Raritan Bay and Sandy Hook Bay
Highlands, New Jersey
Coastal Storm Risk Management
Feasibility Study

Appendix B

Engineering

July 2015

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, Highlands, New Jersey

Coastal Storm Risk Management Feasibility Study

Appendix B – Engineering

Prefatory Statement: This product is being released early in the planning process. Feasibility level details will be identified during project optimization, which is after public and agency review of the draft report. Please be advised that this document is subject to revision as the analysis continues.

Contents

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project Borough of Highlands Feasibility Study

Draft Feasibility Report
JULY 2015

Appendix B1:

Civil Design

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project, Borough of Highlands Feasibility Study

Appendix B1: Civil Design

Table of Contents

List of Figures

Chapter 1: Introduction

1.1 Study Area

The Borough of Highlands is located in the northeastern section of Monmouth County, New Jersey and is bounded on the north by Sandy Hook Bay and on the east by the Shrewsbury River. The project study area consists of approximately 1/3 of a square mile of 1,500 densely developed marine, commercial, and residential buildings extending approximately 8,000 feet along low-lying coastal areas from Murray Beach at the western end to the NJ-36 Highlands-Sea Bright Bridge at the eastern end. Shore Drive serves as the southern boundary. Highlands topography is flat for approximately 1,500 feet inshore to the base of a steep grade. For analysis purposes the study area has been divided into four reaches, based on shoreline characteristics and orientation. Reaches 1, 2 and 3 are the bay-fronting sections, and Reach 4 is the river-fronting section. The reach designations can be seen on the alternative plan sheets that follow.

1.2 Shoreline Condition

The shoreline of Highlands is composed primarily of bulkheads, which range in elevation from around +6 feet NGVD at low points to approximately +10 feet NGVD at the highest point. Small marinas, restaurants, and houses characterize the shoreline. Small beaches with public access are also located in the Borough. The existing beaches and bulkheads are relatively stable, although there are portions of deteriorated timber bulkheads which are in need of repair. Based on the Raritan Bay and Sandy Hook Bay, New Jersey Combined Flood Control and Shore Protection, Reconnaissance Study Report (USACE, March, 1993) and New York District site inspection, the existing shoreline and beaches are relatively unchanged due to the hardened condition of the shoreline.

The flat topography of the waterfront fill and low existing bulkhead elevations allow tidal inundation during periods of major storm events. The 100-year tidal flood limit (12.3 feet NGVD including wave setup) would completely submerge Highlands from shoreline to the base of the bluffs, approximately 1,500 feet inland. This largely occurred during Hurricane Sandy. Most of the town’s streets would be below 5 feet of water during a 100-year storm event.

Chapter 2: Survey Data

2.1 Topographic Data

Photogrammetric mapping of the study area is available from the topography that was compiled by stereo photogrammetric methods from the aerial photography flown at 1″=250′ in April 2002. The mapping contains planimetric features such as structures, roads, and soundings and was used as a basis to layout project alternatives and develop associated quantities. Surface utilities were located by the surveyor in the field. Underground utility information was obtained by the surveyor from the various utility companies. The surveyor makes no guarantee that the underground utilities represent all such utilities in the area, either in service or abandoned. The surveyor further does not warrant that the underground utilities are in the exact location indicated and are located as accurately as possible from available information. The surveyor did not physically locate the underground utilities. The following utilities did not provide record information to the surveyor:

New Jersey American Water Company and Verizon. Horizontal data from the survey is referenced to New Jersey State Plane Coordinate System, NAD 83, US Survey Feet and vertical data is referenced to NGVD29, US Survey Feet.

2.2 Bathymetric Data

Bathymetric profiles of the project area were taken in April 2002 and consist of 21 long range lines, each extending approximately 2,500 feet seaward from near the shoreline, and spaced approximately 500 feet apart. Horizontal data from the survey is referenced to New Jersey State Plane Coordinate System, NAD 83, US Survey Feet and vertical data is referenced to NGVD29, US Survey Feet.

2.3 Vertical Datum

To date, project analyses have been conducted in reference to the NGVD29 vertical datum. Future coastal analyses during optimization will be conducted in reference to the NAVD88 vertical datum. The tidal bench mark nearest to the Highlands project site is located on the Sandy Hook Spit and has an NGS designation of: 853 1680 A TIDAL. The NGS data sheet for this tidal bench mark lists the current NAVD88 elevation as 6.41 feet and the NGVD29 elevation as 7.50 feet. Therefore, in order to convert NGVD29 elevations to NAVD88, 1.09 feet will be subtracted from the NGVD29 elevations.

Chapter 3: Project Alternatives

3.1 Preliminary Alternatives Array

During the formulation of preliminary alternatives, the following storm damage reduction features (that include combinations of structural and nonstructural elements) were analyzed:

1. Seawall with closure gates (raised epoxy coated steel sheet pile bulkhead)
   a) With scour protection
   b) With a fronting berm
   c) Existing seawall with capping (existing state bulkhead)
2. Offshore closure structure (rubble mound, navigation gate)
3. Reinforced dune (with buried seawall)
4. Removable fabricated floodwall (inland)
5. Non-structural flood features, including combinations of:
   a) Buyouts (for frequently flooded structures)
   b) Raising
   c) Ringwalls/structural peripheral wall
   d) Flood proofing
6. Beach and dune fill with terminal groins (with buried seawall)
7. Raised road, ground surface, and asphalt areas
8. Setback floodwalls (I-type floodwall)

Various combinations of the above features were included in the selected design approaches. Preliminary alternatives considered for this study include:

- **Alternative 1:** Updated USACE Plan identified in the Pre-Feasibility Study
- **Alternative 2:** Non-Structural Plan
- **Alternative 3:** Offshore Closure Plan
- **Alternative 4:** Beach and Dune Fill Plan
- **Alternative 5:** Hybrid Plan

3.1.1 Alternative 1: Updated USACE Plan identified in the Pre-Feasibility Study

This alternative is an updated version of Alternative Plan 1 from the Pre-Feasibility Report (May 2000), which was considered to be environmentally and economically feasible. Revisions included adding the capped existing state bulkhead feature to Reach 2, as well as the removable fabricated floodwall and associated additional I-type floodwalls to Reach 4. Crest elevations of the structures were also updated to reflect the wave overtopping analysis.

This alternative includes the construction of epoxy coated steel sheet pile bulkheads with watertight joint sealant, either fronting existing bulkheads or non-bulkheaded frontages, totaling 9,470 linear feet along the Highlands shoreline in all reaches, except for a 1,280 foot portion of existing state bulkhead in Reach 2 which will be capped and in Reach 4 where there would be 1,100 feet of inland removable fabricated floodwall (see plan sheets CS101-CS102). Crest elevations of the raised bulkhead will be set at +15' NGVD in Reach 1, decreasing to elevation +13' NGVD in Reaches 2 and 3, and elevation +12' NGVD in Reach 4. Concrete I-type floodwalls totaling 1,195 linear feet will tie into the existing +11 foot contour at the Highlands/Atlantic Highlands border as the western closure and at Bay Avenue near the Route 36 Bridge as the eastern closure. The raised bulkhead will be located along the high water mark, immediately in front of existing seawalls, passing inboard of piers and rimming the shoreline edges of marina areas. Except in the inside perimeter of marina areas, the bulkheads will be fronted by a stone breakwater, constructed at the toe of the bulkhead to reduce wave overtopping.

In Reach 2, 1,280 linear feet of the existing State bulkheads would be capped to an elevation of +13' NGVD, for an increase in the bulkhead's existing height of approximately 1 foot. This minimal increase in height is allowable because a parapet of approximately 10 to 15 degrees will be applied to the cap to reduce wave overtopping impacts. Because the increase in height will be relatively small, a fixed, rather than removable, extension is assumed, simplifying the needed structural connection. The landward side of the capped bulkhead (above grade) will need to be structurally reinforced to avoid the potential of exceeding the design loads of the existing bulkhead with the added loads intercepted by the capping. This reinforcement will include a 1.5- foot thick (average) monolithic section of reinforced concrete along the landside of the existing bulkhead, continuing with a 2-foot thick, 10-foot wide monolithic reinforced concrete slab at grade. A 1,460-foot long portion of raised bulkhead will be constructed in front of the marina's existing bulkhead (with no required breakwater) to tie together the two portions of the capped State bulkhead. The capped bulkhead will connect to a raised bulkhead on both ends of Reach 2 to tie into Reaches 1 and 3. In addition in Reach 3, a seaside restaurant and deck will be raised in place and the restaurant entry will be modified to maintain existing water views and access with the alignment to elevation +12′ NGVD.

In Reach 4, 185 feet of concrete I-type floodwall will be constructed from the eastern end of the raised bulkhead in Reach 3 southwest along the Windansea Restaurant's property line towards Shrewsbury Street, starting at elevation +12′ NGVD and going down to elevation +11′ NGVD near Shrewsbury Street. At Shrewsbury Street, the floodwall will connect to the northwestern end of 1,075 feet of removable fabricated floodwall, installed at a crest elevation of +11′ NGVD along the waterside curb of Shrewsbury Street. A second concrete floodwall will connect the southeastern end of the removable fabricated floodwall, extending 125 feet to the northeast at elevation +11′ NGVD to a section of raised bulkhead set along the shoreline at elevation +11′ NGVD. The alignment's easterly closure will be a concrete I-type floodwall that ties into the +11′ NGVD contour, just seaward of Bay Avenue.

It should be noted that the installation alignment of the removable fabricated floodwall leaves the 12 residential buildings located seaward of Shrewsbury Street, to the southeast of Cornell Street and to the northwest of the park on Bay Avenue, susceptible to flooding. The following structural option for storm damage protection of these 12 structures was considered: an offshore stone dike enclosing the docks and exposed shoreline, including a collinear navigation gate for boating access. This option was determined to be not viable, due to its navigation interference and its very high cost relative to the small amount of shoreline and number of piers and structures actually protected (i.e., the cost could be as much as twice the value of the structures/properties).

Four 25-foot wide closure gates will provide access points through the alignment to boat launch ramps and marina areas along the project's shoreline. As part of the minimum facility costs, ten existing outlets will be updated by placing new flap gates at the outlets. Construction of timber stair walkovers at 27 access points along the raised bulkhead features will allow for continued access to existing piers. These types of access structures can be somewhat unsightly but can be architecturally treated to improve the aesthetic character. Finally, the temporary (removable) nature of the removable fabricated floodwall in Reach 4 provides for a continuous line of protection when erected just prior to and during storms, but allows for waterfront access at all other times and temporary access during storms via a portable ramp over the removable fabricated floodwall.

This alternative meets the overall project objective of reducing storm damage for the entire Borough of Highlands, except for the buildings located seaward of Shrewsbury Street, to the southeast of Cornell Street and to the northwest of the park on Bay Avenue. In general, as most of the project site's shoreline is being raised from existing elevations, water views will be partially obstructed, but not interrupted.

3.1.2 Alternative 2: Non-Structural Plan

This alternative consists of non-structural storm damage reduction features up to +11 feet (see plan sheets CS103-CS104). The storm damage reduction features were determined using a structural analysis that applied a generalized computer algorithm to a structure inventory database. The algorithm uses flood levels along with information about each structure (i.e. ground elevation, main floor elevation, type of construction, etc.) to determine the appropriate method of flood protection, and then determines cost for flood proofing each structure. The existing Highlands Structure Inventory Table was used for the analysis. The algorithm flow chart for type of flood proofing to be assumed can be seen in attached drawing CS122.

It should be noted that this was a screening level analysis. Actual determination of the most appropriate types of flood protection for a specific building (and associated costs), including area constraints, will need to be determined by examining individual structures and site specific conditions.

The non-structural storm damage reduction features considered include the following:

1. Evacuating the building from the flood plain (buyout/relocation);
2. Elevating the building (raising);
3. Constructing various types of barriers, which usually surround the building but are not attached (ringwall/berm);
4. Constructing various types of barriers (surface floodwalls) that surround the exterior surface of the structure and provide removable flood shields at structure openings;
5. Using techniques known as "wet" flood proofing where basement utilities are relocated above ground and adjacent to the structure, but the basements are allowed to flood;
6. Using techniques also known as "wet" flood proofing where major basement utilities are protected with barriers where no room exists on the property for an above grade utility shed, but the basement is allowed to flood; and
7. Using techniques known as "dry" waterproofing where exterior wall surface waterproofing is designed to withstand added hydrostatic loading or foundation walls are rebuilt to accommodate extra hydrostatic loading for structures with basements (for low level, above grade, flooding against the structure).

A total of 991 structures are affected, with protection measures including 17 "dry" flood proofings; 65 "wet" flood proofings (for which 50 require barriers to be constructed around the utilities in the basement and 15 require relocation of the utilities in a shed above ground); 861 raisings; 13 structures with surface floodwalls; and 35 structures with ringwall/berms. The average height of raising for buildings is approximately 4.6 feet. The total length of ringwall/berms and structure surface floodwalls required is approximately 12,820 feet.

This alternative does meet the overall project objective of reducing storm damage in the Borough of Highlands. However, as the measures only protect buildings and structures from flooding, considerable residual damage would remain after a storm (i.e. to the infrastructure, cars, landscaping, and basements of "wet" floodproofed structures), and significant emergency personnel activity would be required. The non-structural features will not obstruct any water views, nor will waterfront access need to be modified.

3.1.3 Alternative 3: Offshore Closure Plan

This alternative combines structural storm damage reduction features in Reach 1 with an offshore breakwater that extends 4,500 linear feet across the Sandy Hook Bay, protecting Reaches 2, 3, and 4 (see plan sheets CS105-CS107).

At the western end of Reach 1, existing ground will be raised using impervious fill to create a raised ground surface totaling 355 square yards at elevation +11' NGVD that will tie into the existing contour near the end of Shore Drive. The side slopes of the raised ground surface will be approximately 1V:3H and will tie into surrounding areas. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. The raised ground will meet a 225 linear feet raised portion of the existing Locust Street. The 225 feet of existing road will be raised to elevation +11' NGVD; regrading will be necessary for access to private driveways. To match existing grades of both the existing Locust Street to the southeast and the mobile home park parking area to the north, transition road approaches will be constructed at a slope of 1V:10H from each end of the raised road.

Approximately 195 feet of concrete I-type floodwall will be constructed from the eastern end of the raised road northeast along an existing fence line at elevation +11' NGVD. The northern end of the floodwall will transition up to elevation +13' NGVD where it will meet the western end of another 1,276-square yard raised ground surface. This raised ground surface will also be capped with 6 inches of topsoil and planted with native vegetation. It will transition to elevation +13.5' NGVD to meet an reinforced dune constructed along the existing shoreline. The reinforced dune will consist of a buried stone seawall (1V:1.5H) covered with sand (1V:5H) and with an impervious earthen core installed along the backside of the seawall. The dunes will be planted with native dune grass to provide additional stabilization. The reinforced dune will continue at elevation +13.5' NGVD for 290 feet to meet a raised bulkhead.

The raised bulkhead will be located along the set back high water mark, immediately in front of existing seawalls. A parapet of approximately 10 to 15 degrees will be applied to the bulkhead to reduce wave overtopping impacts, allowing for a reduction in elevation in comparison to Alternative 1 of 2 feet. The bulkhead will be 460 feet long at a crest elevation +13.5' NGVD, fronted by a breakwater, constructed at the toe of the bulkhead to reduce wave overtopping impacts. In addition, the breakwater will also provide for protection from the isolated historic erosion that is occurring at this location. Another contiguous reinforced dune, again planted with native dune grass, will have a crest elevation of +13.5' NGVD and continue for 305 feet to meet a raised asphalt parking area, 165 feet long. The crest elevation of the raised asphalt area will be at +13.5 feet, with side slopes of 1V:10H, allowing for continued use as a parking area, and for vehicular access to the existing ferry terminal.

Another reinforced dune will continue from the raised parking area along the shoreline for 945 feet at elevation +13.5' NGVD. At the eastern-most pier in Reach 1, the footprint of the reinforced dune will be angled towards the southwestern corner of the existing state bulkhead to allow for continued recreation use of the large existing beach. The dune barrier will transition from elevation +13.5' NGVD to meet a raised bulkhead with a crest elevation of +13' NGVD. The 35-feet of raised bulkhead and its associated breakwater will be constructed in front of an existing seawall that crosses an existing channel that flows along Snug Harbor Avenue. The raised bulkhead will connect with the existing capped state bulkhead at crest elevation +13' NGVD in Reach 2.

At the eastern end of Reach 1, an offshore breakwater will be tied in to the end of the on-shore dune barrier and run parallel to the existing state bulkhead, continuing across the bay and connecting to high ground on the Sandy Hook Spit. The total breakwater alignment is approximately 4,500 feet, crossing a broad shoal area on the spit side. At the location of the existing navigation channel approximately 500 feet from the state bulkhead, a 135-foot wide navigation sector gate will be installed to allow for a 100-foot clear opening for navigation transit when the gate is in the open position. Prior to potential major storm events, the sector gate will be closed during a period of lower tide, sealing the inner basin, providing additional runoff storage leeward of the barrier and protecting Reaches 2, 3, and 4. No additional storm damage reduction features will be constructed in Reaches 2, 3, and 4.

Mean bay-bottom elevation along the breakwater alignment is roughly –3' NGVD or less, except across the navigation channel where it is an average of –18 to –20' NGVD. The crest of the breakwater will be set at elevation +13.5' NGVD. The crest elevation was selected to limit the effect of storm waves, reduce overtopping damage to the leeward side of the breakwater, and avoid water buildup from overtopping wave effects. There is insufficient storage leeward of the breakwater to store storm water runoff buildup to below elevation +6' NGVD with the sector gate closed, therefore a pump station will be required. Based on gross approximations, a 4,000 cfs pump station will prevent residual damages from the closed gate.

Mean armor size for the offshore breakwater will be around 2.6 tons with a double-stone thickness of rough angular armor material. The armor stone will be underlain with a double layer of 500 pound stone, which in turn, will overlie the core and bedding stone structure foundation.

The impermeable core will be a steel or composite sheet pile wall to elevation +10.5' NGVD, and penetrated sufficiently below the Sandy Hook Bay bottom for structural stability. Because of the potential for overtopping, the harbor side of the breakwater will also need to be armored with similar sized armor stone. The crest width will be three stones wide (10 feet) and will cover the sheet pile wall. Breakwater side slopes will be 1V:2H.

Two 25-foot wide closure gates will provide access points through the alignment to boat launch ramps and marina areas along the project's shoreline. As part of the minimum facility costs, one existing outlet will be updated by placing new flap gates at the outlet. The reinforced dunes will require earthen dune walkovers to maintain waterfront access at five points. Likewise, construction of a timber stair walkover will be constructed at one access point along the raised bulkhead feature to allow for continued access to an existing pier. These types of access structures can be somewhat unsightly but can be architecturally treated to improve the aesthetic character.

This alternative meets the overall project objective of reducing storm damage for the majority of the Borough of Highlands. There are less waterfront access impacts and partial water view obstructions when compared to Alternatives 4 and 5, as the offshore breakwater excludes the need for any storm damage reduction features along the shoreline of Reaches 2, 3, and 4. However, the offshore breakwater may impact views across the Sandy Hook Bay and Shrewsbury River from both the eastern and western shorelines of the project site.

3.1.4 Alternative 4: Beach and Dune Fill Plan

The structural storm damage reduction features in this alternative in Reach 1 are the same as those in Alternative 3—with the substitution of beach and dune fill in a portion of the reach (see Figures CS108-CS109). This is the only area where a beach and dune fill section can be accommodated due to the proximity of the existing navigation channel, piers, and shoreline frontage usage.

At the western end of Reach 1, existing ground will be raised using impervious fill to create a raised ground surface totaling 355 square yards at elevation +11' NGVD that will tie into the existing contour near the end of Shore Drive. The side slopes of the raised ground will be approximately 1V:3H and will tie into surrounding areas. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. The raised ground will meet a 225 linear feet raised portion of the existing Locust Street. The 225 feet of existing road will be raised to elevation +11' NGVD; regrading will be necessary for access to private driveways. To match existing grades of both the existing Locust Street to the southeast and the mobile home park parking area to the north, transition road approaches will be constructed at a slope of 1V:10H from each end of the raised road.

Approximately 195 feet of concrete I-type floodwall will be constructed from the eastern end of the raised road northeast along an existing fence line at elevation +11' NGVD. The northern end of the floodwall will transition up to elevation +13' NGVD where it will meet the western end of an L-shaped raised ground surface totaling 2,160 square yards with 1V:3H side slopes tying into surrounding areas. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. It will transition from elevation 13' NGVD to 13.5' NGVD to connect to the backside of a beach and dune fill area that extends 1,100-feet long along the shoreline, sized according to crenulate bay theory. The shoreline will be renourished with beach fill, extending the existing waterline seaward approximately 40 feet and mitigating for the isolated historic erosion that is occurring at this location. (The potential seaward projection of the beach fill was limited due to existing functioning pier structures and the existing navigation channel.)

The beach fill will be built up to elevation +10' NGVD, meeting the dune fill portion, which will have a crest elevation of +13.5' NGVD. Space limitations and under-seepage concerns did not allow for a wide protective dune; therefore, an inner core consisting of a buried sheet-pile seawall will be located inside the dune approximately flush with dune protection, as shown in Figure 1.

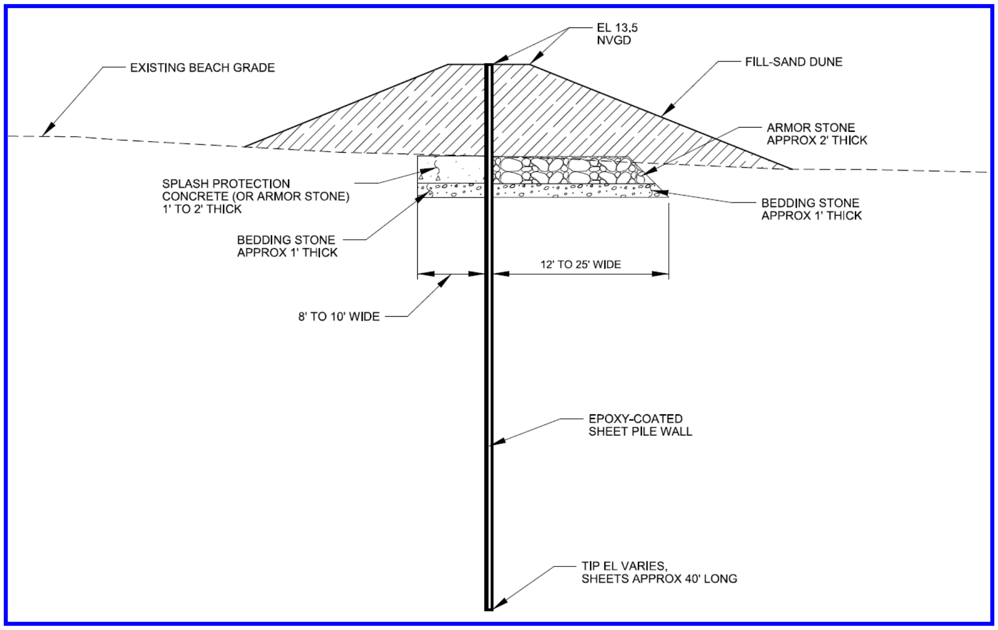


Figure 1: Beach and Dune Fill Typical Section.

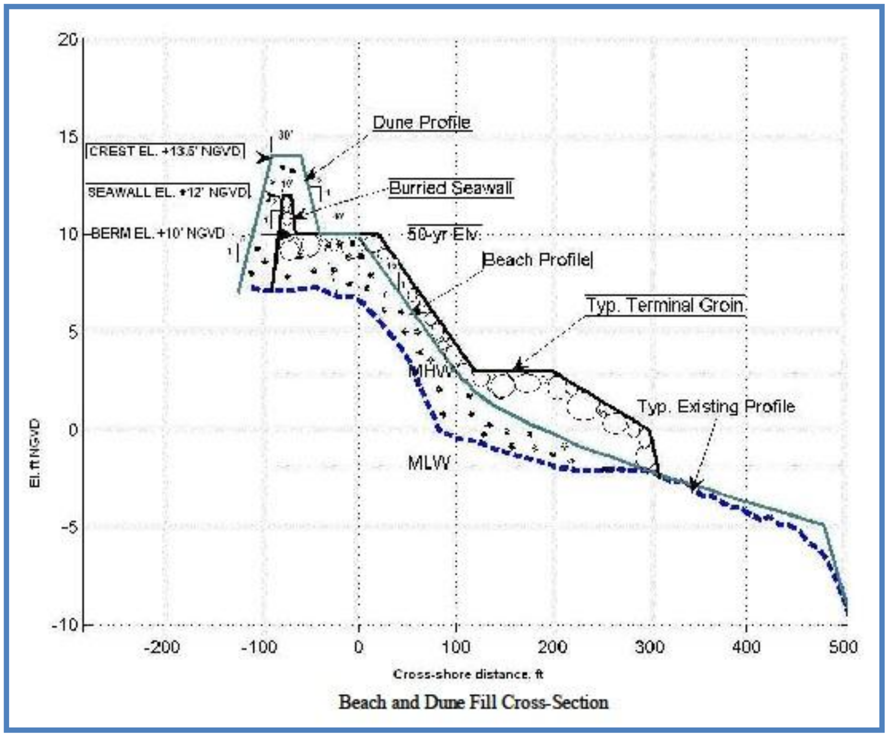


Figure 2: Beach and Dune Fill Typical End Section.

Terminal groins, each approximately 350 feet long will be constructed at either end of the beach and dune fill to create a pocket beach to retain beach fill. Figure 2 shows a typical section for the beach and dune fill.

The crest elevation at the eastern end of the dune fill will continue at elevation +13.5′ NGVD to connect to a raised asphalt parking area, 165 feet long. The crest elevation of the raised asphalt area will be at +13.5′ NGVD, with side slopes of 1V:10H, allowing for continued use as a parking area, and for vehicular access to the existing ferry terminal. A contiguous reinforced dune will be constructed at crest elevation +13.5′ NGVD from the raised asphalt area along the existing shoreline for a total of 945 feet. The reinforced dune will consist of a buried stone seawall (1V:1.5H) covered with sand (1V:5H) and with an impervious earthen core installed along the backside of the seawall. The dunes will be planted with native dune grass to provide additional stabilization.

At the eastern-most pier in Reach 1, the footprint of the reinforced dune will be angled towards the southwestern corner of the existing state bulkhead to allow for continued recreation use of the large existing beach. The dune barrier will transition from elevation +13.5′ NGVD to meet a raised bulkhead with a crest elevation of +13′ NGVD. The 35-feet of raised bulkhead and its associated breakwater will be constructed in front of an existing seawall that crosses an existing channel that flows along Snug Harbor Avenue. The raised bulkhead will connect with the existing capped state bulkhead at crest elevation +13′ NGVD in Reach 2.

In Reach 2, 1,280 linear feet of the existing state bulkhead will be capped to an elevation of +13′ NGVD, for an increase in the bulkhead’s existing height of approximately 1 foot. A parapet of approximately 10 to 15 degrees will be applied to the cap to reduce wave overtopping impacts, allowing for this crest elevation. Because the increase in height will be relatively small, a fixed, rather than removable, extension is assumed, simplifying the needed structural connection. The landward side of the capped bulkhead (above grade) will need to be structurally reinforced to avoid the potential of exceeding the design loads of the existing bulkhead with the added loads intercepted by the capping. This reinforcement will include a 1.5-foot thick (average) monolithic section of reinforced concrete along the landside of the existing bulkhead, continuing with a 2-foot thick, 10-foot wide monolithic reinforced concrete slab at grade. At the center of Reach 2, a buoyant swing gate, similar in design to the “Buoyant Swing Gate” as detailed in the USACE’s Leonardo, NJ Hurricane Storm Damage Reduction Feasibility Study: Closure Gate Assessment and Design (Leonardo Report, April 2002), will be installed at the inlet opening to a marina, tying together the two portions of the capped state bulkhead. The entire gate structure will be 70-feet wide, with a 55-foot wide channel available for navigation transit when the gate is in the open position. Prior to potential major storm events, the swing gate will be closed during a period of lower tide, sealing the existing marina and protecting it from flood waters. The capped bulkhead will connect to a raised bulkhead in Reach 3.

In Reach 3, a 430-foot transition section of raised bulkhead will be constructed at a crest elevation of +13′ NGVD. The raised bulkhead will be located along the set back high water mark, immediately in front of existing seawalls. The associated breakwater will only be constructed for 75 feet from the capped state bulkhead, since the remainder of the raised bulkhead runs along the inside perimeter of an existing marina and a breakwater would interfere with marina operations. To the east, the raised bulkhead will transition to meet a raised asphalt parking area with a crest elevation of +12′ NGVD, continuing for 380 feet across the existing parking areas at the end of Atlantic Street. The side slopes of the raised asphalt area will be 1V:10H, allowing for continued use as parking areas and continued access to the existing marina. Another reinforced dune, again planted with native dune grass, will continue at elevation +12′ NGVD for 145 feet from the raised asphalt area along the existing shoreline. This reinforced dune will then connect to another raised bulkhead, which continues at a crest elevation of +12′ NGVD. This raised bulkhead and its associated breakwater will be constructed for 510 feet. Another 850 feet of contiguous reinforced dune will be constructed at a crest elevation of +12′ NGVD and connect to another section of raised bulkhead, also at crest elevation +12′ NGVD. The raised bulkhead and its associated breakwater will be constructed in front of existing seawall for 635 feet, connecting to a concrete I-type floodwall in Reach 4. A seaside restaurant and deck will be raised in place and the restaurant entry will be modified to maintain existing water views and access with the alignment to elevation +12′ NGVD.

In Reach 4, 140 feet of concrete I-type floodwall will be constructed from the eastern end of the raised bulkhead in Reach 3 southwest along the Windansea Restaurant's property line towards Shrewsbury Street, transitioning from elevation +12′ NGVD to elevation +11′ NGVD. The I-type floodwall will connect to the northwestern end of 1,075 feet of removable fabricated floodwall, installed at a crest elevation of +11′ NGVD along the waterside curb of Shrewsbury Street. The removable fabricated floodwall will connect to the northwestern end of another raised ground surface. The crest will continue at elevation +11′ NGVD. The footprint of this raised ground covers 5,650 square yards of an existing public park located to the north of Bay Avenue. The raised surface will duplicate the existing park features and surfacing, including the raising of a monument at the entrance to the park. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. At the southeastern end of this area, the crest elevation of the raised ground will continue at elevation +11′ NGVD and meet a 415- linear foot raised portion of the existing Bay Avenue to tie into the +11′ NGVD contour along Bay Avenue at the eastern closure of the project. The 415 feet of existing road will be raised to elevation +11′ NGVD; regrading will be necessary for access to driveways and walks. To match existing grades of the existing Bay Avenue to the northwest and close the alignment at the eastern end of the project site, a transition road approach will be constructed at a slope of 1V:10H from the northwestern end of the raised road.

The recommended type of removable fabricated floodwall is the same as that for Alternative 1. It should be noted that the installation alignment of the removable fabricated floodwall leaves the 12 residential buildings located seaward of Shrewsbury Street, to the southeast of Cornell Street and to the northwest of the park on Bay Avenue, susceptible to flooding. The following structural option for storm damage protection of these 12 structures was considered: an offshore stone dike enclosing the docks and exposed shoreline, including a collinear navigation gate for boating access. This option was determined to be not viable, due to its navigation interference and its very high cost relative to the small amount of shoreline and number of piers and structures actually protected (i.e., the cost could be as much as twice the value of the structures/properties).

Three 25-foot wide closure gates will provide access points through the alignment to boat launch ramps and marina areas along the project's shoreline. As part of the minimum facility costs, four existing outlets will be updated by placing new flap gates at the outlets. The reinforced dunes will require earthen dune walkovers to maintain waterfront access at eight points. Likewise, construction of timber stair walkovers will be constructed at nine access points along the raised bulkhead features to allow for continued access to existing piers. These types of access structures can be somewhat unsightly but can be architecturally treated to improve the aesthetic character. The inclusion of the buoyant swing gate will allow for continued access to the marina in Reach 2. The gently sloped (1V:10H) raised parking areas will allow for the continued access to the adjacent waterfront structures. Finally, the temporary (removable) nature of the removable fabricated floodwall in Reach 4 provides for a continuous alignment when erected just prior to and during storms, but allows for waterfront access at all other times and temporary access during storms via a portable ramp over the removable fabricated floodwall.

This alternative meets the overall project objective of reducing storm damage for the majority of the Borough of Highlands, except for the buildings located seaward of Shrewsbury Street, to the southeast of Cornell Street and to the northwest of the park on Bay Avenue. In general, as most of the project site's shoreline is being raised from existing elevations, water views will be partially obstructed but not interrupted.

3.1.5 Alternative 5: Hybrid Plan

This alternative (as shown on plan sheets CS110-CS111) combines the same alignment as Alternative 3 for Reach 1 with the same alignment as Alternative 4 for Reaches 2, 3, and 4.

At the western end of Reach 1, existing ground will be raised using impervious fill to create a raised ground surface totaling 355 square yards at elevation +11' NGVD that will tie into the existing contour near the end of Shore Drive. The side slopes of the raised ground surface will be approximately 1V:3H and will tie into surrounding areas. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. The raised ground will meet a 225 linear feet raised portion of the existing Locust Street. The 225 feet of existing road will be raised to elevation +11' NGVD; regrading will be necessary for access to private driveways. To match existing grades of both the existing Locust Street to the southeast and the mobile home park parking area to the north, transition road approaches will be constructed at a slope of 1V:10H from each end of the raised road.

