4. Structural Assessment

4.1. Introduction

Structural components of the Seawall include Bulkhead Walls ("Bulkhead"), Marginal Wharves ("Bulkhead Wharves" or "Wharves"), and Finger Piers ("Piers"). Other structures supported by or influenced by the Seawall include Bulkhead Buildings, Pier Sheds, and miscellaneous buildings located over or in close proximity to the Seawall. For this study, Bulkhead Wall and Wharf structures are assessed at a screening level by analyzing selected structures and using engineering judgment to characterize expected results for the non-analyzed structures. Piers, sheds, bulkhead buildings and miscellaneous buildings were not analyzed, however, damage to those structures can be assumed to be at least similar to damage levels of supporting structures for purposes of economic analysis.

<u>Structural Assessment of Bulkhead Walls</u> – Forty (40) distinct Bulkhead Wall types were assessed for structural capacity and checked for stability against sliding and overturning. Load cases included static only and static plus five different levels of earthquake loading. Results are reported as Demand/Capacity Ratio's (DCR's) and structural criticality ratings (defined in Section 4.2.8) assigned to each bulkhead type.

The assessment indicates that all Bulkheads are acceptable for static loads, but many were found to be inadequate under varying levels of earthquake load. The number of deficient bulkhead types under analyzed earthquake scenarios is as follows:

- Static load only – 1 (maximum DCR = 1.04)
- Static plus M8.0 San Andreas (median) - 13 (maximum DCR = 2.24)
- Static plus DE (2/3 MCE per 2013 CBC) - 13 (maximum DCR = 2.34)
- Static plus 475 year return period (10%/50yr) - 15 (maximum DCR = 2.66)
- Static plus 975 year return period (5%/50yr) - 17 (maximum DCR = 2.88)
- Static plus MCE (per 2013 CBC) - 20 (maximum DCR = 3.19)

For the most severe seismic loading considered in this study, approximately one-half of the seawall (or 1-1/2 miles) includes bulkheads that are vulnerable to damage. Typical vulnerabilities include failure of concrete and timber piles supporting the bulkhead structure. Bulkheads with identified deficiencies will be considered for retrofit alternatives in Phase 3.

<u>Structural Assessment of Bulkhead Wharves</u> – Advanced screening level structural assessments were completed for five (5) wharves using current displacement based pushover analysis techniques and typical cross sections. Marginal wharfs between Piers 29-31, near Pier 17, near Pier 9, between Piers 26 and 28, and near Pier 38 were selected for analysis. Three earthquake levels were analyzed (72-yr, M8.0 San Andreas median and 975-yr) and results were extrapolated for other wharves using engineering judgment. The analysis focused on the piles only as these were deemed the most critical components for a screening level study, and included load cases both with and without movement of the rock seawall (kinematic loading) to assess impact of seawall movement on structural vulnerability.

In general, bulkhead wharf vulnerabilities consist of non-ductile pile connections to the wharf deck for seismic inertial loading and fracture of the piles below the rock dike under soil lateral sliding (kinematic loading.)

4.2. Structural Assessment Methodology and Criteria

The methodology and criteria used for the structural assessment of the bulkhead wall and associated bulkhead wharfs are presented in this section. Finger pier structures were not a part of this assessment and the performance of finger piers will likely impact the predicted performance of the bulkhead wharf structures.

The bulkhead generally consists of four types of wall: unreinforced concrete bulkhead walls, retaining walls, reinforced concrete cutoff walls, or timber cutoff walls. Descriptions of the various bulkhead types are provided in the following sections. Some portions of the bulkhead are structurally connected to an existing wharf structure. The as-built data for all the bulkhead sections and types were developed during Phase 1 of this study. These data used in this Phase 2 assessment are shown on Figure 4-1.

The bulkhead walls themselves will be assessed for performance items such as stability and lateral displacement. The wall components will be assessed for strength capacity. The wharf piles are assessed for lateral load capacity under earthquake loads where the wharf structure is deemed to provide lateral load resistance to the bulkhead wall.

Based on the results of these assessments, each wall will be assigned a structural rating relative to its performance and strength characteristics. This rating will be the basis for ranking each bulkhead wall section from a structural point of view.

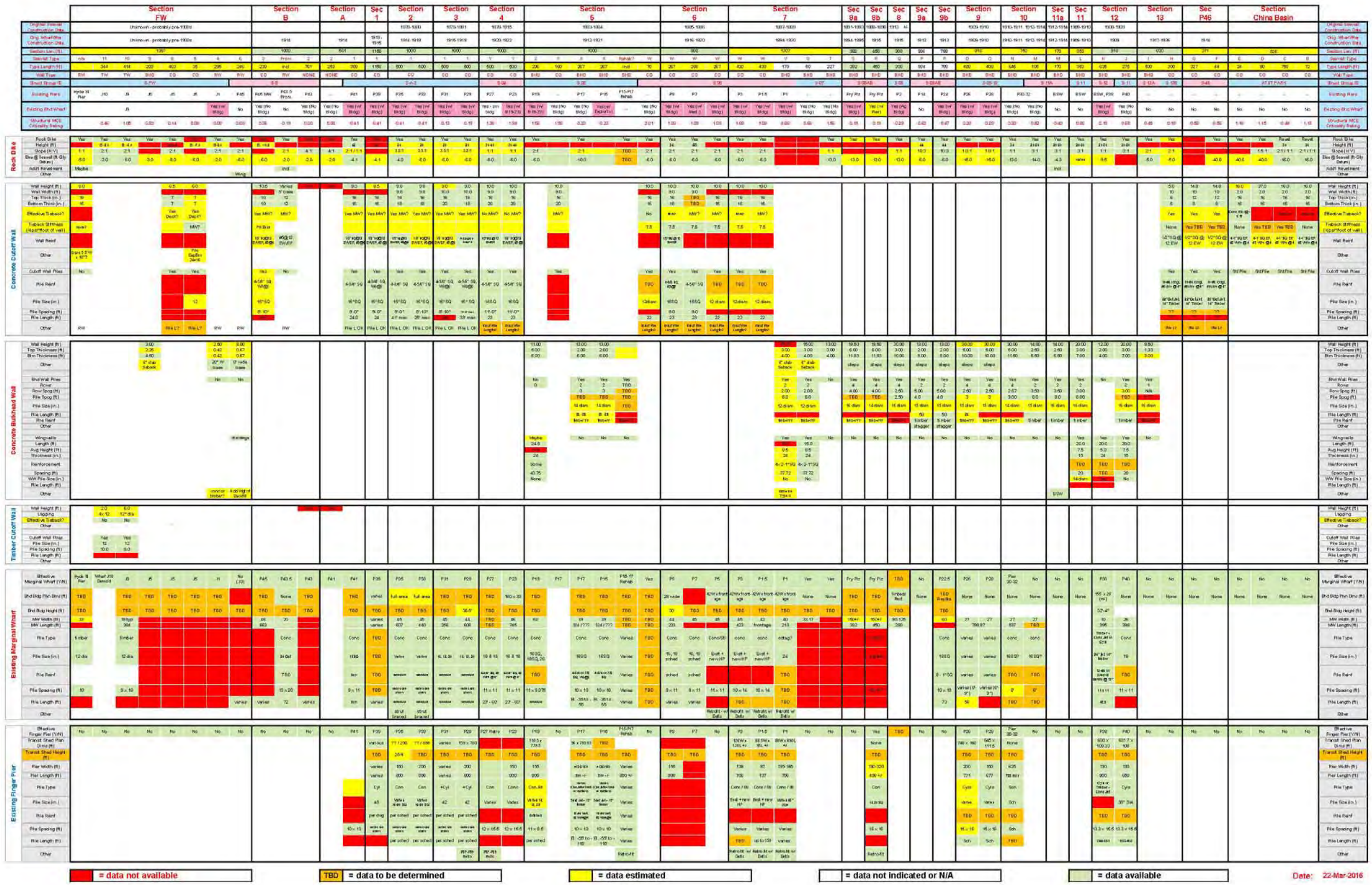


Figure 4-1: Seawall Section and Bulkhead Type Assessment Data Summary

Assessment of Concrete Bulkhead Walls

Concrete bulkhead walls, and their associated components, are assessed for wall stability and structural component strength. The bulkhead wall components typically consist of unreinforced concrete of varying height and thickness. The bulkhead wall is typically founded on a rock dike of varying height, top width, base width and side slopes. Some bulkhead walls are supported on single or multiple rows of timber piles at varying transverse and longitudinal spacing. Some bulkhead walls are supplemented with attached concrete wing walls.

Bulkhead wall stability assessment considers lateral sliding and overturning moment stability. The factor of safety (FOS) for the design basis loading conditions is determined for each wall type considered. The minimum allowable FOS for static loading is 1.5. The minimum allowable FOS for load conditions that include seismic loads is 1.0.

The structural strength design capacity ratio (DCR) is determined for the following bulkhead wall components:

- Bulkhead wall unreinforced concrete
- Bulkhead timber piles, where applicable.
- Wing wall unreinforced or reinforced concrete, where applicable.
- Wing wall timber piles, where applicable
- Marginal wharf timber or concrete piles, where applicable.

Assessment of Concrete Retaining Walls

Concrete retaining walls, and their associated components, are assessed for wall stability and structural component strength.

Similar to the bulkhead walls, retaining wall stability assessment considers lateral sliding and overturning moment stability. The factor of safety (FOS) for the design basis loading conditions is determined for each wall type considered. The minimum allowable FOS for static loading is 1.5. The minimum allowable FOS for load conditions that include seismic loads is 1.0.

The structural strength design capacity ratio (DCR) is determined for the following retaining wall components:

- Retaining wall unreinforced or reinforced concrete
- Wing wall unreinforced or reinforced concrete, where applicable.
- Marginal wharf timber or concrete piles, where applicable.

Assessment of Concrete Cutoff Walls

Concrete cutoff walls, and their associated components, are assessed for wall displacement and structural component strength. The concrete cutoff wall components typically consist of reinforced concrete wall panels of varying height and thickness supported by reinforced concrete plumb piles.

Bulkhead wall displacements at the top of the cutoff wall and at the depth of maximum lateral displacement are determined.

The structural strength design capacity ratio (DCR) is determined for the following bulkhead wall components:

- Cutoff wall reinforced concrete

- Cutoff wall concrete piles, where applicable.
- Marginal wharf timber or concrete piles, where applicable.

Assessment of Timber Cutoff Walls

Timber cutoff walls, and their associated components, are assessed for wall displacement and structural component strength. The timber cutoff wall components typically consist of timber lagging of varying height and size supported by timber plumb piles. The cutoff wall is typically founded on revetment or a rock dike of varying height, top width, base width and side slopes.

Bulkhead wall displacements at the top of the cutoff wall and at the depth of maximum lateral displacement are determined.

The structural strength design capacity ratio (DCR) is determined for the following bulkhead wall components:

- Cutoff wall timber lagging
- Cutoff wall timber piles.
- Marginal wharf timber or concrete piles, where applicable.

Assessment of Marginal Wharf Piles

Pushover analyses were performed for selected marginal wharf piles to determine the lateral load versus displacement response for use in bulkhead and cutoff wall assessment. The lateral load capacity of marginal wharf piles is relatively small for pile displacements that are compatible with acceptable structural performance. Marginal wharf piles provide little contribution to the overall structural performance of the seawall.

The finger piers were not part of this assessment and their seismic performance will likely impact the predicted performance of the bulkhead wharf structures.

Record drawings were reviewed to obtain data for marginal wharf analyses. Marginal wharfs between Piers 29-31, near Pier 17, near Pier 9, between Piers 26 and 28, and near Pier 38 were selected for analysis. The analyses considered design basis seismic inertial loads due to the 72-yr, M8.0 San Andreas and 975-yr earthquakes. Design basis soil lateral sliding was also analyzed for the same marginal wharf structure and design basis seismic events.

Assessment Criteria

Assessment criteria were developed for various loading types applicable to the seawall assessment. These load types are dead, live, soil and earthquake induced loads.

<u>Dead Load</u>

Dead loads are due to self-weight of the seawall structure in addition to any supported structures or equipment. Material weight densities are taken as follows:

- Concrete 150 pcf
- Steel 490 pcf
- Timber 60 pcf
- Soil per geotechnical recommendations

Live Load

Live loads are applied where dictated by area use. A 250 psf maximum live load is applied based on the presence of emergency vehicles near the seawall.

Soil Load

Soil loads consisting of soil active, passive, surcharge and seismic loads are provided by GTC, as presented in Section 2. Load values are presently assumed the same regardless of seawall section.

Earthquake Load

Earthquake ground motion accelerations for five levels of seismic activity are provided by GTC, as presented in Section 2. Load values vary with seawall section and are summarized in the structural assessment subsection for each seawall section. Tables 4-1 and 4-2 present the ground motion accelerations and associated induced seismic pressures, respectively, as a function of seawall location and earthquake event.

Table 4-1: Earthquake Induced Peak Ground Accelerations

Geotech			PGA @ EQ			
Class	72YR	M8.0 SA	DE	475YR	975YR	MCE
A1	0.29	0.37	0.39	0.46	0.51	0.58
A2	0.31	0.37	0.39	0.46	0.51	0.58
B1	0.31	0.37	0.38	0.42	0.44	0.45
B2	0.31	0.37	0.38	0.42	0.44	0.45
C1	0.34	0.37	0.37	0.39	0.41	0.42
C2	0.34	0.37	0.37	0.39	0.41	0.42

Table 4-2: Earthquake Induced Seismic Soil Pressures

Geotech			SEISMIC EARTH PRESSURE / HGT @ EQ			
Class	72YR	M8.0 SA	DE	475YR	975YR	MCE
A1	6	7	8	10	11	13
A2	6	7	8	10	11	13
B1	6	7	8	9	9	9
B2	6	7	8	9	9	9
C1	7	7	7	7	8	9
C2	7	7	7	7	8	9

Design Basis Loads

Design basis loads for this structural strength assessment are based on ultimate limit state (ULS) design and consist of combinations of dead, live, soil and earthquake loads. These combinations, per ASCE 7-10, Section 2.3.2, applicable to this study are:

1. 1.4D + 1.6H (ASCE 7-10, 2.3.2, Equation 1)
2. 1.2D + 1.6L + 1.6H (ASCE 7-10, 2.3.2, Equation 2)
3. 1.2D + 1.6H + 1.0E + 1.0L (ASCE 7-10, 2.3.2, Equation 5)
4. 0.9D + 0.9H + 1.0E (ASCE 7-10, 2.3.2, Equation 7)

where,

D = dead load

H = soil pressure load (non-seismic)

L = live load

E = seismic load due to structure inertial load and seismically-induced soil pressure for three different levels of earthquake shaking

Design basis loads for stability and displacement performance assessment are based on allowable stress design (ASD) and consist of combinations of dead, live, soil and earthquake loads. These combinations, modified from ASCE 7-10, Section 2.4.1, for use in performance assessment in this study are:

1. D + H (ASCE 7-10, 2.4.1, Equation 1)
2. D + L + H (ASCE 7-10, 2.4.1, Equation 2)
3. D + H + E + 0.75L (ASCE 7-10, 2.4.1, Equation 5)
4. D + H + 1.0E (ASCE 7-10, 2.4.1, Equation 7)

Structural Rating

Each seawall section and wall type was given a numerical structural rating as a function of its structural assessment results. Ratings for structural stability, strength, and/or displacement performance have been quantified as a function of the assessment results. The rating formulation is:

Overall rating, $R = 1.0 / [(K_s * R_s + \mathrm{Sum}(K_{ei} * R_{ei})) / \mathrm{Sum}(K_s + K_{ei})] – 1.0$

where,

R = overall rating for wall section, 1.00 max

K_s = relative weighting factor for wall static condition

K_{ei} = relative weighting factor for each wall seismic condition under consideration

R_s = load condition rating for wall static condition

R_{ei} = load condition rating for each wall seismic condition under consideration

The load condition ratings (“Ri”) could be a function of minimum factor of safety or maximum demand capacity ratios (DCR) for each load condition type. For example, if a seawall section assessed FOS is greater than 1.0 or its DCR is less than 1.0, this item would get a performance or strength rating of 1.0 because it is not susceptible to damage or failure under that load scenario.

Since seismic failure is the primary concern of this study, the weighting factors for earthquake load should be relatively high compared to weighting factors for static load. It could even be argued that static load should not be a consideration since static loads are a present day condition of the existing seawall.

Ratings for the earthquake load conditions should consist of results for the three levels of earthquake shaking (L1 (475-yr), L2 (975-yr), L3 (MCE)). There are at least three possible scenarios: 1) weight the L2 event higher than the other two, 2) weight the L3 event higher than the other two, weight all seismic events equally. The first scenario would bias the rating result towards wall sections under a major design level earthquake event; the second scenario would bias the rating result towards sections that are deemed more detrimental under the most onerous MCE event. The third scenario would bias the result towards less onerous seismic events.

As an example, consider a bulkhead wall with a minimum FOS for stability for static, L1, L2 and L3 as 2.0, 1.0, 0.8, and 0.5, respectively (effective "DCRs are 0.5, 1.0, 1.25 and 2.0). The wall has associated maximum DCRs for component strength of 0.5, 0.7, 1.2 and 1.5. Furthermore, the weighting factors for static, L1, L2 and L3 are selected as 1, 4, 10, and 2, respectively.

The following calculations are used to determine the overall rating factor:

For the earthquake events L1, L2 and L3,

$K_{L11} = 4$, $K_{L21} = 10$, $K_{L31} = 1$, $R_{L11} = 1.0$, $R_{L21} = 0.80$, $R_{L31} = 0.50$

$K_{L12} = 4$, $K_{L21} = 10$, $K_{L31} = 1$, $R_{L11} = 1.0$, $R_{L21} = 0.833$, $R_{L31} = 0.67$

$R_i = (K_s * R_s + K_{L1i} * R_{L1i} + K_{L2i} * R_{L2i} + K_{L3i} * R_{L3i}) / (K_s + K_{L1i} + K_{L2i} + K_{L3i})$

$R_1 = (1*0.5+ 4*1.00 + 10*1.25 + 2*2.00) / (1+4+10+2) – 1 = 0.235$ (for stability)

$R_2 = (1*0.5+ 4*0.7 + 10*1.2 + 2*1.5) / (1+4+10+2) – 1 = 0.076$ (for strength)

$R = \max(R_1, R_2) = 0.235$

A rating factor below zero indicates the seawall section is not structurally critical. An overall rating that exceeds zero indicates some seawall structural criticality. The magnitudes of the ratings are not important but the relative values are; structural criticality increases with an increasing rating.

The weighting factors, especially the earthquake level weighting factors, may be varied to study sensitivity in the results. The selection of these weighting factors for seismic loads is not expected to significantly alter the study results, at least with respect to the structural ranking of seawall sections for potential retrofit.

4.3. Cutoff Walls

Record drawings indicate that concrete and timber cutoff walls exist in some seawall sections of the northern seawall. The concrete cutoff wall components typically consist of reinforced concrete wall panels of varying height and thickness supported by reinforced concrete plumb piles. The timber cutoff wall components typically consist of timber lagging of varying height and size supported by timber plumb piles. The cutoff wall is typically founded on revetment or a rock dike of varying height, top width, base width and side slopes.

Cutoff walls are structurally assessed using Shoring software, supplemented by hand calculations. Each type of cutoff wall geometry is input into the Shoring software. This includes the wall height, plumb pile spacing, and factors to account for soil arching for applied and resisting soil pressures on the supporting piles below grade. The design basis active and passive soil profiles, along with seismically induced soil pressures as recommended by our geotechnical engineer, are input into the Shoring software. The supporting piles and the cutoff wall panel are checked for code-based adequacy for the maximum pile shear and bending moment. The maximum applied soil pressure is assumed to check the adequacy of the cutoff wall panel.

Structure criticality ratings are determined for the static load and the five levels of seismic loading for subsequent comparison with other seawall section structure.

4.4. Bulkhead Walls

Record drawings indicate that concrete bulkhead walls exist in some seawall sections of the northern seawall. The bulkhead wall components typically consist of unreinforced concrete of varying height and thickness. The bulkhead wall is typically founded on a rock dike of varying height, top width, base width and side slopes. Some bulkhead walls are supported on single or multiple rows of timber piles at varying transverse and longitudinal spacing. Some bulkhead walls are supplemented with attached concrete wing walls; the wing wall concrete may or may not be reinforced and the wing walls may or may not be pile supported.

Bulkhead walls are structurally assessed by spreadsheet calculation for each applicable seawall section type. Each type of bulkhead wall geometry is input into the spreadsheet. This includes the wall height and width variations, supporting bulkhead wall pile spacing, and wing wall geometry and spacing including wing wall supporting piles. The design basis active and passive soil profiles, along with seismically induced soil pressures as recommended by our geotechnical engineer, are input into the assessment spreadsheet along with marginal wharf pile resistance, where applicable. The bulkhead wall sliding and overturning stability is determined and the bulkhead wall component strengths are evaluated for structural adequacy. This includes the supporting piles and wing wall structure, where applicable.

Structure criticality ratings are determined for the static load and the five levels of seismic loading for subsequent comparison with other seawall section structure.

4.5. Retaining Walls

Record drawings indicate that concrete retaining walls exist in a few seawall sections of the northern seawall. Two such walls exist at Fisherman's Wharf and one such wall was incorporated into the design of the new Pier 43.5 promenade. These retaining walls are evaluated in the same manner as the seawall bulkhead walls discussed above.

4.6. Bulkhead Structure Assessment

Each Bulkhead structural type was assessed for structural strength of its primary components and, in the case of bulkhead and retaining wall seawall types, also assessed for sliding and overturning stability. This assessment was performed for static only and static plus five different levels of seismic loading. A structural criticality rating was assigned to each seawall section type as a function of each assessment. A positive rating indicates some sort of structural deficiency determined by the assessments in this study. Where seawall structural stability is checked, the lowest factor of safety (FOS) against sliding or overturning stability is determined. A lack of stability is indicated by a FOS less than 1.0. The worst structural strength deficiency is indicated by a demand-to-capacity ratio (DCR) greater than 1.0.

The following sections summarize the seawall structural assessment for each seawall section type found along the northern seawall. The criticality rating table for each section type is color coded to indicate the criticality for each level of earthquake loading, as follows:

Red – A very high level of structural instability or a very high level structural strength deficiency was determined for this seawall section and type. A seismic retrofit is strongly indicated.

Orange – A high level of structural instability or high level of structural strength deficiency was determined for this seawall section and type. A seismic retrofit is indicated.

Yellow – A relatively low level of structural instability or structural strength deficiency was determined for this seawall section and type. A seismic retrofit may be indicated.

Green – No level of structural instability or structural strength deficiency was determined for this seawall section and type. A seismic retrofit is not needed.

Blue – A high level of structural stability and a low level of structural strength demand was indicated. A seismic retrofit is not needed.

Seawall Section FW (Fisherman's Wharf) – Bulkhead Assessment

This seawall section consists of seven different bulkhead types. An assessment of all bulkhead types was performed and summarized here.

<u>Seawall Section FW, Bulkhead Type 11 (Wharf J10)</u>

Seawall Section FW Bulkhead Type 11 is a timber pile and lagging wall as shown on Figure 4-2. This timber cutoff wall panel is 2 feet high, 10 feet wide between supporting timber piles, with 4x12 timber lagging and additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Rock dike elevation at seawall: — -2.0 ft City Datum.
- Wall panel size and spacing: — 2.0 ft high, 10 feet wide, 3.5 inches thick.
- Cutoff wall piles (Yes/No): — Yes, timber 12" diameter at 10 feet spacing.
- Marginal wharf attached (Yes/No): — Yes, Wharf J10, 43 ft wide, piles at 10 feet spacing.
- Finger Pier (Yes/No): — No.

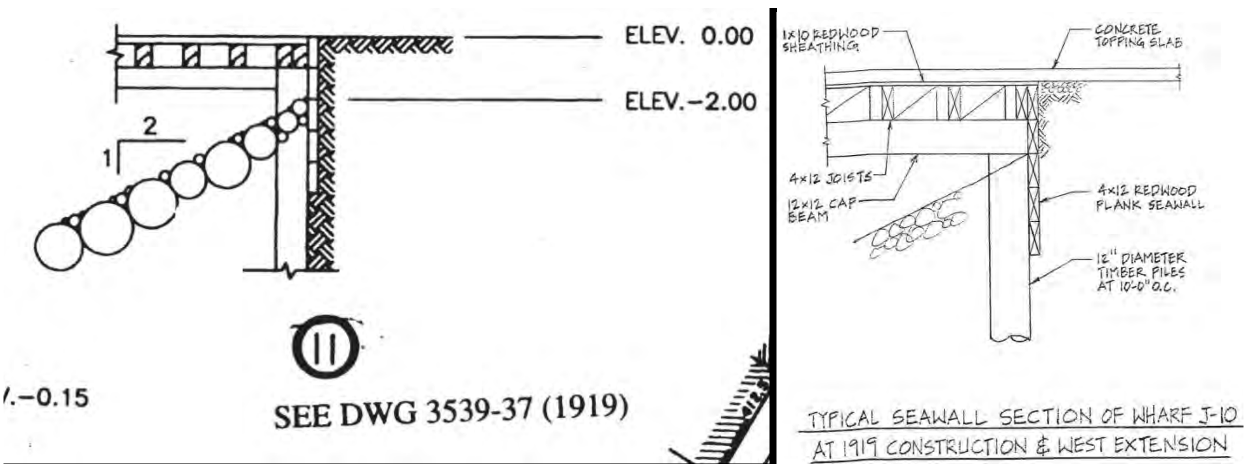


Figure 4-2: Seawall Section FW, Bulkhead Wall Type 11

This cutoff wall section and type has no structural strength deficiencies under static and all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall timber lagging moment capacity. A summary of the results of this assessment is shown on Table 4-3.

Table 4-3: Structural Assessment Summary – Seawall Section FW, Cutoff Wall Type 11

	Seawall Section: FW			Wall Type: 11		
	Static	**EQ-M8.0 SA**	**EQ-DE**	**EQ-475**	**EQ-975**	**EQ-MCE**
Panel Strength DCR	**0.42**	**0.49**	**0.50**	**0.52**	**0.53**	**0.54**
Pile Strength DCR	**0.13**	**0.16**	**0.18**	**0.20**	**0.21**	**0.23**
Criticality Rating	**-0.58**	**-0.51**	**-0.50**	**-0.48**	**-0.47**	**-0.46**

The structural criticality ratings range from -0.58 to -0.46 indicating that retrofit of this seawall section and type is not needed.

Seawall Section FW, Bulkhead Type 10 (Wharf J9)

Seawall Section FW Type 10 is a timber pile and lagging wall as shown on Figure 4-3. This timber cutoff wall panel is about 4.25 feet high, 8 feet wide between supporting timber piles, with timber log lagging and additional components as follows:

- Applicable Geotechnical Soil Profile: A1
- Rock dike elevation at seawall: -4.25 ft City Datum.
- Wall panel size and spacing: 4.25 ft high, 8 feet wide, 12 inch diameter thick.
- Cutoff wall piles (Yes/No): Yes, timber 12” diameter at 8 feet spacing.
- Marginal wharf attached (Yes/No): Yes, Wharf J9, 18 ft wide, piles at 9 feet spacing.
- Finger Pier (Yes/No): No.

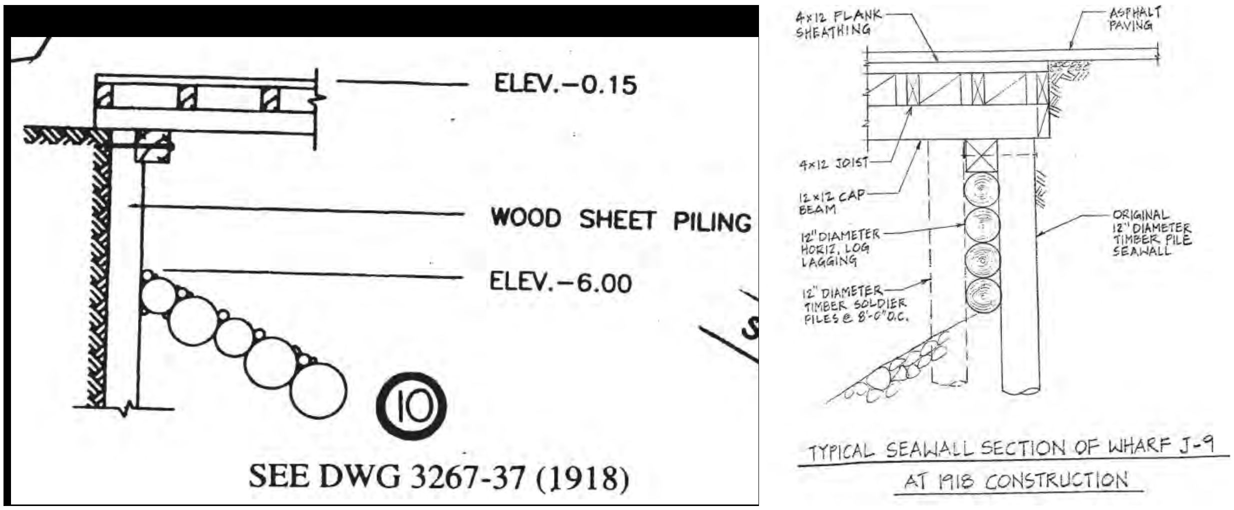


Figure 4-3: Seawall Section FW, Bulkhead Wall Type 10

This cutoff wall section and type has structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall timber soldier pile moment capacity. A summary of the results of this assessment is shown on Table 4-4.

Table 4-4: Structural Assessment Summary – Seawall Section FW, Cutoff Wall Type 10

	Seawall Section: FW			Bulkhead Wall Type: 10		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.13	0.15	0.15	0.16	0.16	0.17
Pile Strength DCR	0.74	1.20	1.53	1.71	1.86	2.05
Criticality Rating	-0.26	0.20	0.53	0.71	0.86	1.05

The structural criticality ratings range from -0.26 to +1.05 indicating that retrofit of this seawall section and type piles are needed.

