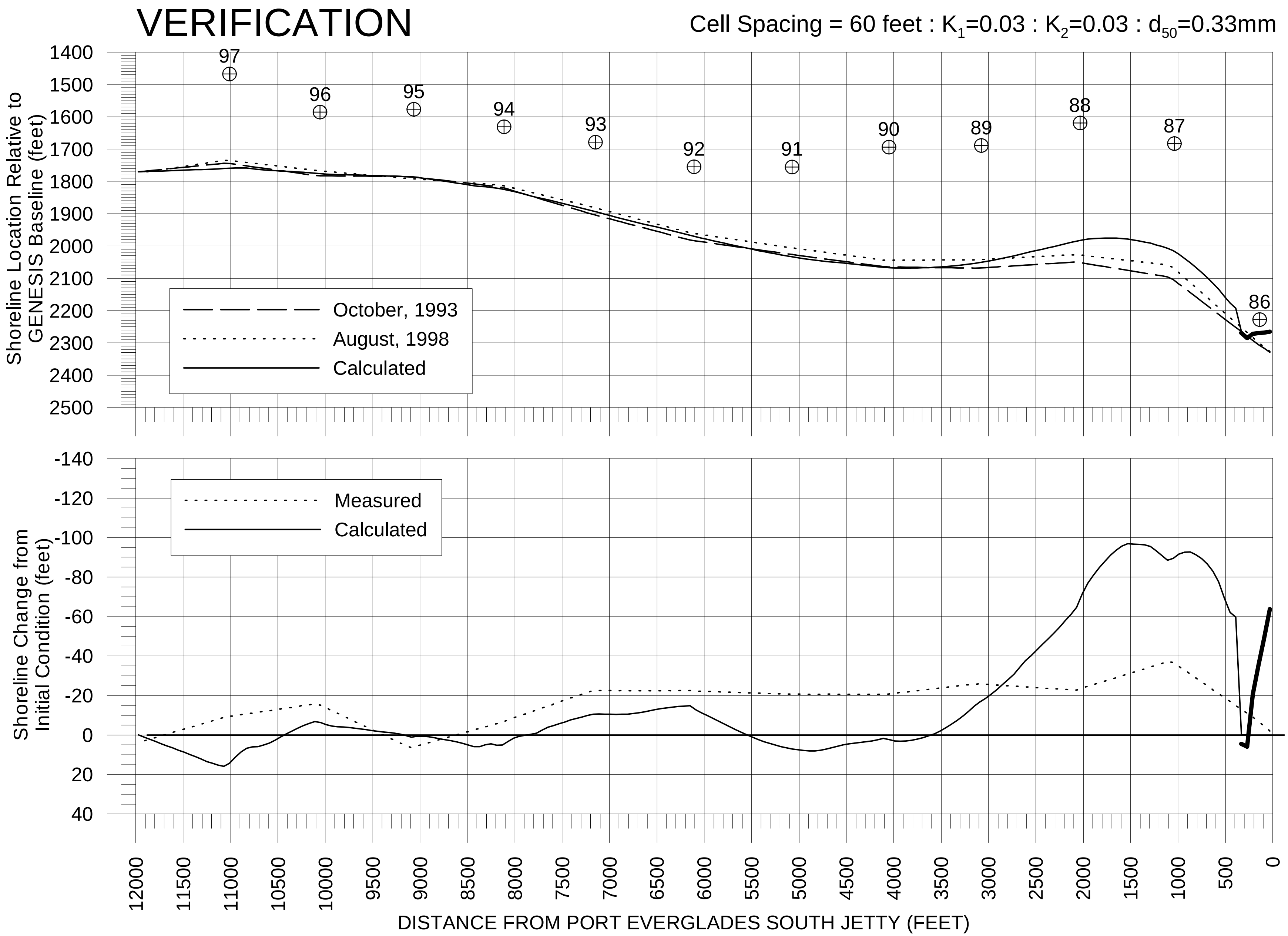

Figure B-20: John U. Lloyd GENESIS model verification.

Verification of Hollywood/Hallandale GENESIS Model

B-119. For completeness, the calibration parameters developed for the high-resolution model were verified with the low-resolution model. The low-resolution model verification was performed for the period between October 1993 and August 1998. This is the same verification period used for the high-resolution model. Of interest to the low-resolution model is the reach of shoreline from the southern end of John U. Lloyd to northern Hollywood.

B-120. Model Domain. The grid cell spacing for the Hollywood/Hallandale model was set at 200 feet. This provided for the 40,000 feet of shoreline to be represented in the 200-grid cell model. The northern boundary of the GENESIS grid corresponds approximately R-97which is some 11,000 feet south of the Port Everglades south jetty. The southern boundary was set about 5,000 feet south of the Broward-Dade County Line. The grid orientation was identical to that for the high-resolution model at 2 degrees east-of-north.

B-121. Physical Input Data. Physical input data for the model was taken from recent beach survey and geotechnical data. The average berm elevation was set at +7 feet, NGVD and the depth of closure was assumed to –12 feet, NGVD, on average, along the entire Segment III shoreline. The median grain size of the beach sediments was assumed to be 0.33 mm.

B-122. Verification. The results of the low-resolution model verification indicated that the calibration coefficients K_1 and K_2 at a 0.03 slightly under-predicted the magnitude of average sediment transport along the study shoreline compared to that computed for the John U. Lloyd model and that documented in the Inlet Management Plan. Therefore, for the low-resolution model, these parameters were modified to 0.07. The increase in the calibration coefficient values was necessary due to the increase in the depth of closure compared to the John U. Lloyd model. Both models produce average net transport rates of about 42,000 cubic yards per year.

Environmental Effects from Shoreline Erosion

B-123. The erosional stress and sediment deficit along the Segment III shoreline has resulted in chronic shoreline recession and dune loss. Shoreline and dune erosion reduces the dry beach area necessary for successful marine turtle nesting. The most notable area along the Segment III shoreline where the loss of beach has had an impact upon sea turtle nesting habitat is at the northern end of John U. Lloyd Beach State Park and the Naval Surface Warfare Center shoreline. As discussed in previous sections, this reach of shoreline is highly erosional. Historical shoreline and beach profile data indicate that when this reach of shoreline is in an eroded condition, the beach is characterized by minimal dry beach area and high steep bluffs along the back beach. Such beach conditions are problematic to sea turtle nesting and nesting success. Nests that are deposited along a section of shoreline in such a condition, if not relocated, are susceptible to disturbance from the erosion and inundation during periods of high tides. In the most sever instance, the beach conditions are such that turtles are unable to successfully deposit a nest resulting in a “false-crawl”.

B-124. There is evidence at John U. Lloyd that suggests that marine turtle nesting and nesting success is related to beach condition. For example, Table B-13 includes sea turtle nesting data along the northernmost 1,600 feet of the John U. Lloyd reach shoreline for the three years between 1999 and 2001. The data clearly indicate a continued reduction in sea turtle nesting along the reach of shoreline over the period. Inspection of the beach condition data that represents the same period clearly indicates continual degradation of the dry beach area. At present, there most of the beach section if inundated to the base of the bluff at high tide.

Table B-13: John U. Lloyd Beach State Park Sea Turtle Nesting Data.

From Jetty to 1,100 feet south (Jetty to ½-way between RR5 and RR6)		
	No. of Nests	Percent of Total Nests
2001	2	1
2000	12	4
1999	18	9
From Jetty to 1,600 feet south (Jetty to RR5)		
	No. of Nests	Percent of Total Nests
2001	7	3
2000	21	7
1999	33	16

Problem Summary

B-125. Based upon field inspections, historical hydrographic and topographic survey data, performance monitoring of past beach fills, an updated sediment budget analysis of Port Everglades, and the calibrated and verified GENESIS model, it is determined that the authorized (previously constructed) reaches of the Segment III shoreline require additional sand nourishment. The areas include the northern reach of John U. Lloyd Beach State Park and the entire Hollywood/Hallandale shoreline. Both of these areas have been nourished twice previously. The analyses also indicate that there are localized areas of these past projects that have performed poorly due to higher than average erosional stress. These areas include the 2,800 feet of shoreline immediately south of the Port Everglades and the southern terminus of the John U. Lloyd and northern terminus of the Hollywood/Hallandale beach fills. The former routinely experiences erosion rates exceeding 30 ft/yr with maximum recession rates approaching 50 ft/yr. Along the latter areas, past beach fill projects have been impacted by high shoreline recession rates due to end loss effects. The high erosional stress and resultant dry beach losses also affect the quality of marine turtle nesting habitat. Implementation of the authorized project with modifications is proposed to address the identified problems.

PROTECTIVE BEACH DESIGN AND COSTS

B-126. This section addresses the dimensions and costs for (1) the reevaluation of the authorized (previously constructed) project (2) the implementation of the reevaluated authorized project with modifications. The reevaluation of the authorized project is based upon the physical and economic conditions for the entire 50-year project life beginning in 1976. The implementation of the reevaluated project with modifications is based upon 1998/2001 physical and economic conditions. The National Economic Development (NED) plan is formulated from the reevaluation of the authorized project.

B-127. The proposed project modifications include a reduced design section at John U. Lloyd and beach fill tapers at the northern and southern ends of the Hollywood/ Hallandale fill. The beach fill tapers are engineering modifications intended to reduce end losses from the design section and increase the project renourishment interval. A groin field is also proposed along the northern end of the John U. Lloyd reach. This project modification is intended to improve shoreline stability along the highly erosional shoreline immediately downdrift of Port Everglades, thus reducing the required project's advance nourishment volume and average annual cost.

B-128. The benefit of mechanical sand bypassing at Port Everglades to the Segment III Shore Protection Project was also investigated. The purpose of this evaluation was to demonstrate the physical and economical benefits of sand bypassing to the Segment III shoreline and Federal shore protection project.

Reevaluation of the Authorized Federal Project (NED Plan)

B-129. Project Length. The authorized Federal project in Segment III includes two reaches of shoreline between Port Everglades and the Broward/Dade County Line. These include the 8,100 feet of shoreline for the Port Everglades south jetty to about R-94 and the 27,500 feet of shoreline from about R-101 to the Broward/Dade County Line (R-128). The north terminus of the fill will abut the south jetty structure. A full design section will be constructed and maintained to the Broward/Dade County line.

B-130. Berm Elevations. The design berm elevation varies along the Segment III project shoreline to approximate the natural berm elevation along the existing beach. Along the John U. Lloyd Beach State Park shoreline between the south jetty of Port Everglades and R-94, the design berm elevation is +10 feet NGVD. The design berm elevation for the Hollywood/Hallandale shoreline reach is +7 ft NGVD.

B-131. Berm Widths. Various design beach widths were considered for purposes of reevaluation the dimensions of the authorized project. The design berm widths of the beach fill project along both the John U. Lloyd and Hollywood/Hallandale shoreline reach were varied between 25 and 75 feet. These berm widths are defined as a seaward translation of the pre-project mean high water line.

B-132. Beach Slopes. The beach profile shape varies along the entire Segment III shoreline. The typical profile shape along the Segment can be described with equivalent slopes. Design beach slopes along the northern John U. Lloyd shoreline reach are generally equivalent to 1:10 and 1:30 above and below the mean low water elevation, respectively. Along the Hollywood/Hallandale shoreline reach, the design beach slopes are 1:10 and 1:45 above and below the mean low water elevation, respectively. These beach slopes are generally equivalent to the trend of the beach profile shape above and below the mean low water line.

B-133. Design Fill Volume. The design beach volume is that portion of the beach fill that provides the permanent storm damage and recreation benefits to the project area. The design volume for each alternative was determined using the design berm width, elevation and translated profile. For the purposes of this formulation, profiles from the August 2001 survey are assumed to represent typical beach conditions and were used in the profile translation. Along those areas were beach conditions are severely over-eroded (i.e., northern John U. Lloyd and northern Hollywood) a beach profile shape was derived from measured beach profiles were sufficient sand resources are available to represent healthy profile conditions.

B-134. The optimum design beach volume is that which maximizes net primary benefits for variations in berm width. To reevaluate the authorized project dimensions the design beach volumes for mean high water shoreline extension of 25 to 75 ft were computed water extensions were developed assuming pre-construction shoreline conditions. The design beach volume and estimated average annual cost associated with each of these berm widths is included in Table B-14. Details of the cost estimates are included in Sub-Appendix B-2.

B-135. Advance Nourishment Volume and Renourishment Interval. A sacrificial volume of fill material, termed "advance nourishment" will be placed in addition to the design beach volume to offset erosion anticipated after the project's construction. The volumetric requirement for the advance nourishment is determined by historical ("background") volume loss rates along the project area, end losses associated with the project itself, and the renourishment interval.

B-136. The historical volume loss rate is based on beach profile changes measured between 1989 and 1998 and the results of the sediment budget developed for Port Everglades. The average annual beach volume change rate along the two reaches of the authorized Segment III project shoreline is 130,000 cy/yr. This volume change includes 53,000 cubic yards per year of erosion along the northern 8,100 feet of John U. Lloyd and 77,000 cubic yards of erosion along Hollywood/Hallandale.

Table B-14: Project dimensions and costs for reevaluation of authorized project.

Design Berm Width (feet)	Design Beach Volume (cubic yards)
25	892,090
50	1,381,660
75	1,907,800

B-137. The project renourishment interval is the number of elapsed years between programmed replacements of the advance nourishment volumes. The optimum renourishment interval is defined as that which minimizes the average annual equivalent cost of project implementation. Table B-15 presents the average annual equivalent project costs for a 50-ft design section and renourishment intervals from 5 to 7 years. Average annual equivalent costs were computed using a 6 and 1/8 percent interest rate and a 50-year project life. Considering the placement of advance nourishment along the entire project shoreline, the most cost effective renourishment interval is six years. The details of each of the project cost estimates outlined in Table B-14 are included in Sub-Appendix B-2.

Table B-15: Renourishment interval optimization for the Segment III reevaluated project cost.

Renourishment Interval (years)	Average Annual Cost		
	25-ft Design Berm	50-ft Design Berm	75-ft Design Berm
5	$2,710,000	$3,169,000	$3,854,000
6	$2,692,000	$3,151,000	$3,835,000
7	$2,834,000	$3,293,000	$3,977,000

B-138. <u>Future Renourishment Volume.</u> After construction of the initial project, performance monitoring of the placed material will be conducted to determine with greater accuracy the future periodic renourishment requirements. For the purposes of this report, it is considered that the future periodic renourishment volume is the same as the advance nourishment volume.

B-139. <u>Overfill Volume.</u> The overfill volume is the additional quantity of material necessary to allow for the textural differences between the native beach and borrow area material. The overfill volume is determined by multiplying the overfill ratio by the required advance and future nourishment volumes. The overfill ratio is only applied to the nourishment volumes because the design beach will theoretically never be exposed to the sorting action of nearshore waves and currents.

B-140. Since past projects along the Segment III shoreline have been constructed from numerous borrow areas and the placement locations of those various sediments are not known exactly, it is difficult to estimate an overfill ratio. For comparative purposes, the overfill ratio is the same for all project considered in the project formulation. Only the volume of the design beach varies to which the overfill ratio is not applied. Therefore, an overfill volume is not applied in this analysis.

B-141. Hardbottom Coverage. The hardbottom coverage is considered in the reevaluation of the authorized project. Estimates of hardbottom impacts are based upon the 1999 location of the hardbottom limit and a profile translation technique. The local depth of closure for each measured beach profile was also considered in estimate the approximate seaward extent of the equilibrium toe-of-fill.

B-142. Project Costs. It is estimated that the unit cost for sand for the initial construction in 1980 was $6.62 per cubic yard. This is based upon estimated costs assuming that previously used sand resources immediately offshore of Segment III are available. For the purposes of comparison, a mobilization cost of $1,000,000 is assumed for all alternatives. It is assumed that the cost of nearshore hardbottom mitigation is $300,000 per acre. This value is based upon the estimated cost to construct limestone boulder mitigation in the nearshore region.

B-143. Costs for project engineering and design, construction administration, maintenance, and project monitoring are estimated as a percentage of contract costs. A contingency of 15 percent is included for all costs estimates.

B-144. Summary. Consideration of project benefits in Appendix D indicates that the 50-ft design berm maximizes the net primary project benefits. Therefore, the 50-ft design beach section with a requirement for renourishment every six years is the NED plan. The economics of implementing the NED for the remainder of the project life are developed in the following section.

Implementation of the Reevaluated (NED) Plan

B-145. Based upon economic considerations, an ECL (pre-project shoreline) extension of 50 feet was found to provide the maximum net primary project benefits along the entire Segment III shoreline. Implementation of this plan will require replacement of portions of the design section and advance nourishment along the entire Segment III shoreline.

Evaluation of John U. Lloyd as Separable Element

B-146. It is noted that the density of shorefront development along Segment III is highly variable. The densest and most valuable shorefront development in Segment III is in Hollywood and Hallandale. Thus, these shoreline reaches generate most of the Segment III storm damage reduction benefits for the Segment III. Since Segment III was initially constructed as a continuous segment, the reevaluation treated the project as such. Thus, the John U. Lloyd reach was not evaluated as a separable element. For the purposes of implementation, however, an additional analysis was conducted to confirm that the John U. Lloyd reach is justified as a separable project element. This analysis included consideration of the separable costs and benefits of the John U. Lloyd reach.

B-147. There is a relatively small amount of development along the John U. Lloyd project reach. The most notable development at that location is infrastructure associated with the Naval Surface Warfare Facility immediately downdrift of the Port Everglades south jetty. There are also scattered structures and other infrastructure associated with John U. Lloyd Beach State Park and Nova University. The John U. Lloyd project output includes storm damage reduction, recreation, and environmental enhancement and preservation. The latter two outputs are considered incidental.

B-148. The separable element evaluation for John U. Lloyd included consideration of three project alternatives. These are the 50-ft design berm as identified in the Segment III reevaluation, a 25-ft design berm, and a 0-ft design berm. The latter is essentially the periodic nourishment alternative where the pre-project shoreline is reestablished and maintained. The design berm would be situated along the previously constructed section of the John U. Lloyd reach between the south jetty and R-94. Six years of advance fill is applied to each alternative. Advance fill is distributed according to historical erosion patterns and predicted sand loss rates. An allowance for overfill is also included. The overfill volumes were developed from the sediment compatibility analysis discussed below. A design berm wider than 50-ft is not considered due to the increased nearshore hardbottom impacts that would be associated with a wider berm. It is noted that reestablishment and maintenance of a 50-ft design berm along John U. Lloyd would impact approximately 10 acres of nearshore hardbottom based upon 2001 conditions.

B-149. Project costs were formulated according to global unit cost estimates developed for the reevaluation of the Segment III project. The unit cost of sand is assumed to vary from $9.79 per cubic yard for the proposed renourishment activity to $15.00 per cubic yard for future nourishment activities where sand sources may be located at more distance areas than existing sources. A separable mobilization cost of $250,000 is assumed to provide for the establishment of sand handling equipment at the John U. Lloyd project area. It is assumed that since John U. Lloyd is an integral element to Segment III and it is planned that this project reach will be constructed coincident with the Hollywood/ Hallandale, only the incremental increase in project costs associated with the incremental mobilization and the sand placement is considered.

B-150. Table B-16 summarizes the sand volumes and average annual cost to implement the separable John U. Lloyd alternatives project. The average annual project costs are based upon a 6 and 1/8 percent interest rate for the remaining 24 years of the project life. The details of the cost formulation are included in Sub-appendix B-3.

Table B-16: Summary of JUL reach alternative sand volumes and costs.

	Project Extension		
	0-ft	25-ft	50-ft
JUL Reach Volumes (cy)	483,000	624,000	697,000
JUL Hardbottom Impacts (acres)	5.0	8.5	10.0
JUL Reach Average Annual Costs	$1,410,000	$1,735,000	$1,895,000

B-151. As discussed in Appendix D, there are sufficient storm damage reduction benefits along the John U. Lloyd reach to justify sand placement at that location as a separable Segment III project element. However, reestablishment and maintenance of the 50-ft NED design berm at John U. Lloyd does not maximize the separable net primary benefits along that reach. Instead, reestablishment of pre-project shoreline conditions and periodic nourishment sufficient to maintain the pre-project shoreline produces the maximum net primary benefits. Therefore, the John U. Lloyd project will only include the reestablishment of the pre-project shoreline and the placement of periodic nourishment.

B-152. It is noted that this project configuration significantly reduces the potential nearshore hardbottom impacts along the John U. Lloyd shoreline. There are, however, approximately 5 acres of unavoidable nearshore hardbottom impacts associated with the periodic nourishment plan. The configuration and performance of the John U. Lloyd project along with additional Segment III modifications are detailed in following discussion.

Plan Implementation

B-153. <u>Design Fill Volume.</u> The design beach volume required to implement the reevaluated plan without the 50-ft design beach section at John U. Lloyd in 2002 is estimated to be approximately 576,600 cubic yards. The design volume was determined using the design berm widths, elevation and translated profile represented by August 1998 beach conditions. This volume is inclusive of the volume of fill behind the Erosion Control Line (ECL).

B-154. <u>Advance Nourishment Volume and Renourishment Interval.</u> The volume of advance fill required to implement the reevaluated plan is based on beach profile changes measured between 1989 and 1998 and the results of the sediment budget developed for Port Everglades. The average annual beach volume change rate along the two reaches of the authorized Segment III project shoreline is 130,000 cy/yr.

B-155. The optimal renourishment interval for the remaining project life is reevaluated to minimized project costs. As before, the optimal renourishment interval is determined by comparison of average annual costs of various interval periods. In this analysis, renourishment interval is 5, 6 and 7 years were considered. The total average annual cost of each of these alternatives is included in Table B-17. The details of the cost comparisons are included in Sub-Appendix B-4.

B-156. To accommodate expected sand losses over the six-year renourishment cycle 780,000 cubic yards of sand will be placed as advance fill. This does not include volumes required for overfill and endlosses.

Table B-17: Re-optimization of renourishment interval for plan implementation.

Renourishment Interval (years)	Average Annual Cost
5	$4,680,000
6	$4,471,000
7	$4,692,000

B-157. <u>Future Renourishment Volume.</u> After construction of the 2002 project, performance monitoring of the placed material will be conducted to determine with greater accuracy the future periodic renourishment requirements. For the purposes of this report, it is considered that the future periodic renourishment volume is the same as the advance nourishment volume. The future renourishment volume required from offsite sand sources would be greatly reduced if sand bypassing is implemented at Port Everglades.

B-158. Overfill Volume. A sediment compatibility analysis was conducted for each borrow area and the existing beach material to evaluate potential overfill requirements. The composite grain size distributions were used to represent the potential offshore borrow areas (see Appendix E). Appendix E identifies seven of borrow areas that can be utilized for this project, though only Borrow Areas II, III, IV, and VI will be considered for use in Segment III because of the proximity of the borrow areas to the project Segment and compatibility.

B-159. For this study, a modified equilibrium method was used to formulate overfill ratios for each of the borrow areas (Munez-Perez, et al, 1999). The original equilibrium method of Dean (1991) employs a shape factor that is a function of mean grain size. This method does not, however, take into account the effects of nearshore hardbottom or reef features upon beach profile shape. The modified equilibrium method uses a shape factor that is a function of grain size, depth of hardbottom, and the cross-shore width of the hardbottom. The estimated overfill volumes are shown in Table B-18. Borrow Areas III and VI are fully compatible with the Segment III beaches. Borrow Areas II and IV require an overfill density of 1.22 cubic yard per linear foot of beach and 1.25 cy per linear foot of beach along the Segment III shoreline, respectively.

Table B-18: Estimated overfill ratios for Segment III.

Borrow Area		John U. Lloyd	Hollywood/ Hallandale
Number	Grain Size, d_{50} (mm)	0.33 mm	0.34 mm
II	0.28	1.22	1.25
III	0.34	1.00	1.00
IV	0.28	1.22	1.25
VI	0.38	1.00	1.00

B-160. An overfill allowance is only added to the advance fill volume as this is the portion of the project that is provided as a transportable volume of sand. It is estimated that the maximum advance fill volume for the Segment III project will be 780,000 cubic yards. It is not known, however, how the material from the borrow areas will be distributed along the Segment III shoreline as the project is constructed. Because of this and the fact the overfill ratios vary between the borrow areas, it is assumed that the beach fill material will be placed uniformly along the Segment III shoreline from all of the borrow areas according to the distribution of the borrow areas volumes. That is, every foot of shoreline in Segment III will have a fraction of sand from each of the five borrow areas. Although this assumption is probably not realistic due to construction limitations, it is proposed in an attempt to formulate a meaningful overfill volume.

B-161. The distribution of sand volumes available in each of the four borrow areas is summarized in Table B-19. Of the sand volume proposed for the Segment III shoreline, about 55.5 percent of the fill will be derived from borrow areas II and IV for which an overfill allowance is required. Applying the assumption proposed in the preceding paragraph allows an "effective" overfill ratio for the entire Segment III project to be computed through a weighted averaging technique. According to the results presented in Table B-19, 108,000 cubic yards of sand are required to be added to the advance fill volume of the project to accommodate the textural differences found between the native Segment III beach material and the sediments in borrow areas II and IV. This equates to an overall overfill ratio of about 1.14.

Table B-19: Computation of overfill for Segment III shoreline.

Borrow Area	Borrow Area Volume Distribution Available for Segment III	John U. Lloyd			Hollywood/Hallandale		
		Base Advance Fill Volume (cy)	Overfill Factor	Adjusted Volume (cy)	Base Advance Fill Volume (cy)	Overfill Factor	Adjusted Volume (cy)
II	49.9%	158,800	1.22	198,400	230,700	1.25	288,300
III	37.3%	118,800	1.00	118,800	172,500	1.00	172,500
IV	5.6%	17,800	1.22	22,200	25,900	1.25	32,300
VI	7.1%	22,600	1.00	22,600	32,900	1.00	32,900
Total	100.0%	318,000		362,000	462,000		526,000

B-162. <u>End Loss Reduction - Beach Fill Tapers/Transitions.</u> The previously constructed beach fills along John U. Lloyd and Hollywood/ Hallandale experienced high sand loss rates at the terminal points of the fill in southern John U. Lloyd and northern Hollywood. End losses were particularly prominent during the first year after construction and are largely attributable to planform equilibration. The currently authorized project does not specifically include a project element that addresses end losses for the terminal ends of the fill sections. Considering documented high, end loss rates from previously constructed projects, beach fill tapers and transitions will be added to the authorized project to decrease end losses. Beach fill tapers will be incorporated into the design at the northern end of the Hollywood/Hallandale project reach while the fill will be to the transitioned to the adjacent shorelines at the southern ends of John U. Lloyd and Hollywood/Hallandale. It is noted that a taper is defined a fill transition extends beyond the design reach and requires additional fill material to meet performance requirements. A transition, on the other hand, includes the tapering of advance fill along areas of decreasing transport potential or advantageous changes in shoreline orientation.

B-163. The terminal ends of the authorized Segment III beach fill reaches have been generally located at R-94 for the southern end of the John U. Lloyd reach, R-101 for the northern end of the Hollywood/Hallandale reach, and R-128 for the southern end of the Hollywood/Hallandale reach. Following construction of the most recent beach fills along these areas, the shoreline position at R-93 retreated about 60 feet during the first year following project construction, and retreated about 5.4 ft/yr over the next ten years. At R-101, shoreline receded nearly 100 feet during the first year following construction and averaged about 20 ft/yr of recession between 1991 and 1998. In both instances the design beach section was impacted by erosion within 2 years following project construction. The intended renourishment interval was eight years.

B-165. <u>Southern End of John U. Lloyd.</u> The elimination of the design section at John U. Lloyd and the orientation of the shoreline along central John U. Lloyd minimizes the need for a formal taper at the southern end of that project reach. In this instance, the advance fill will simply be transitioned to the natural alignment of the downdrift shoreline at a point of decreased shoreline erosion potential (approx. R-92).

B-166. <u>Hollywood/Hallandale.</u> To evaluate and optimize beach fill transitions necessary to maintain the design beach section along the Hollywood/Hallandale shoreline, the calibrated and verified low-resolution GENESIS model was employed. The simulations were executed for a six-year period assuming all cross-shore equilibration was complete at the initiation of the simulation. Advance fill was added to the model based on the previously determined demands of each reach. The taper and transitions were evaluated based upon their ability to maintain the design beach while minimizing the volume of sand used in initial construction. At the northern and southern ends of the Hollywood/ Hallandale reach, tapers and transitions alone would not meet the requirement of maintaining the design section through the proposed nourishment interval. Therefore, a limited volume of sand was bulged at the terminal ends of the fill along with the tapers to ensure the performance criteria were met. The volume of the bulges were added to the estimated tapers volumes and reported with the total fill volume requires to address end losses.

B-167. <u>Northern End of Hollywood/Hallandale.</u> At the northern end of the Hollywood/Hallandale beach fill project, the optimum taper configuration included approximately 117,000 cubic yards of fill and extends approximately 2,000 feet north of the design beach. This would result in sand placement along about 70 percent of the Dania Beach shoreline. This terminal fill area is the most problematic of all those along the Segment III project shoreline. Taper configurations of 1,000 feet, 1,500 feet, and 2,000 feet were considered in the analysis. As indicated by the predicted results depicted in Figure B-21, a taper of at least 2,000 feet in length with some bulge will be required to maintain the design beach section for six years. Due to environmental considerations and the predicted adequate performance of the 2,000 ft taper, larger taper configurations were not considered. The expected area of hardbottom coverage with the tapers and additional sand at north Hollywood is estimated to be about 1.5 acres.

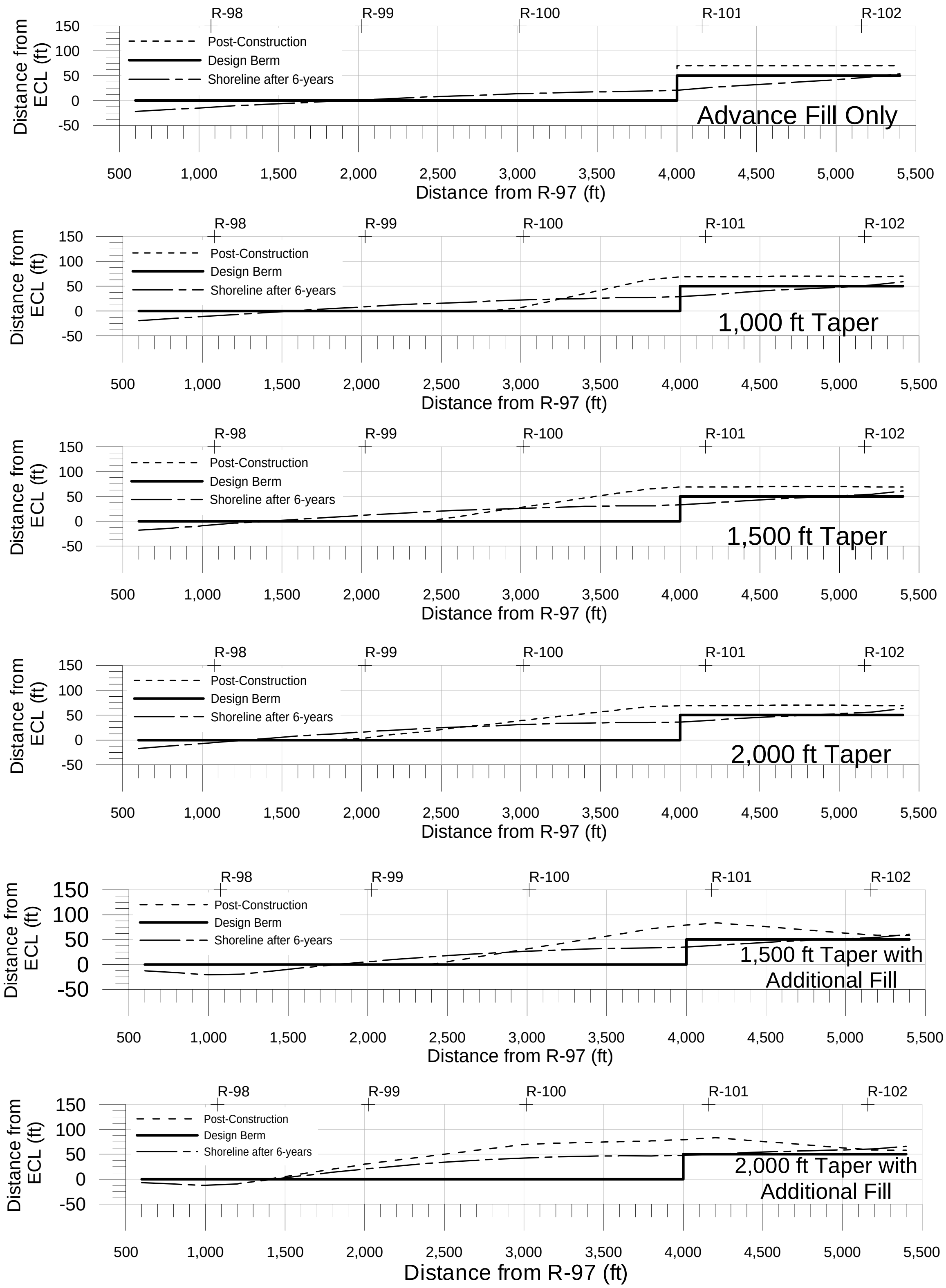

Figure B-21: Predicted performance of taper alternatives at the northern end of the Hollywood/Hallandale beach fill.

B-168. <u>Southern End of Hollywood/Hallandale.</u> The southern end of the project will be situated at the Broward/Miami-Dade County line. Terminal end fill losses at the southern end of the Hollywood/Hallandale beach fill will be addressed with the advance fill. Due to the natural curvature of the Dade County shoreline immediately south of the project area and the recent advance of that shoreline due to past Broward County and Sunny Isles (Dade County) beach fills, the terminal end of the fill will be exposed to reduced transport potential. GENESIS model predictions indicate that the advance fill tapered and terminated at the County line will maintain the require design beach over the renourishment interval. Some additional material will be added to the advance fill along the southernmost 1,500 feet of the southern end of the project to benefit the terminal end performance. The GENESIS results of the terminal end evaluation are depicted in Figure B-22.

B-169. The results of this analysis demonstrate the limited effectiveness of a beach fill without engineered tapers and transitions. As expected, end losses from a beach fill without tapers are predicted to be extremely high immediately following construction. As a result, the design beach section is impacted by localized shoreline retreat within the first or second year following construction.

B-170. In all, 137,300 cubic yards of sand will be required to address the anticipated end losses at the northern and southern ends of Hollywood/Hallandale. This sand volume is added to the total sand requirement to implement the optimal re-evaluated plan.

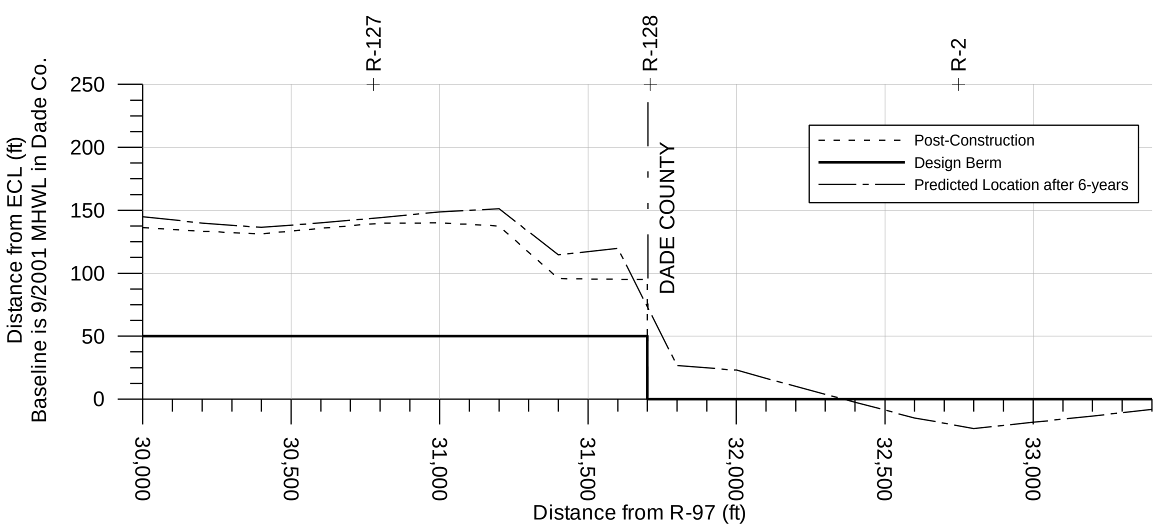

Figure B-22: Predicted performance of southern terminal end of the Hollywood/Hallandale beach fill.

B-171. Hardbottom Coverage. It is estimated that approximately 7.56 acres of nearshore hardbottom will be impacted by the placement of sand associated with the implementation of the NED plan. Estimates of hardbottom impacts are based upon the 2001 location of the hardbottom limit and a profile translation technique. The local depth of closure for each measured beach profile was also considered in estimate the approximate seaward extent of the equilibrium toe-of-fill.

B-172. Project Costs. Project costs required to implement the reevaluated authorized project were formulated using a percent rate of 6 and 1/8 for the remaining 24 years of the project life.

B-173. It is estimated that the unit cost for sand for the 2002 construction will be $9.79 per cubic yard. This cost estimate was developed by the Jacksonville District Cost Engineering Branch. The beach nourishment costs include $1,000,000 for mobilization and demobilization and $9.79 per cubic yard for material dredging. These costs were developed assuming a medium size hopper-dredge with rock separation capability, a 15 mile one-way steaming distance between the borrow areas, rock disposal area, and the beach, nearshore sand pumpout facility, and a pipeline booster. The locations of the proposed borrow areas relative to the project shoreline are shown in Appendix E. Results of the hopper-dredge estimate are presented at the end of this appendix.