Approximately 195 feet of concrete I-type floodwall will be constructed from the eastern end of the raised road northeast along an existing fence line at elevation +11' NGVD. The northern end of the floodwall will transition up to elevation +13' NGVD where it will meet the western end of another 1,276-square yard raised ground surface. This raised ground surface will also be capped with 6 inches of topsoil and planted with native vegetation. It will transition to elevation +13.5' NGVD to meet a reinforced dune constructed along the existing shoreline. The reinforced dune will consist of a buried stone seawall (1V:1.5H) covered with sand (1V:5H) and with an impervious earthen core installed along the backside of the seawall. The dunes will be planted with native dune grass to provide additional stabilization. The reinforced dune will continue at elevation +13.5′ NGVD for 290 feet to meet a raised bulkhead.

The raised bulkhead will be located along the set back high water mark, immediately in front of existing seawalls. A parapet of approximately 10 to 15 degrees will be applied to the bulkhead to reduce wave overtopping impacts, allowing for a reduction in elevation in comparison to Alternative 1 of 2 feet. The bulkhead will be 460 feet long at a crest elevation +13.5′ NGVD, fronted by a breakwater, constructed at the toe of the bulkhead to reduce wave overtopping impacts. In addition, the breakwater will also provide for protection from the isolated historic erosion that is occurring at this location. Another contiguous reinforced dune, again planted with native dune grass, will have a crest elevation of +13.5′ NGVD and continue for 305 feet to meet a raised asphalt parking area, 165 feet long. The crest elevation of the raised asphalt area will be at +13.5 feet, with side slopes of 1V:10H, allowing for continued use as a parking area, and for vehicular access to the existing ferry terminal.

Another reinforced dune will continue from the raised parking area along the shoreline for 945 feet at elevation +13.5′ NGVD. At the eastern-most pier in Reach 1 the footprint of the reinforced dune will be angled towards the southwestern corner of the existing state bulkhead to allow for continued recreation use of the large existing beach. The dune barrier will transition from elevation +13.5′ NGVD to meet a raised bulkhead with a crest elevation of +13′ NGVD. The 35-feet of raised bulkhead and its associated breakwater will be constructed in front of an existing seawall that crosses an existing channel that flows along Snug Harbor Avenue. The raised bulkhead will connect with the existing capped state bulkhead at crest elevation +13′ NGVD in Reach 2.

In Reach 2, 1,415 linear feet of the existing state bulkhead will be capped to an elevation of +13′ NGVD, for an increase in the bulkhead's existing height of approximately 1 foot. A parapet of approximately 10 to 15 degrees will be applied to the cap to reduce wave overtopping impacts, allowing for a crest elevation +13′ NGVD. Because the increase in height will be relatively small, a fixed, rather than removable, extension is assumed, simplifying the needed structural connection. The landward side of the capped bulkhead (above grade) will need to be structurally reinforced to avoid the potential of exceeding the design loads of the existing bulkhead with the added loads intercepted by the capping. This reinforcement will include a 1.5-foot thick (average) monolithic section of reinforced concrete along the landside of the existing bulkhead, continuing with a 2-foot thick, 10-foot wide monolithic reinforced concrete slab at grade. At the center of Reach 2, a buoyant swing gate, similar in design to the "Buoyant Swing Gate" as detailed in the USACE's Leonardo, NJ Hurricane Storm Damage Reduction Feasibility Study: Closure Gate Assessment and Design (Leonardo Report, April 2002), will be installed at the inlet opening to a marina, tying together the two portions of the capped state bulkhead. The entire gate structure will be 70-feet wide, with a 55-foot wide channel available for navigation transit when the gate is in the open position. Prior to potential major storm events, the swing gate will be closed during a period of lower tide, sealing the existing marina and protecting it from flood waters. The capped bulkhead will connect to a raised bulkhead in Reach 3.

In Reach 3, a 430-foot transition section of raised bulkhead will be constructed at a crest elevation of +13′ NGVD. The raised bulkhead will be located along the set back high water mark, immediately in front of existing seawalls. The associated breakwater will only be constructed for 75 feet from the capped state bulkhead, since the remainder of the raised bulkhead runs along the inside perimeter of an existing marina and a breakwater would interfere with marina operations. To the east, the raised bulkhead will transition to meet a raised asphalt parking area with a crest elevation of +12′ NGVD, continuing for 380 feet across the existing parking areas at the end of Atlantic Street. The side slopes of the raised asphalt area will be 1V:10H, allowing for continued use as parking areas and continued access to the existing marina. Another reinforced dune, again planted with native dune grass, will continue at elevation +12′ NGVD for 145 feet from the raised asphalt area along the existing shoreline. This reinforced dune will then connect to another raised bulkhead, which continues at a crest elevation of +12′ NGVD. This raised bulkhead and its associated breakwater will be constructed for 510 feet. Another 850 feet of contiguous reinforced dune will be constructed at a crest elevation of +12′ NGVD and connect to another section of raised bulkhead, also at crest elevation +12′ NGVD. The raised bulkhead and its associated breakwater will be constructed in front of existing seawall for 635 feet, connecting to a concrete I-type floodwall in Reach 4. A seaside restaurant and deck will be raised in place and the restaurant entry will be modified to maintain existing water views and access with the alignment to elevation +12′ NGVD.

In Reach 4, 140 feet of concrete I-type floodwall will be constructed from the eastern end of the raised bulkhead in Reach 3 southwest along the Windansea Restaurant's property line towards Shrewsbury Street, transitioning from elevation +12′ NGVD to elevation +11′ NGVD. The I-type floodwall will connect to the northwestern end of 1,075 feet of removable fabricated floodwall, installed at a crest elevation of +11′ NGVD along the waterside curb of Shrewsbury Street. The removable fabricated floodwall will connect to the northwestern end of another raised ground surface. The crest will continue at elevation +11′ NGVD. The footprint of this raised ground covers 5,650 square yards of an existing public park located to the north of Bay Avenue. The raised surface will duplicate the existing park features and surfacing, including the raising of a monument at the entrance to the park. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. At the southeastern end of this area, the crest elevation of the raised ground will continue at elevation +11′ NGVD and meet a 415- linear foot raised portion of the existing Bay Avenue to tie into the +11′ NGVD contour along Bay Avenue at the eastern closure of the project. The 415 feet of existing road will be raised to elevation +11′ NGVD; regrading will be necessary for access to driveways and walks. To match existing grades of the existing Bay Avenue to the northwest and close the alignment at the eastern end of the project site, a transition road approach will be constructed at a slope of 1V:10H from the northwestern end of the raised road.

The recommended type of removable fabricated floodwall is the same as that for Alternative 1. It should be noted that the installation alignment of the removable fabricated floodwall leaves the 12 residential buildings located seaward of Shrewsbury Street, to the southeast of Cornell Street, and to the northwest of the park on Bay Avenue, susceptible to flooding. The following structural option for storm damage protection of these 12 structures was considered: an offshore stone dike enclosing the docks and exposed shoreline, including a collinear navigation gate for boating access. This option was determined to be not viable, due to its navigation interference and its very high cost relative to the small amount of shoreline and number of piers and structures actually protected (i.e., the cost could be as much as twice the value of the structures/properties).

Three 25-foot wide closure gates will provide access points through the alignment to boat launch ramps and marina areas along the project's shoreline. In addition, three existing outlets will be replaced by new tide gates as part of minimum facility costs. The reinforced dunes will require earthen dune walkovers to maintain waterfront access at ten points. Likewise, construction of timber stair walkovers will be constructed at ten access points along the raised bulkhead features to allow for continued access to existing piers. These types of access structures can be somewhat unsightly but can be architecturally treated to improve the aesthetic character. The inclusion of the buoyant swing gate will allow for continued access to the marina in Reach 2. The gently sloped (1V:10H) raised parking areas will allow for the continued access to the adjacent waterfront structures. Finally, the temporary nature of the removable fabricated floodwall in Reach 4 provides for a continuous alignment when erected just prior to and during storms, but allows for waterfront access at all other times and temporary access during storms via a portable ramp over the removable fabricated floodwall.

This alternative meets the overall project objective of reducing storm damage for the majority of the Borough of Highlands, except for the buildings located seaward of Shrewsbury Street, to the southeast of Cornell Street, and to the northwest of the park on Bay Avenue. In general, as most of the project site's shoreline is being raised from existing elevations, water views will be partially obstructed, but not interrupted.

3.2 Final Alternatives Array

Alternative 1 (the pre-feasibility plan), Alternative 4 (the dune and beachfill plan), and Alternative 5 (the environmental impact minimization and avoidance plan) were considered. Of the three alternatives, Alternative 5 had the highest BCR and the highest net benefits. Accordingly, Alternative 5 was developed further into five variants, Alternatives 5A to 5E.

- **Alternative 5A:** Alternative 5 with Perimeter Bulkhead in lieu of Buoyant Swing Gate
- **Alternative 5B:** Alternative 5 with Raised Bulkhead and Non-Structural Measures in lieu of Removable Flood Wall, Target Elevation of 12 feet NGVD
- **Alternative 5C:** Alternative 5 with Raised Bulkheads and Non-Structural Measures in lieu of Removable Flood Wall, Target Elevation of 13.2 feet NGVD
- **Alternative 5D:** Alternative 5 with Raised Bulkheads in lieu of Removable Flood Wall, Target Elevation of 13.9 feet NGVD
- **Alternative 5E:** Alternative 5A and Alternative 5D Combined

The alignments for Alternative 5A-5E have been further refined to match the existing topographic features from the 2002 survey. In addition, features in various locations have been modified based on input received at multiple public meetings conducted between March and May 2014.

3.2.1 Alternative 5A: Alternative 5 with Perimeter Bulkhead in lieu of Buoyant Swing Gate

This alternative is shown on plan sheets CS112-CS113.

For features in Reach 1, the design elevation is set at 13.5 ft NGVD. See paragraph 4.1 for information on the western tie-in and the private development that has been proposed for the area referred to as the Bollerman property. This private development is assumed to serve as the western end of the project alignment and the raised bulkhead for Reach 1 is assumed to begin at the eastern edge of the development. Raised bulkheads are proposed throughout Reach 1 and will include a stone breakwater and concrete splash pad along the entire length. The seaside rock berm will provide toe protection against erosion and will act as a breakwater to reduce wave action. The breakwater is 12 ft wide and 2 ft thick and will be placed on top of a 6 in. layer of bedding material on geotextile. The concrete splash pad will be placed on the landside to protect against erosion from overtopping. The splash pad is 10 ft wide and 2 ft thick and will be placed on top of a 1 ft layer of bedding material on geotextile. At two locations in Reach 1, sand fill will be placed over the raised bulkhead to improve the aesthetics since they are located along existing beach areas. The sand fill is 12 ft wide at the crown with 1V:5H side slopes to tie into the surrounding area. The dune fills will be planted with native vegetation to help protect against erosion.

For features in Reach 2, the design elevation is set at 13 ft. NGVD. Refer to the description from Alternative 5 for details on the capping of the existing state bulkhead. In lieu of a buoyant swing gate across the opening of the Captain's Cove Marina, this alternative proposes a raised bulkhead that is setback on the landward side of the existing perimeter bulkhead. No breakwater is proposed for the setback wall as wave action is reduced within the marina and toe protection will be provided by the existing wall that is left in place. A concrete splash pad is included on the landside of the setback wall to protect against erosion from overtopping.

A raised bulkhead installed in front of the existing marina wall (as assumed for Alternative 1) was considered but was determined to be undesirable after discussions with the owner. The existing marina is very narrow and interior bulkheads would further reduce the available wet footprint. In addition, due to the increased wall height, walkways would be needed along the interior perimeter and would again reduce the operating width of the marina. Instead, the proposed bulkhead has been setback on the landward side to minimize impacts to the marina. One existing residential structure on the east side of the marina is located too close for a setback wall to be feasible. Consequently, at this location, approximately 100 lf of the raised bulkhead will be installed inside the existing marina wall. Also, due to the setback, traffic along Washington Avenue will likely need to be converted to one direction only; however, roadside parking will remain.

For features in Reach 3, the design elevation is set at 12 ft. NGVD. Raised bulkheads are proposed throughout Reach 3 and will include a stone breakwater and concrete splash pad along the entire length. At two locations in Reach 3, sand fill will be placed over the raised bulkhead to improve the aesthetics since they are located along existing beach areas. The dune fills will be planted with native vegetation to help protect against erosion. In addition, a boat launch facility that utilizes a 35 ton travel lift will need to be raised in place to the new design elevation and will require the construction of an approach ramp to tie into the existing parking lot. Also, the Inlet Café Restaurant and the seaside deck of the Windandsea restaurant will be raised in place to mitigate viewshed impacts to their dining areas.

For features in Reach 4, the design elevation is set at 11 ft. NGVD. Refer to the description from Alternative 5 for details on the removable flood wall. The Eastern Tie-In will consist of sea wall tying in the edge of the Veteran's Memorial Park to high ground at the bluff. A steel and reinforced concrete closure structure and hydraulic gate or gates will be required to allow access along Bay Avenue while maintaining the alignment. This tie-in was selected as the most economical option and reducing the number of conflicts with landowners, including the Twin Lights and Gateway Marinas.

The reinforced dunes will require earthen dune walkovers to maintain waterfront access at seven points. Likewise, construction of timber stair walkovers will be constructed at access points along the raised bulkhead features to allow for continued access to existing piers.

3.2.2 Alternative 5B: Alternative 5 with Raised Bulkhead and Non-Structural Measures in lieu of Removable Flood Wall, Target Elevation of 12 feet NGVD

This alternative is shown on plan sheets CS114-CS115.

This alternative consists of the same storm damage reduction features and access features as Alternative 5A, except the buoyant swing gate is used at Captain's Cove Marina in Reach 2 and the fabricated floodwall is removed in Reach 4. In lieu of the fabricated floodwall, protection is provided by the following features: bulkheading to elevation 12.0 ft. NGVD along the existing shoreline in Reach 4 and raising of the 16 structures landward of the bulkhead with reinforced concrete foundations. The reinforced raised foundations are necessary to withstand the wave overtopping forces possible with a 12.0 ft. NGVD elevation bulkhead.

The reinforced dunes will require earthen dune walkovers to maintain waterfront access at seven points. Likewise, construction of timber stair walkovers will be constructed at access points along the raised bulkhead features to allow for continued access to existing piers.

3.2.3 Alternative 5C: Alternative 5 with Raised Bulkheads and Non-Structural Measures in lieu of Removable Flood Wall, Target Elevation of 13.2 feet NGVD

This alternative is shown on plan sheets CS116-CS117.

This alternative consists of the same storm damage reduction features and access features as Alternative 5A, except the buoyant swing gate is used at Captain's Cove Marina in Reach 2 and the fabricated floodwall is removed in Reach 4. In lieu of the fabricated floodwall, protection is provided by the following features: bulkheading to elevation 13.2 ft. NGVD along the existing shoreline in Reach 4 and raising of the 16 structures landward of the bulkhead with standard block foundations. The raised standard block foundations are adequate to withstand the wave overtopping forces possible with a 13.2 ft. NGVD elevation bulkhead.

The reinforced dunes will require earthen dune walkovers to maintain waterfront access at seven points. Likewise, construction of timber stair walkovers will be constructed at access points along the raised bulkhead features to allow for continued access to existing piers.

3.2.4 Alternative 5D: Alternative 5 with Raised Bulkheads in lieu of Removable Flood Wall, Target Elevation of 13.9 feet NGVD

This alternative is shown on plan sheets CS118-CS119.

This alternative consists of the same storm damage reduction features and access features as Alternative 5A, except the buoyant swing gate is used at Captain's Cove Marina in Reach 2 and the fabricated floodwall is removed in Reach 4. In lieu of the fabricated floodwall, protection is provided by the following features: bulkheading to elevation 13.9 ft. NGVD along the existing shoreline in Reach 4. The existing foundations of structures landward of the bulkhead are adequate to withstand the wave overtopping forces possible with a 13.9 ft. NGVD elevation bulkhead.

The reinforced dunes will require earthen dune walkovers to maintain waterfront access at seven points. Likewise, construction of timber stair walkovers will be constructed at access points along the raised bulkhead features to allow for continued access to existing piers.

3.2.5 Alternative 5E: Alternative 5A and Alternative 5D Combined

This alternative is shown on plan sheets CS120-CS121.

This alternative consists of the same storm damage reduction features and access features as Alternative 5A combined with Alternative 5D. In lieu of a buoyant swing gate across the opening of the Captain's Cove Marina in Reach 2, protection is provided by a raised bulkhead that is setback on the landward side of the existing perimeter bulkhead. In lieu of the fabricated floodwall in Reach 4, protection is provided by bulkheading to elevation 13.9 ft. NGVD along the existing shoreline. As noted above, the existing foundations of structures landward of the bulkhead are adequate to withstand the wave overtopping forces possible with a 13.9 ft. NGVD elevation bulkhead.

The reinforced dunes will require earthen dune walkovers to maintain waterfront access at seven points. Likewise, construction of timber stair walkovers will be constructed at access points along the raised bulkhead features to allow for continued access to existing piers.

Chapter 4: Project Tie-Ins

4.1 Western Tie-In

During the final alternative design phase of this study, a private developer submitted preliminary plans to the Borough of Highlands that proposes a new development at the western end of the project area (approximately 600 linear feet). This area is referred to as the Bollerman property. The preliminary plan includes a multi-use development consisting of 49 residential units located in 11 buildings, a 5,735 square foot restaurant, a 590 square foot office space and reconstructs the existing marina to include 129 slips. A combination of raised ground surfaces and new bulkheads are proposed as part of the development to serve as the alignment against flooding. For the final alternative analysis, it was assumed that this private development will serve as the western tie into high ground and will prevent flood water from flanking around the overall alignment. During the next phase of this study, the design heights for the alignment will be optimized to maximize the return on investment. After final design heights have been determined, the Bollerman development will need to be reexamined to ensure that a continuous and complete alignment is provided at the western tie-in. The preliminary grading plan for the development has been included as an attachment.

4.2 Eastern Tie-In

The Eastern Tie-In will consist of an epoxy-coated sheet pile sea wall from the alignment along the center of Veteran's Memorial Park to high ground at the bluff. A steel and reinforced concrete closure structure and hydraulic gate or gates will be required to allow access along Bay Avenue while maintaining the alignment. This tie-in was selected as the most economical option and reduced the number of conflicts with landowners, including the Twin Lights and Gateway Marinas.

Chapter 5: Access

5.1 Timber Stair Walkover

Along raised bulkhead features, construction of timber stair walkovers will be necessary to allow for continued access to existing piers. The preliminary location of timber stair walkovers can be seen on the individual alternative plan sheets. Figures 3-5 show different views of a typical timber stair walkover for the project.

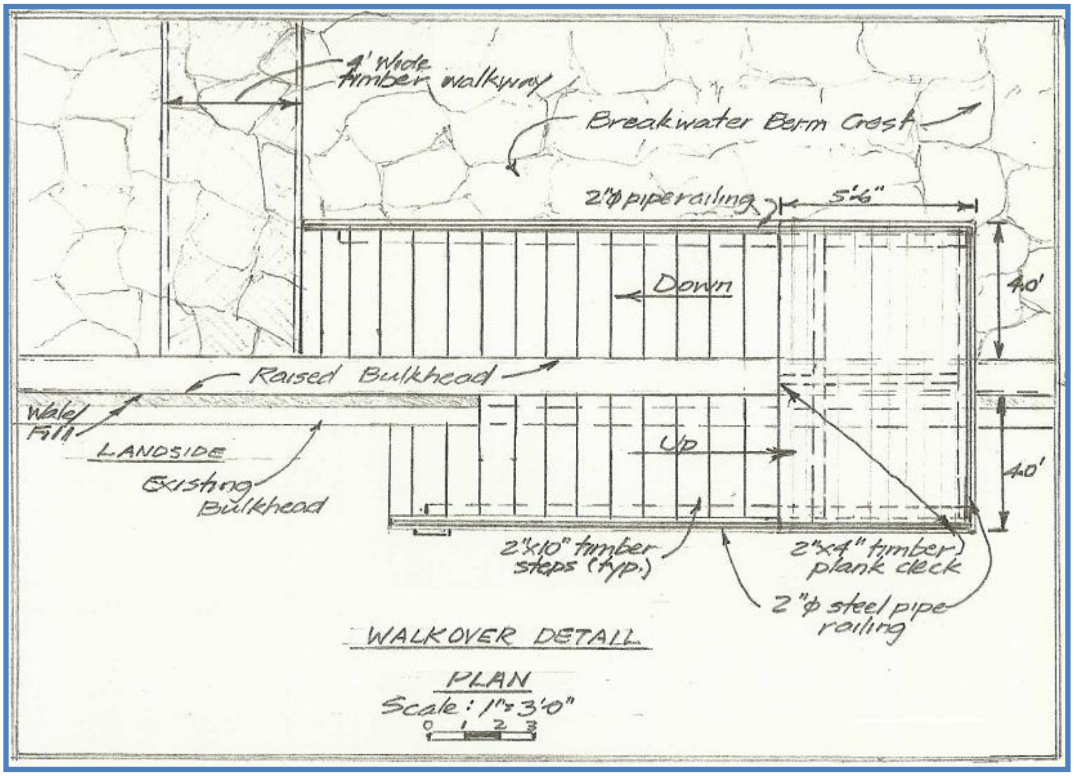


Figure 3: Typical Timber Stair Walkover Plan View.

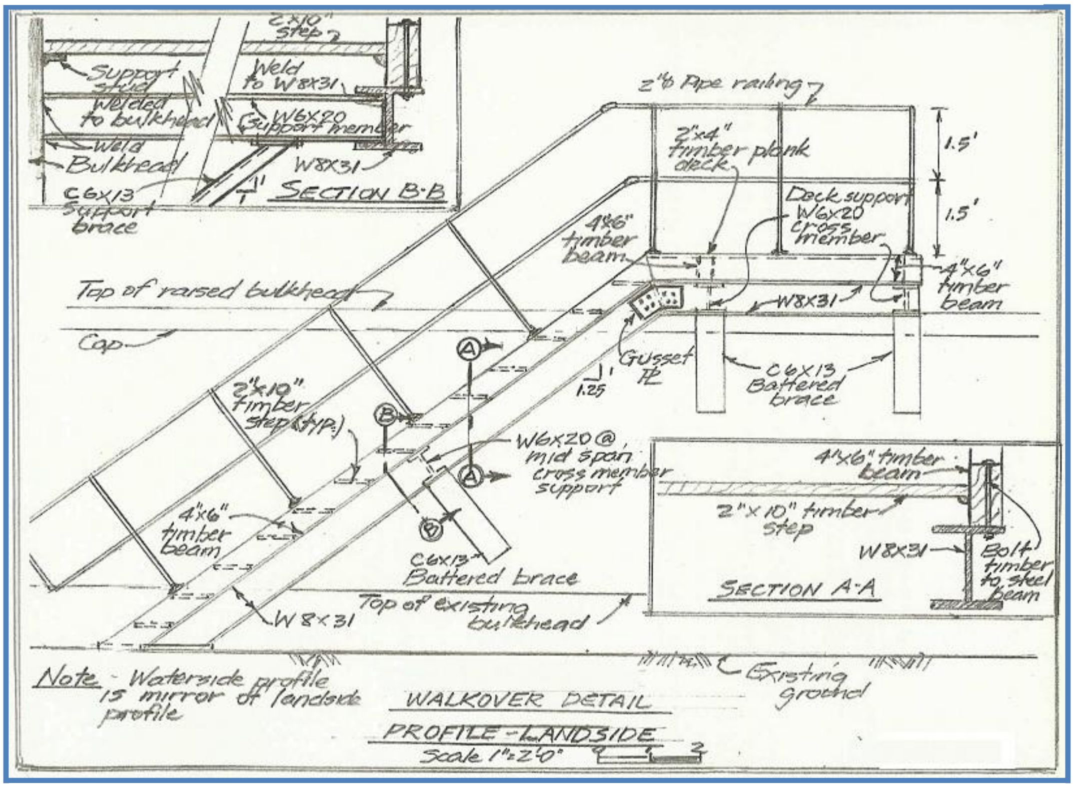


Figure 4: Typical Timber Stair Walkover Profile View.

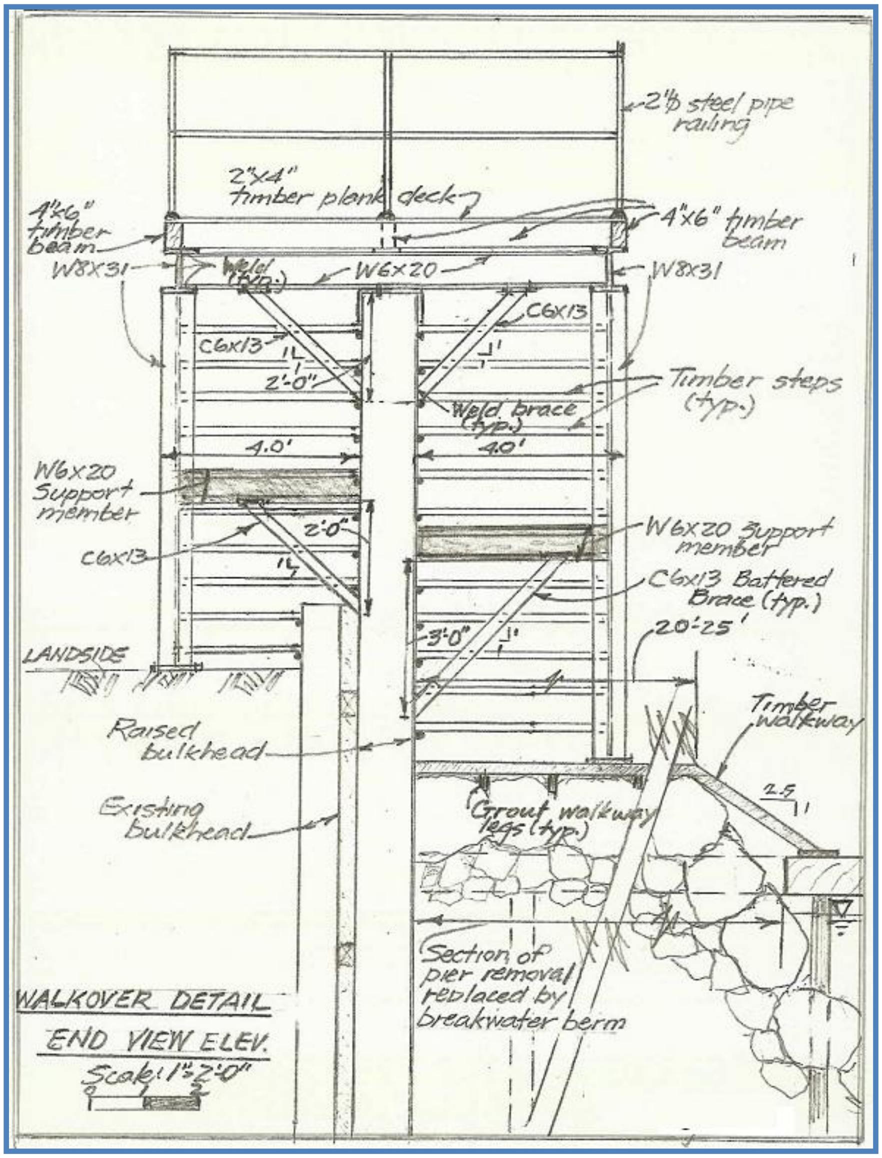


Figure 5: Timber Stair Walkover End View.

5.2 Earthen Ramp Walkover

Along reinforced dune features, construction of earthen ramp walkovers will be necessary to allow for continued access to seaside beaches. The preliminary location of earthen ramp walkovers can be seen on the individual alternative plan sheets. Figure 6 shows a typical earthen ramp walkover for the project.

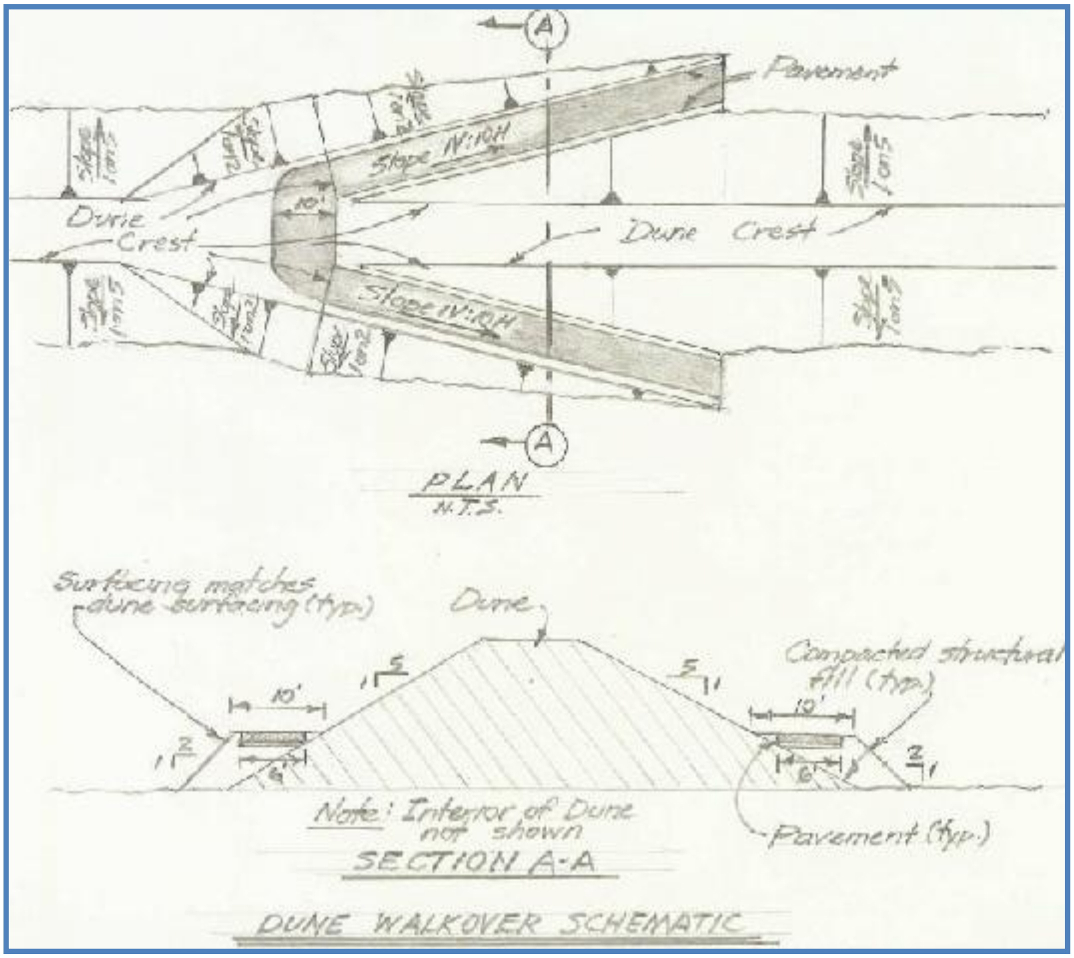


Figure 6: Typical Earthen Ramp Walkover.

Chapter 6: Right-of-Way

The proposed alternatives will require acquisition of a right-of-way corridor wide enough to allow for the footprint of all permanent design features as well as enough room for future flood event monitoring and recurring inspection activities. For the final alternatives array, the following assumptions for right-of-way acquisition were used:

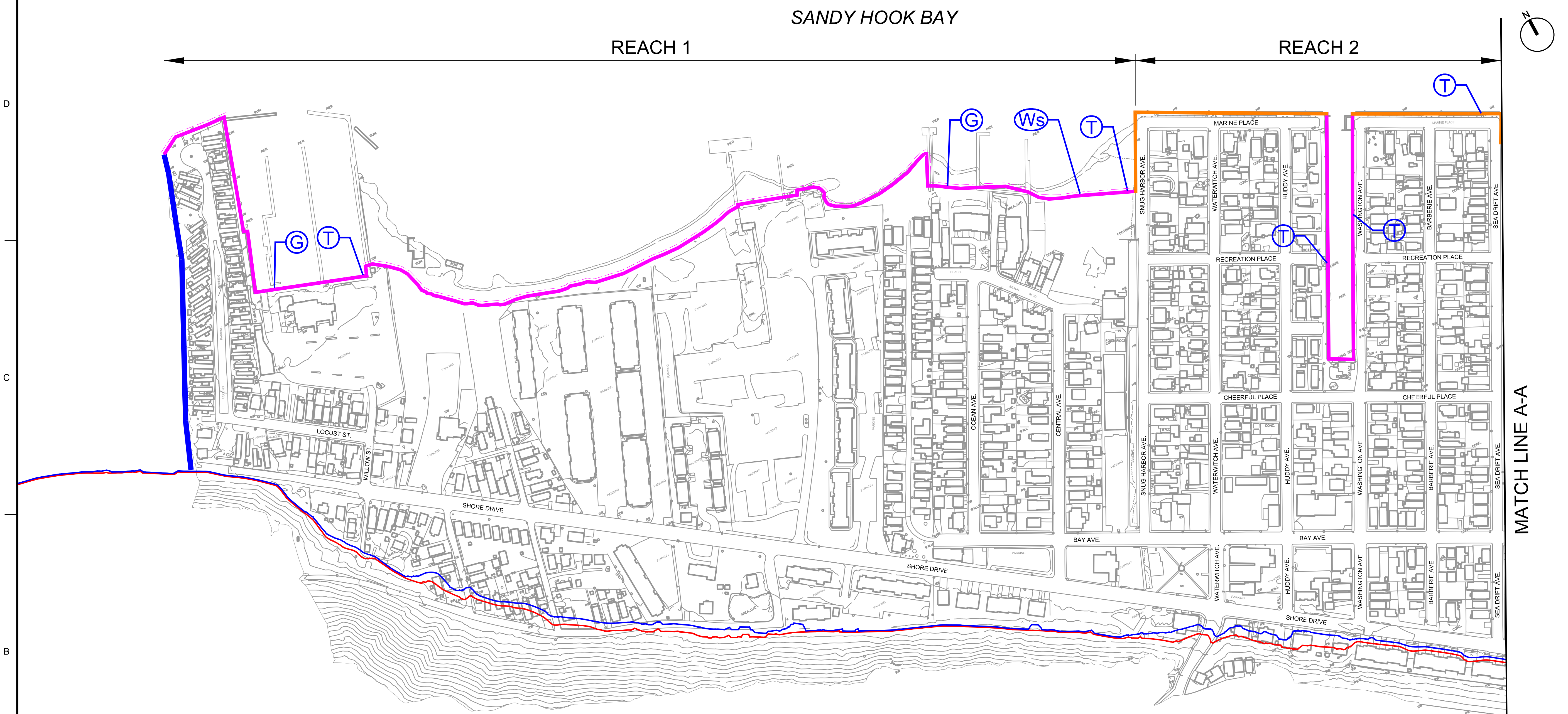
- Permanent easement: 15 feet outside of any permanent design feature.
- Temporary easement: 5 feet beyond the permanent easement for construction limits.