Seawall Section FW, Bulkhead Type 9 (Wharf J5)

Seawall Section FW Type 9 is a concrete bulkhead wall as shown on Figure 4-4. This unreinforced concrete bulkhead wall is 3 feet high, assumed 2.25 feet wide at its top, 4.5 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile A1
- Bulkhead wall piles (Yes/No): No.
- Wing walls (Yes/No): No.
- Wing wall piles (Yes/No): No.
- Marginal wharf attached (Yes/No): Yes, Wharf J5, wharf pile data unknown, assume 2 piles at 9 ft transverse spacing.
- Finger Pier (Yes/No): No.

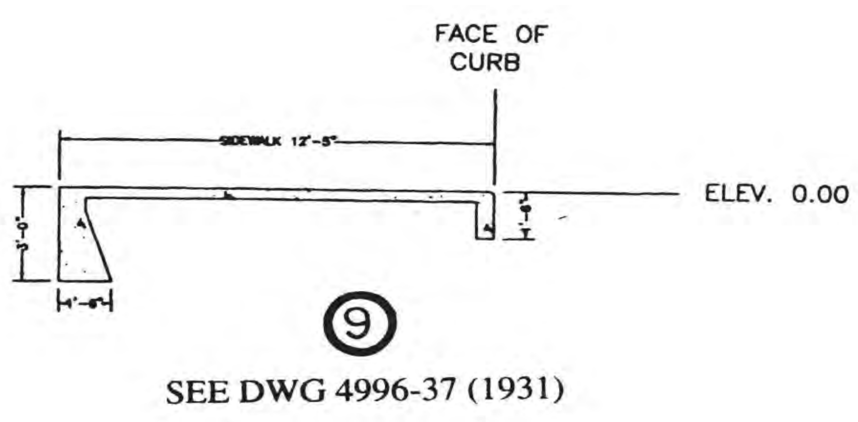


Figure 4-4: Seawall Section FW, Bulkhead Wall Type 9

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the wharf J5 timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-5.

Table 4-5: Structural Assessment Summary – Seawall Section FW, Bulkhead Wall Type 9

	Seawall Section: FW	Bulkhead Wall Type: 9				
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	16.38	2.62	2.51	2.15	1.97	1.75
FOS-Overturning	43.21	4.01	3.83	3.26	2.96	2.62
Strength DCR	0.08	0.44	0.46	0.54	0.59	0.66
Criticality Rating	-0.92	-0.56	-0.54	-0.46	-0.41	-0.34

The structural criticality ratings range from -0.92 to -0.34indicating that retrofit of this seawall section and type is not needed.

<u>Seawall Section FW, Bulkhead Type 8 (Wharf J5)</u>

Seawall Section FW Type 8 is a concrete cutoff wall as shown on Figure 4-5. This reinforced concrete cutoff wall panel is 9 feet high, 9 feet wide between supporting piles, 7 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Rock dike elevation at seawall: — -8.0 ft City Datum.
- Wall panel size and spacing: — 6.5 ft high, width unknown, 7 inches thick.
- Wall panel reinforcement: — Unknown.
- Cutoff wall piles (Yes/No): — Yes, but pile data unknown, assumed 12 inch diameter.
- Pile reinforcement: — Unknown.
- Marginal wharf attached (Yes/No): — Yes, Wharf J5.
- Finger Pier (Yes/No): — No.

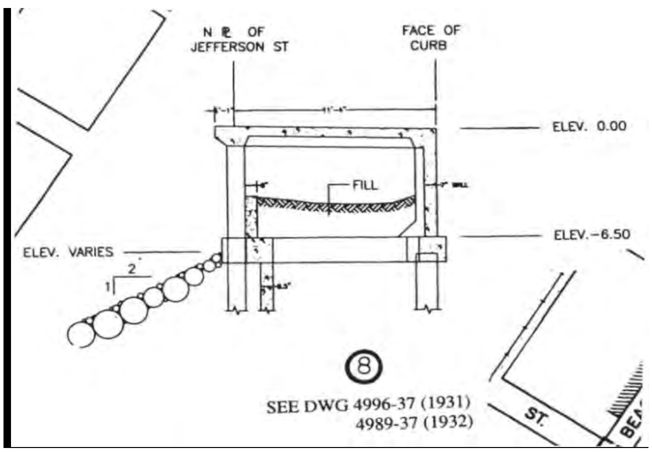


Figure 4-5: Seawall Section FW, Bulkhead Wall Type 8

This cutoff wall section and type has structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall moment capacity. A summary of the results of this assessment is shown on Table 4-6.

Table 4-6: Structural Assessment Summary – Seawall Section FW, Cutoff Wall Type 8

	Seawall Section: FW Bulkhead Wall Type: 8					
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.59	0.71	0.72	0.75	0.77	0.80
Pile Strength DCR	0.62	0.82	0.96	1.02	1.07	1.14
Criticality Rating	-0.38	-0.18	-0.04	0.02	0.07	0.14

The structural criticality ratings range from -0.38 to +0.14 indicating that retrofit of this seawall section and type is needed.

Seawall Section FW, Bulkhead Type 5 (Wharf J5)

Seawall Section FW Type 5 is a concrete cutoff wall as shown on Figure 4-6. This reinforced concrete cutoff wall panel is 6 feet high and assumed 7 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile: A1
- Rock dike elevation at seawall: -6.0 ft City Datum.
- Wall panel size and spacing: 5.5 ft high, width unknown, assumed 6 inches thick. Wall spans vertically from pile cap below.
- Wall panel reinforcement: Unknown, assumed #4@ 6 inch spacing.
- Cutoff wall piles (Yes/No): Yes, but pile data unknown, assumed 12 inch diameter.
- Pile reinforcement: Unknown.
- Marginal wharf attached (Yes/No): Yes, Wharf J5.
- Finger Pier (Yes/No): No.

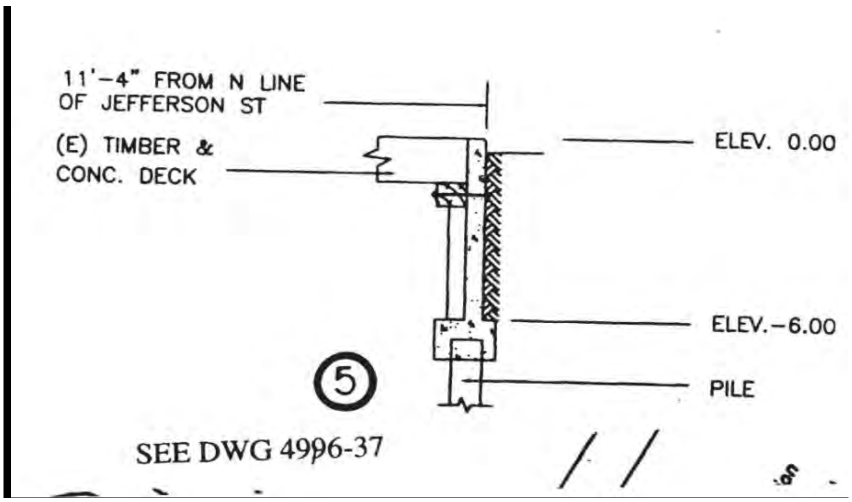


Figure 4-6: Seawall Section FW, Bulkhead Wall Type 5

This cutoff wall section and type has structural strength deficiencies under static and all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall panel moment capacity. A summary of the results of this assessment is shown on Table 4-7.

Table 4-7: Structural Assessment Summary – Seawall Section FW, Cutoff Wall Type 5

	Seawall Section: FW		Bulkhead Wall Type: 5			
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.42	0.51	0.52	0.54	0.56	0.58
Pile Strength DCR	0.50	0.72	0.87	0.94	1.01	1.08
Criticality Rating	-0.50	-0.28	-0.13	-0.06	0.01	0.08

The structural criticality ratings range from -0.50 to +0.08 indicating that retrofit of this seawall section and type is needed.

Seawall Section FW, Bulkhead Type 4 (J1)

Seawall Section FW Type 4 is a concrete retaining wall as shown on Figure 4-7. This assumed unreinforced retaining wall is assumed 2.5 feet high, assumed 5 inches wide at its top, with an assumed 20 inch wide base with additional components as follows:

- Applicable Geotechnical Soil Profile A1
- Retaining wall piles (Yes/No): No.
- Wing walls (Yes/No): No.
- Wing wall piles (Yes/No): No.
- Marginal wharf attached (Yes/No): No.
- Finger Pier (Yes/No): No.

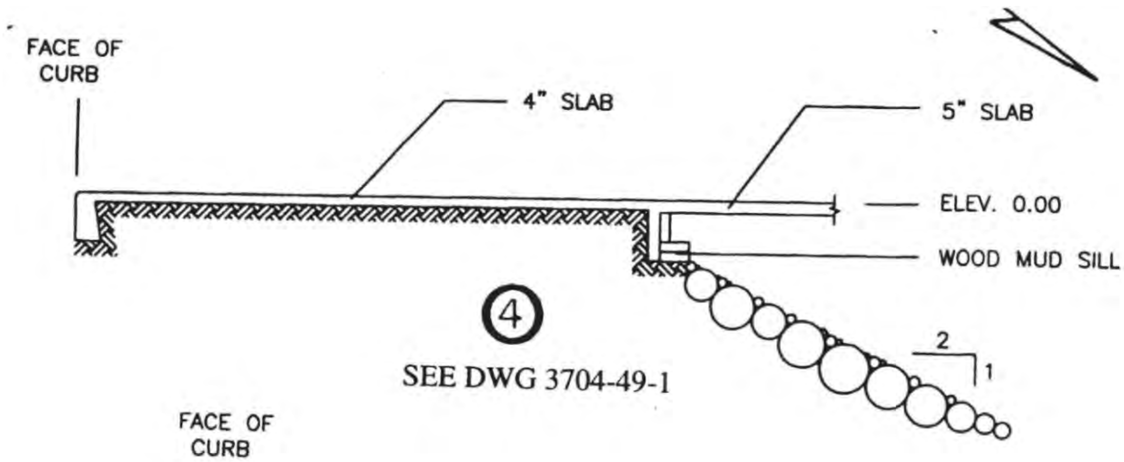


Figure 4-7: Seawall Section FW, Retaining Wall Type 4

This retaining wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study due to the presence of the 4 inch slab tieback assumed connected to the top of the wall. This wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall unreinforced concrete shear capacity. A summary of the results of this assessment is shown on Table 4-8.

Table 4-8: Structural Assessment Summary – Seawall Section FW, Bulkhead Wall Type 4

	Seawall Section: FW	Bulkhead Wall Type:4				
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.00	1.37	1.31	1.17	1.10	1.00
FOS-Overturning	6.55	2.66	2.54	2.24	2.10	1.89
Strength DCR	0.12	0.15	0.15	0.16	0.17	0.17
Criticality Rating	-0.67	-0.27	-0.24	-0.15	-0.09	0.00

The structural criticality ratings range from -0.67 to +0.00 indicating that retrofit of this seawall section and type is not needed.

Seawall Section FW, Bulkhead Type 6 (Pier 45)

Seawall Section FW Type 6 is a concrete retaining wall as shown on Figure 4-8. This assumed unreinforced retaining wall is assumed 6 feet high, assumed 8 inches wide at its top, with a 6 feet wide, 8 inch thick base and wing walls assumed 8 inches thick at 10 feet assumed spacing. Additional components are as follows:

- Applicable Geotechnical Soil Profile: A1
- Retaining wall piles (Yes/No): No.
- Wing walls (Yes/No): Yes, spacing unknown (assume 10 feet).
- Wing wall piles (Yes/No): No.
- Marginal wharf attached (Yes/No): No.
- Finger Pier (Yes/No): No.

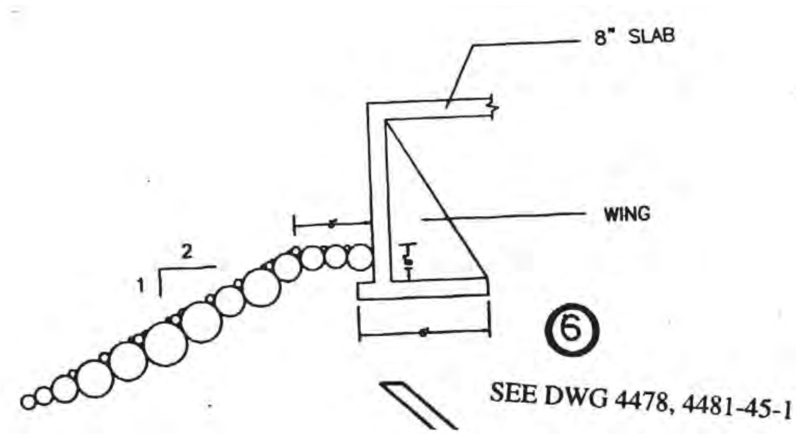


Figure 4-8: Seawall Section FW, Retaining Wall Type 6

This retaining wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study due to the presence of the 8 inch slab tieback assumed connected to the top of the wall. This wall section and type has structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the retaining wall unreinforced concrete moment capacity. A summary of the results of this assessment is shown on Table 4-9.

Table 4-9: Structural Assessment Summary – Seawall Section FW, Bulkhead Wall Type 6

	Seawall Section: FW		Bulkhead Wall Type:6			
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	2.72	1.34	1.29	1.16	1.09	1.00
FOS-Overturning	10.77	5.12	4.87	4.34	4.09	3.71
Strength DCR	0.65	1.04	1.08	1.18	1.24	1.34
Criticality Rating	-0.35	0.04	0.08	0.18	0.24	0.34

The structural criticality ratings range from -0.35 to +0.34 indicating that retrofit of this seawall section and type stem wall is needed.

Seawall Section B

<u>Seawall Section B, Bulkhead Type 3</u>

Seawall Section B Type 3 is a concrete cutoff wall as shown on Figure 4-9. This reinforced concrete cutoff wall panel is 9 feet high, 9 feet wide between supporting piles, 10 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile A1
- Rock dike elevation at seawall: -6.0 ft City Datum assumed.
- Wall panel size and spacing: 10.5 ft high, 8'-10" wide, assumed 10 inches thick.
- Wall panel reinforcement: 1/2" SQ at 12" spacing each way, each face.
- Cutoff wall piles (Yes/No): Yes, concrete16" square at 8'-10" spacing.
- Pile reinforcement: Four 5/8" SQ bars w/ W3 @ 6" pitch.
- Marginal wharf attached (Yes/No): Yes, adjacent to Pier 45.
- Finger Pier (Yes/No): No.

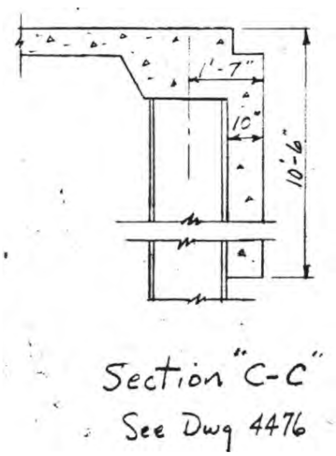


Figure 4-9: Seawall Section B, Bulkhead Wall Type 3

This cutoff wall section and type has structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall concrete panel moment capacity. A summary of the results of this assessment is shown on Table 4-10.

Table 4-10: Structural Assessment Summary – Seawall Section B, Cutoff Wall Type 3

	Seawall Section: B	Bulkhead Wall Type: 3				
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.94	1.18	1.20	1.26	1.30	1.36
Pile Strength DCR	0.28	0.39	0.47	0.53	0.57	0.61
Criticality Rating	-0.06	0.18	0.20	0.26	0.30	0.36

The structural criticality ratings range from =-0.06 to +0.36 indicating that retrofit of this seawall section and type is needed.

Seawall Section B, Bulkhead Type 43.5 Promenade

This seawall section and type was constructed in 2012 for the Pier 43.5 Promenade and replaced the existing seawall at this location. Seawall Section B Type 43.5Prom is a concrete retaining wall as shown on Figure 4-10. This reinforced concrete retaining wall is 5 feet high with a 5 feet wide base, both 12 inches thick, with additional components as follows:

- Applicable Geotechnical Soil Profile	B1
- Retaining wall piles (Yes/No):	No.
- Wing walls (Yes/No):	No.
- Wing wall piles (Yes/No):	No.
- Marginal wharf attached (Yes/No):	No.
- Finger Pier (Yes/No):	No.

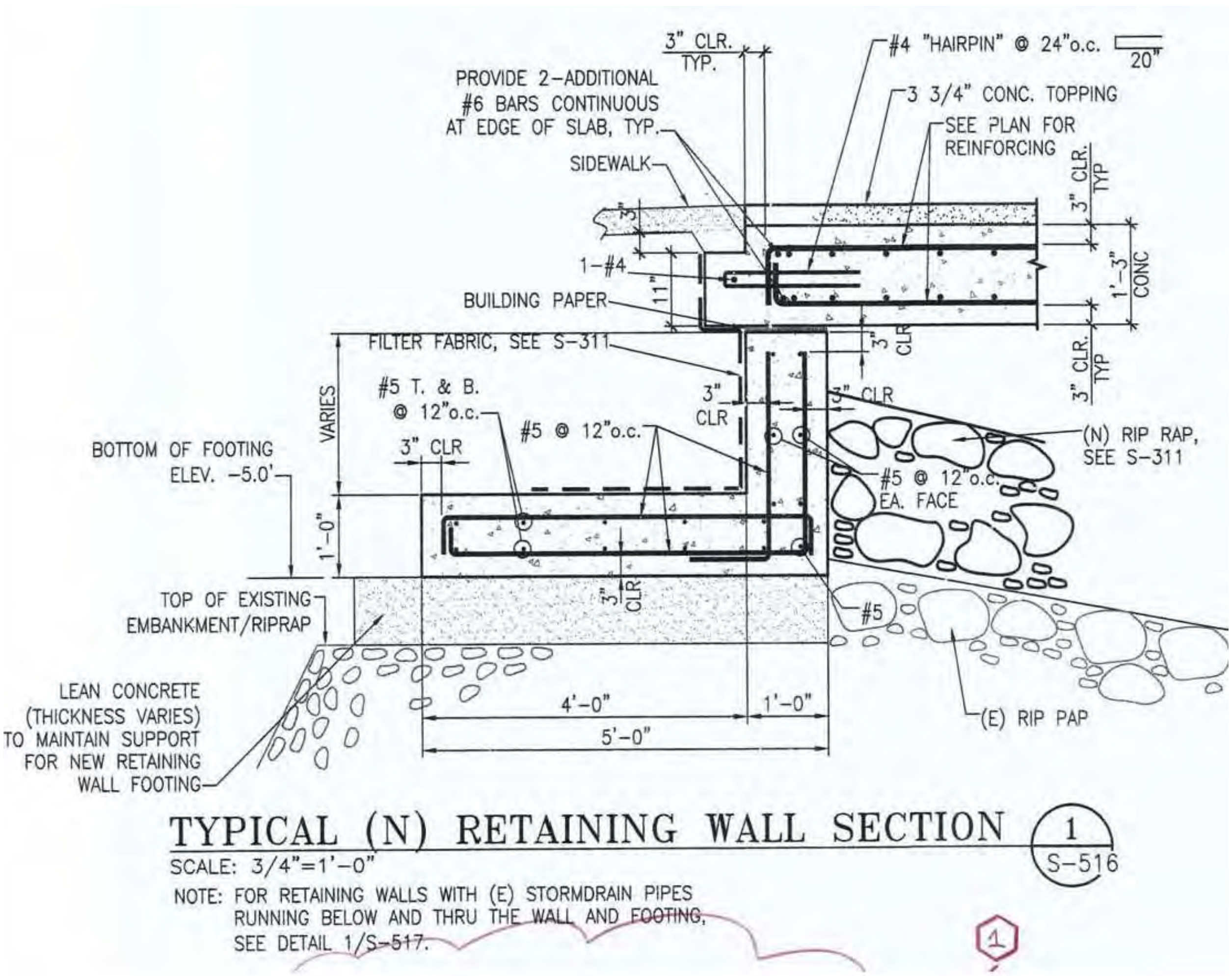


Figure 4-10: Seawall Section B, Retaining Wall Type 43.4 Promenade

This retaining wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this. This wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the retaining wall concrete moment capacity. A summary of the results of this assessment is shown on Table 4-11.

Table 4-11: Structural Assessment Summary – Seawall Section B, Bulkhead Wall Type 43.5 Promenade

	Seawall Section: B			Bulkhead Wall Type:43.5 Prom		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.60	1.42	1.37	1.28	1.25	1.24
FOS-Overturning	5.01	1.85	1.78	1.66	1.62	1.61
Strength DCR	0.02	0.07	0.07	0.08	0.08	0.08
Criticality Rating	-0.72	-0.29	-0.27	-0.22	-0.20	-0.19

The structural criticality ratings range from -0.72 to -0.19 indicating that retrofit of this seawall section and type is not needed.

Seawall Section B, Bulkhead Type 2

Seawall Section B Type 2 consists of revetment only as shown on Figure 4-11 and fronts what remains of Pier 43. Thus, no structural assessment is performed. No structural retrofit of this seawall section and type is indicated.

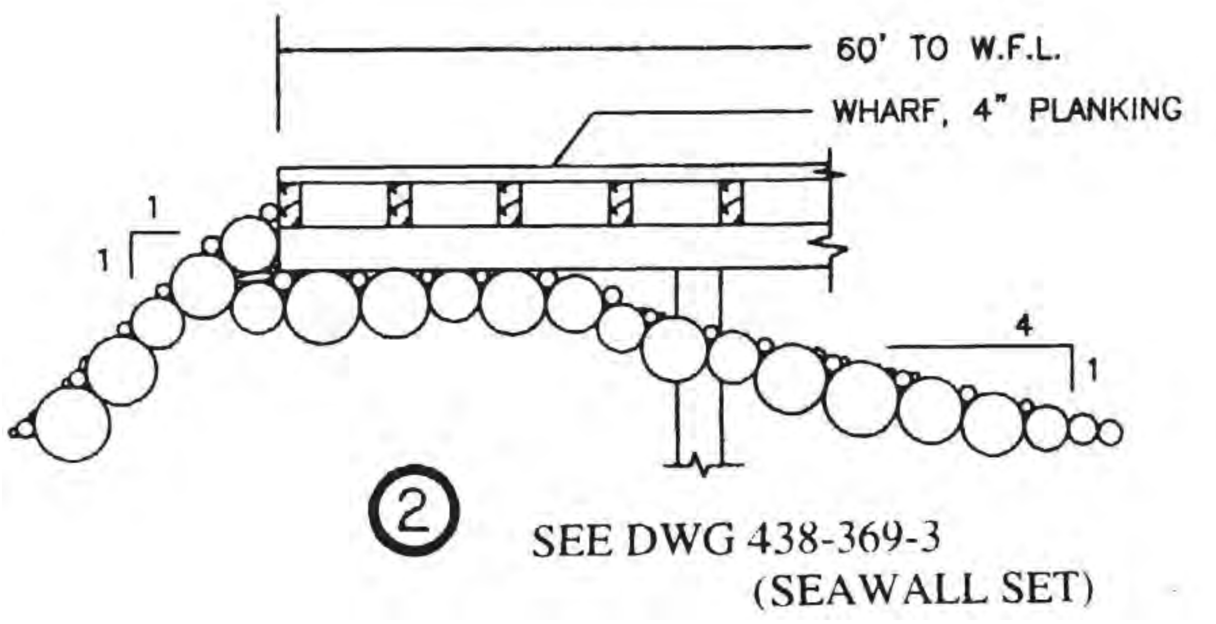


Figure 4-11: Seawall Section B, Bulkhead Type 2

Seawall Section A

<u>Seawall Section A, Bulkhead Type 2</u>

Seawall Section A Type 2, identical to Seawall Section B, Seawall Type 2, consists of revetment only as shown on Figure 4-12. Thus, no structural assessment is performed. No structural retrofit of this seawall section and type is indicated.

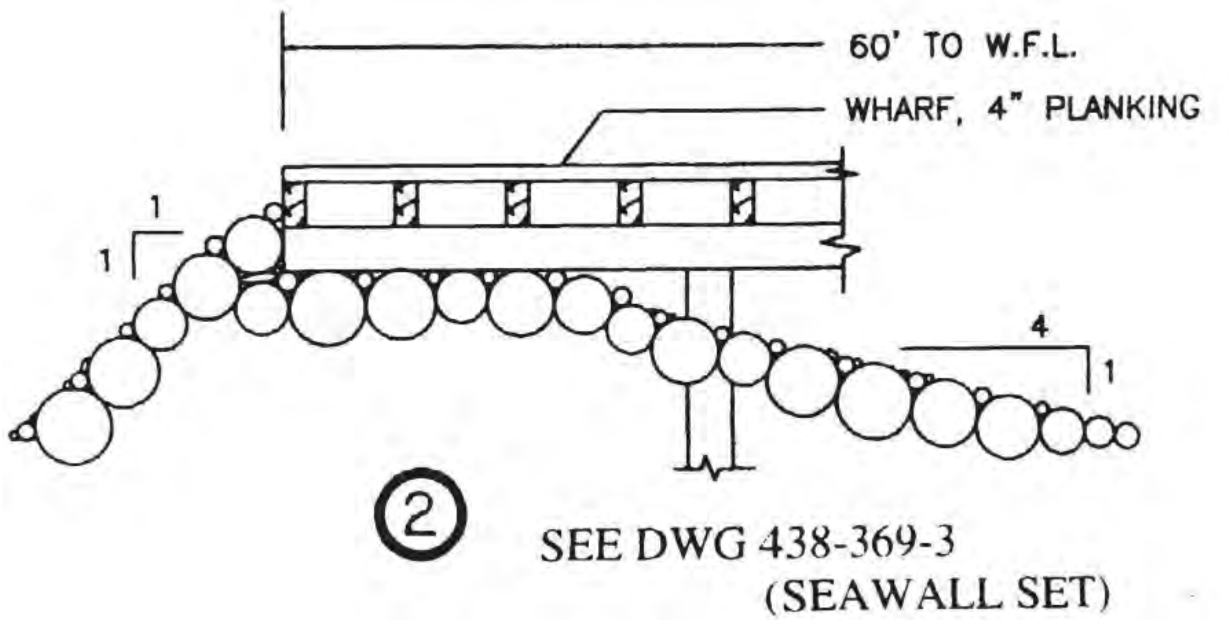


Figure 4-12: Seawall Section A, Bulkhead Type 2

Seawall Section A, Bulkhead Type 1

Seawall Section A Type 1 is a concrete cutoff wall as shown on Figure 4-13. This reinforced concrete cutoff wall panel is 9 feet high, 9 feet wide between supporting piles, 16 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Rock dike elevation at seawall: — -4.1 ft City Datum
- Wall panel size and spacing: — 9.0 ft high, 9.0 ft wide, 16 inches thick.
- Wall panel reinforcement: — 1/2" SQ at 12" spacing each way, each face.
- Cutoff wall piles (Yes/No): — Yes, concrete16" square at 9'-0" spacing.
- Pile reinforcement: — Four 3/4" SQ bars w/ W6 @ 6" pitch
- Marginal wharf attached (Yes/No): — Yes.
- Finger Pier (Yes/No): — Yes, Pier 41(Piers 35 and 39 similar) .

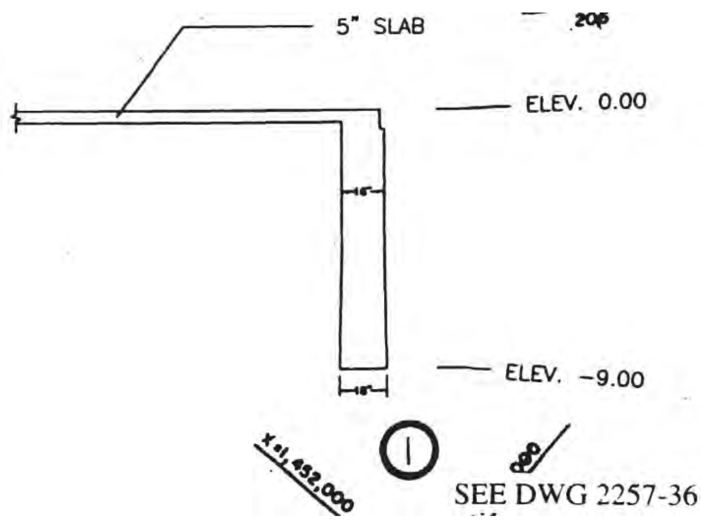


Figure 4-13: Seawall Section A, Bulkhead Wall Type 1

This cutoff wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall concrete panel moment capacity. A summary of the results of this assessment is shown on Table 4-12.

Table 4-12: Structural Assessment Summary – Seawall Section A, Cutoff Wall Type 1

	Seawall Section: A	Bulkhead Wall Type: 1				
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.42	0.60	0.62	0.66	0.68	0.72
Pile Strength DCR	0.10	0.22	0.30	0.33	0.36	0.40
Criticality Rating	-0.58	-0.40	-0.38	-0.34	-0.32	-0.28

The structural criticality ratings range from -0.58 to -0.28 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 1

<u>Seawall Section 1, Bulkhead Type 1</u>

Seawall Section 1 Type 1 is a concrete cutoff wall, the same as for Section A, Seawall Type 1 (Figure 4-14) with geotechnical soil profile A1.

The structural criticality ratings range from -0.58 to -0.28 indicating that retrofit of this seawall section and seawall type is not needed.

Seawall Section 2

<u>Seawall Section 2, Bulkhead Type 1</u>

Seawall Section 2 Type 1 is a concrete cutoff wall, the same as for Section A, Seawall Type 1 (Figure 4-14) with geotechnical soil profile A1.

The structural criticality ratings range from -0.58 to -0.28 indicating that retrofit of this seawall section and seawall type is not needed.

