Montauk Point, New York

Hurricane & Storm Damage Reduction

Final Feasibility Report
& Environmental Impact Statement

US Army Corps of Engineers
New York District

New York State Department
of Environmental Conservation

FINAL - October 2005

Montauk Point, New York
Hurricane & Storm Damage Reduction - Feasibility Study

Syllabus

If allowed to continue, progressive instability of the Montauk Point bluff would result in the irrecoverable loss of the historic Montauk Point Lighthouse and its associated structures, along with archaeological resources. The implication would be the total loss of all historical properties, both buried and above ground. Once this information is lost, it can never be recovered, and future study of the complex would be impossible. The alternative plans developed for this feasibility report are superior to the no action plan as they provide substantial storm damage protection.

The alternative plans included five significantly different measures: stone revetment, offshore breakwater with beach fill, T-groins with beach fill, beach fill, and relocation of the lighthouse. The stone revetment is the most reliable and cost effective structural solution. Because of the steep terrain in the area, the cost of relocation is prohibitive. In addition, relocation would have adverse effects on the surrounding archeological resources, would degrade existing habitats and historic views, and also effect recreational use of the area. Also, a replacement light tower would have to be constructed, as the lighthouse, in its current location, continues to serve as a functioning aid to navigation.

Therefore, the selected plan consists of the construction of a stone revetment with a 73-year storm design (Alternative Plan 2B). This level of design was chosen based on an economic optimization of a wide range of designs to reduce the risk of losses due to storm damages.

- Stone revetment, 840-feet in length, with a crest width of 40-feet at elevation +25 feet NGVD, and 1V:2H side slopes.
- 12.6-ton quarrystone armor units extending from the crest down to embedded toe.
- Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure.
- The bottom of the armor stone layer in the toe is located at a depth of 12-feet from the existing bottom.
- A heavily embedded toe is incorporated to protect against breaking waves, provide long-term stone stability, and scour at the toe of the structure. Stone sub-layers are specified in accordance with standard Corps design procedures.

The selected NED plan is also the locally preferred plan. The local sponsor, New York State Department of Environmental Conservation is willing to provide all items of local cooperation, and is in full support of the selected plan.

The proposed work will have no significant impact on the quality of the environment in the project area. Special consideration was given to the effects of the selected plan on fishing, surfing, and cultural experiences. Most impacts associated with this project will be temporary, and none of the impacts are regarded as significant.

The land that will be protected by implementation of this recommended project is deeded to the Montauk Historical Society (MHS). The MHS is a private, not for profit association that is not part of any state or local government. This land is held open, for use by all on equal terms, regardless of origin or home area. Existing Corps policy indicates that there is no Federal interest in protection of a property owned by a single private non-profit entity.

However, although the MHS is clearly a single, private landowner, they must, by deed restriction and State charter, act as a public entity akin to agencies of State and local governments. The MHS must accomplish a public education mission to stay in operation, must follow Federal National Historic Preservation requirements for maintenance work, and membership and enjoyment of the benefits of the facility and educational programs are open to all, with no restriction, for a fee. Under the deed and charter, the MHS cannot structure and constrain uses of the property, nor can anyone who cares to join the MHS and enjoy the benefits of the facility and water resources project be excluded.

In light of these facts, New York District requested a waiver to the single landowner policy from the Assistant Secretary of the Army (Civil Works) and was granted an exception allowing the completion of the feasibility study with a view towards pursuing a cost-shared construction project for Montauk Point, New York.

The first cost of the selected plan is estimated to be $13,722,000 at October 2004 price levels. The total benefits attributed to this selected plan are estimated at $1,578,700 while the annual costs are $889,300. Therefore, the benefit to cost ratio is 1.8 to 1, with total net benefits of $689,400.

The cost-sharing for construction of this storm damage reduction project is as follows:

50% Federal Share	$6,861,000
50% Non-Federal Share	$6,861,000
Total Project First Cost	$13,722,000

An annual revetment maintenance cost of $52,300 will be a 100% Non-Federal expense.

Montauk Point, New York

Feasibility Main Report – Sections

Paragraph # Page #

Feasibility Main Report – Sections

Paragraph # Page #

Feasibility Main Report – Sections

Paragraph # Page #

Feasibility Report – Appendices and Environmental Impact Statement

---

Montauk Point, New York

1. Study Authorities

This feasibility study was conducted under the authority of a resolution adopted by the Committee on Environment and Public Works of the U.S. Senate on May 15, 1991.

> *"Resolved by the Committee on Environment and Public Works of the United States Senate, that the Secretary of the Army is hereby requested to review the report of the Chief of Engineers on Fire Island to Montauk Point, New York, published as House Document Number 86-425, 86th Congress, 2nd session, and other pertinent reports, to determine whether modifications of the recommendations contained therein are advisable at the present time, with a view to preserving, restoring, and protecting Montauk Point and vicinity, including the historic Montauk Lighthouse and associated facilities, from erosion, environmental degradation, and coastal storm damage."*

Another resolution, also dated May 15, 1991 authorized the study of interim emergency protection works until a comprehensive project was formulated, designed and constructed:

> *"Resolved by the Committee on Environment and Public Works of the United States Senate, that the Committee recognizes that unacceptable cultural and historic impacts would result from loss of historic property to structures in the vicinity of the Montauk Lighthouse, Montauk, New York and in recognition, the Secretary of the Army is requested to review the report of the Chief of Engineers on Fire Island to Montauk Point, New York, published as House Document Number 86-425, 86th Congress, 2nd session, and other pertinent reports, to determine what interim emergency protection works can be carried out to serve as protection for the lighthouse and bluff until a comprehensive study determines the best environmental, cultural and economical plan to enhance and protect this important resource."*

The Reconnaissance Report, dated February 1993, determined that, *"In view of the limited protection afforded by the U.S. Coast Guard and the Montauk Historical Society in 1990, 1992 and 1993, no additional interim emergency measures are warranted at this time"*. This feasibility study confirmed the findings of the Reconnaissance Report. Therefore, the feasibility of a comprehensive project was explored.

2. Feasibility Cost Sharing Agreement

The feasibility study for Montauk Point, New York was initiated on April 25, 2000 and cost shared on a 50% Federal, 50% non-Federal basis at a total cost of $900,000. The non-Federal cost sharing partner is the New York State Department of Environmental Conservation (NYSDEC).

3. Feasibility Study Purpose

The Feasibility Study is the second phase of the Corps of Engineers planning process and follows a favorable Reconnaissance Report and execution of a Feasibility Cost Sharing Agreement between the Corps of Engineers and the non-Federal sponsor, NYSDEC.

The purpose of the Feasibility Study is to fully evaluate all reasonable solutions to identified problems. The Feasibility Report documents the planning, engineering, design, real estate, environmental activities and NEPA documentation required to support a decision on Federal participation in the construction of a project. The Feasibility Report is a complete decision document that provides the basis for recommending the potential implementation of a project; to be followed by a value engineering study, preparation of a Design Documentation Report and completion of Plans & Specifications, during the Preconstruction Engineering and Design phase, upon execution of a Design Agreement.

4. Previous Reports

The New York District completed a Reconnaissance Report for Montauk Point, New York in February 1993. Headquarters USACE certified the Reconnaissance Report to be in accord with Administration policy in May 1993. This report recommended that a cost-shared feasibility study be conducted. The potential recommended plan of improvement identified in the reconnaissance report entailed the placement of a 770-foot long stone revetment to cover the most critically eroding area of Montauk Point.

5. Study Area

The study area is located in Suffolk County, New York, between the Atlantic Ocean and Block Island Sound at the easternmost end of the south fork of Long Island (Figure 1). Montauk is in the Town of East Hampton. The study area includes the entire historic Montauk Point Lighthouse Complex situated on a high bluff underlain with glacial till, approximately 70-feet above Mean Sea Level (MSL). The lighthouse is the focal point of the historic complex and surrounding facilities, and acts as a junction marker for ships headed for New York Harbor or Long Island Sound. The area surrounding the lighthouse is operated by the Montauk Historical Society as a State park and is used primarily by fishermen, surfers and sightseers. The lighthouse property includes a museum that serves to educate visitors about the history of lighthouses (with historic artifacts) as navigational aids for over 200 years of our nation's history. The Montauk Historical Society (non-profit 501-C-3) is dedicated to the protection, preservation and educational development of this nationally significant historic site. Through programs, exhibits, publications and special events, the story of this site is conveyed to the public. Membership in the Montauk Historical Society and visitation to the lighthouse is fee based and open to all without any discrimination. Fees help maintain the properties and overall operation.

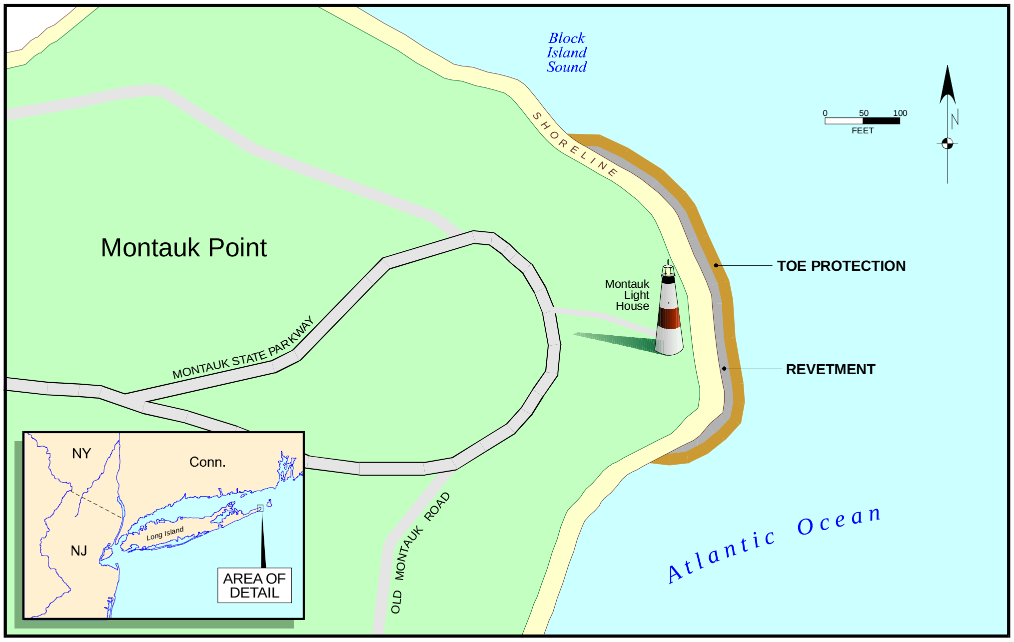


Figure 1 – Study Area

The critical area of study consists of the fronting bluff covering about 900-feet of shoreline. The ownership of the property was transferred from the U.S. Coast Guard to the Montauk Historical Society (in accordance with HR 3675, Department of Transportation and Related Agencies Appropriations Act, 1997, Sec. 341, Conveyance of Light Station, Montauk, New York). All surrounding property is owned by the State of New York.

Continued ownership of the property is subject to the condition that the Montauk Historical Society maintains the Montauk Light Station in accordance with the provisions of the National Historic Preservation Act of 1966, amended (16 U.S.C. 470 et seq.) and other applicable laws. All rights, title and interest would revert to the United States if the Montauk Light station ceases to be maintained in accordance with the National Historic Preservation Act as a nonprofit center for public benefit for interpretation and preservation of the material culture of the United States Coast Guard, maritime history of Montauk and Native American and colonial history.

The bluff and beach along this entire area are considered to be critical elements of the stability of the lighthouse. Erosion control structures are required to protect the bluff faces from the forces of oncoming waves. The area of concern consists of 2,300 feet of shoreline, extending from the pivotal point of the adjacent bluff to the south to a beach area to the north (Figure 2). The entire area must be considered in order to prevent adverse impacts from this project, and to make certain it is environmentally sustainable.

Study Area

Figure 2 – Area of Concern

6. Background & History

Montauk Point is located in Suffolk County, approximately 125 miles east of New York City. The Point separates the Atlantic Ocean to the south from Block Island Sound to the north. The Montauk Point Lighthouse acts as a junction marker for ships headed for New York Harbor or Long Island Sound. The Montauk Point Light Station was authorized for construction in 1792 by President George Washington. Construction was initiated in June 1796 and completed in November 1796 at a cost of $22,300. The lantern is about 80-feet above the ground. The lantern was lit with sperm oil until the 1860s, kerosene until the 1940s and, finally, electricity with a 300,000-candlepower lamp. When the light was completed it was located 300-feet from the edge of the cliff. Presently the lighthouse is less than 120-feet from the edge of the bluff, and other major structures are now precariously situated within 50-feet of the bluff edge.

The Montauk Point Lighthouse is listed on the National Register of Historic Places. Since its construction, the lighthouse has served as an important navigation aid for the first land encountered by ships headed for New York Harbor and Long Island Sound, as well as other ports on the eastern seaboard. Continued erosion has been recognized as a problem for many decades and various efforts have been made to stabilize the shoreline with limited success. The following is a historical account/review of the area (Figures 3 thru 9 illustrate historical shoreline evolution on a qualitative basis).

1792 The lighthouse is authorized for construction by President George Washington on land previously utilized by the Montaukett Indians. The shoreline is approximately 200-feet seaward of the present position.

Figure 3 - Montauk Point, 1878

Figure 4 - Montauk Point, 1928

1946 A 700-foot stone revetment is constructed at the bluff toe, with vegetative plantings along the upper half of the cliff (New York District, 1944). The crest elevation is +20 feet MSL, tapering down to +15 feet MSL at both ends. The crest width is 23-feet with a core and double armor layer of 4 to 8-ton stone. The base layer is 8-ton stone. Since its construction, this entire seawall has completely failed and is now 10 to 70-feet seaward of the existing bluff toe. Most of the stone is at an elevation of about mean high water, with remnants present as rubble along the seaward extent of the present structure toe.

Figure 5 - Montauk Point With Revetment, Circa 1946

Figure 6 - Montauk Point, 1950s

1960s Department of Transportation places rubble over the edge of the bluff just to the south of the lighthouse. After the October 1991 storm, the rubble slides down the slope, due to scouring of the bluff toe. Most of the rubble is subsequently cleared away during the construction of the revetment in 1992.

Figure 7 - Montauk Point, 1960s

1971 The first terracing project is constructed along the bluff slope by Ms. Georgina Reid, a locally renouned preservationist. The construction is on U.S. Coast Guard property just north of the lighthouse.

1972 U.S. Coast Guard places gabions along about 280 feet of the point above the failed 1946 seawall along the toe of the bluff. The gabion system subsequently settles gradually and the crest is of insufficient elevation (only up to +15 feet MSL) to provide protection. It is significantly damaged by the Halloween Storm of 1991.

1980s Terracing and beach grass plantings continue through the 1970s and 1980s. The vegetation includes beach grasses, bushes, seedlings, shrubs and wildflowers up to five feet in height. Dense foliage occupies most of the north end of the point. The lower east side of the bluff is reshaped to a more stable angle, terraced with lumber and secured by steel stakes to provide a flat surface for the beach grass. The vegetation appears to hold the bluff face against the forces of ground seepage, rainfall and runoff. Terracing efforts subsequently deteriorate due to the impacts of major storms in the early 1990s.

1990 The Montauk Historical Society and the New York State Department of Parks and Recreation construct a revetment along Turtle Cove, south of the lighthouse. A 6-feet deep, 15-feet wide trench is excavated for the toe of 263 lineal feet of revetment. Geotextile fabric is placed in the trench and a base layer of 50-pound stone is placed on the fabric. Up to 14,000 pound stones are placed on the base stone up to an elevation of +20 feet MSL. The revetment subsequently settles to a crest elevation of +5 to +10 feet MSL during the October 1991 storm and is no longer adequate as a shore protection structure.

1992 After severe erosion due to the Halloween Storm of 1991 (The Perfect Storm), a new revetment is constructed by the U.S. Coast Guard landward of the old revetment. An emergency construction effort commences along about 300 feet of shoreline. The crest elevation is +25 feet MSL, with 1-3 ton stones placed on the slope above a 14 feet wide berm crest at elevation +18 MSL of a single 10-ton armor layer, which slopes down to the existing toe (generally on stone from the 1946 failed revetment). The Montauk Historical Society constructs a 150-foot long structure along the eastern section of Turtle Cove. The design is similar to the Coast Guard section but 5- to 10-ton stone is used.

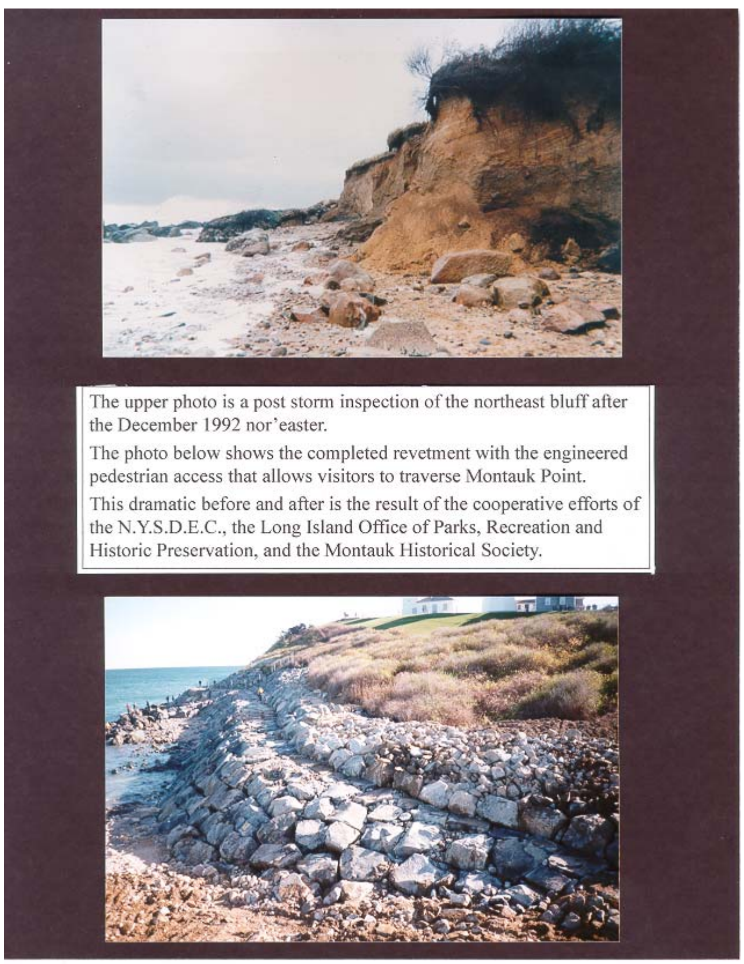


Figure 8 - Montauk Point, 1992

1993 New York District Reconnaissance Study determines sufficient economic justification and Federal interest to conduct a feasibility study.

The same camera angle thirty years later shows a stabilized toe and bluff face. The cooperative efforts of the U.S.A.C.O.E., the U.S.C.G. the N.Y.S.D.E.C., the Long Island Office of Parks, Recreation and Historic Preservation and the Montauk Historical Society have improved the erosion control protection at Montauk Point. (c 1995 Peter Paul Muller Jr.)

Figure 9 - Montauk Point, 1995

7. Existing Conditions

Because the present shore protection measures (somewhat similar to the 1946 revetment that failed) were not designed to withstand significant storm events over a substantial duration, i.e. lack of a buried toe, inadequate stone size, and insufficient overtopping protection, it is expected that the revetment now in place will fail in the foreseeable future. When the lighthouse was originally completed, it was located 300 feet from the edge of the bluff. Presently, the lighthouse is less than 120 feet from the edge of the bluff, and other major structures within the complex are now within 50 feet. As noted during recent site inspections, the current revetment is sustaining damage due to stone movement. Based on stone size and crest elevation, the design level of protection provided by the existing structure is estimated between a 10-year and 15-year frequency storm. Although the existing structure is subject to, and is exhibiting signs of the beginning of the slope failure process, no emergency construction is expected to be necessary prior to potential construction of a comprehensive plan of protection. Monitoring following storm events would determine whether prudent remedial measures should be taken if a comprehensive project is not implemented.

Recent efforts, including terracing, vegetation and improved revetment construction, have decreased the erosion rate. Repeated storm effects, however, will continue to cause erosion at the ends of the structure, eventually compromising the revetment and upper bluff areas. This, in turn, is expected to result in the eventual loss of bluff material, the lighthouse and its adjacent structures.

8. Hydrology - Existing Drainage

Montauk Point Lighthouse is located on a knoll with the surrounding topography sloping away from the lighthouse along a steep gradiant. The site is well vegetated and contains slopes of up to 40 percent grade. Slope lengths are short and show little sign of past erosion. Drainage facilities at the site consist of roof drains, a slotted drain and bluff terraces. The site can be divided into three primary hydrologic drainage areas:

- The bluff area surrounding the lighthouse - runoff from this area flows over the bluff to the Atlantic Ocean.

- Area south of the lighthouse between the bluff and the concrete driveway leading to the lighthouse - runoff from this area flows southwest towards the Atlantic.

- Area north of the lighthouse driveway - runoff from this area flows north towards Long Island Sound.

Sources of runoff at the site include lawn areas, building roofs and paved areas. The site contains minimal facilities for the collection and conveyance of stormwater. Runoff from the lawn areas flows to the Atlantic Ocean via uncontrolled overland flow. No conveyance channels are utilized in directing runoff from lawn areas to specific discharge points. Since most of the slopes within the lawn area are relatively short, runoff can be expected to exist in the form of sheet flow. However, due to the vegetated condition of the unimproved areas of the site, runoff velocities are low enough to prevent rills from developing on sloped areas.

The bluff has been terraced and vegetated to reduce the erosion of the bluff face. In addition to the vegetated terraces, rock outlets have been constructed in areas prone to concentrated flow conditions due to natural drainage patterns or groundwater discharge. Roof drains from the museum have outlets along the slopes surrounding the structure. Although a source of concentrated flow, the roof drain outlets do not appear to cause adverse impacts to the grassed slopes. A third discharge point, located on the south side of the museum, discharges water from the roof drains on the south side of the building. Additionally, some of the roof drains discharge to the lawn area without being conveyed away from the buildings with discharge pipes.

Generally, the drainage facilities at the site appear to be adequate and cause no adverse impacts to the surrounding area. Little evidence of past erosion was observed at the site. Routine maintenance of the drainage facilities and vegetation is needed to prevent occurrence of erosion in the future.

9. <u>Geology</u>

A subsurface exploration program was conducted at Montauk Point Lighthouse to assess the subsurface conditions of the site (reference Engineering Appendix). The results confirm the basic layering scheme presented in the USACE *Reconnaissance Report: Montauk Point, New York,* 1993. That report described a three-layer model, consisting of Montauk till at the base, overlain by (lower) stratified Hempstead gravel (composed of distinct strata of sand, silt and clay) and a surface layer (upper) Hempstead gravel (composed of cohesionless fine sand with little silt). All of Turtle Hill, on which the lighthouse stands, is a slump block that remained after the retreat of a glacier.

10. <u>Historic Shoreline Changes – Erosion Processes</u>

Bluff erosion is caused by a number of forces. At the toe of the bluff, erosion forces include:

- Astronomical and storm tides that allow waves and tidal currents to gradually erode the toe of the bluffs that were exposed with no overlying stone.

- Waves and currents that serve to mobilize and transport sediments away from the shoreline. As the bluff toe erodes and steepens, the upper bluff collapses and slides into the ocean. There is also a net loss of beach material due to littoral transport.

Erosion forces also act on the upper parts of the bluff. These sources of erosion include:

- Water collecting in upland wetlands and ponds, seeps slowly toward the sea, both on the surface and through the soil. Seepage exits on the face of the bluffs, further loosening and moving soil down the bluff face.

- Wave spray and runup erode the bluff face by saturating and washing away sediment.

- Rain adds to the erosion of the sloped bluff face and surface runoff during storms by impinging upon the sediment and washing out large amounts of soil to the beach below. A lack of vegetation on the bluff face could allow the rain and surface water to act directly on the soil. Because of adequate vegetation on the bluffs at Montauk Point, this is not presently happening, but could occur in the future if plant coverage decreases.

- High coastal winds add to the erosion process. Winds will blow loose soil from the face of the bluffs and will cause trees and taller vegetation to sway back and forth, which in turn loosens the soil at their base.

11. Storm-induced Erosion Rates

Because of the steep slopes and high elevations associated with the bluffs at Montauk Point, storms can cause catastrophic bluff failure and erosion of large amounts of soil. In the 1993 Reconnaissance Report, a site survey described erosion measurements that were made in June 1992. The survey indicated that the unprotected (beach fronted) bluff immediately to the north of the lighthouse eroded 20-feet and the unprotected (beach fronted) bluff 800-feet north of the lighthouse receded about 30-feet during the October 1991 storm.

12. Long Term Erosion Rates

Long-term erosion rates along two cross sections using reported historic shorelines and aerial photography were analyzed (Figures 10 & 11). The data was plotted in cross-section view (Figure 12). The historical long-term shoreline recession rate was found to be 2.2-feet per year for the beach and bluff toe and 1.2-feet per year for the top of bluff. In the past 125 years of record (1868-1993), the bluff has receded 150-feet and beach has receded about 330-feet. Erosion rates since 1993 in critical areas of erosion are not pertinent due to the construction of a successfully performing revetment, which is curtailing shoreline retreat. It has been estimated that the average annual erosion rate for the bluff and beach is 6 cubic yards per foot per year, resulting in a total of 5,000 cubic yards of erosion per year in the critical erosion area. The historical data shows that the beach recession rate adjacent to the revetment has been reduced by about 50% since the construction of coastal structures, whereas the bluff recession has stabilized at about 25% of the pre-1945 revetment recession rate due to the terracing construction above the revetment.

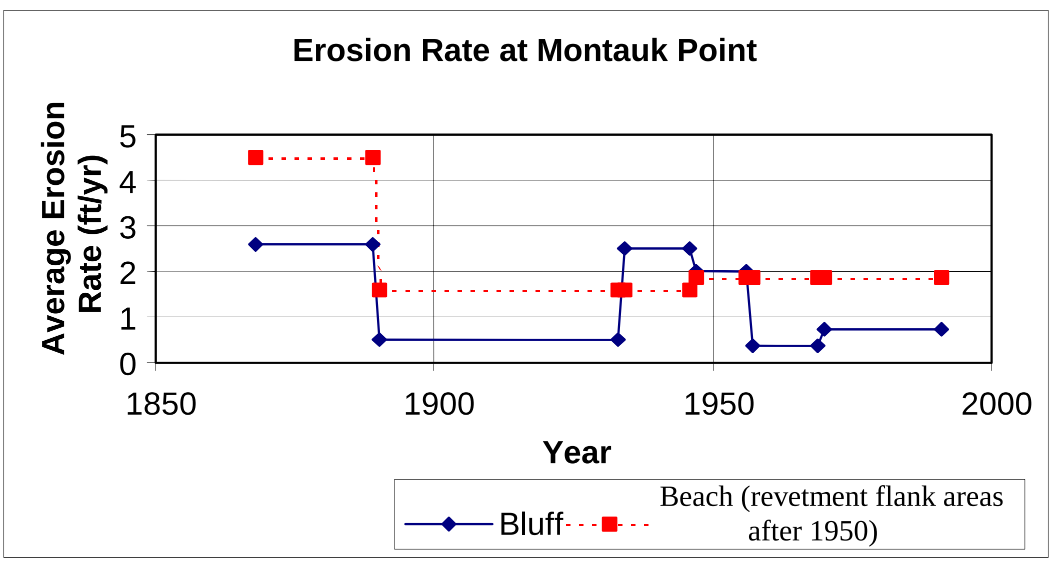


Figure 10 - Average Erosion Rates Since 1868 at Montauk Point (New York District, 1993)

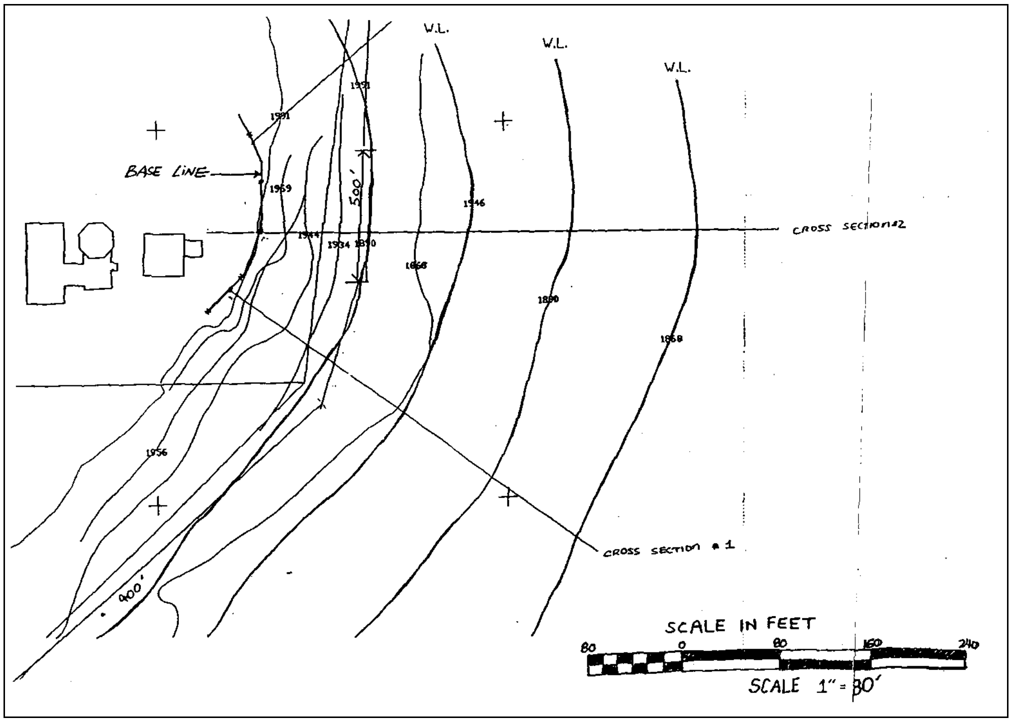


Figure 11 - Shoreline Changes 1865-1992 (New York District, 1993)

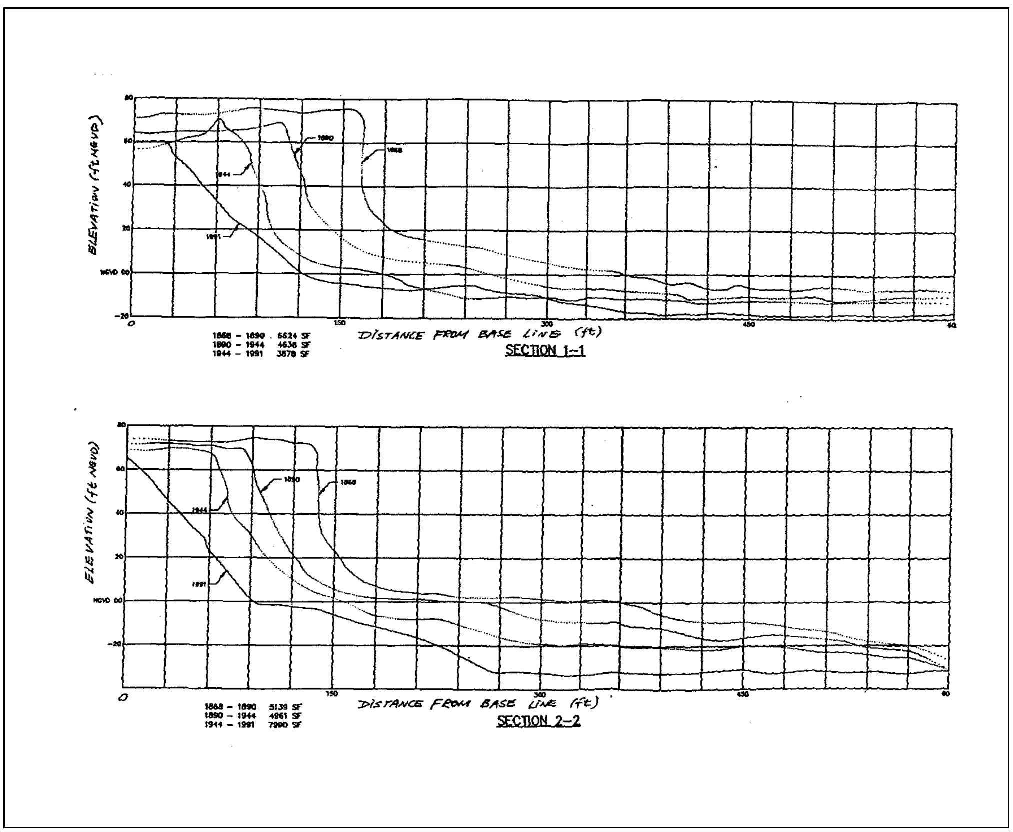


Figure 12 - Shoreline Changes, Cross-Sections

13. Waves

The basis for developing wave characteristics for Montauk Point was an excerpt from a report entitled "Fire Island to Montauk Point Reformulation Project (FIMP), Moffatt & Nichol, June, 2000". The basis of that analysis was the Army Corps of Engineers Wave Information Study, 1976-1994, with adjustments made as necessary based on "observed behavior of longshore transport" as described in a The Coastal and Hydraulics Laboratory (CHL) Progress Report dated January 1997. The wave transformation data used by Moffatt & Nichol for the FIMP study used the offshore WIS waves at Stations 75, 77, and the CHL-modified Stations 79 and 80 for the 1976-1994 time period.

Table 1 indicates that the hindcasted wave height information for storms is, on average, 5.4 feet lower than measured, the periods are 0.5 seconds low, and the directions average about 39 degrees more toward the southeast. These differences are due to a variety of details related to the numerical modeling of waves; however, for purposes of this study it should be noted that extreme waves are, on average about 5.4 feet lower than measured with significantly higher deviation (8 to 12 feet) at the high end of the distribution. These differences, however, become less of a concern in areas such as Montauk Point where the design waves are depth-limited.

Table 1 - Comparison of measured and hindcasted wave characteristics

Event	Measured Peak Wave Height, Hmo (ft)	Hindcast Peak Wave Height, Hmo (ft)	Measured Wave Period, Tp, (s) at Peak	Hindcast Wave Period, Tp, (s) at Peak	Measured Wave Direction, Dm, (deg) at Peak	Hindcast Wave Direction, Dm, (deg) at Peak
12/11/92	30.5	18.7	12.5	12.0	83	133
11/28/93	21.3	18.7	11.5	12.0	151	144
8/19/91	19.0	18.4	16.7	13.0	64	148
1/5/92	20.3	16.1	9.1	14.0	59	133
3/14/93	23.9	15.4	14.3	10.0	155	122
3/4/93	19.7	15.1	10.0	10.0	60	126

Table 2 presents extreme wave heights estimated by Moffatt & Nichol at the 32.8-feet contour irrespective of wave direction based on storm stages developed by CHL in 1996 for the Fire Island to Montauk Point Study. These stages were updated by CHL in 1998 and resulted in no change in the offshore wave development.

Table 2 - Extreme storm statistics produced by the Fire Island to Montauk Reformulation Study

Return Period (yrs)	Significant Wave Height (ft)	Storm Stage (ft, NGVD)	Max. Breaker Height (ft) (-32.8 ft NGVD contour)	Design Significant Wave Height (ft) -32.8 ft depth	Wave Period (s)
2	17.13	4.53	29.12	16.18	13.00
5	20.57	5.38	29.79	16.55	13.15
10	21.03	5.75	30.12	16.73	14.48
25	21.56	6.83	30.53	16.96	16.13
44	21.99	7.20	30.87	17.15	17.10
50	22.11	7.42	30.99	17.22	17.37
73	22.49	7.50	31.38	17.43	18.11
100	22.83	7.60	31.78	17.66	18.66
150	23.26	8.63	32.32	17.96	19.44
200	23.62	9.12	32.70	18.17	20.04
500	24.70	10.83	33.88	18.82	22.23

For development of design waves, it was determined that the waves will be depth-limited at the location of the revetment. Three approach lines (cross-sections) were developed using the most recent (2001) topographic and hydrographic surveys over which the waves at the –32.8 ft contour were transformed. The approach lines are very similar in profile view, and wave transformation model test runs indicated that the nearshore wave characteristics are all virtually identical adjacent to the revetment. Therefore one cross-section (SE) was used for detailed wave transformation modeling. The nearshore model SBEACH was employed to perform the wave transformation because it is a one-dimensional model that includes surf zone processes that are very important in this exposed environment.

Nearshore design waves were also developed for comparative purposes using the spectral model STWAVE. Boundary wave spectra were developed using the extreme significant offshore wave heights and wave periods along with waves from the East and South-Southeast. At Montauk, the presence of more northerly exposure is blocked, so the worst storm waves would be more from the East to South-Southeast. For each wave case, the appropriate water level was added to the water depths based on the CHL extreme storm surges.

Table 3 presents the results of the calculations for significant wave heights at the toe of the present structure based on the two numerical models employed. The differences in results at the structure toe are due to slightly different representations of the bottom profile and the wave breaking processes.

Table 3 - Without-Project Storm Significant Wave Heights at Toe of Revetment

Storm Return Period (years)	Wave Height at Toe (ft)	Wave Height at Toe (ft)	Local Wave Direction for Storms from E	Local Wave Direction for Storms from SSE
	SBEACH	**STWAVE**	**Deg from due E**	**Deg from due E**
2	4.36	3.27	+5	-22
5	4.82	4.75	+5	-27
10	5.05	5.19	+5	-25
25	5.41	5.86	+5	-26
50	5.77	6.35	+5	-27
72	6.05	6.55	+5	-27
100	6.40	6.80	+5	-27
500	7.87	8.77	+5	-29

(Wave directions are from STWAVE. At Turtle Cove, the wave directions are –12 deg for easterly storms and –60 deg for south-southeasterly storms)

14. Water Levels

Astronomical tide statistics were reviewed from two sources: the New York District's Reconnaissance Report for Montauk Point, and NOAA Benchmark Sheets for Montauk Point (Fort Pond, New York). The tidal statistics are generally within 0.31 feet for all relevant tidal datums, with the NOAA statistics higher than those used in the reconnaissance report. Using the relationship between current Mean Sea Level and NGVD29 that the NOAA tidal datums were referenced to NGVD29 and are shown in Table 4.

Table 4 - Tidal statistics for Montauk Point

Level	Elevation, MLLW feet (NAN, 1993)	Elevation, MLLW feet (NOAA, 2001)	Elevation, NGVD feet (NOAA-0.8')
Mean Higher High Water (MHHW)	2.4	2.60	1.80
Mean High Water (MHW)	2.0	2.31	1.51
Mean Sea Level (MSL)	1.2	1.24	0.44
Mean Low Water (MLW)		0.18	-0.62
Mean Lower Low Water (MLLW)	0.0	0.00	-0.80

The Coastal and Hydraulics Laboratory (CHL, 1998) refined the storm surge levels for the Fire Island to Montauk Point Reformulation Project that were presented in Table 2. Those levels, which included a tabulation of stage-frequency values for the combination of tropical and extratropical storms, are added to the astronomical Mean Sea Level to produce total water levels shown in Table 5.

The highest observed water level, according to NOAA recorded water levels at Montauk, was +7.90 feet NGVD recorded in 1954. However, this water level was taken offshore and did not include the significant impact of wave setup (refer to Table 5).

Table 5 - Storm tide statistics developed by the Coastal and Hydraulics Laboratory

Return Period (years)	Combined Storm Surge (Tropical plus Extratropical), NGVD feet	Combined Storm Surge + Astronomical MSL, NGVD feet	Wave Setup (from FIMP)	Storm Surge + Wave Setup + Astronomical MSL, NGVD feet	Utilized Storm Stage + Wave Setup NGVD feet
5	4.76	5.20	2.72	7.92	8.10
10	5.34	5.78	2.88	8.66	8.69
25	6.14	6.58	3.19	9.77	9.52
50	6.73	7.17	3.42	10.59	10.34
100	7.33	7.77	3.57	11.34	11.51
500	10.29	10.73	3.88	14.61	14.51

15. Tidal Currents

Tidal currents play a role in transporting sediment along the beach. At a location such as Montauk Point, flows pass around the point as the astronomical tidal wave enters Long Island Sound to the north and the Atlantic Ocean to the south. Currents are very strong along the toe of the revetment and likely enhance the transport of fine sediments that are winnowed from the bluff face after being mobilized and sorted by waves.

16. Slope Stability Analysis

The till exposed in the wave cut bluffs surrounding Montauk Point is a well graded mixture of boulders, sand, gravel, and underlying silt pre-consolidated by the weight of glacial ice (Figure 13). It has a long stand up time for near vertical slopes but gradually erodes and fails with time under annual rainfall and runoff (Figure 14). Under large magnitude wave actions with high storm surges, the dense till will be scoured and result in toe failures of the mid to upper bluff above the revetment for the till and overlying granular soils. The slope stability analysis of existing conditions was performed for sections representing the steepest slopes that are near and surrounding the lighthouse. Results indicated that the slopes and the present conditions are at equilibrium with little or no safety margin.

The upper parts of the slopes, which are near the angle of repose for the granular soils, show the highest potential for failure if existing conditions are even slightly disturbed. The upper slopes would consistently fail if terracing and vegetation stabilization measures, maintained by the Montauk Point Historical Society, were not practiced.

In addition, a greater surface area and volume of material near the shoreline can fail with external disturbance, however this is unlikely to occur due to the very dense nature of the underlying shoreline soil (till).

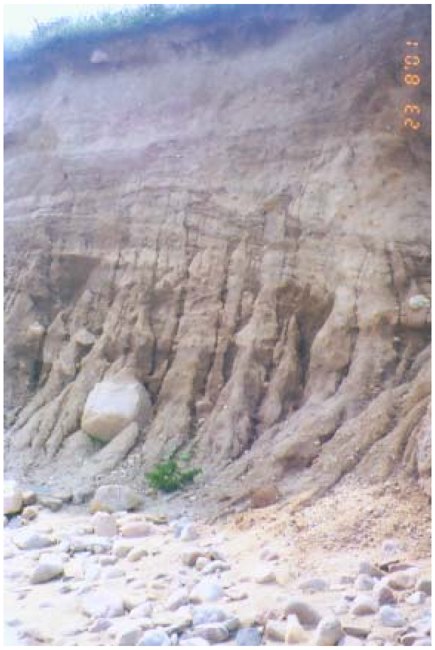


Figure 13 - Typical bluff cross-section with glacial till at Montauk Point

Figure 14 - Eroded bluff south of Turtle Cove with wave-eroded toe and consequent bluff failure

17. Scour, Runup, Overtopping, Wave Attack Forces

The toe of the existing stone revetment consists of stone overlying stiff glacial till. The scour mechanisms associated with glacial till and stone are not predictable using numerical models. Therefore a physical model was built to assess failure mechanisms. Because the revetment toe is glacial till and generally overlain with stone, it is expected that toe scour will be minimal during a given storm event but would be subject to some, but not significant, long-term scour. Runup due to the maximum breaking wave at the toe of the revetment ranges from +22.2 feet NGVD during a 2-year event to +32.0 feet NGVD during a 500-year event (Table 6). Based on the topography data collected in 2001, it appears that the revetment is overtopped along its entire length by the upper 2% of wave runups during all storm events listed. This is consistent with observations that wave runup on the order of several feet deep occurs along the fence at the top of the revetment, which varies as low as elevation +20 feet NGVD.

Table 6 - Without-project, maximum runup & potential for overtopping

Storm Return Period (years)	Max. Water Level at Toe w/Wave Setup (feet, NGVD)	Breaking Wave Height (feet, NGVD)	Max. Runup Level (feet, NGVD)	Revetment Overtopped	Revetment Threatened
2	7.1	7.50	22.2	Entirely	No
5	8.1	8.37	23.4	Entirely	No
10	8.7	8.82	23.4	Entirely	No
25	9.5	9.57	23.7	Entirely	Yes
50	10.2	10.32	24.5	Entirely	Yes
100	11.5	11.38	26.4	Entirely	Yes
500	14.5	13.00	32.0	Entirely	Yes

18. Stability Analysis Evaluation

The stability analysis of existing conditions was performed along three cross sections (Figure 15). These sections represent the steepest slopes that are near and surrounding the lighthouse. Section A-A is the steepest of the three sections.

The stability analysis on each section indicates that the slopes and the present conditions are at equilibrium with little or no safety margin. This is indicated by a factor of safety of approximately 1.0 for Sections A-A and B-B. The upper parts of the slopes, which are near the angle of repose for the granular soils, show the highest potential for failure if existing conditions are even slightly disturbed. At Section A-A, the upper slopes would consistently fail if terracing and vegetation stabilization measures, maintained by the Montauk Point Historical Society, were not practiced.

It is noted that Section C-C is stable with a minimum slope stability factor of safety of 1.25 due to the lower bluff elevation and milder slopes. However, a much larger failure surface and volume of material starting at the shoreline also has a low factor of safety (1.094) in section A-A and can fail with external disturbance, although much less likely due to the very compacted state of the soils near the toe of the bluff below elevation +15 feet NGVD +/-.

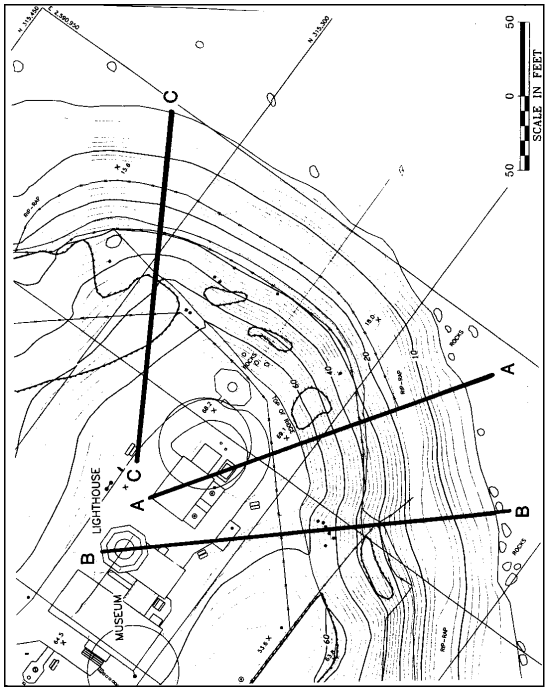


Figure 15 - Locations of cross-sections for slope stability modeling

19. <u>Without-Project Future Conditions</u>

Existing conditions of the revetment show that if no further protection to the fronting bluff occurs, there is a significant threat to the existing bluff protection and ultimately to the treasured resources of the lighthouse, bluff and surroundings. The following describe how the deterioration of the existing bluff protection would lead to direct storm damage and eventual loss of the lighthouse complex and surroundings. Three possible failure modes are considered in determining the remaining life of the existing shore protection structure.

1) Toe erosion at the base of the revetment that would lead to toe stone instability and revetment collapse;

2) Wave action dislodging lighter than required armor stones prevalent and interspersed on the revetment surface; and

3) Wave runup and overtopping that would dislodge the revetment crest stones and lead to revetment collapse.

The exact elevation of the toe of the present structure is not well-defined, but is estimated from photographs and spot elevations in recent topographic surveys obtained by the New York District in 2001. It is noted that failure of the revetment would be followed by bluff failure, which would then threaten the lighthouse. Revetment failure alone will not cause the immediate catastrophic failure to the lighthouse, since the slope stability of the bluff, after revetment failure, has a factor of safety greater than 1. The recession of the bottom profile for the beachfront flanking the revetment is less than the maximum (due to a differing shoreline orientation) based on historical recession rates below the water line.

A corresponding sea level rise (0.01-feet/year) and profile horizontal recession (approximately 1-foot/year historically) for the beachfront flanking the revetment is included. For the revetted area, the recession rates are assumed to be negligible due to the presence of the revetment (recession of the upper part of the profile is assumed, based on performance, to be arrested by vegetative shore protection measures). In addition, erosion immediately adjacent to the revetment will diminish below the historical 1-foot/year rate due to the sheltering effect of the existing revetment, and thus, will negate flanking potential.

These three modes of failure can occur individually or in combination. Because of the uncertainty in predicting the impacts of these three modes of failure (i.e. stone displacement from wave impacts due to undersized stone, erosion of the toe foundation soil (hardened till), or displacement of stone on the upper part of the revetment due to wave runup and overtopping) a physical model of the revetment was undertaken (2002) at the University of Delaware Center for Applied Coastal Engineering. Based on the results of the physical model, the primary mechanism expected to cause bluff failure is the effect of waves, including direct impact and runup/overtopping, on the armor stone. Large-scale slope failure (i.e. that initiated at the shoreline or structure toe) is not expected to occur due to the presence of glacial till and large amounts of stone overlying the soils.

<u>Failure of the structure due to revetment toe erosion</u>

The physical model was not able to exactly replicate the condition of the dense foundation soil (glacial till) overlain with a thin veneer of sand) at the revetment toe, but these conditions in the model were simulated with a hardened bottom. In addition, based on eyewitness accounts from continuous observation over extended periods of time, including severe storms, both storm-induced and long-term toe erosion are considered to be relatively minor in terms of toe stone instability. Although some long-term erosion does occur in the revetted area, it is difficult to compute or otherwise quantify realistic rates. Maintenance practices will tend to protect the base of the structure, and the predominance of dense glacial till overlain by stone will significantly retard toe erosion. Therefore, the toe erosion mode of failure is not considered pertinent to the overall cause of failure.

Failure of armor layer (displacement of armor stone on the revetment slope)

When the water level is elevated by both astronomical tide and storm surge, waves impact the armor stone. The present armor stone will be stable in waves of a certain size, above which they are expected to become damaged (dislodged), resulting in failure of the structure. Because the existing structure is not a recommended type of cross section, i.e. significantly varying stone sizes of one layer with no buried toe, but with stone that is interlocked tightly, and the associated uncertainty in stone performance under storm conditions, a physical model was constructed to replicate the existing revetment as closely as possible in terms of variance in stone size and degree of interlocking. The model tested storm waves ranging from the 2-year return period to the 100-year return period range and very minor displacement of armor stone on the revetment slope was observed. The model included areas of undersized stone interlocked among larger armor units and no failure was observed for the range of waves tested. Thus, this failure mode is not considered pertinent to the overall cause of failure

Failure due to displaced armor from wave overtopping

Additional slope stability analyses were performed to model the reduction of the height of the upper revetment due to wave overtopping and the subsequent wave scour of the underlying soils of a failed revetment. These analyses were performed for three cases: the existing revetment height to an elevation of +18 feet MSL; a revetment height of +14 feet MSL after lowering by initial upper revetment failure; and the failed revetment with a height of +10 feet MSL. These analyses, combined with the physical model results that show upper revetment failure below elevation +10 MSL, indicated that under the latter conditions, the top of the slope would recede landward a distance of approximately 26 feet subsequent to failure of the revetment between elevations +10 feet MSL and +18 feet MSL. The slope profile changes are presented on Figure 16.

The physical model was tested for wave runup and overtopping of the revetment for an approximately 2-year return period storm through an approximately 100-year return period storm. Based on the results of the model test, it was determined that stone displacement, from overtopping of the revetment crest, is anticipated between a 10-year return period storm and a 20-year return period storm, say a 15-year return period storm. This result is substantiated by a semi-empirical analytical method to determine damage threshold exceedance from overtopping (Coastal Engineering Manual 1997 Part VI).

Since the last storm experienced at Montauk Point of this significance was in 1993, there is a likelihood (with a 60% probability) that this 15-year return period storm will occur by the year 2006 and cause significant damage (at least 25% damage level) to the revetment itself.

The previous paragraphs address the potential for significant damage to the revetment. However, this damage in and of itself does not create an immediate threat to the lighthouse since bluff slope failure affecting the lighthouse would not occur just from damage to the revetment. Once the upper sections of the revetment are displaced (2006), the foundation soil underlying the displaced stone would become exposed and subject to subsequent erosion. To determine the extent of erosion of the toe of the upper bluff above the damaged revetment that would cause significant bluff failure to threaten the stability of the lighthouse structure, a slope stability analysis was performed.

The results of this analysis (Figure 16) determined that for significant bluff failure, the damaged crest elevation of the revetment would degrade to approximately +10 feet NGVD (indicated by the physical model from a 10-year return period to a 20-year return period storm) and the upper bluff toe at that elevation would recede horizontally approximately 10 feet. This should cause about 30 feet of loss of the bluff crest and immediately threaten the lighthouse facility at the most critical area to the southeast of the structure.

The period of time estimated for this condition to occur, subsequent to 2006, is an additional 8-10 years, which results from long term erosion at the upper bluff toe (at elevation +10 feet NGVD) with no significant storm occurrence, or from an approximately 10-year return period storm which has a likelihood of occurrence (60% probability) by the year 2015.

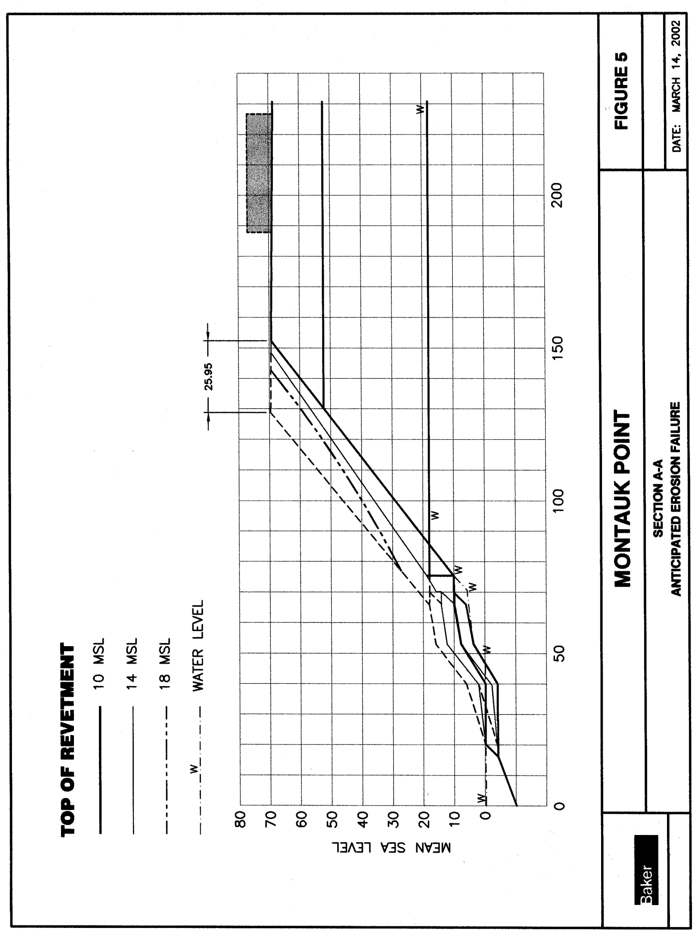


Figure 16 – Slope Profile Changes

Most Likely Without-Project Future Condition

The history of this area indicates that the U.S. Coast Guard and later on, the Montauk Point Historical Society, have attempted to repair the revetment protecting the bluff whenever a severe storm has damaged the protective structure. The most recent storm that damaged the revetment occurred in 1993. Afterwards, the Montauk Point Historical Society raised funds to repair the revetment back to its pre-damaged condition, approximately a 15-year storm design.

Historically, emergency repairs were accomplished in the 1960s, 1970s, 1980s, and 1990s prior to 1993, which has been the consistent practice since the construction of a revetment to protect the bluff in 1944. However, these emergency repairs, financed with limited available local funds, will ultimately not be able to keep the revetment structure intact, which will lead to the eventual loss of the lighthouse complex and surroundings.

20. Problems, Needs, Opportunities, Planning Objectives

Problems

Presently the lighthouse is less than 120-feet from the edge of the bluff and other major structures are within 50-feet of the bluff edge. Continued erosion has been recognized as a problem for many decades and various efforts have been made to stabilize the shoreline with varied success. Over several decades, many erosion control measures have been constructed to protect the lighthouse from the danger of erosion. Presently the existing revetment, built in the early 1990's, provides protection. Because the present shore protection measures were not designed to withstand major storm events over a substantial duration, i.e. lack of buried toe, inadequate stone size, insufficient overtopping protection, it is expected that the revetment now in place will fail in the foreseeable future.

The revetment, in combination with other recent efforts, including terracing, vegetation and improved revetment construction, has decreased the erosion rate. However, the lack of a buried toe and random interspersed inadequate stone size, over time, is leading to loss of adequate stone interlocking and eventual anticipated displacement of upper revetment armor stone from wave overtopping. This will lead to the eventual compromise of the revetment and upper bluff areas, which subsequently, is expected to result in the eventual loss of the lighthouse and its adjacent structures if no corrective action is taken.

Though there have been repeated efforts to halt the progressive erosion of the bluff, these actions have had limited success. All efforts have worked for a time, but none could provide long-term protection. The remaining lands and lighthouse are so important that the State of New York, Montauk Historical Society and local interests are expected to continue to fight the erosion, but only with a scale of protection defined by past practices.

Needs

Erosion has seriously reduced the ability of the shorefront in the project area to provide adequate protection to backshore properties from coastal storms and wave attack. As a result of future projected revetment instability and subsequent bluff erosion, the historic structure, as well as the associated artifacts within the vicinity, will be in critical danger if a long-term protection plan is not implemented.

Opportunities

There have been numerous locally coordinated efforts to fortify the critical shoreline areas by the State, Town and U.S. Coast Guard, in order to protect the Montauk Point complex, a national treasure. Opportunities exist to complement, enhance and augment local efforts in a collaborative planning environment.

Planning Objectives

Planning objectives were identified based on the problems, needs and opportunities as well as existing physical and environmental conditions present in the study area. The main Federal objective is to contribute to National Economic Development (NED) consistent with the nation's environmental policy, pursuant to national environmental statutes, applicable executive orders and other Federal planning requirements. The following general and specific objectives have been identified:

General requirements include:

- Meet the needs and concerns of the public within the study area
- Respond to the public desires and preferences
- Be flexible to accommodate changing economic, social and environmental patterns and changing technologies
- Integrate with and be complementary to other related programs in the study area
- Implement with respect to financial and institutional capabilities and public consensus
- Conform with USACE environmental operating principles

Specific requirements include:

- Protect Montauk Point and vicinity, including the historic lighthouse and associated facilities from erosion, environmental degradation and coastal storm damage
- Reduce the threat of future bluff instability by protecting against wave attack and erosion from ocean impacts
- Provide an economically justified approach for bluff protection at Montauk Point
- Prevent the aggravation of erosion in adjacent areas

Technical constraints include:

- Plans must represent sound, safe and acceptable engineering solutions taking into account the overall littoral system effects
- Plans must be in compliance with Corps of Engineers regulations
- Plans must be realistic and state-of-the-art while not relying on future research
- Maintain proper stone interlocking for bluff protection
- Plans must provide features that minimize the effect of shoreline erosion processes

Economic constraints include:

- Plans must be efficient, make optimal use of resources and not adversely affect other economic systems
- Average annual benefits must exceed the average annual costs

Environmental constraints include:

- Plans must avoid and minimize environmental impacts to the maximum degree practicable
- Plans must consider mitigation or compensation for a potential impact when identified

Regional and Social constraints include:

- All reasonable opportunities for development within the project scope must be weighed, with consideration of state and local interests.
- The needs of other regions must be considered and one area cannot be favored to the detriment of another
- Plans must maintain existing cultural resources to the maximum degree possible, and produce the least possible disturbance to the bluff
- Plans must maintain recreational fishing and surfing experiences

Institutional constraints include:

- Plans must be consistent with existing federal, state and local laws
- Plans must be locally supported and signed by local authorities in the form of a project cooperation agreement, guarantee for all items of local cooperation including cost sharing and all lands, easements and rights-of-way
- Local interests must agree to provide public access to the shore in accordance with Federal and state guidelines and laws
- The plan must be fair and find overall support in the region and state

21. Preliminary Alternatives

Criteria for evaluating preliminary alternatives will include appropriateness to site conditions, compliance with New York State Coastal Zone Management criteria, effectiveness of protection, impacts on environmental and cultural resources, and costs (including interest during construction and maintenance).

The feasibility study must formulate and design long-term protection for the lighthouse complex and surrounding area. Preliminary alternative approaches need to be considered in order to develop the most appropriate form of shoreline stabilization for the area.

Preliminary cost estimates are included so that the most cost effective and efficient solutions, considering coastal processes impacts, can be selected for detailed design and economic optimization.

Alternatives that are feasible approaches to storm protection and shoreline stabilization need to address both present and future needs. The present need is to eliminate the threat of erosion and to provide acceptable levels of protection from the impacts of wave attack and storm recession.

<u>Preliminary Alternatives include</u>:

- Alternative # 1 - No Action Plan
- Alternative # 2* - Stone Revetment
- Alternative # 3* - Offshore Breakwater with Beach Fill
- Alternative # 4* - T-groins with Beach Fill
- Alternative # 5* - Beach Fill
- Alternative # 6* - Relocation of the Lighthouse

** Alternatives # 2 thru # 6 are developed at the same storm design. They are designed to withstand a 73-year return period storm. This level of design is commensurate with a project evaluation over a 50-year period, because over 50 years there would be a 50% risk of a 73-year or greater storm event.*

22. Preliminary Alternative #1 - Repair Structure As-Needed (No Action)

The No Action Plan (no Federal action through the Corps of Engineers) would consist of a continuation of the without-project condition, which includes the eventual displacement of the existing revetment and subsequent erosion of the exposed bluff. If allowed to occur, progressive instability of the bluff would result in the irrecoverable loss of the lighthouse and its associated structures, along with archaeological resources.

While the no action plan fails to meet objectives and needs of the project area, it does provide the basis against which project benefits are measured.

- Emergency efforts by the Montauk Point Historical Society to control the erosion are expected to continue, but in the absence of a comprehensive shore protection project, experience shows that their efforts have not solved and would not solve the long-term problem of significant damage to the existing structure complex, with associated threat to the lighthouse from large storm events over an extended period of time (e.g. 50 years).

- It is estimated that the present revetment structure is susceptible to damage from a 10 to 20-year storm frequency event but progressive damage will occur during lesser events. Emergency repairs will not be able to keep the structure intact without efforts to upgrade the structure design.

- The implication of the bluff failure would be the total loss of all historic properties, both buried and above ground. The architectural and archaeological remains at the lighthouse complex are an invaluable resource in terms of information and national cultural heritage. Once this information is lost, it can never be recovered, and future study of the complex would be impossible. The loss of the lighthouse complex violates the National Historic Preservation Act of 1966, as amended. Bluff failure would also lead to an eventual change in habitat types, and in the recreational use potential of the area

- If the lighthouse complex is lost, the Coast Guard would have to construct a new navigation aid to replace the lighthouse.

23. Preliminary Alternative #2 - Stone Revetment

A riprap stone revetment was proposed for long-term erosion control, as shown in Figure 17. The plan consists of 840-feet of revetment protection. The protection covers the most vulnerable bluff area that would directly endanger the lighthouse complex due to bluff failure without the project. The revetment design was based on Engineering Manual 1110-2-1614 "Design of Coastal Revetments, Seawalls and Bulkheads." A heavily embedded toe shall be employed to stand against breaking waves at the toe of the structure. The revetment section features a 40-feet wide crest at +25 feet NGVD, a 1V:2H side slope, 12.6-ton quarrystone armor units extending from the crest down to the embedded toe. Three layers of 4-5 ton armor units are used to construct the splash apron. Filter cloth and sublayers are specified in accordance with standard Corps of Engineers design procedures.

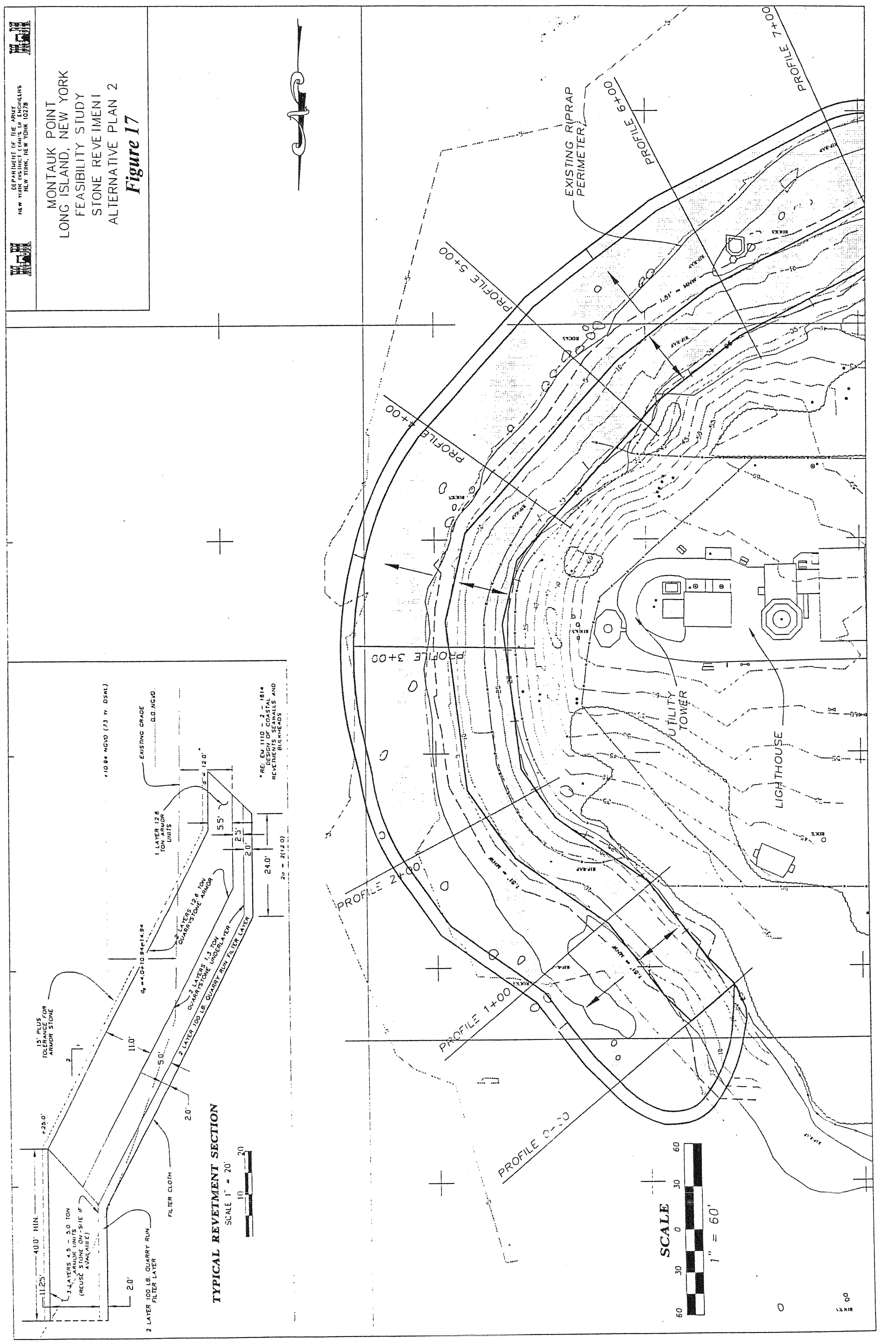
The estimated first cost for the stone revetment is $14,843,000, including 20% contingency, engineering and design, and construction management.

- Revetments are a proven method of shore protection in this area and have a record of acceptance by state and local authorities. Revetment alternatives such as this can utilize much of the stone already on site in the existing structure, thus making good use of existing resources.

- The placement of a stone revetment along the face of the bluff will have a minimal impact on the buried and above ground historic properties. In fact, the addition of stones along the current wall will provide the greatest possible protection for the historic properties and allow them to remain in place for future study.

- The cross section of the revetment can be slightly modified to allow access by fishermen to areas close to the water. It is not expected that a new revetment will change present surfing conditions in any way. There is a revetment in place at Montauk Point now, and the surfing is considered to be good. The proposed revetment will be in the same place as the existing revetment, made of similar rock material, and will be similar to it in all particulars which might effect waves. Wave reflection coefficients are estimated at 13-19 percent less for the proposed revetment alternative than for the existing revetment. The proposed plan will not change wave conditions in any perceivable way.

- Effects of both the existing structure at Montauk and the proposed structure on the littoral sand transport are small and are expected to be local, i.e. are not likely to extend as far alongshore to developed areas such as Ditch Plains or more developed points further west, especially in view of the variable shoreline shape and materials in between. The amount of material that would be contributed downdrift if the proposed revetment alternative was not constructed is very small relative to the total sediment budget and this small amount is added back by some equally minor increase in erosion, a relatively short distance west of the revetment. Any adverse impacts from the proposed revetment alternative are not considered to be significant.



MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2
Figure 17
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
FILTER CLOTH
EXISTING GRADE
EXISTING RIPRAP PERIMETER
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
UTILITY TOWER
LIGHTHOUSE
SCALE
1" = 60'

24. Preliminary Alternative #3 - Offshore Breakwater & Beach Fill

The purpose of an offshore breakwater is to reduce the storm wave height offshore of the revetment toe, thus reducing the wave impact force and runup elevation on the bluff. Shoreline recession would be reduced with the construction of an offshore breakwater. The existing revetment and terracing of the upper bluff would provide a reasonable level of protection with the offshore breakwaters in place. As shown in Figure 18, the breakwater would be a rubble mound structure located about 200-feet offshore at about the – 8 feet NGVD contour. Beach fill would be placed from about the MHWL out to the breakwaters to provide additional toe protection to the existing revetment. Approximately 200,000 cubic yards of beach fill would be placed to a berm elevation of +11 feet NGVD. The required renourishment quantity is estimated at 100,000 cubic yards, every 3 years. The sand is assumed to be acquired via a 4,000 cubic yard hopper dredge from Borrow Area IV, seaward of Shinnecock Inlet, as identified in the Fire Island to Montauk Point Reformulation Study. Three separate structures would be built, two being 300-feet in length and one 500-feet in length, with the longest facing the more severe southeasterly direction. The openings between the structures would allow some tidal circulation but also may induce some dangerous currents concentrated in the gaps.

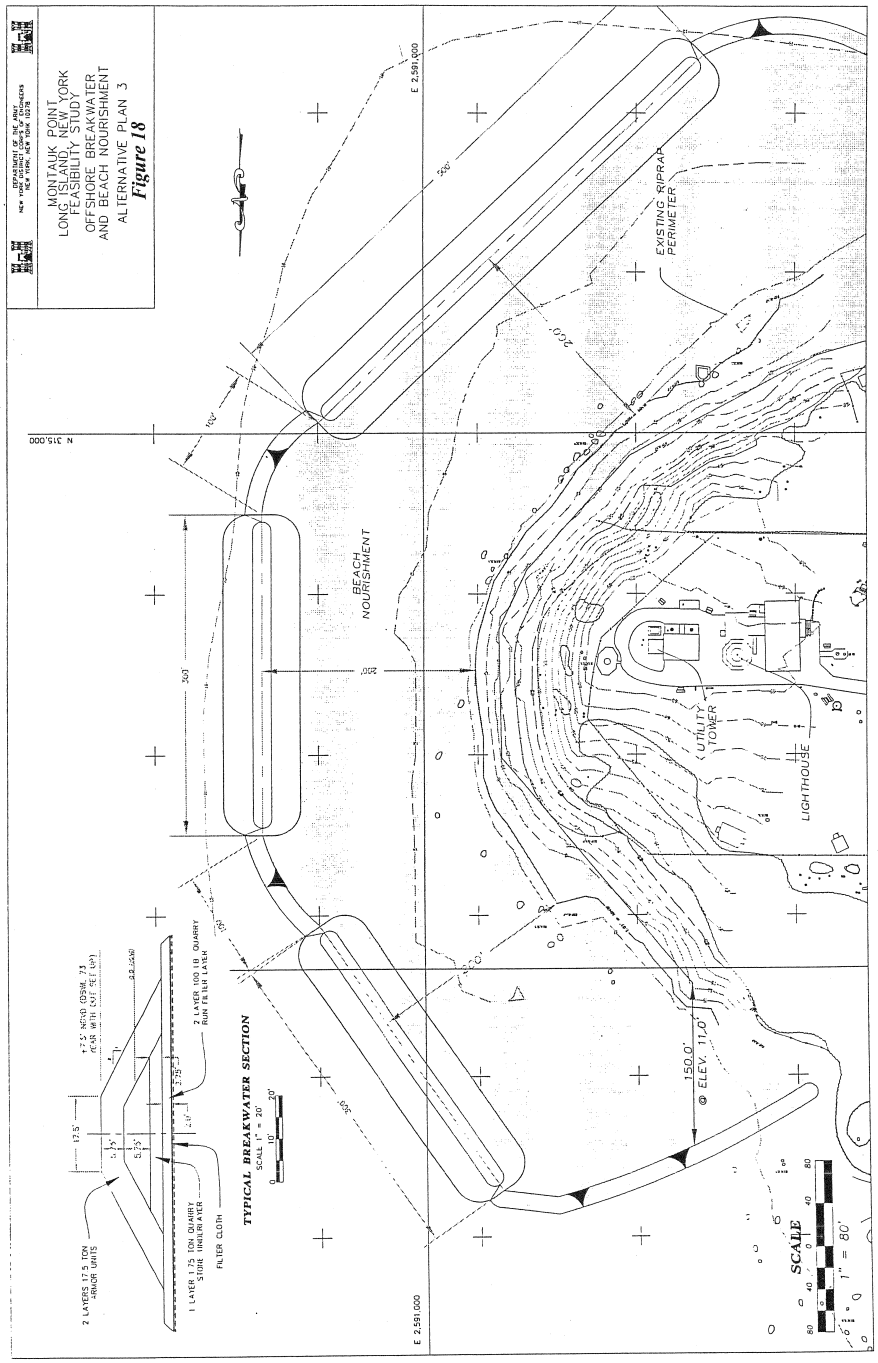
The breakwater design is based on present Corps guidelines. The crest is placed at +7.5 feet NGVD, which is the 73-year water level without wave setup. The armor size is 17.5-tons, placed in two layers on a single layer of 1.75-ton quarrystone underlayer and 2 layers of 100-pound filter stone. The entire structure is built on filter cloth.

The estimated first cost for the offshore breakwater with beach fill is $14,481,000, including 20% contingency, engineering and design, and construction management.

- Breakwaters will be difficult to construct due to difficult site access and in-water construction. Tidal currents are significant and breaking waves arrive from almost all onshore directions. The breakwater requires very large stone and a substantial width and elevation to be effective. The gaps between the breakwaters may induce significant currents that could increase scour to the bottom, potentially compromising the foundation of the breakwaters sometime in the future. Higher surges with waves that submerge the +11 feet berm will not be prevented from damaging the revetment.

- Historic properties will remain protected with the offshore breakwater with beachfill alternative, assuming remedial repairs will be made to the existing revetment, as needed.

- The high currents may cause a safety hazard to swimmers, surfers and fishermen who wade in the area. Surfing activity in the area might be affected by changed reflected wave characteristics.


MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
OFFSHORE BREAKWATER
AND BEACH NOURISHMENT
ALTERNATIVE PLAN 3
Figure 18
BEACH NOURISHMENT
EXISTING RIPRAP PERIMETER
UTILITY TOWER
LIGHTHOUSE
TYPICAL BREAKWATER SECTION
SCALE
1" = 80'

25. Preliminary Alternative #4 - T-Groins with Beach Fill

T-groins, similar to a nearer-to-shore segmented breakwater system with shore-attached groins, are considered as a second breakwater alternative. Similar to the breakwater alternative presented, the purpose of T-groins is to reduce the storm wave height, thus reducing the wave impact force and runup elevation on the bluff. The consistent beach and shoreline recession would be reduced with the construction of T-groins and beach fill. The existing revetment and terracing of the upper bluff would provide a reasonable level of protection with the T-groins in place.

As shown in Figure 19, the T-groin system would be a rubble mound structure located about 100-feet offshore at about the –5 feet NGVD contour. Five separate shore-parallel structures would be built, each being 150-feet in length. A groin would be extended from the center of the shore-parallel breakwater segment to shore, creating individual littoral cells.

Beach fill is placed from shore out to the centerline of the shore-parallel breakwaters to provide erosion protection to the bluff toe to a berm elevation of +11 feet NGVD. Approximately 125,000 cubic yards of beach fill would be placed. The required renourishment quantity is estimated at 100,000 cubic yards every 3 years. The sand is assumed to be trucked in from an upland borrow source. It is expected that embayments in the fill would quickly form as waves and tides re-mold the fill material. The openings between the structures would allow some tidal circulation but also may induce some dangerous currents concentrated in the gaps.

The T-groin design is based on present Corps guidelines. The shore-parallel structure crest is placed at +11 feet NGVD and the groin section crest is placed at +8 feet NGVD. The armor size is 17.5-tons in the shore-parallel structures, placed in two layers on a single layer of 1.75-ton quarrystone underlayer and 2 layers of 100-pound filter stone. The armor size is 4.5-tons in the groins, placed in two layers on 900-pound quarrystone underlayer. The entire structure is built on filter cloth.

The estimated first cost for the T-groins with beach fill is $12,094,000, including 20% contingency, engineering and design, and construction management.

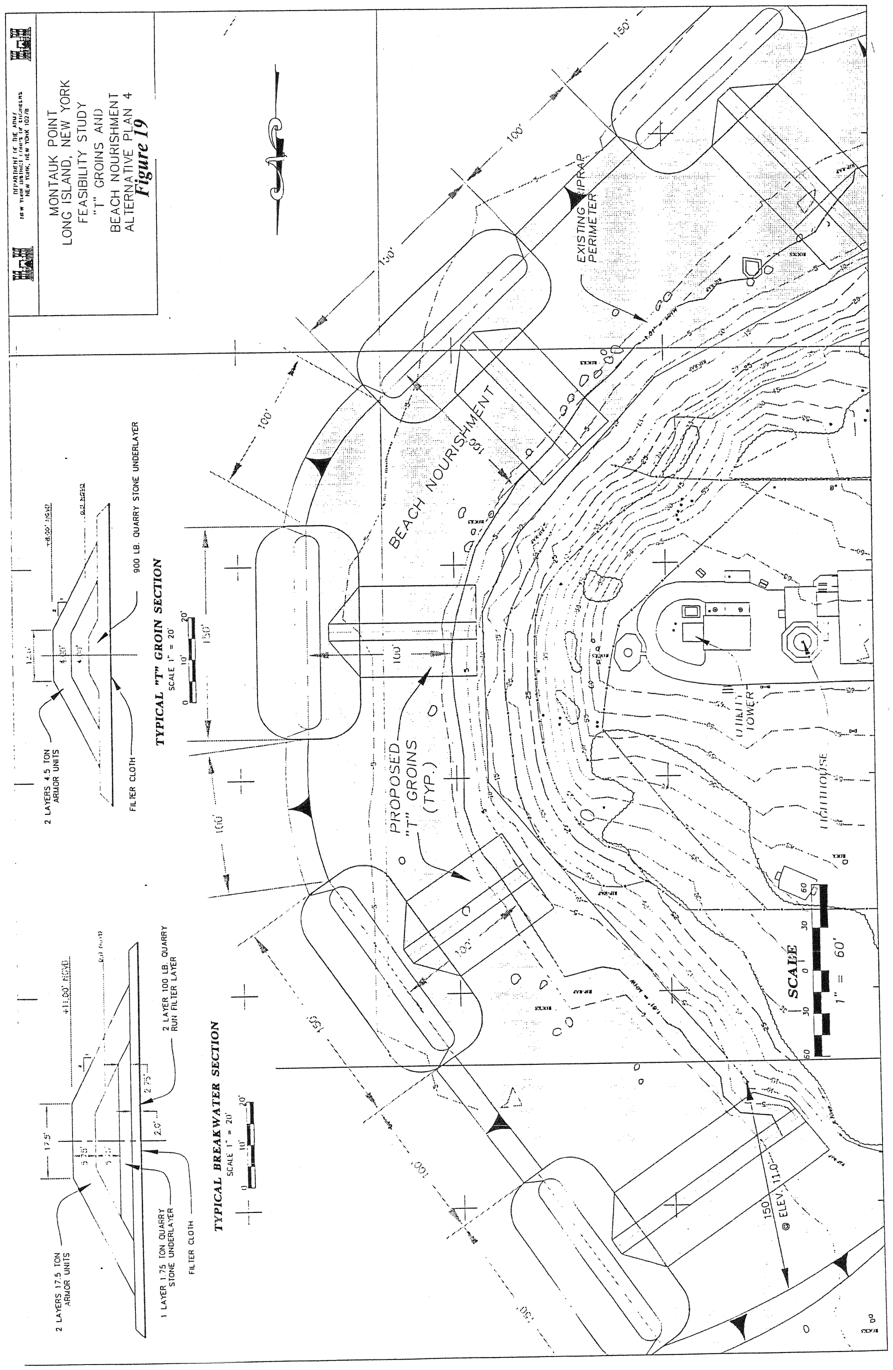
- T-groins will be difficult to construct due to difficult site access, however, land-based equipment can be utilized. Tidal currents are significant and breaking waves arrive from almost all onshore directions. The shore-parallel structures would require very large stone and a substantial width and elevation to be effective. The gaps between the shore-parallel structures may induce significant currents that could scour the bottom, potentially compromising the foundation of the t-groins sometime in the future.

- In this option, the protective beach fill will require renourishment at a rate that is difficult to predict until it is constructed and monitored. Higher surges with waves that submerge the +11 feet berm will not be prevented from damaging the revetment.

- The placement of T-groins along the face of the bluff of Montauk Point will have a minimal negative impact on the buried and above ground historic properties present. The addition of groins along the current wall will, in fact, protect the resources and allow them to remain in place, allowing for their future study.

- The high currents may cause a safety hazard to swimmers, surfers and fishermen who wade in the area. Surfing activity in the area may be affected by changed reflected wave characteristics.

- Impacts stemming from periodic removal of fill at the borrow site could occur. There would probably be seasonal constraints due to essential fish habitat concerns.


MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
"T" GROINS AND
BEACH NOURISHMENT
ALTERNATIVE PLAN 4
Figure 19
TYPICAL "T" GROIN SECTION
SCALE 1" = 20'
2 LAYERS 4.5 TON ARMOR UNITS
FILTER CLOTH
900 LB. QUARRY STONE UNDERLAYER
TYPICAL BREAKWATER SECTION
SCALE 1" = 20'
2 LAYERS 17.5 TON ARMOR UNITS
1 LAYER 1.75 TON QUARRY STONE UNDERLAYER
FILTER CLOTH
2 LAYER 100 LB. QUARRY RUN FILTER LAYER
BEACH NOURISHMENT
PROPOSED
"T" GROINS
(TYP.)
EXISTING
PERIMETER
UTILITY
TOWER
LIGHTHOUSE
SCALE
1" = 60'

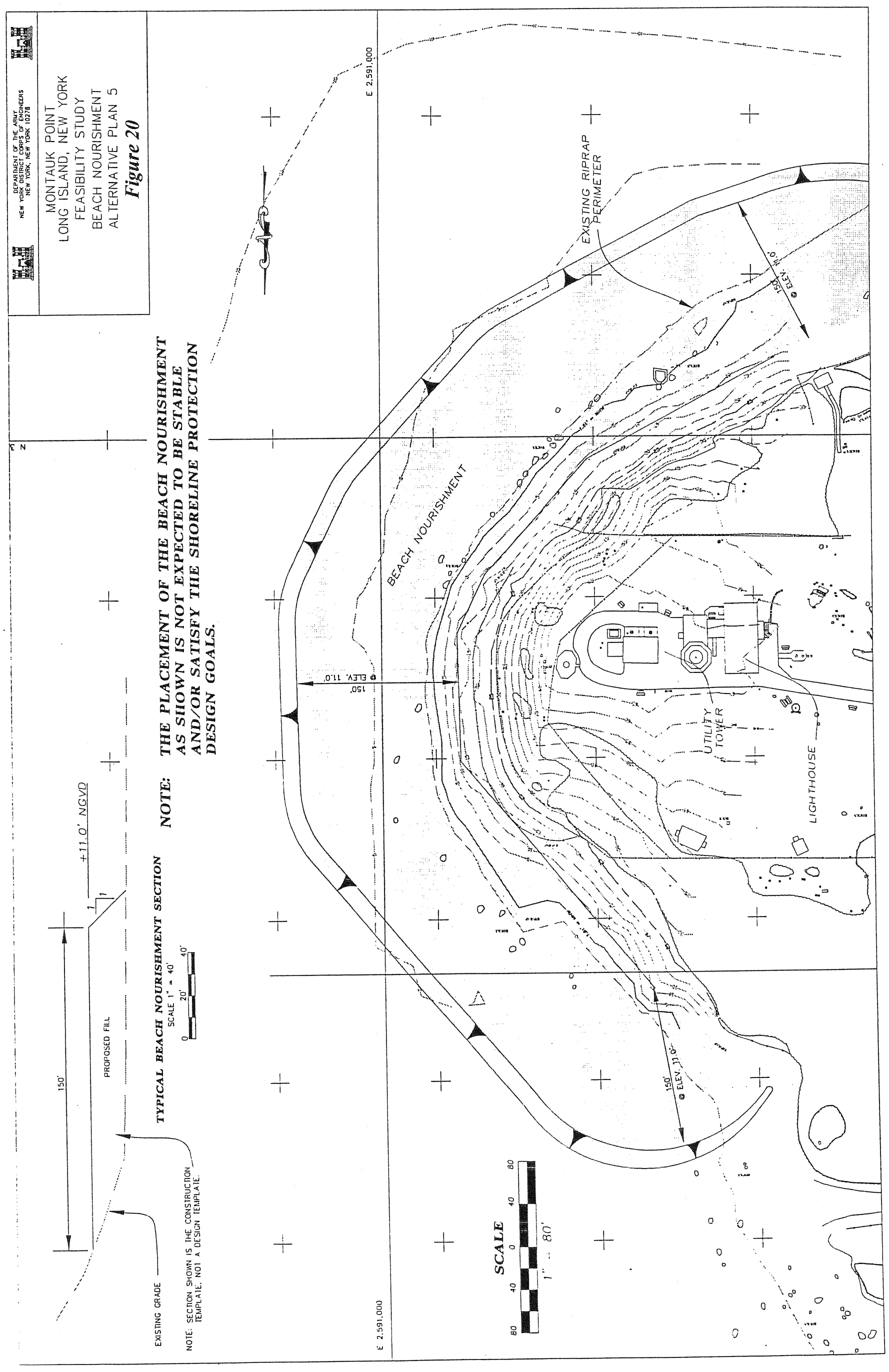
26. Preliminary Alternative #5 - Beach Fill

Beach fill or nourishment without containment structures is illustrated in Figure 20. For this design, a construction berm with an elevation of +11 feet NGVD and 150-feet in width is created. Approximately 200,000 cubic yards of beach fill would be placed. The sand is assumed to be acquired via a 4,000 cubic yard hopper dredge from Borrow Area IV, seaward of Shinnecock Inlet, as identified in the Fire Island to Montauk Point Reformulation Study.

- This alternative is not considered feasible for many reasons. High longshore transport rates will remove the fill rapidly at an unpredictable rate and the area will require constant renourishment. A berm at +11 feet NGVD will provide some short-term reduction in the recession of the toe of the bluff, but will not impede higher water levels and waves from impacting the bluff face and therefore will not provide adequate storm damage protection. Seasonal beach surveys (potentially monthly) will be required during the first two to three years after construction to refine the design of the beach fill cross section and to estimate the renourishment requirements. Because of the lack of adequate storm damage protection, this beach fill alternative will not be considered further.

- It expected that a beach nourishment project will change surfing conditions in the area by reducing wave reflection characteristics from the existing stone structures and by filling out the offshore beach profile to a more gradual slope.

- Impacts stemming from periodic removal of fill at the borrow site could occur. There would probably be seasonal constraints due to essential fish habitat concerns. Recreational fishing at the placement site might also be affected.