For raised bulkheads, permanent easement offsets are taken from the toe of the seaside breakwater. On the landside, offsets are taken from the back of the concrete splash pad. For reinforced dunes, permanent easement offsets are taken from the toe of the seaside and landside slopes. Permanent and temporary easements are shown on the project drawings for Alternatives 5A-5E. Two acres for contractor staging were assumed. The actual location(s) for contractor staging will need to be determined during a future phase of the project. See Appendix D, Real Estate Plan for additional right-of-way details.

Chapter 7: Utilities

Utility costs at this point in the study have been captured in the items associated with the "Minimum Facility Analysis". This analysis calls for the extension or modification of existing outlets through the alignment using flap gates, duck bill valves and control manholes with sluice gates. See Appendix X, Hydrology and Hydraulics for additional details on the "Minimum Facility Analysis".

It is not anticipated that many conflicts with utilities will be encountered since the majority of the proposed design features will be installed on the seaward side of the existing protection. However, as the design of the raised bulkhead feature is further refined during the course of the study, areas of potential conflict with existing utilities will need to be identified. In addition, recommendations resulting from the interior drainage analysis to be conducted after the TSP milestone may also require new infrastructure or modifications to existing infrastructure that could potentially conflict with existing utilities.



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- RAISED BULKHEAD WITH BREAKWATER BERM
- RAISED BULKHEAD
- I TYPE FLOODWALL
- G CLOSURE GATE
- T REPLACE EXISTING OUTLET WITH NEW TIDE GATE
- CAPPED EXISTING STATE BULKHEAD
- REMOVABLE FABRICATED FLOODWALL
- Ws TIMBER STAIR WALKOVER

0 300' 600'

SCALE: 1" = 300'

US Army Corps of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

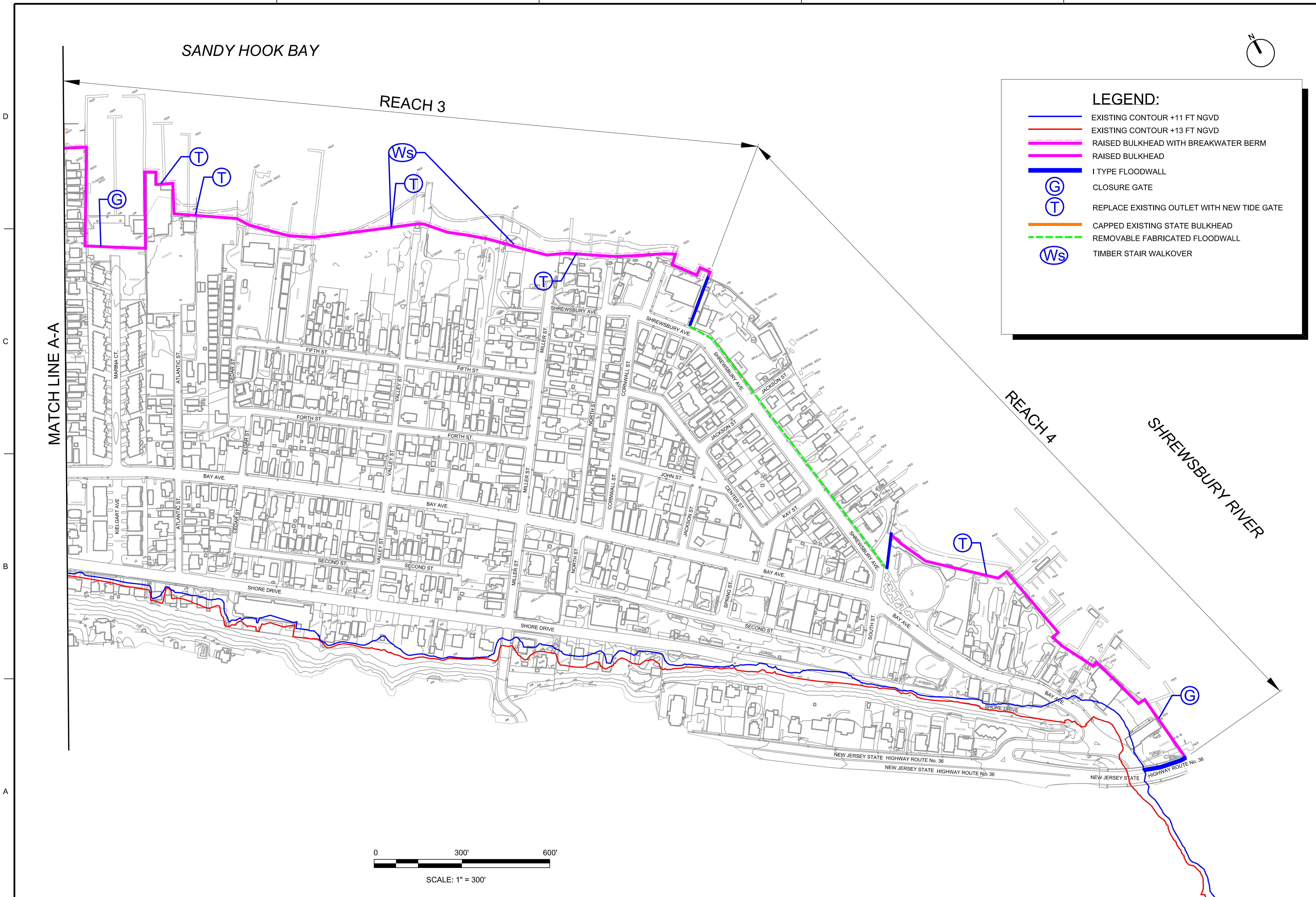
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 1_1 OF 2.dgn

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 1

SHEET IDENTIFICATION
CS101



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 1_2 OF 2.dgn

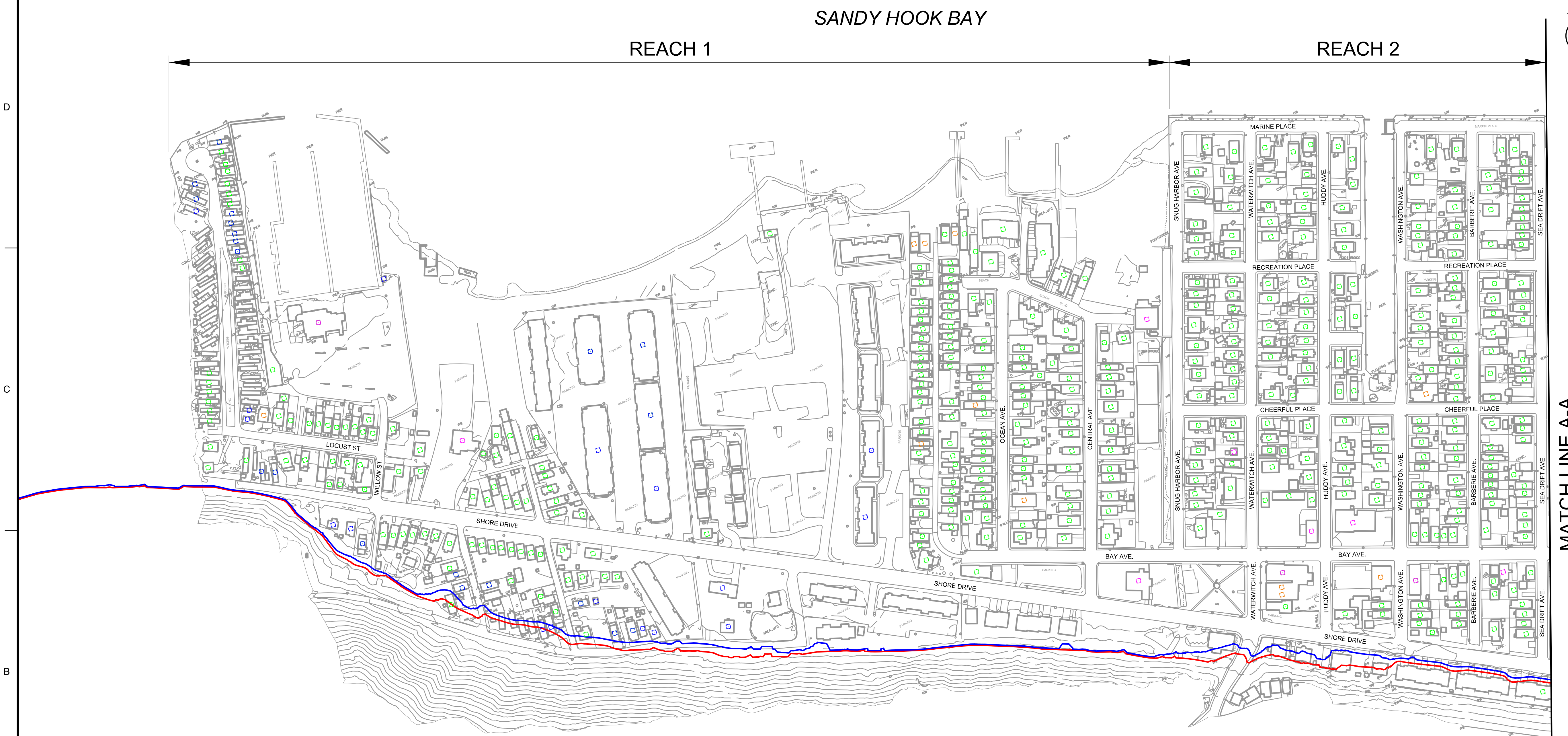
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 1

SHEET IDENTIFICATION
CS102



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- DRY FLOODPROOFING
- WET FLOODPROOFING
- RAISE STRUCTURE
- RINGWALL AROUND PROPERTY
- FLOODWALL AROUND STRUCTURE SURFACE

0 300' 600'

SCALE: 1" = 300'

US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
PLOT SCALE:
PLOT DATE: No
SIZE:
FILE NAME: HLND_TSP ALT 2_1 OF 2.dgn
DATE:
SOLICITATION NO.:
CONTRACT NO.:
FILE NUMBER:

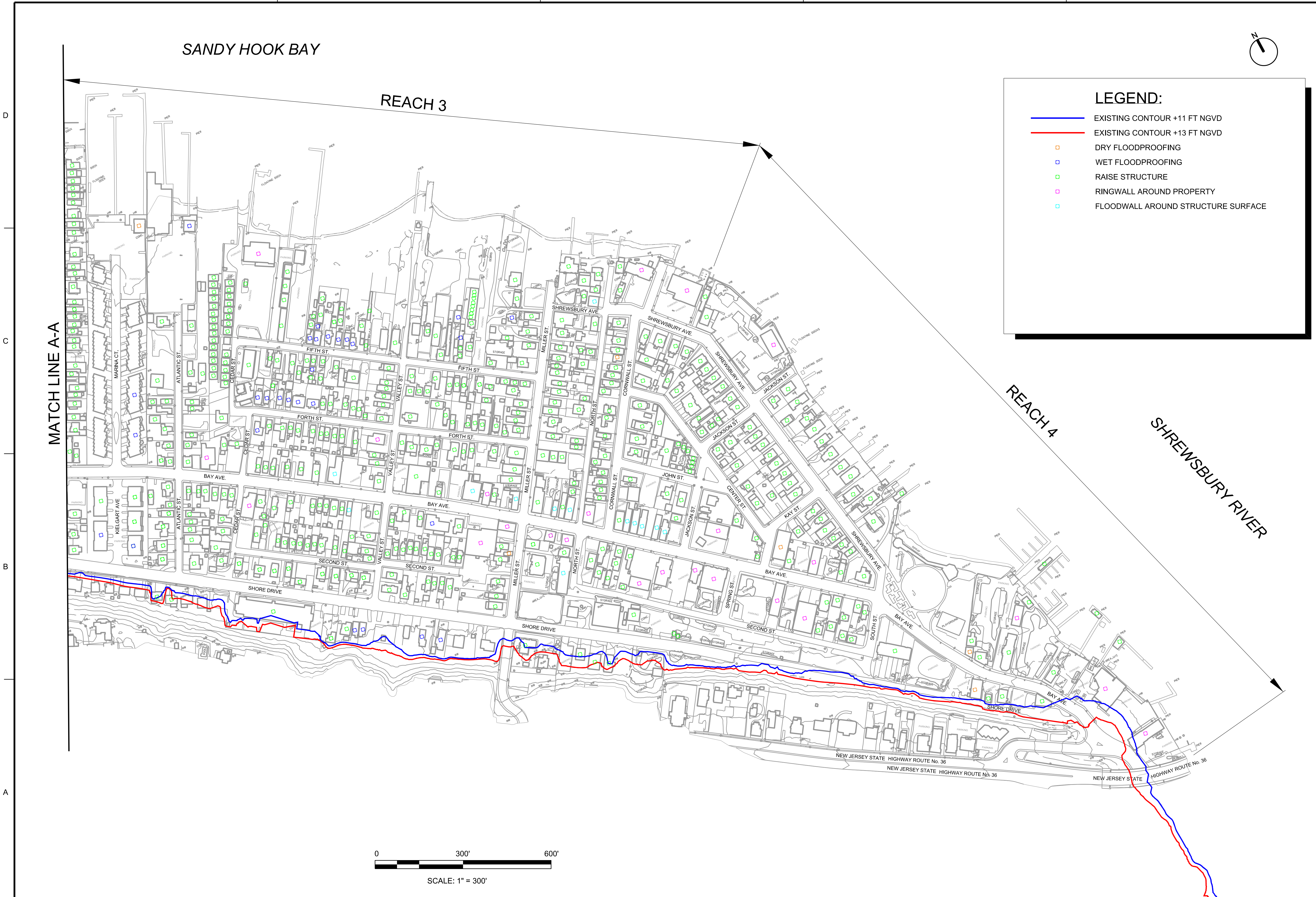
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 2

SHEET IDENTIFICATION
CS103



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

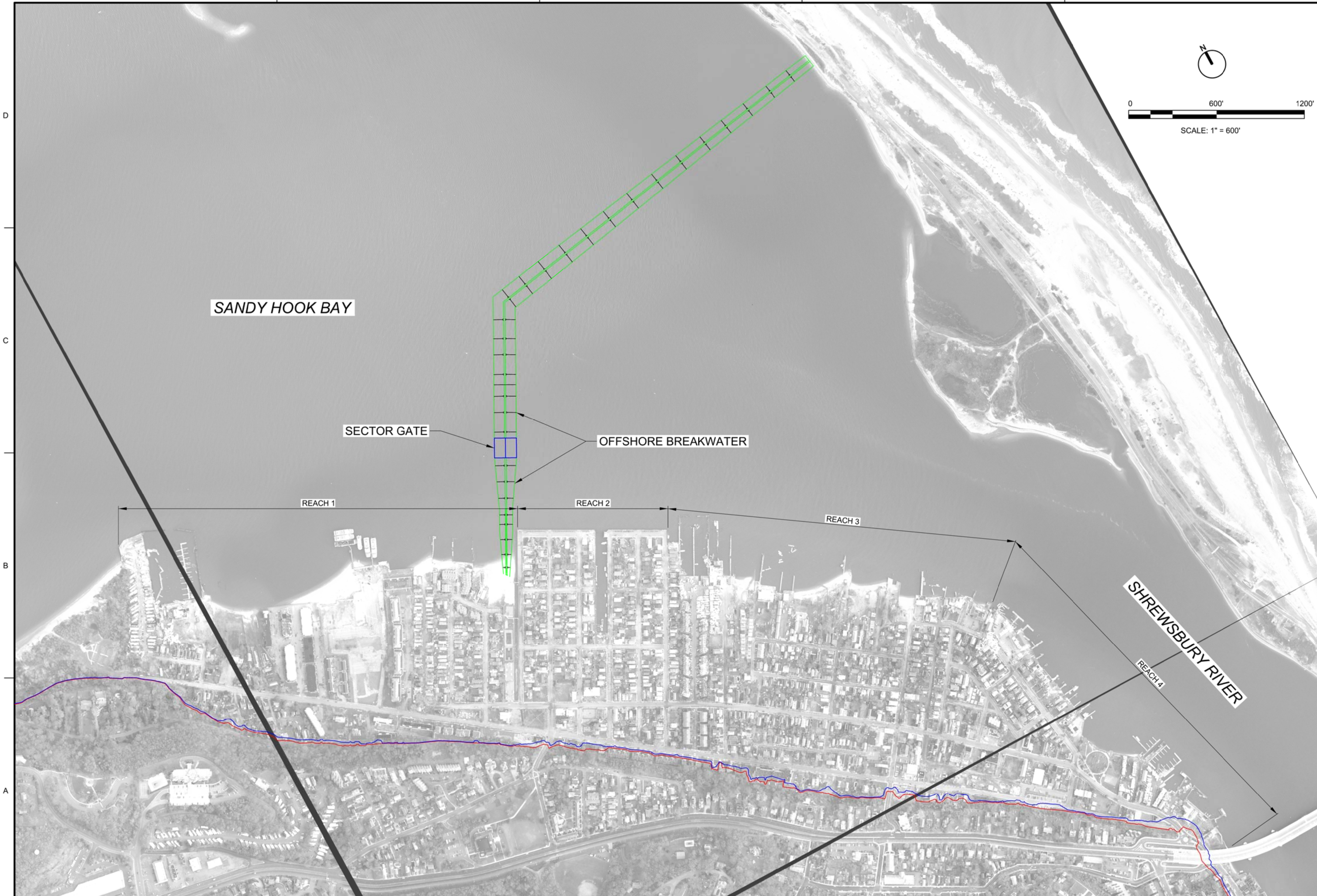
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
PLOT SCALE:
PLOT DATE: No
SIZE:
FILE NAME: HLND_TSP ALT 2_2 OF 2.dgn
DATE:
SOLICITATION NO.:
CONTRACT NO.:
FILE NUMBER:

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 2

SHEET IDENTIFICATION
CS104



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

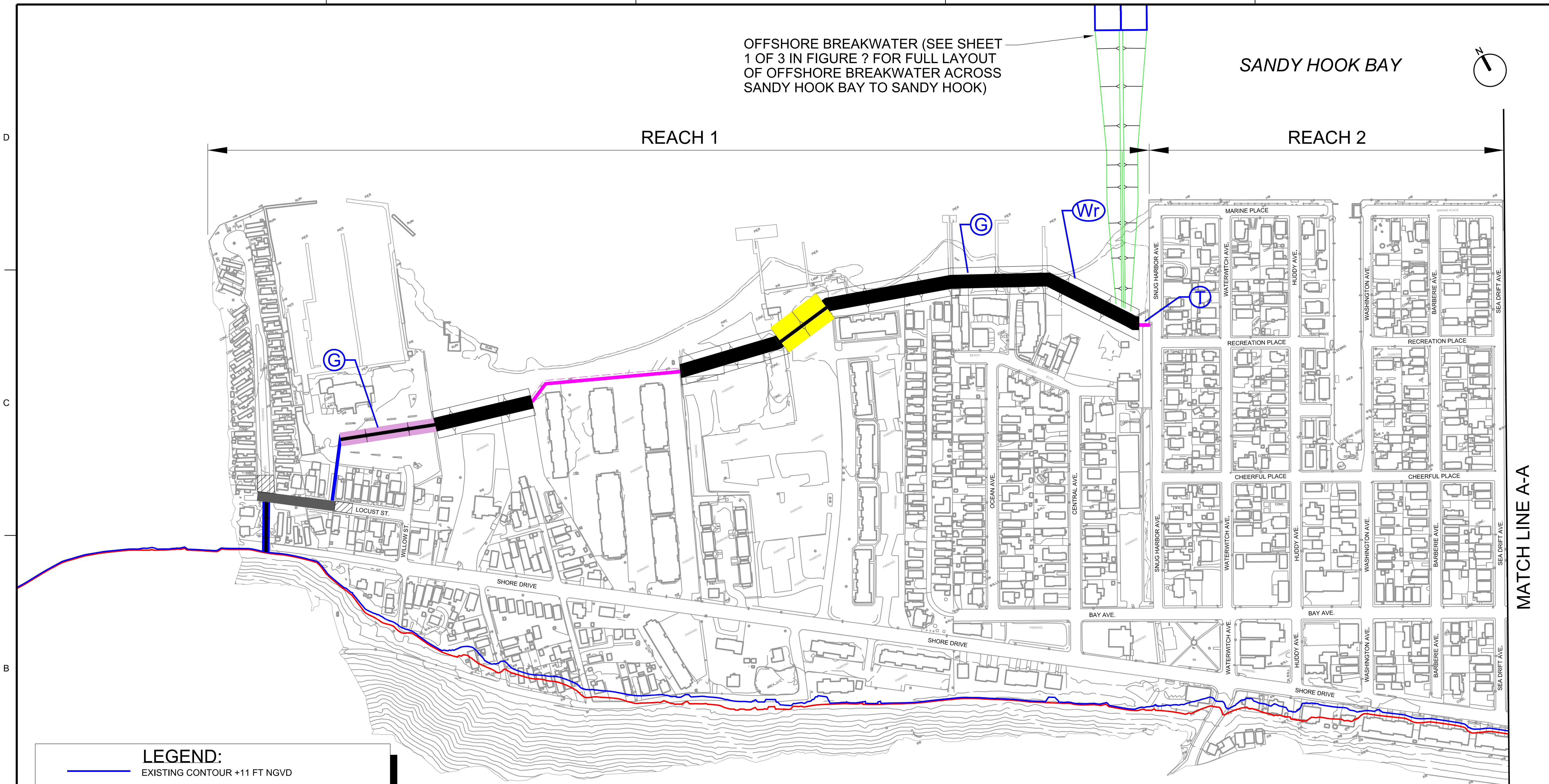
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY:		DATE:
DWN BY:	CKD BY:	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE:	PLOT DATE: No	FILE NUMBER:
SIZE:	FILE NAME: HLND_TSP ALT 3_1 OF 3.dgn	

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 3

SHEET IDENTIFICATION
CS105



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- RAISED BULKHEAD WITH BREAKWATER BERM
- RAISED BULKHEAD
- SECTOR GATE
- I TYPE FLOODWALL
- T — REPLACE EXISTING OUTLET WITH NEW TIDE GATE
- ONSHORE DUNE BARRIER WITH BURIED REVETMENT
- RAISED ASPHALT PARKING AREA
- RAISED GROUND SURFACE
- RAISED ROADWAY
- TRANSITION ROAD APPROACHES
- G — CLOSURE GATE
- Wr — EARTHEN RAMP WALKOVER
- Ws — TIMBER STAIR WALKOVER

US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

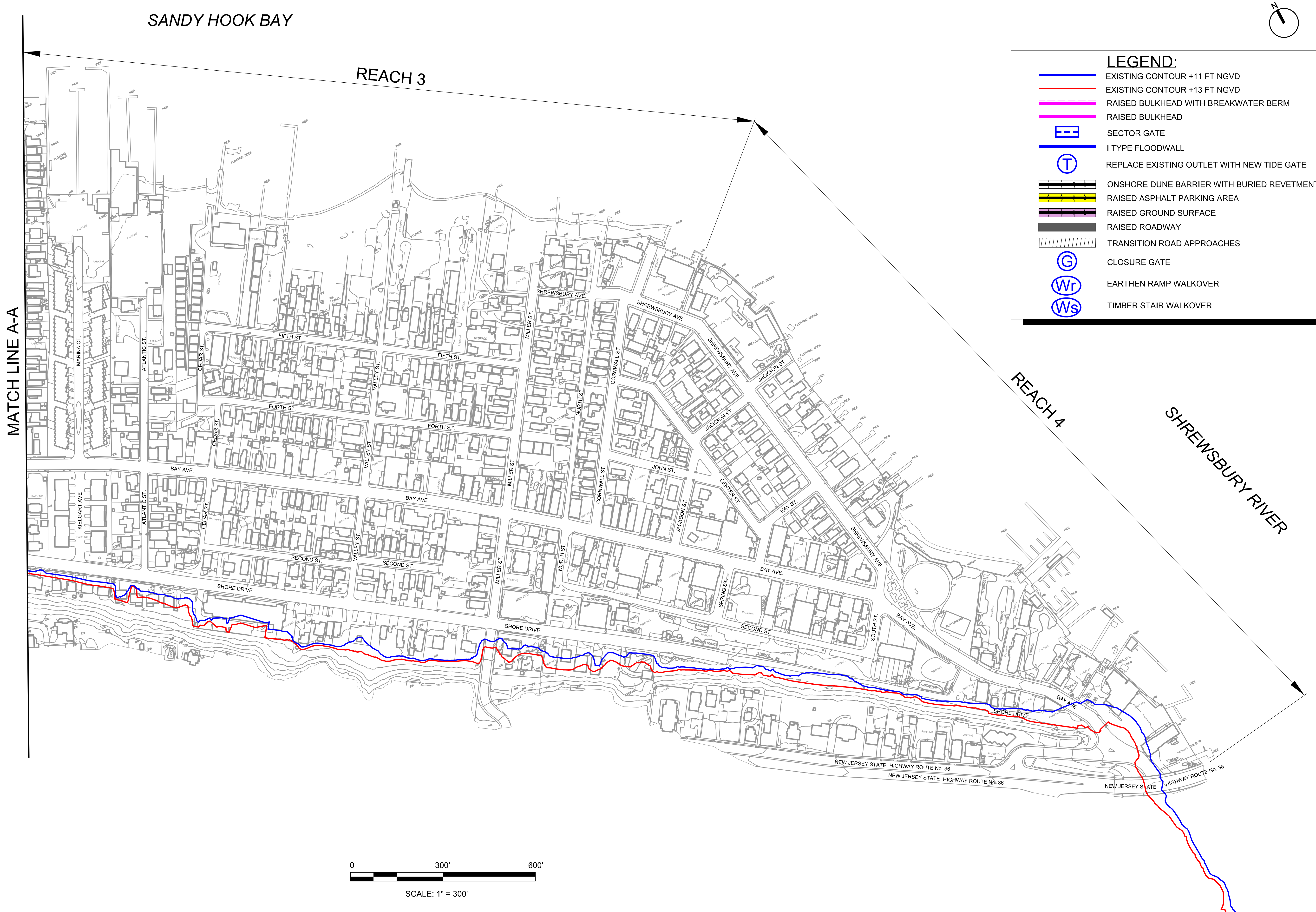
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 3_2 OF 3.dgn

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 3

SHEET IDENTIFICATION
CS106



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- RAISED BULKHEAD WITH BREAKWATER BERM
- RAISED BULKHEAD
- SECTOR GATE
- I TYPE FLOODWALL
- T REPLACE EXISTING OUTLET WITH NEW TIDE GATE
- ONSHORE DUNE BARRIER WITH BURIED REVETMENT
- RAISED ASPHALT PARKING AREA
- RAISED GROUND SURFACE
- RAISED ROADWAY
- TRANSITION ROAD APPROACHES
- G CLOSURE GATE
- Wr EARTHEN RAMP WALKOVER
- Ws TIMBER STAIR WALKOVER

US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY:		DATE:
DWN BY:	CKD BY:	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE:	PLOT DATE: No	FILE NUMBER:
SIZE:	FILE NAME: HLND_TSP ALT 3_3 OF 3.dgn	

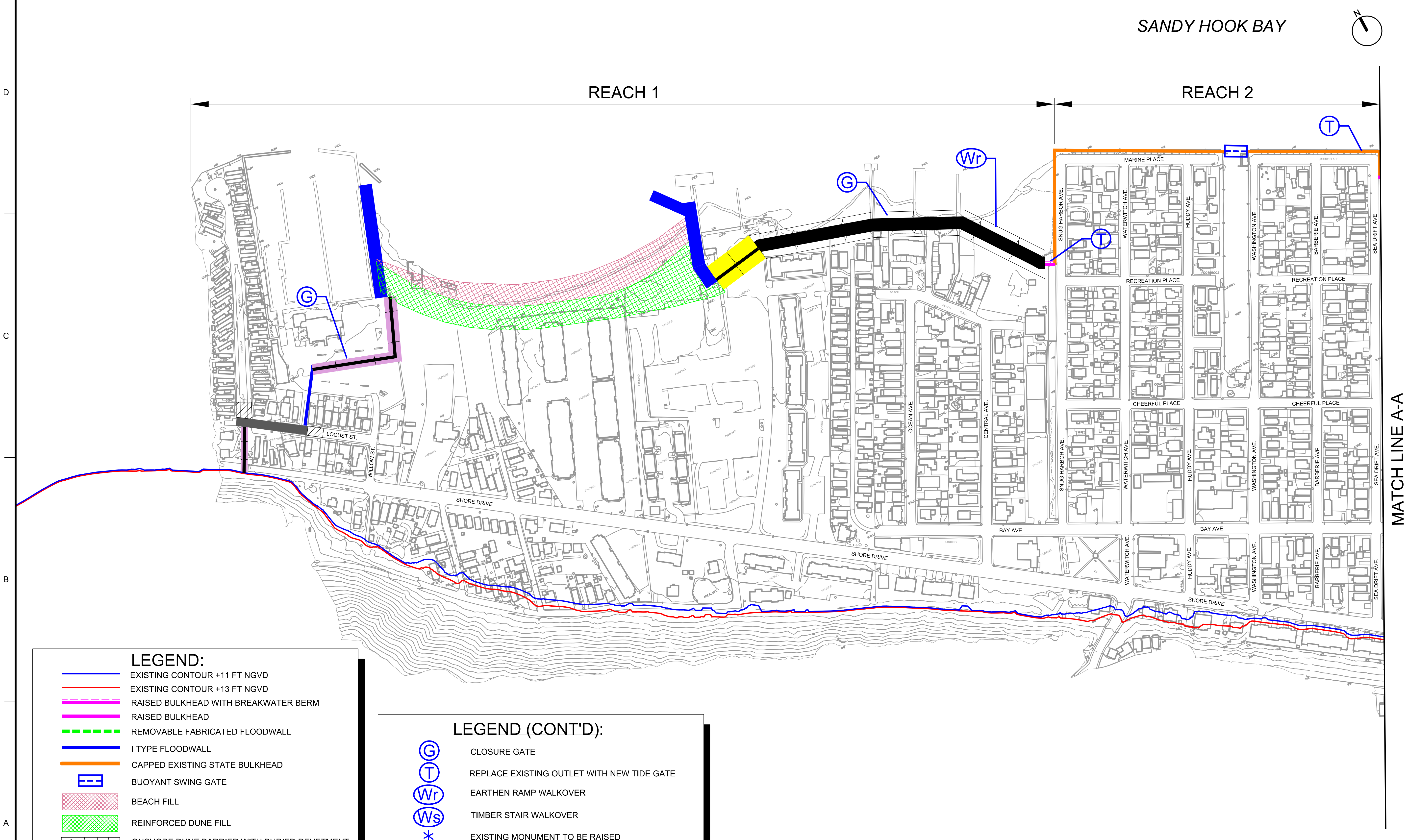
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 3

SHEET IDENTIFICATION
CS107



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- RAISED BULKHEAD WITH BREAKWATER BERM
- RAISED BULKHEAD
- REMOVABLE FABRICATED FLOODWALL
- I TYPE FLOODWALL
- CAPPED EXISTING STATE BULKHEAD
- BUOYANT SWING GATE
- BEACH FILL
- REINFORCED DUNE FILL
- ONSHORE DUNE BARRIER WITH BURIED REVETMENT
- RAISED ASPHALT PARKING AREA
- RAISED GROUND SURFACE
- RAISED ROADWAY
- TRANSITION ROAD APPROACHES
- TERMINAL GROIN

LEGEND (CONT'D):

- G — CLOSURE GATE
- T — REPLACE EXISTING OUTLET WITH NEW TIDE GATE
- Wr — EARTHEN RAMP WALKOVER
- Ws — TIMBER STAIR WALKOVER
- * — EXISTING MONUMENT TO BE RAISED

0 300' 600'

SCALE: 1" = 300'

