U.S. Army Corps of Engineers
Baltimore District

Baltimore Metropolitan
Coastal Storm Risk Management Feasibility Study

APPENDIX A

Civil Engineering

FINAL REPORT

February 2024

1 Content

Engineering Appendix- Civil Engineering

Baltimore Coastal Storm Risk Management

Feasibility Study

1. Introduction

1.1. Purpose and Scope of the Appendix

The purpose of this appendix is to present the Civil Engineering investigations/studies conducted for the Baltimore Coastal Storm Risk Management Feasibility Study. This Appendix investigated and evaluated a holistic way of reducing risk to the study area from inundations associated with storm frequencies ranging from the 25-year (4% Annual Exceedance Probability [AEP]) to the 100-year (1% AEP). Many flood risk management structures were assessed, evaluated, and ranked as partially and marginally feasible through the project matrix elimination process. The two-flood risk management structures types selected were floodwall, and road elevation.

This civil engineering design investigation resulted in the preliminary design of these two structures at strategic locations as a product of Hydrologic and Hydraulic (H&H) studies given water surface elevations at multiple control areas critical to the flood risk reduction of the study area. The designs were sufficient to generate baseline quantities and cost estimates to determine the cost of all the structural alternatives within the project for the feasibility study.

2. Existing Conditions

2.1. Study Area

The study area covered by this Appendix includes Locust Point, Inner Harbor, North Patapsco, South Patapsco, Middle Branch and Martin State Airport.

2.2. Site Description

The site area consists of a mix of residential, commercial and transportation infrastructure.

- The Inner Harbor alignment consists of the waterfront of Baltimore Museum of Industry to Canton Waterfront Park. This alignment is approximately 6.3 miles of floodwall. An I-wall was considered for this alternative due to the limited space available on the harbor. See **Figure 1**, Study Area.

- Locust Point alignment consists of Fort McHenry I-95 tunnel and the tunnel ventilation building, US Naval Reserve Building and Domino Sugar Waterfront to the Baltimore Museum of Industry. A T-wall was considered for this alternative. This alignment is approximately 2.3 miles of floodwall. See **Figure 2**, Study Area.
- North Patapsco alignment consists of the Seagirt Marine Terminal Port of Baltimore. A T-wall was considered for this alternative. This alignment is approximately 2.7 miles of floodwall. See **Figure 3**, Study Area.
- South Patapsco alignment consists of the 895 Tunnel and West Ventilation Building. A T-wall was considered for this alternative. This alignment is approximately 0.6 miles of floodwall. See **Figure 4**, Study Area.
- Middle Branch alignment consists in of the Wheelabrator Baltimore Building and is approximately 0.5 miles of floodwall. A T-wall was considered for this alternative. See **Figure 5**, Study Area.
- Martin State Airport consists of the Wilson Point Road and Lynbrook Road. This alignment is approximately 0.75 miles of elevated road. Road elevation was considered for this alternative. See **Figure 6**, Study Area.

US Army Corps of Engineers	Inner Harbor, Baltimore City	Study Area

Figure 1. Study Area - Inner Harbor

Figure 2. Study Area - Locust Point (Fort McHenry, 95 tunnel, West Ventilation Building and Tide Point)

Figure 3. Study Area – Port of Baltimore-Seagirt

Figure 4. Study Area- 895 Tunnel and Ventilation Building

US Army Corps of Engineers	Middle Branch, Baltimore City	Study Area

Figure 5. Study Area- Middle Branch

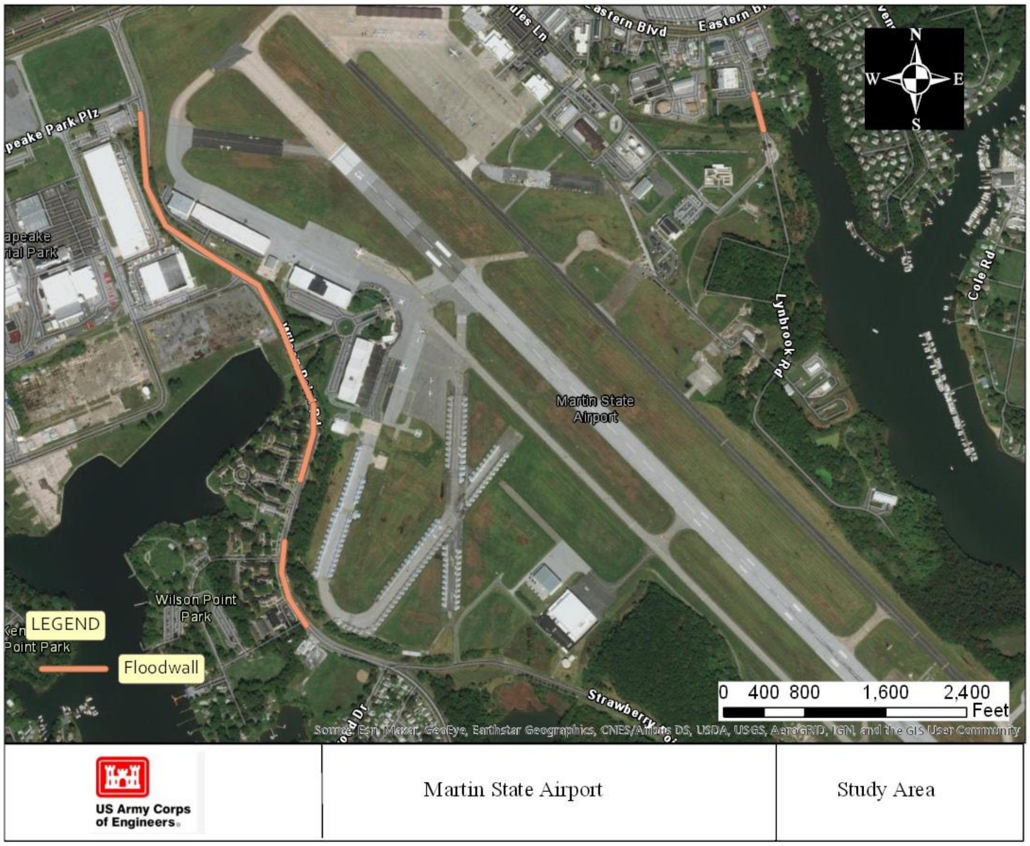


Figure 6. Study Area- Martin State Airport

3. Applicable Design Standards and Criteria

3.1. General

Improvements to site protection from floodwaters are required to follow federal, state, and local standards. Emphasis is on the use of USACE engineering circulars and manuals. For road works standard and specifications from municipal and county should be followed.

3.2. Design Criteria

The floodwalls for all the alternatives were designed to an intermediate sea level rise of a 1% AEP storm. For designing structural alternatives of the project, we used 12.5 feet NAVD88 as the level of performance. The level of performance is 12.5 feet NAVD88, based on the NACCS 100-year WSEL with approximately 95% confidence level and intermediate SLC curve through year 2080. The length of the alignment was estimated utilizing data from the LIDAR survey provided by Planning Division. The Baltimore Metropolitan Area LIDAR (Baltimore City) was provided by the sponsor: Maryland DNR in 2017. The floodwall limits were based on tying into high ground at elevation 12.5 feet and NAVD88 datum. The limit of disturbance used for the construction of the floodwall was 15 feet to each side. Additional temporary easement will be obtained if necessary to ensure constructability of the project. The exact easement for this project will be obtained during the design phase. Martin State Airport proposes to elevate existing roads to serve as flood protection and as an emergency exit for the people living around the area and personnel working on the airport. With the airport being a critical infrastructure, a level of performance of a 1000-year level (0.1% AEP) was evaluated but due to project site constraints it was decided it was not feasible.

In order to prevent erosion on the landside of the floodwall a concrete backsplash has been considered for the critical areas. For further design analysis more detail information is required. To included, but not limited to, detail survey and a detail geotechnical site investigation.

After TSP, it was decided, following the BCR's number that only the I-95 tunnel, I-895 tunnel and the ventilation buildings will proceed for further structural evaluation. After ADM (Agency Decision Milestone) the proposed floodwalls around the tunnels and ventilation building became the recommend plan.

After the Agency Technical Report, the alignments for the Fort McHenry Tunnel (I-95) and Baltimore Harbor Tunnel (I-895) have changed to reduce the span of the closing structures. The closure structures have changed from stop logs to roller gates in order to be more readily deployable during an emergency.

3.3. Civil

AutoCAD Civil 3D and ArcPro GIS were used to create the alignments, cross sections and layouts for the floodwalls and road elevations. Typical cross sections of floodwalls were developed utilizing design guidance from EM 1110-2-2502, Retaining and Floodwalls, Chapter 5- Design of Floodwalls and Levees, FEMA (44 CFR60.3(c)(2)). See **Figure 7,** Typical Cross Sections**.**

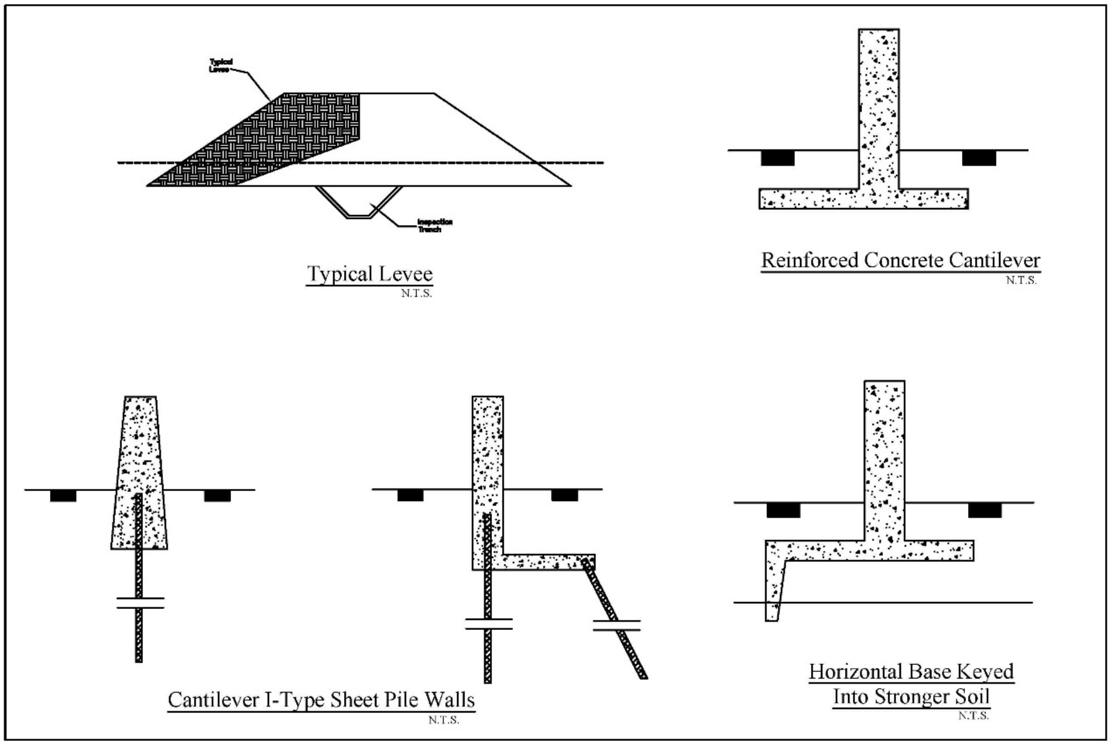


Figure 7, Typical Cross Sections

4. Structural Analysis

4.1. Floodwall Description

The floodwalls considered for the protection of the I-95 and I-895 tunnels and ventilation buildings are cast-in-place reinforced concrete T-walls. Two different types of floodwalls were selected and referenced as Type 1 and Type 2 based on site conditions and floodwall stem height above ground surface. Type 1 floodwall height ranges from 5.5 ft to 6.5 ft while Type 2 varies between 2.5 ft and 3.5 ft. Type 1 floodwalls will be constructed around tunnel entrances and Type 2 will be constructed to protect the tunnel ventilation buildings. Refer to **Table 2** for preliminary design results.

4.2. Analysis and Design of Floodwalls

The concrete T-walls are analyzed for global stability and structural strength based on the requirements established on EM 1110-2-2100 “Stability Analysis of Concrete Structures”, EM 1110-2-2502 “Floodwalls and Other Hydraulic Retaining Walls”, and EM 1110-2-2104 “Strength Design for Reinforced Concrete Hydraulic Structures”.

Five different loading conditions are evaluated during the analysis in accordance with Table B-5 of EM 1110-2-2100, see **Table 1**. An additional loading condition, Design Resiliency Check (DRC), is also evaluated and includes water at the top of the wall. This case was adapted from the USACE New Orleans District Design Guidelines and applies to structures whose primary function is hurricane flood protection. The case was developed to verify the survivability of a structure during major storm events. As shown on **Table 2** and considering the floodwalls as critical structures, EM 1110-2-2502 classifies these loading conditions into three (3) different categories: usual (<10-year recurrence interval), unusual (10 to 750-year recurrence interval), and extreme (>750-year recurrence interval).

The controlling case for the design of the floodwalls was assumed to be the Design Resiliency Check (DRC) case, Water at Top of Wall with Coincident Wave. The wave forces were applied as concentrated loads acting at the top of the floodwalls. These forces were calculated and provided by the Hydraulics and Hydrology Section. Details of these computations are included in Section 4.3 and Section 7 of this appendix.

Load Case	Loading Description	Classification
C1	Surge Stillwater + Coincident Wave	UN/E[1]
C2a	Coincident Pool + OBE	UN
C2b	Coincident Pool + MDE	E
C3	Construction	UN
C4	Normal Operating	UN
Additional Case (DRC)[2]	Water at Top of Wall + Coincident Wave	UN/E

[1] UN = Unusual, E = Extreme; [2] DRC = Design Resiliency Check

Table 1 - Coastal Floodwall Loading Condition Classification

Load Condition Categories	Annual Probability (p)	Return Period (t_r)
Usual	Greater than or equal to 0.10	Less than or equal to 10 years
Unusual (normal structures)	Less than 0.10 but greater than or equal to 0.0033	Greater than 10 years but less than or equal to 300 years
Unusual (critical structures)	Less than 0.10 but greater than or equal to 0.00133	Greater than 10 years but less than or equal to 750 years
Extreme (normal structures)	Less than 0.0033	Greater than 300 years
Extreme (critical structures)	Less than 0.00133	Greater than 750 years

Table 2 - Loading Condition Categories Based on Probability of Occurrence

A set of spreadsheets was developed in Mathcad to analyze the walls considering the DRC as the controlling case. Concrete member sizes were designed based on all vertical, gravity, and horizontal forces acting on the structures. **Figure 8** below provides a schematic of the different forces taken into consideration during the analysis.

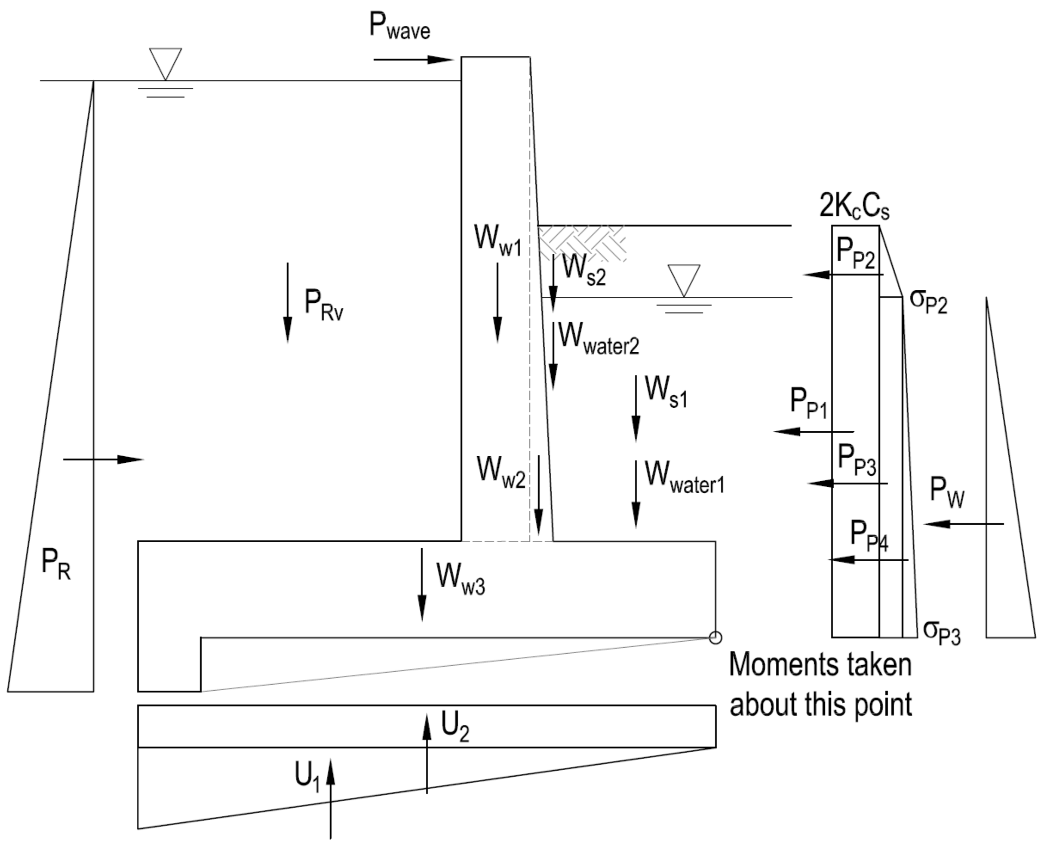


Figure 8 - Forces Acting on Floodwalls

The preliminary design results for T-wall types 1 and 2 are provided in Table 3 below.

Wall Type	Footing		Stem			Key	
	Width (ft)	Thickness (in)	Height* (ft)	Thickness at Crest (in)	Thickness at Base (in)	Depth (ft)	Thickness (in)
1	15	24	8.5	18	18	4	12
2	10	18	5.5	14	14	2	12

* From top of footing

Table 3 - T-wall Preliminary Design Results

4.3. <u>Computed Wall Forces</u>

For floodwalls, the Goda formulation for computing wave forces is used. A definition sketch is shown in Figure 9. Hydraulic inputs for these computations are the incoming wave height, wave period and the surge level. Moreover, the geometrical parameters of the structure (bottom elevation, top of wall, etc.) are inputs for this computation.

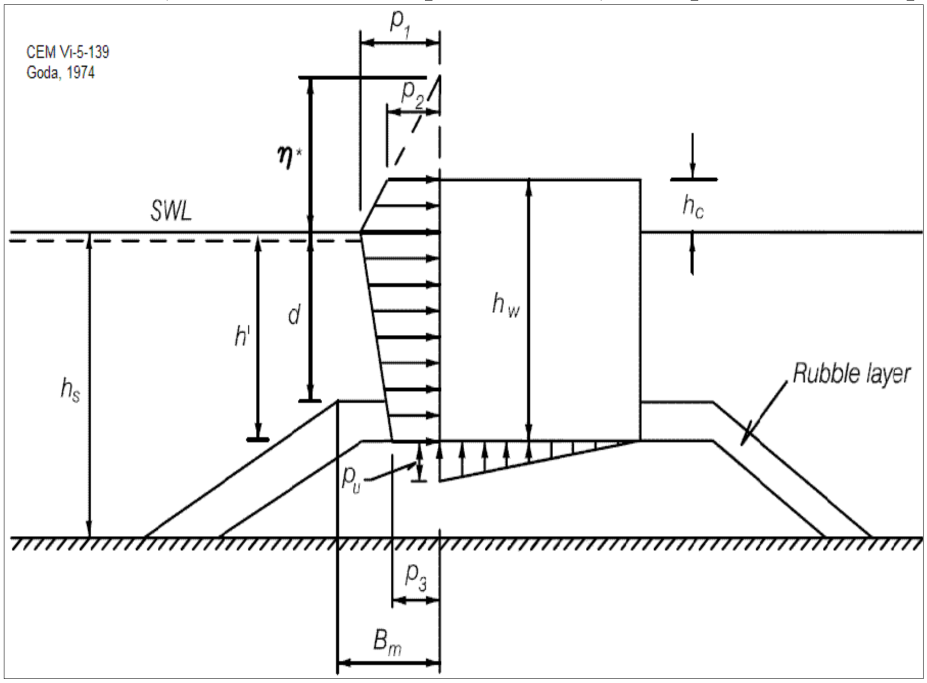


Figure 9 – Definition sketch of wave force calculations (Source: Coastal Engineering Manual, 2001)

It is assumed, the site wave would be depth-limited with mild slope or even flats. The computed wall force and moment is as shown below:

F_H =	2,446	lbf/ft
M_H =	8,205	lbf*ft/ft

Detail of Wall Forces Calculation is shown in section 6.

