# Appendix F

# Revetment Engineering Design Analysis

# *REVETMENT DESIGN & STABILITY REPORT*
# *DENNIS P. COLLINS PARK*
# *BAYONNE, HUDSON COUNTY, NEW JERSEY*

*Hudson County Chromate Site 174*
*West First Street*
*Bayonne, Hudson County, New Jersey*
*SRP Program Interest No. G000011472*

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**Revetment Design & Stability Report**
**Dennis P. Collins Park**
**Bayonne, Hudson County, New Jersey**

## Introduction

This report summarizes the development and design of the revetment to be constructed as part of the site improvements at Dennis Collins Park - HCC Site 174 in Bayonne, New Jersey. The existing project site is located along the Kill van Kull and is fronted by a rock revetment along approximately 1,137 feet of shoreline. The adjacent upland is a public park with recreational amenities including tennis courts, basketball courts, walking paths, and grassy areas. The existing revetment extends from the existing upland grade to the water line, and exhibits deterioration including slope failure and exposed underlayers.

## Background

A brief discussion of the general design criteria relative to the performance and structural integrity of rock revetments is presented to frame the detailed design discussion presented below. Structural failure of the revetment may be caused by any individual or combination of the following processes:

- Undermining – Wave action or high velocity currents cause scouring of the erodible soil at the toe of the structure thereby creating a scour hole into which armor stone slides resulting in slope failure.
- Armor Layer Failure – The armor stone on the front face of the slope are unable to withstand the wave and/or current forces imparted on them during the design condition thereby causing them to be displaced and exposing underlayers.
- Wave Overtopping – Waves breaking on the slope create a rush of water up and over the revetment crest and subsequent scour of the upland erodible soil. The crest stones fall into the resultant scour hole thereby lowering the crest elevation resulting in greater wave overtopping and upland scour.

The following design concepts are considered to address the potential failure modes:

*Undermining*

- Appropriate toe protection should be included in the design to mitigate wave or current induced scour
- The revetment toe should be extend vertically below the scour elevation resulting from the design storm

*Armor Layer Failure*

- Rocks should be sized to withstand the wave forces associated with the design storm
- A relatively narrow rock gradation should be incorporated into the final design
- At least two layers of armor stone should be placed along the exposed slope to provide redundancy
- Rocks should be placed with at least three contact points for stability
- An appropriate foundation including bedding stone and a filter fabric should be incorporated into the design
- The revetment slope should be no steeper than 1.5H:1V
- The effect of rock shape/type on interlocking (e.g. granite, limestone, etc.), design, and construction should be considered
- The final design specification should require that rocks used in the revetment be defect-free so that the rocks do not breakdown into smaller sizes

*Wave Overtopping*

- The revetment crest and/or upland grade should be increased to mitigate overtopping effects during the design storm
- A splash pad consisting of armor stone and bedding stone should extend a minimum of 5 feet landward at the crest to mitigate upland scour

**Design Criteria**

*Selection of the Design Event*

The selection of the design event is typically based on an acceptable probability of that event occurring within the length of time that the structure and its components are intended to serve their given purpose or design life. The selection of an appropriate design event quantifies, acknowledges, and accepts a particular level of risk that the storm event (i.e. the particular combination of wave conditions and water levels that the structure is required to accommodate) might be equaled or exceeded within the design life of the structure.

A design service life of 50 years and a 100-year return period storm event were selected for the revetment design. The design life typically adopted for waterfront structures is between 25 and 50 years. The selected design event was the 100-year return interval storm based on Client feedback, which is one that is expected to be equaled or exceeded on average once every 100 years. As a 100-year return interval storm occurs randomly in any particular timeframe, rather than at a cyclical interval, it has a probability of occurrence within than time frame. The probability of occurrence of various return period events are summarized in Table 1. For example, a 100-year return period event has a 39.3% chance of occurrence within any 50-year period.