US Army Corps of Engineers®
New York District

MARK	DATE	APPR.	DESCRIPTION

MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

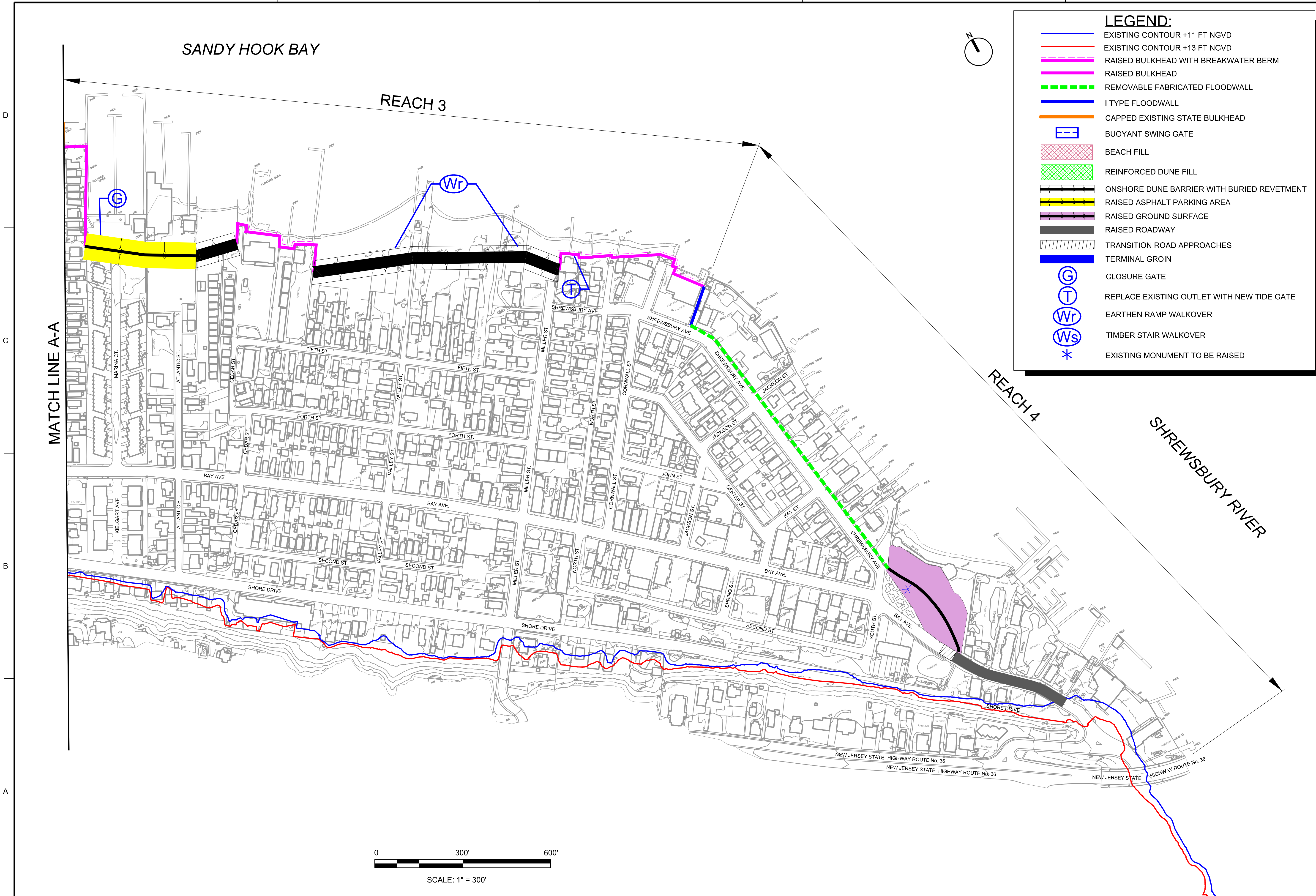
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 4_1 OF 2.dgn

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 4

SHEET IDENTIFICATION
CS108



US Army Corps of Engineers®
New York District

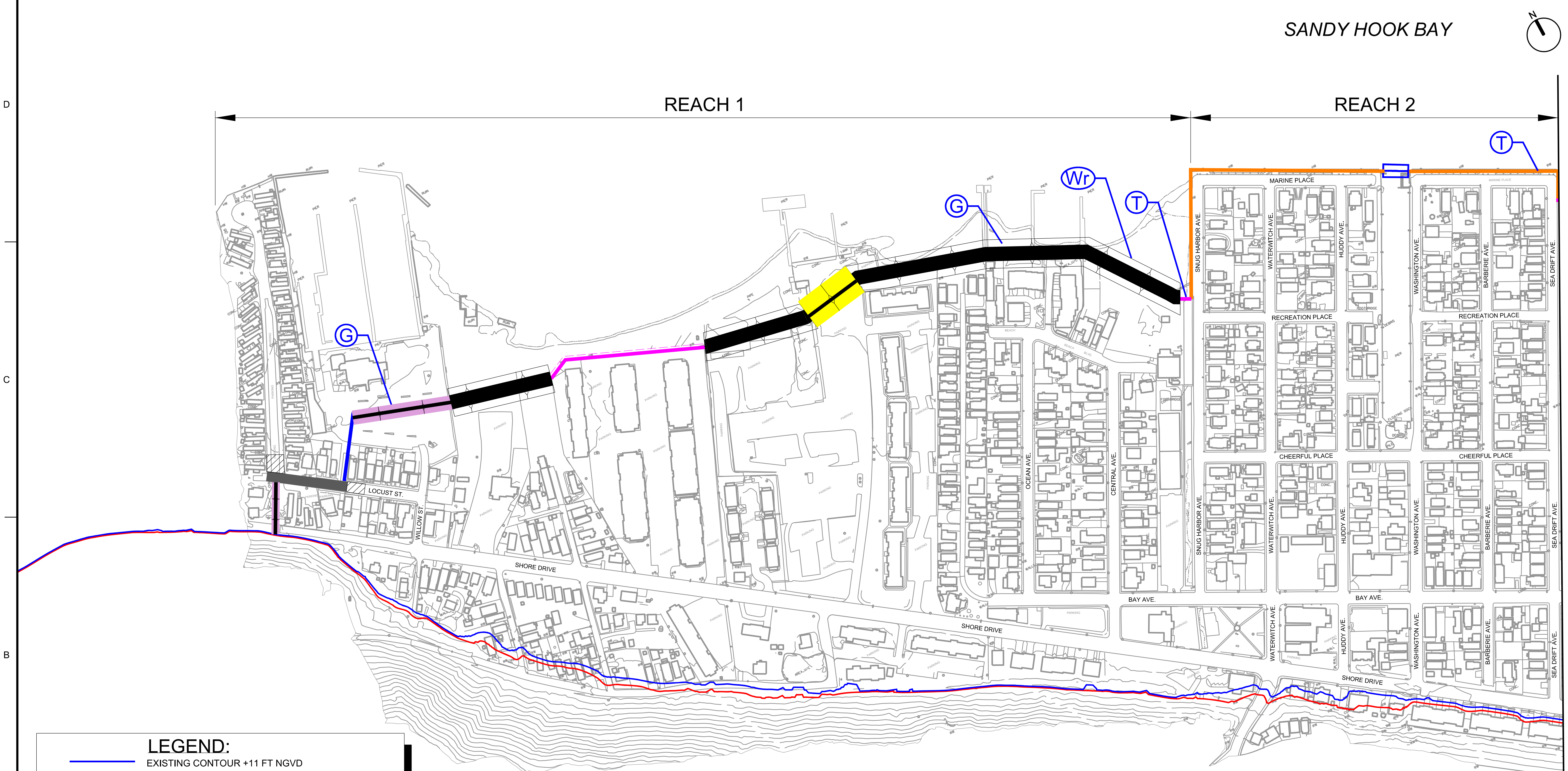
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 4

SHEET IDENTIFICATION
CS109



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- RAISED BULKHEAD WITH BREAKWATER BERM
- RAISED BULKHEAD
- REMOVABLE FABRICATED FLOODWALL
- I TYPE FLOODWALL
- CAPPED EXISTING STATE BULKHEAD
- BUOYANT SWING GATE
- ONSHORE DUNE BARRIER WITH BURIED REVETMENT
- RAISED ASPHALT PARKING AREA
- RAISED GROUND SURFACE
- RAISED ROADWAY
- TRANSITION ROAD APPROACHES

LEGEND (CONT'D):

- G CLOSURE GATE
- T REPLACE EXISTING OUTLET WITH NEW TIDE GATE
- Wr EARTHEN RAMP WALKOVER
- Ws TIMBER STAIR WALKOVER
- * EXISTING MONUMENT TO BE RAISED

0 300' 600'

SCALE: 1" = 300'

US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5_1 OF 2.dgn

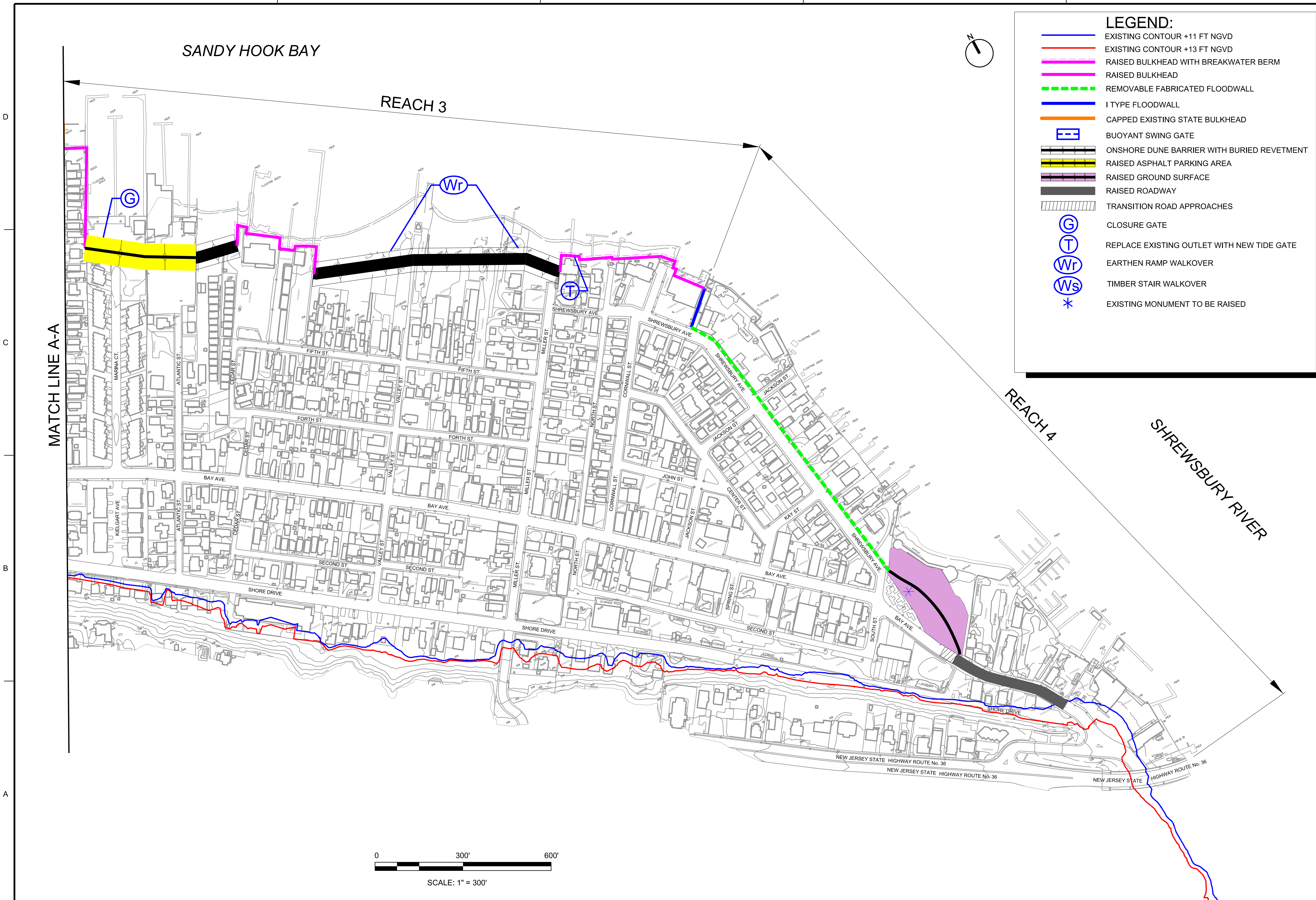
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5

SHEET IDENTIFICATION
CS110



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR.
MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5_2 OF 2.dgn

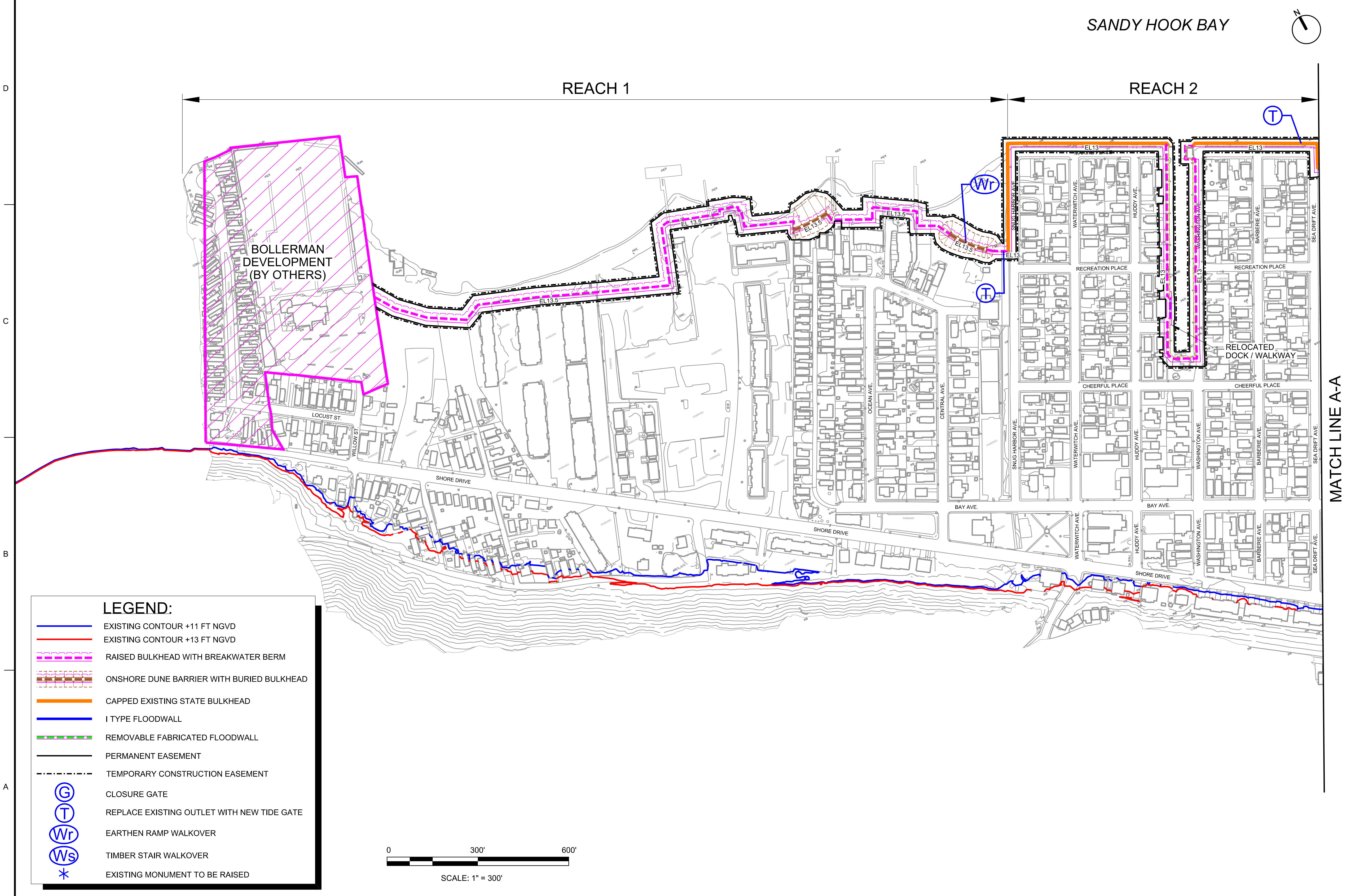
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5

SHEET IDENTIFICATION
CS111



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

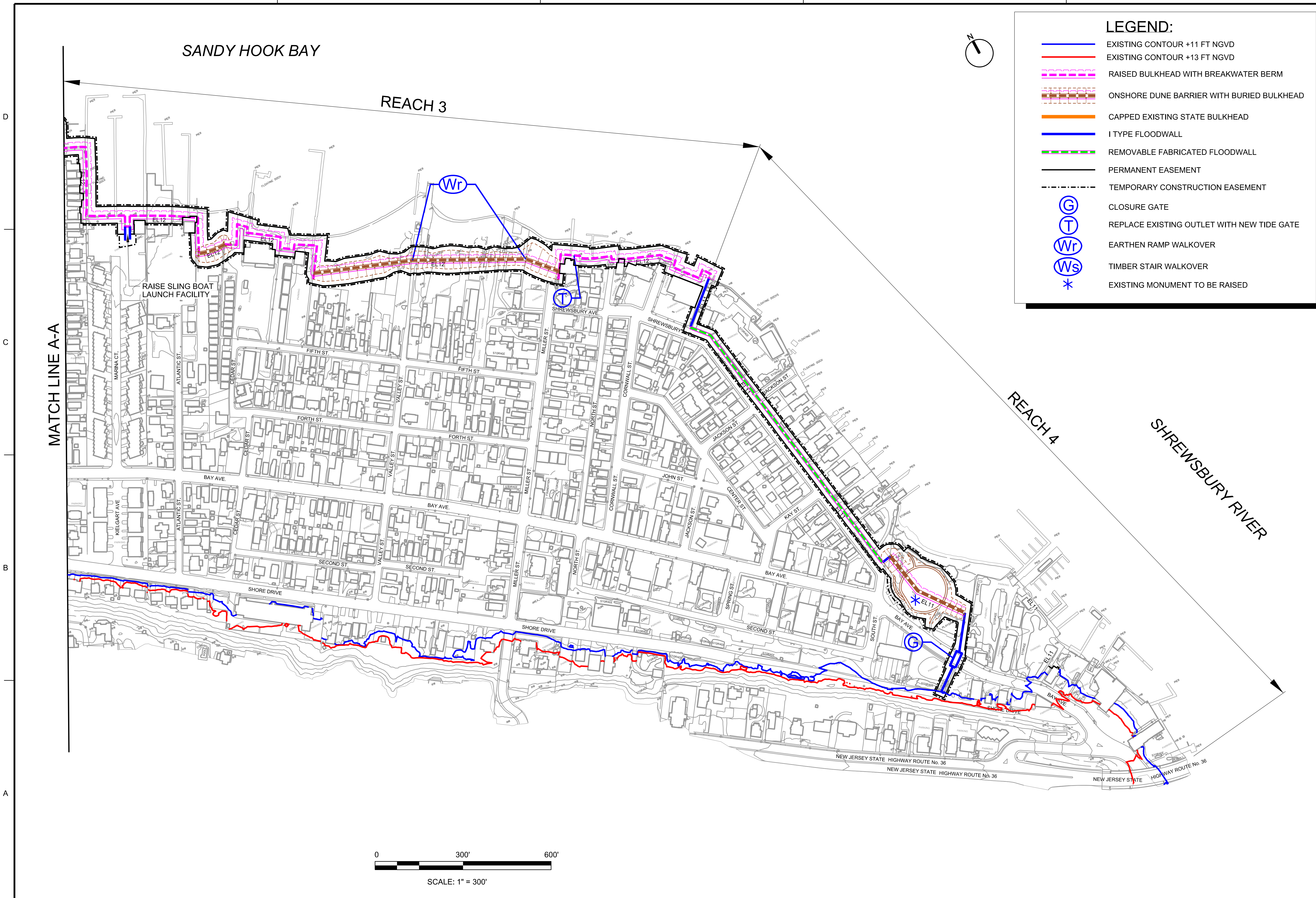
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5A_1 OF 2.dgn

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5A

SHEET IDENTIFICATION
CS112



US Army Corps of Engineers® New York District

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

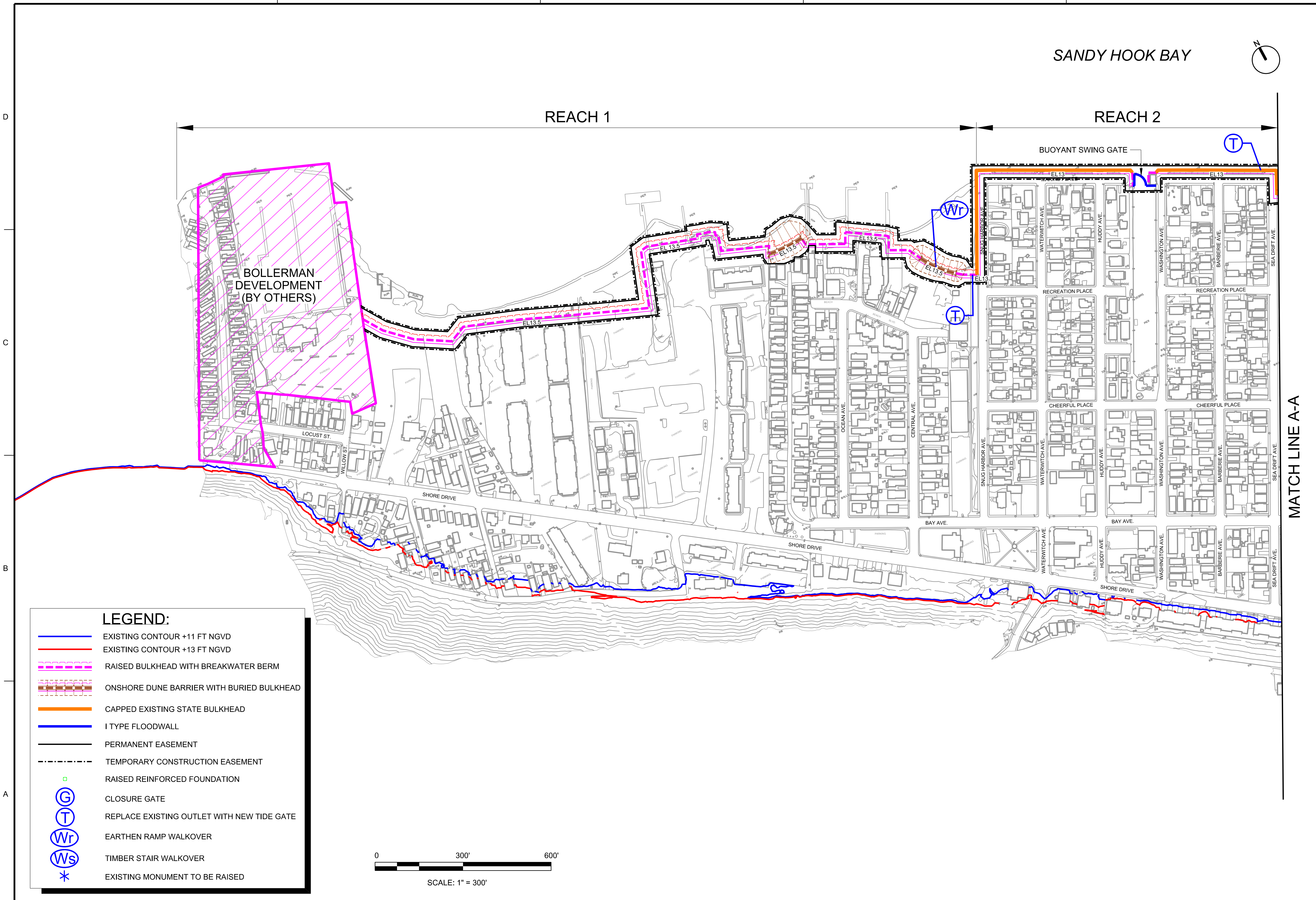
DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5A_2 OF 2.dgn

MARK | DESCRIPTION | DATE | APPR

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5A

SHEET IDENTIFICATION
CS113



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

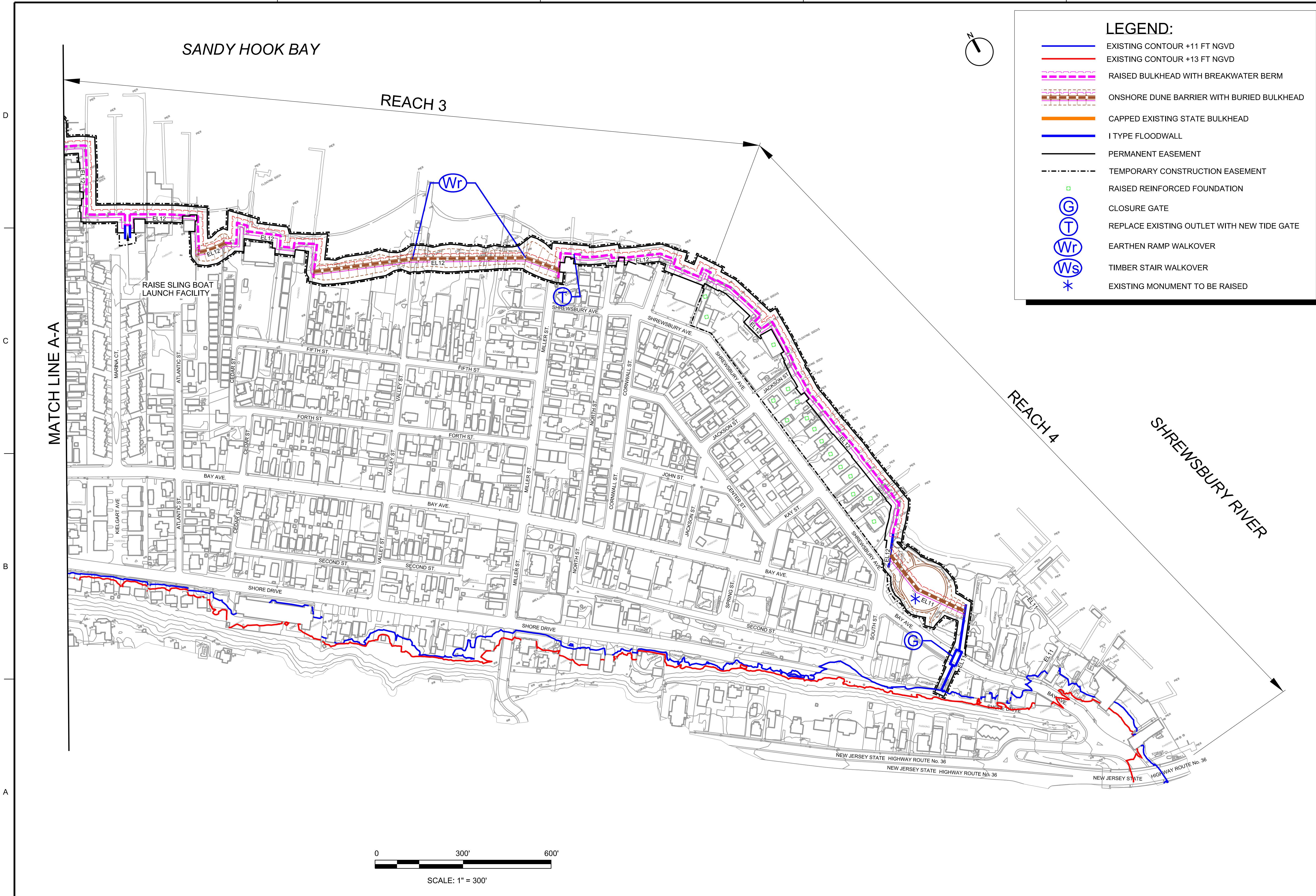
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5B_1 OF 2.dgn

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5B

SHEET IDENTIFICATION
CS114



US Army Corps of Engineers® New York District

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

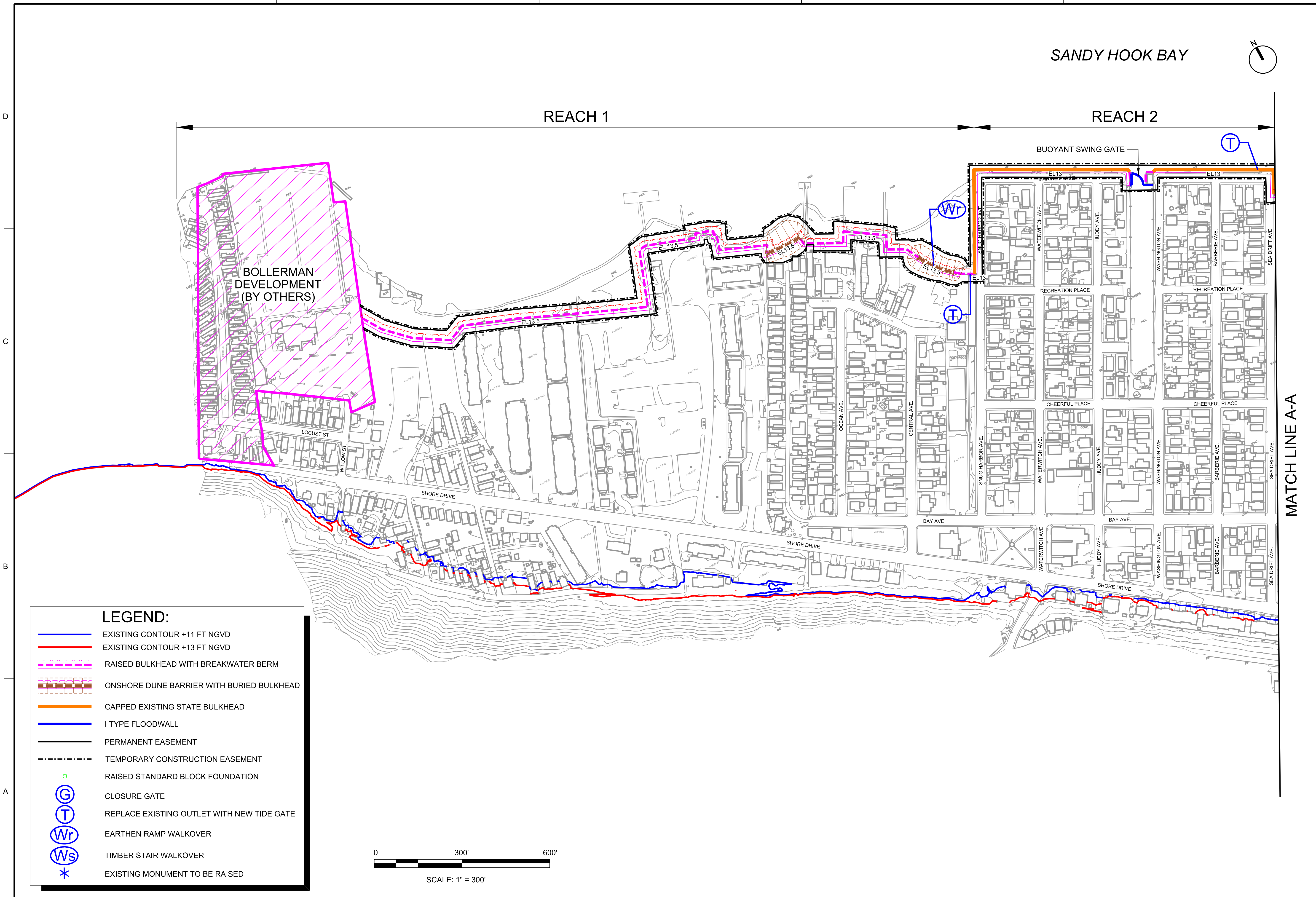
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY:		DATE:
DWN BY:	CKD BY:	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE:	PLOT DATE: No	FILE NUMBER:
SIZE:	FILE NAME: HLND_TSP ALT 5B_2 OF 2.dgn	

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5B

SHEET IDENTIFICATION
CS115



US Army Corps of Engineers® New York District

MARK | DATE | DESCRIPTION | APPR

DESIGNED BY: | DWN BY: | CKD BY: | SUBMITTED BY: | PLOT SCALE: | PLOT DATE: | SIZE: | FILE NAME: HLND_TSP ALT 5C_1 OF 2.dgn | DATE: | SOLICITATION NO.: | CONTRACT NO.: | FILE NUMBER:

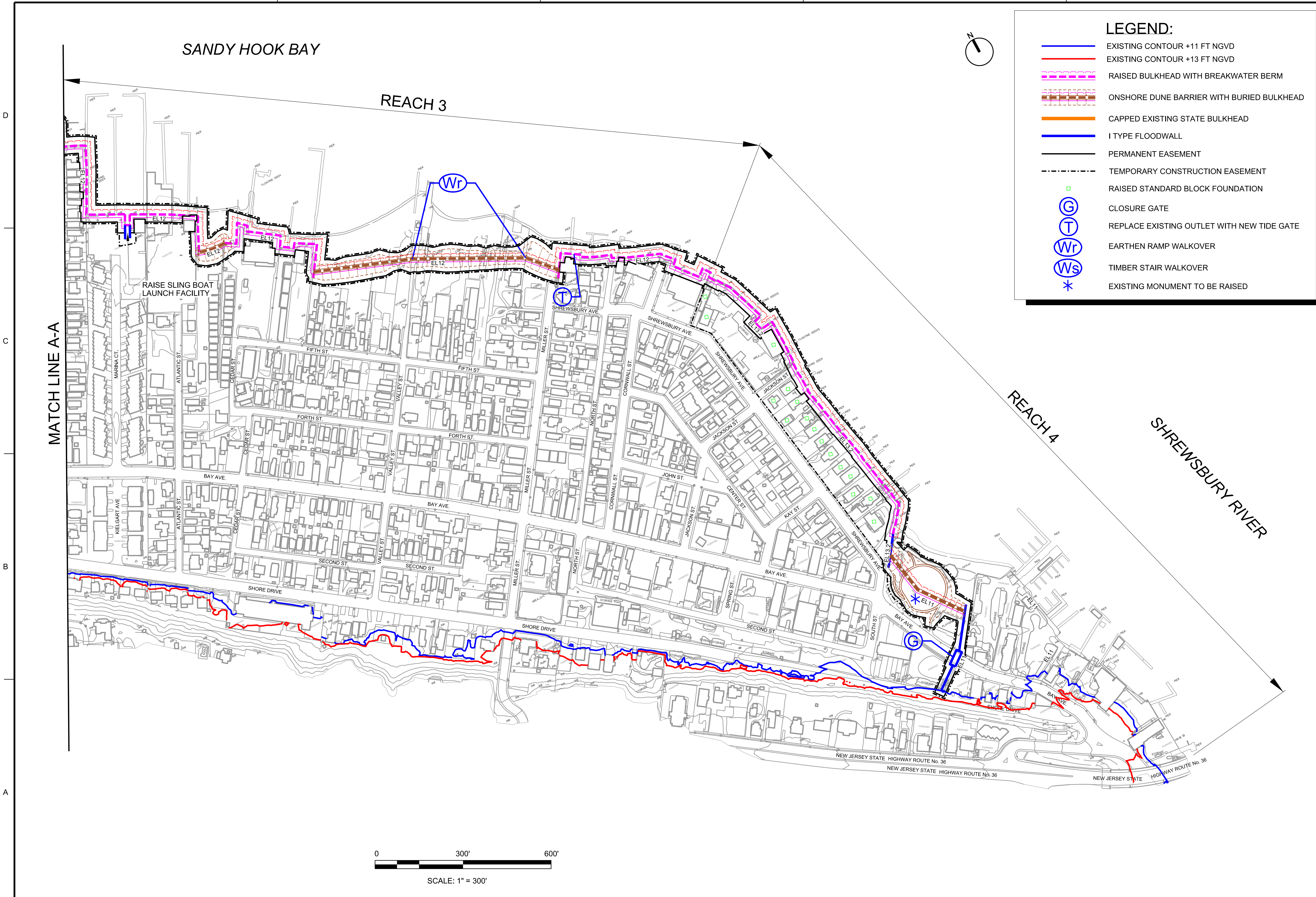
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5C