B-174. It is noted that following the 2002 project, most cost effective sand resources offshore of Broward County will be depleted. Future sand resources for Segment III nourishments will have to be imported from distant domestic offshore sites (i.e., Palm Beach or Martin Counties), foreign sites (The Bahamas or other Caribbean nations), and /or upland sites. Future sand will, therefore, be more expensive than the current identified sources. For the purposes of this investigation, it is assumed that future sand placed along the Segment III shoreline will cost up to $15.00 per cubic yard.

B-175. The cost of nearshore hardbottom mitigation is $300,000 per acre. This estimated is based upon actual cost of similar nearshore hardbottom mitigation in south Florida.

B-176. Cost estimates for monitoring were provided by the Broward County, Florida Department of Planning and Environmental Protection. Engineering, design, supervision and administration were based upon contract amounts agreed upon by Broward County and the joint-venture consulting engineer team.

B-177. The total average annual cost to implement the reevaluated plan for the remaining 24 years of the project life cycle without modifications is $4,471,000. The details of the cost estimate for this plan are included in Sub-Appendix B-4.

Modifications to the Reevaluated Project

B-178. Modifications are proposed to the reevaluated project, to be implemented during the 2002 construction that would improve project performance and reduce project costs. The justification, dimensions, and benefits of these modifications are discussed in the following paragraphs.

Fill Dania Gap (R-94 through R-101)

B-179. The previously constructed beach fills along John U. Lloyd and Hollywood/ Hallandale experienced high sand loss rates at the terminal points of the fill in south John U. Lloyd and north Hollywood. End losses were particularly prominent during the first year after construction and are largely attributable to dramatic planform equilibration caused by inadequate fill transitions. The currently authorized project does not specifically include a project element that addresses the terminal ends of the fill sections. Beach fill tapers, however, have been added to the reevaluated plan as engineering features for purposes of reducing the effects of fill end losses.

B-180. An alternative method by which to reduce endlosses from the southern end of the John U. Lloyd project reach and the northern end of the Hollywood/Hallandale project reach would be to construct a continuous design section between the two projects, thereby eliminated the terminal ends of those project reaches. This would consist of placing a design section between R-94 and R-101. Considering that the optimum design berm width along the adjacent reaches that varies between 0 ft at John U. Lloyd and 50 feet at the northern end of Hollywood, a design section tapered between 0 and 50-ft between R-94 and R-101 is considered. Alternate berm configurations would require complicated transitions and would not be cost effective or environmentally acceptable to implement.

B-181. Creation of a design section along this reach of shoreline would potentially produce additional storm damage reduction, loss of land, and recreational benefits for the project. Likewise, the addition of this project reach would increase the overall average annual project costs. To evaluate the economic efficiency of this proposed project modification, the incremental primary benefits and costs over the remaining 24-years of the project life are compared. If the incremental primary benefits are greater than the incremental project costs, then the modification would be economically feasible. The average annual project costs and benefits used to evaluate modifications to the reevaluated NED plan are based upon a percent rate of 6 and 1/8 for the remaining 24 years of the project life.

B-182. The incremental additional sand volume required to construct the design beach section with advance nourishment would be approximately 360,000 cubic yards. This sand volume is a combination of the design beach, advance nourishment, and overfill. It would be expected that shoreline change would be similar to pre-project conditions. That is, the feeding effects due to the perturbations of beach fill along the adjacent shorelines would be eliminated. Therefore, pre-project loss rates were used to estimate advance fill requirements. The overfill volume was developed from the sediment compatibility described above. It is estimated that a fill of these dimensions would cover about 13 acres of nearshore hardbottom in southern John U. Lloyd and Dania Beach areas.

B-183. Project Costs. The total average annual cost to implement the reevaluated plan with a fill section between R-94 and R-101 is $5,206,000. This results in an incremental increase in average annual project costs over implementation of the reevaluated NED plan of $735,000. The details of this cost estimate are included in Sub-appendix B-5.

B-184. Economic Note. As discussed in Appendix D, constructing and maintaining a full design section does not generate incremental storm damage prevention benefits that equate to at least 50 percent of the incremental costs. It is more cost effective and less impactive to nearshore hardbottom to construct the beach fill with at transition at John U. Lloyd and a taper at the northern end of Hollywood. The Dania shoreline will receive an added beach width due to the construction of the beach fill tapers and will be maintained through sand losses from the adjacent projects.

Groins

B-185. Modifications to the Segment III authorized project are also proposed for the northernmost shoreline along John U. Lloyd (JUL) Beach State Park. Following both the 1977 and 1989 beach fills along this reach of shoreline, recession rates along the northernmost 2,800 feet of the project have consistently exceeded 30 ft/yr. Locally, maximum shoreline recession rates have exceeded 50 ft/yr. Measured shoreline change rates associated with the 1989 beach fill at JUL are shown in Figure B-24.

B-186. To date, only advance fill has been placed in attempt to offset the erosion rate immediate to this area.   Advance fill volumes placed during the projects, however, have not provided long-term protection of the design beach section at that location. In fact, the design section along the northern 2,800 feet of the John U. Lloyd shoreline has been impacted by shoreline recession within the first two years following construction of both the 1977 and 1989 projects.

B-187. In addition to advance fill, a measure to reduce the sand loss rate from the northern John U. Lloyd shoreline included sand tightening the south jetty as part of the 1989 renourishment project. Although the jetty sand-tightening most likely reduced the sand loss rate to the inlet, the shoreline immediately downdrift of the inlet continued to erode more or less at historical rates. This may suggest that the sand loss rates to the inlet were relatively low compared to alongshore and offshore sand losses prior to the sand-tightening project.

B-188. The extent of the most highly erosional shoreline is consistent with the acceleration of southerly alongshore sand transport potential immediately downdrift of Port Everglades. The uncalibrated north, south, and net alongshore sand transport potential is presented in Figure B-23. This curve was developed from a weighted averaged of the alongshore sand transport potential computed for each wave condition simulated in the refraction/diffraction analysis. The uncalibrated CERC longshore sand transport (LST) equation was used to formulate the transport potential patterns.

B-189. The extent of the highest measured shoreline erosion and the limits of the steepest gradient in the alongshore sand transport is also evident in the residual shoreline configuration following recession of the most recent JUL beach fill project. Inspection of the aerial photograph also included in Figure B-24 reveals an unusual curvature in the 1998 shoreline between the jetty and R-89. This curvature is the result of the extreme erosional stress produced by the steep transport gradient. The agreement between the limits of this shoreline curvature, the extent of the steepest gradient in sand transport potential, and the highest measured erosional signal from the 1989 project is striking and supports a high confidence in the understanding of the shoreline change problem at this location.

B-190. In theory, the potential for high sand loss rates along the northernmost 2,800 feet of the John U. Lloyd shoreline can be addressed in two principle manners. First, the advance fill volume can be designed to meet the large annual erosion rate. Techniques similar to this have been attempted in the past. The volume of advance fill placed to protect the design beach, however, has not been sufficient to meet the annual sand requirement. Due to the steep gradient in sand transport potential, a large percentage of sand placed as advance fill would need to be concentrated along a very localized reach of shoreline. This would result in an unusually wide beach fill that would be susceptible to accelerated planform adjustment.

B-191. The second approach would consist of stabilizing a portion of the shoreline with structures and place the advance nourishment along the southern end and downdrift of the structure field. A structure field with advance fill would stabilize the most highly erosional reach of shoreline while providing adequate sand fill to nourish the downdrift shoreline. This method would translate the shoreline recession potential to a point downdrift of the structure field, an area with lower erosion potential. This would reduce the total amount of advance fill required for the project. In the absence of sand bypassing at Port Everglades, the structure field must be configured to maximize shoreline stability and minimize the amount of advance fill required to maintain the required design beach.

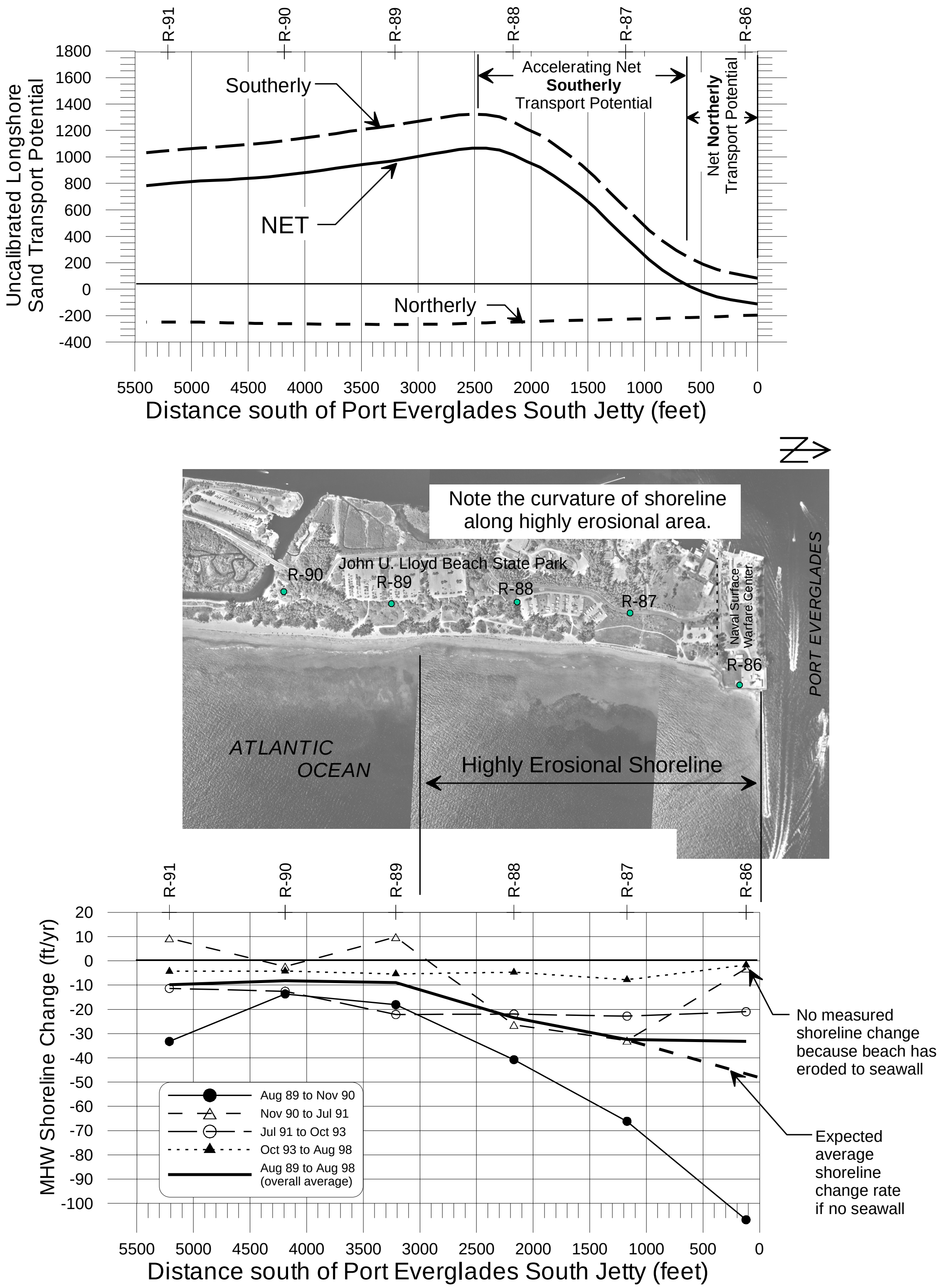

Figure B-23: Alongshore sand transport potential and measured shoreline change along the northern reach of John U. Lloyd Beach State Park (1989-1998).

B-192. To evaluate the expected performance of project configurations intended to address the erosion problem at John U. Lloyd, with and without shore stabilizing structure alternatives are simulated with the calibrated GENESIS model. The alternatives considered include the pre-project shoreline (i.e., ECL) as a baseline with (1) advance fill only, (2) 2 groins with advance fill, and (3) 10 groins with advance fill. The location and quantity of advance fill for each alternative was configured to maximize protection of the design beach while minimizing the quantity of advance fill. The two-groin alternative was configured so as to stabilize the northernmost 700 feet of shoreline where the net sand transport potential is to the north. This project configuration would minimize sand placement immediately adjacent to the inlet jetty and sand transport towards the inlet, thus reducing the potential for inlet related sand losses. The 10-groin alternative was configured to stabilize the entire reach of shoreline defined by the largest measured shoreline recession and the steepest gradient in alongshore sand transport potential (i.e., about 2,800 feet immediate to the inlet). This alternative would stabilize the most highly erosional section of shoreline and translate the feeder beach characteristics of the shoreline to an area where the alongshore sand transport potential is lower. (Note the area of reduced uniform southerly sand transport potential approximately 2,800 feet south of the inlet in Figure B-24.) The 10-groin configuration would also benefit future sand bypassing activities by stabilizing the most highly erosional section of shoreline and allowing bypassed sand to be placed far downdrift of the inlet.

B-193. <u>Advance Nourishment Only.</u> As a baseline for comparison, an advance fill only project configuration was considered. The project included sand fill to construct the design beach and advance fill sufficient to protect the design beach for a six-year period. The project configuration was simulated with the GENESIS model to demonstrate its effectiveness in maintaining a design beach.

B-194. The results of the advance nourishment only simulation are presented in Figure B-24. This alternative would include the placement of about 362,500 cubic yards of advance fill along the John U. Lloyd Beach State Park shoreline. It is interesting to note that this volume is similar to the volume of sand placed as advance fill along John U. Lloyd during the previous two projects. Unlike those projects, however, the model results suggest that approximately 90 percent of the required advance fill should be placed along the northern 3,000 feet of shoreline. This finding supports the idea that the John U. Lloyd shoreline is a strong feeder beach. With the advance fill in this concentrated configuration, the model indicates that the design beach would be protected from recession for about six years.

B-195. Although this analysis indicates that the pre-project beach would be maintained with such a beach fill configuration, accelerated losses to the offshore and inlet due to the wide fill section are not considered. The unusually wide beach fill immediately adjacent to the inlet's south jetty would most likely increase the potential for accelerated sand losses to the inlet and offshore areas. It is estimated that an average of at least 15,000 cubic yards per year of sand would be lost to the inlet with this project configuration.

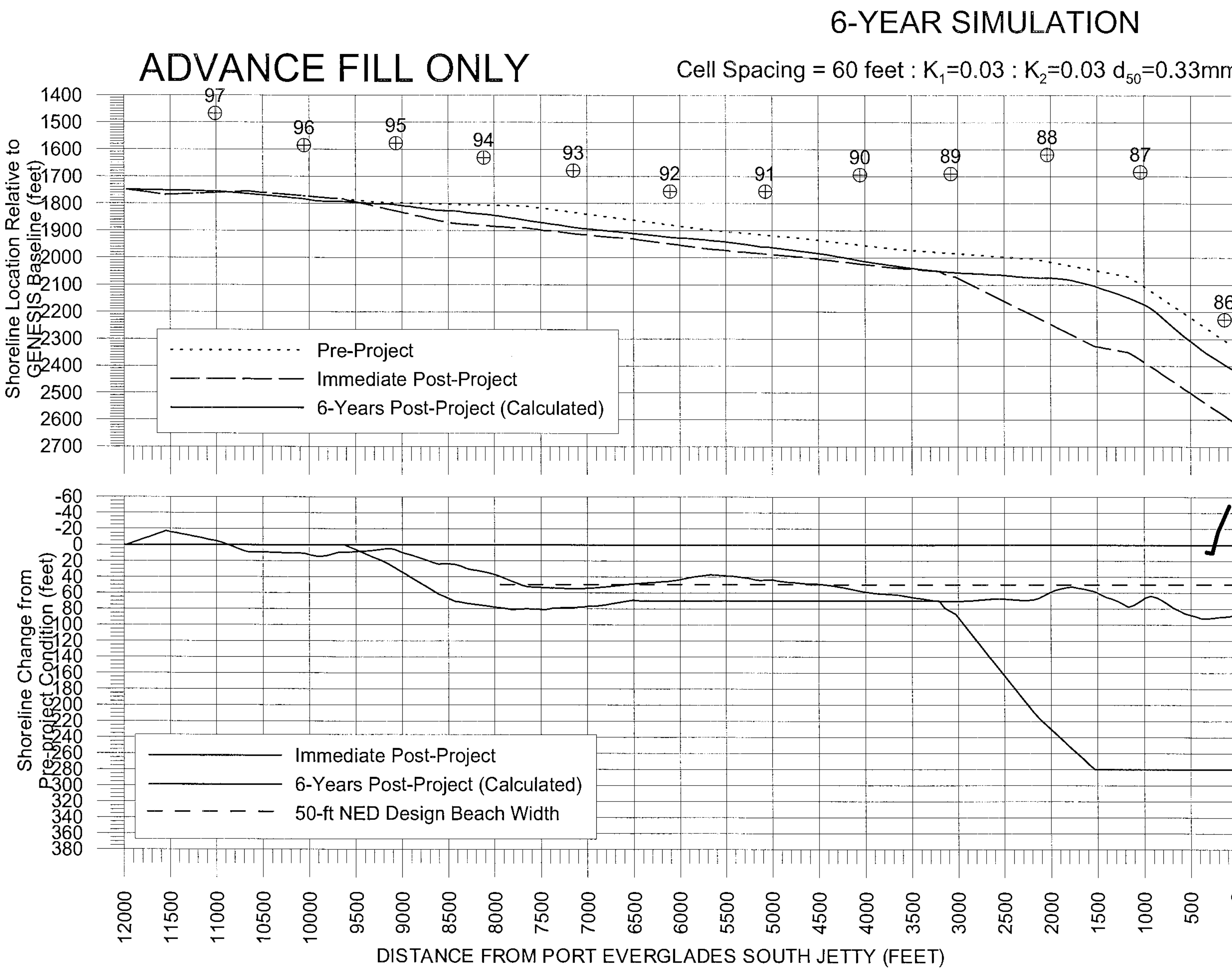

Figure B-24: GENESIS results for advance fill only alternative.

It is expected that this rate may be much higher during the early part of the project life when beach widths are at their maximum widths. Therefore, alternate project configurations are considered to reduce the advance fill volume and minimize the amount of sand fill placed immediately adjacent to the inlet. These project configurations would be intended to maintain the pre-project shoreline with sand retaining groins in place of advance fill along the most highly erosional section of shoreline.

Two-Groin Alternative

B-196. The two groin alternative would include the construction of two T-head groins within 700 feet of the Port Everglades south jetty and a spur attached to the south jetty. The configuration would address the shoreline instabilities associated with the net northerly sand transport potential along this reach of shoreline. Inspection of net alongshore sand transport potential curve in Figure B-23 indicates a nodal point in sand transport potential approximately 700 feet south of the inlet. Other investigations that have considered inlet hydraulics suggest that this nodal point may be located between 1,000 and 3,000 feet south of the inlet (Coastal Tech., 1994). Net sand transport north of the nodal point is to the north while south of the nodal point net transport is to the south. Net southerly transport accelerates rapidly from the nodal point to about 2,800 feet south of the inlet.

B-197. It is proposed that the southernmost groin be positioned just north of the nodal point's northernmost predicted position. The full advance fill section would be constructed immediately south of the southern groin. Advance fill would transition from the south groin to the south jetty. The groins and spur would reduce the sand loss rate to the inlet and protect the Naval Surface Warfare Center upland infrastructure.

B-198. Dimensions. The location and spacing of the groins were designed following the methods outlined in the SPM (1984) and by Bodge (1998). The spacing, length and crest elevations of the groins were designed to maintain the minimum design beach cross-section without the need for advance nourishment within the groin field. The groin spacing to active groin length ratio of 3:1 was used to configure the groin field. The active groin length is measured from the crest of the active beach berm (which is approximately the +6 ft NGVD elevation along the groin field shoreline) to the seaward end of the groin. The Shore Protection Manual suggests that groins be spaced using a ratio between 2:1 to 3:1 (USACE, 1984). The 3:1 ratio was used for this project to minimize the number of groins. A graphical concept of the two-groin structure configuration is presented in Figure B-25.

B-199. Design of the active groin lengths considered (1) the minimum width of the design beach cross-section and (2) the expected equilibrated slope of the beach cross-section. The design beach cross-section requires that the mean high water line be maintained at the pre-project shoreline as represented by the Erosion Control Line (ECL). The expected post-project equilibrated slope of the beach fill is approximately 1 vertical to 10 horizontal above the mean water level. A design active groin length of approximately 100 feet meets the above design criteria. The groin spacing to active groin length ratio of 3:1 requires an average distance between groins of approximately 300 feet.

B-200. The total length of each groin will be longer than the active groin length. The added section of each groin will be extended landward of the active portion of the groin to protect against flanking during storm events. The landward end of each groin will be completely covered by the beach fill. Total groin lengths will vary from approximately 100 to 180 feet.

B-201. A T-head will be constructed at the seaward end of each groin. The T-heads will serve to reduce the potential for the generation of rip currents along the groin stems and protect the seaward terminus of the groins. The T-head lengths for the northern and southern groin will be approximately 160 and 140 feet, respectively. The design procedures used to determine the size, shape, and configuration of the T-heads were taken from Bodge (1998).

B-202. The crest elevation of the T-heads and seaward end of the groin stems will be +4 ft NGVD. The crest elevation of the landward end of each groin stem will be +6 ft NGVD.

B-203. The groins will be of rubble mound construction to minimize wave reflection and the generation of rip currents. The side slopes of the groins will be 1V:2H. The groins will be primarily comprised of two layers of armor stone with a central section of core and chinking stone. The core and chinking stone will be placed where possible to partially sand tighten portions of the structures. The cross-section of the landward portions of the groins is not large enough to allow for placement of sufficient core and chinking material to provide for a sand-tight core. The landward portion of each groin, however, will be buried by sand associated with the design beach section. Because the cross-sectional area of the seaward ends of the groins is larger that the typical stem section, sufficient core and chinking material will be placed to provide sand tightness.

B-204. <u>Stone Sizes.</u> Armor stone sizes were determined using Hudson's stability equation and the design, depth limited breaking wave height. A 10-year design storm condition was used to estimate the required armor stone size. In southern Broward County, the 10-yr storm surge has been estimated to be approximately 4.0 ft NGVD (FEMA, 1978: WIS, 1982).

B-205. The controlling elevation at the seaward end of the groins is about –5.0 ft NGVD. During a 10-year storm event, the water depth at the seaward ends of the groins is expected to average about 9.0 feet. Assuming a breaking wave height to water depth ratio of 0.78, the design, depth limited breaking wave height is approximately 7.0 ft.

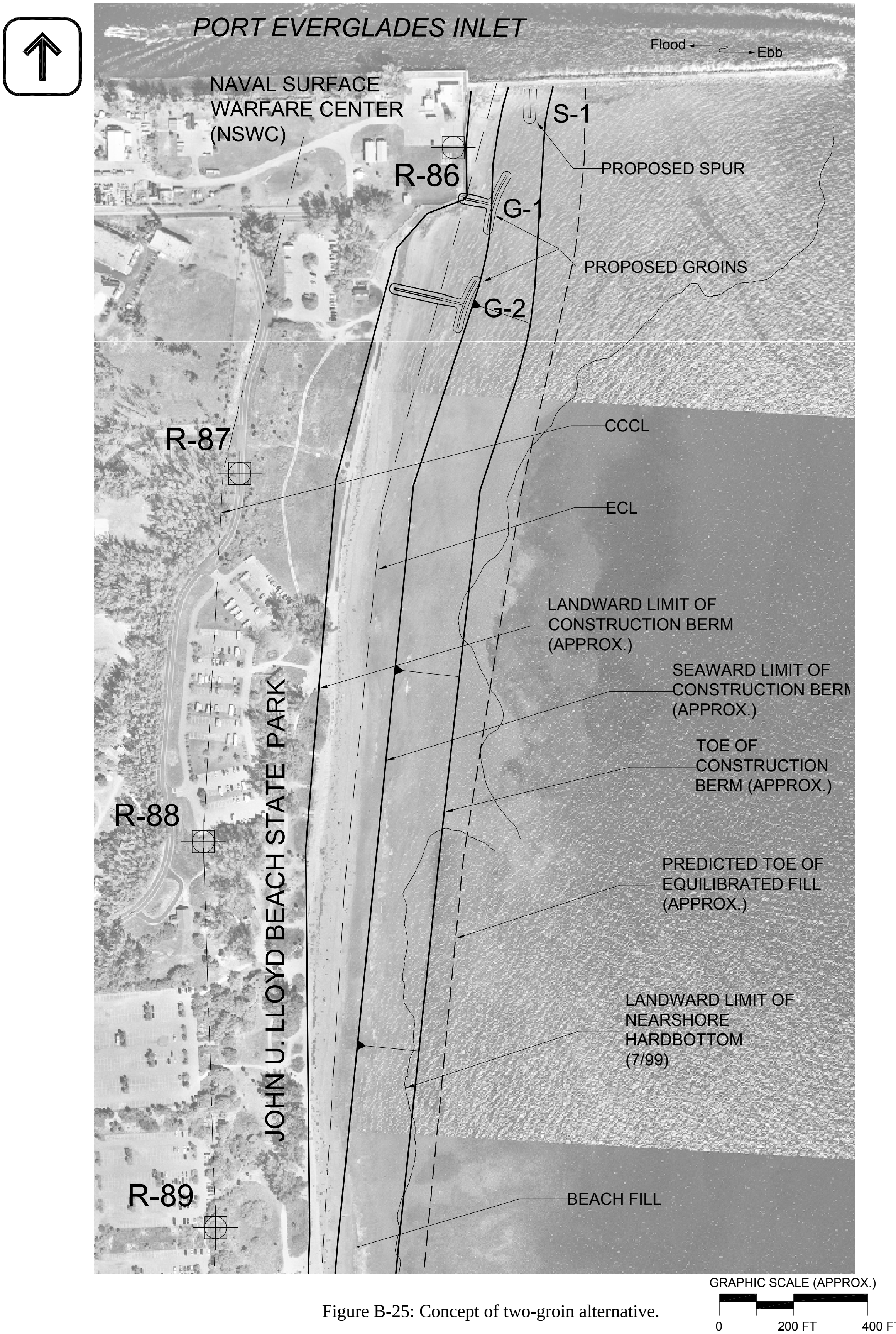

Figure B-25: Concept of two-groin alternative.

B-206. Rough, angular quarried granite with a unit weight of 165 lb/ft^3 will be used for the armor stone. The stability coefficient (K_D) for this material, two layers of armor stone, and breaking wave conditions is 1.6 (Table 7-8, SPM). The required armor stone weight for the groins will range from 1.5 to 2.5 tons with 50 percent of the individual stones weighing 2.0 tons or more. The core and chinking stone used in selected structures will consist of well-graded stone with a minimum unit weight of 165 lb/ft^3. The core and chinking stone will be a well graded material varying in size between 6 and 18 inches. The two-groin alternative would require about 5,300 tons of granite stone.

B-207. Foundation Conditions. The structures will be underlain by sand. A rigid structure foundation, however, will be required beneath the groins and the jetty spur to protect underwater cable infrastructure associated with the Naval Surface Warfare Center. The cables extend from the Navy's upland facility to the offshore areas to support underwater acoustic equipment. The cables are simply lying upon the sea floor with no structural protection. It is estimated that the replacement cost of the cable field is on the order of $350 million. To minimize the risk of damage to these cables, stone filled marine foundation mattresses will be placed as the foundations for the structures. The mattresses will distribute the load of the rock groin uniformly upon the seafloor and cables, thus minimized the loading forces upon the cables.

B-208. Cable Field Protection. In addition to the marine mattress foundations beneath the groins, large cable HDPE conduit (3 to 4, 18-inch conduits) will be installed from the NSWC building across the nearshore area to a point beyond the active sand transport limit. These conduits will be used to install new cables and rerun repaired cables from the facility to the offshore areas. This will prevent the deployment of cables across the beachface, a practice that has historically created a hazard to recreational beach use and resulted in frequent breaks in the cables that require costly repairs. The cables will be anchored with the same type of marine mattresses used as groin foundations.

B-209. Groin Construction. The groin field will be constructed in the summer. Most of the groin field construction activity will be land based. Due to restricted access, the jetty spur may be constructed from a barge that is mobilized to the interior of the Port Everglades entrance. If a barge is used, equipment and materials will access the jetty spur across the south jetty of Port Everglades.

B-210. Model Simulations. To evaluate the benefit of the two-groin alternative, the alternative project configuration was simulated with calibrated GENESIS model. It is noted that the GENESIS model cannot explicitly simulate the shore stabilizing features of the proposed jetty spur. To model the spur, it is assumed that the south jetty would be impermeable to sand transport. The T-head groins also cannot be explicitly modeled with GENESIS. To model the T-head, groins lengths and permeabilities are adjusted in the model to match the shore stabilizing characteristics of the groins.

B-211. The results of the six-year GENESIS simulation for the two-groin alternative are presented in Figure B-26. Comparisons of the pre-project and calculated post-construction shoreline locations indicate that the groin field, with adequate advance nourishment, will provide a uniformly wide beach along the JUL shoreline. The results also indicate that the shoreline will maintain the design beach section, on average. The results of this simulation also demonstrate the benefits of stabilizing that reach of shoreline commonly susceptible to net northerly sand transport. Sand placed in this area is highly susceptible to transport into the inlet and to the offshore areas. Stabilizing this reach of shoreline with groins would reduce the required volume along the northernmost reach of shoreline with minimal impact to the downdrift shoreline.

B-212. In sum, it is estimated from the GENESIS results that the two-groin configuration may reduce the advance fill requirement by about 12 percent. Assuming the local, average-annual sand loss rate along the John U. Lloyd shoreline is about 53,000 cubic yards per year, the two-groin alternative would require the equivalent of about 46,700 cubic yards per year of advance fill. In the net, this would reduce the annual advance fill requirement by about 6,300 cubic yards. Considering overfill and the advance fill volumes for Hollywood/Hallandale, this modified Segment III project would require 983,400 cubic yards of fill in addition to that required to reestablish the design beach.

B-213. <u>Project Costs.</u> It is estimated that the mobilization and unit cost for sand for the 2001 construction will be same for all alternatives considered (i.e., $1,000,000 and $9.79 per cubic yard, respectively). Likewise the cost of future sand placement is estimated to be $15.00 per cubic yard, plus mobilization.

B-214. The cost to construct the groin field is based upon the estimated prices to place granite stone in the marine environment. Based upon recent project is south Florida similar to the proposed works, it is estimated that granite stone for T-head construction costs about $75 per tons in place. This cost includes material purchase, transport, and placement is the design configuration.

B-215. Foundation requirements for the proposed project include both marine stone filled mattresses and a geogrid composite material. Based upon recent bid prices for similar foundation works, it is estimated that the in-place costs for marine mattresses and geogrid composite material is $15.00 and $2.50 per square foot.

B-216. All other project related costs such as monitoring and engineering and design and supervision and administration are identical to all modification alternatives considered in this report.

B-217. <u>Future Maintenance of Groins.</u> The groin field was designed for a 10-year storm surge event with no damage. Because the 10-year event is expected to be exceeded during the remaining 24-year project life, maintenance of the groin field will be required.

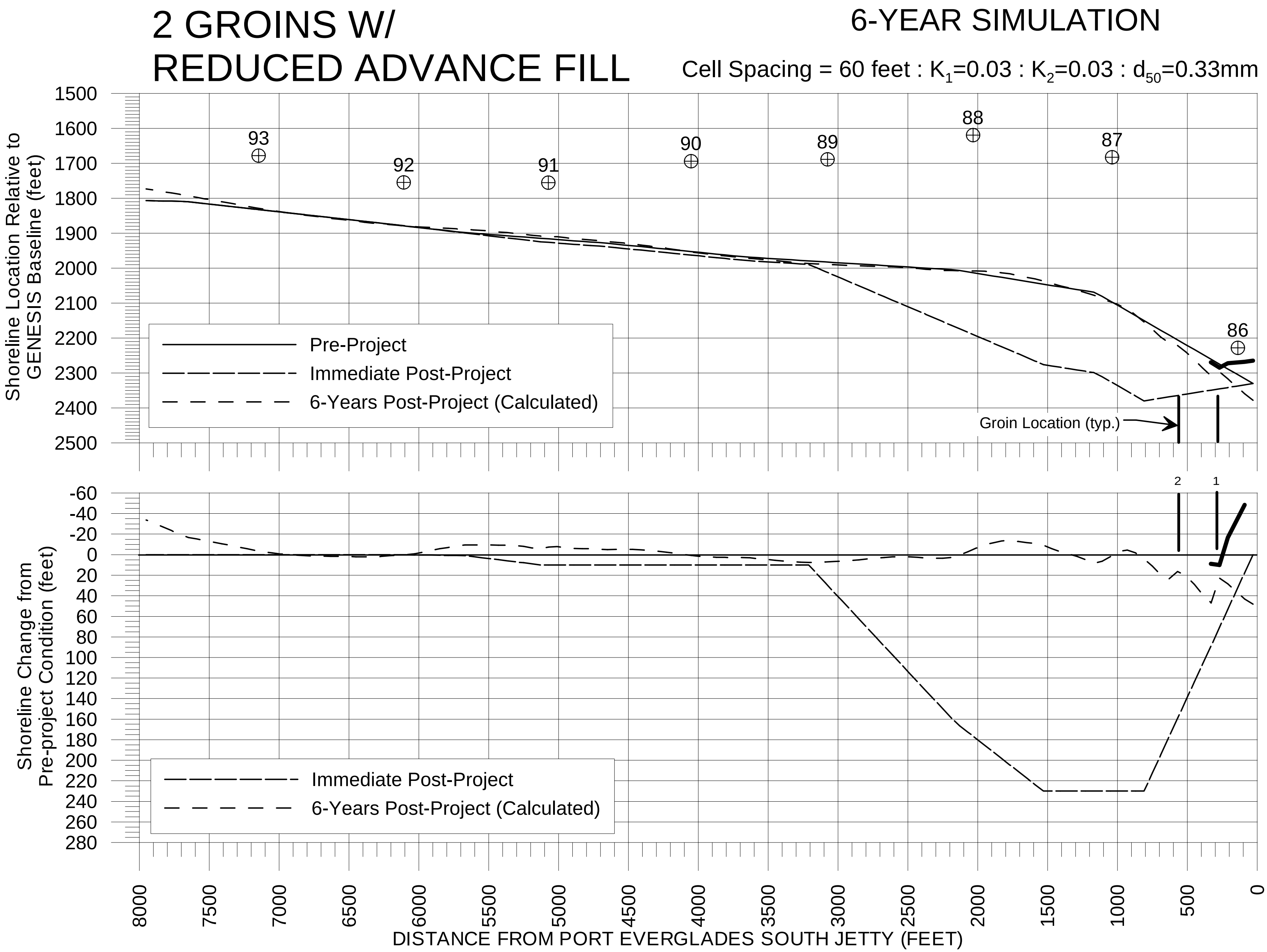

Figure B-26: GENESIS results for two-groin alternative.

B-218. The future maintenance requirements and costs were calculated using a probabilistic approach. The approach involves the development of a relationship between expected structure damage and storm events that exceed the design storm event. Using Table 7-9, Page 7-211 of the Shore Protection Manual (SPM, 1984), the expected structure damage for a storm event exceeding the design storm can be estimated. A probabilistic relationship between structure damage and the occurrence of a storm that exceeds the design storm is determined by tabulating damage estimates for various storm frequencies greater than the design storm. Total damages are computed by integrating the annual probability of damage over the life of the project. The cost to repair annual is assumed to be a percentage of the initial construction cost of the groin field.

B-219. Table B-20 summarizes the various storms considered in this analysis and the level of damage expected from each storm event. The annual expected maintenance cost for the groin field is 1 percent of the initial groin field construction cost.

B-220. Cost Summary. The total average annual cost to implement the modified reevaluated plan to include two groins and a jetty spur is $4,429,000. Project costs required to implement the reevaluated authorized project were formulated using a percent rate of 6 and 1/8 for the remaining 24 years of the project life. The details of the cost estimate for this plan are included in Sub-Appendix B-6.

Table B-20: Expected damage to the groin field for various storms exceeding the design storm.

Storm Return Period (yrs.)	Prob. of Occur.	Surge (ft)	Breaking Wave Hgt. (H) (ft)	H/H_D	Damage (%) (from Table 7-9, SPM)	Assumed Damage (%)
10	0.1000	4.0	6.3	1.00	0 to 5	0
15	0.0667	4.5	6.6	1.05	5 to 10	7.5
20	0.0500	5.0	7.0	1.10	5 to 10	10
35	0.0286	5.5	7.4	1.15	10 to 15	12.5
50	0.0200	6.0	7.8	1.21	10 to 15	15
75	0.0133	6.5	8.2	1.29	15 to 20	20
100	0.0100	7.0	8.6	1.35	20 to 30	30

Ten-Groin Alternative

B-221. For completeness, a ten-groin alternative is also considered to extend the shore stabilizing features of a structural field throughout the most highly erosional section of shoreline. The purpose and physical benefit of the extended groin field would be to stabilize the most highly erosional section of shoreline and apply advance fill along areas of shoreline with lower net longshore sand transport potential (i.e., south of a point some 2,800 feet south of the inlet). The ten-groin alternative would include ten T-head groins placed along about 2,800 feet of shoreline and a jetty spur. The alongshore extent of the groin field was developed to be consistent with the limits of the most highly erosional section of shoreline as described in the preceding paragraphs and detailed in Figure B-24. The location and spacing of the groins were designed following the methods outlined in the SPM (1984) and by Bodge (1998). The physical characteristics of the structures for the ten-groin alternative would be identical to those describe above for the two-groin alternative. A graphical concept of the ten-groin structure configuration is presented in Figure B-27.