**Table 1: Probability of Occurrence of Various Return Interval Events**

| Number of years within the period | Return Period [years] | | | | | | |
|---|---|---|---|---|---|---|---|
| | 5 | 10 | 25 | 50 | 100 | 200 | 500 |
| 1 | 18.1% | 9.5% | 3.9% | 2.0% | 1.0% | 0.5% | 0.2% |
| 2 | 33.0% | 18.1% | 7.7% | 3.9% | 2.0% | 1.0% | 0.4% |
| 5 | 63.2% | 39.3% | 18.1% | 9.5% | 4.9% | 2.5% | 1.0% |
| 10 | 86.5% | 63.2% | 33.0% | 18.1% | 9.5% | 4.9% | 2.0% |
| 25 | 99.3% | 91.8% | 63.2% | 39.3% | 22.1% | 11.7% | 4.9% |
| 50 | 100.0% | 99.3% | 86.5% | 63.2% | 39.3% | 22.1% | 9.5% |
| 100 | 100.0% | 100.0% | 98.2% | 86.5% | 63.2% | 39.3% | 18.1% |
| 200 | 100.0% | 100.0% | 100.0% | 98.2% | 86.5% | 63.2% | 33.0% |

*Water Levels*

The astronomical tide and the storm surge levels were evaluated for the revetment design. The astronomical tide is the daily rising and falling of the water in response to the gravitational pull of the moon, sun, and other astronomical bodies. The storm surge is the increase in water levels due to surface winds and atmospheric pressure fluctuations from low frequency events. The astronomical tides at National Oceanic and Atmospheric Administration (NOAA) tide station #8519483, Bergen Point West Reach, NY were adopted due to their close proximity to the project site, as illustrated in Figure 1.


**Figure 1: NOAA Tide Station #8519483**

The astronomical tide levels referenced to the North American Vertical Datum of 1988 (NAVD) are summarized in Table 2.

**Table 2: Summary of Astronomical Tide Elevations**

| Astronomical Tide Level | Elevation [feet, NAVD] |
|---|---|
| Mean Higher High Water (MHHW) | +2.62 |
| Mean High Water (MHW) | +2.30 |
| Mean Sea Level (MSL) | -0.12 |
| Mean Low Water (MLW) | -2.68 |
| Mean Lower Low Water (MLLW) | -2.89 |

Storm surge elevations were adopted from Federal Emergency Management Agency (FEMA) Preliminary Flood Insurance Study (FIS) #34017CV000B for Hudson County, New Jersey dated December 20, 2013. The elevations for transect #24, which is located along the project site shoreline, were adopted due to their close proximity to the project site and are summarized in Table 3. The storm surge elevations below are from the preliminary FIS, which are higher than the storm surge elevations in the effective FIS. They were adopted at the direction of the City to be conservative as a result of the impacts of Hurricane Sandy in October 2012.

**Table 3: Summary of Storm Surge Elevations**

| Return Period [years] | Elevation [feet, NAVD] |
|---|---|
| 10 | +7.0 |
| 25 | +8.5 |
| 50 | +9.7 |
| 100 | +10.9 |
| 500 | +13.8 |

*Design Waves*

Design waves for the proposed revetment include wind-generated waves and vessel-generated waves. The design waves were established using methods outlined in the U.S. Army Corps of Engineers' Coastal Engineering Manual. Wind-generated waves depend on the wind speed, duration, fetch length, and water depth. Wind speeds were approximated by using published coastal wind speeds (USACE, 2002), as illustrated in Figure 2 and Figure 3. The wind speeds at milepost 2550 were taken for the project site and are summarized in Table 4

**Table 4: Summary of Fastest-Mile Wind Speeds**

| Return Period [years] | Wind Speed [mph] |
|---|---|
| 10 | 50 |
| 25 | 75 |
| 50 | 90 |
| 100 | 100 |