SHEET IDENTIFICATION
CS116



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5C_2 OF 2.dgn

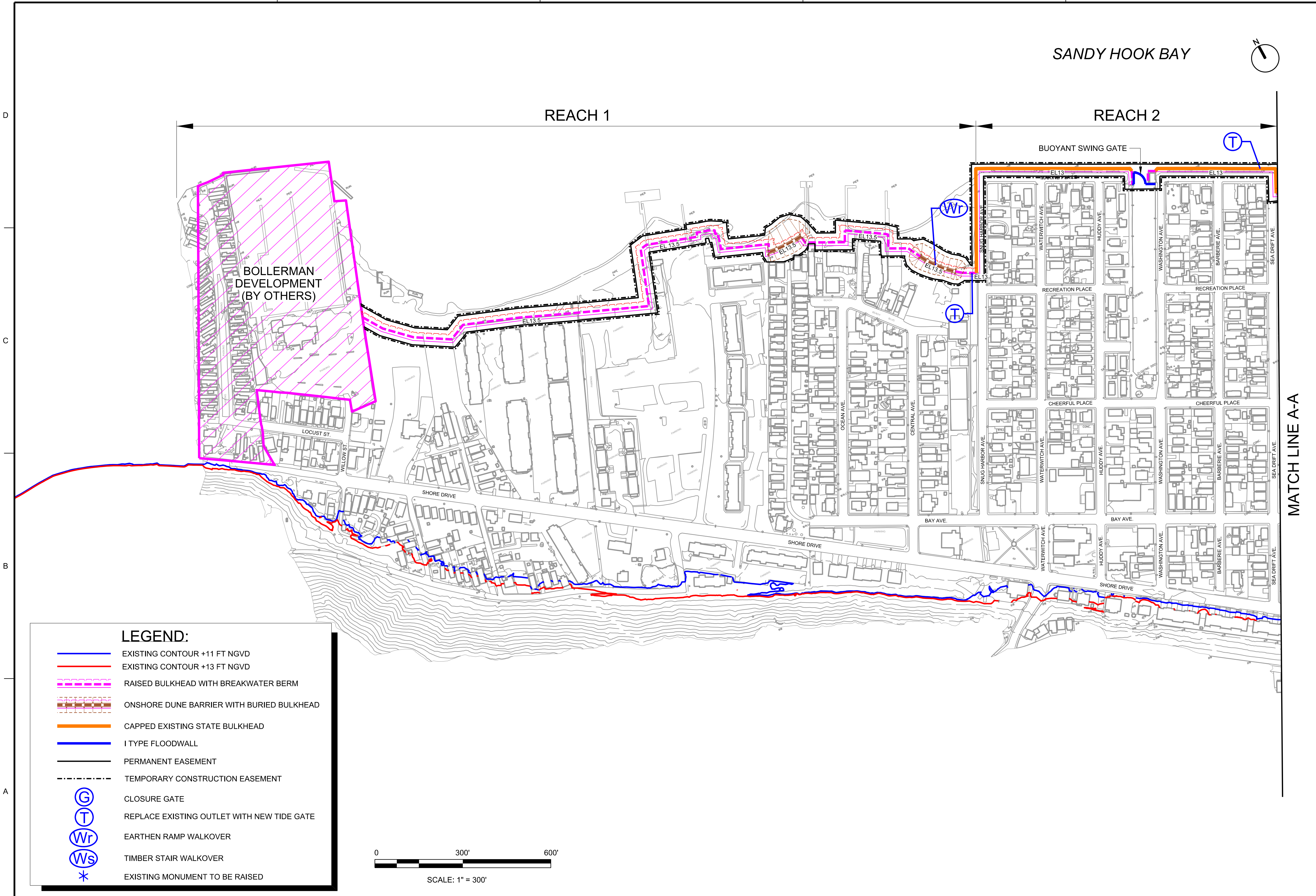
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5C

SHEET IDENTIFICATION
CS117



US Army Corps of Engineers®
New York District

MARK	DATE	DESCRIPTION	APPR

MARK	DATE	DESCRIPTION	APPR

DESIGNED BY:		DATE:
DWN BY:	CKD BY:	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE:	PLOT DATE: No	FILE NUMBER:
SIZE:	FILE NAME: HLND_TSP ALT 5D_1 OF 2.dgn	

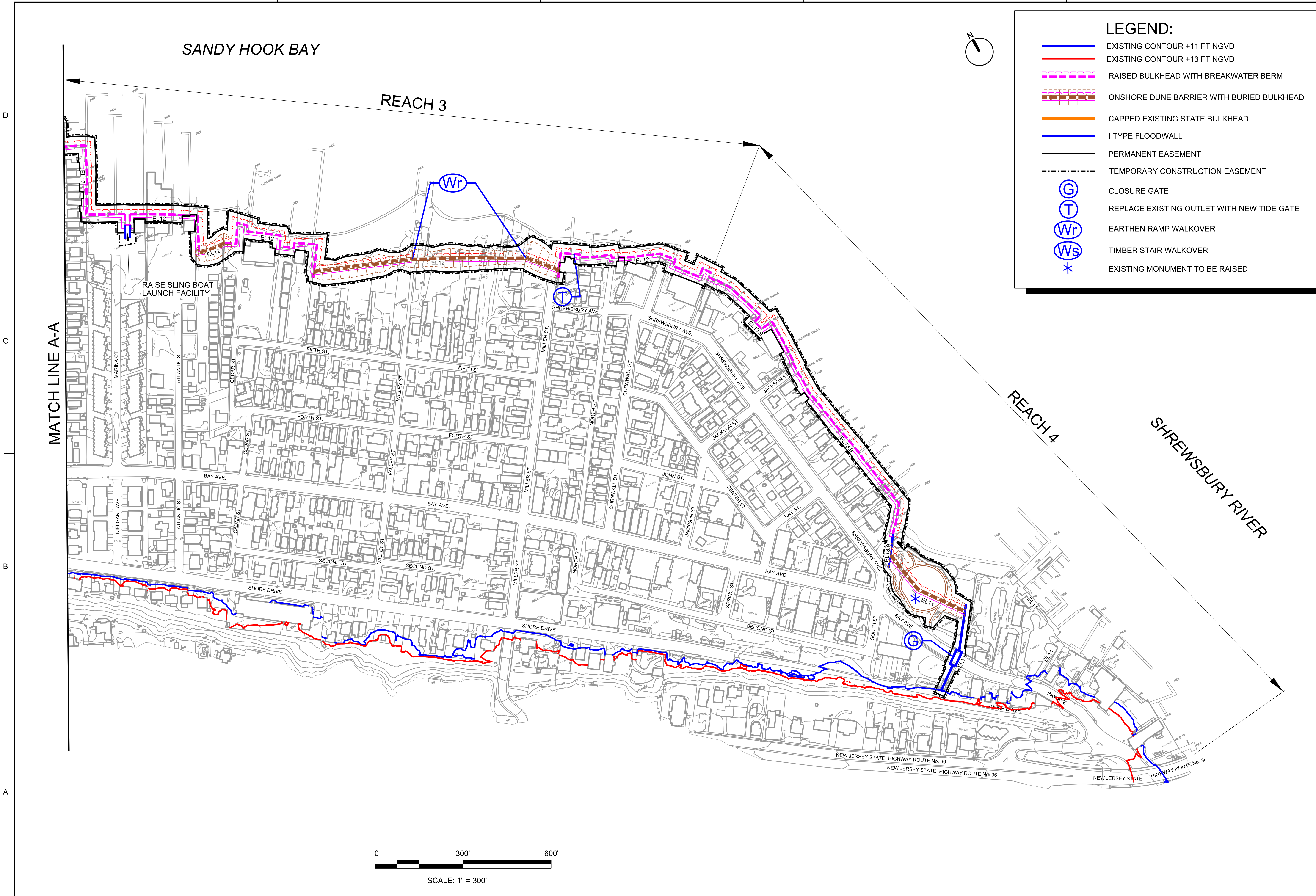
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5D

SHEET IDENTIFICATION
CS118



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

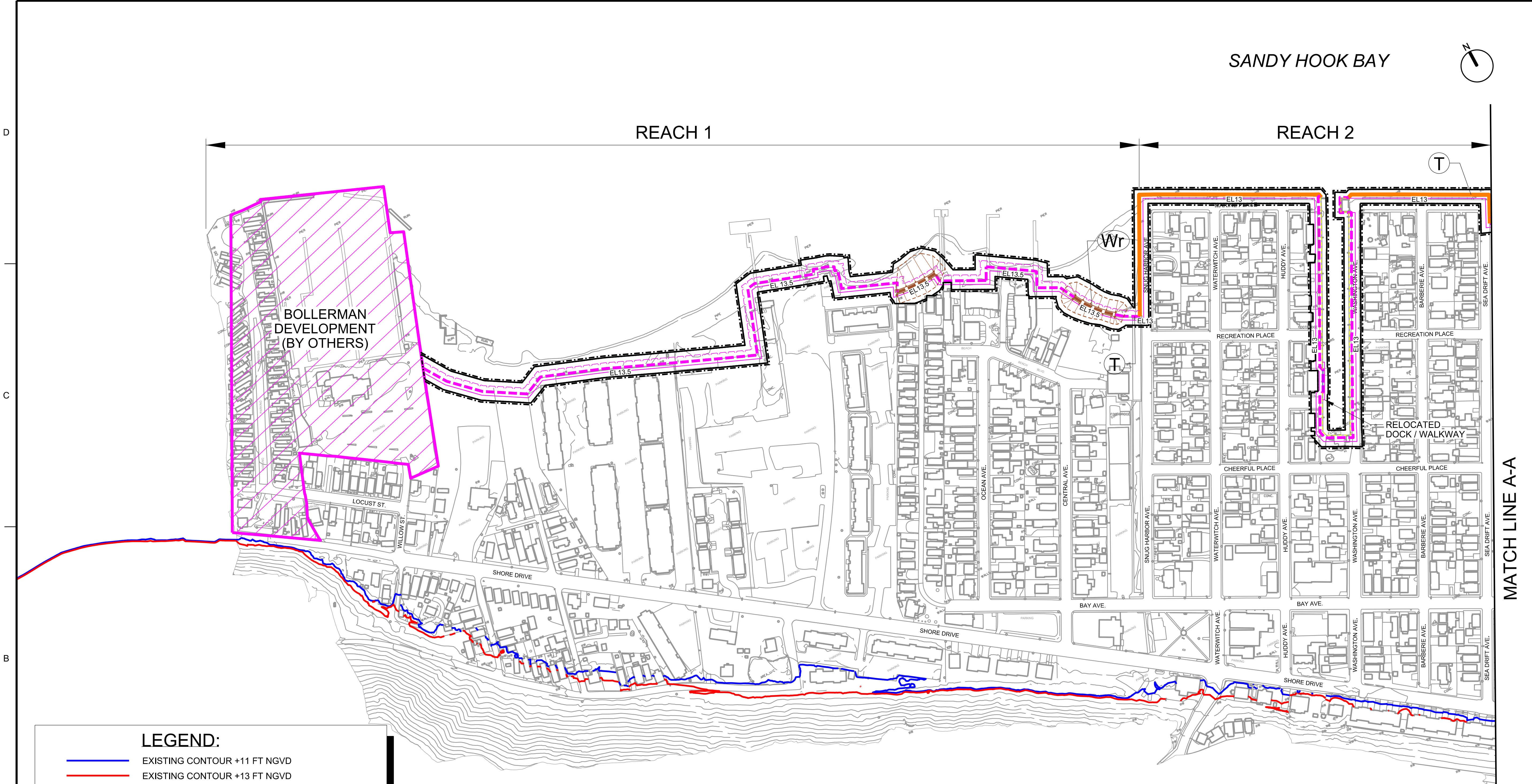
U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY:		DATE:
DWN BY:	CKD BY:	SOLICITATION NO.:
SUBMITTED BY:		CONTRACT NO.:
PLOT SCALE:	PLOT DATE: No	FILE NUMBER:
SIZE:	FILE NAME: HLND_TSP ALT 5D_2 OF 2.dgn	

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5D

SHEET IDENTIFICATION
CS119



LEGEND:

- EXISTING CONTOUR +11 FT NGVD
- EXISTING CONTOUR +13 FT NGVD
- RAISED BULKHEAD WITH BREAKWATER BERM
- ONSHORE DUNE BARRIER WITH BURIED BULKHEAD
- CAPPED EXISTING STATE BULKHEAD
- I TYPE FLOODWALL
- PERMANENT EASEMENT
- TEMPORARY CONSTRUCTION EASEMENT
- G CLOSURE GATE
- T REPLACE EXISTING OUTLET WITH NEW TIDE GATE
- Wr EARTHEN RAMP WALKOVER
- Ws TIMBER STAIR WALKOVER
- * EXISTING MONUMENT TO BE RAISED

0 300' 600'

SCALE: 1" = 300'

US Army Corps of Engineers®
New York District

MARK	DATE	DESCRIPTION	APPR

DESIGNED BY: | DWN BY: | CKD BY: | SUBMITTED BY: | PLOT SCALE: | PLOT DATE: | FILE NAME: | SIZE: | SOLICITATION NO.: | CONTRACT NO.: | FILE NUMBER: | DATE:

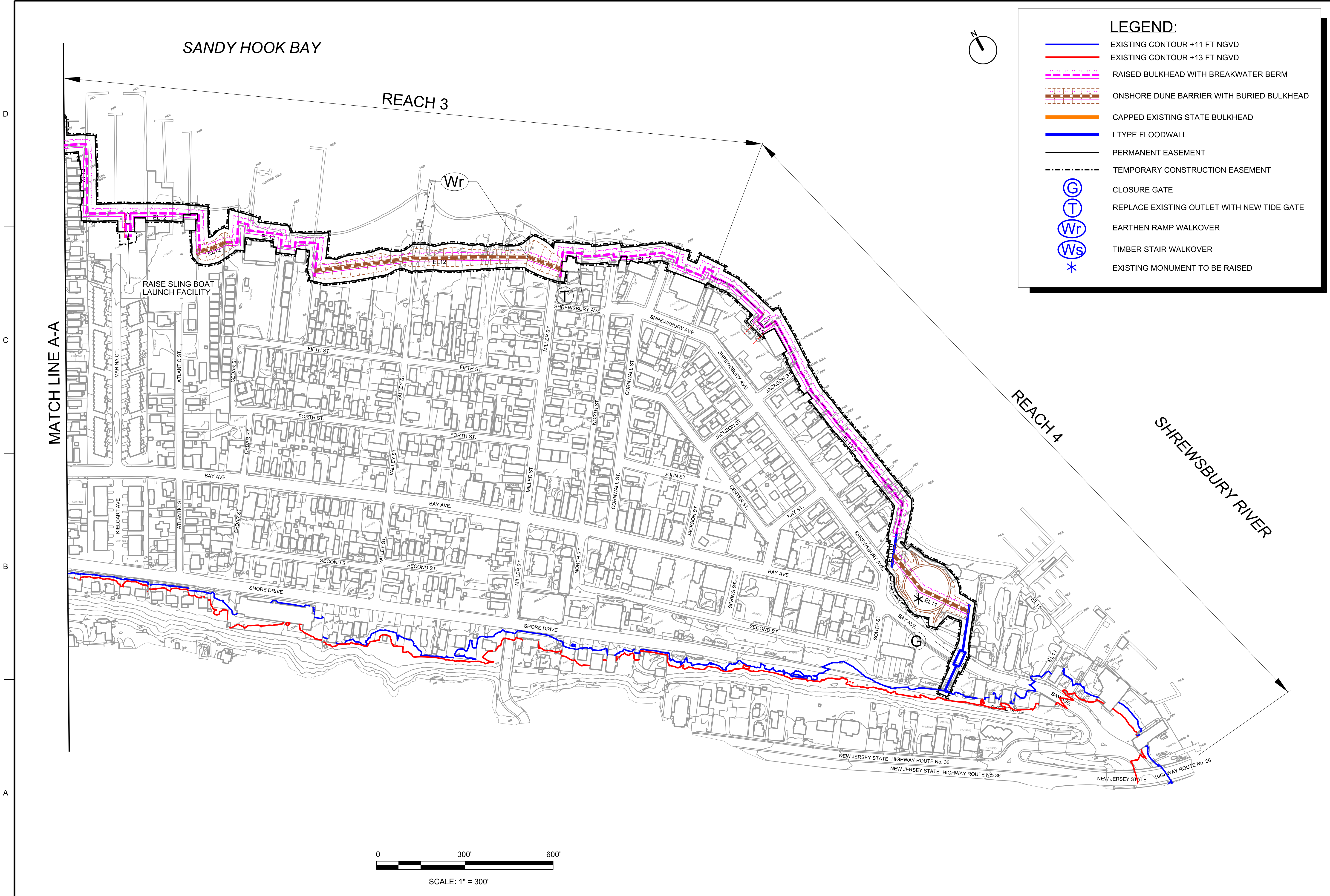
U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5E

SHEET IDENTIFICATION
CS120



US Army Corps of Engineers®
New York District

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U.S. ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
NEW YORK, NEW YORK 10278

U.S. ARMY CORPS OF ENGINEERS
ST. LOUIS DISTRICT
ST. LOUIS, MISSOURI 63103

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
PLOT SCALE: | PLOT DATE: No | FILE NUMBER:
SIZE: | FILE NAME: HLND_TSP ALT 5E_2 OF 2.dgn

HIGHLANDS FEASIBILITY STUDY
HIGHLANDS, NEW JERSEY
SANDY HOOK BAY

CONCEPTUAL FLOOD PROTECTION
ALTERNATIVE 5E

SHEET IDENTIFICATION
CS121

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project Borough of Highlands Feasibility Study

Draft Feasibility Report

JULY 2015

Appendix B2:

Coastal Engineering

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project, Borough of Highlands Feasibility Study

Appendix B2: Coastal Engineering

Table of Contents

List of Figures

List of Tables

1. Chapter 1: FEMA Stage Frequency and Wave Frequency for Existing Conditions (2021)

1.1 Stage Frequency

Stage-frequency curves for existing conditions (2013) were acquired from FEMA for the project location. The stage-frequency curves for the entire region were developed through surge and wave modeling of a suite of synthetic design storms using the ADCIRC (ADvanced CIRCulation)+SWAN (Simulating WAves Nearshore) models. More information on how FEMA develops stage-frequency can be found at http://www.r3coastal.com/home/storm-surge-study. The stage frequency data were taken directly from FEMA without manipulation, although an adjustment was made to get the stage data into the NAVD88 datum. The FEMA stage-frequency curves are referenced to the MSL datum, so a shift to the NAVD88 datum was necessary for this particular project. The datum conversion from the Mean Sea Level (MSL) datum to the NAVD88 datum was calculated to be 0.24 feet. This conversion factor was used since the Sandy Hook gauge is located relatively close to the project site. Table 1 contains the datum information for the Sandy Hook Gauge.

1.2 Wave Height and Wave Period Frequency

The raw ADCIRC+SWAN output, which includes peak surge elevation and associated significant wave heights and mean wave periods, was processed to estimate statistical wave parameters. Figure 1 shows the peak surge elevation each of the synthetic storms plotted against the associated significant wave height and peak wave period at nearby Leonardo. From this trend, we can estimate the wave heights for different surge elevations. Plugging the percent chance of flooding in a given year still water surface elevations gives the associated waves for each return period. The results of this regression analysis give the required wave-frequency information the project site. Table 2 contains the resulting stage and wave frequency curves for the project site.

Table 1: Tidal Datums

Elevations on Station Datum

Station: 8531680, Sandy Hook, NJ
T.M.: 75 W
Status: Accepted (Apr 17 2003)
Epoch: 1983-2001
Units: Feet
Datum: STND

Datum	Value	Description
MHHW	7.74	Mean Higher-High Water
MHW	7.41	Mean High Water
MTL	5.06	Mean Tide Level
MSL	5.09	Mean Sea Level
DTL	5.13	Mean Diurnal Tide Level
MLW	2.71	Mean Low Water
MLLW	2.51	Mean Lower-Low Water
NAVD88	5.33	North American Vertical Datum of 1988
STND	0.00	Station Datum
GT	5.22	Great Diurnal Range
MN	4.70	Mean Range of Tide
DHQ	0.33	Mean Diurnal High Water Inequality
DLQ	0.19	Mean Diurnal Low Water Inequality
HWI	0.29	Greenwich High Water Interval (in hours)
LWI	6.64	Greenwich Low Water Interval (in hours)
Maximum	12.60	Highest Observed Water Level
Max Date & Time	09/12/1960 13:00	Highest Observed Water Level Date and Time
Minimum	-2.20	Lowest Observed Water Level
Min Date & Time	02/02/1976 16:00	Lowest Observed Water Level Date and Time
HAT	9.11	Highest Astronomical Tide
HAT Date & Time	10/16/1993 12:48	HAT Date and Time
LAT	1.14	Lowest Astronomical Tide
LAT Date & Time	01/21/1996 19:36	LAT Date and Time

Tidal Datum Analysis Periods

01/01/1983 - 12/31/2001

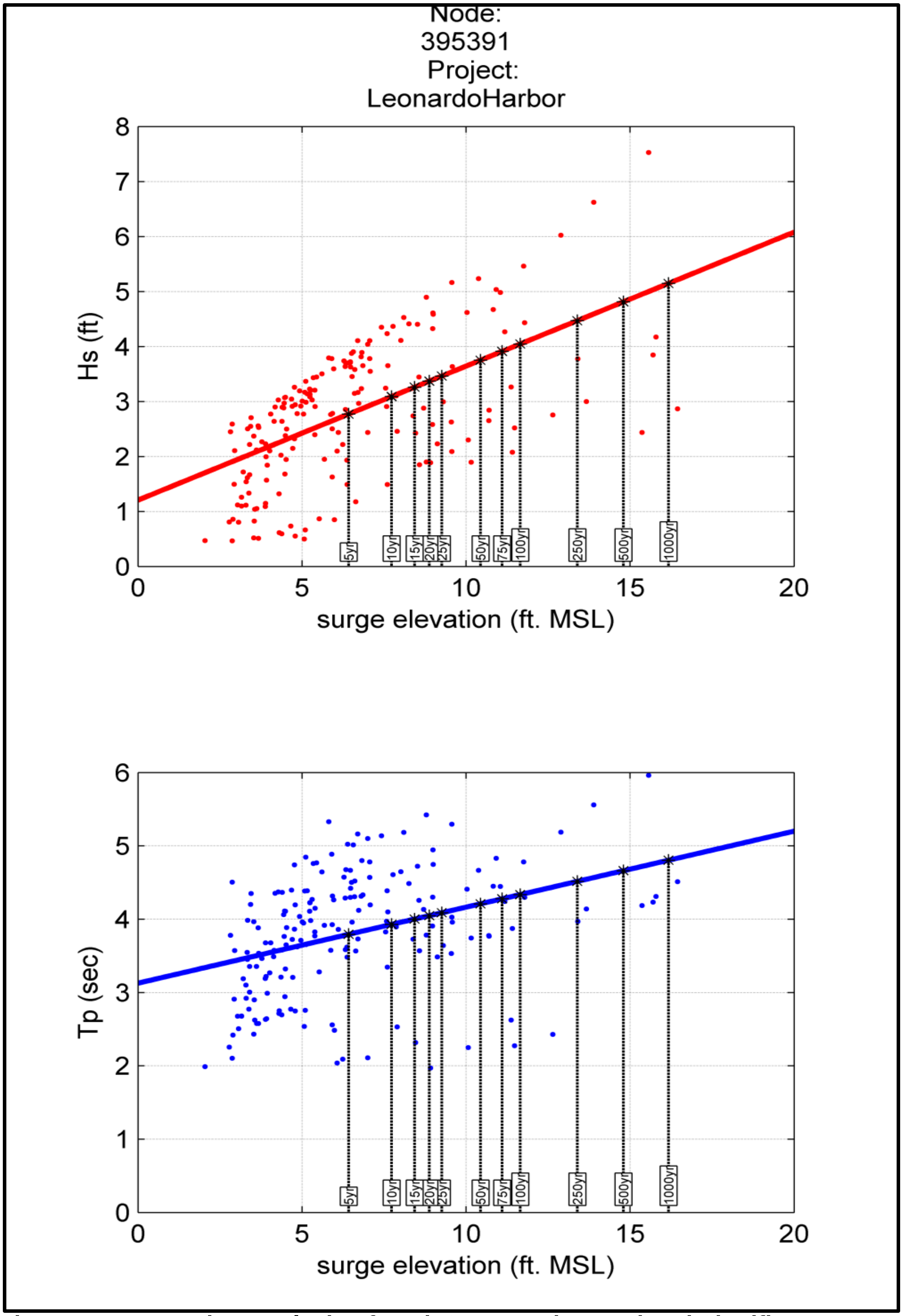


Figure 1: Regression Analysis of Peak Surge and Associated Significant Wave Height (Hs) and Peak Wave Period (Tp) for the Leonardo Project Location

Table 2: Stage and Wave Frequencies in 2014 (assumed to represent 2021 conditions)

Chance of Flooding in a Given Year	FEMA 2014 Offshore Node 395391 Mean Still Water Elevation in ft. MSL	2014 Average Onshore Mean Still Water Elevation in ft. NAVD88 including wave effects	Significant Wave Height, Hs, in ft.	Peak Wave Period, Tp, in seconds
20%	6.6	7.9	2.8	3.8
10%	7.9	8.3	3.1	3.9
7%	8.6	8.9	3.3	4.0
5%	9.1	9.3	3.4	4.0
4%	9.5	9.7	3.5	4.1
2%	10.6	10.8	3.7	4.2
1.3%	11.3	11.5	3.9	4.3
1%	11.9	12.0	4.0	4.3
0.4%	13.6	13.8	4.6	4.5
0.2%	15	15.3	4.8	4.7
0.1%	16.4	16.8	5.1	4.8

Chapter 2: FEMA Stage Frequency for Future Conditions (2071)

2.1 Sea Level Change

The Department of the Army Engineering Circular ER1100-2-8162 (31 Dec 2013) requires that future sea level change (SLC) projections must be incorporated into the planning, engineering design, construction and operation of all civil works projects. The project team should evaluate structural and non-structural components of the proposed alternatives in consideration of the “low,” “intermediate” and “high” potential rates of future SLC for both “with” and “without project” conditions.

SLC considers the effects of (1) the “regional” rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth’s crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids, and (2) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend.

2.1.1 *Vertical Land Movement*

Highlands, New Jersey is located in an area that experiences positive land subsistence due to geological processes; therefore, the net relative sea level rise at Highlands is greater than the eustatic SLR. Said differently, when land in Highlands subsides as water surface elevation increases, the net local SLR is greater in Highlands than at a location experiencing an increase in water surface elevation only. When calculating the intermediate and high rates of sea level rise, the local rate of VLM must first be determined.

The local rate of VLM, which is considered to be constant through time, is determined by subtracting the NRC/IPCC eustatic SLC value (1.7 mm/yr) from the local mean sea level trend. Recall that the two components figuring into the local mean sea level include the eustatic SLC value and the local rate of VLM. The mean rate of SLC at the Sandy Hook station is +3.9 mm/year.

The local rate of VLM at Sandy Hook is calculated from the relationship: VLMSandy Hook = [local rate of SLC] – [eustatic rate of SLC], or VLMSandy Hook = 3.9 mm/yr – 1.7 mm/yr = 2.2 mm/yr.

This local rate of VLM is added back into the sea level rise computations after the eustatic portion has been determined from NRC curves I and III.

At Sandy Hook, the local rate of VLM accounts for a total of 0.57 ft in year 2071 (the 50th year of the project).

2.1.2 *Historic (or Low) Rate of Sea Level Change*

The historic rate of future sea-level rise is determined directly from gauge data gathered in the vicinity of the project area. The nearest NOAA tide gauges from which tide data can be evaluated include: The Battery and Montauk Point gauges in New York, and the Sandy Hook gauge in New Jersey. Of these three locations, tide conditions at Sandy Hook (NOAA Station #8531680) best represent the conditions experienced in Highlands. A 75-year record (1932 to 2006) of tide data gathered at Sandy Hook, NJ indicates a mean sea level trend (eustatic SLR + the local rate of VLM) of +3.9 mm/year (Figure 2).

At Sandy Hook, the Historic (or Low) Rate of SLC, including VLM, accounts for a total of 1.01 ft in year 2071 (the 50th year of the project), and is shown in Table 3 and Figure 3.

2.1.3 *Intermediate Rate of Sea Level Change*

The intermediate rate of local mean SLC is estimated by considering the modified NRC projections and adding the appropriate value to the local rate of vertical land movement. The intermediate rate of local sea level rise is based on the modified NRC Curve I since its value is comparable to that of the IPCC projection. The intermediate rate of sea level rise is computed using the equation

$$E(t_2) – E(t_1) = 0.0017(t_2 – t_1) + b(t_2^2 – t_1^2) + \text{local VLM}$$

where t_1 and t_2 represent the start and end dates of the projected time horizon in years, relative to 1992 (for both the intermediate and high rates of SLR, the NRC curves accelerate upward over time beginning in the year 1992 when the curves were developed; therefore, it is necessary to estimate SLR for a particular time horizon relative to 1992), and b is a constant value of 2.71E^{-5} for the intermediate rate. .

At Sandy Hook, the Intermediate Rate of SLC, including VLM, accounts for a total of 1.57 ft in year 2071 (the 50th year of the project), and is shown in Table 3 and Figure 3.

2.1.4 *High Rate of Sea Level Change*

The high rate of local mean SLR is estimated by determining the modified NRC Curve III value and adding it to the local rate of vertical land movement. This high rate scenario exceeds the 2001 and 2007 IPCC projections and considers the potential rapid loss of ice from Antarctica and Greenland. The NRC Curve III is also based on the general equation E(t) = 0.0017t + bt2; however, the constant b changes to b = 1.13E-4, and has the same initial date of 1992.

At Sandy Hook, the Intermediate Rate of SLC, including VLM, accounts for a total of 3.32 ft in year 2071 (the 50th year of the project) , and is shown in Table 3 and Figure 3.