B-222. Stabilizing this northern reach of shoreline with T-head groins would allow the placement of advance fill beyond the direct of the influence of the inlet. Results of the refraction/diffraction and longshore sand transport potential analysis suggest that generally uniform southerly sand transport potential develops about 2,800 feet south of the inlet. North of that point, there is a strong acceleration in southerly sand transport potential. Such accelerations in transport usually result in highly erosional and unstable shoreline conditions.

B-223. The centroid of concentrated advance fill would be relocated approximately 1,600 feet south from that for the advance fill only alternative. The advance fill for the ten-groin alternative would be configured to meet the sand feeding requirements that naturally maintain shoreline stability along the downdrift shoreline. Approximately 50 percent of the advance fill would be placed along the southern half of the groin field and the remainder would be placed along approximately 1,500 feet of shoreline immediately downdrift of the groin field.

B-224. The ten-groin project configuration was also simulated with the calibrated GENESIS model. The results of the GENESIS are presented in Figure B-28. Comparisons of the pre-project and calculated post-construction shoreline locations indicate that the ten-groin structural field, with adequate advance nourishment, would also maintain the design beach section along the along the John U. Lloyd shoreline, on average. The project configuration, however, is not expected to greatly reduce the off-site sand requirements; thus, it would not significantly reduce long-term off-site sand requirements compared to the two-groin alternative. It does, however, provide shoreline stability along the historically erosional reach of shoreline with minimal sand placement in the vicinity of the south jetty. Minimizing sand placement in the vicinity of the south jetty would reduce the potential from sand losses to the inlet.

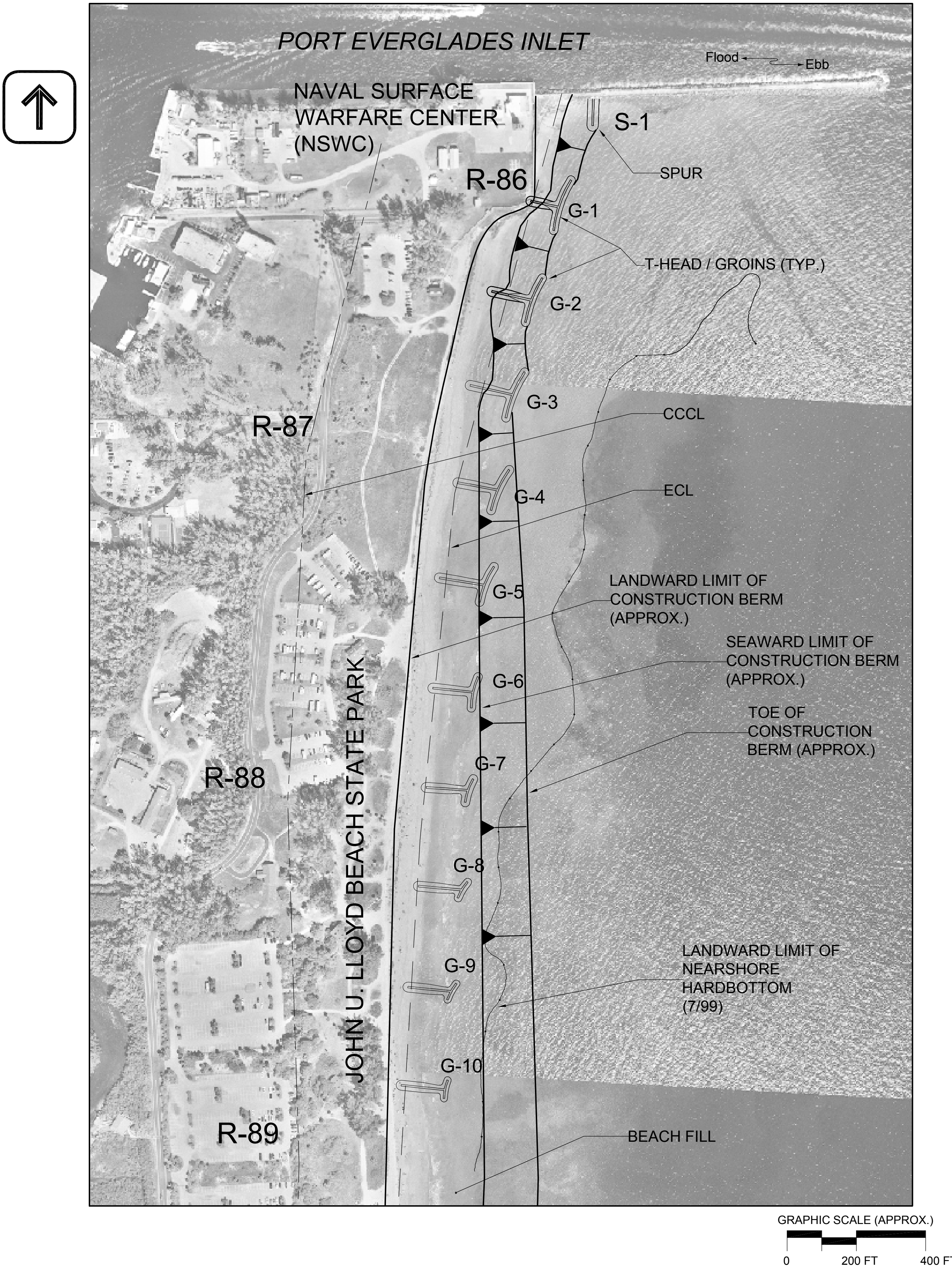

Figure B-27: Concept of ten-groin project alternative.

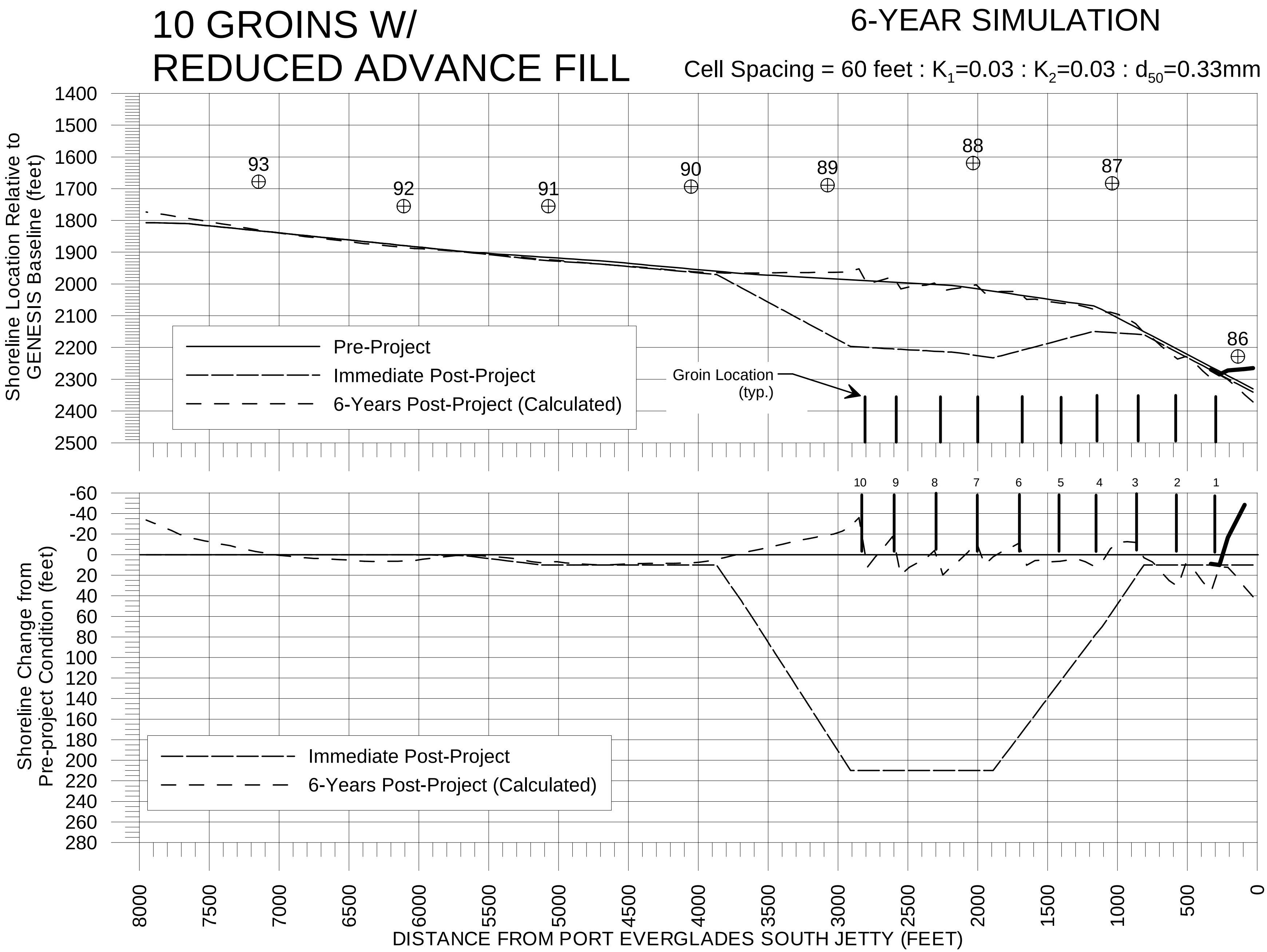

Figure B-28: GENESIS results for ten-groin alternative.

B-225. In sum, it is estimated from the GENESIS results that the ten-groin configuration may reduce the advance fill requirement by about 22 percent. Assuming the local, average-annual sand loss rate along the John U. Lloyd shoreline is about 53,000 cubic yards per year, the ten-groin alternative would require about 41,300 cubic yards per year of advance fill. In the net, this would reduce the annual advance fill sand requirement by about 11,700 cubic yards. The advance fill volume requirement for the John U. Lloyd shoreline reach over the six-year optimum interval is estimated to be about 247,800 cubic yards. An additional 34,300 cubic yards of sand would be required for overfill at John U. Lloyd. Therefore, the total advance fill and overfill volume for Segment III with project modification would be 946,500 cubic yards.

B-226. Project Costs. All unit costs for the ten-groin alternative are assumed to be identical to those developed for the two-groin alternative. The economic difference between the two structural alternatives will be based solely upon the differences in the physical requirements of the two configurations. For instance, the ten-groin alternative requires less annual fill from an off-site location but would require more stone material for the added groins. The ten-groin alternative would require an estimated 22,000 tons of granite stone.

B-227. All other project related costs such as monitoring and engineering, design, and supervision and administration are also identical to all modification alternatives considered in this report. Similarly, the cost of the annual maintenance of the groins is assumed to be approximately 1 percent of the initial cost of the groins.

B-228. Cost Summary. The total average annual cost to implement the modified reevaluated plan with ten groins is $4,432,000. Project costs required to implement the reevaluated authorized project were formulated using a percent rate of 6 and 1/8 for the remaining 24 years of the project life. The details of the cost estimate for this plan are included in Sub-Appendix B-5.

B-229. Summary. Although the ten-groin alternative demonstrates a net economic benefit (i.e., cost reduction) over the two-groin alternative, it is currently the position of the State of Florida's Department of Environmental Protection and Department of Parks and Recreation (the upland land owner) that structural stabilization of the northern 2,800 feet of the John U. Lloyd Beach State Park shoreline is not in the best interest of the State and would not be permitted. Nonetheless, the results of this analysis demonstrate the physical and economic benefits of this project configuration. However, without the consent of the State of Florida, this alternative cannot be considered for implementation at this time.

Mechanical Sand Bypassing at Port Everglades

B-230. Cost-effective sand sources for Segment III beach renourishment will become more important in the future as nearby offshore sand deposits are depleted. One alternative future sand source is sand bypassing at Port Everglades. Although the economic benefit of sand bypassing is often related to reduced maintenance at navigation projects, sand bypassing at Port Everglades would provide both physical and economic benefits to the Segment III Federal Shore Protection Project. The physical benefits would include access to a reliable future sand source that is compatible with the native sediments of the Segment III shoreline and reduced sand shoaling within the Port Everglades navigation project. These latter benefits are not considered in this analysis. The economic benefits would include an overall reduction in the cost to maintain the Segment III project.

B-231. The principle benefit of sand bypassing is the reduced need for offsite sand sources to maintain the design beach section. Following the 2002 nourishment of Segments II and III, cost effective sand sources offshore of Broward County will be essentially depleted. The only other alternatives for offshore sands would be domestic deposits offshore of more northern counties (i.e. Palm Beach and Martin), Federal sand deposits offshore of Martin County, or foreign deposits from the Bahamas or other Caribbean nations. Another alternative would be trucking sand from upland areas. All of these future sand source alternatives will be very expensive compared to the cost of bypassed sand. Additionally, bypassed sand will have almost identical textural and color characteristics as the Segment III sands.

B-232. The calibrated GENESIS model and the Port Everglades sediment budget were used to evaluate the physical benefits of sand bypassing at Port Everglades. In the model it is assumed that sand could be captured and mechanically transported across the inlet at a reliable average annual rate. At present, Port Everglades is a complete littoral barrier. That is, no sand is transported across the inlet from the updrift to downdrift shoreline. Additionally, sand is lost to the inlet from the Segment III shoreline during periods of northerly sand transport.

B-233. Recent estimates suggest that sand is currently accreting along the updrift shoreline at over 65,000 cubic yards per year. Beach volume changes measured along the southern Broward County Segment II shoreline for the period between 1980 and 1996 and between 1993 and 1996 are summarized in Table B-21. Figure B-29 depicts the cumulative beach volume change from the north jetty to a point 7,000 feet north thereof. These measured shoreline changes reveal the pronounced accretion that occurs along the updrift shoreline. Most of this accretion is due to impoundment by the large shoal immediately north of the inlet. This shoal was created from side cast material from an earlier inlet-deepening project. This large shoal essentially acts as a highly effective submerged groin that impounds sand across the entire beach profile along the updrift shoreline. It is expected that the sand transport rate across this shoal is relatively low compared to typical rates of the area; thus explaining the low measured shoaling rates within the Port Everglades entrance channel. It is likewise expected that this shoal will require modification to increase the sand transport rates immediate to the inlet where bypassing activities would be potentially staged.

B-234. For the purposes of this evaluation, it is assumed that approximately 44,000 cubic yards per year of sand could be routinely bypassed across the inlet (Coastal Tech., 1996). Considering the documented accretion rate along the updrift shoreline, the actual rate may be much higher.

B-235. As demonstrated by the advance fill only alternative and measured shoreline change rates, it is estimated that approximately 46,700 cubic yards of sand are required to maintain the design section for the two-groin alternative. Therefore, considering the assumed bypassing rate of 44,000 cubic yards per year, the equivalent of approximately 2,700 cubic yards per year of offsite advance fill, plus overfill, would be required for this project. These numbers are, of course, expected to vary depending upon the ultimate productivity of sand bypassing operations. The advance fill was configured to maximize the benefits of sand bypassing in terms of maintaining the required design section. During the model simulations, the bypassed sand is added to the downdrift shoreline along the southern end of the groin field and immediately downdrift of the southernmost groin. The sand is added at a constant rate equivalent to 44,000 cubic yards per year. No overfill allowance is required for sand that is bypassing since it is essentially native beach material.

B-236. The GENESIS simulation results for the two-groin alternative are included in Figure B-30. In this simulation, bypassed sand is discharged immediately downdrift of the southern groin. These results demonstrate the benefit of sand bypassing at Port Everglades to the Segment III shoreline but also demonstrate the potential problems with discharging bypassed sand too close to the inlet. The model results suggest that sand will be trapped along the northern reach of shoreline and not transported to the downdrift shoreline. This situation would increase the potential for bypassed sand to be transported towards and lost to the inlet. Based upon these results, it is expected that sand must be discharged at a more southerly location. Structural stabilization of the shoreline north of a selected discharge point will be required.

Table B-21: Measured beach volume change immediately north of Port Everglades (adapted from Coastal Tech., 1994).

	Monument	Distance (Feet)	1993-1996 Profile Volume Change (cy/ft/yr)	1993-1996 Volume Change (cy/yr)	1980-1996 Profile Volume Change (cy/ft/yr)	1980-1996 Volume Change (cy/yr)
Shoreline Immediately North of Port Everglades						
	78					
		1,066	5.6	5,970	2.5	2,670
	79					
		1,121	8.2	9,190	3.0	3,360
	80					
		1,082	12.9	13,960	6.3	6,820
	81					
		1,043	10.7	11,160	8.1	8,450
	82					
		1,025	7.0	7,180	7.6	7,790
	83					
		892	10.8	9,630	9.0	8,030
	84					
		680	9.6	6,530	7.9	5,370
	N. Jetty					

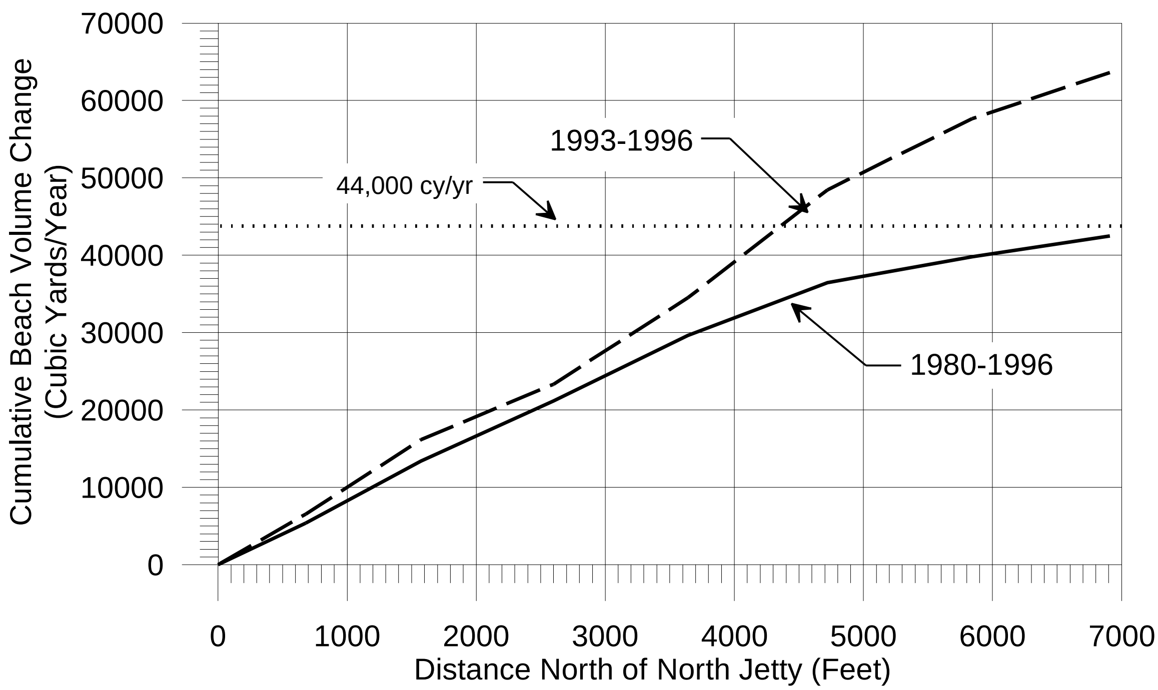

Figure B-29: Cumulative beach volume change north of Port Everglades.

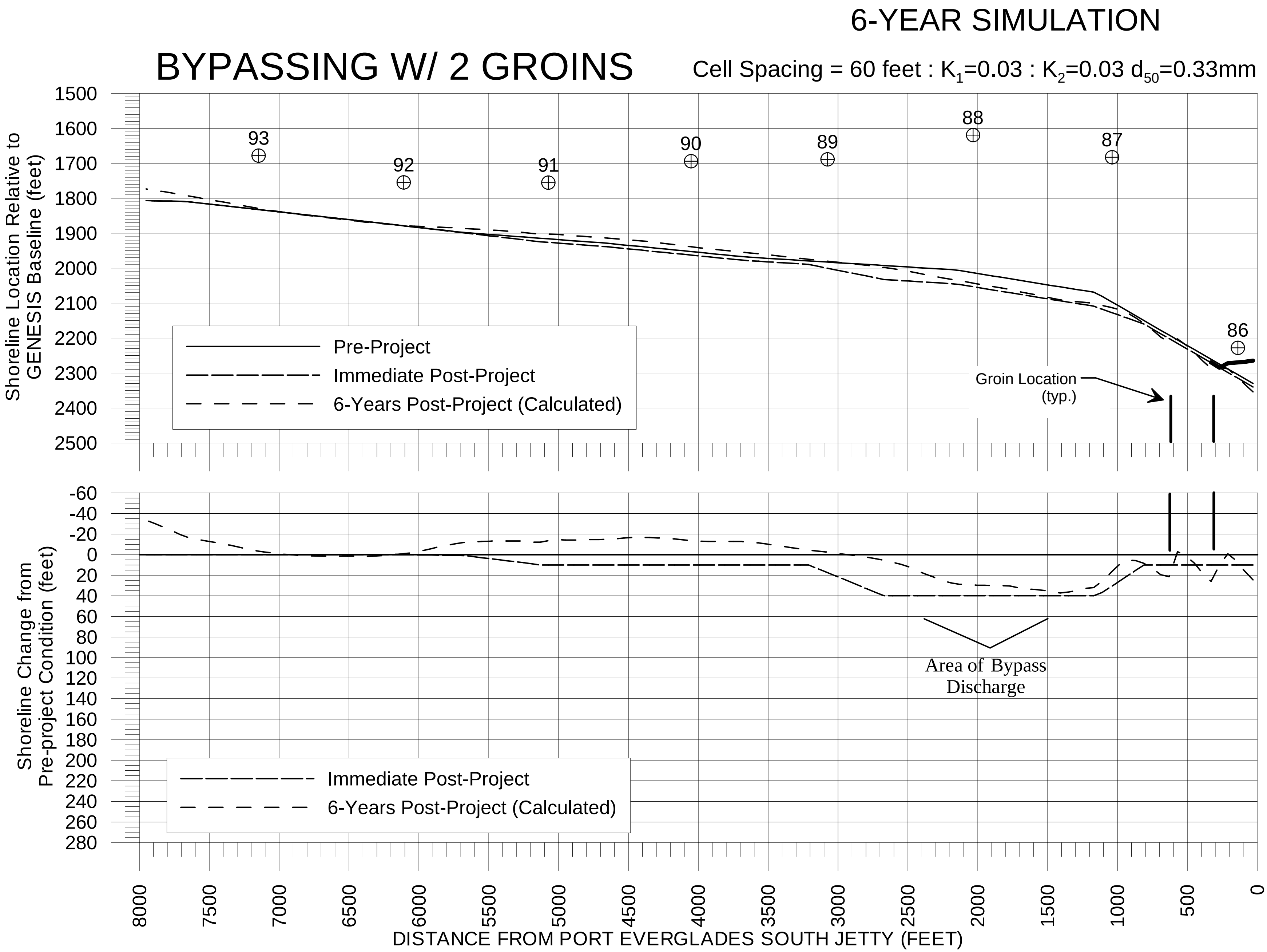

Figure B-30: GENESIS results for two-groin project alternative with inlet sand bypassing.

B-237. Project Costs. The project cost associated with implementation of a sand bypass operation at Port Everglades would include the initial capital layout for the sand bypassing infrastructure, inlet jetty and nearshore shoal modifications, and the annual cost to bypass sand and maintain the bypassing equipment. It is expected that the bypassing infrastructure would include either fixed or mobile sand collection plant, a dedicated pipeline installed beneath the navigation channel of Port Everglades, and numerous discharge points along the southern shoreline. Discharge locations would be situated within 3,000 to 4,000 feet of the south of the south jetty. For the purposes of this investigation it is assumed that annual maintenance costs are incorporated in the unit cost of the bypassed sand. Sand bypassing with the two-groins alternative is assumed not to require any modifications to the proposed groin field.

B-238. It is assumed that the initial cost to construct the sand-bypassing infrastructure would be approximately $7,000,000. This is conservatively high compared to estimates outlined in the Port Everglades Inlet Management Plan (Coastal Tech., 1994). The unit cost of bypassed sand once the bypassing infrastructure is in place and operational is assumed to be about $3.50 per cubic yard.

B-239. The total average annual cost to implement the modified reevaluated plan with implementation of sand bypassing at Port Everglades in year six is $4,287,000. Even considering the initial cost of the bypassing infrastructure, the proposed bypassing plan with two groins at John U. Lloyd represents an average annual cost reduction of approximately $184,000 per year compared to the reevaluated NED plan. There is an average annual cost saving of $142,000 per year over the two-groin no bypassing alternative. This significant cost reduction is due to the lower unit cost of bypassed sand compared to the expected cost of future off-site sand resources. The details of the cost estimate for this plan are included in Table B-5-3 (Sub-Appendix B-5).

SUMMARY

B-239. Based upon the average annual costs of alternate project modifications outlined in Table B-22 and results from analyses of beach monitoring data, calculated wave refraction/diffraction patterns, computed longshore sand transport potential, and a GENESIS shoreline change model, it is recommended that the NED include reconstruction of the pre-project shoreline at John U. Lloyd and reestablishment of a 50-ft extension of the ECL along the Hollywood/Hallandale shoreline. The plan shall include 6 years of advance fill placed along the previously constructed reaches of John U. Lloyd (south jetty of Port Everglades to R-94) and Hollywood/Hallandale Beach (R-101 to R-128). In addition to the renourishment of those shoreline reaches, it is recommended that beach fill transitions be constructed along the southern end of the John U. Lloyd reach and at the northern and southern ends of the Hollywood/ Hallandale reach to reduce endlosses and protect the design section. A two-groin and jetty spur structural field is also recommended for construction along the northern 700 feet of the John U. Lloyd shoreline to stabilize that section of shoreline and reduced sand losses to the Port Everglades. It is also recommended that sand bypassing be implemented at Port Everglades following construction of the recommended project to provide an alternative sand source for future maintenance of the Segment III Shore Protection Project. Implementation of sand bypassing at Port Everglades, along with construction of two groins at John U. Lloyd would reduce the average annual cost of the Segment III project to about $4,287,000. This equates to an average annual cost savings of $184,000 compared to the reevaluated NED plan.

Table B-22: Annualized cost summary for project modifications.

Project Plan	AVERAGE ANNUAL COST
Reevaluated NED Plan with Added Beach Fill Tapers	$4,471,000
Modifications to the Authorized Plan (R-94 to R-101) ***	
Design Section along Dania and Southern JUL (R-94 to R-101)	$5,206,000
Modifications to the Authorized Plan (Groin Field)	
Two-Groin Alternative	$4,429,000
Ten-Groin Alternative	$4,432,000
Modifications to the Authorized Plan (Bypassing)	
Two-Groin Alternative with Future Sand Bypassing at Port Everglades	$4,287,000

Notes:

GENERAL: Project benefits are the same for all alternatives included in this table, except for the project that would include a design section between R-94 and R-101 (see note below).

*** This project modification results in increased project costs and primary benefits. The incremental increase in primary benefits, however, is less than the incremental increase in project costs. Thus, this modification is not

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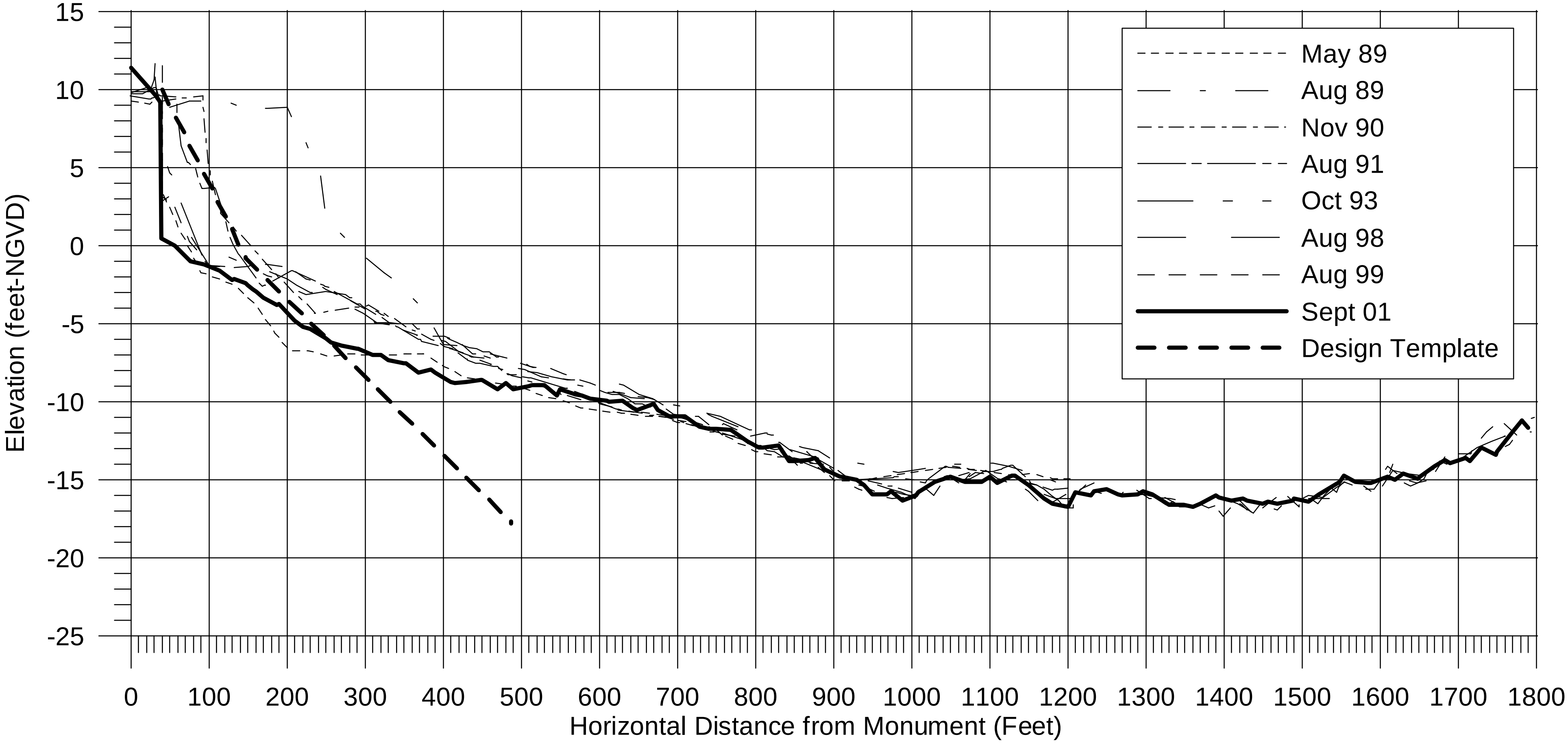
SUB-APPENDIX B-1

SEGMENT III

HISTORICAL BEACH PROFILE DATA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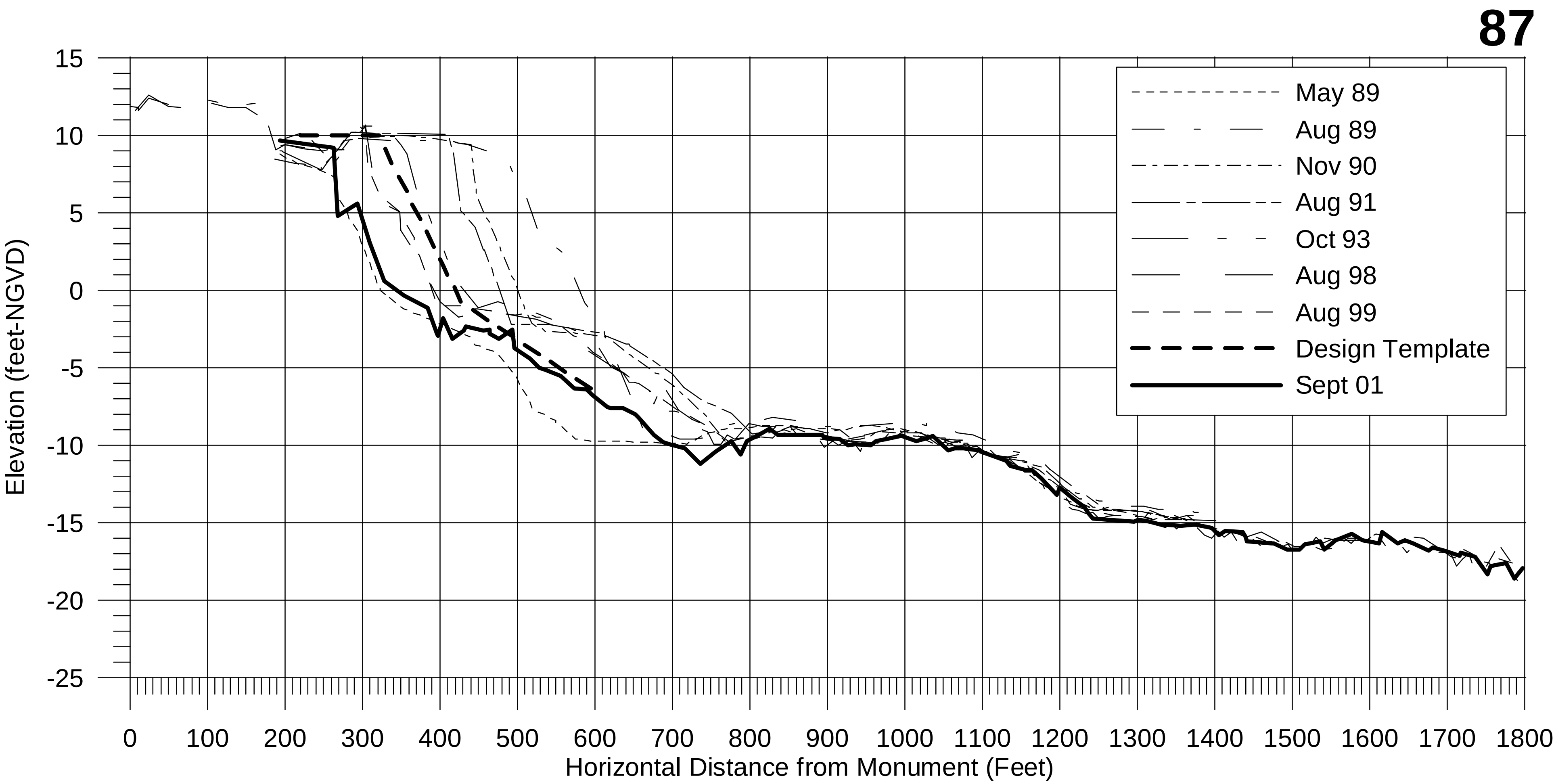
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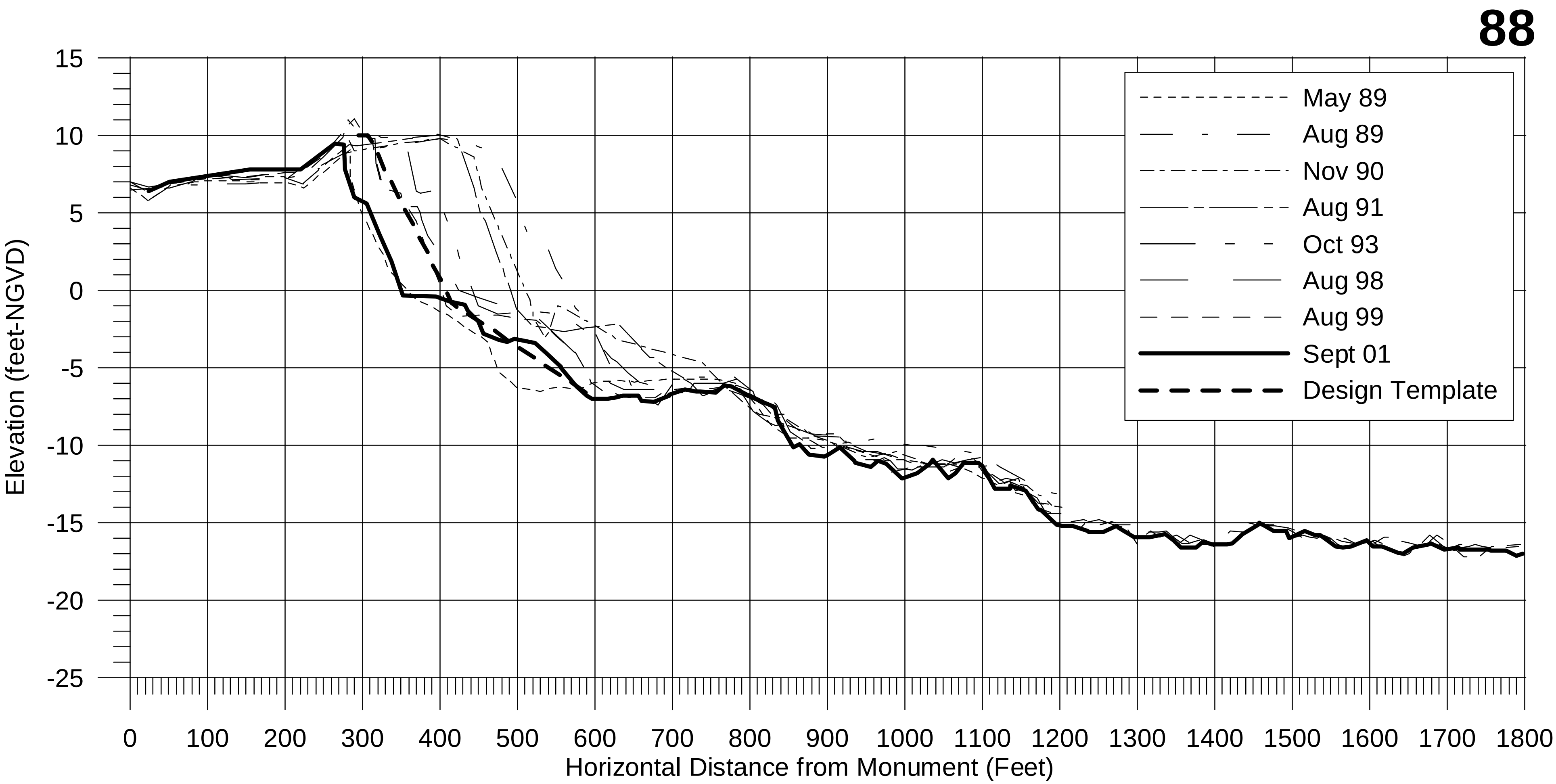