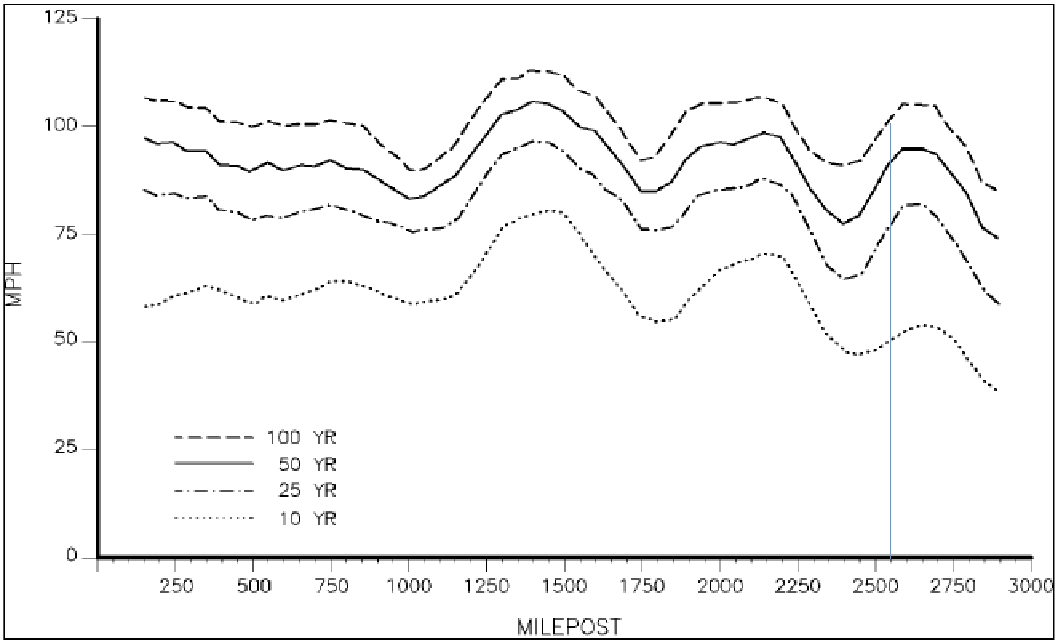


**Figure 2: Extreme Fastest-Mile Coastal Wind Speeds**

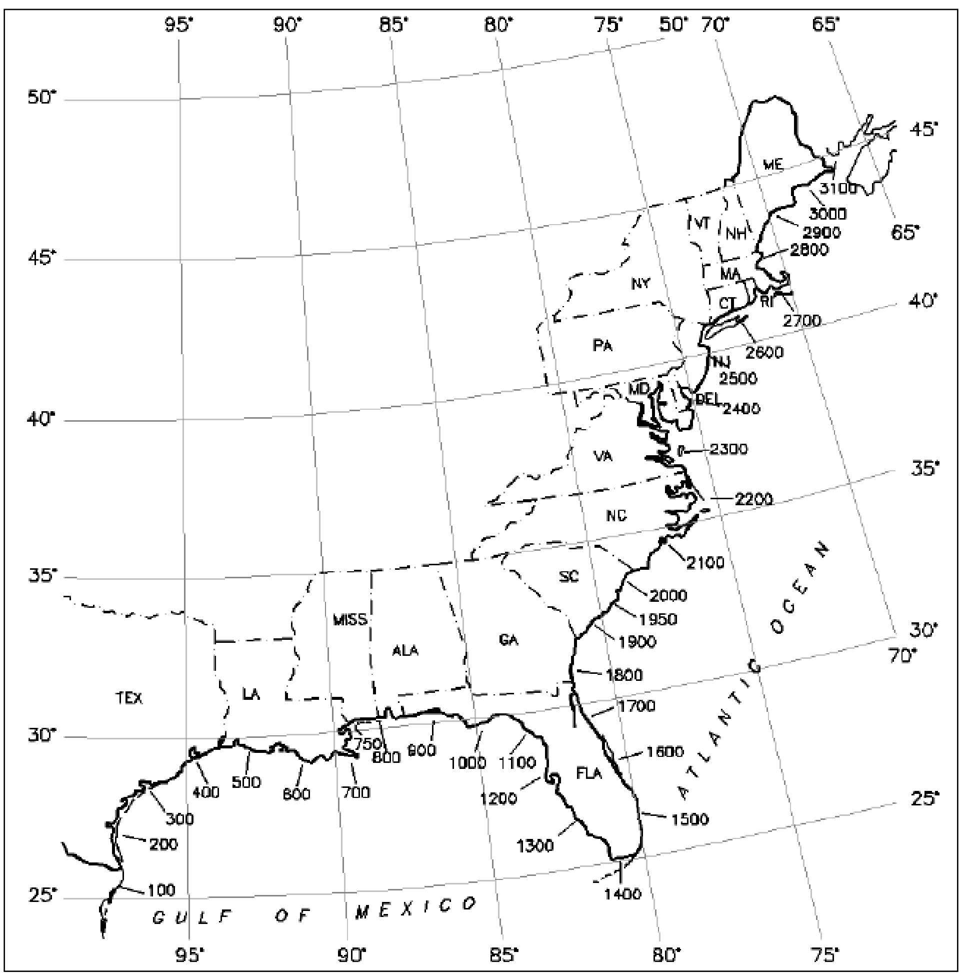


**Figure 3: Milepost Map for Coastal Wind Speeds**

The longest fetch at the project site is approximately 7,800 feet from the southwest, as illustrated in Figure 4. Fetch-limited wave heights and wave periods for various return period events are summarized in Table 5 and were calculated using equation II-3-36 from the Coastal Engineering Manual as follows:

$$\frac{gH_{m_o}}{u_*^2} = 4.13x10^{-2}\left(\frac{gX}{u_*^2}\right)^{\frac{1}{2}}$$

and

$$\frac{gT_p}{u_*} = 0.651\left(\frac{gX}{u_*^2}\right)^{\frac{1}{3}}$$

$$u_* = \sqrt{C_D U_{10}^2}$$

$$C_D = 0.001(1.1 + 0.35U_{10})$$

Where

X = straight line fetch distance over which the wind blows (meters)
$H_{m_o}$ = energy-based significant wave height (meters)
$C_D$ = drag coefficient
$U_{10}$ = wind speed at 10 meter elevation (meters/second)
$u_*$ = friction velocity (meters/second)