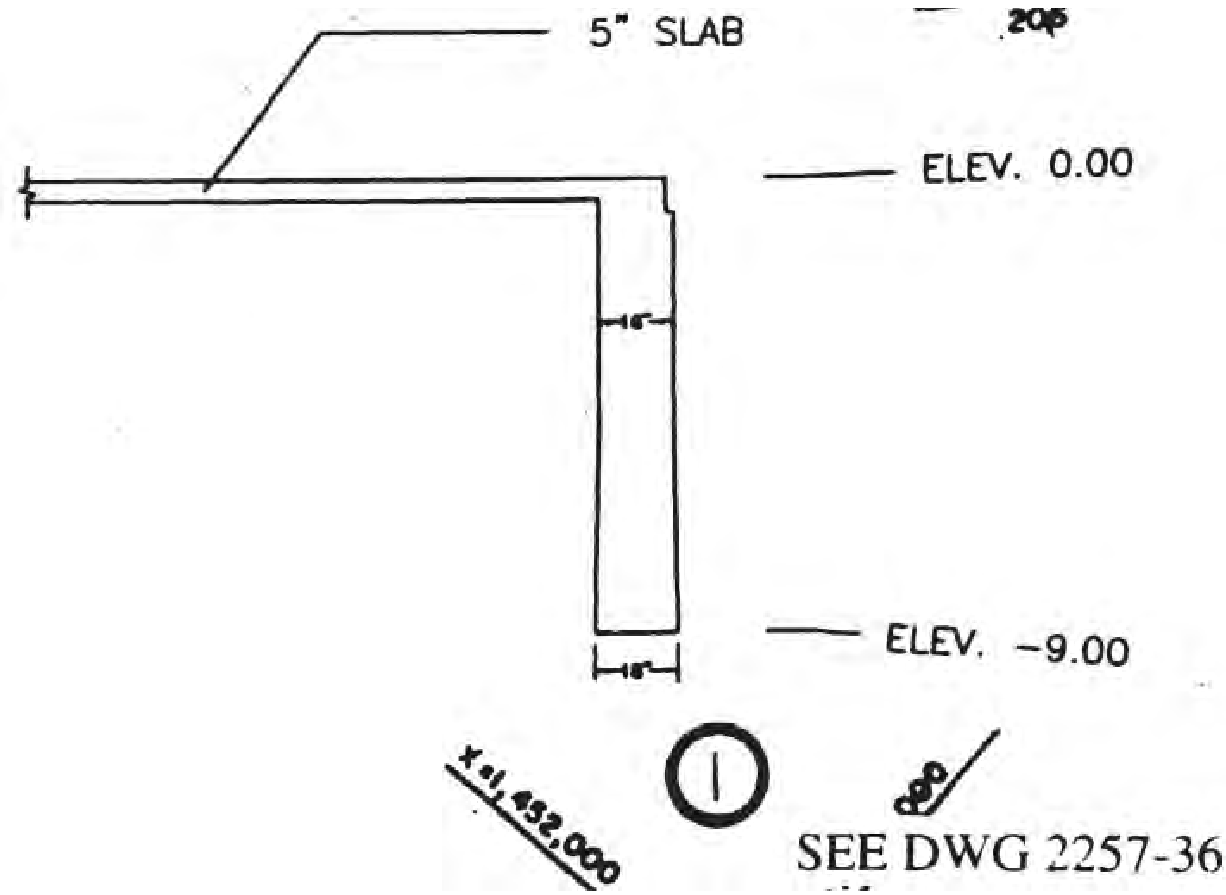


Figure 4-14: Seawall Sections 1 and 2, Bulkhead Wall Type 1

Seawall Section 3

<u>Seawall Section 3, BulkheadType 1</u>

Seawall Section 3 Type 1 is a concrete cutoff wall as shown on Figure 4-15. This reinforced concrete cutoff wall panel is 9 feet high, 10 feet wide between supporting piles, 16 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile — B1
- Rock dike elevation at seawall: — -6.0 ft City Datum
- Wall panel size and spacing: — 9.0 ft high, 10.0 ft wide, 16 inches thick.
- Wall panel reinforcement: — Assumed 1/2" SQ at 12” spacing each way, each face.
- Cutoff wall piles (Yes/No): — Yes, concrete16” square at 10’-0” spacing.
- Pile reinforcement: — Four 5/8" SQ bars w/ W3 @ 6” pitch
- Marginal wharf attached (Yes/No): — Yes.
- Finger Pier (Yes/No): — Yes, Pier 31 (Pier 35 similar).

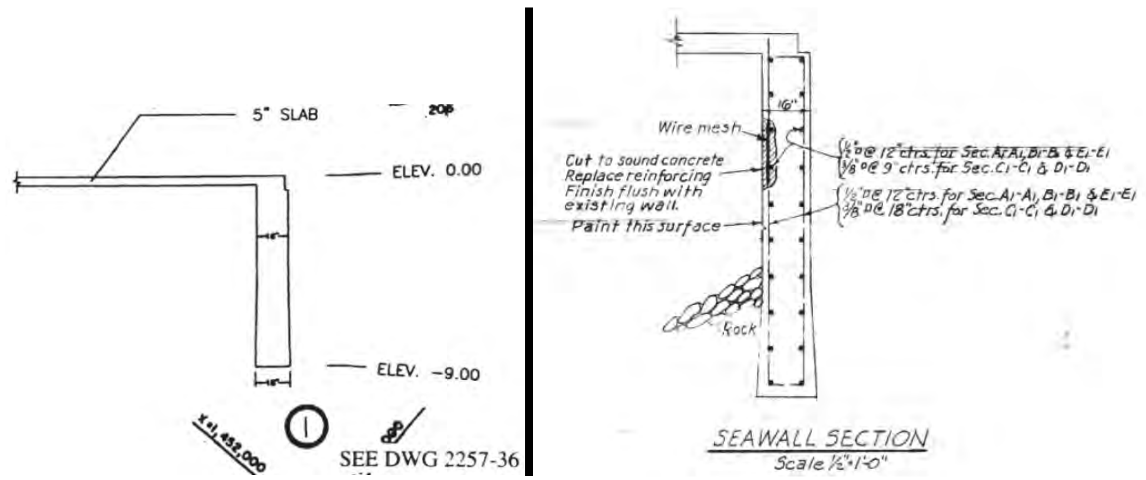


Figure 4-15: Seawall Section 3, Bulkhead Wall Type 1

This cutoff wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall concrete panel moment capacity. A summary of the results of this assessment is shown on Table 4-13.

Table 4-13: Structural Assessment Summary – Seawall Section 3, Cutoff Wall Type 1

	Seawall Section: 3	Bulkhead Wall Type: 1				
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.49	0.66	0.68	0.70	0.70	0.70
Pile Strength DCR	0.26	0.47	0.63	0.66	0.68	0.69
Criticality Rating	-0.51	-0.33	-0.32	-0.30	-0.30	-0.30

The structural criticality ratings range from -0.51 to -0.30 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 4

Seawall Section 4, Bulkhead Type Y

Seawall Section 4 Type Y is a concrete cutoff wall as shown on Figure 4-16. This reinforced concrete cutoff wall panel is 10 feet high, 9 feet wide between supporting piles, 16 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile — B2
- Rock dike elevation at seawall: — -6.0 ft City Datum
- Wall panel size and spacing: — 10.0 ft high, 9.0 ft wide, 16 inches thick.
- Wall panel reinforcement: — 1/2" SQ at 12” spacing each way, each face.
- Cutoff wall piles (Yes/No): — Yes, concrete16” square at 9’-0” spacing.
- Pile reinforcement: — Four 5/8" SQ bars
- Marginal wharf attached (Yes/No): — Yes.
- Finger Pier (Yes/No): — Yes, Piers 23 and 27.

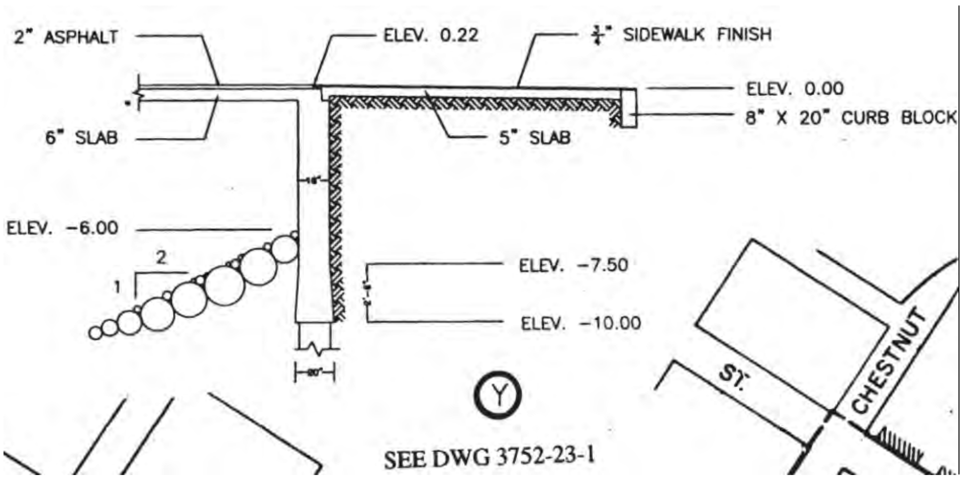


Figure 4-16: Seawall Section 4, Bulkhead Wall Type Y

This cutoff wall section and type has structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall supporting concrete pile moment and shear capacity. A summary of the results of this assessment is shown on Table 4-14.

Table 4-14: Structural Assessment Summary – Seawall Section 4, Cutoff Wall Type Y

	Seawall Section: 4			Bulkhead Wall Type: Y		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.61	0.82	0.82	0.83	0.84	0.86
Pile Strength DCR	0.90	1.55	2.07	2.23	2.34	2.38
Criticality Rating	-0.10	0.55	1.07	1.23	1.34	1.38

The structural criticality ratings range from -0.10 to +1.38 indicating that retrofit of this seawall section and type piles is needed.

Seawall Section 5

Seawall Section 5, Bulkhead Type Z

Seawall Section 5 Type Z is a concrete bulkhead wall as shown on Figure 4-17. This unreinforced concrete bulkhead wall is 11 feet high, 5 feet wide at its top, 6 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile	B1
- Bulkhead wall piles (Yes/No):	No.
- Wing walls (Yes/No):	Yes, at 20 foot centers.
- Wing wall piles (Yes/No):	No.
- Marginal wharf attached (Yes/No):	Yes.
- Finger Pier (Yes/No):	Yes, Pier 19.

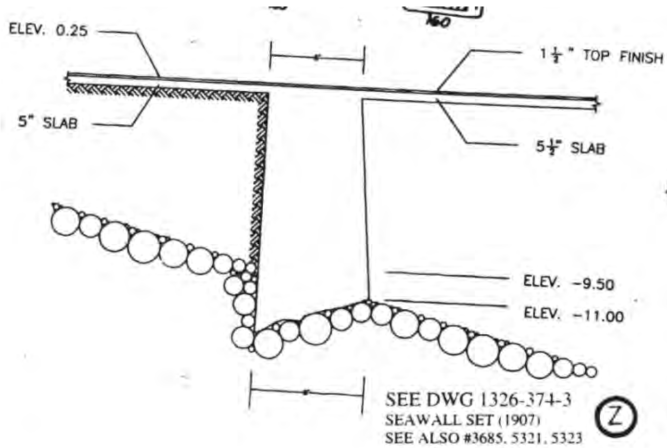


Figure 4-17: Seawall Section 5, Bulkhead Wall Type Z

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed unstable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the marginal wharf concrete pile shear capacity. A summary of the results of this assessment is shown on Table 4-15.

Table 4-15: Structural Assessment Summary – Seawall Section 5, Bulkhead Wall Type Z

	Seawall Section: 05			Bulkhead Wall Type: Z		
	Static	**EQ-M8.0 SA**	**EQ-DE**	**EQ-475**	**EQ-975**	**EQ-MCE**
FOS-Sliding	2.69	0.87	0.84	0.79	0.76	0.75
FOS-Overturning	5.22	0.95	0.92	0.85	0.82	0.81
Strength DCR	0.39	0.88	0.91	0.97	1.00	1.01
Criticality Rating	-0.61	0.15	0.18	0.27	0.31	0.33

The structural criticality ratings range from -061 to +0.33 indicating that retrofit of this seawall section and type wall stability is needed.

Seawall Section 5, Bulkhead Type Y

Seawall Section 5 Type Y is a concrete bulkhead wall, identical to that of Seawall Section 4 Type Y, as shown on Figure 4-18.

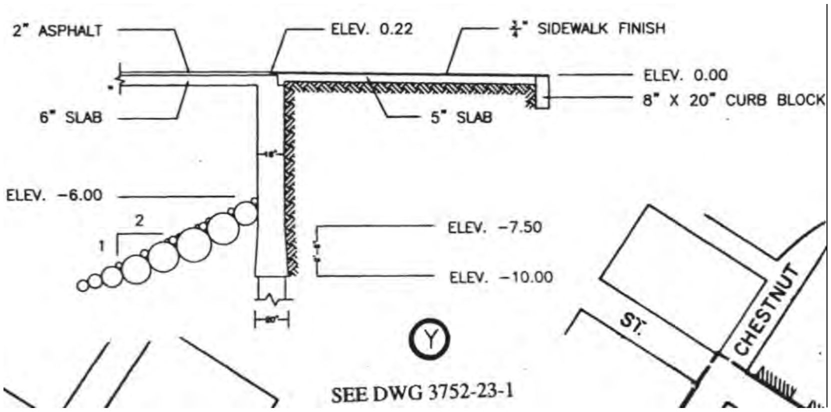


Figure 4-18: Seawall Section 5, Bulkhead Wall Type Y

Seawall Section 5, Bulkhead Type X

Seawall Section 5 Type X is a concrete bulkhead wall as shown on Figure 4-19. This unreinforced concrete bulkhead wall is 13 feet high, 2 feet wide at its top, 6 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — B1
- Bulkhead wall piles (Yes/No): — Yes, two rows 3 feet apart, timber piles at 6 feet centers.
- Wing walls (Yes/No): — Yes, at 20 foot centers.
- Wing wall piles (Yes/No): — Yes, single pile supports each wing wall.
- Marginal wharf attached (Yes/No): — No.
- Finger Pier (Yes/No): — Yes, Pier 15-17.

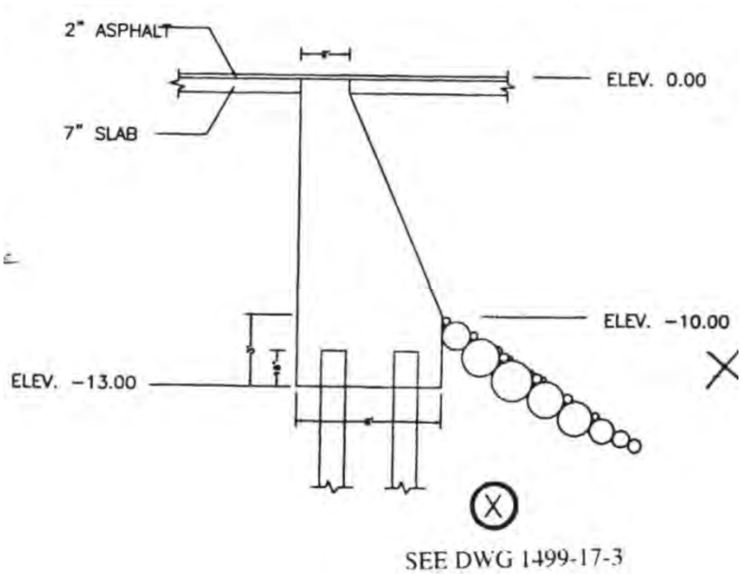


Figure 4-19: Seawall Section 5, Bulkhead Wall Type X

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the marginal wharf pile shear capacity. A summary of the results of this assessment is shown on Table 4-16.

Table 4-16: Structural Assessment Summary – Seawall Section 5, Bulkhead Wall Type X

	Seawall Section: 05			Wall Type: X		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	11.34	3.94	3.82	3.57	3.47	3.43
FOS-Overturning	29.58	6.01	5.81	5.34	5.17	5.09
Strength DCR	0.29	0.58	0.60	0.64	0.66	0.66
Criticality Rating	-0.71	-0.42	-0.40	-0.36	-0.34	-0.34

The structural criticality ratings range from -0.71 to -0.34 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 5, Bulkhead Type W

Seawall Section 5 Type W is a concrete cutoff wall as shown on Figure 4-20. This reinforced concrete cutoff wall panel is 10 feet high, assumed 9 feet wide between supporting piles, 16 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile B1
- Rock dike elevation at seawall: -6.0 ft City Datum
- Wall panel size and spacing: 10.0 ft high, 9.0 ft wide, 16 inches thick.
- Wall panel reinforcement: Unknown.
- Cutoff wall piles (Yes/No): Yes, assume16" square at assumed 9'-0" spacing.
- Pile reinforcement: Assume four 5/8" SQ bars
- Marginal wharf attached (Yes/No): Yes.
- Finger Pier (Yes/No): No.

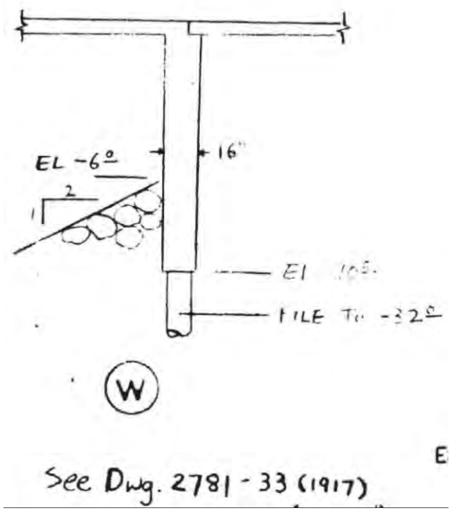


Figure 4-20: Seawall Section 5, Bulkhead Wall Type W

This cutoff wall section and type has structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall supporting concrete pile moment and shear capacity. A summary of the results of this assessment is shown on Table 4-17.

Table 4-17: Structural Assessment Summary – Seawall Section 5, Cutoff Wall Type W

	Seawall Section: 5			Wall Type: W		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.61	0.82	0.82	0.83	0.84	0.86
Pile Strength DCR	0.90	1.54	1.92	1.94	2.01	2.09
Criticality Rating	-0.10	0.54	0.92	0.94	1.01	1.09

The structural criticality ratings range from -0.10 to +1.09 indicating that retrofit of this seawall section and type piles is needed.

Seawall Section 6

Seawall Section 6 Type W is a concrete cutoff wall as shown on Figure 4-21, similar to Seawall Section 5 Type W except that larger reinforcement is indicated in the wall support piles and the assumed geotechnical soil conditions differ. This reinforced concrete cutoff wall panel is 10 feet high, assumed 9 feet wide between supporting piles, 16 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile	C2
- Rock dike elevation at seawall:	-6.0 ft City Datum
- Wall panel size and spacing:	10.0 ft high, 9.0 ft wide, 16 inches thick.
- Wall panel reinforcement:	1/2" SQ at 12" spacing each way, each face.
- Cutoff wall piles (Yes/No):	Yes, 16" square at assumed 9'-0" spacing.
- Pile reinforcement:	Four 3/4" SQ bars
- Marginal wharf attached (Yes/No):	Yes.
- Finger Pier (Yes/No):	No.

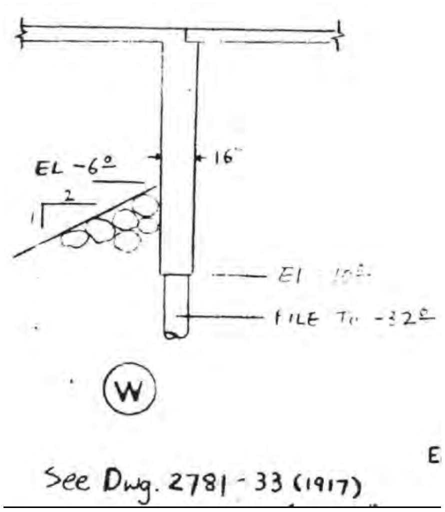


Figure 4-21: Seawall Section 6, Bulkhead Wall Type W

This cutoff wall section and type has structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall supporting concrete pile moment and shear capacity. A summary of the results of this assessment is shown on Table 4-18.

Table 4-18: Structural Assessment Summary – Seawall Section 6, Cutoff Wall Type W

	Seawall Section: 6			Bulkhead Wall Type: W		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.49	0.66	0.68	0.70	0.70	0.70
Pile Strength DCR	0.88	1.66	2.30	2.48	2.60	3.01
Criticality Rating	-0.12	0.66	1.30	1.48	1.60	2.01

The structural criticality ratings range from -0.12 to 2.01 strongly indicating that retrofit of this seawall section and type piles is needed.

Seawall Section 7

Seawall Section 7, Bulkhead Type W

Seawall Section 7 Type W is a concrete bulkhead wall, identical to that of Seawall Section 6 Type W, as shown on Figure 4-22.

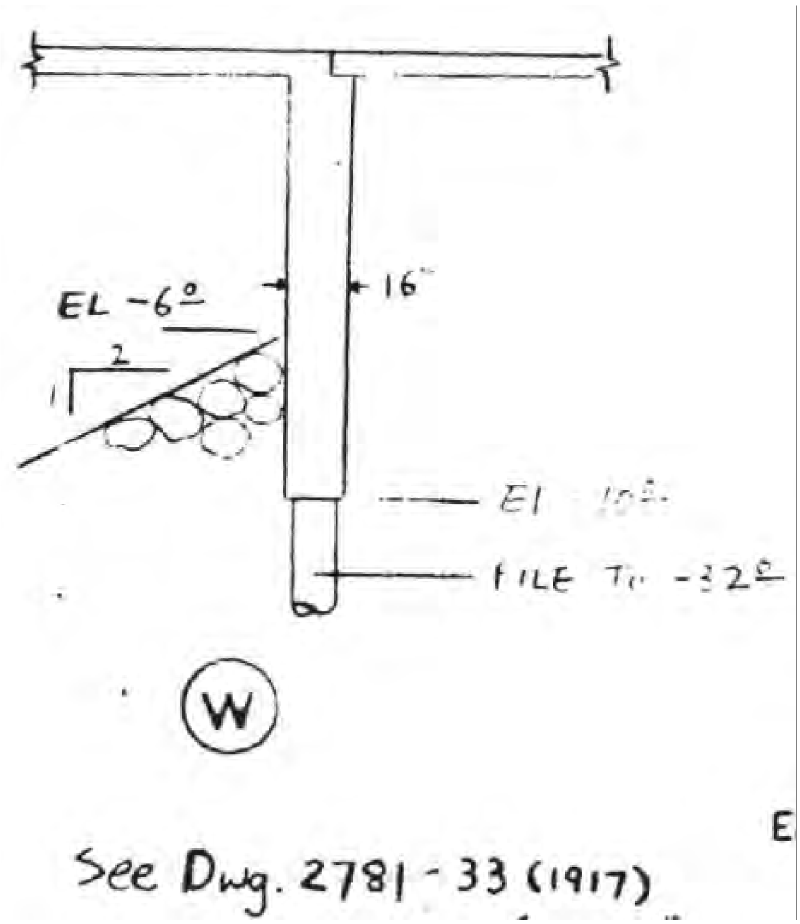


Figure 4-22: Seawall Section 7, Bulkhead Wall Type W

Seawall Section 7, Bulkhead Type V

Seawall Section 7 Type V is a concrete bulkhead wall as shown on Figure 4-23. Actual data on this seawall type was not available but is assumed to be similar to Type U (following). This unreinforced concrete bulkhead wall is assumed to be 15 feet high, 3 feet wide at its top, 4 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — C2
- Bulkhead wall piles (Yes/No): — Yes, but details not known, presumed same as Type U following.
- Wing walls (Yes/No): — Yes, assumed up to 37'-8.625" centers.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes, plus new wharf built over existing
- Finger Pier (Yes/No): — Yes, Pier 1.

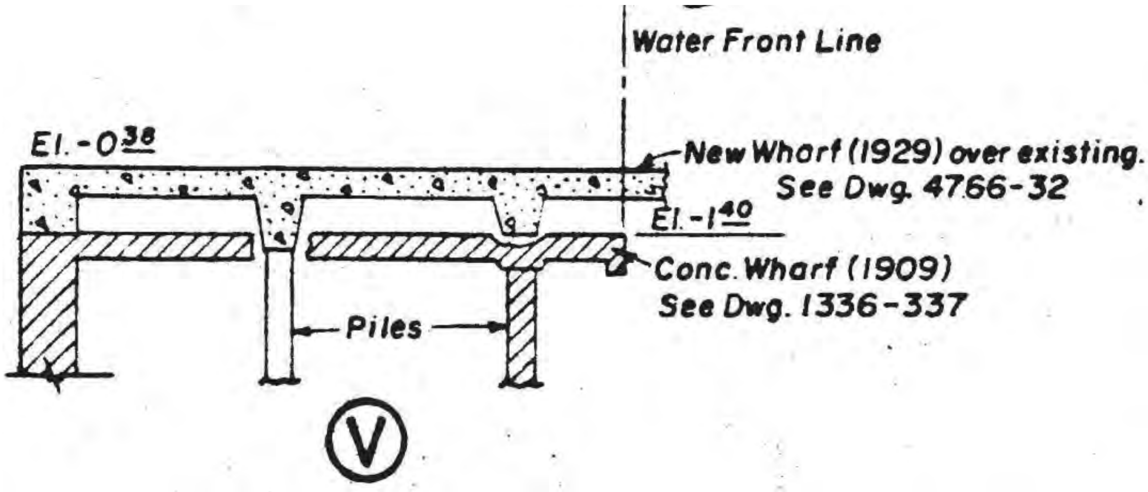


Figure 4-23: Seawall Section 7, Bulkhead Wall Type V

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the marginal wharf pile shear capacity. A summary of the results of this assessment is shown on Table 4-19.

Table 4-19: Structural Assessment Summary – Seawall Section 7, Bulkhead Wall Type V

	Seawall Section: 07			Wall Type: V		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.54	1.52	1.52	1.48	1.42	1.38
FOS-Overturning	6.82	1.79	1.79	1.74	1.65	1.60
Strength DCR	0.39	0.73	0.73	0.75	0.77	0.80
Criticality Rating	-0.61	-0.27	-0.27	-0.25	-0.23	-0.20

The structural criticality ratings range from -0.61 to -0.20 indicating that retrofit of this seawall section and type is not needed.

<u>Seawall Section 7, Bulkhead Type U</u>

Seawall Section 7 Type U is a concrete bulkhead wall as shown on Figure 4-24. This unreinforced concrete bulkhead wall is 15 feet high, 3 feet wide at its top, 4 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile C2
- Bulkhead wall piles (Yes/No): Yes, two rows 2 feet apart, 12” diameter timber piles at 6 feet centers.
- Wing walls (Yes/No): Yes, up to 37’-8.625” centers.
- Wing wall piles (Yes/No): No.
- Marginal wharf attached (Yes/No): Yes.

- Finger Pier (Yes/No): No.

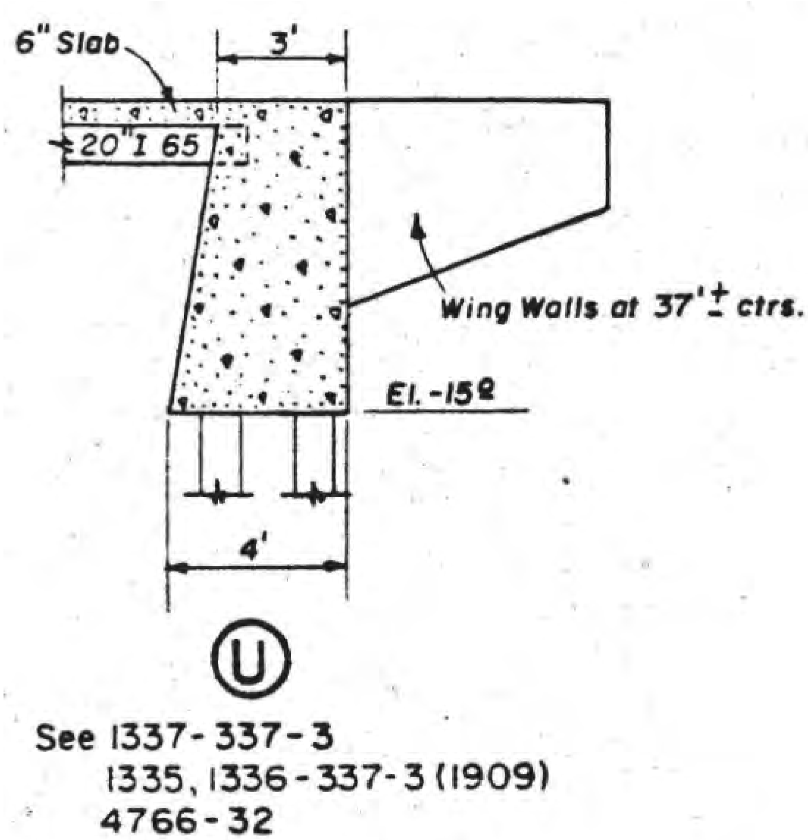


Figure 4-24: Seawall Section 7, Bulkhead Wall Type U

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-20.

Table 4-20: Structural Assessment Summary – Seawall Section 7, Bulkhead Wall Type U

	Seawall Section: 07			Wall Type: U		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.36	1.44	1.44	1.41	1.35	1.31
FOS-Overturning	6.23	1.64	1.64	1.59	1.51	1.46
Strength DCR	0.43	0.81	0.81	0.82	0.86	0.88
Criticality Rating	-0.57	-0.19	-0.19	-0.18	-0.14	-0.12

The structural criticality ratings range from -0.57 to -0.12 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 7, Bulkhead Type T

Seawall Section 7 Type T is a concrete bulkhead wall as shown on Figure 4-25. This unreinforced concrete bulkhead wall is 15 feet high, 3 feet wide at its top, 4 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — C2
- Bulkhead wall piles (Yes/No): — No.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes.
- Finger Pier (Yes/No): — No.

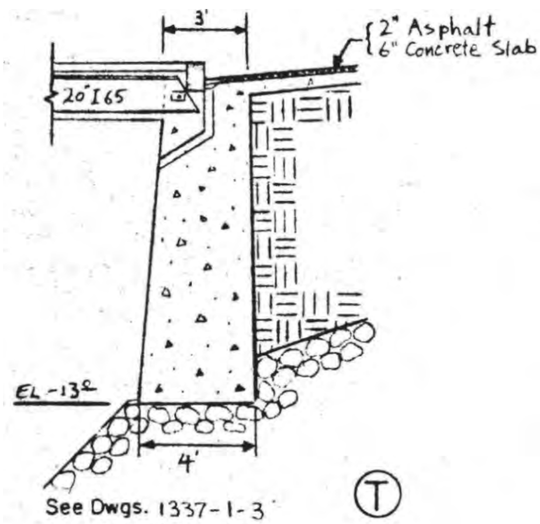


Figure 4-25: Seawall Section 7, Bulkhead Wall Type T

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity.