4.4. References

a. EM 1110-2-2100- Stability Analysis of Concrete Structures
b. EM 1110-2-2502- Floodwalls and Other Hydraulic Retaining Walls
c. EM 1110-2-2104- Strength Design for Reinforced Concrete Hydraulic Structures

US Army Corps of Engineers

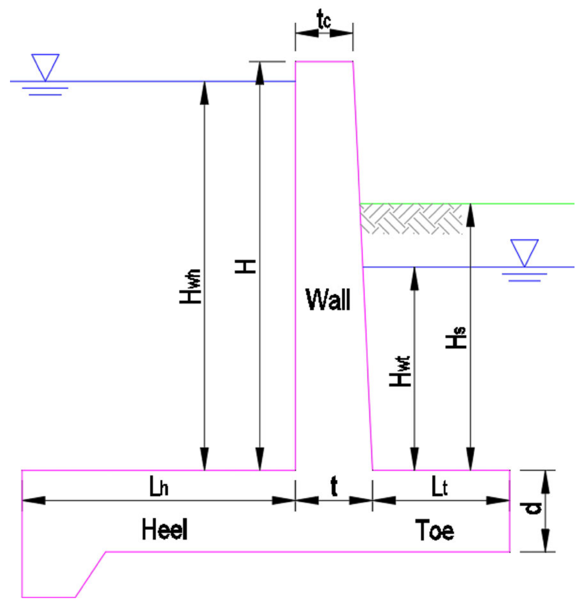
BALTIMORE COASTAL
TYPE 1 FLOODWALL (TUNNEL) - 12.5' LEVEL
STABILITY CHECK & STRENGTH DESIGN
WATER AT TOP OF WALL & WAVE - EXTREME CASE

Analysis By: Nestor Delgado, P.E.
Checked By: Joseph Cervantes

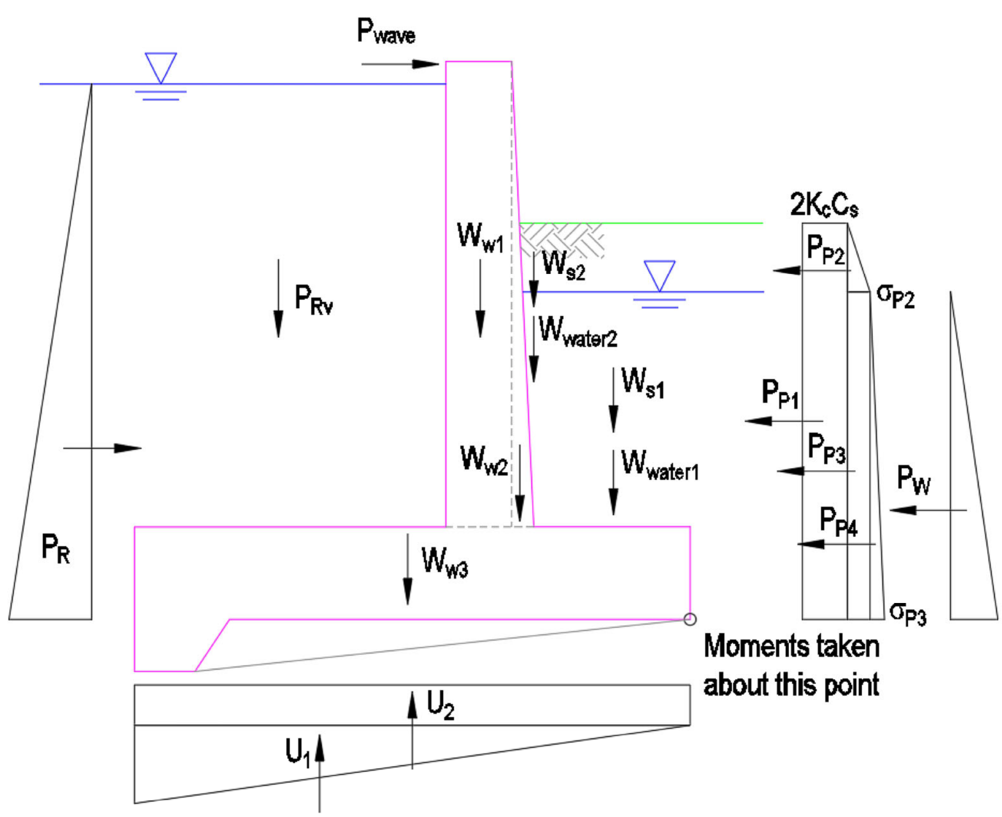
Updated: Sep 2023

REFERENCES

1. IBC 2012 with reference on ASCE7-10
2. ACI 318-14, ACI 530-13 and UFC 3-301-01 with changes on May 15, 2014
3. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 with 2016 Interim Revisions (referred to as LRFD)
4. EM 1110-2-2100: Stability Analysis of Concrete Structures
5. EM 1110-2-2502: Retaining and Flood Walls
6. EM 1110-2-2104: Strength Design for Reinforced Concrete Hydraulic Structures.
7. Engineering and Construction Bulletin (ECB) No. 2017-2: Revision and Clarification of EM 2100 and EM 2502



Notes:
-Driving side is reservoir side
-Soil on top of heel not taken into consideration due to possible erosion during high water event



US Army Corps of Engineers

BALTIMORE COASTAL
TYPE 1 FLOODWALL (TUNNEL) - 12.5' LEVEL
STABILITY CHECK & STRENGTH DESIGN
WATER AT TOP OF WALL & WAVE - EXTREME CASE

Analysis By: Nestor Delgado, P.E.
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WALL DIMENSIONS

Top of wall elevation $EL_{TW} := 12.5 \cdot ft$

Water elev. (heel) $SWEL_h := 12.5 \cdot ft$

Water elev. (toe) $SWEL_t := 6 \cdot ft$

Footing thickness $d := 24 \cdot in$

Heel length $L_h := 7.5 \cdot ft$

Toe length $L_t := 6 \cdot ft$

Key depth $K := 4 \cdot ft$

Key thickness $t_k := 1 \cdot ft$

Equipment Surcharge $S := 0 \cdot plf$

Compaction load $C_{load} := 0 \cdot plf$

Soil elev. (heel) $EL_{s\_h} := 6 \cdot ft$

Soil elev. (toe) $EL_{s\_t} := 6 \cdot ft$

Bottom of footing elevation $EL_{bs} := 2 \cdot ft$

Wall thickness at crest $t_c := 18 \cdot in$

Wall thickness at base $t := 18 \cdot in$

Wall height $H := EL_{TW} - EL_{bs} - d = 8.5\ ft$

Soil height $H_s := EL_{s\_t} - EL_{bs} - d = 2\ ft$

Water height (toe) $H_{wt} := SWEL_t - EL_{bs} - d = 2\ ft$

Water height (heel) $H_{wh} := SWEL_h - EL_{bs} - d = 8.5\ ft$

Base Length $L_{base} := L_t + L_h + t = 15\ ft$

Wall Batter $Bat := \dfrac{t - t_c}{H} = 0$

PROPERTIES AND COEFFICIENTS

Soil moist unit weight $\gamma_m := 110 \cdot psf$

Soil saturated unit weight $\gamma_s := 120 \cdot psf$

Concrete unit weight $\gamma_c := 150 \cdot psf$ (Stability) $\gamma_{cb} := 150 \cdot psf$ (Bearing stress)

Water unit weight $\gamma_w := 62.4 \cdot psf$

Shear strength parameter (Backfill) $\phi_1 := 30 \cdot deg$

Sliding parameter $\phi_s := 30 \cdot deg$

Cohesion (Backfill) $c := 0 \cdot psf$

Cohesion (Footing Soil) $c_s := 0 \cdot psf$

Bearing Capacity $Q_{allowable} := 2000 \cdot plf$ (See bearing capacity calculations)

Horizontal Seismic Coefficient $k_h := 0$

Slope inclination Angle $\beta_a := 0 \cdot deg$ $\beta_p := 0 \cdot deg$ $(passive)$

At-rest coefficient $K_o := (1-\sin(\phi_1)) \cdot (1+\sin(\beta_a))$ $K_o = 0.5$

Sliding factor of safety $FS_{sliding} := 1.1$

GLOBAL STABILITY

WEIGHT OF CONCRETE:

Stem:

$W_{w1} := -\gamma_c \cdot H \cdot t_c = -1.9\ kip$ $x_{w1} := L_t + \frac{t_c}{2} = 6.8\ ft$

$W_{w2} := -\gamma_c \cdot \frac{1}{2} \cdot H \cdot (t - t_c) = 0\ kip$ $x_{w2} := L_t + \frac{2}{3} \cdot (t - t_c) = 6\ ft$

Footing:

$W_{w3} := -\gamma_c \cdot (L_{base}) \cdot d = -4.5\ kip$ $x_{w3} := \frac{L_{base}}{2} = 7.5\ ft$

SOIL FORCES:

Vertical Forces:

$L_{atw} := Bat \cdot H_{wt} = 0\ ft$

$L_{ats} := Bat \cdot H_s = 0\ ft$

$W_{s1} := -(\gamma_s - \gamma_w) \cdot H_{wt} \cdot L_t + -\gamma_m \cdot (H_s - H_{wt}) \cdot L_t = -0.69\ kip$ $x_{s1} := \frac{L_t}{2} = 3\ ft$

$W_{s2} := -(\gamma_s - \gamma_w) \cdot \frac{1}{2} \cdot H_{wt} \cdot L_{atw} - \gamma_m \cdot (H_s - H_{wt}) \cdot \left(\frac{L_{ats} + L_{atw}}{2}\right) = 0\ kip$ $x_{s2} := L_t + \frac{2}{3} \cdot (L_{ats}) = 6\ ft$

Horizontal Forces

Resisting / Passive Wedge

$$A_p := (1 + k_h \cdot \tan(\phi_s)) \cdot \tan(\phi_s) + \frac{2 \cdot c_s \cdot (1 + \tan(\phi_s) \cdot \tan(\beta_p))}{\gamma_m \cdot \left(\frac{1}{ft}\right) \cdot (H_s + d)}$$ $A_p = 0.577$

$$c_{1p} := \frac{2 \cdot \tan(\phi_s) \cdot (\tan(\phi_s) - k_h) + \frac{4 \cdot c_s \cdot (\tan(\phi_s) - \tan(\beta_p))}{\gamma_m \cdot \left(\frac{1}{ft}\right) \cdot (H_s + d)}}{A_p}$$ $c_{1p} = 1.2$

$$c_{2p} := \frac{\tan(\phi_s)\cdot(1+\tan(\phi_s)\cdot\tan(\beta_p)) + (\tan(\beta_p)-k_h) + \dfrac{2\cdot c_s\cdot(1+\tan(\phi_s)\cdot\tan(\beta_p))}{\gamma_m\cdot\left(\dfrac{1}{ft}\right)\cdot(H_s+d)}}{A_p} \qquad c_{2p}=1$$

$$\alpha_p := \operatorname{atan}\left(\frac{-c_{1p}+\sqrt{c_{1p}^{\,2}+4\cdot c_{2p}}}{2}\right) \qquad \alpha_p = 30\ deg$$

Passive pressure coefficient
for seismic (Moist condition)

$$K_{PO} := \left(\frac{1+\tan(\phi_s)\cdot\cot(\alpha_p)}{1-\tan(\phi_s)\cdot\tan(\alpha_p)}\right)\cdot\left(\frac{\tan(\alpha_p)}{\tan(\alpha_p)-\tan(\beta_p)}\right) \qquad K_{PO}=3$$

$$K_P := \textbf{if}\left(k_h > .001, K_{PO}, K_{PO}\right) \qquad K_P = 3$$

Passive pressure for seismic
(Saturated condition)

$$K_{PBO} := \left(\frac{1+\tan(\phi_s)\cdot\cot(\alpha_p)}{1-\tan(\phi_s)\cdot\tan(\alpha_p)}\right)\cdot\left(1+\left(\frac{\tan(\alpha_p)}{\tan(\alpha_p)-\tan(\beta_p)}-1\right)\cdot\left(\frac{\gamma_m}{\gamma_s-\gamma_w}\right)\right)$$

$$K_{PBO}=3$$

$$K_{PB} := \textbf{if}\left(k_h > .001, K_{PBO}, K_{PBO}\right) \qquad K_{PB}=3$$

$$K_c := \frac{1}{2\cdot\sin(\alpha_p)\cdot\cos(\alpha_p)\cdot(1-\tan(\phi_s)\cdot\cos(\alpha_p))}\cdot\frac{\tan(\alpha_p)}{\tan(\alpha_p)-\tan(\beta_p)} \qquad K_c = 2.3$$

$$P_{P1} := -(2\cdot K_c\cdot c\cdot(H_s+d))\cdot ft = 0\ kip \qquad y_{PP1} := \frac{(H_s+d)}{2} = 2\ ft$$

$$\sigma_{P2} := K_P\cdot\gamma_m\cdot(H_s-H_{wt}) = 0\ ft\cdot ksf$$

$$P_{P2} := -\left(\left(\frac{1}{2}\right)\cdot\sigma_{P2}\cdot(H_s-H_{wt})\right) = 0\ kip \qquad y_{PP2} := (H_{wt}+d)+\frac{1}{3}\cdot(H_s-H_{wt}) = 4\ ft$$

$$\sigma_{P3} := \begin{cases} K_P\cdot\gamma_m\cdot(H_s-H_{wt}) + K_{PB}\cdot(\gamma_s-\gamma_w)\cdot(H_{wt}+d) & \text{if } H_{wt} < H_s \\ K_{PB}\cdot(\gamma_s-\gamma_w)\cdot(H_{wt}+d) & \text{else} \end{cases} = 0.7\ ft\cdot ksf$$

$$P_{P3} := -\left(\begin{array}{l} \text{if } H_{wt} < H_s \\ \quad \sigma_{P2} \cdot (H_{wt} + d) \\ \text{else} \\ \quad \frac{1}{2} \cdot \sigma_{P3} \cdot (H_{wt} + d) \end{array} \right) = -1.4 \ kip$$

$$y_{PP3} := \begin{array}{l} \text{if } H_{wt} < H_s \\ \quad \left(\frac{H_{wt} + d}{2} \right) \\ \text{else} \\ \quad \left(\frac{1}{3} \cdot (H_{wt} + d) \right) \end{array} = 1.3 \ ft$$

$$P_{P4} := \begin{array}{l} \text{if } H_{wt} < H_s \\ \quad -\left(\frac{1}{2} \cdot (\sigma_{P3} - \sigma_{P2}) \cdot (H_{wt} + d) \right) \\ \text{else} \\ \quad 0 \ \mathbf{kip} \end{array} = 0 \ kip$$

$$y_{PP4} := \frac{1}{3} \cdot (H_{wt} + d) = 1.3 \ \mathbf{ft}$$

WATER FORCES

Vertical Forces:

$$W_{water1} := -\gamma_w \cdot H_{wt} \cdot L_t = -0.7 \ kip \qquad x_{water1} := \frac{L_t}{2} = 3 \ \mathbf{ft}$$

$$W_{water2} := -\gamma_w \cdot \frac{1}{2} \cdot H_{wt} \cdot L_{atw} = 0 \ \mathbf{lbf} \qquad x_{water2} := L_t + \frac{1}{3} \cdot L_{atw} = 6 \ \mathbf{ft}$$

$$W_{water3} := -\gamma_w \cdot L_h \cdot H_{wh} = -4 \ kip \qquad x_{water3} := \frac{L_h}{2} + t + L_t = 11.3 \ \mathbf{ft}$$

Uplift Forces:

$$\Delta h := H_{wh} - H_{wt} = 6.5 \ \mathbf{ft}$$

$$L_s := L_h + t + L_t + d + min(H_s, H_{wt}) = 19 \ \mathbf{ft}$$

Uplift at heel $\quad U_h := \mathbf{if}(H_{wh} > 0.1 \cdot \mathbf{ft}, H_{wh} + d, 0) = 10.5 \ ft$

Uplift at toe $\quad U_t := \mathbf{if}\left(H_{wt} > 0.1 \cdot \mathbf{ft}, (H_{wh} + d) - \frac{\Delta h \cdot L_{base}}{L_s}, 0\right) = 5.4 \ ft$

$$U_1 := \gamma_w \cdot \frac{1}{2} \cdot (U_h - U_t) \cdot (L_{base}) = 2.4 \ kip \qquad x_{U1} := \frac{2}{3} \cdot (L_{base}) = 10 \ \mathbf{ft}$$

$$U_2 := \gamma_w \cdot U_t \cdot (L_{base}) = 5 \ kip \qquad x_{U2} := \frac{L_{base}}{2} = 7.5 \ \mathbf{ft}$$

Horizontal Forces:

Resisting:

$P_w := -\left(\frac{1}{2}\right) \cdot \gamma_w \cdot (H_{wt} + d)^2 = -0.5\ kip$

$y_w := \frac{H_{wt} + d}{3} = 1.3\ ft$

Driving:

$P_R := \frac{1}{2} \cdot \gamma_w \cdot (H_{wh} + d)^2 = 3.4\ kip$

$y_R := \frac{1}{3} \cdot (H_{wh} + d) = 3.5\ ft$

Wave Forces

$EL_{wave} := 9.36 \cdot ft$ (Wave force elevation)

$P_{wave} := 2.45 \cdot kip$ (Wave force)

$y_{wave} := EL_{wave} - EL_{bs} = 7.36\ ft$

RESULTANT FORCES

Moments about toe

Resisting Moment

$$M_r := W_{w1} \cdot x_{w1} + W_{w2} \cdot x_{w2} + W_{w3} \cdot x_{w3} + W_{s1} \cdot x_{s1} + W_{s2} \cdot x_{s2} + W_{water1} \cdot x_{water1} + W_{water2} \cdot x_{water2} + P_{P1} \cdot y_{PP1} + P_{P2} \cdot y_{PP2} + P_{P3} \cdot y_{PP3} + P_{P4} \cdot y_{PP4} + P_w \cdot y_w + W_{water3} \cdot x_{water3} = -98.2\ kip \cdot ft$$

Overturning Moment

$$M_o := P_R \cdot y_R + P_{wave} \cdot y_{wave} + U_1 \cdot x_{U1} + U_2 \cdot x_{U2} = 91.8\ kip \cdot ft$$

ΣFy Vertical Force

$$F_y := W_{w1} + W_{w2} + W_{w3} + W_{s1} + W_{s2} + W_{water1} + W_{water2} + W_{water3} + U_1 + U_2 = -4.4\ kip$$

ΣFx Horizontal Force

$$F_x := P_{P1} + P_{P2} + P_{P3} + P_{P4} + P_w + P_R + P_{wave} = 4\ kip$$

Location of Resultant

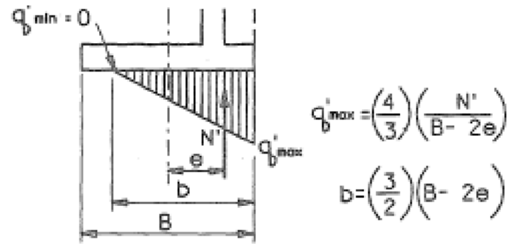
$Resultant := \frac{M_r + M_o}{F_y} = 1.5\ ft$

$Eccentricity := \frac{L_{base}}{2} - Resultant = 6\ ft$

$Middle\_Third := \frac{L_{base}}{6} = 2.5\ ft$

If the resultant fall outside the middle one-third of the base (eccentricity,e>B/6) then the maximum stress is:

$$\sigma_{max} := \frac{4}{3} \cdot \left(\frac{F_y}{L_{base} - 2 \cdot |Eccentricity|} \right) \qquad \sigma_{max} = -1999 \ \frac{lbf}{ft}$$



The base in compression is:

$$L_{bcomp} := \frac{3}{2} \cdot \left(L_{base} - 2 \cdot |Eccentricity| \right) \qquad L_{bcomp} = 4.4 \ ft$$

Percent of base in compression:

$$\%Compression := min\left(\left(\frac{L_{bcomp}}{L_{base}} \cdot 100 \right) \%, 100\% \right) = 29\%$$

$$Check_R := \textbf{if}\left(\%Compression > 0\%, \text{“OK”}, \text{“NOT\_GOOD”} \right)$$

Check compression at Base

$Check_R = \text{“OK”}$

Note: For Extreme Cases, the location of the resultant should lie within the base