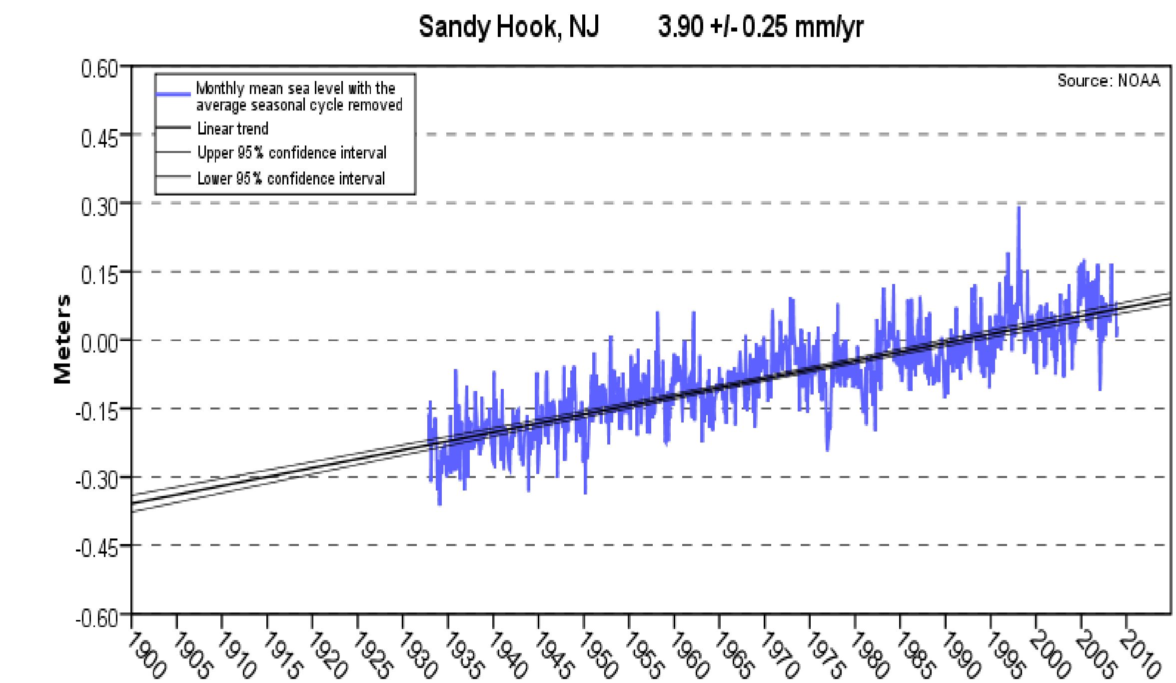


Figure 2: Mean Sea Level Trend at Sandy Hook, NJ (NOAA Station #8531680)

Table 3: Sea Level Change Estimates for Highlands, NJ			
USACE Curves computed using criteria in ER 1100-2-816231 Dec 13			
Gauge: 8531680, NJ, Sandy Hook: 75 yrs. All values are in feet.			
Year	USACE Low	USACE Intermediate	USACE High
1992	0	0	0
2021	0.37	0.45	0.68
2031	0.50	0.63	1.06
2041	0.63	0.84	1.52
2051	0.75	1.07	2.05
2061	0.88	1.31	2.65
2071	1.01	1.57	3.32

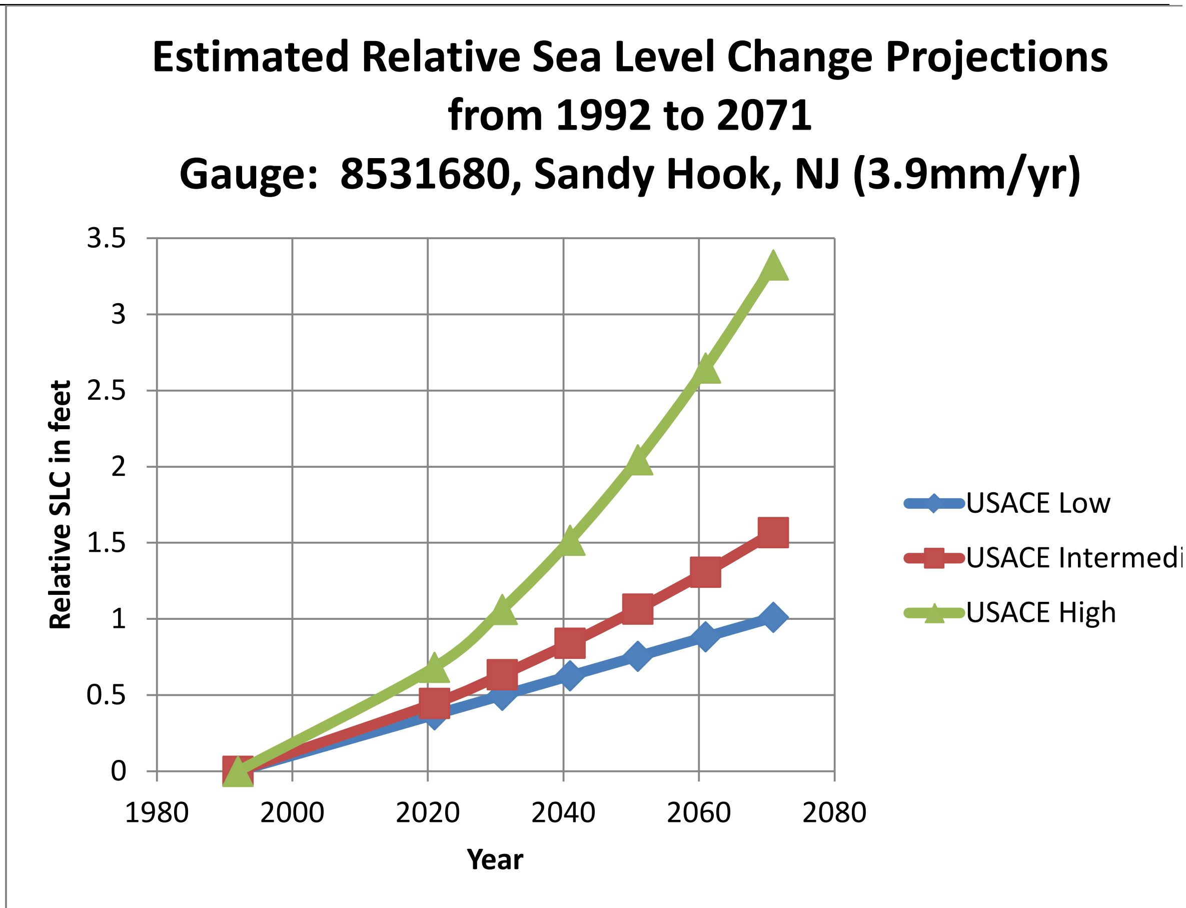


Figure 3: Relative sea level rise at NOAA Sandy Hook gage

2.2 FEMA Stage Frequency for Future Conditions with Sea Level Change

To determine future condition state-frequency data, the incremental SLC rates are added directly to the base condition curve. Significant wave heights and peak wave periods for future conditions were developed by plugging in the future condition surge values into the same trend lines developed for 2021 conditions. The higher future condition surge elevations produce larger waves. Table 4 contains the stage-frequency and wave-frequency data for the project site for the 2071 condition, for low, intermediate, and high SLC rate.

The methodology described above gives information for one example location. The stage- frequency and wave-frequency curves were developed for all structure locations using the same methodology described above.

Chapter 3: Comparison to North Atlantic Comprehensive Coastal Study (NACCS) Stage Frequency Results

3.1 North Atlantic Comprehensive Coastal Study (2014)

The USACE North Atlantic Coast Comprehensive Study (NACCS) sought to quantify existing and future forcing for use in assessing potential engineering projects that would reduce flooding risk and increase resiliency. In the NACCS, rigorous regional statistical analyses and detailed high-fidelity numerical hydrodynamic modeling were conducted for the northeast Atlantic coastal region from Virginia to Maine in order to quantify coastal storm wave, wind and water level extremal statistics. The stage-frequency values predicted in this (NACCS) study were compared to the 2014 FEMA stage-frequency values

3.2 Comparison and Discussion

Two locations were selected for comparison: The location of the NOAA Sandy Hook Gauge (#8531680), and a location in the vicinity of the shoreline of the study area. These are shown on Figure 4. The Sandy Hook Gauge comparison is shown on Figure 5, and the Highlands comparison is shown on Figure 6.

Generally the match of NACCS and FEMA in the vicinity of the the 0.01 percent chance of occurrence is excellent. NACCS used more storms and FEMA was more interested in that range of storms than in the more frequent events, so out of all the sections of the curves to match, this part makes the use of the FEMA curve for this region of the curves makes sense.

In the vicinity of the 0.005 percent chance of occurrence, there is less than 0.4 ft of difference, with the NACCS still water elevations being higher. The fact that the difference is slight lends security to the use of the FEMA data.

The differences at the more frequent end of the curve seem more drastic, until one considers that FEMA's mission is not generally focused on small flooding events, so money and time restrictions would have lead them to focus on the more severe storms during calibration. USACE accounted for this in its localized wave and stage modeling at the shoreline. Results were obtained for each and every structure location in the structure inventory (i.e., each house had its own stage-frequency, with and without wave effects). This data was averaged over the entire inventory and this average stage frequency is shown on Figure 6, being referred to as FEMA with wave effects. This curve represents exactly what was input into the economics model. Remarkably, this curve matches the shape of the NACCS curve exceptionally well in this region of the frequency curve.

3.3 Conclusion

Further refinement of the damages and costs could be obtained using the more robust NACCS Study; however the differences in the still water surface elevations are less than 0.6 ft. at a maximum, and on average less than 0.3 ft., which is well within the uncertainty of the modeling results. It is concluded therefore that sticking with the FEMA water surface elevations is an acceptable choice for the remainder of the Feasibility Study, and that a different plan is not likely to have been selected if the NACCS curves were implemented now. It is recommended that the NACCS curves be implemented during the Design Phase.

Table 4: Future Stage-Frequencies

Year	Sea Level Change Scenario	Chance of Flooding in a Given Year	FEMA 2014 Offshore Node 261708 Mean Still Water Elevation in ft. NAVD	2014 Average Onshore Mean Still Water Elevation in ft. NAVD88 including wave effects	Significant Wave Height, Hs, in ft.	Peak Wave Period, Tp, in seconds
2071	Low/Historic	20%	7.1	8.2	2.9	3.9
2071	Low/Historic	10%	8.4	8.7	3.2	4.0
2071	Low/Historic	7%	8.9	9.4	3.4	4.1
2071	Low/Historic	5%	9.5	9.7	3.5	4.1
2071	Low/Historic	4%	9.8	10.1	3.6	4.2
2071	Low/Historic	2%	11.0	11.3	3.9	4.3
2071	Low/Historic	1.3%	11.7	12.0	4.1	4.3
2071	Low/Historic	1%	12.2	12.4	4.2	4.4
2071	Low/Historic	0.4%	13.9	14.2	4.6	4.6
2071	Low/Historic	0.2%	15.5	15.8	5.0	4.7
2071	Low/Historic	0.1%	16.8	17.3	5.3	4.9
2071	Intermediate	20%	7.7	8.8	3.0	3.9
2071	Intermediate	10%	9.0	9.3	3.4	4.0
2071	Intermediate	7%	9.5	10.0	3.5	4.1
2071	Intermediate	5%	10.1	10.3	3.6	4.1
2071	Intermediate	4%	10.4	10.7	3.7	4.2
2071	Intermediate	2%	11.6	11.9	4.0	4.3
2071	Intermediate	1.3%	12.3	12.6	4.2	4.3
2071	Intermediate	1%	12.8	13.0	4.3	4.4
2071	Intermediate	0.4%	14.5	14.8	4.7	4.6
2071	Intermediate	0.2%	16.1	16.4	5.1	4.8
2071	Intermediate	0.1%	17.4	17.9	5.4	4.9
2071	High	20%	9.4	10.5	3.4	4.1
2071	High	10%	10.7	11.0	3.7	4.2
2071	High	7%	11.2	11.7	3.9	4.3
2071	High	5%	11.8	12.0	4	4.3
2071	High	4%	12.1	12.4	4.1	4.3
2071	High	2%	13.3	13.6	4.4	4.5
2071	High	1.3%	14.0	14.3	4.5	4.5
2071	High	1%	14.5	14.7	4.7	4.6
2071	High	0.4%	16.2	16.5	5.1	4.8
2071	High	0.2%	17.8	18.1	5.4	4.9
2071	High	0.1%	19.1	19.6	5.8	5.1

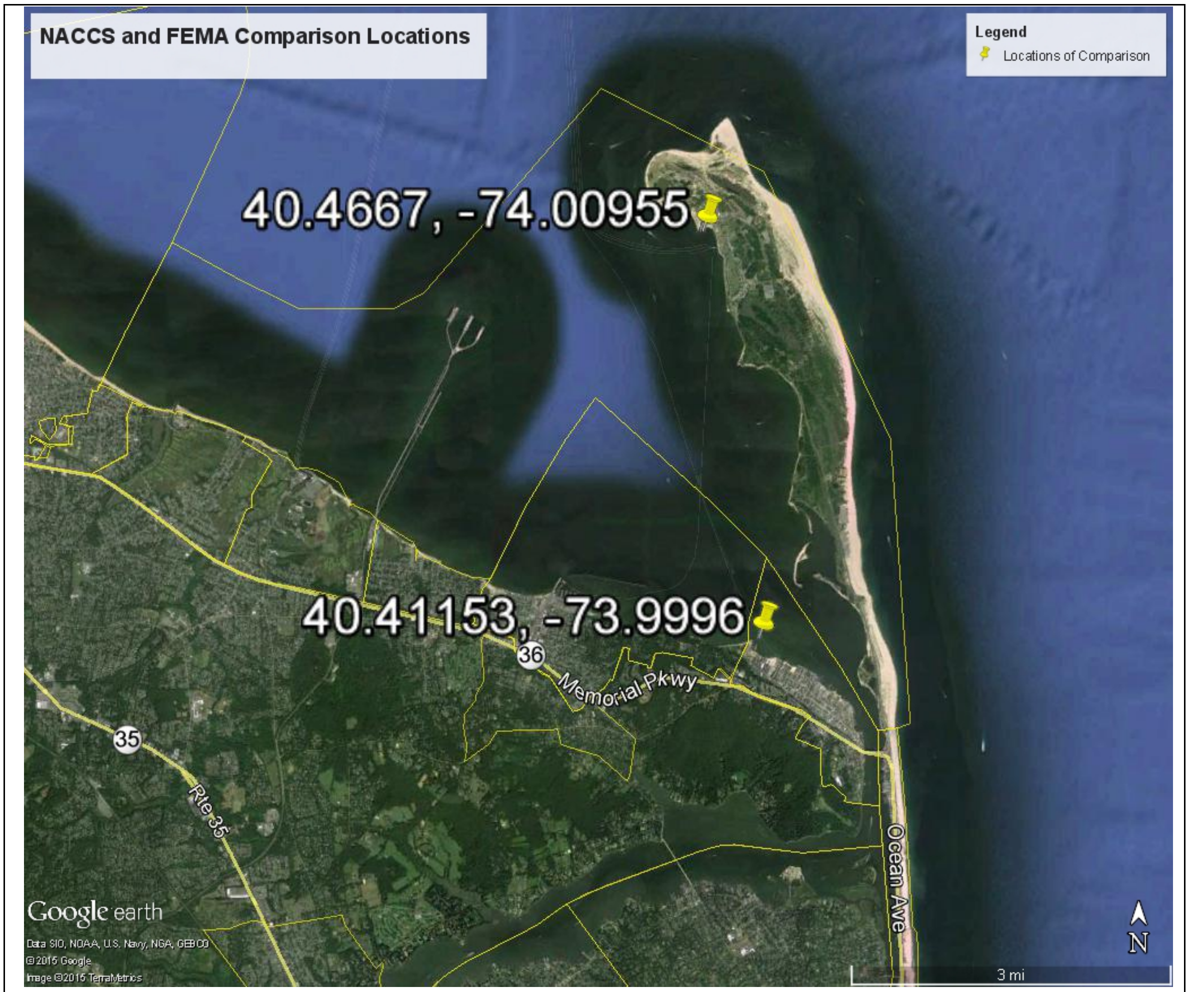


Figure 4: Location of NACCS and FEMA Comparison Nodes

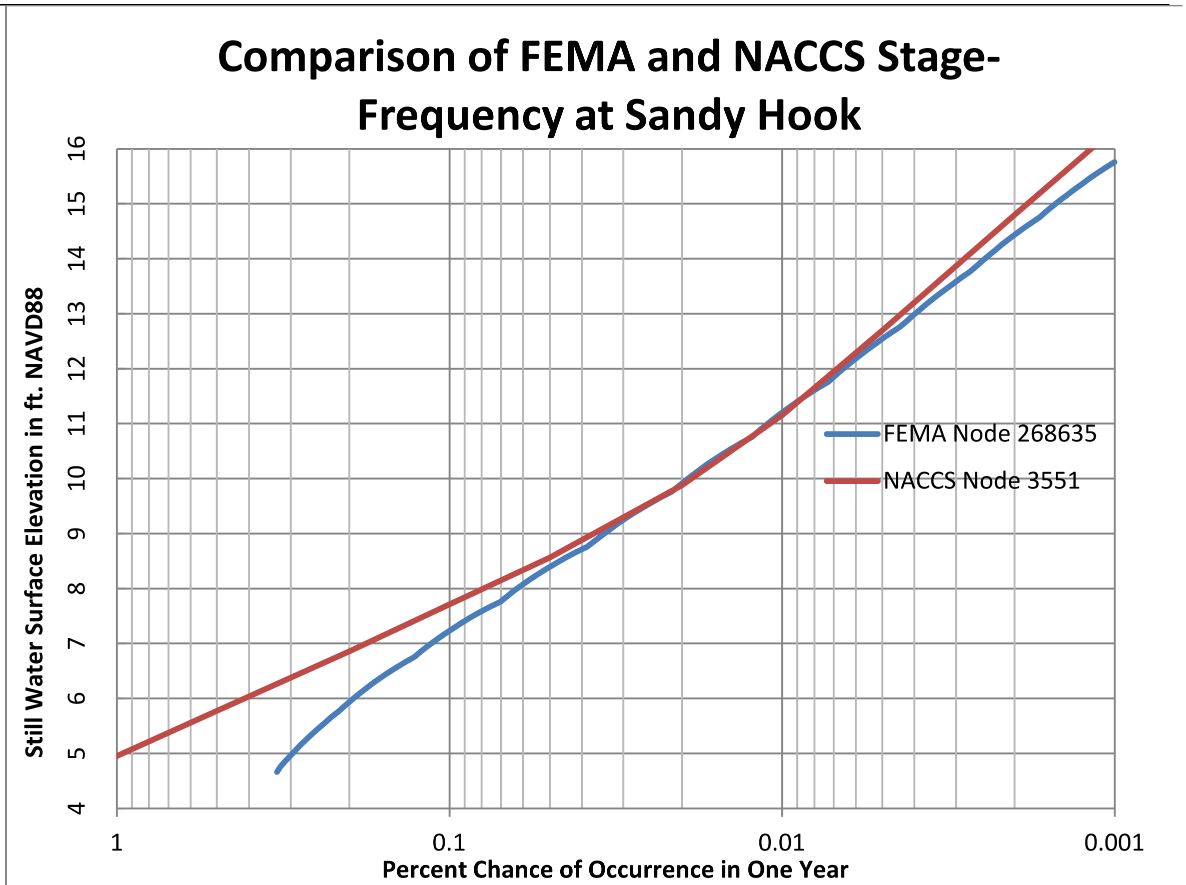


Figure 5: Comparison of NACCS and FEMA Stage-Frequency at Sandy Hook

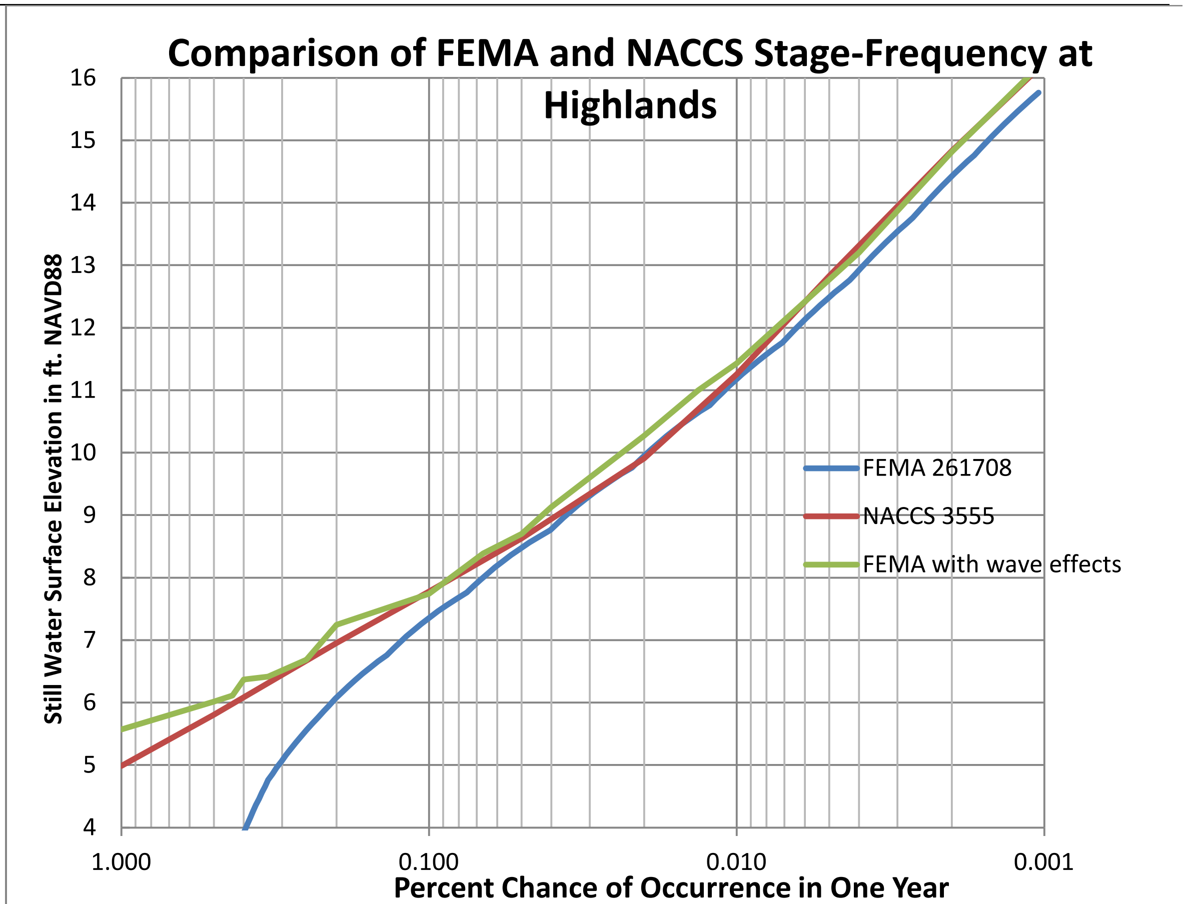


Figure 6:Comparison of NACCS and FEMA Stage-Frequency at Highlands

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project Borough of Highlands Feasibility Study

Draft Feasibility Report
July 2015

Appendix B3:

Geotechnical Engineering

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project, Borough of Highlands Feasibility Study

Appendix B3: Geotechnical Engineering

Table of Contents

List of Figures

List of Tables

This page is intentionally left blank.

Chapter 1: General

1.1 Scope of Geotechnical Investigation and Design.

This Geotechnical Appendix presents the results of studies and investigations completed by the New York District (CENAN) and St. Louis District (CEMVS) Corps of Engineers for the Highlands project.

CENAN completed a site specific, field geotechnical exploration and soils sampling/testing program in 2013. CEMVS used the results of this exploration and testing program to make project recommendations.

CEMVS reviewed geological data posted on-line by the New Jersey Department of Environmental Protection (NJDEP). CEMVS also reviewed soils survey data posted on line by the National Resources Conservation Service (NRCS) of the US Department of Agriculture.

CEMVS reviewed the Final Geotechnical Engineering and Foundation Investigation Report prepared by Hardesty and Hanover, LLP for the Route 36 Highlands Bridge Replacement over the Shrewsbury River. This report was prepared for the New Jersey Department of Transportation and submitted to Jacobs Civil, Inc in January, 2007.

Chapter 2: Background Geological and Soils Information

2.1 Results of Search of NJDEP On-Line Resources.

The NJDEP on-line GIS database now contains a Geological layer. Figure 1 is a screen shot from the website. The NJDEP site identifies the major geologic units that outcrop throughout the state of New Jersey.

Figure 1: Surficial Geology NJDEP

The soils from the shore line to about 4th street are identified as “Kml”. This unit is the Mount Laurel Formation and is described as being “quartz sand, fine to coarse-grained, and slightly glauconitic.” Glauconitic refers to a greenish micaceous mineral in the sands.

From 4th Street and further inland, the soils are identified as “Kns” the Navesink formation which is clayey, glauconitc sand. And further inland, the soils are identified as “Krbsh”, the Sandy Hook Member which is clayey, micaceous, fine grained, quartz sand.

2.2 Results of Search of NRCS On-Line Resources.

The National Resource Conservation Service (NCRS) website provides all soil surveys throughout the state of New Jersey. The soil survey report for Monmouth County is available and represents conditions as they existed in 1983. The General Soils Map from this report is shown on Figure9. The Highland Project is located in an area dominated by surface soils that belong in the Tinton, Phalanx, and Urban Land series.

The Tinton series consists of well drained soils on uplands and terraces. The Phalanx series consists of well drained soils on uplands. The Tinton and Phalanx series are probably not the dominant series in the shoreline region of the Highlands project.

The Urban Land series consists of areas more than 85 percent of which are covered by impermeable surfaces such as dwellings, roads and streets, shopping centers, parking lots and industrial parks. Based on the development apparent in the Highlands area, the Urban Land series must be the dominant series. The manmade improvements shield the true nature of the sub-surface soils. Onsite investigations and evaluations are needed for most uses.

The NRCS now maintains an interactive website for their soil surveys. In this tool, the area identified in the 1983 report as being dominated by the Tinton, Phalanx, and Urban Land series is now identified as ”UdauB”, Udorthents-Urban Land Complex. An image of the project area in the current tool tool showing and the dominant UdauB series is included in Figure 2.

Figure 2: 2014 NCRS Soil Survey – “UdauB”, Udorthents-Urban Land Complex

The "UdauB", Udorthents-Urban Land Complex description refers to 12 inches of loam underlain by 12 to 72 inches of loamy sand, all of which are well drained. Its parent material is identified as buildings, pavement, and other impervious surfaces!

Chapter 3: Detailed Site Specific Soils Exploration and Testing Programs

3.1 CENAN Geotechnical Exploration and Soils Sampling/Testing Program.

Neither the NJDEP nor the NCRS descriptions are adequate for this project. The best advice occurs in the 1983 NRCS description which recommends that "Onsite investigations and evaluations are needed for most uses".

In January and February of 2013, the Baltimore District Corps of Engineers (CENAB) completed 17 borings along the proposed alignment of the Highlands project for CENAN. These borings are named HL-08-01 through HL-08-17. These borings may be found at the end of this section. Each boring was advanced vertically 30 to 32 feet below ground surface with a CME-55 (Central Mining Equipment) drill rig. The soils were sampled with a standard 1-3/8 inch split spoon sampler driven by an automatic trip hammer (140-lb weight falling 30-inches). All samples were visually classified by the USACE Unified Soils Classification System.

CENAN provided the coordinates of the as drilled boring location to CEMVS. These latitude and longitude coordinates were measured using a hand-held GPS device and should be considered approximate. CEMVS plotted the horizontal boring locations within the Google Earth application. Those locations are shown on Figure 3. No vertical elevations have been provided for the as-drilled locations. A virtual tour of the project using the "Street View" capability of Google Earth indicates the area is relatively flat.

The information from this exploration and testing program was entered into the gINT data base and the CENAB standard Form 1836 was plotted for each boring. CEMVS assembled these 1836 forms side by side assuming that the ground surface at each boring was the same. The assembled borings were inspected to determine continuity of major soil units between borings. The standard penetration blow counts in the sands were contoured and compared between the borings in order to develop a more nuanced interpretation of the foundation. Based on these interpretations, the foundation along the proposed alignment was separated into five discrete geotechnical reaches containing similar soils, thickness and density. Figure 7 through 10 present these graphical constructions and the general boundaries between these reaches. In general, beginning at the ground surface, the stratigraphy consists of:

- Zero to two or zero to four feet of pavement and/or manmade fill. Those borings where the fill extends to a depth of 4-feet may indicate low lying areas that have been filled.
- Below the manmade fill, a layer of sand ranging from poorly graded sands (SP), sands with silt (SP-SM), to silty sands (SM), exist to a depth of 25 to 30-feet. Within this sand layer, some borings showed thin, non-continuous layers of silt (ML) or sands (SW). These sands exhibit widely varying gradations (course to fine) and varying density (very loose to medium dense).
- Below the sands, a layer of fine grained soils, silts (ML) or clays (CL or CH) exist to the bottom of the boring.

The field standard penetration blow counts measured within each boring were studied and contoured according to standard ASTM description of blow counts versus assumed density. Those ASTM assumptions and an assumed range of the shear strength of sands per Meyerhof (see section 3.3 below) are provided below.

0 blows (Weight of Hammer) to 4 blows:	Very loose.	Ø < 30°
4 to 10 blows:	Loose.	30° < Ø < 35°
10 to 30 blows:	Medium	35° < Ø < 40°
30 to 50 blows:	Dense	40° < Ø < 45°
Greater than 50 blows:	Very dense	Ø > 45°

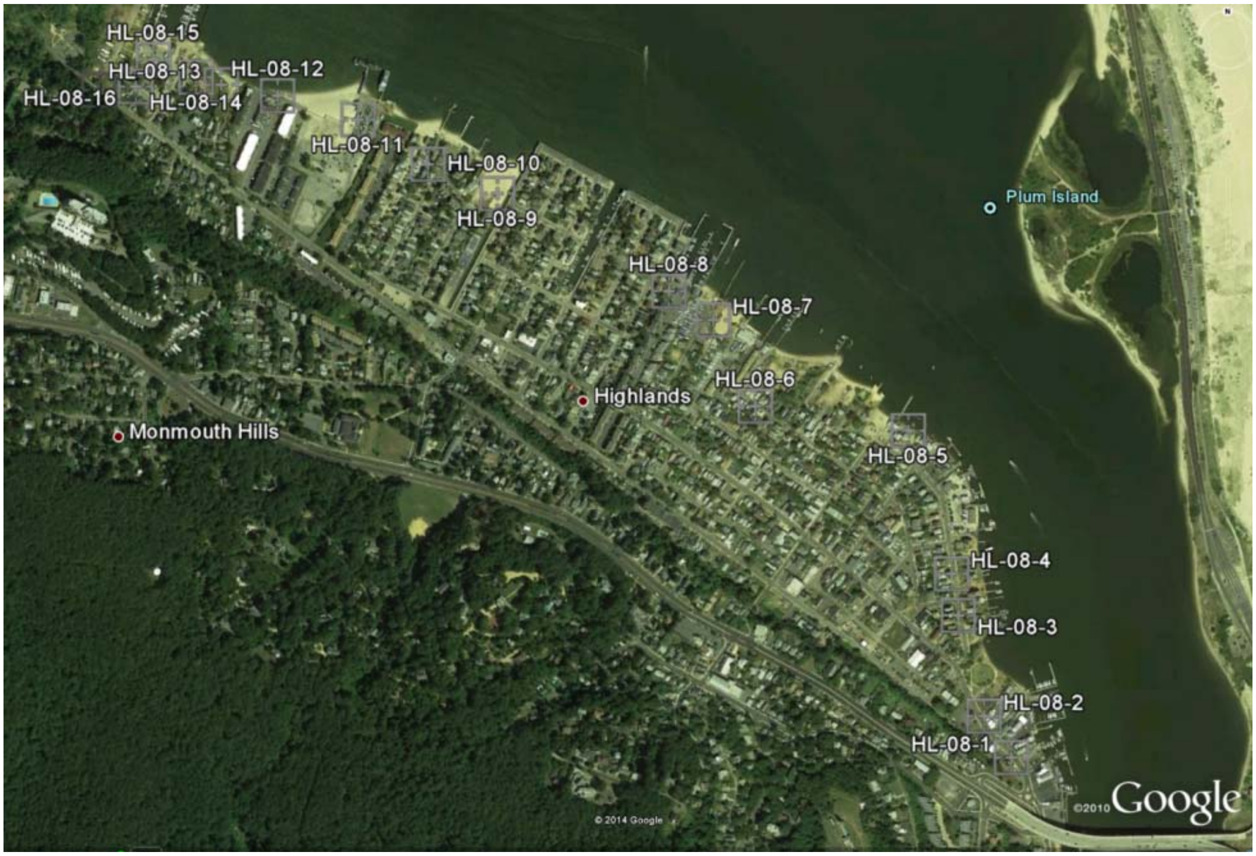


Figure 3: Locations of CENAN Highlands Exploration

The five discrete geotechnical reaches are identified on Figure 6 and described below:

Reach 1. (Figure B10) Includes area between borings HL-08-01 HL-08-03.
- 4-feet of manmade fill
- 4 to 6-feet of very loose sands.
- 13 to 15-feet of medium dense sands.
- ML/CL layer at depth.

Reach 2. (Figure B10) Includes area between borings HL-08-03 HL-08-06.
- 4-feet of manmade fill
- 6-feet of loose sands.
- 7-feet of very loose sands

8 to 14-feet of medium dense sands.
ML/CL layer at depth.

Reach 3. (Figure B9) Includes area between borings HL-08-06 HL-08-09.
6-feet of manmade fill
6-feet of loose sands.
8-feet of very loose sands
15-feet of loose sands.
ML/CL layer at depth.

Reach 4. (Figure B8) Includes area between borings HL-08-09 HL-08-13.
3-feet of manmade fill
4-feet of loose sands.
10-feet of very loose sands
14-feet of medium sands.
ML/CL layer at depth.

Reach 5 (Figure B7). Includes area between borings HL-08-13 HL-08-16.
2-feet of manmade fill
5-feet of loose sands.
7-feet of very loose sands
6-feet of loose sands.
ML/CL layer at depth.

3.2 Review of Site Specific Soils Testing.

CENAN completed a limited amount of soils testing on samples obtained during their Geotechnical Exploration and Soils Sampling/Teesting program. The results of the testing is summarized in Table B1. Certain results from the limited testing program provide some information on the shear strength of the foundation materials.

The Tri-Axial test on the Shelby Tube sample taken from a depth of 28 to 30 feet in boring HL-08-15. Although the visual classification on the plotted 1836 form identifies this layer as a silt (MH), the laboratory classification based on Atterberg limits testing and mechanical sieve analyses classify the sample as a silty sand (SM). The Tri-Axial Consolidated – Undrained with Pore Pressure measurements (CU w/pp) measures an internal friction angle of 26.2^{o} with a cohesion intercept of 3.89 PSI (0.28 TSF).

Two unconfined compression tests (UCT) were completed on clay samples obtained from boring HL-08-04 (30 to 32 feet bgs) and HL-08-05 (28 to 30 feet bgs). The sample from boring HL-08-04 classifies as a CL clay although it was visually identified as an SC. The strength test on this CL sample yielded an undrained shear strength (Cohesion) of .21 TSF. The sample from boring HL-08-05 classifies as a CH clay although it was visually identified as an SC. The strength test on this CH sample measured an undrained shear strength (Cohesion) of 1.07 TSF.

3.3 Results of Route 36 Exploration and Soils Sampling/Testing Program.

CEMVS reviewed the Final Geotechnical Engineering and Foundation Investigation Report prepared by Hardesty and Hanover, LLP for the Route 36 Highlands Bridge Replacement over the Shrewsbury River. This report was prepared for the New Jersey Department of Transportation and submitted to Jacobs Civil, Inc in January, 2007. Although the exploration completed for this major infrastructure project is located just beyond the eastern extent of the Highlands project, the bridge exploration provides insights into the foundation conditions existent below the 32-foot deep borings completed for the Highlands project.