A summary of the results of this assessment is shown on Table 4-21.

Table 4-21: Structural Assessment Summary – Seawall Section 7, Bulkhead Wall Type T

	Seawall Section: 07			Wall Type: T		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.71	1.47	1.47	1.43	1.37	1.33
FOS-Overturning	10.24	2.40	2.40	2.32	2.20	2.13
Strength DCR	0.33	0.59	0.59	0.61	0.63	0.65
Criticality Rating	-0.67	-0.32	-0.32	-0.30	-0.27	-0.25

The structural criticality ratings range from -0.67 to -0.25 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 8a

Seawall Section 8a Type S is a concrete bulkhead wall as shown on Figure 4-26. This unreinforced concrete bulkhead wall is 17'-10" high, 6 feet wide at its top, 13'-10" wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — C2
- Bulkhead wall piles (Yes/No): — Yes, four rows 4 feet apart, 16" diameter timber piles at 4 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes, structure and pile information not available.
- Finger Pier (Yes/No): — Yes, Ferry Plaza.

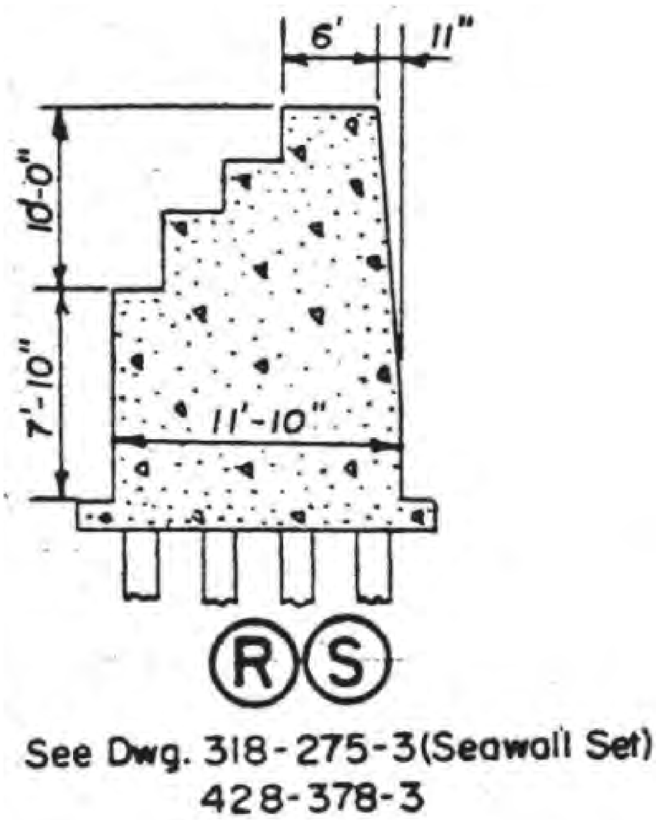


Figure 4-26: Seawall Section 8a, Bulkhead Wall Type S

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under all levels of earthquake loading considered in this study assuming an adequate structural connection to the marginal wharf exists. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-22.

Table 4-22: Structural Assessment Summary – Seawall Section 8a, Bulkhead Wall Type S

	Seawall Section: 08a			Wall Type: S		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	4.23	1.45	1.45	1.40	1.35	1.31
FOS-Overturning	10.38	2.06	2.06	1.99	1.89	1.84
Strength DCR	0.23	0.84	0.84	0.87	0.92	0.95
Criticality Rating	-0.76	-0.16	-0.16	-0.13	-0.08	-0.05

The structural criticality ratings range from -0.76 to -0.05 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 8b

Seawall Section 8b Type R is a concrete bulkhead wall as shown on Figure 4-27. This unreinforced concrete bulkhead wall is17'-10" high, 6 feet wide at its top, 13'-10" wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile C2
- Bulkhead wall piles (Yes/No): Yes, four rows 4 feet apart, 16" diameter timber piles at 4 feet longitudinal centers.
- Wing walls (Yes/No): No.
- Wing wall piles (Yes/No): No.
- Marginal wharf attached (Yes/No): Yes, structure and pile information not available.
- Finger Pier (Yes/No): Yes, Ferry Plaza.

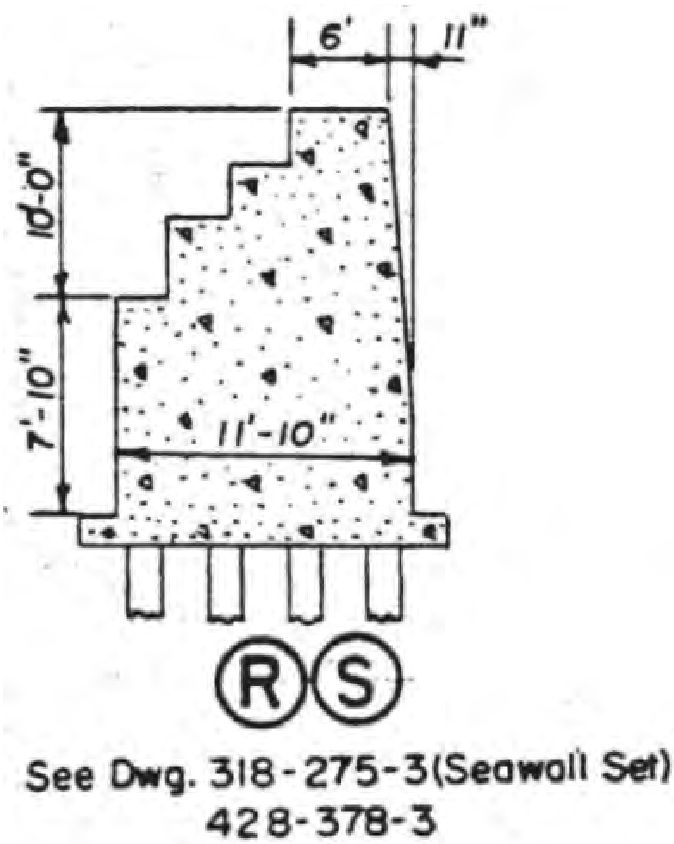


Figure 4-27: Seawall Section 8b, Bulkhead Wall Type R

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under all levels of earthquake loading considered in this study assuming an adequate structural connection to the marginal wharf exists. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-23.

Table 4-23: Structural Assessment Summary – Seawall Section 8b, Bulkhead Wall Type R

	Seawall Section: 08b			Wall Type: R		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	4.23	1.45	1.45	1.40	1.35	1.31
FOS-Overturning	10.38	2.06	2.06	1.99	1.89	1.84
Strength DCR	0.23	0.84	0.84	0.87	0.92	0.95
Criticality Rating	-0.76	-0.16	-0.16	-0.13	-0.08	-0.05

The structural criticality ratings range from -0.76 to -0.05 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 8

Seawall Section 8 Type Q is a concrete bulkhead wall as shown on Figure 4-28. This unreinforced concrete bulkhead wall is 30 feet high, 3 feet wide at its top, 10 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile: C2
- Bulkhead wall piles (Yes/No): Yes, four rows 2'-6" apart, 16" diameter timber piles at 2'-6" longitudinal centers.
- Wing walls (Yes/No): No.
- Wing wall piles (Yes/No): No.
- Marginal wharf attached (Yes/No): Yes, structure and pile information not available or TBD.
- Finger Pier (Yes/No): No (Pier 2 not directly connected).

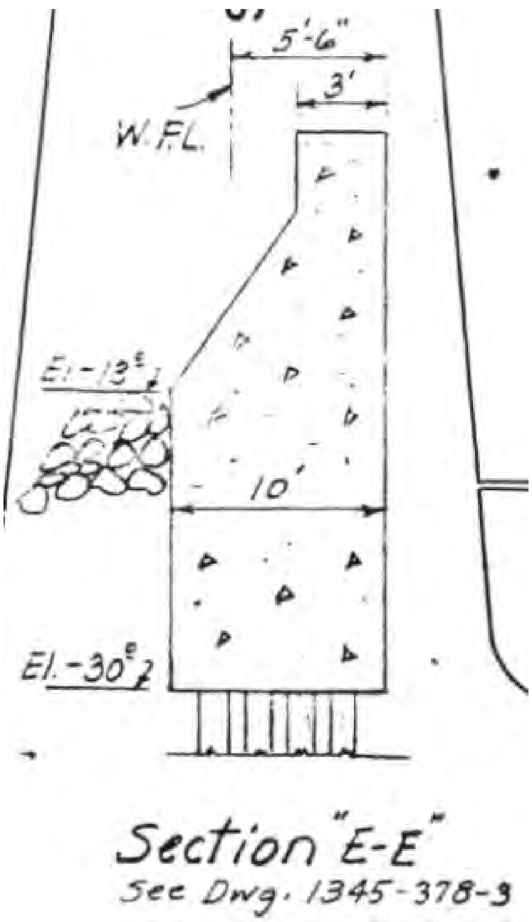


Figure 4-28: Seawall Section 8, Bulkhead Wall Type Q

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-24.

Table 4-24: Structural Assessment Summary – Seawall Section 8, Bulkhead Wall Type Q

	Seawall Section: 08			Wall Type: Q		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	2.57	1.52	1.52	1.50	1.45	1.41
FOS-Overturning	2.46	1.22	1.22	1.20	1.15	1.11
Strength DCR	0.56	1.04	1.04	1.05	1.09	1.12
Criticality Rating	-0.44	0.04	0.04	0.05	0.09	0.12

The structural criticality ratings range from -0.44 to 0.12 indicating that retrofit of this seawall section and type is needed.

Seawall Section 9a

Seawall Section 9a Type P is a concrete bulkhead wall as shown on Figure 4-29. This unreinforced concrete bulkhead wall is 13 feet high, 2 feet wide at its top, 8 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — C2
- Bulkhead wall piles (Yes/No): — Yes, two rows 5 feet apart, 15" diameter timber piles at 4 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes, structure and pile information not available or TBD.
- Finger Pier (Yes/No): — No (Pier 2 not directly connected).

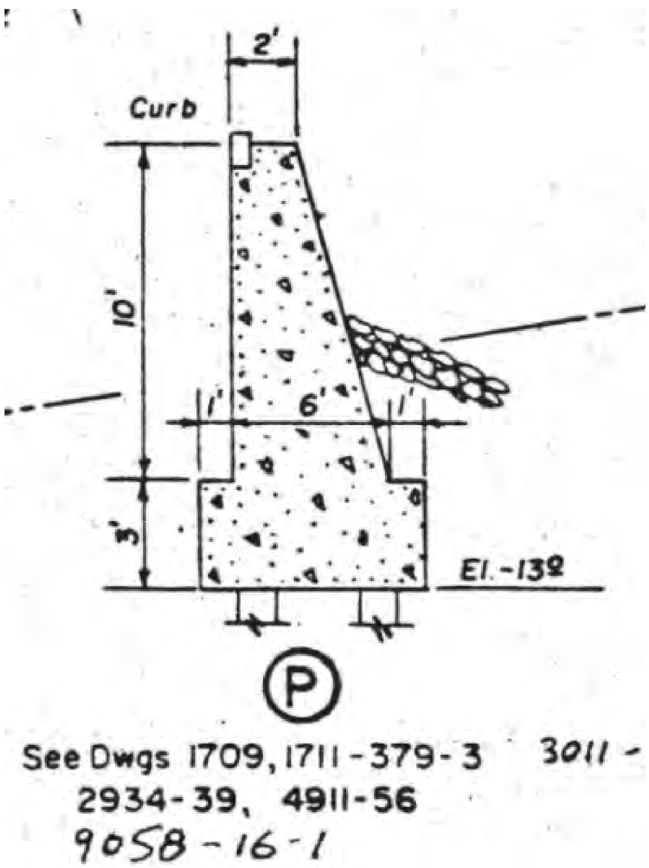


Figure 4-29: Seawall Section 9a, Bulkhead Wall Type P

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-25.

Table 4-25: Structural Assessment Summary – Seawall Section 9a, Bulkhead Wall Type P

	Seawall Section: 09a			Wall Type: P		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	6.86	2.93	2.93	2.86	2.75	2.67
FOS-Overturning	12.25	4.28	4.28	4.18	3.97	3.83
Strength DCR	0.06	0.15	0.15	0.16	0.19	0.21
Criticality Rating	-0.85	-0.66	-0.66	-0.65	-0.64	-0.62

The structural criticality ratings range from -0.85 to -0.62 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 9b

Seawall Section 9b Type P is a concrete bulkhead wall as shown on Figure 4-30. This unreinforced concrete bulkhead wall is 13 feet high, 2 feet wide at its top, 8 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — B1
- Bulkhead wall piles (Yes/No): — Yes, two rows 5 feet apart, 15” diameter timber piles at 4 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — No.
- Finger Pier (Yes/No): — No (Pier 14 not directly connected).

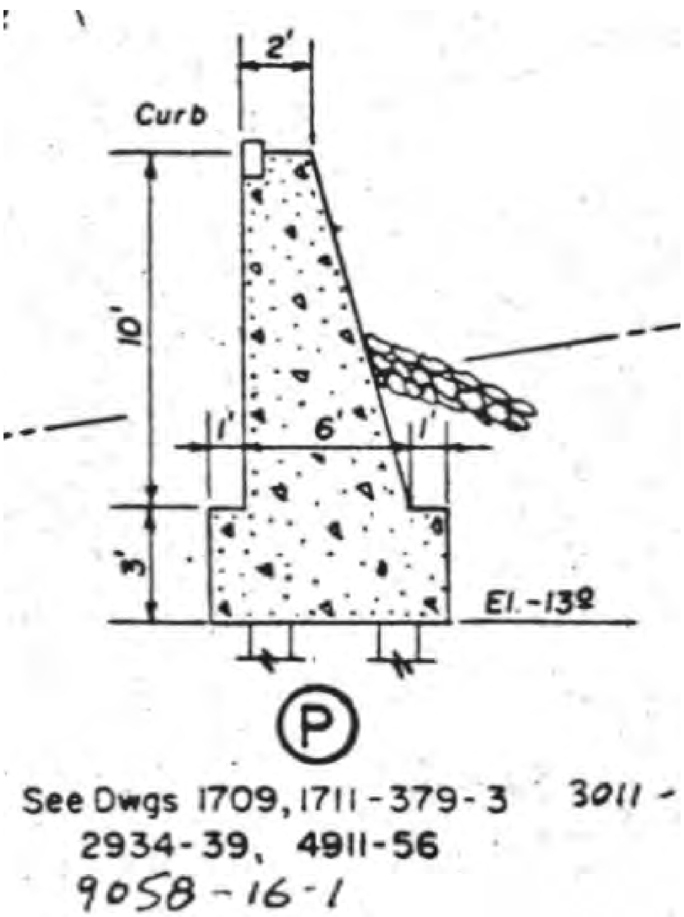


Figure 4-30: Seawall Section 9b, Bulkhead Wall Type P

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under all levels of earthquake loading considered in this study assuming an adequate structural connection to the marginal wharf exists. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-26.

Table 4-26: Structural Assessment Summary – Seawall Section 9b, Bulkhead Wall Type P

	Seawall Section: 09b			Wall Type: P		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	13.21	4.02	3.90	3.62	3.52	3.47
FOS-Overturning	33.44	5.56	5.38	4.94	4.77	4.69
Strength DCR	0.19	0.19	0.19	0.19	0.19	0.19
Criticality Rating	-0.81	-0.75	-0.74	-0.72	-0.72	-0.71

The structural criticality ratings range from -0.81 to -0.71 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 9

Seawall Section 9 Type O is a concrete bulkhead wall as shown on Figure 4-31. This unreinforced concrete bulkhead wall is 30 feet high, 5 feet wide at its top, 10 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — B1
- Bulkhead wall piles (Yes/No): — Yes, four rows 2'-6" apart, 15" diameter timber piles at 3 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes..
- Finger Pier (Yes/No): — Yes, Piers 26 and 28.

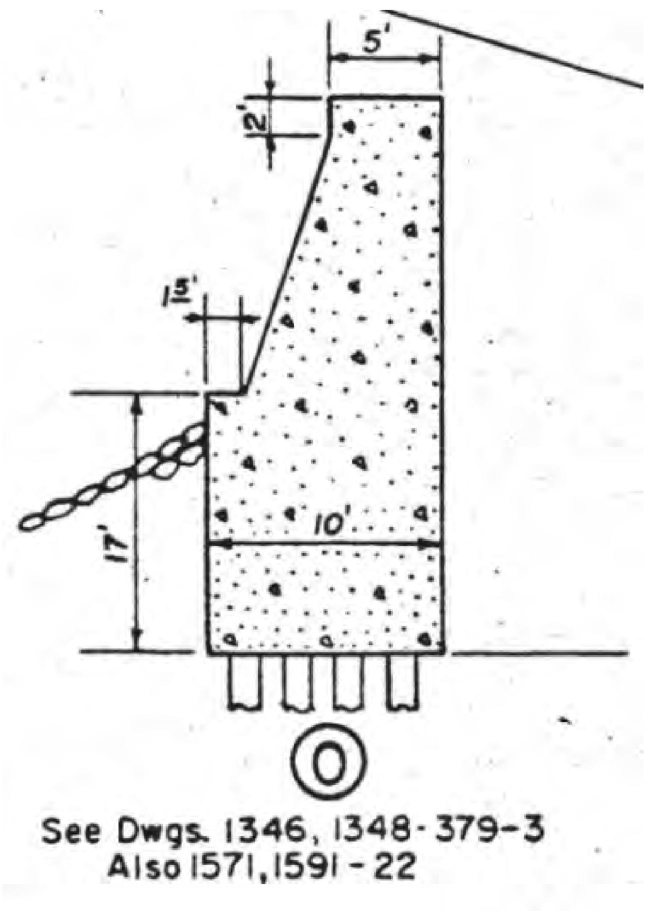


Figure 4-31: Seawall Section 9, Bulkhead Wall Type O

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed unstable in overturning for all levels of earthquake loading considered in this study. This wall section and type has structural strength deficiencies under static load and all levels of earthquake loading considered in this study and assumes an adequate structural connection to the marginal wharf. The critical structural element is the seawall supporting pile shear capacity. A summary of the results of this assessment is shown on Table 4-27.

Table 4-27: Structural Assessment Summary – Seawall Section 9, Bulkhead Wall Type O

	Seawall Section: 09			Wall Type: O		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	2.38	1.28	1.24	1.18	1.16	1.15
FOS-Overturning	2.34	0.91	0.88	0.82	0.80	0.80
Strength DCR	0.69	1.28	1.32	1.39	1.42	1.43
Criticality Rating	-0.31	0.28	0.32	0.39	0.42	0.43

The structural criticality ratings range from -0.31 to +0.43 indicating that retrofit of this seawall section and type is needed.

Seawall Section 10

<u>Seawall Section 10, Bulkhead Type N</u>

Seawall Section 10 Type N is a concrete bulkhead wall as shown on Figure 4-32. This unreinforced concrete bulkhead wall is 30 feet high, 5 feet wide at its top, 11.5 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — A2
- Bulkhead wall piles (Yes/No): — Yes, four rows 2'-8" apart, 14" diameter timber piles at 3 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes.
- Finger Pier (Yes/No): — Yes, Pier 30-32.

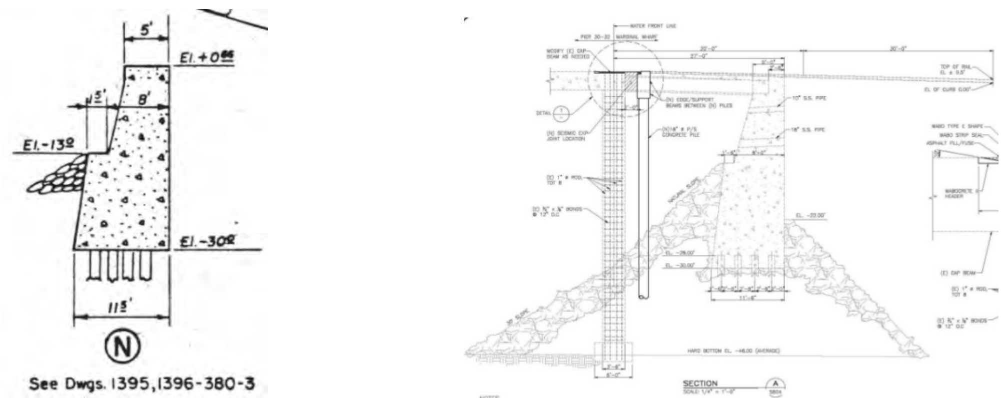


Figure 4-32: Seawall Section 10, Bulkhead Wall Type N

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength and assumes an adequate structural connection to the adjacent marginal wharf. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-28.

Table 4-28: Structural Assessment Summary – Seawall Section 10, Bulkhead Wall Type N

	Seawall Section: 10			Wall Type: N		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	2.57	1.41	1.36	1.24	1.18	1.09
FOS-Overturning	2.91	1.23	1.18	1.05	0.98	0.89
Strength DCR	0.58	1.14	1.18	1.31	1.39	1.52
Criticality Rating	-0.42	0.14	0.18	0.31	0.39	0.52

The structural criticality ratings range from -0.42 to +0.52indicating that retrofit of this seawall section and type is needed.

Seawall Section 10, Bulkhead Type M

Seawall Section 10 Type M is a concrete bulkhead wall as shown on Figure 4-33. This unreinforced concrete bulkhead wall is 14 feet high, 2.5 feet wide at its top, 6.5 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — A2
- Bulkhead wall piles (Yes/No): — Yes, two rows 2'-10" apart, 15" diameter timber piles assumed at 8 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — Yes.
- Finger Pier (Yes/No): — Yes, Pier 32 (of 30-32).

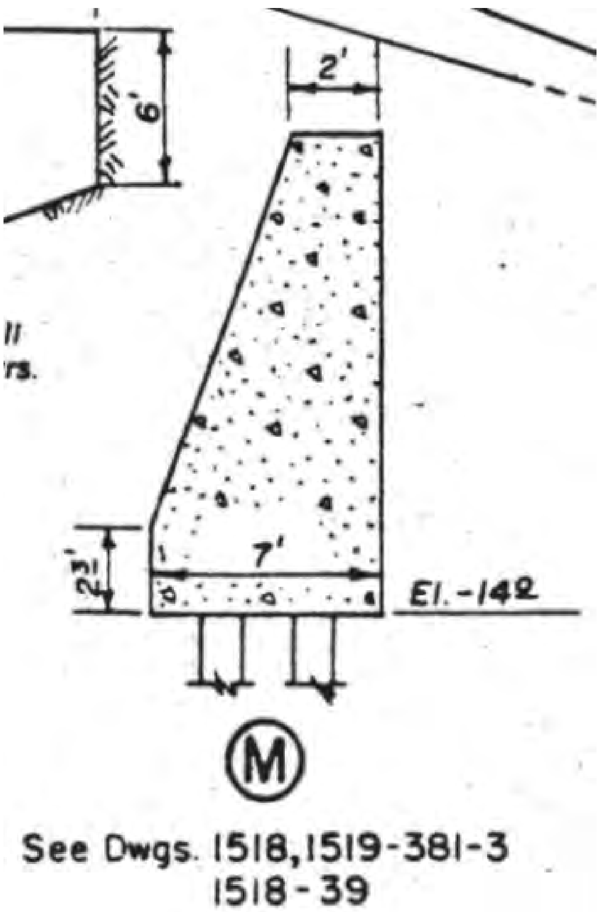


Figure 4-33: Seawall Section 10, Bulkhead Wall Type M

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength and assumes an adequate structural connection to the adjacent marginal wharf. This wall is deemed unstable in overturning for the MCE level of earthquake loading considered in this study. This wall section and type has a structural strength deficiencies under the three highest levels of earthquake loading considered in this study. The critical structural element is the marginal wharf pile shear capacity. A summary of the results of this assessment is shown on Table 4-29.

Table 4-29: Structural Assessment Summary – Seawall Section 10, Bulkhead Wall Type M

	Seawall Section: 10			Wall Type: M		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.65	1.31	1.26	1.11	1.03	0.93
FOS-Overturning	6.87	1.39	1.33	1.15	1.05	0.94
Strength DCR	0.44	0.95	0.99	1.10	1.18	1.29
Criticality Rating	-0.56	-0.05	-0.01	0.10	0.18	0.29

The structural criticality ratings range from -0.56 to +0.29 indicating that retrofit of this seawall section and type is needed.

Seawall Section 11a

Seawall Section 11a Type M is a concrete bulkhead wall as shown on Figure 4-34. This seawall type was retrofitted with additional revetment as a part of the Brannan Street Wharf (BSW) project in 2013. This unreinforced concrete bulkhead wall is 14 feet high, 2.5 feet wide at its top, 6.5 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Bulkhead wall piles (Yes/No): — Yes, two rows 3'-6" apart, 15" diameter timber piles assumed at 8 feet longitudinal centers.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — No (BSW is not connected.)
- Finger Pier (Yes/No): — No

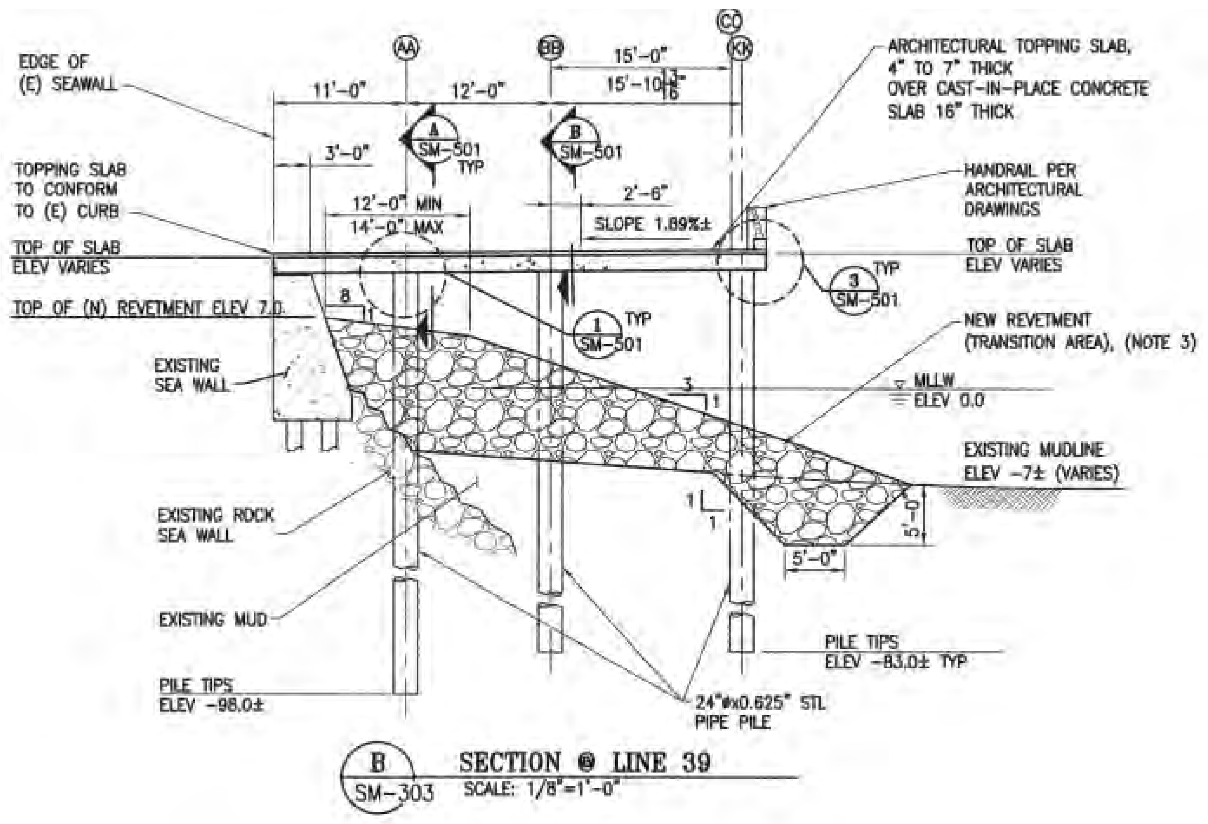


Figure 4-34: Seawall Section 11a, Bulkhead Wall Type M

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-30.

Table 4-30: Structural Assessment Summary – Seawall Section 11a, Bulkhead Wall Type M

	Seawall Section: 11a			Wall Type: M		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	5.51	2.30	2.21	1.97	1.85	1.68
FOS-Overturning	7.53	2.49	2.37	2.08	1.93	1.74
Strength DCR	0.06	0.46	0.49	0.60	0.67	0.77
Criticality Rating	-0.82	-0.54	-0.51	-0.40	-0.33	-0.23

The structural criticality ratings range from -0.82 to -0.23 indicating that retrofit of this seawall section and type is not needed.

