APPENDIX G (DRAFT):

FAIRFIELD AND NEW HAVEN COUNTIES, CT COASTAL STORM RISK MANAGEMENT (CSRM) FEASIBILITY STUDY

USE OF I-95 EMBANKMENT IN NEW HAVEN, CT AS A CSRM FEATURE, POTENTIAL FAILURE MODES ANALYSIS

1. Introduction

As part of the Fairfield and New Haven Counties, CT Coastal Storm Risk Management (CSRM) feasibility study, one of the alternatives being considered is a floodwall parallel to and just southeast of the I-95 embankment in New Haven, CT. The I-95 embankment is very wide and generally has a higher elevation than the Still Water Level (SWL) elevations being considered for protection alternatives for this study. Therefore, consideration was given to using the I-95 embankment as a CSRM structure in addition to floodwalls. In order to assess the suitability of using the I-95 embankment in this matter, the feasibility design team conducted a potential failure modes analysis (PFMA) to brainstorm failure scenarios if the embankment was used as a flood barrier.

The failure scenarios developed in this assessment considered how well the I-95 embankment would perform as a CSRM structure. Damage to the I-95 embankment itself due to wave erosion, sloughing, or bridge stability issues during a storm event was not considered “failure” for the purposes of this analysis as long as the damage did not result in flooding of the protected areas.

Note that this PFM was performed with limited information available to the team at the time of the PFMA. Additional information (foundation, as-builts of embankments and bridges, etc.) could affect the conclusions of this PFMA.

2. Background

The section of the I-95 embankment being considered for use as a CSRM feature is approximately 1.5 miles long, running the entire length of Long Wharf and up northwest where the embankment roadway transfers from I-95 to the Oak Street Connector (Figure 1). There are three roadway underpasses that penetrate the embankment and have roadway elevations lower than the required elevation for flood protection (El. 15[1]). Bridges connected to I-95 are placed over each of these underpasses. These underpasses include the southern end of Long Wharf Drive which curves northwest to Sargent Drive, Canal Dock Road located near the Long Wharf jetty, and a third located further north at Brewery St. Additionally there are two entrance ramps connecting Long Wharf Drive to I-95 (Figure 2). All three of these roadways and the entrance ramps are planned to have closure structures placed across them to prevent inundation into the areas northwest of the I-95 embankment.

Six known outfalls are located perpendicular to the embankment across the entire width of the embankment near Long Wharf (Figure 3). Two of the larger outfalls (referred to as the North and South outfalls, both 72 inch diameter) are utilized by the city to drain neighborhoods on the land side of the I-95 embankment during large flood events. It is not clear if these outfalls pass through the embankment or are largely located beneath the embankment. The latter case is more likely. Aerial photos indicate additional drainage features for the I-95 roadway are embedded within the slopes of the embankment, however the extent of these features along the length of the embankment is unknown.

[1] All elevations are in feet, NAVD88 unless otherwise noted.

The crest of the embankment varies from approximately El. 8 to El. 31, resulting in embankment heights varying from approximately 4 feet up to 28 feet. Most of the embankment is higher than the El. 15 flood protection elevation, however a portion of the embankment is lower than El. 15 (Figure 2). A flood wall is being considered near the embankment along this area to meet the El. 15 flood protection requirement. The embankment width varies from approximately 150 feet to 300 feet. The slopes of the embankment vary from 1V:5H to 1V:2H. Slopes are vegetated, with no stone protection, and appear stable (Figure 4). Embankment cross sections are shown on Figure 5 though Figure 8.

Figure 1: Extent of I-95 Embankment Analyzed for Risk Assessment

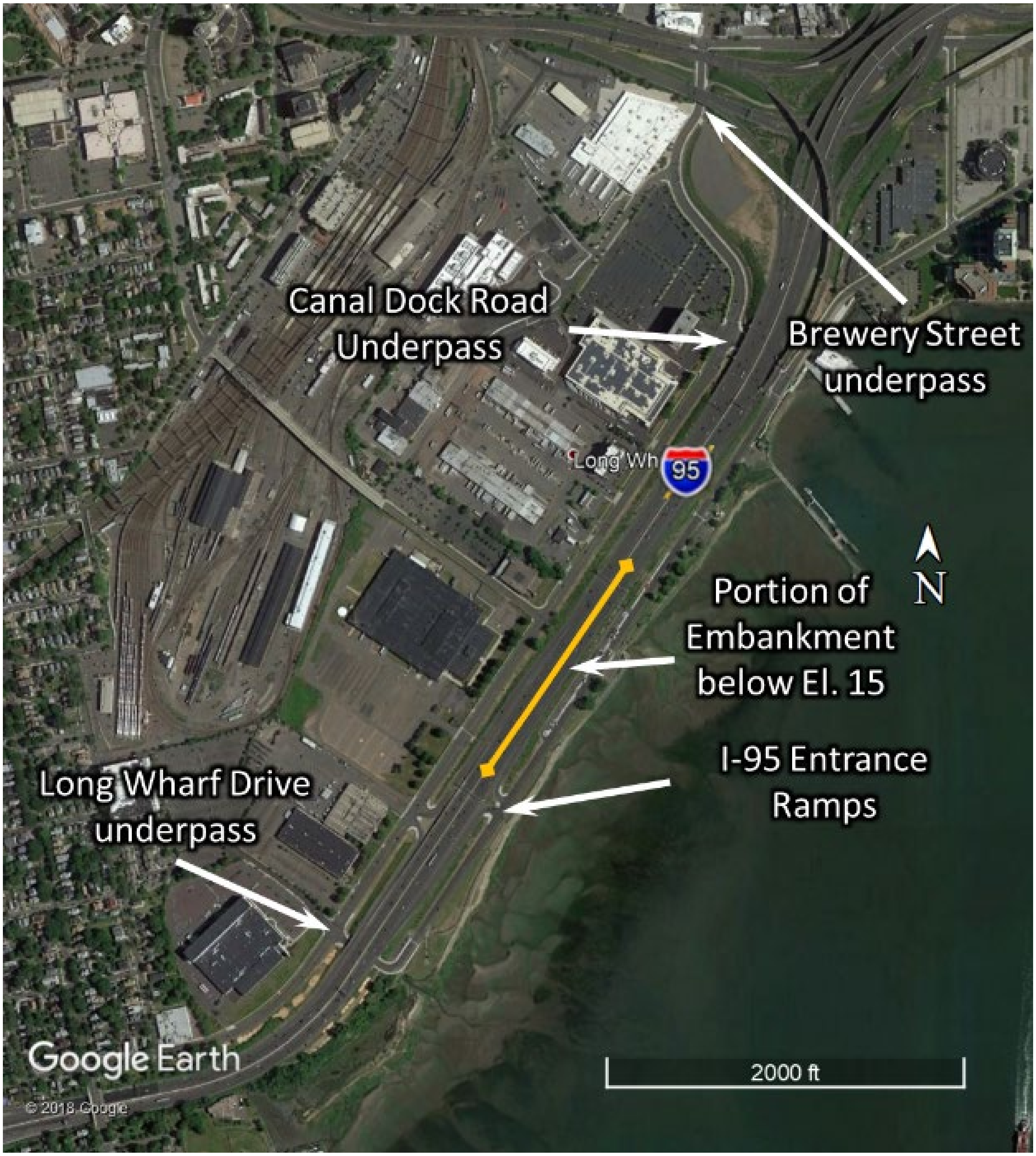

Figure 2: Significant features along area of interest.