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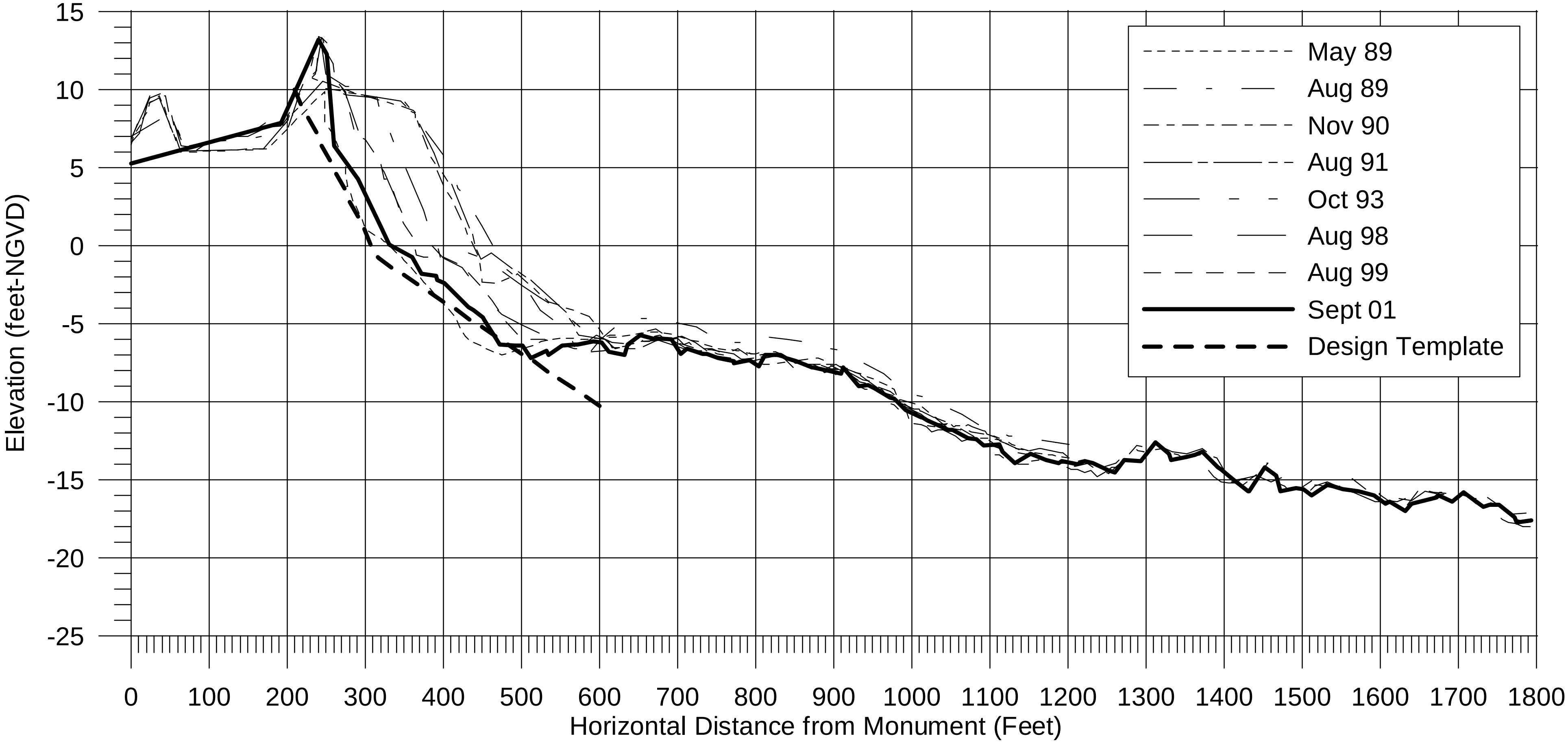
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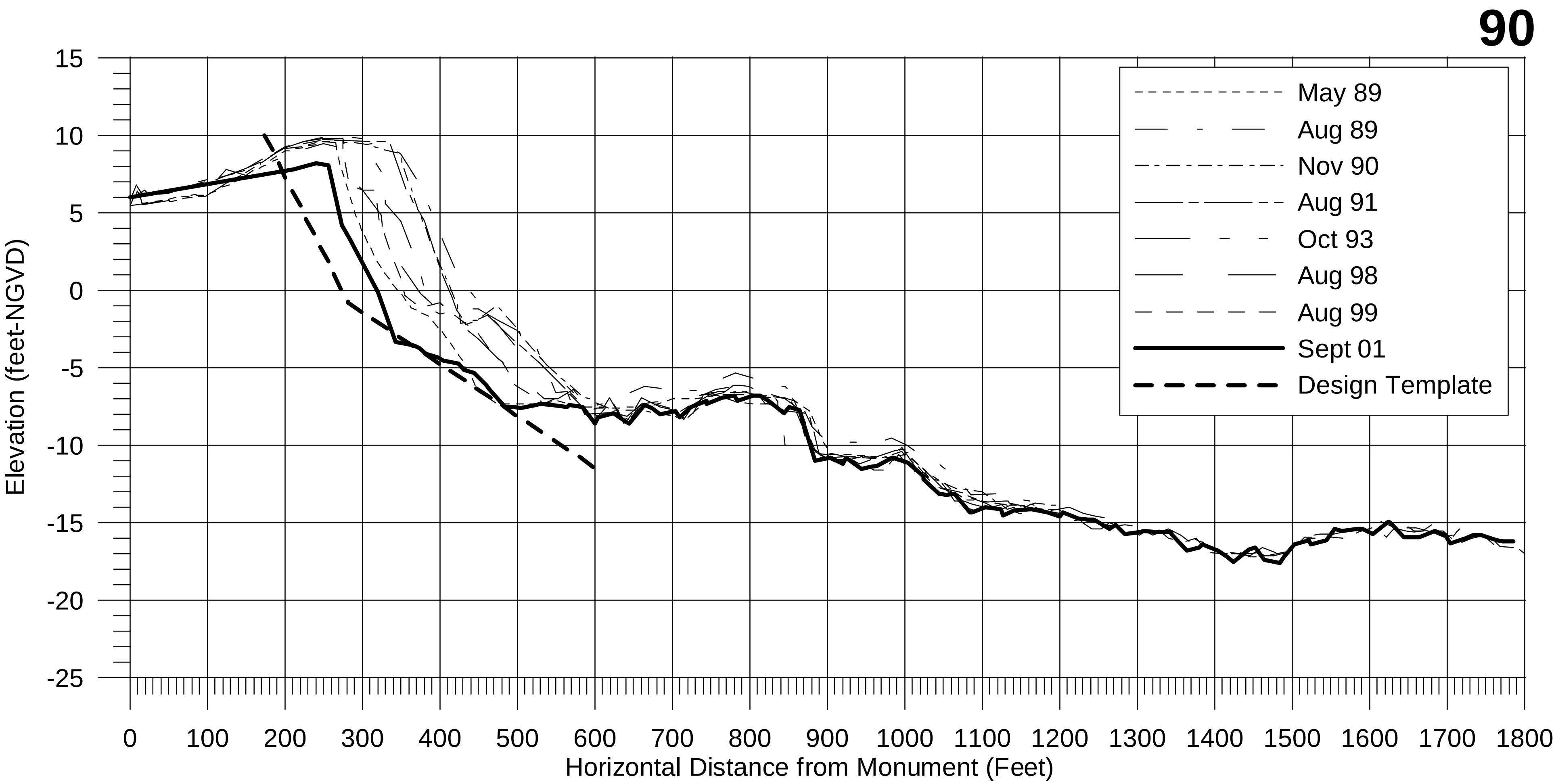
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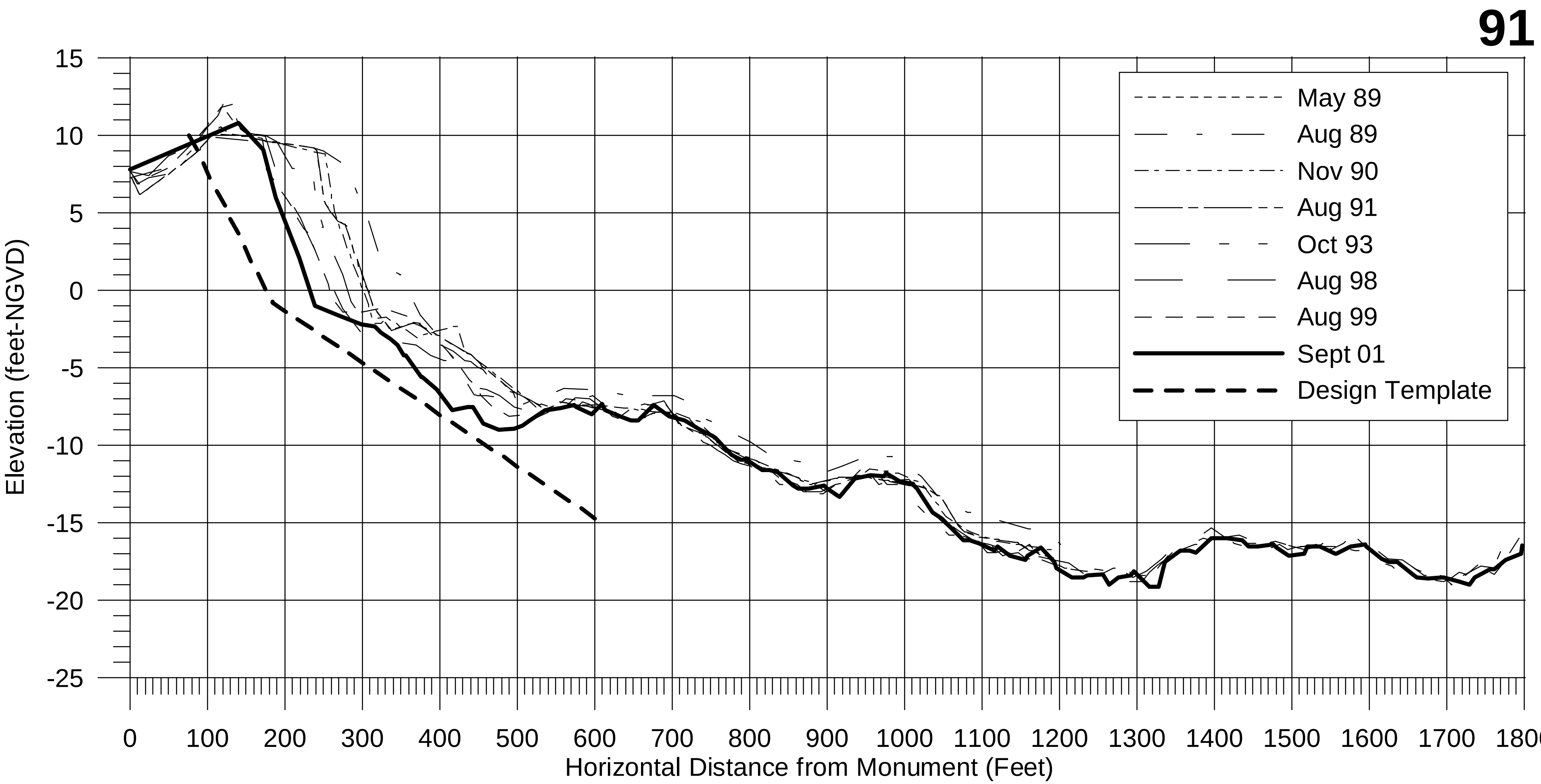

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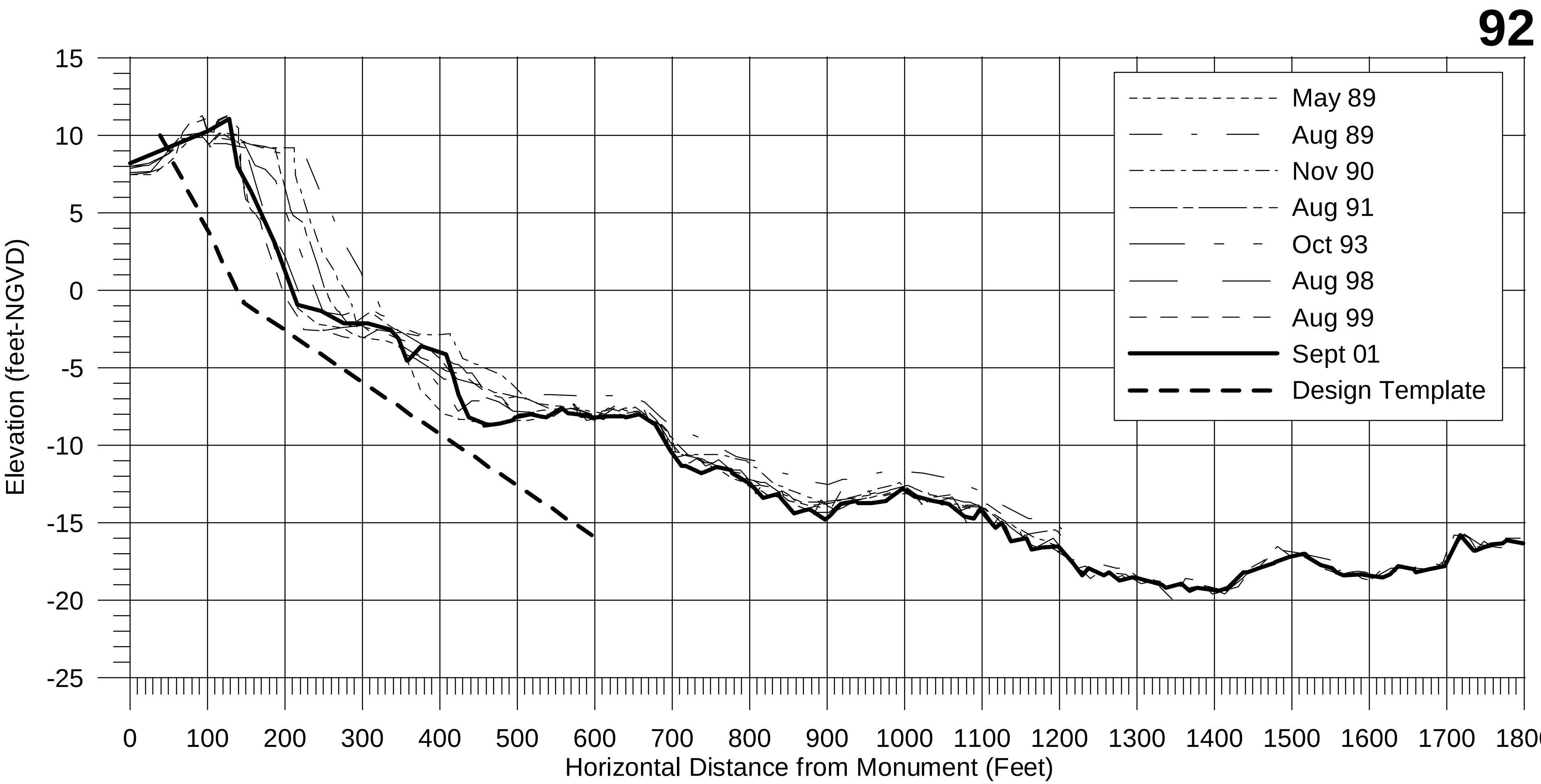
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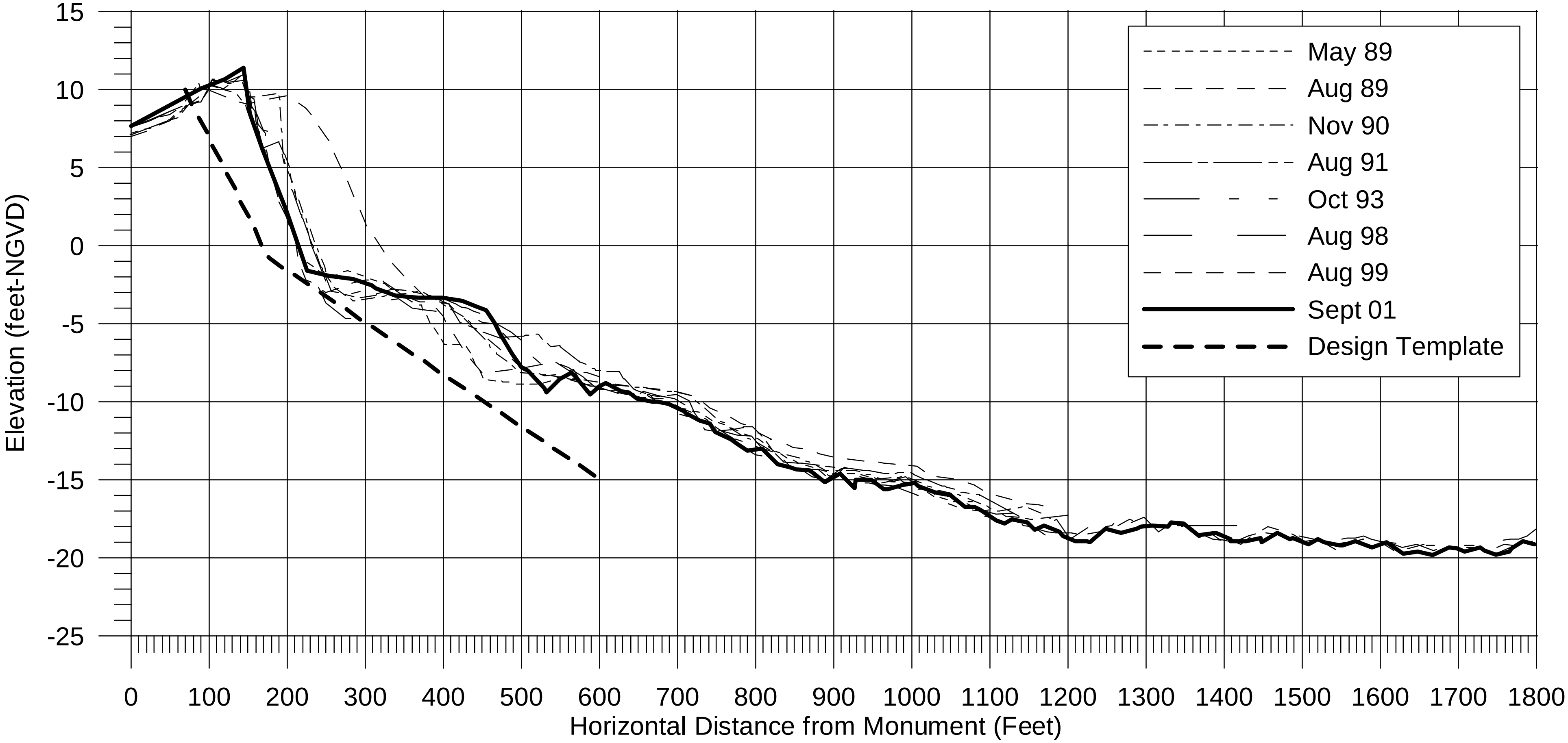

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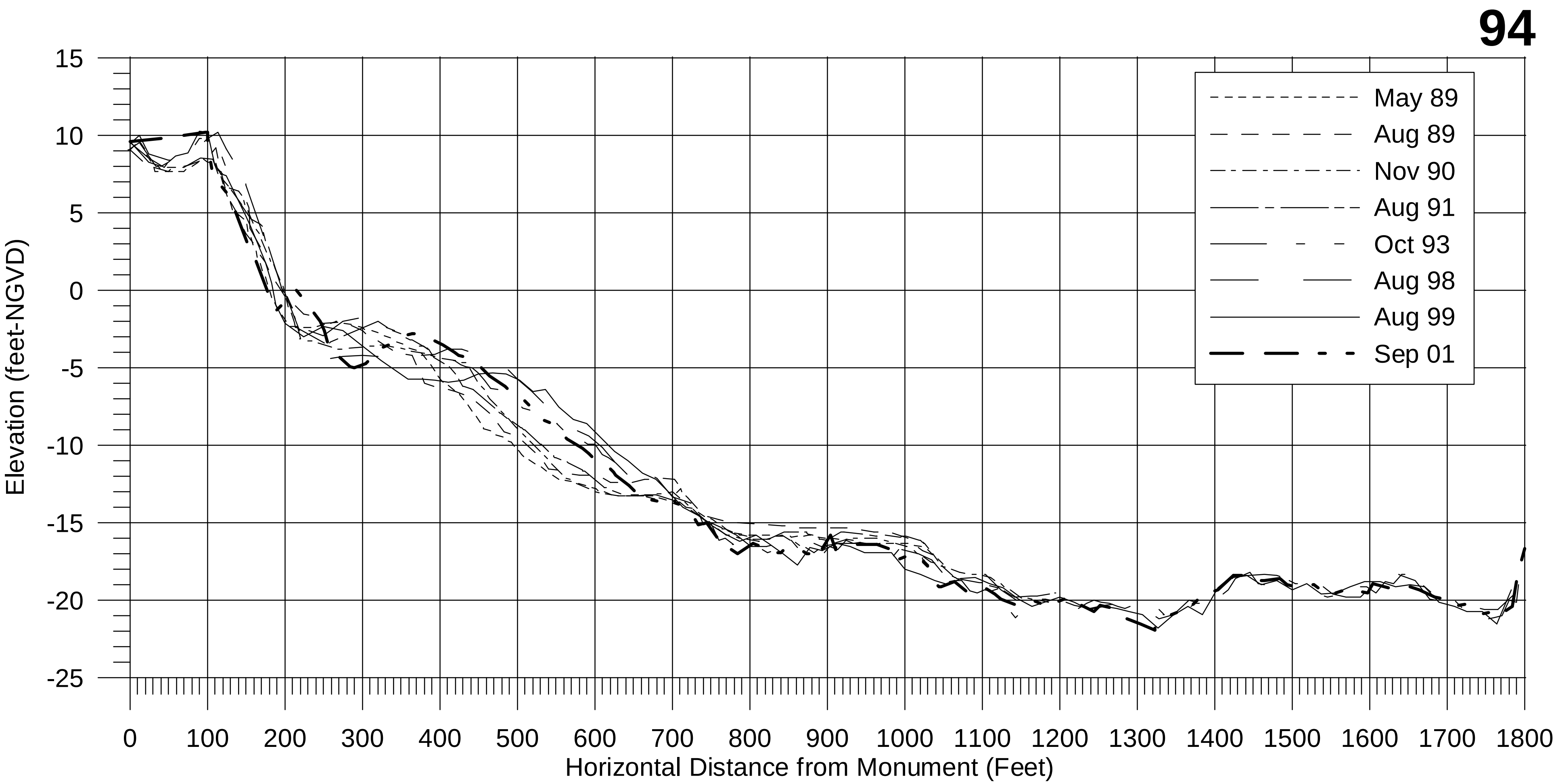
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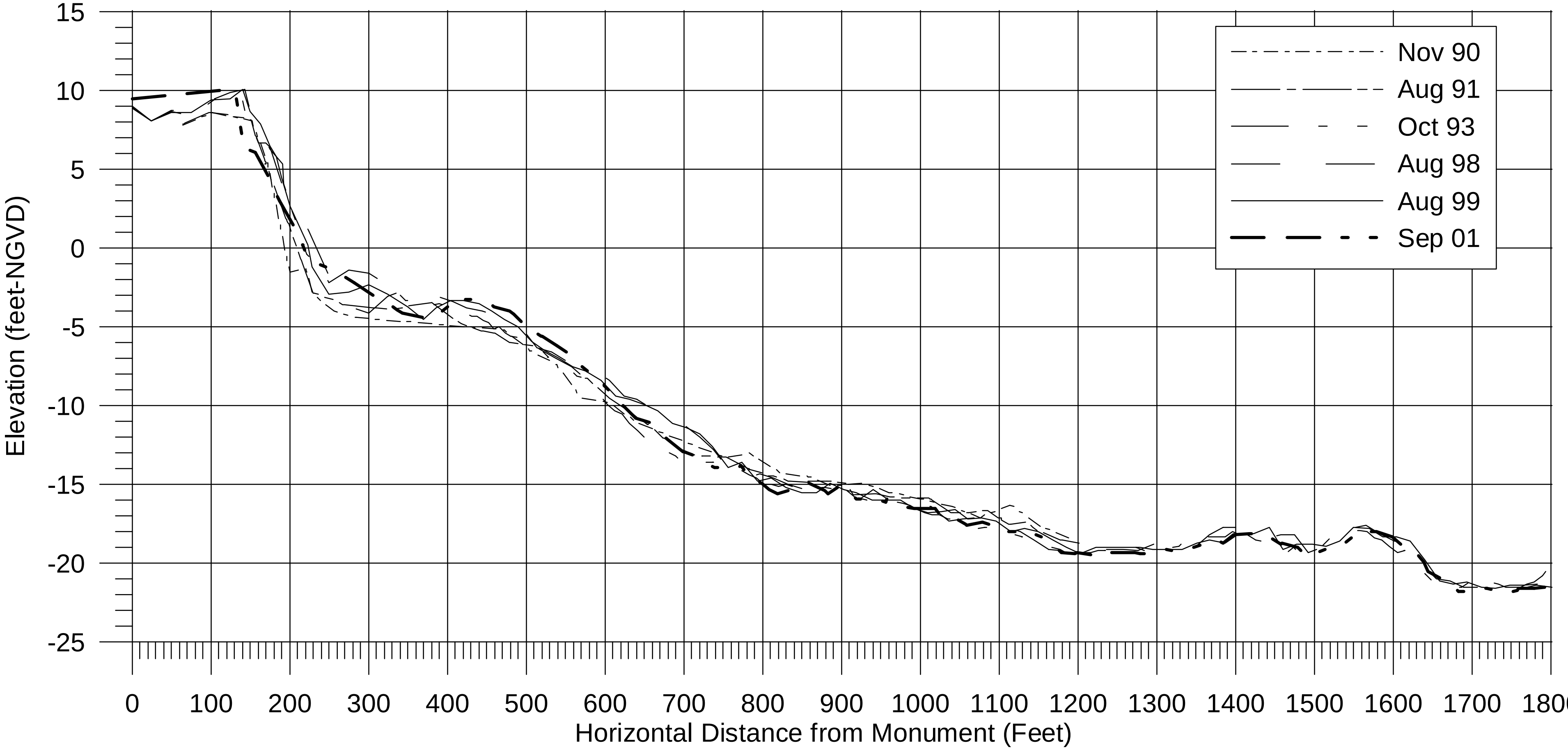

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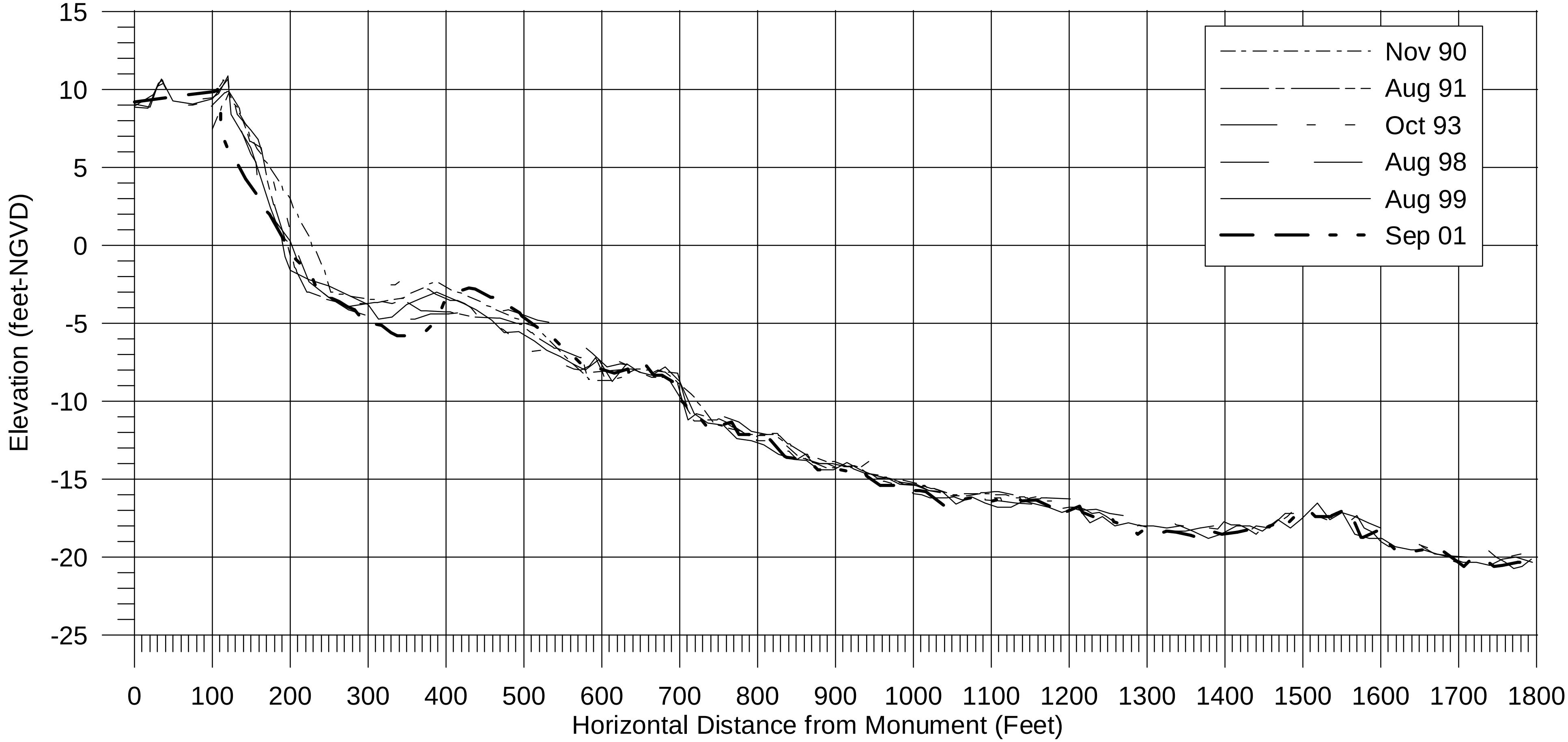

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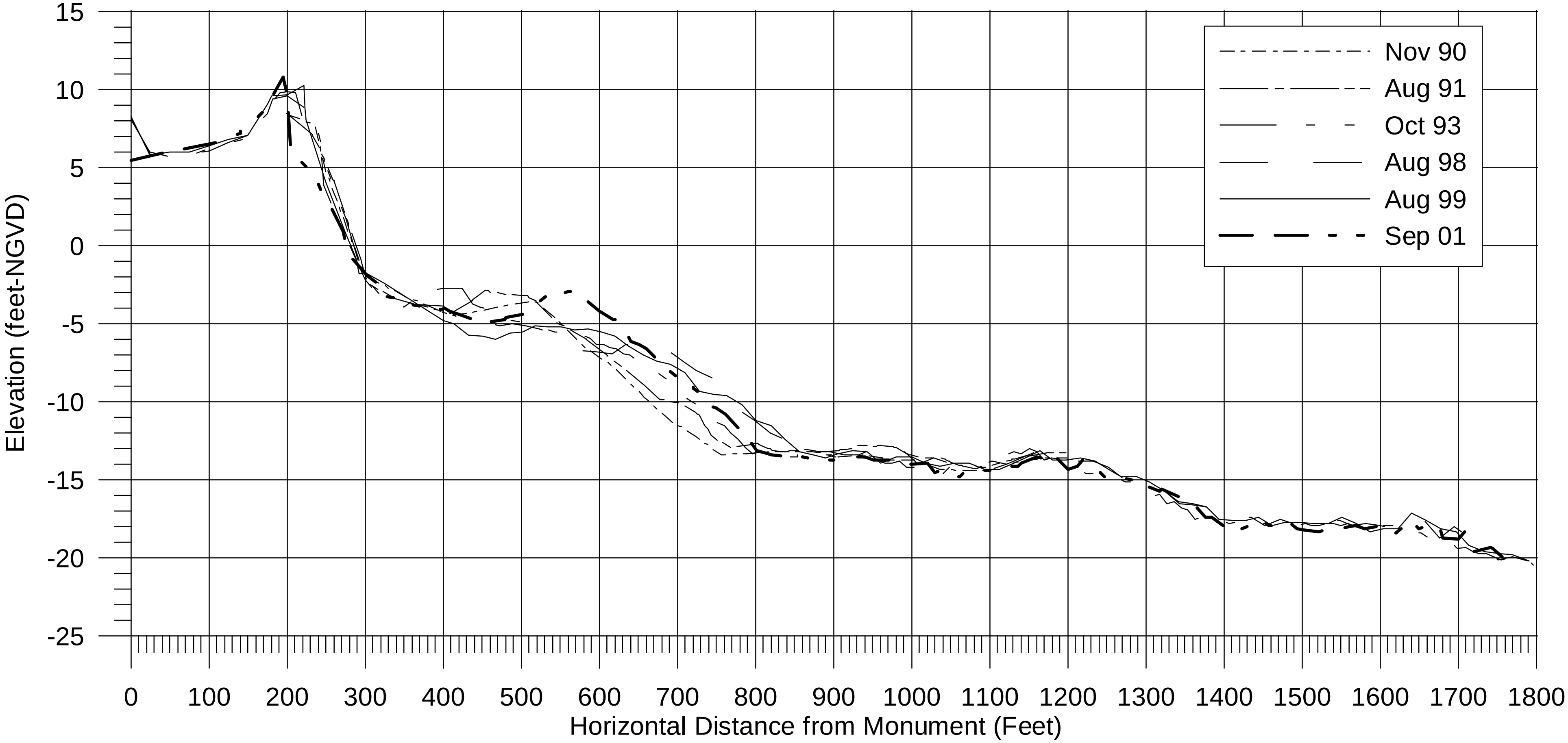