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
BEACH NOURISHMENT
ALTERNATIVE PLAN 5
Figure 20
NOTE: THE PLACEMENT OF THE BEACH NOURISHMENT AS SHOWN IS NOT EXPECTED TO BE STABLE AND/OR SATISFY THE SHORELINE PROTECTION DESIGN GOALS.
TYPICAL BEACH NOURISHMENT SECTION
SCALE 1" = 40'
+11.0' NGVD
150'
PROPOSED FILL
EXISTING GRADE
NOTE: SECTION SHOWN IS THE CONSTRUCTION TEMPLATE, NOT A DESIGN TEMPLATE.
EXISTING RIPRAP PERIMETER
BEACH NOURISHMENT
ELEV. 11.0'
150'
UTILITY TOWER
LIGHTHOUSE
E 2,591,000
SCALE
1" = 80'

27. Preliminary Alternative #6 - Relocation of the Lighthouse

Moving the Montauk Point Light Station, a National Register listed property, would preserve the existing structures, but allow for the eventual destruction of the bluff. Prior to the relocation of the existing buildings, the arrangement and relationships of the structures on the landscape as well as the view to and from the lighthouse and bluff would be documented. In addition, subsurface archeological investigations would be required at the current site as well as at the new lighthouse location.

The preliminary estimated cost for moving the Montauk Point Lighthouse and undertaking the required archeological investigations would be approximately $20 million. In addition, the required creation of raised grades landward of the present location of the lighthouse would add an additional cost of $7 million and reduce parking facilities. The overall project would take approximately six years to complete, with a total cost of approximately $27 million.

- The moving of the lighthouse itself is a precarious task at best. Unlike the Cape Hatteras Lighthouse (which rested on a relatively flat, level surface that permitted the National Park Service to move the structure for a cost of approximately $12 Million) the Montauk Point Lighthouse rests upon a hill on top of the bluff. Raised grades would have to be built to raise the level of the ground to the west of the bluff up to the lighthouse grade to insure a stable move.

- The relocation of the Montauk Point Lighthouse will have an adverse effect on the above and below ground resources. Moving the Lighthouse would have an adverse impact on the archaeological resources and compromise the integrity of the lighthouse and associated structures.

- Environmental degradation of habitats and historic views would continue. Relocating the lighthouse could lead to an eventual change in the recreational use potential of the area.

The moving of the Lighthouse was given considerable weight during the Feasibility phase of the project. However, several factors contributed to the decision not to make this proposal the preferred alternative. They included: a) the overall cost of the alternative b) the engineering requirements of having to build up land to meet the hill of Montauk Point to create a level moving surface, c) the destruction of a National Register Landmarked complex by moving it, the setting is destroyed thus violating the spirit of the National Historic Preservation Act of 1966, as amended, d) the loss of value to the Town of Easthampton, Montauk Point and Montauk Point State Parks, as several hundred thousand visitors come to this area each year, in part to see “the end”, i.e. Montauk Point Lighthouse, e) the New York State Office of Parks, Recreation and Historic Preservation (see Letter Number 01), the Regulatory Agency that would have to approve any move of a National Register structure has already stated, and has done so throughout the entire process, that they would not approve the moving of the Lighthouse, which would lead to the destruction of the Lighthouse complex area.

Additionally, while the Montauk Historical Society maintains the lighthouse complex, the U.S. Coast guard still operates the beacon and the foghorn as working aids-to-navigation. If the lighthouse were not present, the U.S. Coast Guard would likely erect a tower of which to mount a replacement beacon. As per the agreement signed during the transfer of the property from the Federal Government to the Montauk Historical Society, If the Montauk Historical Society fails to protect or maintain the lighthouse, the property would revert back to the USCG.

28. Selected Preliminary Alternative – Stone Revetment

Based on the advantages and disadvantages of each of the alternatives discussed, including an evaluation of environmental quality, other social effects, regional economic development, and national economic development (see Table 7), as well as the estimated costs of construction and periodic nourishment required with the potential alternatives (see Table 7A), and comparison of net benefits (see Table 7B), the selected plan for protection of Montauk Point and the lighthouse complex and bluff is the stone revetment.

Table 7 – Plan Evaluation Matrix

	Environmental Quality	Other Social Effects	Regional Economic Development	National Economic Development
Alternative 1 - No Action Plan	-	-	-	-
Alternative 2 - Stone Revetment	0	0	+	+
Alternative 3 - Offshore breakwater with beach fill	-	-	+	-
Alternative 4 - T Groins with Beach Fill	-	-	+	-
Alternative 5 - Beach fill only	-	-	+	-
Alternative 6 - Relocation of Lighhouse	-	0	0	-
+ Indicates a net positive influence or effect				
0 Indicates no positive or negative effect				
- Indicates a net negative influence or effect				
The alternative plans have been evaluated based upon four accounts to facilitate plan selection.				
Based upon these evaluations the revetment alternative is the selected NED plan.				
The Environmental Quality account displays non-monetary effects on significant cultural and natural resources.				
The Other Social Effects account registers plan effects relevant to planning process but not captured in other three accounts.				
The Regional Economic Development account regsiters changes in regional economic activity.				
The National Economic Development account displays changes in economic value of national output of goods and services.				

Table 7A - Preliminary Alternatives Construction Cost Estimates

October 2004 Price Levels

FIRST COST AND ANNUAL COST SUMMARY – Selection of alternative

	Alternative #2 Stone Revetment	Alternative #3 Offshore Breakwater and Beach Fill	Alternative #4 T-Groins and Beach Fill
Total First Cost	**$ 14,843,000**	$ 14,481,000	$ 12,094,000
Interest During Construction @ 5.375%	**$ 949,000**	$ 752,000	$ 629,000
Total Investment Cost	**$ 15,792,000**	$ 15,233,000	$ 12,723,000

	Alternative #2 Stone Revetment	Alternative #3 Offshore Breakwater and Beach Fill	Alternative #4 T-Groins and Beach Fill
Annualized Total Investment Cost Based on 50-year design life Annual interest of 5.375%	**$ 916,000**	$ 884,000	$ 738,000
Annualized Maintenance Cost	**$ 55,000**	$ 57,000	$ 47,000
Annualized Periodic Nourishment Cost Based on 50-year design life Annual interest of 5.375% 100,000 cy nourishment every 3 years	**Zero Cost $0**	$ 502,000	$ 502,000
Total Annual Cost	**$ 971,000**	$ 1,443,000	$ 1,287,000

Table 7B - Screening of Preliminary Alternatives

	Alternative #2 Stone Revetment	Alternative #3 Offshore Breakwater and Beach Fill	Alternative #4 T-Groins and Beach Fill
Total Annual Cost	**$971,000**	$1,443,000	$1,287,000
Total Annual Benefits *	$1,578,700	$1,578,700	$1,578,700
Total Net Benefits	**$607,700**	$135,700	$291,700

** Alternatives # 2 thru # 4 are developed at the same storm design, as they are each designed to withstand a 73-year return period storm. The benefits claimed are the same because each of the alternatives will protect the same land to the same degree, and each alternative avoids the same average annual project damages.*

Of the potential alternatives discussed above, the stone revetment alternative is the plan that maximizes net benefits. Revetments are a proven method of shore protection in this area and have a record of acceptance by state and local agencies. By re-using some of the stone already on site in the existing structure, cost savings will be realized. Preliminary design variations in the revetment cross-section were considered to evaluate the impacts on construction costs. The cross-section of the preliminary revetment alternative consists of the construction of a revetment section with a crest width of 40-feet at elevation +25 feet NGVD, 1V:2H side slopes, and 12.6-ton quarrystone armor units extending from the crest down to the embedded toe. A heavily embedded toe is incorporated to protect against breaking waves and scour at the toe of the structure.

The embedded toe was designed in accordance with EM 1110-2-1614 entitled "Design of Coastal Revetments, Seawalls and Bulkheads" (1995). Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure. Sublayers are specified in accordance with standard design procedures.

The estimated first cost for the selected preliminary alternative, the stone revetment, is $14,843,000.

The comparison of feasible solutions results in the selection of a revetment as the best approach to protecting the bluff and lighthouse. Three alternative revetments, based upon three different levels of protection, have been analyzed during the Feasibility Study to determine the most economical revetment design.

29. Design Optimization of the Stone Revetment Alternative

Design variations in the selected preliminary alternative, i.e. the stone revetment, were considered to economically optimize the construction cost relative to the economic benefits (provide the greatest net economic benefits). The design will provide long-term storm damage protection for the economic life of 50-years and will comply with all design criteria and constraints. Three (3) alternatives were considered for optimization.

Embedded toe design will be in accordance with EM 1110-2-1614 entitled "Design of Coastal Revetments, Seawalls and Bulkheads (1995). Sublayers are specified in accordance with standard design procedures.

Final Improvement Designs – Stone Revetment – 3 Alternatives

For the three alternative revetment sizes developed as part of the optimization, the higher two levels of protection have a heavily embedded toe to protect against breaking waves and scour at the base of the structure. Sublayers are specified in accordance with standard design procedures. It is noted that because the revetment improvement is founded on dense till or stone, no filter cloth is required to underlie the improvement.

This is a design refinement from the preliminary design where filter cloth was included. In addition, the following refinements to the preliminary revetment alternative were made:

1) Quantities changed slightly based on additional cross sections taken.

2) Mobilization and demobilization costs increased to include temporary construction berms at each end of the revetment to facilitate revetment construction.

3) Contingencies were slightly reduced due to the more detailed level of design.

These design refinements do not affect plan formulation (comparison of alternatives) and selection of the stone revetment alternative.

The following sections describe the three variations of the revetment alternative used in order to optimize the design.

30. Alternative 2A: Stone Revetment with 150-year Storm Design

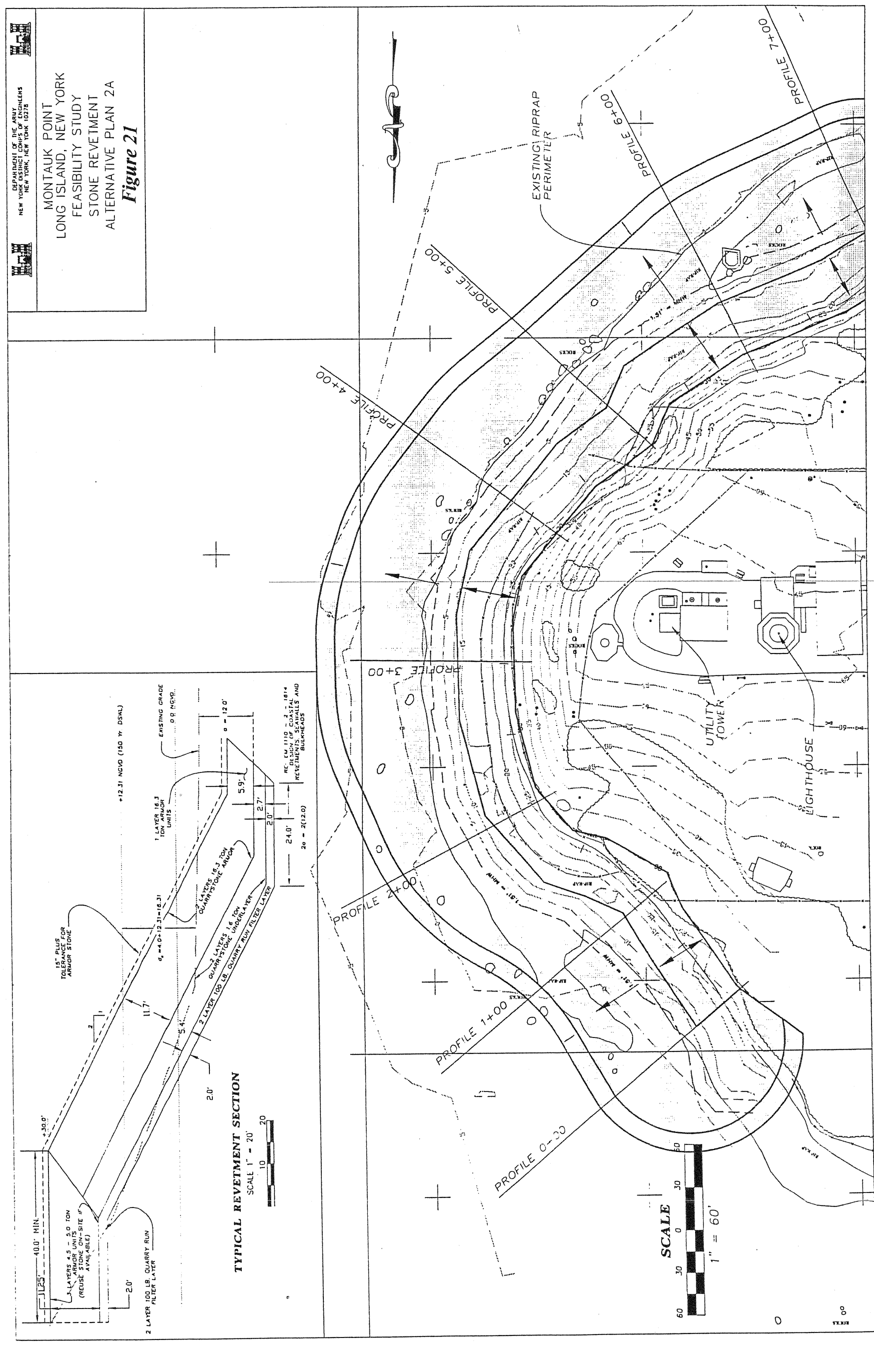
- The design wave for the structure is $H_{150\ Yr.}$ = 14.6-feet based on the average toe elevation near the improved revetment toe of elevation –4 feet NGVD.

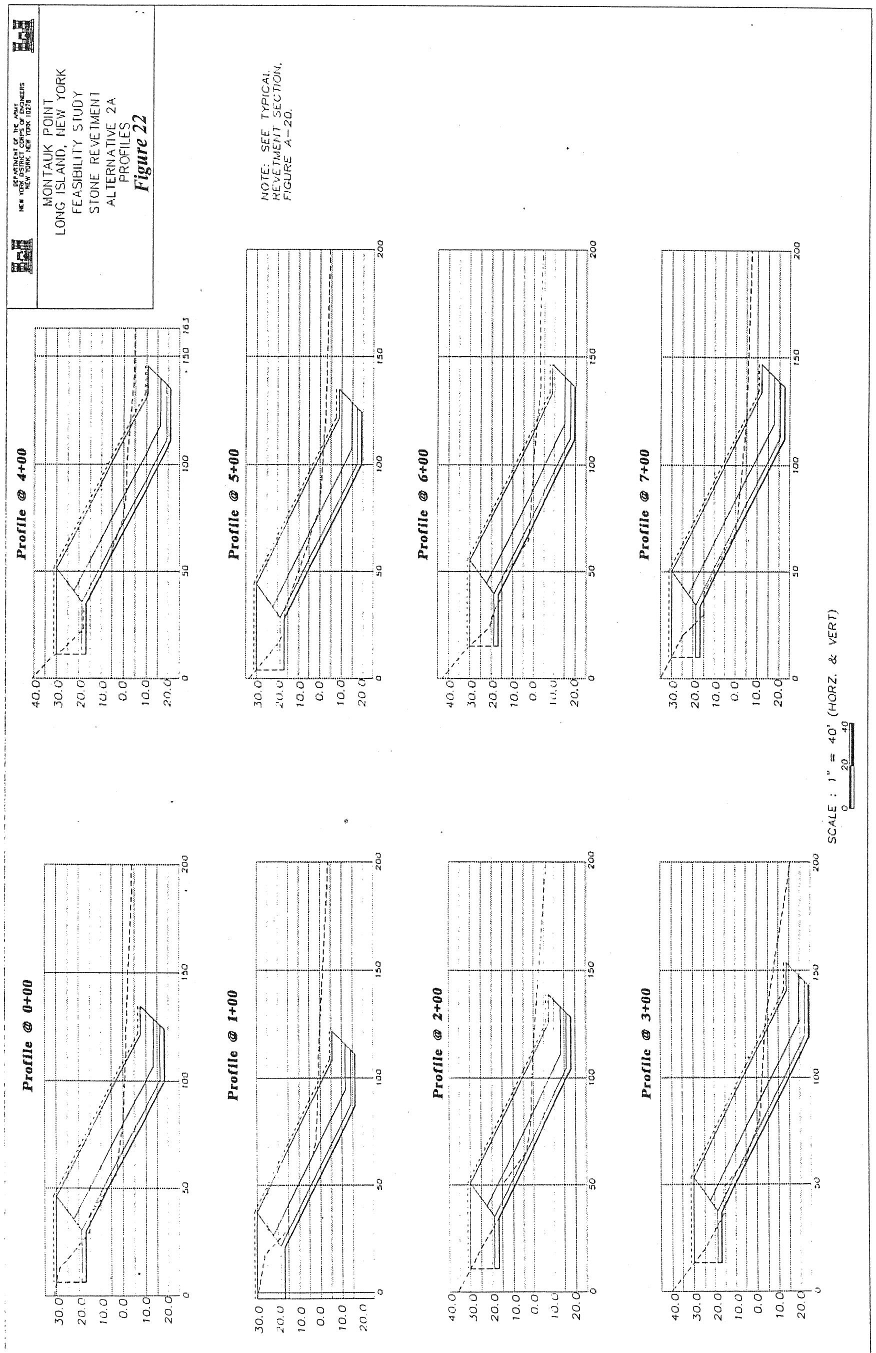
- The cross-section of the revetment shown in Figure 21 consists of the construction of a revetment section with a crest width of 40-feet at elevation +30 feet NGVD, 1V:2H side slopes, and 16.3-ton quarrystone armor units extending from the crest down to the embedded toe.

- According to Engineering Manual guidance, the bottom of the armor stone layer in the toe is located 12 feet below existing grade at the toe (the stone crest at approximately –10 feet with the average toe elevation at –4 feet NGVD).

- Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure.

- Cross-sections of this revetment alternative along the existing profiles are shown in Figure 22.


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2A
Figure 21
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
EXISTING RIPRAP PERIMETER
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
UTILITY TOWER
LIGHTHOUSE
SCALE
1" = 60'


DEPARTMENT OF THE ARMY NEW YORK DISTRICT CORPS OF ENGINEERS NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2A
PROFILES
Figure 22
NOTE: SEE TYPICAL REVETMENT SECTION, FIGURE A-20.
Profile @ 0+00
Profile @ 1+00
Profile @ 2+00
Profile @ 3+00
Profile @ 4+00
Profile @ 5+00
Profile @ 6+00
Profile @ 7+00
SCALE : 1" = 40' (HORZ. & VERT)

31. Alternative 2B: Stone Revetment with 73-year Storm Design

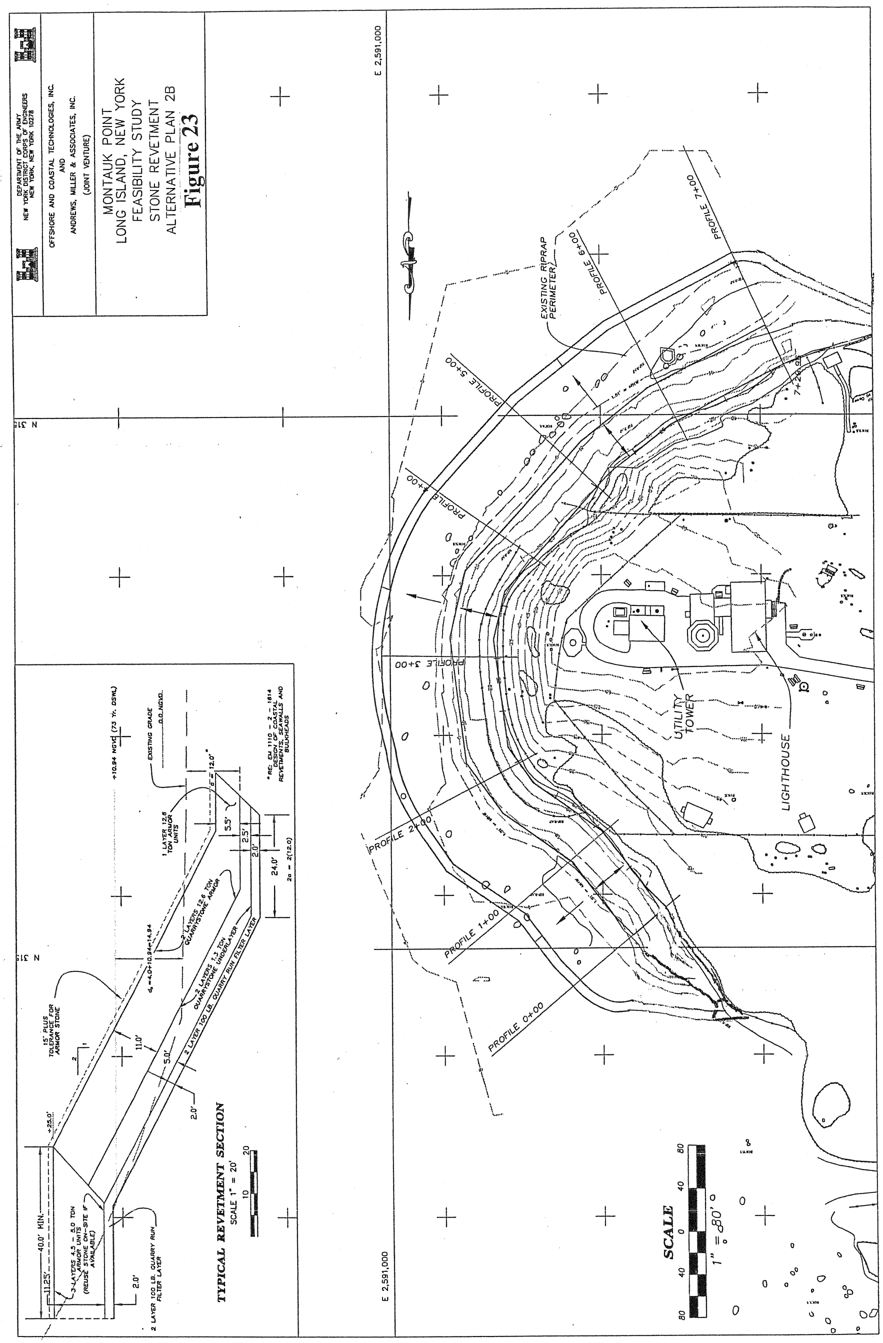
- The design wave for the structure is $H_{73\ Yr.}$ = 13.4 feet based on the average toe elevation near the improved revetment toe of elevation –4 feet NGVD.

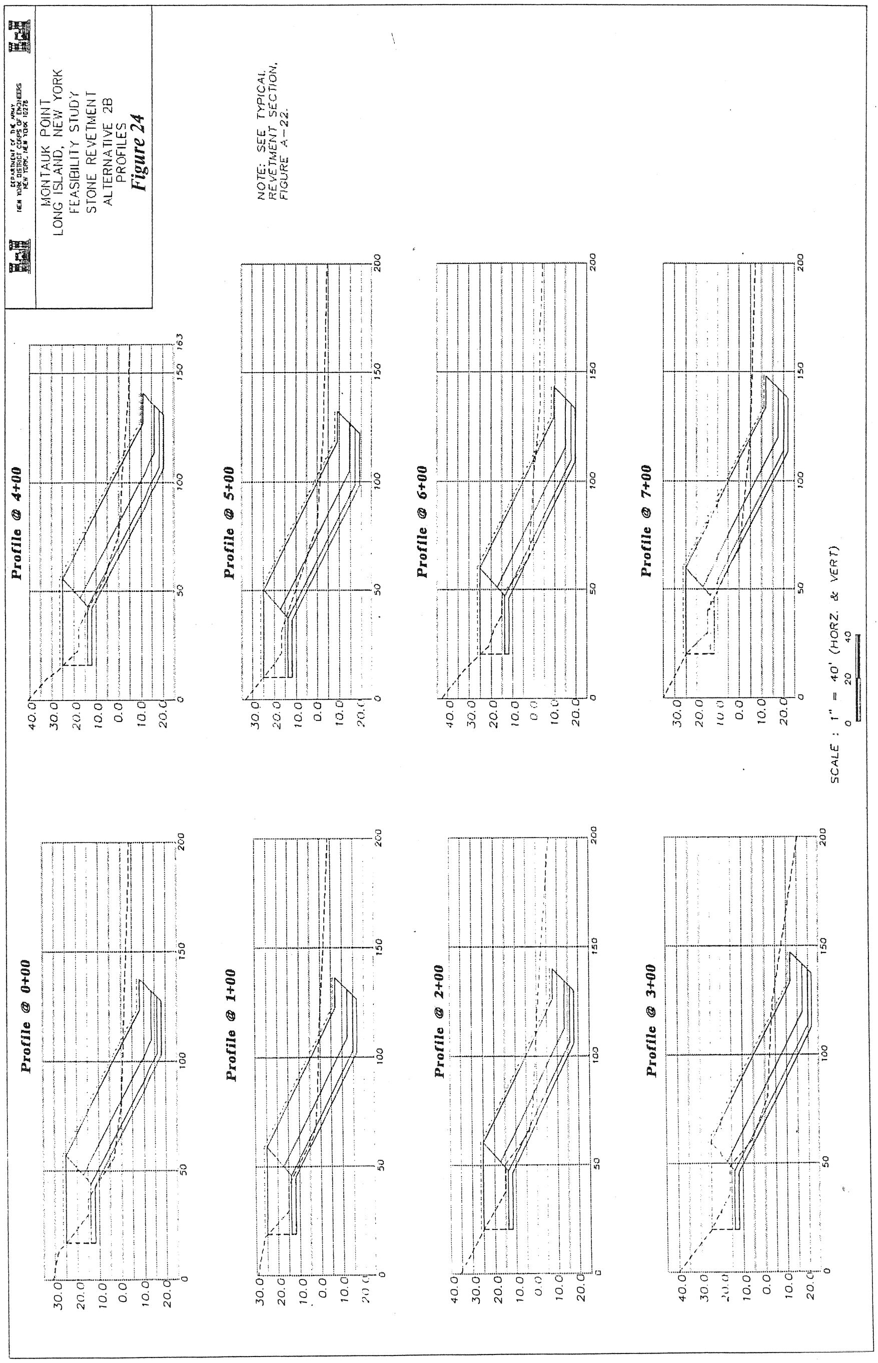
- The cross-section of the revetment shown in Figure 23 consists of the construction of a revetment section with a crest width of 40 feet at elevation +25 feet NGVD, 1V:2H side slopes, and 12.6-ton quarrystone armor units extending from the crest down to the embedded toe.

- The bottom of the armor stone layer in the toe is located 12 feet below existing grade at the toe (average toe elevation at –4 feet NGVD).

- Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure.

- Cross-sections of this revetment alternative along the existing profiles are shown in Figure 24.


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
OFFSHORE AND COASTAL TECHNOLOGIES, INC.
AND
ANDREWS, MILLER & ASSOCIATES, INC.
(JOINT VENTURE)
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2B
Figure 23
EXISTING RIPRAP PERIMETER
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
UTILITY TOWER
LIGHTHOUSE
E 2,591,000
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
SCALE
1" = 80'


DEPARTMENT OF THE ARMY NEW YORK DISTRICT CORPS OF ENGINEERS NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2B
PROFILES
Figure 24
NOTE: SEE TYPICAL REVETMENT SECTION, FIGURE A-22.
Profile @ 0+00
Profile @ 1+00
Profile @ 2+00
Profile @ 3+00
Profile @ 4+00
Profile @ 5+00
Profile @ 6+00
Profile @ 7+00
SCALE : 1" = 40' (HORZ. & VERT)

32. Alternative 2C: Stone Revetment with 15-year Storm Design

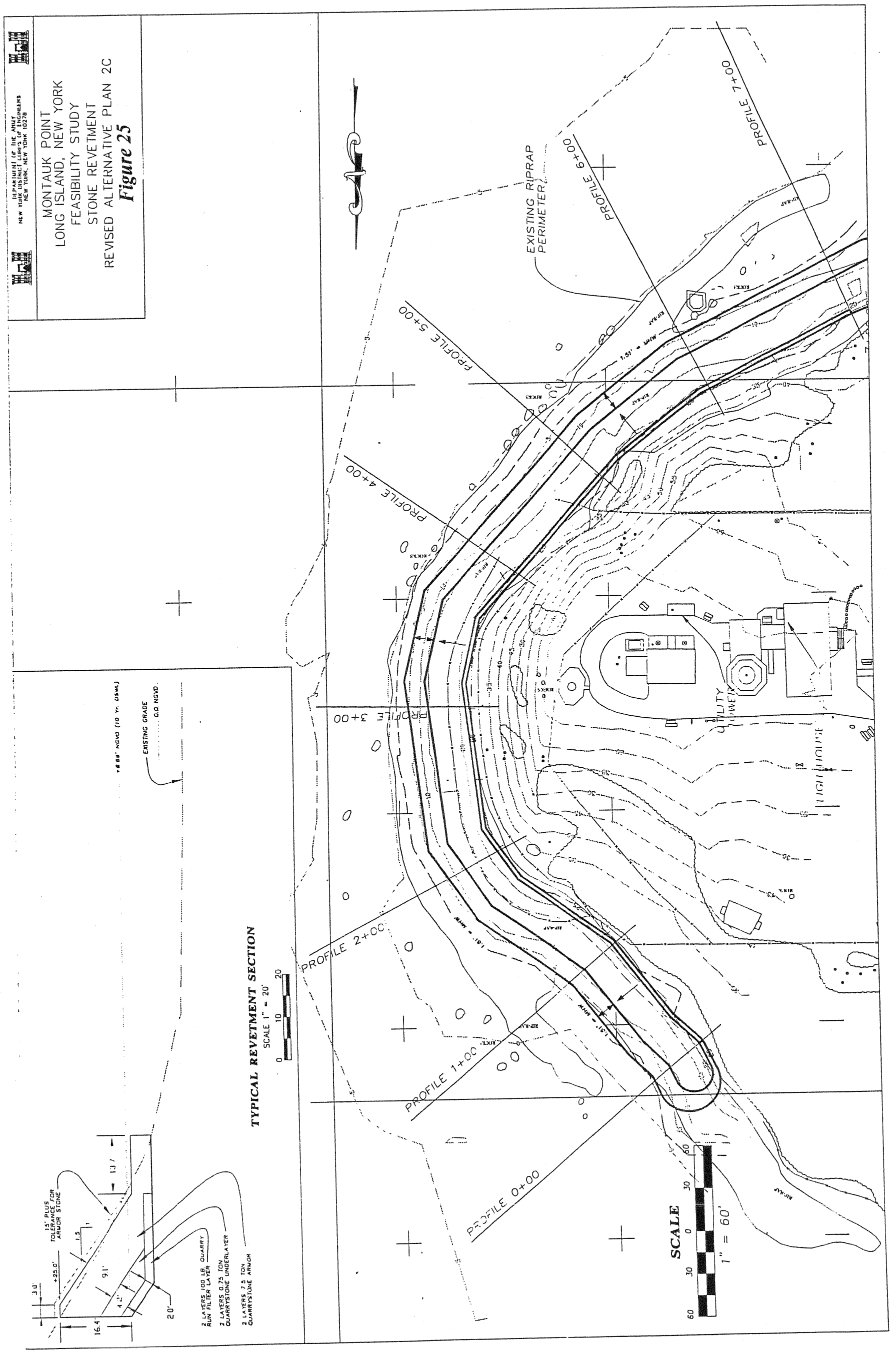
- The design wave for the structure is $H_{15\ Yr.}$ = 9.2 feet based on the average toe elevation near the improved toe of elevation –1 feet NGVD.

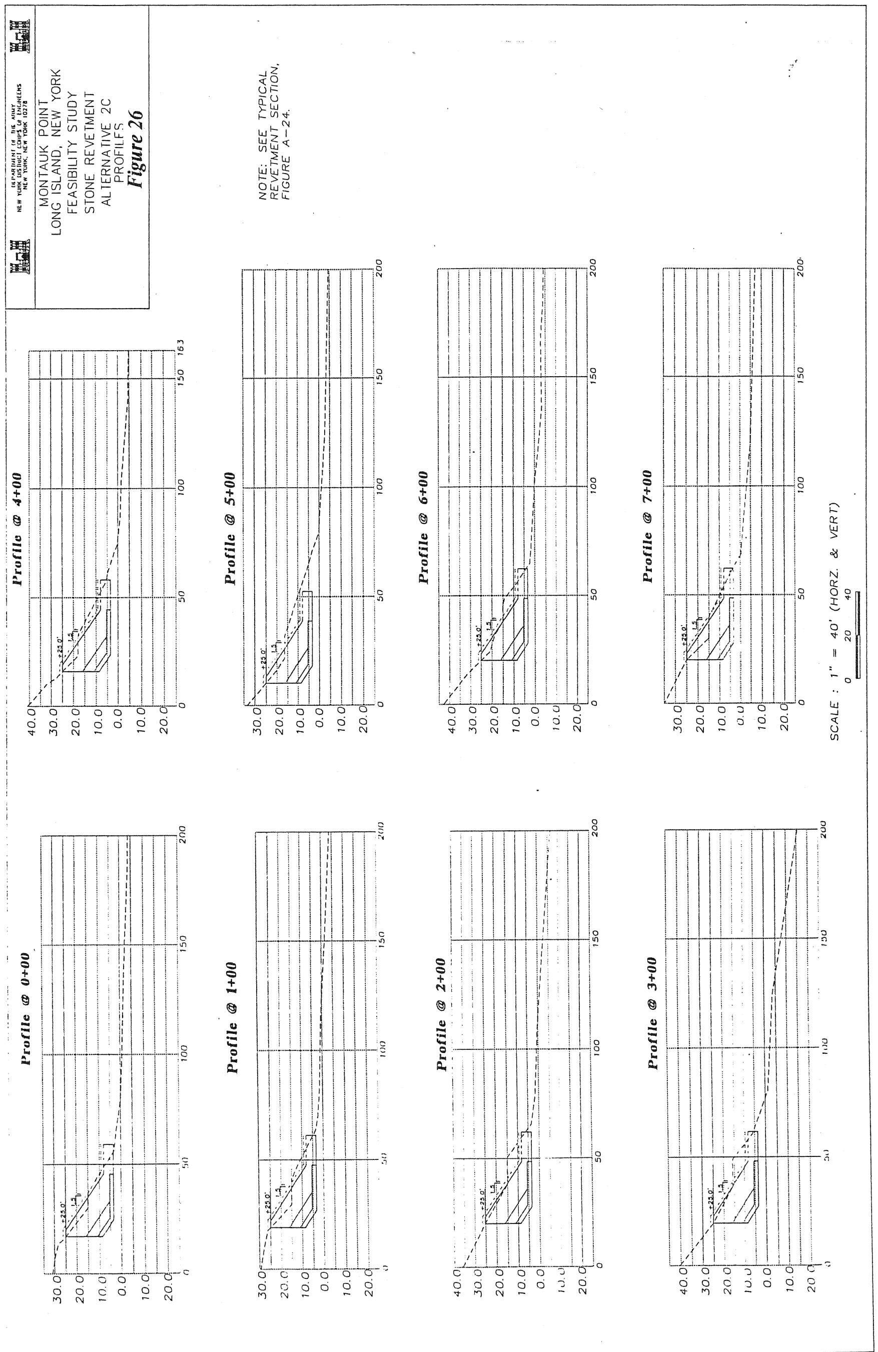
- The cross-section of the revetment shown in Figure 25 consists of a revetment section with a crest width of 3 feet at elevation +25 feet NGVD, 1V:1.5H side slopes, and 7.5-ton quarrystone armor units extending from the crest down to the toe.

- The toe will be built up from the existing toe with large stone and will not require an excavated buried toe. It is assumed that some stones can be re-used in the proposed revetment from the present structure.

- Cross-sections of this revetment alternative along the existing profiles are shown in Figure 26.

Since the costly buried toe is not essential for the 15-year storm design, a narrow berm was developed to provide better foundation on the existing toe stone. In order to construct the narrow berm, an offshore adjacent rubble mound stone temporary structure will be required from which land-based construction equipment will operate.


MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
REVISED ALTERNATIVE PLAN 2C
Figure 25
TYPICAL REVETMENT SECTION
EXISTING GRADE
EXISTING RIPRAP PERIMETER
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
UTILITY
LIGHTHOUSE
SCALE
1" = 60'


MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2C
PROFILES
Figure 26
NOTE: SEE TYPICAL REVETMENT SECTION, FIGURE A-24.
Profile @ 0+00
Profile @ 1+00
Profile @ 2+00
Profile @ 3+00
Profile @ 4+00
Profile @ 5+00
Profile @ 6+00
Profile @ 7+00
SCALE : 1" = 40' (HORZ. & VERT)

33. Coastal Analyses of the 3 Stone Revetment Alternatives

Wave Runup

Wave runup level, as input to overtopping, determined the design crest level of the structure. Table 8 presents the runup elevations for the revetment alternatives and the presently existing structure.

Table 8 - Runup Elevations for Existing Revetment and Improvement Plans

RETURN PERIOD	RUNUP ELEVATION (PERMEABLE) FEET, NGVD			
Years	Existing	Plan 2A 150-year	Plan 2B 73-year	Plan 2C 15-year
2	22.20	33.18	33.18	23.78
5	23.42	36.73	36.73	27.21
10	23.35	38.20	38.20	28.49
25	23.69	40.19	37.85	30.22
44	24.46	42.13	37.70	31.97
50	24.49	42.31	37.62	32.10
73	25.39	44.05	37.85	33.39
100	26.43	44.38	38.32	34.75
150	27.98	44.51	39.21	36.69
200	29.16	44.78	40.00	38.12
500	32.02	45.91	42.30	41.52

Runup is developed from representative composite slopes of the structures template. Therefore, these hypothetical values represent a smooth composite slope. The presence of the berm at a lower elevation (as with the existing revetment) and steeper composite slope (from a relatively narrow berm and shallow toe depth) reduces the runup, but not the overtopping rate above the berm crest, due to the large berm crest width of the improvement. The effect of a steeper slope and shallower structure toe cause the runup elevations associated with Plan 2C to be lower than those for the other plans.

The calculations indicate that runup elevations exceed the existing revetment crest (+18 feet NGVD) at all listed return periods using maximum design wave conditions. Field observations confirm that 'green water' frequently reaches the top of the revetment.

Figure 27 also confirms that wave runup (from the highest segment of the wave group for the more frequent storms, all the way to nearly all the waves on the 73-year return period storm) exceeds the crest elevation of the existing structure, from the 2-year through the 500-year storm, even when permeability is accounted for.

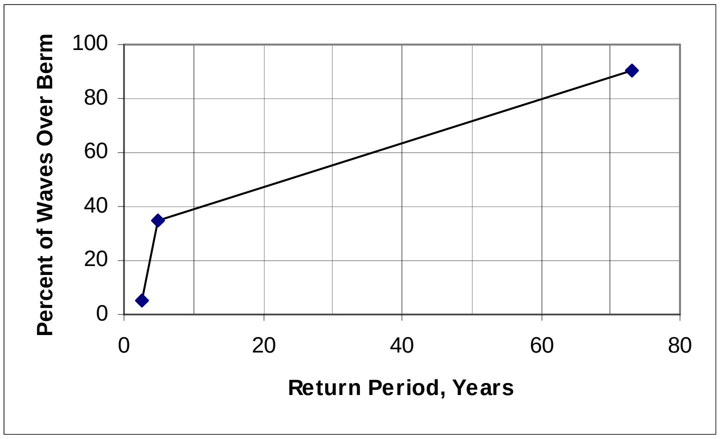


Figure 27- Percent of wave runup exceeding existing berm elevation

Wave Overtopping

Wave overtopping occurs when the structure crest height is lower than the runup level. Overtopping discharge is a very important design parameter because it determines the crest level and the design of the upper part of the structure. In the Montauk Point case, overtopping must be limited at design storm levels so as to avoid failure of the revetment from the top (as observed in the field and in the model test for the existing structure). The relevant critical levels (based on Coastal Engineering Manual criteria from physical modeling of damages sustained with paved and unpaved revetments) at Montauk Point is 100 litres/s/m (0.1 cu m/s/m). This is a critical threshold for damage of vegetative terracing immediately above the revetment stone; however, lower levels of damage can be initiated at the 50 litres/s/m threshold.

Table 9 - Overtopping Rates for Existing Revetment and Improvement Plans

RETURN PERIOD	OVERTOPPING RATES (LITRES/S/M)			
Years	Existing	Plan 2A	Plan 2B	Plan 2C
2	17	1	2	12
5	41	1	4	24
10	90	3	8	48
25	266	6	18	120
44	589	10	33	227
50	728	11	37	274
73	1430	18	60	460
100	2903	27	96	780
150	7479	47	175	1517
200	16221	70	280	2560
500	130783	223	1060	8073

The results show that the critical level for significant damage initiation of the vegetative terracing is exceeded above a 200-year event for Plan 2A, a 100-year event for Plan 2B and greater than a 15-year event for Plan 2C. The existing structure exhibits damaging overtopping rates during events greater than a 10-year level.

Wave Reflection

Wave reflection affects the nearshore wave conditions immediately fronting the structure, and potentially along neighboring beaches. Incident energy is partly dissipated by wave breaking, surface roughness and porous flow through the stone structure.

Table 10 presents a comparison of reflection coefficients for the three Improvement Plans and the existing structure. This indicates that the reflected wave will be reduced at all return periods for all three final improvement alternative plans versus the existing structure because of the flatter structure slopes and more porous rock layering from larger stone sizes. The reductions range from 13-19% for Plans 2A and 2B and 3-5% for Plan 2C.

Table 10 - Reflection Coefficients for Existing Revetment and Improvement Plans

RETURN PERIOD	REFLECTION COEFFICIENT		
Years	**Existing Structure**	**Plans 2A and 2B**	**Plan 2C**
2	0.57	0.46	0.54
5	0.57	0.45	0.54
10	0.57	0.47	0.55
25	0.58	0.48	0.56
44	0.59	0.49	0.56
50	0.59	0.50	0.56
73	0.59	0.50	0.57
100	0.59	0.50	0.57
150	0.59	0.51	0.57
200	0.59	0.51	0.57
500	0.60	0.52	0.58

Wave Scour

Wave scour occurs at the toe of the structure due to the concentration of currents formed by the interaction of incident waves with the down rush from preceding waves. The extensive scour protection toe design included in the Final Improvement Alternative Plans 2A and 2B will prevent adverse scour (including both storm and long term).

Adjacent Impacts

Potential longshore effects include the impact of the new structures on neighboring beaches. Because a revetment has been in place at Montauk Point for nearly 60 years, the sediment that would have become littoral supply adjacent beaches has been stabilized at the Point. The replacement of the existing structure with a new design would not alter that function. The seaward translation of the new structures (Plans 2A and 2B) results in the need for a transition revetment section (which has been included in the project costs) to prevent local erosion at the ends of the project where both wave diffraction and longshore sediment demand will tend to increase erosion at those areas under the improved condition.

Slope Stability Analysis of Improvement Plans

Slope stability analysis was performed on Alternative 2B to evaluate “with project conditions”. Figure 28 shows that the factor of safety for the critical failure surface is 1.46 through the revetment and 1.202 in the bluff above the revetment.

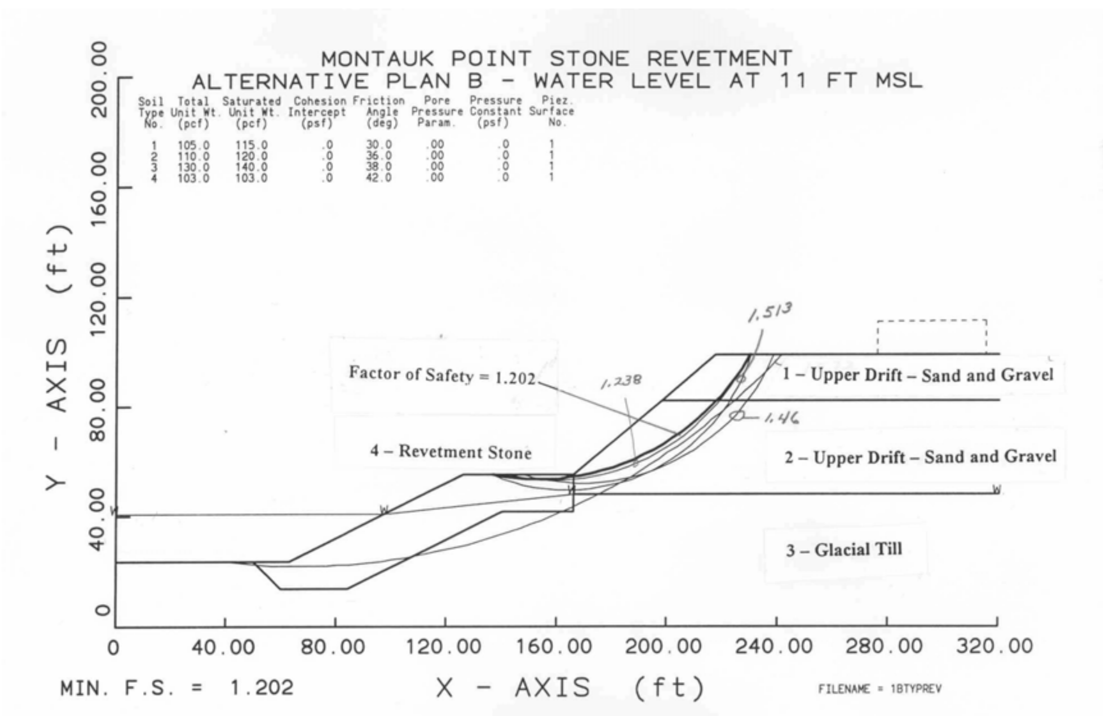


Figure 28 – Slope Stability Analysis for Stone Revetment Alternative 2B

This alternative was then examined for toe of slope saturation due to wave runup for a 100-year return period. The factor of safety for the critical failure surface through the revetment remained the same. The factor of safety for the critical failure surface through the bluff above the revetment decreased to 1.103 indicating that, for design storm exceedance, some repair above the revetment may be needed.

34. Performance Evaluation

- Alternative 2A – 150 year Storm Design

Based on the analysis of direct wave impact and runup/overtopping damages, Alternative 2A will provide protection from the 150-year storm event.

During this event, damages to the revetment due to direct wave impact are estimated to be between 0- to 5- percent, which is generally referred to as a no-damage condition. Wave overtopping during the 150-year storm event is limited to 47-litres/s/m, which is significantly below 100-litres/s/m, which is the estimated threshold of significant damage to unpaved promenades or reinforced vegetative terracing.

As a measure of uncertainty, if the 150-year water level is increased to include 0.7-feet of sea level rise in 50 years and ¾ standard deviation of storm surge, the overtopping rate increases to be 118-litres/s/m for the paved promenade. This rate is just slightly above the threshold of significant damage to unpaved promenades, but much less than the threshold of significant damage to paved promenades (200-litres/m/s), which is the case for the 150-year design with the 40-foot wide paved promenade berm crest. Therefore, there is a large safety factor including uncertainty throughout the period of analysis.

- Alternative 2B – 73 year Storm Design

Alternative 2B will provide protection from the 73-year storm event.

During this event, damages to the revetment due to direct wave impact are estimated to be between 0- to 5- percent (no-damage condition). Wave overtopping during the 73-year storm event is limited to 60-litres/s/m which is significantly below 100-litres/s/m, which is the estimated threshold of damage to unpaved promenades.

As a measure of uncertainty, if the 73-year water level is increased to include 0.7-feet of sea level rise in 50 years and ¾ standard deviation of storm surge, the overtopping rate is calculated to be 162-litres/s/m for the paved promenade, which is within tolerable limits. This rate is less than the threshold of significant damage to paved promenades, i.e. 200-litres/s/m. Therefore, including uncertainty throughout the period of analysis, there is a reasonable safety factor (greater than 75% certainty).

➤ Alternative 2C – 15 year Storm Design

Based on potential runup/overtopping damages, the level of protection provided by Alternative 2C, with an unpaved promenade (berm crest is only 3 feet wide), is estimated to be on the order of a 15-year storm event. The wave overtopping during this event is estimated to be 70 litres/s/m which is just below the threshold of damage to unpaved promenades. As a measure of uncertainty, if the 15-year water level is increased to include 0.7 feet of sea level rise and one standard deviation of storm surge increase, the overtopping rate is calculated to be 251-litres/s/m. This yields a 60% probability of significant damage to the unpaved promenade (overtopping in excess of 100-litres/s/m) and a 10% probability of structure failure (overtopping in excess of 200-litres/s/m) with uncertainty included.

35. Total Quantities & Annual Costs

All subsequent estimates are based on October 2004 price levels for labor, materials, equipment, 2000 topographic surveys and beach profiles. Quantities for the three alternative design levels of improvement have been developed from the detailed plans shown in the feasibility report, as well as detailed design data reflected in accompanying support documents. The quantities for the alternative revetment designs for the Montauk Point erosion control project were computed as follows and are presented in Table 11:

Table 11 – Initial Construction Quantities

Materials	Alternative 2A 150-year protection Crest elevation +30 Ft.	Alternative 2B 73-year protection Crest Elevation +25 Ft.	Alternative 2C 15-year protection Crest Elevation +25 Ft.
Armor Stone (tons)	57,100	46,700	15,600*
Armor Stone Rehandled (tons)	19,100	19,300	1,000
Underlayer (tons)	23,700	18,600	1,000
Bedding Stone (tons)	12,100	11,100	11,500
Excavation (cubic yards)	34,200	32,000	15,000

** Includes construction of cofferdam offshore and reuse in revetment. Alternative cost also includes the disposal of 7,300 tons of unusable existing armor stone to be disposed on site at the structure toe. Also included is 8,000 square feet of temporary exposed bank protection during construction.*

Studies indicate that with Alternatives 2A and 2B, damages to the revetment and the bluff would be reduced significantly and that damages from storm exceedence are greatly reduced compared to Alternative 2C, where storm exceedance damages are high.

Alternative 2A – 150 year Storm Design

- The economic evaluation of Alternative 2A (150-year storm design) with a revetment height of +30 feet NGVD considered the impacts of storm events ranging from a 2-year event to a 200-year event. Wave impact damages are initiated slightly at the 15-year return period storm and overtopping damages are initiated at the 200-year return period storm.

- The total first cost is $15,998,900, plus $1,057,000 for interest during construction, for a total investment cost of $17,055,900.

- The total annual cost of Alternative 2A is estimated to be $1,050,400. Refer to Engineering Quantities and Cost Appendix for more details.

Alternative 2B – 73 year Storm Design

- The economic evaluation of Alternative 2B (73-year storm design) with a revetment height of +25 feet NGVD considered the impacts of storm events ranging from a 2-year event to a 200-year event. Wave impact damages are initiated, slightly, at the 5-year storm event and minor overtopping damages are initiated at the 73-year storm event

- The total first cost is $13,722,900, plus $712,700 for interest during construction, for a total investment cost of $14,435,600.

- The total annual cost of Alternative 2B is estimated to be $889,300. Refer to Engineering Quantities and Cost Appendix for more details.

Alternative 2C – 15 year Storm Design

- The economic evaluation of Alternative 2C (15-year storm design) with a revetment height of +25 feet NGVD considered the impacts of storm events ranging from a 2-year event to a 200-year event. Wave impact damage is initiated slightly at the 2-year return period storm and overtopping damage is initiated at the 15-year return period storm.

- The total first cost is $5,804,000, plus $301,400 for interest during construction, for a total investment cost of $6,105,400.

- The total annual cost of Alternative 2C is estimated to be $524,700. Refer to Engineering Quantities and Cost Appendix for more details.

Table 12 summarizes the First Costs and Annual Costs for Alternatives 2A, 2B, and 2C.

Table 12 – Stone Revetment – 3 Alternatives - Construction Cost Estimates

October 2004 Price Levels

FIRST COSTS & ANNUAL COSTS SUMMARY

	Alternative #2A 150-year protection	**Alternative #2B 73-year protection**	Alternative #2C 15-year protection
Total First Cost	$ 15,998,900	**$ 13,722,900**	$ 5,804,000
Interest During Construction @ 5.375%	$ 1,057,000	**$ 712,700**	$ 301,400
Total Investment Cost	$ 17,055,900	**$ 14,435,600**	$ 6,105,400
Annualized Investment Cost Based on 50-year design life Annual interest of 5.375%	$ 988,900	**$ 837,000**	$ 354,000
Annualized Revetment Maintenance Cost	$ 61,500	**$ 52,300**	$ 170,700
Total Annual Cost	$ 1,050,400	**$ 889,300**	$ 524,700

The NED plan was chosen based on the economic evaluation discussed in the next section of this report.

36. Economic Analysis

The feasibility study was conducted under the study authorities noted in this report. In addition, Section 110 of the National Historic Preservation Act of 1966, as amended (NHPA), imposes a duty to maintain and preserve historic properties. At the present time, this duty is presently borne directly by the Montauk Point Historical Society, the current owners of the Montauk Point Lighthouse complex. However, through the operation of a reversionary interest, as provided for in the land transfer (a quitclaim dated 18 September 1998 from the U.S. Coast Guard to the Montauk Point Historical Society), this duty ultimately falls on the Federal Government. Section 110 of the NHPA imposes duties only on federal agencies.

As a federal agency, the Coast Guard was required to preserve and maintain the property in accordance with the NHPA. The transfer of the property from the Coast Guard to the Historical Society would have been an adverse impact on the property under Section 110 of the NHPA, because the historic property would have passed to an entity, the Historical Society, that was not a Federal agency and therefore not required to adhere to the NHPA, removing the legal protection the historic property enjoyed under federal ownership. To remedy this adverse impact, the Coast Guard included a condition in the transfer agreement that requires the Historic Society to preserve/maintain the property under the NHPA, effectively making the Historical Society act as a Federal agency with regards to the preservation of the property.

Alternative ways to follow Section 110 of the NHPA at Montauk Point therefore include:

- Provide mitigation for adverse impacts following a storm event that causes damage to the bluff and other features of the historic property, or

- Take steps now to protect the integrity and significance of the historic property, thereby avoiding the costs of Section 110 compliance that would have been triggered by storm damage.

- Through a combination of Section 110 of the NHPA and the nature of the land conveyance, there is indeed a statutory duty to perform the cultural resources mitigation at Montauk Point. If triggered by coastal storm damage such mitigation would incur a cost; therefore, avoiding that cost should, therefore be counted as a benefit.

If the Federal government is not mandated to follow Section 110 of the NHPA and the nature of the land conveyance, then the most likely future without-project scenario is that the bluff will erode and the historic Montauk Point Lighthouse complex will collapse.

The economic analysis that follows below is based on this assumption.

The proxy used to place a depreciated replacement value of the historic Montauk Point Lighthouse complex is based on the calculations for the costs of cultural mitigation. Moving the Montauk Point Lighthouse complex, a National Register listed property, will potentially preserve the existing structures, but allow for the eventual destruction of the bluff point and buried cultural resources. These archaeological materials, which are associated with the historic and prehistoric use of the bluff, must be documented and recovered. Prior to moving the structures, each structure would need to be documented on engineering drawings and in photographs so that they can be rebuilt properly on the new site. Subsurface archeological excavations would be performed to recover artifacts both at the present lighthouse site and at the new site. Alternatively, all of these costs could be avoided by protecting the property from the storm damage.

Existing Conditions

The lighthouse complex and the surrounding Montauk Point State Park are valued State properties. Montauk Point Lighthouse complex and the State Park annual attendance figures averaged 106,723 and 904,185 persons, respectively in the 1995-2002 period. The lighthouse complex does not have a parking lot, and visitors must use the state parking lot. The average attendance for the state park only is 797,462. These figures were obtained from Montauk Point Lighthouse and Montauk State Park offices. Recent census data indicate that the populations for Long Island and New York's five boroughs have increased by 8.4% in ten years. The population for the surveyed area increased from 9,931,776 (1990 Census) to 10,762,191 (2000 Census). The economic analysis assumes the lighthouse and state park attendance will remain stable.

Without-Project Future Conditions

The Montauk Point Lighthouse complex sits on a high bluff underlain with glacial till, approximately 70-feet above Mean Sea Level (MSL). It is estimated that once the upper sections of the revetment that protects the bluff are displaced by a 15-year or greater storm event, the foundation soil underlying the displaced stone will become exposed and subject to subsequent erosion. To determine the extent of this erosion at the toe of the upper bluff above the damaged revetment that would cause significant bluff failure, a slope stability analysis was performed. The results of this analysis determined that for significant bluff failure, the damaged crest elevation of the revetment would have to degrade to approximately elevation +10 NGVD and the upper bluff toe at this +10 NGVD elevation recede horizontally approximately 10 feet. This is anticipated to cause approximately 26-30 feet of loss of the bluff crest which will immediately threaten the lighthouse facility at the most critical area to the southeast of the lighthouse. The period of time estimated for this condition to occur, subsequent to revetment failure, is an additional 10 years of long-term erosion at the upper bluff toe (at el. +10 NGVD). A decision tree analysis was applied to calculate the probability of revetment failure for any given year through the 50-year period of economic analysis due to a 15-year or greater storm event. When revetment failure occurs, the bluff crest will erode at an average rate of 3 feet per year. The lighthouse complex will be immediately threatened after 10 years, or 30 feet of erosion at the bluff crest.

Proxy for Depreciated Replacement Value of Montauk Lighthouse Complex

The proxy used to place an economic value of the historic Montauk Point Lighthouse complex is based on the hypothetical calculations for the costs of cultural mitigation of the site. The economic analysis assumes that cultural mitigation of the site will be initiated after the revetment that protects the bluff is displaced. The estimated cost for moving the Montauk Point Lighthouse complex and complete cultural mitigation of the complex is $20 million. This figure does not take into account the required creation of raised grades landward of the present location of the lighthouse for the move, which would add an additional cost of $7 million. The raised grade would be necessary to maintain the lighthouse elevation because the existing bluff elevation decreases significantly as one move away from the shorefront. The overall mitigation process would take approximately six years to complete, with a total cost of $27 million.

Local Costs Foregone

The lighthouse complex is situated on 3 acres of land, specifically a bluff that has an appraised value of $12 million. It is estimated that the top of the bluff will erode at a rate of 3 feet per year when the revetment fails. Because of the complexity of actually replacing the bluff surface, a prorated amount of the appraised value of land lost was used as a proxy for the local costs forgone for this loss in the without-project condition. The average annual local costs forgone are $74,100.

Recreation Loss Value

Another without-project consequence of storm damage to the bluff would be loss of visitations to the lighthouse. Visitation losses associated with the lighthouse's closure were assessed using the Travel Cost Estimate of Willingness to Pay. The lighthouse has a log in which visitors indicate the places where they are traveling from during their visit. A recent sample from the log was used to estimate the round-trip distance from each origin. The values of losses are the costs in cents per mile to operate an automobile, plus the opportunity costs of time spent in travel and on site. Surveys were conducted to determine the number of visitors that make the trip to Montauk, New York exclusively to visit the lighthouse. Based on the survey, 47% of the people sampled indicated that visiting the Montauk Lighthouse complex was the reason they drove to Montauk, New York. The remaining 53% of the people indicated that visiting the Montauk Lighthouse complex was part of their itinerary on their visit to Long Island, New York. The travel costs attributed to this category were prorated at 25% of their total travel costs.

Lighthouse visitations will be lost when the existing revetment is damaged by a 15-year or greater storm event, followed by 10 years of erosion to the bluff. If the revetment is damaged in year 2005, the lighthouse visitations will be lost starting in year 2015. Since the base year is 2009, the lighthouse visitations will be lost from 2015 through 2058. The $3,040,200 generated per year from lighthouse visitations from 2015 through 2058 is discounted to the first year that visitations are lost, year 2015. This was done to convert 44 years of lost visitations into a one-year equivalent loss that will occur in 2015. Similar calculations converted the lost visitations into one-year equivalents losses that will occur in years 2016 through 2058. The average annual lighthouse visitations are calculated to be $882,700.

The Montauk Point Lighthouse complex resides within the Montauk Point State Park. The Montauk Point Lighthouse complex offers a unique experience that is not found elsewhere in the New York metropolitan area. Part of the state park experience is its connection with the lighthouse complex. There will be a reduction to the overall aesthetics and recreational value of the state park visitations if the lighthouse complex did not exist. The average annual reduced state park recreational experience would be $198,200.

<u>With-Project Conditions</u>

Preliminary screening of various alternatives identified that the Stone Revetment Plan is the most feasible alternative both economically and environmentally in providing protection to Montauk Point and its vicinity. Three design levels were considered, the 15-year, 73-year, and 150-year alternatives, to determine the optimal plan. The three alternatives provide protection to the Montauk Point Lighthouse complex until storm exceedance starts to displace the armor stones at the upper portion of the stone revetment for each storm protection design. Residual damages were calculated for the three alternatives and used for plan evaluation.

The existing revetment has been in place since 1994. In the with-project condition, construction will commence in 2008 and will be completed by January 2010. The 15-year storm design, therefore, is pertinent through 2007, with the improved level of protection pertinent from 2008, thereafter. With-project damages were calculated for the following storm damage categories: Storm damage to the lighthouse complex, and local costs foregone for the land loss value due to erosion. With-project damages were also calculated for two recreation loss categories: lost lighthouse visitations, and loss of State Park visitation benefits.

<u>Benefits</u>

Benefits are estimated to be annual damages in the without-project conditions minus any residual damages in the with-project alternatives. The benefits claimed are avoided storm damage costs when compared to the existing condition, specifically avoided loss of the lighthouse complex and its associated costs for the preservation of artifacts, local costs foregone for the loss of land value, and avoided lost visitation benefits to the lighthouse and to the State Park.

The project benefits for the three alternatives are summarized in Table 13 below. All benefits are discounted using a 5 $^{3}/_{8}$ percent interest rate and amortized over the 50-year period of analysis.

Table 13 - Benefit Summary (Oct. 2004 P.L., 5.375% discount rate)

Description	Without-Project Damages	Residual Damages - 15yr storm design	Benefits - 15yr storm design	Residual Damages - 73yr storm design	Benefits - 73yr storm design	Residual Damages - 150yr storm design	Benefits - 150yr storm design
Storm Damage Reduction							
Lighthouse Complex	$518,452	$318,655	$199,797	$33,617	$484,835	$15,732	$502,720
Local Costs Foregone	$74,100	$60,402	$13,698	$19,226	$56,520	$12,636	$61,464
Subtotal	$592,600	$379,100	$213,500	$52,800	$541,400	$28,400	$564,200
Recreation							
Lighthouse Visitation	$882,662	$432,527	$450,135	$35,530	$847,132	$15,007	$867,655
Park Visitation	$198,153	$97,100	$101,053	$7,976	$190,177	$3,369	$194,784
Subtotal	$1,080,800	$529,600	$551,200	$43,500	$1,037,300	$18,400	$1,062,400

Table 14 summarizes the annual cost for the stone revetment alternatives.

Table 14 - Cost Summary (Oct. 2004 P.L., 5.375% discount rate)

Description	15yr storm design	73yr storm design	150yr storm design
Total First Cost	$5,804,000	$13,722,900	$15,998,900
Interest During Construction	$301,400	$712,700	$1,057,000
Total Investment Cost	$6,105,400	$14,435,600	$17,055,900
Annual Investment Cost	$354,000	$837,000	$988,900
Annual Revetment Maintenance Cost	$170,700	$52,300	$61,500
Total Annual Cost	$524,700	$889,300	$1,050,400

Conclusion – NED Plan Selection

Planning Guidance Notebook, ER 1105-2-100, 22 April 2000, Chapter 3-4b(4)(a), reads in pertinent part,

> *"The Corps participates in single purpose projects formulated exclusively for hurricane and storm damage reduction, with economic benefits equal to or exceeding the costs, based solely on damage reduction benefits, or a combination of damage reduction benefits and recreation benefits. Under current policy, recreation must be incidental in the formulation process and may not be more than fifty percent of the total benefits required for justification. If the criterion for federal participation project cost sharing is met, then all recreation benefits are included in the benefit to cost analysis."*

Federal participation in this recreation benefit generating shore protection project is warranted since the recreation benefits are incidental, and when combined with, and limited to, an equivalent amount of primary hurricane and storm damage benefits, they produce an economically justified project.

One way to test this is shown in Table 15 below. Lines 1 and 2 shows the storm damage reduction benefits and incidental recreation benefits respectively. Line 3 shows the incidental recreation benefits limited to an equivalent amount of the storm damage reduction benefits. The incidental recreation benefits are limited because the storm damage reduction benefits must be at least 50 percent of the total benefits used for project evaluation. The sum of these two benefits is displayed in Line 4, and when compared to the annual project costs are used to determine if an alternative is economically justified. The 73-year and 150-year designs are economically justified because their net benefits are positive, and therefore have benefit-to-cost ratios (BCR) greater than one (Lines 6 and 7).

The 73-year design is the National Economic Development (NED) plan because it has the greatest net benefits (Line 6). All recreation benefits (Line 2) are included in the total benefits, total net benefits and final BCR (lines 8, 9 and 10) because the criterion for Federal participation project cost sharing with limited recreation benefits has been met.

Table 15 - NED Plan Selection (Oct. 2004 P.L., 5.375% discount rate)

Description	**15yr Storm Design**	***73yr Storm Design***	**150yr Storm Design**
1. Annual Storm Damage Benefits	$213,500	**$541,400**	$564,200
2. Annual Recreation Benefits	$551,200	**$1,037,300**	$1,062,400
3. Annual Recreation Benefits Used for Project Justification	$213,500	**$541,400**	$564,200
4. Total Benefits Used for Project Justification	$427,000	**$1,082,800**	$1,128,400
5. Annual Costs	$524,700	**$889,300**	$1,050,400
6. Net Benefits	-$97,700	**$193,500**	*$78,000*
7. BCR	0.8	**1.2**	1.1
8. Total Benefits		**$1,578,700**	
9. Total Net Benefits		**$689,400**	
10. Final BCR		**1.8**	

37. The Selected Plan – Stone Revetment - Alternative 2B

Based on maximum net excess benefits, the selected plan consists of the construction of a stone revetment with a 73-year storm design (Figures 30, 31 and 32). Project features include:

- Stone revetment, 840-feet in length, with a crest width of 40-feet at elevation +25 feet NGVD, and 1V:2H side slopes.
- 12.6-ton quarrystone armor units extending from the crest down to embedded toe.
- Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure.
- The bottom of the armor stone layer in the toe is located at a depth of 12-feet from the existing bottom.
- A heavily embedded toe is incorporated to protect against breaking waves, provide long-term stone stability, and scour at the toe of the structure. Stone sub-layers are specified in accordance with standard design procedures.

The cost estimate for the construction of the revetment was approached from the viewpoint of heavy stonework and earthwork operations characterized by large cranes and excavators, loaders and haul trucks. Approximately 840-linear feet of revetment will be constructed along the Montauk Point shoreline.

Productivity considerations were based on the relative configuration of the existing revetment and bank, wave and tide conditions, stone size, placement criteria, distance of truck-delivered stone material from off-site and on-site stockpiles, access, haul roads, entrances, and construction easements.

A construction access berm will be constructed adjacent to the slope of the existing stone revetment ends. This construction will be completed on both the northerly and southerly ends of the revetment. The elevation of the access berms will be +8 feet NGVD. There is one access road, and one alternative access road, designated at each end of the revetment.

Two separate crews are anticipated to perform the work. One crew will operate on the northerly end and the other operating on the southerly end of the revetment. Each crew should have one large power excavator for stiff digging and one large crane for stone removal and placement. The excavation and stone placement construction will be conducted from the construction access berm at elevation +8 feet NGVD. No access via water is proposed. Excavation and stone placement will be performed by the same crew, as there is not enough room on the construction berm for two crews to work at one location concurrently.

Ten (10) 38-ton trucks with 16 to 23.5 cubic yard trailers are anticipated to be used for hauling the bedding, underlayer and armor stone from the quarry to the project site. Two (2) 25-ton (16 to 19 cubic yards) off-highway trucks are proposed to deliver stone from the stockpile area to the work area.

Excavated bottom material from the revetment toe area will be transported directly to a Dredge Material Placement (DMP) site on-site within the grounds of Montauk State Park using the 25-ton off-highway dump trucks. The exact site of the DMP area is to be determined.

It is assumed that about 19,300 tons of existing revetment stone will be re-used in the new revetment. Any unusable stone from the existing revetment will be placed overlying the restored ocean bottom after the buried toe is constructed.

It is estimated that the stone revetment would have a useful life expectancy of 50 years.

First costs include the charges arising from the construction of the stone revetment, as well as the costs of contingencies, engineering, design, supervision and administration, and are summarized in Table 16.

Table 17 provides the Fully Funded Costs for the selected plan initial construction escalated to the midpoint of construction, January 2009.

Table 16 – Cost Summary Details – Project First Cost

Revetment (October 2004 price level)

Mobilization, Demobilization		$ 600,300
Armor Stone	46,700 tons	$5,944,000
Armor Stone Rehandled	19,300 tons	$1,304,700
Underlayer Stone	18,600 tons	$2,383,300
Bedding Stone	11,000 tons	$1,198,800
Excavation	32,000 cubic yards	$ 591,600
Sub-Total Revetment		**$12,022,700**
Lands & Damages		$32,000
Planning, Engineering & Design		$630,000
Construction Management		$1,038,200
TOTAL PROJECT FIRST COST		**$13,722,900**

Table 17 - FULLY FUNDED COSTS

		Project First Costs				Fully Funded Estimate			
Current MCACES Estimate						**Feature Mid Point: JANUARY 2009**			
Effective Pricing Level October 2004 Account		Cost ($)	Contingency ($)	Contingency (%)	Total	%	Cost ($)	Contingency ($)	Total
10	Seawall & Revetment	$10,454,400	$1,568,300	15%	$ 12,022,700	7.78%	$11,268,000	$1,690,300	$ 12,958,300
01	Lands and Damages*	$30,000*	$2,000*	7%	$32,000*	18.42%	$35,500	$2,400	$37,900
30	Engineering & Design	$547,800	$ 82,200	15%	$630,000	18.42%	$648,700	$97,300	$746,100
31	Construction Management	$902,800	$135,400	15%	$1,038,200	20.99%	$1,092,300	$163,800	$1,256,100
	Total Project Cost	**$11,935,000**	**$1,787,900**		**$13,722,900**		**$13,044,500**	**$1,953,900**	**$14,998,400**

Note:
Acct 01, 30, 31 escalation using EC11-2-187 dtd 28 Apr 2005 Table A Class 1
*Acct 01 – Costs for lands has been determined to be $0, Administrative costs are $32k.

38. Policy Exemption for Private Non-Profit Landowner

The land that will be protected by implementation of this recommended project is deeded to the Montauk Historical Society (MHS). The MHS is a private, not for profit association that is not part of any state or local government. This land is held open, for use by all on equal terms, regardless of origin or home area. Existing Corps policy (ER 1165-2-130, ER 1165-2-123) indicates that there is no Federal interest in protection of a property owned by a single private non-profit entity. However, although the MHS is clearly a single, private landowner, they must, by deed restriction and State charter, act as a public entity akin to agencies of State and local governments. The MHS must accomplish a public education mission to stay in operation, must follow Federal National Historic Preservation requirements for maintenance work, and membership and enjoyment of the benefits of the facility and educational programs are open to all, with no restriction, for a fee. Under the deed and charter, the MHS cannot structure and constrain uses of the property, nor can anyone who cares to join the MHS and enjoy the benefits of the facility and water resources project be excluded.

In light of these facts, a waiver to the single landowner policy from the Assistant Secretary of the Army (Civil Works) was granted on 29 June 2005 allowing the completion of the feasibility study with a view towards pursuing a cost-shared construction project for Montauk Point, New York.

39. Project Construction Cost-Sharing

The cost-sharing for this project is 50% Federal and 50% Non-Federal (see Table 18).

Table 18 – Cost Apportionment

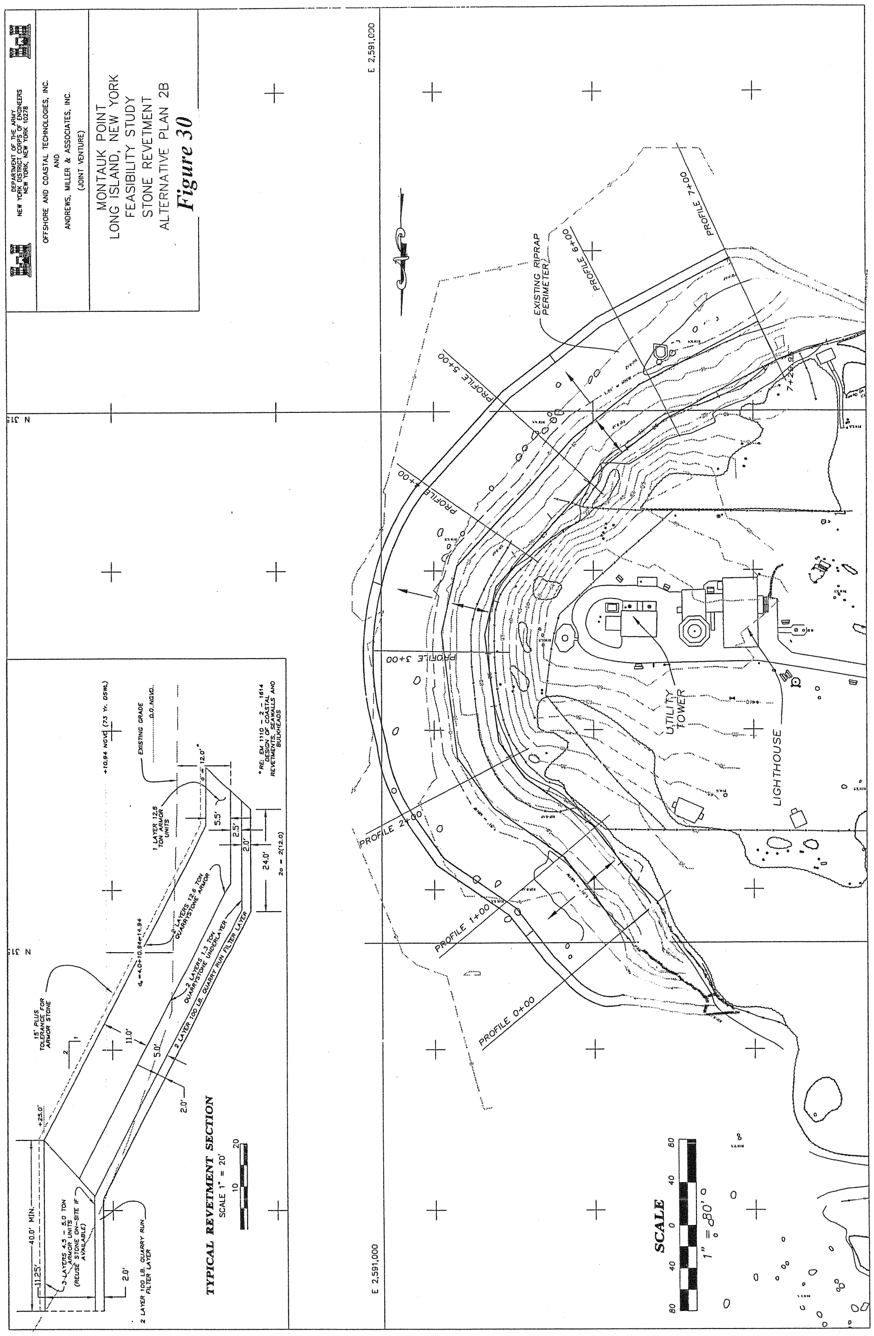
Cost-Shared Items

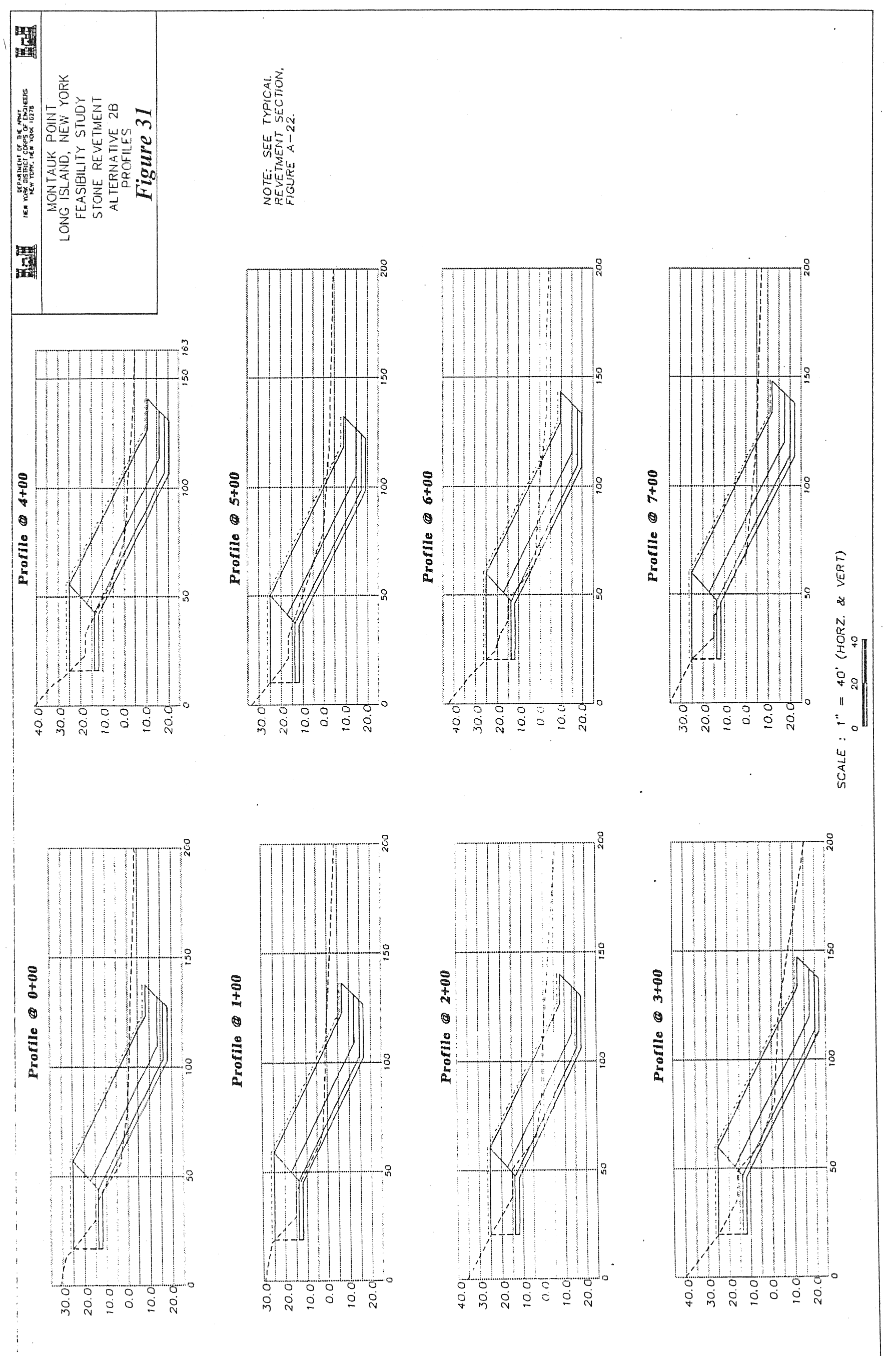
	Federal Share 50%	Non-Federal Share 50%	Total Cost
Cash Contribution	$6,861,000	$6,861,000	$13,722,000
Real Estate Lands * & Damages	$0	$0	$0

Non Cost-Shared Items

	Federal Share 0%	Non-Federal Share 100%	Total Cost
Annual Revetment Maintenance	$0	$52,300	$52,300

** Value of easements to be obtained are estimated to be $0. Administrative and incidental costs associated with easements to be obtained are estimated to be $32k, and are included in cash contribution cost.*


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
OFFSHORE AND COASTAL TECHNOLOGIES, INC.
AND
ANDREWS, MILLER & ASSOCIATES, INC.
(JOINT VENTURE)
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2B
Figure 30
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
EXISTING GRADE
EXISTING RIPRAP PERIMETER
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
UTILITY TOWER
LIGHTHOUSE
E 2,591,000
SCALE
1" = 80'


MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2B
PROFILES
Figure 31
NOTE: SEE TYPICAL REVETMENT SECTION, FIGURE A-22.
Profile @ 0+00
Profile @ 1+00
Profile @ 2+00
Profile @ 3+00
Profile @ 4+00
Profile @ 5+00
Profile @ 6+00
Profile @ 7+00
SCALE : 1" = 40' (HORZ. & VERT)

Profile @ 5+00

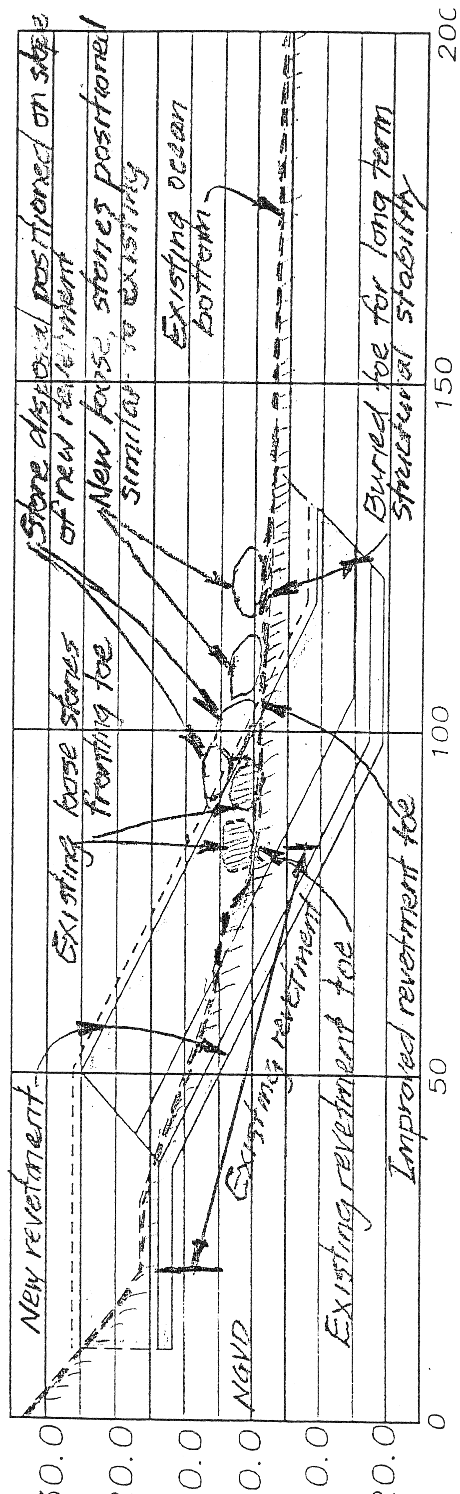


Figure 32

40. Environmental & Cultural Resources Impacts of The Selected Plan

An Environmental Impact Statement (EIS) has been completed and is enclosed with this feasibility report. The proposed work will have no significant impact on the quality of the environment in the project area. Most impacts associated with this project will be temporary, and none of the impacts are regarded as significant (refer to the EIS for additional information).

Topography, geology and soils

Implementation of the revetment is expected to result in significant benefits to the existing topography by stabilizing the bluff and shoreline.

Water Resources

The construction of the revetment would not impact regional hydrology or groundwater resources because the revetment construction would occur at the surface of the bluff and along the Montauk Point shoreline. Implementation of the proposed revetment is expected to result in significant long-term benefits to the existing hydrology and groundwater flow by stabilizing the bluff and shoreline.

Surface Water

During construction of the revetment, a temporary increase in turbidity of nearby surface water is expected. However, the suspended materials would be expected to settle out quickly or would be rapidly transported away by the strong tidal currents. Following completion of in-water construction activities, water quality would be expected to quickly return to pre-construction conditions. No significant long-term impacts on surface water quality are expected.

Wetlands

No direct or indirect impacts to freshwater wetlands, coastal ponds, or interdunal swales in the project area are expected due to construction of the stone revetment. The new revetment would essentially replace the existing revetment within the existing footprint. The minor, temporary and localized suspended sediment generated by revetment construction would quickly settle out of the water column, and would not result in significant sedimentation in the project area or the adjacent unvegetated marine wetlands.

Wildlife

The Fish and Wildlife Coordination Act (FWCA) (16 USC 662(a)) provides that whenever the waters of any stream or other body of water are proposed to be impounded, diverted, the channel deepened or otherwise controlled or modified, the District shall consult with the U.S. Fish and Wildlife Service (USFWS), the Nation Marine Fisheries

Service (NMFS) as appropriate, and the agency administering the wildlife resources of the state. A FWCAR was submitted to the USACE by the USFWS (refer to Environmental Impact Statement - Appendix E). The FWCAR incorporates consultations with the NYSDEC and NMFS, regarding existing fish and wildlife resources, anticipated impacts, and recommendations for avoidance and minimization of impacts. Overall, the USFWS concluded the impacts to fish and wildlife resources occurring within the footprint of the proposed construction area would be minimal. The NYSDEC also concurred with the FWCAR's conclusions and recommendations.

Benthic Resources

Construction of the project would impose a one-time, temporary impact on existing benthic communities at the nearshore area of the Project area. The USFWS's FWCAR concluded that, due to the amount of data supporting the rapid recovery of benthic organisms, there will be limited impacts to the subtidal benthic community as a result of project implementation except in areas of direct stone placement where infaunal communities would be replaced with epifaunal communities.

Finfish and Shellfish

Construction of the project would impose a one-time, temporary impact on the existing finfish and shellfish species at the nearshore area of the project area. The USFWS concluded within their FWCAR that negative impacts to finfish are not expected as a result of implementation of the project. Similar to the finfish species in the project area, recolonization by shellfish species is expected to occur after completion of the proposed project.

Essential Fish Habitat (EFH)

Temporary impacts on EFH are predicted during periods of active construction. Habitat would be temporarily degraded during construction, as a result of elevated suspended sediment levels, temporarily lowering visual feeding efficiency, and irritating gill tissue. However, the suspended sediments are expected to settle quickly out of the water column. Therefore, no long-term adverse impacts on the water quality aspects of EFH are expected.

Birds

The project would result in the temporary disturbance to those species of birds that may utilize the existing revetment for resting, however the new revetment would mimic the old revetment in material and design and immediate reestablishment of resting use is expected. Negative impacts to pelagic seabirds are not expected due to the high mobility and use of deeper water habitats by these species. Following construction, bird species are expected to resume their normal habits consistent with post-construction habitat availability in and within the vicinity of the project area.

Mammals

Construction of the proposed project could have minor short-term impacts on terrestrial mammal populations occurring in the area. Construction equipment traveling over terrestrial habitat could result in the temporary disturbance of habitat and possible mortality of less mobile, burrowing, and/or denning species of mammals during construction activities. The return of ground dwelling species may be reduced, depending on the level of soil compaction that results from construction equipment traveling over terrestrial habitat. Construction activities may also cause the temporary and permanent displacement of more mobile species due to increased human activity and habitat alterations. All of these potential impacts are expected to be of minimal significance because vegetated environments would not be impacted by the project. Following construction, wildlife species are expected to resume their normal habits consistent with post-construction habitat availability in and within the vicinity of the project area.

Federal Species of Concern

Although several species of Federally listed endangered and threatened species of animals and plants can be expected to occur in the general vicinity of the project area at any time, no impacts to these species are expected to occur as a result of construction of the project. The FWCAR concluded that no Federally-listed or proposed endangered or threatened species under the jurisdiction of the USFWS are known to exist within the project impact area and that no habitat in the project area is currently designated or proposed critical habitat in accordance with the provisions of the Endangered Species Act.

State Species of Concern

Although animal and plant species are unlikely to be impacted by the proposed project, the District will conduct pre-construction surveys for state-listed plants and birds and will coordinate with the NYSDEC regarding proper survey protocols as recommended in the USFWS's FWCAR. Further coordination with the NYSDEC would be initiated regarding recommendations to minimize and avoid disturbance if listed species are encountered.

Economy and Income

The project is expected to have a beneficial, long-term effect on the economic characteristics associated with the project area through the protection of Montauk Point from inevitable future erosion and storm damage. Such protection would preserve the bluff top and the Lighthouse complex for continued use by seasonal and permanent residents, and would result in a continuing contribution by the diverse recreational facilities located within the project area to various aspects of the local economy, including the continued demand for seasonal housing, restaurants, and local businesses in support of the recreational uses of the project area. .

Cultural Resources

Based on the results of previous cultural resource investigations, several of the archaeological sites uncovered around the Lighthouse are eligible for inclusion on the National Register of Historic Places (NRHP) under several of the prescribed criteria. Furthermore, the entire Lighthouse Complex itself is eligible as a National Register District, possessing integrity and significance based upon the characteristics of location, setting, feeling, association and design, including "a significant concentration, linkage, or continuity of sites, buildings, structures, or objects, united historically or aesthetically by plan or physical development." Because the Lighthouse property possesses all of these elements, the District encourages the Montauk Point Historical Society to apply for this status.