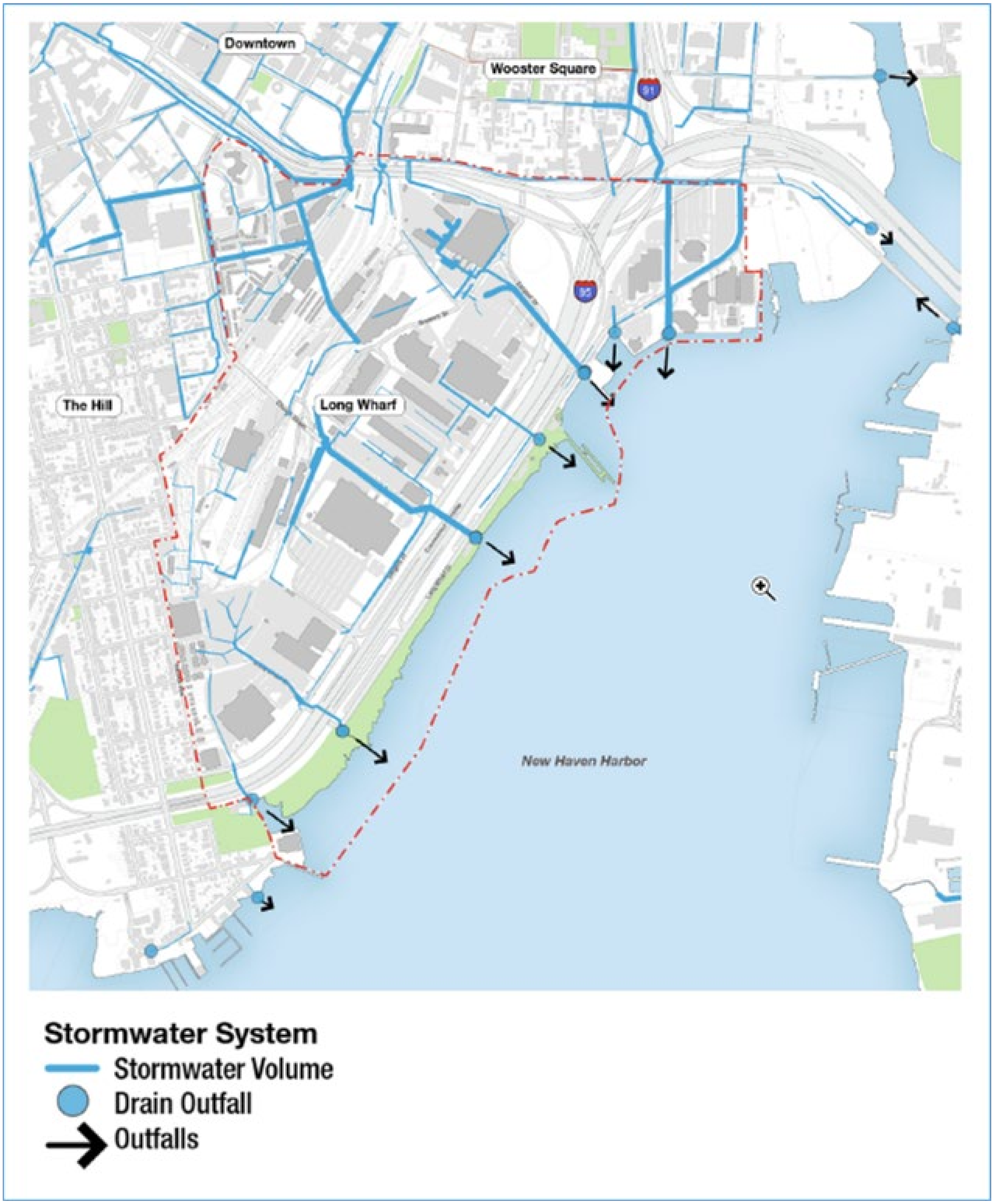

Figure 3: Known Outfalls along Long Wharf

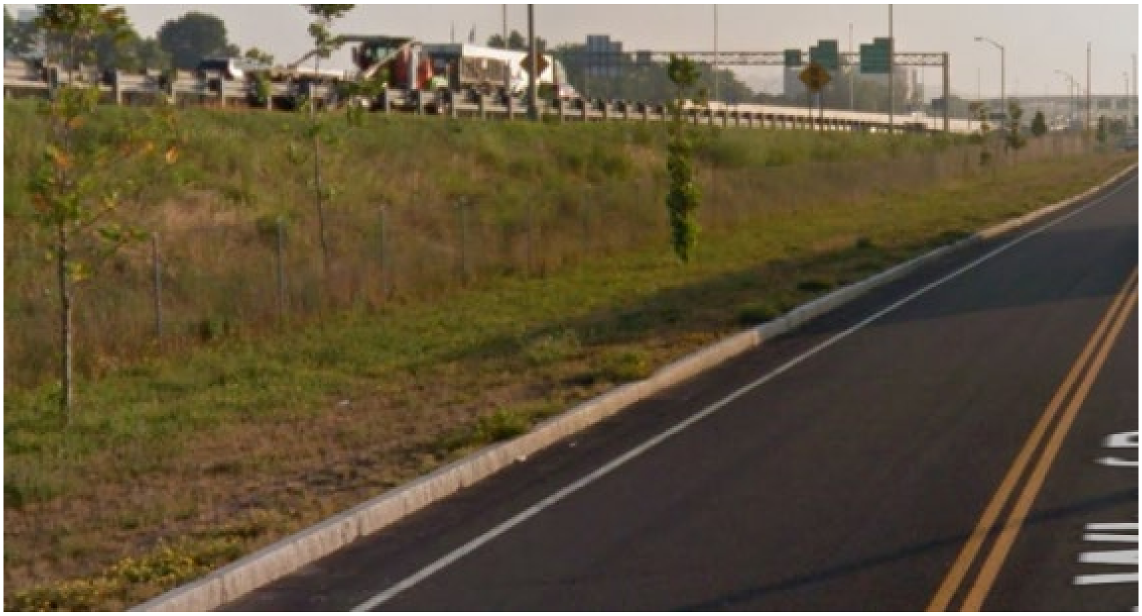

Figure 4: View of ocean side of I-95 embankment from Long Wharf Drive overpass looking northeast.

Figure 5: Locations of cross sections shown on subsequent figures

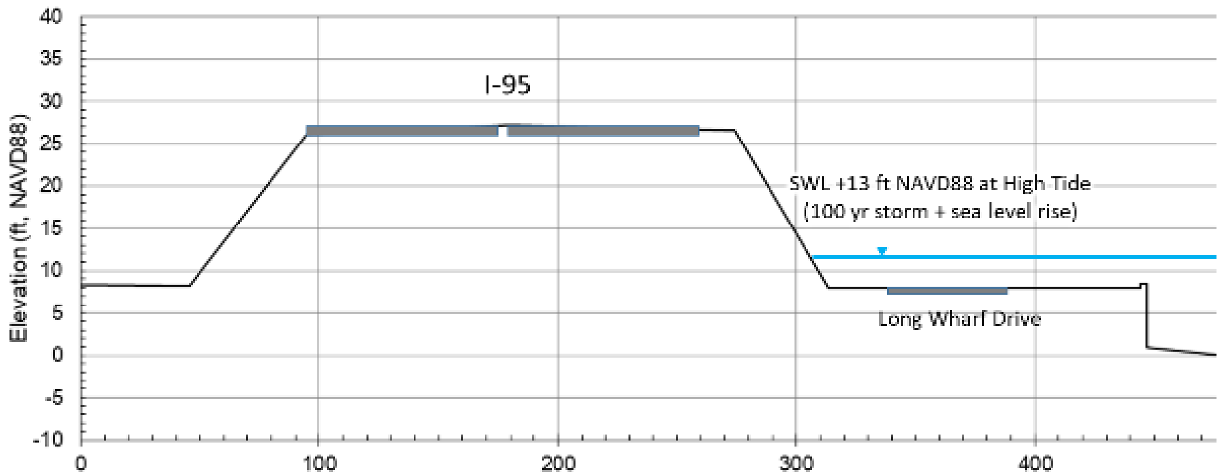

Figure 6: Cross section of I-95 embankment near Long Wharf Drive Underpass (see Figure 5 for location)

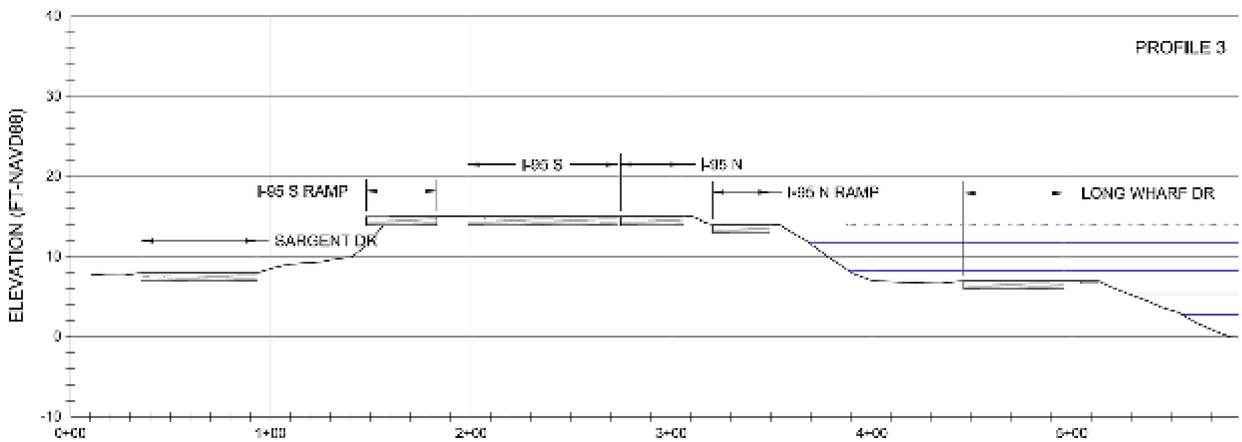

Figure 7: I-95 embankment section near entrance ramp (see Figure 5 for location)

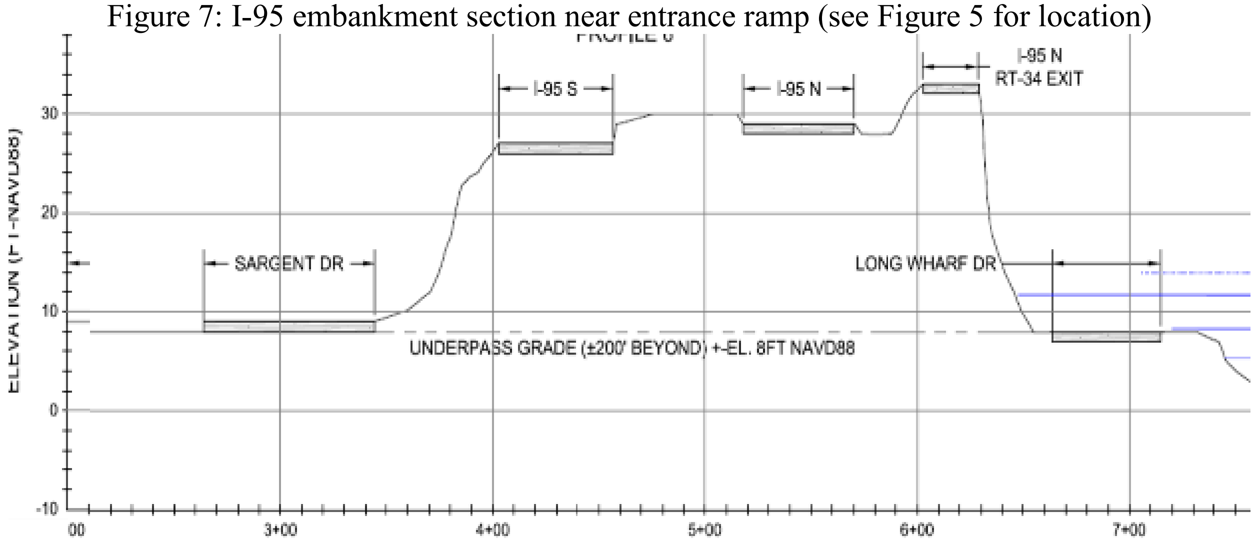

Figure 8: Embankment Section near Canal Dock Road Underpass (see Figure 5 for location)

The Long Wharf area was originally an extensive tidal marsh and the land currently present was created with fill, mostly in conjunction with the construction of the original rail line (originally along the shore) and the construction of Interstate 95 (I-95). The area was incrementally filled between the late 1700s and the early 1950s. Construction of Interstate 95 during the 1950s involved filling along the highway alignment followed by hydraulically filling of areas landward of the roadway with dredge spoils (from dredging of the New Haven Harbor). Figure 9 shows a photo of the construction of the I-95 embankment.

Figure 9: Construction of the I-95 embankment circa 1950s

Information on embankment materials is limited. While there are many available explorations that were conducted in the area over the years, only one available boring, advanced during a 2000 exploration program, penetrated the I-95 embankment. (PB-5, Figure 10). Soil descriptions of the embankment materials indicate that the embankment is made primarily of compacted fine sand with silt. The embankment materials appear to have been placed directly on top of a thick layer of organic silts (approximately 30 to 40 feet). Additional discussions regarding foundation materials below the embankment is included in the main portion of the Geotechnical Engineering Appendix.

Figure 10: Approximate location of available I-95 Embankment Soil Borings

As-built drawings of bridge overpasses in the area of concern were not readily available. However, based on preliminary design reports, the bridges are supported on pile caps, with Mechanically Stabilized Earth (MSE) wingwalls at either end (Figure 11). An MSE retaining wall is a composite structure consisting of alternating layers of compacted backfill (possibly lightweight fill) and soil reinforcement elements, fixed to a wall facing. The stability of the wall system is derived from the interaction between the backfill and soil reinforcements, involving friction and tension. The wall facing is relatively thin, with the primary function of preventing erosion of the structural backfill.