Stress distribution Under the Footing (Negative is compression)

$$\sigma_{toeface} := \frac{F_y}{L_{base}} \cdot \left(1 + \frac{6 \cdot Eccentricity}{L_{base}} \right)$$ *The stress at the toe* $\sigma_{toeface} = -1002 \ \frac{lbf}{ft}$

$$\sigma_{heelface} := \frac{F_y}{L_{base}} \cdot \left(1 - \frac{6 \cdot Eccentricity}{L_{base}} \right)$$ *The stress at the heel* $\sigma_{heelface} = 415 \ \frac{lbf}{ft}$

Bearing Factor of Safety

$$Check_B := \textbf{if}\left(Q_{allowable} > |\sigma_{toeface}|, \text{“OK”}, \text{“NO\_GOOD”} \right)$$

Check bearing stress

$Check_B = \text{“OK”}$

Sliding Factor of Safety:

$\theta := \text{atan}\left(\frac{K}{L_{base}}\right)$ $\theta = 14.9\ deg$

$F_{xn} := F_x \cdot \sin(\theta)$ $F_{xn} = 1\ kip$

$F_{xs} := F_x \cdot \cos(\theta)$ $F_{xs} = 3.9\ kip$

$F_{ys} := F_y \cdot \sin(\theta)$ $F_{ys} = -1.1\ kip$

$F_{yn} := -F_y \cdot \cos(\theta)$ $F_{yn} = 4.3\ kip$

$$FS_s := \left| \frac{\tan(\phi_s) \cdot (F_{xn} + F_{yn}) + c_s \cdot L_{base} \cdot (1 \cdot ft)}{F_{xs} + F_{ys}} \right|$$ $FS_s = 1.12$

$Check_{FS} := \text{if}\left(FS_s > FS_{sliding}, \text{"OK"}, \text{"NOT\_GOOD"}\right)$

Check Sliding

$Check_{FS} = \text{"OK"}$

Reinforced Concrete Design

EM1110-2-2104, Strength Design for Reinforced Hydraulic Structures

Load Factors
(Table 3-1, pg. 3-10)

Permanent Loads

Dead Load	$LF_D := 1.6$
Vertical Earth Load	$LF_{EV} := 1.6$
Lateral Earth Load	$LF_{EH} := 1.6$
Hydrostatic Load (Companion)	$LF_{Hsc} := 1.6$

Temporary Loads

Hydrostatic Load (Principal)	$LF_{Hsp} := 1.6$
Surcharge Load	$LF_{ES} := 1.6$
Operating Equipment	$LF_Q := 1.6$

Dynamic Loads

Earthquake Load	$LF_{EQ} := 1.0$

Material & Section Properties

Concrete strength	$f_c := 4000 \cdot psi$
Steel strength	$f_y := 60000 \cdot psi$
Member unit width	$b := 12 \cdot in$
Resistance Factor, Bending	$\phi_b := 0.9$
Resistance Factor, Shear	$\phi_v := 0.75$

Distance to footing bottom steel	$d_{fb} := d - 4.5 \cdot in$	$d_{fb} = 19.5\ in$
Distance to footing top steel	$d_{ft} := d - 4.5 \cdot in$	$d_{ft} = 19.5\ in$
Distance to wall steel	$d_w := t - 4.5 \cdot in$	$d_w = 13.5\ in$
Distance to key steel	$d_k := t_k - 4.5 \cdot in$	$d_k = 7.5\ in$

Wall Stem - Recompute Lateral Forces for Structural Design of Wall Stem

Horizontal Forces

Reservoir Pressure

$$P_{Rws} := \frac{1}{2} \cdot \gamma_w \cdot H_{wh}^{\ 2} = 2.25\ kip$$

$$y_{Rws} := \frac{H_{wh}}{3} = 2.83\ ft$$

Factored Design Shear

$$V_{uWS} := LF_{Hsp} \cdot P_{Rws} = 3.61\ kip$$

Wall Stem

$$V_{cstem} := 2 \cdot \sqrt{f_c \cdot \frac{lbf}{in^2}} \cdot b \cdot d_w = 20.49\ kip$$

$$\phi V_{nstem} := \phi_v \cdot V_{cstem} = 15.37\ kip$$

$$Check_{WS} := \mathbf{if}\left(V_{uWS} < \phi V_{nstem}, \text{“OK”}, \text{“NO\_GOOD”}\right)$$

Check wall shear strength

$$Check_{WS} = \text{“OK”}$$

Summation of Moments

Max Flexural Moment

Reservoir Forces:

$$M_{ws} := P_{Rws} \cdot y_{Rws} = 6.39\ kip \cdot ft$$

Factored Design Moment

$$M_{uWS} := LF_{Hsp} \cdot M_{ws} = 10.22\ kip \cdot ft$$

$$k_u := 1 - \sqrt{1 - \frac{M_{uWS}}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_w^{\ 2}}} = 0.0185$$

$$A_{sws} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_w}{f_y} = 0.2\ in^2$$

Minimum Steel, ACI 350-06, Section10.5

$$A_{smin1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_w = 0.54\ in^2$$

$$A_{smin2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_w = 0.51\ in^2$$

$$A_{Amim} := \max\left(A_{smin1}, A_{smin2}\right) = 0.54\ in^2$$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$$\rho_b := 0.85 \cdot 0.85 \cdot \frac{f_c}{f_y} \cdot \frac{87000 \cdot \frac{lbf}{in^2}}{87000 \cdot \frac{lbf}{in^2} + f_y} = 0.0285$$

$$A_{smax} := 0.25 \cdot \rho_b \cdot b \cdot d_w = 1.2\ in^2$$

Temperature and Shrinkage Steel (USACE EM1110-2-2104, Section 2-8)

$$A_{ts} := \frac{.0028 \cdot b \cdot t}{2} = 0.3\ in^2$$

Footing

Equations assume base is in compression and d>Lh

$$\sigma_{toeshear} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h + t + d\right) = -624.2\ \frac{lbf}{ft}$$

$U_h = 10.5\ ft$ $\quad$ $U_t = 5.4\ ft$

$$U_{tshear} := U_h - \frac{U_h - U_t}{L_{base}} \cdot \left(L_h + t + d\right) = 6.7\ ft$$

$$V_{uLLtoe1} := \frac{-1}{2} \cdot \left(\sigma_{toeface} + \sigma_{toeshear}\right) \cdot \left(L_t - d\right) \qquad V_{uLLtoe1} = 3.3\ kip$$

$$V_{uLLtoe2} := \frac{1}{2} \cdot \left(U_{tshear} + U_t\right) \cdot \gamma_w \cdot \left(L_t - d\right) \qquad V_{uLLtoe2} = 1.5\ kip$$

$$V_{uLLtoe3} := -\left(\gamma_s \cdot H_{wt} + \gamma_c \cdot d + \gamma_m \cdot \left(H_s - H_{wt}\right) + S\right) \cdot \left(L_t - d\right) \qquad V_{uLLtoe3} = -2.2\ kip$$

$$V_{uLLtoe} := V_{uLLtoe1} + V_{uLLtoe2} + V_{uLLtoe3} \qquad V_{uLLtoe} = 2.6\ kip$$

$$\sigma_{heelshear} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h - d\right) = -104.7\ \frac{lbf}{ft}$$

$$U_{hshear} := U_h - \frac{U_h - U_t}{L_{base}} \cdot \left(L_h - d\right) \qquad U_{hshear} = 8.6\ ft$$

$$V_{uLLheel1} := \frac{-1}{2} \cdot \left(\sigma_{heelface} + \sigma_{heelshear}\right) \cdot \left(L_h - d\right) \qquad V_{uLLheel1} = -0.9\ kip$$

$$V_{uLLheel2} := \frac{1}{2} \cdot \left(U_{hshear} + U_h\right) \cdot \gamma_w \cdot \left(L_h - d\right) \qquad V_{uLLheel2} = 3.3\ kip$$

$V_{uLLheel3} := -\left(\gamma_w \cdot H_{wh} + \gamma_c \cdot d\right) \cdot \left(L_h - d\right)$ $\qquad V_{uLLheel3} = -4.6\ kip$

$V_{uLLheel} := V_{uLLheel1} + V_{uLLheel2} + V_{uLLheel3}$ $\qquad V_{uLLheel} = -2.1\ kip$

Shear may include the static and seismic components. Use the maximum load factor between the static and seismic components to calculate the ultimate shear, conservative assumption.

$LF := \max\left(LF_{EV}, LF_{Hsp}, LF_{EQ}\right) = 1.6$

$V_{uLL.footing} := LF \cdot \max\left(\left|V_{uLLtoe}\right|, \left|V_{uLLheel}\right|\right)$ $\qquad V_{uLL.footing} = 4.2\ kip$

$V_{cfooting} := 2 \cdot \sqrt{f_c \cdot \frac{lbf}{in^2}} \cdot b \cdot min\left(d_{fb}, d_{ft}\right)$ $\qquad V_{cfooting} = 29.6\ kip$

$\phi V_{nfooting} := \phi_v \cdot V_{cfooting}$ $\qquad \phi V_{nfooting} = 22.2\ kip$

$Check_F := \textbf{if}\left(V_{uLL.footing} < \phi V_{nfooting}, \text{"OK"}, \text{"NO\_GOOD"}\right)$

Check footing shear strength

$Check_F = \text{"OK"}$

Footing - Bottom Reinforcement

$\sigma_{toeM} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h + t\right) = -435.3\ \frac{lbf}{ft}$ $\qquad U_{tmom} := U_h - \frac{U_h - U_t}{L_{base}} \cdot \left(L_h + t\right) = 7.42\ ft$

$M_{uLLtoe1} := -\left(\left(\sigma_{toeface} - \sigma_{toeM}\right) \cdot \frac{L_t^2}{3} + \sigma_{toeM} \cdot \frac{L_t^2}{2}\right) = 14.64\ kip \cdot ft$

$M_{uLLtoe2} := \left(\left(U_{tmom} - U_t\right) \cdot \frac{L_t^2}{6} + U_t \cdot \frac{L_t^2}{2}\right) \cdot \gamma_w = 6.8\ kip \cdot ft$

$M_{uLLtoe3} := -\left(\gamma_s \cdot H_{wt} + \gamma_m \cdot \left(H_s - H_{wt}\right) + \gamma_c \cdot d + S\right) \cdot \frac{L_t^2}{2} = -9.72\ kip \cdot ft$

Moment may include the static and seismic components. Use the maximum load factor between the static and seismic components to calculate the ultimate moment, conservative assumption.

$$M_{uLLtoe} := LF \cdot \left(M_{uLLtoe1} + M_{uLLtoe2} + M_{uLLtoe3}\right) = 18.74 \ kip \cdot ft$$

$$LF = 1.6$$

$$k_{utoe} := 1 - \sqrt{1 - \frac{\left|M_{uLLtoe}\right|}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_{fb}^{\,2}}} = 0.0162$$

$$A_{stoe} := \frac{0.85 \cdot f_c \cdot k_{utoe} \cdot b \cdot d_{fb}}{f_y} = 0.22 \ in^2$$

Minimum Steel, ACI 350-06, Section10.5

$$A_{smintoe1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_{fb} = 0.78 \ in^2$$

$$A_{smintoe2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_{fb} = 0.74 \ in^2$$

$$A_{Amimtoe} := \max\left(A_{smintoe1}, A_{smintoe2}\right) = 0.78 \ in^2$$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$$A_{smaxtoe} := 0.25 \cdot \rho_b \cdot b \cdot d_{fb} = 1.67 \ in^2$$

Footing - Top Reinforcement

$$\sigma_{heelM} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h\right) = -293.6 \ \frac{lbf}{ft}$$

$$U_{hmom} := U_h - \frac{U_h - U_t}{L_{base}} \cdot \left(L_h\right) = 7.9 \ ft$$

$$M_{uLLheel1} := -\left(\left(\sigma_{heelM} - \sigma_{heelface}\right) \cdot \frac{L_h^{\,2}}{6} + \sigma_{heelface} \cdot \frac{L_h^{\,2}}{2}\right)$$

$$M_{uLLheel1} = -5 \ kip \cdot ft$$

$$M_{uLLheel2} := \left(\left(U_h - U_{hmom}\right) \cdot \frac{L_h^{\,2}}{3} + U_{hmom} \cdot \frac{L_h^{\,2}}{2}\right) \cdot \gamma_w$$

$$M_{uLLheel2} = 16.9 \ kip \cdot ft$$

$$M_{uLLheel3} := -\left(\gamma_w \cdot H_{wh} + \gamma_c \cdot d\right) \cdot \frac{L_h^{\,2}}{2}$$

$$M_{uLLheel3} = -23.4 \ kip \cdot ft$$

Moment may include the static and seismic components. Use the maximum load factor between the static and seismic components to calculate the ultimate moment, conservative assumption.

$LF = 1.6$ $\quad d_{ft} = 1.6\ ft$

$M_{uLLheel} := LF \cdot (M_{uLLheel1} + M_{uLLheel2} + M_{uLLheel3}) = -18.3\ kip \cdot ft$

$k_{uheel} := 1 - \sqrt{1 - \dfrac{|M_{uLLheel}|}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_{ft}^2}} = 0.0159$

$A_{sheel} := \dfrac{0.85 \cdot f_c \cdot k_{uheel} \cdot b \cdot d_{ft}}{f_y} = 0.21\ in^2$

Minimum Steel, ACI 350-06, Section10.5

$A_{sminheel1} := \dfrac{200 \cdot \dfrac{lbf}{in^2}}{f_y} \cdot b \cdot d_{ft} = 0.78\ in^2$

$A_{sminheel2} := 3 \cdot \dfrac{\sqrt{f_c \cdot \dfrac{in^2}{lbf}} \cdot \dfrac{lbf}{in^2}}{f_y} \cdot b \cdot d_{ft} = 0.74\ in^2$

$A_{Amimheel} := \max(A_{sminheel1}, A_{sminheel2}) = 0.78\ in^2$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$A_{smaxheel} := 0.25 \cdot \rho_b \cdot b \cdot d_{ft} = 1.67\ in^2$

Temperature and Shrinkage Steel

$A_{tsF} := \dfrac{0.0028 \cdot b \cdot d}{2} = 0.4\ in^2$

Key Reinforcement

$P_{PK} := -\left(\left(\dfrac{1}{2}\right) \cdot K_{PB} \cdot (\gamma_s - \gamma_w) \cdot K^2\right) = -1.4\ kip$

$y_{PPK} := -\left(\dfrac{2}{3} \cdot K\right) = -2.67\ ft$

Factored Design Shear

$V_{uK} := -(LF_{EH} \cdot P_{PK})$ $\quad V_{uK} = 2.2\ kip$ $\quad d_k = 0.6\ ft$

$V_{cK} := 2 \cdot \sqrt{f_c \cdot \dfrac{lbf}{in^2}} \cdot b \cdot d_k$ $\quad V_{cK} = 11.4\ kip$

$\phi V_{nK} := \phi_v \cdot V_{cK}$ $\quad \phi V_{nK} = 8.5\ kip$

$Check_{Key} := \mathbf{if}\ (V_{uK} < \phi V_{nK}, \text{“OK”}, \text{“NO\_GOOD”})$

Check key shear strength

$Check_{Key} = \text{“OK”}$

Factored Design Moment

$$M_{uK} := LF_{EH} \cdot P_{PK} \cdot y_{PPK} = 5.9\ kip \cdot ft$$

$$k_{uK} := 1 - \sqrt{1 - \frac{M_{uK}}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_k^{\,2}}} = 0.0349$$

$$A_{sK} := \frac{0.85 \cdot f_c \cdot k_{uK} \cdot b \cdot d_k}{f_y} = 0.18\ in^2$$

Minimum Steel, ACI 350-06, Section 10.5

$$A_{sminK\_1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_k = 0.3\ in^2$$

$$A_{sminK\_2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_k = 0.28\ in^2$$

$$A_{smin\_K} := \max\left(A_{sminK\_1}, A_{sminK\_2}\right) = 0.3\ in^2$$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$$\rho_b = 0.0285 \qquad A_{smax\_K} := 0.25 \cdot \rho_b \cdot b \cdot d_k = 0.64\ in^2$$

Temperature and Shrinkage Steel (USACE EM1110-2-2104, Section 2-8)

$$A_{tK} := \frac{.0028 \cdot b \cdot t_k}{2} = 0.2\ in^2$$

Summary

Stability

Toe Stress $\sigma_{toeface} = -1002 \ \frac{lbf}{ft}$ Negative is Compression

Heel Stress $\sigma_{heelface} = 415 \ \frac{lbf}{ft}$

Check bearing stress $Check_B = \text{“OK”}$

Sliding factor of safety $FS_s = 1.1$ $Check_{FS} = \text{“OK”}$

Wall

Shear $Check_{WS} = \text{“OK”}$

Minimum Steel	Calculated steel	Maximum steel	Temperature and Shrinkage
$A_{Amim} = 0.54 \ in^2$	$A_{sws} = 0.17 \ in^2$	$A_{smax} = 1.15 \ in^2$	$A_{ts} = 0.3 \ in^2$

Wall Reinforcement - #6 @ 8" Vert. EF, As = 0.66in2; #5@9" Hor. EF, As = 0.41in2

Footing

Shear $Check_F = \text{“OK”}$

Bottom Steel Requirements

Minimum Steel	Calculated steel	Maximum steel	Temperature and Shrinkage
$A_{Amimtoe} = 0.78 \ in^2$	$A_{stoe} = 0.22 \ in^2$	$A_{smaxtoe} = 1.67 \ in^2$	$A_{tsF} = 0.4 \ in^2$

Top Steel Requirements

Minimum Steel	Calculated steel	Maximum steel
$A_{Amimheel} = 0.78 \ in^2$	$A_{sheel} = 0.21 \ in^2$	$A_{smaxheel} = 1.67 \ in^2$

Footing Reinforcement - #7 @ 8" Transv. T&B, As = 0.90in2; #6@10" Long. T&B, As = 0.53in2

Key

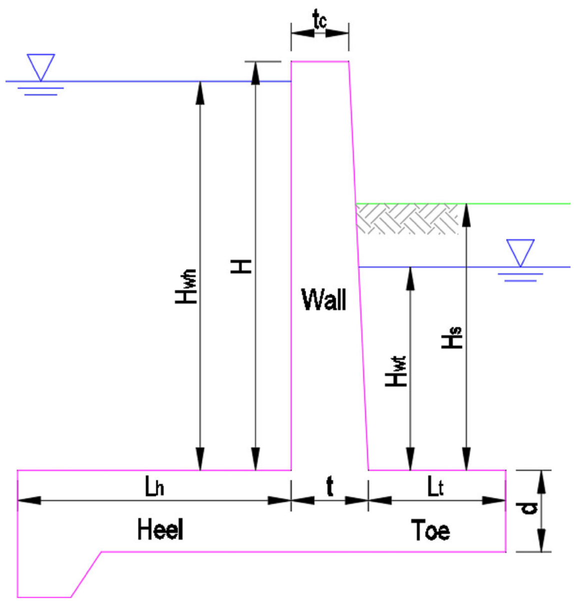
Shear $Check_{Key} = \text{“OK”}$

Minimum Steel	Calculated steel	Maximum steel	Temperature and Shrinkage
$A_{smin\_K} = 0.3\ in^2$	$A_{sK} = 0.18\ in^2$	$A_{smax\_K} = 0.64\ in^2$	$A_{tK} = 0.2\ in^2$

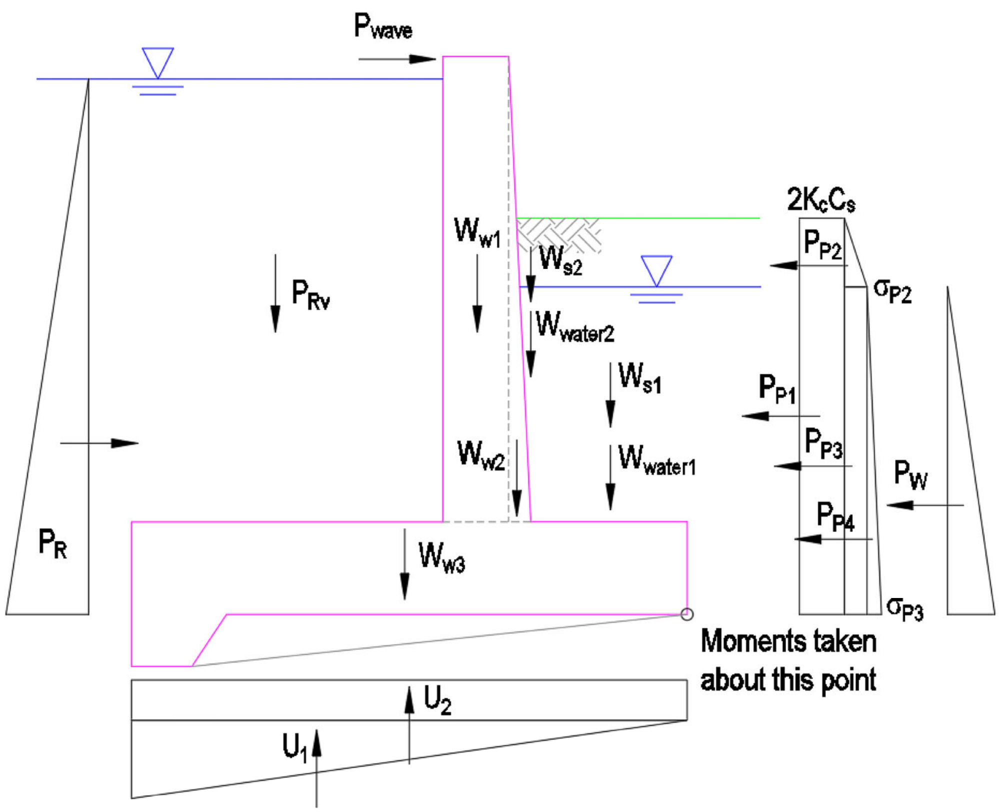
Key Reinforcement - #5 @ 9" EW EF, As = 0.41in2