The design wind-generated wave height is fetch limited and was calcuated to be 4.0 feet with a corresponding 2.4 second wave period during the 100-year return period event.

**Table 5: Design Wave Heights and Periods**

| Return Period [years] | Wave Height [feet] | Wave Period [s] |
|---|---|---|
| 10 | 1.6 | 1.7 |
| 25 | 2.5 | 2.0 |
| 50 | 3.3 | 2.2 |
| 100 | 4.0 | 2.4 |

**Figure 4: Longest Fetch - Southwest**

The wave height of vessel-generated waves depends mostly on the boat speed and type. Unique of vessel-generated waves, the wave height decreases rapidly with the distance of the vessel from the shoreline. The two main types of waves generated by moving vessels are primary waves and secondary waves (Schiereck, 2012). Primary waves are typically minor in wide channels like the Kill van Kull; therefore, only secondary waves were evaluated in the revetment design.

General methods (PIANC, 1987) for calculating the secondary wave heights generated in inland waterways for revetment design are as follows:

$$H = h\alpha_1 F_h^4 \left(\frac{S}{h}\right)^{-0.33}$$

Where

$F_h$ = Froude number, $F_h = \frac{V_s}{\sqrt{gh}}$
$S$ = Distance between vessel's side and the point of interest
$\alpha_1$ = Coefficient depending on vessel type
$h$ = Channel depth
$V_s$ = Vessel speed
$T$ = Secondary wave period, $T = 0.82 V_s \frac{2\pi}{g}$

The coefficient $\alpha_1$ is based on laboratory and field tests in deep water. For this project, $\alpha_1$ was taken to be 0.7 for tankers/cargo ships and 1.0 for tug boats. Although today's cargo ships are capable of speeds greater than 20 knots in open seas, typical vessel speeds for inland travel along the Kill van Kull were assumed to be between 8 and 14 knots. While the cargo ships can be assumed to travel in the center 300 feet of the channel, the distance between the vessel's side and the project site shoreline was taken as a minimum of 164 feet (50 meters) and a maximum of 984 feet (300 meters) based on the Kill van Kull channel width (USACE, 2018). Channel depth was taken as 55 feet relative to MLLW (USACE, 2018).

The vessel-generated wave heights presented below are conservative. Large vessels (tankers and tugs) capable of creating significant waves typically transit the Kill van Kull along the channel centerline, between 500 feet and 800 feet from the project shoreline. The seabed adjacent to the project site is fronted by a wide, shallow shelf that would induce wave breaking of large vessel-generated waves before reaching the shoreline during normal conditions. Vessels are assumed to not be traveling during storm conditions with elevated water levels that would allow vessel-generated waves to reach the shoreline.

Secondary wave heights for a tanker, for a range of velocities and distances from the vessel, are illustrated in Figure 5. A tanker traveling along the near edge of the channel with a speed of 14 knots would create a maximum secondary wave height of approximately 2.7 feet along the project shoreline. A tanker traveling along the far edge of the channel with a speed of 14 knots would create a maximum secondary wave height of approximately 1.5 feet along the project shoreline. Wave periods for a tanker traveling with a speed of 14 knots are 3.8 seconds.

Secondary wave heights for a tug, for a range of velocities and distances from the vessel, are illustrated in Figure 6. A tug traveling along the near edge of the channel with a speed of 14 knots would create a maximum secondary wave height of approximately 3.8 feet along the project shoreline. A tug traveling along the far edge of the channel with a speed of 14 knots would create a maximum secondary wave height of approximately 2.1 feet along the project shoreline. Wave periods for a tug traveling with a speed of 14 knots are 3.8 seconds.

Storm waves during the design storm are greater than vessel-generated waves, and therefore control the revetment design.