Seawall Section 11

Seawall Section 11 Type L is a concrete bulkhead wall as shown on Figure 4-36. This unreinforced concrete bulkhead wall is 20 feet high, 3 feet wide at its top, 7 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Bulkhead wall piles (Yes/No): — Yes, two rows 3 feet apart, 16" diameter timber piles assumed at 6 feet longitudinal centers.
- Wing walls (Yes/No): — Yes, at 20 feet spacing.
- Wing wall piles (Yes/No): — Yes.
- Marginal wharf attached (Yes/No): — No (BSW is not connected.)
- Finger Pier (Yes/No): — No

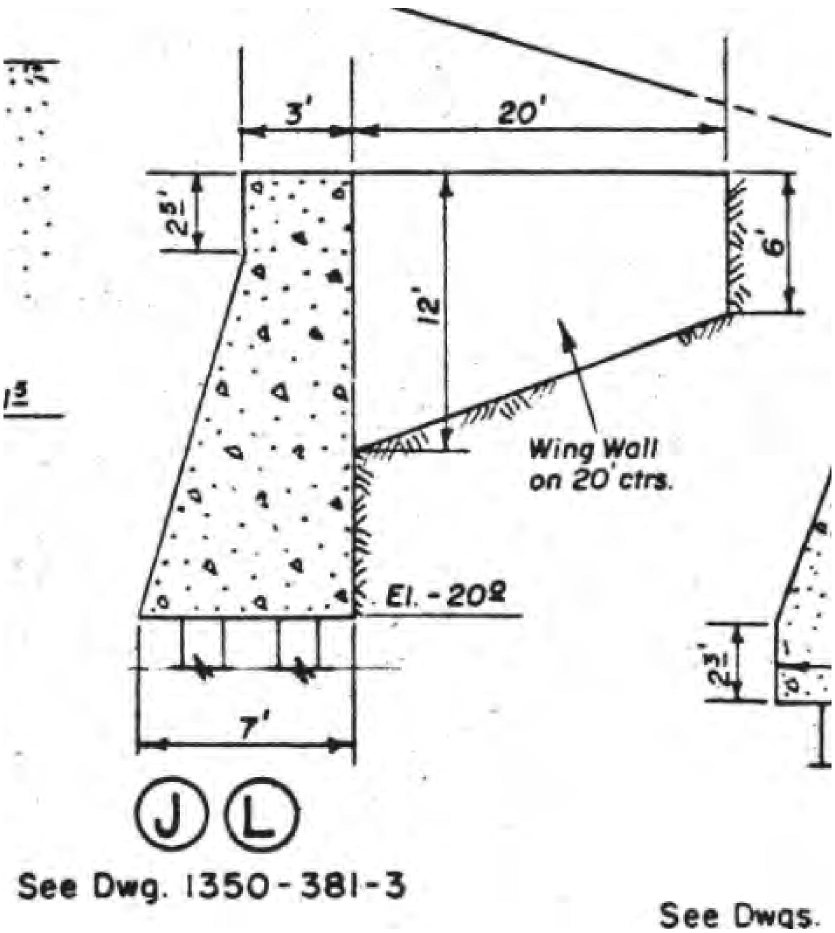


Figure 4-35: Seawall Section 11, Bulkhead Wall Type L

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed unstable for the three highest levels of earthquake loading considered in this study. This wall section and type has structural strength deficiencies under all levels of earthquake loading considered in this study. The critical structural element is the seawall wing wall supporting timber pile shear capacity.

A summary of the results of this assessment is shown on Table 4-31.

Table 4-31: Structural Assessment Summary – Seawall Section 11, Bulkhead Wall Type L

	Seawall Section: 11			Wall Type: L		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	2.54	1.29	1.24	1.12	1.06	0.98
FOS-Overturning	2.83	1.09	1.04	0.92	0.86	0.78
Strength DCR	0.72	1.86	1.92	2.13	2.25	2.45
Criticality Rating	-0.28	0.86	0.92	1.13	1.25	1.45

The structural criticality ratings range from -0.28 to +1.45 strongly indicating that retrofit of this seawall section and seawall type is needed.

Seawall Section 12

Seawall Section 12, Bulkhead Type K (at Brannan Street Wharf)

Seawall Section 12 Type K is a concrete bulkhead wall as shown on Figure 4-36. This unreinforced concrete bulkhead wall is 12 feet high, 2 feet wide at its top, 4 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile	A1
- Bulkhead wall piles (Yes/No):	No.
- Wing walls (Yes/No):	Yes, at 20 foot spacing.
- Wing wall piles (Yes/No):	Yes.
- Marginal wharf attached (Yes/No):	No (BSW is not connected).
- Finger Pier (Yes/No):	No.

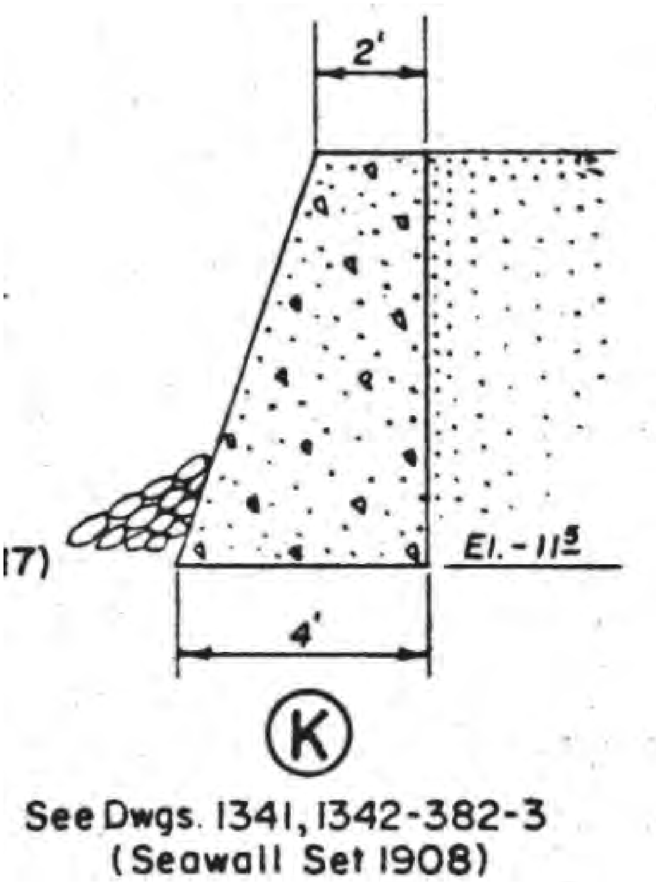


Figure 4-36: Seawall Section 12, Bulkhead Wall Type K (at BSW)

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed unstable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall wing wall shear capacity. A summary of the results of this assessment is shown on Table 4-32.

Table 4-32: Structural Assessment Summary – Seawall Section 12, Bulkhead Wall Type K (Aat BSW)

	Seawall Section: 12	Wall Type: K (BSW)				
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	1.43	0.61	0.58	0.52	0.49	0.44
FOS-Overturning	4.18	1.28	1.22	1.07	0.99	0.89
Strength DCR	0.18	0.18	0.18	0.18	0.18	0.20
Criticality Rating	-0.30	0.65	0.72	0.93	1.06	1.26

The structural criticality ratings range from -0.30 to +1.26 indicating that retrofit of this seawall section and type is needed.

Seawall Section 12, Bulkhead Type K (at Pier 38)

Seawall Section 12 Type K is a concrete bulkhead wall as shown on Figure 4-37. This unreinforced concrete bulkhead wall is 12 feet high, 2 feet wide at its top, 4 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile	A1
- Bulkhead wall piles (Yes/No):	No.
- Wing walls (Yes/No):	Yes, at 20 foot spacing.
- Wing wall piles (Yes/No):	Yes.
- Marginal wharf attached (Yes/No):	Yes (at Pier 38).
- Finger Pier (Yes/No):	Yes, Pier 38.

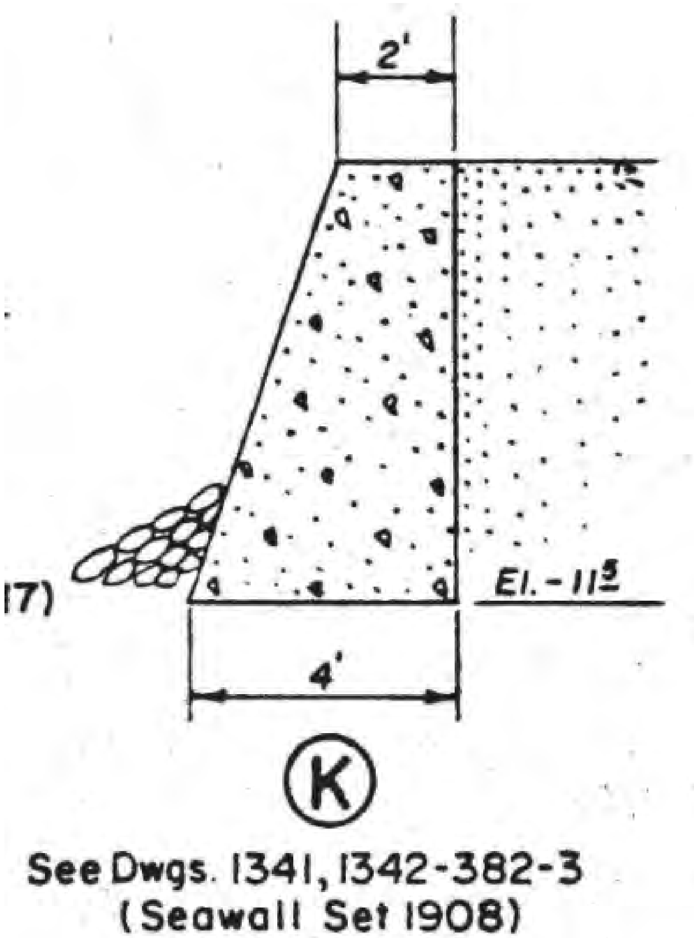


Figure 4-37: Seawall Section 12, Bulkhead Wall Type K (at Pier 38)

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed unstable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall unreinforced concrete moment capacity. A summary of the results of this assessment is shown on Table 4-33.

Table 4-33: Structural Assessment Summary – Seawall Section 12, Bulkhead Wall Type K (at Pier 38)

	Seawall Section: 12 (P38)			Wall Type: K		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	1.43	0.45	0.43	0.38	0.35	0.31
FOS-Overturning	4.18	0.69	0.66	0.57	0.52	0.46
Strength DCR	0.18	0.21	0.23	0.27	0.30	0.35
Criticality Rating	-0.30	1.24	1.34	1.66	1.88	2.19

The structural criticality ratings range from -0.30 to +2.19 strongly indicating that retrofit of this seawall section and type is needed.

Seawall Section 12, Bulkhead Type J

Seawall Section 12 Type J is a concrete bulkhead wall as shown on Figure 4-38. This unreinforced concrete bulkhead wall is 20 feet high, 3 feet wide at its top, 7 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Bulkhead wall piles (Yes/No): — Yes, 2 rows 3 feet apart at 6 feet longitudinal spacing.
- Wing walls (Yes/No): — Yes, at 20 foot spacing.
- Wing wall piles (Yes/No): — Yes.
- Marginal wharf attached (Yes/No): — Yes (at Pier 38).
- Finger Pier (Yes/No): — No..

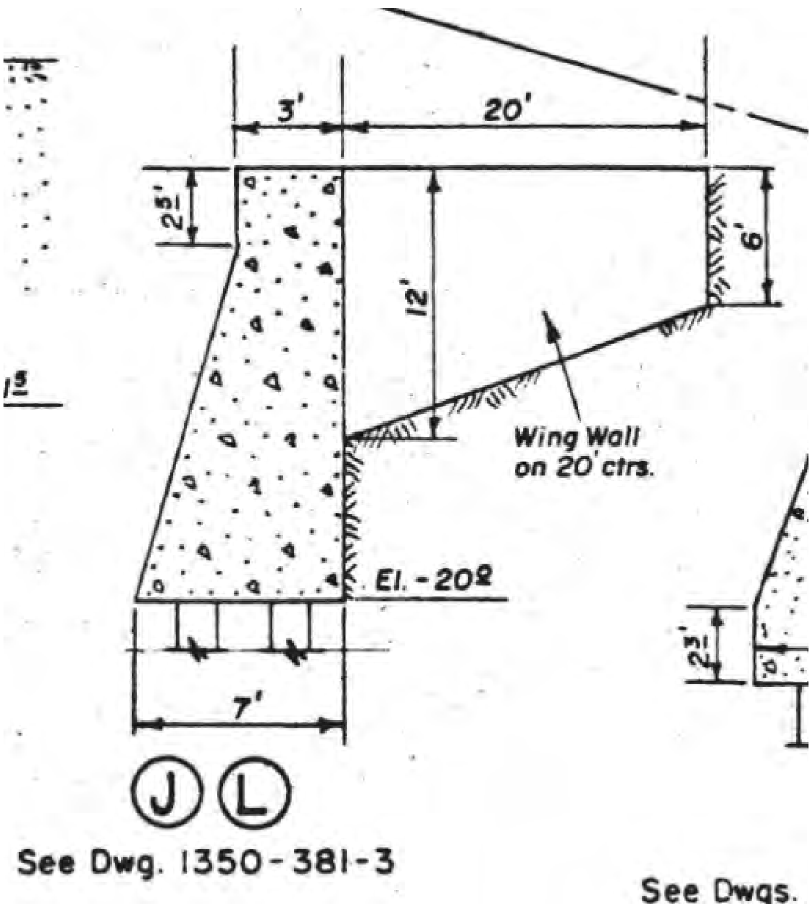


Figure 4-38: Seawall Section 12, Bulkhead Wall Type J

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed stable for all levels of earthquake loading considered in this study. This wall section and type has structural strength deficiencies under the three highest levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-34.

Table 4-34: Structural Assessment Summary – Seawall Section 12, Bulkhead Wall Type J

	Seawall Section: 12			Wall Type: J		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	3.40	1.52	1.46	1.31	1.23	1.12
FOS-Overturning	5.79	1.64	1.57	1.37	1.26	1.13
Strength DCR	0.39	0.89	0.93	1.05	1.12	1.23
Criticality Rating	-0.61	-0.11	-0.07	0.05	0.12	0.23

The structural criticality ratings range from -0.61 to +0.23 indicating that retrofit of this seawall section and type is needed.

Seawall Section 13

Seawall Section 13, Bulkhead Type I

Seawall Section 13 Type I is a concrete bulkhead wall as shown on Figure 4-39. This unreinforced concrete bulkhead wall is 9.5 feet high, 1'-4" wide at its top, 3 feet wide at its base with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Bulkhead wall piles (Yes/No): — Yes, single row, assumed at 6 feet longitudinal spacing.
- Wing walls (Yes/No): — No.
- Wing wall piles (Yes/No): — No.
- Marginal wharf attached (Yes/No): — No.
- Finger Pier (Yes/No): — No.

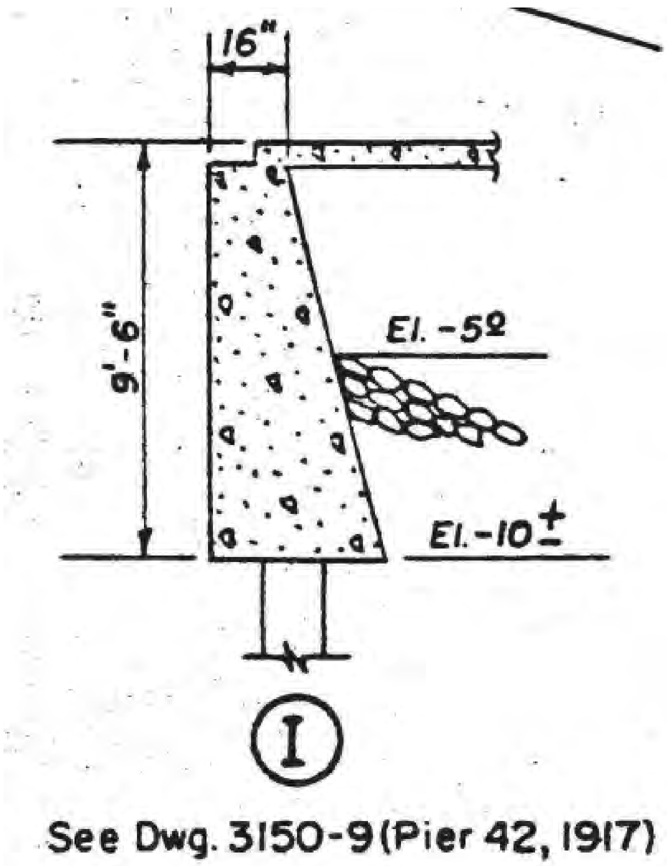


Figure 4-39: Seawall Section 13, Bulkhead Wall Type I

This bulkhead wall was assessed for wall overturning and sliding stability and structural strength. This wall is deemed unstable for all levels of earthquake loading considered in this study. This wall section and type has no structural strength deficiencies under static load and all levels of earthquake loading considered in this study. The critical structural element is the seawall supporting timber pile shear capacity. A summary of the results of this assessment is shown on Table 4-35.

Table 4-35: Structural Assessment Summary – Seawall Section 13, Bulkhead Wall Type I

	Seawall Section: 13			Wall Type: I		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
FOS-Sliding	5.47	2.61	2.51	2.25	2.12	1.94
FOS-Overturning	1.55	0.61	0.58	0.51	0.48	0.43
Strength DCR	0.15	0.48	0.50	0.58	0.62	0.70
Criticality Rating	-0.35	0.64	0.72	0.95	1.09	1.32

The structural criticality ratings range from -0.35 to +1.32 strongly indicating that retrofit of this seawall section and type for stability is needed.

Seawall Section 13, Bulkhead Type H

Seawall Section 13 Type H is a concrete cutoff wall as shown on Figure 4-40. This reinforced concrete cutoff wall panel is 5 feet high, 10 feet wide between supporting piles, 6 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile A1
- Rock dike elevation at seawall: Unknown, assumed -5.0 ft City Datum based on wall height.
- Wall panel size and spacing: 5.0 ft high, 10.0 ft wide, 6 inches thick.
- Wall panel reinforcement: 1/2" SQ at 12” spacing each way, each face.
- Cutoff wall piles (Yes/No): Yes, 22” octagonal jacket on 14” timber piles at 10’-0” spacing.
- Pile reinforcement: Ten #6 longitudinal bars, W6@4”
- Marginal wharf attached (Yes/No): No.
- Finger Pier (Yes/No): No.

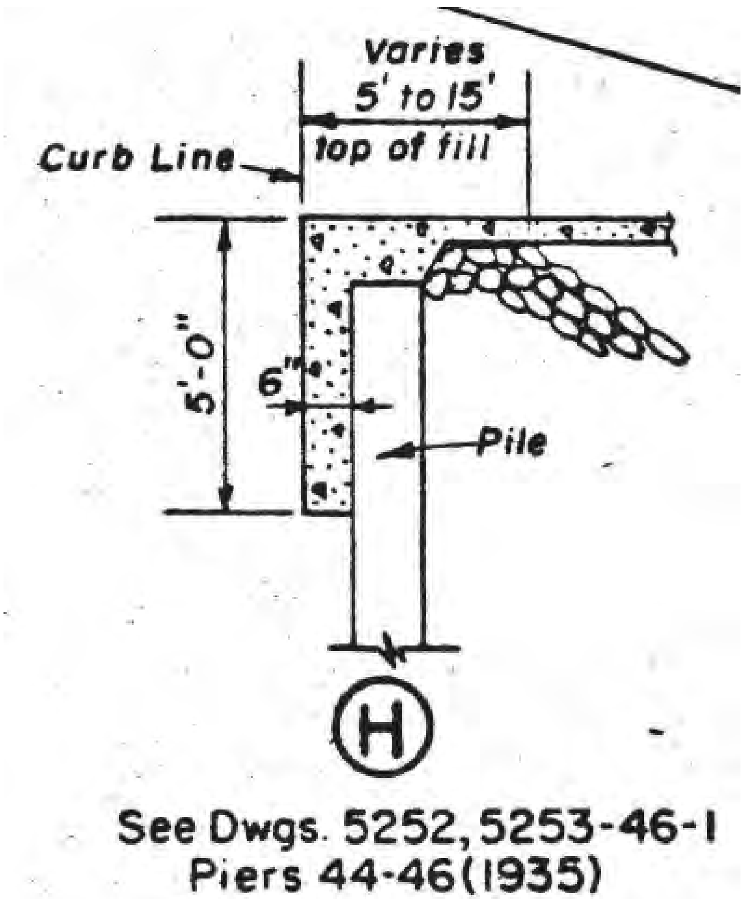


Figure 4-40: Seawall Section 13, Bulkhead Wall Type H

This cutoff wall section and type has structural strength deficiency under the highest three levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall panel moment capacity. A summary of the results of this assessment is shown on Table 4-36.

Table 4-36: Structural Assessment Summary – Seawall Section 13, Cutoff Wall Type H

	Seawall Section: 13			Wall Type: H		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.80	0.97	0.99	1.04	1.06	1.10
Pile Strength DCR	0.11	0.15	0.17	0.19	0.20	0.21
Criticality Rating	-0.20	-0.03	-0.01	0.04	0.06	0.10

The structural criticality ratings range from -0.20 to +0.10 indicating that retrofit of this seawall section and type panels may be needed.

Seawall Section P46

Seawall Section P46 Type G is a concrete cutoff wall as shown on Figure 4-41 and Figure 4-42. This reinforced concrete cutoff wall panel is 14'-10" high with effective tie-backs at Elevations 0.0 and -5.16 feet, 10 feet wide between supporting piles, 8 inches thick with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Rock dike elevation at seawall: — Unknown, assumed -14.83 ft City Datum based on wall height.
- Wall panel size and spacing: — 14.83 ft high, 10.0 ft wide, 8 inches thick.
- Wall panel reinforcement: — Assumed 1/2" SQ at 12" spacing each way, each face.
- Cutoff wall piles (Yes/No): — Yes, 22" octagonal jacket on 14" timber piles at 10'-0" spacing.
- Pile reinforcement: — Ten #6 longitudinal bars, W6@4"
- Marginal wharf attached (Yes/No): — No.
- Finger Pier (Yes/No): — No.

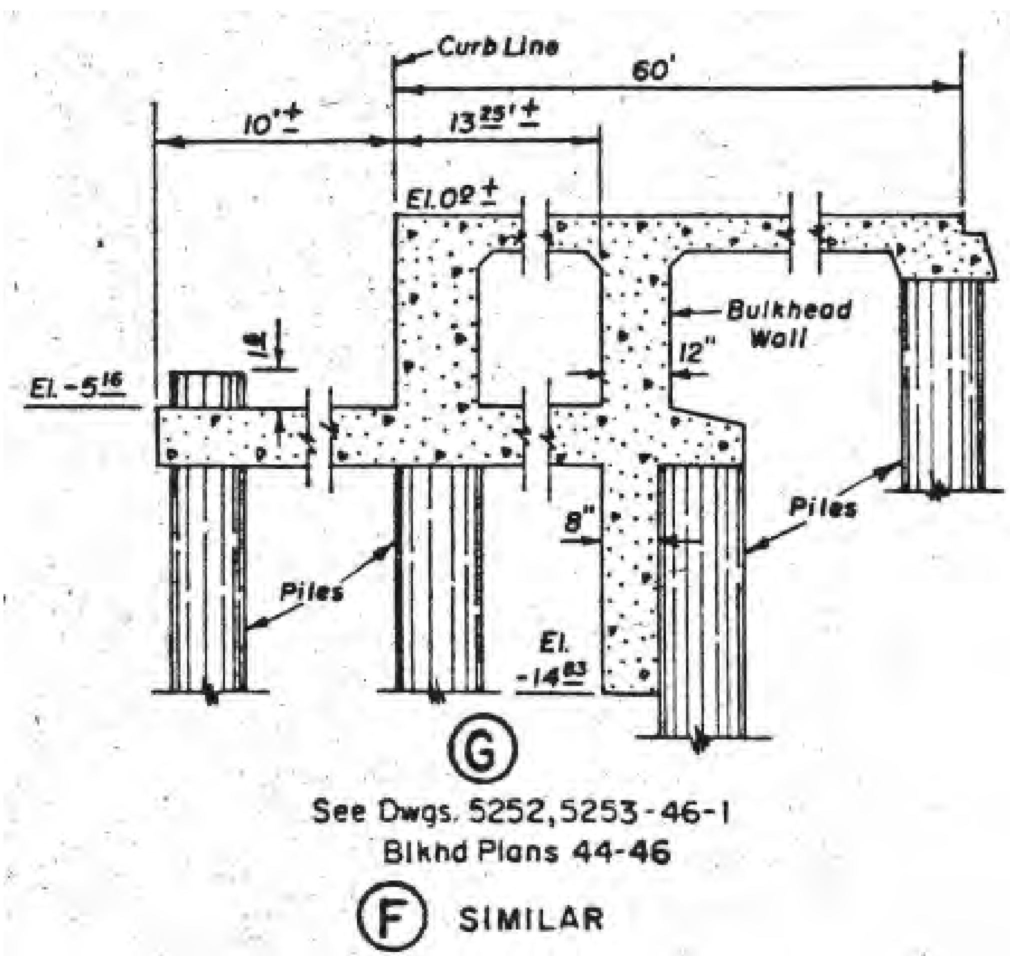


Figure 4-41: Seawall Section P46, Bulkhead Wall Type G

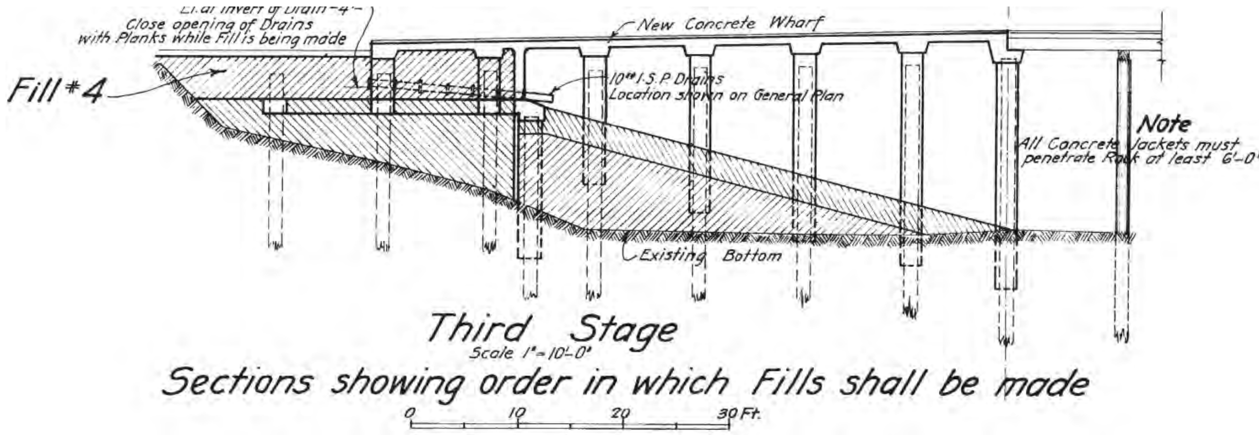


Figure 4-42: Seawall Section P46, Bulkhead Wall Type G

This cutoff wall section and type has no structural strength deficiency under static and all levels of earthquake loading considered in this study. The critical structural element is the seawall cutoff wall panel moment capacity. A summary of the results of this assessment is shown on Table 4-37.

Table 4-37: Structural Assessment Summary – Seawall Section P46, Cutoff Wall Type G (Type F Similar)

	Seawall Section: P46			Wall Type: G		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Panel Strength DCR	0.27	0.36	0.37	0.39	0.40	0.42
Pile Strength DCR	0.10	0.15	0.18	0.20	0.20	0.22
Criticality Rating	-0.73	-0.64	-0.63	-0.61	-0.60	-0.58

The structural criticality ratings range from -0.73 to -0.58 strongly indicating that retrofit of this seawall section and type is not needed.

<u>**Seawall Section P46, Seawall TypeF**</u>

Seawall Section P46 Type F is a concrete bulkhead wall, identical to that of Seawall Section P46 Type G, as shown on Figure 4-41 and Figure 4-42 above.

Seawall Section AT&T

<u>Seawall Section AT&T, Bulkhead TypeE</u>

Seawall Section AT&T Type E is a concrete sheet pile wall as shown on Figure 4-43. This reinforced concrete sheet pile is 16 feet high, 16 inches thick, with an effective tie-back at Elevations -6.0 feet, with additional components as follows:

- Applicable Geotechnical Soil Profile A1
- Rock dike elevation at seawall: Unknown, assumed -16 ft City Datum based on wall height.
- Wall panel size and spacing: Assumed 16 ft high, 2.0 ft wide, 16 inches thick.
- Sheet pile reinforcement: Four 1" SQ longitudinal bars each face, W3@4"
- Marginal wharf attached (Yes/No): No.
- Finger Pier (Yes/No): No.

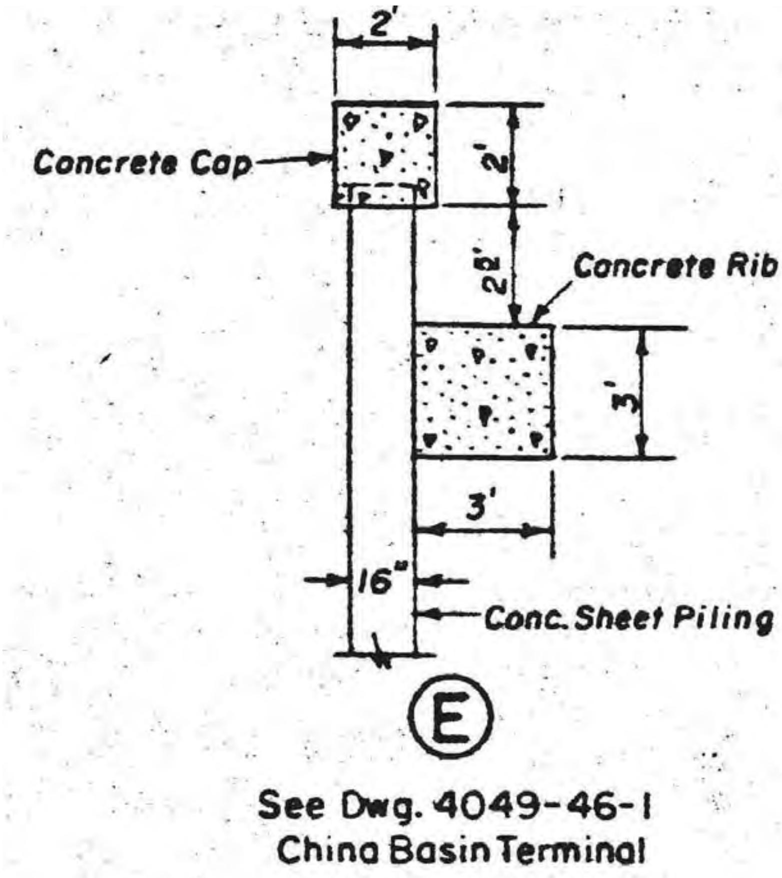


Figure 4-43: Seawall Section AT&T, Bulkhead Wall Type E

This cutoff wall section and type has structural strength deficiency under static and all levels of earthquake loading considered in this study. The critical structural element is the seawall concrete sheet pile moment capacity. A summary of the results of this assessment is shown on Table 4-38.