Construction of the project will not significantly impact the buried cultural resources that are located at the Lighthouse complex, and, in fact, will help to preserve the cultural resources that have been identified by reducing the potential for further erosion of the bluff face. However, it is the recommendation of the District that archaeological monitoring be conducted during the construction phase of the project. Archaeological monitoring during the removal and replacement of the revetment stones will ensure that buried archaeological materials are not disturbed. If previously unidentified archaeological materials are uncovered during construction, the on-site archaeologist would evaluate their significance. If any identified archaeological sites are determined to be potentially eligible for the NRHP, work will be halted and consultation with the New York State Office of Preservation will occur. Upon completion of consultation, if a finding of no-significance is determined, the project will continue after the materials are recorded.

Land use and zoning

Construction, operation, and maintenance of the revetment would not have any direct or indirect impacts on the existing land use and zoning in the project area. The existing land uses in the area would not change as a result of the project. Zoning designations would not be changed, nor would any homes or businesses be removed or displaced.

Coastal Zone Management

As required under the Federal Coastal Zone Management Act of 1972, the District reviewed the proposed Project in relation to the applicable policies of the New York State CMP and determined that it is consistent with all relevant policies. The New York State CMP Consistency Statement is provided as Appendix F of this EIS.

Hazardous, Toxic and Radioactive Wastes (HTRW)

No impacts to any HTRW sites are expected to occur as a result of the proposed project because no sites have been identified in the project area. The District would implement standard guidelines for the storage and cleanup of hazardous materials in the project area during construction. In addition, as recommended by the USFWS, an oil-spill contingency plan would be developed and coordinated prior to any construction.

Navigation

Construction and replacement of the existing revetment is limited to the nearshore area of the project area. Due to the proximity of the revetment to the shore and the absence of Federal or state navigational channels near the project area, no navigational channels would be impacted as a result of the proposed project. Construction of the proposed project would have a long-term beneficial impact in securing the integrity of Turtle Hill Plateau where the Lighthouse and associated facilities presently stand.

Aesthetic and Scenic Resources

Long-term impacts to aesthetic and scenic resources resulting from the construction of the revetment are expected to be of minimal significance to natural and manmade landscapes. The proposed project would be consistent with the existing revetment structure in the project area and would result in very low levels of change in the surrounding landscape that would not attract undue visual attention.

Short-term impacts to aesthetic and scenic resources during the construction phase are also expected to be of minimal significance. However, the District recognizes that construction equipment operating and traveling through the project area during the 2-year construction period could have a negative effect on the scenic resources as well as the relatively quiet and peaceful setting normally provided by Montauk Point State Park. As a result, the District has coordinated with the Montauk Historical Society and NYSOPRHP to develop a plan that would minimize impacts to these aesthetic resources. Currently, the plan includes limiting the time of day when equipment and heavy-duty trucks access the area to off-peak visitation hours. This would reduce the number of encounters that visitors would have with construction equipment traveling to and from the staging areas and revetment. Although these off-peak hours have not yet been determined, a seasonal schedule would be developed in coordination with the Montauk Historical Society and NYSOPRHP.

Recreation

Construction of the project would result in short-term, direct impacts to recreational uses, such as use of pedestrian trails and the revetment for fishing, by temporarily limiting and/or blocking access to the beachfront and the existing revetment. These short-term, direct impacts would primarily affect recreational fishing because surfcasting from the existing revetment is a popular activity at Montauk Point. As a result of this potential impact, the District has coordinated with the Montauk Surfcasters Association and the New York Sport Fishing Federation to develop a plan that would minimize impacts on access to the revetment by fishermen during construction and enhance access after construction. The District has developed a construction schedule that will allow fishermen limited access to the revetment area during the initial stages of construction.

Both organizations understand the importance of ensuring that there is a strong, stable, and long-lasting revetment wall at Montauk Point and offered their full support of the project. Access impacts during construction would be reduced by allowing limited access to the current revetment for fishing during the construction period to the maximum extent practicable, without causing a safety hazard. By initiating construction on the south end of the revetment while having a delayed construction start date on the north end of the revetment, a few additional months of access to the revetment by fishermen would be possible. However, eventually the entire revetment and staging areas immediately adjacent to the northern and southern ends of the revetment would need to be closed to the public. During this time, fishermen would still be able to fish from the adjacent beach areas.

The Surfrider Foundation, Long Island Chapter, raised concerns regarding the impact of the proposed project on recreational surfing. In response to the Surfrider Foundation's concerns, the District performed modeling to determine the potential effect of implementation of the proposed project on offshore waves. The results of this modeling determined that the reflection coefficient for the existing revetment ranged from 0.30 to 0.33, whereas the reflection coefficient for the proposed revetment would range from 0.25 to 0.28, an approximate 15 percent reduction from that of the existing revetment. This reduction is due to the milder front slope and the greater porosity of the thick layers of randomly placed stone of the proposed revetment. Based upon the modeling results, the District believes that implementation of the proposed project would have little to no impact on the quality or surfability of the waves in the offshore waters of Montauk Point, and may, in fact, have less impact than the existing structure.

Overall, implementation of the stone revetment alternative would not result in a significant short-term loss of recreational use of Montauk Point. Although the revetment wall would be closed to the public, the Turtle Hill plateau and adjacent beach front areas would remain open and usable by the public. Long-term impacts on recreation due to implementation of the proposed project are considered to be beneficial, primarily as a result of the long-term preservation of Montauk Point State Park and the Lighthouse complex.

Transportation

The stone revetment alternative is expected to have limited, short-term impacts to transportation within the project area. Such impacts would be associated with construction of the revetment, and would include the added presence of construction related vehicles through Montauk Point State Park, and along access roads from the bluff top down to the shoreline. Construction-related vehicles are expected to include slow-moving, heavy-duty construction equipment, as well as worker's vehicles. The added presence of construction-related vehicles may result in increased traffic and impediments to normal traffic flow in the project area. To help alleviate this impact during construction of the project, flagmen would be available and construction signs would be posted. In addition, the District has coordinated with the Montauk Historical Society and NYSOPRHP to develop a plan that would limit the time of day when equipment and heavy-duty trucks access the area to off-peak visitation hours. This would reduce congestion along the Montauk State Park Highway (the only road in and out of the park). Although these off-peak hours have not yet been determined, a seasonal schedule would be developed in coordination with the Montauk Historical Society and NYSOPRHP. Following construction, the stone revetment alternative is not expected to have any impacts to transportation conditions in the Project area. In addition, all roads would be monitored during the construction phase and returned to their pre-construction condition.

Air Quality

General Conformity under the Clean Air Act, Section 176 has been evaluated for the project described above according to the requirements of 40 CFR 93, Subpart B. The requirements of this rule are not applicable to this proposed project because total direct and indirect emission of from this project/action have been estimated that Ozone (NOx & VOC's) 19.66 tons are below the conformity threshold value established at 40 CFR 93.153(b) of 25 tons per year, and the proposed project/action is not considered regionally significant under 40 CFR 93.153(i). No short-term or long-term impacts to air quality are expected to occur as a result of construction or maintenance of the stone revetment alternative.

Noise

Project construction would result in a minor, temporary increase in noise generation as a result of the use of construction equipment. After construction, the stone revetment is expected to have no impact on noise.

Unavoidable adverse environmental effects

The construction of the project would result in certain unavoidable adverse impacts on the environmental resources located within the project area. Temporary and localized adverse environmental effects that may occur during construction include: an increase in traffic, an increase in noise levels due to construction equipment, an increase of turbidity and sedimentation into water resources during construction, loss of less mobile wildlife including shellfish and other benthic organisms, and disruption of aesthetic, visual, and recreational resources.

However, implementation of the project is expected to generate numerous long-term beneficial impacts that would offset temporary adverse environmental impacts. These long-term beneficial impacts include the protection of the most vulnerable portion of the bluff area from failure, offering protection to the Turtle Hill plateau, the Lighthouse and associated structures, and other historically important resources. This protection would provide long-term protection to the socioeconomics of the area through the preservation the aesthetic, visual, historic, and recreational appeal that the project area currently offers. In addition, implementation of the project is expected to offer protection to valuable interdunal pond communities that exist along the northern shore of Montauk Point.

41. Real Estate Plan

The construction of the new revetment will require three tracts ecompassing two individual affected ownerships, namely the Montauk Historical Society (a not-for-profit educational institution that administers the Montauk Lighthouse Museum and which obtained title to same via a quitclaim deed from the United States of America dated 18 September 1998 (one tract), and the State of New York (two tracts). Reference Figures 33 and 34 for required real estate easements.

The two State-owned tracts are located along the shoreline at the base of the cliff, adjacent to either side of the Montauk Historical Society property. Approximately 1.81 acres of land is required for the revetment. In addition, approximately 2.33 acres will be required for 2 temporary work areas adjacent to the revetment. Access to the Project site will be via existing State roads (Montauk Highway) and local interior roads on either Sponsor-owned or Montauk Historical Society lands, including portions of the planned Temporary Work Areas (1.37 acres). The Sponsor will be responsible for obtaining the required real estate interests.

The project is not expected to require any facility or utility relocations, nor any relocation of displaced persons, residences, businesses or farms under the provisions of Public Law 91-646. Similarly the project does not require acquisition of real property interests for borrow areas, nor will disposal areas will be required for any purpose.

A summary of the acreage needed for the Project and the uses thereof is as follows:

Table 19 – Real Estate Summary

Interest	Acreage
Perpetual non-Standard Revetment Easement	1.81 acres
Temporary Work Area Easement:	3.70 acres
Total:	5.51 acres

Under the doctrine of "offsetting benefits" as applied to the construction of a stone revetment to protect the underlying fee owners' upland and improvements (i.e., the Montauk Point Lighthouse Complex and the adjacent State-owned lands) the value of the easement estates to be obtained and the land to be provided directly by the Sponsor is estimated to be Zero ($0) dollars. The administrative and incidental costs associated with the noted easements to be obtained is estimated to be $32,000.

Insofar as Montauk Historical Society, the landowner of the single easement tract to be acquired, holds title to its land under a Quitclaim Deed from the United States of America and is a "willing seller," no condemnations are anticipated. The landowner, Montauk Historical Society, is strongly supportive of the project.

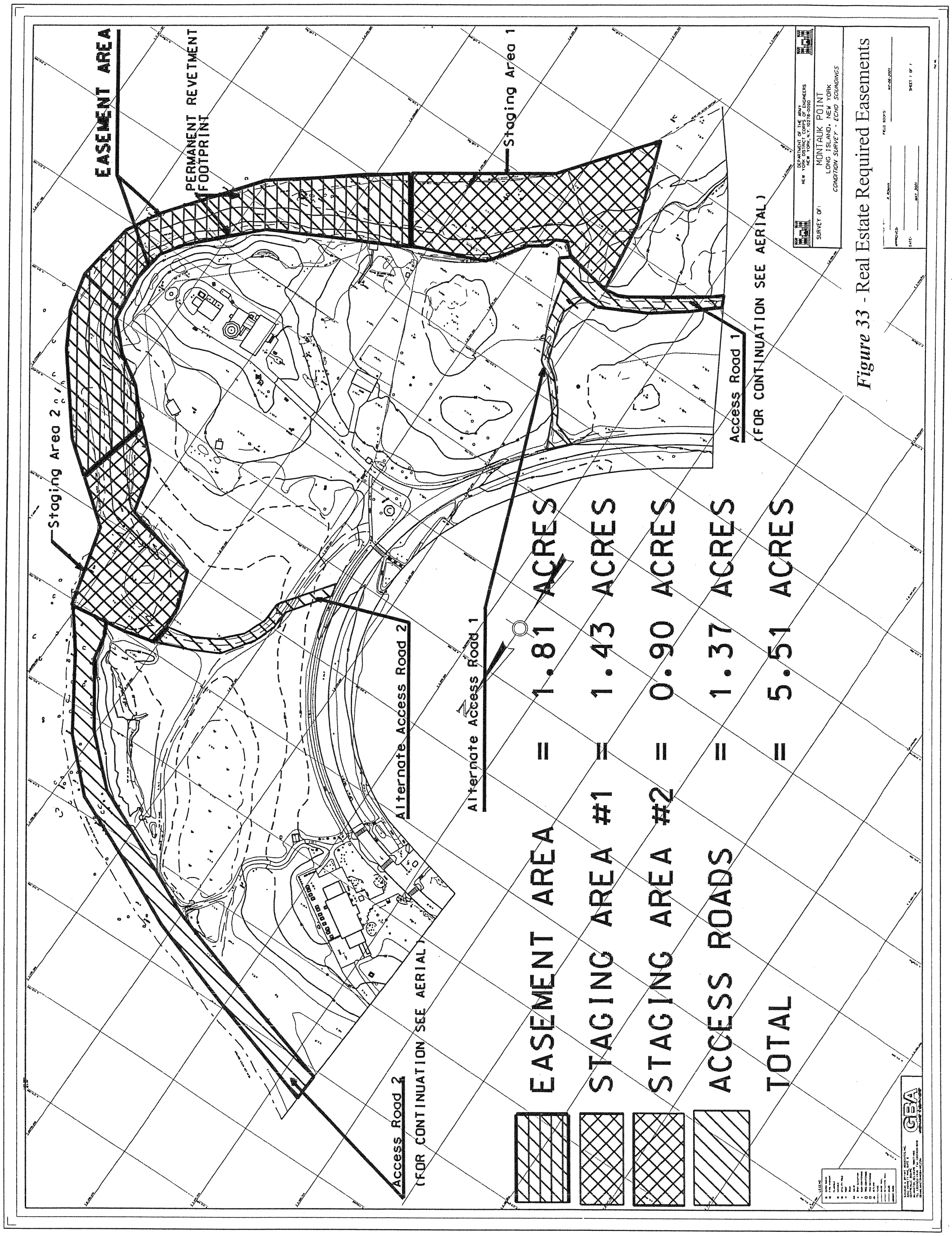


Figure 33 - Real Estate Required Easements

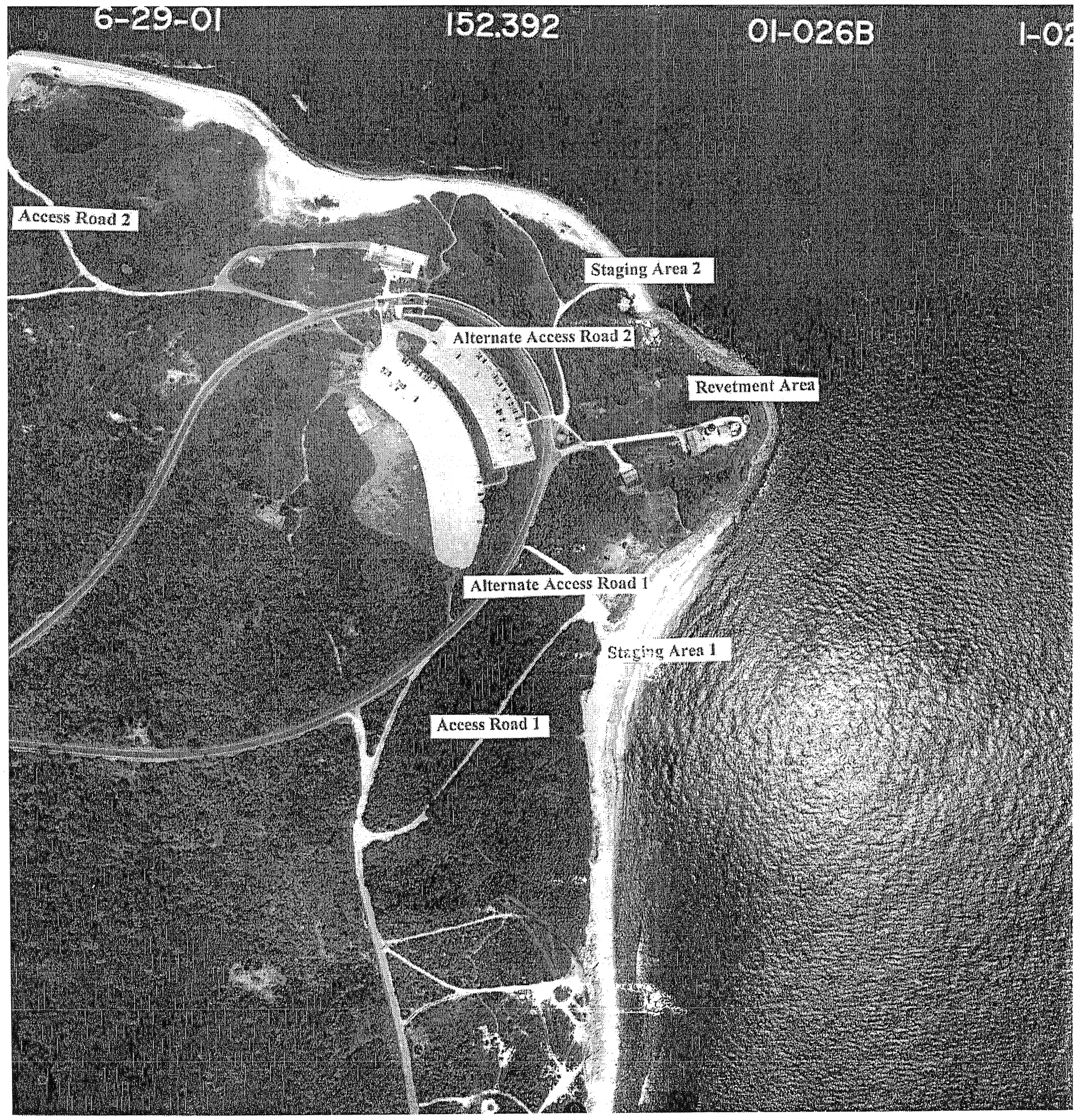


Figure 34 – Real Estate Required Easements (Aerial Map)

42. <u>Project Construction Schedule</u>*

The Design Phase (Planning, Engineering and Design) is anticipated to be initiated in February 2006 and to be completed by September 2007. The estimated time of construction is 2-years. Construction is anticipated to commence January 2008 and be completed by January 2010.

* *NOTE: The project schedule shown below assumes that Federal funding is provided by Congress, as has been done in the past.*

Completion of Feasibility Report – 6 months

August 2005	Draft Report & Draft EIS – public & agency review
October 2005	Final Report & Final EIS
December 2005	Report Approval and Authorization to Proceed PED
January 2006	Execution of Design Agreement with Sponsor

Planning, Engineering & Design Phase – 20 months

February 2006	Value Engineering
	Design Documentation Report (Engineering) Plans & Specifications Initiation - Design & Review Coordination – Environmental, Permits, Real Estate Execute Project Cooperation Agreement with Sponsor Completion of Final P&S Real Estate Acquisition
September 2007	BCO Certification

Construction Contracting Phase – 4 months

October 2007	Construction Contracting - Advertise for Bids
January 2008	Award Contract

Project Construction – 2 years

January 2008	Notice to Proceed – Initiation of Construction
	Construction of Project
January 2010	Project Completion

43. Operation & Maintenance Requirements – Non Federal Sponsor

An Operations and Maintenance Manual will be developed prior to construction, which will detail the local operations of the proposed project. As per ER 1110-2-2902, the following is presented to cover the operation, maintenance, repair, rehabilitation and replacement plan for the project: Pertaining to coastal structures:

Operation and Inspection

Insure the proper functioning of all features requiring operation or adjustment as prescribed in the operations and maintenance manual. Inspect the structures incorporated into the shore protection project (such as, but not limited to, groins, revetments, seawalls, bulkheads, breakwaters, closure structures, and sand bypassing systems) prior to the storm season, immediately following each major storm, and otherwise at intervals not exceeding 90 days. During such inspections, be certain that:

(a) Post storm condition surveys are made as required by the operations and maintenance manual.
(b) No loss, displacement, or cracking of cap stone has occured which affects the stability of the structure.
(c) No undue settlement has occurred which affects the stability of the structure.
(d) There are no encroachments upon the structure that might endanger the structure or hinder its function or repair.
(e) Care is being exercised to prevent accumulation of trash and debris adjacent to the structures.
(f) No toe scour or flanking erosion exist which may endanger stability or functioning of the structure.
(g) All drainage systems on the bluff are in good working condition.
(h) All vegetative plantings covering the bluff slope above the revetment are in good condition.
(i) No excessive loss of materials such as bedding stones, underlayer stones or armor units exist that may endanger stability or functioning of the structures.
(j) No floating plant or boats are allowed to lie against or tie up to the structures unless they are designed for such use or it is necessary for repair efforts.

Maintenance

The possibility of one coastal storm closely following another requires that coastal structures, particularly those which provide storm protection, be maintained to the extent practicable in a state of readiness. Measures to eliminate unauthorized encroachments and to effect repairs found necessary by inspection shall be undertaken immediately. All repairs shall be accomplished by methods acceptable to the District Commander or an authorized representative.

44. Local Cooperation

The NYSDEC, Montauk Historical Society and NYS Parks have been fully involved in project discussions and public meetings throughout plan formulation. A kick-off meeting was held June 2000 to introduce the project, review the study process, and perform a site visit. A public Environmental Scoping Meeting was also held November 2001. The Corps participated in many meetings throughout the study process, both formal and informal, that focused upon the problem at Montauk Point and its proposed alternatives. These meetings have been held with NYSDEC officials as well as with Federal, State and local agencies. There have been separate meetings with representatives of the Surfrider Foundation, who have opposed the project in spite of analysis for this study concluding no significant adverse effects are to be expected to surfing with the project in place.

The project sponsor, the New York State Department of Environmental Conservation (NYSDEC), the Montauk Historical Society, and NYS Parks are in full support of the selected plan of improvement. There is strong local, public and Congressional support for the project.

The project sponsor is prepared to execute a Design Agreement, for the completion of the plans and specifications phase, which will reflect the recommendations of this Feasibility Report.

The project sponsor shall be required to comply with all applicable Federal laws and policies and other requirements. A fully coordinated Project Cooperation Agreement (PCA) package (to include sponsor's financing plan) will be prepared subsequent to the approval of the feasibility phase, which will reflect the recommendations of the Feasibility Study. The non-Federal sponsor has indicated support of the recommendations presented in this Feasibility Report and the desire to execute a PCA for the recommend plan.

The local sponsor shall be required to:

(1) Enter into an agreement which provides, prior to construction, 25 percent of pre-construction engineering and design (PED) costs;
(2) Provide, during the first year of construction, any additional funds needed to cover the non-federal share of PED costs;
(3) Provide all lands, easements, and rights-of-way and perform or ensure the performance of any relocations determined by the Federal Government to be necessary for the initial construction, periodic nourishment, operation, and maintenance of the project.
(4) Provide, during construction, any additional amounts as are necessary to make its total contribution equal to 50 percent of initial project costs assigned to storm damage.
(5) For so long as the project remains authorized, operate, maintain and repair the completed project, or functional portion of the project, at no cost to the Federal Government, in a manner compatible with the project's authorized purposes and in accordance with applicable Federal and State laws and regulations and any specific directions prescribed by the Federal Government.

(6) Give the Federal Government a right to enter, at reasonable times and in a reasonable manner, upon property that the Non-Federal Sponsor, now or hereafter, owns or controls for access to the project for the purpose of inspecting, operating, maintaining, repairing, replacing, rehabilitating, or completing the project. No completion, operation, maintenance, repair, replacement, or rehabilitation by the Federal Government shall relieve the Non-Federal Sponsor of responsibility to meet the Non-Federal Sponsor's obligations, or to preclude the Federal Government from pursuing any other remedy at law or equity to ensure faithful performance;

(7) Hold and save the United States free from all damages arising from the initial construction, periodic nourishment, operation, maintenance, repair, replacement, and rehabilitation of the project and any project-related betterments, except for damages due to the fault or negligence of the United States or its contractors;

(8) Keep and maintain books, records, documents, and other evidence pertaining to costs and expenses incurred pursuant to the project in accordance with the standards for financial management systems set forth in the Uniform Administrative Requirements for Grants and Cooperative Agreements to State and Local Governments at 32 Code of Federal Regulations (CFR) Section 33.20;

(9) Perform, or cause to be performed, any investigations for hazardous substances that are determined necessary to identify the existence and extent of any hazardous substances regulated under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), Public Law 96-510, as amended, 42 U.S.C. 9601-9675, that may exist in, on, or under lands, easements, or rights-of-way that the Federal Government determines to be required for the initial construction, operation, and maintenance of the project. However, for lands that the Federal Government determines to be subject to the navigation servitude, only the Federal Government shall perform such investigations unless the Federal Government provides the Non-Federal Sponsor with prior specific written direction, in which case the Non-Federal Sponsor shall perform such investigations in accordance with such written direction;

(10) Assume complete financial responsibility for all necessary cleanup and response costs of any CERCLA regulated materials located in, on, or under lands, easements, or rights-of-way that the Federal Government determines to be necessary for the initial construction, periodic nourishment, operation, or maintenance of the project;

(11) Agree that the Non-Federal Sponsor shall be considered the operator of the project for the purpose of CERCLA liability, and to the maximum extent practicable, operate, maintain, and repair the project in a manner that will not cause liability to arise under CERCLA;

(12) If applicable, comply with the applicable provisions of the Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970, Public Law 91-646, as amended by Title IV of the Surface Transportation and Uniform Relocation Assistance Act of 1987 (Public Law 100-17), and the Uniform Regulations contained in 49 CFR Part 24, in acquiring lands, easements, and rights-of-way, required for the initial construction, periodic nourishment, operation, and maintenance of the project, including those necessary for relocations, borrow materials, and dredged or excavated material disposal, and inform all affected persons of applicable benefits, policies, and procedures in connection with said Act;

(13) Comply with all applicable Federal and State laws and regulations, including, but not limited to: Section 601 of the Civil Rights Act of 1964, Public Law 88-352 (42 U.S.C. 2000d), and Department of Defense Directive 5500.11 issued pursuant thereto; Army Regulation 600-7, entitled "Nondiscrimination on the Basis of Handicap in Programs and Activities Assisted or Conducted by the Department of the Army"; all applicable Federal labor standards requirements including, but not

limited to, 40 U.S.C. 3141-3148 and 40 U.S.C. 3701-3708 (revising, codifying and enacting without substantial change the provisions of the Davis-Bacon Act (formerly 40 U.S.C. 276a *et seq.*), the Contract Work Hours and Safety Standards Act (formerly 40 U.S.C. 327 *et seq.*) and the Copeland Anti-Kickback Act (formerly 40 U.S.C. 276c *et seq.*)); and Section 402 of the Water Resources Development Act of 1986, as amended (33 U.S.C. 701b-12), requiring non-Federal preparation and implementation of flood plain management plans;

(14) Provide and maintain necessary access roads, parking areas, and other public use facilities, open and available to all on equal terms;

(15) Recognize and support the requirements of Section 221 of Public Law 91-611, Flood Control Act of 1970, as amended, and Section 103 of the Water Resources Development Act of 1986, Public Law 99-662, as amended, which provides that the Secretary of the Army shall not commence the construction of any water resources project or separable element thereof, until the non-Federal sponsor has entered into a written agreement to furnish its required cooperation for the project or separable element; and

(16) Do not use Federal funds to meet the non-Federal sponsor's share of total project costs unless the Federal granting agency verifies in writing that the expenditure of such funds is expressly authorized by statute.

45. Financial Analysis of the Non-Federal Sponsor

The New York State Department of Environmental Conservation (NYSDEC) has stated its intention to act as the non-Federal partner and has requested that funds for design be included in the upcoming New York State Budget. NYSDEC has successfully served as the non-Federal partner on numerous projects within the New York District. In view of their past performance as a partner, it is the assessment of the District that the NYSDEC has more than adequate financial capability to fund its obligation for project construction.

46. Conclusion & Recommendations

Conclusion: If allowed to continue, progressive instability of the Montauk Point bluff would result in the irrecoverable loss of the historic Montauk Point Lighthouse and its associated structures, along with archaeological resources. The implication would be the total loss of all historical properties, both buried and above ground. Once this information is lost, it can never be recovered, and future study of the complex would be impossible. The alternative plans developed for this feasibility report are superior to the no action plan as they provide substantial storm damage protection.

The alternative plans included five significantly different measures: stone revetment, offshore breakwater with beach fill, T-groins with beach fill, beach fill, and relocation of the lighthouse. The stone revetment is the most reliable and cost effective structural solution. Because of the steep terrain in the area, the cost of relocation is prohibitive. In addition, relocation would have adverse effects on the surrounding archeological resources, would degrade existing habitats and historic views, and also effect recreational use of the area. Also, a replacement light tower would have to be constructed, as the lighthouse, in its current location, continues to serve as a functioning aid to navigation.

Therefore, the selected plan consists of the construction of a stone revetment with a 73-year storm design (Alternative Plan 2B). This level of design was chosen based on an economic optimization of a wide range of designs to reduce the risk of losses due to storm damages.

- Stone revetment, 840-feet in length, with a crest width of 40-feet at elevation +25 feet NGVD, and 1V:2H side slopes.

- 12.6-ton quarrystone armor units extending from the crest down to embedded toe.

- Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure.

- The bottom of the armor stone layer in the toe is located at a depth of 12-feet from the existing bottom.

- A heavily embedded toe is incorporated to protect against breaking waves, provide long-term stone stability, and scour at the toe of the structure. Stone sub-layers are specified in accordance with standard Corps design procedures.

The selected NED plan is also the locally preferred plan. The local sponsor, New York State Department of Environmental Conservation is willing to provide all items of local cooperation, and is in full support of the selected plan.

The proposed work will have no significant impact on the quality of the environment in the project area. Special consideration was given to the effects of the selected plan on fishing, surfing, and cultural experiences. Most impacts associated with this project will be temporary, and none of the impacts are regarded as significant.

The land that will be protected by implementation of this recommended project is deeded to the Montauk Historical Society (MHS). The MHS is a private, not for profit association that is not part of any state or local government. This land is held open, for use by all on equal terms, regardless of origin or home area. Existing Corps policy indicates that there is no Federal interest in protection of a property owned by a single private non-profit entity.

However, although the MHS is clearly a single, private landowner, they must, by deed restriction and State charter, act as a public entity akin to agencies of State and local governments. The MHS must accomplish a public education mission to stay in operation, must follow Federal National Historic Preservation requirements for maintenance work, and membership and enjoyment of the benefits of the facility and educational programs are open to all, with no restriction, for a fee. Under the deed and charter, the MHS cannot structure and constrain uses of the property, nor can anyone who cares to join the MHS and enjoy the benefits of the facility and water resources project be excluded.

In light of these facts, New York District requested a waiver to the single landowner policy from the Assistant Secretary of the Army (Civil Works) and was granted an exception allowing the completion of the feasibility study with a view towards pursuing a cost-shared construction project for Montauk Point, New York.

The first cost of the selected plan is estimated to be $13,722,000 at October 2004 price levels. The total benefits attributed to this selected plan are estimated at $1,578,700 while the annual costs are $889,300. Therefore, the benefit to cost ratio is 1.8 to 1, with total net benefits of $689,400.

The cost-sharing for construction of this storm damage reduction project is as follows:

50% Federal Share	$6,861,000
50% Non-Federal Share	$6,861,000
Total Project First Cost	$13,722,000

An annual revetment maintenance cost of $52,300 will be a 100% Non-Federal expense.

Recommendations: I have reviewed and evaluated, in light of the public interest, the information related to storm damage reduction at Montauk Point, New York. I find that the selected NED plan of improvement, the stone revetment, as developed in this report is based on a thorough analysis and evaluation of the various practical alternative courses of action for achieving this project's objectives.

I recommend authorization of the selected stone revetment plan for Montauk Point, with such modifications thereof as in the discretion of the Commander, HQUSACE, as may be advisable.

The recommendations contained herein reflect the information available at this time and current departmental policies governing formulation of individual projects. They do not reflect program and budgeting priorities inherent in the formulation of a national civil works construction program, nor the perspective of highest review levels within the Executive Branch. Consequently, the recommendations may be modified before they are transmitted to the Congress as proposals for authorization and implementing funding. However, prior to transmittal to Congress, the sponsor, interested Federal agencies, and other parties will be advised of any modifications and will be afforded an opportunity to comment further.

Richard J. Polo, Jr.
Colonel, U.S. Army
District Engineer

APPENDIX A: ENGINEERING & DESIGN

Storm Damage Reduction - Feasibility Study
Montauk Point, New York

DRAFT ENGINEERING AND DESIGN
APPENDIX A

Prepared For:

U.S. Army Corps of Engineers
New York District

October 2005

Table Of Contents

List of Tables

List of Figures

List of Sub-Appendices

Erosion Control Feasibility Study
Montauk Point, New York

1. Description of Project Area and Vicinity

A-1 Montauk Point is located in Suffolk County, approximately 125 miles east of New York City. The point separates the Atlantic Ocean to the south from Block Island Sound to the north. (Figure A-1). The Montauk Point Lighthouse acts as a junction marker for ships headed for New York Harbor or Long Island Sound.

A-2 The Montauk Point Light Station was authorized for construction in 1792 by President George Washington. Construction was initiated in June 1796 and completed in November 1796 at a cost of $22,300. The lantern is about 80 feet above the ground. The lantern was lit with sperm oil until the 1860's, kerosene until the 1940's and, finally, electricity with a 300,000-candlepower lamp.

A-3 When the light was completed it was 300 feet from the edge of the cliff. Presently the lighthouse is less than 120 feet from the edge of the bluff and other major structures are within 50 feet of the bluff edge. Continued erosion has been recognized as a problem for many decades and various efforts have been made to stabilize the shoreline with varied success.

A-4 The study area includes the historic Montauk Point Lighthouse that sits on a high bluff underlain with glacial till, approximately 70 feet above Mean Sea Level (MSL). The study area includes steep slopes and shorelines surrounding the bluff, detailed in Figure A-2.

A-5 The critical area of study consists of the bluff from the southwest side of the point to the northwest side of the point, covering about 900 feet of shoreline. The ownership of most of the property was recently transferred from the U.S. Coast Guard to the Montauk Historical Society, surrounding property owned by New York State. The reader is referred to the Real Estate Appendix for further information on ownership of the lands and restrictions to uses.

A-6 The bluff and beach along this entire area are considered to be critical elements of the stability of the lighthouse. Erosion control structures are required to protect the bluff faces from the forces of oncoming waves.

A-7 As with any coastal project, updrift and downdrift areas need to be examined and considered in the formulation of a shore protection plan. In this case, it is estimated that the area of concern consists of 2300 feet of shoreline, extending from the pivotal point of shoreline orientation of the adjacent bluff to the south to a beach area to the north. The entire area must be considered in order to prevent adverse impacts from this project.

Montauk Point, New York
Site Location

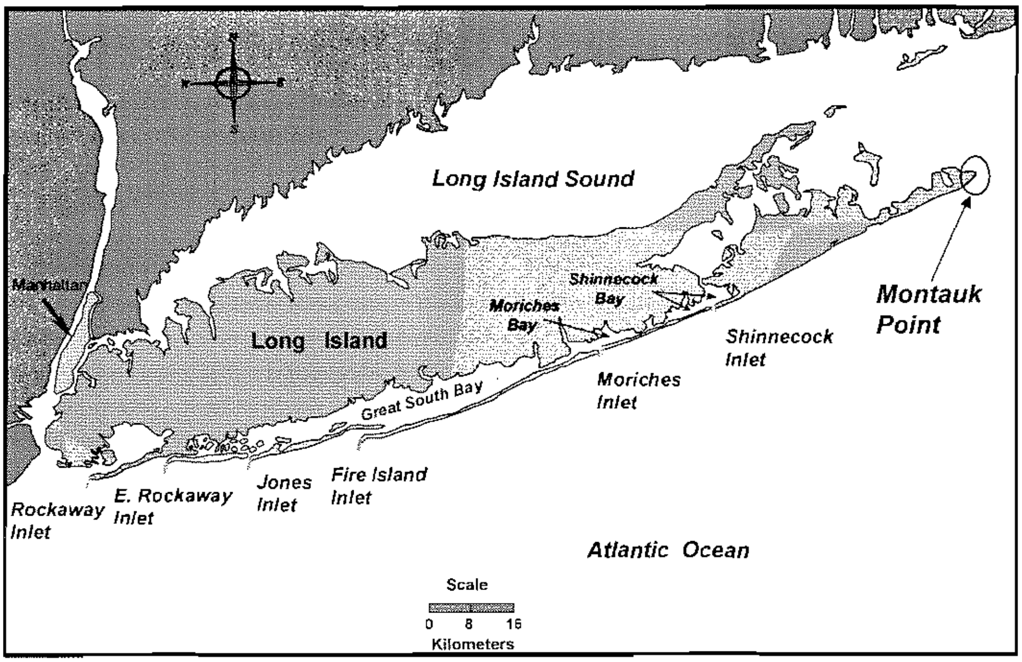
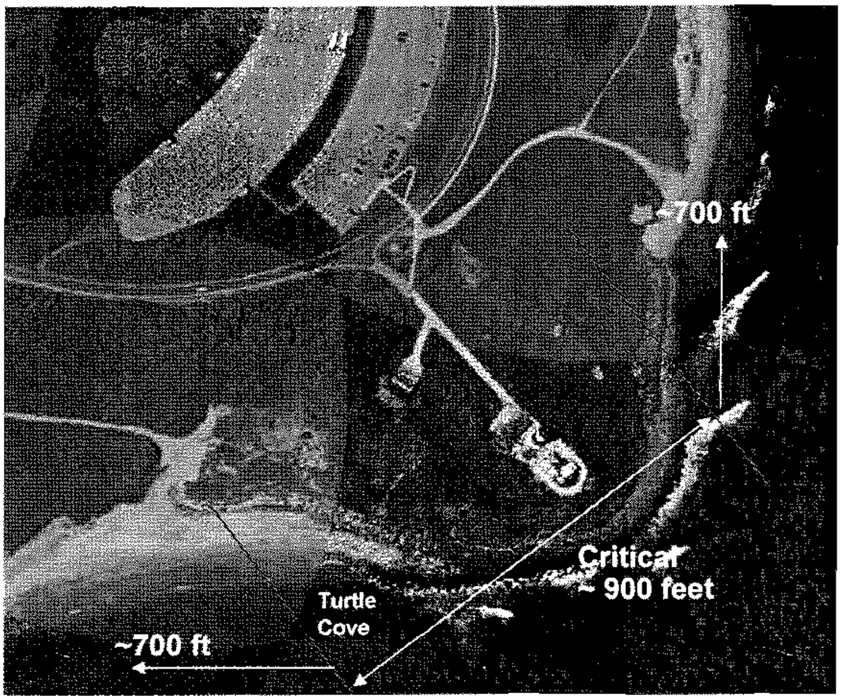


Figure A-1. Site Location



Study Area

Figure A-2. Study Area

2. Coastal History and Status of Project Area

A-8 A brief history of the shore protection treatments is as follows:

1792 The lighthouse is authorized by George Washington on land previously used by Montaukett Indians. The shoreline is approximately 200 feet seaward of the present (2001) position

1946 A 700-ft stone revetment is constructed at the bluff toe, with vegetative plantings along the upper half of the cliff (New York District, 1944). The crest elevation is +20 ft MSL, tapering down to +15 ft MSL at both ends. The crest width is 23 feet with a core and double armor layer of 4 to 8 ton stone. The base layer is 8 ton stone. Since its construction, this entire seawall has completely failed and is now 10 to 70 feet seaward of the existing bluff toe. Most of the stone is at an elevation of about mean high water, with remnants present as rubble along the southern extent of the present structure toe.

1960's Department of Transportation places rubble over the edge of the bluff just to the south of the lighthouse. After the October 1991 storm, the rubble slides down the slope due to scouring of the bluff toe. Most of the rubble is subsequently cleared away during the construction of the revetment in 1992 (see below).

1971 The first terracing project is constructed along the bluff slope by Ms. Georgina Reid. The construction is on U.S. Coast Guard property just north of the lighthouse.

1972 U.S. Coast Guard places gabions along about 280 feet of the point above the failed 1946 seawall along the toe of the bluff. The gabion system subsequently settles gradually and the crest is of insufficient elevation (only up to about +15 feet MSL) to provide protection. It is significantly damaged by the Halloween Storm of 1991.

1980's Terracing and beach grass plantings continue through the 1970's and 1980's. The vegetation includes beach grasses, bushes, seedlings, shrubs and wildflowers up to five feet in height. Dense foliage occupies most of the north end of the point. The lower east side of the bluff is reshaped to a more stable angle, terraced with lumber and secured by steel stakes to provide a flat surface for the beach grass. The vegetation appears to hold the bluff face against the forces of ground seepage, rainfall and runoff. Terracing efforts subsequently deteriorate due to the impacts of major storms in the early 1990's.

1990 The Montauk Historical Society and the New York State Department of Parks and Recreation construct a revetment along Turtle Cove, south of the lighthouse. A 6-ft deep, 15-ft wide trench is excavated for the toe of 263 lineal feet of revetment. Geotextile fabric is placed in the trench and a base layer of 50-pound stone is placed on the fabric. Up to 14,000 pound stones are placed on the base stone up to an elevation of +20 feet MSL. The revetment subsequently settles to a crest elevation of +5 to +10 feet MSL during the October 1991 storm and is no longer adequate as a shore protection structure.

1992 After severe erosion due to Hurricane Bob and the Halloween Storm of 1991 (The Perfect Storm), a new revetment is constructed by the U.S. Coast Guard landward of the old revetment. An emergency construction effort commences along about 300 feet of shoreline. The crest elevation is +25 feet MSL, with 1-3 ton stones placed on the slope above a 14 foot wide berm crest at elevation +18 MSL of a single 10 ton armor layer, which slopes down to the existing toe (generally on stone from the 1946 failed revetment). The Montauk Historical Society constructs a 150 foot long structure along the eastern section of Turtle Cove. The design is similar to the Coast Guard section but 5 to 10 ton stone is used.

1993 New York District Reconnaissance Study determines sufficient economic justification and Federal interest to continue study.

Because the present shore protection measures, (somewhat similar to the 1946 revetment that failed), were not designed to withstand significant storm events over a substantial duration, i.e. lack of a buried toe, inadequate stone size, and insufficient overtopping protection, it is expected that the revetment now in place will fail in the foreseeable future.

3. Existing Drainage

A-9 Montauk Point Lighthouse is located on a knoll with the surrounding topography sloping away from the lighthouse steeply. The site consists predominately of vegetative cover with some pavement and roof areas. The site is well vegetated and contains slopes of up to 40 percent grade. Slope lengths are short and show little sign of past erosion.

A-10 A site reconnaissance was done with Greg Donohue, Erosion Control Specialist of the Montauk Historical Society, to locate and assess the effectiveness of known drainage facilities. Drainage facilities at the site consist of roof drains, a slotted drain and bluff terraces.

A-11 The site can be divided into three primary drainage areas. The first area is the bluff area surrounding the lighthouse. Runoff from this area flows over the bluff to the Atlantic Ocean. The second area is located south of the lighthouse between the bluff and the concrete driveway leading to the lighthouse. Runoff from this area flows southwest towards the Atlantic. The third area is located north of the lighthouse driveway and runoff from this area flows north towards Long Island Sound.

A-12 The current surface drainage pattern is illustrated on Figure A-3. Sources of runoff at the site include lawn areas, building roofs and paved areas. The site contains minimal facilities for the collection and conveyance of storm water.

A-13 Runoff from the lawn areas flows to the Atlantic Ocean via uncontrolled overland flow. No conveyance channels are utilized in directing runoff from lawn areas to specific discharge points. Since most of the slopes within the lawn area are relatively short, runoff can be expected to exist in the form of sheet flow. However, due to the vegetated condition of the unimproved areas of the site, runoff velocities are low enough to prevent rills from developing on sloped areas.

A-14 The bluff has been terraced and vegetated to reduce the erosion of the bluff face. In addition to the vegetated terraces, rock outlets have been constructed in areas prone to concentrated flow conditions due to natural drainage patterns or groundwater discharge.

A-15 Roof drains from the museum have outlets along the slopes surrounding the structure. Although a source of concentrated flow, the roof drain outlets do not appear to cause adverse impacts to the grassed slopes. The roof drains are open and free of sod buildup at the outlet points. Two outlet points, consisting of 4-inch PVC pipe, are located on the north side of the museum. A third discharge point, located on the south side of the museum, discharges water from the roof drains on the south side of the building. Additionally, some of the roof drains discharge to the lawn area without being conveyed away from the buildings with discharge pipes.

A-16 Roof drains from the communications tower outlet to cisterns located on the south side of the building. It could not be determined, through observation and interviews with museum personnel, where the discharge point for the cisterns is located. It is assumed that the cisterns tie into the discharge for the roof drains on the south side of the museum.

A-17 East of the lighthouse a four-inch diameter drain is located on the concrete apron between the lighthouse and the communications tower. Although the capacity of this type of drain is low, excess runoff produced by large rainfall events can overtop the drain and discharge to the lawn area. No signs of erosion due to this anticipated condition were evident during the site reconnaissance.

A-18 Runoff from the concrete driveway leading to the lighthouse is contained within concrete curbs and is directed to a 3-inch slotted drain (trench drain) near the admissions booth. The discharge point for the slotted drain was not visible due to heavy vegetation. Regardless, this drain is insufficient to handle the amount of runoff from the concrete driveway. Additionally, the slotted drain is clogged with dirt and debris and appears to be nonfunctional. Evidence of an existing erosion channel was observed north of the slotted drain. This erosion channel is located between the walking path that leads to the beach and the fence that surrounds the site. The channel is currently obscured by dense brush, which may aid in stabilizing the gully. It is expected that during large rainfall events the area near the admissions booth will become inundated with water. This condition can lead to concentrated flow conditions that may produce an erosion channel.

A-19 An analysis of runoff potential was conducted to assess the adequacy of the slotted drain (Sub-Appendix A-2). Runoff potential was compared to the assumed capacity of the existing drain. It was found that the slotted drain is capable of handling runoff from a ten-year rainfall event. The adequacy of the drain is contingent on the proper maintenance of the structure. Due to the condition of the drain it was assumed that the inlet capacity of the grate controls the overall capacity of the drain. Manufacturer data approximating the configuration of the in-place drain was used to estimate the capacity of the drain.

A-20 Generally, the drainage facilities at the site appear to be adequate and cause no adverse impacts to the surrounding area. Little evidence of past erosion was observed at the site. Routine maintenance of the drainage facilities and vegetation is needed to prevent occurrence of erosion in the future.

A-21 Routine maintenance of the drain is needed to prevent clogging. Replacement of the slotted drain with a structure less prone to clogging, such as a shallow catch basin, is advisable. A replacement catch basin could be outlet in the existing erosion gully. It is recommended that a rock apron or other energy dissipating devise be installed at the end of the outlet pipe to prevent additional erosion caused by concentrated flow. Maintenance of vegetation is important for continued drainage control in all areas subject to runoff. However, these drainage improvements are beyond the scope of this project since it relates to a surface runoff problem that does not adversely affect the proposed improvements.

A-22 The drainage capacity is not in need of upgrading for events greater than a 10-year return period because the combination of the 10-year event drainage capacity and the infiltration rate of the sandy soil have historically prevented any serious erosion from happening during events with return periods greater than 10 years.

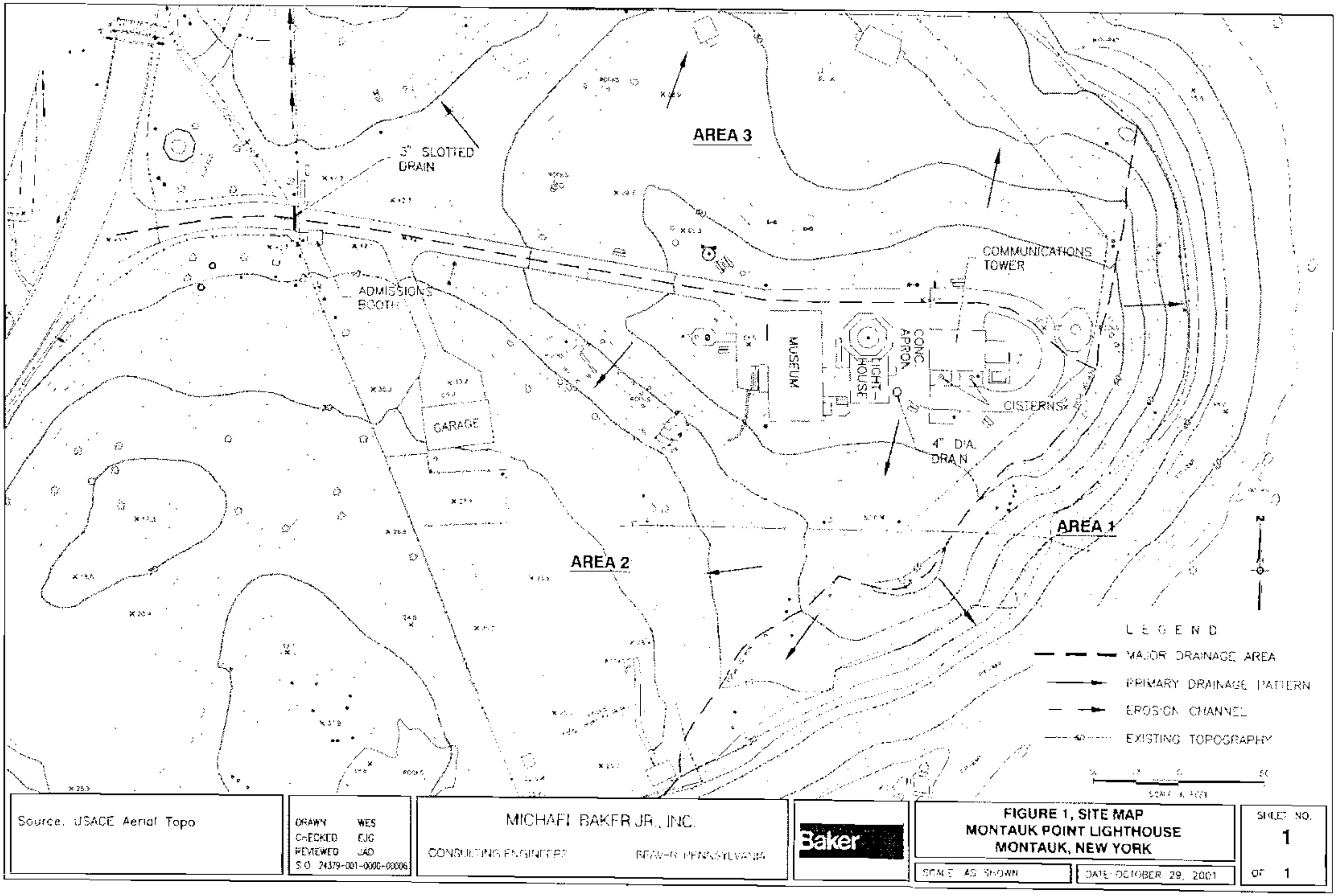


Figure A-3. Site Plans and Drainage

4. Geotechnical Investigation

4.1 Subsurface Exploration Program

A-23 A subsurface exploration program was conducted at Montauk Point lighthouse to assess the subsurface conditions of the site. Test borings were advanced using hollow stem augers in conjunction with split spoon sampling. Standard penetration testing in accordance with ASTM D1586 was performed by recording blow counts on the split spoon sampler. Correlation between the number of blows required to drive the sampler one foot and soil strength parameters can be made.

A-24 Three test borings were proposed for the subsurface exploration at the site. Two borings, intended to be advanced to a depth of 85 feet, were located atop the bluff in the vicinity of the lighthouse. These two borings were to be advanced using continuous split spoon sampling in the top ten feet and split spoon sampling on five-foot centers to the termination depth. A third boring, on the beach area, was to be advanced to a depth of 20 feet using continuous split spoon sampling. The borings were logged with respect to blow counts and soils classified according to USCS visual-manual classification methods ASTM D2488.

A-25 Test Boring TB-1 was drilled northeast of the lighthouse between the communications tower and the bluff. The initial attempt in advancing this boring was met with refusal at 23 feet. The boring was relocated approximately 10 feet west and attempted again. The second attempted reached a depth of 31 feet before meeting refusal. Refusal was likely due to cobbles, boulders, or dense gravel. Following the second attempt it was decided to move to the second boring.

A-26 Test boring TB-2 was drilled just southeast of the lighthouse tower. The boring was advanced to at depth of 49.5 feet before encountering refusal. Refusal was defined as less than 0.1 foot of spoon advance for greater than 100 blows. The boring was then relocated approximately 15 feet west and attempted again the boring was advanced to 41 feet. The relocated boring was advanced to 41 feet.

A-27 Boring TB-3 was proposed near the toe of the bluff southwest of the lighthouse location. Six attempts were made to advance this boring to the termination depth. The large amount of cobbles and boulders contained in the beach sand prevented the boring from being advanced more than 2.5 feet in any of the attempted locations.

A-28 Test boring logs are shown in Sub-Appendix A-3 including log locations.

A-29 Laboratory testing including sieve analysis (ASTM D1140), Atterberg Limits (ASTM D4318), gradation (ASTM C136) and moisture content (ASTM D2216) were completed in a geotechnical laboratory on two sets of samples. The first set were spoon samples taken from borings TB-1 and TB-2 in the soils above the glacial till. The second set was bag samples taken during the geophysical investigation from the till exposed in eroded faces at Montauk. The test results are shown in Sub-Appendix A-3.

4.2 Geophysical Investigation

A-30 To supplement the drilling program, provide a continuous profile across the bluff, and assist in estimating soil conditions beneath the revetment, NDT Engineering, Inc performed a geophysical study on November 27, 2001. The geophysical study utilized seismic refraction and electrical resistively profiling methods to provide a continuous profiling of subsurface layers including the Montauk till surface and the groundwater table.

A-31 The geophysical survey consisted of placing two seismic lines on the site and producing energy waves with a seis-gun device. The velocity of the seismic waves was recorded with a seismograph device linked to a series of geophone receivers spaced evenly along the length of the seismic line.

A-32 A copy of the NDT report with seismic results is included in Sub-Appendix A-6. It should be noted that NDT presents data relative to surface not NGVD. A plot of the seismic results converted to elevation is also in Sub-Appendix A-6.

A-33 The refraction data indicate a velocity contrast at a depth of approximately 50 feet on both lines SL-1 and SL-2. This contrast was interpreted as the interface between an upper layer of sand and gravel and a lower layer of relatively compact glacial till. The resistivity data also indicated a contrast at this depth and is interpreted as the presence of a true or perched water table near the upper surface of the till. Two thin surface layers were also revealed by the resistivity data, interpreted as relatively dry sand and gravel. Underlying this material, but above the water table, is sand and gravel with some silts and clays, containing sufficient moisture to decrease the resistivity. The lowest layer, coincident with the till layer shown by the seismic data, indicates the influence of increased water content. The top of the till is indicated by a jump in velocity from 1600 to 6000 fps indicating an increase in soil density.

A-34 The interpretation of the resistivity data at the top of the till appears to be inconsistent. Although the resistivity interface appears at the same depth for both lines and coincides with the top of the till, it is opposite in magnitude. That is, the change from the upper layer to the lower layer is from 106 ohm-meters to 90 ohm-meters on Line SL-1 and from 31 ohm-meters to 133 ohmmeters on Line SL-2.

A-35 These results confirm the basic layering scheme presented in the USACE *Reconnaissance Report: Montauk Point, New York*, 1993. That report described a three layer model, consisting of Montauk Till at the base, overlain by (lower) stratified Hempstead Gravel (composed of distinct strata of sand, silt and clay) and a surface layer (upper) Hempstead Gravel (composed of cohesionless fine sand with little silt). The Montauk Till is contained within a complex of deposits referred to as the "lower drift" in *Eastern Long Island Geology*, by Les Sirkin, 1995. The Hempstead Gravel is presumed to be a member of the "upper drift". The significance of this is that the lower drift was extensively deformed by subsequent glacial activity. All of Turtle Hill, on which the lighthouse stands, is a slump block that remained after the retreat of the glacier. Furthermore, the Hempstead gravel would be expected to contain "rip-ups" or inclusions of the lower drift, considerably complicating the stratigraphy and distorting the contact between the two drift deposits. Therefore, the somewhat uneven contact shown in the refraction results is not unexpected. The presence of "rip-ups", seen in the beachfront cliffs on site, further complicates interpretation of all subsurface information. This may not be, then, just a simple case of two or three horizontal layers.

A-36 For stability modeling, however, the interface revealed by the refraction results, combined with a presumed water table indicated by the resistivity model, should be adequate for the current level of investigation. The soil unit parameters described under "Stability Analysis" are reasonable engineering properties based on current knowledge. The cementation noted in gathering the samples is reported as a phenomenon of "case hardening" by salt rinds upon exposure to sea spray and is not inserted into the stability model.

5. Erosion

5.1 Processes

A-37 Bluff erosion is caused by a number of forces. At the toe of the bluff, erosional forces include:

- Astronomical and storm tides that allow waves and tidal currents to gradually erode the toe of the bluffs that were exposed with no underlying stone.
- Waves and currents that serve to mobilize and transport sediments away from the shoreline. As the bluff toe erodes and steepens, the upper bluff collapses and slides into the ocean. There is also a net loss of beach material due to littoral transport.

A-38 Erosion forces also act on the upper parts of the bluff. These sources of erosion include:

- Water collecting in upland wetlands and ponds and then seeping slowly toward the sea, both on the surface and through the soil. Seepage exits on the face of the bluffs, further loosening and moving soil down the bluff face.
- Wave spray and runup eroding the bluff face by saturating and washing away sediment.
- Rain eroding the sloped bluff face during storms by impinging upon the sediment and washing out large amounts of soil to the beach below. A lack of vegetation on the bluff face could allow the rain and surface water to act directly on the soil. Because of adequate vegetation on the bluffs at Montauk Point, this is not presently happening but could occur in the future if plant cover decreases.
- High coastal winds, which add to the erosion process. Winds will blow loose soil from the face of the bluffs and will cause trees and taller vegetation to sway back and forth, which in turn loosen the soil at their base.

5.2 Storm-Induced Erosion Rates

A-39 Because of the steep slopes and high elevations associated with the bluffs at Montauk Point, storms can cause some bluff failure and erosion of soil. In the 1993 Reconnaissance Report, a site survey described erosion measurements that were made in June 1992. The survey indicated that the unprotected (beach fronted) bluff immediately to the north of the lighthouse eroded 20 feet and the unprotected (beach fronted) bluff 800 feet north of the lighthouse receded about 30 feet during the October 1991 storm.

A-40 Long term erosion rates along two cross sections using reported historic shorelines and aerial photography were analyzed (Figure A-4). The data was plotted in cross-section view (Figure A-5) and averaged as shown in Figure A-6. The historical long-term shoreline recession rate was found to be 2.2 feet per year for the beach and bluff toe and 1.2 foot per year for the top of the bluff (New York District, 1993). In the past 125 years of record, the bluff has receded 150 feet and beach has receded about 330 feet. Erosion rates since 1993 in critical areas of erosion are not pertinent due to the construction of a successfully performing revetment at this region which is curtailing shoreline retreat. It has been estimated that the average annual erosion rate for the bluff and beach is 6 cubic yards per foot per year, resulting in a total of 5,000 cubic yards of erosion per year in the critical erosion area. The historic data shows that the beach recession rate adjacent to the revetment has been reduced by about 50% since the construction of coastal structures, whereas the bluff recession has stabilized at about 25% of the pre-1945 revetment recession rate due to the terracing construction above the revetment.

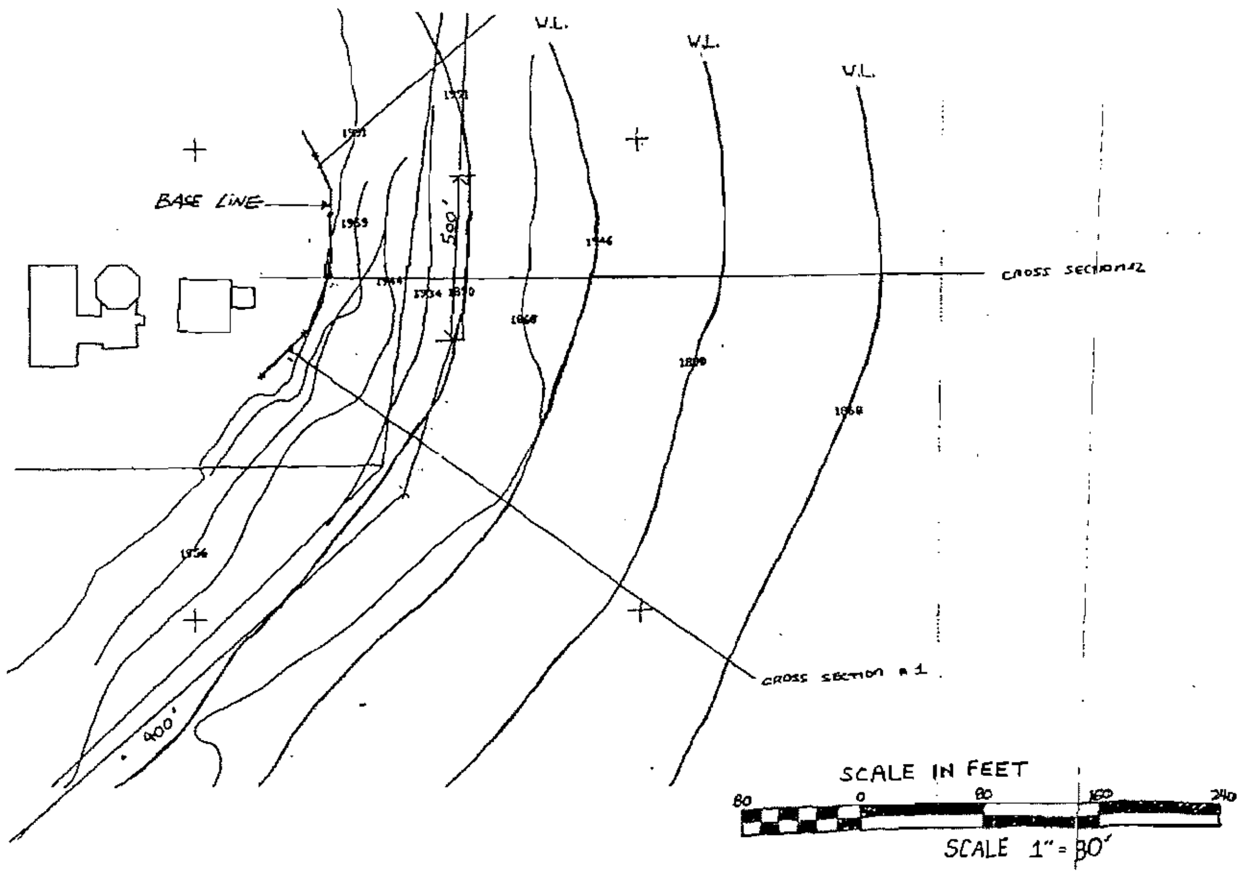


Figure A-4. Shoreline Changes 1865-1992 (New York District, 1993).

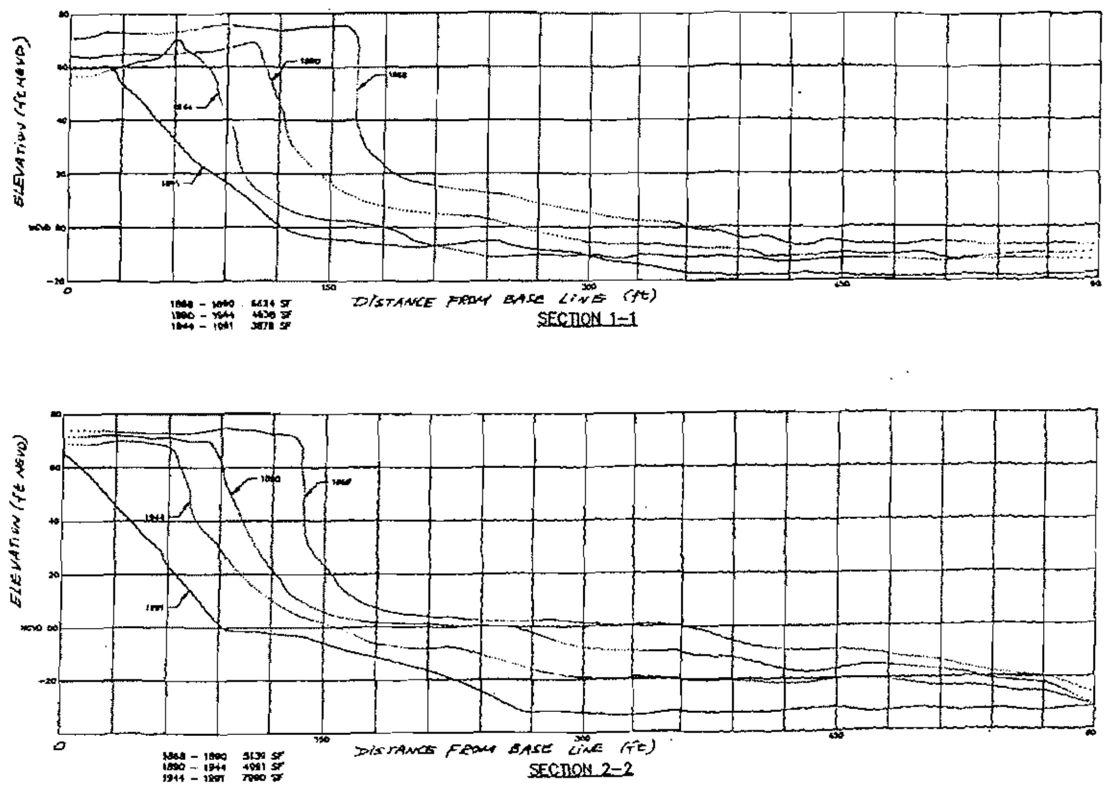


Figure A-5. Shoreline Changes along cross sections shown in Figure A-4.

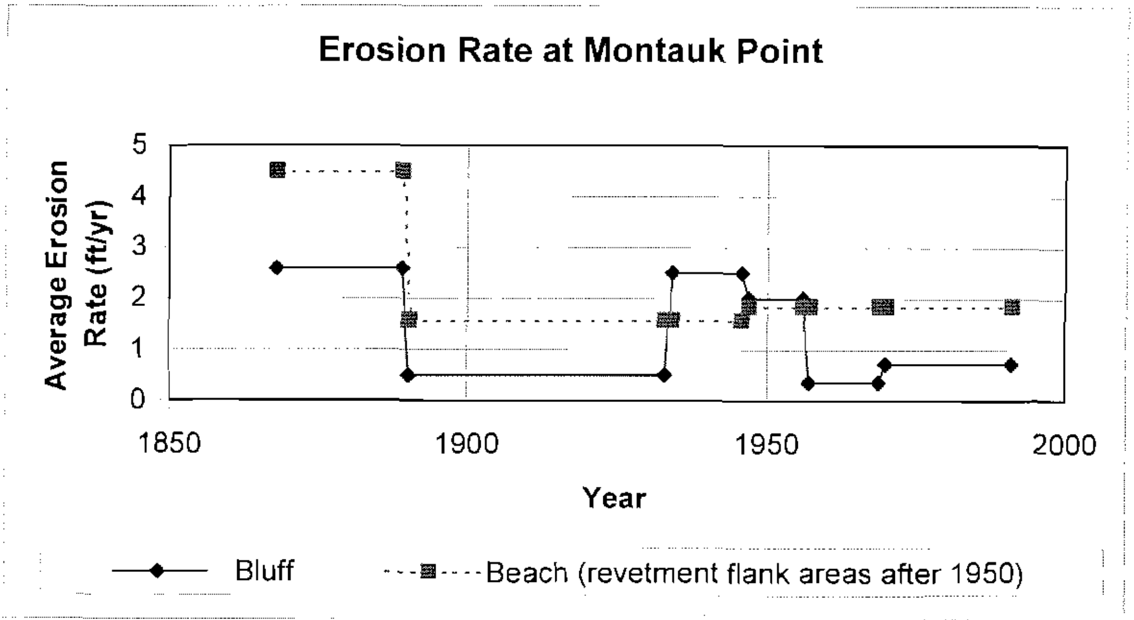


Figure A-6. Average Erosion Rates Since 1868 at Montauk Point (New York District, 1993)

6. Waves and Water Forces

6.1 Waves

A-41 The basis for developing wave characteristics for Montauk Point was an excerpt from a report entitled "Fire Island to Montauk Point Reformulation Project (FIMP), Moffatt & Nichol, June, 2000". The basis of that analysis was the Army Corps of Engineers Wave Information Study, 1976-1994, with adjustments made as necessary based on "observed behavior of longshore transport" as described in a CHL Progress Report dated January 1997. The wave transformation data used by Moffatt & Nichol for the FIMP study used the offshore WIS waves at Stations 75 & 77, and the CHL-modified Stations 79 and 80 for the 1976-1994 time period.

A-42 The offshore WIS waves were transformed to the boundary of a nearshore wave model for the Montauk Point area. The model was used by CHL for shoreline change predictions in the January 1997 report. The Moffatt & Nichol report provides tables of wave height/direction distributions. The largest waves at Montauk arrive from the ESE to SSW direction range, with periods of 9-15 seconds.

A-43 The hindcasted wave peaks were tabulated in a letter to OCTI from Rebecca Brooks of the Coastal Engineering Research Center dated 14 March 1996 and are compared to measurements obtained from the NOAA website at Buoy 44025 in Table A-1. The only years of overlap between measurements and the hindcast are 1991-1993 and include some significant events.

Table A-1. Comparison of measured and hindcasted wave characteristics

Event	Measured Peak Wave Height, Hmo (ft)	Hindcast Peak Wave Height, Hmo (ft)	Measured Wave Period, Tp, (s) at Peak	Hindcast Wave Period, Tp, (s) at Peak	Measured Wave Direction, Dm, (deg) at Peak	Hindcast Wave Direction, Dm, (deg) at Peak
12/11/92	30.5	18.7	12.5	12.0	83	133
11/28/93	21.3	18.7	11.5	12.0	151	144
8/19/91	19.0	18.4	16.7	13.0	64	148
1/5/92	20.3	16.1	9.1	14.0	59	133
3/14/93	23.9	15.4	14.3	10.0	155	122
3/4/93	19.7	15.1	10.0	10.0	60	126

A-44 Table A-1 indicates that the hindcasted wave height information for storms is, on average, 5.4 feet lower than measured, the periods are 0.5 seconds lower, and the directions average about 39 degrees more toward the southeast. These differences are due to a variety of details related to the numerical modeling of waves; however, for purposes of this study it should be noted that extreme waves are, on average about 5.4 feet lower than measured with significantly higher deviation (8 to 12 feet) at the high end of the distribution. These differences, however, become less of a concern in areas such as Montauk Point where the design waves are depth-limited. Note that the hindcast reports an event on 9/9/91 that does not appear in the buoy record, and the list of extreme hindcasted heights did not show a peak from the Halloween Storm of October 1991.

A-45 Table A-2 presents extreme wave heights estimated by Moffatt & Nichol at the 32.8-ft contour irrespective of wave direction based on storm stages developed by CHL in 1996 for the Fire Island to Montauk Point Study. These stages were updated by CHL in 1998 as developed in Section 6.3 (Table A-6) and resulted in no change in the offshore wave development.

Table A-2. Extreme storm statistics produced by the Fire Island to Montauk Reformulation Study.

Return Period (yrs)	Significant Wave Height (ft)	Storm Stage (ft, NGVD)	Max. Breaker Height (ft) (-32.8 ft NGVD contour)	Design Significant Wave Height (ft) -32.8 ft depth	Wave Period (s)
2	17.13	4.53	29.12	16.18	13.00
5	20.57	5.38	29.79	16.55	13.15
10	21.03	5.81	30.12	16.73	14.48
25	21.56	6.33	30.53	16.96	16.13
44	21.99	6.77	30.87	17.15	17.10
50	22.11	6.92	30.99	17.22	17.37
73	22.49	7.42	31.38	17.43	18.11
100	22.83	7.94	31.78	17.66	18.66
150	23.26	8.63	32.32	17.96	19.44
200	23.62	9.12	32.70	18.17	20.04
500	24.70	10.63	33.88	18.82	22.23

A-46 For development of design waves, it was determined that the waves will be depth-limited at the location of the revetment. Three approach lines (cross-sections) were developed using the most recent (2001) topographic and hydrographic surveys over which the waves at the –32.8 ft contour were transformed (Figures A-7 and A-8). The approach lines are very similar in profile view, and wave transformation model test runs indicated that the nearshore wave characteristics are all virtually identical adjacent to the revetment. Therefore one cross-section (SE) was used for detailed wave transformation modeling. The nearshore model SBEACH was employed to perform the wave transformation because it is a one-dimensional model that includes surf zone processes that are very important in this exposed environment.

A-47 Nearshore design waves were also developed for comparative purposes using the spectral model STWAVE. Boundary wave spectra were developed using the extreme significant offshore wave heights (Col. 2, Table A-2) and wave periods (Col 7, Table A-2) along with waves from the East and South-Southeast per Table A-1. Some storm wave directions in Table A-1 are more from the East-Northeast but those data were measured at Buoy 44025 where there is more exposure to the Northeast. At Montauk, the presence of more northerly exposure is blocked, so the worst storm waves would be more from the East to South-Southeast. For each wave case, the appropriate water level was added to the water depths based on the CHL extreme storm surges presented in the Section 6.3.

A-48 Table A-3 presents the results of the calculations for significant wave heights at the toe of the present structure based on the two numerical models employed. The differences in results at the structure toe are due to slightly different representations of the bottom profile and the wave breaking processes.

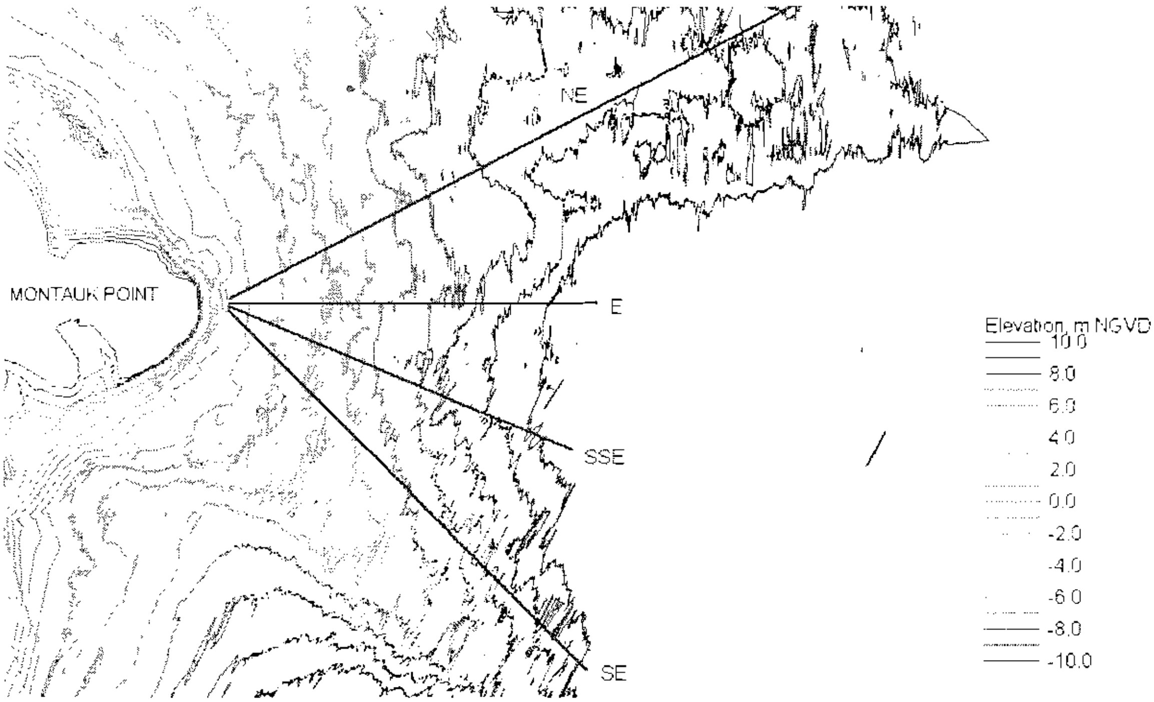


Figure A-7. Location of Beach Profile Lines Considered for SBEACH Wave Transformation Analysis

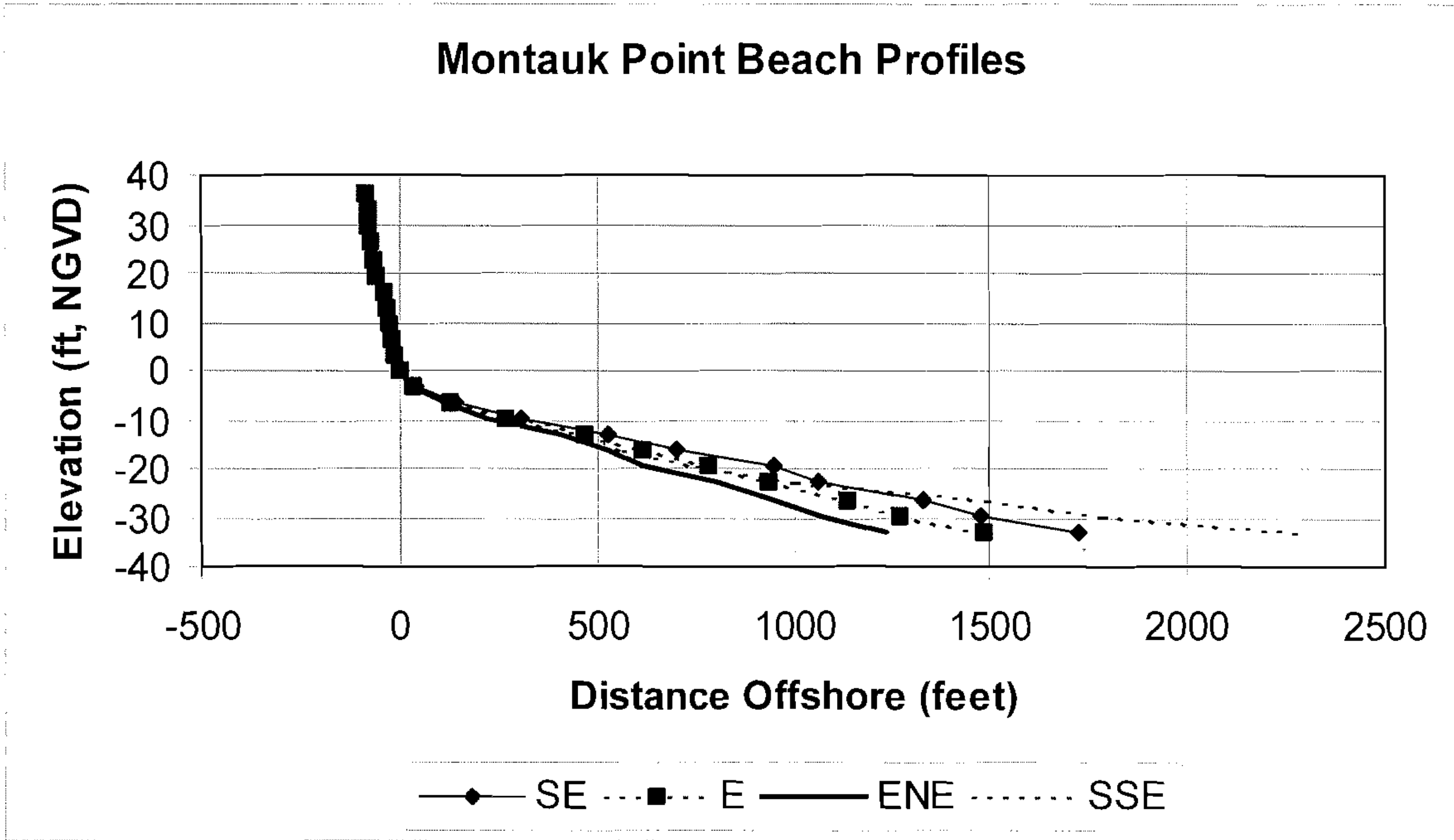


Figure A-8. Beach Profiles Considered for Wave Transformation Analysis

Table A-3. Without-Project Storm Significant Wave Heights at Toe of Revetment

Storm Return Period (years)	Wave Height at Toe (ft)	Wave Height at Toe (ft)	Local Wave Direction for Storms from E	Local Wave Direction for Storms from SSE
	SBEACH	STWAVE	Deg from due E	Deg from due E
2	4.36	3.27	+5	-22
5	4.82	4.75	+5	-27
10	5.05	5.19	+5	-25
25	5.41	5.86	+5	-26
50	5.77	6.35	+5	-27
72	6.05	6.55	+5	-27
100	6.40	6.80	+5	-27
500	7.87	8.77	+5	-29

(Note that wave directions are from STWAVE. At Turtle Cove, the wave directions are –12 deg for Easterly storms and –60 deg for South-Southeasterly storms)

6.2 Tidal Currents

A-49 Tidal currents play a role in transporting sediment along the beach. At a location such as Montauk Point, flows pass around the point as the astronomical tidal wave enters Long Island Sound to the north and the Atlantic Ocean to the south. Currents are very strong along the toe of the revetment and likely enhance the transport of fine sediments that are winnowed from the bluff face after being mobilized and sorted by waves.

A-50 The Tidal Current Tables published by the National Ocean Service provide maximum ebb and flood tidal currents for locations 1.2 miles east and 1 mile northeast of Montauk Point. The currents are summarized in Table A-4.

Table A-4. Published Tidal Current Information for the Montauk Point area.

Location	Maximum Flood Speed	Maximum Flood Direction	Maximum Ebb Speed	Maximum Ebb Direction
1.2 miles east of Montauk Point	2.8 kt	346 deg	2.8 kt	162 deg
1.0 miles northeast of Montauk Point	2.4 kt	356 deg	1.9 kt	145 deg

6.3 *Water Levels*

A-51 Astronomical tide statistics were reviewed from two sources: the New York District's Reconnaissance Report for Montauk Point, and NOAA Benchmark Sheets for Montauk Point (Fort Pond, New York). The tidal statistics are generally within 0.31 feet for all relevant tidal datums, with the NOAA statistics higher than those used in the reconnaissance report. Using the relationship between current Mean Sea Level and NGVD29, the NOAA tidal datums were referenced to NGVD29 and are shown in Table A-5.

Table A-5. Tidal statistics for Montauk Point.

Level	Elevation, MLLW feet (NAN, 1993)	Elevation, MLLW feet (NOAA, 2001)	Elevation, NGVD feet (NOAA-0.8')
Mean Higher High Water (MHHW)	2.4	2.60	1.80
Mean High Water (MHW)	2.0	2.31	1.51
Mean Sea Level (MSL)	1.2	1.24	0.44
Mean Low Water (MLW)		0.18	-0.62
Mean Lower Low Water (MLLW)	0.0	0.00	-0.80

A-52 The Coastal and Hydraulics Laboratory (CHL, 1998) refined the storm surge levels for the Fire Island to Montauk Point Reformulation Project that were presented in Table A-2. Those levels, which included a tabulation of stage-frequency values for the combination of tropical and extratropical storms, are added to the astronomical Mean Sea Level to produce total water levels shown in Table A-6. However, the storm stages from Table A-2 (plus setup) are very close to the updated values from Table A-6 and, for continuity with the offshore wave development (from Table A-2), will be used for wave design.

A-53 The highest observed water level, according to NOAA recorded water levels at Montauk, was +7.90 feet NGVD recorded in 1954. However, this water level was taken offshore and did not include the significant impact of wave setup (refer to Table A-6).

Table A-6. Storm tide statistics developed by the Coastal and Hydraulics Laboratory

Return Period (years)	Combined Storm Surge (Tropical plus Extratropical), NGVD feet	Combined Storm Surge + Astronomical MSL, NGVD feet	Wave Setup (from FIMP)	Storm Surge + Wave Setup + Astronomical MSL, NGVD feet	Utilized Storm Stage * + Wave Setup NGVD feet
5	4.76	5.20	2.72	7.92	8.10
10	5.34	5.78	2.88	8.66	8.69
25	6.14	6.58	3.19	9.77	9.52
50	6.73	7.17	3.42	10.59	10.34
100	7.33	7.77	3.57	11.34	11.51
500	10.29	10.73	3.88	14.61	14.51

* From Table A-2.

7. Scour, Runup, Overtopping, and Wave Attack Forces for Without-Project Conditions

A-54 The toe of the existing stone revetment consists of stone overlying stiff glacial till. The scour mechanisms associated with glacial till and stone are not predictable using numerical models. Therefore a physical model was built to assess failure mechanisms. In that model, both a sand and hard bottom were tested. For the sand bottom tests, sand was placed on a fixed (hard bottom) floor at the toe of the structure and was allowed to move through a 1.5-hour (prototype) storm condition. A trough 40 feet wide and 4 feet deep formed in the same indicating that sand or even small rocks could be eroded during a storm, however, this is not the general condition of the existing revetment toe. It should be noted that observations at the site and discussions with Mr. Greg Donohue indicate the firm glacial till, covered with a thin veneer of sand, is much more resistant to erosion than the sand in the physical model.

A-55 Using the offshore wave and corresponding water level conditions listed in Table A-2, wave runup levels were calculated using the method outlined in the Coastal Engineering Manual (1998) from van der Meer and Janssen (1995) for a revetment with a composite slope. An average structure slope of 1:1.25 is estimated from topographic data.

A-56 Because the revetment toe is glacial till and generally overlain with stone, it is expected that toe scour will be minimal during a given storm event but would be subject to some, but not significant, long-term scour. The runup (average of the highest 2%) due to the maximum breaking wave at the toe of the revetment ranges from 22.2 feet NGVD during a 2-year event to 32.0 feet NGVD during a 500-year event. Based on the topography (Figure A-3) data collected in 2001, it appears that the revetment is overtopped along its entire length by the upper 2% of wave runups during all storm events listed. This is consistent with observations of Greg Donohue that wave runup on the order of several feet deep occurs along the fence at the top of the revetment, which varies as low as elevation +20 feet NGVD.

Table A-7. Without-project, maximum runup and potential for overtopping.

Storm Return Period (years)	Max. Water Level at Toe w/Wave Setup (ft, NGVD)	Breaking Wave Height (ft.) Based on SPM fig. 7.4 *	Max. Runup Level (ft, NGVD)	Revetment Overtopped	Revetment Threatened
2	7.1	7.50	22.2	Entirely	No
5	8.1	8.37	23.4	Entirely	No
10	8.7	8.82	23.4	Entirely	No
25	9.5	9.57	23.7	Entirely	Yes
50	10.2	10.32	24.5	Entirely	Yes
100	11.5	11.38	26.4	Entirely	Yes
500	14.5	13.00	32.0	Entirely	Yes

* Toe at el.(–)1 ft NGVD

8. Slope Stability Analysis

8.1 General Information

A-57 The till exposed in the wave cut bluffs surrounding Montauk Point is a well graded mixture of boulders, sand, gravel, and underlying silt preconsolidated by the weight of glacial ice (Figure A-9). It has a long stand up time for near vertical slopes but gradually erodes and fails with time under annual rainfall and runoff (Figure A-10). Under large magnitude wave actions with high storm surges, the dense till will be scoured and result in toe failures of the mid to upper bluff above the revetment for the till and overlying granular soils.

A-58 To simulate the pattern of erosion and slope failure, stability runs were accomplished of PCSTABL6 software using layer elevations from the borings and the results of the geophysical survey. Assumed soil parameters were derived from the standard penetration tests and observed composition of the till (phi = 38 degrees) and the overlying sand and gravel (phi = 30 to 36 degrees). Cohesion was assigned a value of zero due to the lack of plasticity and small percentage of clay size particles.

A-59 PCSTABL6 uses, in this application, the Bishop method of slices along with an iterative process for generation of potential failure surfaces. This iterative process identifies those critical failure circles with the lowest factors of safety.