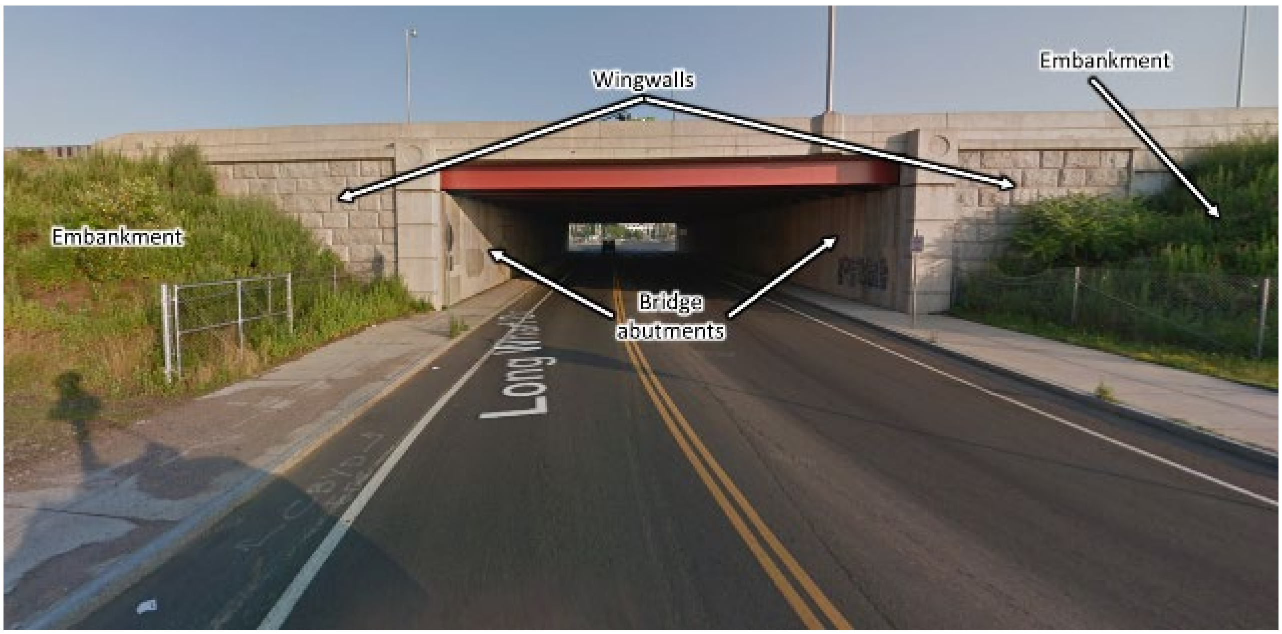

Figure 11: Bridge overpass at Long Wharf Drive viewed from ocean side.

3. Potential Failure Modes Analysis (PFMA)

A failure mode is a unique set of conditions and/or sequence of events that could result in failure, where failure is "characterized by the sudden, rapid, and uncontrolled release of impounded water" (FEMA 2003). A Potential Failure Mode Analysis (PFMA) is the process of identifying and fully describing potential failure modes. A PFMA for the I-95 embankment was conducted in April 2019. A facilitator guided the team members in developing the potential failure modes, based on the team's understanding of the project vulnerabilities resulting from the data review and current field conditions. The risk assessment team was comprised of the following individuals:

Erik Matthews;	Geotechnical Engineer, Facilitator
Doug Fransioli;	Geotechnical Engineer
Thuyen Nguyen;	Structural Engineer
Michael Sears;	Structural Engineer
Lisa Winter;	Coastal Engineer
Byron Rupp;	Planning Study Manager

A review of background data was initially conducted by the PFMA team. Additionally, the team assumed that all bridge overpasses and low spots along the embankment would have flood protection structures (closure structures, seawalls) that would provide protection at those locations. The team was then allowed to focus on the potential weaknesses in the embankment itself, assuming that the new structures added would perform as designed.

The team then "brainstormed" potential failure modes for the I-95 embankment and the bridge overpass structures, which if they occurred could lead to compromising of the embankment. The following list of potential failure modes was developed by the team.

Potential Failure Mode (PFM)	PFM Description
1	Excess Pore pressures behind I-95 bridge abutment leads to failure of abutments
2	Wave loading on bridge superstructure leads to structural failure
3	Closure structure supports load bridge abutments leading to bridge failure
4	Water loading on MSE bridge wingwalls leads to failure of walls and embankment
5	Wave attack on I-95 embankment results in failure
6	Wave overtopping of I-95 embankment results in failure
7	Concentrated Leak Erosion along outfall structures results in failure of embankment
8	Concentrated Leak Erosion along bridge abutment structures results in failure of embankment
9	Backward Erosion Piping through I-95 embankment

Those potential failure modes that were judged to be non-risk drivers (specifically PFM 2 and PFM 3) were excluded from further consideration. For the remaining 7 PFMs, the pertinent background and performance data for each potential failure mode were discussed. Then, a complete description was prepared from initiation to breach. The discussion was then expanded to listing factors, data, or conditions that suggest the failure mode is more likely or less likely to occur and establishing the appropriate level of consequences. Lastly, any recommendations for risk-reduction actions, instrumentation and monitoring, additional data, or analysis were discussed.

Failure modes fell into three categories:

1) Structural instability of bridge overpasses (PFM 1, 2, 3, and 4)
2) Wave attack/overtopping of I-95 embankment (PFM 5 and 6)
3) Embankment instability caused by through seepage (PFM 7, 8, and 9)

Each PFM is discussed in more detail in the following paragraphs. All PFMs looked at loading events up to the 100-yr storm surge/wave loading (El. 13). Worksheets used to brainstorm all of the non-excluded PFMs are located at the end of this report.

3.1 PFM 1: Excess Pore pressures behind I-95 bridge abutment leads to failure of abutments

The team considered a conceptual failure mode that could occur during a severe storm. Wind setup during the storm increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. 8 (the base of the bridge abutments at Long Wharf Drive and

Canal Dock Road). Seepage initiates through the bridge abutment wing wall fascia and into the structural fill behind either of the bridge abutment walls. Over repeated tidal cycles during the storm the SWL approaches a maximum of El. 13. Hydrostatic pressure builds up behind the abutment walls. There are no weep holes in the abutment walls to reduce pressures. Hydrostatic pressures build to a point where the abutment walls become unstable. One or both of the walls rotates, destabilizing the embankment and wingwalls, creating an opening at the ends of the closure structure. Water flowing around the closure structures erodes embankment material, resulting in a larger opening for water to flow through and flood the interior.

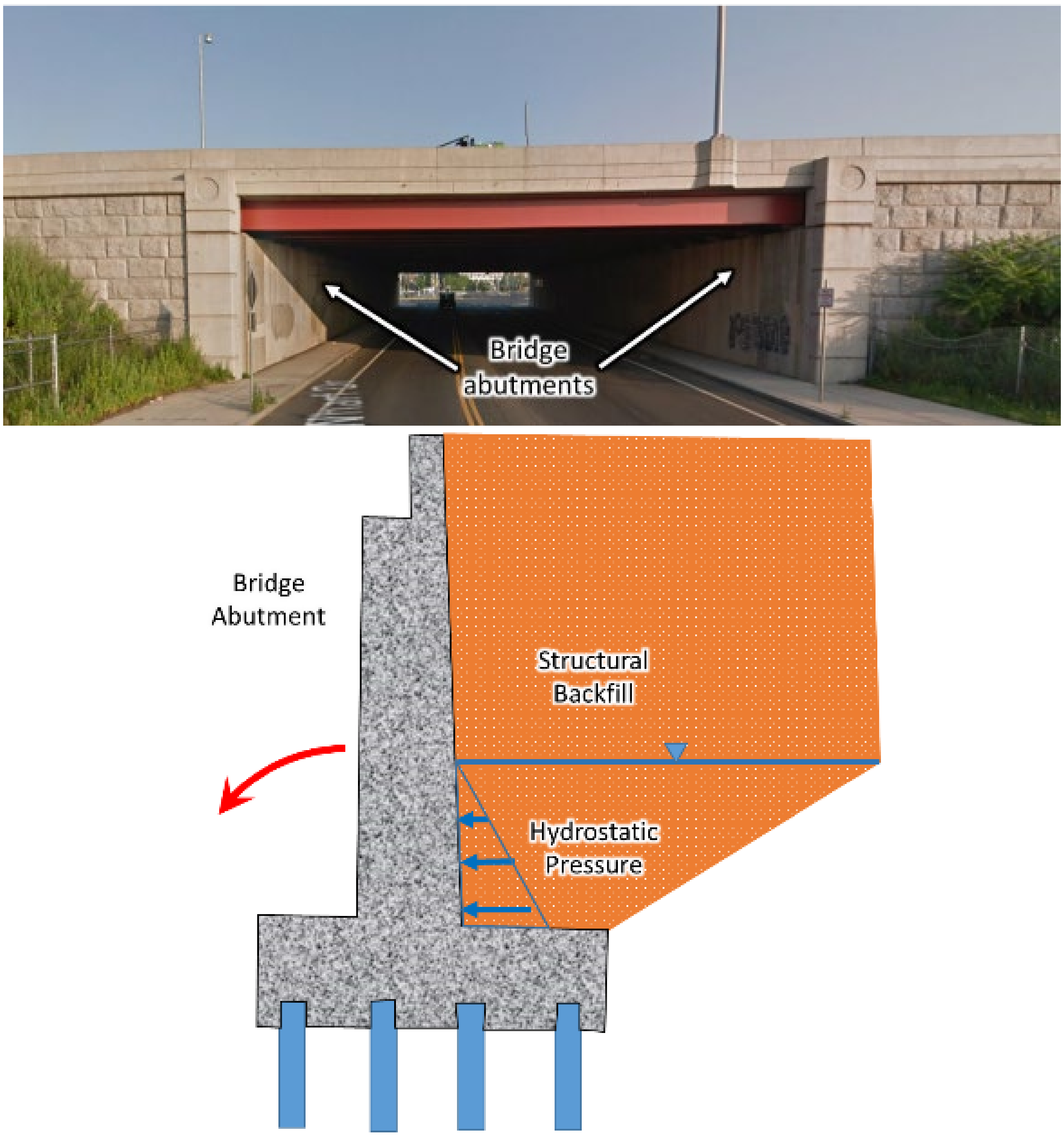

Figure 12: PFM 1 Failure sketch (assumed configuration of abutments)

The duration of loading expected during an extreme storm event (above El. 8) would be short (less than 12 hours). The team considered that this duration of loading would not be sufficient to saturate the structural fill behind the wall. Additionally, the maximum loading resultant would only extend a maximum of a third of the way up the wall, and would not be sufficient to cause the wall to fail. The bridges were also renovated within the last five years, and are assumed to have utilized the latest design and construction techniques. The team concluded that it was unlikely that the expected depth and duration of loading during an extreme event would be sufficient to result in instability of the abutment walls.

3.2 PFM 2: Wave loading on bridge superstructure leads to structural failure - EXCLUDED

This PFM considers whether wave loading on the bridge superstructure during an extreme storm event could lead to failure of the embankment or compromise the closure structures.

Figure 13: Photo showing typical bridge superstructure

During a severe storm, waves would intermittently hit the closure structure to be installed in front of the bridge (the closure structure would be designed to withstand this wave attack). The waves would intermittently splash over the closure structure, resulting in spray hitting the superstructure, but resulting in no significant additional wave loading. The bridge superstructure is significantly higher than top of the proposed closure structures for both overpasses. Given this, the team decided that this PFM was not a credible failure mode.

3.3 PFM 3: Closure structure supports load bridge abutments leading to bridge failure - EXCLUDED

This PFM considers whether the new closure structure supports could load the existing bridge abutments resulting in failure of the bridges. As the closure structures will be designed independent of the bridge abutment, there is no additional loading on the bridge abutments expected. The team determined that this PFM was not credible and was excluded.

3.4 PFM 4: Water loading on MSE bridge wingwalls leads to failure of walls and embankment

The team considered a conceptual failure mode that could occur during a severe storm. Wind setup during the storm increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. +8 ft NAVD88 (the base of the bridge abutments at Long Wharf Drive and Canal Dock Road). Seepage initiates through the bridge abutment wing wall fascia and into the structural fill behind the walls. Over repeated tidal cycles during the storm the SWL approaches a maximum of El. +13 ft NAVD88. Hydrostatic pressure builds up within the MSE backfill over repeated tide cycles, resulting in instability and failure of the MSE wall. Failure of the MSE wall exposes the embankment material and structural fill behind the wall to wave attack. The embankment then fails due to wave erosion, causing flooding of the interior.