Figure B12 is the Geologic Subsurface Profile created by Hardesty and Hanover, LLP for the Route 36 Bridge. On the Highlands side of the bridge, at bridge project station 106+00, the profile indicates a 3 to 5 foot thick layer of Tidal Marsh materials near elevation -10. Immediately below the Tidal Marsh layer is the Navesink Formation (45-foot thick) which is underlain by the Wendnah – Mt. Laurel formation (50-foot thick). The Hardesty and Hanover report describe the Tidal Marsh deposit as a layer of soft, organic, clayey silt. Although occurring at a different elevation, the clay (CH), silt (ML), and elastic silt (MH) layer encountered near the bottom of most of the Highlands borings represents the Tidal Marsh layer.

If the Highland project borings have encountered the Tidal Marsh layer, than it is likely this layer will be underlain by the Navesink and the Wendnah – Mt. Laurel formations as identified in the Rt 36 profile. This is useful for estimating the foundation conditions for 95-feet below that identified by the Highland 32-foot deep borings.

Table 3. Relationship Among Relative Density, Penetration Resistance and Angle of Internal Friction of Cohesionless Soils

State of Packing	Relative Density	Standard Penetration Resistance N	Static Cone Resistance q_c	Angle of Internal Friction ϕ'
	Percent	blows/ft	tsf or kgf/cm^2	degrees
Very Loose	< 20	< 4	< 20	< 30
Loose	20 - 40	4 - 10	20 - 40	30 - 35
Compact	40 - 60	10 - 30	40 - 120	35 - 40
Dense	60 - 80	30 - 50	120 - 200	40 - 45
Very Dense	> 80	> 50	> 200	> 45

Table 1: Shear Strength of Sands versus Standard Penetration Blow Count
(Virginia Tech)

3.4 Shear Strength and Unit Weight of Foundation Materials.

Table 1 from the document titled "*Shear Strength Correlations for Geotechnical Engineering*" (Virginia Tech Department of Civil Engineering, 1989, Duncan, Horz, and Yang) is presented below in Figure B4. The table presents the estimated shear strength of sands given its density as estimated by the standard penetration blow counts. These have been summarized above for the various layers and densities obtained from the site specific exploration program. For the very loose, loose, and medium dense sands encountered in the CENAN exploration program, internal friction angles of 30^o to 35^o are appropriate. The one tri-axial test on the silty sand material yielded a friction angle of 26^o.

The strength testing on the Tidal Marsh layer at the 28 to 30 foot depth yielded two very different samples with widely varying shear strength. The CL material was much weaker (0.21 TSF) than the CH material (1.07 TSF). A higher strength in the CH (fat clay) is not surprising.

More sampling and testing must be completed to correctly identify the locations and nature of the soils in the Tidal Marsh layer.

Table 2 is taken from the Final Geotechnical Engineering and Foundation Investigation Report prepared by Hardesty and Hanover, LLP. This table presents their selected foundation shear strengths for the materials encountered by their exploration and testing program. Their selection of friction angle Ø = 30° for the alluvial deposits (sand) is in line with the Meyerhof recommendations shown in Figure B3 and estimated density of the foundations sands encountered by the Highlands site specific exploration program.

Stratum	Unit weight (pcf)	Shear Strength Parameters	
		Friction ϕ (Deg)	Cohesion (psf)
Stratum A (Alluvial Deposits, Sand)	120	30	--
Stratum A-1 (Tidal marsh)	90	--	300
Stratum B (Upper: Navesink Formation)	120	34	--
Stratum B (Lower: Wenonah-Mount Laurel Aquifer)	125	36	--
Stratum C (Marshalltown-Wenonah Confining Bed)	120	--	2000
Stratum D (Englishtown Aquifer)	130	40	--
Stratum E (Fill)	110	26	--

Table 2: Shear Strength Selections for Rt 36 Bridge Foundation Materials
(Hardesty and Hanover, LLP)

CEMVS-EC-G recommends using a internal friction angle of 26° for the very loose soils and 30° for the medium dense foundation materials. The foundations materials have sufficient shear strength to support the surface features associated with the sand dunes or to support the subterranean features associated with the bulkhead related features.

Chapter 4: Highland Project Features.

4.1 General.

The proposed project includes raising the ground surface, building new concrete I-wall, on shore dunes, or new bulkheads; raising or capping existing bulkheads. Feature selection is based in part on existing installed features and the undeveloped space available along the alignment to construct the proposed features. Table 4 outlines the features included in each of the geotechnical reaches defined above. Table 4 identifies the boring closest to the feature. The table also indicates the analyses needed to complete the feature design. Slope Stability/Seepage Analyses could be done with the commercially available GeoStudio suite of products including the Slope/W and Seep/W applications. Final sheetpile analyses for bulkheads would be done using the USACE program CWLSheet.

Boring	Sample Depth ft bgs*	Test	Class'y	%pass #200	w_{LL}	w_{PL}	Friction Angle	Cohesion (TSF)
HL-08-02	20-21.4	Sieve	SP	2	-	-	-	-
HL-08-04	30-32	Sieve	SC	28	-	-	-	-
		UCT	CL	-	36	15	-	0.21
HL-08-05	28-30	Sieve	SC	30	63	43	-	-
		UCT	CH		63	43	-	1.07
HL-08-12	18-20	Sieve	SM	24	41	27	-	-
HL-08-13	28-30							
HL-08-15	28-30	Sieve	SM	28.7	16	16		
		Cu'	SM				26.2	0.28
bgs – below ground surface								

Table 3: Summary of Soils Testing

Geotechnical Reach	Project Feature	Boring	Slope Stability/ Seepage Analys	CWLSheet
Reach 6	Raised Grd Surf			
	Concr I-Wall	HL-08-15 & -16	Y	Y
	Raised Grd Surf			
Reach 5	On-Shore Dune	HL-08-14 to-13	Y	
	Raised Blk-Head	HL-08-12	Y	Y
	On-Shore Dune	HL-08-11	Y	
	Raised Blk-Head	HL-08-11	Y	Y
	On-Shore Dune	HL-08-10	Y	
	Raised Blk-Head	HL-08-10	Y	Y
	On-Shore Dune	HL-08-09	Y	
Reach 4	Cap Exist Blk-Hd	HL-08-09 & -08	Y	Y
	New Blk-Hd	HL-08-09 & -08	Y	Y
	Cap Exist Blk-Hd	HL-08-09 & -08	Y	Y
Reach 3	New Blk-Hd	HL-08-08	Y	Y
	On-Shore Dune	HL-08-07	Y	
	New Blk-Hd	HL-08-07	Y	Y
Reach 2	On-Shore Dune	HL-08-06 & -05	Y	
	Raised Blk-Head	HL-08-05, -04, -03	Y	Y
Reach 1	On-Shore Dune	HL-08-02 & -01	Y	

Table 4: Proposed Project Feature by Geotechnical Reach

4.2 Details of Project Features.

Raising the Ground. Raising the ground to achieve the required level of protection is the most straightforward technique. The materials used should be of a fine-grained nature to prevent through seepage. An adequate supply of suitable fine-grained borrow material must be identified.

New Concrete I-Wall or new Bulkheads. The concrete I-walls and bulkheads should be designed according to all existing USACE criteria for such structures. These will be supported by sheetpile driven deep enough to provide the necessary lateral support. The foundations materials have sufficient shear strength to provide the necessary lateral support. The sheet piling should be driven deep enough to penetrate the underlying layer of fine grained, Tidal Marsh materials to provide seepage cutoff.

On-Shore Dune. The on shore dune could be designed and constructed with sand. The slope stability of the sand dune and its foundations materials should present little or no problem during design. The bay side slope should be no steeper than 1v:3h. The protected side slope will likely need to be flatter, 1v:4h, to accommodate the through seepage that will occur. Even at this flatter slope, there may be some minor unraveling of the slope caused by the through seepage which may need repairs. The materials used in the dune should be a sand, (SP) as classified by the USACE Unified Classification System. The sand (SP) will have less than 5% fines and will have a much lower risk of internal piping during flood events. An adequate supply of suitable borrow material must be identified. Depending on the required volume, CENAN may consider investigating off-shore borrow sites if land based sites are unavailable.

Capping or Raising Existing Bulkheads. The existing bulkheads may be in poor condition depending on their age and how they were originally constructed. There is potential that even if the existing structure is still in serviceable condition, its existing design cannot be changed or otherwise adjusted to meet current USACE criteria for depth of embedment or height of stickup.

4.3 Additional Geotechnical Information Needed to Complete Design.

The most pressing need is to complete additional high quality exploration that penetrates much deeper into the underlying Navesink formation in the vicinity of the I-wall and bulk head features. These borings should include sample locations at 5-foot centers, with Atterberg limits tests run on all fine grained samples, mechanical sieve analyses run on all coarse grained samples, and all samples classified by the laboratory according to the Unified Classification system. Additional undisturbed samples and tri-axial strength testing of the Tidal Marsh layer should be completed to support the design of the I-wall and bulk head features.

.

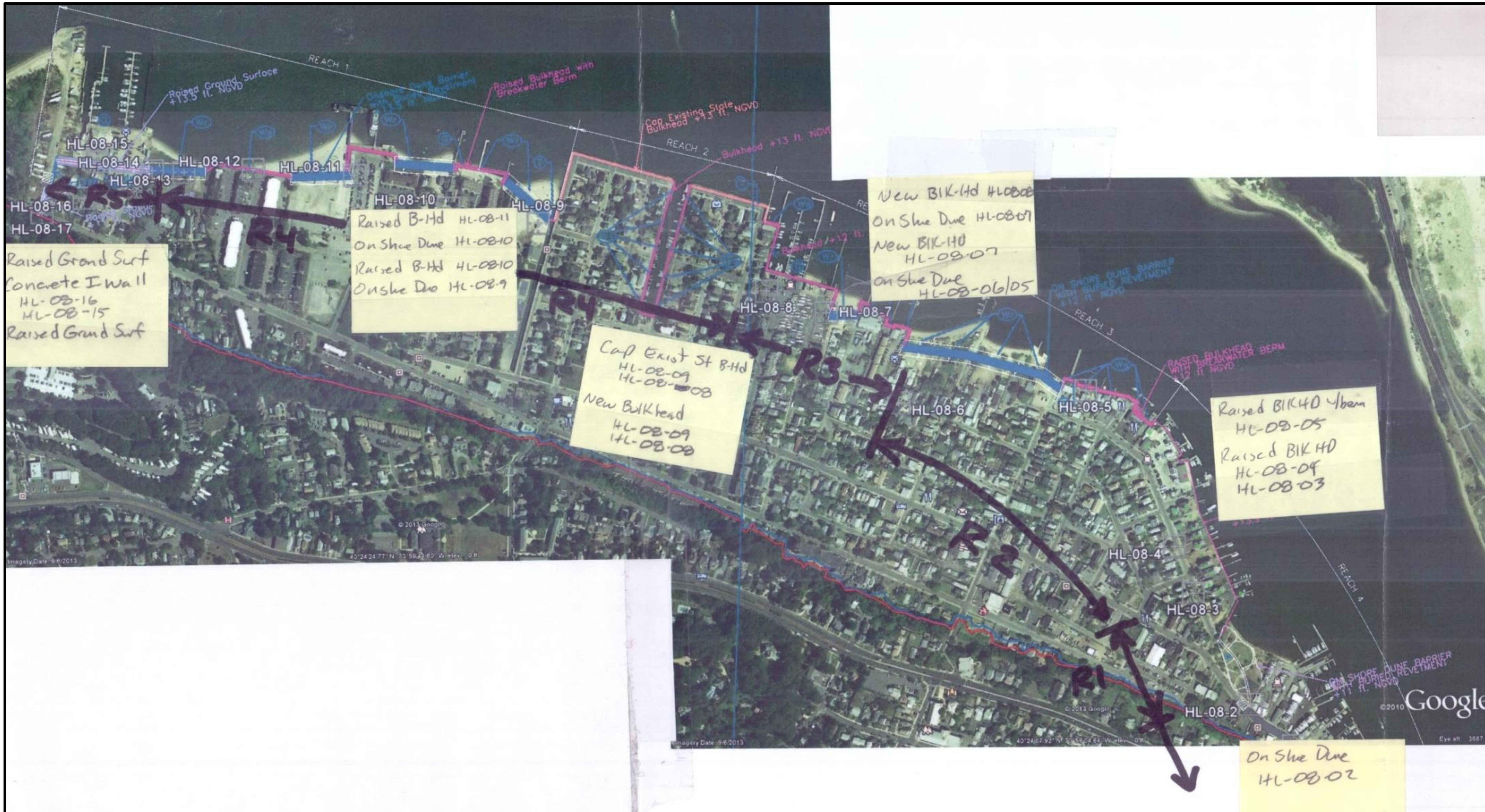


Figure 4: Limits of Geotechnical Reaches Based on Site Specific Exploration and Testing

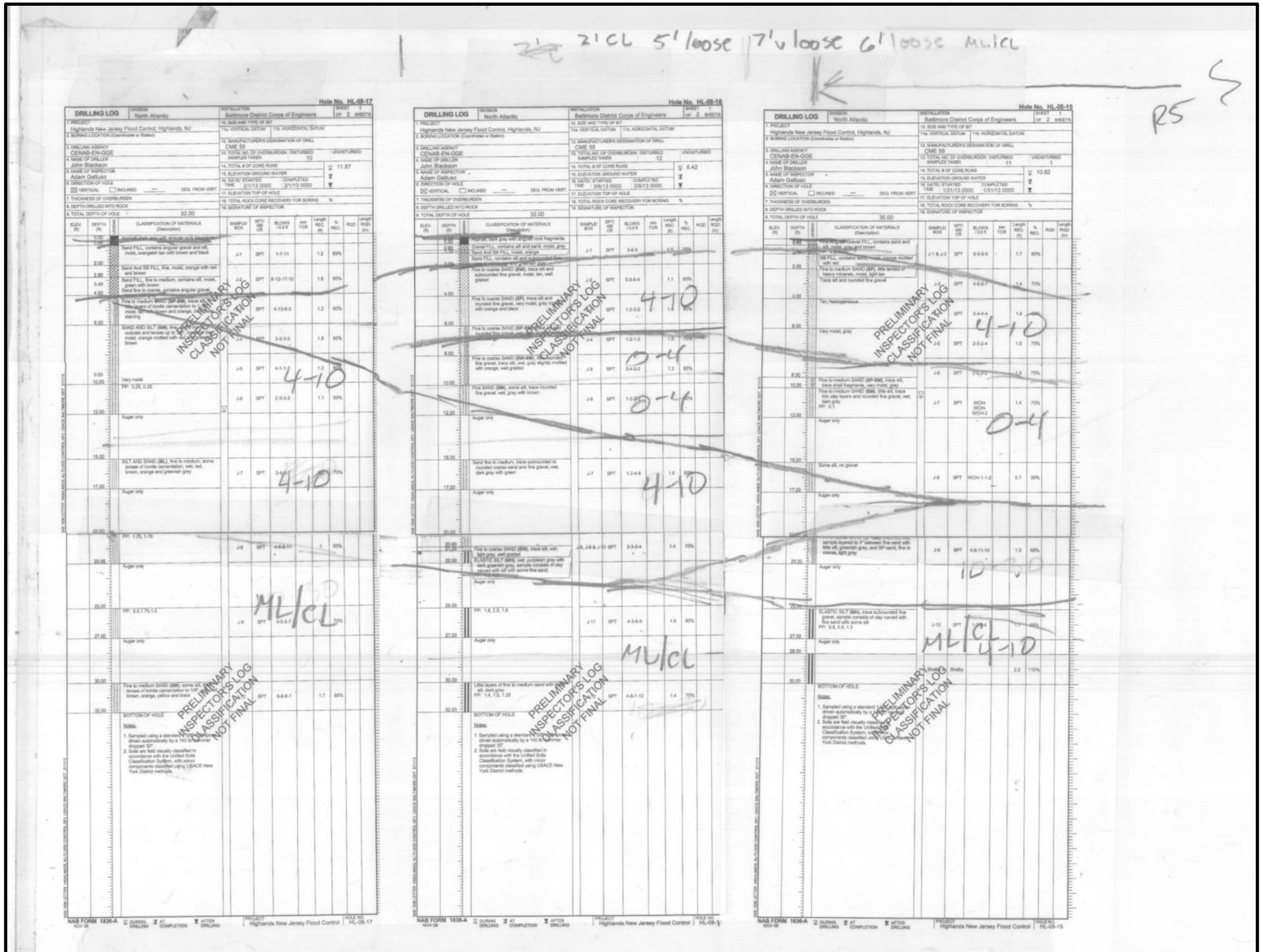


Figure 5: Geotechnical Reach R5

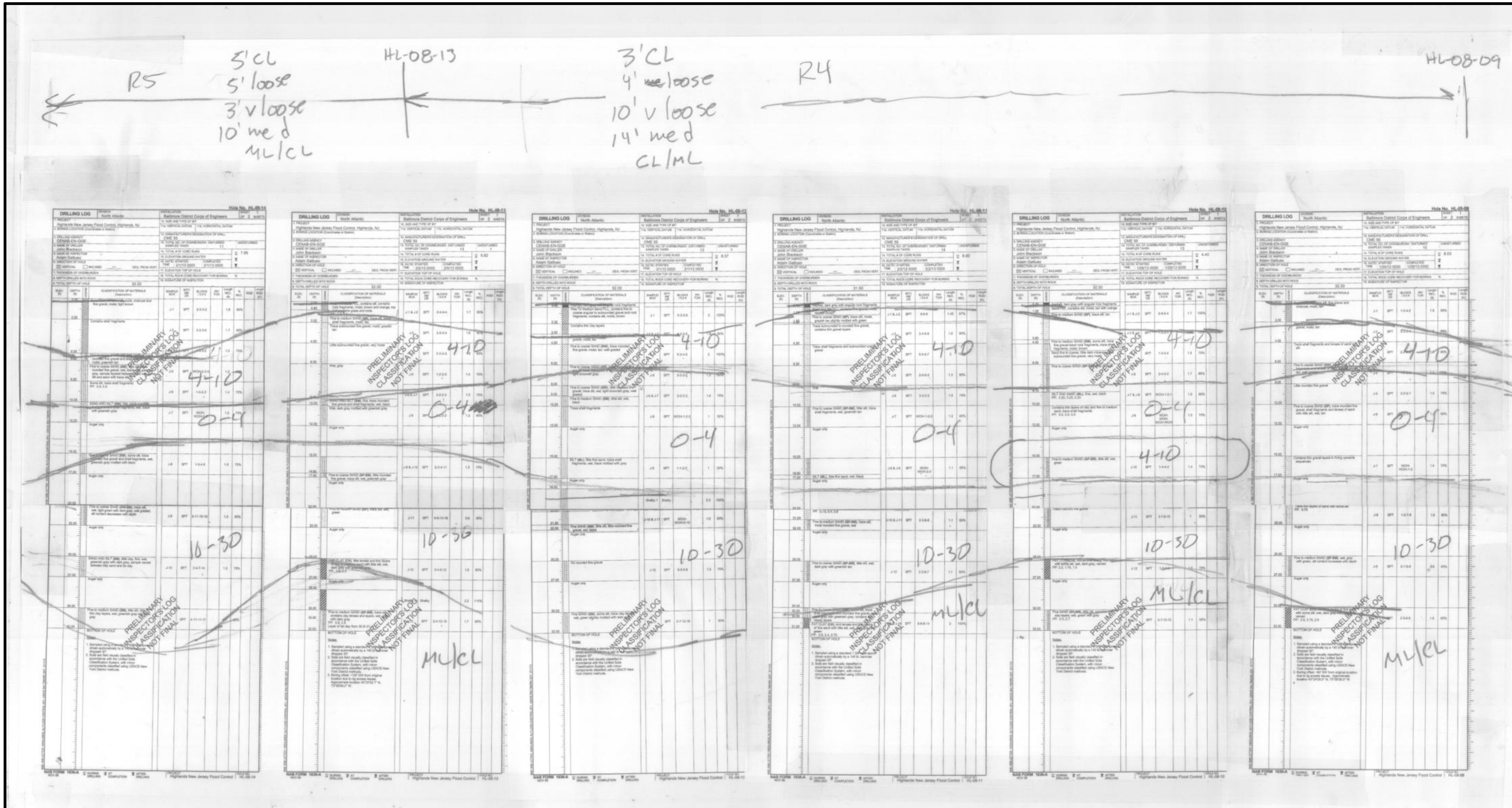


Figure 6: Geotechnical Reach R5 and R4

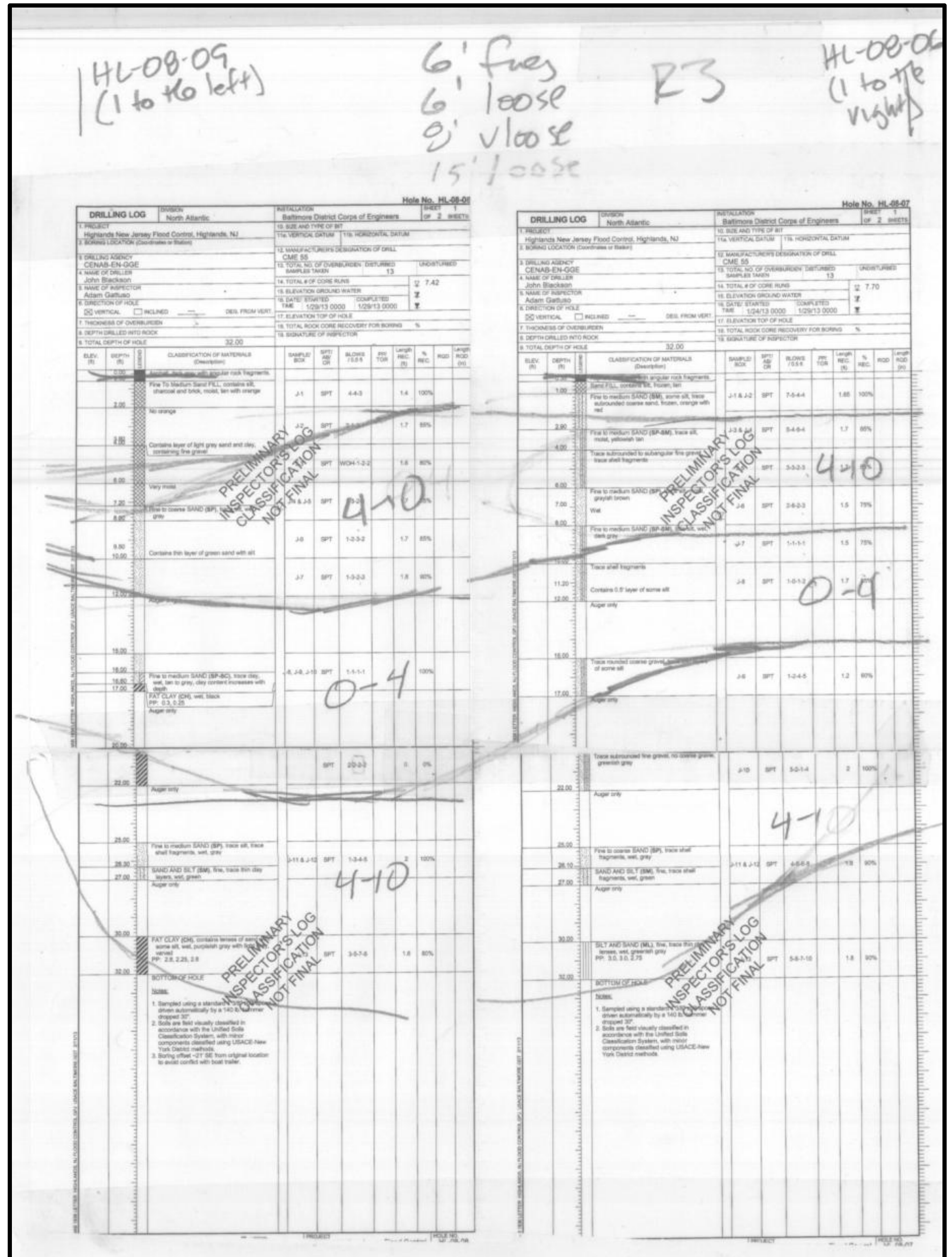


Figure 7: Geotechnical Reach R3

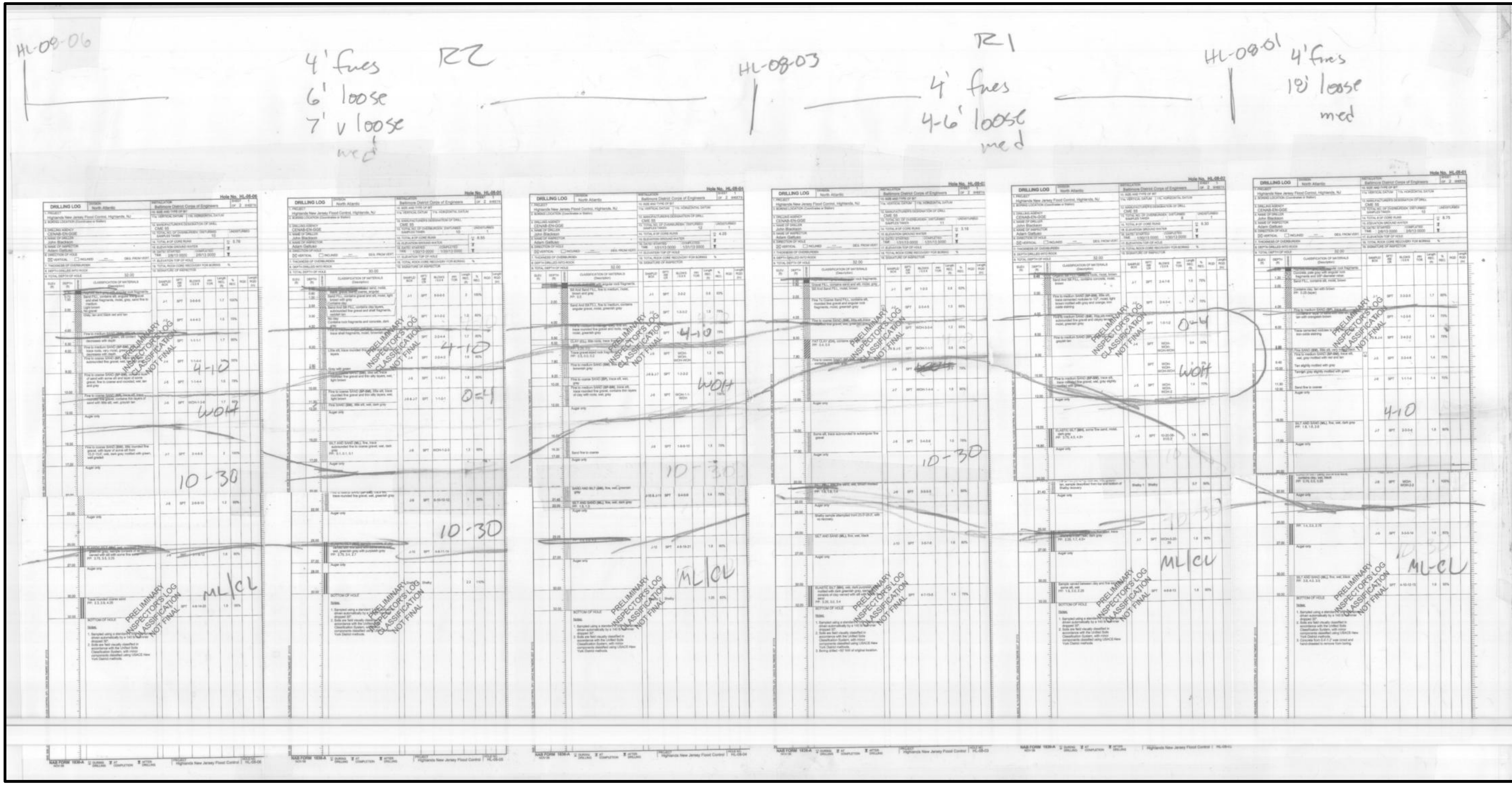


Figure 8: Geotechnical Reaches R2 and R1

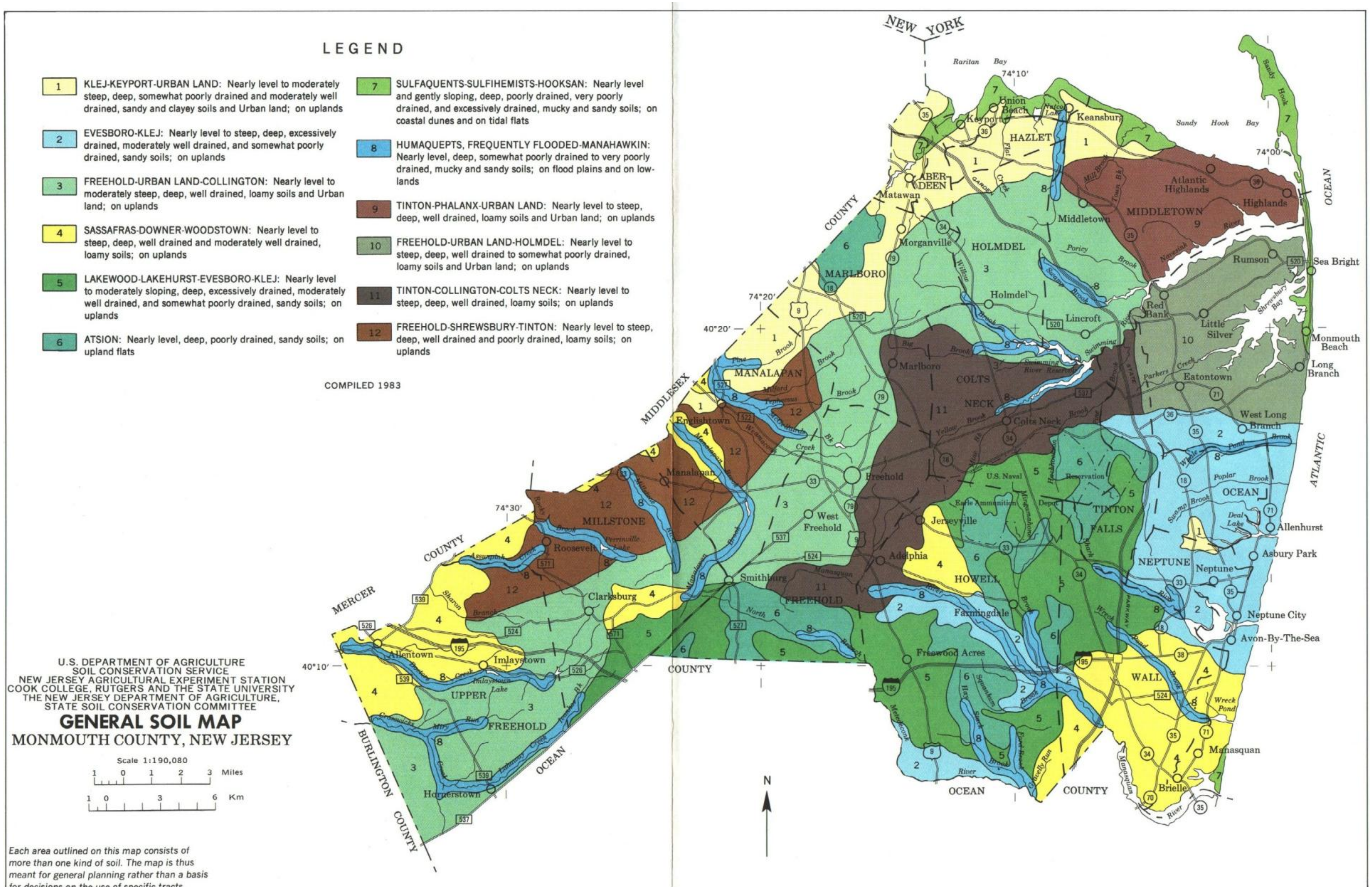


Figure 9: General Soils Map of Monmouth County, 1981

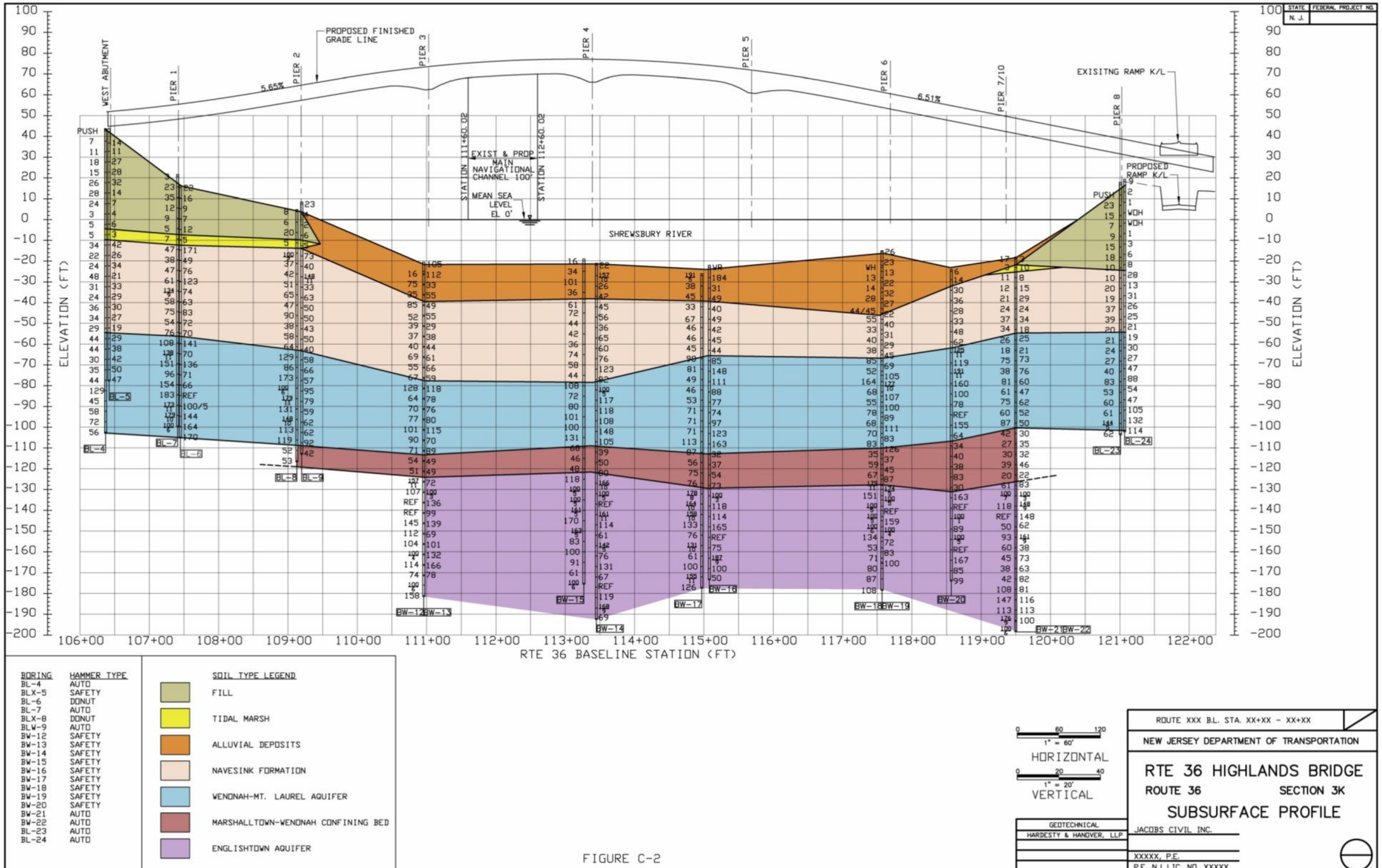


Figure 10: Rt 36 Highlands Bridge Subsurface Profile

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project Borough of Highlands Feasibility Study

Draft Feasibility Report
July 2015

Appendix B4:

Hydrology and Hydraulics

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project, Borough of Highlands Feasibility Study

Appendix B4: Hydrology and Hydraulics

Table of Contents

This page is intentionally left blank.