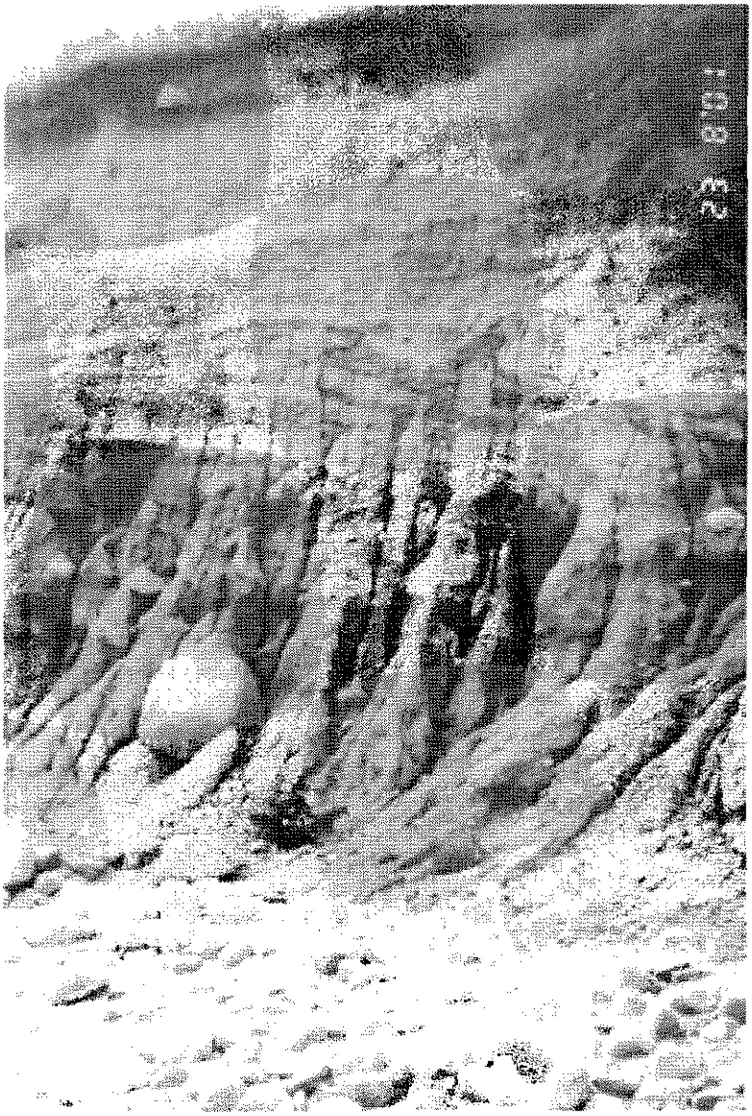


Figure A-9. Typical bluff cross-section with glacial till at Montauk Point.

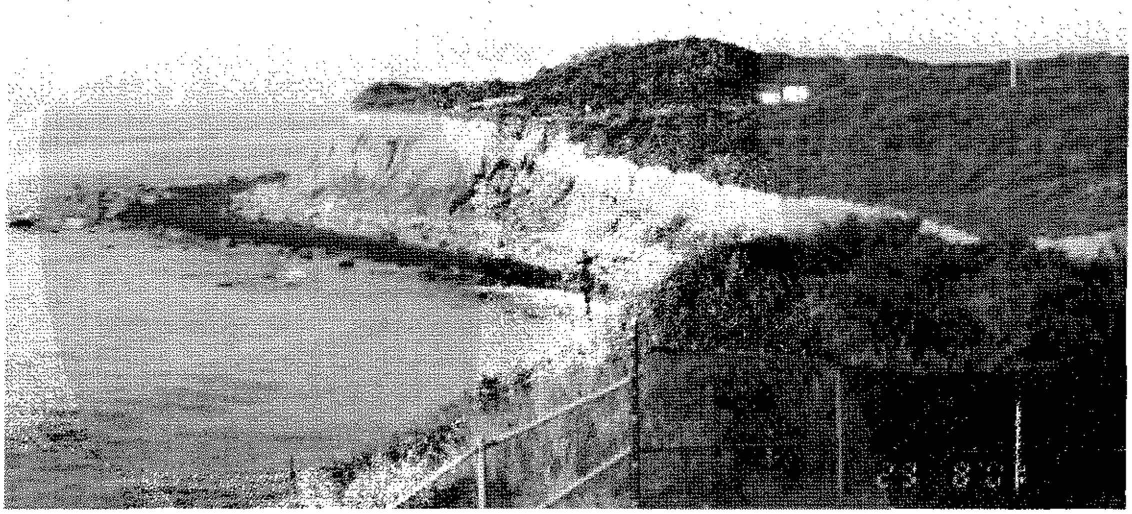


Figure A-10. Eroded bluff south of Turtle Cove with wave-eroded toe and consequent bluff failure.

8.2 Existing Conditions

A-60 The stability analysis of existing conditions were performed on the three cross sections located on Figure A-11 and shown on Figures A-12 to A-14. These sections represent the steepest slopes that are near and surrounding the lighthouse. Section A-A is the steepest of the three sections. The stability analysis on each section indicates that the slopes and the present conditions are at equilibrium with little or no safety margin. This is indicated by a Factor of Safety (FS) of approximately 1.0 for Sections A-A and B-B. The upper parts of the slopes, which are near the angle of repose for the granular soils, show the highest potential for failure if existing conditions are even slightly disturbed. At Section A-A (Figure A-12), the upper slopes would consistently fail if terracing and vegetation stabilization measures, maintained by the Montauk Point Historical Society, were not practiced.

A-61 However, a much larger failure surface and volume of material starting at the shoreline also has a low FS (1.094) in section A-A and can fail with external disturbance, however is unlikely to occur due to the very dense nature of the soils near the toe of the bluff below el. +15 ft. NGVD +/-.

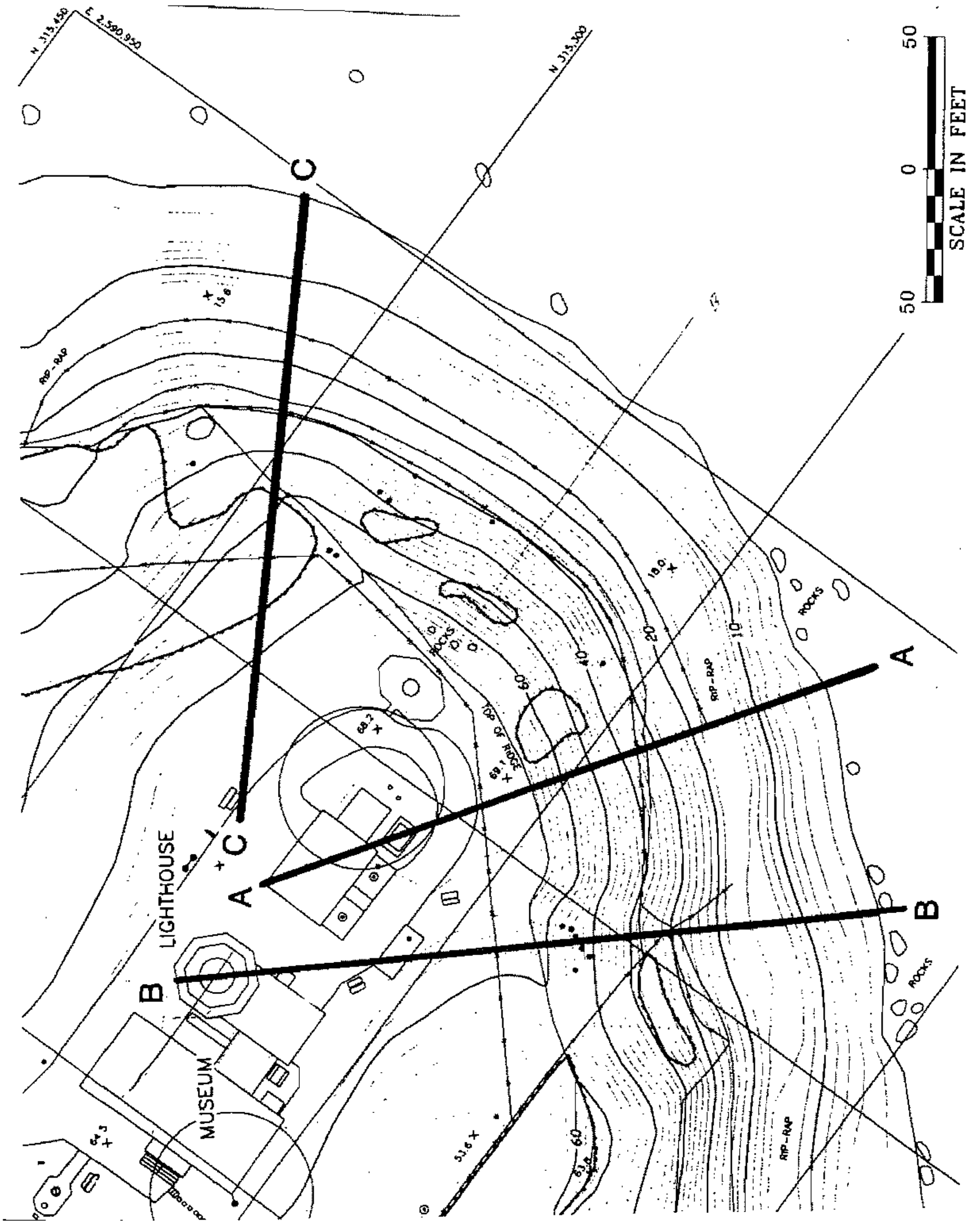


Figure A-11. Locations of cross-sections for slope stability modeling.

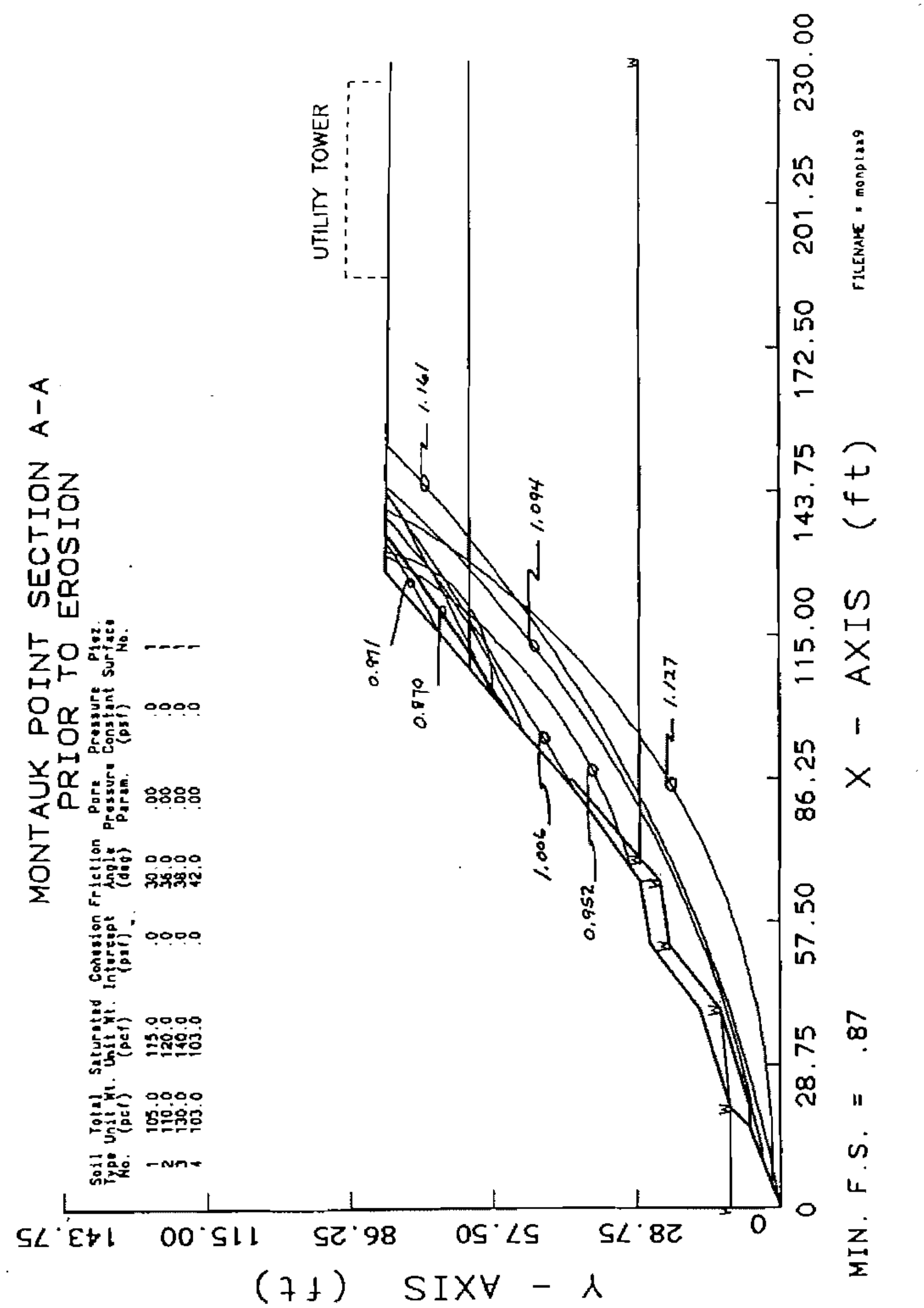


Figure A-12. Slope stability model results for existing conditions, Section A-A.

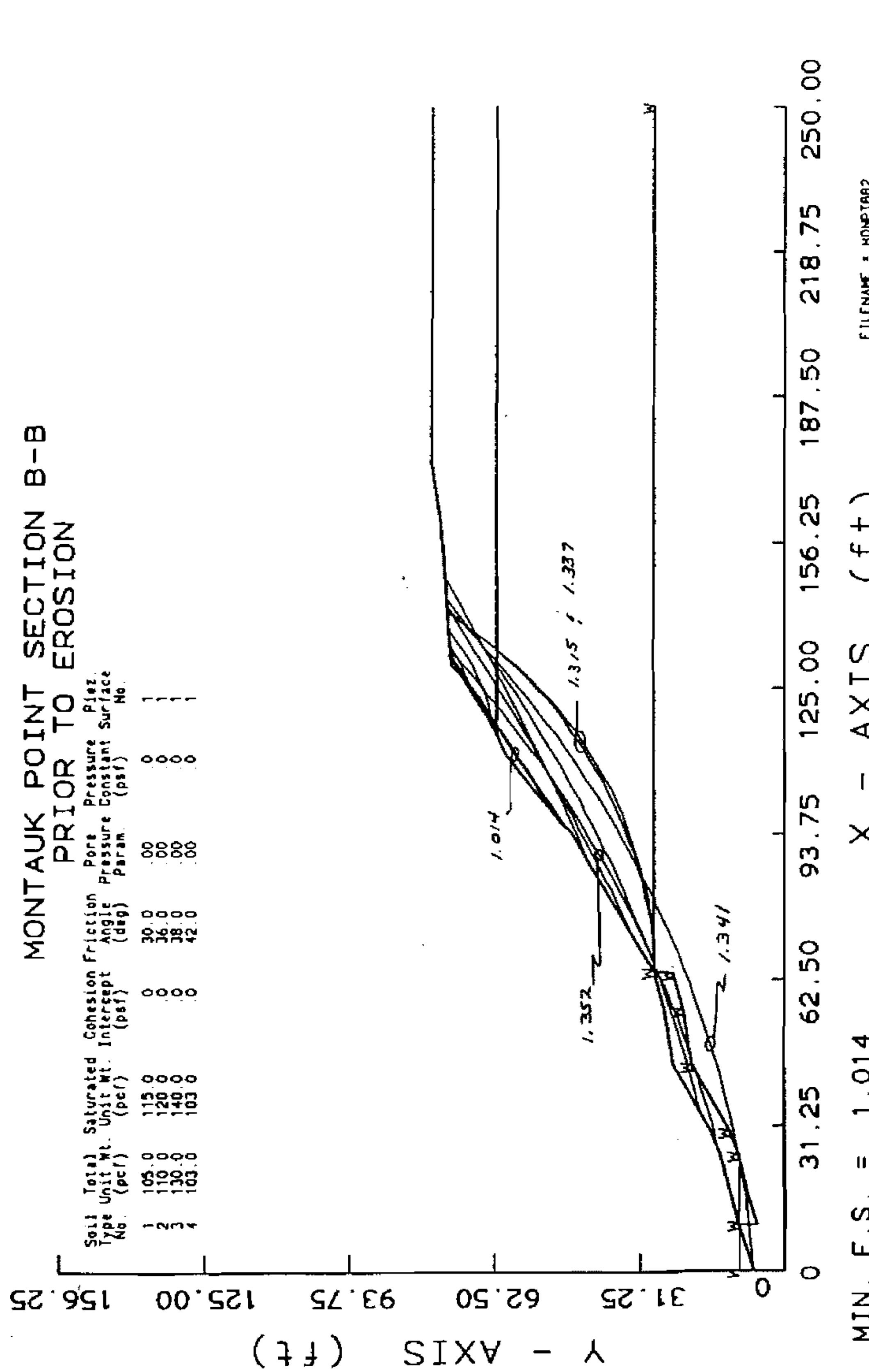


Figure A-13. Slope stability model results for existing conditions, Section B-B.

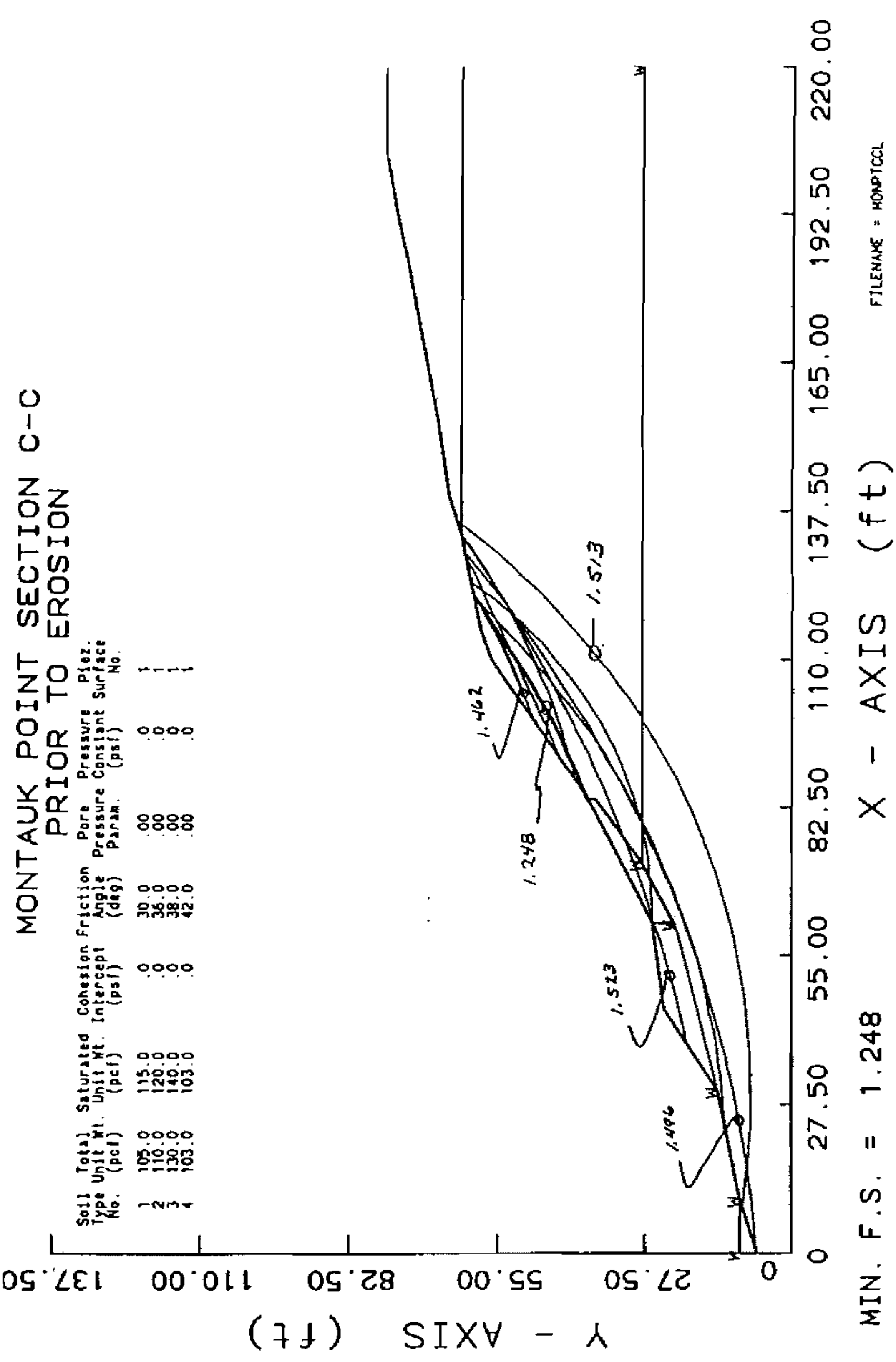


Figure A-14. Slope stability model results for existing conditions, Section C-C

9. Without-Project Future Conditions

A-62 Three possible failure modes are considered in determining the remaining life of the existing shore protection structure. The failure modes are: toe erosion at the base of the revetment that would lead to toe stone instability and revetment collapse; wave action dislodging lighter than required armor stones prevalent and interspersed on the revetment surface; and wave runup and overtopping that would dislodge the revetment crest stones and lead to revetment collapse. The exact elevation of the toe of the present structure is not well defined, but is estimated from photographs such as Figure A-1-8 in Sub-Appendix A-1 and spot elevations in recent topographic surveys obtained by the New York District in 2001. It is noted that failure of the revetment would be followed by bluff failure, which would then threaten the lighthouse. Revetment failure alone will not cause the immediate catastrophic failure of the lighthouse, since the slope stability of the bluff after revetment failure still has a factor of safety greater than > 1.0.

A-63 The recession of the bottom profile for the beach front flanking the revetment is less than the maximum (due to a differing shoreline orientation) based on historical recession rates below the water line. A corresponding sea level rise (0.01 ft/yr) and profile horizontal recession (approximately 1 ft/yr historically, but which will diminish in the future) for the beachfront flanking the revetment is included. For the revetted area, the recession rates are assumed to be negligible due to the presence of the revetment (recession of the upper part of the profile is assumed, based on performance, to be arrested by vegetative shore protection measures). In addition, erosion immediately adjacent to the revetment will diminish below the historical 1 ft./yr rate due to the sheltering effect of the existing revetment, and thus, will negate flanking potential.

A-64 Three modes of failure can occur individually or in combination. Because of the uncertainty in predicting the impacts of these three modes of failure (i.e. stone displacement from wave impacts due to undersized stone, erosion of the toe foundation soil (hardened till), or displacement of stone on the upper part of the revetment due to wave rnnup and overtopping) a physical model was performed. The primary mechanism expected to cause bluff failure is the effect of waves, including direct impact and runup/overtopping, on the armor stone. Large-scale slope failure (i.e. that initiated at the shoreline or structure toe) is not expected to occur due to the presence of glacial till and large amounts of stone overlying the soils.

9.1 Failure of Armor Layer Due to Wave Forces

A-65 When the water level is elevated by both astronomical tide and storm surge, waves impact the armor stone. Although the present armor stone is resistant to smaller waves, large waves can be expected to damage and dislodge the armor, resulting in the failure of the structure. The existing structure is not a recommended type of cross-section, since it only consists of one layer of tightly interlocked stones of varying size, and has no buried toe. Because of the associated uncertainty in stone performance under storm conditions, a physical model was constructed to replicate the existing revetment as closely as possible in terms of variance of stone size and degree of interlocking. The model tested storm waves ranging from the 2-year return period to the 100-year return period range and very minor displacement of armor stone on the revetment slope was observed. The model included areas of undersized stone interlocked among larger armor units and no failure was observed for the range of waves tested. Thus, this failure mode is not considered pertinent.

9.2 Failure of the Structure Toe Due to Erosion

A-66 The physical model was not able to exactly replicate the condition of the dense foundation soil (glacial till of widely-varying gradation overlain with a thin veneer of sand) at the revetment toe, but these conditions in the model were simulated with a hardened bottom. In addition, based on eyewitness accounts from continuous observation over extended periods of time, including severe storms, both storm-induced and long-term toe erosion are considered to be relatively minor in terms of toe stone instability. Although some long-term erosion does occur in the revetted area, it is difficult to compute or otherwise quantify realistic rates. Maintenance practices will tend to protect the base of the structure, and the predominance of dense, glacial till overlain by stone will significantly retard toe erosion. Therefore, the toe erosion mode of failure is not considered pertinent.

9.3 Failure Due to Overtopping.

A-67 Additional stability analyses were performed to model the reduction of the height of the upper revetment due to wave overtopping and the subsequent wave scour of the underlying soils of a failed revetment. These analyses were performed for three cases; the existing revetment height to an elevation of +18 feet MSL, a revetment height of +14 feet MSL after lowering by initial upper revetment failure, and the failed revetment with a height of +10 feet MSL. These analyses, combined with the physical model results that show upper revetment failure to below elevation +10 MSL, indicated that under the latter conditions, the top of the slope would recede landward a distance of approximately 26 feet subsequent to failure of the revetment between elevations +10 feet MSL and +18 feet MSL. The slope profile changes are presented on Figure A-15.

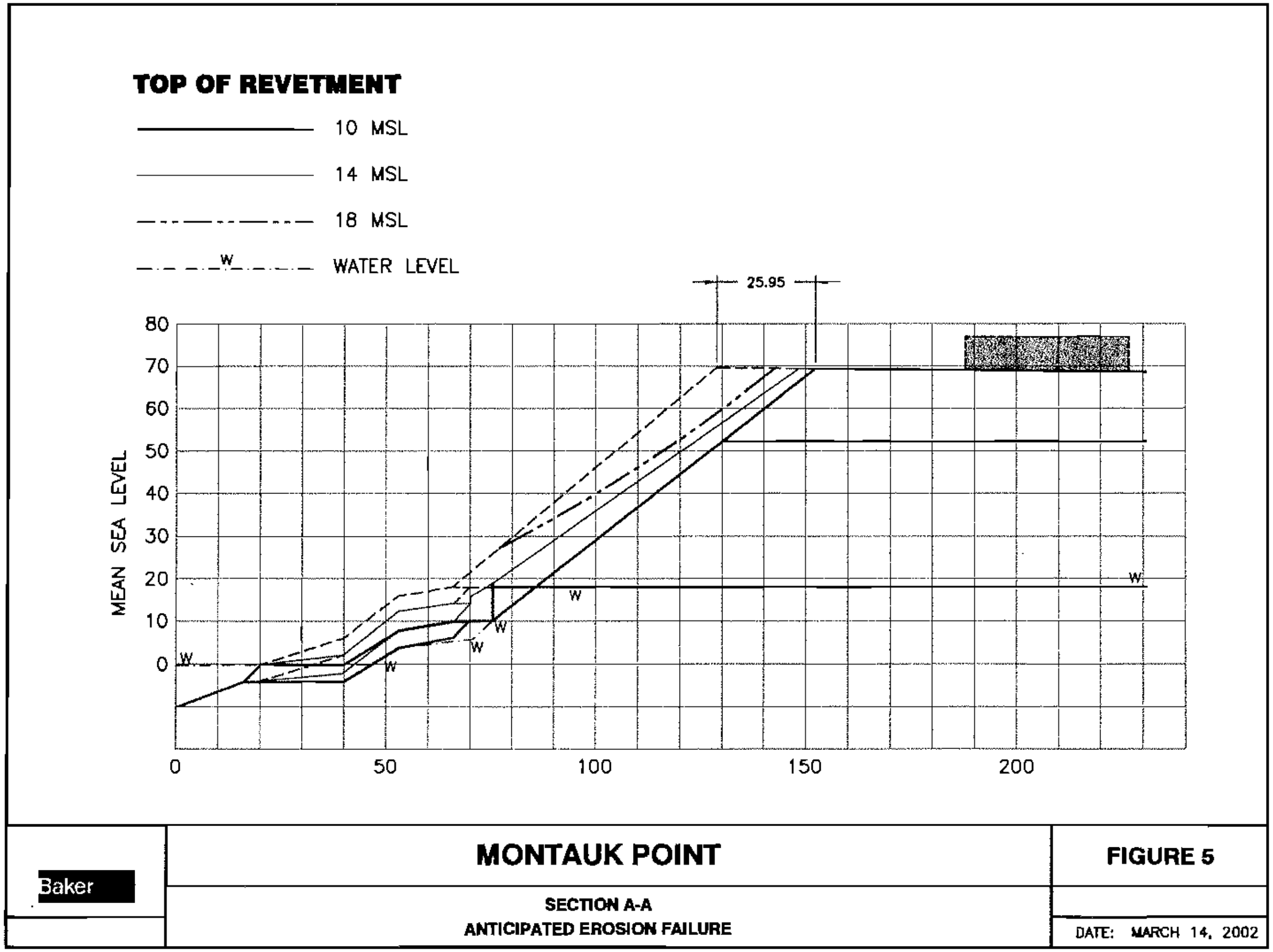


Figure A-15. Slope stability for without-project future conditions.

A-68 The recession of the glacial till behind the failed revetment will be wave eroded, whereby the till slumps, the upper slope slumps, and the cycle is repeated over several years time. The eventual result is the migration of the Turtle Hill bluff until the slope face reaches and undermines the utility tower, lighthouse, and associated structures. Based upon historic recession rates, the upper bluff toe (at +10 ft NGVD) will recede the approximate 10 feet necessary to cause bluff failure, to directly threaten the lighthouse structures, over a period of 8-10 years after the upper sections (above el. +10 feet MSL) of the revetment are displaced.

9.4 Findings

A-69 The physical model was tested for wave runup and overtopping of the revetment for an approximately 2-year return period storm through an approximately 100-year return period storm. Based on the results of the model test, it was determined that stone displacement, from overtopping of the revetment crest, occurs between a 10-year return period storm and a 20-year return period storm, say a 15-year return period storm. This result is substantiated by a semi-empirical analytical method to determine damage threshold exceedance from overtopping (Coastal Engineering Manual 1997 Part VI).

A-70 Since the last storm experience at Montauk Point of this significance was in 1993, there is a likelihood (60% probability) that this 15-year return period storm will occur by the year 2006 to cause significant damage (at least 25% damage level) to the revetment itself. Once the upper sections of the revetment are displaced in the year 2006, the foundation soil underlying the displaced stone will become exposed and subject to subsequent erosion.

A-71 To determine the extent of erosion of the toe of the upper bluff above the damaged revetment that would cause significant bluff failure to threaten the stability of the lighthouse structure, a slope stability analysis was performed. The results of this analysis determined that for significant bluff failure, the damaged crest elevation of the revetment should degrade to approximately +10' NGVD (indicated by the physical model from a 10 year return period to a 20 year return period storm) and the upper bluff toe at that elevation recede horizontally approximately 10 feet. This should cause about 30 feet of loss of the bluff crest and immediately threaten the lighthouse facility at the most critical area to the southeast of the structure.

A-72 The period of time estimated for this condition to occur, subsequent to 2006, is an additional 8-10 years which results from long-term erosion at the upper bluff toe (elevation +10 feet NGVD) with no significant storm occurrence, or from an approximately 10-year return period storm which has a likelihood of occurrence (60% probability) by the year 2015.

A-73 Design revetment concepts for future protection of the area must also consider appropriate transition and tapers to preclude any erosion-induced discontinuities.

A-74 For design of storm protection alternatives, Table A-8 provides water levels and wave characteristics. The design breaking wave height listed in Table A-8 is calculated using Figure 7-4 (SPM 1984) at a bottom elevation of –4' NGVD at the improved revetment toe. The present structure toe is at a bottom elevation of about –1' NGVD, making the design breaking waves slightly lower than those listed in the table.

Table A-8. Water Level and Wave Characteristics

Return Period (yrs)	Offshore Significant Wave Height (ft)	Storm Stage (ft, NGVD)	Storm Stage plus Wave Setup (ft, NGVD)	Design Breaking Wave Height (ft) at Revetment Toe (ft) (-4' NGVD)	Wave Period (s)
2	17.13	4.53	7.07	10.1	13.00
5	20.57	5.38	8.10	10.9	13.15
10	21.03	5.81	8.69	11.4	14.48
25	21.56	6.33	9.52	12.2	16.13
44	21.99	6.77	10.16	12.8	17.10
50	22.11	6.92	10.34	12.9	17.37
73	22.49	7.42	10.94	13.4	18.11
100	22.83	7.94	11.51	13.9	18.66
150	23.26	8.63	12.31	14.6	19.44
200	23.62	9.12	12.86	15.1	20.04
500	24.70	10.63	14.51	16.5	22.23

10. Development of Alternatives

10.1 General Approach

A-75 Alternatives that are feasible approaches to storm protection and shoreline stabilization need to address both present and future needs. The present need is to eliminate the threat of erosion and to provide acceptable levels of protection from the impacts of wave attack and storm recession.

General requirements include:

- Meet the needs and concerns of the public within the study area
- Respond to the public desires and preferences
- Be flexible to accommodate changing economic, social and environmental patterns and changing technologies
- Integrate with and be complementary to other related programs in the study area
- Implement with respect to financial and institutional capabilities and public consensus
- Conform with USACE environmental operating principles

Specific requirements include:

- Protect Montauk Point and vicinity, including the historic lighthouse and associated facilities from erosion, environmental degradation, and coastal storm damage
- Reduce the threat of future bluff instability including those due to wave attack and erosion from ocean impacts
- Provide a cost effective approach for bluff protection
- Prevent the aggravation of erosion in adjacent areas
- Maintain proper stone interlocking for bluff protection

A-76 There are a variety of constraints on a possible solution, thereby limiting the number of feasible solutions.

Technical constraints include:

- Plans must represent sound, safe and acceptable engineering solutions taking into account the overall littoral system effects
- Plans must be in compliance with Corps of Engineers regulations
- Plans must be realistic and state-of-the-art while not relying on future research
- Plans must provide bluff protection
- Plans must provide features that minimize the effect of shoreline erosion processes

Economic constraints include:

- Plans must be efficient, make optimal use of resources and not adversely affect other economic systems
- Average annual benefits must exceed the average annual costs

Environmental constraints include:

- Plans must avoid and minimize environmental impacts to the maximum degree practicable
- Plans must consider mitigation or compensation for a potential impact when identified

Regional and Social constraints include:

- All reasonable opportunities for development within the project scope must be weighed, with consideration of state and local interests
- The needs of other regions must be considered and one area cannot be favored to the detriment of another
- Plans must maintain existing cultural resources to the maximum degree possible, and produce the least possible disturbance to the bluff
- Plans must maintain or improve recreational fishing and surfing experiences

Institutional Constraints include:

- Plans must be consistent with existing federal, state and local laws
- Plans must be locally supported and signed by local authorities in the form of a local cooperation agreement, guarantee for all items of local cooperation including possible cost sharing
- Local interests must agree to provide public access to the beach in accordance with Federal and state guidelines and laws
- The plan must be fair and find overall support in the region and state.

A-77 Criteria for evaluating preliminary alternatives will include appropriateness to site conditions, compliance with New York State Coastal Zone Management criteria, effectiveness of protection, impacts on environmental and cultural resources, and annual cost (including interest during construction and maintenance).

10.2 Alternatives

PRELIMINARY ALTERNATIVE 1 – Repair Structure On As-Needed Basis (No Action Plan)

A-78 The No-Action Plan (no Federal action through the Corps of Engineers) would consist of a continuation of the Without-Project condition. If allowed to occur, progressive instability of the bluff would result in the irrecoverable loss of the Turtle Hill Plateau, the lighthouse, and its associated structures, along with archaeological resources.

A-79 Efforts by the Montauk Historical Society to control the erosion are expected to continue, but in the absence of a comprehensive shore protection project, experience shows that their efforts have not solved and would not solve the long-term problem of significant damage to the existing structure complex with associated threat to the lighthouse from large storm events over an extended period of time (e.g. 50 years). It is estimated that emergency repair costs will continue to be required and there would also be costs to investigate and curate historically significant resources in threatened bluff areas. However, the emergency repair over an extended period of time is not anticipated to provide adequate protection to the lighthouse and bluff, and will therefore leave them vulnerable to failure from storm damage due to expected design exceedance of these limited actions.

A-80 If the lighthouse was lost, the Coast Guard would have to construct a new navigation aid to replace the lighthouse. While the No Action plan fails to meet objectives and needs of the project area, it does provide the basis from which project benefits are measured.

A-81 It is estimated that the present revetment structure is susceptible to damage from a greater than 10-year storm frequency event but periodic damage will occur during lesser events. It is assumed that the Montauk Historical Society will do repairs as they are needed, but ultimately will not be able to keep the structure intact without efforts to upgrade the structure design.

PRELIMINARY ALTERNATIVE 2 – Stone Revetment

A-82 A riprap stone revetment was developed for long term erosion control as shown in Figure A-16. The plan consists of 840 feet of revetment protection. The protection covers the most vulnerable bluff area that would directly endanger the lighthouse complex due to bluff failure without the project.

A-83 The revetment was designed based on the Engineering Manual 1110-2-1614 "Design of Coastal Revetments, Seawalls and Bulkheads." A heavily embedded toe shall be employed to stand against breaking waves at the toe of the structure. As shown in Figure A-16, the revetment section features a 40' wide crest at +25' NGVD, a 1V:2H side slope, and 12.6-ton quarrystone armor units extending from the crest down to the embedded toe. Three layers of 4-5 ton armor units are used to construct the splash apron. Filter cloth and sublayers are specified in accordance with standard Corps of Engineers design procedures. The estimated first cost for the stone revetment is $14,843,000, including 20% contingency, engineering and design, and construction management, as shown in Table A-9.

A-84 Revetments are a proven method of shore protection in this area and have a record of acceptance by state and local authorities. Revetment alternatives such as this can utilize much of the stone to be removed, already on site in the existing structure, thus making good use of existing resources. The cross section can be slightly modified to allow access for fishermen to areas close to the water. It is not expected that a new revetment will change present surfing conditions in any way.

PRELIMINARY ALTERNATIVE 3 – Offshore Segmented Breakwater with Beach Fill

A-85 The purpose of an offshore breakwater is to reduce the storm wave height offshore of the revetment toe, thus reducing the wave impact force and runup elevation on the bluff. Shoreline recession would be reduced with the construction of an offshore breakwater. The existing revetment and terracing of the upper bluff would provide a reasonable level of protection with the offshore breakwaters in place.

A-86 As shown in Figure A-17, the breakwater would be a rubble mound structure located about 200 feet offshore at about the –8 ft NGVD contour. Beach fill would be placed from about the MHWL out to the breakwaters to provide additional toe protection to the existing revetment. Approximately 200,000 cubic yards of beach fill would be placed to a berm elevation of +11 ft. NGVD. The required renourishment quantity is estimated at 100,000 cubic yards, every 3 years. The sand is assumed to be acquired via a 4,000 cubic yard hopper dredge from Borrow Area IV, seaward of Shinnecock Inlet, as identified in the Fire Island to Montauk Point Reformulation Study. Three separate structures would be built, two being 300 feet in length and one 500 feet in length, with the longest facing the more severe southeasterly direction. The openings between the structures would allow some tidal circulation but also may induce some dangerous currents concentrated in the gaps.

A-87 The breakwater design is based on present Corps guidelines. As shown in Figure A-17, the crest is placed at +7.5 ft. NGVD, which is the 73-year water level without wave setup. The armor size is 17.5 tons, placed in two layers on a single layer of 1.75 ton quarrystone underlayer and 2 layers of 100 pound filter stone. The entire structure is built on filter cloth. The estimated first cost for the offshore breakwater with beach fill is $14,841,000, including a 20% contingency, engineering and design, and construction management, as shown in Table A-10.

A-88 Breakwaters will be difficult to construct due to difficult site access and in-water construction. Tidal currents are significant and breaking waves arrive from almost all onshore directions. The breakwater requires very large stone and a substantial width and elevation to be effective. The gaps between the breakwaters may induce significant currents that could increase scour to the bottom, potentially compromising the foundation of the breakwaters sometime in the future. The high currents may also cause a safety hazard to swimmers, surfers and fishermen who wade in the area. Higher surges with waves that submerge the +11 ft berm will not be prevented from damaging the revetment. Finally, the surfing activity in the area may be affected by changed reflected wave characteristics.

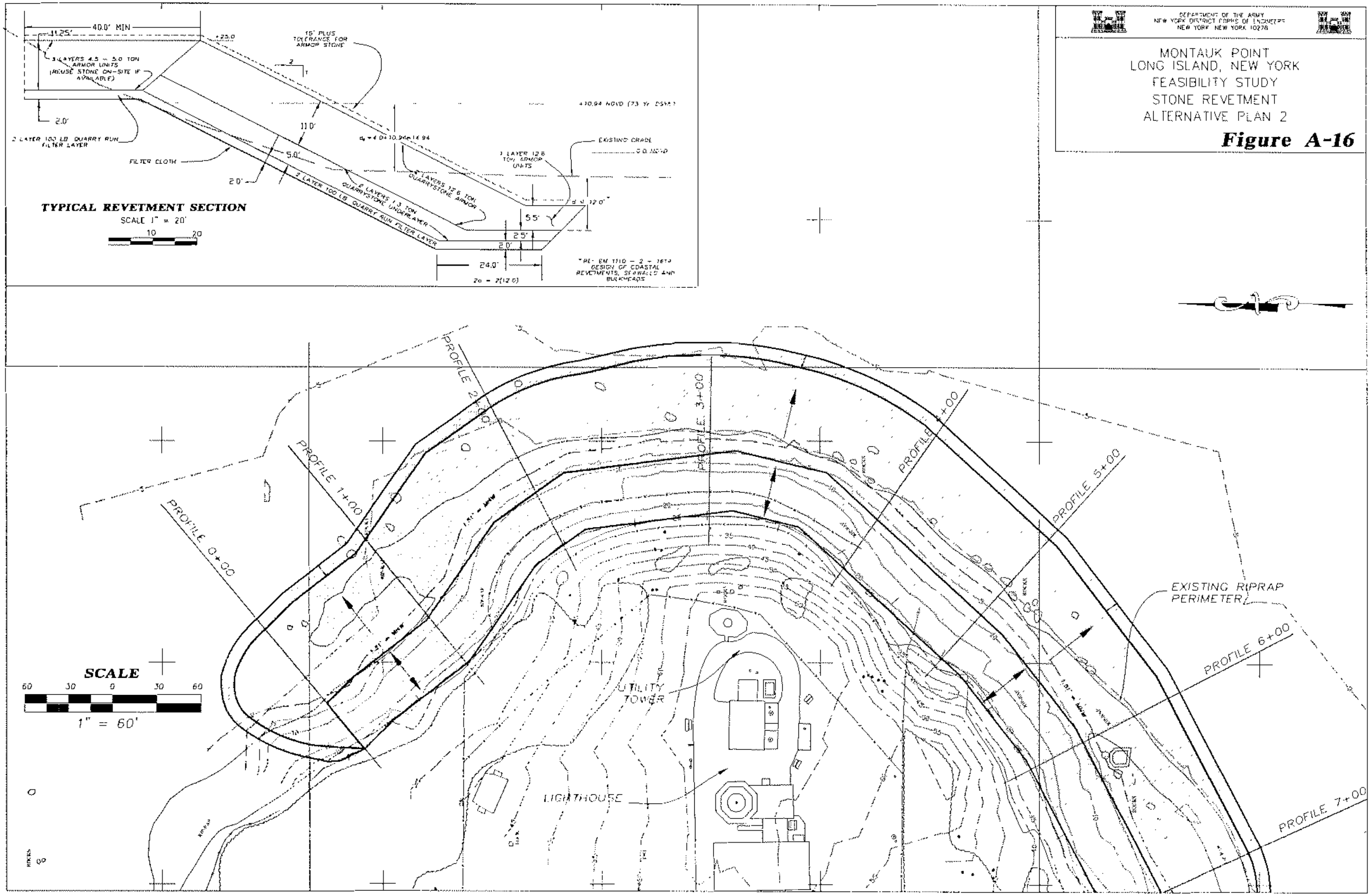
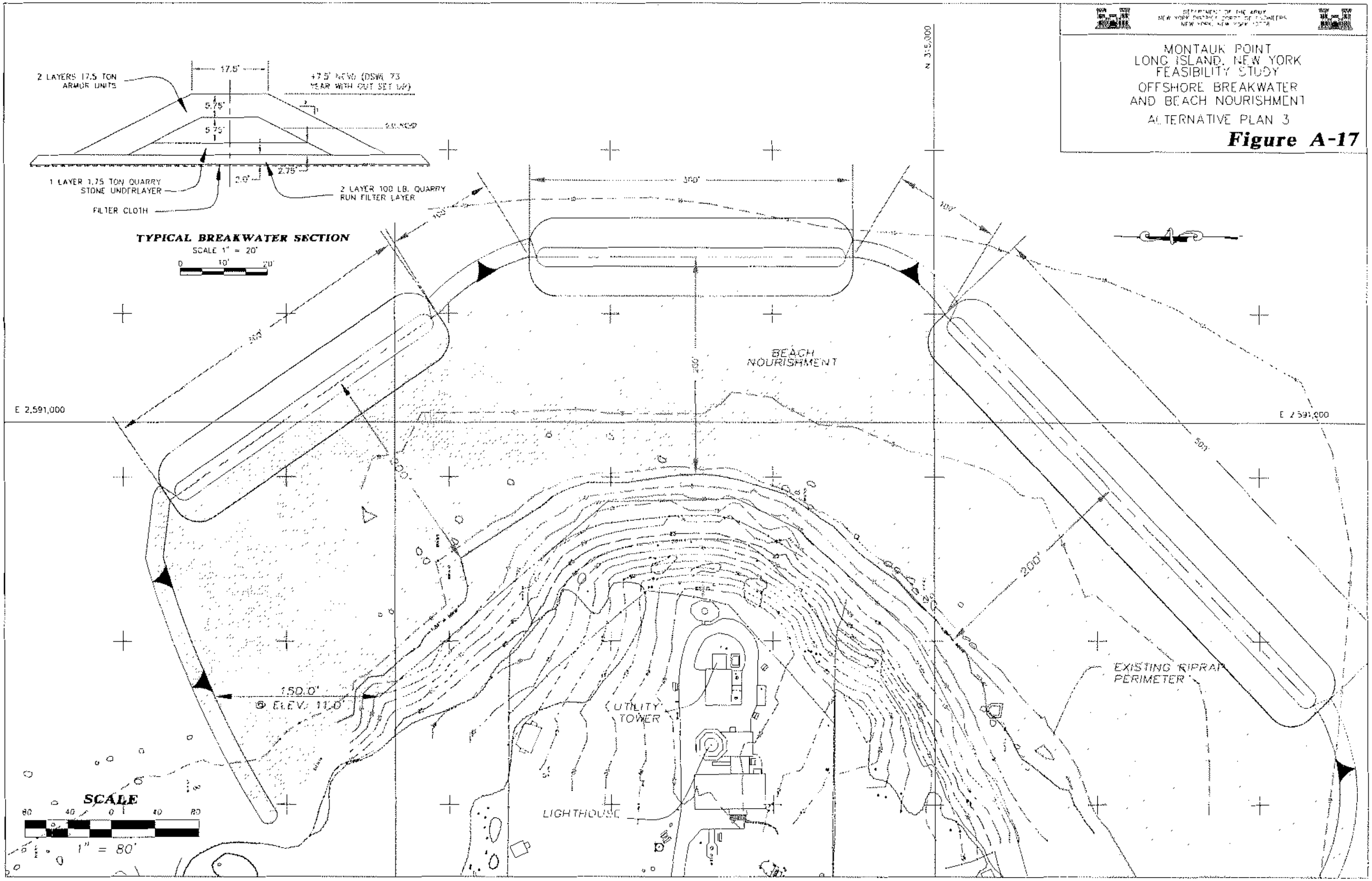

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
40.0' MIN
11.25'
3 LAYERS 4.5 – 5.0 TON ARMOR UNITS (REUSE STONE ON-SITE IF AVAILABLE)
2 LAYER 100 LB QUARRY RUN FILTER LAYER
FILTER CLOTH
15' PLUS TOLERANCE FOR ARMOR STONE
2 LAYERS 12.6 TON QUARRYSTONE ARMOR
2 LAYERS 1.3 TON QUARRYSTONE UNDERLAYER
2 LAYER 100 LB QUARRY RUN FILTER LAYER
EXISTING GRADE
+10.94 NGVD (73 Yr. DSWL)
1 LAYER 12.6 TON ARMOR UNITS
*RE: EM 1110 – 2 – 1614 DESIGN OF COASTAL REVETMENTS, SEAWALLS AND BULKHEADS
SCALE
1" = 60'
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
EXISTING RIPRAP PERIMETER
UTILITY TOWER
LIGHTHOUSE

Figure A-16


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
OFFSHORE BREAKWATER
AND BEACH NOURISHMENT
ALTERNATIVE PLAN 3
2 LAYERS 17.5 TON ARMOR UNITS
17.5'
+7.5' NGVD (DSWL 73 YEAR WITH OUT SET UP)
5.75'
2.75'
2.0'
1 LAYER 1.75 TON QUARRY STONE UNDERLAYER
FILTER CLOTH
2 LAYER 100 LB. QUARRY RUN FILTER LAYER
TYPICAL BREAKWATER SECTION
SCALE 1" = 20'
0
10'
20'
300'
100'
200'
500'
BEACH NOURISHMENT
E 2,591,000
N 315,000
150.0'
@ ELEV. 11.0'
UTILITY TOWER
LIGHTHOUSE
EXISTING RIPRAP PERIMETER
SCALE
80
40
0
40
80
1" = 80'
Figure A-17

PRELIMINARY ALTERNATIVE 4 – T-Groins with Beach Fill

A-89 T-groins, similar to a nearer-to-shore segmented breakwater system with shore-attached groins, are considered as a second breakwater alternative. Similar to the breakwater alternative presented, the purpose of T-groins is to reduce the storm wave height, thus reducing the wave impact force and runup elevation on the bluff. The consistent beach and shoreline recession would be reduced with the construction of T-groins and beach fill. The existing revetment and terracing of the upper bluff would provide a reasonable level of protection with the T-groins in place.

A-90 As shown in Figure A-18, the T-groin system would be a rubble mound structure located about 100 feet offshore at about the –5 ft NGVD contour. Five separate shore-parallel structures would be built, each being 150 feet in length. A groin will be extended from the center of the shore-parallel breakwater segment to shore, creating individual littoral cells. Beach fill is placed from shore out to the centerline of the shore-parallel breakwaters to provide erosion protection to the bluff toe to a berm elevation of +11 ft. NGVD. Approximately 125,000 cubic yards of beach fill will be placed. The required renourishment quantity is estimated at 100,000 cubic yards every 3 years. The sand is assumed to be trucked in from an upland borrow source. It is expected that embayments in the fill will quickly form as waves and tides re-mold the fill material. The openings between the structures would allow some tidal circulation but also may induce some dangerous currents concentrated in the gaps.

A-91 The T-groin design is based on present Corps guidelines. As shown in Figure A-18, the shore-parallel structure crest is placed at +11' NGVD and the groin section crest is placed at +8' NGVD. The armor size is 17.5 tons in the shore-parallel structures, placed in two layers on a single layer of 1.75 ton quarrystone underlayer and 2 layers of 100 pound filter stone. The armor size is 4.5 tons in the groins, placed in two layers on 900 lb. quarrystone underlayer. The entire structure is built on filter cloth. The estimated first cost for the T-groins with beach fill is $12,094,000, including a 20% contingency, engineering and design, and construction management, as shown in Table A-11.

A-92 T-groins will be difficult to construct due to difficult site access, however, land-based equipment can be utilized. Tidal currents are significant and breaking waves arrive from almost all onshore directions. The shore-parallel structures would require very large stone and a substantial width and elevation to be effective. The gaps between the shore-parallel structures may induce significant currents that could scour the bottom, potentially compromising the foundation of the T-groins sometime in the future. The high currents may also cause a safety hazard to swimmers, surfers and fishermen who wade in the area. In this option, the protective beach fill will require renourishment at a rate that is difficult to predict until it is constructed and monitored. Higher surges with waves that submerge the +11 ft. berm will not be prevented from damaging the revetment. Finally, the surfing activity in the area may be affected by changed reflected wave characteristics.

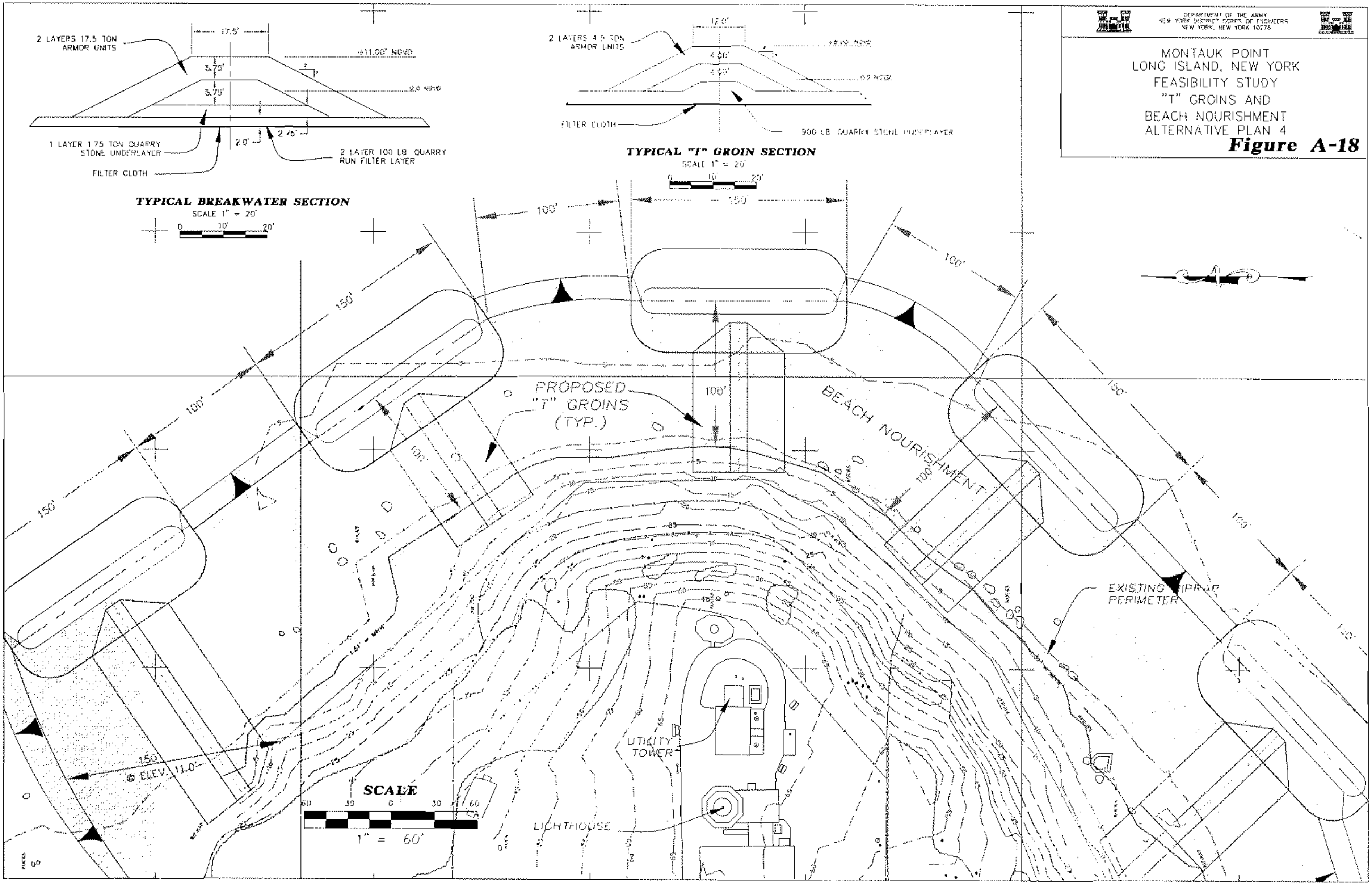

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
"T" GROINS AND
BEACH NOURISHMENT
ALTERNATIVE PLAN 4
2 LAYERS 17.5 TON ARMOR UNITS
17.5'
+11.00' NGVD
5.75'
5.75'
2.0 NGVD
2.0'
2.75'
1 LAYER 1.75 TON QUARRY STONE UNDERLAYER
2 LAYER 100 LB. QUARRY RUN FILTER LAYER
FILTER CLOTH
TYPICAL BREAKWATER SECTION
SCALE 1" = 20'
2 LAYERS 4.5 TON ARMOR UNITS
12.0'
4.00'
4.00'
FILTER CLOTH
900 LB. QUARRY STONE UNDERLAYER
TYPICAL "T" GROIN SECTION
SCALE 1" = 20'
100'
150'
PROPOSED "T" GROINS (TYP.)
BEACH NOURISHMENT
EXISTING RIPRAP PERIMETER
UTILITY TOWER
LIGHTHOUSE
150' @ ELEV. 11.0'
SCALE
1" = 60'

Figure A-18

PRELIMINARY ALTERNATIVE 5 – Beach Nourishment

A-93 Beach nourishment without containment structures is illustrated in Figure A-19. For this design, a construction berm with an elevation of +11' NGVD and 150 feet in width, is created. Approximately 200,000 cubic yards of beach fill will be placed. The sand is assumed to be acquired via a 4,000 cubic yard hopper dredge from Borrow Area IV, seaward of Shinnecock Inlet, as identified in the Fire Island to Montauk Point Reformulation Study.

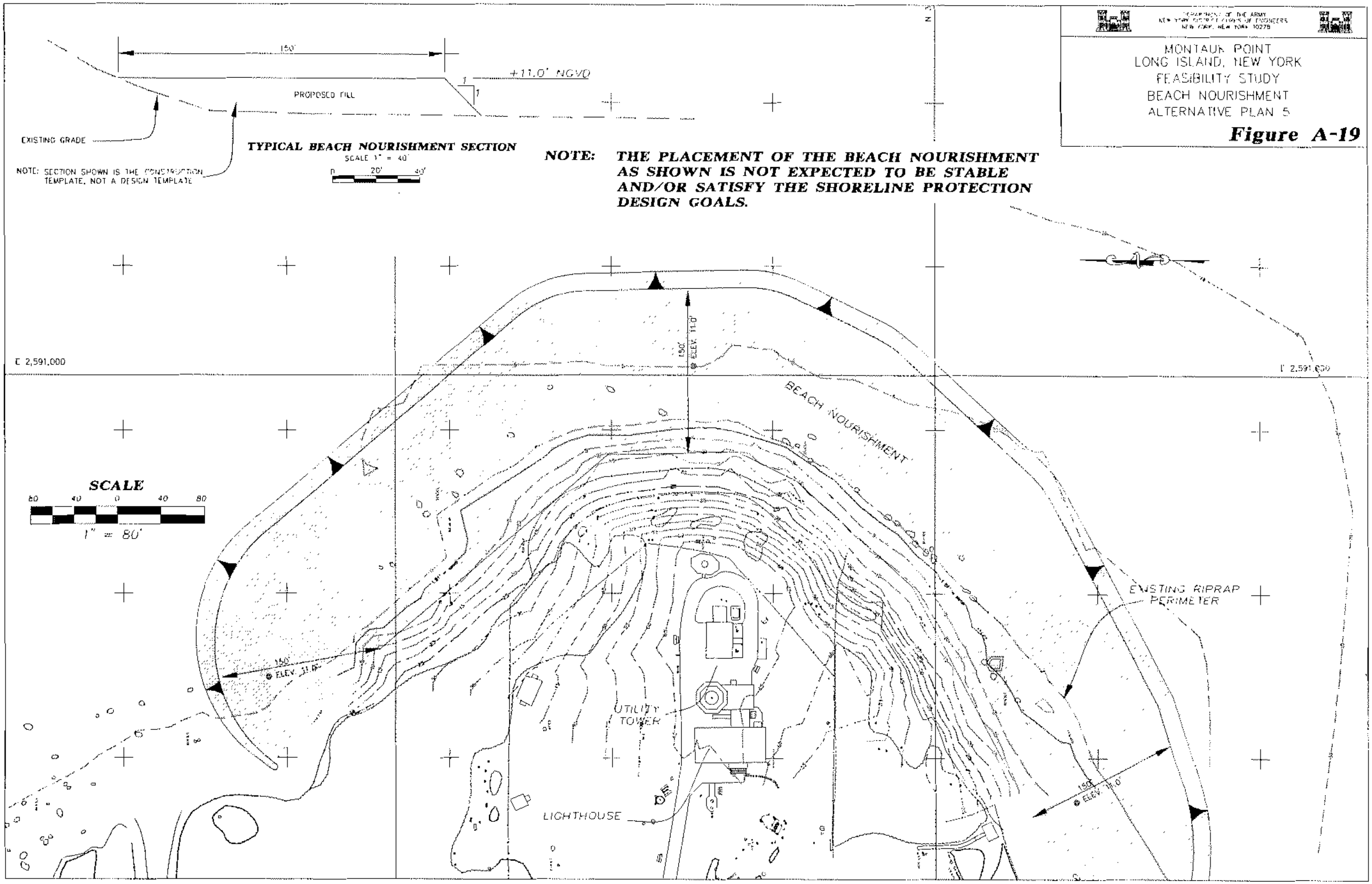
A-94 This alternative is considered not feasible for many reasons. High longshore transport rates will remove the fill rapidly at an unpredictable rate and the area will require constant renourishment. A berm at +11' NGVD will provide some short term reduction in the recession of the toe of the bluff, but will not impede higher water levels and waves from impacting the bluff face and therefore will not provide adequate storm damage protection. Seasonal beach surveys (potentially monthly) will be required during the first two to three years after construction to refine the design of the beach fill cross section and to estimate the renourishment requirements. It expected that a beach nourishment project will change surfing conditions in the area by reducing wave reflection characteristics from the existing stone structures and by filling out the offshore beach profile to a more gradual slope. Because of the lack of adequate storm damage protection, this beach fill alternative will not be considered further.

PRELIMINARY ALTERNATIVE 6 – Relocation of the Lighthouse

A-95 Moving the Montauk Point Light Station, a National Register listed property, would preserve the existing structures, but allow for the eventual destruction of the bluff. Prior to the relocation of the existing buildings, the arrangement and relationships of the structures on the landscape as well as the view to and from the lighthouse and bluff would be documented. In addition, subsurface archaeological investigations would be required at the current site as well as at the new lighthouse location.

A-96 The moving of the lighthouse itself is a precarious task at best. Unlike the Cape Hatteras Lighthouse (which rested on a relatively flat, level surface that permitted the National Park Service to move the structure for a cost of approximately $12 Million), the Montauk Point Lighthouse rests upon a hill on top of the bluff. Raised grades would have to be built to raise the level of the ground to the west of the bluff up to the lighthouse grade to ensure a stable move.

A-97 The preliminary estimated cost for moving the Montauk Point Lighthouse and undertaking the required archaeological investigations would be approximately $19,500.000. This figure does not take into account the creation of raised grades landward of the present location of the lighthouse for the move, which could add an additional cost of $8,600,000 and reduce parking facilities. The overall project would take approximately five years to complete, with a total cost of $26,800,000.


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
BEACH NOURISHMENT
ALTERNATIVE PLAN 5
150'
+11.0' NGVD
1
1
PROPOSED FILL
EXISTING GRADE
NOTE: SECTION SHOWN IS THE CONSTRUCTION TEMPLATE, NOT A DESIGN TEMPLATE
TYPICAL BEACH NOURISHMENT SECTION
SCALE 1" = 40'
0
20'
40'
NOTE: THE PLACEMENT OF THE BEACH NOURISHMENT AS SHOWN IS NOT EXPECTED TO BE STABLE AND/OR SATISFY THE SHORELINE PROTECTION DESIGN GOALS.
E 2,591,000
150'
@ ELEV. 11.0'
BEACH NOURISHMENT
SCALE
80
40
0
40
80
1" = 80'
EXISTING RIPRAP PERIMETER
UTILITY TOWER
LIGHTHOUSE
Figure A-19

10.3 Selected Preliminary Alternative – Stone Revetment

A-98 A summary of the estimated first cost and annual cost of each of the structural alternatives is presented in Table A-12. Based on the advantages and disadvantages of each of the alternatives discussed above and the estimated costs of construction and periodic nourishment required with the offshore breakwater and T-groin alternatives, the selected plan for protection of Montauk Point and the lighthouse complex is the construction of a stone revetment as shown in Figure 16. As shown in Table A-12, the revetment alternative has the lowest annual cost of the alternatives considered. As discussed previously, revetments are a proven method of shore protection in this area and have a record of acceptance by state and local agencies. By re-using some of the stone already on site in the existing structure, cost savings will be realized.

A-99 Preliminary design variations in the revetment cross-section were considered to evaluate the impacts on construction costs. The cross-section of the preliminary revetment alternative, Alternative 2, shown in Figure A-16 is developed at a 73-year level of protection consistent with the level of protection afforded by all structural alternatives. It consists of the construction of a revetment section with a crest width of 40' at elevation +25' NGVD, 1V:2H side slopes, and 12.6-ton quarrystone armor units extending from the crest down to the embedded toe. The design wave for this structure is $H_{73\ Yr.}$ = 13.4' calculated using Figure 7-4 (SPM 1984) and $DSWL_{73\ Yr}$ = +10.94' NGVD. A heavily embedded toe is incorporated to protect against breaking waves and scour at the toe of the structure. The embedded toe was designed in accordance with EM 1110-2-1614 entitled "Design of Coastal Revetments, Seawalls and Bulkheads" (1995). Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure. Sublayers are specified in accordance with standard design procedures. The estimated first cost for the selected preliminary alternative, the stone revetment, is $14,843,000 as shown in Table A-12.

A-100 For a breakdown of the stone revetment design, please refer to Sub-Appendix A-4, Design Calculations.

PRELIMINARY CONSTRUCTION COST ESTIMATE
October 2004 Price Level

ALTERNATIVE 2 - Stone Revetment

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	CONTING	TOTALS
Breakwaters & Seawalls (Revetment)						
Mob.Demob	1	Job	L.S	$200,000	$40,000	
Armor Stone(12.6ton) - New	51,000	TON	$110.68	$5,644,680	$1,128,936	
Armor Stone(4.5ton) - Rehandled	18,500	TON	$58.78	$1,087,430	$217,486	
Underlayer(1.3ton)-New	20,300	TON	$111.42	$2,261,826	$452,365	
Bedding Stone - New	12,200	TON	$94.76	$1,156,072	$231,214	
Excavation	34,300	CY	$16.08	$551,544	$110,309	
Filter Cloth	12,700	SY	$6.44	$81,805	$16,361	
SUBTOTAL						
				$10,983,357		
CONTINGENCY @ 20%					$2,196,671	
TOTAL BREAKWATERS & SEAWALLS	(Revetment)					$13,180,028
ENGINEERING AND DESIGN				$450,000	$90,000	$540,000
CONSTRUCTION MANAGEMENT				$935,000	$187,000	$1,122,000
TOTAL FIRST COST						$14,843,000
INTEREST DURING CONSTRUCTION						
(30 Months @ 5.375%)						$949,000
TOTAL INVESTMENT COST						$15,792,000
ANNUALIZED INVESTMENT COST						$915,630
(Based on 50 Year Design Life and Annual Interest of 5.375%)						
ANNUALIZED MAINTENANCE COST						$54,917
TOTAL ANNUAL COST						$970,547
					Rounded	$971,000

Table A-9. Stone Revetment Preliminary Cost Estimate

PRELIMINARY CONSTRUCTION COST ESTIMATE
October 2004 Price Level

ALTERNATIVE 3 - OFFSHORE BREAKWATER AND BEACHFILL

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	CONTING	TOTALS
Breakwaters & Seawalls						
Mob.Demob	1	Job	L.S	$800,000	$160,000	
Armor Stone(17.5ton) - New	41,200	TON	$110.68	$4,560,016	$912,003	
Underlayer(1.75ton)-New	7,800	TON	$111.42	$869,076	$173,815	
Bedding Stone - New	12,200	TON	$94.76	$1,156,072	$231,214	
Filter Cloth	11,200	SY	$6.44	$72,143	$14,429	
Sand Fill	200,000	CY	$16.00	$3,200,000	$640,000	
Repair Existing Revetment Above El +12.0	5000	TON	$110.68	$553,400	$110,680	
SUBTOTAL						
				$10,657,307		
CONTINGENCY @ 20%					$2,131,461	
TOTAL BREAKWATERS & SEAWALLS						$12,788,768
ENGINEERING AND DESIGN				$500,000	$100,000	$600,000
CONSTRUCTION MANAGEMENT				$910,000	$182,000	$1,092,000
TOTAL FIRST COST						$14,481,000
INTEREST DURING CONSTRUCTION						$752,000
(24 Months @ 5.375%)						
TOTAL INVESTMENT COST						$15,233,000
ANNUALIZED INVESTMENT COST						$884,000
(Based on 50 Year Design Life and Annual Interest of 5.375%)						
ANNUALIZED MAINTENANCE COST						$56,054
ANNUALIZED PERIODIC NOURISHMENT COST						$502,000
(Based on 50 Year Design Life, Annual Interest of 5.375% and 100,000 cy. Nourishment Every 3 yrs)						
TOTAL ANNUAL COST						$1,442,054
					Rounded	$1,443,000

Table A-10. Offshore Breakwater Preliminary Cost Estimate

PRELIMINARY CONSTRUCTION COST ESTIMATE
October 2004 Price Level

ALTERNATIVE 4 - T GROINS AND BEACH NOURISHMENT

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT	CONTING	TOTALS
Breakwaters & Seawalls (T Groins)						
Mob.Demob	1	Job	L.S	$100,000	$20,000	
Armor Stone(17.5ton) - Breakwaters	26,500	TON	$110.68	$2,933,020	$586,604	
Armor Stone(4.5ton) - Groin	12,100	TON	$110.68	$1,339,228	$267,846	
Underlayer(1.75ton) - Breakwater	5,300	TON	$111.42	$590,526	$118,105	
Underlayer (900 lb) - Groin	8,500	TON	$111.42	$947,070	$189,414	
Bedding Stone - Breakwater	8,400	TON	$94.76	$795,984	$159,197	
Filter Cloth	16,100	SY	$6.44	$103,705	$20,741	
Sand Fill	125000	CY	$16.00	$2,000,000	$400,000	
Repair Existing Revetment Above El +12.0	5000	TON	$110.68	$553,400	$110,680	
SUBTOTAL						
				$8,809,533		
CONTINGENCY @ 20%					$1,761,907	
TOTAL BREAKWATERS & SEAWALLS	(Breakwater)					$10,571,440
ENGINEERING AND DESIGN				$500,000	$100,000	$600,000
CONSTRUCTION MANAGEMENT				$768,000	$153,600	$921,600
TOTAL FIRST COST						$12,094,000
INTEREST DURING CONSTRUCTION (24 Months @ 5.375%)						$629,000
TOTAL INVESTMENT COST						$12,723,000
ANNUALIZED TOTAL INVESTMENT COST (Based on 50 Year Design Life and Annual Interest of 5.375%)						$738,000
ANNUALIZED MAINTENANCE COST						$46,815
ANNUALIZED PERIODIC NOURISHMENT COST (Based on 50 Year Design Life, Annual Interest of 5.375% and 100,000 cy. Nourishment Every 3 yrs)						$502,000
TOTAL ANNUAL COST						$1,286,815
					Rounded	$1,287,000

Table A-11. T-Groins and Beach Nourishment Preliminary Cost Estimate

PRELIMINARY CONSTRUCTION COST ESTIMATE
October 2004 Price Level

FIRST COST AND ANNUAL COST SUMMARY

	ALTERNATIVE 2 STONE REVETMENT	ALTERNATIVE 3 OFFSHORE BREAKWATER AND BEACH FILL	ALTERNATIVE 4 T GROINS AND BEACH FILL
TOTAL FIRST COST	$14,843,000	$14,481,000	$12,094,000
INTEREST DURING CONSTRUCTION (@ 5.375%)	$949,000	$752,000	$629,000
TOTAL INVESTMENT COST	$15,792,000	$15,233,000	$12,723,000
ANNUALIZED TOTAL INVESTMENT COST (Based on 50 Year Design Life and Annual Interest of 5.375%)	$915,600	$884,000	$738,000
ANNUALIZED MAINTENANCE COST	$55,000	$57,000	$47,000
ANNUALIZED PERIODIC NOURISHMENT COST (Based on 50 Year Design Life, Annual Interest of 6.125% and 100,000 cy. Nourishment Every 3 yrs)	$0	$502,000	$502,000
TOTAL ANNUAL COST	$971,000	$1,443,000	$1,287,000

Table A-12. First Cost and Annual Cost Summary

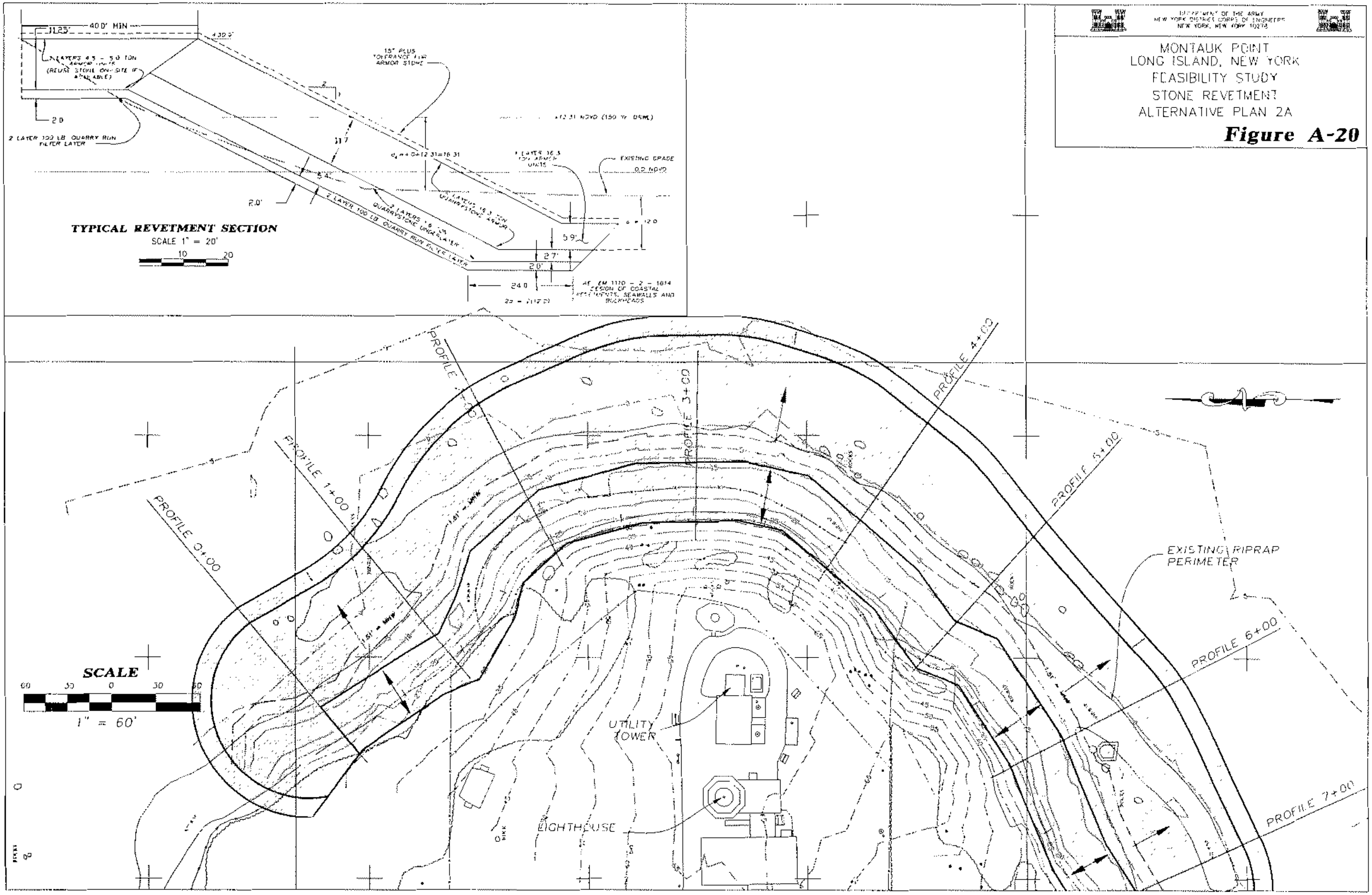
10.4 Final Improvement Designs

A-101 For the three alternative revetment sizes developed as part of the optimization, the higher two levels of protection have a heavily embedded toe to protect against breaking waves and scour at the base of the structure. Embedded toe design will be in accordance with EM 1110-2-1614 entitled "Design of Coastal Revetments, Seawalls and Bulkheads" (1995). Sublayers are specified in accordance with standard design procedures. It is noted that because the revetment improvement is founded on dense till or stone, no filter cloth is required to underlie the improvement. This is a design refinement from the preliminary design, where filter cloth was included. In addition, the following three refinements to the preliminary revetment alternative were made: (1) the quantities changed slightly based on additional cross sections taken. (2) The mobilization and demobilization costs increased to include temporary construction berms at each end of the revetment to facilitate revetment construction. (3) Contingency reduced due to more detailed level of design. The following describes the three variations of the revetment alternative used in order to optimize the design.

Alternative 2A: Stone Revetment with 150-year Level of Protection - The design wave for the structure is $H_{150\ Yr.}$ = 14.6' based on the average toe el. near the improved revetment toe of el. –4' NGVD calculated using Figure 7-4 (SPM 1984) and $DSWL_{150\ Yr}$ = +12.31' NGVD. The cross-section of the revetment shown in Figure A-20 consists of the construction of a revetment section with a crest width of 40' at elevation +30' NGVD, 1V:2H side slopes, and 16.3-ton quarrystone armor units extending from the crest down to the embedded toe. According to Engineering Manual guidance described above, the bottom of the armor stone layer in the toe is located 12 ft. below existing grade at the toe (the stone crest at approximately –10' with the average toe el. at el. –4 NGVD). Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure. Cross-sections of this revetment alternative along the existing profiles are shown in Figure A-21.

Alternative 2B: Stone Revetment with 73-year Level of Protection - The design wave for the structure is $H_{73\ Yr.}$ = 13.4' based on the average toe el. near the improved revetment toe of el. –4' NGVD calculated using Figure 7-4 (SPM 1984) and $DSWL_{73\ Yr}$ = +10.94' NGVD. The cross-section of the revetment shown in Figure A-22 consists of the construction of a revetment section with a crest width of 40' at elevation +25' NGVD, 1V:2H side slopes, and 12.6-ton quarrystone armor units extending from the crest down to the embedded toe. The bottom of the armor stone layer in the toe is located 12 ft. below existing grade at the toe (average toe el. at –4' NGVD). Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones can be re-used in the proposed revetment from the present structure. Cross-sections of this revetment alternative along the existing profiles are shown in Figure A-23.

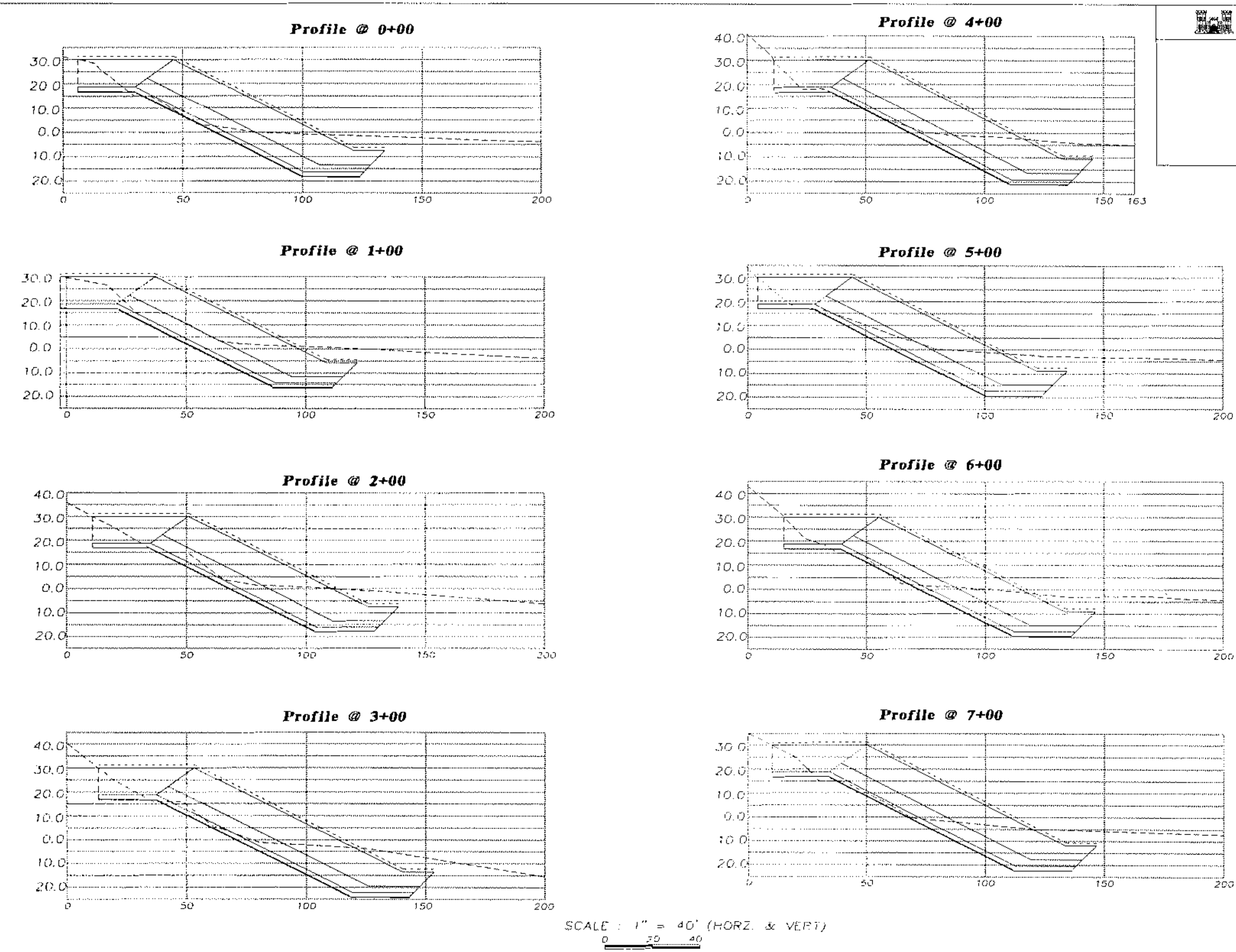
Alternative 2C: Stone Revetment with 15-year Level of Protection - The design wave for the structure is $H_{15\ Yr.}$ = 9.2' based pm the average toe el. near the improved toe of el. –1' NGVD calculated using Figure 7-4 (SPM 1984) and $DSWL_{15\ Yr}$ = +9.05' NGVD. The cross-section of the revetment shown in Figure A-24 consists of a revetment section with a crest width of 3' at elevation +25' NGVD, 1V:1.5H side slopes, and 7.5-ton quarrystone armor units extending from the crest down to the toe. The toe will be built up from the existing toe with large stone and will not require an excavated buried toe. It is assumed that some stones can be re-used in the proposed revetment from the present structure. Cross-sections of this revetment alternative along the existing profiles are shown in Figure A-25. Since the costly buried toe is not essential for the 15-year level of protection, a narrow berm was developed to provide better foundation on the existing toe stone. In order to construct the narrow berm, an offshore adjacent rubble mound stone temporary structure will be required from which land-based construction equipment will operate.


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2A
Figure A-20
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
40.0' MIN
2 LAYER 100 LB QUARRY RUN FILTER LAYER
EXISTING GRADE
24.0
SCALE
1" = 60'
PROFILE 0+00
PROFILE 1+00
PROFILE
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
EXISTING RIPRAP PERIMETER
UTILITY TOWER
LIGHTHOUSE

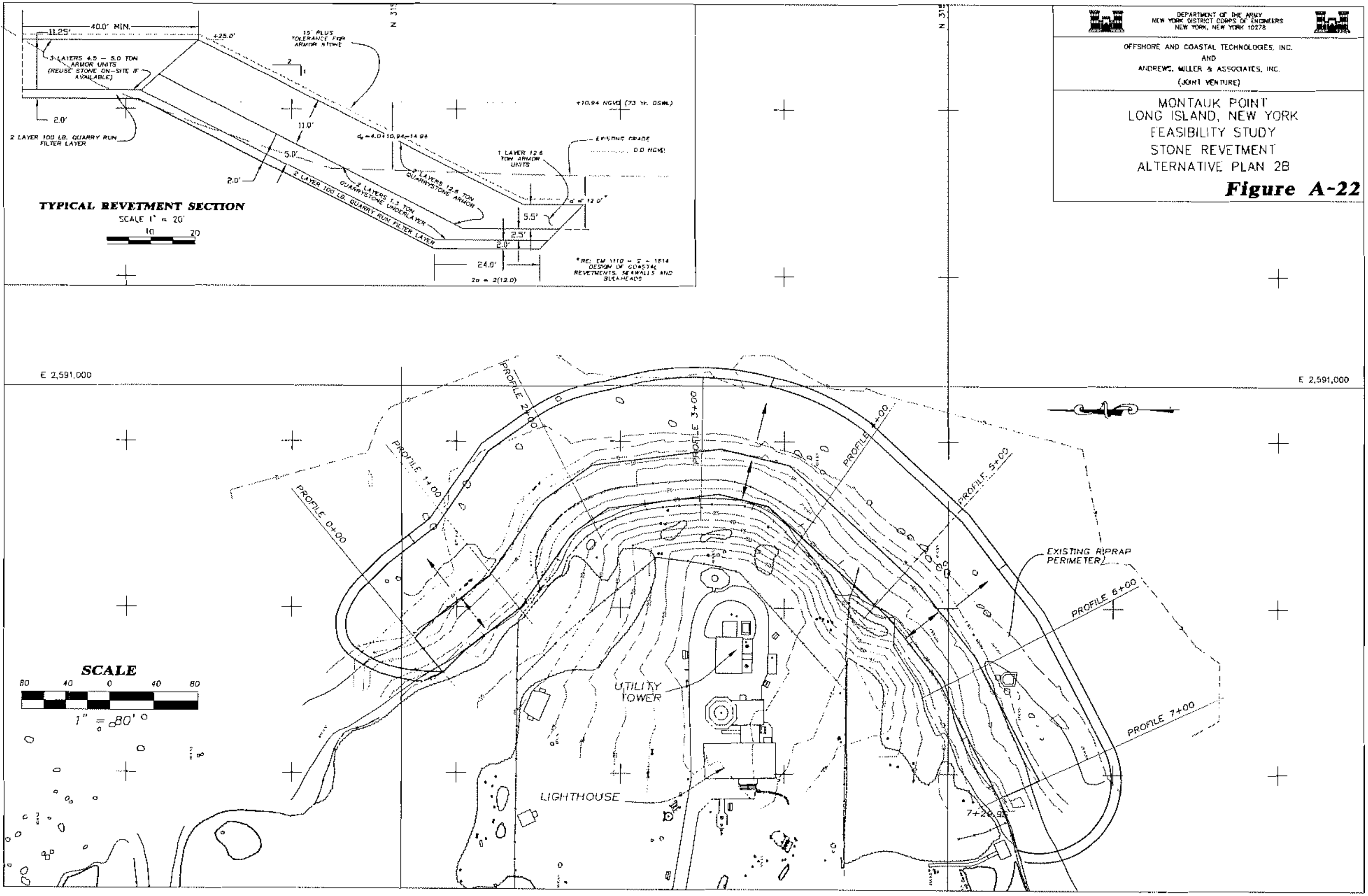
DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278

MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2A
PROFILES

Figure A-21



NOTE: SEE TYPICAL REVETMENT SECTION, FIGURE A-20.