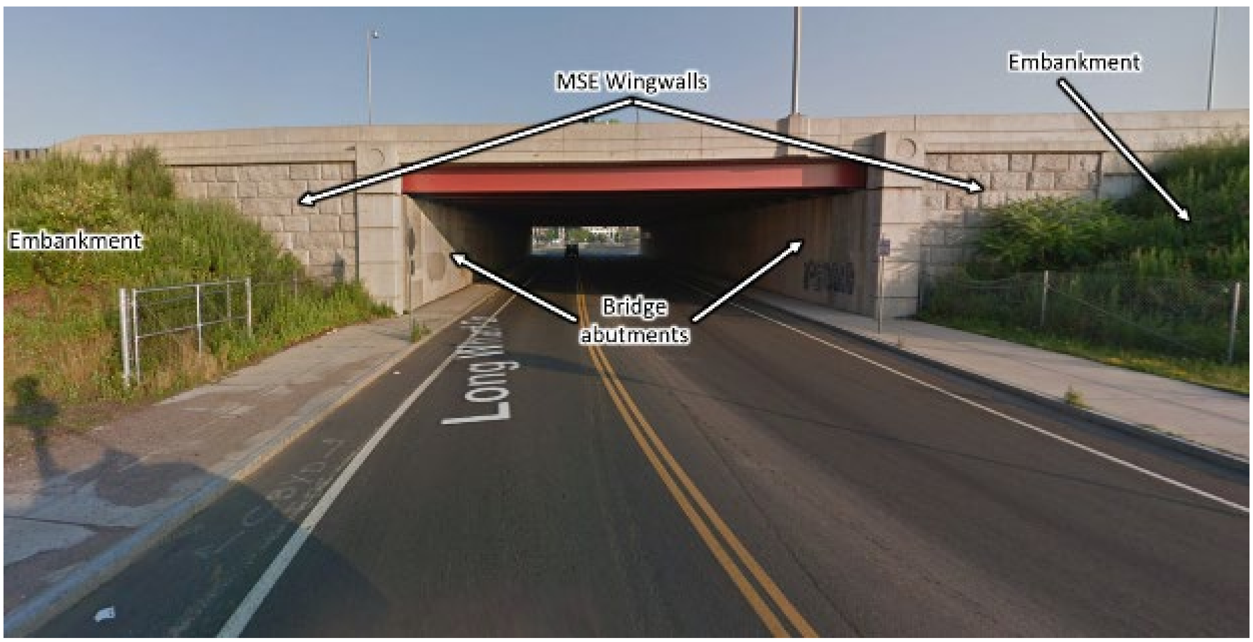

Figure 14: Photo showing location of MSE wingwalls

As discussed previously, the bridges have Mechanically Stabilized Earth (MSE) wingwalls at either end (Figure 14). An MSE retaining wall is a composite structure consisting of alternating layers of compacted backfill (possibly lightweight fill) and soil reinforcement elements, fixed to a wall facing (Figure 14). The wall facing is relatively thin, with the primary function of preventing erosion of the structural backfill.

The team considered whether the MSE facing would be pervious enough to allow water to enter the backfill (which may be lightweight fill) resulting in excess hydrostatic pressures within the backfill that would lead to MSE wall instability. The team concluded that while it was possible that some seepage could enter the joints in the fascia during high water loading, the duration of loading would not be sufficient to fully saturate the wall. Additionally, the MSE wall structure has multiple layers of geogrid within the backfill that would provide resiliency. The team therefore concluded that it was very unlikely that loading during an extreme event would lead to instability of the MSE walls.

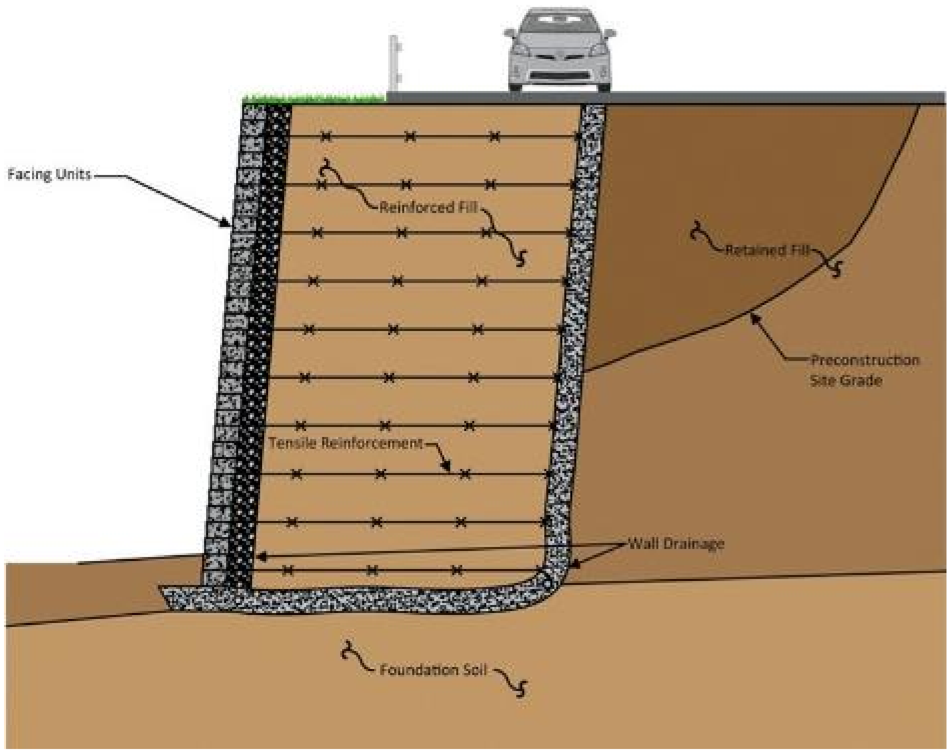

Figure 15: Typical MSE wall section (not specific to this project)

3.5 PFM 5: Wave attack on the embankment results in erosion and breach of embankment

As the SWL rises above El. +8 ft NAVD88 (the toe of the I-95 embankment), breaking waves would begin attacking the I-95 embankment. The embankment is vegetated but does not have any slope protection (Figure 16). The granular materials of the embankment are vulnerable to erosion. The progression of this PFM would consist of undermining of the seaward slope of the embankment, resulting in progressive sloughing. As the SWL rises further to El. 13 (maximum loading), wave attack would accelerate. Sloughing would continue to the landward side of the embankment, resulting in breach and flooding of the interior.

The PFMA team concluded that the embankment would be vulnerable to wave attack. However, given that there is anywhere from 150-300 ft of land seaward of the toe of the embankment, waves that impact the embankment would have already broken and lost most of their energy. Additionally, waves would only be impacting the embankment for a relatively short duration (mid-to-peak tide cycle) and would be more likely to wash against the embankment rather than break on the embankment. Finally, even if erosion of the embankment did occur, the duration of wave attack would not be sufficient to erode through the entire 150-300 ft of embankment.

The team therefore concluded that it was unlikely that wave loading during an extreme event would lead to breach of the embankment. Damage due to wave attack could be effectively controlled by installing slope protection on the seaward slope of the embankment.

Figure 16: Typical condition of seaward face of I-95 embankment

3.6 PFM 6: Wave runup and overtopping of the embankment results in erosion and breach of embankment

Similar to PFM 5, this PFM would require the SWL to rise above El. +8 (the toe of the I-95 embankment). As the SWL continues to rise, water from breaking waves would begin washing up on the I-95 embankment. As the SWL increases, if runup exceeds the embankment crest, overwash will occur. Increasing water surface would result in an increase of wave overwash, initiating erosion of the embankment at the crest. As overwash continues, downcutting of the embankment could accelerate and continue to the landward side of the embankment.

The mean height of wave runup is estimated at 2 ft above the design level of protection, or El. 15. This magnitude of runup would only occur periodically during the peak of the storm. The majority of the embankment that would be subject to wave runup has a crest elevation at or above the maximum runup elevation. Wave runup would be intermittent, and be most likely to result in minor flooding of the roadway. The embankment crest carries the I-95 roadway, and the asphalt would resist the minor expected overwash. While overwash could cause some localized erosion near the crest, it would not be expected to lead to breach of the 150-300 ft wide embankment.

The team therefore concluded that the probability that wave overtopping during an extreme event would lead to breach of the embankment was remote.

3.7 PFM 7: Concentrated Leak Erosion along penetration (storm sewer pipe, conduit, other pipe) through I-95 embankment leads to failure of embankment

The term "Concentrated Leak Erosion" refers to flow through cracks in an embankment rather than flow through the pores of intact embankment soil. Where there is an opening through which concentrated leakage occurs, the walls of the opening may be eroded by the leaking water, leading to enlargement of the crack and subsequent breach.

This PFM considers whether poor compaction around a conduit through the I-95 embankment has resulted in loose or poorly compacted zones around the perimeter and along the length of the conduit. The SWL rises above El. 8 (the base of the I-95 embankment), and concentrated seepage begins to flow through the loose zones along the pipe. Sufficient gradients and velocities exist to move particles, resulting in erosion of soil along the pipe. Flow exits unfiltered on the landside of the I-95 embankment. The embankment materials are able to support the widening crack. Crack filling/self-healing does not occur, and a flow limiter is not present. The rate of erosion increases as the pipe continues to enlarge. The embankment breaches, resulting in flooding of the interior.

As shown previously on Figure 3, there are several known storm sewer outfalls along the I-95 embankment. Based on limited as-built information available during the PFMA, as well as visual inspection of some of the outfall locations, it appears that these outfalls all pass well below the I-95 embankment (Figure 17). The PFMA team concluded that for this PFM to occur, a penetration would have to extend through the entire embankment at the base (maximum loading). Whether penetrations that meet this criteria exist along the embankment is unknown. The embankment is assumed to consist primarily of a silty sand that may have sufficient fines to hold a crack open. However, maximum gradients at future predicted high tide (with sea level rise) are very low (0.025) and would not be sufficient to initiate concentrated leak erosion. The duration of loading during the 100 year storm would not be sufficient to fully saturate the embankment, which would be required for this PFM to progress.

The team concluded that the probability that CLE along a penetration during an extreme event would lead to breach of the embankment was very unlikely primarily due to the low gradients and the short duration of loading.