REFERENCES

1. IBC 2012 with reference on ASCE7-10
2. ACI 318-14, ACI 530-13 and UFC 3-301-01 with changes on May 15, 2014
3. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 with 2016 Interim Revisions (referred to as LRFD)
4. EM 1110-2-2100: Stability Analysis of Concrete Structures
5. EM 1110-2-2502: Retaining and Flood Walls
6. EM 1110-2-2104: Strength Design for Reinforced Concrete Hydraulic Structures.
7. Engineering and Construction Bulletin (ECB) No. 2017-2: Revision and Clarification of EM 2100 and EM 2502



Notes:
-Driving side is reservoir side
-Soil on top of heel not taken into consideration due to possible erosion during high water event



Analysis By: Nestor Delgado, P.E.
Checked By: Joseph Cervantes

Updated: Sep 2023

WALL DIMENSIONS

Top of wall elevation $EL_{TW} := 12.5 \cdot ft$

Water elev. (heel) $SWEL_h := 12.5 \cdot ft$

Water elev. (toe) $SWEL_t := 9 \cdot ft$

Footing thickness $d := 18 \cdot in$

Heel length $L_h := 5 \cdot ft$

Toe length $L_t := 3.5 \cdot ft$

Key depth $K := 2 \cdot ft$

Key thickness $t_k := 1 \cdot ft$

Equipment Surcharge $S := 0 \cdot plf$

Compaction load $C_{load} := 0 \cdot plf$

Soil elev. (heel) $EL_{s\_h} := 9 \cdot ft$

Soil elev. (toe) $EL_{s\_t} := 9 \cdot ft$

Bottom of footing elevation $EL_{bs} := 5.5 \cdot ft$

Wall thickness at crest $t_c := 14 \cdot in$

Wall thickness at base $t := 14 \cdot in$

Wall height $H := EL_{TW} - EL_{bs} - d = 5.5\ ft$

Soil height $H_s := EL_{s\_t} - EL_{bs} - d = 2\ ft$

Water height (toe) $H_{wt} := SWEL_t - EL_{bs} - d = 2\ ft$

Water height (heel) $H_{wh} := SWEL_h - EL_{bs} - d = 5.5\ ft$

Base Length $L_{base} := L_t + L_h + t = 9.67\ ft$

Wall Batter $Bat := \dfrac{t - t_c}{H} = 0$

PROPERTIES AND COEFFICIENTS

Soil moist unit weight $\gamma_m := 110 \cdot psf$

Soil saturated unit weight $\gamma_s := 120 \cdot psf$

Concrete unit weight $\gamma_c := 150 \cdot psf$ (Stability) $\gamma_{cb} := 150 \cdot psf$ (Bearing stress)

Water unit weight $\gamma_w := 62.4 \cdot psf$

Shear strength parameter (Backfill) $\phi_1 := 30 \cdot deg$

Sliding parameter $\phi_s := 30 \cdot deg$

Cohesion (Backfill) $c := 0 \cdot psf$

Cohesion (Footing Soil) $c_s := 0 \cdot psf$

Bearing Capacity $Q_{allowable} := 2000 \cdot plf$ (See bearing capacity calculations)

Horizontal Seismic Coefficient $k_h := 0$

Slope inclination Angle $\beta_a := 0 \cdot deg$ $\beta_p := 0 \cdot deg$ $(passive)$

At-rest coefficient $K_o := (1 - \sin(\phi_1)) \cdot (1 + \sin(\beta_a))$ $K_o = 0.5$

Sliding factor of safety $FS_{sliding} := 1.1$

GLOBAL STABILITY

WEIGHT OF CONCRETE:

Stem:

$W_{w1} := -\gamma_c \cdot H \cdot t_c = -1\ kip$ $x_{w1} := L_t + \dfrac{t_c}{2} = 4.1\ ft$

$W_{w2} := -\gamma_c \cdot \dfrac{1}{2} \cdot H \cdot (t - t_c) = 0\ kip$ $x_{w2} := L_t + \dfrac{2}{3} \cdot (t - t_c) = 3.5\ ft$

Footing:

$W_{w3} := -\gamma_c \cdot (L_{base}) \cdot d = -2.2\ kip$ $x_{w3} := \dfrac{L_{base}}{2} = 4.8\ ft$

SOIL FORCES:

Vertical Forces:

$L_{atw} := Bat \cdot H_{wt} = 0\ ft$

$L_{ats} := Bat \cdot H_s = 0\ ft$

$W_{s1} := -(\gamma_s - \gamma_w) \cdot H_{wt} \cdot L_t + -\gamma_m \cdot (H_s - H_{wt}) \cdot L_t = -0.4\ kip$ $x_{s1} := \dfrac{L_t}{2} = 1.8\ ft$

$W_{s2} := -(\gamma_s - \gamma_w) \cdot \dfrac{1}{2} \cdot H_{wt} \cdot L_{atw} - \gamma_m \cdot (H_s - H_{wt}) \cdot \left(\dfrac{L_{ats} + L_{atw}}{2}\right) = 0\ kip$ $x_{s2} := L_t + \dfrac{2}{3} \cdot (L_{ats}) = 3.5\ ft$

Horizontal Forces

Resisting / Passive Wedge

$$A_p := (1 + k_h \cdot \tan(\phi_s)) \cdot \tan(\phi_s) + \frac{2 \cdot c_s \cdot (1 + \tan(\phi_s) \cdot \tan(\beta_p))}{\gamma_m \cdot \left(\dfrac{1}{ft}\right) \cdot (H_s + d)}$$ $A_p = 0.577$

$$c_{1p} := \frac{2 \cdot \tan(\phi_s) \cdot (\tan(\phi_s) - k_h) + \dfrac{4 \cdot c_s \cdot (\tan(\phi_s) - \tan(\beta_p))}{\gamma_m \cdot \left(\dfrac{1}{ft}\right) \cdot (H_s + d)}}{A_p}$$ $c_{1p} = 1.2$

$$c_{2p} := \frac{\tan(\phi_s)\cdot(1+\tan(\phi_s)\cdot\tan(\beta_p)) + (\tan(\beta_p)-k_h) + \dfrac{2\cdot c_s\cdot(1+\tan(\phi_s)\cdot\tan(\beta_p))}{\gamma_m\cdot\left(\dfrac{1}{ft}\right)\cdot(H_s+d)}}{A_p} \qquad c_{2p}=1$$

$$\alpha_p := \operatorname{atan}\left(\frac{-c_{1p}+\sqrt{c_{1p}^{\ 2}+4\cdot c_{2p}}}{2}\right) \qquad \alpha_p = 30\ deg$$

Passive pressure coefficient
for seismic (Moist condition)

$$K_{PO} := \left(\frac{1+\tan(\phi_s)\cdot\cot(\alpha_p)}{1-\tan(\phi_s)\cdot\tan(\alpha_p)}\right)\cdot\left(\frac{\tan(\alpha_p)}{\tan(\alpha_p)-\tan(\beta_p)}\right) \qquad K_{PO}=3$$

$$K_P := \textbf{if}\ (k_h > .001\ ,K_{PO}\ ,K_{PO}) \qquad K_P = 3$$

Passive pressure for seismic
(Saturated condition)

$$K_{PBO} := \left(\frac{1+\tan(\phi_s)\cdot\cot(\alpha_p)}{1-\tan(\phi_s)\cdot\tan(\alpha_p)}\right)\cdot\left(1+\left(\frac{\tan(\alpha_p)}{\tan(\alpha_p)-\tan(\beta_p)}-1\right)\cdot\left(\frac{\gamma_m}{\gamma_s-\gamma_w}\right)\right)$$

$$K_{PBO}=3$$

$$K_{PB} := \textbf{if}\ (k_h > .001\ ,K_{PBO}\ ,K_{PBO}) \qquad K_{PB}=3$$

$$K_c := \frac{1}{2\cdot\sin(\alpha_p)\cdot\cos(\alpha_p)\cdot(1-\tan(\phi_s)\cdot\cos(\alpha_p))}\cdot\frac{\tan(\alpha_p)}{\tan(\alpha_p)-\tan(\beta_p)} \qquad K_c = 2.3$$

$$P_{P1} := -(2\cdot K_c\cdot c\cdot(H_s+d))\cdot ft = 0\ kip \qquad y_{PP1} := \frac{(H_s+d)}{2} = 1.8\ ft$$

$$\sigma_{P2} := K_P\cdot\gamma_m\cdot(H_s-H_{wt}) = 0\ ft\cdot ksf$$

$$P_{P2} := -\left(\left(\frac{1}{2}\right)\cdot\sigma_{P2}\cdot(H_s-H_{wt})\right) = 0\ kip \qquad y_{PP2} := (H_{wt}+d)+\frac{1}{3}\cdot(H_s-H_{wt}) = 3.5\ ft$$

$$\sigma_{P3} := \begin{array}{|l} \text{if } H_{wt} < H_s \\ \quad K_P\cdot\gamma_m\cdot(H_s-H_{wt})+K_{PB}\cdot(\gamma_s-\gamma_w)\cdot(H_{wt}+d) \\ \text{else} \\ \quad K_{PB}\cdot(\gamma_s-\gamma_w)\cdot(H_{wt}+d) \end{array} = 0.6\ ft\cdot ksf$$

$$P_{P3} := -\left(\begin{array}{l} \text{if } H_{wt} < H_s \\ \quad \sigma_{P2} \cdot (H_{wt}+d) \\ \text{else} \\ \quad \frac{1}{2} \cdot \sigma_{P3} \cdot (H_{wt}+d) \end{array}\right) = -1.1\ kip$$

$$y_{PP3} := \begin{array}{l} \text{if } H_{wt} < H_s \\ \quad \left(\frac{H_{wt}+d}{2}\right) \\ \text{else} \\ \quad \left(\frac{1}{3} \cdot (H_{wt}+d)\right) \end{array} = 1.2\ ft$$

$$P_{P4} := \begin{array}{l} \text{if } H_{wt} < H_s \\ \quad -\left(\frac{1}{2} \cdot (\sigma_{P3} - \sigma_{P2}) \cdot (H_{wt}+d)\right) \\ \text{else} \\ \quad 0\ \mathbf{kip} \end{array} = 0\ kip$$

$$y_{PP4} := \frac{1}{3} \cdot (H_{wt}+d) = 1.2\ \mathbf{ft}$$

WATER FORCES

Vertical Forces:

$$W_{water1} := -\gamma_w \cdot H_{wt} \cdot L_t = -0.4\ kip \qquad x_{water1} := \frac{L_t}{2} = 1.8\ \mathbf{ft}$$

$$W_{water2} := -\gamma_w \cdot \frac{1}{2} \cdot H_{wt} \cdot L_{atw} = 0\ \mathbf{lbf} \qquad x_{water2} := L_t + \frac{1}{3} \cdot L_{atw} = 3.5\ \mathbf{ft}$$

$$W_{water3} := -\gamma_w \cdot L_h \cdot H_{wh} = -1.7\ kip \qquad x_{water3} := \frac{L_h}{2} + t + L_t = 7.2\ \mathbf{ft}$$

Uplift Forces:

$$\Delta h := H_{wh} - H_{wt} = 3.5\ \mathbf{ft}$$

$$L_s := L_h + t + L_t + d + min\left(H_s, H_{wt}\right) = 13.2\ \mathbf{ft}$$

Uplift at heel $\quad U_h := \mathbf{if}\left(H_{wh} > 0.1 \cdot \mathbf{ft}, H_{wh} + d, 0\right) = 7\ ft$

Uplift at toe $\quad U_t := \mathbf{if}\left(H_{wt} > 0.1 \cdot \mathbf{ft}, \left(H_{wh} + d\right) - \frac{\Delta h \cdot L_{base}}{L_s}, 0\right) = 4.4\ ft$

$$U_1 := \gamma_w \cdot \frac{1}{2} \cdot \left(U_h - U_t\right) \cdot \left(L_{base}\right) = 0.8\ kip \qquad x_{U1} := \frac{2}{3} \cdot \left(L_{base}\right) = 6.4\ \mathbf{ft}$$

$$U_2 := \gamma_w \cdot U_t \cdot \left(L_{base}\right) = 2.7\ kip \qquad x_{U2} := \frac{L_{base}}{2} = 4.8\ \mathbf{ft}$$

Horizontal Forces:

Resisting:

$P_w := -\left(\frac{1}{2}\right) \cdot \gamma_w \cdot (H_{wt} + d)^2 = -0.4\ kip$

$y_w := \frac{H_{wt} + d}{3} = 1.2\ ft$

Driving:

$P_R := \frac{1}{2} \cdot \gamma_w \cdot (H_{wh} + d)^2 = 1.5\ kip$

$y_R := \frac{1}{3} \cdot (H_{wh} + d) = 2.3\ ft$

Wave Forces

$EL_{wave} := 10.78 \cdot ft$ (Wave force elevation)

$P_{wave} := 1.5 \cdot kip$ (Wave force)

$y_{wave} := EL_{wave} - EL_{bs} = 5.28\ ft$

RESULTANT FORCES

Moments about toe

Resisting Moment

$$M_r := W_{w1} \cdot x_{w1} + W_{w2} \cdot x_{w2} + W_{w3} \cdot x_{w3} + W_{s1} \cdot x_{s1} + W_{s2} \cdot x_{s2} + W_{water1} \cdot x_{water1} + W_{water2} \cdot x_{water2} + P_{P1} \cdot y_{PP1} + P_{P2} \cdot y_{PP2} + P_{P3} \cdot y_{PP3} + P_{P4} \cdot y_{PP4} + P_w \cdot y_w + W_{water3} \cdot x_{water3} = -29.9\ kip \cdot ft$$

Overturning Moment

$M_o := P_R \cdot y_R + P_{wave} \cdot y_{wave} + U_1 \cdot x_{U1} + U_2 \cdot x_{U2} = 29.4\ kip \cdot ft$

ΣFy Vertical Force

$F_y := W_{w1} + W_{w2} + W_{w3} + W_{s1} + W_{s2} + W_{water1} + W_{water2} + W_{water3} + U_1 + U_2 = -2.2\ kip$

ΣFx Horizontal Force

$F_x := P_{P1} + P_{P2} + P_{P3} + P_{P4} + P_w + P_R + P_{wave} = 1.6\ kip$

Location of Resultant

$Resultant := \frac{M_r + M_o}{F_y} = 0.2\ ft$

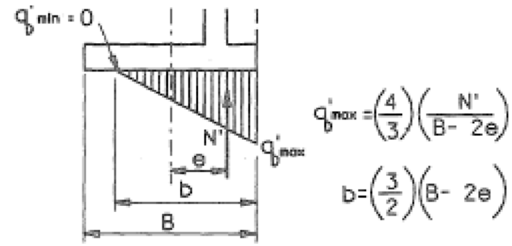
$Eccentricity := \frac{L_{base}}{2} - Resultant = 4.6\ ft$

$Middle\_Third := \frac{L_{base}}{6} = 1.6\ ft$

If the resultant fall outside the middle one-third of the base (eccentricity,e>B/6) then the maximum stress is:

$$\sigma_{max} := \frac{4}{3} \cdot \left(\frac{F_y}{L_{base} - 2 \cdot \left| Eccentricity \right|} \right)$$

$$\sigma_{max} = -6820 \ \frac{lbf}{ft}$$



The base in compression is:

$$L_{bcomp} := \frac{3}{2} \cdot \left(L_{base} - 2 \cdot \left| Eccentricity \right| \right)$$

$$L_{bcomp} = 0.7 \ ft$$

Percent of base in compression:

$$\%Compression := min\left(\left(\frac{L_{bcomp}}{L_{base}} \cdot 100 \right)\%, 100\% \right) = 7\%$$

$$Check_R := \textbf{if} \left(\%Compression > 0\%, \text{"OK"}, \text{"NOT\_GOOD"} \right)$$

Check compression at Base

$Check_R = \text{"OK"}$

Note: For Extreme Cases, the location of the resultant should lie within the base

Stress distribution Under the Footing (Negative is compression)

$$\sigma_{toeface} := \frac{F_y}{L_{base}} \cdot \left(1 + \frac{6 \cdot Eccentricity}{L_{base}} \right)$$ *The stress at the toe* $$\sigma_{toeface} = -898 \ \frac{lbf}{ft}$$

$$\sigma_{heelface} := \frac{F_y}{L_{base}} \cdot \left(1 - \frac{6 \cdot Eccentricity}{L_{base}} \right)$$ *The stress at the heel* $$\sigma_{heelface} = 433 \ \frac{lbf}{ft}$$

Bearing Factor of Safety

$$Check_B := \textbf{if} \left(Q_{allowable} > \left| \sigma_{toeface} \right|, \text{"OK"}, \text{"NO\_GOOD"} \right)$$

Check bearing stress

$Check_B = \text{"OK"}$

Sliding Factor of Safety:

$\theta := \text{atan}\left(\dfrac{K}{L_{base}}\right)$ $\theta = 11.7\ deg$

$F_{xn} := F_x \cdot \sin(\theta)$ $F_{xn} = 0.3\ kip$

$F_{xs} := F_x \cdot \cos(\theta)$ $F_{xs} = 1.6\ kip$

$F_{ys} := F_y \cdot \sin(\theta)$ $F_{ys} = -0.5\ kip$

$F_{yn} := -F_y \cdot \cos(\theta)$ $F_{yn} = 2.2\ kip$

$$FS_s := \left|\frac{\tan(\phi_s)\cdot(F_{xn}+F_{yn}) + c_s \cdot L_{base}\cdot(1\cdot ft)}{F_{xs}+F_{ys}}\right|$$ $FS_s = 1.32$

$Check_{FS} := \textbf{if}\left(FS_s > FS_{sliding}, \text{“OK”}, \text{“NOT\_GOOD”}\right)$

Check Sliding

$Check_{FS} = \text{“OK”}$

Reinforced Concrete Design

EM1110-2-2104, Strength Design for Reinforced Hydraulic Structures

Load Factors
(Table 3-1, pg. 3-10)

Permanent Loads

Dead Load	$LF_D := 1.6$
Vertical Earth Load	$LF_{EV} := 1.6$
Lateral Earth Load	$LF_{EH} := 1.6$
Hydrostatic Load (Companion)	$LF_{Hsc} := 1.6$

Temporary Loads

Hydrostatic Load (Principal)	$LF_{Hsp} := 1.6$
Surcharge Load	$LF_{ES} := 1.6$
Operating Equipment	$LF_Q := 1.6$

Dynamic Loads

Earthquake Load	$LF_{EQ} := 1.0$

Material & Section Properties

Concrete strength	$f_c := 4000 \cdot psi$
Steel strength	$f_y := 60000 \cdot psi$
Member unit width	$b := 12 \cdot in$
Resistance Factor, Bending	$\phi_b := 0.9$
Resistance Factor, Shear	$\phi_v := 0.75$

Distance to footing bottom steel	$d_{fb} := d - 4.5 \cdot in$	$d_{fb} = 13.5\ in$
Distance to footing top steel	$d_{ft} := d - 4.5 \cdot in$	$d_{ft} = 13.5\ in$
Distance to wall steel	$d_w := t - 4.5 \cdot in$	$d_w = 9.5\ in$
Distance to key steel	$d_k := t_k - 4.5 \cdot in$	$d_k = 7.5\ in$

Wall Stem - Recompute Lateral Forces for Structural Design of Wall Stem

Horizontal Forces

Reservoir Pressure

$P_{Rws} := \frac{1}{2} \cdot \gamma_w \cdot H_{wh}^{2} = 0.94\ kip$

$y_{Rws} := \frac{H_{wh}}{3} = 1.83\ ft$

Factored Design Shear

$V_{uWS} := LF_{Hsp} \cdot P_{Rws} = 1.51\ kip$

Wall Stem

$V_{cstem} := 2 \cdot \sqrt{f_c \cdot \frac{lbf}{in^2}} \cdot b \cdot d_w = 14.42\ kip$