Table 4-38: Structural Assessment Summary – Seawall Section AT&T, Cutoff Wall Type E

	Seawall Section: AT&T			Wall Type: E		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Sheet Pile Strength DCR	1.04	1.46	1.76	1.90	2.03	2.18
Criticality Rating	0.04	0.46	0.76	0.90	1.03	1.18

The structural criticality ratings range from +0.04 to +1.18 indicating that retrofit of this seawall section and type is needed.

Seawall Section AT&T, Bulkhead TypeD

Seawall Section AT&T Type D is a concrete sheet pile wall as shown on Figure 4-44. Top of backfill soil is at Elevation -8.0 ft City Datum. This reinforced concrete sheet pile is 37 feet high, 16 inches thick, with an effective tie-back at Elevations -8.0 feet, with additional components as follows:

- Applicable Geotechnical Soil Profile: A1
- Rock dike elevation at seawall: Assumed -37 ft City Datum based on wall height.
- Wall panel size and spacing: Assumed 37 ft high, 2.0 ft wide, 16 inches thick.
- Sheet pile reinforcement: Four 1" SQ longitudinal bars each face, W3@4"
- Marginal wharf attached (Yes/No): No.
- Finger Pier (Yes/No): No.

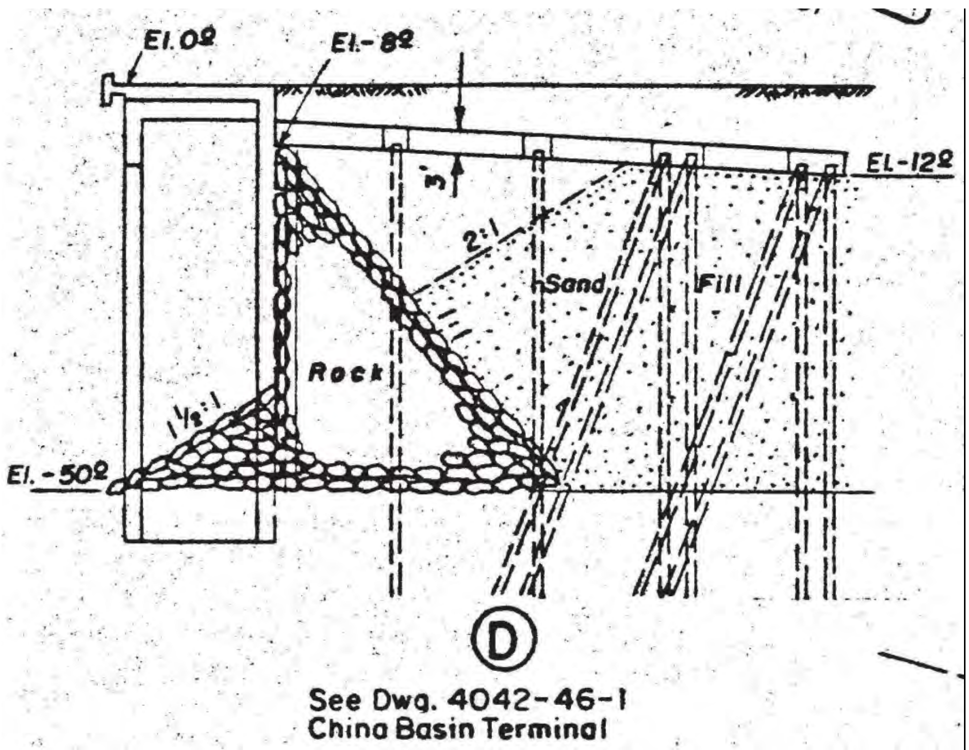


Figure 4-44: Seawall Section AT&T, Bulkhead Wall Type D

This cutoff wall section and type has structural strength deficiency under static and all levels of earthquake loading considered in this study. The critical structural element is the seawall concrete sheet pile moment capacity. A summary of the results of this assessment is shown on Table 4-39.

Table 4-39: Structural Assessment Summary – Seawall Section AT&T, Cutoff Wall Type D

	Seawall Section: AT&T			Wall Type: E		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Sheet Pile Strength DCR	0.90	1.36	1.69	1.85	1.99	2.15
Criticality Rating	-0.10	0.36	0.69	0.85	0.99	1.15

The structural criticality ratings range from -0.10 to +1.15 strongly indicating that retrofit of this seawall section and type is needed.

<u>Seawall Section AT&T, Bulkhead TypeC</u>

Seawall Section AT&T Type C is a concrete sheet pile / arch wall as shown on Figure 4-45. Top of backfill soil is at Elevation -5.0 ft City Datum. This reinforced concrete sheet pile is 16 feet high, 16 inches thick, with an effective tie-back at Elevations -0.0 feet, with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Rock dike elevation at seawall: — Assumed -16 ft City Datum based on wall height.
- Wall panel size and spacing: — Assumed 16 ft high, 2.0 ft wide, 16 inches thick.
- Sheet pile reinforcement: — Four 1" SQ longitudinal bars each face, W3@4"
- Marginal wharf attached (Yes/No): — No.
- Finger Pier (Yes/No): — No.

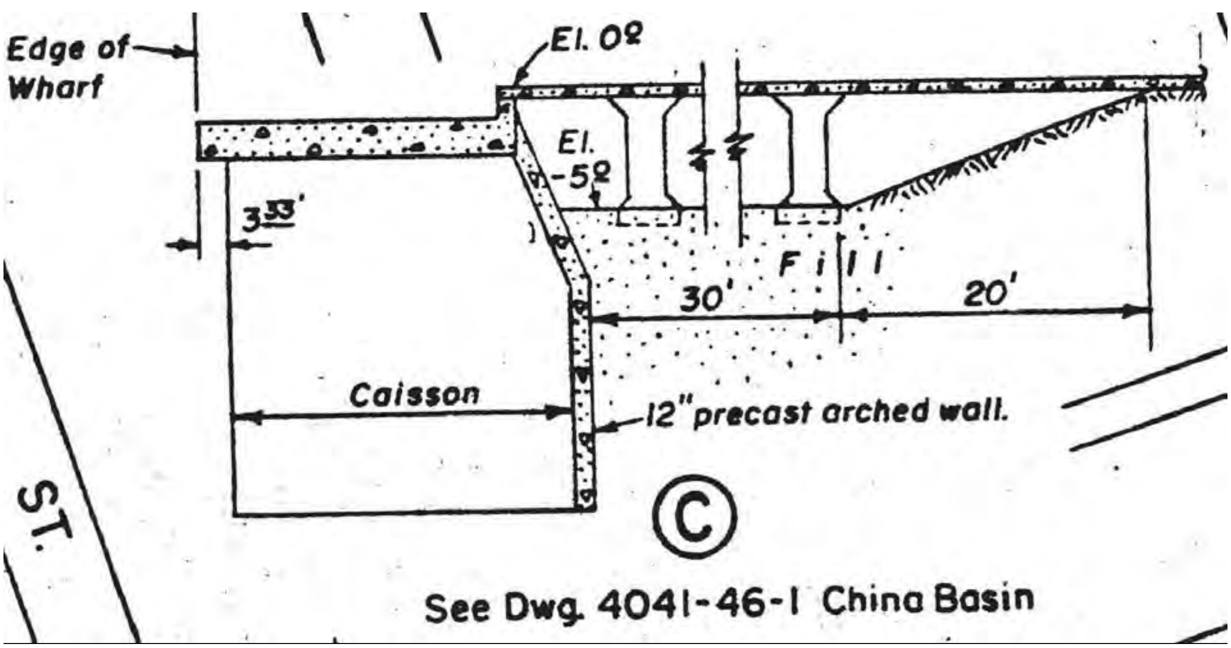


Figure 4-45: Seawall Section AT&T, Bulkhead Wall Type C

This cutoff wall section and type has no structural strength deficiency under all levels of earthquake loading considered in this study. The critical structural element is the seawall concrete sheet pile moment capacity.

A summary of the results of this assessment is shown on Table 4-40.

Table 4-40: Structural Assessment Summary – Seawall Section AT&T, Cutoff Wall Type C

	Seawall Section: AT&T			Wall Type: E		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Sheet Pile Strength DCR	0.38	0.43	0.47	0.49	0.50	0.52
Criticality Rating	-0.62	-0.57	-0.53	-0.51	-0.50	-0.48

The structural criticality ratings range from -0.62 to -0.48 indicating that no retrofit of this seawall section and type is needed

<u>Seawall Section AT&T, Bulkhead TypeB</u>

Seawall Section AT&T Type B is a concrete sheet pile wall as shown on Figure 4-46. This reinforced concrete sheet pile is 16 feet high, 16 inches thick, with an effective tie-back at Elevations -6.0 feet, with additional components as follows:

- Applicable Geotechnical Soil Profile — A1
- Rock dike elevation at seawall: — Unknown, assumed -16 ft City Datum based on wall height.
- Wall panel size and spacing: — Assumed 16 ft high, 2.0 ft wide, 16 inches thick.
- Sheet pile reinforcement: — Four 1” SQ longitudinal bars each face, W3@4”
- Marginal wharf attached (Yes/No): — No.
- Finger Pier (Yes/No): — No.

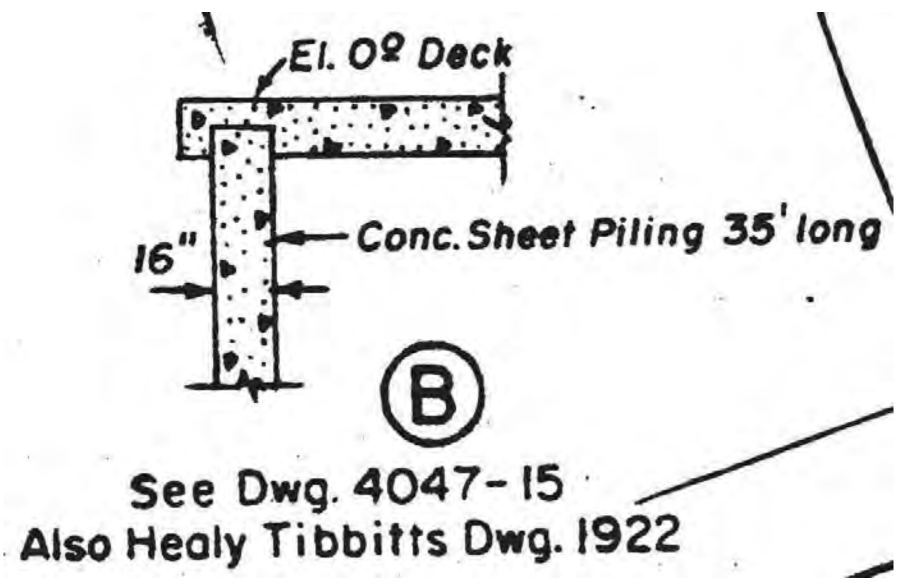


Figure 4-46: Seawall Section AT&T, Bulkhead Wall Type B

This cutoff wall section and type has structural strength deficiency under static and all levels of earthquake loading considered in this study. The critical structural element is the seawall concrete sheet pile moment capacity.

A summary of the results of this assessment is shown on Table 4-41.

Table 4-41: Structural Assessment Summary – Seawall Section AT&T, Cutoff Wall Type B

	Seawall Section: AT&T			Wall Type: B		
	Static	EQ-M8.0 SA	EQ-DE	EQ-475	EQ-975	EQ-MCE
Sheet Pile Strength DCR	1.04	1.46	1.76	1.90	2.03	2.18
Criticality Rating	0.04	0.46	0.76	0.90	1.03	1.18

The structural criticality ratings range from +0.04 to +1.18 indicating that retrofit of this seawall section and type is needed.

4.7. Bulkhead Walls Criticality Rating Summary

Table 4-42 summarizes the criticality ratings assigned to each seawall section and type. These are also summarized by loading type on Figure 4-47 through Figure 4-52 and by location for MCE seismic loading on Figure 4-53 and Figure 4-54.

The static results indicate that all seawall structure is not deemed critical with respect to static load.

Table 4-42: Structural Assessment Summary – Bulkhead Walls Structural Criticality Ratings Summary

Seawall Section	Seawall Type	Seawall Structural Criticality Rating						
		Static	Static + Surcharge	EQ-1906	EQ-DE	EQ-475	EQ-975	EQ-MCE
FW	11	-0.58		-0.51	-0.50	-0.48	-0.47	-0.46
FW	10	-0.26		0.20	0.53	0.71	0.86	1.05
FW	9	-0.92		-0.56	-0.54	-0.46	-0.41	-0.34
FW	8	-0.38		-0.18	-0.04	0.02	0.07	0.14
FW	5	-0.50		-0.28	-0.13	-0.06	0.01	0.08
FW	4	-0.67		-0.27	-0.24	-0.15	-0.09	0.00
FW	6	-0.63		-0.26	-0.22	-0.14	-0.08	0.00
B	3	-0.06		0.18	0.20	0.26	0.30	0.36
B	43.5 Prom	-0.72		-0.29	-0.27	-0.22	-0.20	-0.19
B	2	-1.00		-1.00	-1.00	-1.00	-1.00	0.00
A	2	-1.00		-1.00	-1.00	-1.00	-1.00	0.00
A	1	-0.66		-0.51	-0.50	-0.47	-0.45	-0.41
1	1	-0.66		-0.51	-0.50	-0.47	-0.45	-0.41
2	1	-0.66		-0.51	-0.50	-0.47	-0.45	-0.41
2	1	-0.66		-0.51	-0.50	-0.47	-0.45	-0.41
3	1	-0.39		-0.18	-0.17	-0.14	-0.13	-0.13
3	1	-0.39		-0.18	-0.17	-0.14	-0.13	-0.13
4	Y	-0.10		0.55	1.07	1.23	1.34	1.38
4	Y	-0.10		0.55	1.07	1.23	1.34	1.38
5	Z	-0.61		0.15	0.18	0.27	0.31	0.33
5	Y	-0.10		0.55	1.07	1.23	1.34	1.38
5	X	-0.61		-0.12	-0.09	-0.03	0.00	0.01
5	X	-0.61		-0.12	-0.09	-0.03	0.00	0.01
5	P1517							
5	W	-0.12		0.66	1.30	1.48	1.60	2.01
6	W	-0.10		0.54	0.92	0.94	1.01	1.09
6	W	-0.10		0.54	0.92	0.94	1.01	1.09
6	W	-0.10		0.54	0.92	0.94	1.01	1.09
7	W	-0.10		0.54	0.92	0.94	1.01	1.09
7	W	-0.10		0.54	0.92	0.94	1.01	1.09
7	V	-0.72		-0.34	-0.34	-0.32	-0.30	-0.28
7	U	-0.70		-0.31	-0.31	-0.29	-0.26	-0.24
7	T	-0.73		-0.32	-0.32	-0.30	-0.27	-0.25
8a	S	-0.76		-0.31	-0.31	-0.29	-0.26	-0.24
8b	R	-0.76		-0.31	-0.31	-0.29	-0.26	-0.24
8	Q	-0.59		-0.18	-0.18	-0.16	-0.13	-0.10
9a	P	-0.81		-0.66	-0.66	-0.65	-0.64	-0.62
9b	P	-0.81		-0.75	-0.74	-0.72	-0.72	-0.71
9	O	-0.57		0.10	0.14	0.22	0.24	0.26
9	O	-0.57		0.10	0.14	0.22	0.24	0.26
10	N	-0.61		-0.19	-0.15	-0.05	0.02	0.12
10	M	-0.73		-0.23	-0.20	-0.10	-0.03	0.07
11a	M(revd)	-0.82		-0.57	-0.55	-0.49	-0.46	-0.40
11	L	-0.61		-0.08	-0.04	0.08	0.16	0.28
12-BSW	K	-0.30		0.65	0.72	0.93	1.06	1.26
12-P38	K	-0.30		1.24	1.34	1.66	1.88	2.19
12	J	-0.71		-0.34	-0.32	-0.24	-0.19	-0.11
13	I	-0.35		0.64	0.72	0.95	1.09	1.32
13	H	-0.20		-0.03	-0.01	0.04	0.06	0.10
P46	G	-0.73		-0.64	-0.63	-0.61	-0.60	-0.58
P46	F	-0.73		-0.64	-0.63	-0.61	-0.60	-0.58
AT&T	E	0.04		0.46	0.76	0.90	1.03	1.18
AT&T	D	-0.10		0.36	0.69	0.85	0.99	1.15
AT&T	C	-0.62		-0.57	-0.53	-0.51	-0.50	-0.48
AT&T	B	0.04		0.46	0.76	0.90	1.03	1.18

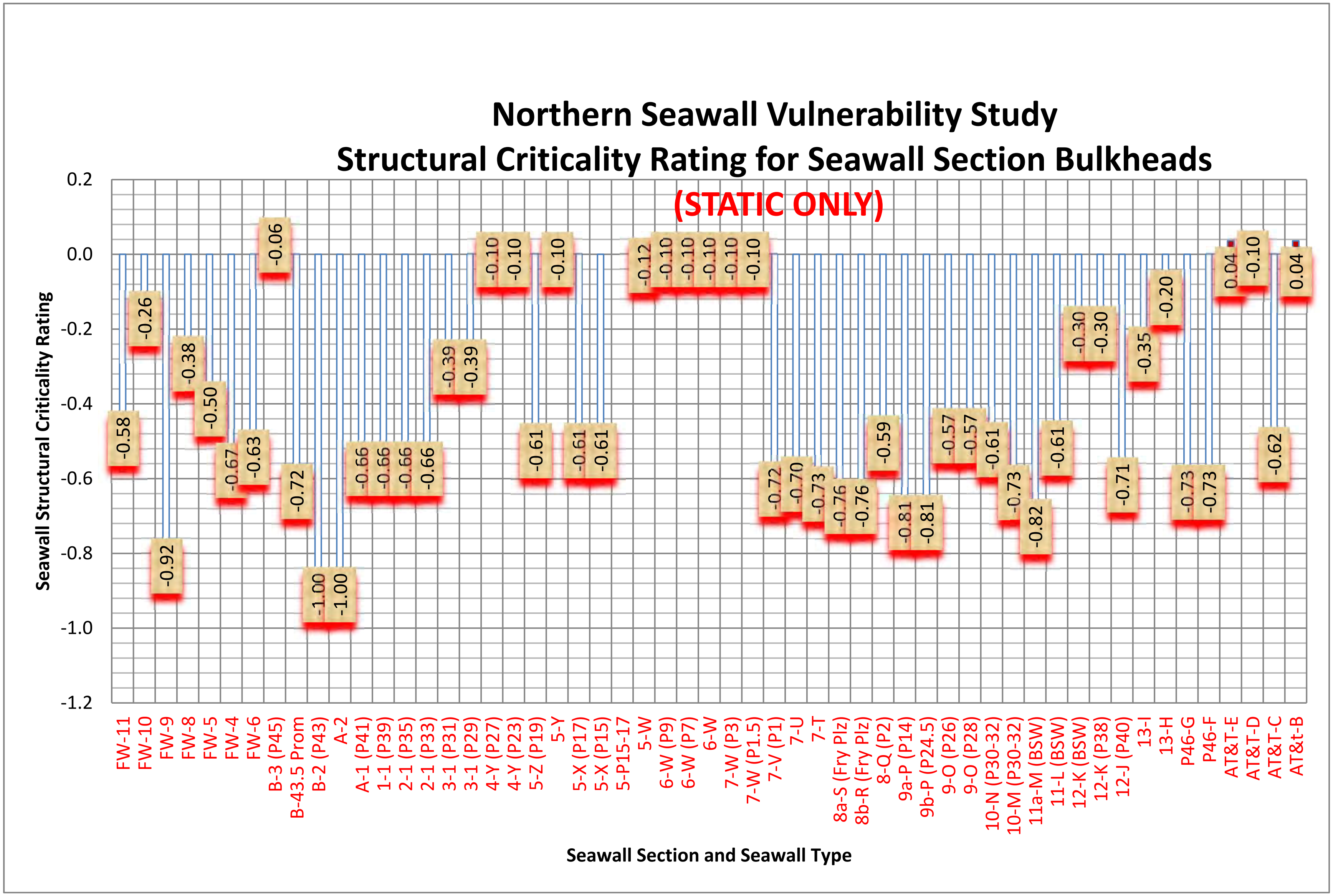


Figure 4-47: Seawall Structural Criticality Ratings, Bulkhead Walls – Static Loading

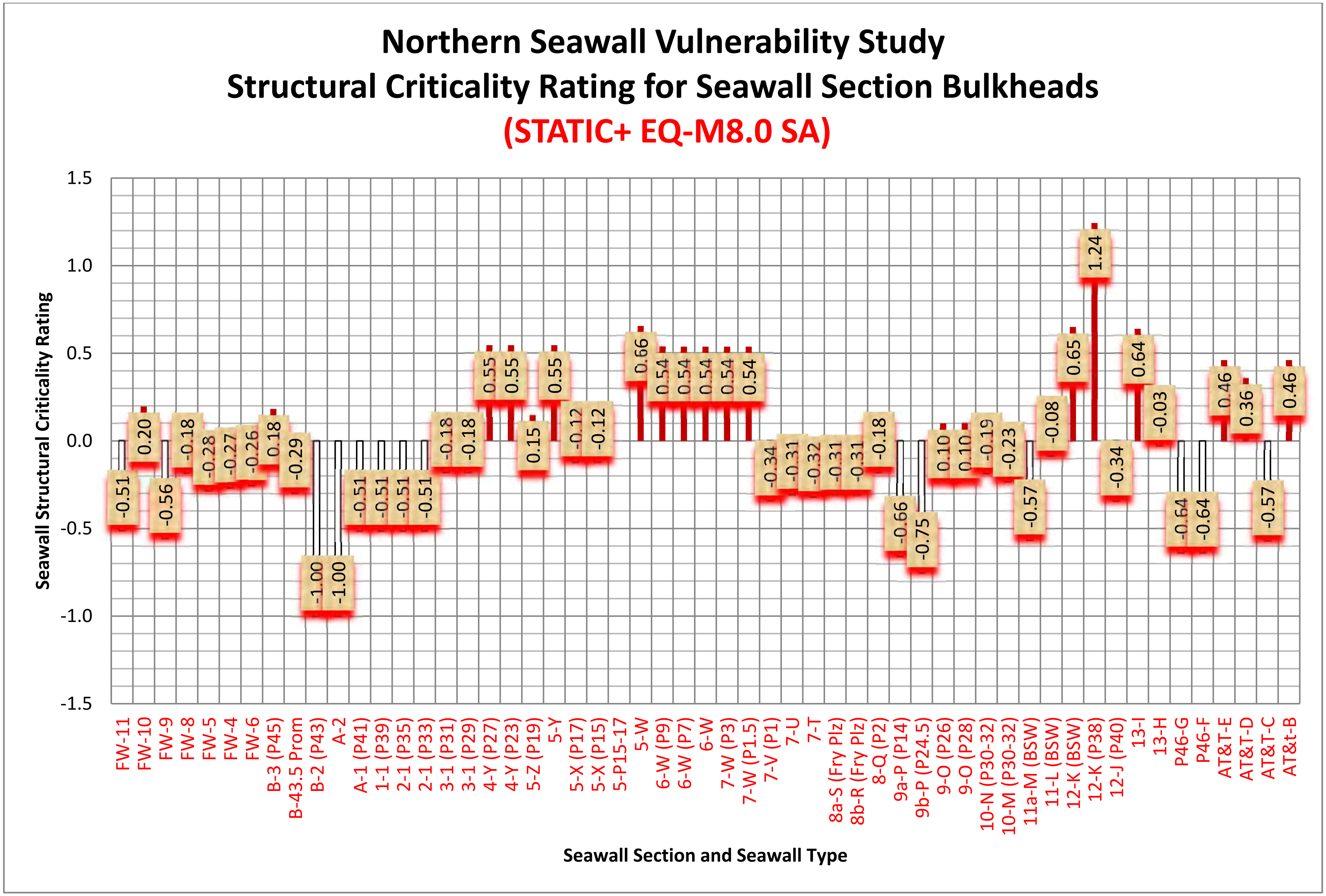


Figure 4-48: Seawall Structural Criticality Ratings, Bulkhead Walls – Static + M8.0 San Andreas Earthquake Loading

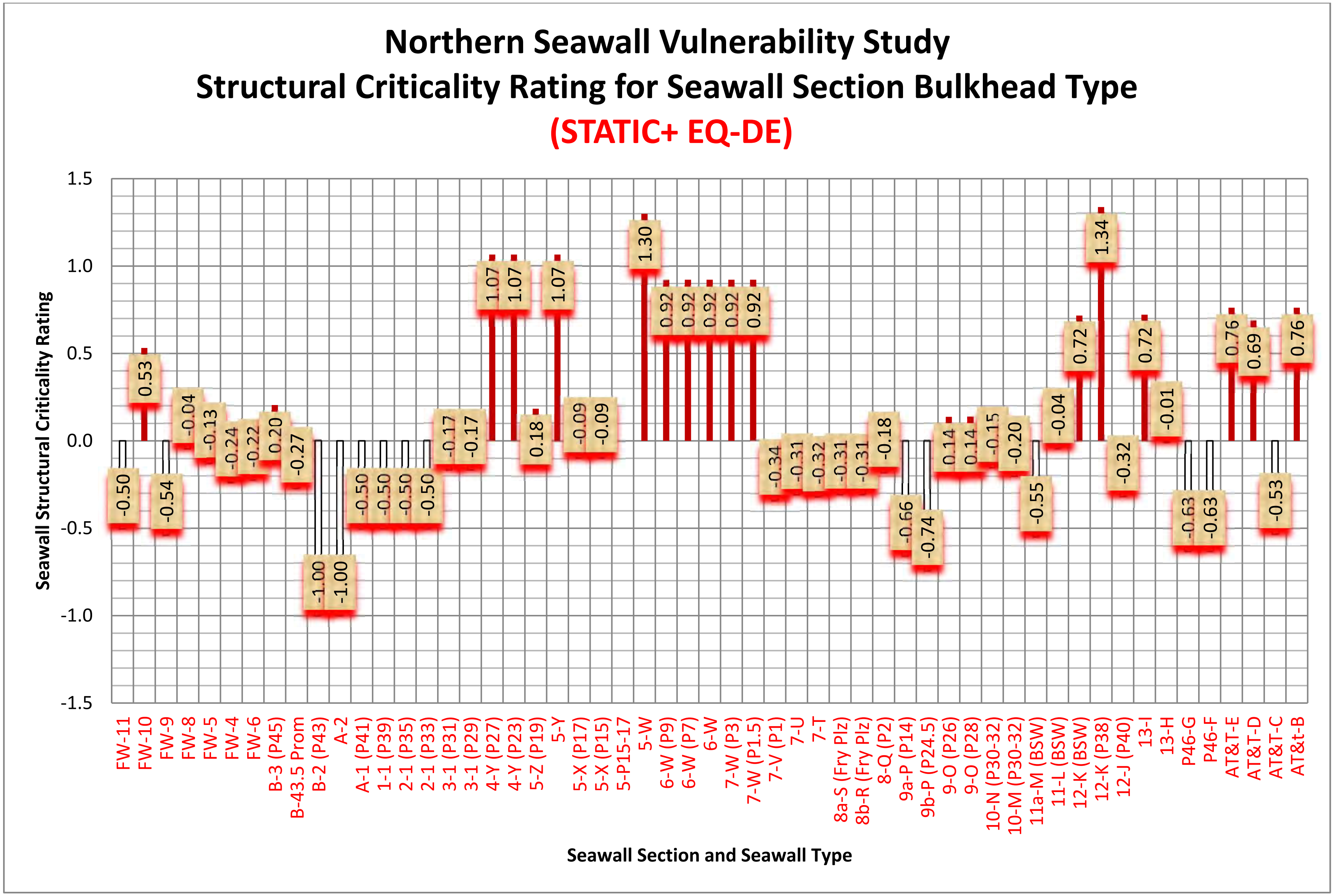


Figure 4-49: Seawall Structural Criticality Ratings, Bulkhead Walls – Static + Design Earthquake (DE) Loading

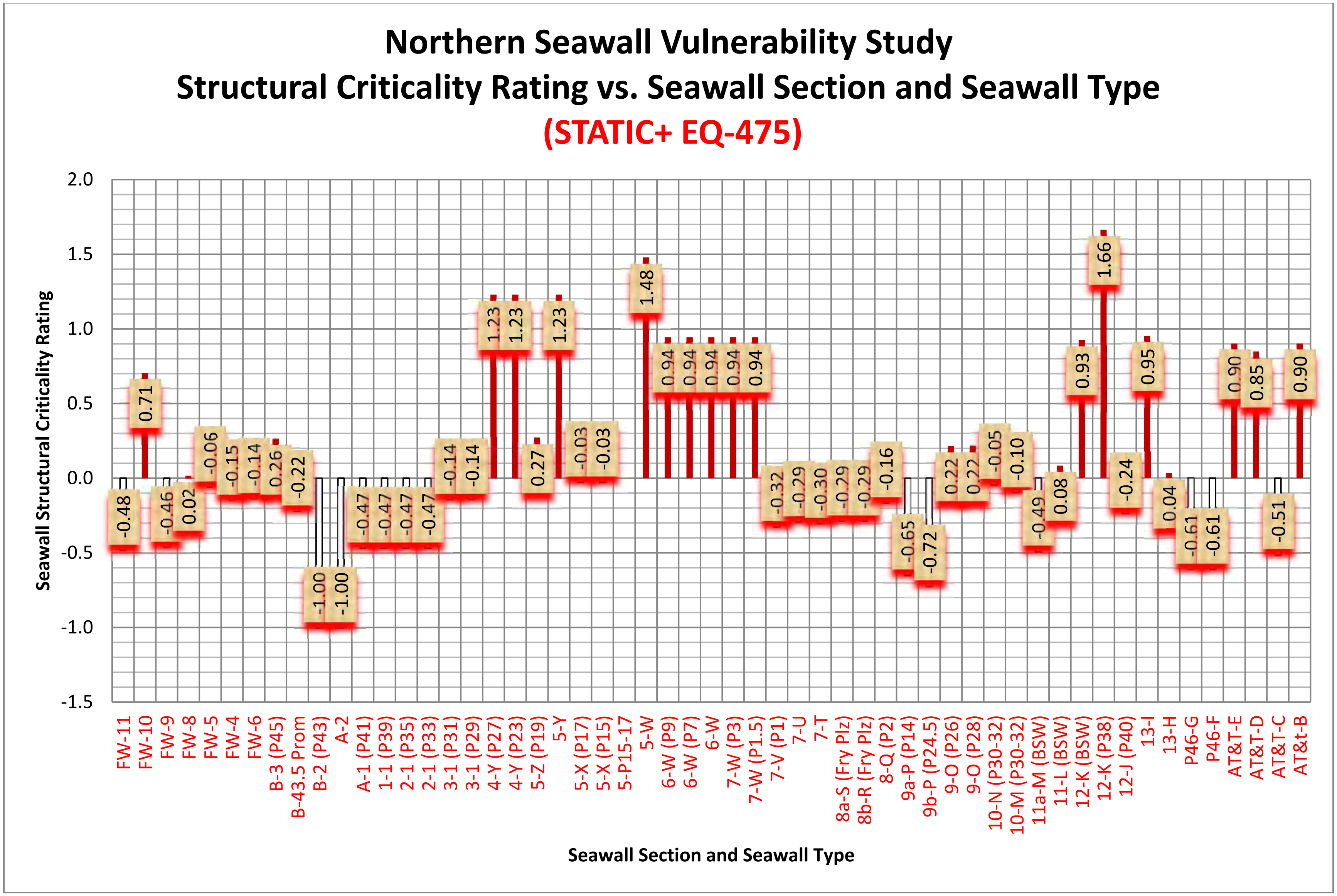


Figure 4-50: Seawall Structural Criticality Ratings, Bulkhead Walls – Static + 475 Year Earthquake (DE) Loading