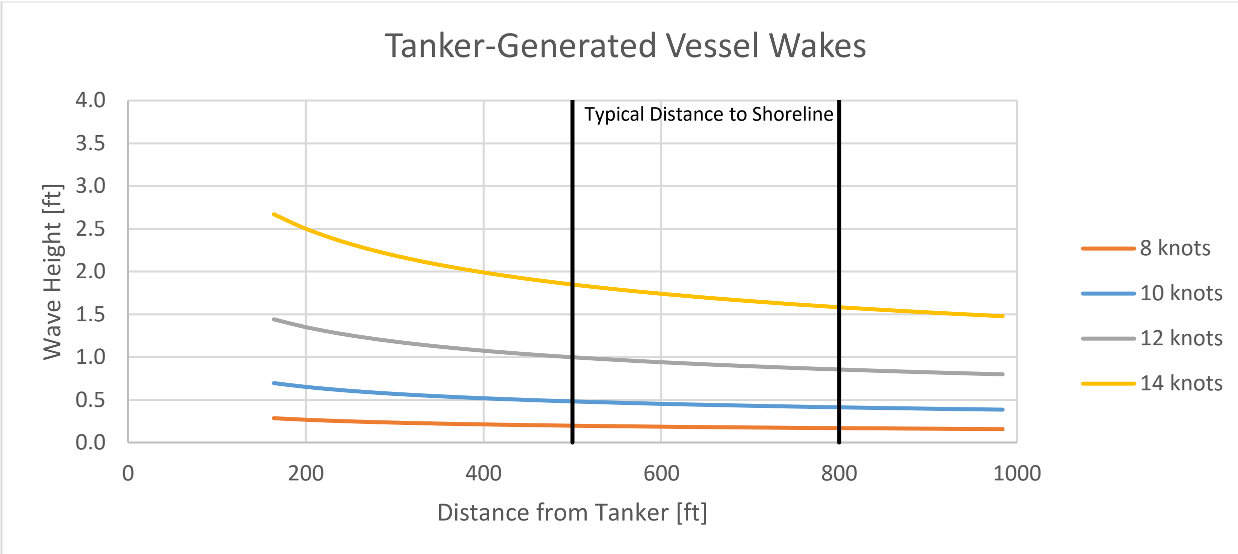


**Figure 5: Tanker-Generated Secondary Waves**

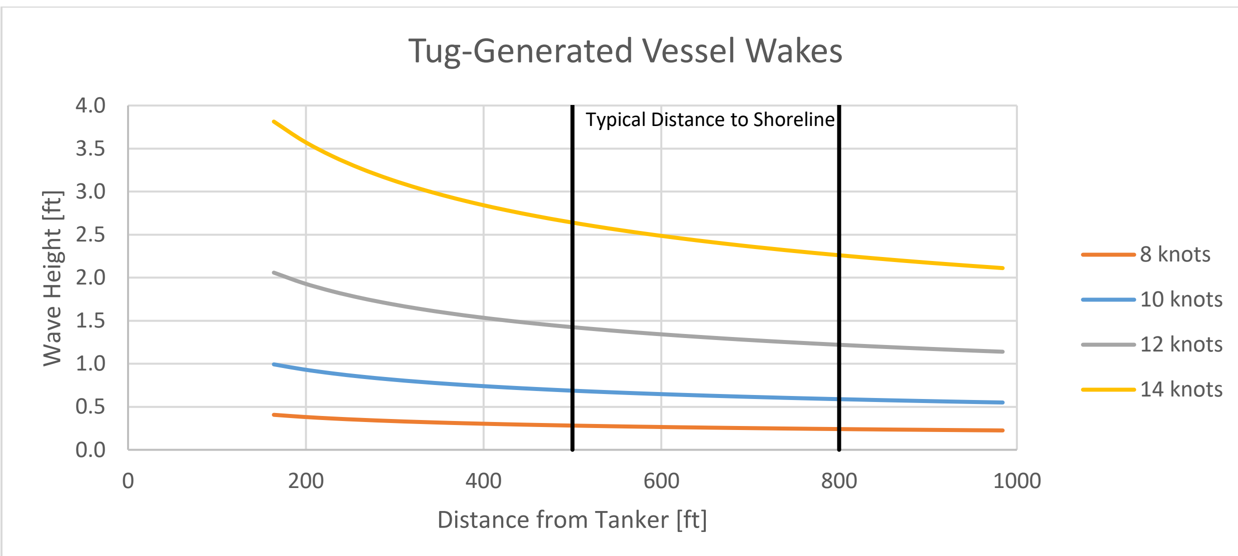


**Figure 6: Tug-Generated Secondary Waves**

**Structural Design of the Revetment**

A structural design of the revetment was performed based on the coastal engineering analysis described above and additional design constraints of the project site. APTIM performed a site visit on June 25, 2018 to evaluate the existing conditions of the project site including the revetment layout, slope, and design/construction constraints. The existing armor stone was also assessed to evaluate the armor layer thickness and size of the individual armor stone. The armor stone was a mixture of granite rocks with concrete debris. The armor stone appeared to be installed with one layer along an approximate 2H:1V slope. Representative measurements (length, width, thickness) of a representative sample of armor stones were also obtained to calculate approximate armor stone weight based on an assumed rock density of 165 pounds per cubic foot for granite. The results indicated that the average armor stone weight was 0.25 tons. Post-Hurricane Sandy repair plans for the revetment were reviewed and indicated that damaged areas of the revetment were to be repaired with 1.2 ton armor stone. However, armor stone of this size was not observed along the shoreline.

*Design Constraints*

The revetment design and layout was developed to address regulatory, Client, City, and physical requirements of the project. Two design alternatives were evaluated for the revetment: repair or replacement. The repair alternative included an overlay of the existing revetment with new armor stone. However, environmental regulations required that the existing MHWL be maintained as part of any modifications to the existing revetment; therefore, a revetment overlay was not further evaluated as the proposed structure would create a net fill of the Kill van Kull. The replacement alternative includes installation of a new revetment, which requires excavation and removal of the existing revetment to allow installation of the new revetment design template while also maintaining the existing MHWL.

*Armor Layer*

The armor layer was designed using the Hudson equation as described in the Coastal Engineering Manual (USACE, 2002), which calculates the minimum rock size required to provide stability from incident waves during the design storm as follows:

$$W_{50} = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot\theta}$$

Where

$W_{50}$ = The 50$^{th}$ percentile (median) weight of the stone
$\gamma_r$ = Density of the stone
$H$ = Design wave height at the toe of the structure
$S_r$ = Specific gravity of stone
$K_D$ = Stability coefficient
$\cot\theta$ = Design slope of the revetment