$\phi V_{nstem} := \phi_v \cdot V_{cstem} = 10.81\ kip$

$Check_{WS} := \mathbf{if}\left(V_{uWS} < \phi V_{nstem}, \text{“OK”}, \text{“NO\_GOOD”}\right)$

Check wall shear strength

$Check_{WS} = \text{“OK”}$

Summation of Moments

Max Flexural Moment

Reservoir Forces:

$M_{ws} := P_{Rws} \cdot y_{Rws} = 1.73\ kip \cdot ft$

Factored Design Moment

$M_{uWS} := LF_{Hsp} \cdot M_{ws} = 2.77\ kip \cdot ft$

$k_u := 1 - \sqrt{1 - \frac{M_{uWS}}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_w^{2}}} = 0.0101$

$A_{sws} := \frac{0.85 \cdot f_c \cdot k_u \cdot b \cdot d_w}{f_y} = 0.1\ in^2$

Minimum Steel, ACI 350-06, Section10.5

$A_{smin1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_w = 0.38\ in^2$

$A_{smin2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_w = 0.36\ in^2$

$A_{Amim} := \max\left(A_{smin1}, A_{smin2}\right) = 0.38\ in^2$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$$\rho_b := 0.85 \cdot 0.85 \cdot \frac{f_c}{f_y} \cdot \frac{87000 \cdot \frac{lbf}{in^2}}{87000 \cdot \frac{lbf}{in^2} + f_y} = 0.0285$$

$$A_{smax} := 0.25 \cdot \rho_b \cdot b \cdot d_w = 0.8\ in^2$$

Temperature and Shrinkage Steel (USACE EM1110-2-2104, Section 2-8)

$$A_{ts} := \frac{.0028 \cdot b \cdot t}{2} = 0.24\ in^2$$

Footing

Equations assume base is in compression and d>Lh

$$\sigma_{toeshear} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h + t + d\right) = -622.4\ \frac{lbf}{ft}$$

$U_h = 7\ ft$ $\qquad U_t = 4.4\ ft$ $\qquad U_{tshear} := U_h - \frac{U_h - U_t}{L_{base}} \cdot \left(L_h + t + d\right) = 5\ ft$

$$V_{uLLtoe1} := \frac{-1}{2} \cdot \left(\sigma_{toeface} + \sigma_{toeshear}\right) \cdot \left(L_t - d\right) \qquad V_{uLLtoe1} = 1.5\ kip$$

$$V_{uLLtoe2} := \frac{1}{2} \cdot \left(U_{tshear} + U_t\right) \cdot \gamma_w \cdot \left(L_t - d\right) \qquad V_{uLLtoe2} = 0.6\ kip$$

$$V_{uLLtoe3} := -\left(\gamma_s \cdot H_{wt} + \gamma_c \cdot d + \gamma_m \cdot \left(H_s - H_{wt}\right) + S\right) \cdot \left(L_t - d\right) \qquad V_{uLLtoe3} = -0.9\ kip$$

$$V_{uLLtoe} := V_{uLLtoe1} + V_{uLLtoe2} + V_{uLLtoe3} \qquad V_{uLLtoe} = 1.2\ kip$$

$$\sigma_{heelshear} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h - d\right) = -48.8\ \frac{lbf}{ft}$$

$$U_{hshear} := U_h - \frac{U_h - U_t}{L_{base}} \cdot \left(L_h - d\right) \qquad U_{hshear} = 6.1\ ft$$

$$V_{uLLheel1} := \frac{-1}{2} \cdot \left(\sigma_{heelface} + \sigma_{heelshear}\right) \cdot \left(L_h - d\right) \qquad V_{uLLheel1} = -0.7\ kip$$

$$V_{uLLheel2} := \frac{1}{2} \cdot \left(U_{hshear} + U_h\right) \cdot \gamma_w \cdot \left(L_h - d\right) \qquad V_{uLLheel2} = 1.4\ kip$$

$V_{uLLheel3} := -\left(\gamma_w \cdot H_{wh} + \gamma_c \cdot d\right) \cdot \left(L_h - d\right)$ $\qquad V_{uLLheel3} = -2\ kip$

$V_{uLLheel} := V_{uLLheel1} + V_{uLLheel2} + V_{uLLheel3}$ $\qquad V_{uLLheel} = -1.2\ kip$

Shear may include the static and seismic components. Use the maximum load factor between the static and seismic components to calculate the ultimate shear, conservative assumption.

$LF := \max\left(LF_{EV}, LF_{Hsp}, LF_{EQ}\right) = 1.6$

$V_{uLL.footing} := LF \cdot \max\left(\left|V_{uLLtoe}\right|, \left|V_{uLLheel}\right|\right)$ $\qquad V_{uLL.footing} = 2\ kip$

$V_{cfooting} := 2 \cdot \sqrt{f_c \cdot \dfrac{lbf}{in^2}} \cdot b \cdot min\left(d_{fb}, d_{ft}\right)$ $\qquad V_{cfooting} = 20.5\ kip$

$\phi V_{nfooting} := \phi_v \cdot V_{cfooting}$ $\qquad \phi V_{nfooting} = 15.4\ kip$

$Check_F := \textbf{if}\left(V_{uLL.footing} < \phi V_{nfooting}, \text{“OK”}, \text{“NO\_GOOD”}\right)$

Check footing shear strength

$Check_F = \text{“OK”}$

Footing - Bottom Reinforcement

$\sigma_{toeM} := \sigma_{heelface} + \dfrac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot \left(L_h + t\right) = -415.9\ \dfrac{lbf}{ft}$

$U_{tmom} := U_h - \dfrac{U_h - U_t}{L_{base}} \cdot \left(L_h + t\right) = 5.36\ ft$

$M_{uLLtoe1} := -\left(\left(\sigma_{toeface} - \sigma_{toeM}\right) \cdot \dfrac{L_t^{\,2}}{3} + \sigma_{toeM} \cdot \dfrac{L_t^{\,2}}{2}\right) = 4.51\ kip \cdot ft$

$M_{uLLtoe2} := \left(\left(U_{tmom} - U_t\right) \cdot \dfrac{L_t^{\,2}}{6} + U_t \cdot \dfrac{L_t^{\,2}}{2}\right) \cdot \gamma_w = 1.81\ kip \cdot ft$

$M_{uLLtoe3} := -\left(\gamma_s \cdot H_{wt} + \gamma_m \cdot \left(H_s - H_{wt}\right) + \gamma_c \cdot d + S\right) \cdot \dfrac{L_t^{\,2}}{2} = -2.85\ kip \cdot ft$

Moment may include the static and seismic components. Use the maximum load factor between the static and seismic components to calculate the ultimate moment, conservative assumption.

$M_{uLLtoe} := LF \cdot (M_{uLLtoe1} + M_{uLLtoe2} + M_{uLLtoe3}) = 5.57\ kip \cdot ft$ $\qquad LF = 1.6$

$k_{utoe} := 1 - \sqrt{1 - \frac{|M_{uLLtoe}|}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_{fb}^2}} = 0.01$ $\qquad A_{stoe} := \frac{0.85 \cdot f_c \cdot k_{utoe} \cdot b \cdot d_{fb}}{f_y} = 0.09\ in^2$

Minimum Steel, ACI 350-06, Section10.5

$A_{smintoe1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_{fb} = 0.54\ in^2$ $\qquad A_{smintoe2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_{fb} = 0.51\ in^2$

$A_{Amimtoe} := \max(A_{smintoe1}, A_{smintoe2}) = 0.54\ in^2$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$A_{smaxtoe} := 0.25 \cdot \rho_b \cdot b \cdot d_{fb} = 1.15\ in^2$

Footing - Top Reinforcement

$\sigma_{heelM} := \sigma_{heelface} + \frac{\sigma_{toeface} - \sigma_{heelface}}{L_{base}} \cdot (L_h) = -255.3\ \frac{lbf}{ft}$ $\qquad U_{hmom} := U_h - \frac{U_h - U_t}{L_{base}} \cdot (L_h) = 5.7\ ft$

$M_{uLLheel1} := -\left((\sigma_{heelM} - \sigma_{heelface}) \cdot \frac{L_h^2}{6} + \sigma_{heelface} \cdot \frac{L_h^2}{2}\right)$ $\qquad M_{uLLheel1} = -2.5\ kip \cdot ft$

$M_{uLLheel2} := \left((U_h - U_{hmom}) \cdot \frac{L_h^2}{3} + U_{hmom} \cdot \frac{L_h^2}{2}\right) \cdot \gamma_w$ $\qquad M_{uLLheel2} = 5.1\ kip \cdot ft$

$M_{uLLheel3} := -(\gamma_w \cdot H_{wh} + \gamma_c \cdot d) \cdot \frac{L_h^2}{2}$ $\qquad M_{uLLheel3} = -7.1\ kip \cdot ft$

Moment may include the static and seismic components. Use the maximum load factor between the static and seismic components to calculate the ultimate moment, conservative assumption.

$LF = 1.6 \qquad d_{ft} = 1.1\ ft$

$M_{uLLheel} := LF \cdot (M_{uLLheel1} + M_{uLLheel2} + M_{uLLheel3}) = -7.3\ kip \cdot ft$

$k_{uheel} := 1 - \sqrt{1 - \frac{|M_{uLLheel}|}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_{ft}^2}} = 0.0131$

$A_{sheel} := \frac{0.85 \cdot f_c \cdot k_{uheel} \cdot b \cdot d_{ft}}{f_y} = 0.12\ in^2$

Minimum Steel, ACI 350-06, Section10.5

$A_{sminheel1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_{ft} = 0.54\ in^2$

$A_{sminheel2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_{ft} = 0.51\ in^2$

$A_{Amimheel} := \max(A_{sminheel1}, A_{sminheel2}) = 0.54\ in^2$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$A_{smaxheel} := 0.25 \cdot \rho_b \cdot b \cdot d_{ft} = 1.15\ in^2$

Temperature and Shrinkage Steel

$A_{tsF} := \frac{0.0028 \cdot b \cdot d}{2} = 0.3\ in^2$

Key Reinforcement

$P_{PK} := -\left(\left(\frac{1}{2}\right) \cdot K_{PB} \cdot (\gamma_s - \gamma_w) \cdot K^2\right) = -0.3\ kip$

$y_{PPK} := -\left(\frac{2}{3} \cdot K\right) = -1.33\ ft$

Factored Design Shear

$V_{uK} := -(LF_{EH} \cdot P_{PK})$

$V_{uK} = 0.6\ kip$

$d_k = 0.6\ ft$

$V_{cK} := 2 \cdot \sqrt{f_c \cdot \frac{lbf}{in^2}} \cdot b \cdot d_k$

$V_{cK} = 11.4\ kip$

$\phi V_{nK} := \phi_v \cdot V_{cK}$

$\phi V_{nK} = 8.5\ kip$

$Check_{Key} := \mathbf{if}\ (V_{uK} < \phi V_{nK}, \text{"OK"}, \text{"NO\_GOOD"})$

Check key shear strength

$Check_{Key} = \text{"OK"}$

Factored Design Moment

$$M_{uK} := LF_{EH} \cdot P_{PK} \cdot y_{PPK} = 0.7\ kip \cdot ft$$

$$k_{uK} := 1 - \sqrt{1 - \frac{M_{uK}}{0.425 \cdot \phi_b \cdot f_c \cdot b \cdot d_k^2}} = 0.0043$$

$$A_{sK} := \frac{0.85 \cdot f_c \cdot k_{uK} \cdot b \cdot d_k}{f_y} = 0.02\ in^2$$

Minimum Steel, ACI 350-06, Section 10.5

$$A_{sminK\_1} := \frac{200 \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_k = 0.3\ in^2$$

$$A_{sminK\_2} := 3 \cdot \frac{\sqrt{f_c \cdot \frac{in^2}{lbf}} \cdot \frac{lbf}{in^2}}{f_y} \cdot b \cdot d_k = 0.28\ in^2$$

$$A_{smin\_K} := \max\left(A_{sminK\_1}, A_{sminK\_2}\right) = 0.3\ in^2$$

Maximum Steel, USACE EM1110-2-2104, Section 3-5

$$\rho_b = 0.0285 \qquad A_{smax\_K} := 0.25 \cdot \rho_b \cdot b \cdot d_k = 0.64\ in^2$$

Temperature and Shrinkage Steel (USACE EM1110-2-2104, Section 2-8)

$$A_{tK} := \frac{.0028 \cdot b \cdot t_k}{2} = 0.2\ in^2$$

Summary

Stability

Toe Stress $\sigma_{toeface} = -898\ \frac{lbf}{ft}$ Negative is Compression

Heel Stress $\sigma_{heelface} = 433\ \frac{lbf}{ft}$

Check bearing stress $Check_B = \text{"OK"}$

Sliding factor of safety $FS_s = 1.3$ $Check_{FS} = \text{"OK"}$

Wall

Shear $Check_{WS} = \text{"OK"}$

Minimum Steel	Calculated steel	Maximum steel	Temperature and Shrinkage
$A_{Amim} = 0.38\ in^2$	$A_{sws} = 0.07\ in^2$	$A_{smax} = 0.81\ in^2$	$A_{ts} = 0.24\ in^2$

Wall Reinforcement - #6 @ 8" Vert. EF, As = 0.66in2; #5@9" Hor. EF, As = 0.41in2

Footing

Shear $Check_F = \text{"OK"}$

Bottom Steel Requirements

Minimum Steel	Calculated steel	Maximum steel	Temperature and Shrinkage
$A_{Amimtoe} = 0.54\ in^2$	$A_{stoe} = 0.09\ in^2$	$A_{smaxtoe} = 1.15\ in^2$	$A_{tsF} = 0.3\ in^2$

Top Steel Requirements

Minimum Steel	Calculated steel	Maximum steel
$A_{Amimheel} = 0.54\ in^2$	$A_{sheel} = 0.12\ in^2$	$A_{smaxheel} = 1.15\ in^2$

Footing Reinforcement - #7 @ 8" Transv. T&B, As = 0.90in2; #6@10" Long. T&B, As = 0.53in2

Key

Shear $Check_{Key} = \text{“OK”}$

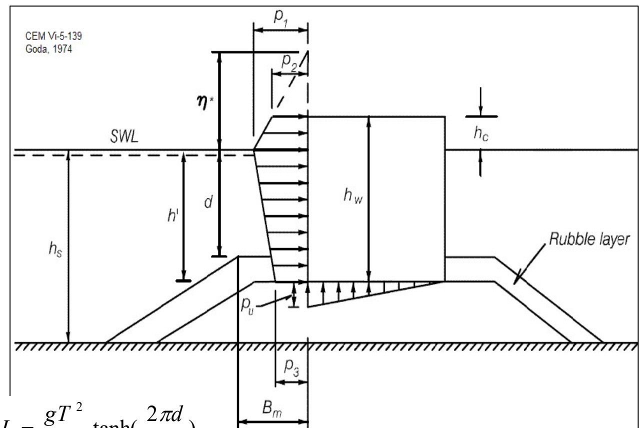
Minimum Steel	Calculated steel	Maximum steel	Temperature and Shrinkage
$A_{smin\_K} = 0.3\ in^2$	$A_{sK} = 0.02\ in^2$	$A_{smax\_K} = 0.64\ in^2$	$A_{tK} = 0.2\ in^2$

Key Reinforcement - #5 @ 9" EW EF, As = 0.41in2

Wave Loading on Walls and Caissons

Input Wave and Water Level Variables from Input Sheet			
H_{mo} =	4	ft	Wave Height at Structure Toe
T_p =	4.6686	s	Spectral Peak Wave Period
h =	6.5	ft	Depth of water at 5Hs from wall (max)
$cot\ \alpha$ =			Cotangent of slope of berm
γ_w =	64	pcf	Specific Weight of Water
γ_r =	165	pcf	Specific Weight of Toe Stone
g =	32.2	ft/s^2	Acceleration of Gravity
β =	0	deg	Wave Direction
S_r =			Specific Gravity of Toe Stone
H_B/h_B =	0.6153846		

Computed Variables			
h_S =	6.5	ft	Water Depth At Structure Toe
h_b =	6.5	ft	Water Depth at 5Hs from Structure
h' =	6.5	ft	Depth of Still Water Above Fill
d =	6.5	ft	Depth of Still Water Above Rock
B =	1	ft	Width of Caisson
h_C =	0	ft	Height of Wall Above SWL
h_W =	6.5	ft	Height of Wall Above Fill
Bm =	5	ft	Width of Rock Forward of Wall
H_{design} = **1.80** H_s	7.2	ft	Design Wave, see note at the bottom of page
$L_{op} = g\ T^2/2\pi$	111.7	ft	Deep Water Wave Length
L_p =	63.4	ft	Local Wave Length
k_p =	0.099	1/ft	Wave Number
$k_p h_s$ =	0.644		
α_*	0.000		
α_1	0.896		
α_2	0.000		
α_3	0.823		
λ_1	1.000		
λ_2	1.000		
λ_3	1.000		
η^*	10.800	ft	
p_1 =	412.7	psf	Pressure at SWL
p_2 =	412.7	psf	Pressure at Top of Wall
p_3 =	339.8	psf	Pressure at Ground or Top of Fill
p_u =	339.8	psf	Uplift Pressure at Seaward Edge
F_H =	2446	lbf/ft	$F_H = (p_1+p_2)/2*h_c + (p_1+p_3)/2*h'$
M_H =	8205	lbf*ft/ft	$M_H = (2p_1 + p_3)h'^2/6 + (p_1 + p_2)h'h_c/2 + (p_1 + 2p_2)h_c^2/6$
F_H =	2201	lbf/ft	$F_H = U_{FH}\ F_H$
M_H =	6646	lbf*ft/ft	$M_H = U_{MH}[1/6(2p_1 + p_3)h'^2 + 1/2(p_1 + p_2)h'h_c + 1/6(p_1 + 2p_2)h_c^2]$
M_U =	82	lbf*ft/ft	$M_H = U_{MH}[1/6(2p_1 + p_3)h'^2 + 1/2(p_1 + p_2)h'h_c + 1/6(p_1 + 2p_2)h_c^2]$
M_G =	328	lbf*ft/ft	$M_H = U_{MH}[1/6(2p_1 + p_3)h'^2 + 1/2(p_1 + p_2)h'h_c + 1/6(p_1 + 2p_2)h_c^2]$
$M_G/(M_H+M_U)>1?$	0.05	not stab	



$$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$$

eqn VI-5-147 $\eta^* = 0.75(1 + \cos\beta)\lambda_1 H_{design}$

eqn VI-5-148 $p_1 = 0.5(1 + \cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_* \cos^2\beta)\rho_w g H_{design}$

eqn VI-5-149 p_2 = if $\eta^*>h_c$ then $(1 - h_c/\eta^*)p_1$ else 0

eqn VI-5-150 $p_3 = \alpha_3 p_1$

Force reduction for bias and uncertainty:

Moment reduction for bias and uncertainty (moment taken at the heel of the caisson):

Wave Loading on Walls and Caissons

Table VI-5-55: Uncertainty and Bias of Horizontal Wave induced force					
Stochiastic	Mean	No Model Tests		Model Test Performed	
Variable	Value	Stand. Dev.		Stand. Dev.	
X_i	μx_i	σx_i	$\sigma x_i/\mu x_i$ (%)	σx_i	$\sigma x_i/\mu x_i$ (%)
U_{FH}	0.90	0.25	0.22	0.05	0.055
U_{MH}	0.81	0.40	0.49	0.10	0.12
U_{MU}	0.72				

Notes

1: Design wave height

Design wave height defined as the highest wave in the design sea state at a location just in front of the breakwater. If seaward of a surf zone Goda (1985) recommends for practical design a value of 1.8*Hs to be used corresponding to the 0.15% exceedence value for Raileigh distributed wave heights. This corresponds to H1/250 (mean of the heights of the waves included in 1/250 of the total number of waves, counted in descending order of height from the higherst wave). Goda's recommendation includes a safety factor in terms of positive bias as discussed in Table VI-5-55.

If within the surf zone, Hdesign is taken as the highest of the random breaking waves at a distance 5*Hs seaward of the structure. The CEM is not clear on how this wave heigth should be determined. Taking Hdesign=1.8*Hs also for within the surf zone, will lead to a conservative estimate of the wave forces. To make a more accurate assessment of the wave heigths, the approach presented by Battjes and Groenendijk (Wave height distributions on shallow foreshores, 2000), could be applied. However this approach should be studied in more detail.