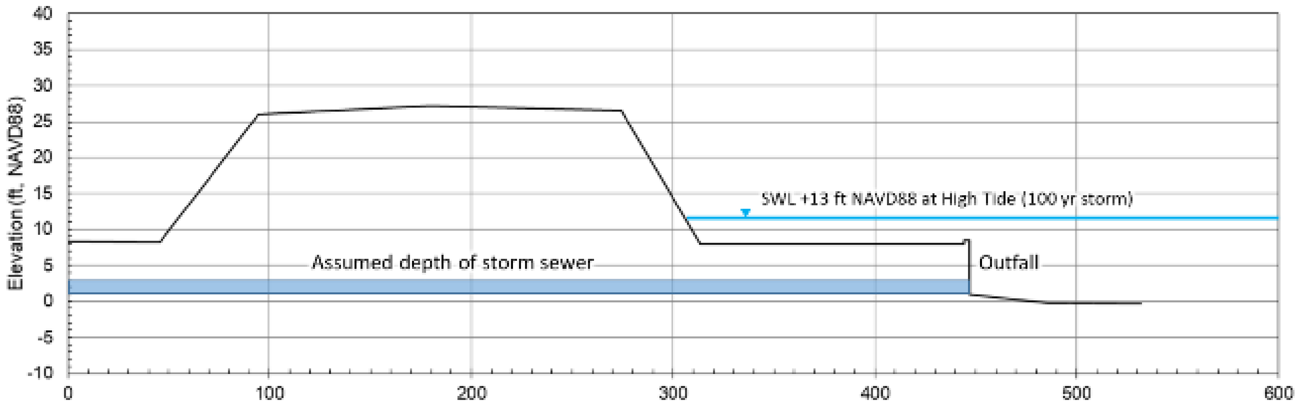

Figure 17: Storm sewer outfall near Long Wharf Drive Bridge under I-95 embankment (other outfalls similar)

3.8 PFM 8: Concentrated leak erosion along the highway embankment/bridge abutment contact leads to embankment failure and breach

This PFM is similar to PFM 7, however a crack due to poor compaction of structural backfill along the backfill/bridge abutment contact is considered (Figure 18). During the 100-year storm, the SWL rises above El. 8 (the base of the bridge abutments at Long Wharf Drive and Canal Dock Road). Concentrated seepage begins to flow through the crack, eroding soil along the crack. There is no landward side filter to hinder the movement of particles. Over repeated tidal cycles during the storm the SWL approaches a maximum of El. 13, the rate of erosion increases, and the crack continues to grow. Increasing flow through the crack leads to gross enlargement of the crack and downcutting of the embankment. The embankment breaches, resulting in flooding of the interior.

The nature of the structural backfill material is not known, however it is assumed to be a free draining granular material that would likely not hold a crack. Wingwalls at the landward side would prevent exit of eroded embankment material (crack would not be able to enlarge). Maximum gradients at future predicted high tide (with sea level rise) are very low (0.025) and would not be sufficient to initiate concentrated leak erosion. Bridges were rebuilt in the last five years and assumed to utilize the latest design and construction techniques, therefore poor compaction or defects are unlikely.

The team concluded that the probability that CLE along the bridge abutment / embankment interface during an extreme event would lead to breach of the embankment was very unlikely given the available information.

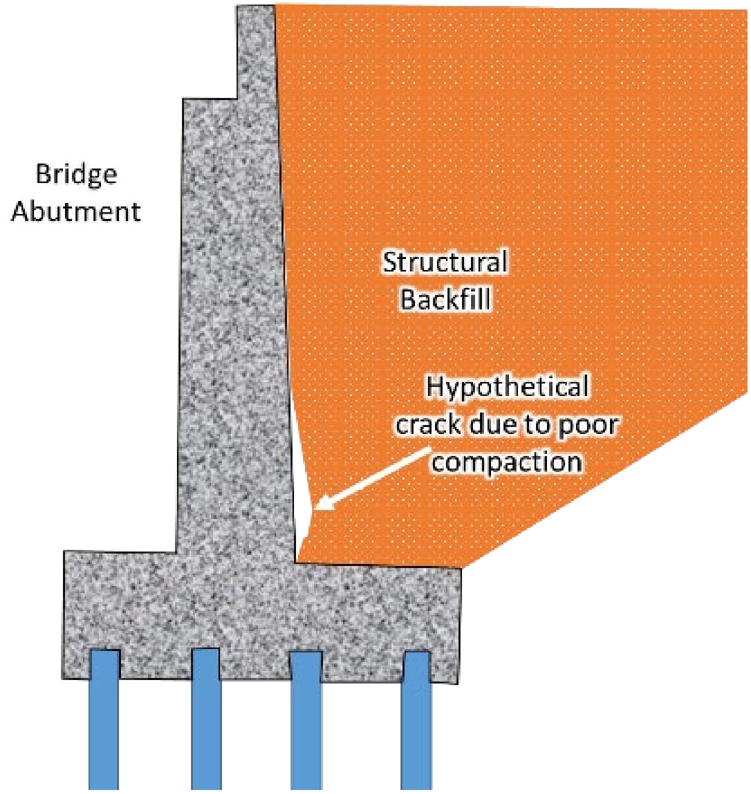

Figure 18: Section of bridge abutment showing hypothetical crack through structural backfill along face of bridge abutment.

3.9 PFM 9: Backward Erosion Piping through highway embankment leads to embankment failure and breach

Backward erosion piping (BEP) occurs when soil erosion (particle detachment) begins at a seepage exit point and erodes backwards (towards the impounded water), supporting a “pipe” or “roof” along the way. As the erosion continues, the seepage path gets shorter, and flow concentrates, leading to higher gradients, more flow, and higher potential for erosion to continue. Four conditions must exist for BEP to occur: 1) flow path or source of water; 2) unprotected or unfiltered exit; 3) erodible material within the flow path; and 4) continuous stable roof forms allowing a pipe to form. Backward erosion piping occurs in cohesionless soils or those with a low plasticity index (PI). The erosion process begins at a free surface on the landside or downstream side of the embankment.

For this PFM, the team considered that the 100-year SWL loading would result in seepage through the embankment. Once the embankment was fully saturated, seepage flow would exit at the landward side of the I-95 embankment. Gradients would have to be sufficient to result in a pipe developing and progressing from the landward to the seaward slope. The embankment materials would need to contain enough fines material to hold a roof. There is no flow limiter on the landside slope. Gross enlargement of the pipe would result in collapse of the embankment. The embankment would breach, resulting in flooding of the interior.

In order for this PFM to progress, the embankment would need to become fully saturated during the storm event. The embankment material appears to consist of a silty fine sand material that may be susceptible to BEP. However, based on the visual descriptions of soils collected from the 2000 boring (PB-5), the material could have up to 35% silt, which would make the material less susceptible to BEP. While the maximum loading on the embankment would be El. 13, the duration of loading above El. 8 (the base of the embankment) would only be a few hours, which would not be sufficient to saturate the entire 150-300 ft width of the embankment. Maximum gradients at future predicted high tide (with sea level rise) are very low (0.025) and would not be sufficient to initiate BEP.

The team concluded that the probability that BEP through the embankment during an extreme event would lead to breach of the embankment was remote given the short duration of loading and the low expected seepage gradients.

4. Conclusions

Based on the qualitative analysis described in this memorandum, the team consensus is that the I-95 embankment could be incorporated into the New Haven CSRM as a storm risk management measure. Floodgates will be required at the bridges just as they would be for a floodwall in this area.

Of all the PFMs considered in the PFMA, the team was most concerned about wave attack on the embankment. Given the relatively short duration of wave loading during an extreme event, erosion of the seaward face of the embankment would be expected, but would not lead to significant erosion back into the I-95 embankment. This type of erosion damage would occur during the 100-year event even for the existing (i.e. no project) condition, and placing flood barriers along the bridge underpasses would not make wave attack on the embankment any worse. Placing properly sized riprap or other slope protection would mitigate this damage.

Seepage related PFMs were not considered to be of high concern given the available information, as loading is not expected to be of sufficient duration to allow these PFMs to progress to failure. Without additional information, it is unclear what seepage mitigation measures would be needed if they were deemed necessary.

PFMs related to bridge stability are very unlikely to occur, and even if they did, would be unlikely to result in subsequent breach of the embankment. Drains could be installed at the base of the abutment walls to reduce seepage pressures if necessary.

There is uncertainty in this assessment as the makeup and condition of the embankment, foundation, and bridge structures was unknown as the PFM was performed with limited information available to the team at the time of the PFMA. Additional information (foundation, as-builts of embankments and bridges, etc.) could affect the conclusions of this PFMA.

5. Recommendations

The PFMA was conducted with limited available information. While the team did receive a large package of as-built drawings and reports, much of the information that would have provided clarification for some of the PFMs could not be readily located. Information regarding the nature of the embankment fill materials is unclear, as are the configurations of the bridge overpasses. It is recommended that the following investigations/analyses be performed during the design phase of the project:

- Perform a thorough review of all available as-built and design information be conducted. This should be done in coordination with personnel at the CT DOT who are knowledgeable regarding this stretch of the I-95 embankment. This should include identifying penetrations which may exist through or underneath the I-95 embankment, as well as a thorough review of all bridge overpass as-built drawings.
- Conduct subsurface borings on the landward and seaward sides of the embankment to better define the nature of the fill materials within the embankment. Due to the high traffic volume, explorations within the embankment itself may not be feasible. If there are other available historic borings that contain this information, then additional borings may not be necessary.
- A seepage and slope stability analysis of the embankment should be conducted as part of the design phase to confirm the assumptions made for the seepage related failure modes discussed above.
- While this PFMA indicates that the embankment would perform adequately during the 100-year design storm, it is critical that the state and federal DOT have complete acceptance of the use of this embankment as a CSRM feature.
- During the design phase, it is strongly recommended that all changes or improvements to the I-95 embankment be closely coordinated with the CT DOT. This should be initiated at the beginning of the design phase, as well as at multiple points during the process.
- It is recommended that during the design phase, a more robust PFMA be conducted, and include CT DOT and others as appropriate.

NEW HAVEN SHORELINE POTENTIAL FAILURE MODE ANALYSIS JUDGED TO BE SIGNIFICANT OR CREDIBLE	
PFM # 1: Excess Pore pressures behind bridge abutments leads to destabilization and failure of abutments	
Loading and Duration: 1/100 ACE, SWL El. +13 ft NAVD88 (includes storm surge and 50-yr sea level rise). Maximum loading occurs at the peak of the storm (high tide)	
Detailed Description of Failure Mode: An approaching storm results in storm surge (wind setup) which increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. +8 ft NAVD88 (the base of the bridge abutments at Long Wharf Drive and Canal Dock Road). Seepage initiates through the bridge abutment wing wall fascia and into the structural fill behind either of the bridge abutment walls. Over repeated tidal cycles during the storm the SWL approaches a maximum of El. +13 ft NAVD88, hydrostatic pressure builds up behind the abutments. There are no weep holes in the abutment walls to reduce pressures. Hydrostatic pressures build to a point where the abutment walls become unstable. One or both of the walls rotates, destabilizing the embankment and wingwalls, creating an opening at the ends of the closure structure. Water flowing around the closure structures erodes embankment material, resulting in a larger opening for water to flow thorough and flood of the interior.	
Conditions making PFM Likely Or Unfavorable Factors	**Conditions making PFM Unlikely Or Favorable Factors**
• Likely not designed for excess hydrostatic pressures, as there are no apparent drainage features (e.g. weep holes) in the visible portion of the wall • Structural fill behind the walls are pervious granular materials that would saturate more rapidly • Relatively short seepage path from embankment/wingwall face to abutment walls • MSE wall fascia are modular and joints between fascia would allow seepage to enter	• Storm duration would only be a few days. • Tidal fluctuations would make the period of maximum loading very short, making full saturation of the fill behind the wall less likely • Wing walls may be more pervious and would allow excess pressures to dissipate when tides recede. • Limited information about embankment fill suggest that it is a silty sand that would be less pervious • Bridges were rebuilt in the last five years and assumed to utilize the latest design and construction techniques • Maximum loading would only extend a third of the way up the abutment wall (at high tide and peak of storm) • Wingwall fascia may not be very pervious, and therefore may only allow limited seepage
Other Considerations and Concerns: • Available preliminary design calculations show that backfill behind wall was to have a maximum unit weight of 65 lb/cf • Assumed that wing walls are MSE[1] walls with fascia on the outer surface that would allow seepage through the joints between the units 1. A Mechanically Stabilized Earth (MSE) retaining wall is a composite structure consisting of alternating layers of compacted backfill and soil reinforcement elements, fixed to a wall facing. The stability of the wall system is derived from the interaction between the backfill and soil reinforcements, involving friction and tension. The wall facing is relatively thin, with the primary function of preventing erosion of the structural backfill. The result is a coherent gravity structure that is flexible and can carry a variety of heavy loads.	
Knowledge Gaps and Data Uncertainties: • We do not have as-builts of existing bridge abutments and wingwalls	