The revetment design incorporated granite for the armor stone. Local quarries near the project site typically produce granite boulders for use as rip rap and armor stone with a density between 160-165 pounds per cubic foot. As stated above, the design wave height at the toe of the structure is 4.0 feet. The specific gravity was calculated assuming the water in the Kill van Kull is brackish with a density of 64 pounds per cubic foot to be conservative. The stability coefficient was taken as 2.0 for randomly placed, angular stone. The design slope of the revetment was taken as 1.5H:1V to be consistent with similar revetment structures in New York Harbor and construction limitation of working with granite.

The armor rock size ($W_{50}$) was calculated to be approximately 900 pounds, which corresponds to a nominal (cube root) 21" diameter stone ($D_{50}$). An armor layer thickness of 2 armor stones (42") was adopted for the design. However, design guidance (USACE, 1994) recommends adding a minimum of 6" to the armor layer thickness to account for ice forces which may impact the revetment. Therefore, a total armor layer thickness of 48", with an armor stone diameter of 24", was adopted for the design.

*Foundation*

The foundation of the revetment is a transitional layer of small stone and fabric placed between the underlying soil and the structure. The foundation prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor stone to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils. In areas above the waterline, the foundation layer prevents surface water from causing erosion and washout beneath the armor stone.

Geotechnical borings (Kimball, 1998) immediately upland of the proposed revetment were reviewed to evaluate the in-situ soil conditions along the project shoreline. The underlying soil generally consists of a silty sand and sandy silt layer over a sand layer. Blow counts range from 10 to 30, indicating medium to dense soils.

A two-part foundation system was incorporated into the design consisting of a bedding stone layer and a geotextile fabric. The bedding stone gradation was taken as between $W_{50}/200$ and $W_{50}/20$, which corresponds to a gradation of 4" to 8" ($D_{50}$ = 6"). The geotextile fabric will be placed directly on the excavated soil and covered with a 12" layer of the bedding stone. The geotextile shall be a woven, monofilament fabric made of high-tenacity polypropylene yarns, which allows hydraulic flow while also limiting soil transmission, ultraviolet and biological deterioration, and rotting.

*Undermining*

Proper toe protection is required to mitigate the effects of scour at the seabed due to wave breaking. Toe protection is supplemental armoring of the bottom surface in front of the revetment to prevent waves from scouring and undermining the slope, which may result in a reduction of slope stability and potential structural failure of the revetment. Design guidance states that the toe protection be designed for the maximum depth of scour, which can be taken as one wave height (4.0 feet). Alternatively, a sufficient volume of material can be placed along the structure's toe to fall into the scour hole to provide an equal level of protection.