MDOT
MARYLAND DEPARTMENT OF TRANSPORTATION™

MARYLAND PORT ADMINISTRATION

BALTIMORE COSTAL STORM RISK MANAGEMENT FEASIBILITY ANALYSIS

BALTIMORE CITY, MARYLAND

US Army Corps of Engineers ®
Baltimore District

MARK	DESCRIPTION	DATE	APPR

DESIGN BY:		ISSUE DATE: 12/01/2021	
DRAWN BY:	CKD BY:	SOLICITATION NO.: --	
SBMT BY:	PLOT DATE:	CONTRACT NO.: --	
PLOT SCALE: 1"= 1'		FILE / MAP NUMBER:	
SIZE: 22" x 34"	FILE NAME: 20240123- STRUCTURAL PLAN.DWG		

US ARMY CORPS OF ENGINEERS
BALTIMORE DISTRICT
BALTIMORE, MARYLAND

ENGINEERING DIVISION

BALTIMORE DISTRICT
COASTAL STORM RISK MANAGEMENT
AT BALTIMORE, MARYLAND

MAPPING

KEY MAP SHEET

SHEET IDENTIFICATION

GI101

1 2 3 4 5

D

C

B

A

PROP. FLOODWALL

LOD LIMIT OF DISTURBANCE

EX. BUILDING

SURVEY NOTES

1. ALL COORDINATE VALUES ARE IN U.S. SURVEY FEET AND REFERENCE THE MARYLAND STATE PLANE NORTH ZONE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83)
2. ALL ELEVATIONS ARE IN U.S. SURVEY FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
3. LIDAR PROVIDED BY MARYLAND DNR, 2017.

US Army Corps of Engineers ®
Baltimore District

MARK	DESCRIPTION	DATE	APPR

US ARMY CORPS OF ENGINEERS
BALTIMORE DISTRICT
BALTIMORE, MARYLAND

ENGINEERING DIVISION

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BALTIMORE DISTRICT
COASTAL STORM RISK MANAGEMENT
AT BALTIMORE, MARYLAND
MAPPING

KEY MAP SHEET

SHEET IDENTIFICATION

GI102

D

C

B

A

PROP. FLOODWALL

LOD LIMIT OF DISTURBANCE

EX. BUILDING

SURVEY NOTES
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3. LIDAR PROVIDED BY MARYLAND DNR, 2017.

0 40 80 Feet

SCALE: 1"=40'

US Army Corps of Engineers ®
Baltimore District

MARK	DESCRIPTION	DATE	APPR

DESIGN BY:
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SBMT BY: PLOT DATE:
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US ARMY CORPS OF ENGINEERS
BALTIMORE DISTRICT
BALTIMORE, MARYLAND

ENGINEERING DIVISION

BALTIMORE DISTRICT
COASTAL STORM RISK MANAGEMENT
AT BALTIMORE, MARYLAND
MAPPING
VENTILATION BUILDING

SHEET IDENTIFICATION
GI103

BALTIMORE
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

LIFE SAFETY RISK ASSESSMENT APPENDIX

November 2022

1 Contents

1. Life Safety Risk Assessment Introduction

The United States Army Corps of Engineers (USACE) recognizes that risks to human life are a fundamental component of all flood risk management studies and must receive explicit consideration in the planning process. Current USACE guidance (ER 1105-2-101, PCB 2019-4, ECB 2019-03, ECB 2019-15, and the January 2021 Policy Directive – Comprehensive Documentation of Benefits in Decision Documents) on risk assessments in planning studies specifies how studies should be performed on new or existing dams and levees. This risk assessment's purpose is to make sure that the feasibility level designs follow the four Tolerable Risk Guidelines (TRG):

a. TRG 1 – Understanding the Risk
b. TRG 2 – Building Risk Awareness
c. TRG 3 – Fulfilling Daily Responsibilities
d. TRG 4 – Actions to Reduce Risk

While all these guidelines are important, TRGs 1 and 4 are critical to Planning studies. The risk assessment below is the first step to Understanding the Risk (TRG 1) of the proposed features and makes recommendations on changes that could Reduce the Risk (TRG 4). An additional benefit of the risk assessment is the identification of areas of concern in the proposed design that may require extra attention during design or changes to design to ensure minimal risk to the public.

For this study, the life safety risk consideration was accomplished by performing a feasibility screening level Potential Failure Mode Analysis and qualitative life loss estimate.

This life safety risk assessment focused on structural measures consisting of floodwalls along the tunnel entrances on the I-895 and I-95 and their associated transportation critical facilities in Baltimore, MD. The main Feasibility Study Report covers the other alternatives and provides the context for this specific aspect of the study. The design was at approximately 10% level when the life safety risk assessment was complete on July 29, 2022.

2. Findings and Recommendations

The life safety risk assessment consisted of data review on all available information at the time of the risk assessment, a facilitated Potential Failure Mode Analysis (PFMA), and a qualitative assessment of risk. Due to the preliminary nature of the design, the Potential Failure Modes (PFMs) identified cannot be evaluated in detail at this time.

2.1 Potential Failure Modes

Based on the preliminary design, the team brainstormed 25 PFMs associated with the floodwalls and anticipated closure structures required for accessing the ventilation buildings. The full list of failure modes can be found in Section 5.2.

2.2 Uncertainty

The following were major points of uncertainty documented during the PFMA.

- Hydrologic Loading uncertainty centered around how wave heights are incorporated into the water surface elevations for the design 100-year event and the height, type (breaking, nonbreaking), and duration of the waves.
- Vessel and debris (for example, floating cars) impact frequency and magnitude of loadings during a hurricane event are uncertain and may need to be defined probabilistically.
- The lack of site-specific subsurface information leads to uncertainty in soil types, foundation performance and type, seepage pressures and uplift, and geotechnical or structural measures required.
- Stillwater or wave overtopping depths and durations are uncertain.
- Bearing capacity of soil types is unknown.
- Groundwater elevations are unknown.
- Preliminary designs, based on presumptive values, assumptions, and insufficient geotechnical and hydrologic loading information, add to the uncertainty in their evaluation and assessment.
- There is no "with project condition" modeling available to confirm the proposed project does not transfer risk to nearby unprotected locations.

2.3 Recommendations

Throughout the PFMA, discussions were held on recommendations for consideration including the following.

- Vessel and debris (for example, floating cars) impact loadings need to be established for the floodwall designs [Hurricane and Storm Damage Risk Reduction System - Design Guidelines (Interim, March 2012)].
- Develop wave information and confirm if wave loading is the controlling load case rather than the resiliency load case of water surface at top of wall.
- Floodwall height may need to be increased to meet wave overtopping criteria which would change the dimensions of the wall and may impact cost.
- EM 1110-2-2502 (Draft, 10 April 2020) and Hurricane and Storm Damage Risk Reduction System - Design Guidelines (Interim, March 2012) recommend a sheet pile cutoff to manage seepage under a floodwall and may be required depending on the water differential, wall base width, and foundation soil.
- Recommend reviewing the load cases from the current and previous EM 1110-2-2502 and the Hurricane and Storm Damage Risk Reduction System - Design Guidelines (Interim, March 2012), and documenting each load case that will be used in the floodwall designs for clarity.
- Recommendations for the Maryland (MD) Transportation Authority closure plan: consider timing and intensity of storms, winds, etc. that could impact the ability to set closures, install pumps or generators, etc., during the storm surge rise time (Fig. 3.3). Evaluate water levels and intensity of storm to ensure safety of employees in

implementing closure measures. If storm is high intensity may want to implement closures earlier.

- Evaluate existing slopes and retaining structures in the vicinity of tunnel entrances or buildings for additional surcharge loadings from floodwall construction and coastal water loading.
- Consider increasing foundation sizes to allow for raising floodwalls in the future.
- Ensure Operation and Maintenance (O&M) manual discusses need for proper maintenance of pumps to address interior drainage, ensure generators are operating properly, and there is sufficient fuel for operating pump stations.
- Survey the area and ensure the stormwater system does not introduce water from the flood side of the floodwall into the protected area.
- For the stem concrete design: recommend looking at minimum steel required. If in excess of 1.33 of steel required, go with the minimum. This ensures that if the concrete section cracks, the steel provided has the same moment capacity as an uncracked section and the steel can handle the pressure.
- Provide sheet piling and armoring at floodwall and high-ground transition areas.
- Perform consequence modeling between the 35% and 65% design level to estimate possible life loss if the floodwalls were to breach prior to overtopping.
- Confirm "with project conditions" do not transfer flood risk to nearby unprotected areas.

2.4 Potential Cost Impacts

During the PFMA, the team discussed areas where costs could increase including the following:

- Vessel and debris (for example, floating cars) impact loadings may require a deep foundation for certain floodwalls, depending on magnitude of loadings.
- If floodwall height needs to be increased to meet wave overtopping criteria, the dimensions of the floodwall may change and impact cost. Small changes in floodwall heights and extents will not likely significantly increase cost.
- Including sheet pile cutoff at base of floodwall to control seepage will likely increase estimated cost.
- Closures are expected for access to ventilation buildings and present additional costs. Closures will also require a substantial structure on each end (abutments) to withstand the thrust and a more significant foundation design or treatment such as deep foundations to transfer the load.
- There is the potential for stormwater system mitigation to ensure the protected area is not inundated through existing stormwater systems which may increase cost.
- Considerations for increasing the base of the floodwalls for possibly raising the floodwall height in the future could increase costs.
- Including sheet piling and armoring at floodwall and high-ground transition areas will increase costs.

3 Background and Loading Hazards

3.1 Geotechnical/Geology

Site-specific geotechnical investigations have not yet been performed for the study locations. There is some limited information geotechnical information available from other Baltimore area projects. At this stage in design, the Project Delivery Team (PDT) assumes that the geotechnical conditions will be similar to those encountered for the construction of the Seagirt Terminal Bert-IV (Patapsco North Planning Unit). The expected geotechnical subsurface profile is shown in Figure 3.1 and the estimated design parameters are shown in Table 3-1.

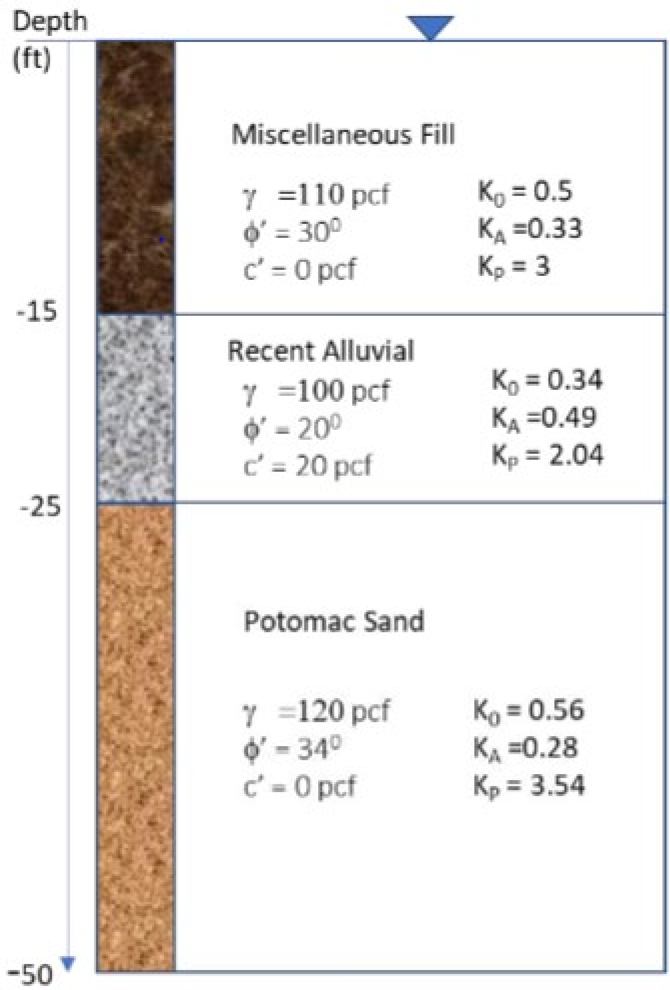


Figure 3.1: Assumed subsurface profile

Table 3-1: Estimated Geotechnical Design Parameters

Soil Type	Bulk Density (pcf)	Effective Cohesion (c')	Effective Friction Angle (ϕ')	Lateral Earth Pressure Coefficient		
				At Rest (K_o)	Active (K_a)	Passive (K_p)
Miscellaneous Fill	110	0	30	0.50	0.33	3.00
Recent Alluvial	100	20	20	0.34	0.49	2.04
Potomac Sand	120	0	34	0.56	0.28	3.54

Additional details for the geotechnical site conditions can be found in Appendix D of the Baltimore Coastal Storm Risk Management Feasibility Study Report.

3.2 Seismic Hazard

In 2011, there was a seismic event, with an epicenter in Virginia, which caused cubicle walls to shake and resulted in small cracks in buildings in the Baltimore area. Due to this, a seismic hazard curve for site class condition DE was developed using the Risk Management Center (RMC) Seismic Hazard Curves Toolbox Version 1.1 and is shown in Figure 3.2 below. The Peak Ground Acceleration (PGA) for the 1/1,000 Annual Expected Probability (AEP) is 0.2 g.

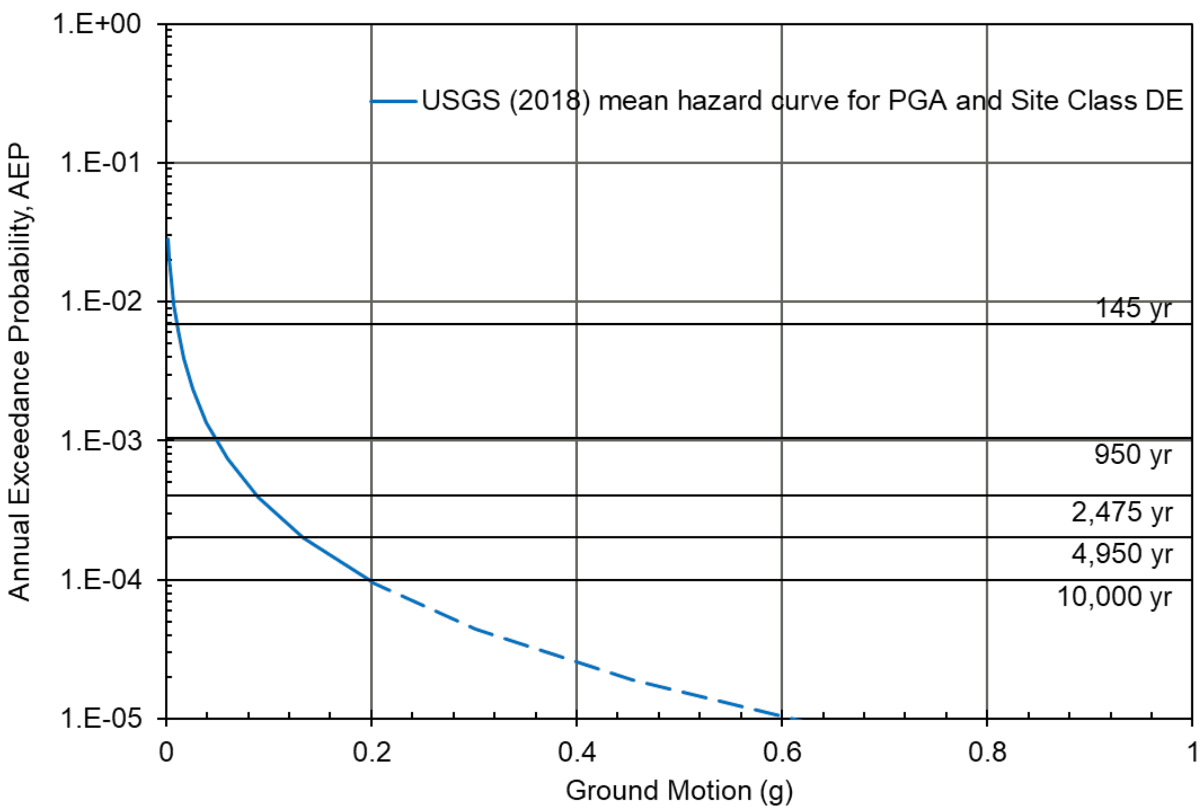


Figure 3.2: Site Class DE Seismic Hazard Curve

For PGA values less than or equal to 0.2 g, liquefaction is not a concern in saturated sands. Earthquake ground motion of this PGA will not lead to instability of the wall due to the robustness of the design. Additionally, the floodwalls will not retain water and are only anticipated to be loaded during tropical storm or hurricane events. Due to the lack of constant loading on a floodwall, seismic related potential failure modes will require a joint event to occur wherein a storm event loads the wall and earthquake ground motion causes additional loadings on a floodwall. The joint probability of occurrence of these two events is remote. Furthermore, life loss in Ft. McHenry and Harbor Tunnels due to floodwall instability and breach is not expected for certain coastal floodwater heights because of Maryland Department of Transportation (MDOT) tunnel closure plans. Seismic potential failure modes were excluded from further consideration due to the remote joint probability of occurrence and low probability of life loss within the tunnels.

3.3 Hydraulic Hazard

The hydraulic hazard for the floodwall and closure designs is under development; the risk assessment is based upon water surface elevation at Save Point 5944 which has the highest base water surface elevation plus 96 random tides plus intermediate sea level rise. It was the risk assessment (RA) team's understanding that the 96 random tides waves are considered as part of the water surface elevation used in design. However, actual wave heights and durations are unknown. The AEPs of different confidence intervals at this location are shown in Table 3-2

Table 3-2: Annual Exceedance Probability of Water Levels at NACCS Save Point 5944

NACCS Save Point 5944	Annual Exceedance Probability (1 in x) [Water Level in feet, NAVD 88]												
	1	2	5	10	20	50	100	200	500	1,000	2,000	5,000	10,000
Confidence Limit 84 for AEP	4.43	5.26	5.81	6.33	7.09	8.48	9.84	11.25	13.22	14.43	15.38	16.39	17.04
Confidence Limit 95 for AEP	5.19	6.34	6.88	7.38	8.14	9.62	11.00	12.40	14.37	15.58	16.53	17.54	18.19
Confidence Limit 98 for AEP	5.66	7.08	7.62	8.10	8.86	10.39	11.76	13.17	15.14	16.34	17.30	18.31	18.96
Expected Value AEP	2.94	3.49	4.13	4.74	5.46	6.66	7.94	9.33	11.30	12.51	13.46	14.47	15.12

Loading duration on the floodwalls during a coastal loading event is expected to be short based on historic coastal events. Hurricane Isabel in 2003 resulted in extreme water levels with storm surges along the Chesapeake Bay and its tributaries. As shown in the graphic below, while the storm surge created by Hurricane Isabel at the Baltimore, MD gage was high, the duration was approximately 12 hours as shown in Figure 3.3 below. A duration of 12 hours potentially limits the time for risk reduction measures by MDOT during a tropical storm or hurricane event, considering the life-safety of MDOT or contractor personnel.

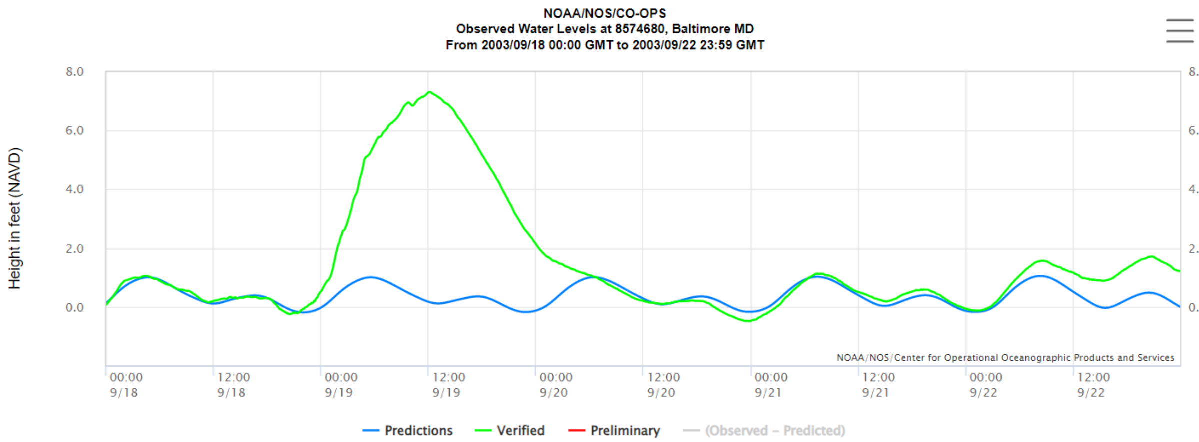


Figure 3.3 Water storm surge due to Hurricane Isabel in 2003

Additional coastal events that impacted Baltimore, MD include Tropical Storm Ernesto in 2006 which resulted in a lower peak water surface elevation but with a longer duration of storm surge as shown in Figure 3.4 below.