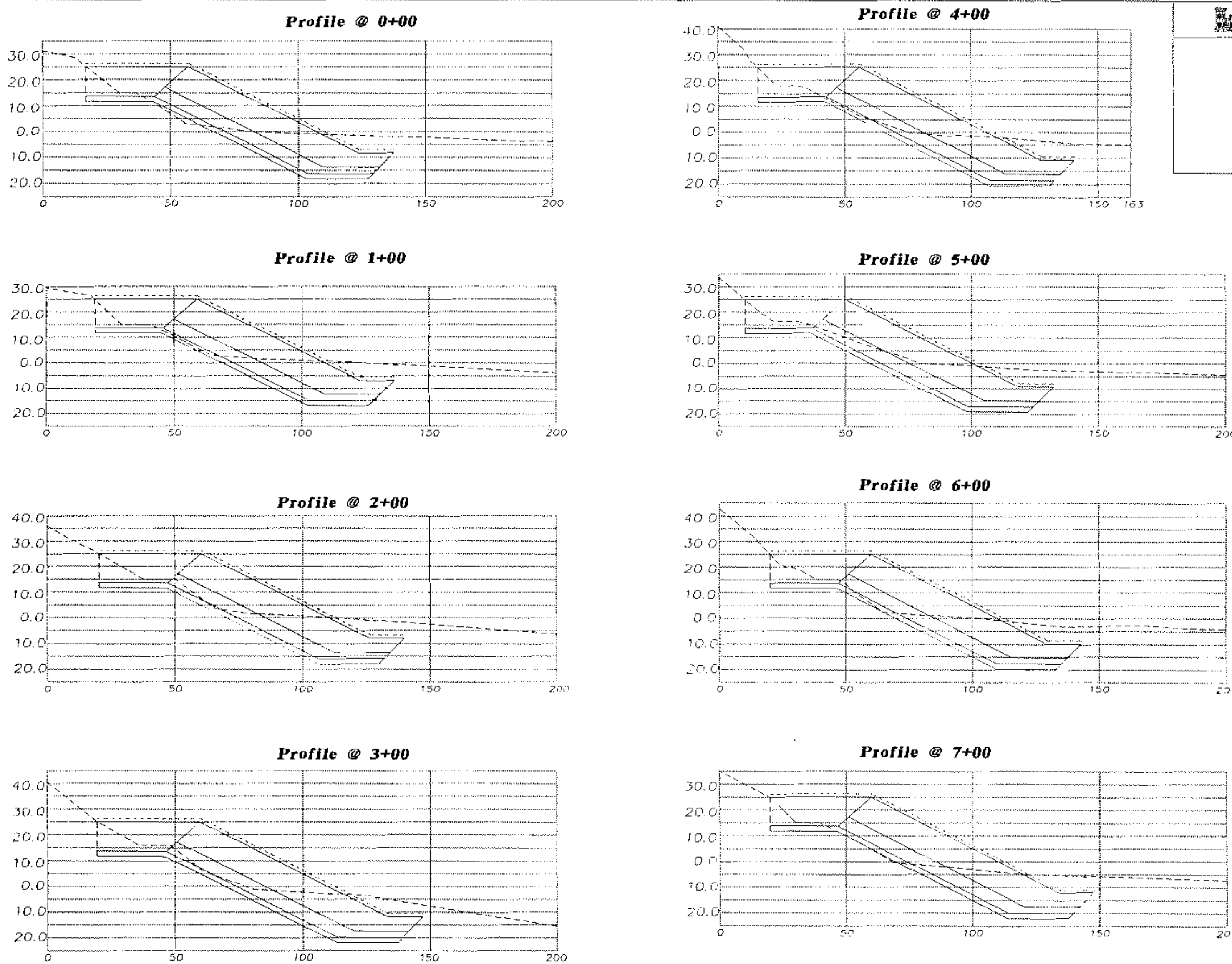


DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278

OFFSHORE AND COASTAL TECHNOLOGIES, INC.
AND
ANDREWS, MILLER & ASSOCIATES, INC.
(JOINT VENTURE)

MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE PLAN 2B

Figure A-22



DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278

MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2B
PROFILES

Figure A-23

NOTE: SEE TYPICAL REVETMENT SECTION, FIGURE A-22.

SCALE : 1" = 40' (HORZ & VERT)

0 20 40

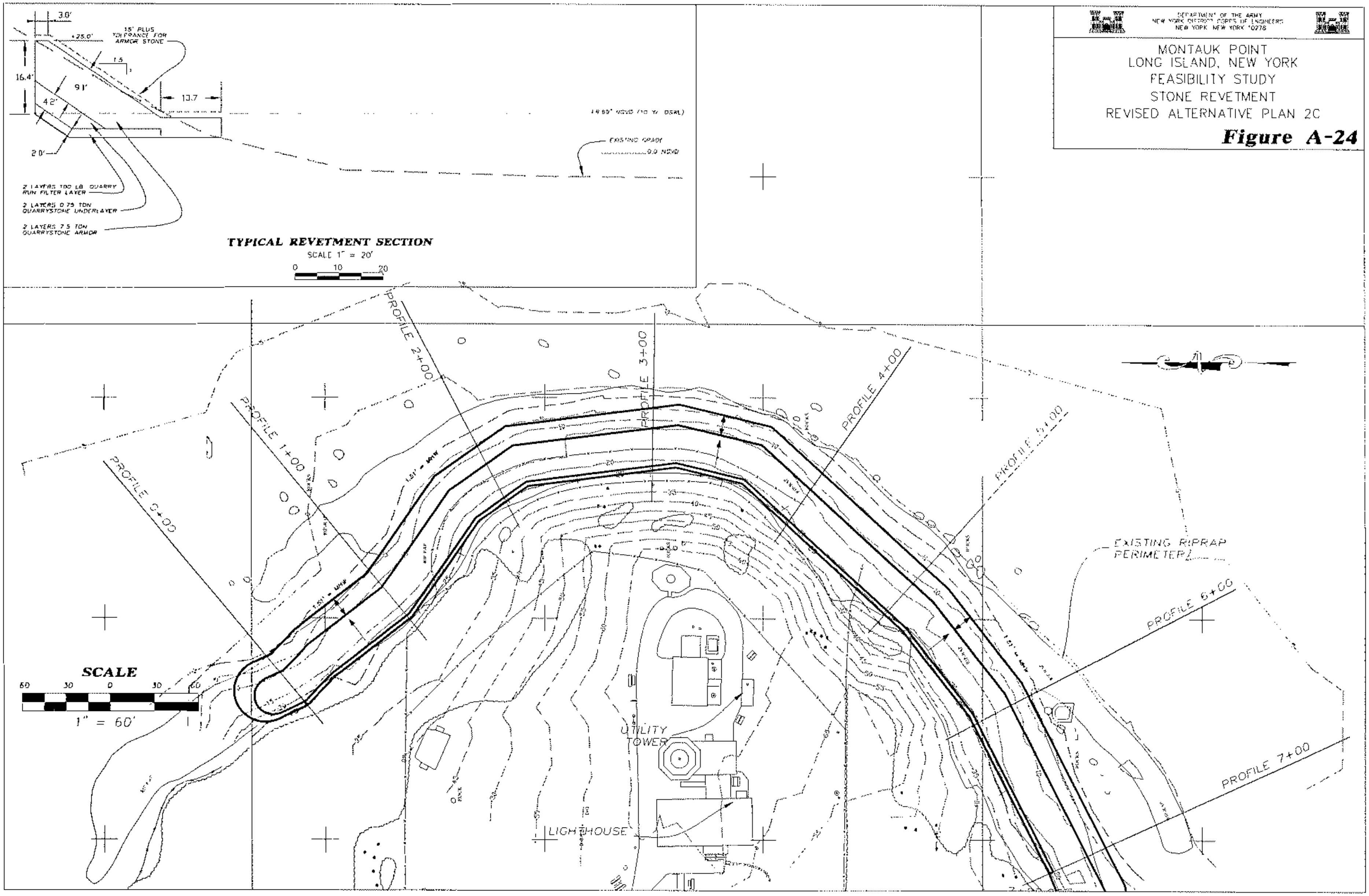

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278
MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
REVISED ALTERNATIVE PLAN 2C
TYPICAL REVETMENT SECTION
SCALE 1" = 20'
15' PLUS TOLERANCE FOR ARMOR STONE
2 LAYERS 100 LB. QUARRY RUN FILTER LAYER
2 LAYERS 0.75 TON QUARRYSTONE UNDERLAYER
2 LAYERS 7.5 TON QUARRYSTONE ARMOR
EXISTING GRADE
SCALE
1" = 60'
PROFILE 0+00
PROFILE 1+00
PROFILE 2+00
PROFILE 3+00
PROFILE 4+00
PROFILE 5+00
PROFILE 6+00
PROFILE 7+00
EXISTING RIPRAP PERIMETER
UTILITY TOWER
LIGHTHOUSE

Figure A-24

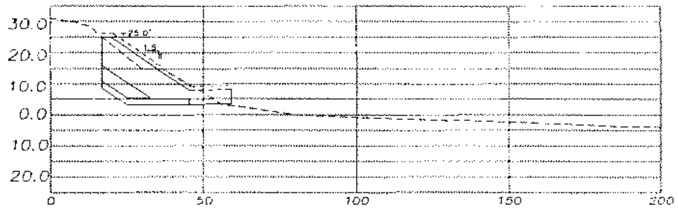
DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
NEW YORK, NEW YORK 10278

MONTAUK POINT
LONG ISLAND, NEW YORK
FEASIBILITY STUDY
STONE REVETMENT
ALTERNATIVE 2C
PROFILES

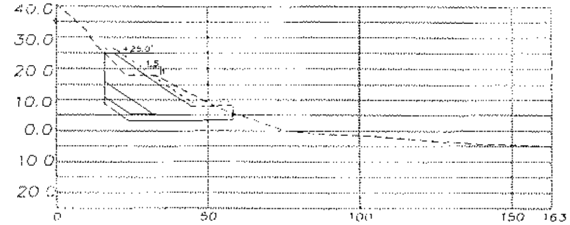
Figure A-25

NOTE. SEE TYPICAL REVETMENT SECTION, FIGURE A-24.

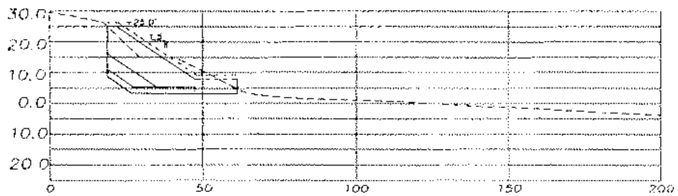
Profile @ 0+00



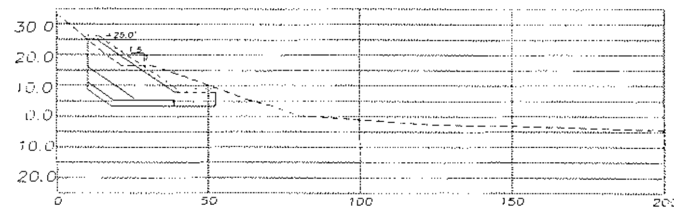
Profile @ 4+00



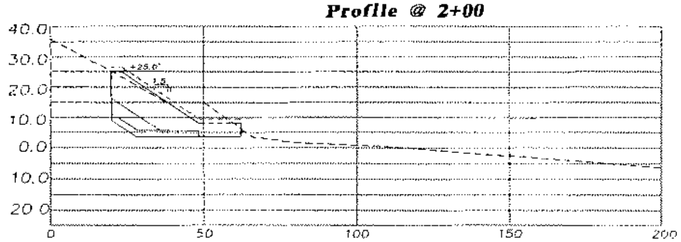
Profile @ 1+00



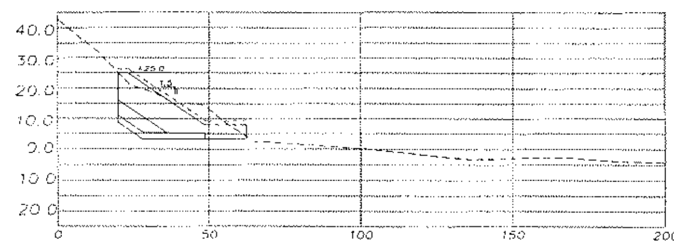
Profile @ 5+00



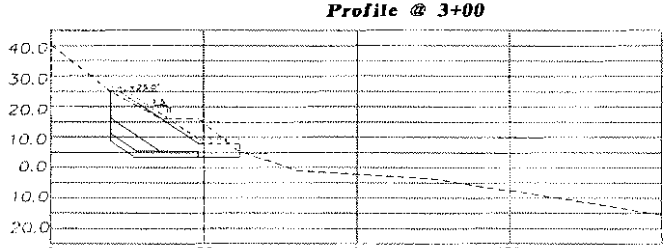
Profile @ 2+00



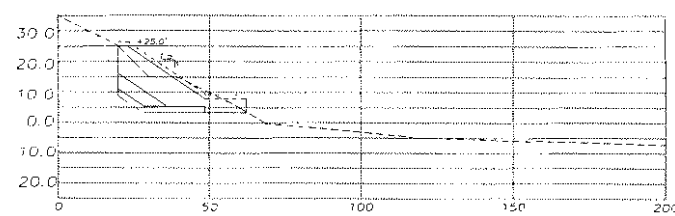
Profile @ 6+00



Profile @ 3+00



Profile @ 7+00



SCALE : 1" = 40' (HORZ & VERT)

0 20 40

10.5 Coastal Analysis of Improvement Plans

A-102 According to the Coastal Engineering Manual (EM 1110-2-1100, Part IV, Draft 30 Sep 01), design conditions for coastal structures require acceptable levels of hydraulic responses in terms of wave runup, overtopping, scour and reflection.

A-103 *Wave runup* level is one of the most important factors affecting the design of coastal structures because it determines the design crest level of the structure that limits wave overtopping.

A-104 Wave runup is calculated according to the methods outlined in the Coastal Engineering Manual (Draft 30 September 01). The method used here is described in Section VI-5-2 of that document. First, through the calculation of surf similarity parameter, assuming irregular waves, the 2% runup is calculated using the formula in the CEM on Figure VI-5-3. Surf similarity parameters at Montauk exceed a value of 4.0 for all storm events examined, so surface roughness effects on wave runup are negligible.

The surf-similarity parameter is a number that is related to the type of breaking wave. For irregular waves it is defined as:

$$\xi_{op} = \frac{\tan ;}{\sqrt{S_{op}}}$$

Where: $$S_{om} = \frac{2\pi}{g} \frac{Hs}{Tp}$$

Hs = significant wave height at the structure toe

Tp = wave period at peak of wave spectrum

g = acceleration due to gravity

$;$ = bottom slope

For: ξ . breaking waves are spilling

$.5 < \xi_{op} < 3$ breaking waves are plunging

$3 < \xi_{op} < 3.5$ breaking waves are collapsing

$\xi_{op} > 3.5$ breaking waves are surging

A-105 To account for the composite slope conditions that are created by the presence of the berm at the top of the revetment (existing and proposed), the method of de Waal and van der Meer is used as given on Page VI-5-12 of the CEM. A factor, gamma, is determined from two other factors that account for the width of the berm and the elevation of the berm relative to the water level.

A-106 To examine the effect of permeability on wave runup, the CEM presents data from Delft Hydraulics in Table VI-5-12. The figure presents two best-fit lines through laboratory data for the ratio of the two-percent runup to the wave height as a function of surf similarity parameter. The percentage reduction between the two lines is used here to scale down the runup due to expected structural permeability. Because of the scatter in the underlying data, a reduction factor of 0.74 was used for all cases examined here.

A-107 Once the runup magnitudes are calculated they are added to the still water level corresponding to each recurrence level to determine absolute runup elevations relative to the project datum, NGVD29.

A-108 Table A-13 presents the runup elevations for the Final Improvement Plans and the existing structure. The presence of the berm at a lower elevation (as with the existing revetment) and steeper composite slope (from a relatively narrow berm and shallow toe depth) reduces the runup, but not the overtopping rate above the berm crest, due to the large berm crest width of the improvement. The effect of a steeper slope and shallower structure toe cause the runup elevations associated with Plan 2C to be lower than those for the other plans. The calculations indicate that runup elevations exceed the existing revetment crest (+18 feet NGVD) at all listed return periods using maximum design wave conditions. Field observations confirm that 'green water' frequently reaches the top of the revetment. Figure A-26 also confirms that wave runup (from the highest segment of the wave group for the more frequent storms, all the way to nearly all the waves on the 73 year return period storm) exceeds the crest elevation of the existing structure, from the 2 year thru the 500 year storm, even when permeability is accounted for.

A-109 *Wave overtopping* occurs when the structure crest height is lower than the runup level. Overtopping discharge is a very important design parameter because it determines the crest level and the design of the upper part of the structure. In the Montauk Point case, overtopping must be limited at design storm levels so as to avoid failure of the revetment from the top (as observed in the field and in the model test for the existing structure).

A-110 Critical levels of average overtopping discharges are provided in Table VI-5-6 of the CEM. The relevant critical levels (based on Coastal Engineering Manual criteria from physical modeling of damages sustained with paved and unpaved revetments) at Montauk are 100 liters/s/m (0.1 cu m/s/m). This is a critical threshold for damage of vegetative terracing immediately above the revetment stone; however, lower levels of damage can be initiated at the 50 liters/s/m threshold.

A-111 The overtopping rate can be calculated from the many approaches described in the CEM. The situation closest to the Montauk Point structure is presented in Table VI-5-12, which summarizes a formula developed semi-empirically by Pedersen (1996) for layered, permeable, rock-armored slopes with a berm in front of a crown wall (in this case analogous to the bluff atop the revetment). Table A-14 presents overtopping rates calculated using the Pedersen method outlined in the CEM on Table VI-5-12.

A-112 The results show that the critical level for significant damage initiation of the vegetative terracing is exceeded above a 200-year event for Plan 2A, a 100-year event for Plan 2B and greater than a 15-year event for Plan 2C. The existing structure exhibits damaging overtopping rates during events greater than a 10-year level.

A-113 *Wave reflection* affects the nearshore wave conditions immediately fronting the structure, and potentially along neighboring beaches. Incident energy is partly dissipated by wave breaking, surface roughness and porous flow through the stone structure.

A-114 The CEM equation VI-5-38 for wave reflection was originally formulated by Seelig (1983) and improved with coefficients for 1-layer and 2-layer rock structures with an underlayer by Allsop (1990).

The reflection coefficient is defined as:

$$C_r = \frac{a\xi_{op}^2}{b+\xi_{op}^2}$$

where:

ξ_{op} = surf-similarity parameter

a = 0.64 for 1- or 2- layer structures

b = 7.22 for 1- layer of armor on stone underlayer

= 8.85 for 2- layers of armor on stone underlayer.

A-115 Allsop's coefficients are valid within the range of surf similarity parameters that occur during storms at Montauk Point. Table A-15 presents a comparison of reflection coefficients for the three Improvement Plans and the existing structure.

A-116 The wave reflection coefficients in Table A-15 indicate that the reflected wave will be reduced at all return periods for all three Final Improvement Alternative Plans versus the existing structure because of the flatter structure slopes and more porous rock layering. The reductions range from 13-19% for Plans 2A and 2B to 3-5% for Plan 2C.

A-117 *Wave scour* occurs at the toe of the structure due to the concentration of currents formed by the interaction of incident waves with the down rush from preceding waves. The extensive scour protection toe design included in the Final Improvement Alternative Plans 2A and 2B will prevent adverse scour (including both storm and long term).

A-118 *Adjacent Impacts*. Potential longshore effects include the impact of any new structures on neighboring beaches. Because a revetment similar to the recommended plan has been in place at Montauk Point for nearly 60 years, there is essentially no change from existing adjacent impacts due to implementation of the recommended plan. The sediment that would have become littoral supply to adjacent beaches from the area immediately behind the existing revetment has been stabilized at the Point during its functional life. The replacement of the existing structure with the recommended design would not alter this function. The recommended plan includes appropriate tie-backs on either side to minimize local erosion at the ends of the project due to longshore sediment demand. See Subappendix A-7 for further discussion.

A-119 *Level of Protection*. Based on the analysis of direct wave impact and runup/overtopping damages, Alternative 2A will provide protection from the 150-yr. storm event. During this event, damages to the revetment due to direct wave impact are estimated to be between 0- to 5- percent which is generally referred to as a no-damage condition. Wave overtopping during the 150-yr. storm event is limited to 47 liters/s/m which is significantly below 100 liters/s/m, which is the estimated threshold of significant damage to unpaved promenades. As a measure of uncertainty, if the 150-yr water level is increased to include 0.7 feet of sea level rise in 50 years and ¾ standard deviation of storm surge, the overtopping rate increases to be 118 liters/s/m for the *paved* promenade. This rate is just slightly above the threshold of significant damage to *unpaved* promenades, but much less than the threshold of significant damage to *paved* promenades (200 liters/m/s). Therefore, there is a large safety factor including uncertainty throughout the project life.

A-120 Alternative 2B will provide protection from the 73-yr. storm event. During this event, damages to the revetment due to direct wave impact are estimated to be between 0- to 5- percent (no-damage condition). Wave overtopping during the 73-yr. storm event is limited to 60 litres/s/m which is significantly below 100 litres/s/m, the estimated threshold of damage to *unpaved* promenades. As a measure of uncertainty, if the 73-yr water level is increased to include 0.7 feet of sea level rise in 50 years and ¾ standard deviation of storm surge, the overtopping rate is calculated to be 162 liters/s/m for the *paved* promenade. This rate is less than the threshold of significant damage to *paved* promenades, i.e. 200 liters/s/m. Therefore, including uncertainty throughout the project life, there is a reasonable safety factor (greater than 75% certainty).

A-121 Based on potential runup/overtopping damages, the level of protection provided by Alternative 2C with an *unpaved* promenade, is estimated to be on the order of a 15-yr. storm event. The wave overtopping during this event is estimated to be 70 litres/s/m, which is just below the threshold of damage to *unpaved* promenades. As a measure of uncertainty, if the 15-yr water level is increased to include 0.7 feet of sea level rise and one standard deviation of storm surge increase, the overtopping rate is calculated to be 251 litres/s/m. This yields a 60% probability of significant damage to the *unpaved* promenade (overtopping in excess of 100 litres/s/m) and a 10% probability of structure failure (overtopping in excess of 200 litres/s/m) with uncertainty included.

A-122 Damages to the existing revetment can be expected to continue and will require continued maintenance. The revetment damage maintenance costs are parameters that have been recurring since construction of the existing revetment. The quantitative assessment of wave-induced maintenance costs is based on the records of recent revetment maintenance operations increased to account for increasing damages due to a worsening without project condition and to account for increased damages due to sea level rise.

A-123 The assumptions used in the economic evaluation regarding revetment and bluff damage frequency were based on the results of engineering studies to assess the ability of the alternatives to withstand the design wave conditions in the area and reduce the runup and overtopping along the bluff face. The studies indicate that with Alternatives 2A and 2B, damages to the revetment and the bluff would be reduced significantly and that damages from storm excedence are greatly reduced from Alternative 2C where storm excedence damages are high.

A-124 The economic evaluation of Alternative 2A (150 year storm design level of protection) with a revetment height of +30 feet NGVD considered the impacts of storm events ranging from a 2-yr. event to a 200-yr. event. Wave impact damages are initiated slightly at the 15 year return period storm and overtopping damages are initiated at the 200 year return period storm. The total annual cost of Alternative 2A is estimated to be **$1,050,400**. Refer to the Quantities and Cost Appendix C.

A-125 The economic evaluation of Alternative 2B (73 year storm design level of protection) with a revetment height of +25 feet NGVD considered the impacts of storm events ranging from a 2-yr. event to a 200-yr. event. Wave impact damages are initiated, slightly, at the 5 year storm event and minor overtopping damages are initiated at the 73 year storm event. The total annual cost of Alternative 2B is estimated to be **$889,300**. Refer to the Quantities and Cost Appendix C for detail cost tables.

A-126 The economic evaluation of Alternative 2C (15 year storm design level of protection) with a revetment height of +25 feet NGVD considered the impacts of storm events ranging from a 2-yr. event to a 200-yr. event. Wave impact damage is initiated slightly at the 2 year return period storm and overtopping damage is initiated at the 15 year return period storm. The total annual cost of Alternative 2C is estimated to be **$524,700**. Refer to the Quantities and Cost Appendix C for detail cost tables.

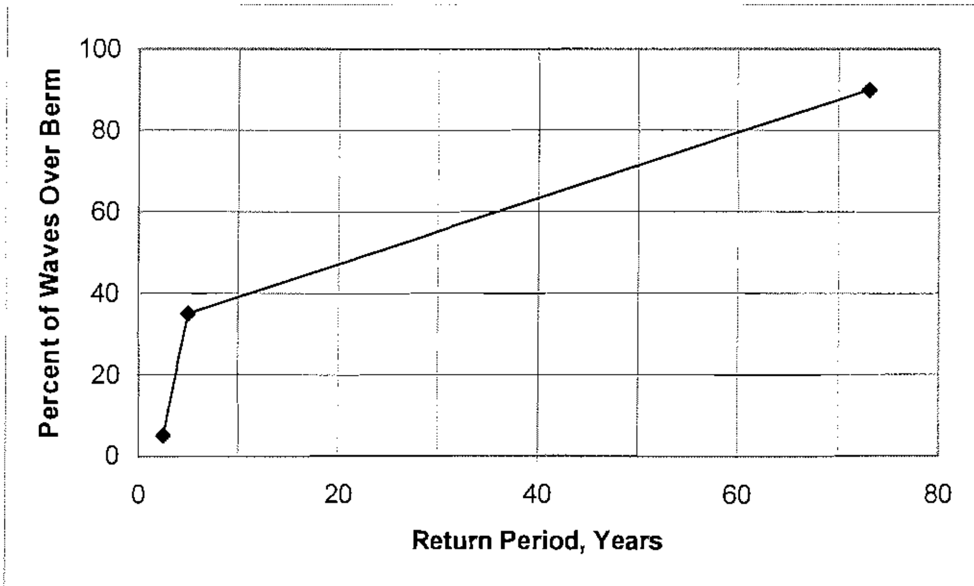


Figure A-26. Percent of wave runup exceeding the existing berm elevation based upon physical model test studies performed for this feasibility study.

Table A-13. Runup Elevations for Existing Revetment and Improvement Plans

RETURN PERIOD	RUNUP ELEVATION (PERMEABLE) FT, NGVD			
(Years)	Existing	Plan 2A	Plan 2B	Plan 2C
2	22.20	33.18	33.18	23.78
5	23.42	36.73	36.73	27.21
10	23.35	38.20	38.20	28.49
25	23.69	40.19	37.85	30.22
44	24.46	42.13	37.70	31.97
50	24.49	42.31	37.62	32.10
73	25.39	44.05	37.85	33.39
100	26.43	44.38	38.32	34.75
150	27.98	44.51	39.21	36.69
200	29.16	44.78	40.00	38.12
500	32.02	45.91	42.30	41.52

Table A-14. Overtopping Rates for Existing Revetment and Improvement Plans

RETURN PERIOD	OVERTOPPING RATES (LITERS/S/M)			
(Years)	Existing	Plan 2A	Plan 2B	Plan 2C
2	17	1	2	12
5	41	1	4	24
10	90	3	8	48
25	266	6	18	120
44	589	10	33	227
50	728	11	37	274
73	1430	18	60	460
100	2903	27	96	780
150	7479	47	175	1517
200	16221	70	280	2560
500	130783	223	1060	8073

Table A-15. Reflection Coefficients for Existing Revetment and Improvement Plans

RETURN PERIOD	REFLECTION COEFFICIENT		
(Years)	Existing Structure	Plans 2A and 2B	Plan 2C
2	0.57	0.46	0.54
5	0.57	0.45	0.54
10	0.57	0.47	0.55
25	0.58	0.48	0.56
44	0.59	0.49	0.56
50	0.59	0.50	0.56
73	0.59	0.50	0.57
100	0.59	0.50	0.57
150	0.59	0.51	0.57
200	0.59	0.51	0.57
500	0.60	0.52	0.58

10.6 Slope Stability Analysis of Improvement Plans

A-128 Slope stability analysis was performed on Alternative 2B to evaluate "with project " conditions. Figure A-27 shows that the Factor of Safety for the critical failure surface is 1.46 through the revetment and 1.202 in the bluff above the revetment. This alternative was then examined for toe of slope saturation due to wave runup for a 100-year return period. The Factor of Safety for the critical failure surface through the revetment remained the same. The Factor of Safety for the critical failure surface through the bluff above the revetment decreased to 1.103 indicating that, for design storm exceedance, some repair above the revetment may be needed.

11. Monitoring

A-129 Monitoring of the revetment, as part of non-Federal maintenance of the structure, is required throughout the life of the project, i.e. 50 years, to assure that the revetment remains as built and is functioning properly with no flanking at each end and no stone displacement. This monitoring should be accomplished by on-site inspections regularly throughout the year. Such inspections are part of the existing operating practice for the site, and it is assumed that these will be continued throughout the project life at no additional cost to the project.

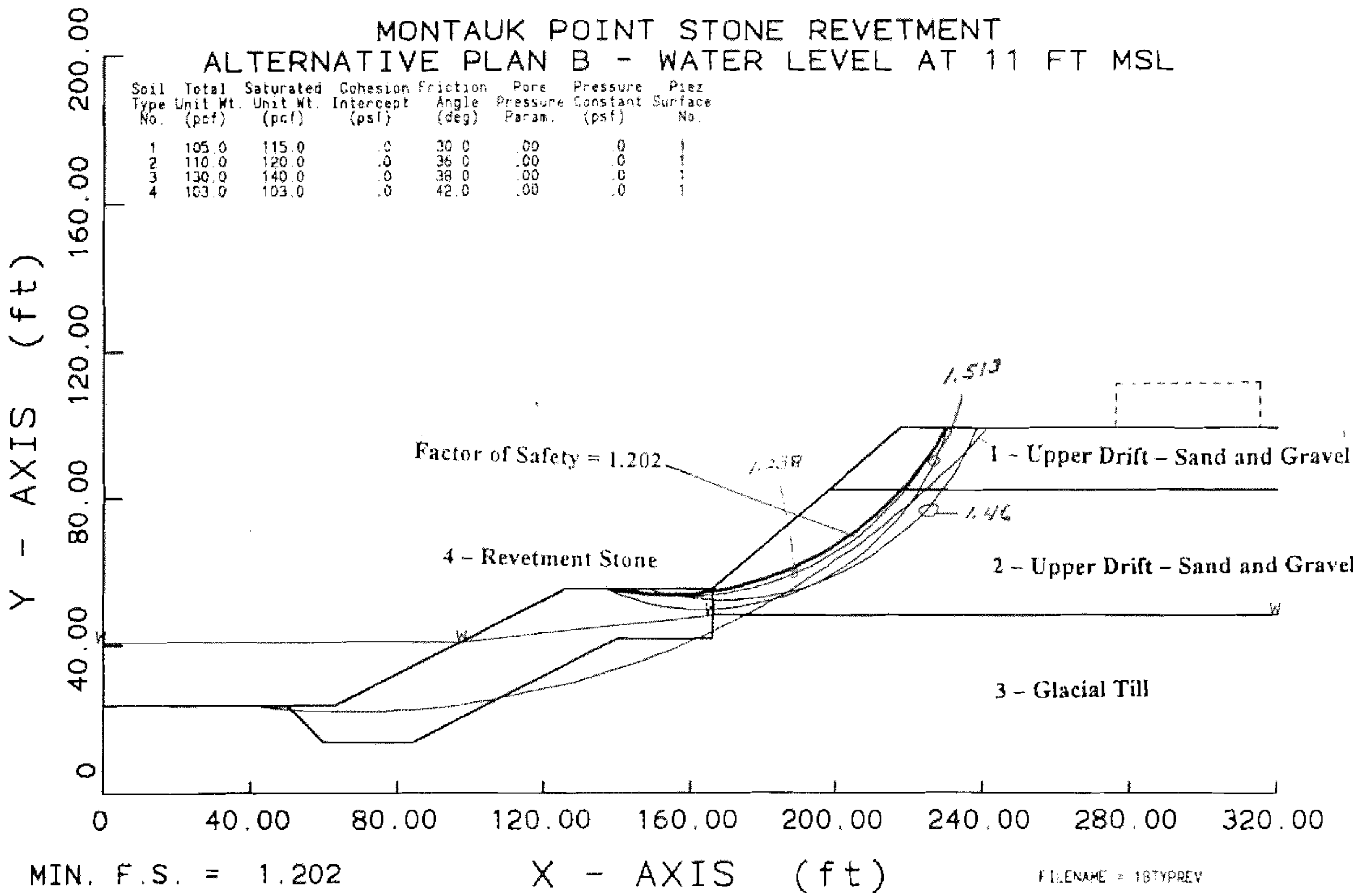


Figure A-27. Slope Stability Analysis for Stone Revetment Alternative "B"

Computation Tables for Runup and Overtopping

Alt 2A and 2B - Runup

						1:2 slope	
Return Pe	DSWL (ft)	ds (ft)	Tp(s)	Hb	Sop	Eop	R2% (2)
2	7.1	11.1	13.0	9.88	0.01141	4.682	35.21
5	8.1	12.1	13.2	10.77	0.01206	4.553	38.65
10	8.7	12.7	14.5	11.30	0.01049	4.882	39.83
25	9.5	13.5	16.1	12.02	0.00904	5.257	41.43
44	10.2	14.2	17.1	12.64	0.00843	5.445	43.11
50	10.3	14.3	17.4	12.73	0.00820	5.521	43.22
73	10.9	14.9	18.1	13.26	0.00790	5.626	44.75
100	11.5	15.5	18.7	13.80	0.00770	5.699	46.35
150	12.3	16.3	19.4	14.51	0.00752	5.765	48.55
200	12.9	16.9	20.0	15.04	0.00734	5.837	50.13
500	14.5	18.5	22.2	16.47	0.00652	6.193	53.70

(1) Use 0.89 to be consistent with Interim Report 2 Final Improvement
(2) Use Pilarczyk (1990) in CEM Fig VI-5-3 for
(3) rb is reduction due to berm width, rdh is reduction due to berm
(4) Gma is the total reduction due to berm width and height limited between 0.6
(5) Reduction in Runup due to Permeability taken from ratio of curves in Fig VI-5-12
Reductions due to roughness do not apply because surf similarity parameters

Alt 2C - Runup

						1:1.5 slope	
Return Pe	DSWL (ft)	ds (ft)	Tp(s)	Hb	Sop	Eop	R2% (2)
2	7.1	8.1	13.0	7.37	0.00851	7.226	22.52
5	8.1	9.1	13.2	8.28	0.00927	6.922	25.80
10	8.7	9.7	14.5	8.83	0.00819	7.365	26.72
25	9.5	10.5	16.1	9.56	0.00719	7.860	27.98
44	10.2	11.2	17.1	10.19	0.00680	8.083	29.39
50	10.3	11.3	17.4	10.28	0.00663	8.188	29.43
73	10.9	11.9	18.1	10.83	0.00645	8.300	30.75
100	11.5	12.5	18.7	11.38	0.00635	8.367	32.15
150	12.3	13.3	19.4	12.10	0.00627	8.415	34.09
200	12.9	13.9	20.0	12.65	0.00617	8.486	35.45
500	14.5	15.5	22.2	14.11	0.00558	8.920	38.31

Existing Condition Structure - Runup

						1:1.25 slope	
Return Pe	DSWL (ft)	ds (ft)	Tp(s)	Hb	Sop	Eop	R2% (2)
2	7.1	8.1	13.0	7.37	0.00851	8.672	20.39
5	8.1	9.1	13.2	8.28	0.00927	8.307	23.51
10	8.7	9.7	14.5	8.83	0.00819	8.839	24.12
25	9.5	10.5	16.1	9.56	0.00719	9.433	24.97
44	10.2	11.2	17.1	10.19	0.00680	9.701	26.09
50	10.3	11.3	17.4	10.28	0.00663	9.827	26.06
73	10.9	11.9	18.1	10.83	0.00645	9.961	27.16
100	11.5	12.5	18.7	11.38	0.00635	10.041	28.34
150	12.3	13.3	19.4	12.10	0.00627	10.099	30.02
200	12.9	13.9	20.0	12.65	0.00617	10.184	31.16
500	14.5	15.5	22.2	14.11	0.00558	10.705	33.27

Alt 2A - Runup On Composite

rb	rdh	Gma	Permea Factor	Runup (ft,
0.5	1.00	1.00	0.7	33.1
0.4	1.00	1.00	0.7	36.7
0.4	1.00	1.00	0.7	38.2
0.4	1.00	1.00	0.7	40.1
0.4	1.00	1.00	0.7	42.1
0.4	1.00	1.00	0.7	42.3
0.4	1.00	1.00	0.7	44.0
0.4	0.89	0.95	0.7	44.3
0.4	0.74	0.89	0.7	44.5
0.4	0.64	0.85	0.7	44.7
0.3	0.44	0.79	0.7	45.9

Alt 2B - Runup On Composite

rdh	Gma	Permea Factor	Runup (ft,
1.00	1.00	0.7	33.1
1.00	1.00	0.7	36.7
1.00	1.00	0.7	38.2
0.83	0.92	0.7	37.8
0.68	0.86	0.7	37.7
0.66	0.85	0.7	37.6
0.56	0.81	0.7	37.8
0.47	0.78	0.7	38.3
0.38	0.74	0.7	39.2
0.32	0.73	0.7	40.0
0.20	0.69	0.7	42.3

Alt 2C - Runup On Composite

rb	rdh	Gma	Permea Factor	Runup (ft,
0.1	1.00	1.00	0.7	23.7
0.1	1.00	1.00	0.7	27.2
0.1	1.00	1.00	0.7	28.4
0.0	1.00	1.00	0.7	30.2
0.0	1.00	1.00	0.7	31.9
0.0	1.00	1.00	0.7	32.1
0.0	0.84	0.98	0.7	33.3
0.0	0.70	0.97	0.7	34.7
0.0	0.55	0.96	0.7	36.6
0.0	0.45	0.96	0.7	38.1
0.0	0.27	0.95	0.7	41.5

Existing Cond. - Runup On Composite

rb	rdh	Gma	Permea Factor	Runup (ft,
0.4	1.00	1.00	0.7	22.2
0.4	0.71	0.88	0.7	23.4
0.4	0.55	0.82	0.7	23.3
0.3	0.39	0.76	0.7	23.6
0.3	0.29	0.73	0.7	24.4
0.3	0.28	0.73	0.7	24.4
0.3	0.21	0.72	0.7	25.3
0.3	0.16	0.71	0.7	26.4
0.3	0.11	0.70	0.7	27.9
0.3	0.08	0.70	0.7	29.1
0.3	0.03	0.71	0.7	32.0

PEDERSON Overtopping - Plans 2B and 2A

							Plan 2B			Plan 2A		
							Crest +25	cot a = 2		Crest +30	cot a = 2	
Return Per	DSWL (ft)	ds (ft)	Tp(s)	ds/gT2	Hb/ds (1)	Hb	Ac (ft)	Rc (ft)	q(l/s/m)	Ac (ft)	Rc (ft)	q(l/s/m)
2	7.1	11.1	13.0	0.00204	0.93	9.879	17.9	17.9	2	22.9	22.9	1
5	8.1	12.1	13.2	0.002157	0.93	10.769	16.9	16.9	4	21.9	21.9	1
10	8.7	12.7	14.5	0.001876	0.94	11.303	16.3	16.3	8	21.3	21.3	3
25	9.5	13.5	16.1	0.001617	0.94	12.015	15.5	15.5	18	20.5	20.5	6
44	10.2	14.2	17.1	0.001508	0.95	12.638	14.8	14.8	33	19.8	19.8	10
50	10.3	14.3	17.4	0.001467	0,95	12.727	14.7	14.7	37	19.7	19.7	11
73	10.9	14.9	18.1	0.001412	0.96	13.261	14.1	14.1	60	19.1	19.1	18
100	11.5	15.5	18.7	0.001377	0.96	13.795	13.5	13.5	96	18.5	18.5	27
150	12.3	16.3	19.4	0.001345	0.96	14.507	12.7	12.7	175	17.7	17.7	47
200	12.9	16.9	20.0	0.001312	0.97	15.041	12.1	12.1	280	17.1	17.1	70
500	14.5	18.5	22.2	0.001166	0.97	16.465	10.5	10.5	1060	15.5	15.5	223

Average overtopping rate calculated using Pederson (1996) for rock permeable slope fronting crown wall (bluff) and irregular waves

Alternatives 2C and Existing Structure Using Toe at -1'NGVD

							Plan 2C			Existing Structure		
							Crest=+25	cot a=1.5		Crest +18	cot a=1.25	
Return Per	DSWL (ft)	ds (ft)	Tp(s)	ds/gT2	Hb/ds (1)	Hb	Ac (ft)	Rc (ft)	q(l/s/m)	Ac (ft)	Rc (ft)	q(l/s/m)
2	7.1	8.1	13.0	0.001488	0.92	7.70	17.9	17.9	12	10.9	10.9	17
5	8.1	9.1	13.2	0.001622	0.92	8.37	16.9	16.9	24	9.9	9.9	41
10	8.7	9.7	14.5	0.001433	0.91	8.82	16.3	16.3	48	9.3	9.3	90
25	9.5	10.5	16.1	0.001258	0.91	9.57	15.5	15.5	120	8.5	8.5	266
44	10.2	11.2	17.1	0.00119	0.91	10.10	14.8	14.8	227	7.8	7.8	589
50	10.3	11.3	17.4	0.001159	0.91	10.32	14.7	14.7	274	7.7	7.7	728
73	10.9	11.9	18.1	0.001128	0.91	10.81	14.1	14.1	460	7.1	7.1	1430
100	11.5	12.5	18.7	0.00111	0.91	11.38	13.5	13.5	780	6.5	6.5	2903
150	12.3	13.3	19.4	0.001097	0.91	12.11	12.7	12.7	1517	5.7	5.7	7479
200	12.9	13.9	20.0	0.001079	0.91	12.70	12.1	12.1	2560	5.1	5.1	16221
500	14.5	15.5	22.2	0.000977	0.91	13.40	10.5	10.5	8073	3.5	3.5	130783

Sub-Appendix A-1

Historic Photographs

Figure A-1-1 Montauk Point, 1878.

Figure A-1-2. Montauk Point, 1928.

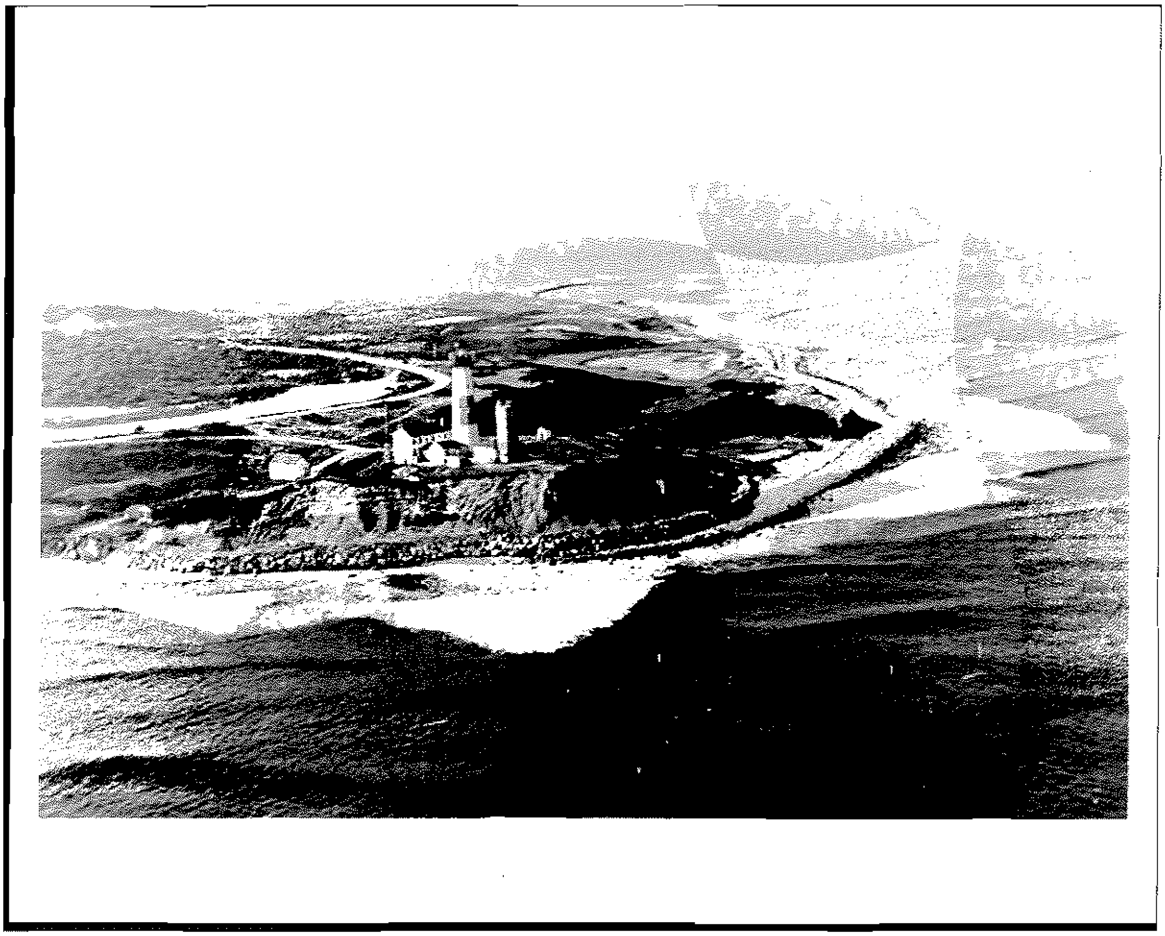
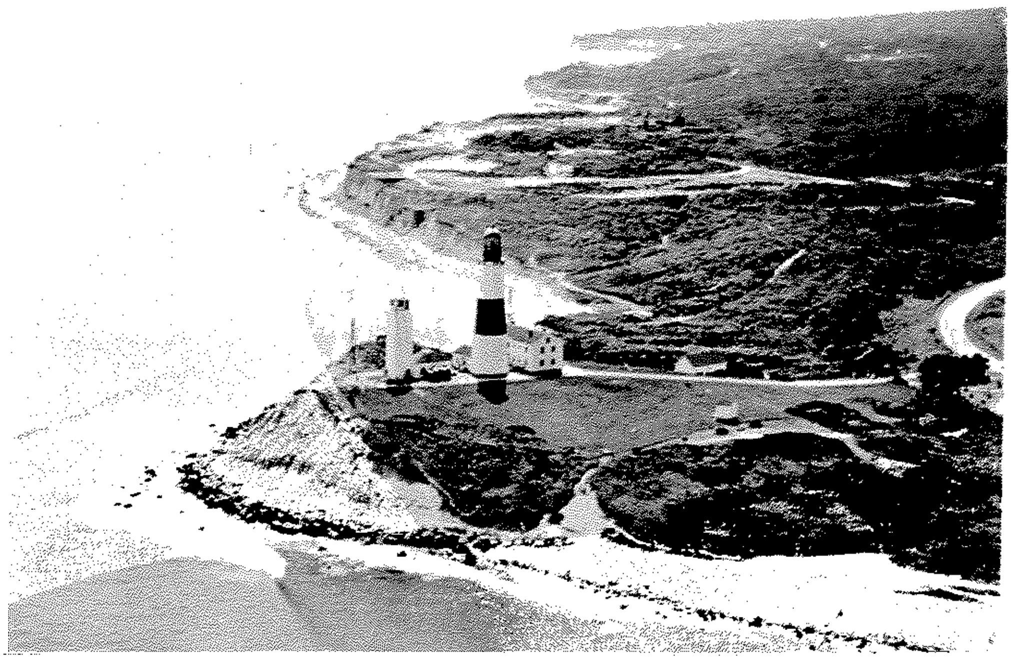


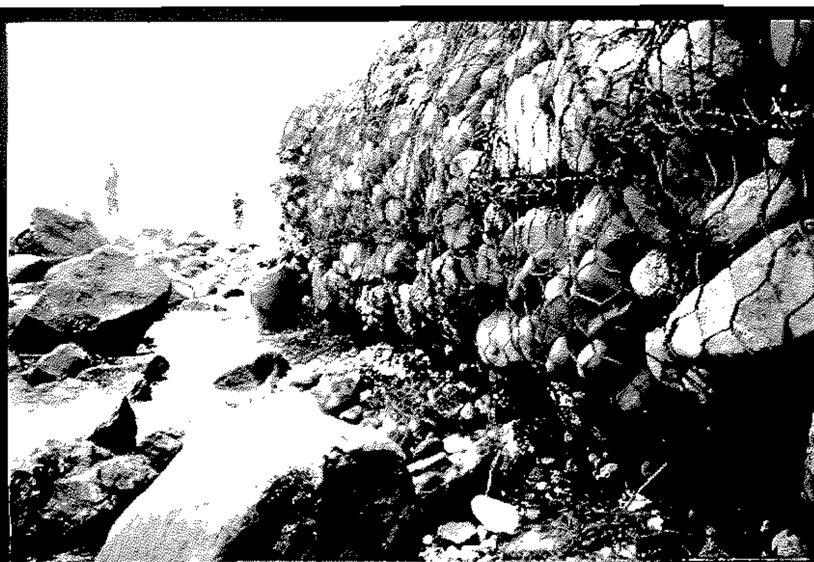
Figure A-1-3. Montauk Point With Revetment, Circa 1946.

Figure A-1-4. Montauk Point, 1950s.



This 1960s aerial of Montauk Point shows the U.S.A.C.O.E. revetment and the untouched bluff face before Giorgina Reids pilot project on the eastern slope

Figure A-1-5. Montauk Point, 1960s.



U.S. Coast Guard gabion structure before the 'Perfect Storm' of October 1991. The gabions provided ample toe protection for about 15 years. The three-day nor'easter destroyed the protection on the northern flank of the project.

Figure A-1-6. Toe of Montauk Point before and after 1991 storm.

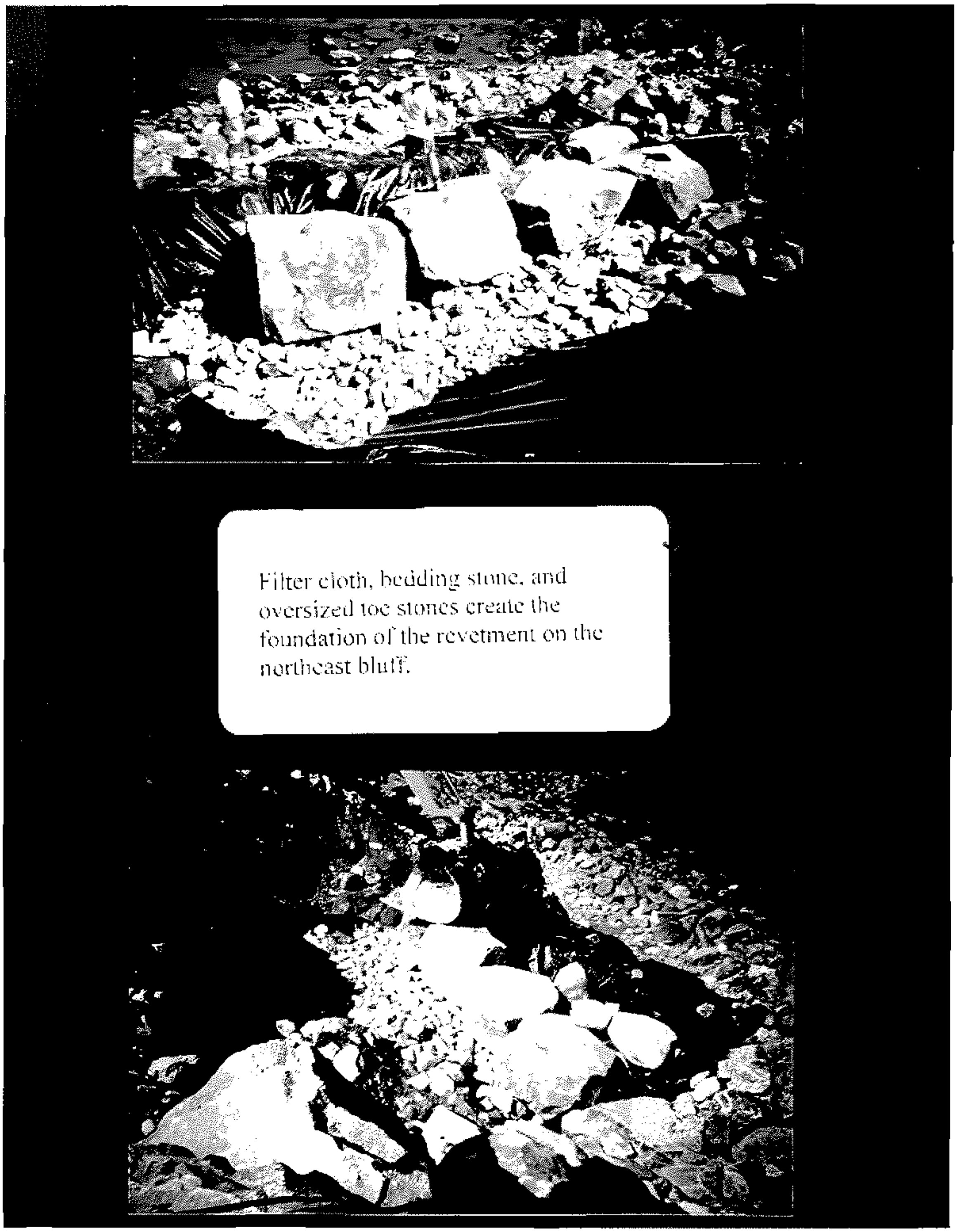
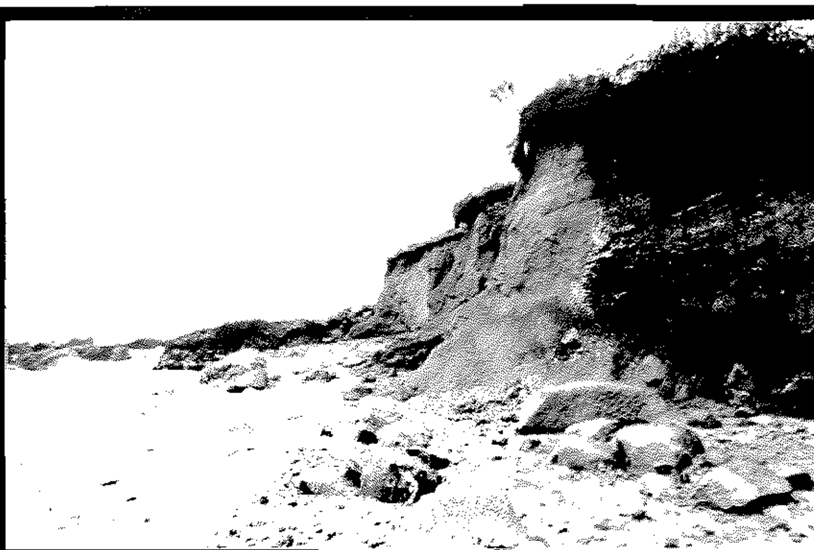


Figure A-1-7. Construction of revetment, 1992.



The upper photo is a post storm inspection of the northeast bluff after the December 1992 nor'easter.

The photo below shows the completed revetment with the engineered pedestrian access that allows visitors to traverse Montauk Point.

This dramatic before and after is the result of the cooperative efforts of the N.Y.S.D.E.C., the Long Island Office of Parks, Recreation and Historic Preservation, and the Montauk Historical Society.

Figure A-1-8. Montauk Point before and after December 1992 northeaster.

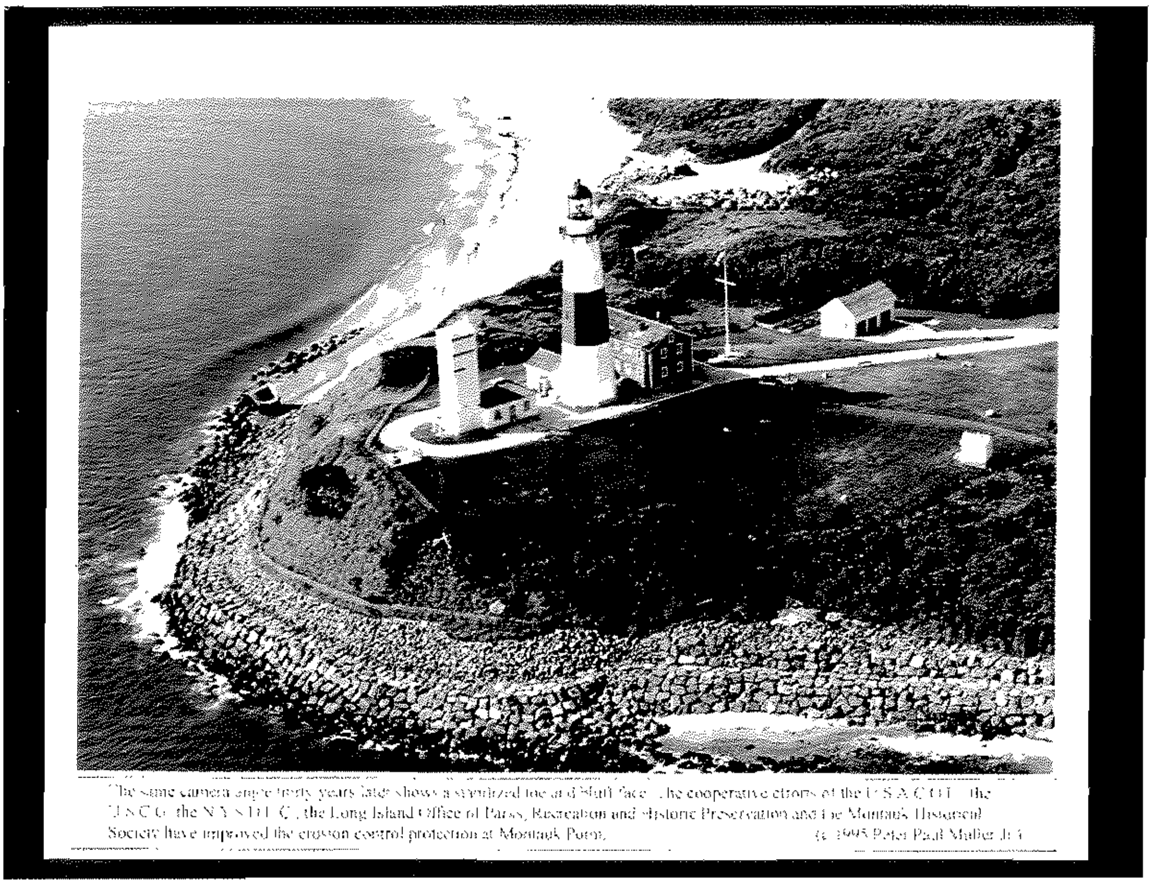


Figure A-1-9. Montauk Point, 1995.

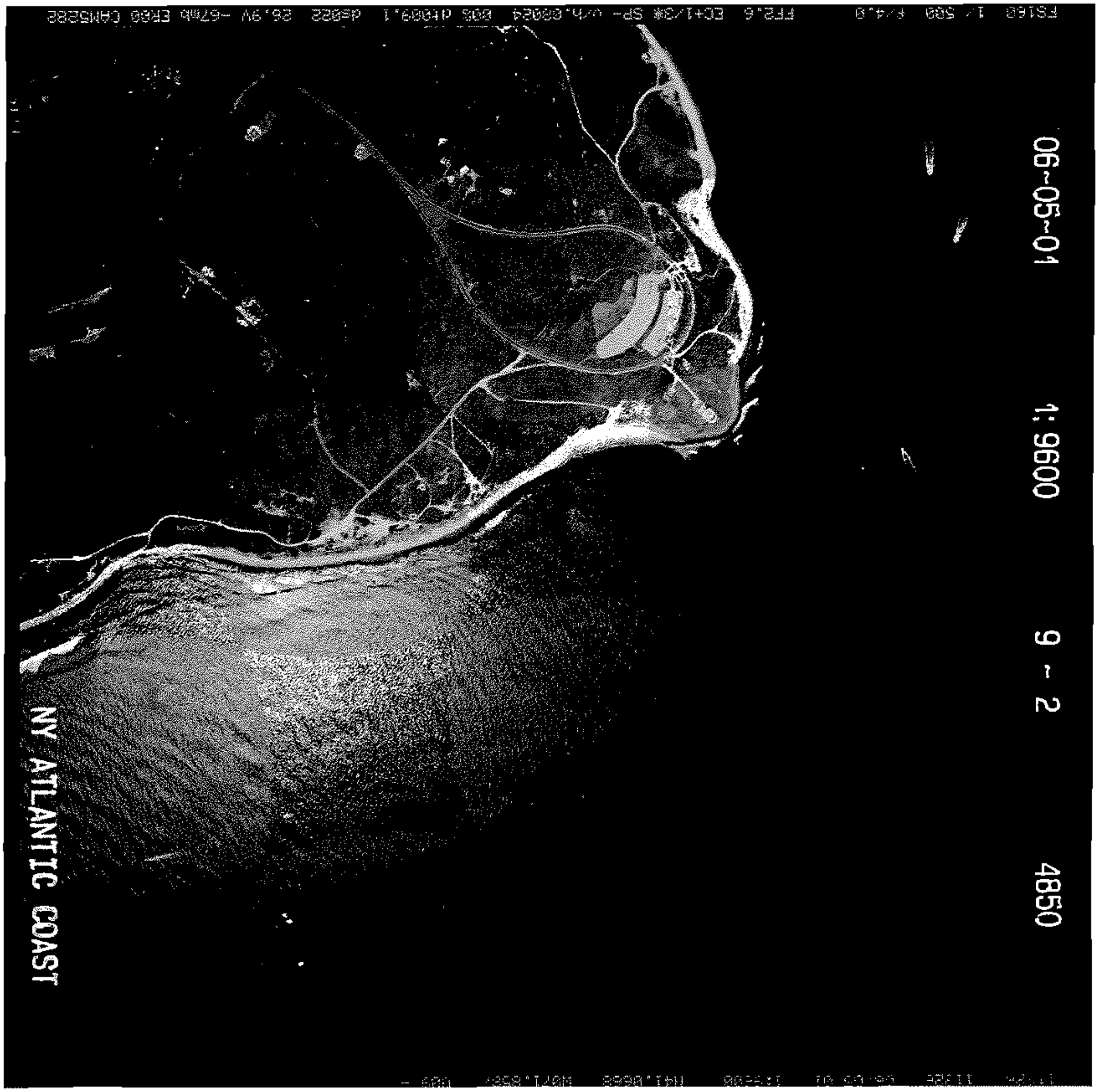


Figure A-1-10. Montauk Point, 2001.

Sub-Appendix A-2

Surface Drainage Calculation Sheets

S.O. No. 23773 -001 - 0000 - 00006

Subject: MONTAUK POINT LIGHTHOUSE

RUNOFF CALCULATION

Sheet No. 1 of 4

Drawing No.

Computed by EJG Checked By JM Date 01/28/03

Baker

USING RATIONAL METHOD TO CALCULATE PEAK FLOW ANTICIPATED RUNOFF AND DRAIN CAPACITY FOR A 10-YEAR RAINFALL EVENT.

$Q = CIA$

WHERE: Q = PEAK FLOW
C = RUNOFF CURVE NUMBER
I = RAINFALL INTENSITY
A = DRAINAGE AREA

C = 0.98 FOR CONCRETE PAVEMENT.

I = 6.7 in/hr FROM RAINFALL INTENSITY CURVE USING CONSERVATIVE TIME OF CONCENTRATION (t_c) OF 5 MINUTES.

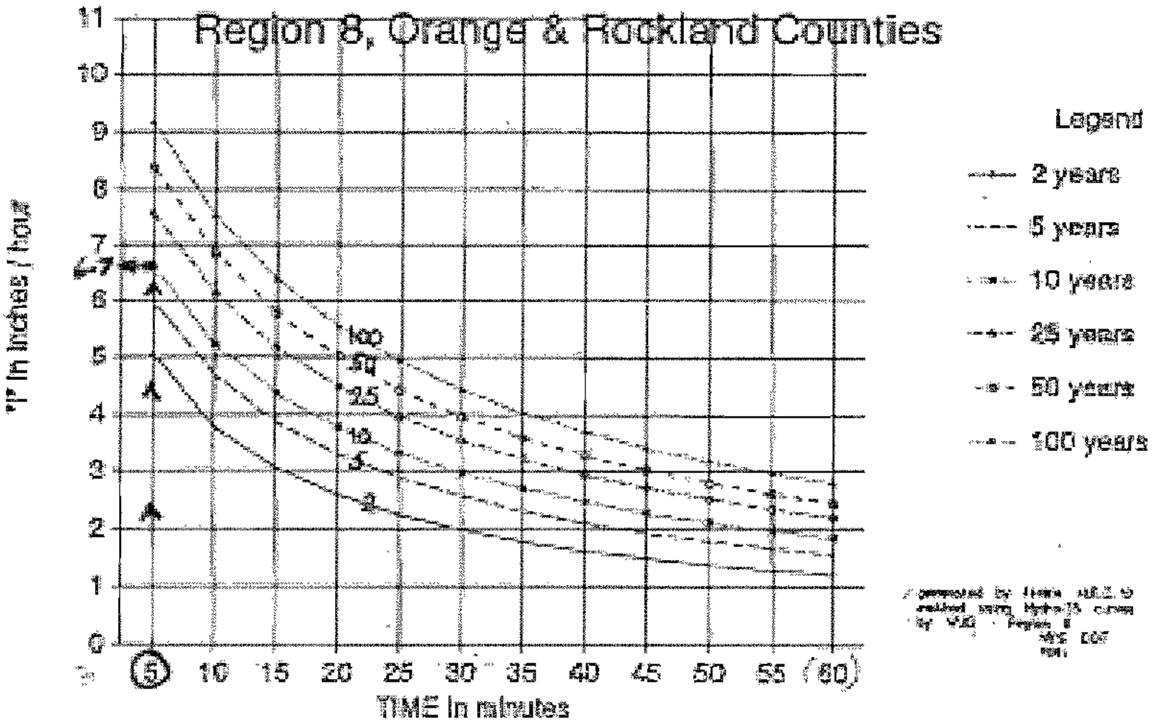
A = 0.11 AC AS MEASURED FROM AERIAL TOPO MAP.

$\therefore\ Q = CIA = 0.98\,(6.7\ \text{in/hr})(0.11\ \text{ac})$

$Q = 0.72$ CFS

SINCE APPROXIMATE CAPACITY OF DRAIN IS 1.27 CFS (SEE SHEET 4 OF 4). THE EXISTING SLOTTED TRENCH DRAIN IS ADEQUATE WITH PROPER MAINTENANCE.

RAINFALL INTENSITY

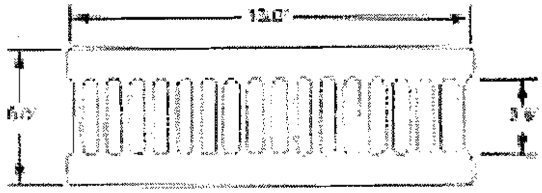


POLYDRAIN

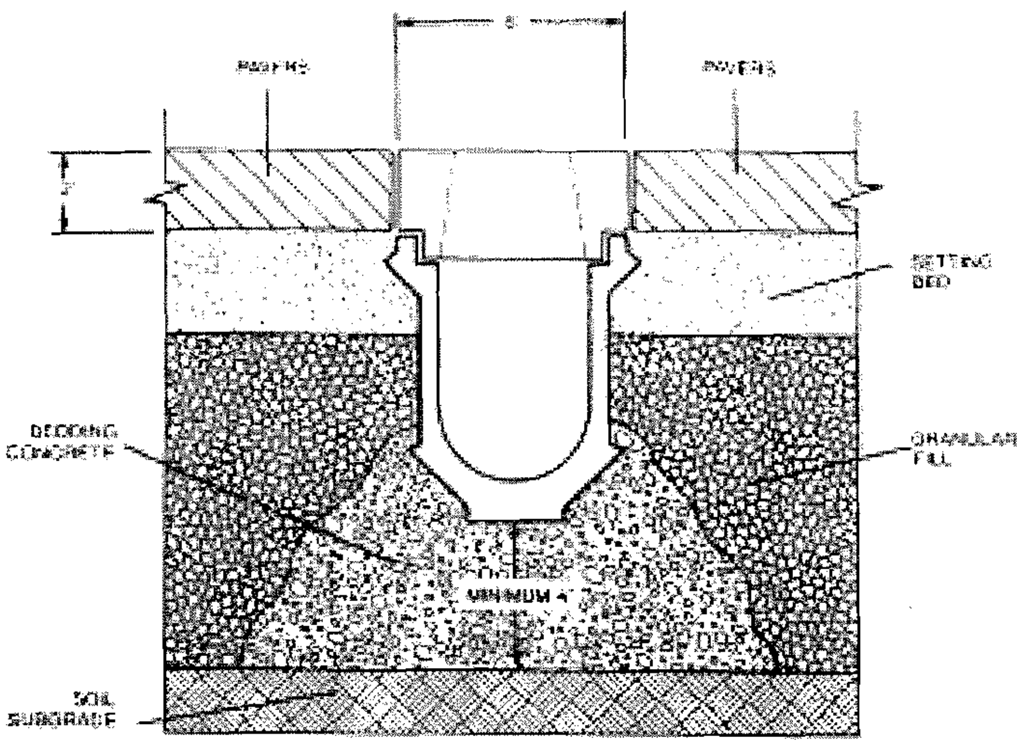
No. 313 Polydyn Grate

* APPROX. CONFIGURATION OF EXISTING DRAIN.

PLAN VIEW



CROSS SECTIONAL VIEW



SCALE

NOTE: No. 313 grate adds 2.0" to overall depth of channel

POLYDRAIN

[illegible]

TABLE 6-1
ABT, INC.
GRATING INFLOW DATA

GRATING NUMBER	GRATE DESCRIPTION	AVG. GRATE SLOT LENGTH (INCHES)	AVG. GRATE SLOT WIDTH (INCHES)	SLOT OPENINGS PER METER	PERCENT OPEN AREA	GRATE INFLOW (GPM) PER METER (GRATE INFLOW (GPM) PER FOOT)			
						PERMISSIBLE HEAD			
						1/32"	1/16"	1/8"	1/4"
310	POLYMER CONCRETE OVERLAY	2.545	0.325	42	17.0	44 (13)	62 (19)	87 (27)	123 (38) *
410/412	GALVANIZED PERFORATED	3.250	N/A	765	19.8	41 (12)	58 (18)	81 (25)	115 (35)
420/422	GALVANIZED SLOTTED	3.100	0.351	37	21.8	44 (14)	62 (19)	88 (27)	125 (38)
430/432	VINYL COATED	3.070	0.295	37	17.9	38 (11)	51 (16)	73 (22)	103 (32)
440/442	STAINLESS STEEL SLOTTED	3.130	0.351	37	21.5	44 (14)	62 (19)	88 (27)	125 (38)
482/484	STAINLESS STEEL (PERFORATED)	0.250	N/A	765	19.8	41 (12)	58 (18)	81 (25)	115 (35)
500/502	DUCTILE IRON INLAY/FRAME & GRATE	4.031	0.702	34	52.4	110 (34)	156 (48)	221 (68)	313 (96)
503/513	DUCTILE IRON INLAY/FRAME & GRATE	5.469	0.702	36	44.0	95 (29)	134 (41)	190 (58)	269 (82)
504/514	LS. DUCTILE IRON INLAY/FRAME & GRATE	3.438	0.656	38	42.2	91 (28)	128 (39)	181 (56)	257 (79)
532	GRAY IRON GRATE & FRAME	3.470	0.413	32	23.6	45 (15)	70 (21)	99 (30)	140 (43)
534	LS. DUCTILE IRON GRATE & GRAY IRON FRAME	1.640	0.750	42	32.0	91 (28)	128 (39)	181 (56)	257 (78)
720	FIBERGLASS (BARS 1 IN. ON CENTER)	1.000	0.735	38	61.4	125 (38)	178 (55)	251 (77)	356 (109)
722	FIBERGLASS (BARS 0.6 IN. ON CENTER)	2.880	0.325	65	32.0	93 (28)	131 (40)	185 (57)	262 (80)
604/614	600 SERIES CATCH BASIN GRATE	9.875	0.365	34	36.7	85 (26)	121 (37)	171 (52)	242 (74)
652/654	GALV/DUCTILE IRON GRATE FOR 6 IN. CHANNEL	7.250	0.550	32	34.3	84 (26)	119 (37)	168 (52)	238 (73)
655	FIBERGLASS (BARS 1 1/2 IN. ON CENTER)	8.817	0.892	27	57.4	226 (69)	320 (98)	453 (138)	640 (196)
657	FIBERGLASS (BARS 1.0 IN. ON CENTER)	8.817	0.400	40	38.9	151 (46)	214 (65)	302 (92)	428 (131)
662	GRAY IRON INLAY GRATE FOR 12 IN. CHANNEL	5.910	1.190	34	47.8	245 (75)	347 (106)	491 (151)	694 (211)

* 38 gpm/ft = 0.085 CFS/FT
LENGTH OF DRAIN = 15 FT.
∴ (0.085 CFS/FT)(15 FT) = 1.27 CFS

Sub-Appendix A-3

Boring Logs and Sieve Analyses

ENGINEERS FIELD BORING LOG

BORING NO.	TB-1
SHEET	1 OF 2
DATE: START	08/22/01
END	08/22/01
O.G. ELEV.	68.2

PROJECT NAME Montauk Point Lighthouse
LOCATION: Montauk, NY
STATION OFFSET BASELINE
COORDINATES: NORTH: 319,217 EAST: 2,590,609
INSPECTOR (SIGNED) Eric Gleason DRILLERS NAME/COMPANY Carl Pedersen/Land, Air, Water Env.
EQUIPMENT USED Mobile B-61 HDX
DRILLING METHODS 4 1/4" I.D. Hollow Stem Augers w/ 1 3/8" I.D. Split Spoon
CASING: SIZE: DEPTH: WATER: DEPTH: TIME: DATE:
CHECKED BY: DATE DEPTH: TIME: DATE:
S.O. NUMBER: 23492-001-0000 FILE: Montauk NOT ENCOUNTERED ☒ INCLINATION (DEGREES): 0

DEPTH (FT.)	SAMPLE NO. AND TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (FT.) / RQD (FT.)	RECOVERY (%) / RQD (%)	POCKET PENET or TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–2.0	S-1	5, 8, 7, 8	1.2	60				SANDY SILT (ml, a-4), tan to brown, moist, medium dense, non-plastic, homogeneous, sand is fine	
2.0–4.0	S-2	7, 5, 5, 4	1.6	80				2.0 WELL GRADED SAND (sw, a-1-b), tan to yellow brown, moist, loose, non plastic, homogeneous, sand is fine to medium, trace subangular gravel is coarse granite and quartz	EL. 66.2
4.0–6.0	S-3	3, 3, 3, 3	1.0	50					
6.0–8.0	S-4	3, 3, 3, 10	1.3	65					
8.0–10.0	S-5	12, 15, 13, 14	0.7	35				8.0 SANDY SILT (ml, a-4), light brown, moist, medium dense, non-plastic, homogeneous, sand is fine	EL. 60.2
10.0–14.0	A-N							11.0 POORLY GRADED SAND WITH GRAVEL (sp, a-3), gray, dry, dense, non-plastic, homogeneous, sand is fine, subangular gravel is fine to coarse granite fragments (glacial till)	EL. 57.2
14.0–16.0	S-6	10, 17, 17, 16	0.4	20					
16.0–19.0	A-N								
19.0		13, 11	0.3	15				19.0 WELL GRADED SAND WITH GRAVEL (sw, a-1-b), brown, moist, medium dense	EL. 49.2

Two attempts at different locations were made to advance TB-1 to the termination depth.

BkR_8 (LD4)
(02/01 Baker)

ENGINEERS FIELD BORING LOG

BORING NO.	TB-1
SHEET 2 OF 2	
DATE: START	08/22/01
END	08/22/01
O.G. ELEV.	68.2

PROJECT NAME: Montauk Point Lighthouse
LOCATION: Montauk, NY
STATION: OFFSET: BASELINE:
COORDINATES, NORTH: 315,217 EAST: 2,590,568
INSPECTOR (SIGNED): Eric Olsan DRILLERS NAME/COMPANY: Carl Pedersen/Land, Air, Water Env.
EQUIPMENT USED: Mobile B-61 HDX
DRILLING METHODS: 4 1/4" I.D. Hollow Stem Augers w/ 1 3/8" I.D. Split Spoon
CASING: SIZE: DEPTH: WATER: DEPTH: TIME: DATE:
CHECKED BY: DATE: DEPTH: TIME: DATE:
S.O. NUMBER: 22492-001-0000 FILE: Montauk NOT ENCOUNTERED: INCLINATION (DEGREES): 0

DEPTH (FT.)	SAMPLE NO. AND TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (FT.) / RQD (FT.)	RECOVERY (%) / RQD (%)	POCKET PENET or TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
	S-7	13 10						non plastic, homogeneous, sand is fine to medium, rounded gravel is fine to medium granite. (glacial till)	
21.0									
	A-N								
24.0	S-8	6 8 12 27	1.1	55				24.0 WELL GRADED SAND (sw, a-1-b): tan to yellow brown, moist, medium dense, non-plastic, homogeneous, sand is fine to medium, trace rounded gravel is medium granite. (glacial till)	EL. 44.2
26.0									
	A-N							27.0 POORLY GRADED SAND WITH GRAVEL (sp, a-1-b): yellow brown, moist, very dense, non-plastic, homogeneous, sand is fine to medium, rounded gravel is medium to coarse granite and quartz. (glacial till)	EL. 41.2
29.0	S-9	24 27 50 42	1.0	50					
31.0								31.0 End of Boring at 31.0'	EL. 37.2 Auger Refusal

Two attempts at different locations were made to advance TB-1 to the termination depth.

BKW_E (LD4)
(02/01 Rusted)

ENGINEERS FIELD BORING LOG

BORING NO. TB-2
SHEET 1 OF 3
DATE: START 08/22/01
END 08/23/01
O.G. ELEV. 66.5

PROJECT NAME: Montauk Point Lighthouse
LOCATION: Montauk, NY
STATION ______ OFFSET ______ BASELINE ______
COORDINATES: NORTH: 315,167 EAST: 2,560,738
INSPECTOR (SIGNED) Eric Gilson DRILLERS NAME/COMPANY Carl Pedersen/Karo, Air, Water Env.
EQUIPMENT USED Mobile B-61 HDX
DRILLING METHODS 4 1/4" I.D. Hollow Stem Augers w/ 1 3/8" I.D. Split Spoon
CASING: SIZE: ______ DEPTH: ______ WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: ______ DATE: ______ DEPTH: ______ TIME: ______ DATE: ______
S.O. NUMBER: 23492-001-0000 FILE: Montauk NOT ENCOUNTERED ☒ INCLINATION (DEGREES): 0

DEPTH (FT.)	SAMPLE NO. AND TYPE CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (FT.) / RQD (FT.)	RECOVERY (%) / RQD (%)	POCKET PENET or TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0–2.0	S-1	5, 5, 9, 7	1.3	65				SANDY SILT WITH GRAVEL (ml, a-4) brown, gray and black, moist, medium dense, non-plastic, homogeneous, sand is fine, subrounded gravel is fine to medium granite, coal, and quartz (fill)	
2.0–4.0	S-2	3, 4, 2, 1	0.4	20				2.0 SANDY SILT (ml, a-4), tan to brown, loose to medium dense, non-plastic, homogeneous, sand is fine.	← EL. 64.5
4.0–6.0	S-3	3, 3, 4, 6	1.7	85					
6.0–8.0	S-4	4, 6, 8, 9	1.6	80					
8.0–10.0	S-5	4, 5, 5, 8	1.4	70					
10.0–14.0	A-N								
14.0–16.0	S-6	9, 9, 15, 34	1.5	75				14.0 POORLY GRADED SAND WITH SILT (sp-sm, a-3), tan to yellow brown, moist, medium dense, non-plastic, homogeneous, sand is fine, trace subrounded gravel is fine to medium granite and quartz	← EL. 52.5
16.0–18.0	S-7	43, 15, 16, 30	1.2	60				16.5 SANDY SILT (ml, a-4), light brown, moist, dense, non-plastic, homogeneous, sand is fine, trace rounded gravel is fine to coarse granite and quartz	← EL. 50.0
18.0–19.0	A-N								
19.0		11, 13	1.2	60				19.0 POORLY GRADED SAND (sp, a-1-b), yellow brown, moist, medium dense to	← EL. 47.5

Two attempts at different locations were made to advance TB-2 to the termination depth

BKR_E (4.04)
(03/01 Baker)

ENGINEERS FIELD BORING LOG

BORING NO. TB-2
SHEET 2 OF 3
DATE: START 08/22/01
END 08/23/01
O.G. ELEV. 66.5

PROJECT NAME Montauk Point Lighthouse
LOCATION: Montauk, NY
STATION OFFSET BASELINE
COORDINATES: NORTH: 315,167 EAST: 2,580,738
INSPECTOR (SIGNED) Eric Orsan DRILLERS NAME/COMPANY Carl Pedersen/Land, Air, Water Env
EQUIPMENT USED Mobile B-61 HDX
DRILLING METHODS 4 1/4" I.D. Hollow Stem Augers w/ 1 3/4" I.D. Split Spoon
CASING: SIZE: DEPTH: WATER: DEPTH: TIME: DATE:
CHECKED BY: DATE: DEPTH: TIME: DATE:
S.O. NUMBER: 23492-001-0200 FILE: Montauk NOT ENCOUNTERED ☒ INCLINATION (DEGREES): 0

DEPTH (FT.)	SAMPLE NO. AND TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (FT.) / RQD (FT.)	RECOVERY (%) / RQD (%)	POCKET PENET or TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
21.0	S-8	15 17						dense, non-plastic, sand is fine, trace rounded gravel is fine to medium granite (glacial till)	
24.0	A-N								
26.0	S-9	4 9 13 20	1.0	50				25.5 SILTY SAND (sm a-1-b), brown, moist, medium dense, non plastic, homogeneous, sand is fine to medium	EL. 41.0
29.0	A-N								
31.0	S-10	6 7 9 14	0.8	40				29.0 WELL GRADED SAND (sw a-1-b), tan to yellow brown, moist, medium dense, sand is fine to medium	EL. 37.5
34.0	A-N								
36.0	S-11	15 16 22 26	1.0	50				34.0 POORLY GRADED SAND (sp a-3), tan, moist, dense, non-plastic, homogeneous, sand is fine	EL. 32.5
39.0	A-N								
		53 21	1.2	60				39.0 ELASTIC SILT WITH SAND (mt a-4), gray brown, moist, dense to very dense, ...	EL. 27.5

Two attempts at different locations were made to advance TB-2 to the termination depth

BKR E 5/04
(C) 01 Baker

ENGINEERS FIELD BORING LOG

BORING NO.	TB-2
SHEET	3 OF 3
DATE: START	08/22/01
END	08/23/01
O.G. ELEV.	66.5

PROJECT NAME Montauk Point Lighthouse
LOCATION: Montauk, NY
STATION OFFSET BASELINE
COORDINATES: NORTH: 315,167 EAST: 2,540,738
INSPECTOR (SIGNED) Eric Glaser DRILLERS NAME/COMPANY Carl Pedersen/Land, Air, Water Env.
EQUIPMENT USED Mobile B-61 HDX
DRILLING METHODS 4 1/4" I.D. Hollow Stem Augers w/ 1 3/8" I.D. Split Spoon
CASING: SIZE: DEPTH: WATER: DEPTH: TIME: DATE:
CHECKED BY: DATE: DEPTH: TIME: DATE:
S.O. NUMBER: 23492-001-0000 FILE: Montauk NOT ENCOUNTERED ☒ INCLINATION (DEGREES): 0

DEPTH (FT.)	SAMPLE NO. AND TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (FT.) / RQD (FT.)	RECOVERY (%) / RQD (%)	POCKET PENET or TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
40	S-12	24						homogeneous, sand is fine. (glacial till)	
41.0									
	A-4								
44.0									
44.5	S-13	112 107/0.1	0.5	83				44.3 POORLY GRADED GRAVEL AND SAND (gp-sp, a-1-a) gray, dry, very dense, non-plastic, homogeneous, sand is fine, angular gravel is medium to coarse granite fragments. (glacial till)	EL 22.2
	A-4								
49.0									
49.5	S-14	100 100/0.0	0.5	100				49.5 End of Boring at 49.5'	EL 17.0

Two attempts at different locations were made to advance TB-2 to the termination depth.

ENGINEERS FIELD BORING LOG

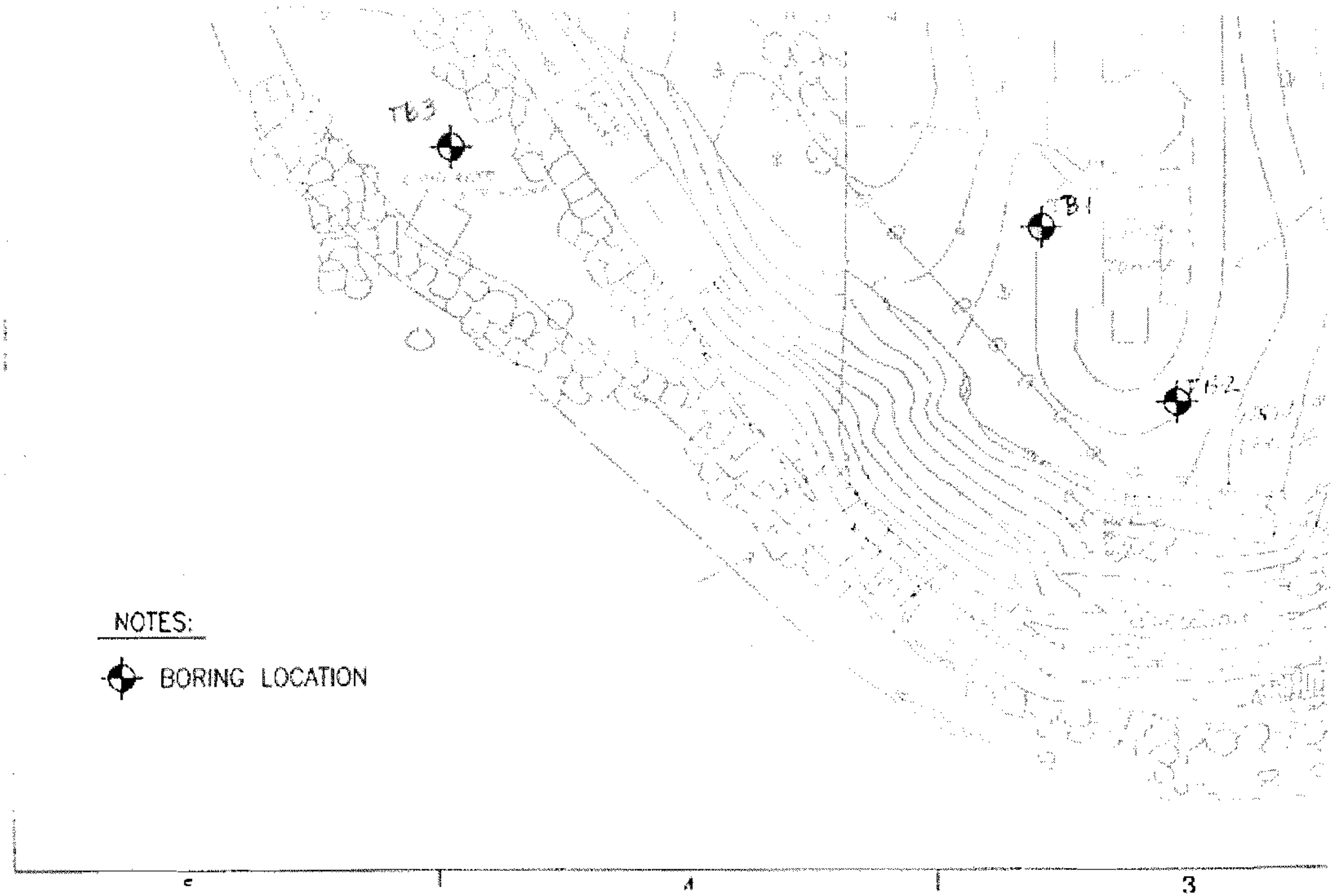
ENG_F (LD4)
(02-01 Baker)

BORING NO.	TB-3
SHEET	1 OF 1
DATE: START	08/24/01
END	08/24/01
O.G. ELEV.	4.0

PROJECT NAME Montauk Point Lighthouse
LOCATION: Montauk, NY
STATION OFFSET BASELINE
COORDINATES: NORTH: 314,850 EAST: 2,590,600
INSPECTOR (SIGNED) Eric Gilson DRILLERS NAME/COMPANY Carl Pedersen/Land, Air, Water Env
EQUIPMENT USED Mobile B-61 HDX
DRILLING METHODS 4 1/4" I.D. Hollow Stem Augers w/ 1 3/8" I.D. Split Spoon
CASING: SIZE: DEPTH: WATER: DEPTH: 2.0 TIME: 0 hrs DATE: 08/24/01
CHECKED BY: DATE: DEPTH: TIME: DATE:
S.O. NUMBER: 23492-001-0000 FILE: Montauk NOT ENCOUNTERED ☐ INCLINATION (DEGREES): 0

DEPTH (FT.)	SAMPLE NO. AND TYPE CORE RUN	BLOWS/0.5 FT ON SAMPLER	RECOVERY (FT.) / RQD (FT.)	RECOVERY (%) / RQD (%)	POCKET PENET or TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0	S-1	1 2 3 5	0.2	10				WELL GRADED SAND WITH GRAVEL (sw a-1-b), black, tan, brown and gray wet, loose, non-plastic, homogeneous, sand is fine to medium, rounded gravel is fine to coarse granite, quartz and chert 2.0	EL. 2.0
2.0 – 2.4	S-2	25/0.4	0.2	50				Granite Boulder 2.4	EL. 1.6
								End of Boring at 2.4'	

Six attempts at different locations were made to advance TB-3 to the termination depth. Boulders were consistently encountered within two feet of the surface.