The structure toe elevation varies but is approximately 0.0 feet NAVD, with a maximum scour elevation of -4.0 feet NAVD. The bottom of the revetment elevation was placed at -3.0 feet NAVD with a 1 foot horizontal extension of bedding stone. A single horizontal row of armor stone was also added to the toe to provide additional toe protection and slope stability.

*Overtopping*

The revetment crest elevation was designed to account for the upland soil capping work and armor stone size. The upland capping work includes 2 feet of clean fill over the existing grade, which ranges from +6.0 feet NAVD to +10 feet NAVD. As a result, crest elevations of +8.0 feet NAVD, +10.0 feet NAVD, and +12 feet NAVD were adopted for the design. Overtopping rates were calculated using the Runup and Overtopping on Impermeable Structures module within the Automated Coastal Engineering System (ACES) program, version 1.07. Runup and overtopping rates for multiple return period storm events were evaluated and are summarized in Table 6. It is noted that the structure is submerged in different locations during different return period storm events and not subject to runup and overtopping.

**Table 6: Overtopping Rates [$m^3/s/m$]**

| **Return Period [years]** | **Crest Elevation [ft, NAVD]** | | |
|---|---|---|---|
| | **+8.0** | **+10.0** | **+12.0** |
| 10 | 0.01 | 0.00 | 0.00 |
| 25 | Submerged | 0.10 | 0.00 |
| 50 | Submerged | 0.30 | 0.01 |
| 100 | Submerged | Submerged | 0.16 |

The overtopping rates were compared to published values in the Coastal Engineering Manual (Table VI-5-6) for public and structural safety immediately upland of the revetment as illustrated in Figure 7. The calculated overtopping rates are shown to be unsafe at any speed for vehicles traveling and very dangerous for pedestrians walking immediately upland of the revetment. However, it assumed that no vehicles or pedestrians will be immediately upland of the revetment during the design storm. The calculated overtopping rates may cause damage to the area immediately upland of the revetment that is not paved. The revetment design incorporates a 4 foot wide splash area (2 armor stones) and a 2 foot wide section of bedding stone 12” below the proposed upland grade to provide stabilization from overtopping.