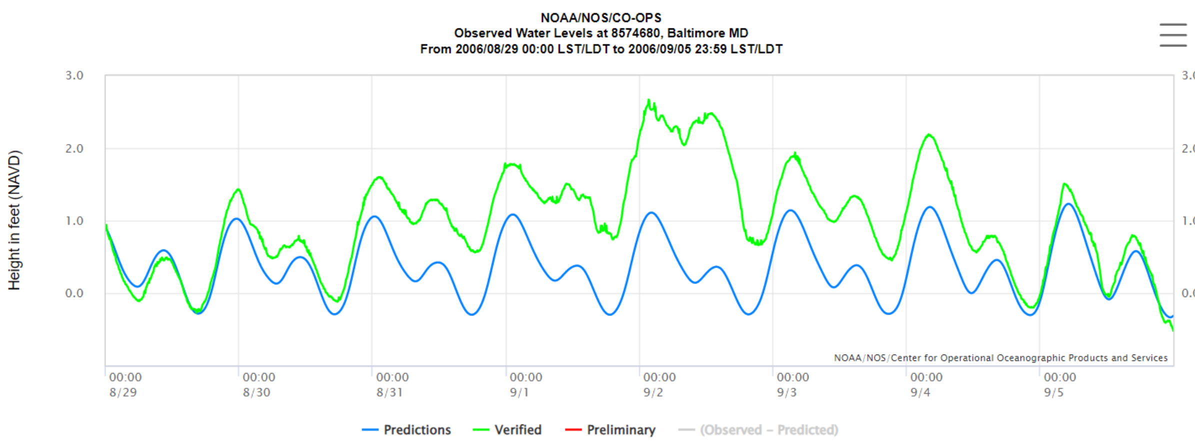


Figure 3.4 Water storm surge due to Tropical Storm Ernesto in 2006

Similar lower storm surge with longer loading durations occurred during Tropical Storm Hanna in 2008 (Figure 3.5) and Hurricane Irene in 2011 (Figure 3.6).

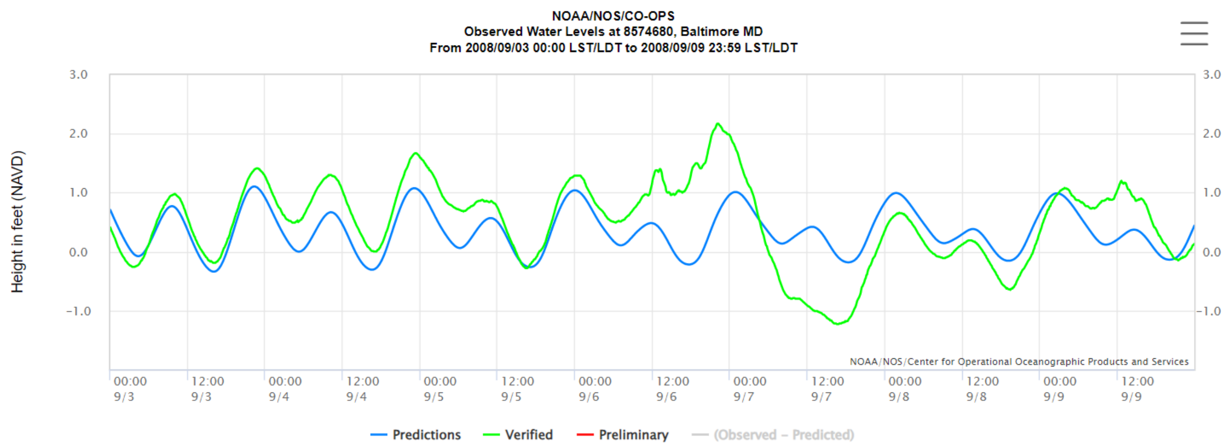


Figure 3.5 Water storm surge due to Tropical Storm Hanna in 2008

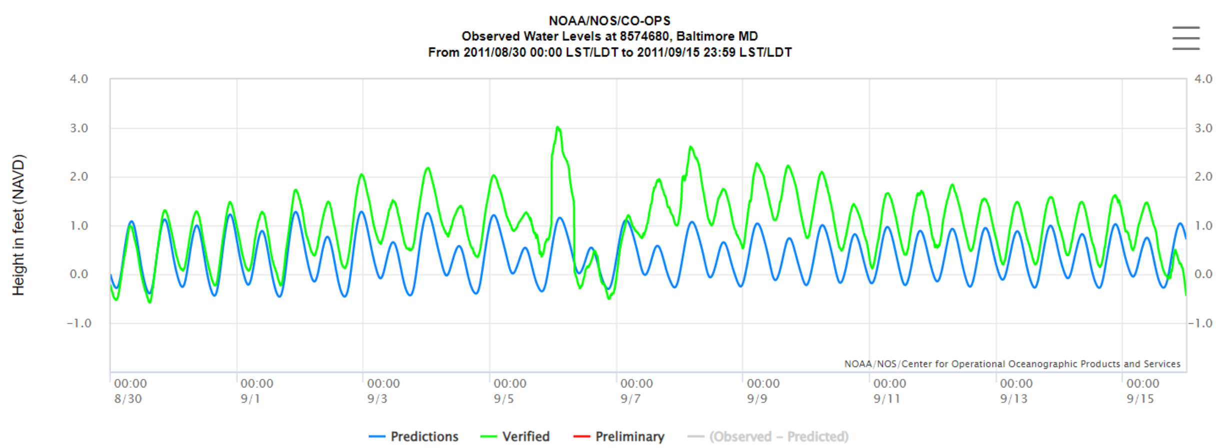


Figure 3.6 Water storm surge due to Hurricane Irene in 2011

4 Consequences

Breach models and consequence models were not completed at the time of this PFMA. Due to the alignment of the floodwalls along the tunnel entrances and for flood risk management of the ventilation buildings, consequences will be described qualitatively.

It is expected during a coastal event where flooding is predicted from storm surge, the ventilation buildings will not have personnel inside. Thus, the population at risk associated with the floodwalls is limited to the population within vehicles in the tunnels at the time of breach or overtopping. Evacuation of an inundated tunnel is unlikely.

Due to the decrease in vehicular traffic during the COVID-19 pandemic, FY2019 average daily tunnel traffic values are used in this risk assessment. The Fort McHenry Tunnel had an average daily traffic volume of approximately 141,100, inclusive of northbound and southbound traffic.

The Baltimore Harbor Tunnel had an average daily traffic volume of approximately 47,500, including both northbound and southbound traffic.

Neither the Baltimore Harbor Tunnel nor the Fort McHenry Tunnel are used as evacuation routes during hurricane events. The published evacuation routes from the City of Baltimore mapping site is provided in Figure 4.1.

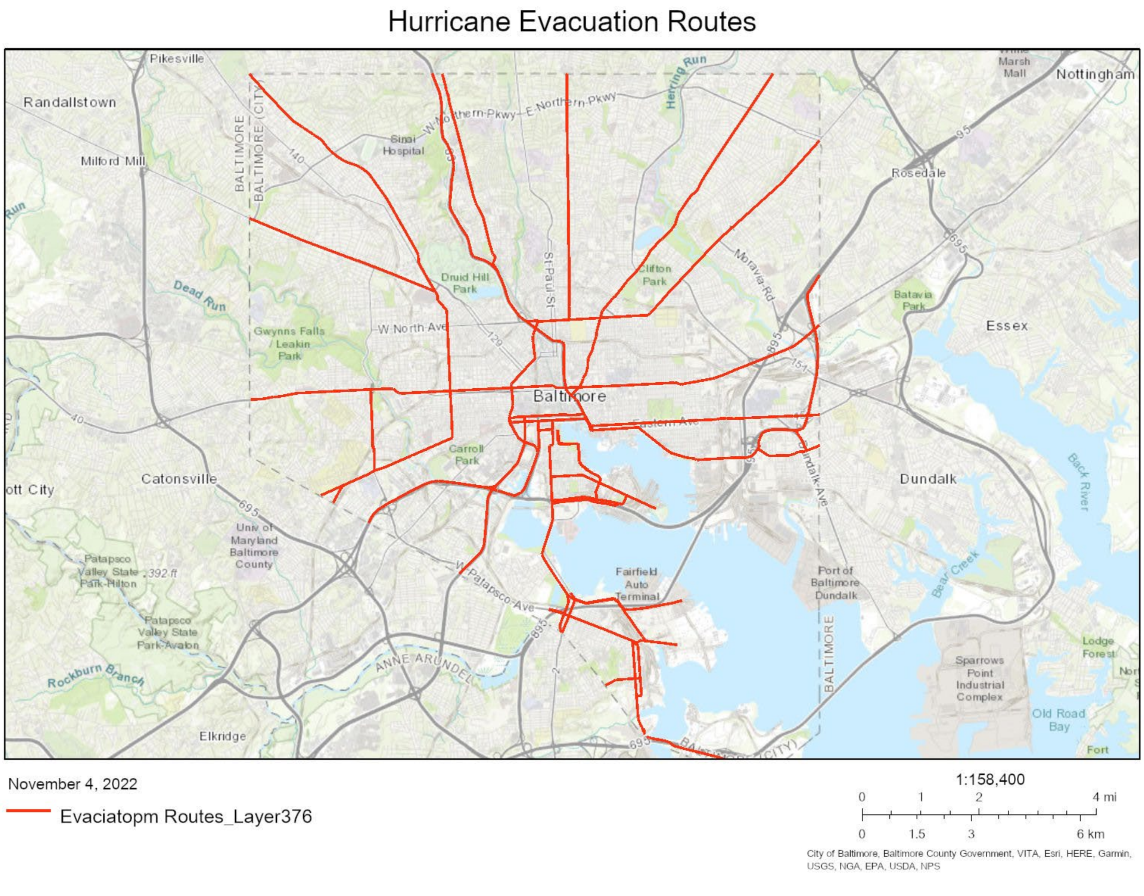


Figure 4.1 Hurricane Evacuation Routes

Based on the Maryland Transportation Authority's Flood Preparedness for the Fort McHenry and Baltimore Harbor Tunnels, tunnel closures in preparation for coastal events are based on water levels and begin at water level 6 feet, North American Vertical Datum, 1988 (NAVD88). Tunnel tubes are progressively closed as water levels increase with full closure of the Baltimore Harbor Tunnel at water level 8 feet, NAVD88. The Fort McHenry Tunnel begins closures at water level 8 feet, NAVD88 with tunnel use limited to emergency and responder traffic at water level 11 feet, NAVD88. The Fort McHenry Tunnel is closed to all traffic at water level 12 feet, NAVD88.

Due to the expected closure of the Baltimore Harbor Tunnel when the water surface elevation is approximately 4.2 feet below the top of the proposed floodwall, life loss in the Baltimore Harbor Tunnel is not expected.

The Fort McHenry Tunnel closures also begin when the water surface elevation is approximately 4.2 feet below the top of the proposed floodwall, the tunnel is closed to public use with approximately 1.2 feet remaining to top of wall and closed to all traffic when the water surface is near top of wall (~0.2 feet below top of wall). Because the Fort McHenry Tunnel is expected to be closed when the water surface is below the top of the proposed floodwall the likelihood of life loss is low.

It is expected that for life loss to occur, a breach of a floodwall would need to occur at water levels below the evacuation thresholds, while public or emergency response traffic is still allowed in the tunnels. This life safety risk will be evaluated in conjunction with future risk assessment efforts using more detailed consequence modeling.

5 Potential Failure Mode Analysis (PFMA)

A potential 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 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.

A PFMA was conducted by the following personnel:

Name	Role/Discipline	Organization
Troy Cosgrove, P.E.	Facilitator/Geotechnical Engineer	MVD Levee Safety Center, Branch Chief
Emily Calla, P.E.	Facilitator/Hydraulic Engineer	MVD Levee Safety Center
Richard Allwes, P.E.	Structural Engineer	Risk Management Center
Trent Porter, E.I.T.	Geotechnical Engineer	LRH-DSMMCX
Chun-Yi Kuo, P.E.	Geotechnical Engineer/PDT	Baltimore District
Dan Risley, P.E.	Hydraulic Engineer	Baltimore District/ Hydraulics and Hydrology Section Chief
Nestor Delgado-Velez, P.E.	Structural Engineer/PDT	Philadelphia District

On July 29, 2022, a scaled-down Potential Failure Mode Analysis (PFMA) was performed to inform the design of the structural measures consisting of floodwalls along the tunnel entrances on the I-895 and I-95 and their associated transportation critical facilities in Baltimore, MD. The scaled-down nature of the PFMA is used to meet project requirements while being commensurate with the size and scope of the study. No actual risks exist now because the project is in the preliminary design stage. The intention of the PFMA session is to mitigate future risk by identifying key items of concern that should be addressed during design and cost risks in development of the total project cost.

5.1 Design Background

The design at the time of the risk assessment was approximately 10% complete. The top of floodwall design is elevation 12.5 NAVD88 which was based on the 100-year water surface elevation with intermediate 2080 sea level rise at save point, North Atlantic Coast Comprehensive Study (NACCS) 5944, with the highest water surface elevation. This elevation is between the 50% confidence interval for events with return periods of 500 years and 1000 years. The length of the alignment was estimated utilizing LIDAR survey data.

The floodwalls considered for the protection of the I-95 and I-895 tunnels will be cast-in-place concrete T-walls. There are two types of T-walls being considered for the floodwall alignment, the primary difference is the height of the walls. Type 1 T-walls are taller and will be used along the tunnel entrances while Type 2 T-walls are shorter and will be used along the ventilation buildings. The floodwalls are expected to tie into high ground. Current design characteristics of each floodwall type are shown in Table 5-1 and current floodwall alignments are shown in Figure 5.1 and Figure 5.2. There do not appear to be any floodwalls along the eastern portal of the Fort McHenry tunnel. Additionally, there appear to be several areas in which construction of the floodwalls may be difficult due to space limitations caused by existing infrastructure in the area (McComas Street and existing retaining walls with possible tieback systems near the Fort McHenry tunnel).

Figure 5.1: Fort McHenry, I-95 tunnel, and West ventilation building

Figure 5.2: I-895 Tunnel and Ventilation Building Alignment

Table 5-1: T-wall type characteristics

Wall Type	Footing		Stem			Key	
	Width (ft)	Thickness (in)	Height (ft)	Thickness at Crest (in)	Thickness at Base (in)	Depth (ft)	Thickness (in)
1	11.5	18	8.2	12	18	2	12
2	6.67	14	5.2	10	14	1.5	12

The typical T-wall section does not include sheet pile cutoff for seepage mitigation. Splash pads were also not included in the typical section. It is uncertain whether the identification of these protective measures is required at this level of design. However, there are large portions of area on the landside of the alignment which will have concrete or asphalt on the land side. If these areas are expected to serve as splash pads, their thickness should be verified to ensure they are capable of withstanding the water-jet forces of the overtopping flows. The inclusion of a sheet pile cutoff would help to mitigate erosion of the foundation soil on the waterside of the T-wall and help limit uplift beneath the wall. An additional measure that may help improve the robustness of the design could come in the design of the concrete stems. Looking at minimum steel required in the design guidance and, if in excess of 1.33 of steel required, choose to use the minimum steel required. This ensures that if the concrete section cracks, the steel provided has the same moment capacity as an uncracked section. This is not a requirement and current design follows USACE guidance for hydraulic reinforced-concrete structures. However, this is not likely to create much of an additional cost and would further reduce risk associated with the project. There will also need to be gates designed to provide access to the ventilation buildings. The current design analysis assumed a worst-case scenario of water loading to the top of wall and wave force information was not provided. The Coast Storm Manual (v1 and v2) indicates that the following information should be provided to the structural engineers for a complete analysis of T-Wall stability:

- Breaking or non-breaking waves
- Wave frequency
- Wave height
- Depth of water

It should be ensured that uplift pressures are adequately accounted for stability during the design. Typically, when the wall base is not in 100% compression, it is assumed that there is full hydrostatic uplift for the portion of the base that is not in compression. When full hydrostatic uplift is taken into account, the base changes for the preliminary design from 78% compression to roughly 50% compression. This should be considered in design and preliminary cost estimates.

A subsurface site investigation has not been conducted at this time. Current analysis is based on available geotechnical data in the vicinity of the study area. The current design has been analyzed for global stability with water loading to the top of floodwall and the stability analyses need to be reevaluated because of uplift when the bases are not in 100 percent compression. The current subsurface profile (Figure 3.1) indicates that the foundation beneath the floodwall will have fill composed of dredging material. Typical dredged material includes sandy silt. Beneath the fill materials are alluvial materials composed of clayey sands and silty sands which are granular.

There is some potential for floodwall impact by vessels, barges, or floating cars. There are a large number of ships and barges in the areas. However, a majority of the ship traffic draft in excess of 30 feet and may run aground prior to impacting a floodwall. Personal watercrafts and other smaller vessels are most likely to impact the floodwalls. However, there is potential for barge impact due to barge's drafting near 8 feet if loaded and less if unloaded. A coal terminal facility is in the vicinity of the project which could cause loaded and unloaded barges to be present. There is likely a SOP for mooring or removing all watercraft other than personal craft which will help limit the potential for floodwall impact. There is also some potential for floating cars or other vehicles to impact the wall. There is a parking lot near the proposed alignment at the Harbor Tunnel where imported/exported cars are temporarily stored. There is also a vehicle parking and storage facility near the Fort McHenry Tunnel. It is expected that many of these vehicles will be moved in advance of the storm limiting the potential for impact with a floodwall.

5.2 Brainstorming PFMs

- PFM 01: Overtopping (OT) due to Stillwater and waves of Floodwall Leads to Scour, Erosion, Undermining, and Instability (Floodwalls founded on soil – no scour pad) Leads to Breach.
- PFM 02: Sliding Instability of Floodwall Leads to Breach.
- PFM 03: Overturning Instability of Floodwall Leads to Breach.
- PFM 04: Floodwall Instability due to Bearing Pressures Exceed Soil Bearing capacity which Leads to Breach.
- PFM 05: Global Instability of Floodwall (Foundation soil rotational failure) Leads to Breach.
- PFM 06: Concentrated Leak Erosion (CLE) between the Floodwall Base and Soil Interface Leads to Breach.
- PFM 07: CLE through the Floodwall/High Ground Tie In Leads to Breach.
- PFM 08: Wave and/or Stillwater overtopping at the floodwall tie-in leads to breach.
- PFM 09: Backward Erosion Piping (BEP) through the Floodwall/High Ground Tie In Leads to Breach.
- PFM 10: Floodwall Stem Failure Leads to Breach.
- PFM 11: BEP through the foundation of the Floodwall Leads to Breach.
- PFM 12: Floodwall Stem Failure due to Debris/Vessel Impact Loading Leads to Breach.
- PFM 13: Failure of gate/closure used as access to ventilation building for Fort McHenry Tunnel.
- PFM 14: Failure of gate/closure used as access to ventilation building for Harbor Tunnel.
- PFM 15: Failure of gate/closure used as access to ventilation building for Harbor Tunnel (Eastern Side).
- PFM 16: Operational Failure of gates/closures at ventilation buildings and tunnels.
- PFM 17: CLE Along Utilities/pipelines under floodwall Leads to Breach.
- PFM 18: CLE along unknown/undocumented utility/pipeline under floodwall leads to breach.
- PFM 19: Electrical/mechanical failure leads to inoperability of gates at ventilation buildings and tunnels.
- PFM 20: Continuous wave overwash of floodwalls leads to inundation of ventilation buildings.
- PFM 21: Continuous wave overwash of floodwalls leads to inundation of tunnels.
- PFM 22: Reverse loading of floodwall leads to rotation/failure of floodwall.
- PFM 23: Failure of pressurized pipeline leads to undermining and collapse of floodwall.
- PFM 24: Stormwater or sewer system allows for water to enter the protected area from the flood side.
- PFM 25: Debris/Vessel Impact Loading Leads to Floodwall Instability and Breach.