• Only limited subsurface information and minimal data on fills used behind bridge
One (or more) design alternatives to prevent this failure mode from occurring • Install weep holes in abutment walls to relieve hydrostatic pressure
Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc) • Obtain As-built drawings for bridge abutments and wingwalls • Obtain additional information on the nature of the embankment and structural fill materials

Figure 19: Long Wharf Drive Bridge Overpass, I-95 embankment

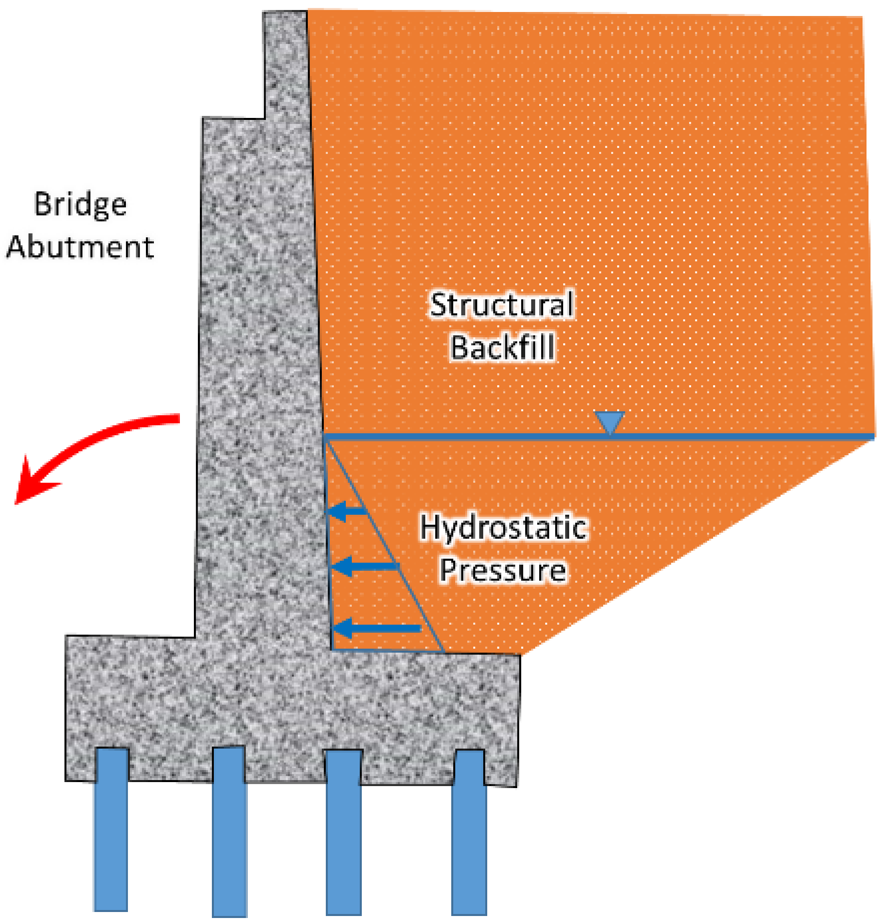

Figure 20: Potential Failure mode sketch

Figure 21: Canal Dock Road Bridge under I-95

NEW HAVEN SHORELINE POTENTIAL FAILURE MODE ANALYSIS JUDGED TO BE SIGNIFICANT OR CREDIBLE	
PFM # 4: Water loading on MSE[1] bridge wingwalls leads to failure of walls and embankment	
<u>Loading and Duration:</u> 1/100 ACE, SWL El. +13 ft NAVD88 (includes storm surge and 50-yr sea level rise). Maximum loading occurs at the peak of the storm (tidal)	
<u>Detailed Description of Failure Mode</u>: An approaching storm results in storm surge (wind setup) which increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. +8 ft NAVD88 (the base of the bridge abutments at Long Wharf Drive and Canal Dock Road). Seepage initiates through the bridge abutment wing wall fascia and into the structural fill behind either of the bridge abutment walls. Over repeated tidal cycles during the storm the SWL approaches a maximum of El. +13 ft NAVD88. Hydrostatic pressure builds up within the MSE backfill over repeated tide cycles, resulting in instability and failure of the MSE wall. Failure of the MSE wall exposes the embankment material and structural fill behind the wall to wave attack. The embankment then fails due to wave erosion, causing flooding of the interior.	
Conditions making PFM Likely Or Unfavorable Factors	**Conditions making PFM Unlikely Or Favorable Factors**
• Likely not designed for excess hydrostatic pressures • Embankment soil and structural fill behind the walls are pervious granular materials that would saturate more rapidly • MSE wall fascia are modular and joints between fascia would allow seepage to enter • MSE wingwall has never experienced the level of loading of concern	• Storm duration would only be a few days. • Tidal fluctuations would make the period of maximum loading very short, making full saturation of the fill behind the wall less likely • Wing walls are more pervious and would allow excess pressures to dissipate when tides recede. • Limited information about embankment fill suggest that it is a silty sand that would be less pervious • Bridges were rebuilt in the last five years and assumed to utilize the latest design and construction techniques • Maximum loading would only extend a third of the way up the MSE wall (at high tide) • MSE wall has multiple geogrid layers built in to provide resiliency
<u>Other Considerations and Concerns:</u> • Available preliminary design calculations show that backfill behind wall was to have a <u>maximum</u> unit weight of 65 lb/cf • Assumed that wing walls are MSE[1] walls with fascia on the outer surface that would allow seepage through the joints between the units • Note that this PFM would contribute to PFM 1 or PFM 5.	
<u>Knowledge Gaps and Data Uncertainties:</u> • We do not have as-builts of existing bridge abutments and wingwalls • Only limited subsurface information and minimal data on fills used behind bridge	
One (or more) design alternatives to prevent this failure mode from occurring • Install weep holes in MSE bridge wingwalls to relieve hydrostatic pressure	
Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc) • Obtain As-built drawings for bridge abutments and wingwalls • Obtain additional information on the nature of the embankment and structural fill materials	

1. A Mechanically Stabilized Earth (MSE) retaining wall is a composite structure consisting of alternating layers of compacted backfill and soil reinforcement elements, fixed to a wall facing. The stability of the wall system is derived from the interaction between the backfill and soil reinforcements, involving friction and

tension. The wall facing is relatively thin, with the primary function of preventing erosion of the structural backfill. The result is a coherent gravity structure that is flexible and can carry a variety of heavy loads.

Figure 22: Long Wharf Drive Bridge under I-95 embankment

Figure 23: Canal Dock Road Bridge under I-95

NEW HAVEN SHORELINE POTENTIAL FAILURE MODE ANALYSIS JUDGED TO BE SIGNIFICANT OR CREDIBLE	
PFM # 5: Wave attack on the embankment during the 100 yr loading results in erosion and breach of embankment	
<u>Loading and Duration:</u> 1/100 ACE, SWL El. +13 ft NAVD88 (includes storm surge and 50-yr sea level rise). Maximum wave heights up to 4 ft, mostly in 2-3 ft range.	
<u>Detailed Description of Failure Mode</u>: An approaching storm results in storm surge (wind setup) which increases the height of the still water level (SWL) during normal tidal cycles. As the SWL rises above El. +8 ft NAVD88 (the toe of the I-95 embankment) and water from breaking waves begins attacking the I-95 embankment. Wave energy impacting the slope is sufficient to erode the embankment surface, which has no riprap protection. The granular materials of the embankment are vulnerable to erosion. The seaward slope is undermined at the water level, resulting in progressive sloughing of the embankment. As the SWL rises further to El. +13 ft NAVD88, wave attack accelerates and sloughing continues to the landward side of the embankment. The embankment is breached, resulting in flooding of the interior.	
Conditions making PFM Likely Or Unfavorable Factors	**Conditions making PFM Unlikely Or Favorable Factors**
• Water side of embankment has no slope protection and is vulnerable to wave erosion • Wave heights have shorter period (6 sec) which would result higher frequency of wave attack • Embankment materials are assumed to be silty sands and would be highly vulnerable to erosion due to wave attack • Scour at toe of embankment at lower flood depths could destabilize the embankment slope • Embankment not designed for wave loading	• Waves would already be broken when impacting the embankment, resulting in less energy. • Embankment is over 200 ft wide, resulting in a much longer time of wave attack that would be needed to breach the embankment. • Cyclical loading during a storm event could allow for intervention at low tide • The relatively short duration of wave loading would damage the embankment slope, but would not be sufficient to breach the embankment. • Most waves would be depth limited and would have broken before reaching the road embankment. • Embankment slope fairly shallow (1V:2H)
<u>Other Considerations and Concerns:</u> • Initial erosion damage could be mitigated during the storm by dumping stone in areas that are experiencing wave erosion	
<u>Knowledge Gaps and Data Uncertainties:</u> • Embankment fill materials unknown • Limited wave modeling of area available	
One (or more) design alternatives to prevent this failure mode from occurring • Embankment slope protection • Wave attenuation structures	
Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc) • Perform site specific wave analysis • Obtain additional information regarding embankment material properties	

Figure 24: Typical condition of I-95 embankment

NEW HAVEN SHORELINE
POTENTIAL FAILURE MODE ANALYSIS
JUDGED TO BE SIGNIFICANT OR CREDIBLE

PFM # 6: Wave runup and overtopping of the embankment during the 100 yr loading results in erosion and breach of embankment

Loading and Duration: 1/100 ACE, SWL El. +13 ft NAVD88 (includes storm surge and 50-yr predicted sea level rise). Maximum runup elevations range +17 ft NAVD88 (rough armored slope) to +21 ft NAVD88 (smooth slope)

Detailed Description of Failure Mode: An approaching storm results in storm surge which increases the height of the still water level (SWL) during normal tidal cycles. As the SWL rises above El. +8 ft NAVD88 (the toe of the I-95 embankment) and water from breaking waves begins washing up on the I-95 embankment. As the SWL increases, runup exceeds the embankment crest, and embankment overwash occurs. As the SWL continues to increase, wave overwash increases. Erosion of the embankment at the crest initiates. As overwash continues, downcutting of the embankment accelerates and continues to the landward side of the embankment. The embankment is breached, resulting in flooding of the interior.