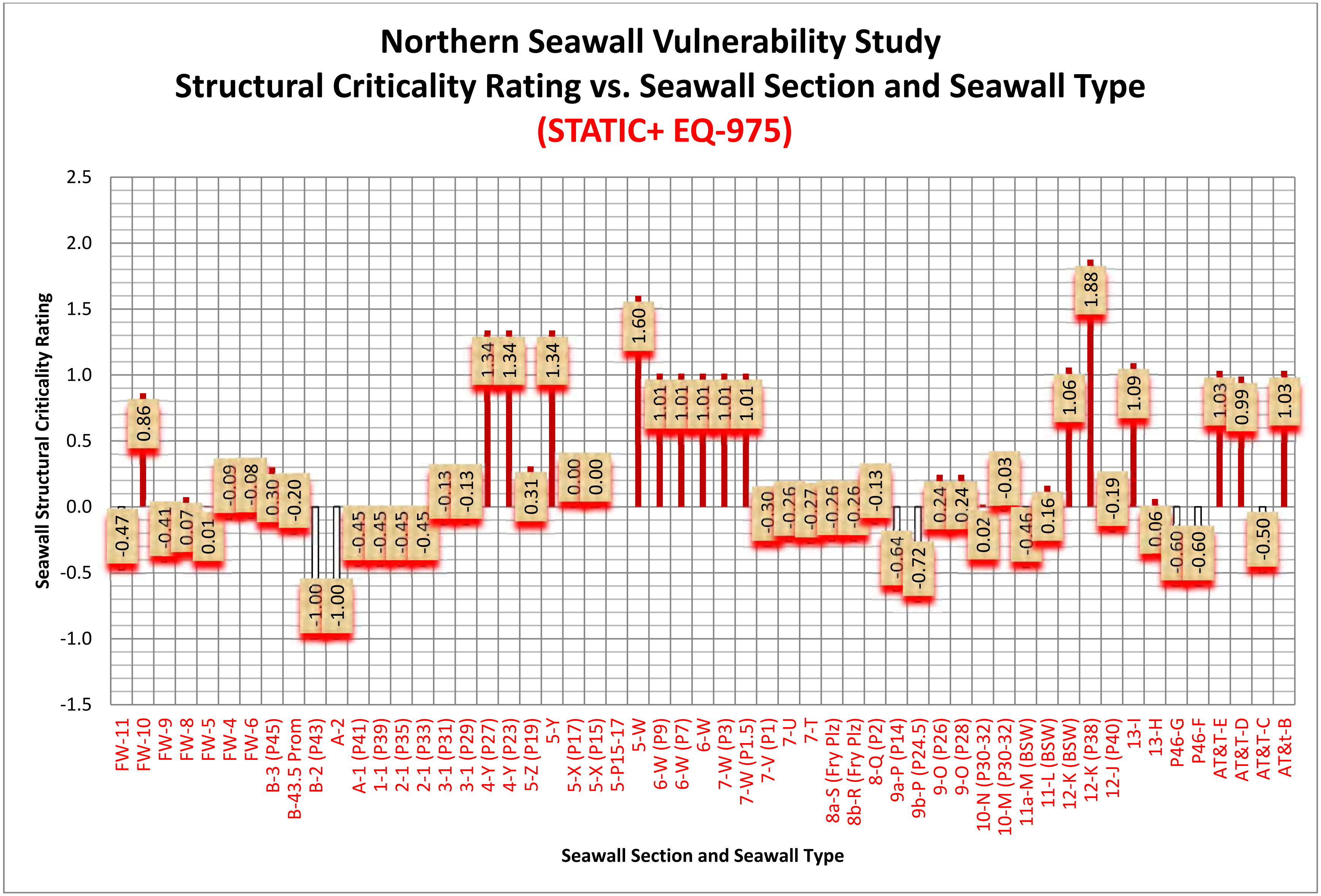


Figure 4-51: Seawall Structural Criticality Ratings, Bulkhead Walls – Static + 975 Year Earthquake Loading

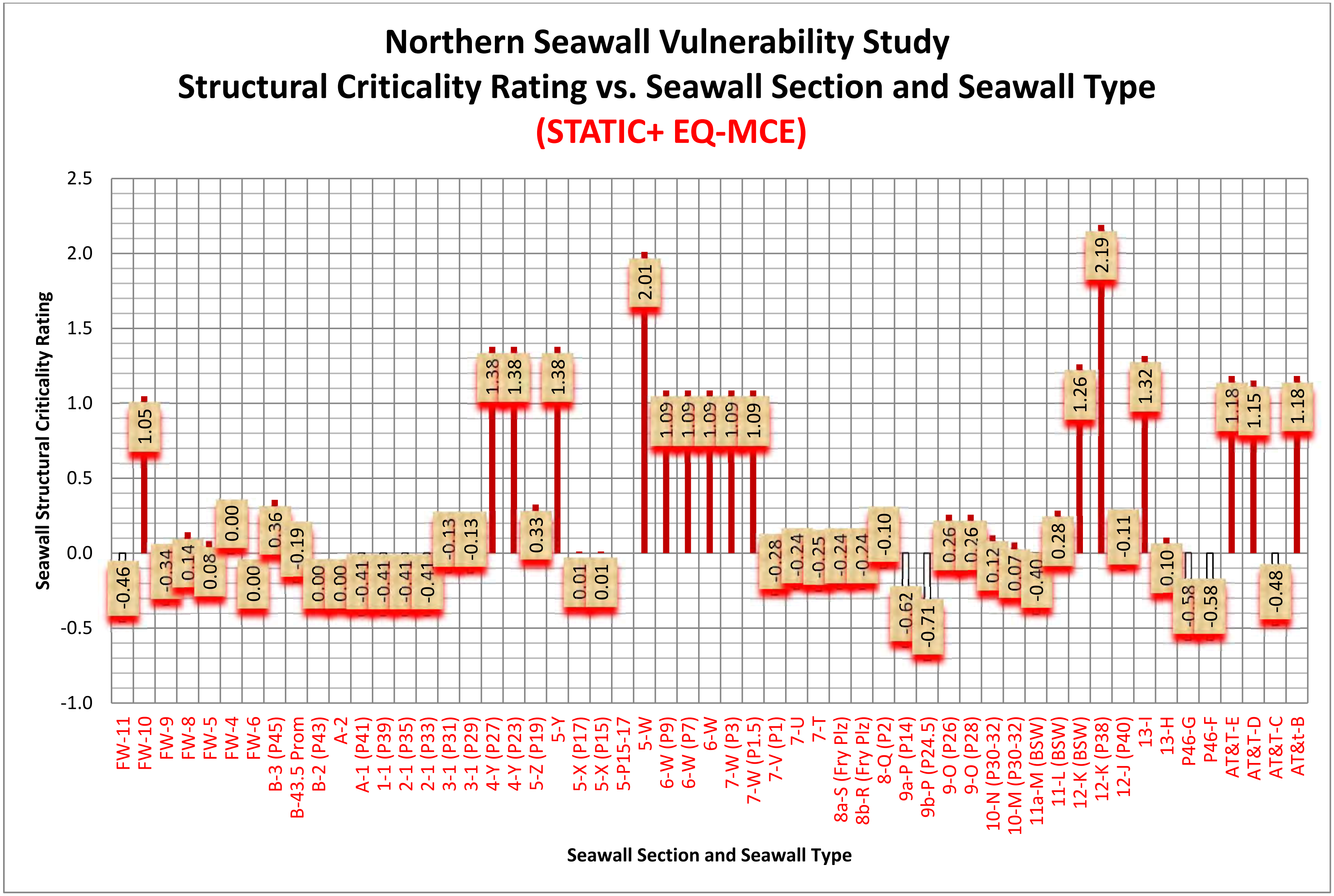


Figure 4-52: Seawall Structural Criticality Ratings, Bulkhead Walls – Static + Maximum Considered Earthquake (MCE) Loading

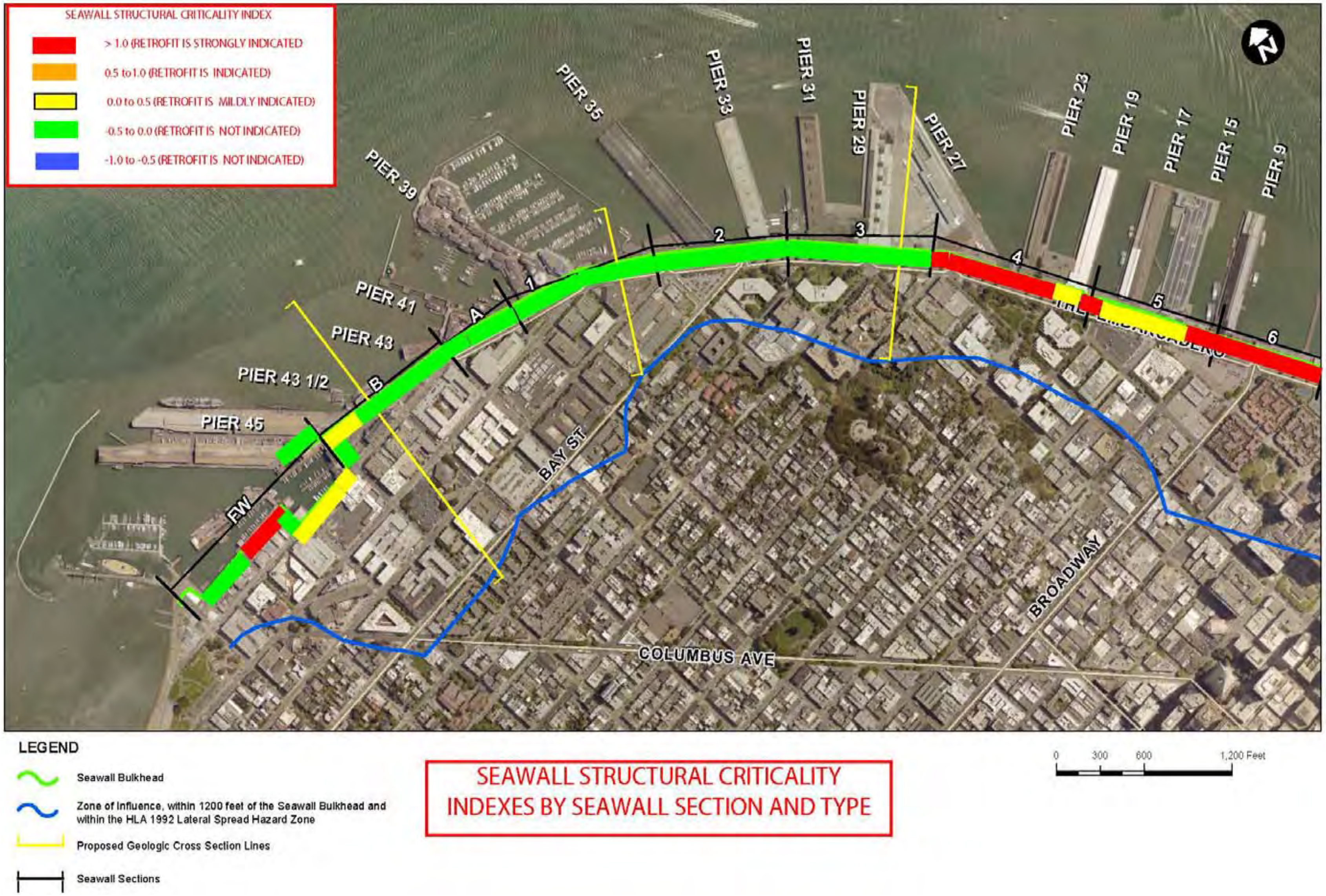


Figure 4-53: Bulkhead Wall Structural Criticality Ratings by Location, Section FW to Section 6

Figure 4-54: Bulkhead Wall Structural Criticality Ratings by Location, Section 6 to Section China Basin

4.8. Marginal Wharf Assessments

The assessment of marginal bulkhead wharfs under the design basis seismic events is considered in this section. This assessment did not include the additional impacts of the seismic response of connecting finger piers.

Five representative bulkhead wharf segments were analyzed for seismic loads applied perpendicular to the seawall. Two types of seismically induced load were considered:

1. Seismic inertial loads consisting of peak ground acceleration (PGA) inertial forces of structure plus seismically induced soil pressures. Supported buildings are assumed to experience seismic inertial loads consisting of the peak response spectral acceleration (at T = 0.7 sec). This may or may not be conservative but was deemed acceptable for use in this study.
2. Seismically induced soil lateral sliding of the lower clay layer located beneath the seawall rock dike.

All analysis cases were developed for non-linear analysis and included a static load case, a seismic inertial loads case (where recommended by GTC), and for the soil lateral sliding case, a pushover analysis.

The five bulkhead wharf segments that were analyzed were selected based on the following criteria:

1. Segment is a Port property.
2. Segment supports a building structure that is occupied by Port or public personnel.
3. Sufficient data for the segment structure has been provided to GHD for use in this assessment.

The five bulkhead wharf segments selected were:

1. Bulkhead wharf structure between Piers 29 and 31. This wharf structure and the supporting building is approximately 165 feet wide and represents the bulkhead wharf of maximum width into the bay. The seawall at this location consists of a concrete cut-off wall supported on a rock dike.
2. Bulkhead wharf structure near Pier 17. This wharf structure has no building or other major structure supported on it. The seawall at this location consists of a moderately sized pile supported bulkhead seawall supported on a rock dike.
3. Bulkhead wharf structure near Pier 9. This wharf structure supports a building structure that serves as leased space to a large number of public personnel. The seawall at this location consists of a concrete cut-off wall supported on a rock dike.
4. Bulkhead wharf structure between Piers 26 and 28. This wharf structure supports a building structure that serves as leased space to the public. The seawall at this location consists of a relatively large pile supported bulkhead seawall supported on a rock dike.
5. Bulkhead wharf structure near Pier 38. This wharf structure supports a building structure that serves as leased space to the public. The seawall at this location consists of a relatively small bulkhead seawall supported on a rock dike. The bulkhead is not pile supported.

Each model consists of frame elements modeling the wharf and seawall piles and the wharf deck beams, and plate elements to model the wharf deck, seawall bulkhead walls or cutoff wall panels, and as load collectors for supported building structure. No attempt was made to model the building structure correctly with respect to structure stiffness since this complexity was deemed outside of the scope of work for this study.

The pile sizes, material types (concrete or timber), pile dimension and concrete reinforcement (where applicable) details were determined from the record drawings in our possession. For non-linear analysis, plastic hinges may occur in these elements. The hinge definitions were determined for each pile type and specific hinges assigned in the analysis model as appropriate. Non-linear hinges for reinforced concrete piles were determined in SAP based on the moment-curvature Caltrans Equivalent Pile Method. Reinforced concrete shear hinges were developed based on the recommendations ASCE 41-06. Data for this hinge development follows for each analysis segment. Non-linear hinges for timber piles (both shear and moment) were determined based on the recommendations of ASCE 41-06.

The non-linear analyses account for soil-structure interaction. Soil lateral (p-y) springs are included in the model at regular intervals (typically 12 inch spacing). The springs were determined by LPILE analysis documented previously in this calculation volume. These LPILE analysis results were converted to appropriate spring input values and imported into each SAP model.

Pier 29-31 Bulkhead Wharf:

This bulkhead wharf (Figures 4-55 and 4-56) has significant building structure located on it, extending about 190 feet towards the bay. The bulkhead wharf piles consist of fifteen transverse rows of16 to18 inch square reinforced concrete piles with four longitudinal reinforcing bars inside a W3 wire spiral. Bulkhead wharf pile spacing is nominally 15 feet transverse and 9 feet longitudinal. The ductile capacity of these piles is very limited due to the minimal size of the spiral confinement.

Earthquake Inertial Load – The bulkhead wharf piles have sufficient capacity to resist the 72-year, M8.0 San Andreas-level and 975-year seismic inertial events. The expected seismic deck demand is 1.8, 2.1 and 2.4 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile maximum displacement capacities at the pile head are 3.88 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads. The controlling pile head displacement DCRs are 0.45, 0.55 and 0.62 inch for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile is the first row of 18” square concrete piles supporting the bulkhead wharf and the critical location is at the pile head just below the wharf deck.

Earthquake Induced Liquefaction and Soil Lateral Sliding - The bulkhead wharf piles (18” square) that penetrate the base of the rock dike or are founded directly into the unstable soils below will fail below the rock dike due to soil lateral sliding. The location of failure is actually at the pile head just below the wharf deck. The maximum soil lateral sliding displacement demands are 5, 50 and 114 inches for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum displacement capacity of the piles at the zone of soil lateral sliding is 2.2 inches, resulting in DCRs that are 2.3, 23 and 52 for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum axial load in the critical piles is 67 kips. The available skin friction of these piles embedded 13 feet in the rock dike is about 50 kips. Thus, the piles will most likely fail below the rock dike and there is insufficient axial load carrying capacity remaining in the piles to support the wharf deck. Significant settlement should be expected. The critical pile axial pile load DCR is dependent on the pile size but is greater than 1.3. The critical pile shear load DCR is 1.8.

Conclusion:

The maximum DCR of 0.62 under seismic inertial loading indicates that this seawall section has no need for retrofit for seismic inertial load. However, bulkhead wharf pile head connections deemed damaged or otherwise deficient should be retrofitted to accommodate the expected displacement and member force displacement demands. Wharf deck beam connections at the seawall should be retrofitted to accommodate the expected seismic loads and/or relative displacements.

The maximum DCR of 52 under soil lateral sliding indicates that this seawall section cannot accommodate the soil lateral sliding displacement demand. The failure of the associated piling will most likely produce damage to bulkhead wharf structure above that is supported by these rows of piles. The inner two rows of 16" square piles, followed by the outer four rows of 20" square piles are expected to fail under soil lateral sliding with the remainder of the piling following thereafter. As each row of piles fails, the two bays of bulkhead wharf and supported building structure will be subject to significant downward displacement and associated structural damage.

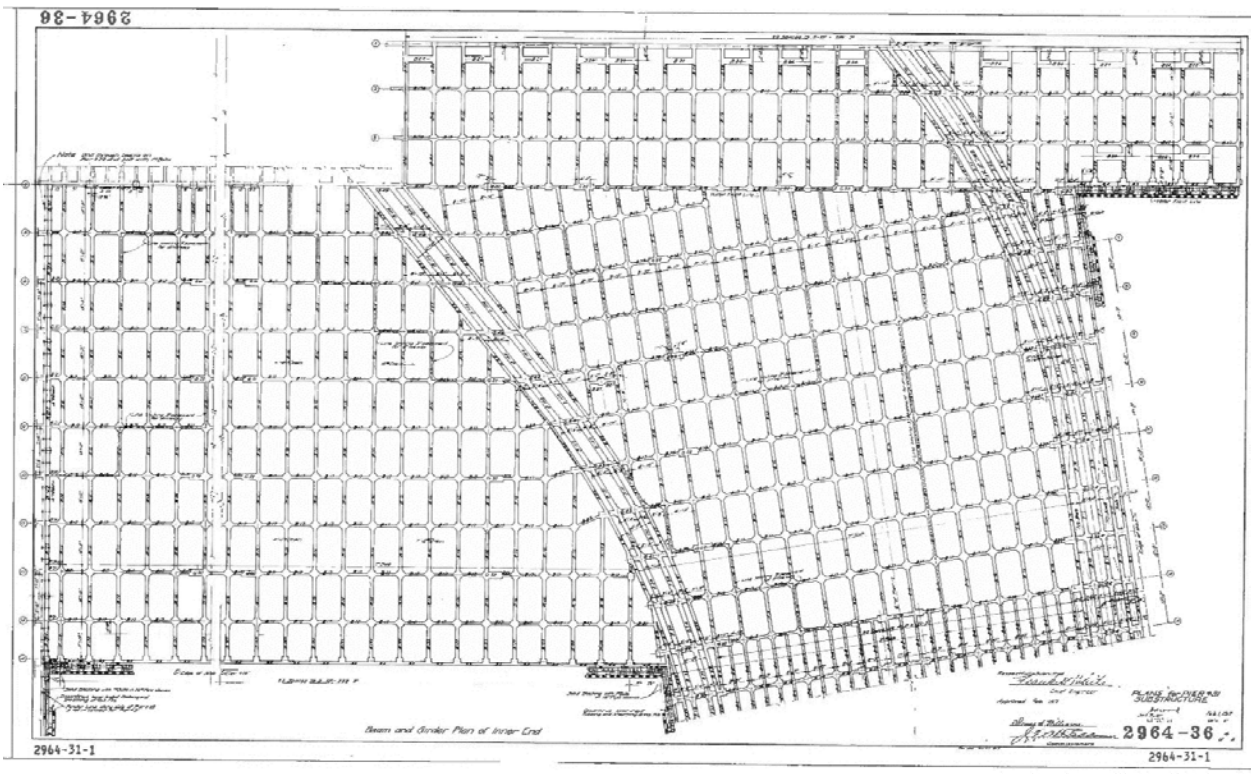


Figure 4-55: Section 3 Bulkhead Wharf Between Piers 29 & 31 – Plan View (Port Drawing 2964-31-1)

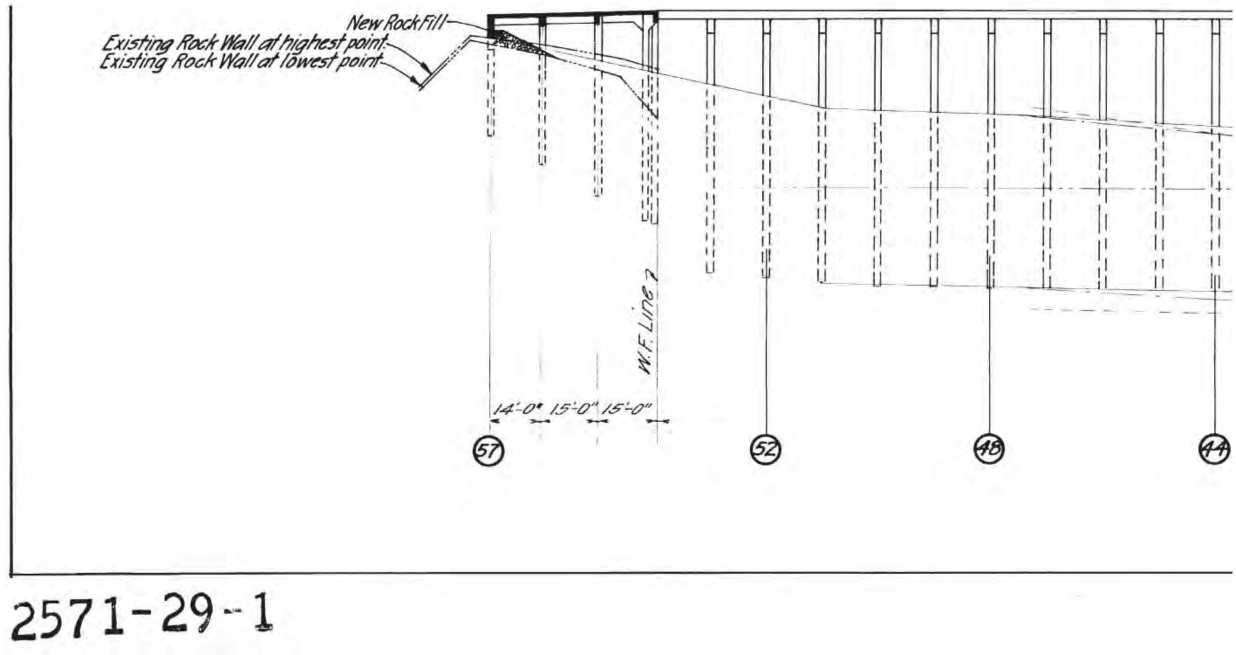


Figure 4-56: Section 3 Bulkhead Wharf Between Piers 29 & 31 – Section View (Port Drawing 2571-29-1)

Pier 17 Bulkhead Wharf:

This bulkhead wharf (Figure 4-57) has no significant building structure located on it. The bulkhead wharf piles consist of four transverse rows of 16 or 18 inch square reinforced concrete piles with four longitudinal reinforcing bars inside a W3 wire spiral. Bulkhead wharf pile spacing is nominally 11 feet transverse and 11 feet longitudinal. The ductile capacity of these piles is very limited due to the minimal size of the spiral confinement.

Earthquake Inertial Load – The bulkhead wharf piles have insufficient capacity to resist the M8.0 San Andreas level seismic inertial event. The expected seismic deck demand is 2.7, 3.1 and 3.5 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile maximum displacement capacities at the pile head are 2.21, 2.21, and 2.45 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The controlling pile head displacement DCRs are 1.22, 1.39 and 1.42 for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile is the 12" timber pile supporting the seawall bulkhead and the critical location is at the pile head just below the base of the seawall bulkhead. Once this pile fails at the pile head, there is no axial pile load capacity available to support the seawall and adjacent structure.

Earthquake Induced Liquefaction and Soil Lateral Sliding - The bulkhead wharf piles (16" square) that penetrate the base of the rock dike will fail below the rock dike due to soil lateral sliding. The maximum soil lateral sliding displacement demands are 5.0, 32.5 and 74.5 inches for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum displacement capacity of the piles at the zone of soil lateral sliding is 10.65 inches, resulting in DCRs of 0.47, 3.1 and 7.0 for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum axial load in the critical piles is 62 kips. The available skin friction of these piles embedded 13 feet in the rock dike is about 39 kips. Thus, if the piles fail below the rock dike, there is insufficient axial load carrying capacity remaining in the piles to support the wharf deck and pile settlement should be expected. The critical pile axial pile load DCR is 1.6. The critical pile shear load DCR is 0.2.

Conclusion:

The maximum DCR of 1.42 under seismic inertial loading indicates that this seawall section has a strong need for retrofit for seismic inertial load. Deficient bulkhead wharf pile head connections should be retrofitted to accommodate at least 2 times the present displacement capacity demand. Alternatively, since this seawall section has no major building structures on it, consideration could be given to removing this section of marginal wharf.

Bulkhead wharf pile head connections deemed damaged or otherwise deficient should be retrofitted to accommodate the expected displacement and member force displacement demands. Wharf deck beam connections at the seawall should be retrofitted to accommodate the expected seismic loads and/or relative displacements.

The maximum DCR of 7.0 under soil lateral sliding indicates that this seawall section cannot accommodate the soil lateral sliding demand. The failure of the associated piling will most likely produce damage to bulkhead wharf structure above that is supported by these rows of piles. The outer two rows of piles are expected to fail under soil lateral sliding subjecting approximately half of the bulkhead wharf structure to significant displacement and associated structural damage.