5.3 Evaluating PFMs

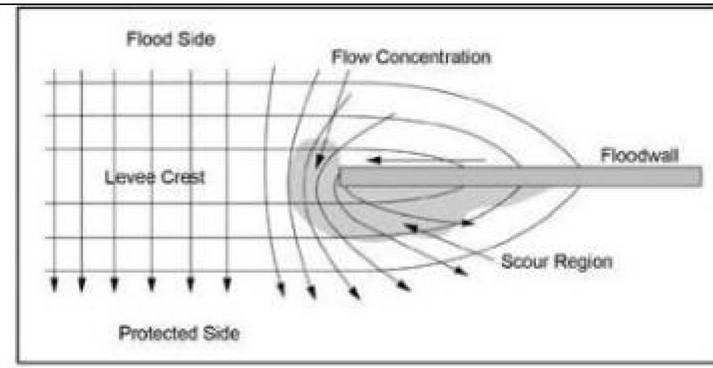
Many of the brainstormed PFMs are typically managed with designed defensive measures, adhering to published engineering standards, construction Quality Assurance (QA), or Emergency Action Plans (EAP). A more thorough risk assessment (i.e., Semi-Quantitative Risk Assessment – SQRA) will occur during the pre-construction engineering and design (PED) phase of the project.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 01: Overtopping of Floodwall due to Stillwater and Waves Leads to Scour, Erosion, Undermining, Instability, and Breach (Floodwalls founded on soil – no scour pad)	This failure mode covers both wave and stillwater overtopping of the floodwall. Depth of overtopping and duration are key factors. Soil type (clay versus sand) and vegetation cover will impact erodibility. Preliminary design – no splash pad to dissipate force and energy of impinging jet and prevent scour. Seepage pressures and uplift are unknown.	Splash pad on landside of T-wall would mitigate some of these issues and would reduce the amount of scour on the landside of the T-wall. Develop wave overtopping and design wall to get the wall wave overtopping rate to be 0.1 cfs/ft at the design storm for the 90% confidence interval to minimize overwash and interior flooding (HSDRRS guidance for T-walls).	Depth of overtopping and duration are key factors. Do not have a good understanding of wave overtopping at this point. Currently, subsurface information is unavailable and leads to uncertainty in soil types and their erosion resistance.	Walls are designed for 100-yr storm at 95% confidence interval; a larger storm may result in much larger overtopping and wave heights. Development of interior tailwater may reduce scour on landside to make breach less likely. Pumps may be operating at the ventilation buildings and would reduce the amount of water accumulating at the interior of the floodwall. The tunnels have their own pumping systems. Many buildings have parking lots and other hard surfaces that will increase water accumulation within the floodwall-protected area and will potentially increase uplift pressures on the floodwall. In some locations, these hard surfaces will slow overwash erosion rates on the land side and act as a splash pad.
PFM 02: Sliding Instability of Floodwall Leads to Breach	Wave loadings (breaking or nonbreaking) would increase lateral pressure on the floodwalls.	A key is being provided at the heel of the base slab of the floodwalls and will reduce the likelihood of sliding. A sheet pile cutoff wall will reduce underseepage and uplift.	Currently no subsurface information has been obtained leading to uncertainty in soil types.	Walls are designed for 100-yr storm at 95% confidence interval; a larger storm may result in much larger overtopping and wave heights. Geotechnical design should not account for the pressures from the soils.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
	Overtopping depth would result in more load on the floodwalls. Seepage pressures and uplift are unknown. At this point only preliminary design is completed and there is no subsurface information.	Sheet pile cutoff wall would provide some lateral resistance. Floodwall will be designed and analyzed during design to ensure meeting proper factors of safety for stability.	Do not know if the walls will be buried in soil or how much backfill will be placed on the protected side. If the walls are constructed on an existing hardened surface do not know how the walls will be anchored.	The deadweight of the soil should not be accounted for in the stability analysis of the floodwall if the soil could be washed away during the storm. Many buildings have parking lots and other hard surfaces that will increase water accumulation within the floodwall area and will potentially increase uplift pressures on the food wall.
PFM 03: Overturning Instability of Floodwall Leads to Breach	Wave loadings (breaking or nonbreaking) would cause more lateral pressure on the floodwalls. Overtopping depth would result in more load on the wall. Seepage pressures and uplift are unknown. At this point only preliminary design is completed and there is no subsurface information.	A sheet pile cutoff wall would reduce underseepage and uplift. Floodwall will be designed and analyzed during design to ensure meeting proper factors of safety for stability.	Currently no subsurface information has been obtained leading to uncertainty in soil types.	Design should account for 100% uplift for the portion of the base that is not in compression. The hydrostatic pressure and uplift on the key should be included in the stability calculations. Many buildings have parking lots and other hard surfaces that will increase water accumulation within the floodwall area and will potentially increase uplift pressures on the food wall.
PFM 04: Floodwall Instability due to Bearing Pressures Exceed Soil	Wave impacts would cause more lateral pressure on the walls.	A sheet pile cutoff wall would reduce underseepage and uplift.	Currently no subsurface information has been obtained	Design should account for 100% uplift for the portion of the base that is not in compression.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
Bearing capacity which Leads to Breach	Overtopping depth would result in more load on the wall. Seepage pressures and uplift are unknown. At this point only preliminary design is completed and there is no subsurface information.	Floodwall will be designed and analyzed during design to ensure meeting proper factors of safety for stability.	leading to uncertainty in soil types. Bearing capacity of soils is unknown.	The hydrostatic pressure and uplift on the key should be included in the stability calculations. Generally when geotechnical engineers calculate bearing capacity of soil it is considered saturated; likely for a hurricane event the soils would be in a transient seepage state not a steady seepage state.
PFM 05: Global Instability of Floodwall (Foundation soil rotational failure) Leads to Breach	Rapid Drawdown leads to global instability. There could be a failure plane into the tunnel entrance/exit portal or slope. At this point only preliminary design is completed and there is no subsurface information.	Foundation will be analyzed during design to ensure meeting proper factors of safety for stability.	Soils may or may not be saturated depending on the duration of loading. Do not know what the natural groundwater elevation is; there could be high groundwater. May require multiple storms for rapid drawdown to result in consequences due to foundation failure. There may be tie-back walls on the retaining walls of the tunnels.	There could be surcharge on the entrance wall due to loading on structure that was not included in original design. Evaluate existing slopes and retaining structures for additional surcharge loadings from floodwall construction and coastal water loading. For a hurricane event, the soils would likely be in a transient seepage state not a steady seepage state.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 06: CLE between the Floodwall Base and Soil Interface Leads to Breach	Seepage develops along the interface between the floodwall and foundation during coastal loading. There is a 6-ft differential across the wall as currently modeled. Typically, duration of loading is short. Likely not a high head differential.	Good construction practices would limit the existence of the flaw. A sheet pile cutoff wall would reduce the likelihood of flaw existing and act as a flow limiter. Sand filter would prevent progression of CLE.	Do not have information on foundation soils. Duration of loading and potential exit for the CLE is unknown due to lack of details at this point. Head differential across the wall may not result in sufficient gradient to move material. Groundwater elevation may be high due to Patapsco River; the initial analysis has the groundwater surface 1-ft below the soil.	Preliminary design – sheet pile cutoff can be included to control underseepage or provide scour protection for the foundation, if necessary Many buildings have parking lots and other hard surfaces that will increase water accumulation within the floodwall area. Accumulation of water in the floodwall area may lower head differential across the wall.
PFM 07: CLE through the Floodwall/High Ground Tie-In Leads to Breach	Differential Settlement- creates flaw. Flaw could also result from poor compaction. Clay could dry and open a crack (desiccation cracking) but that would be more likely in hot, dry areas.	Typically batter concrete walls. Ensure specifications address proper compaction at tie-in. Specifications need to cover how the slope at tie-in will be laid back prior to construction of the floodwall. The excavation needs to be made to	Do not have subsurface investigation information, do not have information on engineered fill that will be brought to the site.	Likely the tunnel floodwalls will extend to a taper and not tie into high ground.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
	Typically soils near coastline are softer and could result in settlement. Typically, duration of loading is short. Likely not a high head differential.	ensure there is room to properly compact the material. Specifications need to address potential winter shutdown/construction. Lengthen embedment of the floodwall into the high ground to lengthen the seepage path.		
PFM 08: Wave and/or Stillwater overtopping at the floodwall tie-in leads to breach.	Wave overtopping of the embankment to floodwall tie-in. The preliminary design does not address the tie-in of the floodwalls or if any treatment is needed. How well vegetated the embankment is, type of soil, duration of loading and overtopping, velocity of flow, slope of the embankment.	Analyze tie-ins for overtopping potential. Ensure slopes have proper grass cover, turf reinforcement mat or other armoring if needed. Do not allow installation of utility poles or other possible knickpoint. Ensure specifications address proper compaction at tie-in. Specifications need to cover how the slope at tie-in will be laid back prior to construction of the floodwall. The excavation needs to be made to ensure there is room to properly compact the material. Specifications need to address potential winter shutdown/construction.	Do not have subsurface investigation information, do not have information on engineered fill that will be brought to the site. No details have been developed for the tie-in design.	 Figure 12-4. Diagram of Overtopping Erosion at Levee - Floodwall Transition. The image above depicts a potential flow path although the levee crest on the image would be a high ground tie-in. Ensure no trees are permitted near this area. Ensure O&M plan includes prevention of animal burrowing activities and treatment of animal burrows if observed.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 09: BEP through the Floodwall/High Ground Tie-in Leads to Breach	If a sand layer exists within tie-in with floodwall and high ground, there is the potential for BEP to occur. Typically, duration of loading is short. Likely not a high head differential.	Ensure specifications address proper compaction at tie-in. Specifications need to cover how the slope at tie-in will be laid back prior to construction of the floodwall. The excavation needs to be made to ensure there is room to properly compact the material. Specifications need to address potential winter shutdown/construction. Lengthen embedment of the floodwall into the high ground to lengthen the seepage path. Depending on extents of sand layer, over-excavation and/or a sheet pile cutoff may be necessary.	Do not have subsurface investigation information, do not have information on engineered fill that will be brought to the site. No details have been developed for the tie-in design.	Likely the tunnel floodwalls will extend to taper and not tie into high ground.
PFM 10: Floodwall Stem Failure Leads to Breach	Wave impacts would cause more lateral pressure on the walls. Overtopping depth would result in more load on the wall.	Floodwall will be designed and analyzed during design to ensure meeting USACE hydraulic reinforced-concrete design guidance and American Concrete Institute (ACI) 318-14.	Corrosion of flexural reinforcement.	Epoxy coated rebar.
PFM 11: BEP through the foundation of the Floodwall Leads to Breach	Concrete or clay layer could serve as a roof.	A sheet pile cutoff would eliminate this PFM.	Do not have subsurface investigation information.	Preliminary design – Preliminary design – sheet pile cutoff can be included to control underseepage or provide scour protection for the foundation, if necessary.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
	Would need a continuous layer of fine, clean sand. Silt sized particles could allow BEP to develop but need an unfiltered exit. Typically, duration of loading is short. Likely not a high head differential. This PFM includes both a heave/blowout and/or daylight condition.			Typical sands in the area are not expected to be pipeable.
PFM 12: Floodwall Stem Failure due to Debris/Vessel Impact Loading Leads to Breach	Numerous terminals and docks in the area. Pleasure craft in the area. Debris could also include automobiles. Hurricane Agnes photos show numerous cars floating in the river.	Floodwall will be designed and analyzed during design to ensure meeting USACE hydraulic reinforced-concrete design guidance and ACI 318-14. Establish appropriate design criteria for vessel/debris impact loading [Hurricane and Storm Damage Risk Reduction System - Design Guidelines (Interim, March 2012)].	Coast Guard requirements for mooring vessels prior to coastal events. Severity of the impact loading.	
PFM 13: Failure of gate/closure used as access to ventilation building for Fort McHenry Tunnel.	Preliminary design does not include any closures, however, access to the ventilation buildings and tunnel areas will be needed and some type of	Closure/gate will be designed and analyzed during design to ensure meeting proper factors of safety.	No preliminary design available to evaluate. Cost may be higher for closure gate than	Closures should be able to be closed by one or two people without special equipment to minimize something mechanically or electrically misoperating (such as swing gate or roller gate). Closure will require substantial structure on each end (abutments) to withstand the thrust; this may need a more significant foundation design/treatment such as deep foundations to transfer the load.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
	closure system will need to be designed.		floodwall section.	Ability to access the ventilation buildings with large equipment that may dictate the size of the gate.
PFM 14: Failure of gate/closure used as access to ventilation building for Harbor Tunnel.	Preliminary design does not include any closures, however, access to the ventilation buildings and tunnel areas will be needed and some type of closure system will need to be designed.	Closure/gate will be designed and analyzed during design to ensure meeting proper factors of safety.	No preliminary design available to evaluate. Cost may be higher for closure gate than floodwall section.	Closures should be able to be closed by one or two people without special equipment to minimize something mechanically or electrically misoperating (such as swing gate or roller gate). Closure will require substantial structure on each end to withstand the thrust; this may need a more significant foundation design/treatment such as deep foundations to transfer the load. Ability to access the ventilation buildings with large equipment that may dictate the size of the gate.
PFM 15: Failure of gate/closure used as access to ventilation building for Harbor Tunnel (Eastern Side)	Preliminary design does not include any closures, however, access to the ventilation buildings and tunnel areas will be needed and some type of closure system will need to be designed.	Closure/gate will be designed and analyzed during design to ensure meeting proper factors of safety.	No preliminary design available to evaluate. Cost may be higher for closure gate than floodwall section.	Closures should be able to be closed by one or two people without special equipment to minimize something mechanically or electrically misoperating (such as swing gate or roller gate). Closure will require substantial structure on each end to withstand the thrust; this may need a more significant foundation design/treatment such as deep foundations to transfer the load. Ability to access the ventilation buildings with large equipment that may dictate the size of the gate.
PFM 16: Operational Failure of gates/closures at ventilation buildings and tunnels.	Preliminary design does not include any closures, however, access to the ventilation buildings and tunnel areas will be needed and some type of closure system will need to be designed.	Ensure the O&M manual accounts for proper timing of making the closures and ensuring safety of employees and structures from a water loading standpoint and storm intensity standpoint (such as wind speeds).	Experience of personnel in making these types of closures in preparation of or during a coastal event. Pandemic impacting availability of workforce to set closures due to potential illness.	Consider interior drainage within the floodwall area due to rainfall; pumps will need to be properly maintained, fuel for generator and pumps. There may be competing priorities in setting closures with limited staffing available.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 17: CLE Along Utilities/pipelines under floodwall Leads to Breach.	This PFM covers CLE along an existing pipe that is impacted during coastal loading. Typically duration of loading is short. Likely not a high head differential. Utility or pipeline would need to be exposed for there to be an exit point.	Performing seepage analysis along the utility/pipelines to determine if any remedial measures are needed. Remedial measures may include filters on the protected side. Perform ground penetrating radar to confirm location and alignment of utilities/pipelines.	No details concerning utilities were provided due to the preliminary nature of the design.	May need to pass the utility through the sheetpile cutoff wall stem, relocated the utility, or construct utility corridors for passing the utilities through the wall.
PFM 18: CLE along unknown/undocumented utility/pipeline under floodwall leads to breach.	Typically, duration of loading is short. Likely not a high head differential. Utility or pipeline would need to be exposed for there to be an exit point. This could include unknown, abandoned utility.	Perform ground penetrating radar to determine if there are unknown/undocumented utilities or pipelines.	Unsure of available documentation on existing utilities/pipelines.	Installation of sheetpile cutoff wall would find undocumented utilities or pipelines. Excavation could also result in finding undocumented utilities or pipelines.
PFM 19: Electrical/mechanical failure leads to inoperability of gates at ventilation buildings and tunnels.	Preliminary design does not include any closures, however, access to the ventilation buildings and tunnel areas will be needed and some type of closure system will need to be designed.	Periodic exercising of the gates to ensure operability. Maintaining essential spare parts. Develop plan to exercise gates days in advance of coastal loadings to ensure gates are able to operate properly.	No preliminary design available to evaluate.	Closures should be able to be closed by one or two people without special equipment to minimize something mechanically or electrically misoperating (such as swing gate or roller gate).

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
		Ensure personnel or contract is in place to properly maintain the structures.		
PFM 20: Continuous wave overwash of floodwalls leads to inundation of ventilation buildings.	Volume of overwash would need to exceed the capacity of the pumps for the ventilation buildings.	Evaluate the need for temporary or permanent pumps to properly evacuate wave overtopping/overwash.	Unsure of pumping capacity. Unsure of wave overtopping volume. No preliminary design available to evaluate.	Consider interior drainage within the floodwall area due to rainfall; pumps will need to be properly maintained, fuel for generator and pumps.
PFM 21: Continuous wave overwash of floodwalls leads to inundation of tunnels	Volume of overwash would need to exceed the capacity of the pumping system for the tunnels. During a storm there will be increased volume of water entering the tunnels through seepage.	Evaluate the capacity of the existing pumping system and estimate wave overtopping along reach of floodwalls to determine if auxiliary pumps are required.	Unsure of pumping capacity of tunnel system. Unsure of wave overtopping volume.	May want resiliency/excess capacity in pumping system.
PFM 22: Reverse loading of floodwall leads to rotation/failure of floodwall.	During an extreme event, large volume of wave and/or Stillwater overtopping occurs and fills the enclosed protected area, followed by rapid drawdown and the floodwaters are maintained within the protected area.	Temporary or permanent interior pumping to remove overwash/overtopping and rainfall within the protected area. Properly design the floodwalls to minimize wave overtopping which will reduce the volume of overwash into the area.	Unsure if all areas will include pumps to reduce likelihood of reverse loading. How rapid of a drawdown would be required for this to occur.	Ensure the stormwater system (drains/manholes) does not allow water to enter the protected area from the flood side of the wall.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
	Due to the small confined area the problem could be exacerbated due to small volume required to fill the area. There will not be infiltration into the ground due to asphalt/concrete in the majority of the protected areas.		Unsure of wave overtopping volume.	
PFM 23: Failure of pressurized pipeline leads to undermining and collapse of floodwall.	High pressure pipeline that fails due to corrosion or differential settlement. There are aged pipes in the region.	Evaluate the high-pressure pipelines in the area to determine their age and condition. There could be a lined sleeve for pressurized pipes that run under the floodwall.	No details concerning high pressure pipelines were provided due to the preliminary nature of the design.	This is not something that is covered by the design of the floodwall; there would need to be a mitigation plan to address this PFM.
PFM 24: Stormwater or sewer system allows for water to enter the protected area from the flood side.	Floodwaters are permitted to flow uncontrollably through stormwater/sewer lines that back up into the protected area causing impacts to the protected area. Drains under the floodwall may not be able to withstand the pressure of the water resulting in water flowing into the protected area.	Survey the area for stormwater/sewer systems, assess the system and provide closures as needed such as valves.	No preliminary assessment of stormwater/sewer system available to evaluate.	

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 25: Debris/Vessel Impact Loading Leads to Floodwall Instability and Breach	Numerous terminals and docks in the area. Pleasure craft in the area. Debris may also include automobiles. Hurricane Agnes photos show numerous cars floating in the river.	Floodwall will be designed and analyzed during design to ensure meeting USACE hydraulic reinforced-concrete design guidance and ACI 318-14. Establish appropriate design criteria for vessel/debris impact loading.	Coast Guard requirements for mooring vessels prior to coastal events. Severity of the impact loading.	
Seismic Related Potential failure modes	Seismic related potential failure modes have been excluded from consideration. An explanation of the exclusion can be found in section 3.2 of this report.			

While none of the failure modes evaluated stood out as particularly “risk driving”, these failure modes should and will be considered during design of the project and will be re-evaluated once the design is more substantial.

6 Typical Risks

Since the designs are still relatively conceptual in nature a more rigorous risk assessment (e.g., Semi-Quantitative Risk Assessment, Quantitative Risk Assessment) was not performed at this point. Having subsurface data and design at least at the 35-65% level would reduce the uncertainties to the point that the risk assessment may further inform what measures will be needed to ensure compliance with USACE Levee Safety guidelines, so that incremental risks are properly mitigated and managed as low as practicable.

7 Key Limitations

The limitation of the PFMA session and any risk analysis methodology is primarily driven by the availability and the completeness of the information used to assess the risk. With due regards for uncertainty at this point it is recommended that further design is conducted and that at least an SQRA session is completed between the 35-65% design level.

The methodology for the scaled down PFMA seems appropriate for this level of study. It identifies the potential for risks but cannot fully quantify the risk until more information is available on the design and existing conditions.

8 Conclusions

The risk assessment team proposed several recommendations that may dramatically influence the estimated cost of the floodwalls. These recommendations are based on preliminary design and limited information available at the feasibility level; the recommendations do not influence the alignment of the proposed floodwalls. While life loss is unlikely based on the expected tunnel closures, economic consequences could be significant if the tunnels and/or ventilation buildings are inundated. The teams' assessment of low likelihood of life loss is based on the tunnel closure plan provided by the Maryland Transportation Authority, if the tunnels are not closed to traffic as outlined in the flood preparedness plan the likelihood for life loss increases.