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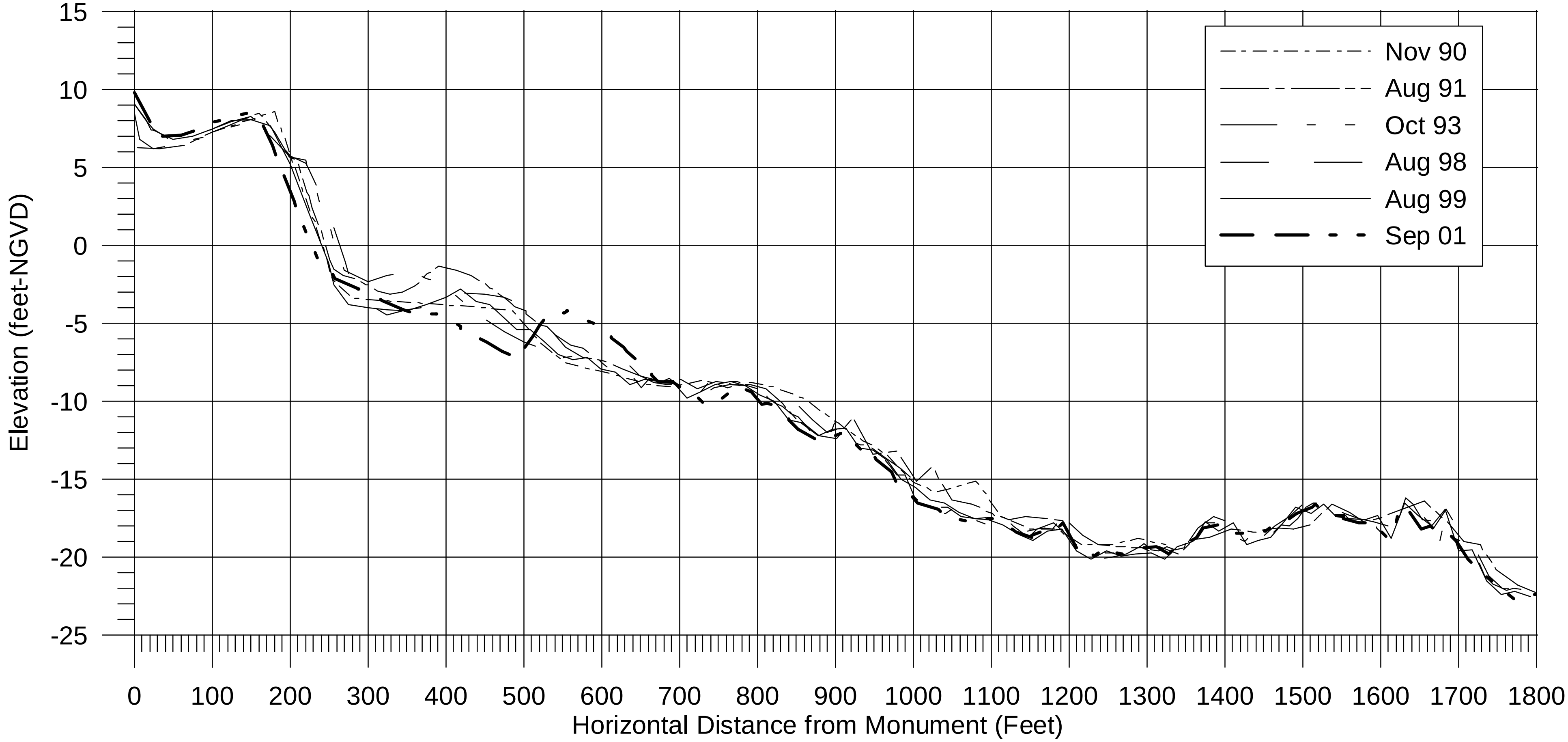
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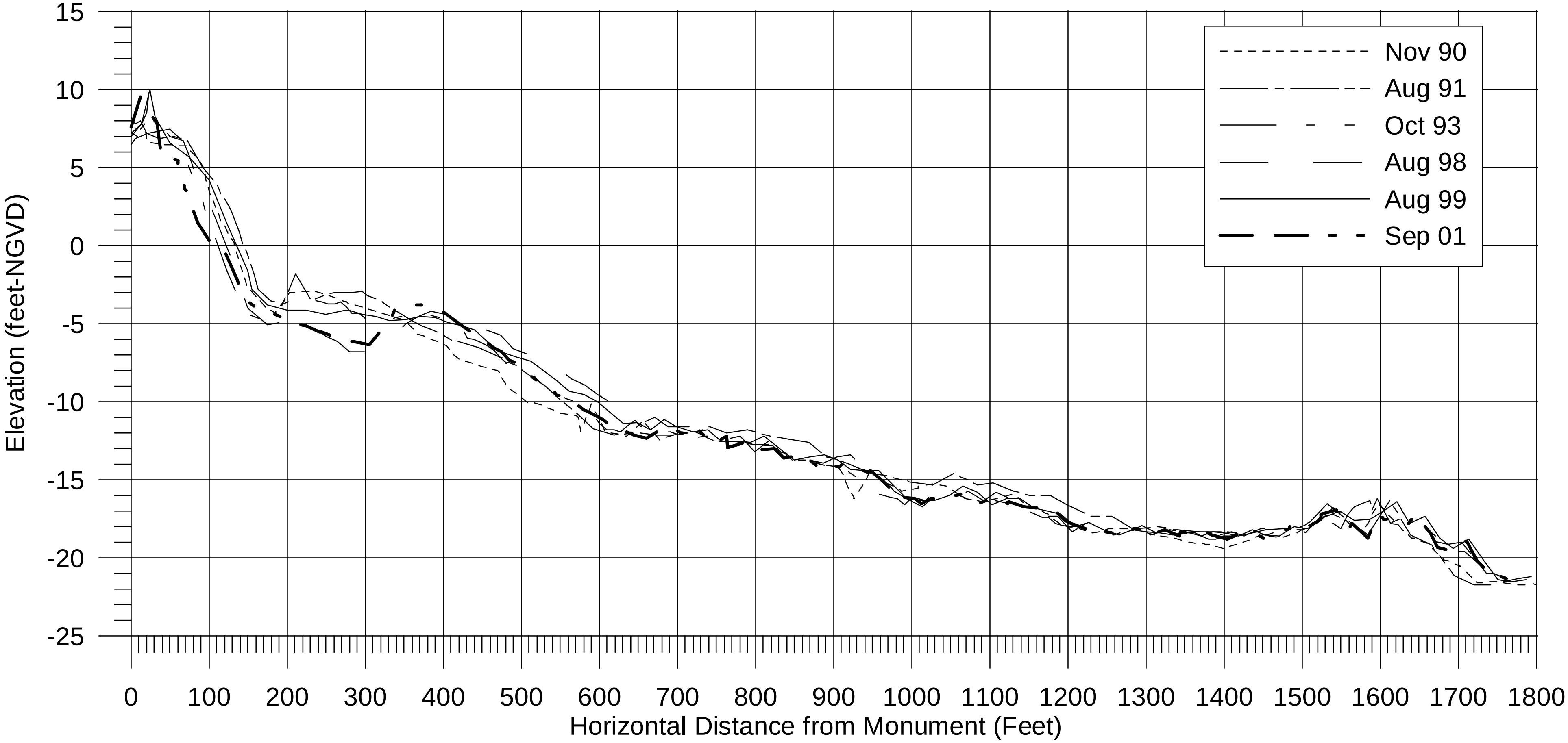

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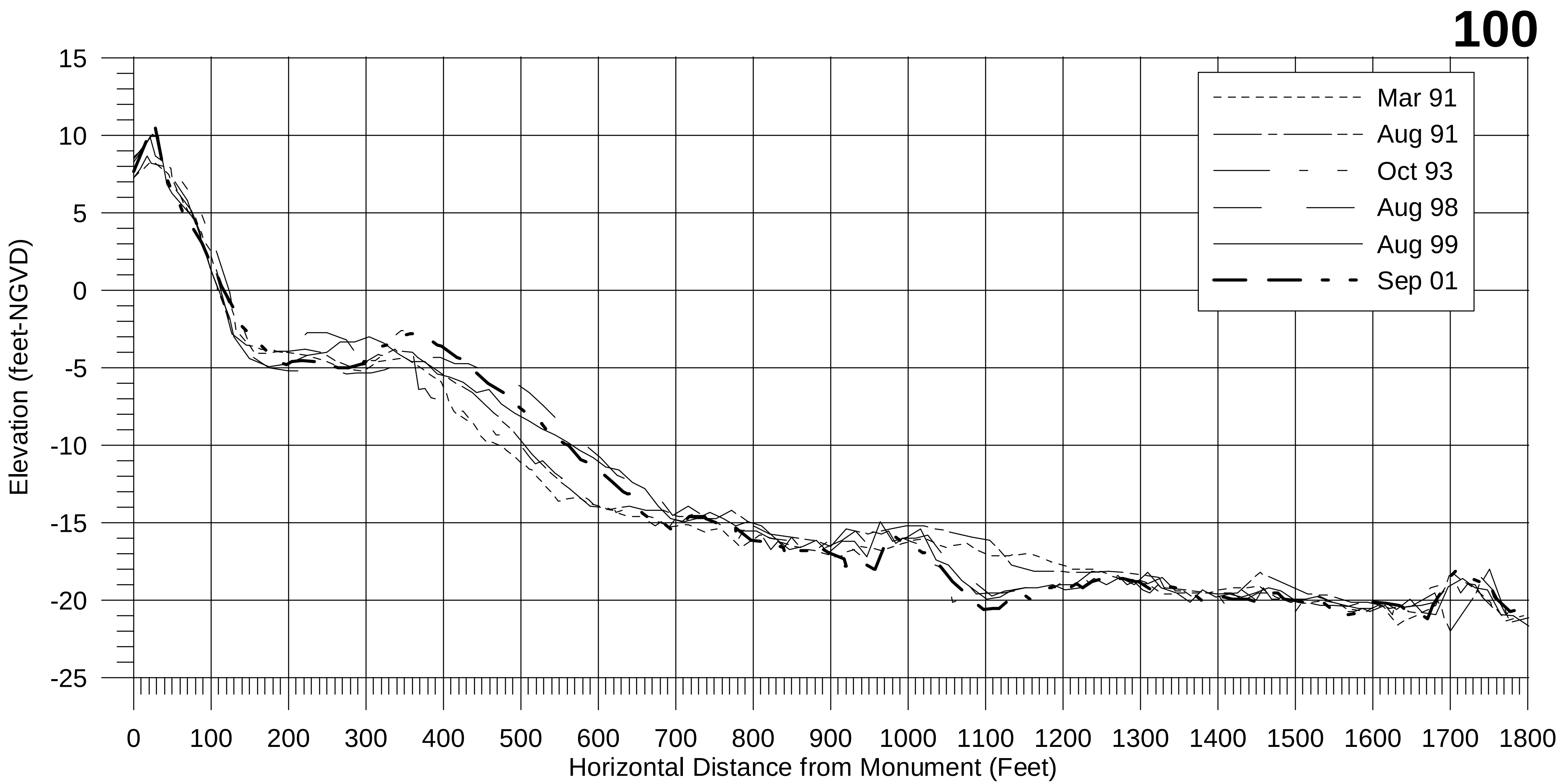
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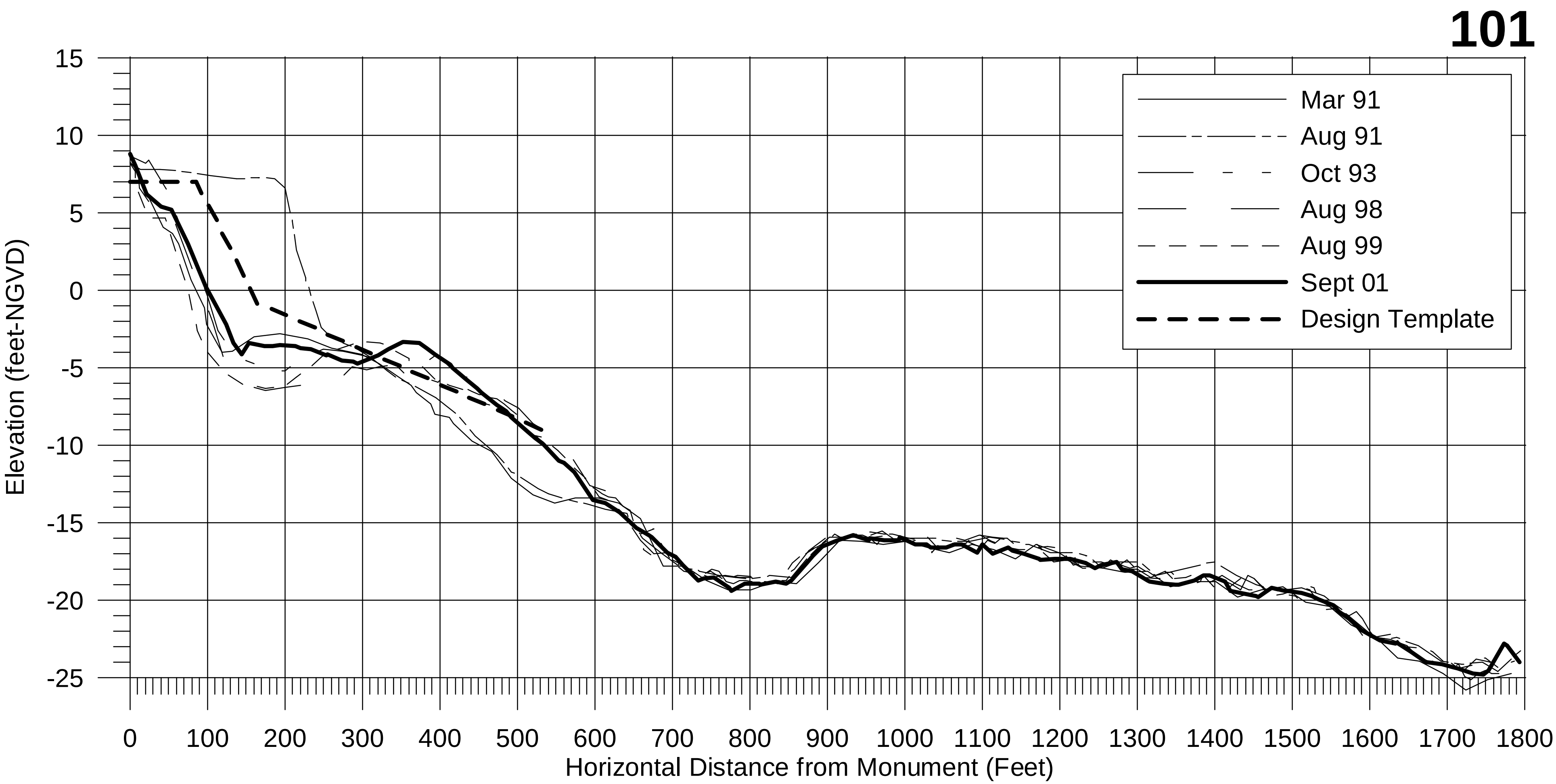

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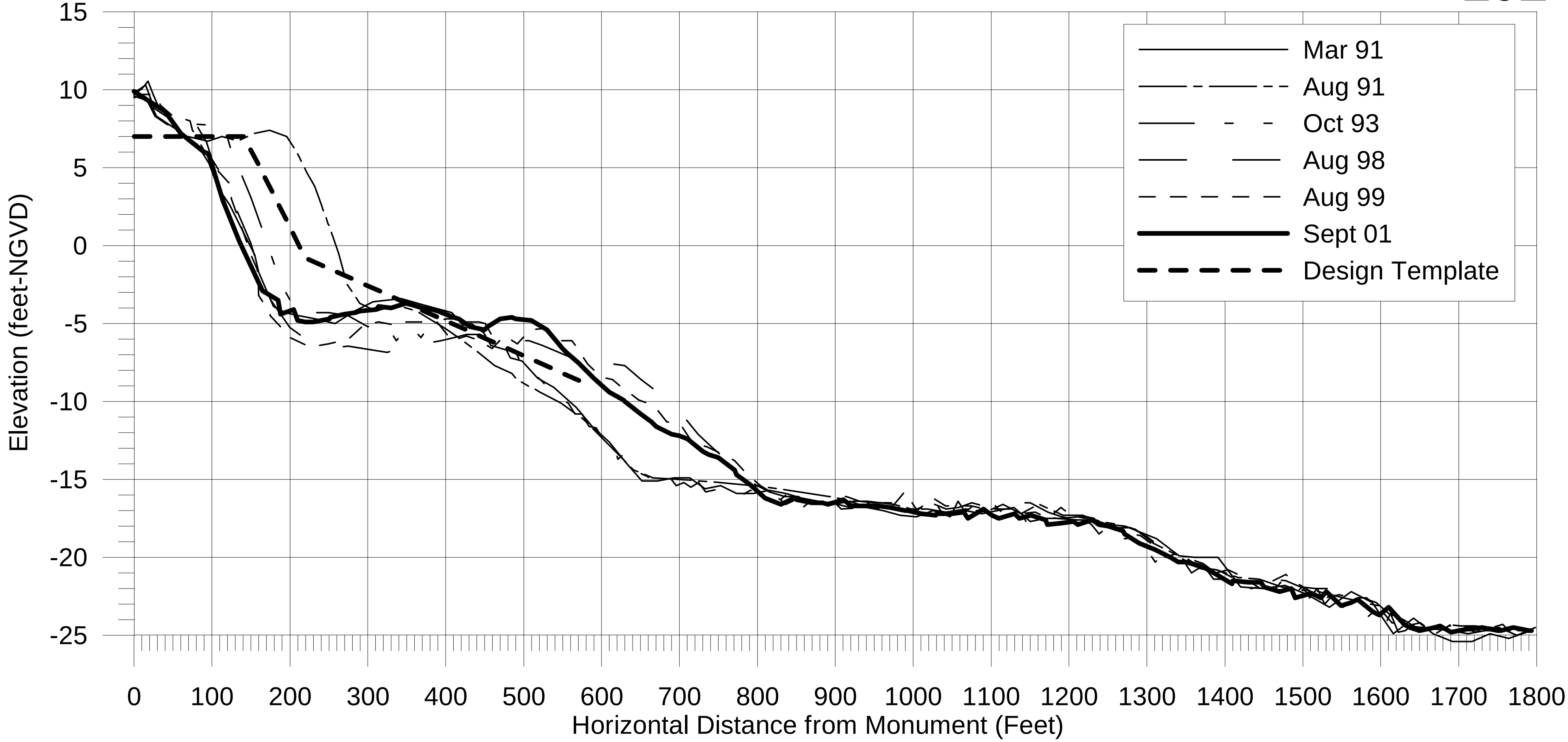

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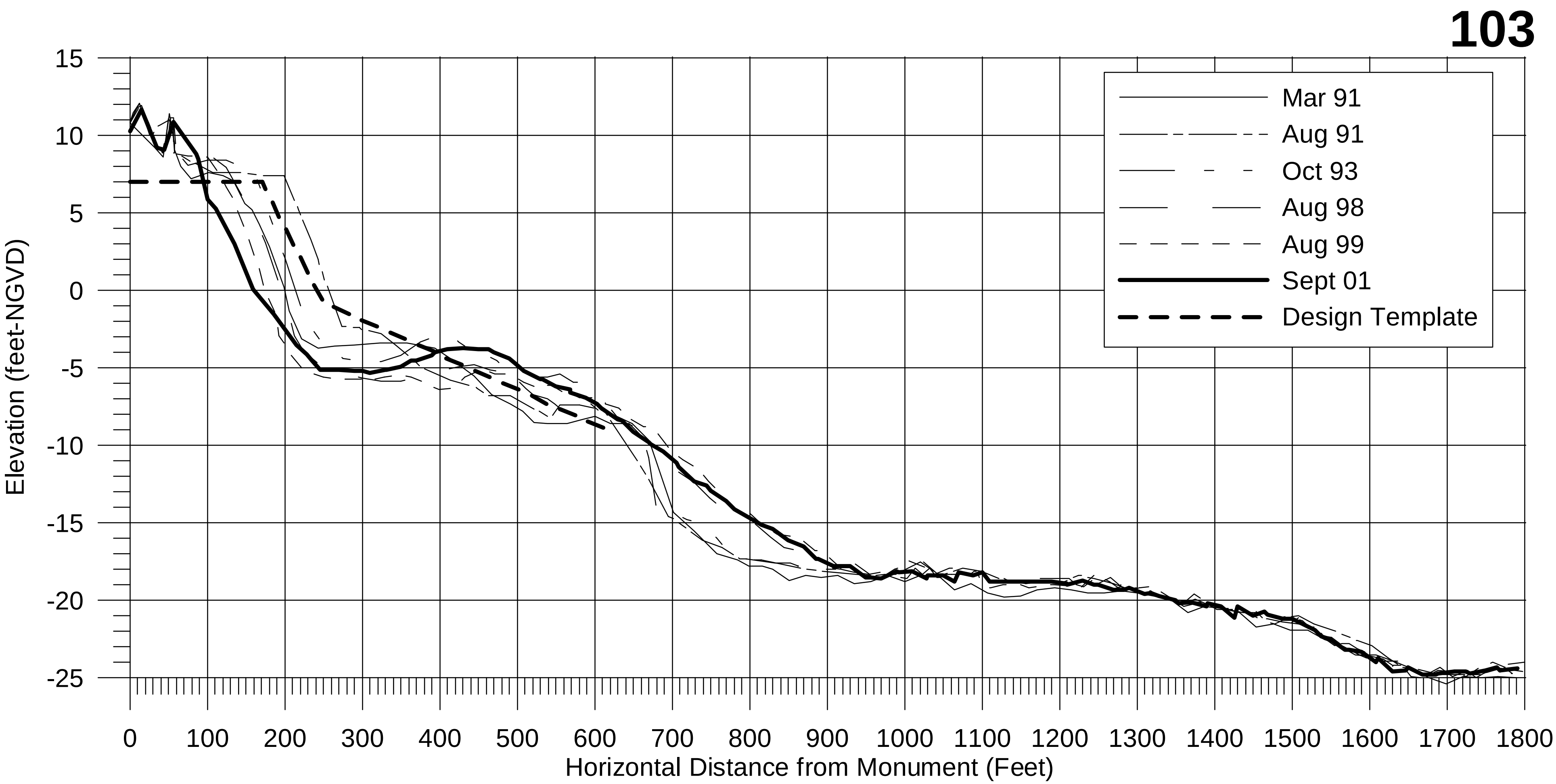

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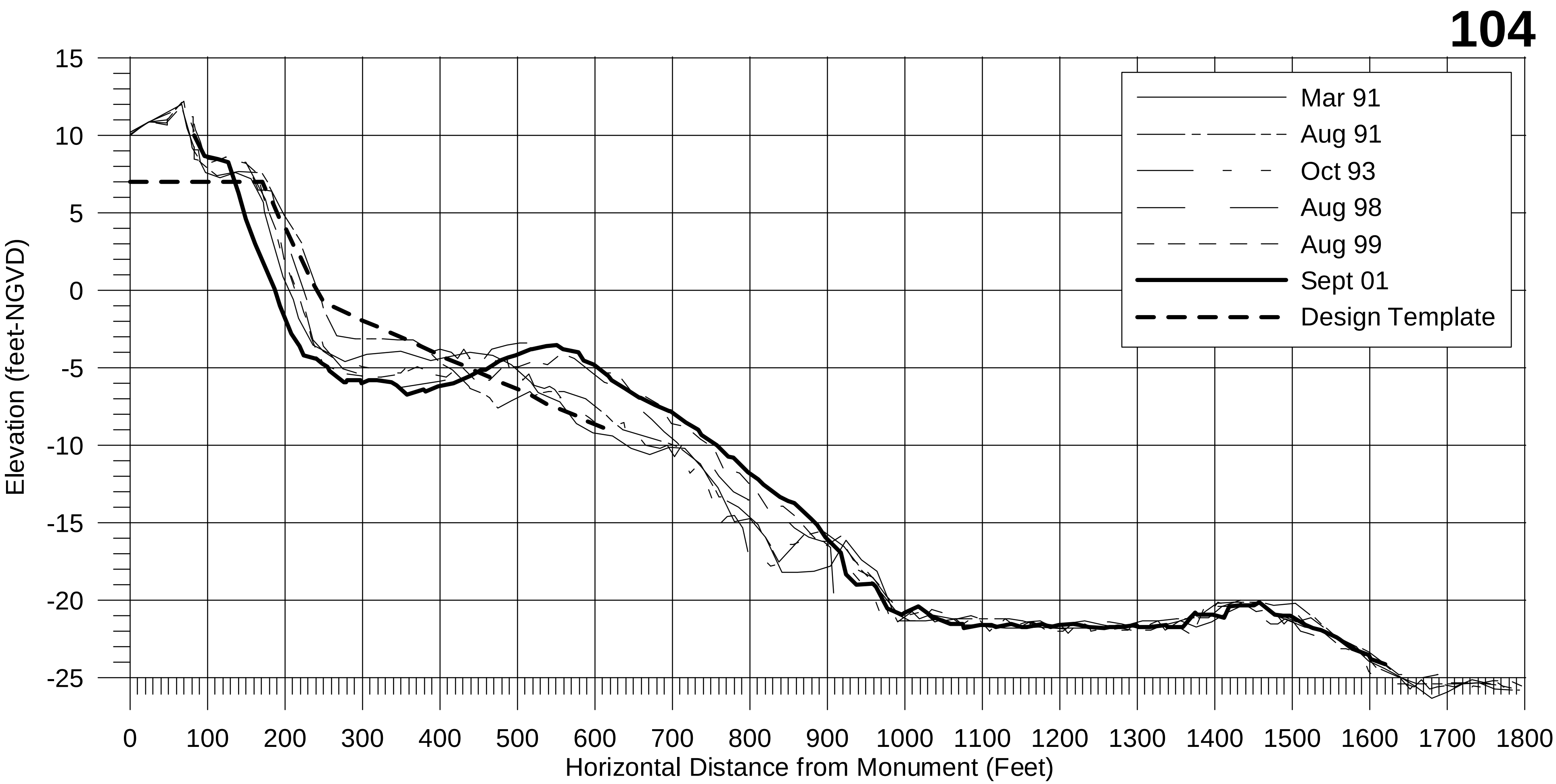
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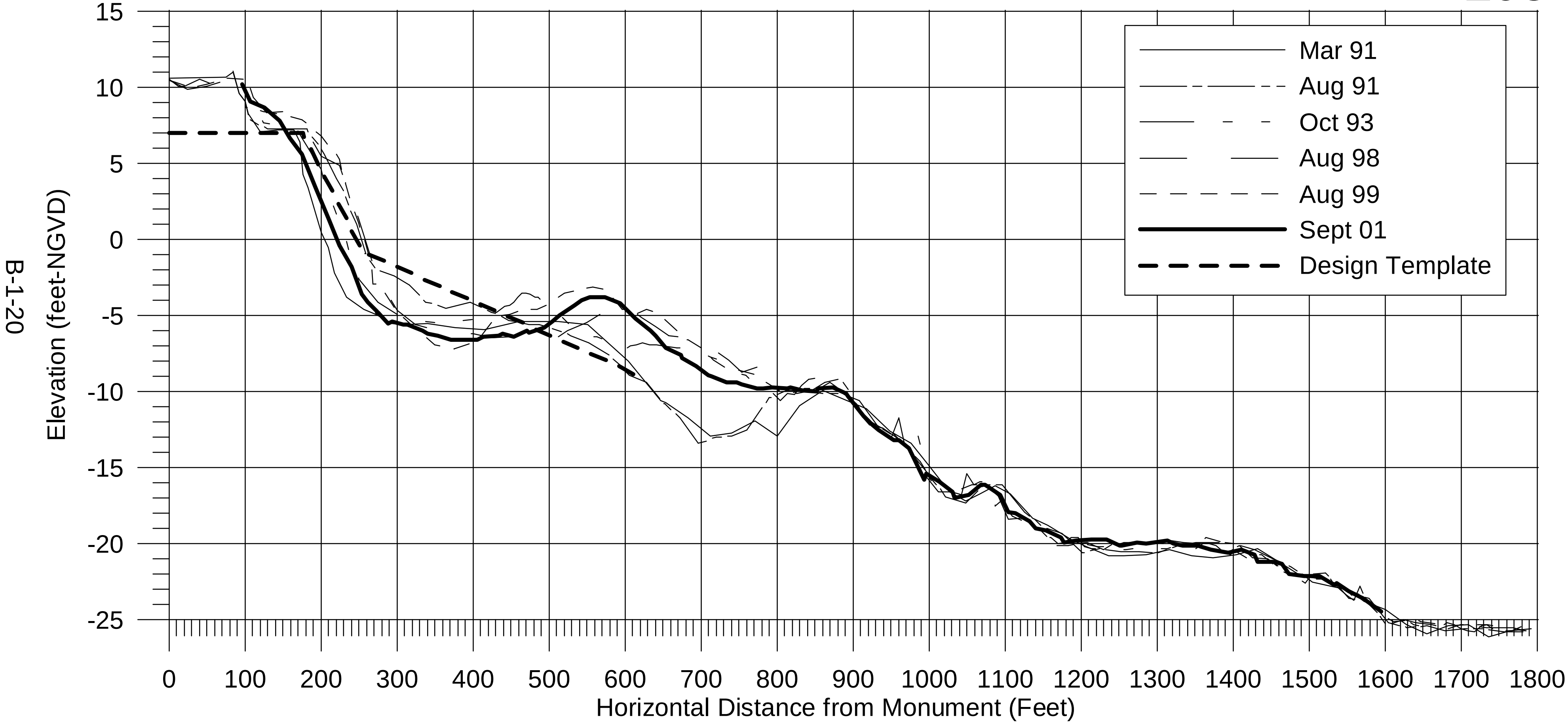

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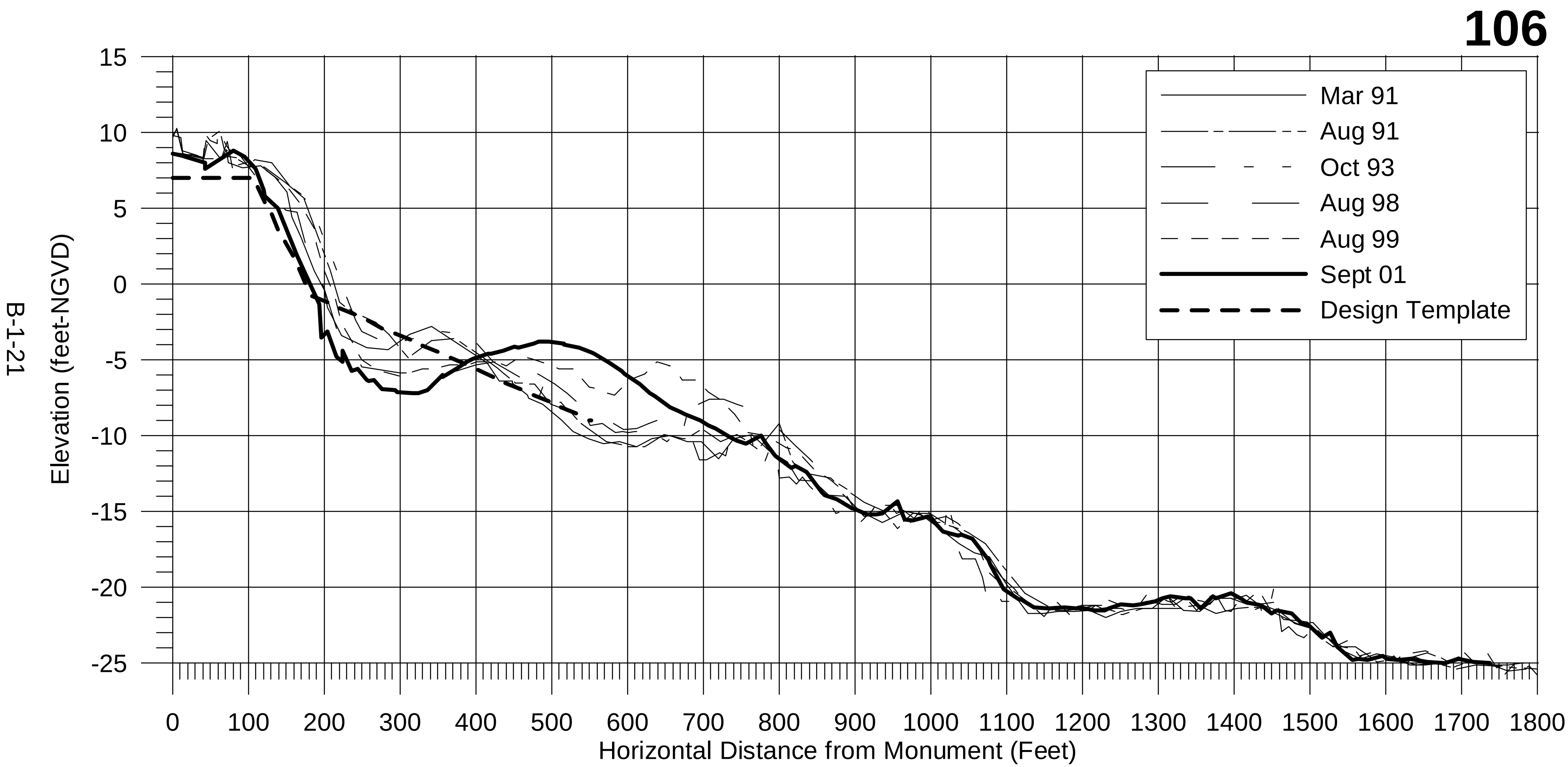
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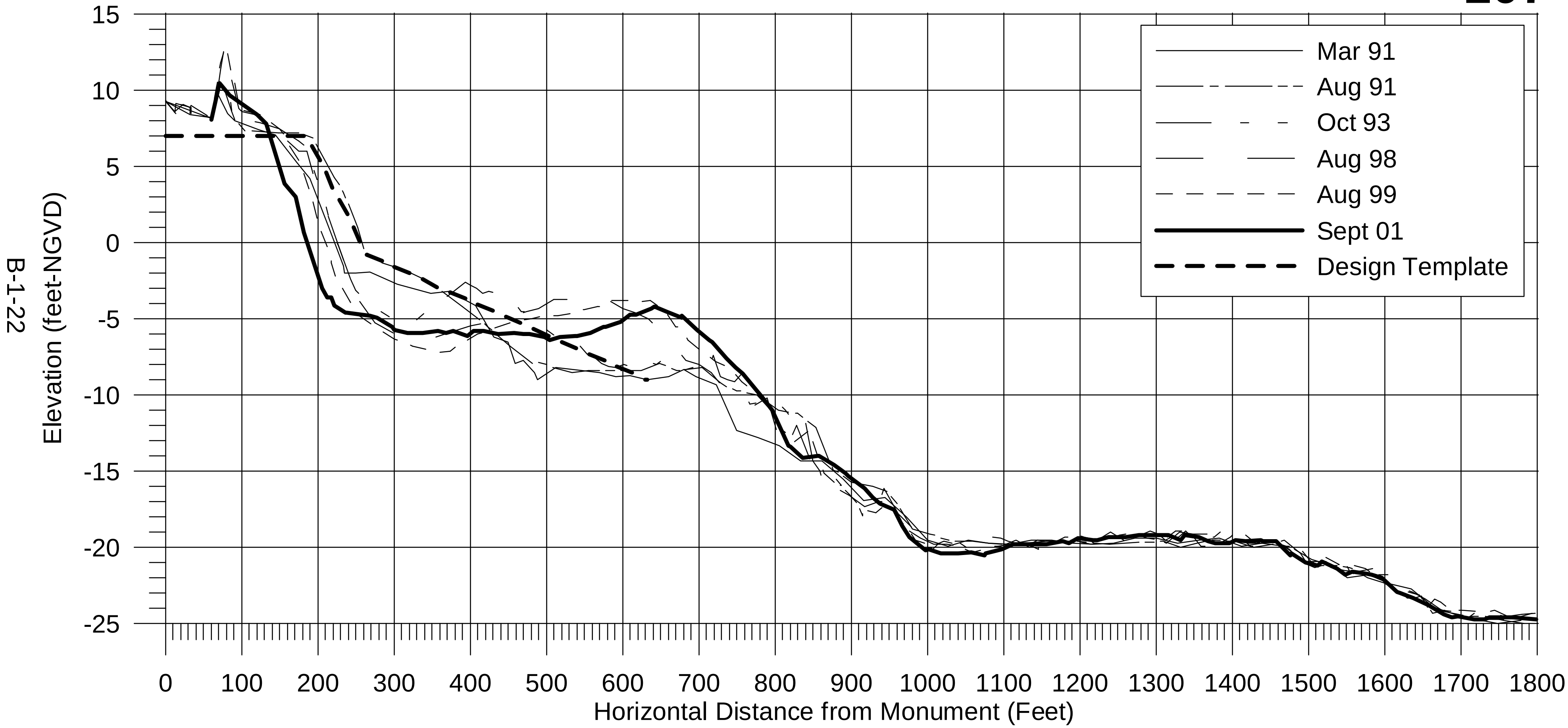

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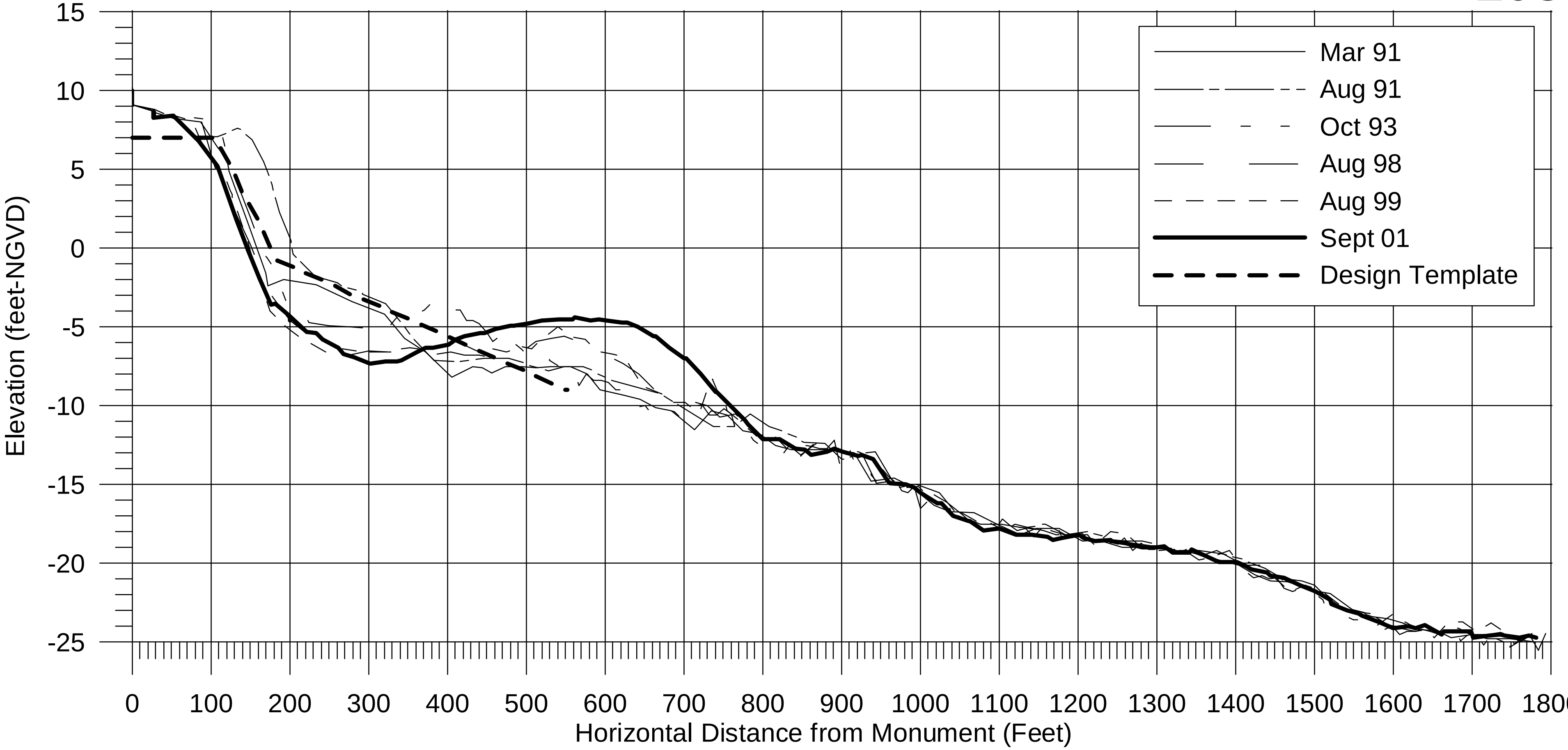