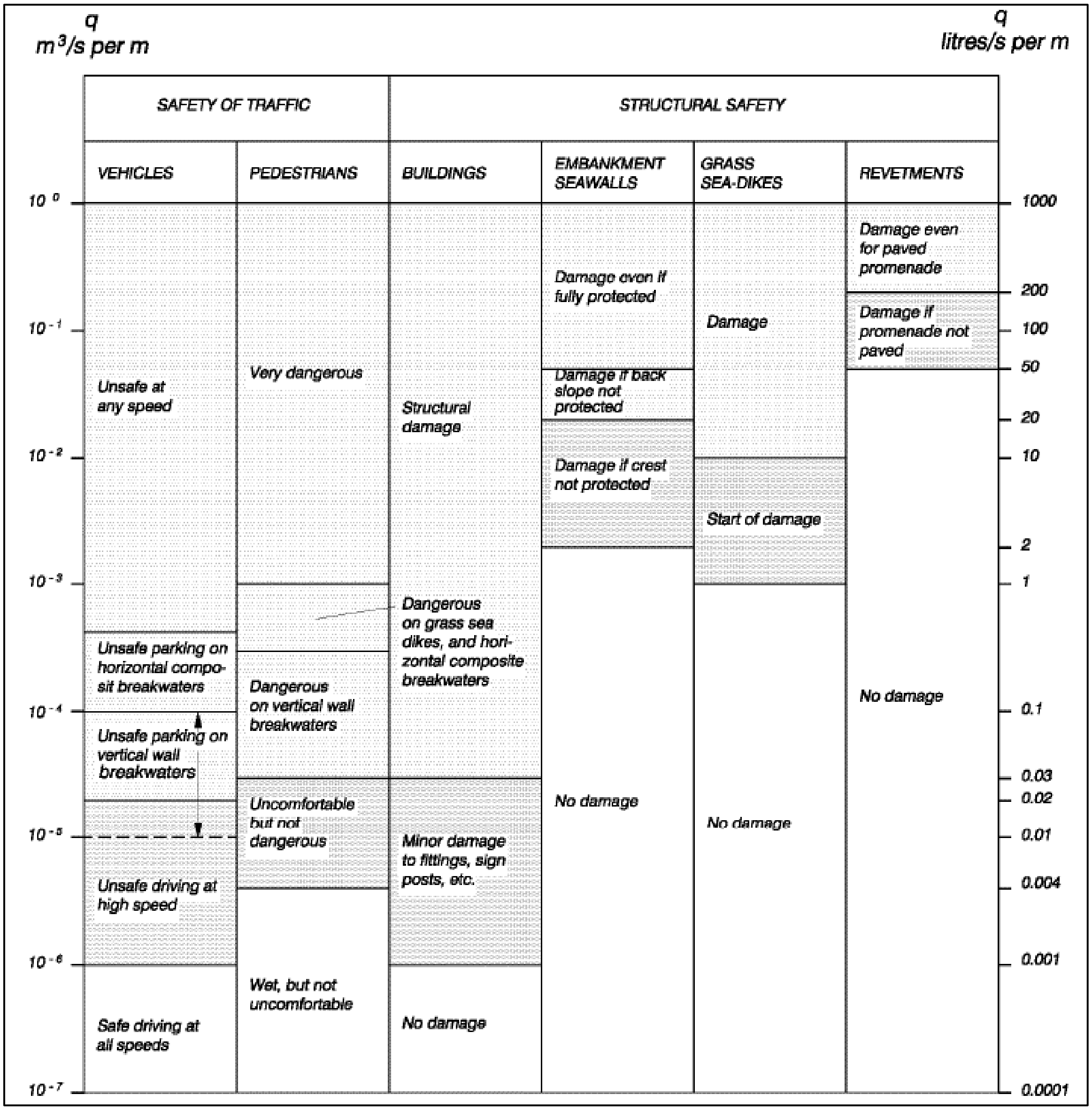


**Figure 7: Critical Values of Average Overtopping Discharges**

## Typical Design Section

The final design section for the revetment at the project site is illustrated in Figure 8. The proposed revetment section will transition to the adjacent revetment/shoreline profile at each end of the proposed structure.

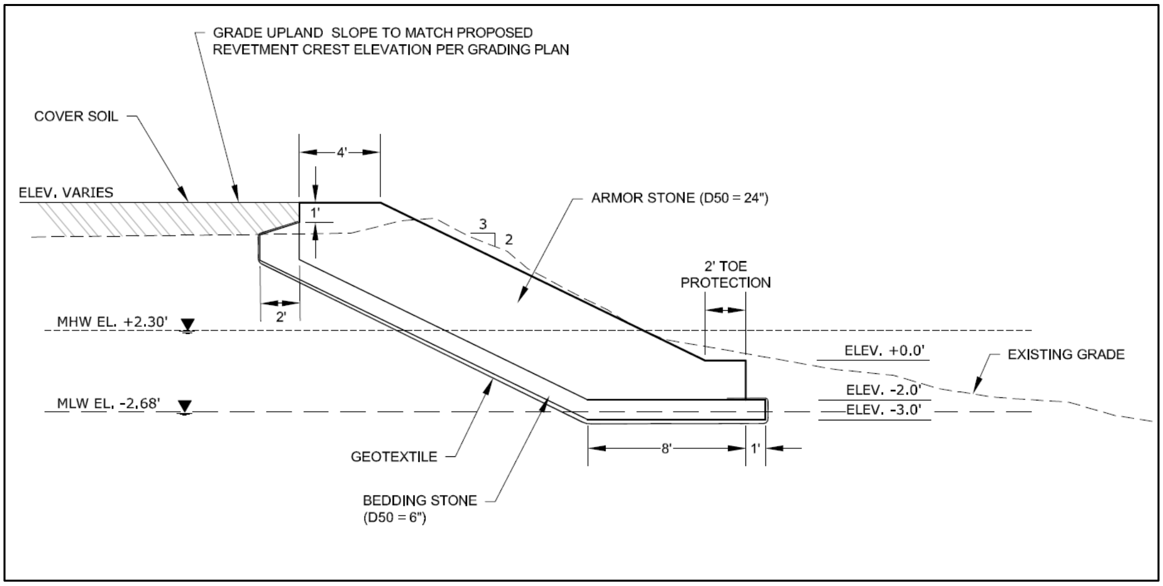


**Figure 8: Typical Revetment Design Section**

## References


1. (FEMA) Federal Emergency Management Agency, 2013. Flood Insurance Study #34017CV000A for Hudson County, New Jersey dated December 20, 2013.

2. (NOAA) National Oceanic and Atmospheric Administration, 2019. Tide station #8519483, Bergen Point West Reach, NY.

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