Conditions making PFM Likely Or Unfavorable Factors	Conditions making PFM Unlikely Or Favorable Factors
• Calculated runup exceeds the embankment height by up to 6 feet above the design level of protection (+15 ft NAVD88) • Embankment materials are assumed to be sands with silt and would be highly vulnerable to erosion due to wave attack • Embankment slope is relatively smooth, which would increase height of runup	• The mean level of expected runup is only 2 ft above the design level of protection, and would only occur at the peak of the storm • Most of embankment is paved with asphalt, which would slow the rate of erosion • Embankment is over 200 ft wide, resulting in a much longer time of wave overwash that would be needed to breach the embankment. • Cyclical loading during a storm event could allow for intervention at low tide (such as placing jersey barriers) • The relatively short duration of wave overwash would damage the embankment crest, but would not be sufficient to breach the embankment. • Most waves would be depth limited and would have broken before reaching the road embankment. • Embankment slope fairly shallow (1V:2H)

Other Considerations and Concerns:

- Initial erosion damage could be mitigated during the storm by dumping stone in areas that are experiencing wave erosion

Knowledge Gaps and Data Uncertainties:

- Embankment fill materials unknown
- Limited wave modeling of area available

One (or more) design alternatives to prevent this failure mode from occurring

- Embankment slope protection
- Wave attenuation structures

Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc)

- Perform site specific wave analysis
- Obtain additional information regarding embankment material properties

Figure 25: Typical condition of I-95 embankment

NEW HAVEN SHORELINE
POTENTIAL FAILURE MODE ANALYSIS
JUDGED TO BE SIGNIFICANT OR CREDIBLE

PFM # 7: Concentrated Leak Erosion along penetration (storm sewer pipe, conduit, other pipe) through I-95 embankment leads to failure of embankment

Loading and Duration: 1/100 ACE, maximum SWL El. +13 ft NAVD88 (includes storm surge and sea level rise over 50 years) at high tide

Detailed Description of Failure Mode: Poor compaction around a conduit through the I-95 embankment results in loose or poorly compacted zones around the perimeter and along the length of the conduit. An approaching storm results in storm surge which increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. +8 ft NAVD88 (the base of the I-95 embankment). Concentrated seepage begins to flow through the loose zones along the pipe. Sufficient gradients and velocities exist to move particles, resulting in erosion of soil along the pipe. Flow exits unfiltered on the landside of the I-95 embankment. The embankment materials are able to support a roof, or the pipe itself acts as a roof. Crack filling/self-healing does not occur, and a flow limiter is not present. The rate of erosion increases as the pipe continues to enlarge. The embankment breaches, resulting in flooding of the interior.

Conditions making PFM Likely Or Unfavorable Factors	Conditions making PFM Unlikely Or Favorable Factors
• Many utilities cross underneath the embankment. Given the intense development in the area it is likely that there are unknown penetrations that pass through the embankment. • Difficult to compact backfill around utilities leading to possible loose zones	• Embankment is very wide (~200 ft) • Likelihood of conduits extending through the embankment near the base is unlikely. • **Maximum** gradients at future predicted high tide (with sea level rise) are of very short duration and very low (0.025) and would not be sufficient to initiate concentrated leak erosion • Tidal fluctuations would make the period of maximum loading very short, making full saturation of the fill behind the wall very unlikely • Limited information about embankment fill suggest that it is a silty sand that might not have sufficient fines to hold a crack • Embankment would not be loaded at low tide.

Other Considerations and Concerns:

- PFM considers penetrations through the I-95 embankment that extend from seaward to landward side of embankment (currently unknown by team).
- Penetrations would have to be consistently at or within 5 ft above the base of the embankment for loading of concern to take place
- Storm sewer outfalls run underneath embankment, but are at a depth of at least 5 ft below the embankment and do not daylight on the land side (necessary for this PFM to progress)
- Maximum gradient calculated assuming $\Delta h=13-8=5$ ft, $\Delta l=200$ ft $i=5/200=0.025$.

Knowledge Gaps and Data Uncertainties:

- We do not have as-builts of existing embankment and penetrations
- Unknown if there are penetrations that extend through the base of the embankment
- Embankment material properties uncertain

One (or more) design alternatives to prevent this failure mode from occurring

- Uncertain without knowing the existence and nature of penetrations through embankment

Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc)

- Obtain As-built drawings and specifications for all utilities/pipes/penetrations crossing the embankment
- Obtain additional information on the nature of the embankment and structural fill materials behind the bridge structures

Figure 26: Storm sewer outfall near Long Wharf Drive Bridge under I-95 embankment (other outfalls similar)

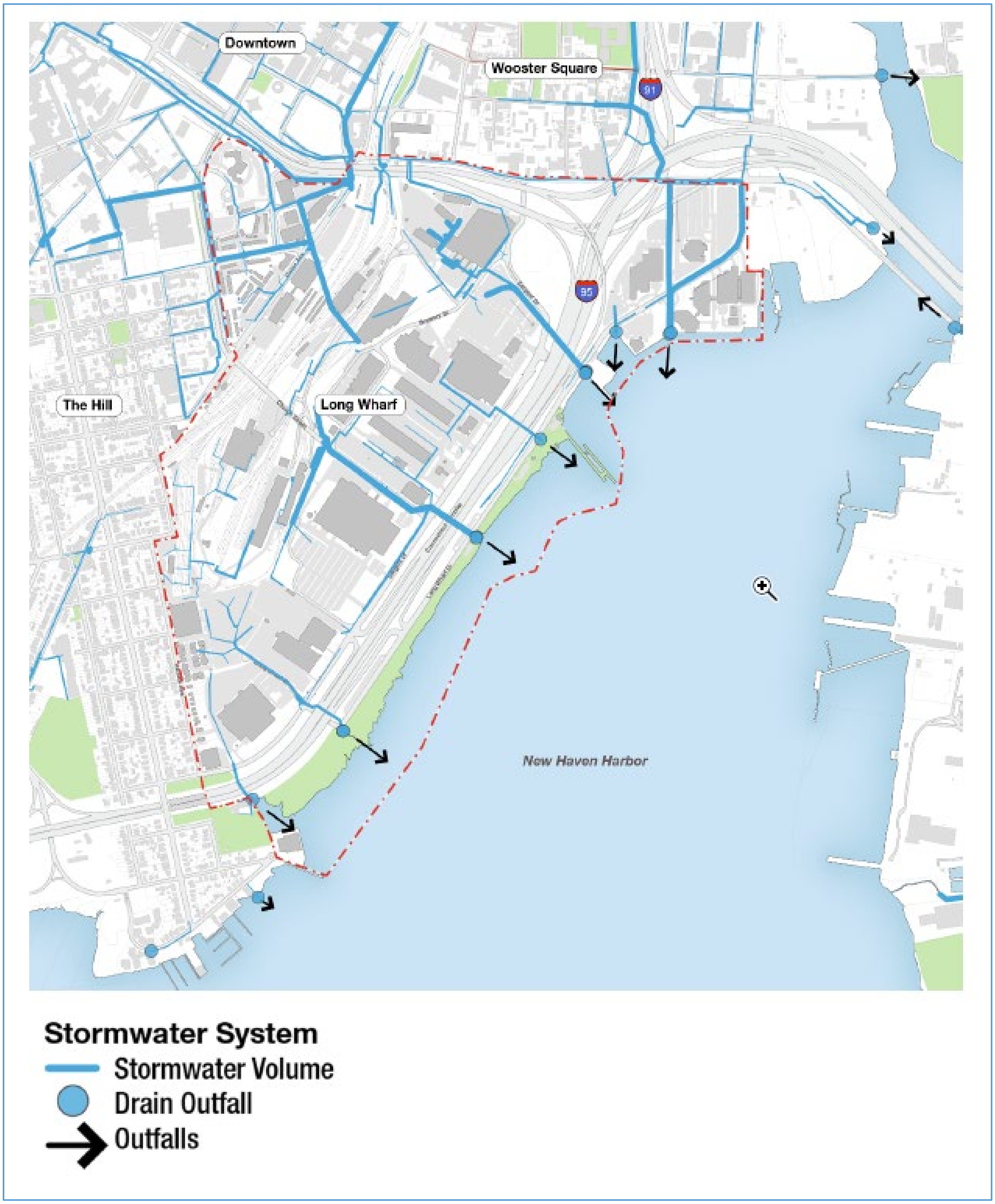

Figure 27: Location of stormwater outfalls

NEW HAVEN SHORELINE
POTENTIAL FAILURE MODE ANALYSIS
JUDGED TO BE SIGNIFICANT OR CREDIBLE

PFM # 8: Concentrated leak erosion along the highway embankment/bridge abutment contact leads to embankment failure and breach

Loading and Duration: 1/100 ACE, maximum SWL El. +13 ft NAVD88 (includes storm surge and 50-yr sea level rise) at high tide

Detailed Description of Failure Mode: Poor compaction of highway embankment along bridge abutment contact results in a crack or defect along the interface. An approaching storm results in storm surge (wind setup) which increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. +8 ft NAVD88 (the base of the bridge abutments at Long Wharf Drive and Canal Dock Road). Concentrated seepage begins to flow through the crack, eroding soil along the crack. There is no landward side filter to hinder the movement of particles. Over repeated tidal cycles during the storm the SWL approaches a maximum of El. +13 ft NAVD88, the rate of erosion increases, and the crack continues to grow. Increasing flow through the crack leads to gross enlargement of the crack and downcutting of the embankment. The embankment breaches, resulting in flooding of the interior.

Conditions making PFM Likely Or Unfavorable Factors	Conditions making PFM Unlikely Or Favorable Factors
• Abutments are vertical walls that would be more difficult to compact against. • Structural fill behind the walls are likely pervious granular materials that would saturate more rapidly • Relatively short seepage path from embankment/wingwall face to abutment walls • Limited information about embankment fill suggest that it is a silty sand that would possibly hold a crack • Construction techniques used to build the embankment unknown	• Storm duration would only be a few days; tidal fluctuations would result in a very short period of loading, making full saturation of the fill behind the wall unlikely • **Maximum** gradients at high tide are of very short duration and very low (0.025[1]) and would not be sufficient to initiate concentrated leak erosion • Backfill behind walls is assumed to be a free draining structural fill that would not hold a crack • Wingwalls at landward side of bridge abutment would prevent exit of eroded embankment material (crack would not be able to enlarge) • Bridges were rebuilt in the last five years and assumed to utilize the latest design and construction techniques, therefore poor compaction or defects are unlikely. • Embankment would not be loaded at low tide, allowing for reduction of seepage pressures.

Other Considerations and Concerns:
- Gradient calculated assuming $\Delta h=13-8=5$ ft, $\Delta l=200$ ft $i=5/200=0.025$.
- Backfill behind abutment wall is unknown but likely free draining granular material that will not hold a crack.

Knowledge Gaps and Data Uncertainties:
- We do not have as-builts of existing bridge abutments and wingwalls
- Only limited subsurface information and minimal data on fills used behind bridge
- Embankment material properties uncertain

One (or more) design alternatives to prevent this failure mode from occurring
- Uncertain without knowing the nature of materials behind walls.

Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc)

- Obtain As-built drawings for bridge abutments and wingwalls
- Obtain additional information on the nature of the embankment and structural fill materials behind the bridge structures

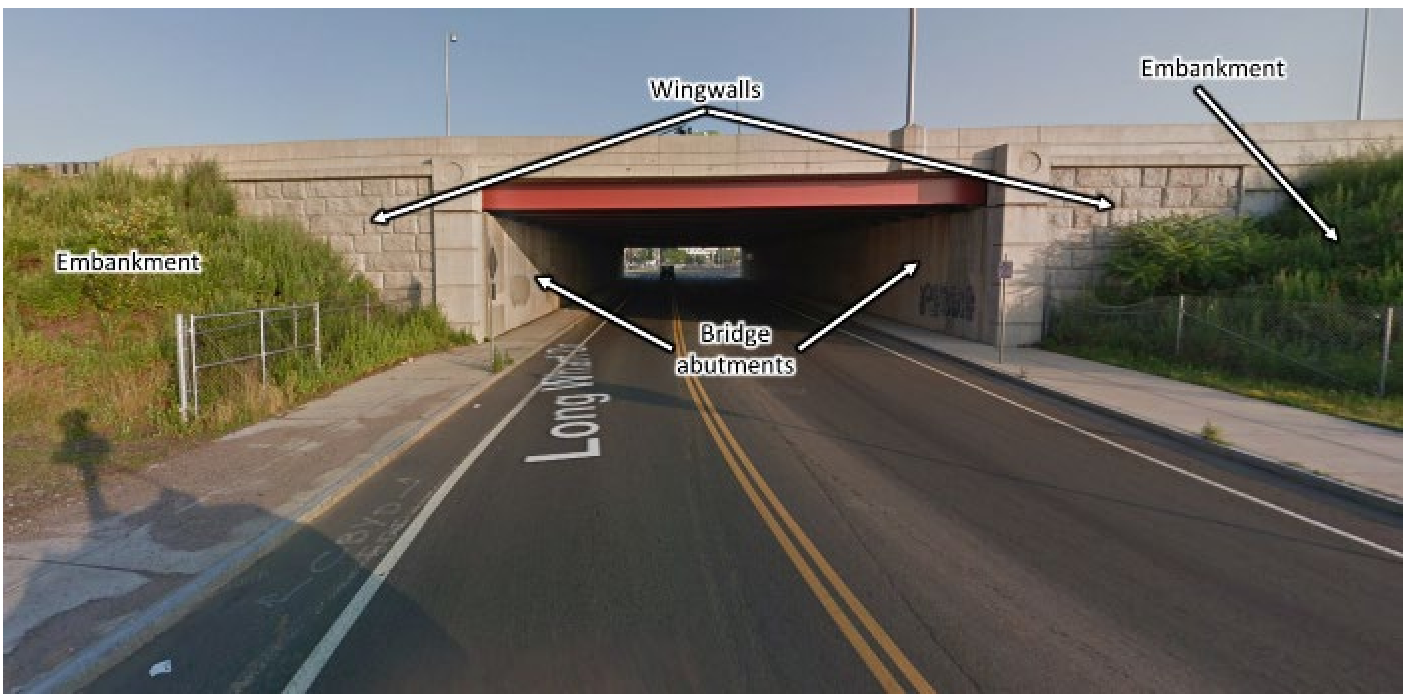

Figure 28: Long Wharf Drive Bridge Under I-95 embankment

Figure 29: Canal Dock Road Bridge under I-95

NEW HAVEN SHORELINE
POTENTIAL FAILURE MODE ANALYSIS
JUDGED TO BE SIGNIFICANT OR CREDIBLE

PFM # 9: Backward Erosion Piping through highway embankment leads to embankment failure and breach

Loading and Duration: 1/100 ACE, maximum SWL El. +13 ft NAVD88 (includes storm surge and 50-yr sea level rise) at high tide

Detailed Description of Failure Mode: I-95 highway embankment materials are continuously susceptible to backward erosion piping (BEP) from seaward to landward through the embankment. An approaching storm results in storm surge which increases the height of the still water level (SWL) during normal tidal cycles. The SWL rises above El. +8 ft NAVD88 (the base of the I-95 embankment), initiating seepage through the embankment. As the SWL continues to rise, horizontal gradients become high enough to initiate BEP of the embankment materials at the landside toe of the embankment. There is an unfiltered exit on the landside slope. A pipe develops and progresses from the landward to the seaward slope. The embankment materials contain enough fines material to hold a roof. There is no flow limiter on the landside slope. The pipe enlarges as flow increases. Gross enlargement of the pipe results in collapse of the embankment. The embankment breaches, resulting in flooding of the interior.

Conditions making PFM Likely Or Unfavorable Factors	Conditions making PFM Unlikely Or Favorable Factors
• Limited subsurface information indicates the embankment may be composed of pervious uniform fine grained sand susceptible to BEP • Possible layers of silt within the embankment could hold a roof. • Construction techniques used to build the embankment unknown	• Storm duration would only be a few days; tidal fluctuations would result in a very short period of loading, making full saturation of the fill through the embankment unlikely • Maximum gradients at high tide are of very short duration and very low (0.025[1]) and would not be sufficient to initiate backward erosion piping. • Based on limited information on the embankment fill, the material is fine sand with 20-35% fines, which would make the material much less susceptible to BEP. • Embankment would not be loaded at low tide, allowing for reduction of seepage pressures.

Other Considerations and Concerns:

- Gradient calculated assuming Δh=13-8=5 ft, Δl=200 ft i=5/200=0.025.

Knowledge Gaps and Data Uncertainties:

- Very limited subsurface information and minimal data on nature and variability of embankment materials

One (or more) design alternatives to prevent this failure mode from occurring

- Uncertain without knowing the nature of embankment materials but gradients are very low.

Recommendations for PED (Subsurface Investigations, Design Studies, Risk Analysis, etc)

- Obtain additional information on the nature of the embankment fill materials

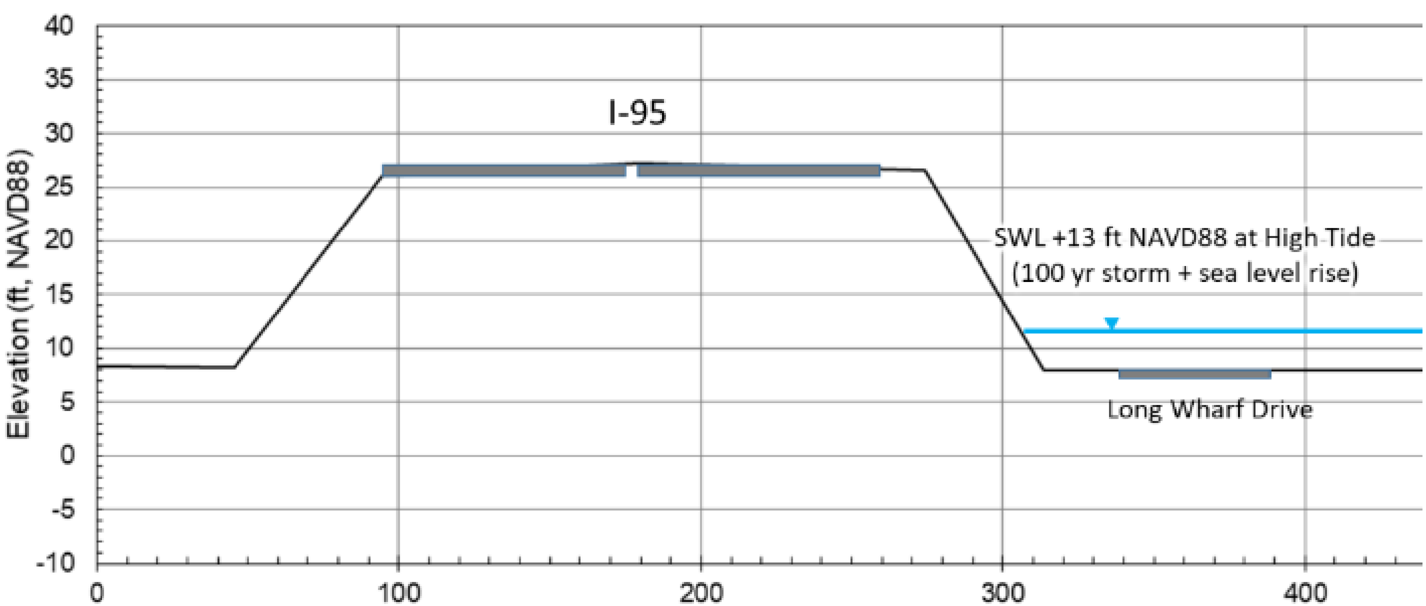

Figure 30: Typical condition of I-95 embankment near southern limit of study (top) and cross sectional profile at this location (bottom)

Figure 31: Location of boring PB-5 through embankment (others shown are at toe of embankment)

T. Paquette BORING FOREMAN	FORM NO. SM-1 ED. 1/71 STATE OF CONNECTICUT DEPARTMENT OF TRANSPORTATION BUREAU OF HIGHWAYS BORING REPORT	SHEET 1 OF 4
J. Freitas/J. O'Brien INSPECTOR	TOWN New Haven, Connecticut	LOCATION Long Wharf Drive
R. Borjeson SOILS ENGINEER	PROJECT NAME I-95 New Haven Harbor Program Management	Guild Drilling Co. BORING CONTRACTOR
	PROJECT NO. 92-505	Parsons Brinckerhoff Quade & Douglas, Inc. CONTRACTING ENGINEER

LOCATION Long Wharf Drive adjacent to the Long Wharf Nature Preserve

			AUGER	CASING	SAMPLER	CORE BAR		
SURFACE ELEV.	18.8						HOLE NO.	PB-5
DATE FINISHED	3/29/00	TYPE		HW	SS	N/A	LINE & STATION	
GROUND WATER OBSERVATIONS		SIZE I.D.		4"	1 3/8"		OFFSET	
AT 8.8 FT.	48 HRS.	HAMMER WT.		300#	140#	BIT	N. COORDINATE	165,475.0
AT FT.	HRS.	HAMMER FALL		24"	30"		E. COORDINATE	551,963.2

DEPTH	CASING BLOWS PER FOOT	SAMPLE DEPTHS FROM - TO	NO.	PEN. INCH	REC. INCH	TYPE	BLOWS PER 6 INCHES ON SAMPLER 0-6	6-12	12-18	18-24	STRATA CHANGE: DEPTH, ELEV.	FIELD IDENTIFICATION OF SOIL, REMARKS (INCL. COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC.)
		0.0' - 2.0'	1	24	18	D	1	3	5	6		Red-brown f SAND, trace c gravel, some silt, dry. (FILL)
5		4.0' - 6.0'	2	24	17	D	8	7	15	15		Red-brown f SAND, some silt. (FILL)
10		9.0' - 11.0'	3	24	19	D	5	12	12	11		Red-brown f SAND, some silt. (FILL)
15		14.0' - 16.0'	4	24	13	D	10	12	14	15		Red-brown m-c SAND, little f gravel, trace shells. (FILL)
											19.0	Top 3": Gray-brown f-c SAND, trace f-m gravel, little silt. (FILL)
20		19.0' - 21.0'	5	24	9	D	9	7	4	4	-0.4	Dark green-gray ORGANIC CLAY, some silt, trace peat fibers.

Figure 32: Log for boring PB-5