TB3
TB1
TB2
NOTES:
BORING LOCATION

Sub-Appendix A-4

Design Calculations

EROSION CONTROL FEASIBILITY STUDY
MONTAUK POINT, NEW YORK

PURPOSE: To develop the Final Improvement Design

REFERENCE:

1. Shore Protection Manual, 1984 Edition
   Coastal Engineering Research Center
2. Coastal Engineering Manual, 2001
   Coastal and Hydraulics Laboratory
3. EM 1110-2-1614, Design of Coastal Revetments, Seawalls and Bulkheads.
4. Erosion Control Feasibility Study, Montauk Point, New York
   First Interim Submission, Final Report, 4 April 2002
5. Erosion Control Feasibility Study, Montauk Point, New York
   Coastal Analysis and Slope Stability Analysis for Improvement Plans, 28 June 2002.

PROCEDURE:

Alternative Plans

The three (3) alternative plans under consideration are as follows:

ALTERNATIVE 2A	ALTERNATIVE 2B	ALTERNATIVE 2C
STONE REVETMENT	STONE REVETMENT	STONE REVETMENT
150-year Level of Protection	73-year Level of Protection	15-year Level of Protection
Crest Elev. +30' NGVD	Crest Elev. +25' NGVD	Crest Elev. +25' NGVD

Design Waves

An analysis of the design wave heights and design Stillwater levels occurring for the 5 yr., 10 yr., 25 yr., 50 yr., 73 yr., 100 yr. and 150 yr. storm events was conducted.

For this analysis, review of the wave data (Reference 4) indicated that the waves from the ESE-SSW directions with a wave period of 12 seconds have the most significant impact on the project area. Review of the existing profile data (Reference 4) indicates that the typical profiles have slopes ranging from 1V:50H to 1V:100H for an average 1V:75H.

Using these design parameters, the methodology, based on Figure 7-4 of Reference 1, was used to determine if the wave spectrum is subject to depth limitation, which would control the wave height for design purposes. For the three alternatives considered, the design wave is depth limited and is shown in Tables A-4-1 and A-4-2

For the purposes of the Feasibility Study, the annual cost of maintenance of the alternatives considered is estimated to be 0.5% of the total first cost of construction. This maintenance is associated with 0% - 5% damage levels up to the design storm. For storm exceedance damage levels to the specific design, damages increase and require major rehabilitation.

In order to determine the quantities and costs of major rehabilitation of each alternative after significant storm events, an analysis of the design wave heights and design still water levels occurring during the 2-yr., 5-yr., 10-yr., 15-yr., 25-yr., 50-yr., 73-yr., 100-yr. and 150-yr.storm events was conducted. The methodology used to determine the design wave heights was based on Figure 7-4 in the SPM (1984). The results of this analysis are presented in Table A-4- 1.

Major storm damage beginning with storm waves that are 80% of the design storm wave (to allow for a damage contingency at a lower initiation of damage) and extending to the 150-yr. storm are annualized to provide the major rehabilitation costs for each alternative.

During significant storm events, damage to the revetment alternatives is possible due to direct wave impact and due to wave runup and overtopping which erodes the bank above the revetment and undermines the revetment.

To evaluate the potential damage to the revetment alternatives from direct wave attack, the methodology presented in Table 7-9 in the SPM (1984) was used which gives $H/H_{D=0}$ as a function of cover-layer damage and type of armor unit; where H is the maximum wave height at the structure toe for a specific storm event and $H_{D=0}$ is the design wave height corresponding to 0- to 5-percent damage. To establish damages from wave impacts, the percentage of damage from Table 7-9 for a specific storm event is multiplied by the first cost of the alternative. It is noted, to capture the true cost of each operation under major rehabilitation, the mob & demob and E & D and construction management costs are initially separated out prior to prorating the percent damages and then added back in to reflect pertinent mob & demob, E and D and construction management which should not be prorated. These results are shown in Table C-8 of Appendix C.

To evaluate the potential damage to the revetment alternatives from wave runup and overtopping, the methodology presented in Table VI-5-6 in the Coastal Engineering Manual (CEM) Chapter VI-5 was used which gives critical values of average overtopping discharge (q, in litres/s per m) as a function of damage to structures. For this analysis, percent damage levels were assigned to the overtopping discharge (q, in litres/s per m) as shown in Table 3. The percentage of damage to the revetment from wave overtopping from Table 3 for a specific storm event is added to the percentage damage from wave impacts. The maximum damage for each damage mechanism is 50%. The results of this analysis are presented in Table C-8 of Appendix C.

The total revetment damage costs resulting from direct wave impact and runup/overtopping for significant storm events for each of the alternatives are presented in Table C-8 of Appendix C. The average annual major rehabilitation costs for each alternative are developed using a damage frequency analysis. Tables C-4 through C-7 present a summary in which the repair costs associated with each storm frequency are used to derive an average annual repair cost for major rehabilitation.

Future impacts on annual maintenance costs and major rehabilitation costs due to sea level rise are considered to be minor given the predicted rate of sea level rise of 0.014 feet per year. For example, at this rate at the mid-point of the project life, 25 years, the rise in water level would be 0.35 feet. Without sea level rise, the proposed design wave for Alternative 2B is H 73 Yr. = 13.4' calculated using Figure 7-4 (SPM 1984) and DSWL 73 Yr = +10.94' NGVD which results in a design armor stone weight of 12.6 tons.

Adding the sea level rise to the DSWL 73 Yr = +10.94' NGVD would result in DSWL 73 Yr + Sea Level Rise = 11.3' NGVD which would result in a design wave for this structure, H 73 Yr. +

Sea Level Rise = 13.8'. This design wave results in a design armor stone weight of 13.8 tons or a 9.5% increase in armor stone weight due to sea level rise through the mid-point of the project life.

Given the small predicted annual increase in sea level rise in the project area, and the standard construction specification for the armor stone to range from 0.75 W to 1.25 W (W = 12.6 tons) with about 50 percent of the individual stones weighing more than W, increasing the armor stone weight to account for future sea level rise is not considered to be warranted.

DESIGN WAVE HEIGHTS PER STORM EVENT
per SPM Figure 7-4

		5-YR.	10-YR.	25-YR.	50-YR.	**73-YR.**	100-YR.	**150-YR.**
ALT 2A								
DSWL		8.10	8.69	9.52	10.34	10.94	11.51	12.31
d @ DSWL		-4.00	-4.00	-4.00	-4.00	-4.00	-4.00	-4.00
ds		12.10	12.69	13.52	14.34	14.94	15.51	16.31
ds/gT^2		0.00261	0.00274	0.00292	0.00309	0.00322	0.00334	0.00352
Hb/ds		0.91	0.91	0.91	0.90	0.90	0.90	0.90
$H_{Storm\ Event}$ (ft.)		10.95	11.48	12.24	12.91	13.37	13.88	**14.60**
ALT 2B								
DSWL		8.10	8.69	9.52	10.34	10.94	11.51	12.31
d @ DSWL		-4.00	-4.00	-4.00	-4.00	-4.00	-4.00	-4.00
ds		12.10	12.69	13.52	14.34	14.94	15.51	16.31
ds/gT^2		0.00261	0.00274	0.00292	0.00309	0.00322	0.00334	0.00352
Hb/ds		0.91	0.91	0.91	0.90	0.90	0.90	0.90
$H_{Storm\ Event}$ (ft.)		10.95	11.48	12.24	12.91	**13.37**	13.88	14.60
ALT 2C								
DSWL		8.10	8.69	9.52	10.34	10.94	11.51	12.31
d @ DSWL		-1.00	-1.00	-1.00	-1.00	-1.00	-1.00	-1.00
ds		9.10	9.69	10.52	11.34	11.94	12.51	13.31
ds/gT^2		0.00196	0.00209	0.00227	0.00245	0.00258	0.00270	0.00287
Hb/ds		0.92	0.91	0.91	0.91	0.91	0.91	0.91
$H_{Storm\ Event}$ (ft.)		8.37	8.82	9.57	10.32	10.81	11.38	12.11
Breaker Travel Distance, x_p								
m =	0.0667	37.0458153	38.8521815	41.3933407	43.6613207	45.2354422	46.9612924	49.3835383
0.052083333								
m =	0.05	38.7373938	40.6262419	43.283435	45.654975	47.3009738	49.1056294	51.6384794
0.055555556								

A summary of the design conditions for each of the final improvement plans is presented in Table A-4-2.

TABLE A-4-2
DESIGN CONDITIONS

Final Improvement Plan	DSWL, Ft. NGVD	Design Wave, Ft.
Alternative 2A Stone Revetment 150-year Level of Protection Crest Elev. +30' NGVD	+12.31	14.6
Alternative 2B Stone Revetment 73-year Level of Protection Crest Elev. +25' NGVD	+10.94	13.4
Alternative 2C Stone Revetment 15-year Level of Protection Crest Elev. +25' NGVD	+9.05	9.2

Armor Size Calculation

Hudson's stability formula was used to determine the required armor stone size using the ACES 1.07 breakwater design module with the following equation:

$$W = \frac{W_r H^3}{K_D (S_r - 1)^3 \text{COT@}}$$

where:

W = weight (lb.) of individual armor unit in the primary cover layer
Wr = unit weight of armor rock (165 lb/cubic ft)
H = design wave height
S_r = specific gravity of armor unit relative to water (2.58)
COT@ = angle of structure side slope measured from the horizontal (degrees)
K_D = stability coefficient that varies primarily with the shape of the armor units, roughness of the armor unit surface, sharpness of edges, and degree of interlocking obtained in placement. K_D values are selected for a breaking wave condition based on depths and slopes at the structure; K_D = 2.0

Armor Thickness

The thickness of the armor layer was computed using ACES 107 – Breakwater Design Using Hudson and Related Equations. The equation used in ACES 1.07 is:

$$\mathbf{r = nK_d(W_a/W_r)^{1/3}}$$

where:

- $\mathbf{r}$ = average thickness (ft)
- $\mathbf{n}$ = number of layers (2)
- $\mathbf{W_a}$ = weight of the individual armor unit
- $\mathbf{W_r}$ = unit weight of the armor unit (165 lb./cubic foot)
- $\mathbf{K_d}$ = layer thickness coefficient (1.0)

The recommended armor stone sizes and thickness determined using ACES 1.07 for each of the final alternative plans are presented in Table A-4-3.

TABLE A-4-3

ARMOR STONE SIZES AND THICKNESS
(ACES 1.07 Output)

ALTERNATIVE 2A - 150-YR. DESIGN LEVEL

Armor Weight/Mass	(Wr):	165.00 lb/ft^3
Wave Height	(H) :	14.60 ft
Stability Coefficient	(Kd):	2.00
Layer Coefficient	(K^):	1.02
Average Porosity	(P) :	38.00 %
Cotangent of Structure Slope	:	2.00
No. Units Comprising Layer Thickness	(n) :	2.00
Single Armor Unit Weight	(W) :	16.31 tons
Minimum Crest Width	(B) :	17.83 ft
Average Layer Thickness	(r) :	11.89 ft
No. of Single Armor Units	(Nr):	37.26 Per 1000 ft^2

ALTERNATIVE 2B - 73-YR. DESIGN LEVEL

Armor Weight/Mass	(Wr):	165.00 lb/ft^3
Wave Height	(H) :	13.40 ft
Stability Coefficient	(Kd):	2.00
Layer Coefficient	(K^):	1.02
Average Porosity	(P) :	38.00 %
Cotangent of Structure Slope	:	2.00
No. Units Comprising Layer Thickness	(n) :	2.00
Single Armor Unit Weight	(W) :	12.61 tons
Minimum Crest Width	(B) :	16.36 ft
Average Layer Thickness	(r) :	10.91 ft
No. of Single Armor Units	(Nr):	44.24 Per 1000 ft^2

ALTERNATIVE 2C - 15-YR. DESIGN LEVEL

Armor Weight/Mass	(Wr):	165.00 lb/ft^3
Wave Height	(H) :	9.20 ft
Stability Coefficient	(Kd):	2.00
Layer Coefficient	(K^):	1.02
Average Porosity	(P) :	38.00 %
Cotangent of Structure Slope	:	1.50
No. Units Comprising Layer Thickness	(n) :	2.00
Single Armor Unit Weight	(W) :	5.4 tons
Minimum Crest Width	(B) :	13.71 ft (a)
Average Layer Thickness	(r) :	9.0 ft (a)
No. of Single Armor Units	(Nr):	63.02 Per 1000 ft^2 (a)

(a) Note - The minimum required armor stone size is 5.4 tons, however, since this alternative involves removal of armor stones in the 5 to 10 ton range which can be reused, the average layer thickness is increased to that associated with a 7.5 ton average armor stone.

Underlayer and Bedding Layers

The recommended underlayer and bedding layer for each of the final improvement plans are presented in Table A-4-4.

TABLE A-4-4
UNDERLAYER AND BEDDING LAYER

Alternative Plan	**Underlayer, W/10**		**Bedding Layer**	
	Weight (Tons)	Thickness (Ft.)	Weight (Lbs)	Thickness (Ft.)
Alternative 2A	1.6	5.4	100	2.0
Alternative 2B	1.3	5.0	100	2.0
Alternative 2C	0.75	4.2	100	2.0

Toe Design

In Alternatives 2A and 2B, a heavily embedded toe is incorporated to protect against breaking waves and scour at the toe of the structure. The embedded toe is designed in accordance with EM 1110-2-1614 entitled "Design of Coastal Revetments, Seawalls and Bulkheads (Reference 3). Filter cloth and sublayers are specified in accordance with standard design procedures. For Alternative 2C, the toe will be built up from the existing toe with large stone and will not be an excavated buried toe.

Sub-Appendix A-5

Two-Dimensional Physical Model Study
And
Interview with Greg Donahue

Introduction

Work done to date on the Montauk Point Feasibility Study has provided numerical estimates of the separate impacts to the existing stone revetment. Such impacts include storm waves, scour and overtopping. No satisfactory numerical modeling methodology exists for combining the effects of these damage mechanisms as they occur in nature. In order to more fully define the without-project condition, an estimate of conditions leading to failure of the existing structure is needed.

This report presents the results of a two-dimensional physical model test of the revetment presently in place at Montauk Point, Long Island, New York. The objective of this work is to better define the failure mechanisms and criteria for the existing revetment at Montauk Point. Failure criteria are expressed in terms of combinations of water level and wave conditions. Failure is assumed to occur when the structure is damaged to about the 25% level. Such a damage level would render the structure susceptible to catastrophic failure in future storm events. The tests were conducted at the University of Delaware Center for Applied Coastal Engineering wave test flume.

To prepare the model test conditions, such as the seafloor and revetment cross-section, the following activities were performed:

- Existing topographic and bathymetric data were reviewed to identify worst-case cross sections for testing.
- A field visit was performed to interview Mr. Greg Donohue, Erosion Control Director, Montauk Point Lighthouse Museum, and to inspect the existing stone revetment structure. The interview and inspection yielded information about the layering of the stone in the structure, the characteristics of the stone, the size distribution of the stone, and more accurate information about the elevations of the stone. A cross section was identified for testing, submitted to the New York District for approval, and then constructed in the wave test facility.

Model Setup

The revetment model was constructed in the University of Delaware Center for Applied Coastal Engineering wave test flume. The wave tank is approximately 8 feet wide, 5 feet deep, and 120 feet long. It is equipped with a hydraulic wave generation system capable of creating realistic irregular wave trains of specified wave height and wave period. The wave tank is divided into two four-foot wide sections. The revetment model was built in one of the sections, while the other section was left open to provide energy dissipation on a rough stone beach.

The model scale of 1:30.48 was selected to insure that the offshore design wave could be developed by the wave generator. The scale factor of 30.48 was used because it is the ratio of one centimeter in the model to one foot in the prototype. This makes it convenient for constructing the model and for converting model measurements in centimeters into prototype conditions in feet.

A floor was constructed in the flume with a 1:50 slope to match the natural offshore bottom slope at Montauk Point. The slope was approximately 40 feet long (1200 cm in the model, or 1200 feet in prototype), running from –4.8 feet NGVD at the toe of the revetment to –28.8 feet NGVD at the seaward end of the slope. With elevated storm water levels, the seaward end of the slope was at a depth of about 10 meters, matching the water depth at which the design waves for the project have been specified.

The model revetment was constructed as shown in Figure A-5-1. This section was developed based on field measurements, discussions with Mr. Greg Donohue, and a design section provided by Mr. Donohue. The selected section has an over-steepened front slope. This section faces approximately due east, and was originally constructed at Montauk Point in the Federal Phase 2 project in the fall of 1992 and spring of 1993.

Special care was used to obtain an accurate simulation of armor stone size, shape, and constructed interlocking. Several sources of quarry stones were used in an attempt to obtain crushed stones with shapes similar to the quarry stones used in the construction of the revetment. The prototype stones are very blocky, with flat faces, and are fit closely in a single layer on the revetment. Crushed stones for the model were hand picked to obtain the most blocky and flat-faced stones for modeling purposes. After constructing a trial section in the wave flume, it was decided that the crushed stones did not adequately simulate the quarried stone used to construct the revetment. Therefore, more regular blocky, flat faced stones were obtained for the model by taking cut sheets of slate, breaking them into appropriate sized blocks, and tumbling them in a cement mixer to round the corners. This resulted in modeled armor stones closely matching the revetment armor stones in shape, weight, and interlocking characteristics.

The model revetment, illustrated in Figures A-5-2 and A-5-3, was carefully constructed so that the model represented realistic conditions, including imperfect stone placement, which could lead to local stone removal by wave attack or runup. A number of stones were carefully placed so that they were not jammed in by their neighbors, as observed in the prototype cross-section. Such weak spots could make it easier for the stones to be lifted from the slope. A few stones on the revetment berm were left unsupported on the seaward edge to examine the possibility of that failure due to non-interlocking. Several stones on the berm were also left unsupported at the rear of the berm (landward edge) to examine the stability of the berm if the support were removed from the bluff area by erosion.

Waves were measured in front of the revetment at 60, 160, and 240 prototype feet from the toe of the revetment. The offshore wave height was measured at the seaward end of the slope in about 33 feet of water. At this location, three gauges were used to provide data for the determination of the onshore component of the wave and the offshore component due to reflections from the revetment.

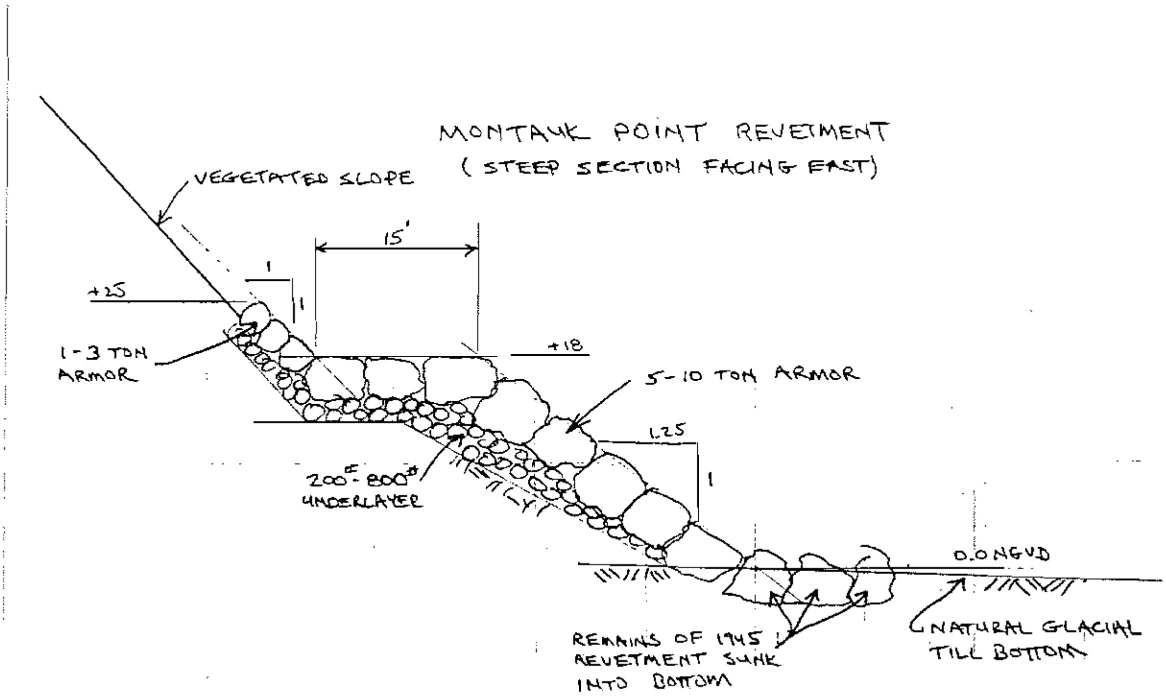


Figure A-5-1. Existing revetment cross-section used for physical modeling.

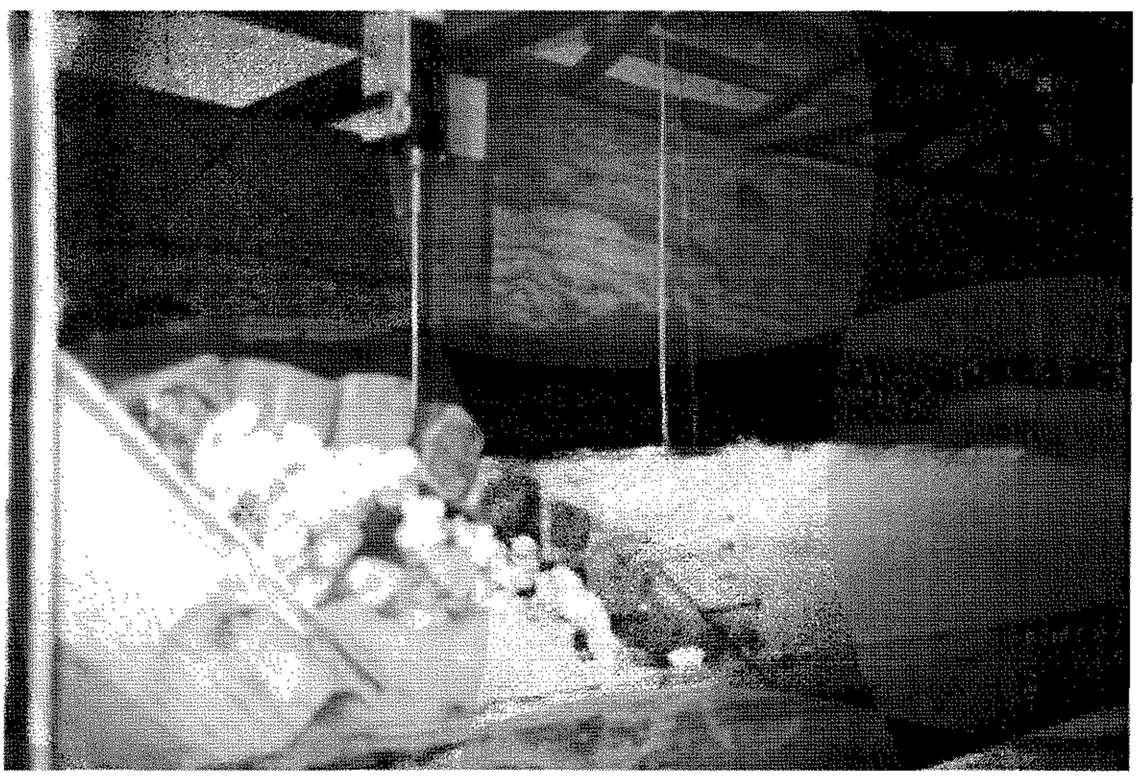


Figure A-5-2. Photograph of the cross-section of the two-dimensional model.

Figure A-5-3. Plan view of revetment in two-dimensional model

Test Results

A total of 16 cases were tested in the revetment model tests, examining a range of events that encompassed the 2-year to the 100-year conditions (Table A-5-1). Water surface elevations were varied from +5.2 feet to +11.2 feet NGVD to represent the full range of possible storm water levels. Note that the water level locally was further elevated at the structure by additional wave setup; however the wave setup in the model is less than occurs in the prototype because the size of the test basin is much smaller. The total water level including wave setup in the model was targeted to be as close as possible to that estimated to occur in nature (Table A-5-1). Wave periods of 13, 15, and 18 seconds (prototype peak spectral period) were used to cover the range of expected storm wave periods. Wave heights at the offshore measurement position, in approximately 33 feet of water, ranged from about 14 to 17 feet (significant incident wave height). At the offshore measurement location, the larger waves in the wave train were observed to be breaking, indicating that they were being depth limited at that location.

Table A-5-1. Extreme storm statistics produced by the Fire Island to Motnauk Point Reformulatin Study

Return Period (yrs)	Significant Wave Height (ft)	Storm Stage (ft, NGVD)	Storm Stage plus Wave Setup (ft, NGVD)	Max. Breaker Height (ft) (-32.8 ft NGVD contour)	Design Significant Wave Height (ft) -32.8 ft depth	Wave Period (s)
2	17.13	4.53	7.07	29.12	16.18	13.00
5	20.57	5.38	8.10	29.79	16.55	13.15
10	21.03	5.81	8.69	30.12	16.73	14.48
25	21.56	6.33	9.52	30.53	16.96	16.13
44	21.99	6.77	10.16	30.87	17.15	17.10
50	22.11	6.92	10.34	30.99	17.22	17.37
73	22.49	7.42	10.94	31.38	17.43	18.11
100	22.83	7.94	11.51	31.78	17.66	18.66

The first 11 tests examined the existing revetment for the full range of water levels and wave periods. Test 12 examined the fate of a layer of sand placed in front of the structure toe under storm conditions. The final tests examined a larger revetment, similar to that proposed for construction. In these final tests, the stone sizes were not carefully simulated for stability testing. These tests examined the effect of the larger revetment on runup and wave reflection. The test conditions are shown in the Table A-5-2.

Existing Revetment Tests

For the 5.2 ft NGVD water level, corresponding to a storm with a return period of <2 years (when wave setup is added by the waves), the maximum runup was about +35 feet on the bluff behind the revetment. About 5% of waves overtopped the revetment berm. No movement of armor stones or stones on the bluff above the berm occurred.

Table A-5-2. Tests cases examined in the physical model.

TEST NO.	WATER LEVEL	WAVE SETUP (measured)	WATER LEVEL (w/setup)	Approx. RETURN PERIOD	WAVE PERIOD	OFFSHORE WAVE HT. (model)	MAX. WAVE@TOE. (measured)	TEST CONDITION
	(ft, NGVD)	(ft)	(ft, NGVD)	(yr)	(sec)	(Hs, ft)	(ft)	
1	5.2	0.1	5.2	<2	13.0	14.2	13.6	Existing Revetment
2	5.2	0.1	5.2	<2	15.0	14.9	14.0	Existing Revetment
3	5.2	0.1	5.2	<2	18.1	17.1	15.6	Existing Revetment
4	7.2	0.3	7.5	3	13.0	14.3	15.4	Existing Revetment
5	7.2	0.3	7.5	3	15.0	14.9	16.0	Existing Revetment
6	7.2	0.3	7.5	3	18.1	17.1	16.9	Existing Revetment
7	9.7	1.2	10.9	73	13.0	14.4	15.6	Existing Revetment
8	9.7	1.2	10.2	73	15.0	14.6	16.9	Existing Revetment
9	9.7	1.2	10.2	73	18.1	16.8	19.0	Existing Revetment
10	11.2	1.2	12.4	>100	18.1	16.7	21.6	Existing Revetment
11	8.2	0.5	8.7	10	18.1	16.9	16.4	Existing Revetment
12	8.2	0.5	8.7	10	18.1	17.0	18.5	Sand Layer at Toe
13	5.2	0.1	5.2	<2	18.1	16.3	15.4	Larger Revetment
14	7.2	0.3	7.5	3	18.1	16.4	15.3	Larger Revetment
15	8.2	0.5	8.7	10	18.1	17.0	16.9	Larger Revetment
16	9.7	1.2	10.9	73	18.1	16.3	18.6	Larger Revetment

For the +7.2 foot water level, corresponding to a storm with a return period in the 3-year range (when wave setup is added by the waves), the maximum runup was to approximately +45 feet on the bluff. About 35% of waves overtopped the revetment berm. The stones armoring the bluff above the berm (the splash protection) were displaced and carried down onto the berm.

For the +9.7 foot water level, which corresponds to a storm return period of approximately 73 years (when wave setup is added by the waves), the maximum runup elevation exceeded +50 feet on the bluff. Over 90% of all waves overtopped the revetment berm. One or two of the smaller unsupported armor stones on the seaward edge of the berm were removed by waves. This was not considered a failure of the structure but just a local repositioning of unsupported smaller stones. Unsupported stones on the back edge of the berm were move landward by waves. The movement of these stones did not lead to unraveling of the armor layer.

For the +11.2 foot MLLW water level, which corresponds to a storm return period exceeding 100 years, the runup again exceeded +50 feet on the bluff. All of the waves overtopped the berm. The armor layer began failing about one-half way through the test, and quickly unraveled, leading to complete failure of the revetment section within about 30 minutes prototype time.

Figure A-5-4 compares the percent of waves which overtopped the berm and impacted the unprotected back berm as a function of storm return period. The figure indicates that at about a 10-year return period, 40% of the runups exceed the berm and would erode the bluff. At that frequency, the bluff would be eroded by consecutive waves during the highest parts of the wave groups, leading to flow rates that would likely undermine the berm, leading to revetment failure from the top down.

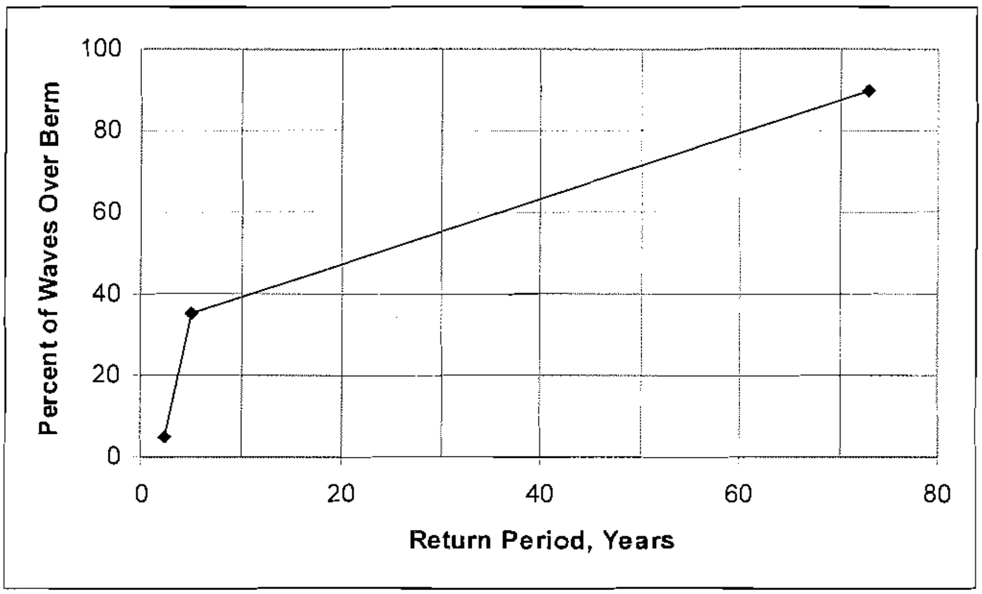


Figure A-5-4. Runup on bluff versus storm return period

Sand Layer Test

The sand layer test consisted of spreading a four-foot thick layer of sand in front of the revetment toe, extending out 120 feet. The storm waves were then run, and the sand layer was observed during and after the test. It was observed that the sand was displaced seaward, leading to a scour hole in front of the toe. In the one and one-half hour (prototype) test, the sand scoured to the wave tank bottom (through all four feet of sand), and extended approximately 40 feet seaward of the toe.

While the scour depth was limited by the fixed wave tank floor, and sediment movement rates are not linearly scaled in the wave tests, the sand layer test indicates that sand will be scoured from in front of the revetment under storm conditions in a relatively short period. It should be emphasized that observations at the site and discussions with Mr. Greg Donohue indicate that the bottom in front of the revetment consists of very firm glacial till, covered with a thin veneer of sand in the summer. The glacial till will be much more resistant to erosion than the sand layer and therefore the test results show only that toe scour might have been a problem had the bottom in front of the revetment been a removable beach sand.

Large Revetment Tests

In order to assess the performance of a larger revetment cross-section, similar to that proposed for a new Federal project, on wave runup and reflection, the revetment section was rebuilt to the following specifications, which are based on preliminary design plans for a new revetment section:

Berm Height:	25' NGVD
Berm Width:	40'
Slope:	1:2
Armor Layer Thickness:	11'
Underlayer Thickness:	5'
Filter Layer Thickness:	2'

The gradation and stone sizes of the armor layer and under layers were not as carefully modeled, as they would have been for armor stability tests. The general geometry, porosity, and stone sizes were maintained, so that the model properly simulated runup and reflection characteristics.

The large revetment had significantly less runup than did the existing revetment. At a water level of +5.2, no waves ran up on the bluff. With a water level of +7.2, a few waves reached the bluff, but did not run up on the bluff. With a water level of +9.7, the maximum runup reached approximately +40, with about 10 percent of the waves reaching the bluff.

It was noted that even when the waves did not reach the top of the berm at +25, water did flow through the armor stones and surge up the bluff behind the armor stones. Therefore, the bluff material should be covered with appropriate filter stones before application of the armor layers.

The reflectivity of the larger revetment was approximately 25 percent under the storm conditions tested. This compares to a measured reflectivity of approximately 30 percent for the existing revetment under similar conditions.

Conclusions

The model tests of the existing revetment demonstrate that the existing revetment armor layer is stable in storm waves with return period up to approximately 73 years. In storm waves with periods exceeding 100 years, the revetment rapidly fails. This assessment assumes that the armor layer remains in a condition similar to that observed in the field and model tested. The model tests do not account for changes in the geometry due to effects such as loss of toe stones, collapse due to loss of bluff support behind the structure, or loss of filter material from beneath the structure.

The pre-model interview with Mr. Greg Donohue of the Montauk Historical Society and field inspection indicated that the existing revetment was constructed very well (i.e. better than average interlocking, careful toe stone placement). These characteristics were replicated to the best degree possible in the tested model, and that this good construction has resulted in better than expected performance by the existing section, both in the model test and in the field.

Runup on the bluff above the top of the revetment was observed in storm waves with return periods in the 2 to 5 year range. This is consistent with observations by Mr. Greg Donohue, who has reported that green water over the top of the berm at +18 feet has damage the chain link fence between the berm and the bluff on several occasions over the past 10 years. During the past ten years, there have been no extreme storm events (since the 1992 storm).

For more extreme storms, such as the 10 to 20 year storm, the runup on the bluff is extreme, and would likely result in failure of the revetment (with lighter stones) by displacement with subsequent lower collapse back into the eroded zone. The armoring of the upper bluff varies so the exact location of the first failure will depend upon the direction of wave attack and the size of the armor protecting the upper bluff. In areas where the bluff is armored heavily, the top of the revetment may fail first.

The sand layer test indicated that sand in front of the toe of the revetment is eroded seaward in storm wave conditions, leaving a scour hole at the toe of the slope. It should be noted that observations at the site and discussions with Mr. Greg Donohue indicate that the bottom in front of the revetment consists of very firm glacial till, covered with a thin veneer of sand in the summer. The glacial till will be much more resistant to erosion than is the sand layer. The existence of armor stones from the 1945 revetment at the toe of the existing revetment indicates that the erosion of material in front of the revetment is relatively slow. The erosion in the cohesive material will tend to be more long term erosion, rather than storm erosion as would occur with a sand bottom.

Model tests of a larger revetment section, based on preliminary plans for a reconstructed section, indicate stability for storms with return periods greater than 100 years (the maximum tested here). For this section, runup is less severe for the larger section. However, the bluff will still need protection against runup erosion damage for design level storms up to an elevation of approximately +40. At the highest tested water level, only one or two waves reached +40' ft NGVD, with most reaching +25 to +30'. This test corresponded to a 100-year level (or slightly greater), and the maximum runup (Rmax) could be interpreted to be about +40 ft NGVD with a smooth plywood bluff slope. An estimate for a rough slope at this return period puts the average of the highest 2% of runups (R2%) at about +25 ft to +30 ft NGVD, which generally agrees with computations done by CEM procedures. The reflection of the larger proposed section is reduced to 25 percent, as compared to 30 percent for the existing structure. This reduction is due to a flatter armor slope, and more porous armor. This difference, although notable from an engineering perspective, should not result in noticeable difference to surfing conditions in the area.

Notes of meeting with Greg Donahue concerning the Montauk Lighthouse revetment history and construction.

Date: 1/29/2002

Attending: Dan Behnke, OCTI
Ed Fulford, Andrews Miller
Gary Williams, Andrews Miller
Greg Donahue

The purpose of the meeting was to obtain information from Greg Donahue concerning the history and construction of the Montauk Lighthouse revetment. The meeting took place at the lighthouse, and included a walking tour of the revetment with Mr. Donahue.

The general history of recent efforts to stabilize the bluffs around the lighthouse was presented by Mr. Donahue as follows:

1970's – Coast Guard construction of gabions

1989 – Historical Society began bluff stabilization, including bluff terracing, engineering and stone placement.

1989-1997 – State, Federal and local efforts constructed revetment in 5 phases, beginning with Phase 1 in 1992, the Phase 2 Federal segment in fall 1992 and spring 1993, Phase 3 in 1993, Phase 4 by the State in 1995, and Phase 5 by the State in 1996 and 1997.

Concerning construction of the existing revetment, Mr. Donahue reported that the sand and cobbles visible at the toe of the revetment is just a thin veneer on top of very firm glacial till. The remains of the old corps revetment, which can be seen seaward of the toe of the existing revetment, has settled into the glacial till bottom. A few of these old stones were attempted to be removed during construction of the existing revetment, and it was extremely difficult to remove them from the bottom, even with heavy construction equipment, because of the adhesion and suction of the clay-rich glacial till.

Based on Mr. Donahue's description of the natural existing bottom material, and observations of the old revetment stones sitting on the bottom at an elevation of approximately 0.0' NGVD, it appears that the existing bottom in front of the revetments is quite resistant to erosion. If erosion had taken place during large storms in the years since the construction of the old corps revetment, the armor stones would have sunk deeper into the bottom, and would not be sitting well above the bottom as they currently do. Long-term erosion may slowly erode away the bottom material, but it appears unlikely that it will significantly erode during a single storm event.

For the Phase 1 construction, filter cloth was laid on the existing slope, a modest layer of filter stone was laid on the filter cloth, and a single layer of armor stone was placed on the filter stone. The toe of the revetment was laid against the existing stones from the old corps revetment that were sunk in the clay bottom. The berm of the revetment is approximately +15-16 feet.

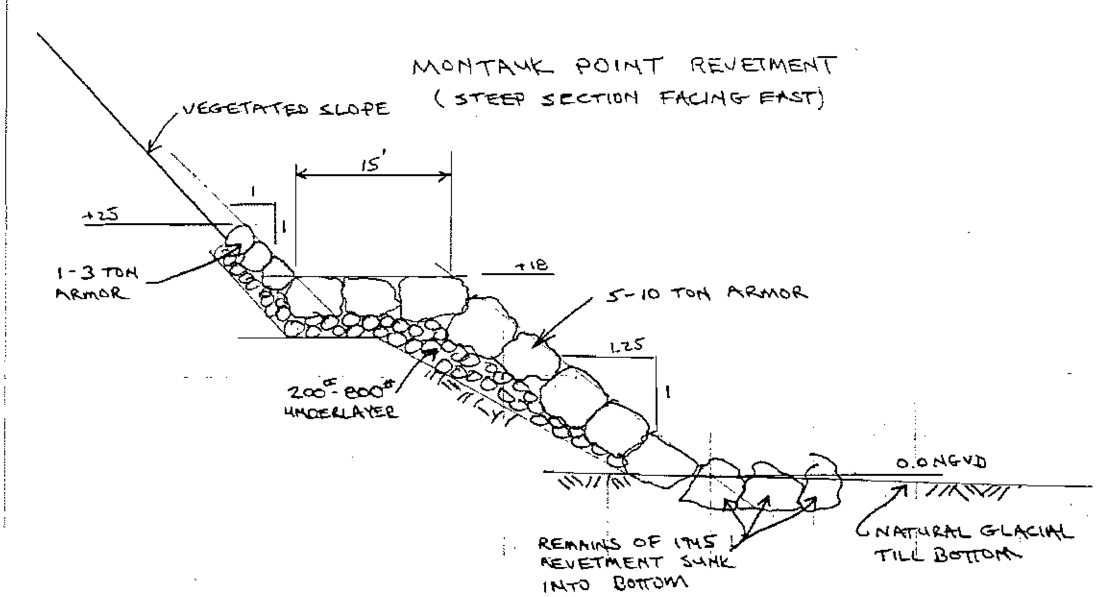
For Phase 2 construction, a construction roadway was built along the toe of the bluff. Filter cloth was laid on the bluff above the construction road. No filter cloth was used behind the construction road or beneath the toe. A layer of filter stone was laid over the construction road.

A single layer of armor stone was laid on the filter stone at approximately 1:1.5 slope. The toe of the filter stone was kept behind the existing stones from the old corps revetment. In some areas, the available distance between the existing stones from the old corps revetment and the construction road was not sufficient for the 1:1.5 slope, so the slope was steepened to fit the revetment in the available space. One such area, which faces due east, was estimated to be the worst case in terms of wave attack and stone stability, because of the steep slope. The slope in this area was estimated to be approximately 1:1.25. The crest elevation is +17', and the crest width is about 15 feet. Above the crest the bluff is protected by a layer of 200 to 500 pound rip rap over a layer of filter cloth. The primary armor stones are specified as 7 to 10 tons. A total of 20 stones were measured to obtain an estimate of the in-place stone size distribution.

For the Phase 3 construction, a toe trench was excavated in the glacial till bottom to –4', filter cloth was laid in the trench, filter stone of 200 –800 pounds was placed over the cloth, and armor stones were placed over the filter stones. The toe of the revetment for Phases 3-5 is behind the old stones from the corps revetment. The stone weights are 7-10 for the lower slopes, and 5-8 tons for the upper slopes/crest. The bluff slopes above the crest are armored with 1000-pound stone over filter cloth. Mr. Donahue showed a large number of photographs of construction of phases 3-5, showing the toe trench, filter cloth, filter stone, single layer of armor stone, etc. The stone fitting is quite tight and uniform, probably leading to greater stability than predicted by Hudson's equation with typical SPM stability coefficients, as long as the section stays intact.

Mr. Donahue reported that the chain link fence on top of the crest of the Phase 2 revetment, at an elevation of +17' to +18' has been damaged several times since construction, and that he has seen solid green water several feet deep over the top of the crest.

The following sketch was made of the section of the Phase 2 section judged to be the most vulnerable to damage due to wave exposure and steep slopes:



Based on my discussions with Mr. Donahue, examinations of photographs taken by Mr. Donahue during construction of the existing revetments, and examinations of the existing revetments, I believe that Mr. Donahue is knowledgeable concerning the construction of the existing revetments, and the conditions at the site. I place high confidence on the information provided by Mr. Donahue concerning the Montauk Lighthouse revetments.

Daniel L. Behnke, P.E.
Senior Coastal Engineer

Sub-Appendix A-6

NDT Engineering Seismic Report

GEOPHYSICAL SURVEY

MONTAUK POINT LIGHTHOUSE

MONTAUK, NEW YORK

PREPARED FOR

MICHAEL BAKER JR., INC

DECEMBER, 2001

NDT ENGINEERING, INC.

NDT ENGINEERING, INC

December 12, 2001

Mr. John Callahan, PG
Michael Baker Jr. Inc.
4301 Dutch Ridge Road
Beaver, Pennsylvania 15009-0280

Dear Mr. Glisan:

In accordance with your letter of authorization to proceed dated November 7, 2001 NDT Engineering conducted a seismic refraction and electrical resistivity geophysical study on November 28, 2001 at the Montauk Point Lighthouse in Montauk, New York. The objectives of this investigation were to evaluate soil layering and depth of ground water table.

This report presents the results and findings of this investigation.

Sincerely,
NDT ENGINEERING, Inc.

Paul S Fisk

Paul S. Fisk
President

P.O. Box 303 SHREWSBURY, MA 01545 Tel (508) 845-1950 Fax (508) 845-1952

TABLE OF CONTENTS

LIST OF FIGURES

.0 RESULTS

The results of the seismic refraction and electrical resistivity measurements are shown on profiles for lines SL 1 and SL 2, Figures 2 and 3 respectively. While the two profiles differ slightly the comments given below apply to both.

The results of the seismic survey show a 1,600ft/sec layer approximately 50ft thick underlain by a layer of 5,600 to 6,000ft/sec. The first layer has a velocity typical of sands and gravels. The second layer velocity (5,600 to 6,000ft/sec) is representative of a relatively compact glacial till. Boring blow counts should be used for soil strength evaluations.

Superimposed on the seismic profile are the results of the electrical resistivity survey. The best fit for the electrical survey is a four-layer case. The topmost layer is about 4feet thick with an electrical resistivity of 3,000ohm-meters underlain by a layer about 8 feet thick with a resistivity of 6,000 ohmmeters. These two layers are relatively dry sands and gravels. The third layer from about 8ft to about 45ft has an electrical resistivity of 106 ohmmeters. This layer is also indicative of sands and gravels with some silts and clays with sufficient moisture content to lower its resistivity. The lowest layer, below a depth of 45ft. is influenced by higher moisture content, this boundary is coincident with the top of the till layer however the resistivity value is due to the water content. In this case a true or perched water table on the top of the till.

2.0 INTRODUCTION AND PURPOSE

A seismic refraction and electrical resistivity investigation was conducted at the Montauk Point lighthouse on November 28, 2001 to evaluate soil layering and depth of ground water table.

3.0 METHODS OF INVESTIGATION

Survey Control
The locations of seismic refraction lines and resistivity sounding are shown on Figures 1 which is a portion of a Corps of Engineers plan provided to NDT Engineering by Michael Baker Jr., Inc. The location of the seismic refraction lines was determined by measurements referenced to roads, fences, buildings and other onsite landmarks. Ground surface elevations were determined from a ground surface contour map provided.

Seismic Refraction
A 24-channel seismic refraction system with geophone sensors spaced at 10 and 20-foot intervals and a "carbon electric industrial blank" energy source was used. to generate seismic energy at the ends, quarter point and center of each survey line in 2 to 3 foot deep holes. Measured travel times (in milliseconds) of compressional "P" wave energy were used to develop travel time plots used as a basis for data interpretation. A discussion of the seismic refraction survey method is included in Appendix A.

Electrical Resistivity Measurements

Electrical resistivity (inverse conductivity) measurements made at ground surface can be used to evaluate subsurface materials. The resistivity of earth materials is related to temperature, water content, salinity or ion content of water and matrix materials. For almost all earth materials the conductivity/resistivity is controlled by the presence of water. Dry sands, gravels and massive unweathered rock are typical relatively high resistivity whereas clays, water-saturated sediments or weathered bedrock have low resistivities.

The "apparent" resistivity value of a particular material, measured in the field, is a function of the material's true resistivity, the thickness of the unit, thickness and resistivity of adjacent layers, and the electrode spacing. Apparent resistivity values are calculated based on the configuration of current and potential electrodes. Interpretation of electrical resistivity data performed by computer inverse modeling.

The field technique used for this investigation was vertical sounding or point test. A resistivity point test is analogous to drilling; the results of a point test consists of vertical profile of units defined by resistivity characteristics, similar to a lithologic sequence developed from drilling data. A point test is conducted by incrementally increasing the spacing between electrodes, maintaining the electrode configuration about a single point. Resistivity measurements obtained at greater electrode separations are sampling deeper in the earth. For this investigation the Lee Partition of the Wenner electrode configuration was used. An electrical current is applied across the outer electrodes and the change in voltage is measured between the inner pair of potential electrodes. The electrode spacings used for this investigation were 3, 5, 7, 10, 20, 30, 40, 50, 70, 100, 120 and160-feet.

TABLES

RESISTIVITY TABLE FOR LEE PARTITION

LOCATION: LONG ISLAND NY   DATE: 112801   CLIENT: BAKER ENG   JOB N4159

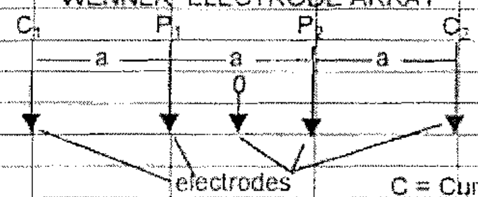
SL1

spacing ft	resistance p1-p2(ohmft	resistance 0-p1(ohmft)	resistance 0-p2(ohmft)	resistiivity 0-p1(ohmft)	resistivity 0-p2(ohmft)	resistivity p1-p2(ohmft)	resistivity p1-p2(ohmm)
3	405.8	204.6	201.3	3858	3796	7649	2332
5	257	125.8	131.7	3954	4139	8074	2461
7	183.7	95.2	88.4	4189	3890	8079	2463
10	130.3	69.1	61.1	4344	3841	8187	2496
20	47.1	24.7	22.4	3105	2816	5919	1804
30	19.8	10.1	9.6	1905	1810	3732	1138
40	12.6	5.6	7.1	1408	1785	3167	965
50	9.4	4	5.5	1257	1729	2953	900
70	5	2.1	2.9	924	1276	2199	670
100	2.1	0.8	1.3	503	817	1319	402
120	1.4	0.5	0.9	377	679	1056	322
160	0.6	0.2	0.6	201	603	603	184

SL2

spacing ft	resistance p1-p2(ohmft	resistance 0-p1(ohmft)	resistance 0-p2(ohmft)	resistiivity 0-p1(ohmft)	resistivity 0-p2(ohmft)	resistivity p1-p2(ohmft)	resistivity p1-p2(ohmm)
3	522.9	275.6	254	5197	4790	9856	3005
5	355.7	186.8	170.1	5871	5346	11174	3407
7	255	137.2	118	6037	5192	11215	3419
10	168.4	94.4	75	5934	4715	10581	3226
20	42	21.5	20.6	2703	2590	5278	1609
30	14.7	6.6	8.2	1245	1546	2771	845
40	4.6	2.03	2.52	510	634	1156	352
50	2.14	9.6	1.18	3017	371	672	205
70	1.58	0.73	0.85	321	374	695	212
100	0.73	0.4	0.33	251	207	459	140
120	0.38	0.21	0.17	158	128	287	87

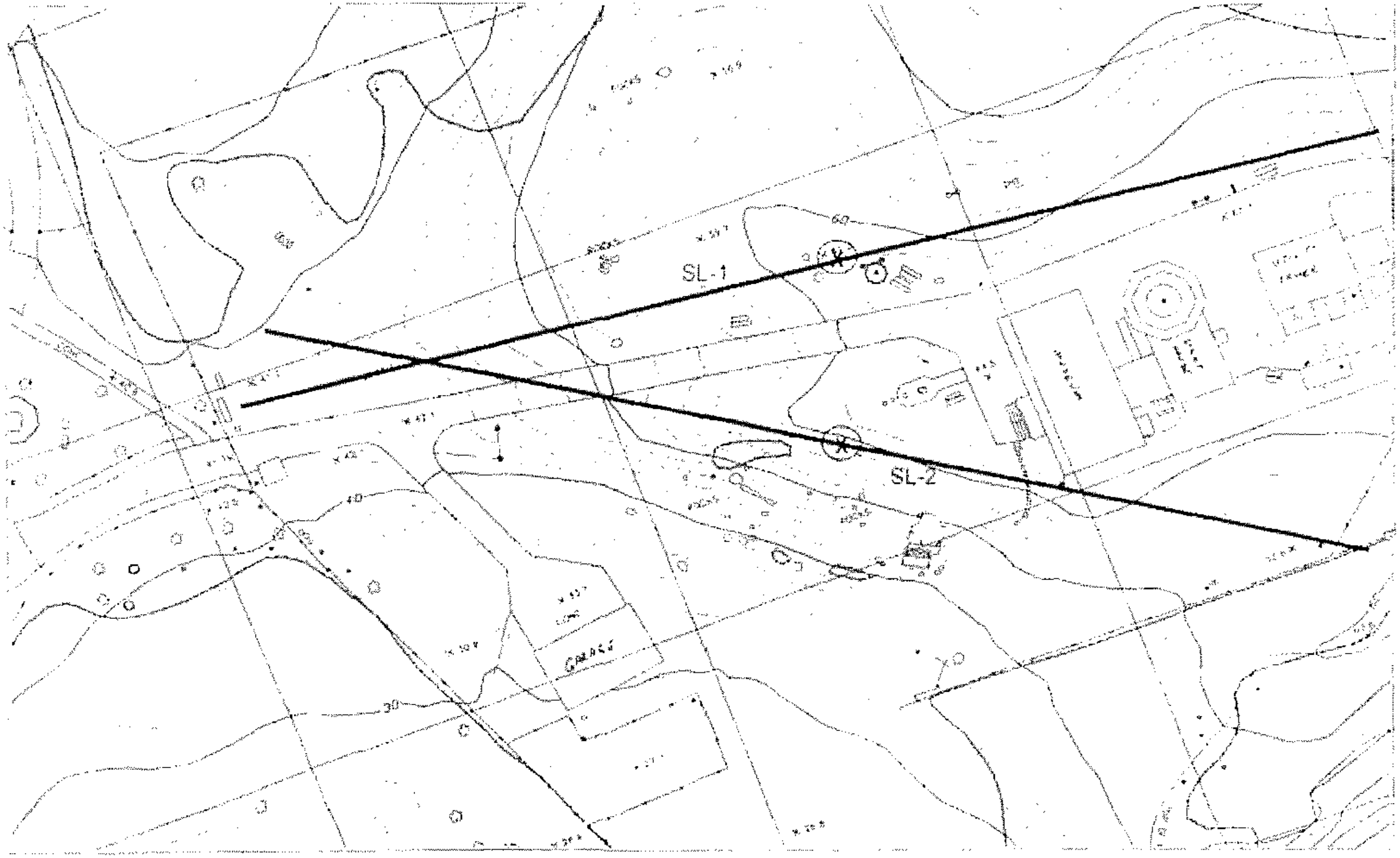
WENNER ELECTRODE ARRAY



C = Current electrode
P = Potential electrode

FIGURES

MONTAUK POINT LIGHTHOUSE AREA OF INVESTIGATION



(X) CENTER OF RESISTIVITY SOUNDING

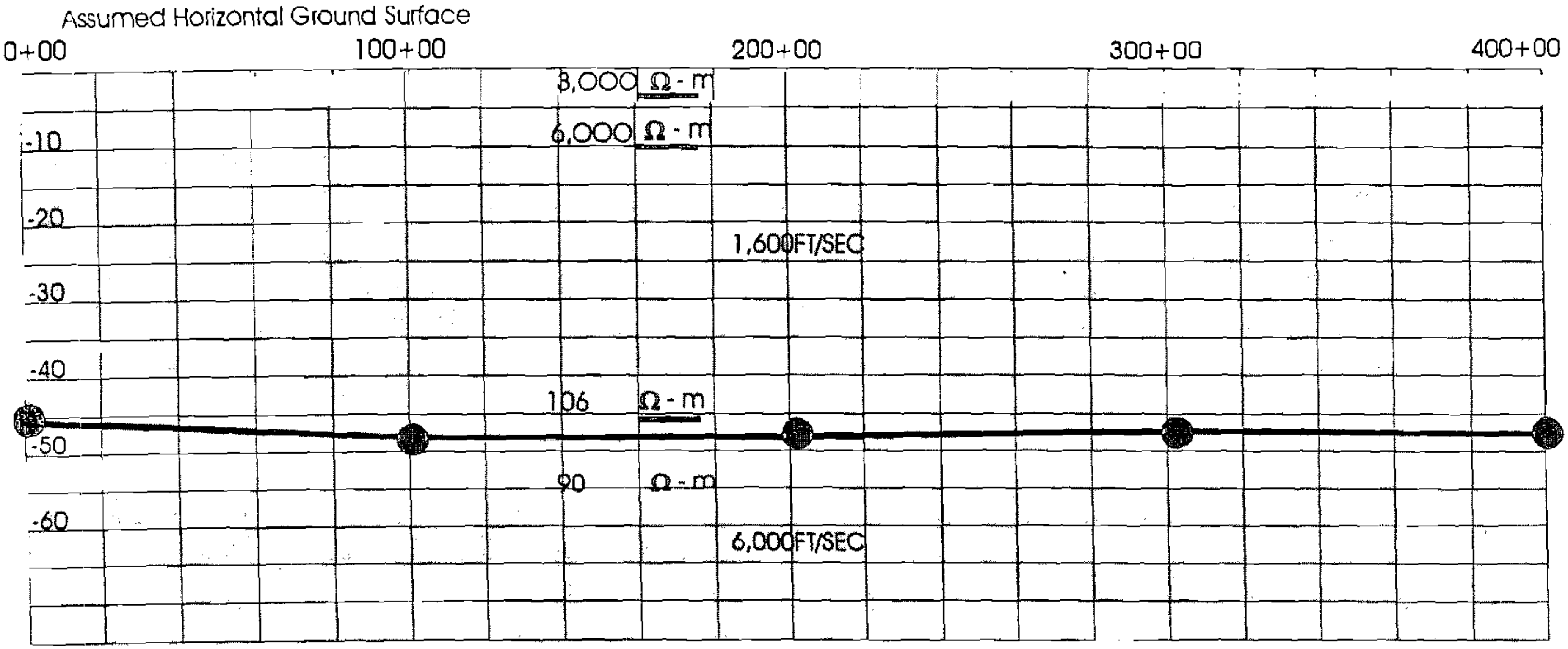
— SEISMIC LINE

FIGURE 1

[redacted]

Seismic and Electrical Sounding Results
Montauk Point Long Island New York

LINE SL-1



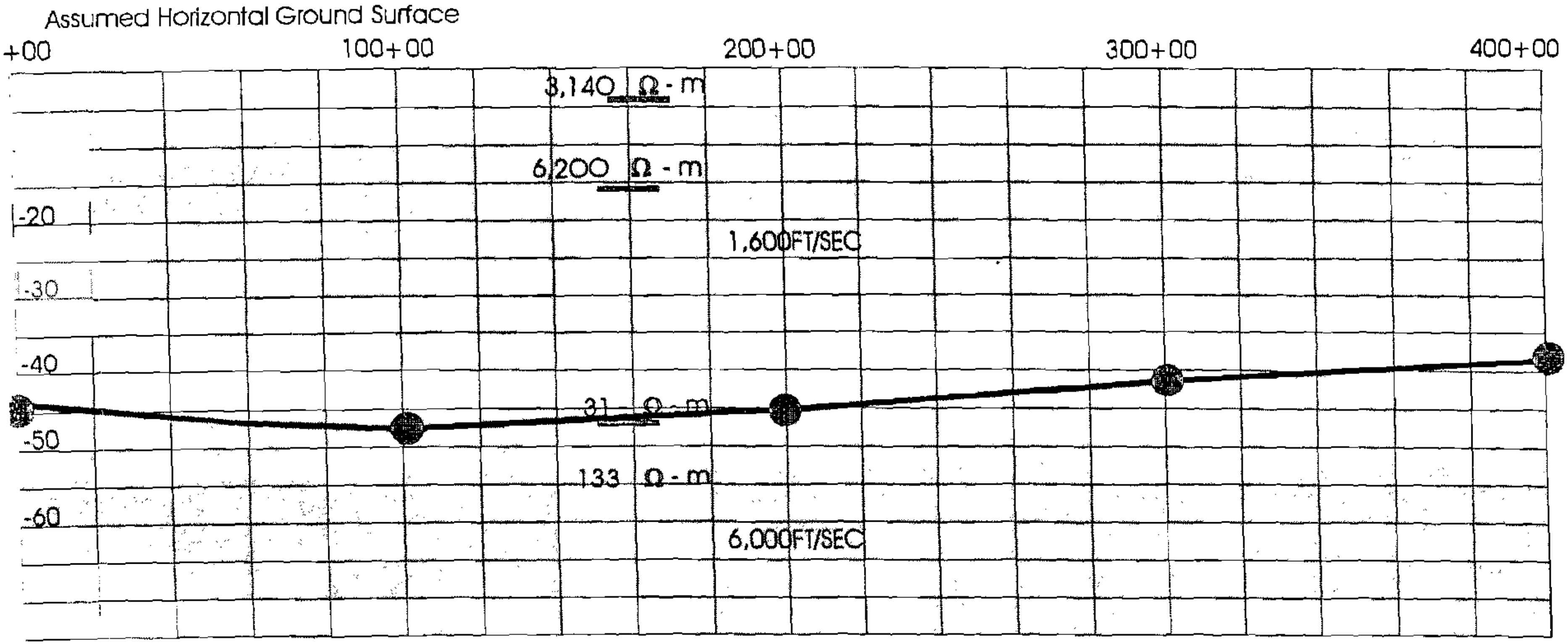
1" = 40' Hor
1" = 20' Ver

Electrical Resistivity Ohm - meters Ω - m
Seismic Velocity in Feet/Second

NDT ENGINEERING INC.

Seismic and Electrical Sounding Results
Montauk Point Long Island New York

LINE SL-2



1" = 40' Hor
1" = 20' Ver

Electrical Resistivity Ohm - meters Ω - m
Seismic Velocity in Feet/Second

NDT ENGINEERING INC.
FIGURE 3

APPENDIX 1

APPENDIX: SEISMIC REFRACTION

OVERVIEW

Seismic exploration methods utilize the natural energy transmitting properties of the soils and rocks and are based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson's ratio) of the materials. Energy is generated at the ends and at the center of the seismic spread. The geophone/hydrophone is in direct contact with the earth/water and converts the earth's motion resulting from the energy generation into electric signals with a voltage proportional to the particle velocity of the ground motion. The field operator can amplify and filter the seismic signals to minimize background noise. Data are recorded on magnetic disk and can be printed in the field. Interpretations are based on the time required for a seismic wave to travel form a source to a series of geophones/hydrophones located at specific intervals along the ground surface. The resultant seismic velocities are used for:

- Material identification.
- Stratigraphic correlation.
- Depth determinations.
- Calculation of elastic moduli values and Poison's ratio.

A variety of seismic wave types, differing in resultant particle motion, are generated by a near surface seismic energy source. The two types of seismic waves for seismic exploration are the compressional (P) wave and the shear (S) wave. Particle motion resulting from a (P-wave) is an oscillation, consisting of alternating compression and dilatation, orientated parallel to the direction of propagation. An S-wave causes particle motion transverse to the direction of propagation. The P-wave travels with a higher velocity of the two waves and is of greater importance for seismic surveying. The following discussions are concerned principally with P-waves.

Possible seismic wave paths include a direct wave path, a reflected wave path or a refracted wave path. These wave paths are illustrated in FIGURE A1. The different paths result in different travel times, so that the recorded seismic waveform will theoretically show three distinct wave arrivals. The direct and refracted wave paths are important to seismic refraction exploration while the reflected wave path is important for seismic reflection studies.

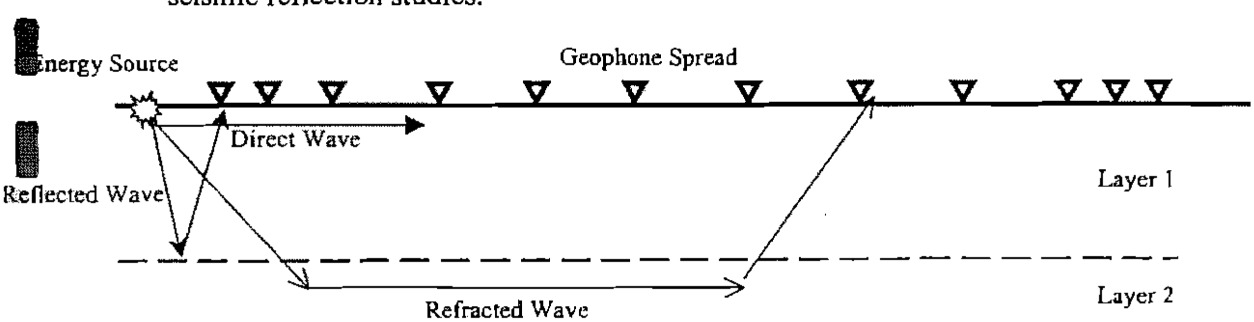


FIGURE A1:
SEISMIC WAVE PATHS FOR DIRECT WAVE, REFLECTED WAVE AND REFRACTED WAVE ILLUSTRATING EFFECTS OF A BOUNDARY BETWEEN MATERIALS WITH DIFFERENT ELASTIC PROPERTIES

At small distances between the energy source and detector the first arriving seismic signals will be direct waves that travel near the ground surface through lower velocity materials. At greater distances, the first arrival will be refracted waves that have taken an incident path through the two layers. The refracted wave arrives before the direct velocity materials compensate for the longer path. Depth calculations are based on the ratio of the layer velocities and the horizontal distance from the energy source to the point that the refracted wave overtakes the direct wave.

Seismic waves incident on the interface between materials of different elastic properties at what is termed the critical angle are refracted and travel along the top of the lower layer. The critical angle is a function of the seismic velocities of the two materials. These same waves are then refracted back to the surface at the same angle. The recorded arrival times of these refracted waves, because they depend on the properties and geometry of the subsurface, can be analyzed to produce a vertical profile of the subsurface. Information such as the number, thickness and depths of stratigraphic layers, as well as clues to the composition of these units can be ascertained.

The first arrivals at the geophones/hydrophones located near the energy source are direct waves that travel through the near surface. At greater distances, the first arrival is a refracted wave. Lower layers typically are higher velocity materials, therefore the refracted wave will overtake both the direct wave and the reflected wave, because of the time gained travelling through the higher velocity material compensates for the longer wave path. Depth computations are based on the ratio of the layer velocities and the distance from the energy source to the point where refracted wave arrivals over take direct arrivals.

Although not the usual case, a constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole data are available.

APPLICATIONS

Seismic refraction technique is an accurate and effective method for determining the thickness of subsurface geologic layers. Applications for engineering design, assessment, and remediation as well as ground water and hydrogeologic studies include:

Continuous profiling of subsurface layers including the bedrock surface
* Water-table depth determinations
* Mapping and general identification of significant stratigraphic layers
* Detection of sinkholes and cavities
* Detection of bedrock fracture zones
* Detection of filled-in areas
* Elastic moduli and Poisson's ratio values for subsurface layers

Seismic refraction investigations are particularly useful because seismic velocities can be used for material identification. FIGURE A2 presents a guide to material identification based on P-wave seismic velocities. In rocks and compacted overburden material, the seismic waves travel from grain to grain so that the measured seismic velocity value is a direct function of the solid material. In porous or fractured rock and most overburden materials the seismic waves travel partly or wholly though the fluid between the grains.

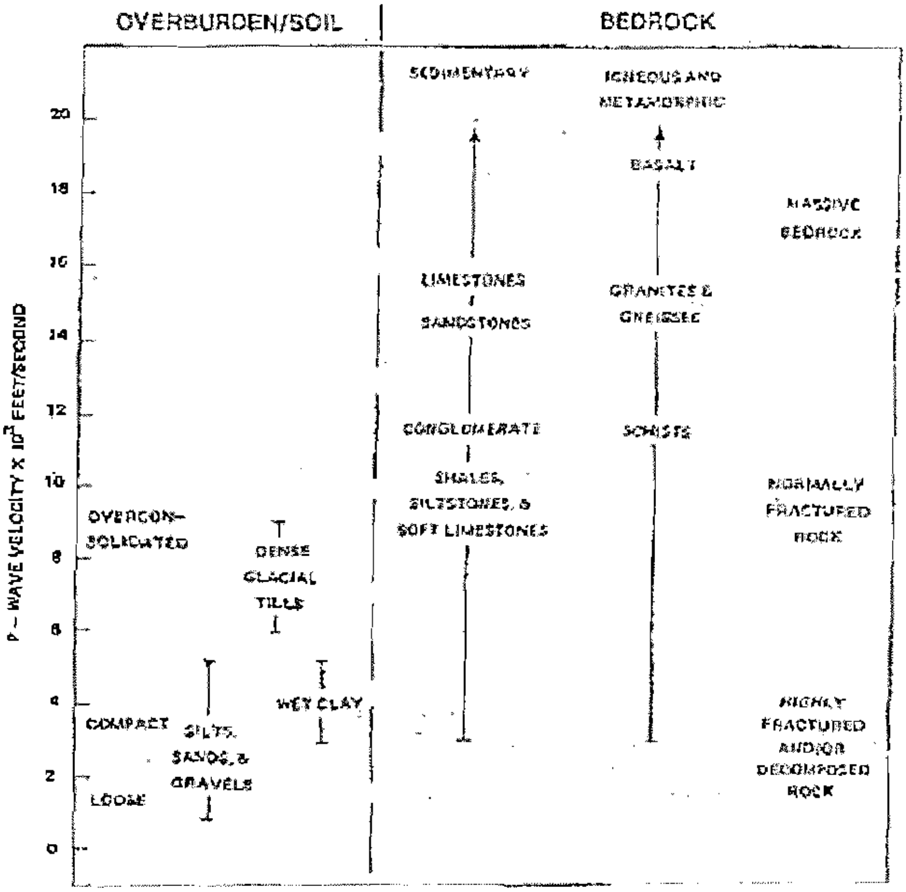


FIGURE A2:
GUIDE TO MATERIAL IDENTIFICATION BY P-WAVE VELOCITY

Seismic compressional wave velocities in unconsolidated deposits are significantly affected by water saturation. The seismic velocity values of unsaturated overburden materials such gravels, sands and silts generally fall in the range of 1,000 to 2,000 ft/sec. When these materials are water saturated, that is when the space between individual grains are 100% filled with water, the seismic velocities range from 4,800 to 5,100 ft/sec, equivalent to the compressional P-wave velocity of sound in water. This is because the seismic wave assumes the velocity of the faster medium, that of water. Even a small decrease in the saturation level will substantially lower the measured P-wave velocity of the material. Because of this velocity contrast between saturated and unsaturated materials, the water table acts as a strong refractor.

Seismic investigations over unconsolidated deposits are used to map stratigraphic discontinuities and to unravel the gross stratigraphy of the subsurface. These can be vertically as in the case of a dense till layer beneath a layer of saturated material or horizontally as in the case of the boundaries of a fill material. Often these boundaries represent significant hydrologic boundaries, such as those between aquifers and aquicludes.

A common use of seismic refraction is the determination of the thickness of a saturated layer in unconsolidated sediments and the depth to relatively impermeable bedrock or dense glacial till. Continuous subsurface profiles and even contour maps of the top of a particular horizon or layer of interest can be developed from a suite of seismic refraction data.

Bedrock velocities FIGURE A2 vary over a broad range depending on variables, which include:

* Rock type
* Density
* Degree of jointing/fracturing
* Degree of weathering

Fracturing and weathering generally reduce seismic velocity values in bedrock. Low velocity zones in seismic data must be evaluated carefully to determine if they are due to overburden conditions or fractured/weathered or perhaps even faulted bedrock.

<u>EQUIPMENT:</u>

The basic equipment necessary to conduct a seismic refraction investigation consists of:

* Energy source
* Seismometers (Geophones/Hydrophones)
* Seismic cables
* Seismograph

Energy sources used for seismic surveys are categorized as either non-explosive or explosive. The energy for a non-explosive seismic signal can be provided by one of the following:

* Sledge Hammer (very shallow penetration)
* Weight Drop
* Seisgun

* Airgun
* Sparker
* Vibrators (for reflection surveys)

Explosive sources can be categorized as:

* Dynamite
* Primers
* Blasting Agents

Choice of energy source is dependent on site conditions, depth of investigation, and seismic technique chosen as well as local restrictions. Explosive sources may be prohibited in urban areas where non-explosive sources can be routinely used. Deeper investigations usually require a larger energy source: therefore, explosives may be required for sufficient penetration.

Geophones/Hydrophones are sensitive vibration detectors, which convert ground motion to an electric voltage for recording the seismic wave arrivals. Seismic cables, which link the geophones/hydrophones and seismograph are generally fabricated with pre-measured locations for the geophones/hydrophones and shot point definitions.

The seismograph can be single channel or multi-channel, although, multi-channel seismographs (12 to 24 channels) are preferred and necessary for all but the simplest of very shallow surveys. The seismograph, amplifies (increases the voltage output of the geophones), conditions/filters the data, and produces analog and digital archives of the data. The analog archive is in the form of a thermal print of the data, which can be printed directly after acquisition in the field. The digital archive is stored on magnetic disk and can be used for subsequent computer processing and enable more extensive and detailed interpretation of seismic data.

<u>ACQUISITION CONSIDERATIONS:</u>

Several concerns arise before data collection, which must be addressed before of any seismic survey:

* Geophone spacing and Spread length
* Energy Source (discussed above)
* On-site utilities and cultural features (buildings, high tension lines, buried utilities, etc.)
* Vibration generating activities
* Geology
* Topography

To acquire seismic refraction data, a specific number of geophones are spaced at regular intervals along a straight line on the ground surface; this line is commonly referred to as a seismic spread. The length of spread determines the depth of penetration; a longer spread is required for a greater depth of penetration. Spread length should be approximately three to five times the required depth of penetration. Required resolution will control the number of geophones in each spread and the distance between each gcophone. Closer spacings and more geophones usually result in more detail and greater resolution.

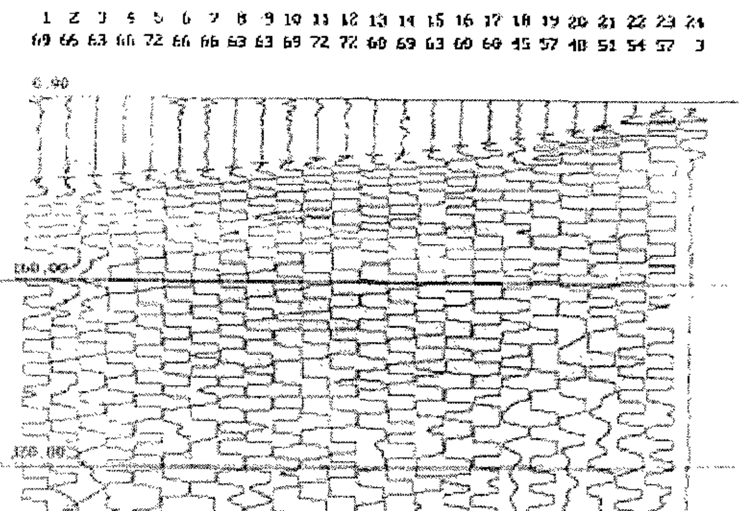
Cultural effects such as vibration generating activities, on-site utilities, and building affect where data can be acquired, and where lines/spreads are located. High volume traffic areas may require nighttime acquisition. If the survey is to be conducted near a building where vibration-sensitive manufacturing is conducted, data acquisition may be constrained to particular time intervals and appropriate energy sources must be used. Over head and buried utilities must be located an avoided, for both safety and induced electrical noise concems. Since the seismic method measures ground vibration, it is inherently sensitive to noise from a variety of sources such as traffic, wind, rain etc. Signal Enhancement, such as record stacking, accomplished by adding a number of seismic signals from a repeated source, causes the seismic signal to "grow" out of the noise level, permitting operation in noisier environments and at greater source to phone spacings.

Knowledge of site geology can be used to determine the energy source. Some geologic materials, such as loose, unsaturated alluvium, do not transmit seismic energy as well and a powerful energy source may be required. Geologic conditions also dictate whether or not drilled shotholes are required. Site geology can also dictate the positioning of seismic lines/spreads. Where a bedrock depression of a feature is suspected, seismic lines should be orientated perpendicular to the suspected trend of the feature. Seismic cross profiles may be necessary to confirm depths to a particular refracting horizon.

The topography of a site dictates whether or not surveyed elevations are required. If possible, refraction profile lines should be positioned along level topography. For highly variable topography, a continuous elevation profile may be required to ensure sufficiently accurate cross-sections and to permit the use of time corrections in the interpretation of the refraction data.

DATA PRESENTATION AND INTERPETATION:

Interpretation of seismic refraction data involves solving a number of mathematical equations with the refraction data as it is presented on a travel-time versus distance chart. Seismic refraction data FIGURE A3 can be processed by plotting the "First Arrival" travel times at each geophone location. The preferred format of data presentation is a graph (Travel Time Plot) illustrated in FIGURE A4, in which travel time in milliseconds is plotted against source-receiver distance. From such a chart, the velocities of each layer can be obtained directly from the increase slope of each straight-line segment. Using the velocities the critical angle of refraction for each boundary can be calculated using Snell's Law. Then, utilizing these velocities, and angles and the recorded distances to crossover points (where line segments cross); the depths and thickness of each layer can be calculated using simple geometric relationships.



Shot point is 0.00

Channel	1	Phone	location	195.00	Arrival	time	71.3	msec
Channel	2	Phone	location	190.00	Arrival	time	70.3	msec
Channel	3	Phone	location	185.00	Arrival	time	68.5	msec
Channel	4	Phone	location	175.00	Arrival	time	66.3	msec
Channel	5	Phone	location	165.00	Arrival	time	63.8	msec
Channel	6	Phone	location	155.00	Arrival	time	63.0	msec
Channel	7	Phone	location	145.00	Arrival	time	62.0	msec
Channel	8	Phone	location	135.00	Arrival	time	59.0	msec
Channel	9	Phone	location	125.00	Arrival	time	56.8	msec
Channel	10	Phone	location	115.00	Arrival	time	54.8	msec
Channel	11	Phone	location	110.00	Arrival	time	51.5	msec
Channel	12	Phone	location	105.00	Arrival	time	49.3	msec
Channel	13	Phone	location	95.00	Arrival	time	47.3	msec
Channel	14	Phone	location	90.00	Arrival	time	45.0	msec
Channel	15	Phone	location	85.00	Arrival	time	43.0	msec
Channel	16	Phone	location	75.00	Arrival	time	39.0	msec
Channel	17	Phone	location	65.00	Arrival	time	37.0	msec
Channel	18	Phone	location	55.00	Arrival	time	33.0	msec
Channel	19	Phone	location	45.00	Arrival	time	28.0	msec
Channel	20	Phone	location	35.00	Arrival	time	25.0	msec
Channel	21	Phone	location	25.00	Arrival	time	20.0	msec
Channel	22	Phone	location	15.00	Arrival	time	12.0	msec
Channel	23	Phone	location	10.00	Arrival	time	8.0	msec
Channel	24	Phone	location	5.00	Arrival	time	7.0	msec

FIGURE A3:
TYPICAL 24 CHANNEL ANALOG SEISMIC REFRACTION RECORD, WITH FIRST ARRIVAL TIMES

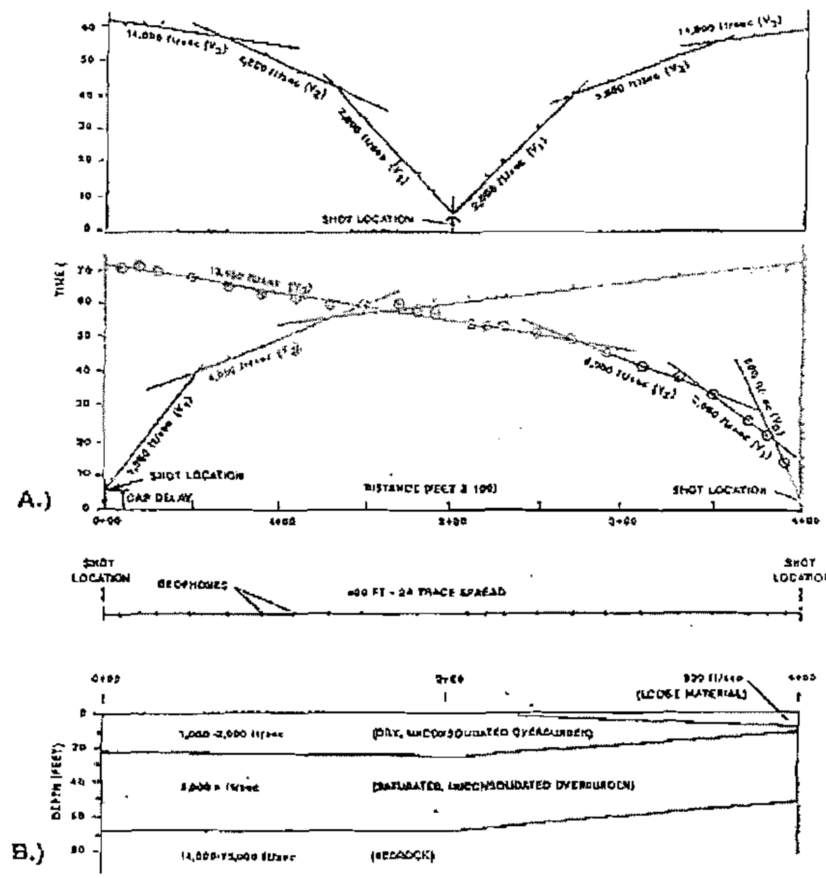


FIGURE A4:
A: TRAVEL-TIME PLOTS; UPPER PLOT IS A CENTER SHOT, LOWER PLOT IS TWO END SHOTS
B: RESULTING PROFILE OF SUBSURFACE MATERIALS SHOWING INTERFACE BETWEEN DIFFERENT SEISMIC VELOCITY LAYERS

The results of any seismic survey, refraction or reflection are usually presented in profile form showing elevations of seismic horizons/layers. Data acquired on a grid basis can be contoured and used to construct isopach maps. Seismic velocities and therefore, generalized material identifications should be presented on refraction profiles along with any test borings used for correlation to establish confidence in the overall subsurface data, both seismic and borings.

Where profiles indicate dipping boundaries, calculation of dips, true depths and true velocities involve more complicated equations. Further more, corrections for differing elevations and varying thicknesses of weathered zones must often be madc. Fracturing and weathering generally reduce seismic velocity values in bedrock. Consequently, travel-time plots with late arrivals must be evaluated carefully to determine if the late arrival times (slower velocities) are due to overburden conditions or fractured/weathered bedrock.

Very thin layers or low velocity zones often complicate the travel-time chart as well. Although not the usual case, one constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole velocity data are available.

ADVANTAGES AND LIMITATIONS:

The seismic refraction technique, when properly employed, is the most accurate of the geophysical methods for determining subsurface layering and materials. It is extremely effective in that as much as 2,000 linear feet or more of profiling can be acquired in a field day. The resulting profiles can be used to minimize drilling and place drilling at locations where borehole information will be maximized resulting in cost-effective exploration. A standard drilling program runs the risk of missing key locations due to drillhole spacing. This risk is substantially reduced when refraction is used.

In summary, the advantages and limitations of the seismic techniques are:

Advantages:

* Material identification
* Subsurface data over broader areas at less cost than drilling
* Relatively accurate depth determination
* Correlation between drillholes
* Preliminary results available almost immediately
* Rapid data processing

Limitations:

* As depth of interest and geophone spacing increases, resolution decreases
* Thin layers may be undetected
* Velocity inversions may add uncertainty to calculations
* Susceptible to noise interference in urban areas, which require use of grounded cables and equipment, signal enhancement and alternative energy sources.

Sub-Appendix A-7

Further Discussion Including Downdrift Impacts, Contribution to Littoral Drift, Surfing Impacts, and Moving the Lighthouse as an Alternative

As Resulted from Public Information Sessions Held in September 2005

Introduction

At a public information session held 19 September 2005, Corps representatives agreed to develop additional explanation for the Feasibility Report regarding the potential project effects on coastal and littoral processes. The following explanation, though lengthy, provides a fairly simple, yet thorough, discussion regarding the coastal and littoral process considerations involved in the plan formulation evaluation.

Coastal Processes. The proposed project alternative to protect Montauk Point will essentially continue the coastal process impacts of the 1992-1993 revetment over the 50-year planning horizon. The effects on coastal processes at the Point from man made interventions date back nearly 60 years. Shoreline erosion rates are a function of many factors including the type of material at each shoreline location, total mass of material at each location, and the wave energy impinging upon it. The Long Island shoreline both north and south of Montauk Point consists of a series of concave and convex shoreline reaches, indicating a variation in erodability alongshore. Historic shoreline mapping shows that the 1870, 1933 and 1938 shorelines, preceding construction of revetments at Montauk Point in the 1940's, all have an indentation at Turtle Cove, similar to the shoreline shape that exists now. This indicates that the Turtle Cove reach erodes at a faster rate than the point, and that this indented curvature was not caused by construction of the revetment. Shorelines also show that the Atlantic shore west of the point has been generally erosive over the 1870-1995 period. The existing and the proposed revetment alternative will act to hold or anchor the eastern most end of the Turtle Cove area. While the proposed revetment alternative will continue to limit the coastal erosion at the Point protecting the Lighthouse complex, forces such as tidal currents and waves will continue to erode the north and south downdrift shores continuing the curvature process, especially immediately adjacent to the revetment.

Downdrift Effects. The erosion of the areas just west of the Point tends to make up for the material covered by the existing revetment that is prevented from entering the littoral system. Since the proposed revetment alternative will extend over virtually the same lineal extent of the shoreline as the existing revetment, the proposed revetment will continue this trend. The effect of both the existing revetment and the proposed revetment on the downdrift, unprotected shoreline is to reduce the amount of sand moving alongshore by a small amount estimated to be approximately 3,000 cy/yr for the south shore. The sediment deficit would tend to increase the erosion rate of shoreline immediately adjacent to the revetment by a similar increment, over a short distance downdrift. Shoreline change mapping for the Turtle Cove area shows approximately a 2-ft/yr increase in retreat rate in the post-revetment period (1938-to 1995) as compared to the pre-revetment period (1870-1938), which may be due in part to the construction of the existing revetment, as well as to all other factors contributing to shoreline erosion. Indications from the shoreline change mapping are that any sediment deficits caused by the revetment are made up within approximately 2,500 feet of the end of the structure, at which point the shorelines changes to a pattern of fluctuation of erosion and accretion. Shorelines north of the existing revetment have a similar response, which would continue with the proposed structure in place. At the same time, for shoreline further westward, the relative impact of the small amount of reduced littoral material due to the revetment also diminishes because the net amount of littoral material moving westerly along the shores gradually increases as the waves and currents act upon the more westerly shores.

The curvature of the immediately downdrift or westerly shore areas will continue but will not pinch off or cause flanking of the protected area during the 50 year project evaluation period. It is also likely that as the curved recession due to waves and currents continues, there may be a tendency toward stabilization as the landform adjusts to the prevailing coastal forces. Subject to the occurrence of future storm events, the rate of erosion observed in relatively recent years could be somewhat reduced.

The following paragraphs give an indication of the volumes of littoral material involved in determining downdrift impacts of the project:

<u>Contribution to Littoral Drift</u>. Erosion of bluffs and beach fronting the Montauk Lighthouse does contribute a small amount of material to the littoral drift that moves along the Atlantic shore southwest of the lighthouse as well as along the Block Island Sound shore northwest of the lighthouse. Based on measurement over the period of record (1868-1993) the average annual erosion rate of bluff and beach as stated in the Feasibility Report, is estimated at 6 cubic yards per year per foot of shoreline. This average includes periods of time when the bluff was covered by a revetment, as well as prior to the construction of protective works. As such, this long-term average is a reasonable estimate for shoreline loss for periods of time covering conditions when the protective works are intact and limit loss, and periods of time when the protective works have lost integrity, resulting in greater amounts of erosion.