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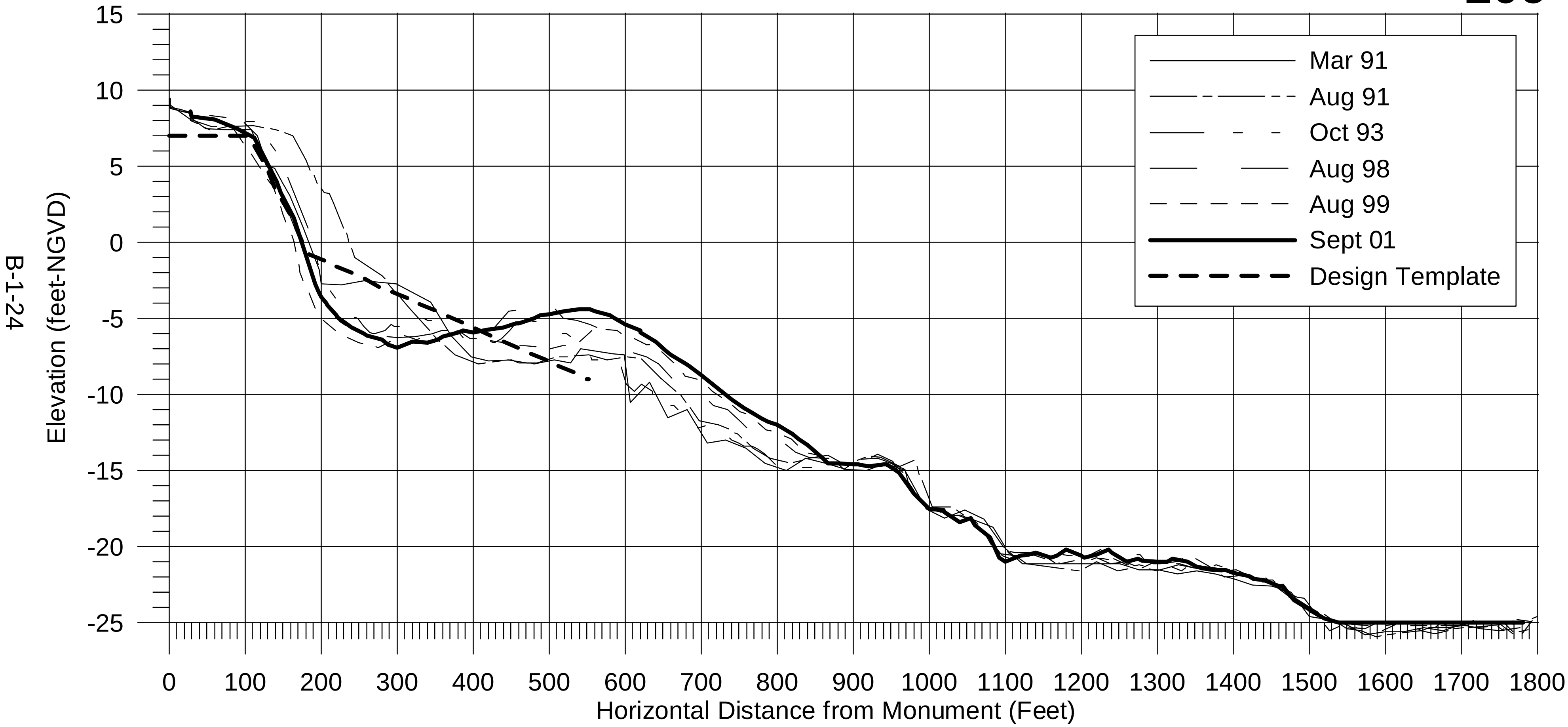
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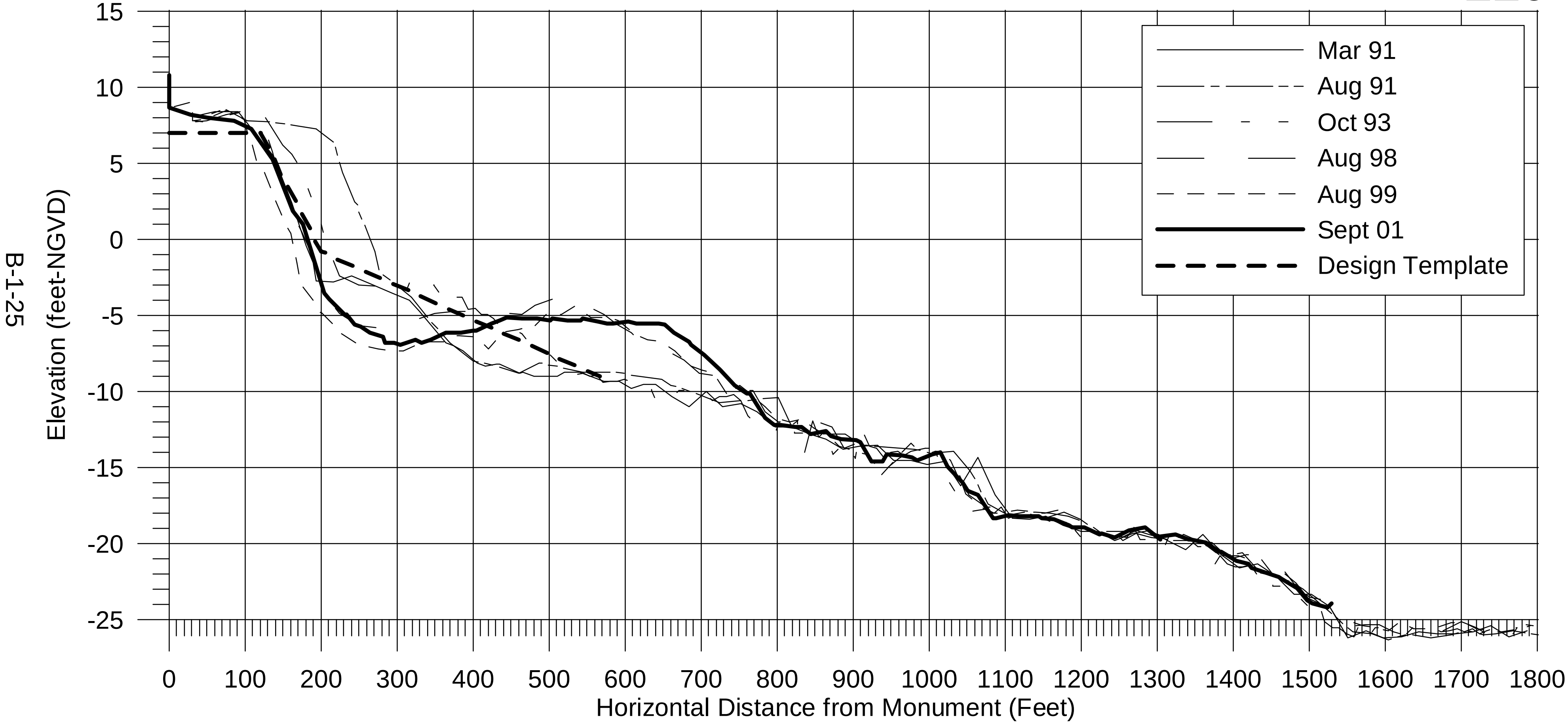
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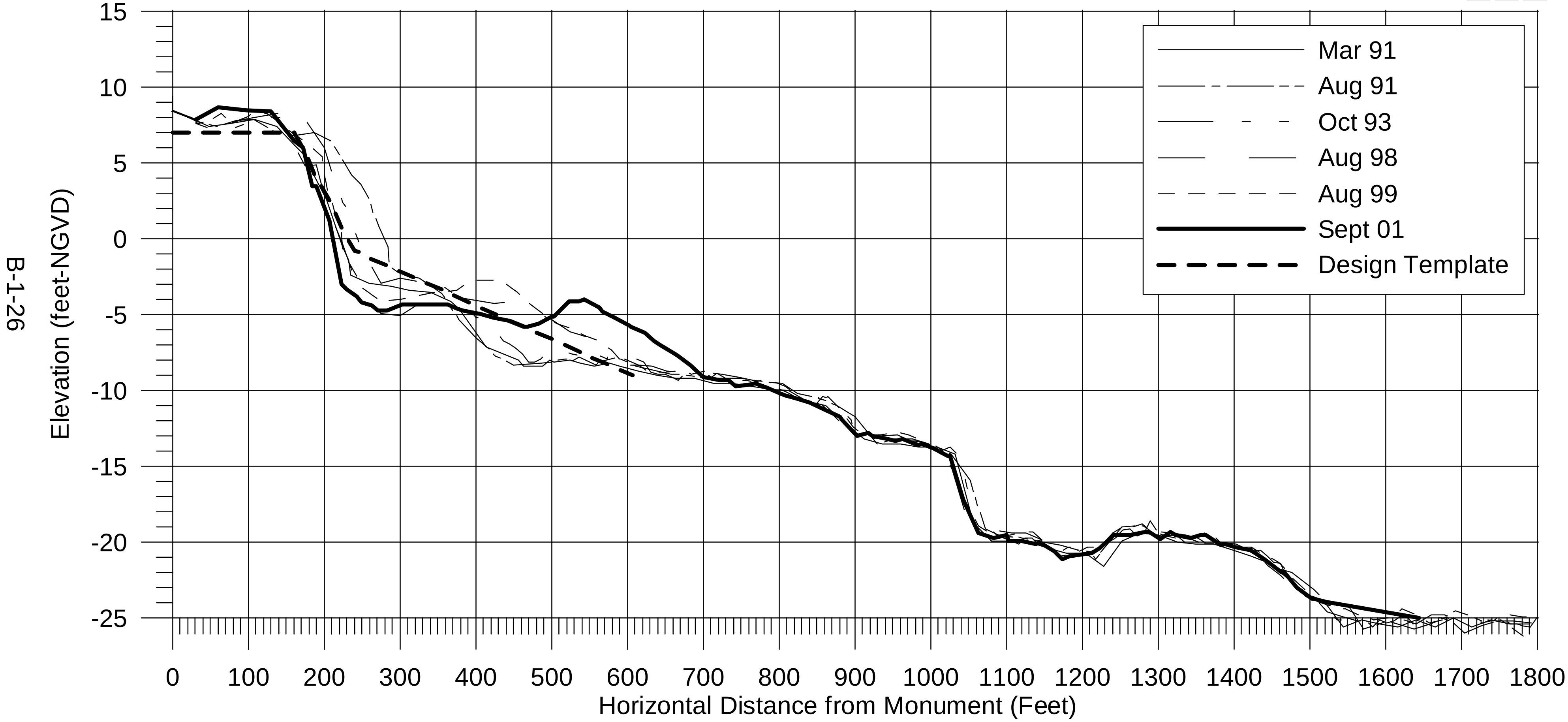
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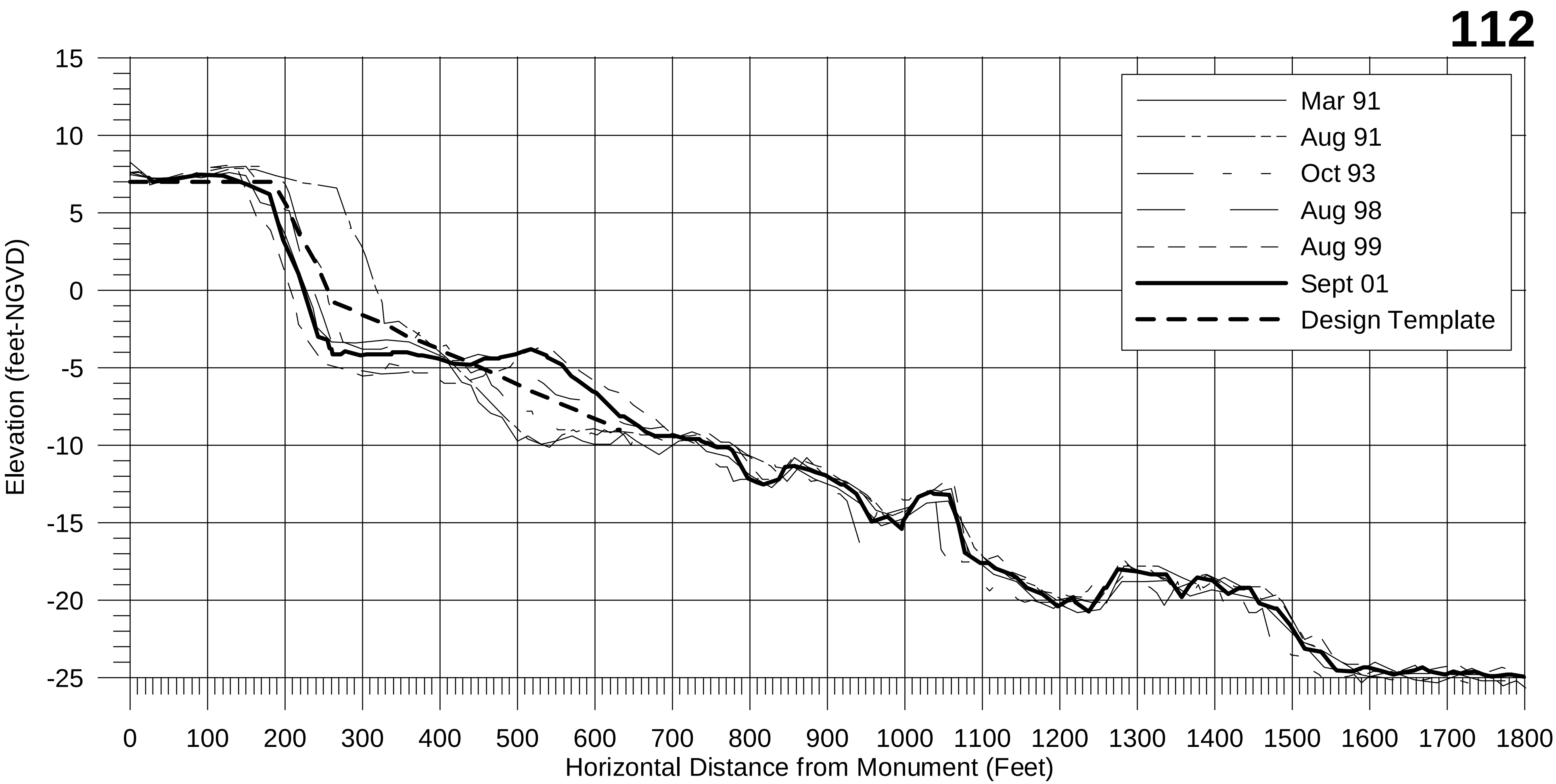
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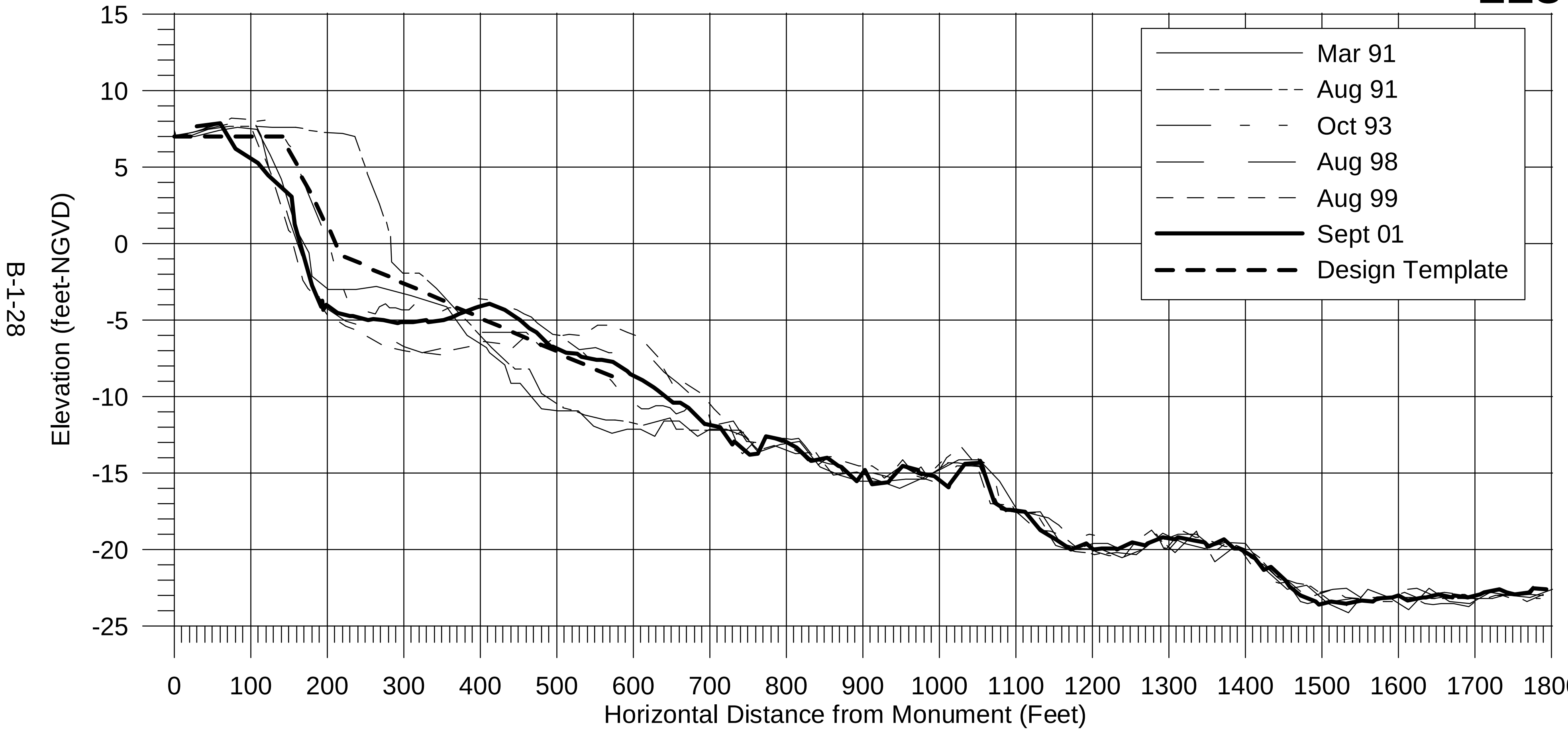

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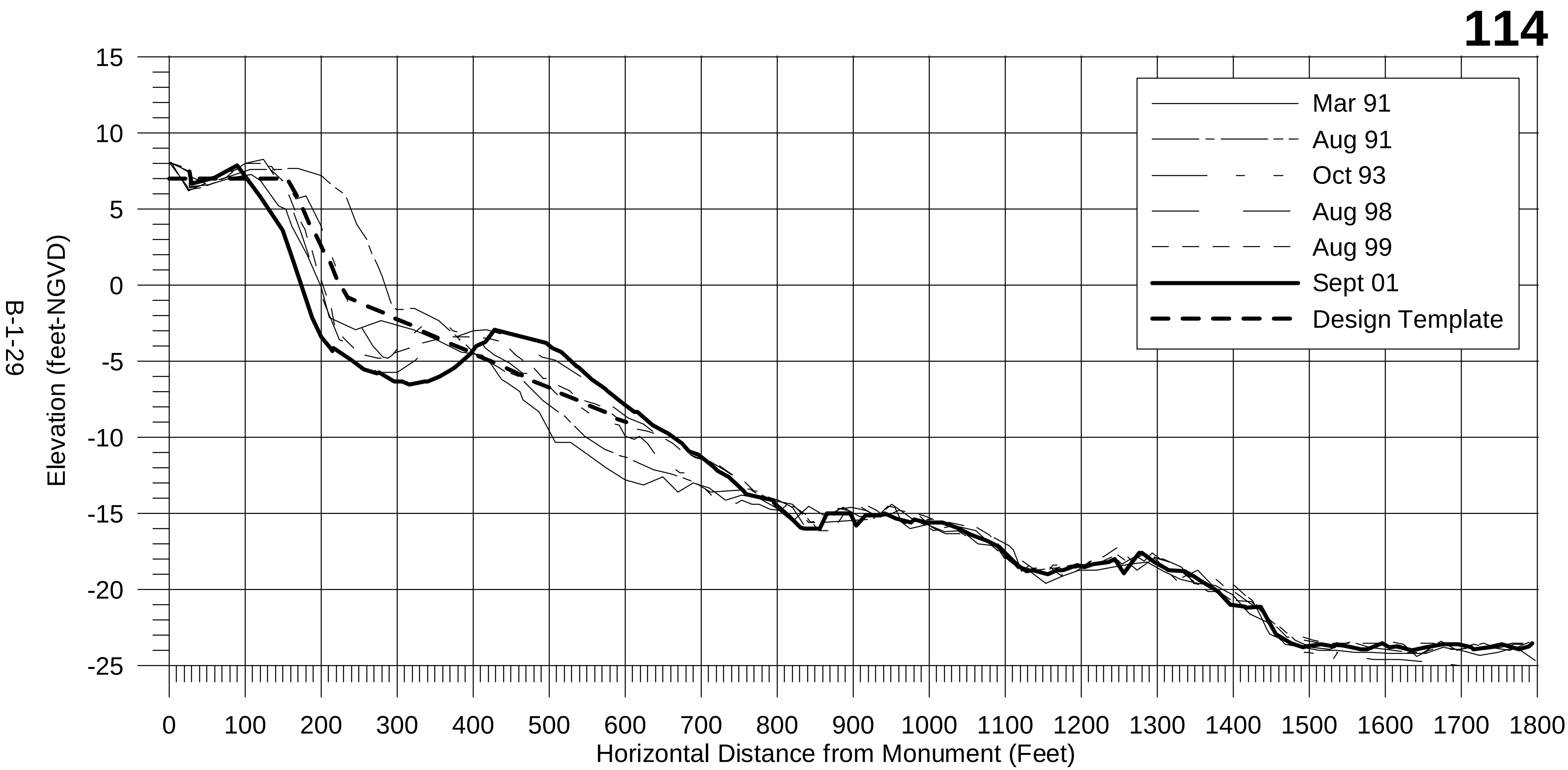

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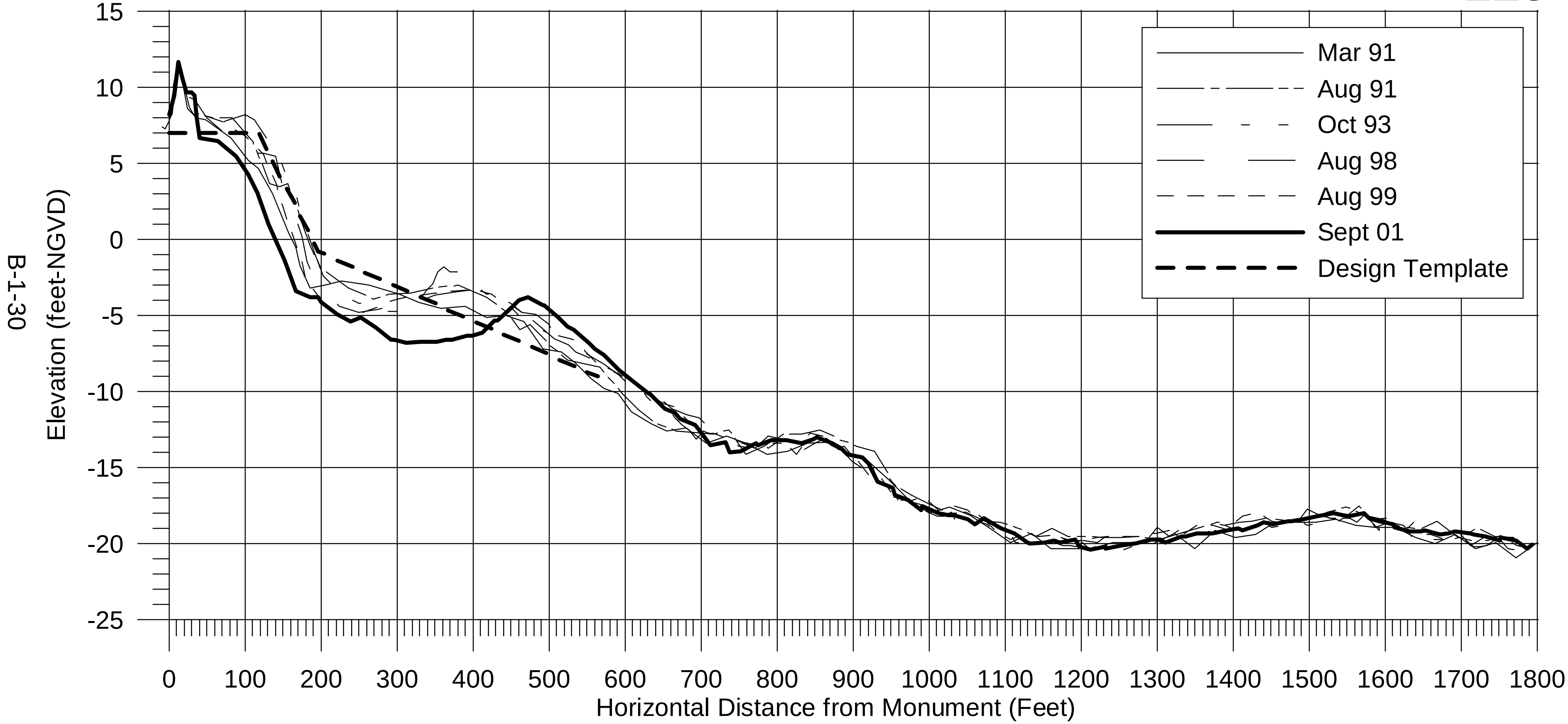
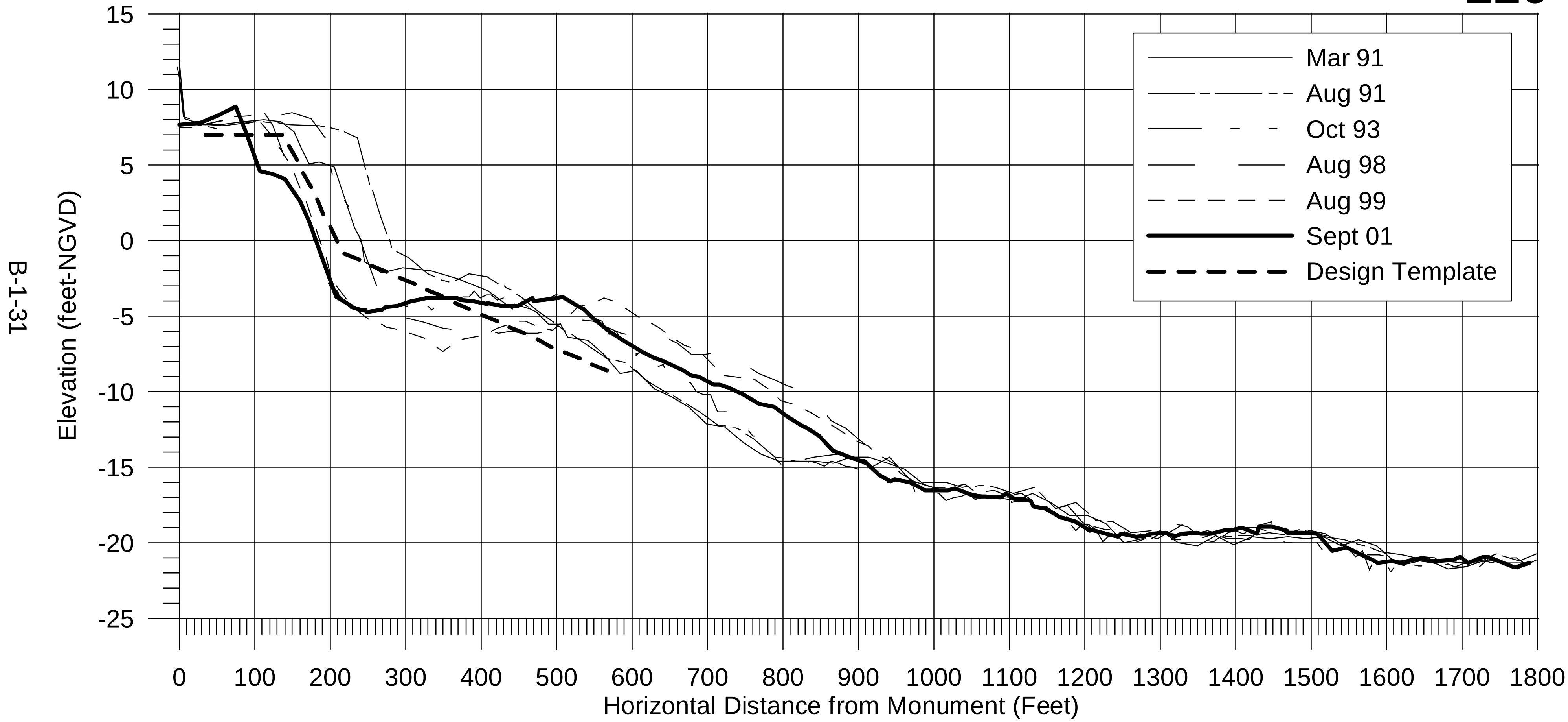
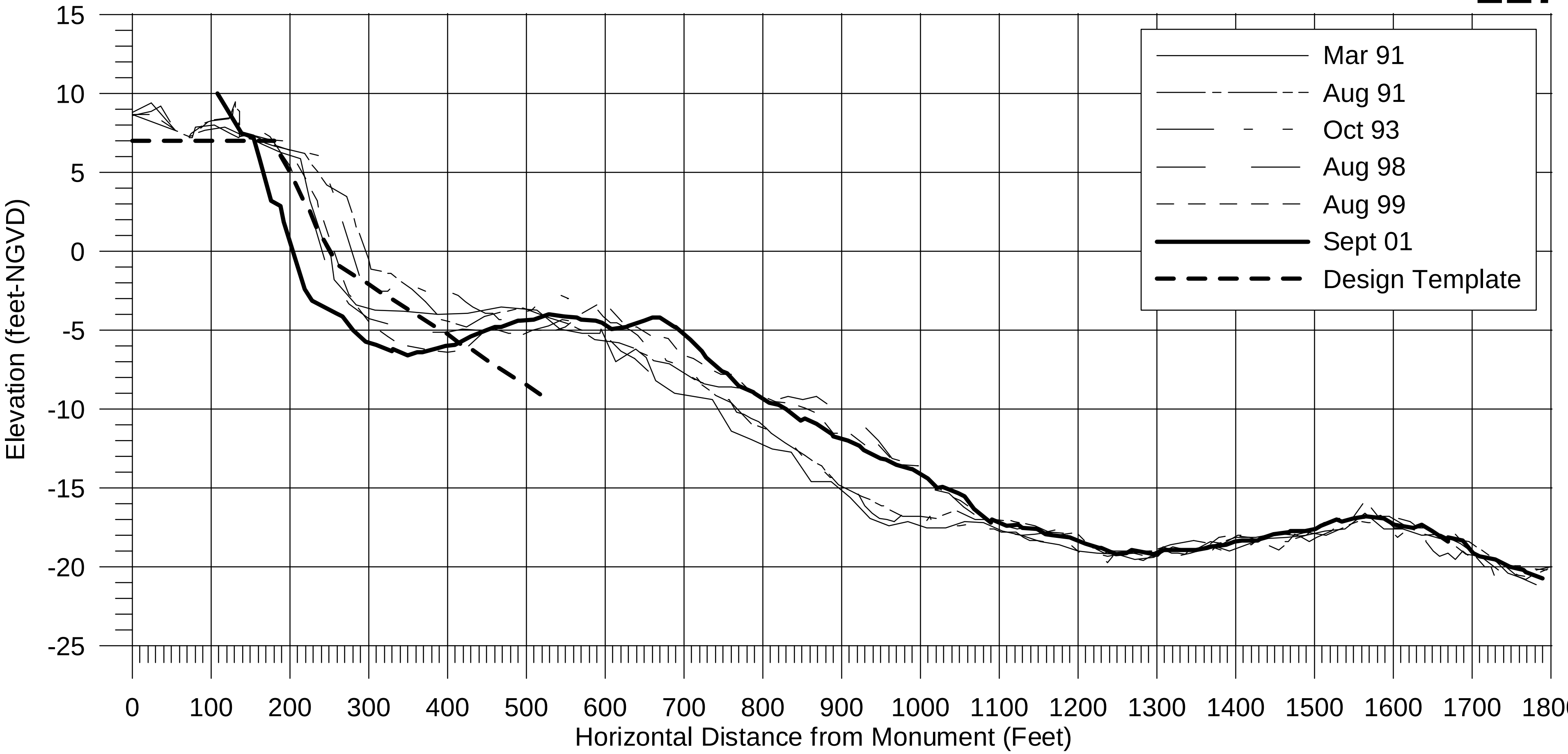
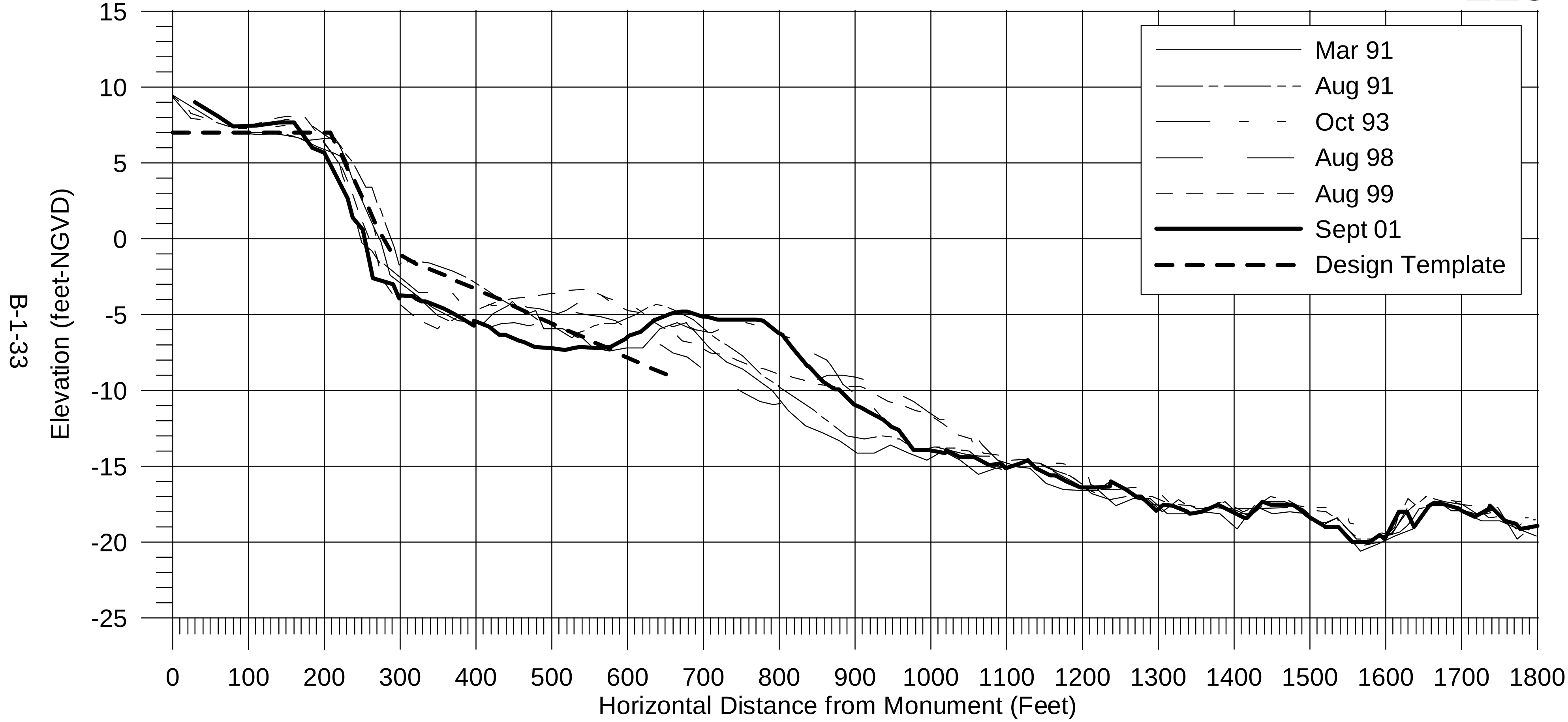
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