Chapter 1: Interior Hydraulic Analysis

1.1 Status of H&H work, Prior to Selection of TSP

For the Pre-TSP version of the Highlands, NJ Feasibility Report, the full Hydrologic and Hydraulic analysis was not completed. None of the non-structural alternatives would require a hydraulic analysis, and every structural alternative will have the same basic footprint and will require the same interior analysis. Therefore, the detailed Interior Drainage Hydraulic Analysis will be postponed until after the TSP milestone is reached.

A "Minimum Facility Analysis (Draft)" for the Interior Hydraulics Design was developed in August 2007, and this report may be referenced for more information regarding the preliminary interior drainage analysis to date for the protected area of the project. The complete technical reference for this document is shown below, and the report may be found on file in the Hydraulics Branch of the New York District or St. Louis District. This report will be superseded by the Interior Drainage Hydraulic Analysis to be performed by St. Louis District after the determination of the TSP.

Reference:

- Interior Flooding Minimum Facility Analysis, Borough of Highlands, Combined Erosion Control and Storm Damage Reduction Study, USACE New York District, August 2007.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project Borough of Highlands Feasibility Study

Draft Feasibility Report

JULY 2015

Appendix B5:

Structural

This page is intentionally left blank.

Raritan Bay and Sandy Hook Bay, New Jersey Combined Erosion Control and Coastal Storm Risk Management Project, Borough of Highlands Feasibility Study

Appendix B5: Structural

Table of Contents

List of Figures

List of Tables

Chapter 1: Overview

The Highlands flood protection alternatives included multiple methods of flood protection. The New York District of the U.S. Army Corps of Engineers formulated a variety of storm damage reduction schemes in order to evaluate various goals such as maximizing the level of risk reduction, reduction of overtopping, maintaining access, waterfront views, or to eliminate the need for protection all together. Engineering judgment and some general calculations were applied to refine each alternative type to be included in the Highlands line of flood protection. In all 10 alternatives, designated as Alternative 1-5 and 5A-5E, were studied. These alternatives included some variation of sheet pile walls, capped sheet pile walls, reinforced concrete walls, removable fabricated walls, different sizes of closure gates for access, and an offshore closure gate. The features were combined with other flood protection or avoidance measures in order to compare the cost benefit of each alternative.

The project extends for approximately 8000ft and includes most of the borough. The western portion will likely tie into high ground just before the privately owned area that will be known as the Bollerman Development. This area will provide its own private flood protection system. The eastern portion will tie into high ground just before the Route 36 Highlands Bridge. Still water level (SWL) of +9.9' NGVD, plus a value of +1.1 feet for the hydrostatic wave force of small surface, wind generated inland waves, was used in the design of each flood risk reduction structure. Project wave action varied from West to East based on exposure to wind driven waves. Final wall heights were determined from overtopping rates based on wave height and frequency. Two overtopping criteria were used to set the crest elevations: the "critical values of average overtopping discharges" (defined by activity) and the "damaging/unsafe condition overtopping threshold rates". The crest elevations allow for modest overtopping without jeopardizing structural or public safety or introducing damage. The projections resulted in final wall elevations that varied between +13.5' NGVD generally near the western end of the project and +11 NGVD towards the eastern end.

As with most any method of permanent flood protection, runoff trapped behind the structure may affect the hydrology and drainage of interior areas. Considerations should be made to include methods to discharge the water behind any method of flood protection without weakening the flood protection system.

Chapter 2: Criteria

For the purposes of the study, general guidance from USACE Engineering Manuals was reviewed in order to generate preliminary wall sections. The draft EC 1110-2-6066 pertaining to I-wall design were also considered during the process, however due to the preliminary nature of the EC cited requirements were not included in this feasibility effort. General principles and guidance based on existing projects was also used to estimate the size and type of flood risk reduction.

Chapter 3: Structure Types

Five basic types of structures were integrated in to the screened alternatives. Each provided its own benefit based on the goal/theme of the alternative. While the structure type would be modified per alternative, the basic concepts of each are as follows.

3.1 Seawall/Bulkhead Modification

This measure would entail raising or capping existing bulkheads. Raised bulkheads would provide risk reduction from coastal flooding to interior structures. Two general methods would be used for this type of flood protection system: sheet pile and capped existing sheet pile.

3.1.1 Sheet Pile

The sheet pile option is the main structural method for risk reduction for each alternative. Sheet pile will be driven into the ground along the required line of protection and with the appropriate stick up to provide flood risk reduction. Sheet pile type and length will ultimately be sized based on loadings from the soil, water, and other boats and debris that could come in contact with the wall during a storm event. General guidance has been used to initially determine an approximate size and depth for this report. Sheets can be expected to be around 40′ long. They will interface with existing bulkheads or sheet pile I walls depending on their location along the project. In most instances, new sheets will be driven directly against the existing walls (Figure 1). Existing sheets will be left in place or removed, while the voids will be filled by some means of compacted fill or flowable fill. In some instances, sheets may be driven on the protected side of the wall (Figure 2). Existing bulkheads would serve only as retaining structures and existing waterfront conditions would be allowed to remain.

To prevent failure in reverse head cases (opposite the direction of the flood load), sheets would be connected to new or existing anchorage systems. Depending on the capacity requirements of the different sheet pile wall sections and the condition/existence of the original anchorages, new anchorage systems may need to be installed to provide adequate support. Sheets may also be driven inland where anchorage would not be required.

Toe protection and armoring would be a key part of any sheet pile section. Toe protection would prevent wash out on the flood side of the wall in addition to providing a wave berm that would help to dissipate any wave forces before they impact the wall. Protected side armoring would provide protection against scour and failure due to overtopping.

To provide resistance to corrosion, sheets may be coated with a paint system or increased in size to provide sacrificial thickness. The paint system would require reapplication periodically to ensure proper adhesion and protection, particularly due to the fact that the walls are subjected to a brackish environment. Increasing the sheet pile section coupled with the application of a paint system should be considered in order to provide corrosion protection and section loss (strength reduction).

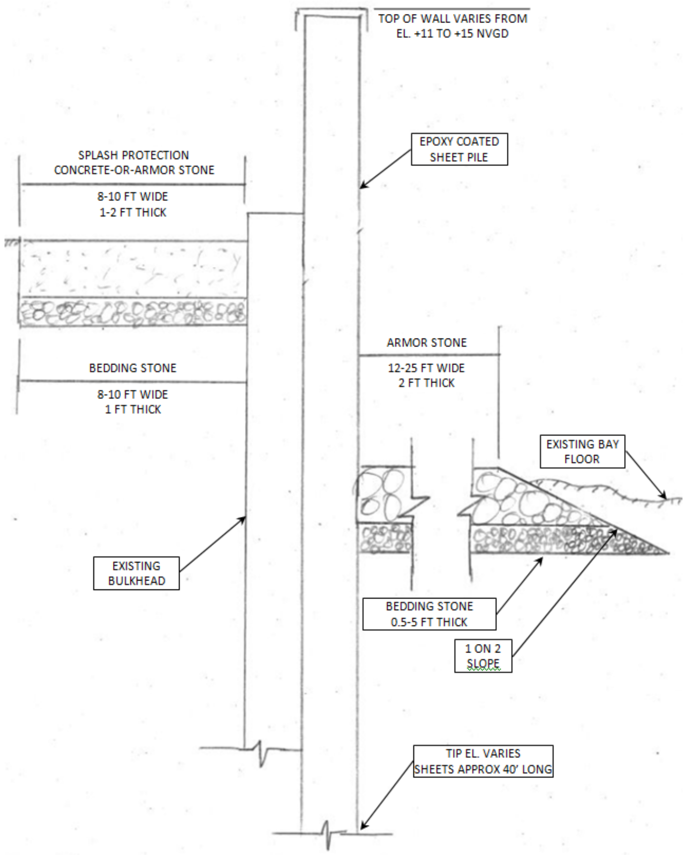


Figure 1: Steel sheet pile I-wall with toe protection and armoring.

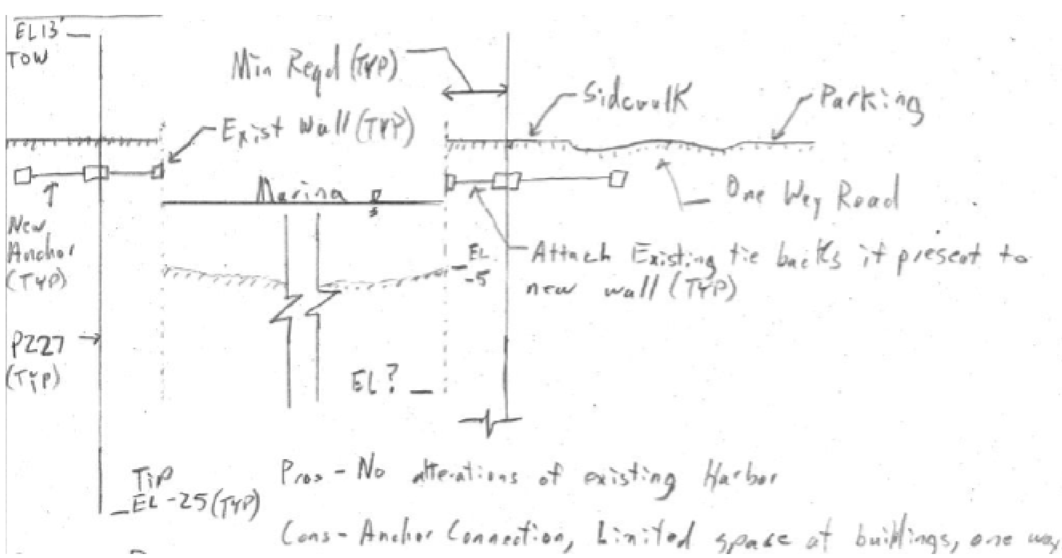


Figure 2: Steel sheet pile I-wall with anchorage (at Captains Cove)

3.1.2 **Concrete Cap**

In some sections of wall, a concrete cap is used as an option for strengthening and raising existing wall sections.

A concrete cap would be the most resilient and esthetically pleasing means of corrosion protection (Figure 3). Most importantly, it can provide extra strength to an existing system. At the State Bulkhead, concrete is fully integrated with the existing line of protection. A concrete "stem" would be poured over the existing sheet pile walls and then attached to a concrete base slab that is poured onto the protected side of the existing line of protection. The new concrete would be positively attached to the existing sheet pile structure and would increase the height of the wall by utilizing the additional height of the concrete. During flood loading the existing sheet piling would mostly serve as a seepage cut off, however, some load may be transferred to the sheets. During the reversed head case, the existing sheet pile and sheet pile anchorage would serve as a retaining structure.

The feasibility of using a concrete cap is dependent on a variety of factors. The strength of the existing foundation is the key factor in the usability of this option. The capacity of the existing sheet pile structure to handle the extra weight of the wall in addition to the added flood loading from the increased height will affect the viability of using a concrete cap. Constructability should be considered when capping an existing sheet pile wall. Many of the existing walls/bulkheads in Highlands are located at the shoreline. Forming and pouring the flood side of this wall could pose potential complications and/or increase construction cost. Special support systems will have to be devised to support the flood side concrete while it cures.

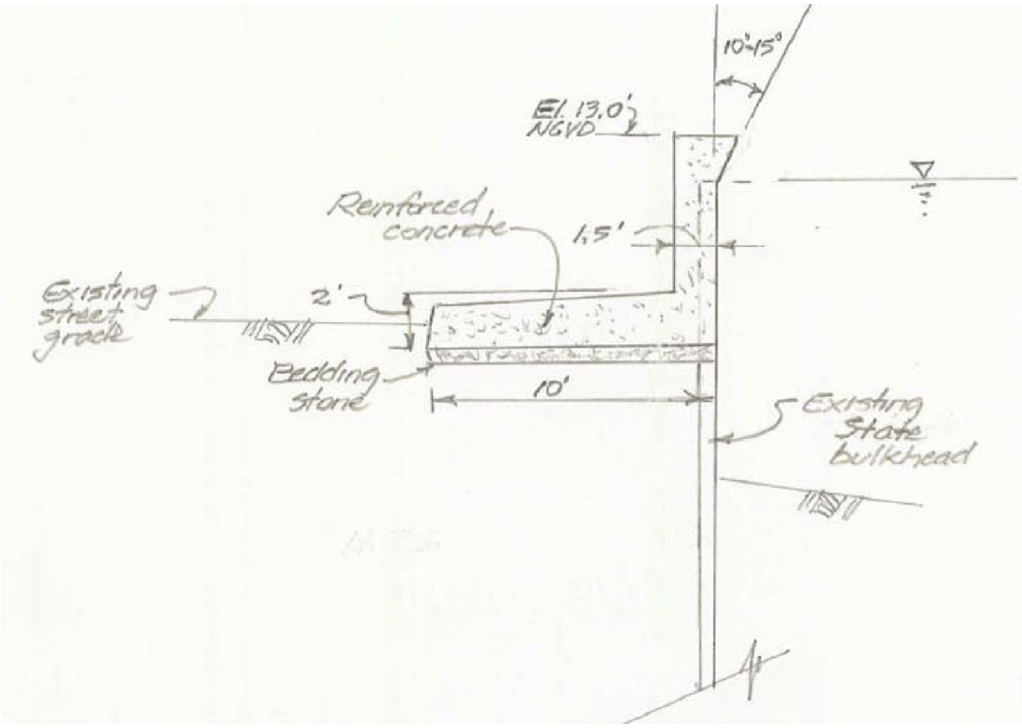


Figure 3: State Bulkhead with reinforced concrete cap.

3.2 Offshore Closure Structure with Navigation Gate

This option would include a 4500' long breakwater embankment that extends from Highlands, across the bay to tie into Sandy Hook Spit. The breakwater would be built to elevation 13.5' and utilize impervious fill or a sheet pile core to provide flood protection. During tidal flood events, closure gates placed across waterways can be closed, and high flows pumped across the closure.

This feature would not only reduce the flooding risk of most of the Borough of Highlands, but also the risk of flooding to those upstream along the bay. At the location of the existing navigation channel, approximately 500 feet from the state bulkhead, a 135-foot wide navigation sector gate (Figure 4) will be installed to allow for a 100-foot clear opening for navigation transit when the gate is in the open position. A sector gate allows for differential head on either side of the line of protection, which would be useful for pre and post storm timeframes in which water has accumulated on the protected side of the gate, but has receded on the flood side. Prior to potential major storm events, the sector gate will be closed during a period of lower tide, sealing the inner basin and providing additional runoff storage leeward of the barrier. Along with the gate itself, a concrete monolith will have to be built into the existing channel in order to support the gate and provide a new access for vessels to pass through. The existing channel is around 20' deep. The gate depth would be sized to handle normal drainage from the bay and maximum required vessel draft. Sheet pile would be used to provide a link between the hardened concrete structure and the breakwater embankment. Consideration for the control of navigation through the structure would have to be considered.

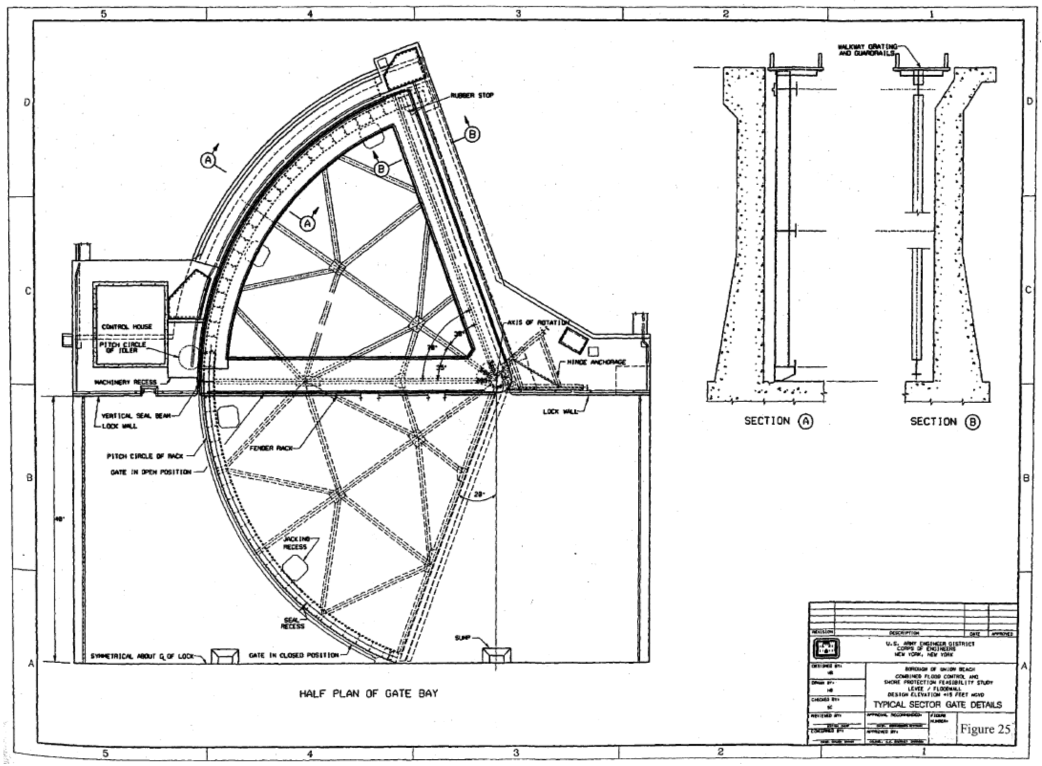


Figure 4: Offshore closure sector gate example.

3.3 Removable Fabricated Floodwall

A removable floodwall is a temporary structure that is erected prior to a flood event. Post-flooding, the barrier walls are stored offsite. It allows for vehicular and pedestrian access, unobstructed views and increased availability of land usage all the while providing flood protection when required. This alternative will be considered for the western half of Reach 4 only.

A preliminary concept was created based on general requirements and information provided by producers of fabricated floodwall systems (Figure 5). A metal sill plate and continuous concrete footing would be the only permanently installed component. The permanent structure would be set flush with the curb grade along the installation alignment. In advance of predicted high flood events, a trained crew will install vertical steel supports at 20-foot intervals with an intermediate support beam set between each parting support at 10-foot intervals. A base plank will be installed, and additional interlocking planks with watertight seals will then be stacked between each of the parting supports up to elevation +11’ NGVD. The planks will be clamped down and squeezed tightly together to create a watertight seal. The height of the removable fabricated floodwall will be approximately 6 feet, with an erection time of approximately 3 hours utilizing three, 3-men crews. A portable ramp will be installed to allow for access over the floodwall after it is erected. The construction of a shed at a nearby public works facility will be required to store the floodwall supports and planks. A preliminary foundation design and stability analysis for the removable fabricated floodwall is based on the following preliminary analysis:

- Geotechnical borings along the proposed wall alignment are not yet available. They will be obtained during future phases of this project. However, since the location is near an existing heavily trafficked roadway and paved sidewalk and the total wall height is less than 6 feet, foundation conditions are anticipated to be satisfactory for the required bearing of the vertical cantilever supports. Therefore, pile support or diagonal bracing are not required. These assumptions will be confirmed in future phases of the project. This preliminary design assumes that the soil is sand with an angle of internal friction of 30 degrees and zero cohesion.
- Hydrodynamic wave forces and earthquake loading are neglected, as they are considered to be minimal.
- The design water level is at the top of wall – elevation +11 feet. (This is the total 50-year storm surge elevation of +9.9 feet, plus a value of +1.1 feet for the hydrostatic wave force of small surface, wind generated inland waves).
- Frost depth is assumed at 38 inches below ground surface.

The foundation is a reinforced concrete slab, 4.5 feet thick immediately below the road ground surface (5 feet thick at the adjacent sidewalk) for a total 10-foot width. Therefore, for the entire length of 1,075 feet, 1,900 cubic yards of reinforced concrete will be required.

The wall stability was analyzed using the USACE's EM-1110-2-2502 Retaining and Flood Walls Manual (1989). Significant overturning forces include the horizontal water pressure and the uplift pore pressure. Weights contributing to a resisting moment include the weight of water and soil over the base, and the weight of concrete. Since site specific soils information has not yet been collected, the preliminary design is conservative.

The hydraulic gradient between the upstream and downstream sides of the wall can cause a phenomenon called boiling. Boiling occurs when the hydraulic gradient exceeds the ratio of the submerged unit weight of soil divided by the density of water. This critical gradient is approximately equal to one for typical soils. The gradient along the shortest flow path in this preliminary design is 0.6, which is acceptable and predicts that boiling should not occur.

Required repaving of the surface on both sides of the wall after its installation will further lower the hydraulic gradient, helping to control seepage and improving wall stability. A sheet pile seepage cutoff could also be considered to help reduce the gradient. The final design should take these factors into account to insure stability during storm events and minimize final construction costs.

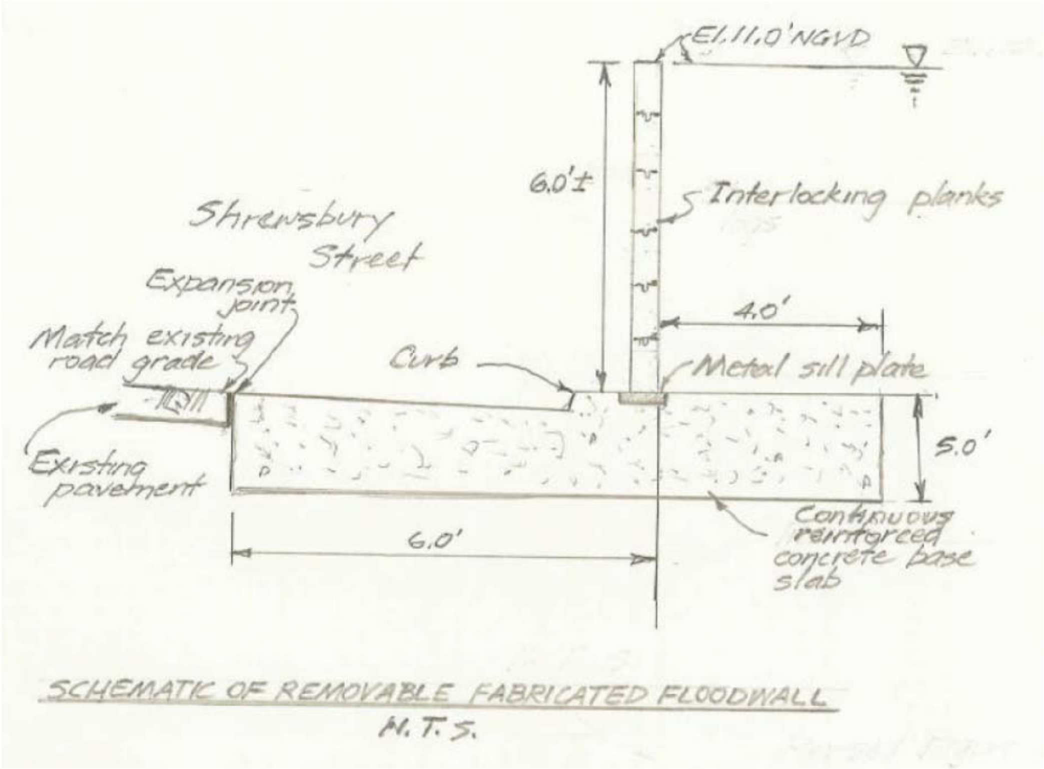


Figure 5: Removable floodwall with foundation concept.

3.4 Setback Concrete Floodwalls (I-type Floodwall)

Floodwalls are intended to provide risk reduction from coastal flooding to interior structures (Figure 6). They follow the same principles as the modification of the existing bulkhead alignment. These structures may provide a cost effective means to prevent flooding of low-lying areas while reducing the impact on nearby structures and limit the land required for rights of way. They would most likely consist of a steel sheet pile integrated into a concrete reinforced stem. The sheet pile provides the foundation for the wall and is used to transfer the flood loads into the soil and to provide a seepage cut off. The concrete portion of the wall works to extend the protection to its final height and provides strength and corrosion resistance above grade. Concrete floodwalls are more esthetically pleasing than a typical sheet pile I wall. Concrete can also be more corrosion resistant and cost effective than similar lengths of standard sheeting.

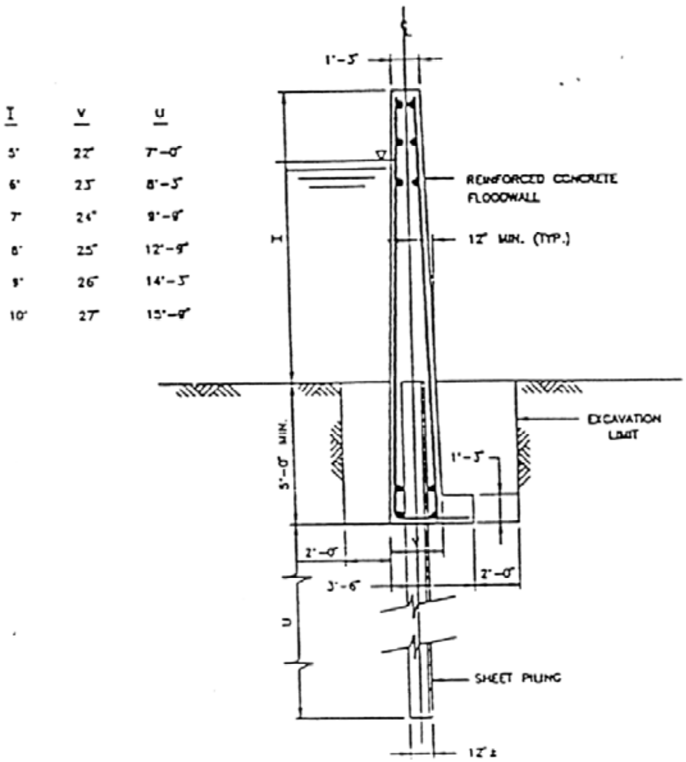


Figure 6: Typical reinforced concrete I-wall.

3.5 Closure Gate

To facilitate access to the flood side of a permanent floodwall system, vehicular and pedestrian closure gates will be included in some of the alternatives (Figure 7). These openings are also used to facilitate operations at the existing marinas to allow loading and unloading of marine vessels. Closure gates require adjacent reinforced concrete abutments to seal against and adequate foundations to support flood loading and the self weight of the structure. The gate abutments are also used to tie the gate structure to the main line of protection. Closure gates are generally made up of welded steel shapes and plating that requires a paint coating to prevent corrosion. Operation of the gate can vary from simple hand tools to vehicle assisted closures.

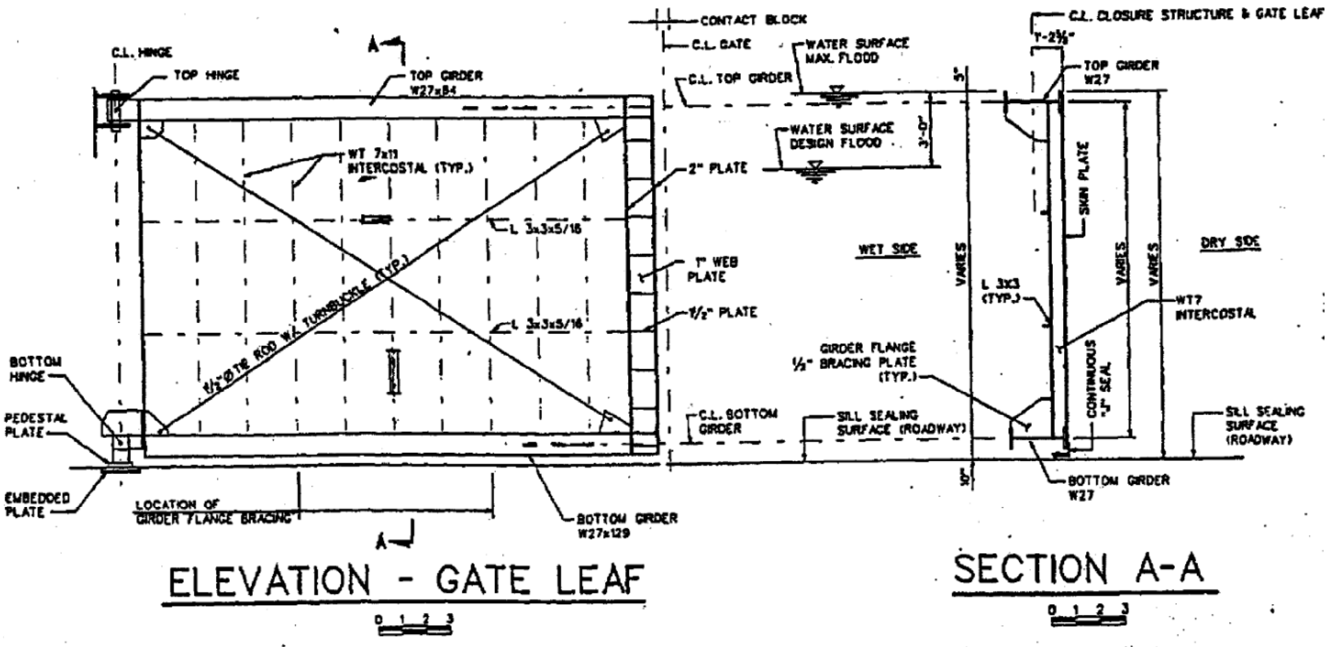


Figure 7: Steel closure gate example- swing gate.

Chapter 4: Alternative Description

Structural alternatives were incorporated into each alternative to provide the reduced flooding risk as required. All alternatives incorporated a raised sheet pile wall over some part of the line of protection except for Alternative 2. Removable floodwalls were incorporated into four alternatives to allow for increased land usage of a portion of Reach 4. Though the offshore closure prevents flood waters from even reaching most of the Borough of Highlands, the alternative still requires different types of flood protection to be construction, just over a smaller portion of the Highlands project area. Closure gates may or may not be used in Alternatives 5a-5e pending determination of the type of risk reduction that will be used at the eastern tie in at the end of Reach 4. The use of all structure types within each alternative are summarized in the table below.

TABLE 1: Structure Types vs. Alternative

	Seawall/Bulkhead Mod.					
	Sheet Pile	Concrete Cap	Offshore Closure	Removable Floodwall	Closure Gate	Setback I-Wall
Alternative 1						
Alternative 2						
Alternative 3						
Alternative 4						
Alternative 5						
Alternative 5A					*	
Alternative 5B					*	*
Alternative 5C					*	*
Alternative 5D					*	*
Alternative 5E					*	*

*Pending final determination of eastern tie in.

Chapter 5: Summary

Each structural flood risk reduction method is a viable option for this project. Estimates have been made as to the size, type and location of each structure based on preliminary engineering analysis of known conditions and requirements of the project site. Considerations during final design should be expanded to include an in depth foundation analysis based on site specific conditions, multiple load combinations including wind, wave and boat loadings, and further coordination with other disciplines to ensure items such as seepage and access are taken into account for each unique wall section. The preliminary data used to create the alternatives in this report has been used to generate basic costs for the materials, construction, and maintenance of the structures themselves. In addition, the requirements of the type of structures applied to a specific reach can assist in determining initial estimates for construction limits, level of difficulty of the construction, and right of way requirements. Once implemented, any final alternative that utilizes these structural methods will achieve the goal of lowering the risk of losses due flooding to the borough of Highlands New Jersey.