The proposed revetment covers approximately 840 linear feet of shoreline, which is virtually identical to the length of the existing revetment. Using the long-term erosion rate of 6 cubic yards (cy) per linear foot per year, approximately 5,000 cy of material can be assumed lost from the beach and bluff per year. Note that the material comprising the bluff is a mix of sand, gravel, and silt, based on boring logs. Some percentage of the eroded material is lost to the net longshore sand transport as it is either too fine, or too coarse. This percentage could be in the 10-30% range, but for this discussion no reduction has been taken for loss of fines or coarse material to the quantity of sediment transport.

The estimated volume of material per year will move either northwest or southwest of the point, except for some finer sediment moved permanently offshore or larger material that settles out. Based on the shape of the point and the length of the revetment, approximately 60% of the eroded material can be expected to move southwest, and approximately 40% can be expected to move northwest, i.e. approximately 3,000 cy/year will be contributed to the littoral drift along the Atlantic shore and approximately 2,000 cy/year along the Block Island Sound shore from the revetted shoreline. The shores to the northwest and southwest tend to make up for the effects of the revetment by contributing substitute littoral material over a relatively short distance downdrift.

For comparative purposes along the Atlantic shore, sediment budget estimates of the net longshore sand transport rate moving westward at Ditch Plains is approximately 70,000 cy per year. At the Montauk beaches, the net longshore transport rate increases to approximately 100,000 cy/yr. There is considerable uncertainty in sediment budget calculations, such that the given amount may differ by +/-40,000 cy/yr. Effects of both the existing structure at Montauk and the proposed structure on the littoral sand transport are small and are expected to be local, i.e. are not likely to extend as far alongshore to developed areas such as Ditch Plains or more developed points further west, especially in view of the variable shoreline shape and materials in between.

The amount of material that would be contributed downdrift if the proposed revetment alternative was not constructed is very small relative to the total sediment budget and this small amount is added back by some equally minor increase in erosion, a relatively short distance west of the revetment. Any adverse impacts from the proposed revetment alternative are not considered to be significant, and would not materially affect areas being considered for protection under the Fire Island Inlet to Montauk Point Reformulation Study.

<u>Impacts to Surfing:</u> There is a revetment in place at Montauk Point now, and the surfing is considered to be good. The proposed revetment will be in the same place as the existing revetment, made of similar rock material, and will be similar to it in all particulars which might effect waves. As shown in the Feasibility Report, Table 10, and discussed in Section 4.0 - Recreation, wave reflection coefficients are estimated at 13-19 percent less for the proposed

revetment alternative than for the existing revetment. The proposed plan will not change wave conditions in any perceivable way.

<u>Moving the Lighthouse:</u> The moving of the Lighthouse was given considerable weight during the Feasibility phase of the project. However, several factors contributed to the decision not to make this proposal the preferred alternative. They included: a) the overall cost of the alternative b) the engineering requirements of having to build up land to meet the hill of Montauk Point to create a level moving surface, c) the destruction of a National Register Landmarked complex - by moving it, the setting is destroyed thus violating the spirit of the National Historic Preservation Act of 1966, as amended, d) the loss of value to the Town of Easthampton, Montauk Point and Montauk Point State Parks, as several hundred thousand visitors come to this area each year, in part to see "the end", i.e. Montauk Point Lighthouse, e) the New York State Office of Parks, Recreation and Historic Preservation (see Letter Number 01), the Regulatory Agency that would have to approve any move of a National Register structure has already stated, and has done so throughout the entire process, that they would not approve the moving of the Lighthouse, which would lead to the destruction of the Lighthouse complex area.

Additionally, while the Montauk Historical Society maintains the lighthouse complex, the U.S. Coast guard still operates the beacon and the foghorn as working aids-to-navigation. If the lighthouse were not present, the U.S. Coast Guard would likely erect a tower on which to mount a replacement beacon. As per the agreement signed during the transfer of the property from the Federal Government to the Montauk Historical Society, If the Montauk Historical Society fails to protect or maintain the lighthouse, the property would revert back to the USCG. Please note that in the analysis of the without and with-project conditions adjustments were made to account for sea level rise.

MONTAUK POINT, N.Y. - ECONOMICS APPENDIX – Oct 2005

General

1. The feasibility study is being conducted under the following study authority: 15 May 91:

"Resolved by the Committee on Environment and Public Works of the United States Senate, that the Secretary of the Army is hereby requested to review the report of the Chief of Engineers on Fire Island to Montauk Point, New York, published as House Document Number 86-425, 86th Congress, 2nd session, and other pertinent reports, to determine whether modifications of the recommendations contained therein are advisable at the present time, with a view to preserving, restoring, and protecting Montauk Point and vicinity, including the historic Montauk Lighthouse and associated facilities, from erosion, environmental degradation, and coastal storm damage."

2. In addition to the study authority, Section 110 of the National Historic Preservation Act of 1966, as amended (NHPA), imposes a duty to maintain and preserve historic properties. At the present time, this duty is presently borne directly by the Montauk Point Historical Society, the current owners of the Montauk Point Lighthouse complex. However, thru the operation of a reversionary interest, as provided for in the land transfer (a quitclaim dated 18 September 1998 from the U.S. Coast Guard to the Montauk Point Historical Society), this duty ultimately falls on the Federal Government. Section 110 of the NHPA imposes duties only on federal agencies. As a federal agency, the Coast Guard was required to preserve and maintain the property in accordance with the NHPA. The transfer of the property from the Coast Guard to the Historical Society would have been an adverse impact on the property under Section 110 of the NHPA, because the historic property would have passed to an entity, the Historical Society, that was not a Federal agency and therefore not required to adhere to the NHPA, removing the legal protection the historic property enjoyed under federal ownership. To remedy this adverse impact, the Coast Guard included a condition in the transfer agreement that requires the Historic Society to preserve/maintain the property under the NHPA, effectively making the Historical Society act as a Federal agency with regards to the preservation of the property.

3. Alternative ways to follow Section 110 of the NHPA at Montauk Point therefore include:

- Provide mitigation for adverse impacts following a storm event that causes damage to the bluff and other features of the historic property, or

- Take steps now to protect the integrity and significance of the historic property, thereby avoiding the costs of Section 110 compliance that would have been triggered by storm damage.

- Through a combination of Section 110 of the NHPA and the nature of the land conveyance, there is indeed a statutory duty to perform the cultural resources mitigation at Montauk Point. If triggered by coastal storm damage such mitigation would incur a cost; therefore, avoiding that cost should, therefore be counted as a benefit.

4. If the Federal government is not mandated to follow Section 110 of the NHPA and the nature of the land conveyance, then the most likely future without-project scenario is that the bluff will erode and the historic Montauk Point Lighthouse complex will collapse. The economic analysis that follows below is based on this assumption.

5. The proxy used to place a depreciated replacement value of the historic Montauk Point Lighthouse complex is based on the calculations for the costs of cultural mitigation. Moving the Montauk Point Lighthouse complex, a National Register listed property, will potentially preserve the existing structures, but allow for the eventual destruction of the bluff point and buried cultural resources. These archaeological materials, which are associated with the historic and prehistoric use of the bluff, must be documented and recovered. Prior to moving the structures, each structure would need to be documented on engineering drawings and in photographs so that they can be rebuilt properly on the new site. Subsurface archeological excavations would be performed to recover artifacts both at the present lighthouse site and at the new site. Alternatively, all of these costs could be avoided by protecting the property from the storm damage.

Existing Conditions

6. The lighthouse complex and the surrounding Montauk Point State Park are valued Federal and State properties respectively. Montauk Point Lighthouse complex and the State Park annual attendance figures averaged 106,723 and 904,185 persons, respectively in the 1995-2002 period. The lighthouse complex does not have a parking lot, and visitors must use the state parking lot. The average attendance for the state park only is 797,462 (904,185-106,723). These figures were obtained from Montauk Point Lighthouse and Montauk State Park offices. Recent census data indicate that the populations for Long Island and New York's five boroughs have increased by 8.4% in ten years. The population for the surveyed area increased from 9,931,776 (1990 Census) to 10,762,191 (2000 Census). The economic analysis assumes the lighthouse and state park attendance will remain stable. Tables 1-3 show lighthouse admissions, parks admissions, and state population data.

TABLE 1. Lighthouse Attendance

Year	Adults	Seniors	Children	Group	Total
1995	90,664		12,998	2,634	106,296
1996	83,184	7,130	13,601	2,647	106,562
1997	78,562	8,916	1,401	2,884	91,763
1998	78,768	8,927	19,896	3,889	111,480
1999	77,079	9,199	19,997	4,397	110,672
2000	78,719	9,330	20,269	5,901	114,219
2001	66,818	8,352	18,720	5,969	99,859
2002	77,615	9,133	20,123	6,062	112,933
Total	631,409	60,987	127,005	34,383	853,784
Avg.	**78,926**	**8,712**	**15,876**	**4,298**	**106,723**

TABLE 2. Montauk State Park

Year	Attendance
1995	905,535
1996	849,165
1997	900,894
1998	916,680
1999	929,585
2000	916,460
2001	906,149
2002	909,010
Total	7,233,478
Avg.	**904,185**

TABLE 3. Population Data

County	1980	1990	2000	2004*	1990-2000 %Change
Nassau	1,321,582	1,287,348	1,334,544	1,339,461	3.7%
Suffolk	1,284,231	1,321,864	1,419,369	1,475,488	7.4%
Bronx	1,168,972	1,203,789	1,332,650	1,365,536	10.7%
Kings	2,231,028	2,300,664	2,465,326	2,475,290	7.2%
New York	1,428,285	1,487,536	1,537,195	1,562,723	3.3%
Queens	1,891,325	1,951,598	2,229,379	2,237,216	14.2%
Richmond	352,029	378,977	443,728	463,314	17.1%
Total	9,677,452	9,931,776	10,762,191	10,919,028	8.4%

*Source: US Census Bureau, 2004 population data are estimates

Without-Project Conditions

7. The Montauk Point Lighthouse complex sits on a high bluff underlain with glacial till, approximately 70-feet above Mean Sea Level (MSL). It is estimated that once the upper sections of the revetment that protects the bluff are displaced by a 15-year or greater storm event, the foundation soil underlying the displaced stone will become exposed and subject to subsequent erosion. To determine the extent of this erosion at the toe of the upper bluff above the damaged revetment that would cause significant bluff failure, a slope stability analysis was performed. The results of this analysis determined that for significant bluff failure, the damaged crest elevation of the revetment should degrade to approximately elevation +10 NGVD and the upper bluff toe at this +10 NGVD elevation recede horizontally approximately 10 ft. This is anticipated to cause approximately 26-30 ft. of loss of the bluff crest which will immediately threaten the lighthouse facility at the most critical area to the southeast of the lighthouse.

8. The period of time estimated for this condition to occur, subsequent to revetment failure, is an additional 10 years of long-term erosion at the upper bluff toe (at el. +10 NGVD). A decision tree analysis was applied to calculate the probability of revetment failure for any given year through the 50-year period of economic analysis due to a 15-year or greater storm event. When revetment failure occurs, the bluff crest will erode at an average rate of 3 feet per year. The lighthouse complex will be immediately threatened after 10 years, or 30 feet of erosion at the bluff crest.

<u>Proxy for Depreciated Replacement Value of Montauk Lighthouse Complex</u>

9. The proxy used to place an economic value of the historic Montauk Point Lighthouse complex is based on the hypothetical calculations for the costs of cultural mitigation of the site. The economic analysis assumes that cultural mitigation of the site will be initiated after the revetment that protects the bluff is displaced. The estimated cost for moving the Montauk Point Lighthouse complex and complete cultural mitigation of the complex is $20,192,000. This figure does not take into account the creation of raised grades landward of the present location of the lighthouse for the move, which could add an additional cost of $6,780,000. The raised grade would be necessary to maintain the lighthouse elevation because the existing bluff elevation decreases significantly as one move away from the shorefront. The overall mitigation process would take approximately six years to complete, with a total cost of $26,972,000 (Oct. 2004 price level), as shown in Table 4. The cost flows for years 1 through 6 were discounted (collapsed) to the first year that mitigation would occur. This was done to convert 6 years of expenditures into an equivalent expenditure that will occur in one year. Table 5 shows the calculations for the one-year equivalent value of the lighthouse complex if the upper section of the revetment is displaced in year 2006. Since this expenditure only happens when a 15-year or greater storm occurs, a decision tree analysis was applied to calculate the probability of occurrence throughout the 50-year period of analysis. For example, the probability for the expenditure to occur in year 0 (base year) is (1/15) = 0.067; year 1 (base year +1) is (14/15)*(1/15) = .062; and so forth up to the fiftieth year. The expected value (sum of the products of the probability of occurrences

multiplied by the one-year equivalent cultural mitigation cost) was then amortized using a 5-$^{3}/_{8}$ percent discount rate and a 50-year period of analysis to calculate the average annual mitigation cost at an October 2004 price level.[1]

10. Another scenario would have the cultural mitigation initiated after the revetment is displaced thereby, exposing the bluff to erosion, in year one and completed by year four. The actual moving of the lighthouse complex would be done in years eight and nine. This scenario would prevent any cultural artifacts from being lost or recorded after the revetment are displaced and allow for a three year lag in which the money for moving the lighthouse will not be needed. The conversion of the expenditure flows for this nine-year time period is shown in Table 6.

Table 4. Cultural Mitigation Costs of Lighthouse Complex[2]

Year	Tasks	Costs
1	Public Hearings	$ 60,000
	Phase 1 Preliminary Survey	$ 60,000
	Coordination	$ 60,000
2	Phase 3 Archaeological Survey	$ 1,000,000
	Coordination	$ 60,000
3	Archaeological Lab Work	$ 200,000
	HABS Work (various)	$ 600,000
	Coordination	$ 60,000
4	Report Write-up	$ 100,000
	Coordination	$ 60,000
	Public Hearings	$ 60,000
5	Site Preparation for moving	$ 6,720,000
	Coordination	$ 60,000
5	Moving Lighthouse	$ 8,876,000
	Coordination	$ 60,000
6	Moving Lighthouse	$ 8,876,000
	Coordination	$ 60,000
	Total	$ 26,972,000

[1] Using the long-term erosion rate of one foot per year at the upper section of the displaced revetment, by year 10, the upper bluff will be in danger of collapse. If a 15-year or greater event will occur in 2005, then 2015 is the estimated date of lighthouse failure. Cultural mitigation will begin in year 2009 because it takes six years to mitigate the project site.

[2] When the Cape Hatteras Lighthouse was moved in 1997, the Park Service had estimated the value of the lighthouse to be $20 million (1997 price level); personal correspondence, Paul Cloyd, P.E., Nation Park Service (2/13/2004). This figure becomes $25.4 million in 2004 price level (Civil Works Construction Cost Index). The District's value of $27 million for the Montauk Lighthouse complex compares similarly to the Cape Hatteras Lighthouse valuation.

Table 5. Montauk Point Lighthouse Complex - Calculation for one-year equivalent value (Oct. 2004 P.L., 5.375% discount rate)

Year		Present Value Factor	Mitigation Cost	Expected Value
2006	BY-4	1.2329639	$ 180,000	$ 221,933
2007	BY-3	1.1700725	$ 1,060,000	$ 1,240,277
2008	BY-2	1.1103891	$ 860,000	$ 954,935
2009	BY-1	1.0537500	$ 220,000	$ 231,825
2010	BY	1.0000000	$ 15,716,000	$ 15,716,000
2011	BY+1	0.9489917	$ 8,936,000	$ 8,480,190
	Total			$ 26,845,000

Table 6. Montauk Point Lighthouse Complex - Calculation for one-year equivalent value (Oct. 2004 discount rate)

CY		Present Value Factor	Mitigation Cost	Expected Value
2006	BY-4	1.2329639	$ 180,000	$ 221,933
2007	BY-3	1.1700725	$ 1,060,000	$ 1,240,277
2008	BY-2	1.1103891	$ 860,000	$ 954,935
2009	BY-1	1.0537500	$ 220,000	$ 231,825
2010	BY	1.0000000		
2011	BY+1	0.9489917		
2012	BY+2	0.9005852		
2013	BY+3	0.8546479	$ 15,716,000	$ 13,431,647
2014	BY+4	0.8110538	$ 8,936,000	$ 7,247,577
	Total			$ 23,328,000

11. The year 1994 was used to initiate the probability calculations for revetment failure because 1993 was the most recent occurrence of a 15-year or greater storm event. Tables 7 & 8 show the expected annual cultural mitigation costs that would be incurred in the without-project condition when the revetment fails and bluff erosion begins for the two mitigation scenarios. These calculations become the proxies for the depreciated replacement value of the Montauk Lighthouse complex.

Table 7. Proxy for Depreciated Replacement Value of Lighthouse Complex - without-project (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Lighthouse Complex in Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.3806404	0.6193596			
2008	15	0.3552644	0.6447356			
2009	16	0.3315801	0.6684199			
2010	17	0.3094747	0.6905253	$26,845,160	0.067	$1,059,795
2011	18	0.2888431	0.7111569	$25,475,834	0.062	$981,786
2012	19	0.2695869	0.7304131	$24,176,355	0.058	$905,221
2013	20	0.2516144	0.7483856	$22,943,160	0.054	$831,231
2014	21	0.2348401	0.7651599	$21,772,868	0.051	$760,591
2015	22	0.2191841	0.7808159	$20,662,271	0.047	$693,803
2016	23	0.2045718	0.7954282	$19,608,324	0.044	$631,158
2017	24	0.1909337	0.8090663	$18,608,136	0.041	$572,789
2018	25	0.1782048	0.8217952	$17,658,967	0.038	$518,705
2019	26	0.1663245	0.8336755	$16,758,213	0.036	$468,831
2020	27	0.1552362	0.8447638	$15,903,405	0.033	$423,027
2021	28	0.1448871	0.8551129	$15,092,199	0.031	$381,110
2022	29	0.1352280	0.8647720	$14,322,372	0.029	$342,869
2023	30	0.1262128	0.8737872	$13,591,812	0.027	$308,078
2024	31	0.1177986	0.8822014	$12,898,517	0.025	$276,502
2025	32	0.1099453	0.8900547	$12,240,585	0.024	$247,905
2026	33	0.1026157	0.8973843	$11,616,214	0.022	$222,056
2027	34	0.0957746	0.9042254	$11,023,690	0.021	$198,731
2028	35	0.0893896	0.9106104	$10,461,391	0.019	$177,717
2029	36	0.0834303	0.9165697	$9,927,773	0.018	$158,809
2030	37	0.0778683	0.9221317	$9,421,374	0.017	$141,820
2031	38	0.0726771	0.9273229	$8,940,806	0.016	$126,571
2032	39	0.0678319	0.9321681	$8,484,750	0.015	$112,899
2033	40	0.0633098	0.9366902	$8,051,958	0.014	$100,652
2034	41	0.0590892	0.9409108	$7,641,241	0.013	$89,691
2035	42	0.0551499	0.9448501	$7,251,474	0.012	$79,889
2036	43	0.0514732	0.9485268	$6,881,589	0.011	$71,129
2037	44	0.0480417	0.9519583	$6,530,571	0.010	$63,307
2038	45	0.0448389	0.9551611	$6,197,457	0.010	$56,325
2039	46	0.0418496	0.9581504	$5,881,335	0.009	$50,097
2040	47	0.0390597	0.9609403	$5,581,339	0.008	$44,545
2041	48	0.0364557	0.9635443	$5,296,644	0.008	$39,597
2042	49	0.0340253	0.9659747	$5,026,471	0.007	$35,190
2043	50	0.0317570	0.9682430	$4,770,079	0.007	$31,267
2044	51	0.0296398	0.9703602	$4,526,766	0.006	$27,774
2045	52	0.0276638	0.9723362	$4,295,863	0.006	$24,667
2046	53	0.0258196	0.9741804	$4,076,738	0.006	$21,903
2047	54	0.0240983	0.9759017	$3,868,791	0.005	$19,446
2048	55	0.0224917	0.9775083	$3,671,450	0.005	$17,261
2049	56	0.0209923	0.9790077	$3,484,176	0.005	$15,320
2050	57	0.0195928	0.9804072	$3,306,454	0.004	$13,595
2051	58	0.0182866	0.9817134	$3,137,797	0.004	$12,063
2052	59	0.0170675	0.9829325	$2,977,744	0.004	$10,702
2053	60	0.0159297	0.9840703	$2,825,854	0.003	$9,493
2054	61	0.0148677	0.9851323	$2,681,712	0.003	$8,421
2055	62	0.0138765	0.9861235	$2,544,922	0.003	$7,468
2056	63	0.0129514	0.9870486	$2,415,110	0.003	$6,623
2057	64	0.0120880	0.9879120	$2,291,920	0.003	$5,873
2058	65	0.0112821	0.9887179	$2,175,013	0.002	$5,208
2059	66	0.0105300	0.9894700	$1,412,045	0.002	$3,159
						$11,412,671
					Annual Damages	$661,714

Table 8. Proxy for Depreciated Replacement Value of Lighthouse Complex - without-project (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Lighthouse Complex in Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.3806404	0.6193596			
2008	15	0.3552644	0.6447356			
2009	16	0.3315801	0.6684199			
2010	17	0.3094747	0.6905253	$23,328,193	0.067	$920,952
2011	18	0.2888431	0.7111569	$22,138,262	0.062	$853,163
2012	19	0.2695869	0.7304131	$21,009,026	0.058	$786,629
2013	20	0.2516144	0.7483856	$19,937,392	0.054	$722,332
2014	21	0.2348401	0.7651599	$16,170,297	0.051	$564,876
2015	22	0.2191841	0.7808159	$15,345,477	0.047	$515,274
2016	23	0.2045718	0.7954282	$14,562,730	0.044	$468,749
2017	24	0.1909337	0.8090663	$13,819,910	0.041	$425,399
2018	25	0.1782048	0.8217952	$13,114,980	0.038	$385,232
2019	26	0.1663245	0.8336755	$12,446,007	0.036	$348,192
2020	27	0.1552362	0.8447638	$11,811,158	0.033	$314,174
2021	28	0.1448871	0.8551129	$11,208,690	0.031	$283,043
2022	29	0.1352280	0.8647720	$10,636,954	0.029	$254,643
2023	30	0.1262128	0.8737872	$10,094,381	0.027	$228,804
2024	31	0.1177986	0.8822014	$9,579,484	0.025	$205,353
2025	32	0.1099453	0.8900547	$9,090,851	0.024	$184,115
2026	33	0.1026157	0.8973843	$8,627,142	0.022	$164,917
2027	34	0.0957746	0.9042254	$8,187,086	0.021	$147,594
2028	35	0.0893896	0.9106104	$7,769,477	0.019	$131,987
2029	36	0.0834303	0.9165697	$7,337,126	0.018	$117,368
2030	37	0.0778683	0.9221317	$6,997,076	0.017	$105,327
2031	38	0.0726771	0.9273229	$6,640,167	0.016	$94,002
2032	39	0.0678319	0.9321681	$6,301,463	0.015	$83,848
2033	40	0.0633098	0.9366902	$5,980,036	0.014	$74,752
2034	41	0.0590892	0.9409108	$5,675,005	0.013	$66,612
2035	42	0.0551499	0.9448501	$5,385,532	0.012	$59,332
2036	43	0.0514732	0.9485268	$5,110,826	0.011	$52,826
2037	44	0.0480417	0.9519583	$4,850,131	0.010	$47,017
2038	45	0.0448389	0.9551611	$4,602,734	0.010	$41,831
2039	46	0.0418496	0.9581504	$4,367,956	0.009	$37,206
2040	47	0.0390597	0.9609403	$4,145,154	0.008	$33,083
2041	48	0.0364557	0.9635443	$3,933,717	0.008	$29,408
2042	49	0.0340253	0.9659747	$3,733,065	0.007	$26,135
2043	50	0.0317570	0.9682430	$3,542,648	0.007	$23,221
2044	51	0.0296398	0.9703602	$3,361,943	0.006	$20,627
2045	52	0.0276638	0.9723362	$3,190,456	0.006	$18,320
2046	53	0.0258196	0.9741804	$3,027,716	0.006	$16,267
2047	54	0.0240983	0.9759017	$2,873,278	0.005	$14,442
2048	55	0.0224917	0.9775083	$2,726,717	0.005	$12,820
2049	56	0.0209923	0.9790077	$2,587,631	0.005	$11,378
2050	57	0.0195928	0.9804072	$2,455,641	0.004	$10,097
2051	58	0.0182866	0.9817134	$2,330,383	0.004	$8,959
2052	59	0.0170675	0.9829325	$2,211,514	0.004	$7,948
2053	60	0.0159297	0.9840703	$2,098,708	0.003	$7,051
2054	61	0.0148677	0.9851323	$1,991,657	0.003	$6,254
2055	62	0.0138765	0.9861235	$1,890,066	0.003	$5,547
2056	63	0.0129514	0.9870486	$1,236,405	0.003	$3,391
2057	64	0.0120880	0.9879120	$193,285	0.003	$495
2058	65	0.0112821	0.9887179	$183,426	0.002	$439
2059	66	0.0105300	0.9894700	$174,069	0.002	$389
						$8,941,820
					Annual Damages	$518,452

12. Of these two proxies for the depreciated value of the lighthouse complex, the economic analysis that follows will use the proxy least favorable to project justification, $518,500, for further analysis to determine if there is a viable solution to protecting Montauk Point and its vicinity.

Local Costs Forgone

13. The lighthouse complex is situated on 3 acres of land, specifically a bluff that has an appraised value of $12 million. It is estimated that the top of the bluff will erode at a rate of 3 feet per year when the revetment fails. Because of the complexity of actually replacing the bluff surface, a prorated amount of the appraised value of land lost was used as a proxy for the local costs forgone for this loss in the without-project condition. The local costs forgone for this land value due to long-term erosion is calculated to be $82,600 per year. The average annual local costs forgone are $74,100 as shown in Table 9. The two numbers differ because the average annual costs take into account the probability that revetment failure will not occur immediately.

Table 9. Local Costs Forgone (Oct. 2004 P.L., 5.375% discount rate)

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value Factor		
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.3806404	0.6193596			
2008	15	0.3552644	0.6447356			
2009	16	0.3315801	0.6684199			
2010	17	0.3094747	0.6905253	1.0000000	$82,600	$57,037
2011	18	0.2888431	0.7111569	0.9489917	$82,600	$55,745
2012	19	0.2695869	0.7304131	0.9005852	$82,600	$54,334
2013	20	0.2516144	0.7483856	0.8546479	$82,600	$52,831
2014	21	0.2348401	0.7651599	0.8110538	$82,600	$51,260
2015	22	0.2191841	0.7808159	0.7696833	$82,600	$49,641
2016	23	0.2045718	0.7954282	0.7304231	$82,600	$47,991
2017	24	0.1909337	0.8090663	0.6931654	$82,600	$46,323
2018	25	0.1782048	0.8217952	0.6578082	$82,600	$44,652
2019	26	0.1663245	0.8336755	0.6242545	$82,600	$42,987
2020	27	0.1552362	0.8447638	0.5924124	$82,600	$41,337
2021	28	0.1448871	0.8551129	0.5621944	$82,600	$39,709
2022	29	0.1352280	0.8647720	0.5335178	$82,600	$38,109
2023	30	0.1262128	0.8737872	0.5063040	$82,600	$36,542
2024	31	0.1177986	0.8822014	0.4804783	$82,600	$35,012
2025	32	0.1099453	0.8900547	0.4559699	$82,600	$33,522
2026	33	0.1026157	0.8973843	0.4327117	$82,600	$32,074
2027	34	0.0957746	0.9042254	0.4106398	$82,600	$30,670
2028	35	0.0893896	0.9106104	0.3896937	$82,600	$29,311
2029	36	0.0834303	0.9165697	0.3698161	$82,600	$27,998
2030	37	0.0778683	0.9221317	0.3509524	$82,600	$26,731
2031	38	0.0726771	0.9273229	0.3330509	$82,600	$25,511
2032	39	0.0678319	0.9321681	0.3160626	$82,600	$24,336
2033	40	0.0633098	0.9366902	0.2999408	$82,600	$23,207
2034	41	0.0590892	0.9409108	0.2846413	$82,600	$22,122
2035	42	0.0551499	0.9448501	0.2701222	$82,600	$21,082
2036	43	0.0514732	0.9485268	0.2563437	$82,600	$20,084
2037	44	0.0480417	0.9519583	0.2432681	$82,600	$19,129
2038	45	0.0448389	0.9551611	0.2308594	$82,600	$18,214
2039	46	0.0418496	0.9581504	0.2190836	$82,600	$17,339
2040	47	0.0390597	0.9609403	0.2079086	$82,600	$16,502
2041	48	0.0364557	0.9635443	0.1973035	$82,600	$15,703
2042	49	0.0340253	0.9659747	0.1872394	$82,600	$14,940
2043	50	0.0317570	0.9682430	0.1776886	$82,600	$14,211
2044	51	0.0296398	0.9703602	0.1686250	$82,600	$13,516
2045	52	0.0276638	0.9723362	0.1600237	$82,600	$12,852
2046	53	0.0258196	0.9741804	0.1518612	$82,600	$12,220
2047	54	0.0240983	0.9759017	0.1441150	$82,600	$11,617
2048	55	0.0224917	0.9775083	0.1367640	$82,600	$11,043
2049	56	0.0209923	0.9790077	0.1297879	$82,600	$10,495
2050	57	0.0195928	0.9804072	0.1231676	$82,600	$9,974
2051	58	0.0182866	0.9817134	0.1168850	$82,600	$9,478
2052	59	0.0170675	0.9829325	0.1109229	$82,600	$9,006
2053	60	0.0159297	0.9840703	0.1052649	$82,600	$8,556
2054	61	0.0148677	0.9851323	0.0998956	$82,600	$8,129
2055	62	0.0138765	0.9861235	0.0948000	$82,600	$7,722
2056	63	0.0129514	0.9870486	0.0899645	$82,600	$7,335
2057	64	0.0120880	0.9879120	0.0853755	$82,600	$6,967
2058	65	0.0112821	0.9887179	0.0810207	$82,600	$6,617
2059	66	0.0105300	0.9894700	0.0768879	$82,600	$6,284
						$1,278,010
					Annual Damages	$74,100

Recreation Loss

14. Another without-project consequence of storm damage to the bluff would be loss visitations to the lighthouse. Visitation losses associated with the lighthouse's closure were assessed using the Travel Cost Estimate of Willingness to Pay. The lighthouse has a log in which visitors indicate the places where they are traveling from to visit. A recent sample from the log was used to estimate the round-trip distance from each origin. The values of losses are the costs in cents per mile to operate an automobile, plus the opportunity costs of time spent in travel and on site. Surveys were conducted to determine the number of visitors that make the trip to Montauk, NY exclusively to visit the lighthouse. Based on the survey, 47% of the people sampled indicated that visiting the Montauk Lighthouse complex was the reason they drove to Montauk, New York. The remaining 53% of the people indicated that visiting the Montauk Lighthouse complex was part of their itinerary on their visit to Long Island, New York. The travel costs attributed to this category were prorated at 25% of their total travel costs.

15. A rate of $0.378 per mile[3] was used for calculating the operating costs per car, as shown in Table 10. Costs per person were calculated using state park figures of 3.5 persons per car. The opportunity cost of time is 1/3 and 1/12 the average wage rate for adults and children, respectively. The hourly wage rate is $18.11[4]. The estimated car driving speed is 40 mph. Tables 11 and 12 show the calculations for the Travel Cost Method. As a result, $3,040,200 in annual visitation losses has been projected for all visitors to the Montauk Point Lighthouse complex including admissions fees.

Table 10. Variable Driving Costs for 2004

Categories	**Per mile**
gas and oil	$ 0.065
maintenance	$ 0.054
tires	$ 0.007
Subtotal cost per mile	$ 0.126
	Per year
depreciation	$ 3,782.00
(15,000 miles annually)	
	Per mile
depreciation	$ 0.252
Total variable costs per mile	**$ 0.378**

[3] U.S. Department of Transportation.
[4] The estimated average payroll tax rate for the region is 30%. The current hourly wage rate is $25.46 (US Dept. of Labor, April 2004) multiplied by the CPI factor to bring the price level to October 2004 (207.3/204.0). The after-tax hourly wage rate is 0.7 x $25.46(207.3/204.0) = $18.11.

Table 11. Montauk Point Lighthouse Travel Cost Method

Per Capita Income	Oct-04	Adult time cost/hr		Child time cost/hr	Annual Admission Fees
NY&NJ metropolitan area	$18.11	$6.04		$1.51	$482,121
Cost per mile	0.378	Avg. time spent			
Round Trip Factor	2	at lighthouse	1 hour		
People per car	3.5	No. Adults per year	88851		
Avg. driving speed	40	No. Children per year	17872		

Residence	No. of people sampled	Multiply Factor	No. of Adults	No. of Children	Miles to Montauk	Travel Cost Per Car	Car Travel Cost per Person	Total Car Travel Cost	Travel time cost per adult	Travel time cost per child	Total travel time cost	Total time cost spent at lighthouse
E. Hampton	40	0.02247191	1997	402	16	$12.10	$3.46	8,288	$4.83	$1.21	$10,128	$12,659
So. Hampton(1)	6	0.003370787	299	60	31	$23.44	$6.70	2,409	$9.36	$2.34	$2,943	$1,899
So. Hampton(2)	7	0.003932584	349	70	45	$34.02	$9.72	4,079	$13.58	$3.40	$4,985	$2,215
Southhold	11	0.006179775	549	110	42	$31.75	$9.07	5,983	$12.68	$3.17	$7,311	$3,481
Riverhead	10	0.005617978	499	100	48	$36.29	$10.37	6,216	$14.49	$3.62	$7,596	$3,165
Brookhaven(1)	73	0.041011236	3644	733	61	$46.12	$13.18	57,669	$18.41	$4.60	$70,466	$23,103
Brookhaven(2)	74	0.041573034	3694	743	67	$50.65	$14.47	64,209	$20.22	$5.06	$78,457	$23,420
Islip	100	0.056179775	4992	1004	74	$55.94	$15.98	95,835	$22.34	$5.58	$117,100	$31,649
Smithtown	16	0.008988764	799	161	76	$57.46	$16.42	15,748	$22.94	$5.73	$19,242	$5,064
Babylon	83	0.046629213	4143	833	83	$62.75	$17.93	89,217	$25.05	$6.26	$109,014	$26,268
Huntington	48	0.026966292	2396	482	88	$66.53	$19.01	54,704	$26.56	$6.64	$66,842	$15,191
Oyster Bay	21	0.011797753	1048	211	95	$71.82	$20.52	25,837	$28.67	$7.17	$31,569	$6,646
So. Oyster Bay	21	0.011797753	1048	211	90	$68.04	$19.44	24,477	$27.17	$6.79	$29,908	$6,646
Hempstead	143	0.080337079	7138	1436	100	$75.60	$21.60	185,194	$30.18	$7.55	$226,287	$45,257
No. Hempstead	19	0.010674157	948	191	103	$77.87	$22.25	25,344	$31.09	$7.77	$30,968	$6,013
Queens	99	0.055617978	4942	994	115	$86.94	$24.84	147,443	$34.71	$8.68	$180,160	$31,332
Brooklyn	40	0.02247191	1997	402	115	$86.94	$24.84	59,573	$34.71	$8.68	$72,792	$12,659
Manhattan	106	0.059550562	5291	1064	116	$87.70	$25.06	159,241	$35.01	$8.75	$194,576	$33,548
Bronx	24	0.013483146	1198	241	120	$90.72	$25.92	37,298	$36.22	$9.06	$45,574	$7,596
Staten Island	12	0.006741573	599	120	120	$90.72	$25.92	18,649	$36.22	$9.06	$22,787	$3,798
Others	827	0.464606742	41281	8303	20	$15.12	$4.32	214,204	$6.04	$1.51	$261,734	$261,734
Total	1780	1	88851	17872				1,301,619			$1,590,437	$563,345
Prorated Travel Cost								$897,749			$1,096,953	$563,345

Table 12. Summary of Travel Cost Method (Oct. 2004 P.L.)		
Prorated Car Travel Cost	$	897,749
Prorated Travel Time Cost	$	1,096,953
Time Spent at Lighthouse Cost	$	563,345
Admissions Cost	$	482,121
Total	$	3,040,167

16. Lighthouse visitations will be lost when the existing revetment is damaged by a 15-year or greater storm event, followed by 10 years of erosion to the bluff. If the revetment is damaged in year 2005, the lighthouse visitations will be lost starting in year 2015. Since the base year is 2009, the lighthouse visitations will be lost from 2015 through 2058. The $3,040,200 per year of lighthouse visitations from 2015 through 2058 is discounted to the first year that visitations are lost, year 2015. This was done to convert 44 years of lost visitations into a one-year equivalent loss that will occur in 2015. Similar calculations converted the lost visitations into one-year equivalents losses that will occur in years 2016 through 2058. These results are shown in Table 13. The average annual lighthouse visitations are calculated to be $882,700 as shown in Table 14.

Table 13. Montauk Point Lighthouse Visitations – Calculation for one-year equivalent value in year n (Oct. 2004 P.L., 5.375% discount rate)

Year	Present Value Factor	Lighthouse Visitations in year n	Lighthouse Visitations Present Value	Lighthouse Visitations 1-yr equivalent value in year n
2010	1			
2011	0.948991696			
2012	0.90058524			
2013	0.854647914			
2014	0.811053774			
2015	0.769683297			
2016	0.730423057	$3,040,167	$2,220,608	$39,185,369
2017	0.693165416	$3,040,167	$2,107,339	$36,964,761
2018	0.657808224	$3,040,167	$1,999,847	$34,857,422
2019	0.624254543	$3,040,167	$1,897,838	$32,857,576
2020	0.592412377	$3,040,167	$1,801,033	$30,959,738
2021	0.562194427	$3,040,167	$1,709,165	$29,158,705
2022	0.533517843	$3,040,167	$1,621,983	$27,449,540
2023	0.506304003	$3,040,167	$1,539,249	$25,827,557
2024	0.480478294	$3,040,167	$1,460,734	$24,288,308
2025	0.455969912	$3,040,167	$1,386,225	$22,827,574
2026	0.43271166	$3,040,167	$1,315,516	$21,441,349
2027	0.410639772	$3,040,167	$1,248,413	$20,125,833
2028	0.389693734	$3,040,167	$1,184,734	$18,877,420
2029	0.369816118	$3,040,167	$1,124,303	$17,692,686
2030	0.350952425	$3,040,167	$1,066,954	$16,568,383
2031	0.333050937	$3,040,167	$1,012,530	$15,501,429
2032	0.316062574	$3,040,167	$960,883	$14,488,899
2033	0.299940758	$3,040,167	$911,870	$13,528,016
2034	0.284641289	$3,040,167	$865,357	$12,616,146
2035	0.270122219	$3,040,167	$821,217	$11,750,789
2036	0.256343743	$3,040,167	$779,328	$10,929,572
2037	0.243268084	$3,040,167	$739,576	$10,150,244
2038	0.230859391	$3,040,167	$701,851	$9,410,669
2039	0.219083645	$3,040,167	$666,051	$8,708,817
2040	0.20790856	$3,040,167	$632,077	$8,042,767
2041	0.197303497	$3,040,167	$599,836	$7,410,690
2042	0.187239381	$3,040,167	$569,239	$6,810,854
2043	0.177688617	$3,040,167	$540,203	$6,241,615
2044	0.168625022	$3,040,167	$512,648	$5,701,412
2045	0.160023746	$3,040,167	$486,499	$5,188,764
2046	0.151861206	$3,040,167	$461,683	$4,702,265
2047	0.144115024	$3,040,167	$438,134	$4,240,582
2048	0.136763961	$3,040,167	$415,785	$3,802,448
2049	0.129787863	$3,040,167	$394,577	$3,386,663
2050	0.123167604	$3,040,167	$374,450	$2,992,086
2051	0.116885034	$3,040,167	$355,350	$2,617,636
2052	0.110922927	$3,040,167	$337,224	$2,262,286
2053	0.105264936	$3,040,167	$320,023	$1,925,061
2054	0.09989555	$3,040,167	$303,699	$1,605,038
2055	0.094800048	$3,040,167	$288,208	$1,301,339
2056	0.089964458	$3,040,167	$273,507	$1,013,131
2057	0.085375524	$3,040,167	$259,556	$739,624
2058	0.081020663	$3,040,167	$246,316	$480,069
2059	0.076887937	$3,040,167	$233,752	$233,752

Table 14. Lighthouse Visitations Damages - without-project

(Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.3806404	0.6193596			
2008	15	0.3552644	0.6447356			
2009	16	0.3315801	0.6684199			
2010	17	0.3094747	0.6905253			
2011	18	0.2888431	0.7111569			
2012	19	0.2695869	0.7304131			
2013	20	0.2516144	0.7483856			
2014	21	0.2348401	0.7651599			
2015	22	0.2191841	0.7808159			
2016	23	0.2045718	0.7954282	$39,185,369	0.067	$1,546,963
2017	24	0.1909337	0.8090663	$36,964,761	0.062	$1,424,545
2018	25	0.1782048	0.8217952	$34,857,422	0.058	$1,305,146
2019	26	0.1663245	0.8336755	$32,857,576	0.054	$1,190,430
2020	27	0.1552362	0.8447638	$30,959,738	0.051	$1,081,515
2021	28	0.1448871	0.8551129	$29,158,705	0.047	$979,098
2022	29	0.1352280	0.8647720	$27,449,540	0.044	$883,554
2023	30	0.1262128	0.8737872	$25,827,557	0.041	$795,014
2024	31	0.1177986	0.8822014	$24,288,308	0.038	$713,432
2025	32	0.1099453	0.8900547	$22,827,574	0.036	$638,628
2026	33	0.1026157	0.8973843	$21,441,349	0.033	$570,334
2027	34	0.0957746	0.9042254	$20,125,833	0.031	$508,219
2028	35	0.0893896	0.9106104	$18,877,420	0.029	$451,915
2029	36	0.0834303	0.9165697	$17,692,686	0.027	$401,031
2030	37	0.0778683	0.9221317	$16,568,383	0.025	$355,172
2031	38	0.0726771	0.9273229	$15,501,429	0.024	$313,946
2032	39	0.0678319	0.9321681	$14,488,899	0.022	$276,971
2033	40	0.0633098	0.9366902	$13,528,016	0.021	$243,879
2034	41	0.0590892	0.9409108	$12,616,146	0.019	$214,321
2035	42	0.0551499	0.9448501	$11,750,789	0.018	$187,971
2036	43	0.0514732	0.9485268	$10,929,572	0.017	$164,523
2037	44	0.0480417	0.9519583	$10,150,244	0.016	$143,693
2038	45	0.0448389	0.9551611	$9,410,669	0.015	$125,219
2039	46	0.0418496	0.9581504	$8,708,817	0.014	$108,863
2040	47	0.0390597	0.9609403	$8,042,767	0.013	$94,404
2041	48	0.0364557	0.9635443	$7,410,690	0.012	$81,643
2042	49	0.0340253	0.9659747	$6,810,854	0.011	$70,398
2043	50	0.0317570	0.9682430	$6,241,615	0.010	$60,505
2044	51	0.0296398	0.9703602	$5,701,412	0.010	$51,817
2045	52	0.0276638	0.9723362	$5,188,764	0.009	$44,198
2046	53	0.0258196	0.9741804	$4,702,265	0.008	$37,529
2047	54	0.0240983	0.9759017	$4,240,582	0.008	$31,702
2048	55	0.0224917	0.9775083	$3,802,448	0.007	$26,621
2049	56	0.0209923	0.9790077	$3,386,663	0.007	$22,199
2050	57	0.0195928	0.9804072	$2,992,086	0.006	$18,358
2051	58	0.0182866	0.9817134	$2,617,636	0.006	$15,031
2052	59	0.0170675	0.9829325	$2,262,286	0.006	$12,155
2053	60	0.0159297	0.9840703	$1,925,061	0.005	$9,676
2054	61	0.0148677	0.9851323	$1,605,038	0.005	$7,546
2055	62	0.0138765	0.9861235	$1,301,339	0.005	$5,722
2056	63	0.0129514	0.9870486	$1,013,131	0.004	$4,166
2057	64	0.0120880	0.9879120	$739,624	0.004	$2,843
2058	65	0.0112821	0.9887179	$480,069	0.004	$1,725
2059	66	0.0105300	0.9894700	$233,752	0.003	$785
						$15,223,407
					Annual Damages	$882,662

17. The Montauk Point Lighthouse complex resides within the Montauk Point State Park. The Montauk Point Lighthouse complex offers a unique experience that is not found elsewhere in the New York metropolitan area. Part of the state park experience is its connection with the lighthouse complex. There will be a reduction to the overall aesthetics and recreational value of the state park visitations if the lighthouse complex did not exist. Per ER1105-2-100, Planning Guidance Notebook, the Unit Day Value method was used to assign visitation values to the state park for the with-project and without-project conditions. It is estimated that the current value for the recreational experience is $6.86. Without the lighthouse complex, the recreational experience is reduced to an estimate of $5.95. The annual benefits lost from state park visitations experience are $682,500 based on 750,000 visitations[5]. Table 15 shows the calculations for the state park recreation values based on Unit Day Value calculations. The average annual reduced state park usage values will be incurred when the existing revetment is damaged by a 15-year or greater storm event, and after 10 years of long-term erosion have occurred to the bluff. Tables 16 shows the one-year equivalent reduced state park visitation usages for years 2015 through 2058 and Table17 shows calculations for the average annual reduced state park recreational experience to be $198,200.

Table 15. State Park Visitations, Unit Day Values

	Without-Project	With-Project
Recreation Experience	10	15
Availability of opportunity	6	14
Carrying capacity	6	6
Accessibility	10	10
Environmental	10	10
Total	42	55
Unit Day Value	$5.95	$6.86

[5] Unit Day Value was used due to study cost considerations. The difference in state park usage value is $0.91 per visit. 750,000 visitations x $0.91 = $682,500 (Oct. 2004 P.L.). Although the actual visitations to the State Park are 797,462, the method of using Unit Day Value to evaluate recreation usage imposes a visitation cap of 750,000 persons.

Table 16. Montauk Point State Park Visitations - Calculation for one-year equivalent value in year n (Oct. 2004 P.L., 5.375% discount rate)

Year	Present Value Factor	State Park Visitations in year n	State Park Visitations Present Value	State Park Visitations 1-yr equivalent value in year n
2010	1			
2011	0.948991696			
2012	0.90058524			
2013	0.854647914			
2014	0.811053774			
2015	0.769683297			
2016	0.730423057	$682,500	$498,514	$8,796,890
2017	0.693165416	$682,500	$473,085	$8,298,376
2018	0.657808224	$682,500	$448,954	$7,825,291
2019	0.624254543	$682,500	$426,054	$7,376,337
2020	0.592412377	$682,500	$404,321	$6,950,283
2021	0.562194427	$682,500	$383,698	$6,545,962
2022	0.533517843	$682,500	$364,126	$6,162,264
2023	0.506304003	$682,500	$345,552	$5,798,138
2024	0.480478294	$682,500	$327,926	$5,452,585
2025	0.455969912	$682,500	$311,199	$5,124,659
2026	0.43271166	$682,500	$295,326	$4,813,459
2027	0.410639772	$682,500	$280,262	$4,518,134
2028	0.389693734	$682,500	$265,966	$4,237,872
2029	0.369816118	$682,500	$252,400	$3,971,906
2030	0.350952425	$682,500	$239,525	$3,719,507
2031	0.333050937	$682,500	$227,307	$3,479,982
2032	0.316062574	$682,500	$215,713	$3,252,674
2033	0.299940758	$682,500	$204,710	$3,036,962
2034	0.284641289	$682,500	$194,268	$2,832,252
2035	0.270122219	$682,500	$184,358	$2,637,984
2036	0.256343743	$682,500	$174,955	$2,453,626
2037	0.243268084	$682,500	$166,030	$2,278,671
2038	0.230859391	$682,500	$157,562	$2,112,641
2039	0.219083645	$682,500	$149,525	$1,955,079
2040	0.20790856	$682,500	$141,898	$1,805,555
2041	0.197303497	$682,500	$134,660	$1,663,657
2042	0.187239381	$682,500	$127,791	$1,528,998
2043	0.177688617	$682,500	$121,272	$1,401,207
2044	0.168625022	$682,500	$115,087	$1,279,934
2045	0.160023746	$682,500	$109,216	$1,164,848
2046	0.151861206	$682,500	$103,645	$1,055,631
2047	0.144115024	$682,500	$98,359	$951,986
2048	0.136763961	$682,500	$93,341	$853,628
2049	0.129787863	$682,500	$88,580	$760,286
2050	0.123167604	$682,500	$84,062	$671,706
2051	0.116885034	$682,500	$79,774	$587,644
2052	0.110922927	$682,500	$75,705	$507,870
2053	0.105264936	$682,500	$71,843	$432,165
2054	0.09989555	$682,500	$68,179	$360,322
2055	0.094800048	$682,500	$64,701	$292,143
2056	0.089964458	$682,500	$61,401	$227,442
2057	0.085375524	$682,500	$58,269	$166,041
2058	0.081020663	$682,500	$55,297	$107,773
2059	0.076887937	$682,500	$52,476	$52,476

Table 17. Park Visitation Damages - without-project design

(Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.3806404	0.6193596			
2008	15	0.3552644	0.6447356			
2009	16	0.3315801	0.6684199			
2010	17	0.3094747	0.6905253			
2011	18	0.2888431	0.7111569			
2012	19	0.2695869	0.7304131			
2013	20	0.2516144	0.7483856			
2014	21	0.2348401	0.7651599			
2015	22	0.2191841	0.7808159			
2016	23	0.2045718	0.7954282	$8,796,890	0.067	$347,284
2017	24	0.1909337	0.8090663	$8,298,376	0.062	$319,802
2018	25	0.1782048	0.8217952	$7,825,291	0.058	$292,998
2019	26	0.1663245	0.8336755	$7,376,337	0.054	$267,245
2020	27	0.1552362	0.8447638	$6,950,283	0.051	$242,794
2021	28	0.1448871	0.8551129	$6,545,962	0.047	$219,802
2022	29	0.1352280	0.8647720	$6,162,264	0.044	$198,353
2023	30	0.1262128	0.8737872	$5,798,138	0.041	$178,476
2024	31	0.1177986	0.8822014	$5,452,585	0.038	$160,161
2025	32	0.1099453	0.8900547	$5,124,659	0.036	$143,368
2026	33	0.1026157	0.8973843	$4,813,459	0.033	$128,037
2027	34	0.0957746	0.9042254	$4,518,134	0.031	$114,092
2028	35	0.0893896	0.9106104	$4,237,872	0.029	$101,452
2029	36	0.0834303	0.9165697	$3,971,906	0.027	$90,029
2030	37	0.0778683	0.9221317	$3,719,507	0.025	$79,734
2031	38	0.0726771	0.9273229	$3,479,982	0.024	$70,479
2032	39	0.0678319	0.9321681	$3,252,674	0.022	$62,178
2033	40	0.0633098	0.9366902	$3,036,962	0.021	$54,749
2034	41	0.0590892	0.9409108	$2,832,252	0.019	$48,114
2035	42	0.0551499	0.9448501	$2,637,984	0.018	$42,198
2036	43	0.0514732	0.9485268	$2,453,626	0.017	$36,934
2037	44	0.0480417	0.9519583	$2,278,671	0.016	$32,258
2038	45	0.0448389	0.9551611	$2,112,641	0.015	$28,111
2039	46	0.0418496	0.9581504	$1,955,079	0.014	$24,439
2040	47	0.0390597	0.9609403	$1,805,555	0.013	$21,193
2041	48	0.0364557	0.9635443	$1,663,657	0.012	$18,328
2042	49	0.0340253	0.9659747	$1,528,998	0.011	$15,804
2043	50	0.0317570	0.9682430	$1,401,207	0.010	$13,583
2044	51	0.0296398	0.9703602	$1,279,934	0.010	$11,633
2045	52	0.0276638	0.9723362	$1,164,848	0.009	$9,922
2046	53	0.0258196	0.9741804	$1,055,631	0.008	$8,425
2047	54	0.0240983	0.9759017	$951,986	0.008	$7,117
2048	55	0.0224917	0.9775083	$853,628	0.007	$5,976
2049	56	0.0209923	0.9790077	$760,286	0.007	$4,983
2050	57	0.0195928	0.9804072	$671,706	0.006	$4,121
2051	58	0.0182866	0.9817134	$587,644	0.006	$3,374
2052	59	0.0170675	0.9829325	$507,870	0.006	$2,729
2053	60	0.0159297	0.9840703	$432,165	0.005	$2,172
2054	61	0.0148677	0.9851323	$360,322	0.005	$1,694
2055	62	0.0138765	0.9861235	$292,143	0.005	$1,285
2056	63	0.0129514	0.9870486	$227,442	0.004	$935
2057	64	0.0120880	0.9879120	$166,041	0.004	$638
2058	65	0.0112821	0.9887179	$107,773	0.004	$387
2059	66	0.0105300	0.9894700	$52,476	0.003	$176
						$3,417,567
					Annual Damages	$198,153

With-Project Conditions

Preliminary Alternatives

18. Preliminary alternative approaches need to be considered in order to develop the most appropriate form of shoreline stabilization for the area. Criteria for evaluating preliminary alternatives will include appropriateness to site conditions, compliance with New York State Coastal Zone Management criteria, effectiveness of protection, impacts on environmental and cultural resources, and costs (including interest during construction and maintenance). Alternatives that are feasible approaches to storm protection and shoreline stabilization need to address both present and future needs. The present need is to eliminate the threat of erosion and to provide acceptable levels of protection from the impacts of wave attack and storm recession. Preliminary cost estimates are included so that the most cost effective and efficient solutions, considering coastal processes impacts, can be selected for detailed design and economic optimization.

19. The initial screening of hurricane storm damage reduction measures resulted in the following alternatives:

- Alternative 1 - No Action. While the no action plan fails to meet objectives and needs of the project area, it does provide the basis against which project benefits are measured.
- Alternative 2 – Stone Revetment. Revetments are a proven method of shore protection in this area and have a record of acceptance by state and local authorities. Revetment alternatives such as this can utilize much of the stone already on site in the existing structure, thus making good use of existing resources.
- Alternative 3 – Offshore Breakwater with Beach Fill. Breakwaters will be difficult to construct due to difficult site access and in-water construction. Tidal currents are significant and breaking waves arrive from almost all onshore directions. The breakwater requires very large stone and a substantial width and elevation to be effective. The gaps between the breakwaters may induce significant currents that could increase scour to the bottom, potentially compromising the foundation of the breakwaters sometime in the future.
- Alternative 4 – T-groins with Beach Fill. T-groins will be difficult to construct due to difficult site access, however, land-based equipment can be utilized. Tidal currents are significant and breaking waves arrive from almost all onshore directions. The shore-parallel structures would require very large stone and a substantial width and elevation to be effective. The gaps between the shore-parallel structures may induce significant currents that could scour the bottom, potentially compromising the foundation of the t-groins sometime in the future.
- Alternative 5 – Beach Fill. This alternative is considered not feasible for many reasons. High longshore transport rates will remove the fill rapidly at an unpredictable rate and the area will require constant renourishment. A berm at +11 feet NGVD will provide some short-term reduction in the recession of the toe

of the bluff, but will not impede higher water levels and waves from impacting the bluff face and therefore will not provide adequate storm damage protection. Seasonal beach surveys (potentially monthly) will be required during the first two to three years after construction to refine the design of the beach fill cross section and to estimate the renourishment requirements. Because of the lack of adequate storm damage protection, this beach fill alternative will not be considered further.

- Alternative 6 – Relocation of the Lighthouse Complex. The moving of the lighthouse itself is a precarious task at best. Unlike the Cape Hatteras Lighthouse (which rested on a relatively flat, level surface that permitted the National Park Service to move the structure for a cost of approximately $12 Million) the Montauk Point Lighthouse rests upon a hill on top of the bluff. Raised grades would have to be built to raise the level of the ground to the west of the bluff up to the lighthouse grade to insure a stable move. The relocation of the Montauk Point Lighthouse will have an adverse effect on the above and below ground resources. Moving the Lighthouse would have an adverse impact on the archaeological resources and compromise the integrity of the lighthouse and associated structures. Environmental degradation of habitats and historic views would continue. This alternative will not be considered further.

<u>Comparison of Alternatives</u>

20. Based on the advantages and disadvantages of each of the alternatives discussed, alternatives 2, 3, and 4 were carried forth for further analysis. Alternatives 2 through 4 are developed at the same storm design for plan comparison. They are designed to withstand a 73-year return period storm. This level of design is commensurate with a project evaluation over a 50-year period, because over 50 years there would be a 50% risk of a 73-year or greater storm event. The benefits claimed are the same because each of the alternatives protects the same structures and land to the same degree, and each alternative prevents the same average annual project damages. The estimated average annual costs were calculated and compared to the average annual benefits. Alternative 3 is the plan that maximizes the net benefits, and therefore will be selected for plan optimization (See Table 18).

Table 18. Comparison of Alternatives

	Alternative 2 Stone Revetment	Alternative 3 Offshore Breakwater and Beach Fill	Alternative 4 T-Groins and Beach Fill
Total Annual Costs	$971,000	$1,443,000	$1,287,000
Total Annual Benefits	$1,578,700	$1,578,700	$1,578,700
Total Net Benefits	$607,700	$135,700	$291,700

Optimization of Selected Plan

21. Preliminary screening of various alternatives identified that the Stone Revetment Plan is the most feasible alternative both economically and environmentally in providing protection to Montauk Point and its vicinity. Three storm design levels were considered, the 15-year, 73-year, and 150-year alternatives, to determine the optimal plan. The three alternatives provide protection to the Montauk Point Lighthouse complex until storm exceedance starts to displace the armor stones at the upper portion of the stone revetment for each storm protection design. Residual damages were calculated for the three alternatives and used for plan evaluation. Table 19 shows the three design alternatives and their associated storm exceedance levels that would cause the upper part of the stone revetment to be displaced, thereby exposing the bluff to erosion.

Table 19. Storm Exceedance for Stone Revetment Alternatives

Storm Design	Storm Exceedance
15 year	0.04
73 year	0.008
150 year	0.005

22. The existing revetment has been in place since 1994. In the with-project condition, construction will commence in 2008 and will be completed by January 2010. The 15-year storm design, therefore, is pertinent through 2007, with the improved storm exceedance design pertinent from 2008, thereafter. With-project damages were calculated for the following storm damage categories: Storm damage to the lighthouse complex, and local costs forgone for the land loss values due to erosion. With-project damages were also calculated for two recreation loss categories: lost lighthouse visitations, and lost state park visitations benefits.

Montauk Point Lighthouse Complex

23. Tables 20-22 show the residual damages that occur to the lighthouse complex under the with-project conditions for the 15-year, 73-year, and 150-year storm design stone revetment alternatives.

Table 20. Lighthouse Complex - 15yr storm design
Residual Damages (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Lighthouse Complex in Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.5646733	0.4353267			
2008	15	0.5420864	0.4579136			
2009	16	0.5204029	0.4795971			
2010	17	0.4995868	0.5004132	$23,328,193	0.040	$552,571
2011	18	0.4796033	0.5203967	$22,138,262	0.038	$370,075
2012	19	0.4604192	0.5395808	$21,009,026	0.037	$354,643
2013	20	0.4420024	0.5579976	$19,937,392	0.035	$338,391
2014	21	0.4243223	0.5756777	$16,170,297	0.034	$274,911
2015	22	0.4073494	0.5926506	$15,345,477	0.033	$260,454
2016	23	0.3910555	0.6089445	$14,562,730	0.031	$246,029
2017	24	0.3754132	0.6245868	$13,819,910	0.030	$231,791
2018	25	0.3603967	0.6396033	$13,114,980	0.029	$217,860
2019	26	0.3459808	0.6540192	$12,446,007	0.028	$204,329
2020	27	0.3321416	0.6678584	$11,811,158	0.027	$191,268
2021	28	0.3188559	0.6811441	$11,208,690	0.026	$178,727
2022	29	0.3061017	0.6938983	$10,636,954	0.025	$166,741
2023	30	0.2938576	0.7061424	$10,094,381	0.024	$155,330
2024	31	0.2821033	0.7178967	$9,579,484	0.023	$144,505
2025	32	0.2708192	0.7291808	$9,090,851	0.022	$134,268
2026	33	0.2599864	0.7400136	$8,627,142	0.021	$124,613
2027	34	0.2495870	0.7504130	$8,187,086	0.020	$115,529
2028	35	0.2396035	0.7603965	$7,769,477	0.019	$107,003
2029	36	0.2300194	0.7699806	$7,337,126	0.018	$98,531
2030	37	0.2208186	0.7791814	$6,997,076	0.018	$91,546
2031	38	0.2119858	0.7880142	$6,640,167	0.017	$84,574
2032	39	0.2035064	0.7964936	$6,301,463	0.016	$78,074
2033	40	0.1953662	0.8046338	$5,980,036	0.016	$72,025
2034	41	0.1875515	0.8124485	$5,675,005	0.015	$66,401
2035	42	0.1800494	0.8199506	$5,385,532	0.014	$61,179
2036	43	0.1728475	0.8271525	$5,110,826	0.014	$56,336
2037	44	0.1659336	0.8340664	$4,850,131	0.013	$51,848
2038	45	0.1592962	0.8407038	$4,602,734	0.013	$47,694
2039	46	0.1529244	0.8470756	$4,367,956	0.012	$43,852
2040	47	0.1468074	0.8531926	$4,145,154	0.012	$40,302
2041	48	0.1409351	0.8590649	$3,933,717	0.011	$37,023
2042	49	0.1352977	0.8647023	$3,733,065	0.011	$33,998
2043	50	0.1298858	0.8701142	$3,542,648	0.010	$31,208
2044	51	0.1246904	0.8753096	$3,361,943	0.010	$28,636
2045	52	0.1197027	0.8802973	$3,190,456	0.010	$26,268
2046	53	0.1149146	0.8850854	$3,027,716	0.009	$24,088
2047	54	0.1103181	0.8896819	$2,873,278	0.009	$22,083
2048	55	0.1059053	0.8940947	$2,726,717	0.008	$20,238
2049	56	0.1016691	0.8983309	$2,587,631	0.008	$18,543
2050	57	0.0976024	0.9023976	$2,455,641	0.008	$16,985
2051	58	0.0936983	0.9063017	$2,330,383	0.008	$15,554
2052	59	0.0899503	0.9100497	$2,211,514	0.007	$14,240
2053	60	0.0863523	0.9136477	$2,098,708	0.007	$13,035
2054	61	0.0828982	0.9171018	$1,991,657	0.007	$11,929
2055	62	0.0795823	0.9204177	$1,890,066	0.006	$10,915
2056	63	0.0763990	0.9236010	$1,236,405	0.006	$6,883
2057	64	0.0733430	0.9266570	$193,285	0.006	$1,037
2058	65	0.0704093	0.9295907	$183,426	0.006	$948
2059	66	0.0675929	0.9324071	$174,069	0.005	$867
						$5,495,880
					Annual Damages	$318,655

Table 21. Lighthouse Complex - 73yr storm design
Residual Damages (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Lighthouse Complex in Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.8936417	0.1063583			
2008	15	0.8864925	0.1135075			
2009	16	0.8794006	0.1205994			
2010	17	0.8723654	0.1276346	$23,328,193	0.008	$110,514
2011	18	0.8653865	0.1346135	$22,138,262	0.007936	$18,686
2012	19	0.8584634	0.1415366	$21,009,026	0.007872512	$18,773
2013	20	0.8515957	0.1484043	$19,937,392	0.007809532	$18,778
2014	21	0.8447829	0.1552171	$16,170,297	0.007747056	$15,989
2015	22	0.8380246	0.1619754	$15,345,477	0.007685079	$15,875
2016	23	0.8313204	0.1686796	$14,562,730	0.007623599	$15,713
2017	24	0.8246699	0.1753301	$13,819,910	0.00756261	$15,510
2018	25	0.8180725	0.1819275	$13,114,980	0.007502109	$15,272
2019	26	0.8115279	0.1884721	$12,446,007	0.007442092	$15,003
2020	27	0.8050357	0.1949643	$11,811,158	0.007382555	$14,708
2021	28	0.7985954	0.2014046	$11,208,690	0.007323495	$14,392
2022	29	0.7922067	0.2077933	$10,636,954	0.007264907	$14,059
2023	30	0.7858690	0.2141310	$10,094,381	0.007206788	$13,711
2024	31	0.7795821	0.2204179	$9,579,484	0.007149133	$13,352
2025	32	0.7733454	0.2266546	$9,090,851	0.00709194	$12,985
2026	33	0.7671586	0.2328414	$8,627,142	0.007035205	$12,612
2027	34	0.7610214	0.2389786	$8,187,086	0.006978923	$12,235
2028	35	0.7549332	0.2450668	$7,769,477	0.006923092	$11,856
2029	36	0.7488937	0.2511063	$7,337,126	0.006867707	$11,421
2030	37	0.7429026	0.2570974	$6,997,076	0.006812765	$11,099
2031	38	0.7369594	0.2630406	$6,640,167	0.006758263	$10,724
2032	39	0.7310637	0.2689363	$6,301,463	0.006704197	$10,353
2033	40	0.7252152	0.2747848	$5,980,036	0.006650564	$9,987
2034	41	0.7194135	0.2805865	$5,675,005	0.006597359	$9,626
2035	42	0.7136582	0.2863418	$5,385,532	0.00654458	$9,271
2036	43	0.7079489	0.2920511	$5,110,826	0.006492224	$8,923
2037	44	0.7022853	0.2977147	$4,850,131	0.006440286	$8,583
2038	45	0.6966670	0.3033330	$4,602,734	0.006388763	$8,251
2039	46	0.6910937	0.3089063	$4,367,956	0.006337653	$7,927
2040	47	0.6855649	0.3144351	$4,145,154	0.006286952	$7,611
2041	48	0.6800804	0.3199196	$3,933,717	0.006236656	$7,304
2042	49	0.6746398	0.3253602	$3,733,065	0.006186763	$7,006
2043	50	0.6692426	0.3307574	$3,542,648	0.006137269	$6,716
2044	51	0.6638887	0.3361113	$3,361,943	0.006088171	$6,436
2045	52	0.6585776	0.3414224	$3,190,456	0.006039466	$6,164
2046	53	0.6533090	0.3466910	$3,027,716	0.00599115	$5,902
2047	54	0.6480825	0.3519175	$2,873,278	0.005943221	$5,648
2048	55	0.6428978	0.3571022	$2,726,717	0.005895675	$5,403
2049	56	0.6377547	0.3622453	$2,587,631	0.00584851	$5,167
2050	57	0.6326526	0.3673474	$2,455,641	0.005801721	$4,939
2051	58	0.6275914	0.3724086	$2,330,383	0.005755308	$4,720
2052	59	0.6225707	0.3774293	$2,211,514	0.005709265	$4,509
2053	60	0.6175901	0.3824099	$2,098,708	0.005663591	$4,306
2054	61	0.6126494	0.3873506	$1,991,657	0.005618282	$4,111
2055	62	0.6077482	0.3922518	$1,890,066	0.005573336	$3,923
2056	63	0.6028862	0.3971138	$1,236,405	0.005528749	$2,580
2057	64	0.5980631	0.4019369	$193,285	0.005484519	$405
2058	65	0.5932786	0.4067214	$183,426	0.005440643	$387
2059	66	0.5885324	0.4114676	$174,069	0.005397118	$369
						$579,795
					Annual Damages	$33,617

Table 22. Lighthouse Complex - 150yr storm design
Residual Damages (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Lighthouse Complex in Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.9322301	0.0677699			
2008	15	0.9275690	0.0724310			
2009	16	0.9229311	0.0770689			
2010	17	0.9183165	0.0816835	$23,328,193	0.005	$69,071
2011	18	0.9137249	0.0862751	$22,138,262	0.004975	$7,464
2012	19	0.9091563	0.0908437	$21,009,026	0.004950125	$7,533
2013	20	0.9046105	0.0953895	$19,937,392	0.004925374	$7,568
2014	21	0.9000874	0.0999126	$16,170,297	0.004900748	$6,473
2015	22	0.8955870	0.1044130	$15,345,477	0.004876244	$6,456
2016	23	0.8911091	0.1088909	$14,562,730	0.004851863	$6,419
2017	24	0.8866535	0.1133465	$13,819,910	0.004827603	$6,364
2018	25	0.8822202	0.1177798	$13,114,980	0.004803465	$6,294
2019	26	0.8778091	0.1221909	$12,446,007	0.004779448	$6,211
2020	27	0.8734201	0.1265799	$11,811,158	0.004755551	$6,116
2021	28	0.8690530	0.1309470	$11,208,690	0.004731773	$6,012
2022	29	0.8647077	0.1352923	$10,636,954	0.004708114	$5,898
2023	30	0.8603842	0.1396158	$10,094,381	0.004684573	$5,778
2024	31	0.8560823	0.1439177	$9,579,484	0.004661151	$5,652
2025	32	0.8518019	0.1481981	$9,090,851	0.004637845	$5,521
2026	33	0.8475429	0.1524571	$8,627,142	0.004614656	$5,386
2027	34	0.8433051	0.1566949	$8,187,086	0.004591582	$5,248
2028	35	0.8390886	0.1609114	$7,769,477	0.004568624	$5,108
2029	36	0.8348932	0.1651068	$7,337,126	0.004545781	$4,943
2030	37	0.8307187	0.1692813	$6,997,076	0.004523052	$4,825
2031	38	0.8265651	0.1734349	$6,640,167	0.004500437	$4,683
2032	39	0.8224323	0.1775677	$6,301,463	0.004477935	$4,541
2033	40	0.8183201	0.1816799	$5,980,036	0.004455545	$4,399
2034	41	0.8142285	0.1857715	$5,675,005	0.004433268	$4,259
2035	42	0.8101574	0.1898426	$5,385,532	0.004411101	$4,120
2036	43	0.8061066	0.1938934	$5,110,826	0.004389046	$3,983
2037	44	0.8020761	0.1979239	$4,850,131	0.0043671	$3,848
2038	45	0.7980657	0.2019343	$4,602,734	0.004345265	$3,715
2039	46	0.7940753	0.2059247	$4,367,956	0.004323539	$3,585
2040	47	0.7901050	0.2098950	$4,145,154	0.004301921	$3,458
2041	48	0.7861544	0.2138456	$3,933,717	0.004280411	$3,333
2042	49	0.7822237	0.2177763	$3,733,065	0.004259009	$3,211
2043	50	0.7783126	0.2216874	$3,542,648	0.004237714	$3,091
2044	51	0.7744210	0.2255790	$3,361,943	0.004216526	$2,975
2045	52	0.7705489	0.2294511	$3,190,456	0.004195443	$2,862
2046	53	0.7666961	0.2333039	$3,027,716	0.004174466	$2,752
2047	54	0.7628627	0.2371373	$2,873,278	0.004153594	$2,646
2048	55	0.7590484	0.2409516	$2,726,717	0.004132826	$2,542
2049	56	0.7552531	0.2447469	$2,587,631	0.004112161	$2,442
2050	57	0.7514768	0.2485232	$2,455,641	0.004091601	$2,344
2051	58	0.7477195	0.2522805	$2,330,383	0.004071143	$2,250
2052	59	0.7439809	0.2560191	$2,211,514	0.004050787	$2,159
2053	60	0.7402610	0.2597390	$2,098,708	0.004030533	$2,070
2054	61	0.7365597	0.2634403	$1,991,657	0.00401038	$1,985
2055	62	0.7328769	0.2671231	$1,890,066	0.003990328	$1,903
2056	63	0.7292125	0.2707875	$1,236,405	0.003970377	$1,257
2057	64	0.7255664	0.2744336	$193,285	0.003950525	$198
2058	65	0.7219386	0.2780614	$183,426	0.003930772	$190
2059	66	0.7183289	0.2816711	$174,069	0.003911118	$182
						$271,324
					Annual Damages	$15,732

Local Costs Forgone

24. Local costs forgone for loss of land value were calculated for the three alternatives based on the different probabilities that the stone revetment will be displaced, thereby exposing the bluff to erosion. The long-term erosion rate that is used is three feet per year at the top of the bluff. Tables 23-25 show the residual damages for local costs forgone for loss of land value for the three alternatives.

Table 23. Local Costs Forgone- 15yr storm design Residual Damages
(Oct. 2004 P.L., 5.375% discount rate)

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value Factor		
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.5646733	0.4353267			
2008	15	0.5420864	0.4579136			
2009	16	0.5204029	0.4795971			
2010	17	0.4995868	0.5004132	1.0000000	$82,600	$41,334
2011	18	0.4796033	0.5203967	0.9489917	$82,600	$40,792
2012	19	0.4604192	0.5395808	0.9005852	$82,600	$40,139
2013	20	0.4420024	0.5579976	0.8546479	$82,600	$39,391
2014	21	0.4243223	0.5756777	0.8110538	$82,600	$38,566
2015	22	0.4073494	0.5926506	0.7696833	$82,600	$37,678
2016	23	0.3910555	0.6089445	0.7304231	$82,600	$36,739
2017	24	0.3754132	0.6245868	0.6931654	$82,600	$35,761
2018	25	0.3603967	0.6396033	0.6578082	$82,600	$34,753
2019	26	0.3459808	0.6540192	0.6242545	$82,600	$33,723
2020	27	0.3321416	0.6678584	0.5924124	$82,600	$32,680
2021	28	0.3188559	0.6811441	0.5621944	$82,600	$31,630
2022	29	0.3061017	0.6938983	0.5335178	$82,600	$30,579
2023	30	0.2938576	0.7061424	0.5063040	$82,600	$29,531
2024	31	0.2821033	0.7178967	0.4804783	$82,600	$28,492
2025	32	0.2708192	0.7291808	0.4559699	$82,600	$27,463
2026	33	0.2599864	0.7400136	0.4327117	$82,600	$26,450
2027	34	0.2495870	0.7504130	0.4106398	$82,600	$25,453
2028	35	0.2396035	0.7603965	0.3896937	$82,600	$24,476
2029	36	0.2300194	0.7699806	0.3698161	$82,600	$23,520
2030	37	0.2208186	0.7791814	0.3509524	$82,600	$22,587
2031	38	0.2119858	0.7880142	0.3330509	$82,600	$21,678
2032	39	0.2035064	0.7964936	0.3160626	$82,600	$20,794
2033	40	0.1953662	0.8046338	0.2999408	$82,600	$19,935
2034	41	0.1875515	0.8124485	0.2846413	$82,600	$19,102
2035	42	0.1800494	0.8199506	0.2701222	$82,600	$18,295
2036	43	0.1728475	0.8271525	0.2563437	$82,600	$17,514
2037	44	0.1659336	0.8340664	0.2432681	$82,600	$16,760
2038	45	0.1592962	0.8407038	0.2308594	$82,600	$16,031
2039	46	0.1529244	0.8470756	0.2190836	$82,600	$15,329
2040	47	0.1468074	0.8531926	0.2079086	$82,600	$14,652
2041	48	0.1409351	0.8590649	0.1973035	$82,600	$14,000
2042	49	0.1352977	0.8647023	0.1872394	$82,600	$13,373
2043	50	0.1298858	0.8701142	0.1776886	$82,600	$12,771
2044	51	0.1246904	0.8753096	0.1686250	$82,600	$12,192
2045	52	0.1197027	0.8802973	0.1600237	$82,600	$11,636
2046	53	0.1149146	0.8850854	0.1518612	$82,600	$11,102
2047	54	0.1103181	0.8896819	0.1441150	$82,600	$10,591
2048	55	0.1059053	0.8940947	0.1367640	$82,600	$10,100
2049	56	0.1016691	0.8983309	0.1297879	$82,600	$9,631
2050	57	0.0976024	0.9023976	0.1231676	$82,600	$9,181
2051	58	0.0936983	0.9063017	0.1168850	$82,600	$8,750
2052	59	0.0899503	0.9100497	0.1109229	$82,600	$8,338
2053	60	0.0863523	0.9136477	0.1052649	$82,600	$7,944
2054	61	0.0828982	0.9171018	0.0998956	$82,600	$7,567
2055	62	0.0795823	0.9204177	0.0948000	$82,600	$7,207
2056	63	0.0763990	0.9236010	0.0899645	$82,600	$6,863
2057	64	0.0733430	0.9266570	0.0853755	$82,600	$6,535
2058	65	0.0704093	0.9295907	0.0810207	$82,600	$6,221
2059	66	0.0675929	0.9324071	0.0768879	$82,600	$5,922
						$1,041,755
				Annual Damages		$60,402

Table 24. Local Costs Forgone - 73yr storm design Residual Damages

(Oct. 2004 P.L., 5.375% discount rate)

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value Factor		
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.8936417	0.1063583			
2008	15	0.8864925	0.1135075			
2009	16	0.8794006	0.1205994			
2010	17	0.8723654	0.1276346	1.0000000	$82,600	$10,543
2011	18	0.8653865	0.1346135	0.9489917	$82,600	$10,552
2012	19	0.8584634	0.1415366	0.9005852	$82,600	$10,529
2013	20	0.8515957	0.1484043	0.8546479	$82,600	$10,476
2014	21	0.8447829	0.1552171	0.8110538	$82,600	$10,398
2015	22	0.8380246	0.1619754	0.7696833	$82,600	$10,298
2016	23	0.8313204	0.1686796	0.7304231	$82,600	$10,177
2017	24	0.8246699	0.1753301	0.6931654	$82,600	$10,039
2018	25	0.8180725	0.1819275	0.6578082	$82,600	$9,885
2019	26	0.8115279	0.1884721	0.6242545	$82,600	$9,718
2020	27	0.8050357	0.1949643	0.5924124	$82,600	$9,540
2021	28	0.7985954	0.2014046	0.5621944	$82,600	$9,353
2022	29	0.7922067	0.2077933	0.5335178	$82,600	$9,157
2023	30	0.7858690	0.2141310	0.5063040	$82,600	$8,955
2024	31	0.7795821	0.2204179	0.4804783	$82,600	$8,748
2025	32	0.7733454	0.2266546	0.4559699	$82,600	$8,537
2026	33	0.7671586	0.2328414	0.4327117	$82,600	$8,322
2027	34	0.7610214	0.2389786	0.4106398	$82,600	$8,106
2028	35	0.7549332	0.2450668	0.3896937	$82,600	$7,888
2029	36	0.7488937	0.2511063	0.3698161	$82,600	$7,670
2030	37	0.7429026	0.2570974	0.3509524	$82,600	$7,453
2031	38	0.7369594	0.2630406	0.3330509	$82,600	$7,236
2032	39	0.7310637	0.2689363	0.3160626	$82,600	$7,021
2033	40	0.7252152	0.2747848	0.2999408	$82,600	$6,808
2034	41	0.7194135	0.2805865	0.2846413	$82,600	$6,597
2035	42	0.7136582	0.2863418	0.2701222	$82,600	$6,389
2036	43	0.7079489	0.2920511	0.2563437	$82,600	$6,184
2037	44	0.7022853	0.2977147	0.2432681	$82,600	$5,982
2038	45	0.6966670	0.3033330	0.2308594	$82,600	$5,784
2039	46	0.6910937	0.3089063	0.2190836	$82,600	$5,590
2040	47	0.6855649	0.3144351	0.2079086	$82,600	$5,400
2041	48	0.6800804	0.3199196	0.1973035	$82,600	$5,214
2042	49	0.6746398	0.3253602	0.1872394	$82,600	$5,032
2043	50	0.6692426	0.3307574	0.1776886	$82,600	$4,855
2044	51	0.6638887	0.3361113	0.1686250	$82,600	$4,682
2045	52	0.6585776	0.3414224	0.1600237	$82,600	$4,513
2046	53	0.6533090	0.3466910	0.1518612	$82,600	$4,349
2047	54	0.6480825	0.3519175	0.1441150	$82,600	$4,189
2048	55	0.6428978	0.3571022	0.1367640	$82,600	$4,034
2049	56	0.6377547	0.3622453	0.1297879	$82,600	$3,883
2050	57	0.6326526	0.3673474	0.1231676	$82,600	$3,737
2051	58	0.6275914	0.3724086	0.1168850	$82,600	$3,595
2052	59	0.6225707	0.3774293	0.1109229	$82,600	$3,458
2053	60	0.6175901	0.3824099	0.1052649	$82,600	$3,325
2054	61	0.6126494	0.3873506	0.0998956	$82,600	$3,196
2055	62	0.6077482	0.3922518	0.0948000	$82,600	$3,072
2056	63	0.6028862	0.3971138	0.0899645	$82,600	$2,951
2057	64	0.5980631	0.4019369	0.0853755	$82,600	$2,834
2058	65	0.5932786	0.4067214	0.0810207	$82,600	$2,722
2059	66	0.5885324	0.4114676	0.0768879	$82,600	$2,613
						$331,590
				Annual Damages		$19,226

Table 25. Local Costs Forgone - 150yr storm design Residual Damages
(Oct. 2004 P.L., 5.375% discount rate)

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value Factor		
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.9322301	0.0677699			
2008	15	0.9275690	0.0724310			
2009	16	0.9229311	0.0770689			
2010	17	0.9183165	0.0816835	1.0000000	$82,600	$6,747
2011	18	0.9137249	0.0862751	0.9489917	$82,600	$6,763
2012	19	0.9091563	0.0908437	0.9005852	$82,600	$6,758
2013	20	0.9046105	0.0953895	0.8546479	$82,600	$6,734
2014	21	0.9000874	0.0999126	0.8110538	$82,600	$6,693
2015	22	0.8955870	0.1044130	0.7696833	$82,600	$6,638
2016	23	0.8911091	0.1088909	0.7304231	$82,600	$6,570
2017	24	0.8866535	0.1133465	0.6931654	$82,600	$6,490
2018	25	0.8822202	0.1177798	0.6578082	$82,600	$6,400
2019	26	0.8778091	0.1221909	0.6242545	$82,600	$6,301
2020	27	0.8734201	0.1265799	0.5924124	$82,600	$6,194
2021	28	0.8690530	0.1309470	0.5621944	$82,600	$6,081
2022	29	0.8647077	0.1352923	0.5335178	$82,600	$5,962
2023	30	0.8603842	0.1396158	0.5063040	$82,600	$5,839
2024	31	0.8560823	0.1439177	0.4804783	$82,600	$5,712
2025	32	0.8518019	0.1481981	0.4559699	$82,600	$5,582
2026	33	0.8475429	0.1524571	0.4327117	$82,600	$5,449
2027	34	0.8433051	0.1566949	0.4106398	$82,600	$5,315
2028	35	0.8390886	0.1609114	0.3896937	$82,600	$5,180
2029	36	0.8348932	0.1651068	0.3698161	$82,600	$5,043
2030	37	0.8307187	0.1692813	0.3509524	$82,600	$4,907
2031	38	0.8265651	0.1734349	0.3330509	$82,600	$4,771
2032	39	0.8224323	0.1775677	0.3160626	$82,600	$4,636
2033	40	0.8183201	0.1816799	0.2999408	$82,600	$4,501
2034	41	0.8142285	0.1857715	0.2846413	$82,600	$4,368
2035	42	0.8101574	0.1898426	0.2701222	$82,600	$4,236
2036	43	0.8061066	0.1938934	0.2563437	$82,600	$4,105
2037	44	0.8020761	0.1979239	0.2432681	$82,600	$3,977
2038	45	0.7980657	0.2019343	0.2308594	$82,600	$3,851
2039	46	0.7940753	0.2059247	0.2190836	$82,600	$3,726
2040	47	0.7901050	0.2098950	0.2079086	$82,600	$3,605
2041	48	0.7861544	0.2138456	0.1973035	$82,600	$3,485
2042	49	0.7822237	0.2177763	0.1872394	$82,600	$3,368
2043	50	0.7783126	0.2216874	0.1776886	$82,600	$3,254
2044	51	0.7744210	0.2255790	0.1686250	$82,600	$3,142
2045	52	0.7705489	0.2294511	0.1600237	$82,600	$3,033
2046	53	0.7666961	0.2333039	0.1518612	$82,600	$2,927
2047	54	0.7628627	0.2371373	0.1441150	$82,600	$2,823
2048	55	0.7590484	0.2409516	0.1367640	$82,600	$2,722
2049	56	0.7552531	0.2447469	0.1297879	$82,600	$2,624
2050	57	0.7514768	0.2485232	0.1231676	$82,600	$2,528
2051	58	0.7477195	0.2522805	0.1168850	$82,600	$2,436
2052	59	0.7439809	0.2560191	0.1109229	$82,600	$2,346
2053	60	0.7402610	0.2597390	0.1052649	$82,600	$2,258
2054	61	0.7365597	0.2634403	0.0998956	$82,600	$2,174
2055	62	0.7328769	0.2671231	0.0948000	$82,600	$2,092
2056	63	0.7292125	0.2707875	0.0899645	$82,600	$2,012
2057	64	0.7255664	0.2744336	0.0853755	$82,600	$1,935
2058	65	0.7219386	0.2780614	0.0810207	$82,600	$1,861
2059	66	0.7183289	0.2816711	0.0768879	$82,600	$1,789
						$217,940
					Annual Damages	$12,636

Recreation Loss

25. Residual loss of Montauk Point Lighthouse visitation benefits was calculated for the three with-project alternatives based on the probability that the stone revetment will be displaced, thereby exposing the bluff to erosion. The long-term erosion rate that is used is three feet per year. Therefore, by the tenth year after the upper sections of the revetment that protects the bluff are displaced the stone revetment the lighthouse will be immediately threatened and closed to the public. Tables 26-28 show the residual lost visitations benefits for the three with-project alternatives.

26. Similarly, residual losses of the Montauk Point State Park visitations benefits were calculated for the three with-project alternatives and are shown in Tables 29-31.