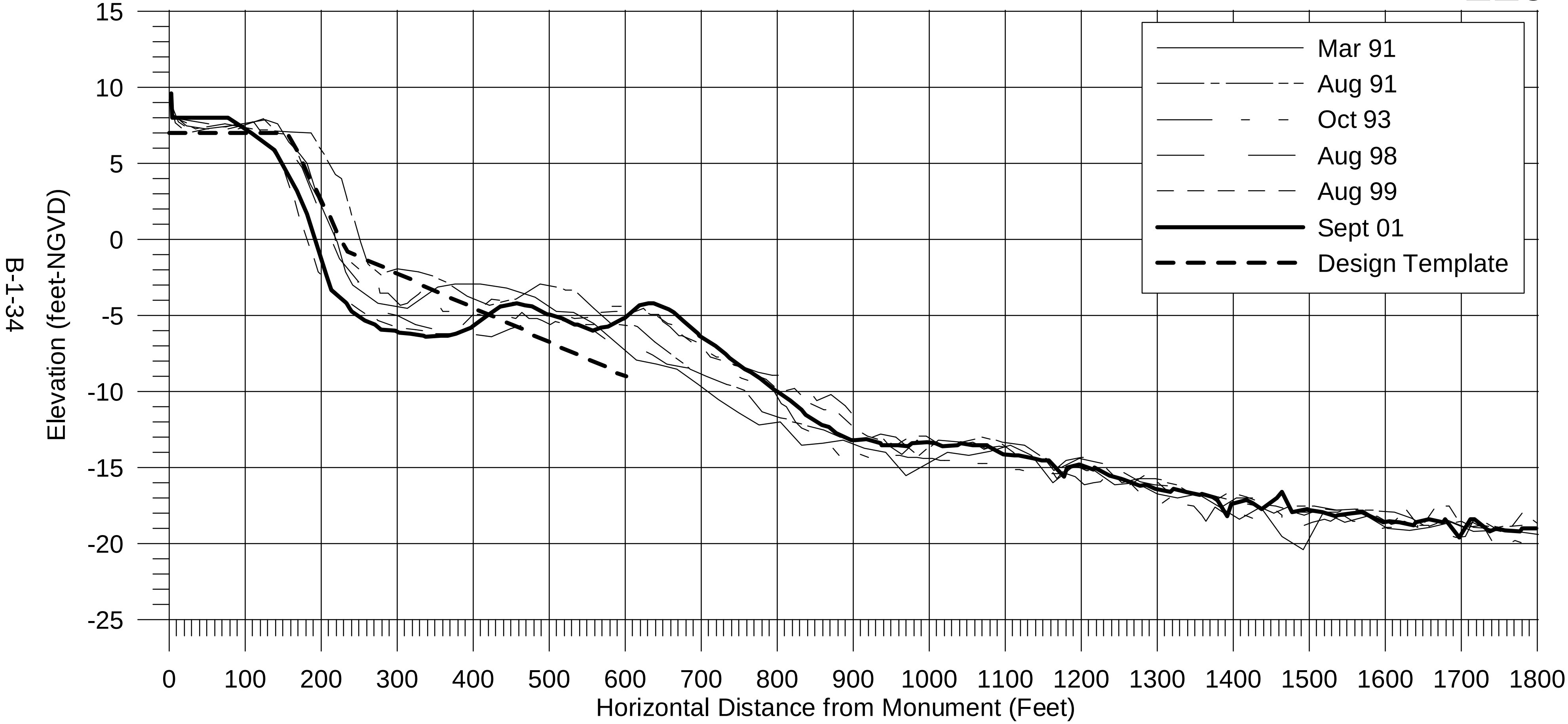

B-1-30
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template


B-1-31
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template


Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template
Elevation (feet-NGVD)
Horizontal Distance from Monument (Feet)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800

B-1-32


B-1-33
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template


B-1-34
Elevation (feet-NGVD)
Horizontal Distance from Monument (Feet)
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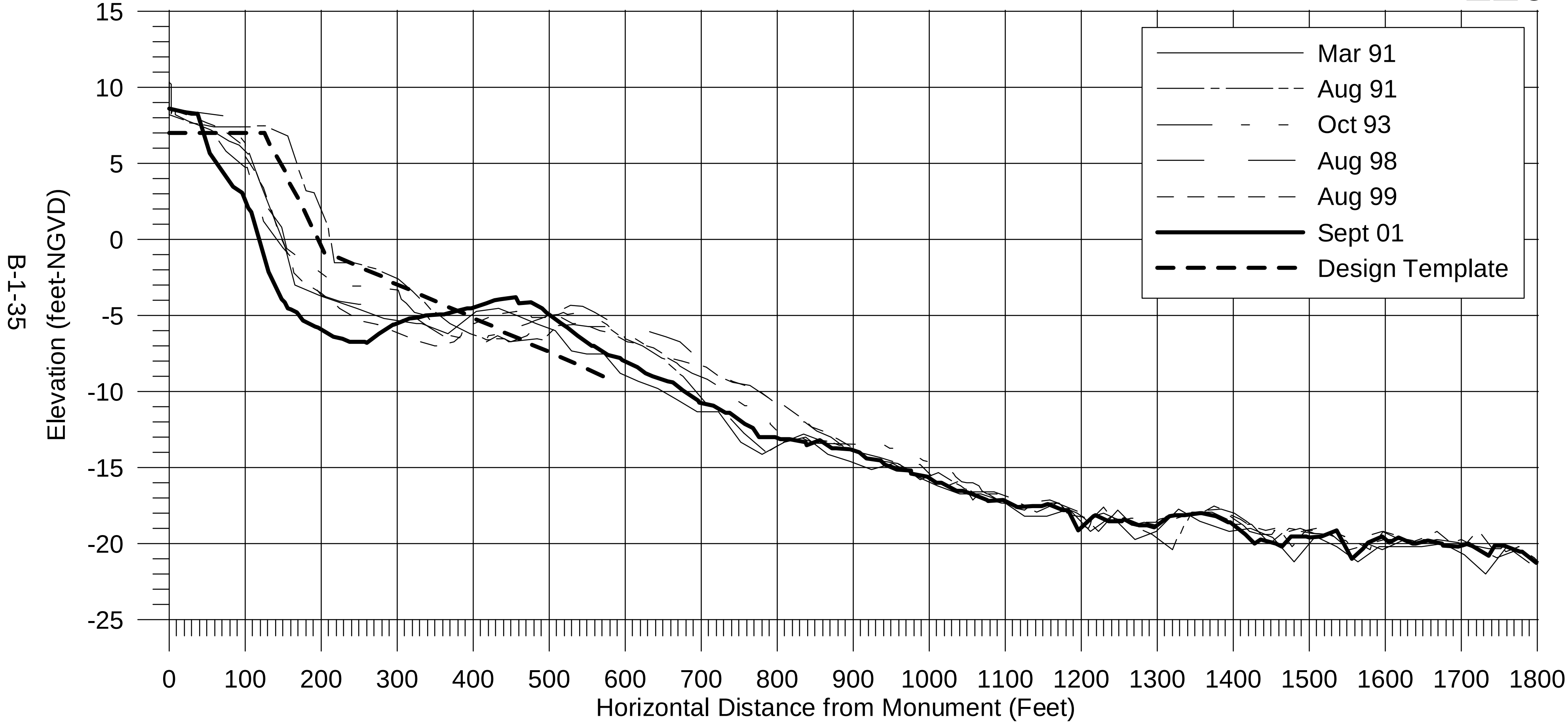
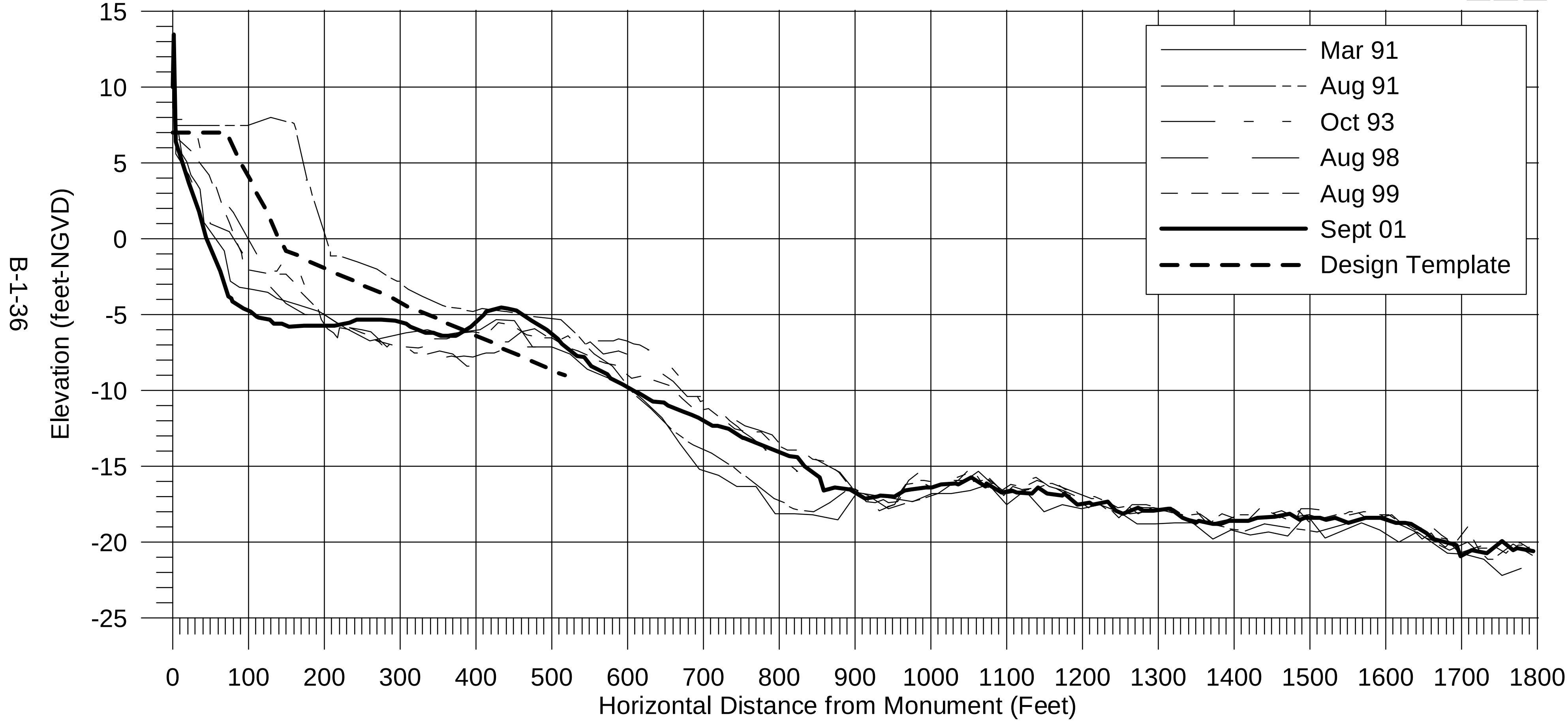
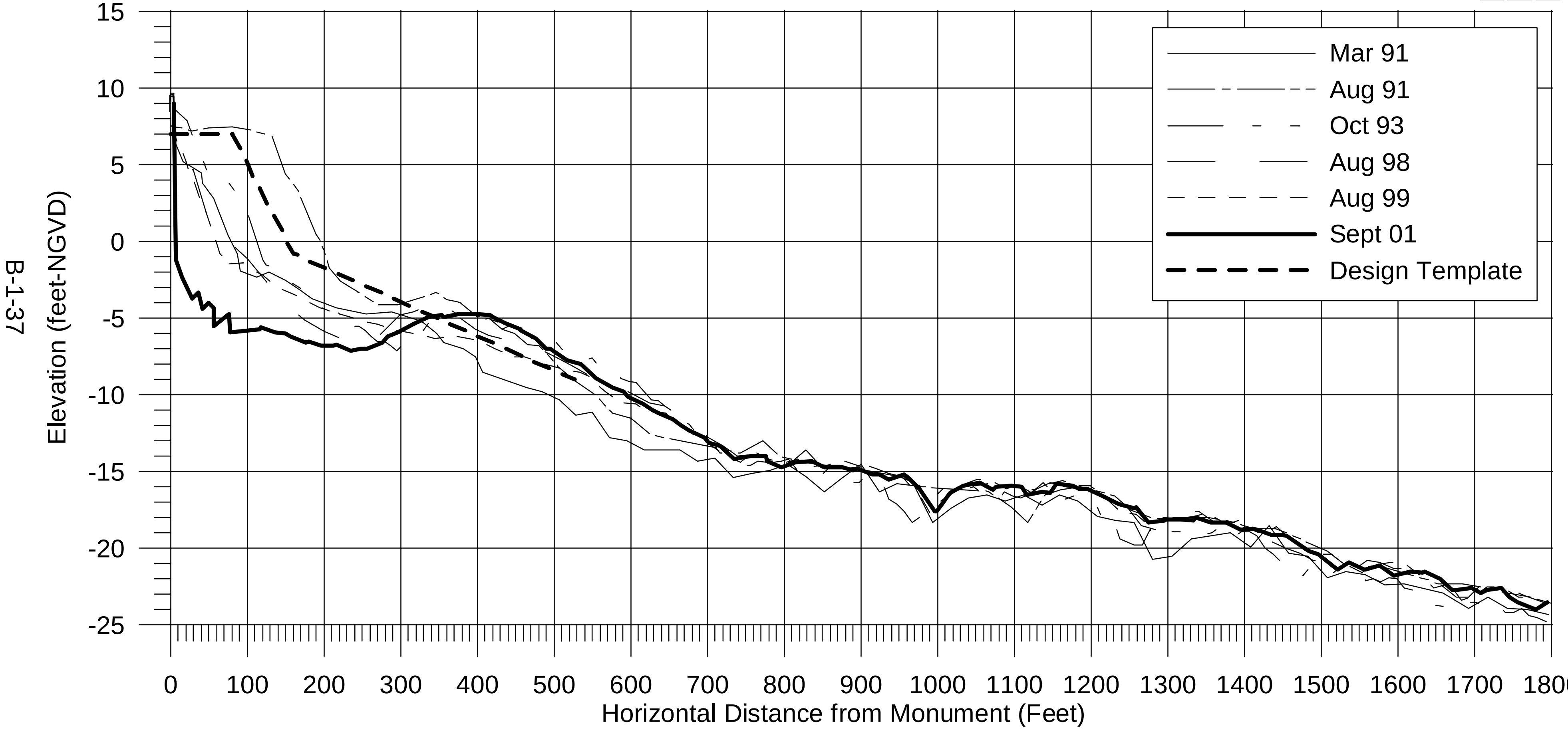
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

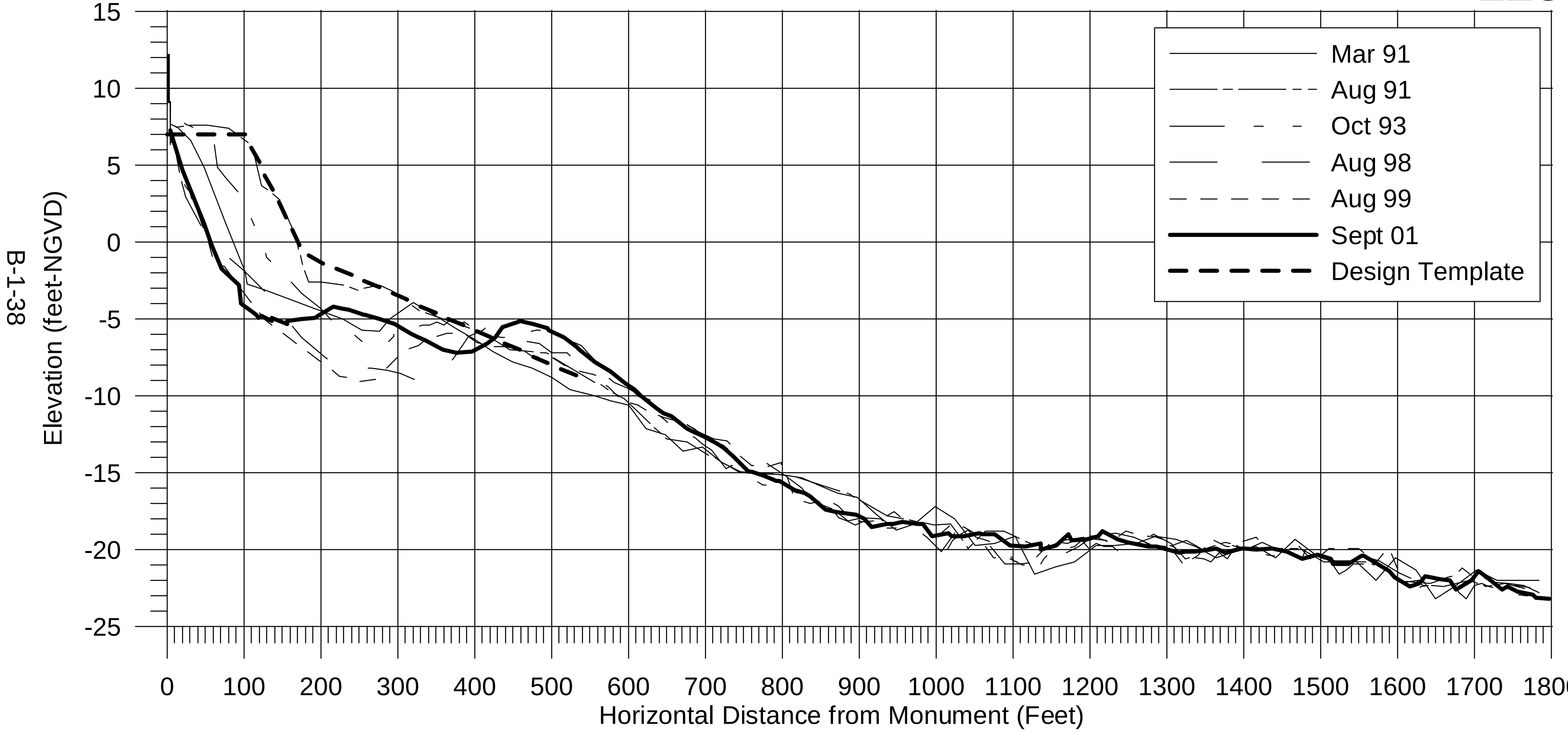

B-1-35
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template


B-1-36
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

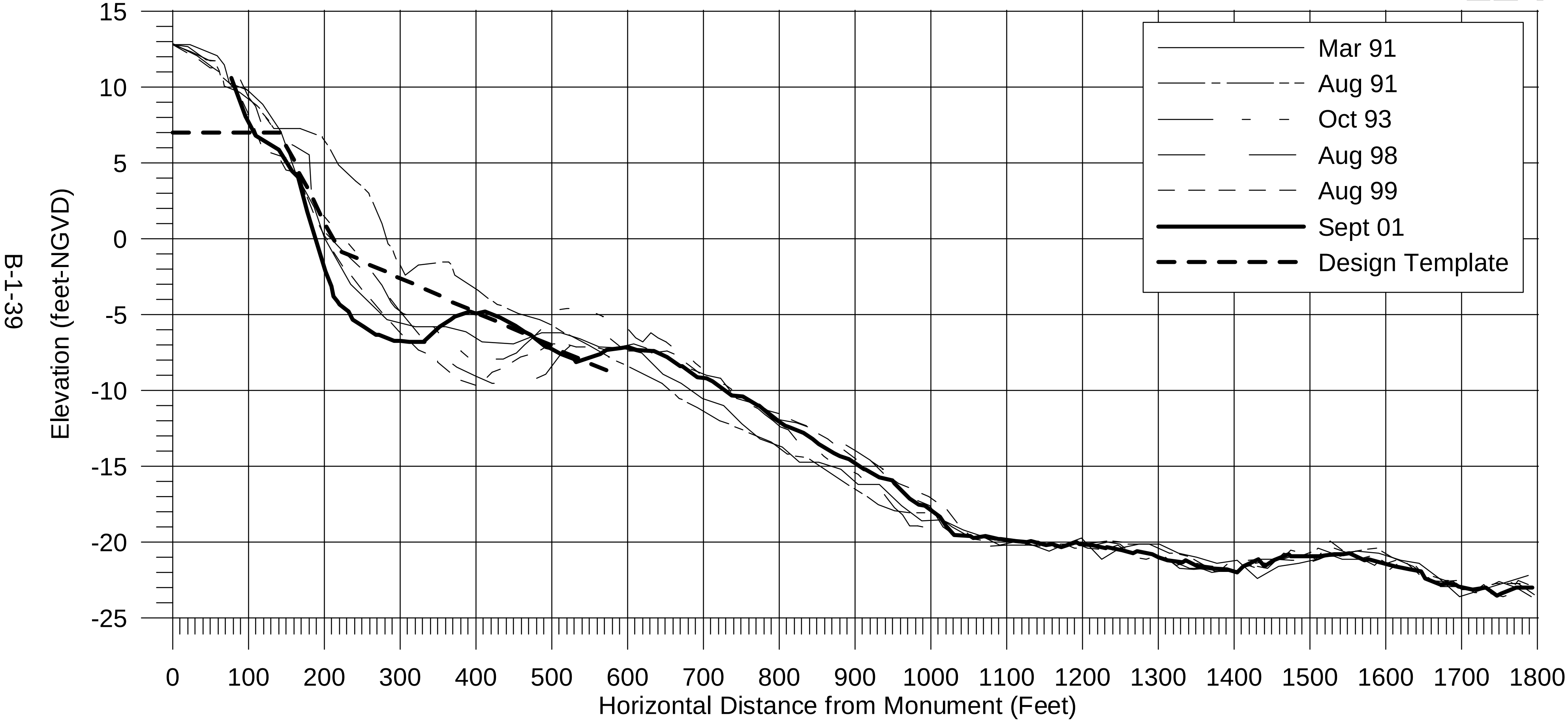


B-1-37
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

123

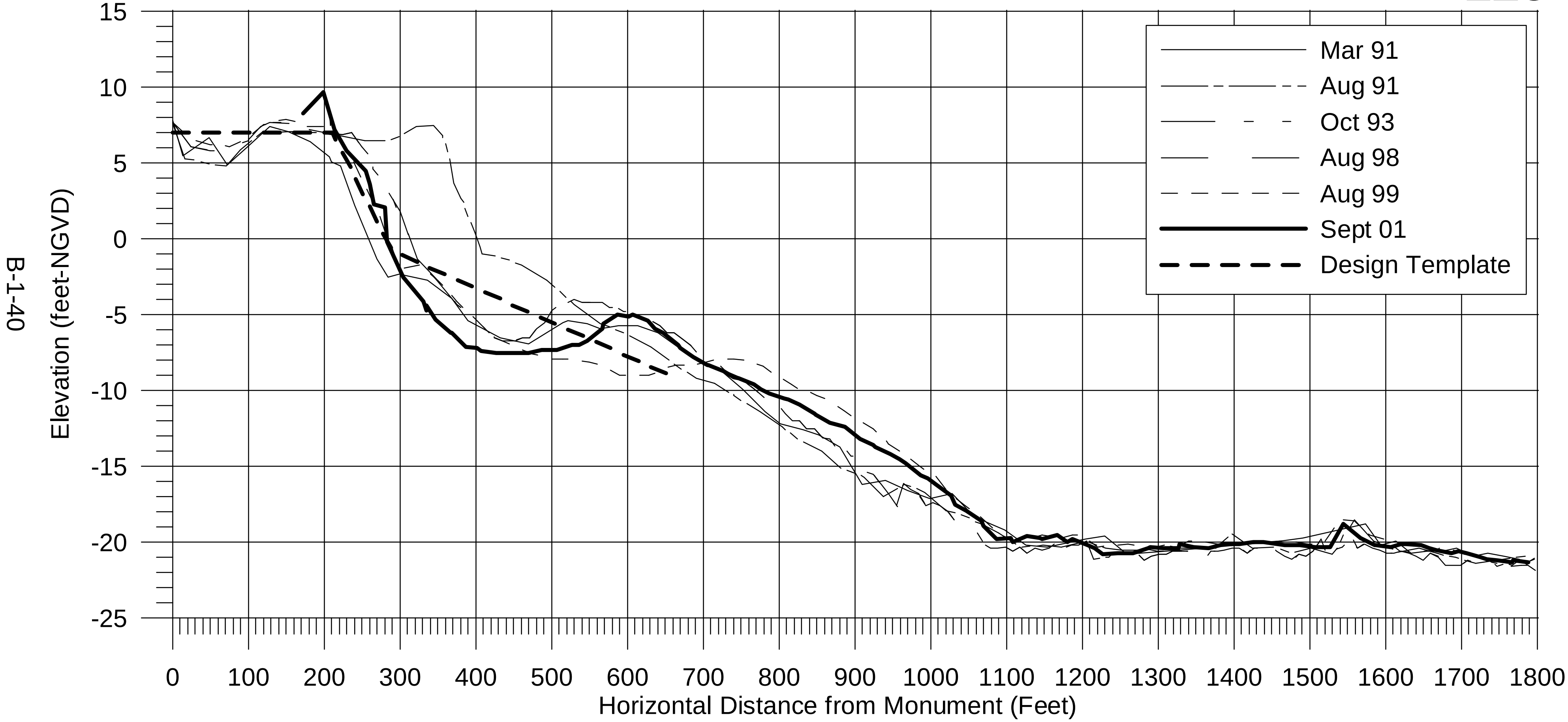


B-1-38
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

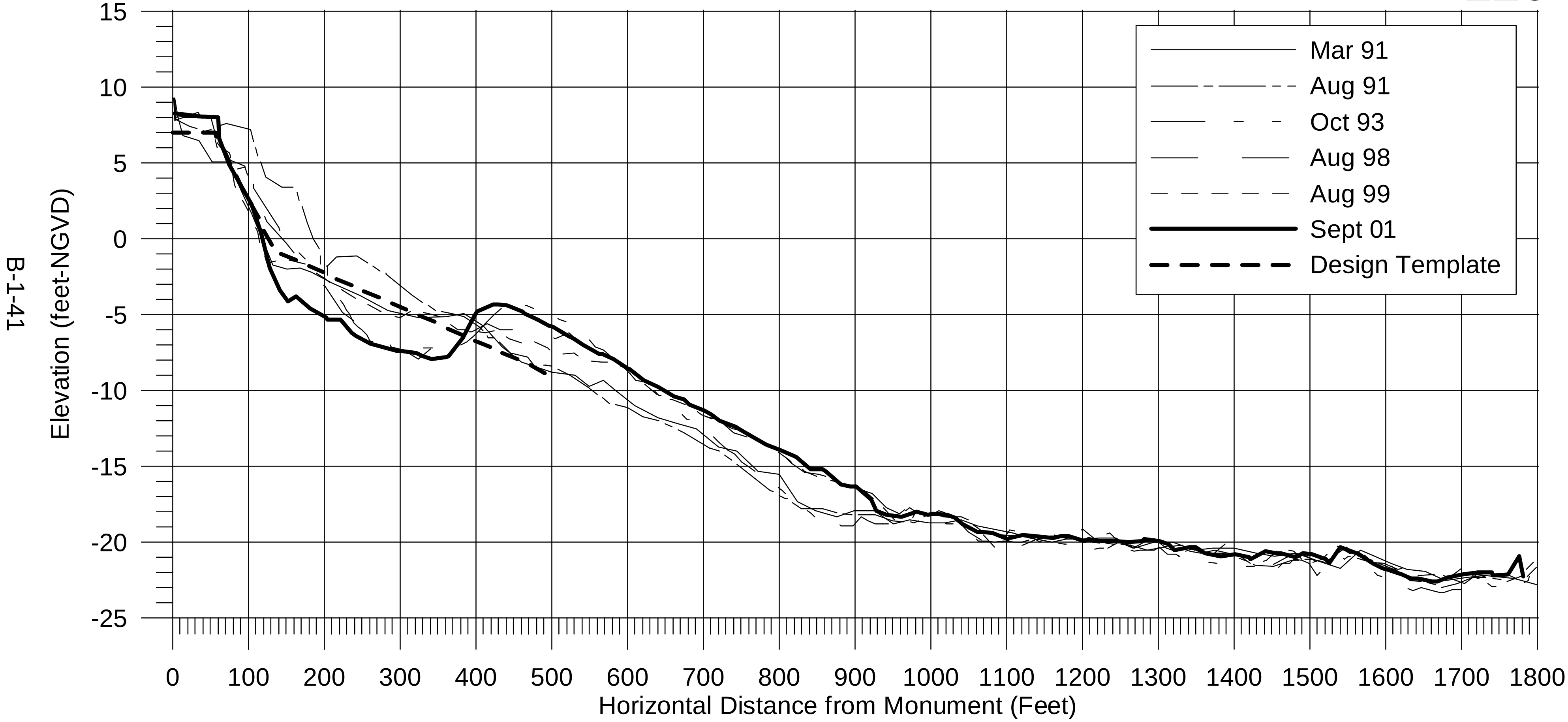


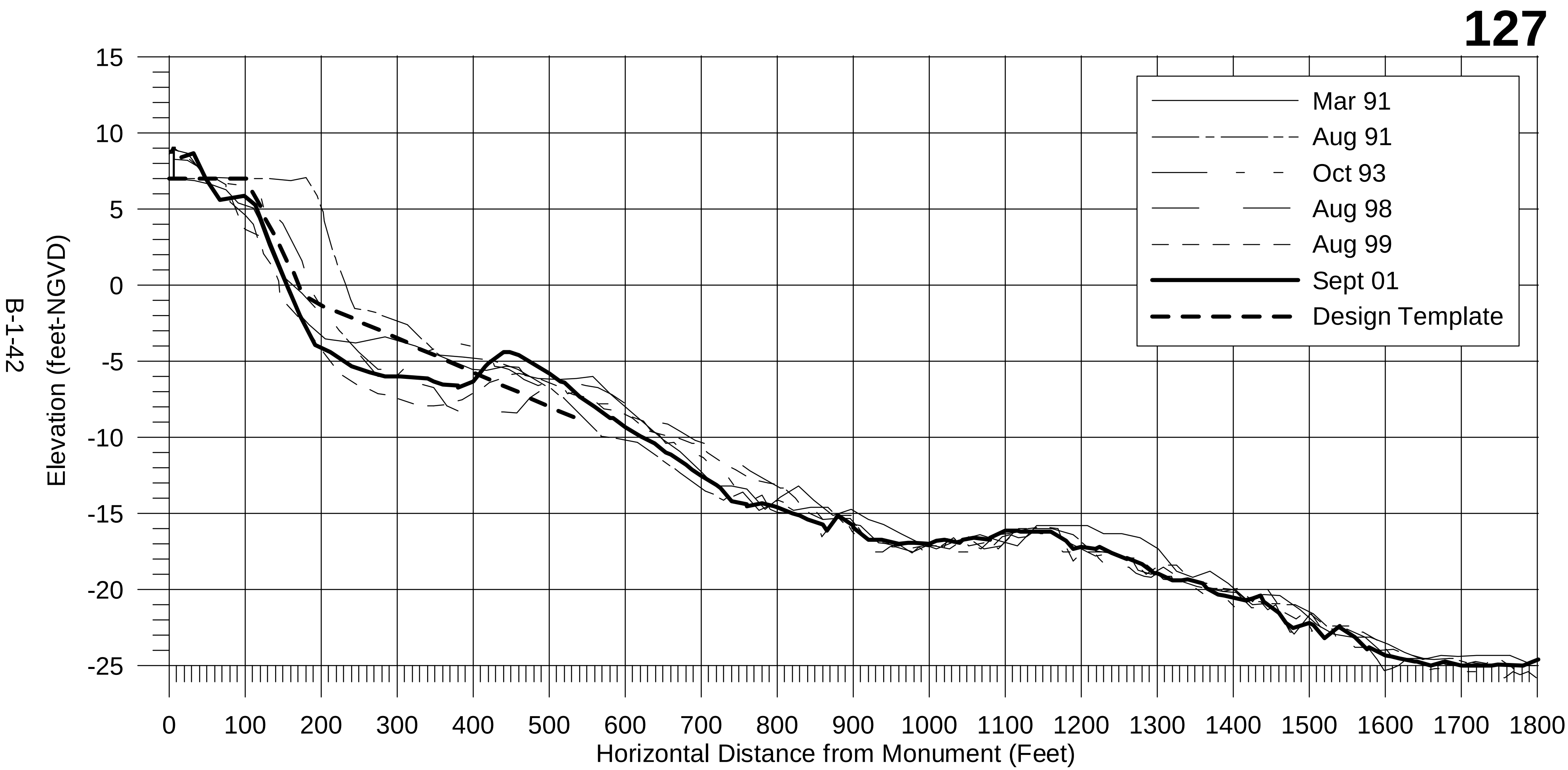
B-1-39
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

125

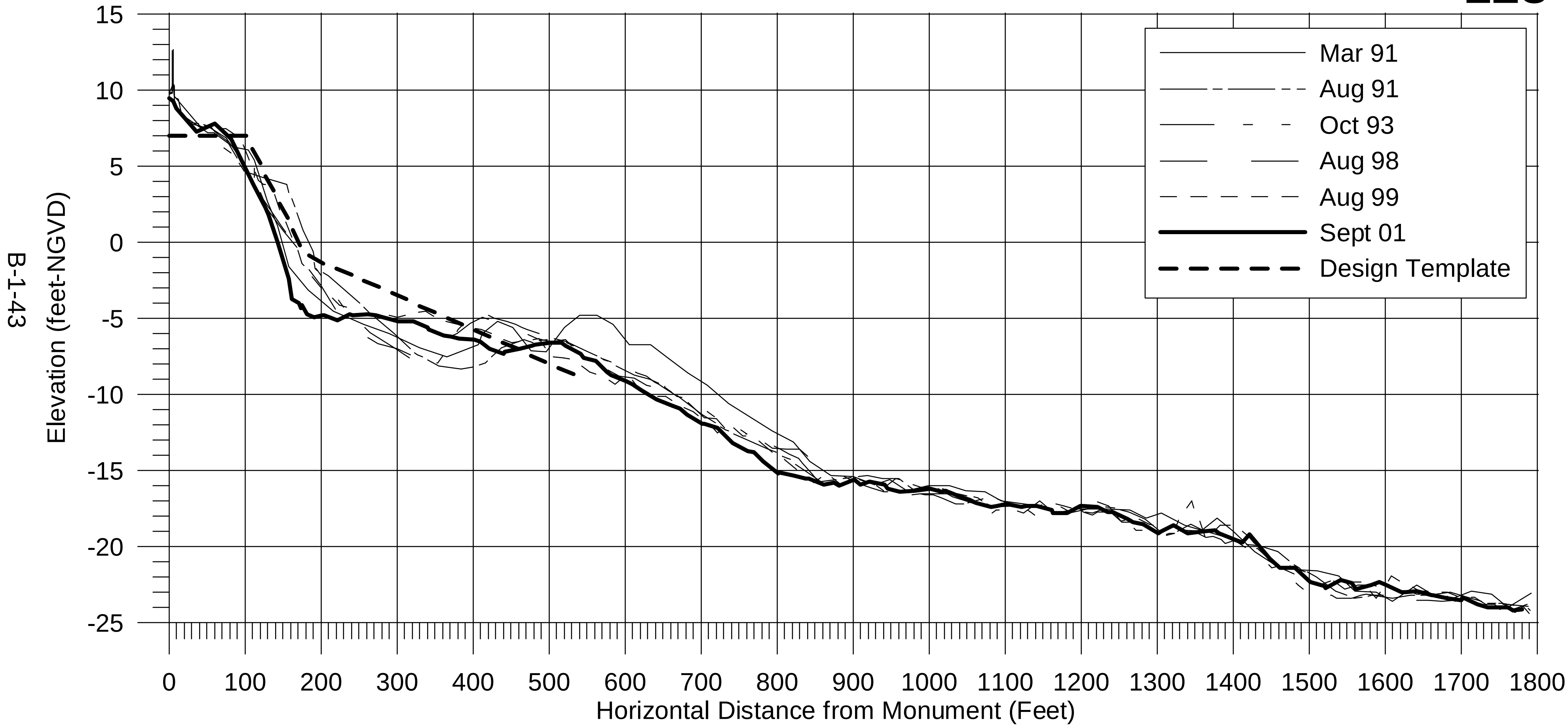


B-1-40
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template


B-1-41
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template



127
B-1-42
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template



B-1-43
Elevation (feet-NGVD)
15
10
5
0
-5
-10
-15
-20
-25
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
Horizontal Distance from Monument (Feet)
Mar 91
Aug 91
Oct 93
Aug 98
Aug 99
Sept 01
Design Template

SUB-APPENDIX B-2

SEGMENT III

DETAILED COST ESTIMATES FOR REEVALUATING THE PROJECT WIDTH AND DETERMINING THE OPTIMAL RENOURISHMENT INTERVAL FOR THE FEDERAL PROJECT

Figure B-2-1: Reevaluation of 50-yr Segment III Federal Project (25-ft Design Berm; 5-yr Interval))

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 5-YEAR RENOURISHMENT INTERVAL 25-ft project														
INTEREST RATE		6.125	%											
				UNIT	RENOURISHMENT YEAR									
ITEM		UNIT	QUANTITY	COST	0	5	10	15	20	25	30	35	40	45
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	892,090	6.62	5906									
RENOURISHMENT														
1	0	CY	650,000	6.62	4303									
2	5	CY	650,000	6.62		4303								
3	10	CY	650,000	6.62			4303							
4	15	CY	650,000	6.62				4303						
5	20	CY	650,000	9.79					6364					
6	25	CY	650,000	15.00						9750				
7	30	CY	650,000	15.00							9750			
8	35	CY	650,000	15.00								9750		
9	40	CY	650,000	15.00									9750	
10	45	CY	650,000	15.00										9750
BEACH TILLING		ACRE	83.0	300	25	25	25	25	25	25	25	25	25	25
HARDBOTTOM MITIGATION		ACRE	6.0	300,000	1800									
SUBTOTAL					13034	5328	5328	5328	7388	10775	10775	10775	10775	10775
CONTINGENCY		15	%		1955	799	799	799	1108	1616	1616	1616	1616	1616
SUBTOTAL (CONTRACT)					14989	6127	6127	6127	8497	12391	12391	12391	12391	12391
E&D+S&A		15	%		2248	919	919	919	1274	1859	1859	1859	1859	1859
TOTAL CONSTRUCTION					17237	7046	7046	7046	9771	14250	14250	14250	14250	14250
SUMMARY-INVESTMENT AND ANNUAL COSTS														
TOTAL CONSTRUCTION COST					17237	7046	7046	7046	9771	14250	14250	14250	14250	14250
INTEREST DURING CONSTRUCTION					58									
TOTAL INVESTMENT COST					17295	7046	7046	7046	9771	14250	14250	14250	14250	14250
PRESENT WORTH OF EACH CONSTRUCTION					17295	5234	3888	2889	2976	3224	2395	1779	1322	982
TOTAL PRESENT WORTH					41983									
AVERAGE ANNUAL COST					2710									