Table 26. Lighthouse Visitations Damages - 15yr with-project design Residual Damages (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.3806404	0.6193596			
2008	15	0.5420864	0.4579136			
2009	16	0.5204029	0.4795971			
2010	17	0.4995868	0.5004132			
2011	18	0.4796033	0.5203967			
2012	19	0.4604192	0.5395808			
2013	20	0.4420024	0.5579976			
2014	21	0.4243223	0.5756777			
2015	22	0.4073494	0.5926506			
2016	23	0.3910555	0.6089445			
2017	24	0.3754132	0.6245868			
2018	25	0.3603967	0.6396033			
2019	26	0.3459808	0.6540192			
2020	27	0.3321416	0.6678584	$30,959,738	0.04	$619,706
2021	28	0.3188559	0.6811441	$29,158,705	0.0384	$582,685
2022	29	0.3061017	0.6938983	$27,449,540	0.036864	$546,002
2023	30	0.2938576	0.7061424	$25,827,557	0.0353894	$510,022
2024	31	0.2821033	0.7178967	$24,288,308	0.0339739	$475,031
2025	32	0.2708192	0.7291808	$22,827,574	0.0326149	$441,240
2026	33	0.2599864	0.7400136	$21,441,349	0.0313103	$408,806
2027	34	0.2495870	0.7504130	$20,125,833	0.0300579	$377,838
2028	35	0.2396035	0.7603965	$18,877,420	0.0288556	$348,404
2029	36	0.2300194	0.7699806	$17,692,686	0.0277014	$320,542
2030	37	0.2208186	0.7791814	$16,568,383	0.0265933	$294,264
2031	38	0.2119858	0.7880142	$15,501,429	0.0255296	$269,559
2032	39	0.2035064	0.7964936	$14,488,899	0.0245084	$246,403
2033	40	0.1953662	0.8046338	$13,528,016	0.0235281	$224,757
2034	41	0.1875515	0.8124485	$12,616,146	0.0225869	$204,572
2035	42	0.1800494	0.8199506	$11,750,789	0.0216835	$185,794
2036	43	0.1728475	0.8271525	$10,929,572	0.0208161	$168,361
2037	44	0.1659336	0.8340664	$10,150,244	0.0199835	$152,212
2038	45	0.1592962	0.8407038	$9,410,669	0.0191841	$137,279
2039	46	0.1529244	0.8470756	$8,708,817	0.0184168	$123,496
2040	47	0.1468074	0.8531926	$8,042,767	0.0176801	$110,797
2041	48	0.1409351	0.8590649	$7,410,690	0.0169729	$99,117
2042	49	0.1352977	0.8647023	$6,810,854	0.016294	$88,392
2043	50	0.1298858	0.8701142	$6,241,615	0.0156422	$78,559
2044	51	0.1246904	0.8753096	$5,701,412	0.0150165	$69,558
2045	52	0.1197027	0.8802973	$5,188,764	0.0144159	$61,333
2046	53	0.1149146	0.8850854	$4,702,265	0.0138392	$53,828
2047	54	0.1103181	0.8896819	$4,240,582	0.0132857	$46,990
2048	55	0.1059053	0.8940947	$3,802,448	0.0127542	$40,772
2049	56	0.1016691	0.8983309	$3,386,663	0.0122441	$35,125
2050	57	0.0976024	0.9023976	$2,992,086	0.0117543	$30,007
2051	58	0.0936983	0.9063017	$2,617,636	0.0112841	$25,375
2052	59	0.0899503	0.9100497	$2,262,286	0.0108328	$21,191
2053	60	0.0863523	0.9136477	$1,925,061	0.0103995	$17,419
2054	61	0.0828982	0.9171018	$1,605,038	0.0099835	$14,026
2055	62	0.0795823	0.9204177	$1,301,339	0.0095841	$10,979
2056	63	0.0763990	0.9236010	$1,013,131	0.0092008	$8,250
2057	64	0.0733430	0.9266570	$739,624	0.0088327	$5,812
2058	65	0.0704093	0.9295907	$480,069	0.0084794	$3,640
2059	66	0.0675929	0.9324071	$233,752	0.0081403	$1,709
						$7,459,851
					Annual Damages	$432,527

Table 27. Lighthouse Visitations Damages - 73yr with-project design Residual Damages (Oct. 2004 P.L. 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.8936417	0.1063583			
2008	15	0.8864925	0.1135075			
2009	16	0.8794006	0.1205994			
2010	17	0.8723654	0.1276346			
2011	18	0.8653865	0.1346135			
2012	19	0.8584634	0.1415366			
2013	20	0.8515957	0.1484043			
2014	21	0.8447829	0.1552171			
2015	22	0.8380246	0.1619754			
2016	23	0.8313204	0.1686796			
2017	24	0.8246699	0.1753301			
2018	25	0.8180725	0.1819275			
2019	26	0.8115279	0.1884721			
2020	27	0.8050357	0.1949643	$30,959,738	0.008	$31,612
2021	28	0.7985954	0.2014046	$29,158,705	0.007936	$31,150
2022	29	0.7922067	0.2077933	$27,449,540	0.0078725	$30,586
2023	30	0.7858690	0.2141310	$25,827,557	0.0078095	$29,933
2024	31	0.7795821	0.2204179	$24,288,308	0.0077471	$29,206
2025	32	0.7733454	0.2266546	$22,827,574	0.0076851	$28,416
2026	33	0.7671586	0.2328414	$21,441,349	0.0076236	$27,572
2027	34	0.7610214	0.2389786	$20,125,833	0.0075626	$26,686
2028	35	0.7549332	0.2450668	$18,877,420	0.0075021	$25,765
2029	36	0.7488937	0.2511063	$17,692,686	0.0074421	$24,816
2030	37	0.7429026	0.2570974	$16,568,383	0.0073826	$23,847
2031	38	0.7369594	0.2630406	$15,501,429	0.0073235	$22,864
2032	39	0.7310637	0.2689363	$14,488,899	0.0072649	$21,872
2033	40	0.7252152	0.2747848	$13,528,016	0.0072068	$20,876
2034	41	0.7194135	0.2805865	$12,616,146	0.0071491	$19,880
2035	42	0.7136582	0.2863418	$11,750,789	0.0070919	$18,888
2036	43	0.7079489	0.2920511	$10,929,572	0.0070352	$17,904
2037	44	0.7022853	0.2977147	$10,150,244	0.0069789	$16,929
2038	45	0.6966670	0.3033330	$9,410,669	0.0069231	$15,966
2039	46	0.6910937	0.3089063	$8,708,817	0.0068677	$15,019
2040	47	0.6855649	0.3144351	$8,042,767	0.0068128	$14,087
2041	48	0.6800804	0.3199196	$7,410,690	0.0067583	$13,174
2042	49	0.6746398	0.3253602	$6,810,854	0.0067042	$12,280
2043	50	0.6692426	0.3307574	$6,241,615	0.0066506	$11,406
2044	51	0.6638887	0.3361113	$5,701,412	0.0065974	$10,554
2045	52	0.6585776	0.3414224	$5,188,764	0.0065446	$9,724
2046	53	0.6533090	0.3466910	$4,702,265	0.0064922	$8,916
2047	54	0.6480825	0.3519175	$4,240,582	0.0064403	$8,131
2048	55	0.6428978	0.3571022	$3,802,448	0.0063888	$7,369
2049	56	0.6377547	0.3622453	$3,386,663	0.0063377	$6,630
2050	57	0.6326526	0.3673474	$2,992,086	0.006287	$5,915
2051	58	0.6275914	0.3724086	$2,617,636	0.0062367	$5,223
2052	59	0.6225707	0.3774293	$2,262,286	0.0061868	$4,554
2053	60	0.6175901	0.3824099	$1,925,061	0.0061373	$3,908
2054	61	0.6126494	0.3873506	$1,605,038	0.0060882	$3,284
2055	62	0.6077482	0.3922518	$1,301,339	0.0060395	$2,683
2056	63	0.6028862	0.3971138	$1,013,131	0.0059911	$2,104
2057	64	0.5980631	0.4019369	$739,624	0.0059432	$1,547
2058	65	0.5932786	0.4067214	$480,069	0.0058957	$1,011
2059	66	0.5885324	0.4114676	$233,752	0.0058485	$495
						$612,784
					Annual Damages	$35,530

Table 28. Lighthouse Visitations Damages - 150yr with-project design Residual Damages (Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.9322301	0.0677699			
2008	15	0.9275690	0.0724310			
2009	16	0.9229311	0.0770689			
2010	17	0.9183165	0.0816835			
2011	18	0.9137249	0.0862751			
2012	19	0.9091563	0.0908437			
2013	20	0.9046105	0.0953895			
2014	21	0.9000874	0.0999126			
2015	22	0.8955870	0.1044130			
2016	23	0.8911091	0.1088909			
2017	24	0.8866535	0.1133465			
2018	25	0.8822202	0.1177798			
2019	26	0.8778091	0.1221909			
2020	27	0.8734201	0.1265799	$30,959,738	0.005	$12,645
2021	28	0.8690530	0.1309470	$29,158,705	0.004975	$12,515
2022	29	0.8647077	0.1352923	$27,449,540	0.004950125	$12,344
2023	30	0.8603842	0.1396158	$25,827,557	0.004925374	$12,135
2024	31	0.8560823	0.1439177	$24,288,308	0.004900748	$11,893
2025	32	0.8518019	0.1481981	$22,827,574	0.004876244	$11,623
2026	33	0.8475429	0.1524571	$21,441,349	0.004851863	$11,328
2027	34	0.8433051	0.1566949	$20,125,833	0.004827603	$11,013
2028	35	0.8390886	0.1609114	$18,877,420	0.004803465	$10,680
2029	36	0.8348932	0.1651068	$17,692,686	0.004779448	$10,333
2030	37	0.8307187	0.1692813	$16,568,383	0.004755551	$9,973
2031	38	0.8265651	0.1734349	$15,501,429	0.004731773	$9,605
2032	39	0.8224323	0.1775677	$14,488,899	0.004708114	$9,229
2033	40	0.8183201	0.1816799	$13,528,016	0.004684573	$8,848
2034	41	0.8142285	0.1857715	$12,616,146	0.004661151	$8,463
2035	42	0.8101574	0.1898426	$11,750,789	0.004637845	$8,077
2036	43	0.8061066	0.1938934	$10,929,572	0.004614656	$7,689
2037	44	0.8020761	0.1979239	$10,150,244	0.004591582	$7,303
2038	45	0.7980657	0.2019343	$9,410,669	0.004568624	$6,918
2039	46	0.7940753	0.2059247	$8,708,817	0.004545781	$6,536
2040	47	0.7901050	0.2098950	$8,042,767	0.004523052	$6,158
2041	48	0.7861544	0.2138456	$7,410,690	0.004500437	$5,784
2042	49	0.7822237	0.2177763	$6,810,854	0.004477935	$5,416
2043	50	0.7783126	0.2216874	$6,241,615	0.004455545	$5,052
2044	51	0.7744210	0.2255790	$5,701,412	0.004433268	$4,696
2045	52	0.7705489	0.2294511	$5,188,764	0.004411101	$4,345
2046	53	0.7666961	0.2333039	$4,702,265	0.004389046	$4,002
2047	54	0.7628627	0.2371373	$4,240,582	0.0043671	$3,665
2048	55	0.7590484	0.2409516	$3,802,448	0.004345265	$3,336
2049	56	0.7552531	0.2447469	$3,386,663	0.004323539	$3,015
2050	57	0.7514768	0.2485232	$2,992,086	0.004301921	$2,702
2051	58	0.7477195	0.2522805	$2,617,636	0.004280411	$2,396
2052	59	0.7439809	0.2560191	$2,262,286	0.004259009	$2,098
2053	60	0.7402610	0.2597390	$1,925,061	0.004237714	$1,808
2054	61	0.7365597	0.2634403	$1,605,038	0.004216526	$1,527
2055	62	0.7328769	0.2671231	$1,301,339	0.004195443	$1,253
2056	63	0.7292125	0.2707875	$1,013,131	0.004174466	$987
2057	64	0.7255664	0.2744336	$739,624	0.004153594	$729
2058	65	0.7219386	0.2780614	$480,069	0.004132826	$478
2059	66	0.7183289	0.2816711	$233,752	0.004112161	$235
						$258,831
					Annual Damages	$15,007

Table 29. Park Visitations - 15yr storm design Residual Damages

(Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.5646733	0.4353267			
2008	15	0.5420864	0.4579136			
2009	16	0.5204029	0.4795971			
2010	17	0.4995868	0.5004132			
2011	18	0.4796033	0.5203967			
2012	19	0.4604192	0.5395808			
2013	20	0.4420024	0.5579976			
2014	21	0.4243223	0.5756777			
2015	22	0.4073494	0.5926506			
2016	23	0.3910555	0.6089445			
2017	24	0.3754132	0.6245868			
2018	25	0.3603967	0.6396033			
2019	26	0.3459808	0.6540192			
2020	27	0.3321416	0.6678584	$6,950,283	0.040	$139,121
2021	28	0.3188559	0.6811441	$6,545,962	0.038	$130,809
2022	29	0.3061017	0.6938983	$6,162,264	0.037	$122,574
2023	30	0.2938576	0.7061424	$5,798,138	0.035	$114,497
2024	31	0.2821033	0.7178967	$5,452,585	0.034	$106,642
2025	32	0.2708192	0.7291808	$5,124,659	0.033	$99,056
2026	33	0.2599864	0.7400136	$4,813,459	0.031	$91,775
2027	34	0.2495870	0.7504130	$4,518,134	0.030	$84,822
2028	35	0.2396035	0.7603965	$4,237,872	0.029	$78,215
2029	36	0.2300194	0.7699806	$3,971,906	0.028	$71,960
2030	37	0.2208186	0.7791814	$3,719,507	0.027	$66,061
2031	38	0.2119858	0.7880142	$3,479,982	0.026	$60,515
2032	39	0.2035064	0.7964936	$3,252,674	0.025	$55,316
2033	40	0.1953662	0.8046338	$3,036,962	0.024	$50,457
2034	41	0.1875515	0.8124485	$2,832,252	0.023	$45,925
2035	42	0.1800494	0.8199506	$2,637,984	0.022	$41,710
2036	43	0.1728475	0.8271525	$2,453,626	0.021	$37,796
2037	44	0.1659336	0.8340664	$2,278,671	0.020	$34,171
2038	45	0.1592962	0.8407038	$2,112,641	0.019	$30,818
2039	46	0.1529244	0.8470756	$1,955,079	0.018	$27,724
2040	47	0.1468074	0.8531926	$1,805,555	0.018	$24,873
2041	48	0.1409351	0.8590649	$1,663,657	0.017	$22,251
2042	49	0.1352977	0.8647023	$1,528,998	0.016	$19,843
2043	50	0.1298858	0.8701142	$1,401,207	0.016	$17,636
2044	51	0.1246904	0.8753096	$1,279,934	0.015	$15,615
2045	52	0.1197027	0.8802973	$1,164,848	0.014	$13,769
2046	53	0.1149146	0.8850854	$1,055,631	0.014	$12,084
2047	54	0.1103181	0.8896819	$951,986	0.013	$10,549
2048	55	0.1059053	0.8940947	$853,628	0.013	$9,153
2049	56	0.1016691	0.8983309	$760,286	0.012	$7,885
2050	57	0.0976024	0.9023976	$671,706	0.012	$6,736
2051	58	0.0936983	0.9063017	$587,644	0.011	$5,697
2052	59	0.0899503	0.9100497	$507,870	0.011	$4,757
2053	60	0.0863523	0.9136477	$432,165	0.010	$3,911
2054	61	0.0828982	0.9171018	$360,322	0.010	$3,149
2055	62	0.0795823	0.9204177	$292,143	0.010	$2,465
2056	63	0.0763990	0.9236010	$227,442	0.009	$1,852
2057	64	0.0733430	0.9266570	$166,041	0.009	$1,305
2058	65	0.0704093	0.9295907	$107,773	0.008	$817
2059	66	0.0675929	0.9324071	$52,476	0.008	$384
						$1,674,694
					Annual Damages	$97,100

Table 30. Park Visitations - 73yr storm design Residual Damages

(Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.8936417	0.1063583			
2008	15	0.8864925	0.1135075			
2009	16	0.8794006	0.1205994			
2010	17	0.8723654	0.1276346			
2011	18	0.8653865	0.1346135			
2012	19	0.8584634	0.1415366			
2013	20	0.8515957	0.1484043			
2014	21	0.8447829	0.1552171			
2015	22	0.8380246	0.1619754			
2016	23	0.8313204	0.1686796			
2017	24	0.8246699	0.1753301			
2018	25	0.8180725	0.1819275			
2019	26	0.8115279	0.1884721			
2020	27	0.8050357	0.1949643	$6,950,283	0.008	$7,097
2021	28	0.7985954	0.2014046	$6,545,962	0.007936	$6,993
2022	29	0.7922067	0.2077933	$6,162,264	0.007872512	$6,866
2023	30	0.7858690	0.2141310	$5,798,138	0.007809532	$6,720
2024	31	0.7795821	0.2204179	$5,452,585	0.007747056	$6,557
2025	32	0.7733454	0.2266546	$5,124,659	0.007685079	$6,379
2026	33	0.7671586	0.2328414	$4,813,459	0.007623599	$6,190
2027	34	0.7610214	0.2389786	$4,518,134	0.00756261	$5,991
2028	35	0.7549332	0.2450668	$4,237,872	0.007502109	$5,784
2029	36	0.7488937	0.2511063	$3,971,906	0.007442092	$5,571
2030	37	0.7429026	0.2570974	$3,719,507	0.007382555	$5,354
2031	38	0.7369594	0.2630406	$3,479,982	0.007323495	$5,133
2032	39	0.7310637	0.2689363	$3,252,674	0.007264907	$4,910
2033	40	0.7252152	0.2747848	$3,036,962	0.007206788	$4,687
2034	41	0.7194135	0.2805865	$2,832,252	0.007149133	$4,463
2035	42	0.7136582	0.2863418	$2,637,984	0.00709194	$4,240
2036	43	0.7079489	0.2920511	$2,453,626	0.007035205	$4,019
2037	44	0.7022853	0.2977147	$2,278,671	0.006978923	$3,800
2038	45	0.6966670	0.3033330	$2,112,641	0.006923092	$3,584
2039	46	0.6910937	0.3089063	$1,955,079	0.006867707	$3,372
2040	47	0.6855649	0.3144351	$1,805,555	0.006812765	$3,163
2041	48	0.6800804	0.3199196	$1,663,657	0.006758263	$2,957
2042	49	0.6746398	0.3253602	$1,528,998	0.006704197	$2,757
2043	50	0.6692426	0.3307574	$1,401,207	0.006650564	$2,561
2044	51	0.6638887	0.3361113	$1,279,934	0.006597359	$2,369
2045	52	0.6585776	0.3414224	$1,164,848	0.00654458	$2,183
2046	53	0.6533090	0.3466910	$1,055,631	0.006492224	$2,002
2047	54	0.6480825	0.3519175	$951,986	0.006440286	$1,825
2048	55	0.6428978	0.3571022	$853,628	0.006388763	$1,654
2049	56	0.6377547	0.3622453	$760,286	0.006337653	$1,488
2050	57	0.6326526	0.3673474	$671,706	0.006286952	$1,328
2051	58	0.6275914	0.3724086	$587,644	0.006236656	$1,172
2052	59	0.6225707	0.3774293	$507,870	0.006186763	$1,022
2053	60	0.6175901	0.3824099	$432,165	0.006137269	$877
2054	61	0.6126494	0.3873506	$360,322	0.006088171	$737
2055	62	0.6077482	0.3922518	$292,143	0.006039466	$602
2056	63	0.6028862	0.3971138	$227,442	0.00599115	$472
2057	64	0.5980631	0.4019369	$166,041	0.005943221	$347
2058	65	0.5932786	0.4067214	$107,773	0.005895675	$227
2059	66	0.5885324	0.4114676	$52,476	0.00584851	$111
						$137,567
					Annual Damages	$7,976

Table 31. Park Visitations - 150yr storm design Residual Damages

(Oct. 2004 P.L., 5.375% discount rate)

Discount Rate 0.05375

	End of year n	Probability that armor stone will be there at end of year n	Probability that armor stone won't be there at end of year n	Present Value of Visitations for Year n	Prob. Of Damage in Year n	Expected Damage in Year n
1994	1	0.9333333	0.0666667			
1995	2	0.8711111	0.1288889			
1996	3	0.8130370	0.1869630			
1997	4	0.7588346	0.2411654			
1998	5	0.7082456	0.2917544			
1999	6	0.6610292	0.3389708			
2000	7	0.6169606	0.3830394			
2001	8	0.5758299	0.4241701			
2002	9	0.5374412	0.4625588			
2003	10	0.5016118	0.4983882			
2004	11	0.4681710	0.5318290			
2005	12	0.4369596	0.5630404			
2006	13	0.4078290	0.5921710			
2007	14	0.9322301	0.0677699			
2008	15	0.9275690	0.0724310			
2009	16	0.9229311	0.0770689			
2010	17	0.9183165	0.0816835			
2011	18	0.9137249	0.0862751			
2012	19	0.9091563	0.0908437			
2013	20	0.9046105	0.0953895			
2014	21	0.9000874	0.0999126			
2015	22	0.8955870	0.1044130			
2016	23	0.8911091	0.1088909			
2017	24	0.8866535	0.1133465			
2018	25	0.8822202	0.1177798			
2019	26	0.8778091	0.1221909			
2020	27	0.8734201	0.1265799	$6,950,283	0.005	$2,839
2021	28	0.8690530	0.1309470	$6,545,962	0.004975	$2,810
2022	29	0.8647077	0.1352923	$6,162,264	0.004950125	$2,771
2023	30	0.8603842	0.1396158	$5,798,138	0.004925374	$2,724
2024	31	0.8560823	0.1439177	$5,452,585	0.004900748	$2,670
2025	32	0.8518019	0.1481981	$5,124,659	0.004876244	$2,609
2026	33	0.8475429	0.1524571	$4,813,459	0.004851863	$2,543
2027	34	0.8433051	0.1566949	$4,518,134	0.004827603	$2,472
2028	35	0.8390886	0.1609114	$4,237,872	0.004803465	$2,398
2029	36	0.8348932	0.1651068	$3,971,906	0.004779448	$2,320
2030	37	0.8307187	0.1692813	$3,719,507	0.004755551	$2,239
2031	38	0.8265651	0.1734349	$3,479,982	0.004731773	$2,156
2032	39	0.8224323	0.1775677	$3,252,674	0.004708114	$2,072
2033	40	0.8183201	0.1816799	$3,036,962	0.004684573	$1,986
2034	41	0.8142285	0.1857715	$2,832,252	0.004661151	$1,900
2035	42	0.8101574	0.1898426	$2,637,984	0.004637845	$1,813
2036	43	0.8061066	0.1938934	$2,453,626	0.004614656	$1,726
2037	44	0.8020761	0.1979239	$2,278,671	0.004591582	$1,639
2038	45	0.7980657	0.2019343	$2,112,641	0.004568624	$1,553
2039	46	0.7940753	0.2059247	$1,955,079	0.004545781	$1,467
2040	47	0.7901050	0.2098950	$1,805,555	0.004523052	$1,382
2041	48	0.7861544	0.2138456	$1,663,657	0.004500437	$1,299
2042	49	0.7822237	0.2177763	$1,528,998	0.004477935	$1,216
2043	50	0.7783126	0.2216874	$1,401,207	0.004455545	$1,134
2044	51	0.7744210	0.2255790	$1,279,934	0.004433268	$1,054
2045	52	0.7705489	0.2294511	$1,164,848	0.004411101	$975
2046	53	0.7666961	0.2333039	$1,055,631	0.004389046	$898
2047	54	0.7628627	0.2371373	$951,986	0.0043671	$823
2048	55	0.7590484	0.2409516	$853,628	0.004345265	$749
2049	56	0.7552531	0.2447469	$760,286	0.004323539	$677
2050	57	0.7514768	0.2485232	$671,706	0.004301921	$607
2051	58	0.7477195	0.2522805	$587,644	0.004280411	$538
2052	59	0.7439809	0.2560191	$507,870	0.004259009	$471
2053	60	0.7402610	0.2597390	$432,165	0.004237714	$406
2054	61	0.7365597	0.2634403	$360,322	0.004216526	$343
2055	62	0.7328769	0.2671231	$292,143	0.004195443	$281
2056	63	0.7292125	0.2707875	$227,442	0.004174466	$222
2057	64	0.7255664	0.2744336	$166,041	0.004153594	$164
2058	65	0.7219386	0.2780614	$107,773	0.004132826	$107
2059	66	0.7183289	0.2816711	$52,476	0.004112161	$53
						$58,106
					Annual Damages	$3,369

Benefits

27. Benefits are estimated to be annual damages in the without-project conditions minus any residual damages in the with-project alternatives. The benefits claimed are avoided storm damage costs when compared to the existing condition, specifically avoided loss of the lighthouse complex and its associated costs for the preservation of artifacts, prevented local costs forgone for loss of land values, avoided lost visitation benefits to the lighthouse and to the State Park. The project benefits for the three alternatives are summarized in Table 32 below. All benefits are discounted using a 5-$^{3}/_{8}$ percent interest rate and amortized over the 50-year period of analysis. Table 33 summarized the annual cost for the stone revetment alternatives.

Table 32. Benefit Summary (Oct. 2004 P.L., 5.375% discount rate)

Description	Without-Project Damages	Residual Damages - 15yr storm design	Benefits - 15yr storm design	Residual Damages - 73yr storm design	Benefits - 73yr storm design	Residual Damages - 150yr storm design	Benefits - 150yr storm design
Storm Damage Reduction							
Lighthouse Complex	$518,452	$318,655	$199,797	$33,617	$484,835	$15,732	$502,720
Local Costs Forgone	$74,100	$60,402	$13,698	$19,226	$56,520	$12,636	$61,464
Subtotal	$592,600	$379,100	$213,500	$52,800	$541,400	$28,400	$564,200
Recreation							
Lighthouse Visitations	$882,662	$432,527	$450,135	$35,530	$847,132	$15,007	$867,655
Park Visitations	$198,153	$97,100	$101,053	$7,976	$190,177	$3,369	$194,784
Subtotal	$1,080,800	$529,600	$551,200	$43,500	$1,037,300	$18,400	$1,062,400

Table 33. Cost Summary (Oct. 2004 P.L., 5.375% discount rate)

Description	15yr storm design	73yr storm design	150yr storm design
Total First Cost	$5,804,000	$13,722,900	$15,998,900
Interest During Construction	$301,400	$712,700	$1,057,000
Total Investment Cost	$6,105,400	$14,435,600	$17,055,900
Annual Investment Cost	$354,000	$837,000	$988,900
Annual Revetment Maintenance Cost	$170,700	$52,300	$61,500
Total Annual Cost	$524,700	$889,300	$1,050,400

Summary

28. The Planning Guidance Notebook, ER 1105-2-100, 22 April 2000, Chapter 3-4b(4)(a), reads in pertinent part,

"The Corps participates in single purpose projects formulated exclusively for hurricane and storm damage reduction, with economic benefits equal to or exceeding the costs, based solely on damage reduction benefits, or a combination of damage reduction benefits and recreation benefits. Under current policy, recreation must be incidental in the formulation process and may not be more than fifty percent of the total benefits required for justification. If the criterion for federal participation project cost sharing is met, then all recreation benefits are included in the benefit to cost analysis."

29. Federal participation in this recreation benefit generating shore protection project is warranted since the recreation benefits are incidental, and when combined with and limited to an equivalent amount of primary hurricane and storm damage reduction benefits, they produce an economically justified project. The incidental recreation benefits are limited because the storm damage reduction benefits must be at least 50 percent of the total benefits used for project evaluation. Table 34 shows that the 73-year design has the highest net benefits among the three alternatives and is therefore, the National Economic Development (NED) plan. After the NED plan is determined, all recreation benefits are included in the final benefit cost ratio (BCR) because the criterion for federal participation project cost sharing with limited recreation benefits has been met.

Table 34. NED Plan Selection (Oct. 2004 P.L., 5.375% discount rate)

Description	15yr Storm Design	*73yr Storm Design*	150yr Storm Design
Annual Storm Damage Benefits	$213,500	*$541,400*	$564,200
Annual Recreation Benefits	$551,200	*$1,037,300*	$1,062,400
Annual Recreation Benefits Used for Project Justification[6]	$213,500	*$541,400*	$564,200
Total Benefits Used for Project Justification[7]	$427,000	*$1,082,800*	$1,128,400
Annual Costs	$524,700	*$889,300*	$1,050,400
Net Benefits	-$97,700	*$193,500*	$78,000
BCR	0.81	*1.22*	1.07
Total Benefits[8]		*$1,578,700*	
Total Net Benefits		*$689,400*	
Final BCR		*1.78*	

[6] Annual recreation benefits limited to an equivalent amount of annual storm reduction damage benefits.
[7] Sum of annual storm damage reduction benefits and annual recreation benefits used for project justification.
[8] Includes all annual recreation benefits.

OCTOBER 2005

MONTAUK POINT EROSION CONTROL
FEASIBILITY STUDY
MONTAUK POINT, NEW YORK

QUANTITIES AND COST

APPENDIX C

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT

Table of Contents

List of Tables

Table **Page**

List of Attachments

Introduction

General

C-1 This document contains the first costs for the Montauk Point Erosion Control Project. Methods for deriving the costs of the various project elements of the recommended plan are discussed.

C-2 The MCACES summary sheets reflecting feasibility level costs is shown in Attachment C-1 at the end of this support document.

Basis of Estimates

C-3 All estimates are based on October 2004 price levels for labor, materials, equipment, 2000 topographic surveys and beach profiles. Quantities for the alternative plans of improvement have been developed from the detailed plans shown in the Feasibility Report, as well as detailed design data reflected in accompanying support documents.

C-4 The quantities for the alternative plans for the Montauk Point erosion control project were computed as follows and are presented in Table C-1:

TABLE C-1 INITIAL CONSTRUCTION QUANTITIES	ALTERNATIVE 2A 150-year Level of Protection Crest Elev. +30' NGVD	ALTERNATIVE 2B 73-year Level of Protection Crest Elev. +25' NGVD	ALTERNATIVE 2C 15-year Level of Protection Crest Elev. +25' NGVD
Material			
Armor Stone (tons)	57,100	46,700	15,600 *
Armor Stone- Rehandled (tons)	19,100	19,300	1,000
Underlayer (tons)	23,700	18,600	1,000
Bedding Stone (tons)	12,100	11,100	11,500
Excavation (cy)	34,200	32,000	15,000

* Includes construction of cofferdam offshore and reuse in revetment. Alternative cost also includes the disposal of 7,300 tons of unusable existing armor stone to be disposed on site at the structure toe. Also included is 8,000 sf. of temporary exposed bank protection during construction.

Construction Quantity Estimate. The 2000 beach profile survey was used as existing conditions, forming the basis for the initial construction quantity estimate. The design cross-section was superimposed on each of the existing beach profiles. Quantity estimates for the alternative levels of protection appear in Table C-1. A detailed quantity estimate for the selected alternative (73-year level of protection) appears in Table C-2.

Armor Stone Construction Tolerance. Armor stone quantities include an additional 15 inch construction template tolerance.

Quarry Stone Source. Tilcon Quarry, CT

TABLE C-2

SELECTED ALTERNATIVE – QUANTITY ESTIMATE

		ARMOR STONE								UNDERLAYER STONE				BEDDING STONE				EXCAVATION			
		4.5 - 5.0 Ton Armor Units				12.6 Ton Quarrystone Armor				1.3 Ton Quarrystone				100 LB. Quarry Run							
STATION	LENGTH	AREA	AREA	VOLUME	VOLUME	AREA	AREA	VOLUME	VOLUME	AREA	AREA	VOLUME	VOLUME	AREA	AREA	VOLUME	VOLUME	AREA	AREA	VOLUME	VOLUME
		Sq. Ft.	AVG.	Cu. Ft.	Cu. Yards	Sq. Ft.	AVG.	Cu. Ft.	Cu. Yards	Sq. Ft.	AVG.	Cu. Ft.	Cu. Yards	Sq. Ft.	AVG.	Cu. Ft.	Cu. Yards	Sq. Ft.	AVG.	Cu. Ft.	Cu. Yards
0-55		0				0				0				0				0			
	55		213.5	11742.5	434.9		500	27500	1018.5		200.5	11027.5	408.4		118.5	6517.5	241.4		503.5	27692.5	1025.6
0+00		427				1000				401				237				1007			
	100		427	42700	1581.5		979	97900	3625.9		389.5	38950	1442.6		236	23600	874.1		1019	101900	3774.1
1+00		427				958				378				235				1031			
	100		427	42700	1581.5		973.5	97350	3605.6		384.5	38450	1424.1		235	23500	870.4		1094.5	109450	4053.7
2+00		427				989				391				235				1158			
	100		427	42700	1581.5		1043	104300	3863.0		412.5	41250	1527.8		244	24400	903.7		1185	118500	4388.9
3+00		427				1097				434				253				1212			
	100		427	42700	1581.5		1079	107900	3996.3		427	42700	1581.5		253	25300	937.0		1184.5	118450	4387.0
4+00		427				1061				420				253				1157			
	100		427	42700	1581.5		1041.5	104150	3857.4		412	41200	1525.9		247	24700	914.8		1202	120200	4451.9
5+00		427				1022				404				241				1247			
	100		427	42700	1581.5		1028.5	102850	3809.3		406.5	40650	1505.6		244.5	24450	905.6		1207.5	120750	4472.2
6+00		427				1035				409				248				1168			
	100		427	42700	1581.5		1060.5	106050	3927.8		425.5	42550	1575.9		250.5	25050	927.8		1097	109700	4063.0
7+00		427				1086				442				253				1026			
	74		213.5	15799	585.1		543	40182	1488.2		221	16354	605.7		126.5	9361	346.7		513	37962	1406.0
7+74		0				0				0				0				0			
TOTAL	**829**			**326442**	**12090**			**788182**	**29192**			**313132**	**11597**			**186879**	**6921**			**864605**	**32022**
			TONS (cy x 1.6)		**19345**				**46707**				**18556**				**11074**				

Work Breakdown Structure

C-5 The estimate was compiled using MCACES and patterned after the Civil Works Template as a model. The estimate makes use of all six reporting levels available in the following format:

Level 1	Construction Element	One of five major account codes used to estimate the total project cost.
Level 2	Sub-element/Segment	An individual segment of construction activity comprising one or more categories of work or features (cost accounts)
Level 3	Feature	A sub-component of a major type of work (cost accounts)
Level 4-6	Sub-Feature, Bid Item, And Assembly	Increasingly detailed levels of descriptions and estimating dependant on the information and design level developed for the Feasibility Report

Project Description

C-6 The project is located at Montauk Point in Suffolk County, approximately 125 miles east of New York City. The Recommended Plan which is fully described in the Feasibility Report, consists of the construction of a stone revetment section, 840 feet in length, with a crest width of 40’ at elevation +25’ NGVD, 1V:2H side slopes, and 12.6-ton quarry armor stone units extending from the crest down to the embedded toe. Three layers of 4-5 ton armor units are used atop the splash apron. It is assumed that some of these stones from the present structure can be re-used in the proposed revetment. . The bottom of the armor stone layer in the toe is located at a depth of 12’ from the existing bottom. A heavily embedded toe is incorporated to protect against breaking waves and scour at the toe of the structure. Stone sub-layers are specified in accordance with standard design procedures.

Formulation of Project First Costs

C-7 First costs include the charges arising from the construction of the stone revetment, as well as the costs of contingencies, engineering, design, supervision and administration. The detailed estimates include such items as: lands, seawalls/revetment, engineering & design and construction management. Given in Attachment C-1 are the MCACES estimate's title, table of contents, and summary pages for the recommended plan of protection.

C-8 Table C-3 provides first cost estimates for the recommended plan (i.e. stone revetment crest at +25 feet NGVD). Table C-4 provides the Fully Funded Costs for the recommended plan initial construction escalated to the midpoint of construction, January 2009. Tables C-6 and C-7 provide first cost estimates for the other two alternative level of protection plans analyzed.

TABLE C-3- RECOMMENDED PLAN
TOTAL FIRST COST - MONTAUK POINT
EROSION CONTROL FEASIBILITY STUDY (Plan 2B)
October 2004 Price Level

Account Code	Description	QTY	UOM	Unit Price	Amount	% Cont'g	Cont'g Amt	Total
10	Breakwaters & Seawalls							
10.46.01	Mob/Demob	1	LS		$ 522,000	15.00%	$ 78,300	$ 600,300
10.46.02.01	Armor Stone	46,700	Ton	$ 110.68	$ 5,168,700	15.00%	$ 775,300	$ 5,944,000
10.46.02.02	Armor Stone Rehandle	19,300	Ton	$ 58.78	$ 1,134,500	15.00%	$ 170,200	$ 1,304,700
10.46.02.03	Underlayer Stone	18,600	Ton	$ 111.42	$ 2,072,400	15.00%	$ 310,900	$ 2,383,300
10.46.02.04	Bedding Stone	11,000	Ton	$ 94.76	$ 1,042,400	15.00%	$ 156,400	$ 1,198,800
10.46.02.05	Excavation	32,000	CY	$ 16.08	$ 514,400	15.01%	$ 77,200	$ 591,600
10	**TOTAL Breakwaters & Seawalls**				**$ 10,454,400**		**$ 1,568,300**	**$ 12,022,700**
01	**LAND & DAMAGES**				$ 30,000	6.67%	$ 2,000	$ 32,000
30	**PLANNING, ENGINEERING, & DESIGN**				$ 547,800	15.00%	$ 82,200	$ 630,000
31	**CONSTRUCTION MANAGEMENT**				$ 902,800	15.00%	$ 135,400	$ 1,038,200
	TOTAL PROJECT FIRST COST				**$ 11,935,000**		**$ 1,787,900**	**$ 13,722,900**

TABLE C-4

*** * * TOTAL FEDERAL COST-SHARED SUMMARIES * * ***

This Estimate is based on the scope contained in the Feasibility Report

Project: Montauk Point, NY

District: New York
POC: P Harimohan

		Current MCACES Estimate Prepared: February 2005 Effective Pricing Level: October 2004				……………Fully Funded Estimate…………..				
Acct. No.	Feature Description	Cost ($K)	Cont. ($K)	Cont. (%)	Total ($K)	%	Midpoint Date	Cost ($K)	Cont. ($K)	Total ($K)
10	Breakwaters & Seawall	10,454.4	1,568.3	15%	12,022.7	7.78%	Jan-09	11,268.0	1,690.3	12,958.3
	Total	10,454.4	1,568.3		12,022.7			11268.0	1,690.3	12,958.3
01	Lands & Damages	30.0	2.0	7%	32.0	18.42%	Jan-08	35.5	2.4	37.9
30---	Engineering & Design	547.8	82.2	15%	630.0	18.42%	Jan-08	648.7	97.3	746.1
31---	Construction Management	902.8	135.4	15%	1,038.2	20.99%	Jan-09	1,092.3	163.8	1,256.1
	Total Federal Cost Summary	11,935.0	1,787.9		13,722.9			13,044.5	1,953.9	14,998.4

Total Federal Costs (50%)	7,499.2
Total Non-Federal Costs (50%)	7,499.2

NOTE:
Acct 1, 30, 31 escalation using EC11-2-187 dtd 28 Apr 2005 Table A Class 1
Acct 10 escalation using EM1110-2-1304 dated Mar 2005 Table A-1

TABLE C-5

TOTAL FIRST COST - MONTAUK POINT
EROSION CONTROL FEASIBILITY STUDY
ALTERNATIVE 2A
October 2004 Price Level

Account Code	Description	QTY	UOM	Unit Price	Amount	% Cont'g	Cont'g Amt	Total
10	Breakwaters & Seawalls							
10.46.01	Mob/Demob	1	LS		$ 522,000	15.00%	$ 78,300	$ 600,300
10.46.02.01	Armor Stone	57,100	Ton	$ 110.68	$ 6,319,828	15.00%	$ 947,974	$ 7,267,800
10.46.02.02	Armor Stone Rehandle	19,100	Ton	$ 58.78	$ 1,122,698	15.00%	$ 168,405	$ 1,291,100
10.46.02.03	Underlayer Stone	23,700	Ton	$ 111.42	$ 2,640,654	15.00%	$ 396,098	$ 3,036,800
10.46.02.04	Bedding Stone	12,100	Ton	$ 94.76	$ 1,146,596	15.00%	$ 171,989	$ 1,318,600
10.46.02.05	Excavation	34,200	CY	$ 16.08	$ 549,936	15.00%	$ 82,490	$ 632,400
10	**TOTAL Breakwaters & Seawalls**				**$ 12,301,712**		**$ 1,845,257**	**$ 14,147,000**
01	**LAND & DAMAGES**				$ 30,000	6.67%	$ 2,000	$ 32,000
30	**PLANNING, ENGINEERING, & DESIGN**				$ 547,800	15.00%	$ 82,200	$ 630,000
31	**CONSTRUCTION MANAGEMENT**				$ 1,034,700	15.00%	$ 155,200	$ 1,189,900
	TOTAL PROJECT FIRST COST				**$ 13,914,212**		**$ 2,084,657**	**$ 15,998,900**

TABLE C-6

TOTAL FIRST COST - MONTAUK POINT
EROSION CONTROL FEASIBILITY STUDY
ALTERNATIVE 2C
October 2004 Price Level

Account Code	Description	QTY	UOM	Unit Price	Amount	% Cont'g	Cont'g Amt	Total
10	Breakwaters & Seawalls							
10.46.01	Mob/Demob	1	LS		$ 522,000	15.00%	$ 78,300	$ 600,300
10.46.02.01	Armor Stone	15,600	Ton	$ 110.68	$ 1,726,608	15.00%	$ 258,991	$ 1,985,600
10.46.02.02	Armor Stone Rehandle	1,000	Ton	$ 58.78	$ 58,780	15.00%	$ 8,817	$ 67,600
10.46.02.03	Underlayer Stone	1,000	Ton	$ 111.42	$ 111,420	15.00%	$ 16,713	$ 128,100
10.46.02.04	Bedding Stone	11,500	Ton	$ 94.76	$ 1,089,740	15.00%	$ 163,461	$ 1,253,200
10.46.02.05	Excavation	15,000	CY	$ 16.08	$ 241,200	15.00%	$ 36,180	$ 277,400
10.46.02.06	Armor Stone Disposal	7,300	CY	$ 30.00	$ 219,000	15.00%	$ 32,850	$ 251,900
10.46.02.07	Bank Protection	8,000	SF	$ 10.00	$ 80,000	15.00%	$ 12,000	$ 92,000
10	**TOTAL Breakwaters & Seawalls**				**$ 4,048,748**		**$ 607,312**	**$ 4,656,100**
01	**LAND & DAMAGES**				$ 30,000	6.67%	$ 2,000	$ 32,000
30	**PLANNING, ENGINEERING, & DESIGN**				$ 547,800	15.00%	$ 82,200	$ 630,000
31	**CONSTRUCTION MANAGEMENT**				$ 422,500	15.00%	$ 63,400	$ 485,900
	TOTAL PROJECT FIRST COST				**$ 5,049,048**		**$ 754,912**	**$ 5,804,000**

C-9 Unit Costs. Unit costs for material and equipment were developed and based on: the Unit Price Book (UPB) associated with MCACES; current New York DOT and N.Y. District bid unit costs (adjusted appropriately for the size of the project, construction period, inflation and profit), actual costs and productions on projects and construction similar in nature; contact with manufacturers, dealers, distributors, and material suppliers in the vicinity of the proposed project; current labor rates for the northern Long Island area and cost estimating judgement based on experience.

C-10 Lump Sum Items. Based on experience, certain items of cost such as mobilization and demobilization were assigned a "lump sum" cost. These items were estimated in this way due to the multiplicity of activities required to accomplish each of these items.

C-11 Market Research. To accurately estimate unit prices for individual work items, manufacturers, distributors, vendors and suppliers, and state agencies were contacted for price information on materials and types of construction. When more than one source of information or price quote was obtained for a single item, the average cost was calculated and used in the MCACES estimate.

C-12 Labor Rates. The labor rates for the estimate were taken from the prevailing Davis-Bacon wage rates for the State of New York for building, heavy, and highway construction as detailed in General Decision Number NY020013. The wage rate data was received in detail, listed by counties, and is current as of October 2004. Wage rates were reviewed and averages were calculated for use in the development for each trade listing of the MCACES model. These average labor and fringe benefit costs were input into the MCACES system in the labor rates database.

C-13 Contingencies. As stated in ER 1110-2-1302 (31 Mar 94), the goal in contingency development is to identify the uncertainty associated with an item of work or task, forecast the risk/cost relationship, and assign a value to this task that would limit the cost risk to an acceptable degree of confidence. Consideration has been given to the level of detail available at the current stage of planning for which this cost estimate has been prepared.

C-14 Based on the current level of design development for the project, the following general contingency factors (%) were used.

- Seawall and revetments 15 % - Cost based on final design but subject to differing condition of the existing revetment at the time of construction.

Estimates of Project Features

C-15 Seawalls and Revetments. Seawalls and revetments represent the only construction feature of the project.

C-16 The estimate for the construction of the revetment was approached from the viewpoint of heavy stonework and earthwork operations characterized by large cranes, loaders and haul trucks. Approximately 840 linear feet of revetment will be constructed along the Montauk Point shoreline.

C-17 Productivity considerations were based on the relative configuration of the existing revetment and bank, wave and tide conditions, stone size, placement criteria, distance of truck-delivered stone material from off-site and on-site stockpiles, access, haul roads, entrances, and construction easements.

C-18 A construction access berm will be adjacent to the slope of the existing stone revetment by temporarily relocating the existing stone. This construction will be completed on both the Northerly and Southerly ends of the revetment. The elevation of the access berms will be +8' NGVD. An additional temporary road is proposed on the Northerly end of the revetment to provide access to the project site.

C-19 Two separate crews are anticipated to perform the work. One crew will operate on the Northerly end and the other operating on the Southerly end of the revetment. Each crew should have one large crane for excavation and stone placement.

C-20 The excavation and stone placement construction will be conducted from the construction berm at el. +8’ NGVD. No access via water is proposed. Excavation and stone placement will be performed by the same crew since there is not enough room on the construction berm for two crews to work at one location concurrently.

C-21 Ten (10) 38-ton trucks with 16-23.5 cy trailers are anticipated to be used for hauling the bedding, underlayer and armor stone from the quarry to the project site. Two (2) 25-ton (16-19 cy) off-highway trucks are proposed to deliver stone from the stockpile area to the work area.

C-22 Excavated bottom material from the revetment toe area will be transported directly to a Dredge Material Placement (DMP) site on-site within the grounds of Montauk State Park using the 25-ton off-highway dump trucks. The exact site of the DMP area is to be determined.

C-23 It is assumed that about 19,300 tons of existing revetment stone will be re-used in the new revetment. Any unusable stone from the existing revetment will be placed overlying the restored ocean bottom after the buried toe is constructed.

C-24 Three (3) deep draft offshore barges are proposed to provide wave protection to the work area. The cost for the barges is included within Preparatory Work under Mob/Demob.

Estimates of Additional Costs

C-25 Planning, Engineering and Design. Costs were developed for all activities associated with the pre-construction, planning, engineering, and design effort. These costs include the preparation of a Design Documentation Report, plans and specifications for the construction contract and engineering support through project construction.

C-26 Construction Management. Costs were developed for all construction management activities from pre-award requirements through final contract closeout.

C-27 Interest During Construction. Interest during construction (IDC) is the cost of construction money invested before the beginning of the period of economic analysis and before the accumulation of benefits by the project. IDC costs have been added to the project cost to determine the total investment costs. Average annual costs were determined based on investment costs which include IDC. Interest during construction was considered for a 24 month construction period at 5.375%.

C-28 Planning Guidance Notebook (EP 1105-2-45, Paragraph 2-6, page 2-2) states that costs incurred during the pre-construction and construction period should be increased by adding compound interest at the applicable project discount rate from the date the expenditures are made to the beginning of the period of analysis (base year). For the purposes of this study, these expenditures were assumed to occur in monthly increments.

Annual Charges

C-29 Period of Analysis. It is estimated that the stone revetment would have a useful life expectancy of 50 years.

Interest and Amortization

C-30 The interest rate used in converting investment costs to equivalent annual costs is the rate set by the

Water Resources Council for the evaluation of Federal Government water resources projects. This rate has been set at 5.375 % for FY2005.

C-31 Amortization is the financial or economic process of recovering an investment in a project. The amortization period is the period of time assumed or selected for economic recovery of the net investment in a project. When combined, interest and amortization become the capital recovery factor which, when applied to project costs, will result in the annual cost of the project investment.

Maintenance

C-32 For the purposes of the Feasibility Study, the annual cost of maintenance of the alternatives considered is estimated to be 0.5% of the total direct (without contingency) first cost of construction. This maintenance is associated with 0% - 5% damage levels up to the design storm.

C-38 Future impacts on annual maintenance costs due to sea level rise are considered to be minor given the predicted rate of sea level rise of 0.014 feet per year. For example, at this rate at the mid-point of the project life, 25 years, the rise in water level would be 0.35 feet. Without sea level rise, the proposed design wave for Alternative 2B is $H_{73\ Yr.}$ = 13.4' calculated using Figure 7-4 (SPM 1984) and $DSWL_{73\ Yr}$ = +10.94' NGVD which results in a design armor stone weight of 12.6 tons. Adding the sea level rise to the $DSWL_{73\ Yr}$ = +10.94' NGVD would result in $DSWL_{73\ Yr + Sea\ Level\ Rise}$ = 11.3' NGVD which would result in a design wave for this structure, $H_{73\ Yr. + Sea\ Level\ Rise}$ = 13.8'. This design wave results in a design armor stone weight of 13.8 tons or a 9.5% increase in armor stone weight due to sea level rise through the mid-point of the project life.

C-39 Given the small predicted annual increase in sea level rise in the project area, and the standard construction specification for the armor stone to range from 0.75 W to 1.25 W (W = 12.6 tons) with about 50 percent of the individual stones weighing more than W, increasing the armor stone weight to account for future sea level rise is not considered to be warranted.

C-40 <u>Annualized Maintenance Costs.</u> Annualized revetment maintenance costs for Plans 2A and 2B are based on 0.5% of the direct first cost based on experience with Corps designed coastal structures. This maintenance will be accomplished from the berm. For Plan 2C, annualized maintenance costs include construction of an offshore rubble mound stone cofferdam to accomplish repairs since no berm is available for maintenance operations. The first cost and annualized cost for this cofferdam is shown in Table C-8.

Table C-8
Annualized Maintenance Cost (Oct. 04 P.L.)
for Montauk 15-Year Plan Construction Berm
(Estimated to be performed every 10 years)

**Cost breakout of construction berm:

600 lf of stone cofferdam

consists of: 7,800 tons of 6-10 ton armor
4,000 tons of bedding

To install and remove:

7,800	tons @	$ 141.66	per ton	$	1,105,000
4,000	tons @	$ 120.00	per ton	$	480,000
Subtotal				$	1,585,000
Contingency	20%			$	317,000
Subtotal				$	1,902,000
S&A				$	100,000
Subtotal				$	2,000,000

Interest Rate	0.05375
Life (years)	50
Capital Recovery Factor	0.05798
Frequency (years)	10
Present Worth Factor	0.5924

Year	Future Worth $	PW Factor	Present Worth $
10	$ 2,000,000	0.5924	$ 1,184,825
20	$ 2,000,000	0.3510	$ 701,905
30	$ 2,000,000	0.2079	$ 415,817
40	$ 2,000,000	0.1232	$ 246,335
		Sum	$ 2,548,882

Interest Rate	5.375%
N Years	50
Capital Recovery Factor	0.057980614

Annualized Cost of Plan 2C Construction Berm $ 148,000

Annual Costs

C-41 Annual Costs. The annual charges include the annualized first cost with interest during construction, and annualized operations and maintenance costs of the revetment within design storm condition. Annual project costs for the recommended plan (i. e. 73-year level of protection, crest elevation +25' NGVD) are presented in Table C-9. Annual costs for two alternative levels of protection are presented in Tables C10 and C-11.

TABLE C-9
TOTAL ANNUAL COST – MONTAUK POINT
EROSION CONTROL FEASIBILITY STUDY
RECOMMENDED PLAN

Total First Cost Interest During Construction (a)	$13,722,900 $712,700
Total Investment Cost	$14,435,600
Annualized Investment Cost (b)	$837,000
Annual Revetment Maintenance (c)	$52,300
Total Annual Cost	$889,300

(a) i = 5.375 % for 24 mo. construction period
(b) I = 5.375 % for 50 yr. period of analysis
(c) i = 0.5% of Direct First Cost (excluding E&D, S&A, and contingency)

TABLE C-10
TOTAL ANNUAL COST – MONTAUK POINT
EROSION CONTROL FEASIBILITY STUDY
ALTERNATIVE 2A

Total First Cost Interest During Construction (a)	$15,998,900 $1,057,000
Total Investment Cost	$17,055,900
Annualized Investment Cost (b) Annual Revetment Maintenance (c,d)	$988,900 $61,500
Total Annual Cost	$1,050,400

(a) i = 5.375 % for 30 mo. construction period
(b) I = 5.375 % for 50 yr. period of analysis
(c) i = 0.5% of Direct First Cost (excluding E&D, S&A, and contingency)
(d) Increase of annual maintenance with increases in level of protection is due to increase of maintenance of rubblemound structures as total quantity of stone increases, and need for larger equipment and slower production rate associated with increase in armor unit weight.

TABLE C-11
TOTAL ANNUAL COST – MONTAUK POINT
EROSION CONTROL FEASIBILITY STUDY
ALTERNATIVE 2C

Total First Cost Interest During Construction (a)	$5,804,000 $301,400
Total Investment Cost	$6,105,400
Annualized Investment Cost (b) Annual Revetment Maintenance (c)	$354,000 $170,700
Total Annual Cost	$524,700

(a) i = 5.375 % for 24 mo. construction period
(b) I = 5.375% for 50 yr. period of analysis
(c) Includes normal annualized maintenance @ 0.5% of the direct first cost excluding E&D, S&A, and contingency ($22,700), plus the cost to construct an offshore temporary construction berm from which to perform the repairs ($148,000) (see Table C-8)

Cost Sharing Responsibilities General

C-42 The basic requirements for the Federal and non-Federal sharing of responsibilities in the construction, operation, and maintenance of Federal water resources projects are set forth in the Water Resources Development Act (WRDA) of 1986 (PL 99-662).

Cost Apportionment

C-43 The Water Resources Development Act of 1986, Section 103, which sets forth cost sharing for hurricane and storm damage reduction projects, states that non-Federal interests must operate, maintain, and rehabilitate the project; must provide lands, easements, rights-of-way, relocations, and disposal areas (LERRD). The non-Federal -share of the project cost is limited to 50% of the first costs.

C-44 The Federal share of the project's total first cost is $6,845,450. This represents 50 % of the total.

C-45 The non-Federal share of the estimated total first cost of the proposed project is $6,845,450. The non-Federal cost typically consists of a number of components including lands, easements, rights-of-way, relocations, and a cash contribution. Since no land acquisition or relocation is involved, the non-Federal share is all cash contribution. The non-Federal share represents 50% of the total project first costs. A breakdown of the Federal and non-Federal cost share is shown in Table C-12 - Cost Apportionment.

Table C-12
Montauk Point
Cost Apportionment
Oct 2004 P.L.

Cost Sharing	**Federal Share**	**Non-Federal Share**	**TOTAL**
Cash Contribution	$ 6,861,450	$ 6,861,450	$ 13,722,900
Total First Cost	$ 6,861,450	$ 6,861,450	$ 13,722,900
Annual Revetment Maintenance		$ 52,300	$ 52,300

MONTAUK POINT EROSION CONTROL
FEASIBILITY STUDY
MONTAUK POINT, NEW YORK

ATTACHMENT C1

MCACES Recommended Plan
(Alternative 2B)

APPENDIX D: REAL ESTATE PLAN

U.S. Army Corps of Engineers

REAL ESTATE PLAN

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

Prepared by:

Department of the Army
New York District
Corps of Engineers

Real Estate Division

October 2005

REAL ESTATE PLAN

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

TABLE OF CONTENTS

Exhibits

Maps

<u>Preamble</u>

A. **Introduction**: Montauk Point is situated on the extreme eastern end of the south fork of Long Island, approximately 125 miles east of New York City. The historic Montauk Point Lighthouse Complex sits on a high bluff approximately 70 feet above Mean Sea Level (MSL). The Montauk Point Light Station was authorized for construction in 1792, and is included in the *National Register of Historic Places*. When the light was completed it was located 300 feet from the edge of the cliff. Presently the lighthouse is less than 120 feet from the edge of the bluff and other major structures are now within 50 feet of the bluff edge. The critical area of study consists of the bluff from the southwest side of the Point to the northwest side of the Point, covering about 900 feet of shoreline. The bluff and beach along this entire area are considered to be critical elements of the stability of the lighthouse. Erosion control structures are required to protect the bluff faces from the forces of oncoming waves. The larger area of concern consists of 2,300 feet of shoreline, extending from the pivotal point of the adjacent bluff to the south to a beach area to the north. The entire area of concern must be considered in order to prevent potential adverse impacts from this Project.

B. **Authorization**: The study is being conducted under the authority of the following resolution, adopted by the Committee on Environment and Public Works of the U.S. Senate on May 15, 1991:

> *"Resolved by the Committee on Environment and Public Works of the United States Senate, that the Secretary of the Army is hereby requested to review the report of the Chief of Engineers on Fire Island to Montauk Point, New York, published as House Document Number 86-425, 86th Congress, 2nd session, and other pertinent reports, to determine whether modifications of the recommendations contained therein are advisable at the present time, with a view to preserving, restoring, and protecting Montauk Point and vicinity, including the historic Montauk Lighthouse and associated facilities, from erosion, environmental degradation, and coastal storm damage."*

C. **Designation**: Montauk Point, New York Storm Damage Reduction Project - Feasibility Study (the "Project")

D. **Location**: The study area is situated in the Village of Montauk in the Town of East Hampton, Suffolk County, New York, between the Atlantic Ocean and Block Island Sound, at the easternmost end of the south fork of Long Island and includes the historic Montauk Point Lighthouse Complex.

E. **Non-Federal Sponsor**: The non-Federal Sponsor is the New York State Department of Environmental Conservation ("NYSDEC," or "the State"). The Project, if approved, will be cost-shared at a ratio of 50% Federal and 50% non-Federal.

1. **Statement of Purpose**: The purpose of this Real Estate Plan is to present the overall plan describing the minimum real estate requirements for the Montauk Point, New York Storm Damage Reduction Project.

This Real Estate Plan is tentative in nature; both the final real property acquisition lines and costs are subject to change after approval of the Decision Document to which this Plan is appended.

2. **Project Purpose and Features**:

A. Purpose: Because existing shore protection measures (somewhat similar to a failed revetment installed in 1946) were not designed to withstand significant storm events over a substantial duration (e.g. lack of a buried toe, inadequate stone size, and insufficient overtopping protection), it is expected that the revetment now in place will fail in the foreseeable future.

Recent efforts, including terracing, vegetation and improved revetment construction, have decreased the erosion rate. Repeated storm effects will continue to cause erosion at the ends of the structure, and the eventual compromise of the revetment and upper bluff areas. This, in turn, is expected to result in the eventual loss of the lighthouse and its adjacent structures if no corrective action is taken.

B. Plan of Improvement: The selected plan for protection of Montauk Point and the lighthouse complex and bluff is the construction of a stone revetment with a crest width of 40-feet at elevation +25 feet NGVD, 1V:2H side slopes, and 12.6-ton quarrystone armor units extending from the crest down to the embedded toe. A heavily embedded toe is incorporated to protect against breaking waves and scour at the toe of the structure.

C. Required Lands, Easements, Rights-of-Way, Relocations and Disposal Areas (LERRD): The construction of the new revetment will require three (3) tracts and two (2) individual affected ownerships, namely the Montauk Historical Society (a Not-for-Profit educational institution which administers the Montauk Lighthouse Museum and which obtained title to same via a quitclaim deed from the United States of America dated 18 September 1998 (one tract), and the State of New York (two tracts). The two State-owned tracts are along the shoreline at the base of the cliff, on the Atlantic Ocean and Block Island Sound, adjacent to either side of the Montauk Historical Society property.

Approximately **1.81 acres** of land is required for the revetment. In addition, approximately **2.33 acres** will be required for two (2) Temporary Work Area adjacent to the revetment (1.43 acres south of the revetment ("Staging Area #1) and 0.90 acre north of the revetment ("Staging Area #2")). Approximately **1.37 acres** will be required for the two Temporary Access Roads, one along Block Island Sound to the north, the other to the south near the Atlantic oceanfront. The total Project requirement is approximately **5.51 acres**. (*See* ***Map 1*** and ***Figure 1***.) Access to the Project site will be via existing State roads (Montauk Highway) and local interior roads on either Sponsor-owned or Montauk Historical Society lands, including portions of the Temporary Work Areas and Roads discussed above. The Sponsor will be responsible for obtaining the required real estate interests.

Although the location of the temporary access roads for construction is currently fixed and defined, future maintenance or repair work *may* require different locations for temporary access roads, due to environmental or similar considerations, including erosion control, but such temporary access roads would be situated on land owned either by the Non-Federal Sponsor or the Montauk Historical Society.

However, if at some time in the future the Non-Federal Sponsor intends to sell or otherwise convey the lands upon which the existing (or planned) access roads are situated, it would have to include a "reservation" providing for such access in any deed of conveyance or similar instrument.

The Project is not expected to require any facility or utility relocations, nor any relocation of displaced persons, residences, businesses or farms under the provisions of Public Law 91-646 (*See* **Paragraph 11** hereof, "PL 91-646 Uniform Relocation Assistance"). Similarly the Project does not require acquisition of real property interests for borrow areas, nor will disposal areas will be required for any purpose.

A summary of the acreage needed for the Project and the uses thereof is as follows:

Interest:	Acreage
Perpetual non-Standard Revetment Easement	1.81
Temporary Work Area Easements:	3.70
Total:	**5.51 acres**

D. Appraisal Information

(i) Highest and Best Use:

The land required for the construction of the revetment is inundated by the Atlantic Ocean at high tide and its highest and best use is "recreational." Insofar as the proposed improvement will protect the Montauk Point Historical Society's upland improvements (i.e., the Lighthouse itself and appurtenant buildings and improvements) as well as the cliff upon which these improvements are sited, the value of the required easement for the revetment and associated temporary work area easement is considered to be subject to an "offsetting benefit" that is greater than the value of the easements themselves.

(ii) Real Estate Costs:

A summary of real estate costs, using a November 2002 valuation (Gross Appraisal) is as follows:

Lands and Damages:

	Acres	$/Acre (fee)	$/Acre (easement)	Est. Cost
Permanent Easements:	1.81	(nominal)	(nominal)	$ 0
Temporary Easements:	3.70	(nominal)	(nominal)	$ 0
TOTAL:	**5.51**			**$ 0**

Administrative Costs:

Planning:	$ 20,000
Incidental Acquisition Costs: (includes mapping & survey, title evidence, tract appraisals, negotiations & closings)	$ 10,000
TOTAL, Administrative Costs:	$ 30,000
Contingencies: (20% of Lands & Damages and Admin costs, *Excluding* Planning costs):	$ 2,000
GRAND TOTAL, Real Estate Costs:	**$ 32,000**

3. **Non-Federal Sponsor Owned Lands**: The non-Federal Sponsor (the State of New York) owns approximately one-third (1/3) of the 1.81 acres required for the perpetual Revetment Easement, as well as unpaved roads thereon that will provide access to the Revetment work area ("Temporary Work Area Easements"). Further, any construction, operation, maintenance, repair or rehabilitation activities seaward of the Mean High Water Line, will be performed in waters of the State of New York. The Sponsor's interests are available for Project purposes.

The balance of the required easement areas is owned in fee by the Montauk Historical Society, a not-for-profit public educational corporation chartered for this purpose by the State of New York. Montauk Historical Society supports the Project and has agreed to make the required easement areas available for Project purposes.

4. **Estates**: There are two estates, one Standard and one non-Standard, to be obtained by the non-Federal Sponsor: perpetual **Revetment Easement** ("non-Standard estate") and **Temporary Work Area Easement** (4 years' duration) ("Standard Estate" No. 15). The complete text of these estates is included in **Exhibit "A."**

The proposed non-standard perpetual Revetment Easement is similar to a standard Flood Protection Levee Easement (**Standard Estate No. 9**), with the words "flood protection levee" replaced by the words "stone revetment."

5. **Existing Federal Projects**: The following Projects are in the vicinity of the subject Project:

1. Fire Island Inlet to Montauk Point, New York Hurricane Protection and Beach Erosion Control Project
2. Lake Montauk Harbor Navigation Improvement and Environmental Restoration Project.

Neither of these two projects affects the subject Montauk Point Project, nor are any lands required for these two projects required for the subject Project, and vice versa.

6. **Federally-Owned Lands**: There are presently no Federal Government owned lands in the Project area.

7. **Navigational Servitude**: Insofar as this Project is for storm damage reduction purposes, the Government will not invoke its rights of Navigational Servitude. Any construction, operation or maintenance activities seaward of the Mean High Water Line, however, will be performed waters of the State of New York, the Project's Non-Federal Sponsor.

8. **Project Maps**: Project Maps are attached hereto. **Map 1** depicts the Project features (Revetment (Permanent Easement) area, Staging areas and temporary Access Roads. **Figure 1** is an aerial photograph depicting the general Project area, as well as the two Project access roads.

9. **Induced Flooding**: No induced flooding is anticipated as a result of this Project.

10. **Baseline Cost Estimate**: A Baseline Cost Estimate in M/CACES Format is attached hereto as **Exhibit "B."**

Under the doctrine of "offsetting benefits" as applied to the construction of a stone revetment to protect the underlying fee owners' upland and improvements (i.e., the Montauk Point Lighthouse Complex and the adjacent State-owned lands) the value of the easement estates to be obtained and the land to be provided directly by the Sponsor is estimated to be **Zero ($0) dollars.** The administrative cost of acquisition is estimated to be approximately **Ten Thousand ($10,000) dollars.** Insofar as Montauk Historical Society, the landowner of the single easement tract to be acquired, holds title to its land under a Quitclaim Deed from the United States of America and is a "willing seller," *no condemnations* are anticipated.

11. **Compliance with Public Law 91-646**: No residences, businesses or farms will be displaced as a result of the construction, operation or maintenance of the Project. Accordingly, no relocation assistance under the provisions of PL91-646 will be required.

12. **Mineral and Timber Activities**: There are no present or anticipated mineral activities or timber harvesting in the Project area and vicinity.

13. **Assessment of the Non-Federal Sponsor's Land Acquisition Experience and Ability**: An Assessment of the non-Federal Sponsor's Real Estate Acquisition Capability is attached hereto as **Exhibit "C."** The Sponsor is considered to be "fully capable."

14. **Zoning**: Application or enactment of zoning ordinances is **not** anticipated for the Project.

15. **Acquisition Schedules**: A schedule of acquisition by the non-Federal Sponsor is attached hereto as **Exhibit "D."** The schedule assumes a Project Cooperation Agreement will be signed in **January 2007**, and forecasts Certification of Project LERRD in **June 2007**.

16. **Facility/Utility Relocations**: The Project will require no Facility or Utility relocations.

17. **Hazardous, Toxic or Radiological Waste ("HTRW")**: As indicated in Paragraph 3.10 of the Project's Preliminary Draft Environmental Impact Statement ("Preliminary DEIS"), there are no known contaminants or HTRW problems associated with the LER required for construction, operation and maintenance of the Project.

18. **Project Support**: The affected underlying fee owners (Montauk Historical Society, and the non-Federal Sponsor, the State of New York), local County and Town officials, and other residents in the Project area, are supportive of this Project.

19. **Notification to Non-Federal Sponsor**: Based on its past sponsorship of other Corps water resource (Civil Works) projects and ongoing discussions during the Project's Feasibility phase, the non-Federal Sponsor is aware of the risks of acquiring LER required for the Project prior to the signing of the Project Cooperation Agreement ("PCA"), and of the other requirements of PL91-646. In accordance with Paragraph 12-31 of Chapter 5 of the Corps of Engineers Real Estate Handbook, ER 405-1-12, formal written notification of the risks of such acquisition, and of the requirement to document expenses associated with acquiring and providing Project LERRD, and of the requirements of PL91-646, will be forwarded to the non-Federal Sponsor during the Project's Preliminary Engineering and Design ("PED") phase.

20. **Historical Sites**: The Montauk Light Station (comprising the Montauk Point Lighthouse and its outbuildings, all of which will be protected by the Project) is listed on the *National Register of Historic Places.*

21. **Other Issues**:

A. Aside from the Montauk Point Lighthouse, a *National Register of Historic Places*-listed structure, and the surrounding support structures (all eligible for the *National Register*), at this time no known historically-significant artifacts have been uncovered in the area of the proposed revetment construction and access areas.

B. There are no known existing encumbrances (i.e. easements, rights-of-way, etc.).

22. **Recommendations**:

A. It is recommended that the "Non-Standard" Perpetual Revetment Easement proposed for use for this Project be approved by HQ, USACE.

B. This report has been prepared in accordance with the Corps of Engineers Regulation ER 405-1-12. It is recommended that this report be approved.

for Noreen D. Dresser
Chief, Real Estate Division

REAL ESTATE PLAN

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

EXHIBIT "A" - ESTATES

Estates

Revetment Easement (non-Standard Estate): a perpetual and assignable right and easement in (the land described in Schedule A) (Tracts Nos. _______, ______ and ______) to construct, maintain, repair, operate, replace and patrol a stone revetment, including all appurtenances thereto; reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

STANDARD ESTATE #15
TEMPORARY WORK AREA EASEMENT: a temporary easement and right-of-way in, over and across the land described in Schedule A (Tract No. ___) for a period not to exceed forty-eight (48) months, beginning with the date of possession of the land is granted to the United States, for use by the United States, its representatives, agents and contractors as a work area including the right to move, store, and remove equipment and supplies and also to erect and remove temporary structures.

REAL ESTATE PLAN

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

EXHIBIT "B" – BASELINE COST ESTIMATES in M/CACES Format

Exhibit B - Chart of Accounts
for
Montauk Point, New York Storm Damage Reduction Project

	(Prepared by CENAN-RE-M March 2003)			
	TOTAL PROJECT COSTS	non-Federal	Federal	Project Cost
01	LANDS AND DAMAGES	$15,000	$15,000	30,000
	Contingencies (20%) (Excludes Planning)			2,000
	TOTAL, Lands and Damages			32,000
	(Rounded to):			**32,000**
01A	**PROJECT PLANNING**	**9,000**	**11,000**	
01A10	REAL ESTATE SUPPLEMENT/PLAN	9,000	9,000	
01A20	PRELIMINARY RE ACQUISITION MAPS		2,000	
01A30	PHYSICAL TAKINGS ANALYSIS			
01A40	PRELIMINARY ATTORNEY'S OPINION OF COMPENSABILITY			
01A50	ALL OTHER RE ANALYSES/DOCUMENTS			
01B	**ACQUISITIONS**	**3,000**	**1,000**	
01B10	BY GOVERNMENT			
01B20	BY LOCAL SPONSOR (LS)	3,000		
01B30	BY GOVT ON BEHALF OF LS			
01B40	REVIEW OF LS		1,000	
01C	**CONDEMNATIONS**	**0**	**0**	
01C10	BY GOVERNMENT			
01C20	BY LS	0		
01C30	BY GOVT ON BEHALF OF LS			
01C40	REVIEW OF LS		0	
01D	**INLEASING**	**0**	**0**	
01D10	BY GOVERNMENT			
01D20	BY LS			
01D30	BY GOVT ON BEHALF OF LS			
01D40	REVIEW OF LS			
01E	**APPRAISAL**	**3,000**	**1,000**	
01E10	BY GOVT (IN HOUSE)			
01E20	BY GOVT (CONTRACT)			
01E30	BY LS	3,000		
01E40	BY GOVT ON BEHALF OF LS			
01E50	REVIEW OF LS		1,000	
01F	**PL 91-646 ASSISTANCE**	**0**	**0**	
01F10	BY GOVERNMENT			
01F20	BY LS	0		
01F30	BY GOVT ON BEHALF OF LS			
01F40	REVIEW OF LS		0	

Exhibit B - Chart of Accounts
for
Montauk Point, New York Storm Damage Reduction Project
(Prepared by CENAN-RE-M March 2003)

01G	**TEMPORARY PERMITS/LICENSES/RIGHTS-OF-ENTRY**	**0**	**0**	
01G10	BY GOVERNMENT			
01G20	BY LS	0		
01G30	BY GOVT ON BEHALF OF LS			
01G40	REVIEW OF LS		0	
01G50	OTHER			
01G60	DAMAGE CLAIMS			
01H	**AUDITS**	**0**	**0**	
01H10	BY GOVERNMENT			
01H20	BY LS			
01H30	BY GOVT ON BEHALF OF LS			
01H40	REVIEW OF LS			
01J	**ENCROACHMENTS AND TRESPASS**	**0**	**0**	
01J10	BY GOVERNMENT			
01J20	BY LS			
01J30	BY GOVT ON BEHALF OF LS			
01J40	REVIEW OF LS			
01K	**DISPOSALS**	**0**	**0**	
01K10	BY GOVERNMENT			
01K20	BY LS			
01K30	BY GOVT ON BEHALF OF LS			
01K40	REVIEW OF LS			
01N00	**FACILITY/UTILITY RELOCATIONS**	**0**	**0**	
01Q00	**RESERVED FOR FUTURE HQUSACE USE**	**0**	**0**	
01R	**REAL ESTATE PAYMENTS**	**0**	**1,000**	
01R1	**LAND PAYMENTS**	0	0	
01R1A	BY GOVERNMENT			
01R1B	BY LS	0		
01R1C	BY GOVT ON BEHALF OF LS			
01R1D	REVIEW OF LS		0	
01R2	**PL 91-646 ASSISTANCE PAYMENTS**	0	0	
01R2A	BY GOVERNMENT			
01R2B	BY LS	0		
01R2C	BY GOVT ON BEHALF OF LS			
01R2D	REVIEW OF LS		0	

Exhibit B - Chart of Accounts
for
Montauk Point, New York Storm Damage Reduction Project
(Prepared by CENAN-RE-M March 2003)

01R3	**DAMAGE PAYMENTS**	0	0	
01R3A	BY GOVERNMENT			
01R3B	BY LS			
01R3C	BY GOVT ON BEHALF OF LS			
01R3D	REVIEW OF LS		0	
01R9	**OTHER**			
01T	**LERRD CREDITING**	**0**	**1,000**	
01T10	LAND PAYMENTS		0	
01T20	ADMINISTRATIVE COSTS		1,000	
01T30	PL 91-646 ASSISTANCE		0	
01T40	ALL OTHER			

REAL ESTATE PLAN

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

EXHIBIT “C” – Assessment of Non-Federal Sponsor’s Real Estate Acquisition Capability

Montauk Point, New York Storm Damage Reduction Project
Assessment of Non-Federal Sponsor's Real Estate Acquisition Capability

I. Legal Authority:

a. Does the sponsor have legal authority to acquire and hold title to real property for project purposes? *YES*
b. Does the sponsor have the power of eminent domain for this project? *YES*
c. Does the sponsor have "quick-take" authority for this project? *YES*
d. Are any of the lands/interests in the land required for the project located outside the sponsor's political boundary? *NO*
e. Are any of the lands/interests in the land required for the project owned by an entity whose property the sponsor cannot condemn? *NO*

II. Human Resources Requirements:

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? *NO*
b. If the answer to IIa is YES, has a reasonable plan been developed to provide such training? *N/A*
c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? *YES*
d. Is the sponsor's projected in-house staffing level sufficient considering its other work load, if any, and the project schedule? *YES*
e. Can the sponsor obtain contractor support, if required, in a timely fashion? *YES*
f. Will the sponsor likely request USACE assistance in acquiring real estate? *NO*

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? *YES*
b. Has the sponsor approved the project/real estate schedule / milestones? *YES*

Exhibit "C"

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? *YES*
b. With regard to this project, the sponsor is anticipated to be: Fully Capable.

V. Coordination:

a. Has this assessment been coordinated with the sponsor? *YES*
b. Does the sponsor concur with this assessment? *YES*

Prepared by:

Stanley H. Nuremburg,
Realty Specialist

Reviewed and Approved by:

Noreen Dean Dresser
Chief, Real Estate Division

Exhibit "C"

REAL ESTATE PLAN

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

EXHIBIT "D" – Real Estate Acquisition Milestones and Schedules

Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

Exhibit D – Schedule of Real Estate Acquisition

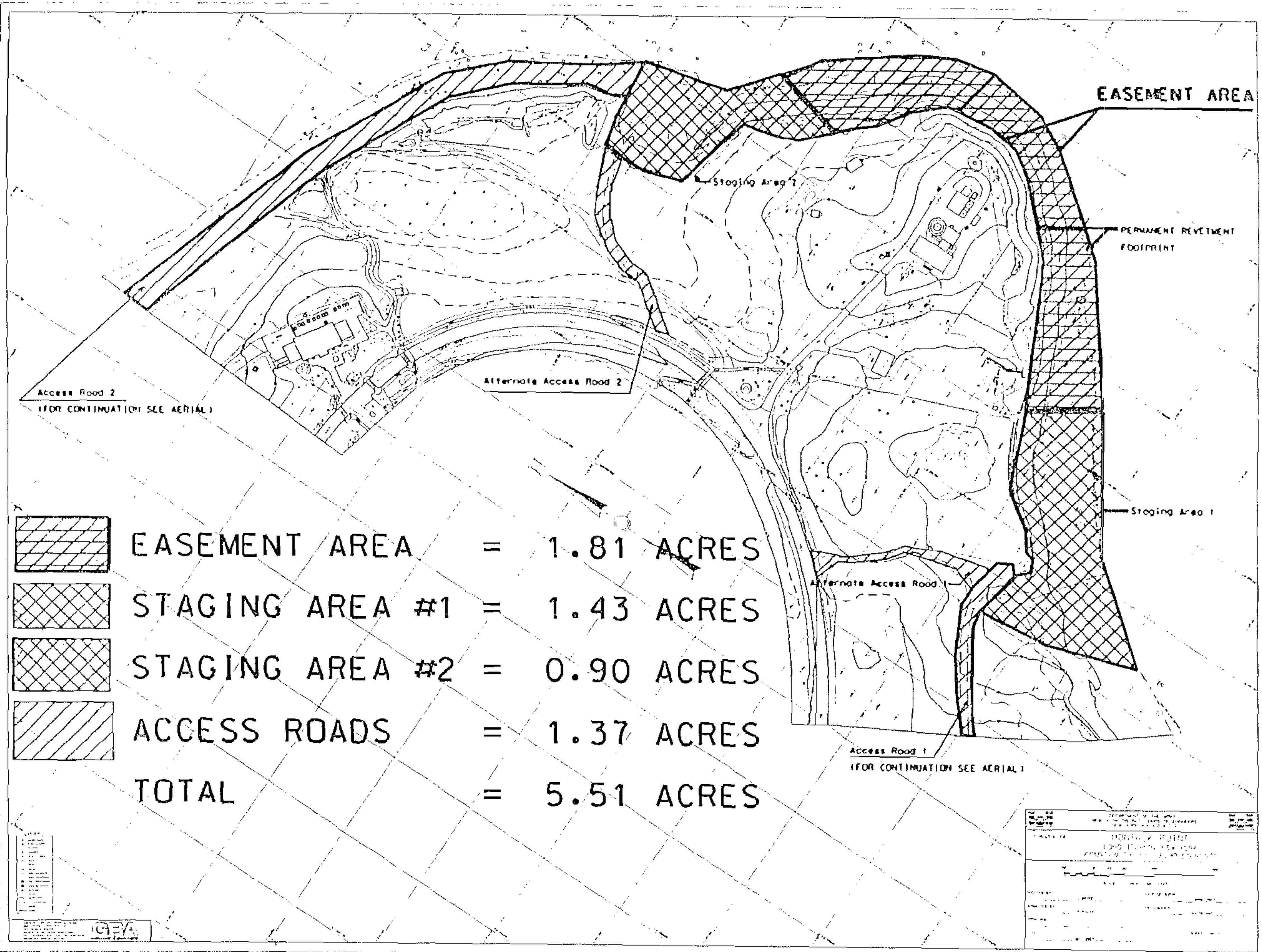
ID	Task Name	Start	Finish
1	Start RE Acquisition	31 Jan 07	15 June 07
2	PCA Signed	31 Jan 07	31 Jan 07
3	Obtain LER (Sponsor)	7 Feb 07	30 April 07
4	Receive Authorization for Entry for Construction from Sponsor	10 May 07	31 May 07
5	Certify RE for Construction	7 June 07	15 June 07

REAL ESTATE PLAN

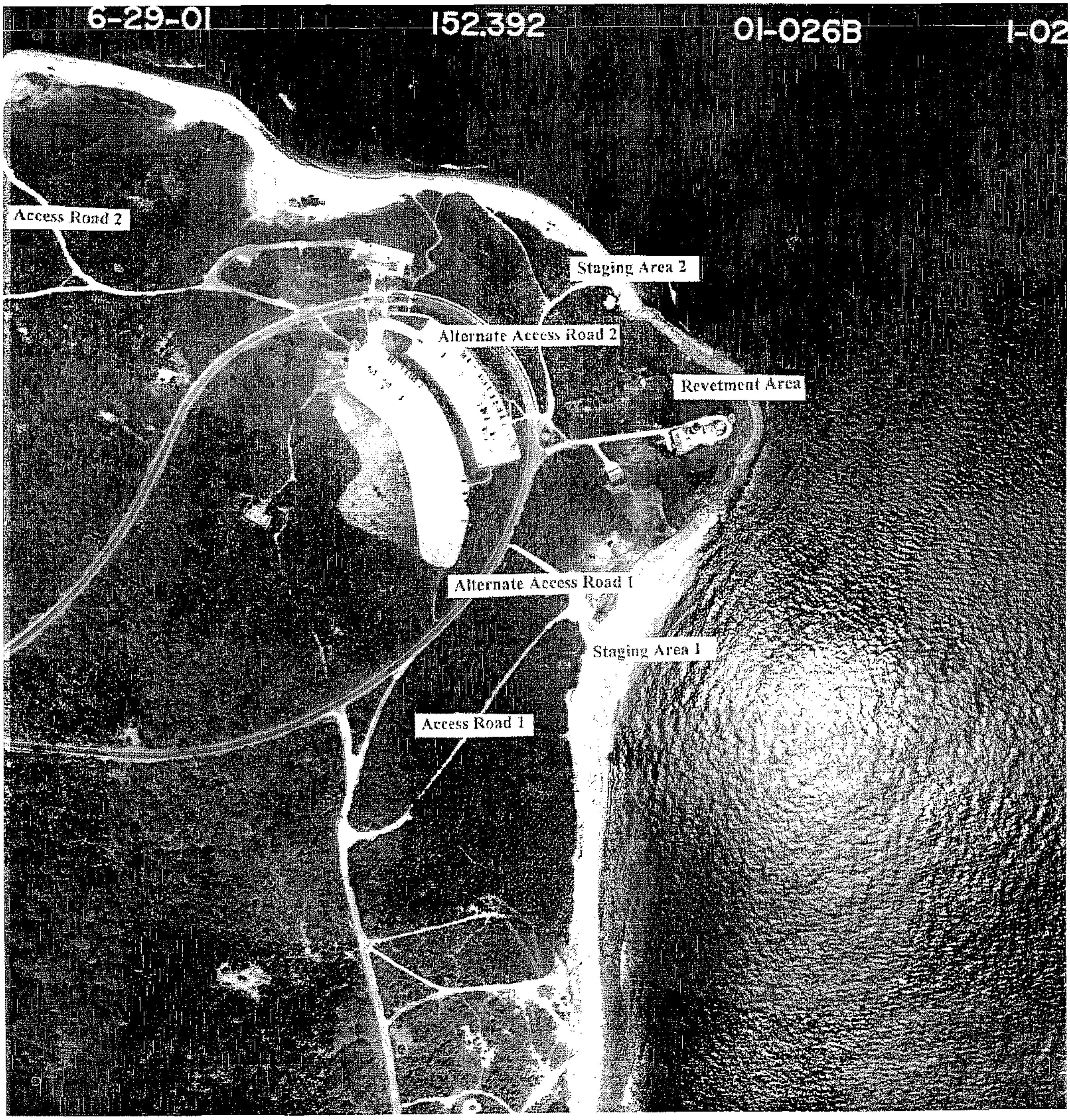
Montauk Point, New York Storm Damage Reduction Project - Feasibility Study

MAPS

Map 1 & Figure 1



Montauk easement 2.dgn 05/27/2003 11:42:50 AM



6-29-01
152.392
01-026B
1-02
Access Road 2
Staging Area 2
Alternate Access Road 2
Revetment Area
Alternate Access Road 1
Staging Area 1
Access Road 1

New York State Department of Environmental Conservation
Division of Water
Bureau of Program Resources and Flood Protection, 4th Floor
625 Broadway, Albany, New York 12233-3507
Phone: (518) 402-8151 • **FAX:** (518) 402-9029
Website: www.dec.state.ny.us

Denise M. Sheehan
Acting
Commissioner

October 5, 2005

Mr. Frank Santomauro, P.E.
Chief, Planning Division
U. S. Army Corps of Engineers
New York District
26 Federal Plaza
New York, NY 10278-0090

Re: Montauk Point, NY
Storm Damage Reduction

Dear Mr. Santomauro:

The New York State Department of Environmental Conservation (Department) has reviewed the *Draft Feasibility Report* and the *Draft Environmental Impact Statement*, and supports the project recommended therein. The Department will request that funds for design of the project be included in the 2006-2007 State Budget. However, we continue to pursue our previously-stated position that project construction funding should be 65% federal - 35% non-federal.

A copy of the letter from the Montauk Historical Society expressing support for the project is enclosed. Department efforts to identify a local cost-sharing partner for the project continue. We understand that the non-federal partners will be responsible to provide all lands, easements, and rights-of-way necessary for the construction of the project prior to federal advertisement for bids for construction.

Please direct inquiries to Project Engineer Rick Tuers, at 518-402-8148, if more information is needed.

Yours Truly,

for Michael Stankiewicz

Michael Stankiewicz, Chief
Flood Protection Structural Programs Section
Bureau of Program Resources and Flood Protection

Enclosure

c: w/o enc. - D. White/G. Donahue - Montauk Historical Society
w/enc. - F. Verga/T. Pfiefer - US Army Corps of Engineers
- R. Tuers/R. Rakoczy- BPR&FP
- E. Star - Region 1- Stony Brook, NY

MONTAUK LIGHTHOUSE

Montauk Historical Society

Post Office Box 943 • Montauk, New York 11954

AUG 2005
RESOURCE MANAGEMENT

August 1, 2005

Mr. Richard Tuers
NY State Department of Environmental Conservation
Coastal Erosion Management Section
Bureau of Flood Protection
625 Broadway, 4th Floor
Albany, NY 12233-3507

Dear Rick,

Please accept this letter in support of the Army Corps of Engineers "Storm Damage Reduction - Feasibility Study: Feasibility Report & Environmental Impact Statement Draft July 2005."

We are grateful that the report stresses the need to protect the Montauk Point Lighthouse Museum Complex. You well know that our mission as a non-profit organization (the protection, preservation and educational development of this nationally significant historic site) is dependent on the continued life of this complex of structures. In fact, Richard F. (Dick) White, Jr. Chairman of the Lighthouse Committee stresses the importance of the timeliness of this project.

We appreciate all of the work that has gone into this study and all of your efforts on behalf of saving this important Cultural Resource.

Sincerely,

Ann Shengold
Museum Director

(631) 668-2544 http://www.montauklighthouse.com Fax: (631) 668-2546

New York State Office of Parks, Recreation and Historic Preservation
Historic Preservation Field Services Bureau
Peebles Island, PO Box 189, Waterford, New York 12188-0189 518-237-8643

OFFICE OF PARKS · RECREATION · HISTORIC PRESERVATION
NEW YORK STATE

Bernadette Castro
Commissioner

September 8, 2005

Dr. Christopher Ricciardi, EIS Coordinator
US Army Corps of Engineers-NY District
Planning Division-Environmental Branch
26 Federal Plaza, Room 2151
New York, NY 10278-0090

RE: Archeology Survey at the Montauk Point Light Station
Lake Montauk
Montauk, Suffolk County, NY
04PR04116 (formerly 02PR04111)

Dear Dr. Ricciardi,

Thank you for requesting the comments of the State Historic Preservation Office (SHPO). We received the Draft Environmental Impact Statement on August 22, 2005 and are reviewing the project in accordance with Section 106 of the National Historic Preservation Act of 1966 and relevant implementing regulations.

L01a

Douglas Mackey of our archeology unit has reviewed the DEIS and concurs with the recommendations regarding archeology issues.

L01b

We understand that moving the lighthouse was explored, but will not take place. We feel strongly that it should not be moved and are pleased that it is not being considered.

Please use the PR number of top of this letter when you refer to this project in future. If you or anyone involved with the project has any questions, please contact me at 518-237-8643, ext. 3252.

Sincerely,

Sloane Bullough

Sloane Bullough
Historic Sites Restoration Coordinator

An Equal Opportunity/Affirmative Action Agency
printed on recycled paper

DEPARTMENT OF THE ARMY
OFFICE OF THE ASSISTANT SECRETARY
CIVIL WORKS
108 ARMY PENTAGON
WASHINGTON DC 20310-0108

29 JUN 2005

MEMORANDUM FOR DIRECTOR OF CIVIL WORKS

SUBJECT: Montauk Point, New York, Storm Damage Reduction Feasibility Study – Policy Exemption for Private Non-Profit Non-Federal Sponsor

I have completed my review of CEMP-NAD memorandum dated June 7, 2005, regarding the request that I grant an exception to existing policy which prohibits the Army Corps of Engineers from cost-sharing water resources projects involving a single, private landowner. Although the Montauk Historical Society (Society) is clearly a single, private, land-owner the Society must, by deed restriction and State charter, act as a public entity akin to agencies of State and local governments. The Society must accomplish its public education mission to stay in operation, must follow Federal National Historic Preservation requirements for maintenance work, and membership and enjoyment of the benefits of the facility and educational programs are open to all, with no restriction, for a $5.00 fee. Under the deed and charter the Society can not structure and constrain uses of the property as envisioned in existing policy guidance nor can anyone who cares to join the Society and enjoy the benefits of the facility (and water resources project) be excluded.

Based upon this analysis I grant the exception to the single landowner policy for this project. However, please note that this project remains a low budget priority. If you have any questions please do not hesitate to contact me. Your staff may contact Mr. Chip Smith at (703) 693-3655.

Geo. P. Dunlop

for John Paul Woodley, Jr.
Assistant Secretary of the Army
(Civil Works)

Printed on Recycled Paper