Figure B-2-2: Reevaluation of 50-yr Segment III Federal Project (25-ft Design Berm; 6-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL 25-ft project													
INTEREST RATE		6.125	%										
					RENOURISHMENT YEAR								
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18	24	30	36	42	48
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	892,090	6.62	5906								
RENOURISHMENT													
1	0	CY	780,000	6.62	5164								
2	6	CY	780,000	6.62		5164							
3	12	CY	780,000	6.62			5164						
4	18	CY	780,000	6.62				5164					
5	24	CY	780,000	9.79					7636				
6	30	CY	780,000	15.00						11700			
7	36	CY	780,000	15.00							11700		
8	42	CY	780,000	15.00								11700	
9	48	CY	780,000	15.00									11700
BEACH TILLING		ACRE	87.5	300	26	26	26	26	26	26	26	26	26
HARDBOTTOM MITIGATION		ACRE	8.0	300,000	2400								
SUBTOTAL					14495	6190	6190	6190	8662	12726	12726	12726	12726
CONTINGENCY		15	%		2174	928	928	928	1299	1909	1909	1909	1909
SUBTOTAL (CONTRACT)					16670	7118	7118	7118	9962	14635	14635	14635	14635
E&D+S&A		15	%		2500	1068	1068	1068	1494	2195	2195	2195	2195
TOTAL CONSTRUCTION					19170	8186	8186	8186	11456	16830	16830	16830	16830
SUMMARY-INVESTMENT AND ANNUAL COSTS													
TOTAL CONSTRUCTION COST					19170	8186	8186	8186	11456	16830	16830	16830	16830
INTEREST DURING CONSTRUCTION					63								
TOTAL INVESTMENT COST					19233	8186	8186	8186	11456	16830	16830	16830	16830
PRESENT WORTH OF EACH CONSTRUCTION					19233	5730	4011	2808	2750	2829	1980	1386	970
TOTAL PRESENT WORTH					41698								
AVERAGE ANNUAL COST					2692								

Figure B-2-3: Reevaluation of 50-yr Segment III Federal Project (25-ft Design Berm; 7-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 7-YEAR RENOURISHMENT INTERVAL 25-ft project													
INTEREST RATE		6.125	%										
					RENOURISHMENT YEAR								
ITEM		UNIT	QUANTITY	UNIT COST	0	7	14	21	28	35	42	49
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	892,090	6.62	5906							
RENOURISHMENT												
1	0	CY	910,000	6.62	6024							
2	7	CY	910,000	6.62		6024						
3	14	CY	910,000	6.62			6024					
4	21	CY	910,000	9.79				8909				
5	28	CY	910,000	15.00					13650			
6	35	CY	910,000	15.00						13650		
7	42	CY	910,000	15.00							13650	
8	49	CY	910,000	15.00								13650
BEACH TILLING		ACRE	92.0	300	28	28	28	28	28	28	28	28
HARDBOTTOM MITIGATION		ACRE	10.0	300,000	3000							
SUBTOTAL					15957	7052	7052	9937	14678	14678	14678	14678
CONTINGENCY		15	%		2394	1058	1058	1490	2202	2202	2202	2202
SUBTOTAL (CONTRACT)					18351	8110	8110	11427	16879	16879	16879	16879
E&D+S&A		15	%		2753	1216	1216	1714	2532	2532	2532	2532
TOTAL CONSTRUCTION					21104	9326	9326	13141	19411	19411	19411	19411
SUMMARY-INVESTMENT AND ANNUAL COSTS												
TOTAL CONSTRUCTION COST					21104	9326	9326	13141	19411	19411	19411	19411
INTEREST DURING CONSTRUCTION					68							
TOTAL INVESTMENT COST					21171	9326	9326	13141	19411	19411	19411	19411
PRESENT WORTH OF EACH CONSTRUCTION					21171	6151	4057	3771	3674	2423	1598	1054
TOTAL PRESENT WORTH					43902							
AVERAGE ANNUAL COST					2834							

Figure B-2-4: Reevaluation of 50-yr Segment III Federal Project (50-ft Design Berm; 5-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS
5-YEAR RENOURISHMENT INTERVAL
50-ft project

INTEREST RATE		6.125	%		RENOURISHMENT YEAR									
ITEM		UNIT	QUANTITY	UNIT COST	0	5	10	15	20	25	30	35	40	45
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	1,381,660	6.62	9147									
RENOURISHMENT														
1	0	CY	650,000	6.62	4303									
2	5	CY	650,000	6.62		4303								
3	10	CY	650,000	6.62			4303							
4	15	CY	650,000	6.62				4303						
5	20	CY	650,000	9.79					6364					
6	25	CY	650,000	15.00						9750				
7	30	CY	650,000	15.00							9750			
8	35	CY	650,000	15.00								9750		
9	40	CY	650,000	15.00									9750	
10	45	CY	650,000	15.00										9750
BEACH TILLING		ACRE	103.5	300	31	31	31	31	31	31	31	31	31	31
HARDBOTTOM MITIGATION		ACRE	13.0	300,000	3900									
SUBTOTAL					18381	5334	5334	5334	7395	10781	10781	10781	10781	10781
CONTINGENCY		15	%		2757	800	800	800	1109	1617	1617	1617	1617	1617
SUBTOTAL (CONTRACT)					21138	6134	6134	6134	8504	12398	12398	12398	12398	12398
E&D+S&A		15	%		3171	920	920	920	1276	1860	1860	1860	1860	1860
TOTAL CONSTRUCTION					24308	7054	7054	7054	9779	14258	14258	14258	14258	14258
SUMMARY-INVESTMENT AND ANNUAL COSTS														
TOTAL CONSTRUCTION COST					24308	7054	7054	7054	9779	14258	14258	14258	14258	14258
INTEREST DURING CONSTRUCTION					76									
TOTAL INVESTMENT COST					24384	7054	7054	7054	9779	14258	14258	14258	14258	14258
PRESENT WORTH OF EACH CONSTRUCTION					24384	5240	3893	2892	2978	3226	2396	1780	1322	982
TOTAL PRESENT WORTH					49094									
AVERAGE ANNUAL COST					3169									

Figure B-2-5: Reevaluation of 50-yr Segment III Federal Project (50-ft Design Berm; 6-yr Interval) (NED Plan)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL 50-ft project													
INTEREST RATE		6.125	%										
					RENOURISHMENT YEAR								
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18	24	30	36	42	48
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	1,381,660	6.62	9147								
RENOURISHMENT													
1	0	CY	780,000	6.62	5164								
2	6	CY	780,000	6.62		5164							
3	12	CY	780,000	6.62			5164						
4	18	CY	780,000	6.62				5164					
5	24	CY	780,000	9.79					7636				
6	30	CY	780,000	15.00						11700			
7	36	CY	780,000	15.00							11700		
8	42	CY	780,000	15.00								11700	
9	48	CY	780,000	15.00									11700
BEACH TILLING		ACRE	108.0	300	32	32	32	32	32	32	32	32	32
HARDBOTTOM MITIGATION		ACRE	15.0	300,000	4500								
SUBTOTAL					19843	6196	6196	6196	8669	12732	12732	12732	12732
CONTINGENCY		15	%		2976	929	929	929	1300	1910	1910	1910	1910
SUBTOTAL (CONTRACT)					22819	7125	7125	7125	9969	14642	14642	14642	14642
E&D+S&A		15	%		3423	1069	1069	1069	1495	2196	2196	2196	2196
TOTAL CONSTRUCTION					26242	8194	8194	8194	11464	16839	16839	16839	16839
SUMMARY-INVESTMENT AND ANNUAL COSTS													
TOTAL CONSTRUCTION COST					26242	8194	8194	8194	11464	16839	16839	16839	16839
INTEREST DURING CONSTRUCTION					81								
TOTAL INVESTMENT COST					26322	8194	8194	8194	11464	16839	16839	16839	16839
PRESENT WORTH OF EACH CONSTRUCTION					26322	5736	4015	2811	2752	2830	1981	1387	971
TOTAL PRESENT WORTH					48804								
AVERAGE ANNUAL COST					3151								

Figure B-2-6: Reevaluation of 50-yr Segment III Federal Project (50-ft Design Berm; 7-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS
7-YEAR RENOURISHMENT INTERVAL
50-ft project

INTEREST RATE		6.125	%									
					RENOURISHMENT YEAR							
ITEM		UNIT	QUANTITY	UNIT COST	0	7	14	21	28	35	42	49
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	1,381,660	6.62	9147							
RENOURISHMENT												
1	0	CY	910,000	6.62	6024							
2	7	CY	910,000	6.62		6024						
3	14	CY	910,000	6.62			6024					
4	21	CY	910,000	9.79				8909				
5	28	CY	910,000	15.00					13650			
6	35	CY	910,000	15.00						13650		
7	42	CY	910,000	15.00							13650	
8	49	CY	910,000	15.00								13650
BEACH TILLING		ACRE	112.5	300	34	34	34	34	34	34	34	34
HARDBOTTOM MITIGATION		ACRE	17.0	300,000	5100							
SUBTOTAL					21305	7058	7058	9943	14684	14684	14684	14684
CONTINGENCY		15	%		3196	1059	1059	1491	2203	2203	2203	2203
SUBTOTAL (CONTRACT)					24500	8117	8117	11434	16886	16886	16886	16886
E&D+S&A		15	%		3675	1217	1217	1715	2533	2533	2533	2533
TOTAL CONSTRUCTION					28175	9334	9334	13149	19419	19419	19419	19419
SUMMARY-INVESTMENT AND ANNUAL COSTS												
TOTAL CONSTRUCTION COST					28175	9334	9334	13149	19419	19419	19419	19419
INTEREST DURING CONSTRUCTION					86							
TOTAL INVESTMENT COST					28261	9334	9334	13149	19419	19419	19419	19419
PRESENT WORTH OF EACH CONSTRUCTION					28261	6157	4061	3773	3676	2424	1599	1055
TOTAL PRESENT WORTH					51006							
AVERAGE ANNUAL COST					3293							

Figure B-2-7: Reevaluation of 50-yr Segment III Federal Project (75-ft Design Berm; 5-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 5-YEAR RENOURISHMENT INTERVAL 75-ft project														
INTEREST RATE		6.125	%											
					RENOURISHMENT YEAR									
ITEM		UNIT	QUANTITY	UNIT COST	0	5	10	15	20	25	30	35	40	45
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	1,907,800	6.62	12630									
RENOURISHMENT														
1	0	CY	650,000	6.62	4303									
2	5	CY	650,000	6.62		4303								
3	10	CY	650,000	6.62			4303							
4	15	CY	650,000	6.62				4303						
5	20	CY	650,000	9.79					6364					
6	25	CY	650,000	15.00						9750				
7	30	CY	650,000	15.00							9750			
8	35	CY	650,000	15.00								9750		
9	40	CY	650,000	15.00									9750	
10	45	CY	650,000	15.00										9750
BEACH TILLING		ACRE	124.5	300	37	37	37	37	37	37	37	37	37	37
HARDBOTTOM MITIGATION		ACRE	28.0	300,000	8400									
SUBTOTAL					26370	5340	5340	5340	7401	10787	10787	10787	10787	10787
CONTINGENCY		15	%		3955	801	801	801	1110	1618	1618	1618	1618	1618
SUBTOTAL (CONTRACT)					30325	6141	6141	6141	8511	12405	12405	12405	12405	12405
E&D+S&A		15	%		4549	921	921	921	1277	1861	1861	1861	1861	1861
TOTAL CONSTRUCTION					34874	7063	7063	7063	9788	14266	14266	14266	14266	14266
SUMMARY-INVESTMENT AND ANNUAL COSTS														
TOTAL CONSTRUCTION COST					34874	7063	7063	7063	9788	14266	14266	14266	14266	14266
INTEREST DURING CONSTRUCTION					95									
TOTAL INVESTMENT COST					34970	7063	7063	7063	9788	14266	14266	14266	14266	14266
PRESENT WORTH OF EACH CONSTRUCTION					34970	5247	3898	2895	2981	3228	2398	1781	1323	983
TOTAL PRESENT WORTH					59702									
AVERAGE ANNUAL COST					3854									

Figure B-2-8: Reevaluation of 50-yr Segment III Federal Project (75-ft Design Berm; 6-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS
6-YEAR RENOURISHMENT INTERVAL
75-ft project

INTEREST RATE		6.125	%		RENOURISHMENT YEAR								
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18	24	30	36	42	48
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	1,907,800	6.62	12630								
RENOURISHMENT													
1	0	CY	780,000	6.62	5164								
2	6	CY	780,000	6.62		5164							
3	12	CY	780,000	6.62			5164						
4	18	CY	780,000	6.62				5164					
5	24	CY	780,000	9.79					7636				
6	30	CY	780,000	15.00						11700			
7	36	CY	780,000	15.00							11700		
8	42	CY	780,000	15.00								11700	
9	48	CY	780,000	15.00									11700
BEACH TILLING		ACRE	129.0	300	39	39	39	39	39	39	39	39	39
HARDBOTTOM MITIGATION		ACRE	30.0	300,000	9000								
SUBTOTAL					27832	6202	6202	6202	8675	12739	12739	12739	12739
CONTINGENCY		15	%		4175	930	930	930	1301	1911	1911	1911	1911
SUBTOTAL (CONTRACT)					32007	7133	7133	7133	9976	14650	14650	14650	14650
E&D+S&A		15	%		4801	1070	1070	1070	1496	2197	2197	2197	2197
TOTAL CONSTRUCTION					36808	8203	8203	8203	11473	16847	16847	16847	16847
SUMMARY-INVESTMENT AND ANNUAL COSTS													
TOTAL CONSTRUCTION COST					36808	8203	8203	8203	11473	16847	16847	16847	16847
INTEREST DURING CONSTRUCTION					100								
TOTAL INVESTMENT COST					36908	8203	8203	8203	11473	16847	16847	16847	16847
PRESENT WORTH OF EACH CONSTRUCTION					36908	5742	4019	2813	2754	2831	1982	1387	971
TOTAL PRESENT WORTH					59408								
AVERAGE ANNUAL COST					3835								

Figure B-2-9: Reevaluation of 50-yr Segment III Federal Project (75-ft Design Berm; 7-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 7-YEAR RENOURISHMENT INTERVAL 75-ft project													
INTEREST RATE		6.125	%										
					RENOURISHMENT YEAR								
ITEM		UNIT	QUANTITY	UNIT COST	0	7	14	21	28	35	42	49
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000	1000	1000	1000
INITIAL FILL		CY	1,907,800	6.62	12630							
RENOURISHMENT												
1	0	CY	910,000	6.62	6024							
2	7	CY	910,000	6.62		6024						
3	14	CY	910,000	6.62			6024					
4	21	CY	910,000	9.79				8909				
5	28	CY	910,000	15.00					13650			
6	35	CY	910,000	15.00						13650		
7	42	CY	910,000	15.00							13650	
8	49	CY	910,000	15.00								13650
BEACH TILLING		ACRE	133.5	300	40	40	40	40	40	40	40	40
HARDBOTTOM MITIGATION		ACRE	32.0	300,000	9600							
SUBTOTAL					29294	7064	7064	9949	14690	14690	14690	14690
CONTINGENCY		15	%		4394	1060	1060	1492	2204	2204	2204	2204
SUBTOTAL (CONTRACT)					33688	8124	8124	11441	16894	16894	16894	16894
E&D+S&A		15	%		5053	1219	1219	1716	2534	2534	2534	2534
TOTAL CONSTRUCTION					38741	9342	9342	13157	19428	19428	19428	19428
SUMMARY-INVESTMENT AND ANNUAL COSTS												
TOTAL CONSTRUCTION COST					38741	9342	9342	13157	19428	19428	19428	19428
INTEREST DURING CONSTRUCTION					104							
TOTAL INVESTMENT COST					38846	9342	9342	13157	19428	19428	19428	19428
PRESENT WORTH OF EACH CONSTRUCTION					38846	6162	4065	3776	3677	2425	1600	1055
TOTAL PRESENT WORTH					61606							
AVERAGE ANNUAL COST					3977							

SUB-APPENDIX B-3

SEGMENT III

DETAILED COST ESTIMATES FOR EVALUATION OF THE JOHN U. LLOYD REACH AS A SEPARABLE PROJECT ELEMENT

Figure B-3-1: Cost to implement JUL periodic nourishment only as separable project element.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL									
INTEREST RATE		6.125	%		RENOURISHMENT YEAR				
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		LS	1	250,000	250	250	250	250
INITIAL FILL		CY	120,600	9.79	1181			
RENOURISHMENT								
2	0	CY	362,500	9.79	3549			
3	6	CY	362,500	15.00		5438		
4	12	CY	362,500	15.00			5438	
5	18	CY	362,500	15.00				5438
BEACH TILLING		ACRE	15.0	300	5	5	5	5
HARDBOTTOM MITIGATION		ACRE	5.0	300,000	1500			
SUBTOTAL					6484	5692	5692	5692
CONTINGENCY		15	%		973	854	854	854
SUBTOTAL (CONTRACT)					7457	6546	6546	6546
TOTAL CONSTRUCTION					7457	6546	6546	6546
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL INVESTMENT COST					7457	6546	6546	6546
PRESENT WORTH OF EACH CONSTRUCTION					7457	4582	3207	2245
TOTAL PRESENT WORTH					17491			
AVERAGE ANNUAL COST					1410			

Figure B-3-2: Cost to implement 25-ft design berm at JUL as separable project element.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL								
INTEREST RATE		6.125	%		RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		LS	1	250,000	250	250	250	250
INITIAL FILL		CY	215,000	9.79	2105			
RENOURISHMENT								
2	0	CY	409,000	9.79	4004			
3	6	CY	409,000	15.00		6135		
4	12	CY	409,000	15.00			6135	
5	18	CY	409,000	15.00				6135
BEACH TILLING		ACRE	30.0	300	9	9	9	9
HARDBOTTOM MITIGATION		ACRE	8.5	300,000	2550			
								409000
SUBTOTAL					8918	6394	6394	6394
CONTINGENCY		15	%		1338	959	959	959
SUBTOTAL (CONTRACT)					10256	7353	7353	7353
TOTAL CONSTRUCTION					10256	7353	7353	7353
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL INVESTMENT COST					10256	7353	7353	7353
PRESENT WORTH OF EACH CONSTRUCTION					10256	5147	3603	2522
TOTAL PRESENT WORTH					21528			
AVERAGE ANNUAL COST					1735			

Figure B-3-3: Cost to implement 50-ft design berm at JUL as separable project element.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL								
INTEREST RATE		6.125	%		RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		LS	1	250,000	250	250	250	250
INITIAL FILL		CY	264,000	9.79	2585			
RENOURISHMENT								
2	0	CY	433,000	9.79	4239			
3	6	CY	433,000	15.00		6495		
4	12	CY	433,000	15.00			6495	
5	18	CY	433,000	15.00				6495
BEACH TILLING		ACRE	45.0	300	14	14	14	14
HARDBOTTOM MITIGATION		ACRE	10.0	300,000	3000			
SUBTOTAL					10087	6759	6759	6759
CONTINGENCY		15	%		1513	1014	1014	1014
SUBTOTAL (CONTRACT)					11600	7772	7772	7772
TOTAL CONSTRUCTION					11600	7772	7772	7772
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL INVESTMENT COST					11600	7772	7772	7772
PRESENT WORTH OF EACH CONSTRUCTION					11600	5441	3808	2666
TOTAL PRESENT WORTH					23515			
AVERAGE ANNUAL COST					1895			

SUB-APPENDIX B-4

SEGMENT III

DETAILED COST ESTIMATES FOR IMPLEMENTATION OF THE REEVALUATED PLAN AND DETERMINING THE OPTIMAL RENOURISHMENT INTERVAL

Figure B-4-1: Implementation of Segment III Reevaluated NED Plan (24-yr; 5-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS
5-YEAR RENOURISHMENT INTERVAL
(PLAN IMPLEMENTATION)

					RENOURISHMENT YEAR				
INTEREST RATE		6.125	%						
ITEM		UNIT	QUANTITY	UNIT COST	0	5	10	15	20
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459				
RENOURISHMENT									
2	0	CY	877,300	9.79	8589				
3	5	CY	877,300	15.00		13160			
4	10	CY	877,300	15.00			13160		
5	15	CY	877,300	15.00				13160	
6	20	CY	877,300	15.00					13160
BEACH TILLING		ACRE	110.5	300	33	33	33	33	33
HARDBOTTOM MITIGATION		ACRE	6.25	300,000	1875				
SUBTOTAL					16956	14193	14193	14193	14193
CONTINGENCY		15	%		2543	2129	2129	2129	2129
SUBTOTAL (CONTRACT)					19499	16322	16322	16322	16322
EASEMENTS		JOB	1	250,000	250				
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342	1342
TOTAL CONSTRUCTION					21556	18129	18129	18129	18129

SUMMARY-INVESTMENT AND ANNUAL COSTS

	0	5	10	15	20
TOTAL CONSTRUCTION COST	21556	18129	18129	18129	18129
INTEREST DURING CONSTRUCTION	78				
TOTAL INVESTMENT COST	21635	18129	18129	18129	18129
PRESENT WORTH OF EACH CONSTRUCTION	21635	13467	10004	7432	5521
TOTAL PRESENT WORTH	58059				
AVERAGE ANNUAL COST	4680				

Figure B-4-2: Implementation of Segment III Reevaluated NED Plan (24-yr; 6-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL (PLAN IMPLEMENTATION)								
INTEREST RATE		6.125	%					
					RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459			
RENOURISHMENT								
2	0	CY	1,025,300	9.79	10038			
3	6	CY	1,025,300	15.00		15380		
4	12	CY	1,025,300	15.00			15380	
5	18	CY	1,025,300	15.00				15380
BEACH TILLING		ACRE	115.0	300	35	35	35	35
HARDBOTTOM MITIGATION		ACRE	7.56	300,000	2268			
SUBTOTAL					18799	16414	16414	16414
CONTINGENCY		15	%		2820	2462	2462	2462
SUBTOTAL (CONTRACT)					21619	18876	18876	18876
EASEMENTS		JOB	1	250,000	250			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					23676	20683	20683	20683
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					23676	20683	20683	20683
INTEREST DURING CONSTRUCTION					86			
TOTAL INVESTMENT COST					23762	20683	20683	20683
PRESENT WORTH OF EACH CONSTRUCTION					23762	14478	10135	7094
TOTAL PRESENT WORTH					55469			
AVERAGE ANNUAL COST					4471			

Figure B-4-3: Implementation of Segment III Reevaluated NED Plan (24-yr; 7-yr Interval)

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 7-YEAR RENOURISHMENT INTERVAL 50-ft project (PLAN IMPLENTATION)								
INTEREST RATE		6.125	%		RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	7	14	21
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459			
RENOURISHMENT								
2	0	CY	1,173,300	9.79	11487			
3	7	CY	1,173,300	15.00		17600		
4	14	CY	1,173,300	15.00			17600	
5	21	CY	1,173,300	15.00				17600
BEACH TILLING		ACRE	119.5	300	36	36	36	36
HARDBOTTOM MITIGATION		ACRE	9.50	300,000	2850			
SUBTOTAL					20831	18635	18635	18635
CONTINGENCY		15	%		3125	2795	2795	2795
SUBTOTAL (CONTRACT)					23956	21431	21431	21431
EASEMENTS		JOB	1	250,000	250			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					26013	23238	23238	23238
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					26013	23238	23238	23238
INTEREST DURING CONSTRUCTION					94			
TOTAL INVESTMENT COST					26107	23238	23238	23238
PRESENT WORTH OF EACH CONSTRUCTION					26107	15327	10110	6668
TOTAL PRESENT WORTH					58213			
AVERAGE ANNUAL COST					4692			

SUB-APPENDIX B-5

SEGMENT III

DETAILED COST ESTIMATE FOR PROJECT MODIFICATION THAT CONSISTS OF CONSTRUCTING A FULL DESIGN BEACH SECTION ALONG DANIA AND SOUTHERN JOHN U. LLOYD

Figure B-5-1: Implementation of Segment III NED Plan with modification of a full design section along southern John U. Lloyd and Dania Beach shorelines.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL									
INTEREST RATE		6.125	%						
ITEM		UNIT	QUANTITY	UNIT COST	RENOURISHMENT YEAR 0	6	12	18
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	797,600	9.79	7809			
RENOURISHMENT								
2	0	CY	1,075,300	9.79	10527			
3	6	CY	1,075,300	15.00		16130		
4	12	CY	1,075,300	15.00			16130	
5	18	CY	1,075,300	15.00				16130
BEACH TILLING		ACRE	140.0	300	42	42	42	42
HARDBOTTOM MITIGATION		ACRE	20.6	300,000	6180			
SUBTOTAL					25558	17172	17172	17172
CONTINGENCY		15	%		3834	2576	2576	2576
SUBTOTAL (CONTRACT)					29391	19747	19747	19747
EASEMENTS		JOB	1	250,000	250			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					31448	21554	21554	21554
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					31448	21554	21554	21554
INTEREST DURING CONSTRUCTION					101			
TOTAL INVESTMENT COST					31550	21554	21554	21554
PRESENT WORTH OF EACH CONSTRUCTION					31550	15088	10561	7393
TOTAL PRESENT WORTH					64592			
AVERAGE ANNUAL COST					5206			

SUB-APPENDIX B-6

SEGMENT III

DETAILED COST ESTIMATES FOR IMPLEMENTATION OF THE REEVALUATED FEDERAL PROJECT WITH GROINS AND FUTURE SAND BYPASSING MODIFICATIONS

Figure B-6-1: Implementation of Segment III NED Plan with two groins and a jetty spur immediately downdrfit of Port Everglades.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL									
INTEREST RATE		6.125	%		RENOURISHMENT YEAR				
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459			
RENOURISHMENT								
2	0	CY	982,400	9.79	9618			
3	6	CY	982,400	15.00		14736		
4	12	CY	982,400	15.00			14736	
5	18	CY	982,400	15.00				14736
BEACH TILLING		ACRE	115.0	300	35	35	35	35
HARDBOTTOM MITIGATION		ACRE	7.56	300,000	2268			
GROINS		TONS	5,300	75.0	398	44	44	44
GROIN FOUNDATION (Mattress)		sq.ft.	22,000	15.0	330			
SUBTOTAL					19107	15814	15814	15814
CONTINGENCY		15	%		2866	2372	2372	2372
SUBTOTAL (CONTRACT)					21973	18186	18186	18186
EASEMENTS		JOB	1	437,500	438			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					24217	19993	19993	19993
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					24217	19993	19993	19993
INTEREST DURING CONSTRUCTION					84			
TOTAL INVESTMENT COST					24301	19993	19993	19993
PRESENT WORTH OF EACH CONSTRUCTION					24301	13995	9797	6857
TOTAL PRESENT WORTH					54950			
AVERAGE ANNUAL COST					4429			

Figure B-6-2: Implementation of Segment III NED Plan with ten groins and a jetty spur immediately downdrift of Port Everglades.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL								
INTEREST RATE		6.125	%		RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459			
RENOURISHMENT								
2	0	CY	946,500	9.79	9266			
3	6	CY	946,500	15.00		14198		
4	12	CY	946,500	15.00			14198	
5	18	CY	946,500	15.00				14198
BEACH TILLING		ACRE	98.0	300	29	29	29	29
HARDBOTTOM MITIGATION		ACRE	6.5	300,000	1950			
GROINS		TONS	21,000	75.0	1575	129	129	129
GROIN FOUNDATION (Mattress)		sq.ft.	22,000	15.0	330			
GROIN FOUNDATION (Geogrid)		sq.ft.	95,000	2.5	238			
SUBTOTAL					19847	15355	15355	15355
CONTINGENCY		15	%		2977	2303	2303	2303
SUBTOTAL (CONTRACT)					22824	17659	17659	17659
EASEMENTS		JOB	1	437,500	438			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					25069	19466	19466	19466
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					25069	19466	19466	19466
INTEREST DURING CONSTRUCTION					82			
TOTAL INVESTMENT COST					25151	19466	19466	19466
PRESENT WORTH OF EACH CONSTRUCTION					25151	13626	9538	6677
TOTAL PRESENT WORTH					54991			
AVERAGE ANNUAL COST					4432			

Figure B-6-3: Implementation of Segment III NED Plan with two groins, a jetty spur, and sand bypassing at Port Everglades.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL								
INTEREST RATE		6.125	%		RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459			
RENOURISHMENT								
2	0	CY	982,400	9.79	9618			
3	6	CY	682,500	15.00		10238		
4	12	CY	682,500	15.00			10238	
5	18	CY	682,500	15.00				10238
BEACH TILLING		ACRE	115.0	300	35	35	35	35
HARDBOTTOM MITIGATION		ACRE	7.56	300,000	2268			
GROINS		TONS	5,300	75.0	398	44	44	44
GROIN FOUNDATION (Mattress)		sq.ft.	22,000	15.0	330			
SUBTOTAL					19107	11316	11316	11316
CONTINGENCY		15	%		2866	1697	1697	1697
SUBTOTAL (CONTRACT)					21973	13013	13013	13013
EASEMENTS		JOB	1	437,500	438			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					24217	14820	14820	14820
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					24217	14820	14820	14820
INTEREST DURING CONSTRUCTION					84			
TOTAL INVESTMENT COST					24301	14820	14820	14820
PRESENT WORTH OF EACH CONSTRUCTION					24301	10374	7262	5083
INITIAL COST OF BYPASS PLANT =$7,000,000						7000		
PRESENT WORTH OF BYPASS PLANT CONSTRUCTION					4900			
PRESENT WORTH OF ANNUAL BYPASSING (44,000 cy/yr @ $3.50/cy starting at YEAR 6)					1264			
TOTAL PRESENT WORTH					53184			
AVERAGE ANNUAL COST					4287			

Figure B-6-4: Implementation of Segment III NED Plan with ten groins, a jetty spur and, sand bypassing at Port Everglades.

ESTIMATE OF CONTRACT AND CONSTRUCTION COSTS 6-YEAR RENOURISHMENT INTERVAL								
INTEREST RATE		6.125	%					
					RENOURISHMENT YEAR			
ITEM		UNIT	QUANTITY	UNIT COST	0	6	12	18
MOBILIZATION		JOB	1	1,000,000	1000	1000	1000	1000
INITIAL FILL		CY	557,600	9.79	5459			
RENOURISHMENT								
2	0	CY	946,500	9.79	9266			
3	6	CY	681,700	15.00		10226		
4	12	CY	681,700	15.00			10226	
5	18	CY	681,700	15.00				10226
BEACH TILLING		ACRE	98.0	300	29	29	29	29
HARDBOTTOM MITIGATION		ACRE	6.5	300,000	1950			
GROINS		TONS	21,000	75.0	1575	129	129	129
GROIN FOUNDATION (Mattress)		sq.ft.	22,000	15.0	330			
GROIN FOUNDATION (Geogrid)		sq.ft.	95,000	2.5	238			
SUBTOTAL					19847	11383	11383	11383
CONTINGENCY		15	%		2977	1708	1708	1708
SUBTOTAL (CONTRACT)					22824	13091	13091	13091
EASEMENTS		JOB	1	437,500	438			
ENVIR. MONITORING		JOB	1	275,000	275	275	275	275
GEOTECHNICAL STUDIES		JOB	1	190,000	190	190	190	190
E&D+S&A		JOB	1	1,342,000	1342	1342	1342	1342
TOTAL CONSTRUCTION					25069	14898	14898	14898
SUMMARY-INVESTMENT AND ANNUAL COSTS								
TOTAL CONSTRUCTION COST					25069	14898	14898	14898
INTEREST DURING CONSTRUCTION					82			
TOTAL INVESTMENT COST					25151	14898	14898	14898
PRESENT WORTH OF EACH CONSTRUCTION					25151	10428	7300	5110
INITIAL COST OF BYPASS PLANT =$7,000,000						7000		
PRESENT WORTH OF BYPASS PLANT CONSTRUCTION					4900			
PRESENT WORTH OF ANNUAL BYPASSING (44,000 cy/yr @ $3.50/cy starting at YEAR 6)					1264			
TOTAL PRESENT WORTH					54153			
AVERAGE ANNUAL COST					4365			

SUB-APPENDIX B-7

SEGMENT III

ENGINEERING COST ESTIMATE FOR OFFSHORE HOPPER-DREDGING, ROCK SEPARATION AND BEACH FILL PLACEMENT

(Note: This estimate was prepared by Jacksonville District COE Cost Engineering staff.)

MOBIL & DEMOB COST:	$458,885	BID QUANTITY	1,800,000	C.Y.
		UNIT COST...	$9.79	PER C.Y.
	Hopper Dredging	EXCAV. COST.	$17,622,000	
CHECKLIST FOR INPUT DATA.		TIME........	11.79	MONTHS

PG 1 OF 12: PROJECT TITLES

PROJECT -	Hopper Dredging	
LOCATION -	Segment III - Alternative 1	
INVIT # -		
DATE OF EST. -	17-Nov-99	
EST. BY -	M Fascher	
MOB. BID ITEM # -	1	
EXCAV. BID ITEM # -	2	

PG 2 OF 12: TYPE OF EST & IND COSTS

TYPE OF EST. -	Planning Estimate	
CONTRACTOR'S O.H. -	16.5%	
CONTRACTOR'S PROFIT -	10.0%	
CONTRACTOR'S BOND -	1.0%	

PG 3 OF 12: EXCAVATION QTY'S

BANK HEIGHT -	7	ft
REQ'D EXCAVATION -	1,800,000	cyds
PAY OVERDEPTH -		cyds
CONTRACT AMOUNT -	1,800,000	cyds
NOT DREDGED -		cyds
NET PAY -	1,800,000	cyds
NONPAY YARDAGE -	540,000	cyds
GROSS YARDAGE -	2,340,000	cyds
LOSSES -	30.0	% of Net Pay
TOTAL BANK HEIGHT -	7.0	ft

PG 4, 5 & 6 OF 12: PRODUCTION

TYPE OF MATERIAL -	3%	MUD
-	94%	SAND
-	3%	GRAVEL
HOPPER CAPACITY -	3,800	cyds
EFF. HOPPER CAP. -	1,950	cyds
DRDGE RATE (ALL HEADS) -	1,202	cy/hr
ACT. DRAGHDS USED -	2	ea
DRDGE RATE USED -	1,202	cy/hr
TURNS/CYCLE -	3	ea
MIN. PER TURN -	6	min
DISPOSAL DIST -	15	mi
TRVL SPD TO DISP -	9.8	mph
TRVL SPD FROM DISP -	10.8	mph
DUMP/CONNECT TIME -	15	min
PUMPOUT RATE -	1800	cy/hr
PIPELINE USED -	13000	lf
CLEANUP -	0%	More Time
% EFF WORK TIME -	86.0%	

PG 7 & 8 OF 12: PLANT OWN. & OPER.

DREDGE SELECTED -	GENERIC MEDIUM	
DREDGE ACQUIS COST -	$16,600,000	
DREDGE CAPITAL IMPROV -	10%	
PROPULSION TUG -	self prop.	/mo
SURVEY VESSEL -	$30,000	/mo
BOOSTER -	$200,000	/mo
CRANE BARGE -	$0	/mo
TENDER TUG -	$40,000	/mo
OTHER MARINE -	$0	/mo
SHORE EQUIP -	$0	/mo

PG 9 OF 12: OTHER ADJUSTMENTS

SPECIAL COST/MO (1ST) -	$0	>
SP COST/MO (2ND-14TH) -	$0	From Sheet D\3
SPECIAL COST LS (1ST) -	$0	>
SP COST LS (2ND-14TH) -	$0	From Sheet E

PG 10 OF 12: LOCAL AREA FACTORS

PRESENT YEAR -	1998	
ECONOMIC INDEX -	5676	
LAF -	0.85	
INTEREST RATE -	6.675%	/yr
TIME PERIOD -	July to December 1998	
PIPELINE AVAILABILITY -	9	mos/yr
BUCKET AVAILABILITY -	10	mos/yr
HOPPER AVAILABILITY -	10	mos/yr
FUEL PRICE -	$1.00	/gal

PG 11 OF 12: DREDGE OPER ADJ FACTORS

PUMP LOAD FACTOR -	50%	
RPR & MAINT. ADJ -	1.00	
JET PUMP USEAGE -	100%	

PG 12 OF 12: TRAVEL & PROVISIONS

FREQ PD TRAVEL -	28	days
RT TRAVEL COST -	$400	
GOVT. PERSONNEL -	3	ea
PROVISIONS & SUPP -	$15	/man

LOADS PER DAY -	3.3	
PRODUCTION -	316	gross cy per hour
OPERATING TIME -	628	hours per month
GROSS PRODUCTION -	198,448	cy per month
PAY PRODUCTION -	152,672	pay cy per month