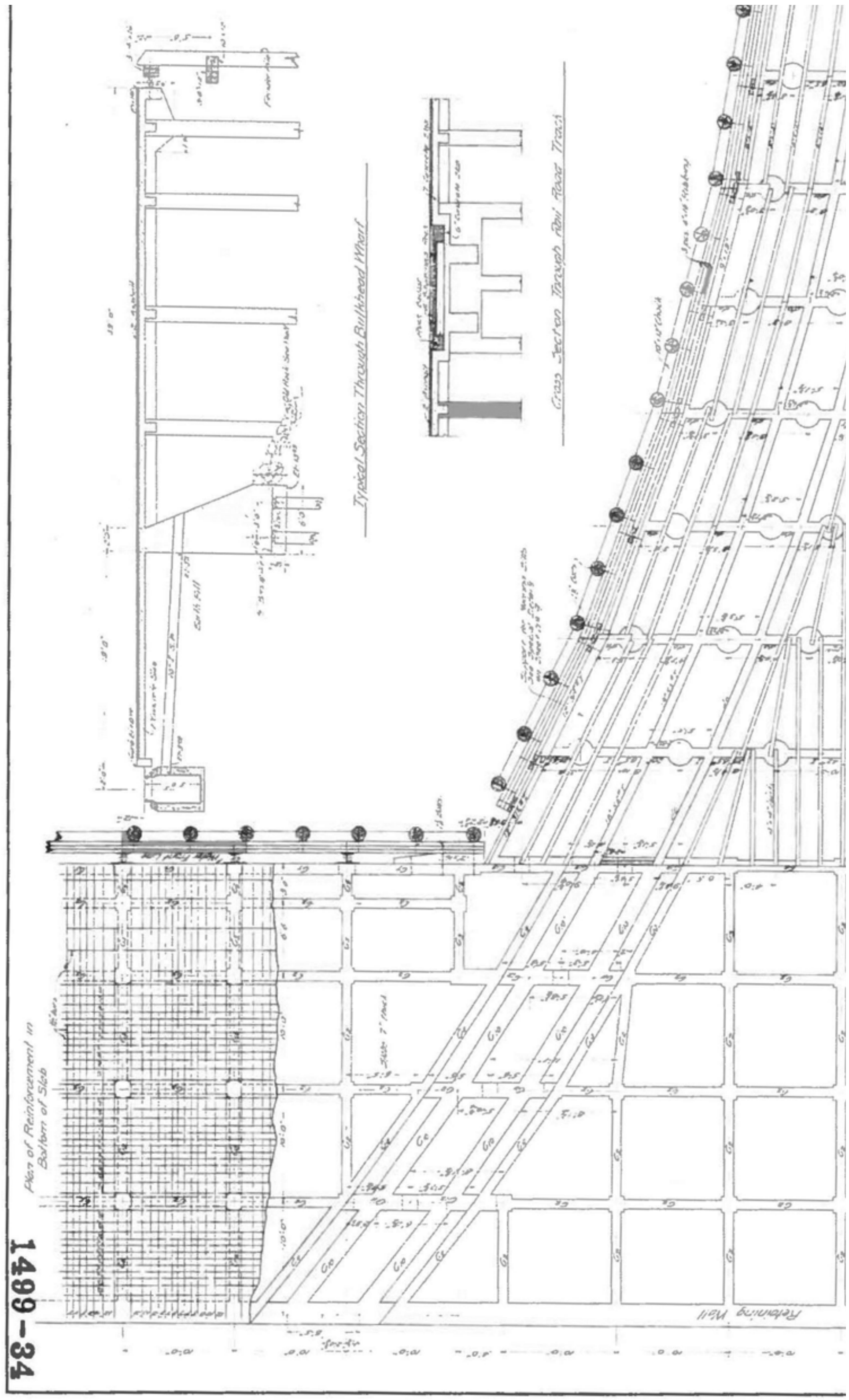


Figure 4-57: Section 5 Bulkhead Wharf Near Pier 17 – Plan and Section View (Port Drawing 1499-17-3)

Pier 9 Bulkhead Wharf:

The bulkhead wharf piles consist of five transverse rows of 16 or 18 inch square reinforced concrete piles with four longitudinal reinforcing bars inside a W3 wire spiral. Bulkhead wharf pile spacing is nominally 11 feet transverse and 11 feet longitudinal (Figure 4-58). The ductile capacity of these piles is very limited due to the minimal size of the spiral confinement.

Earthquake Inertial Load – The bulkhead wharf piles have sufficient capacity to resist the 72-year, M8.0 San Andreas-level and 975-year seismic inertial events. The expected seismic deck demand is 0.92, 1.0 and 1.1 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile maximum displacement capacities at the pile head are 0.57, 0.57, and 0.98 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The controlling pile head displacement DCRs are 1.61, 1.74 and 1.07 for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile is the 16" square pile supporting the seawall cutoff wall and the critical location is at the pile head just below the base of the cutoff wall.

Earthquake Induced Liquefaction and Soil Lateral Sliding - The bulkhead wharf piles (18" square) that penetrate the base of the rock dike will fail below the rock dike due to soil lateral sliding. The maximum soil lateral sliding displacement demands are 6.8, 26 and 59 inches for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum displacement capacity of the piles at the zone of soil lateral sliding is 8.3, 18.3 and 18.3 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively, resulting in DCRs of 0.82, 3.1 and 7.1 for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum axial load in the failed piles is 49 kips. The available skin friction of these piles in the rock dike is about 48 kips. Thus, although the piles have failed below the rock dike, there is only marginally sufficient axial load carrying capacity remaining in the outer two rows of piles to support the wharf deck although some pile settlement should be expected. The critical pile axial pile load DCR is 1.03. The critical pile shear load DCR is 1.12.

Conclusion:

The maximum DCR of 1.74 under seismic inertial loading indicates that this seawall section has a strong need for retrofit for seismic inertial load. Deficient bulkhead wharf pile head connections should be retrofitted to accommodate at least 2 times the present displacement capacity demand. Bulkhead wharf pile head connections deemed damaged or otherwise deficient should be retrofitted to accommodate the expected displacement and member force displacement demands. Wharf deck beam connections at the seawall should be retrofitted to accommodate the expected seismic loads and/or relative displacements.

The maximum DCR of 7.1 under soil lateral sliding indicates that this seawall section cannot accommodate the soil lateral sliding demand. The failure of the associated piling will most likely produce damage to bulkhead wharf structure above that is supported by these rows of piles. The outer two rows of piles are expected to fail under soil lateral sliding subjecting approximately half of the bulkhead wharf structure and supported buildings to significant displacement and associated structural damage.

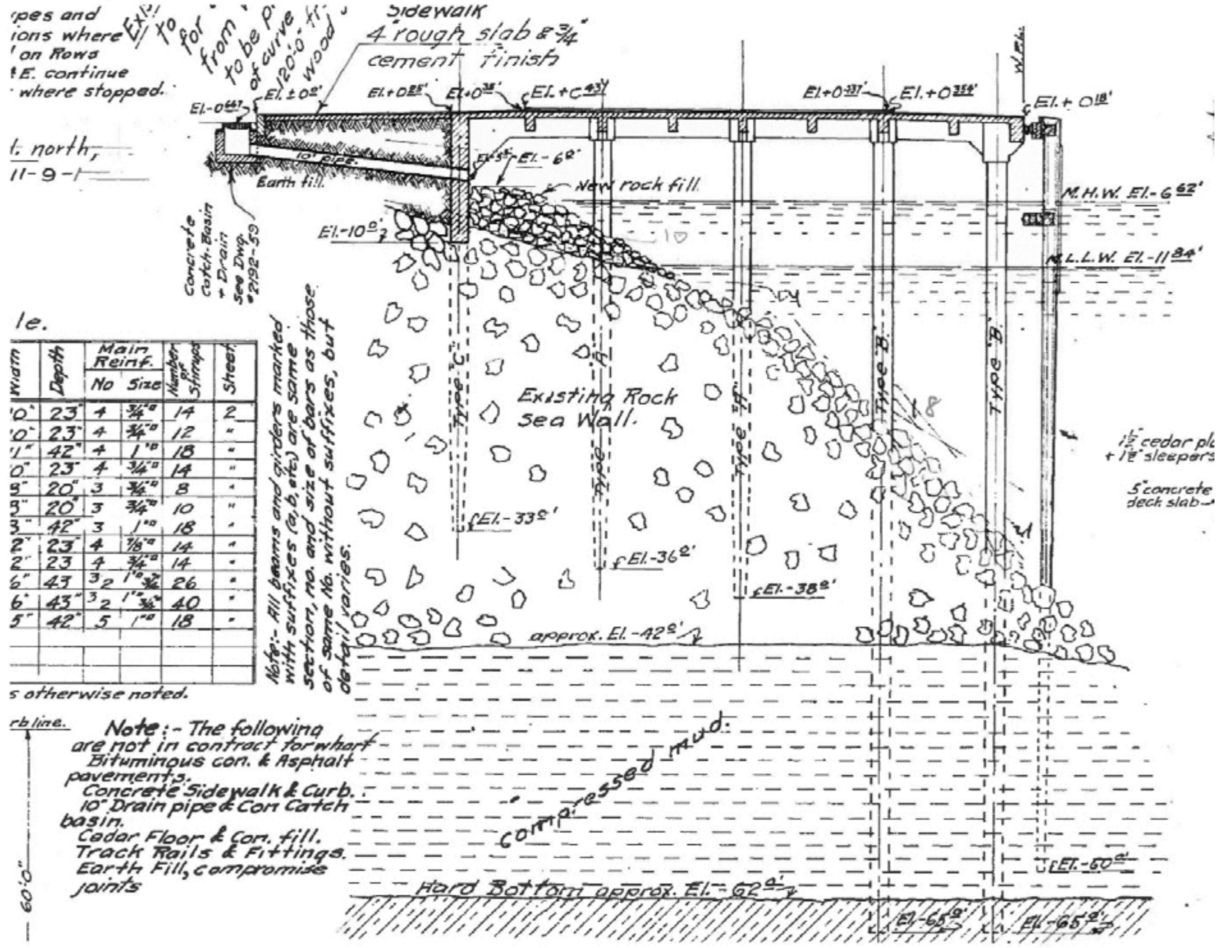


Figure 4-58: Section 6 Bulkhead Wharf Near Pier 9 –Section View (Port Drawing 2781-9-1)

Pier 26-28 Bulkhead Wharf:

The bulkhead piles consist of one 24-inch circular reinforced concrete jacket around a 16-inch timber pile. The reinforced concrete jacket consists of twelve ½-inch diameter longitudinal bars inside 1/8-inch by ¾-inch steel straps at 12 inch spacing. The bulkhead wharf pile longitudinal spacing is nominally 6'-8" (Figure 4-59). The ductile capacity of these piles is very limited due to the minimal size of the spiral confinement.

Earthquake Inertial Load – The bulkhead wharf piles have marginally sufficient capacity to resist the 72-year, M8.0 San Andreas-level and 975-year seismic inertial events. The expected seismic deck demand is 1.83, 3.3 and 3.6 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile maximum deck displacement capacities at the pile head are 2.38, 2.38 and 2.50 inches, respectively. The maximum deck displacement capacity of the piles at the zone of soil lateral sliding is 2.4, 2.4 and 2.5 inches, resulting in DCRs of 0.77, 0.88 and 1.01 for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The critical pile is the 16" diameter timber pile supporting the seawall bulkhead and the critical location is at the pile head just below the base of the bulkhead wall.

Earthquake Induced Liquefaction and Soil Lateral Sliding - The bulkhead wharf timber piles (16" diameter) that penetrate the base of the rock dike will fail below the rock dike due to soil lateral sliding. The maximum soil lateral sliding displacement demands are 7.6, 19 and 54 inches for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum displacement capacity of these piles at the zone of soil lateral sliding is 1.54, 1.54 and 1.90 inches, resulting in DCRs of 4.9, 12 and 28 for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum axial load in the failed piles is 108 kips. The available skin friction of these piles in the rock dike is about 89 kips. Thus, there is insufficient axial load carrying capacity remaining in the piles to support the wharf deck. Significant pile settlement should be expected. The critical pile axial pile load DCR is 1.21 implying that the wharf vertical loads will not be supported when the piles have failed below the rock dike. The critical pile shear load DCR is 2.25.

Conclusions:

The maximum displacement DCR of 1.01 under seismic inertial loading indicates that this seawall section has just sufficient displacement capacity under seismic inertial load. Deficient bulkhead wharf pile head connections should be retrofitted to accommodate at least 2 times the present displacement capacity demand. However, the wharf deck beam connections at the seawall should be retrofitted to accommodate the expected seismic loads and/or relative displacements. Bulkhead wharf pile head connections deemed damaged or otherwise deficient should be retrofitted to accommodate the expected displacement and member force displacement demands.

The maximum DCR of 28 under soil lateral sliding indicates that this seawall section cannot accommodate the soil lateral sliding demand without pile failure. This bulkhead wharf section is a candidate for seismic retrofit to accommodate soil lateral sliding. The failure of the associated piling will most likely produce damage to the bulkhead wharf structure above that is supported by this single row of piles. If the single row of supporting piles is incapable of supporting the vertical load, the entire marginal wharf structure will displace with associated structural damage to the entire bulkhead wharf structure and supported buildings.

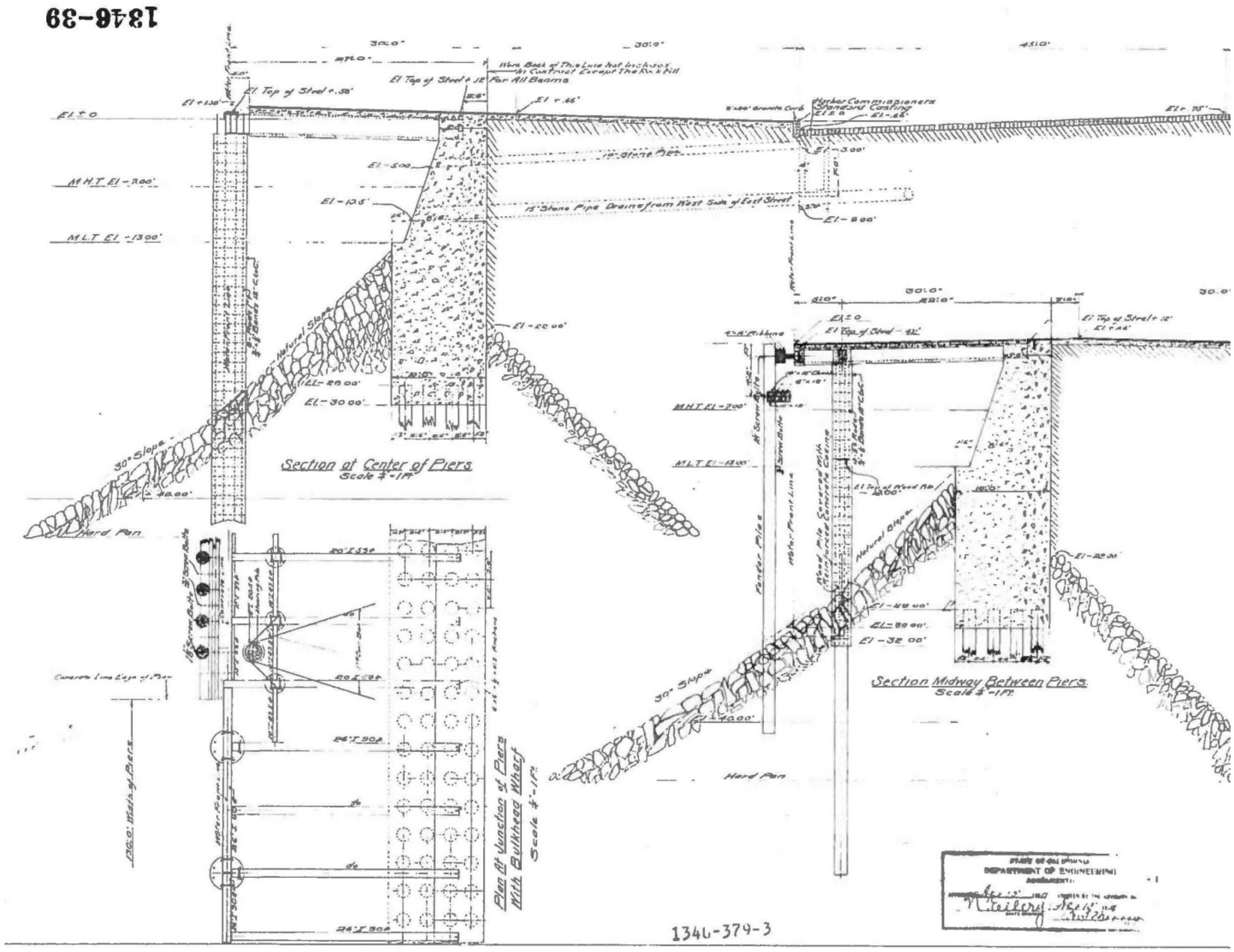


Figure 4-59: Section 9 Bulkhead Wharf Between Piers 26 & 28 –Section View (Port Drawing 1346-379-3)

Pier 38 Bulkhead Wharf:

The bulkhead piles consist of one row of 24-inch circular reinforced concrete jacket around a 16-inch timber pile. The reinforced concrete jacket consists of twelve ½-inch diameter longitudinal bars inside 1/8-inch by ¾-inch steel straps at 12 inch spacing. The bulkhead wharf pile longitudinal spacing is nominally 6'-3" (Figure 4-60). The ductile capacity of these piles is very limited due to the minimal size of the spiral confinement. The pile capacity curves are shown on the following.

<u>Earthquake Inertial Load –</u> The bulkhead wharf piles have sufficient capacity to resist the 72-year, M8.0 San Andreas-level and 975-year seismic inertial events. The expected seismic deck demand is 4.6, 5.5 and 6.8 inches for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The controlling pile DCRs are 0.81, 0.98 and 0.89 for the 72-year, M8.0 San Andreas-level and 975-year earthquake loads, respectively. The critical pile maximum displacement capacities at the pile head are 5.6, 5.6, and 7.6 inches, respectively. The critical pile is the 24" jacketed timber pile supporting the bulkhead wharf and the critical location is at the pile head just below the wharf deck.

<u>Earthquake Induced Liquefaction and Soil Lateral Sliding -</u> The bulkhead wharf timber piles (16" diameter) that penetrate the base of the rock dike will fail below the rock dike due to soil lateral sliding. The maximum soil lateral sliding displacement demands are 2.1, 8 and 57 inches for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum displacement capacity of the piles at the zone of soil lateral sliding is 4.7 inches, resulting in DCRs of 0.45, 1.7 and 12.2 for the 72-year, M8.0 San Andreas-level and 975-year seismic events, respectively. The maximum axial load in the failed piles is 56 kips. The available skin friction of these piles in the rock dike is about 84 kips. Thus, although the piles will have failed below the rock dike, there is sufficient axial load carrying capacity remaining in the piles to support the wharf deck although pile settlement should be expected. The critical pile axial pile load DCR is 0.66. The critical pile shear load DCR is 0.63.

<u>Conclusions:</u>

The maximum DCR of 0.98 under seismic inertial loading indicates that this seawall section has just sufficient displacement capacity under seismic inertial load. However, the wharf deck beam connections at the seawall should be retrofitted to accommodate the expected seismic loads and/or relative displacements. Bulkhead wharf pile head connections deemed damaged or otherwise deficient should be retrofitted to accommodate the expected displacement and member force displacement demands.

The maximum DCR of 12.2 under soil lateral sliding indicates that this seawall section cannot accommodate the soil lateral sliding demand. This bulkhead wharf section is a strong candidate for seismic retrofit to accommodate soil lateral sliding. The single row of piles may be able to continue to support the wharf vertical loads even though the piles have failed below the rock dike. However, any vertical failure of the associated piling will most likely produce damage to bulkhead wharf structure above that is supported by this single row of piles. If the single row of supporting piles is incapable of supporting the vertical load, the entire marginal wharf structure will displace with associated structural damage to the entire bulkhead wharf structure and supported buildings.

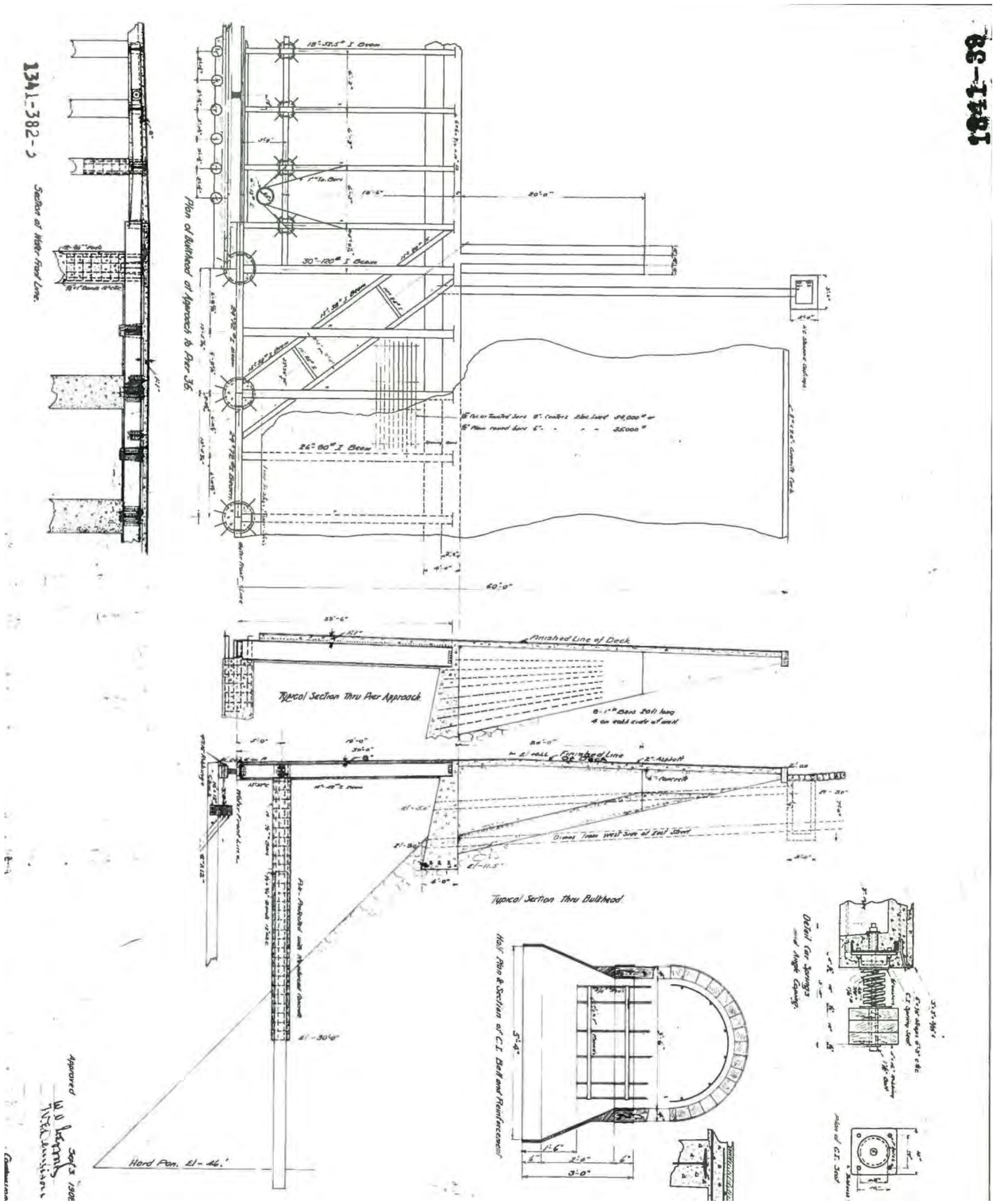


Figure 4-60: Section 12 Bulkhead Wharf Near Pier 38 –Section View (Port Dwg 1341-382-3)

Other Bulkhead Wharfs Supporting Buildings

The following 15 seawall sections have bulkhead wharfs connected to the seawall and directly support structures on the bulkhead wharfs (structures on piers are not included). Five of these were analyzed as a part of this work.

- Section FW, Wharf J1
- Section A, Near Pier 41
- Section 1, Near Pier 39
- Section 2, Near Piers 35 and 33
- Section 3, Near Piers 31 and 29 (analyzed as a part of this work)
- Section 4, Near Piers 23 and 19
- Section 5, Near Piers 23 and 19, Pier 15 (associated with Exploratorium)
- Section 6, Near Piers 7 and 9, (Pier 9 analyzed as a part of this work)
- Section 7, Near Piers 1 thru 3,
- Sections 8a, Ferry Plaza
- Sections 8b, Ferry Plaza
- Section 8, Agricultural Building
- Section 9b, Fire Station
- Section 9, Near Piers 26 and 28 (analyzed as a part of this work)
- Section 12, Near Pier 38 (analyzed as a part of this work

A review of the record drawings with respect to soil lateral sliding potential effects on bulkhead wharf piles follows:

- Section A (Pier 41), bayside piles up to 40 feet in length may be subject to lateral sliding overstress with insufficient axial capacity remaining. Estimate one bay of wharf (about 16% of wharf width of 60 feet) may be subject to significant damage under soil lateral sliding.
- Section B (Pier 43 and 43.5). Pier 43.5 has been recently reconstructed and it is assumed the design accommodates design basis soil lateral sliding. That reconstruction enclosed, but presumably did not accommodate the existing Franciscan Restaurant building. Pier 43, bayside piles up to 40 feet in length may be subject to lateral sliding overstress with insufficient axial capacity remaining. Estimate one bay of wharf (about 20% of wharf width of 50 feet) may be subject to significant damage under soil lateral sliding. Pier 43 has no occupied structures on it but a significant gateway portal does exist. Estimate that none (0%) of the reconstructed Pier 43.5 wharf area will be subject to significant damage under soil lateral sliding. Estimate that all (100%) of the Franciscan Restaurant wharf area will be subject to significant damage under soil lateral sliding. Estimate that all (100%) of the existing Pier 43 wharf area will be subject to significant damage under soil lateral sliding. Other than the Franciscan Restaurant, there are no major occupied structures on the reconstructed Pier 43.5 or the existing Pier 43.
- Section 1 (Pier 39), bayside piles up to 40 feet in length may be subject to lateral sliding overstress with insufficient axial capacity remaining. Estimate one bay of wharf (about 20% of wharf width of 50 feet) may be subject to significant damage under soil lateral sliding.
- Section 2 (Pier 33 and 35), bayside piles up to 40 feet in length may be subject to lateral sliding overstress with insufficient axial capacity remaining. Estimate one bay of wharf (about 16% of

wharf width of 60 feet, 20% of wharf width of 50 feet) may be subject to significant damage under soil lateral sliding.

- Section 3 (Between Piers 29 and 31). This bulkhead wharf section was analyzed as a part of this work. The rock dike appears to be founded at Elev -22 feet. All bayside rows of piles appear to penetrate the rock dike and are thus susceptible to lateral sliding failure. The supporting wharf piles have insufficient vertical support due to the rock dike. Estimate essentially all bays of wharf (about 100% of wharf width) may be subject to significant damage under soil lateral sliding.
- Section 4 (Section 5 similar at P19-23), bayside piles up to 60 feet in length may be subject to lateral sliding overstress with insufficient axial capacity remaining. Estimate one to two bays of wharf (about 50% of wharf width of 46 feet) may be subject to significant damage under soil lateral sliding.
- Section 5 (Near Pier 17). This bulkhead wharf section was analyzed as a part of this work. The rock dike appears to be founded at Elev -46 feet. The two bayside rows of piles appears to penetrate the rock dike and is thus susceptible to lateral sliding failure. The supporting wharf piles have insufficient vertical support due to the rock dike. Estimate one to two bays of wharf (about 50% of wharf width) may be subject to significant damage under soil lateral sliding.
- Section 6 (Near Pier 9). This bulkhead wharf section was analyzed as a part of this work. The rock dike appears to be founded at Elev -46 feet. The two bayside rows of piles appears to penetrate the rock dike and is thus susceptible to lateral sliding failure. The supporting wharf piles have marginally sufficient vertical support due to the rock dike. However, if the piles were to fail, estimate one to two bays of wharf (about 50% of wharf width) may be subject to significant damage under soil lateral sliding.
- Section 7 (Piers 1 thru 3), a single row of bayside piles of unknown length. Most likely subject to lateral soil sliding failure. Height of rock dike is not known. If not well founded in rock dike, estimate that the entire (100%) of the marginal wharf area will be subject to significant damage under soil lateral sliding.
- Sections 8a and 8b (Ferry Plaza), many rows of bayside piles of unknown length. Height of rock dike is unknown. Data on new construction not available. Estimate that about 50% of the marginal wharf area will be subject to significant damage under soil lateral sliding.
- Section 8 (Agricultural Building). The rock dike appears to be founded at Elev -46 feet, the seawall supporting piles are likely to have sufficient vertical support due to the rock dike. All piles appear to penetrate the rock dike and are thus susceptible to lateral sliding failure. The wharf supporting piles are likely to have sufficient vertical support due to the rock dike except for the outermost piles, say the last two bays for a 50 ft wide bulkhead wharf. Piles data for the bulkhead wharf are not available. It is assumed that about 40% of the bulkhead wharf area will be subject to significant damage under soil lateral sliding.
- Section 9a. There are no bulkhead wharf structures in this seawall section. Estimated bulkhead wharf damage is thus zero (0%).
- Section 9b (Fire Station). The fire station appears to have been located on the remnants of Pier 24. The rock dike appears to be founded at Elev -46 feet. All piles appear to penetrate the rock dike and are thus susceptible to lateral sliding failure. The wharf supporting piles are likely to have sufficient vertical support due to the rock dike except for the outermost piles, say the last two bays for a 50 foot wide bulkhead wharf. Estimate two bays of bulkhead wharf (feet, 40% of wharf width of 50 feet) may be subject to significant damage under soil lateral sliding.

- Section 9 (Between Piers 26 and 28). This bulkhead wharf section was analyzed as a part of this work. The rock dike appears to be founded at Elev -46 feet. The single row of piles appears to penetrate the rock dike and is thus susceptible to lateral sliding failure. The supporting wharf piles have insufficient vertical support due to the rock dike. It is assumed that the entire (100%) marginal wharf area will be subject to significant damage under soil lateral sliding.
- Section 10 (Near Pier 30/32). The rock dike appears to be founded at Elev -46 feet. The single row of piles appears to penetrate the rock dike and is thus susceptible to lateral sliding failure. The supporting wharf piles have insufficient vertical support due to the rock dike. It is assumed that the entire (100%) marginal wharf area will be subject to significant damage under soil lateral sliding. There are no major structures on this bulkhead wharf other than the Java Hut nearer Pier 28.
- Sections 11a and 11 (Brannan Street Wharf), Brannan Street Wharf is not structurally connected to the seawall, there is no marginal wharf connected directly to the seawall. Brannan Street Wharf was designed to withstand design basis soil lateral spreading. Height of rock dike is unknown. The seawall piles are assumed to be subject to soil lateral spreading damage. Estimate that none (0%) of the Brannan Street Wharf area will be subject to significant damage under soil lateral sliding.
- Section 12 (Near Piers 38 and 40). This bulkhead wharf section was analyzed as a part of this work. The rock dike appears to be founded at Elev -46 feet. The single row of piles appears to penetrate the rock dike and is thus susceptible to lateral sliding failure. The supporting wharf piles have marginally sufficient vertical support due to the rock dike. However, if the supporting piles were to fail, the entire (100%) marginal wharf area will be subject to significant damage under soil lateral sliding. There are no structures on the bulkhead wharf at Pier 40.
- Sections 13, P46 and China Basin. There are no bulkhead wharf structures in these seawall sections. Estimated bulkhead wharf damage is thus zero (0%).

The bulkhead wharf analyses and extrapolated results indicate varying levels of required retrofit on all seawall sections. This is shown on Figures 4-61 and 4-62 for seismic inertial loading and on Figures 4-63 and 4-64 for seismic soil lateral sliding. The estimated percentage of wharf deck damage due to soil lateral sliding is also shown on Figures 4-63 and 4-64.

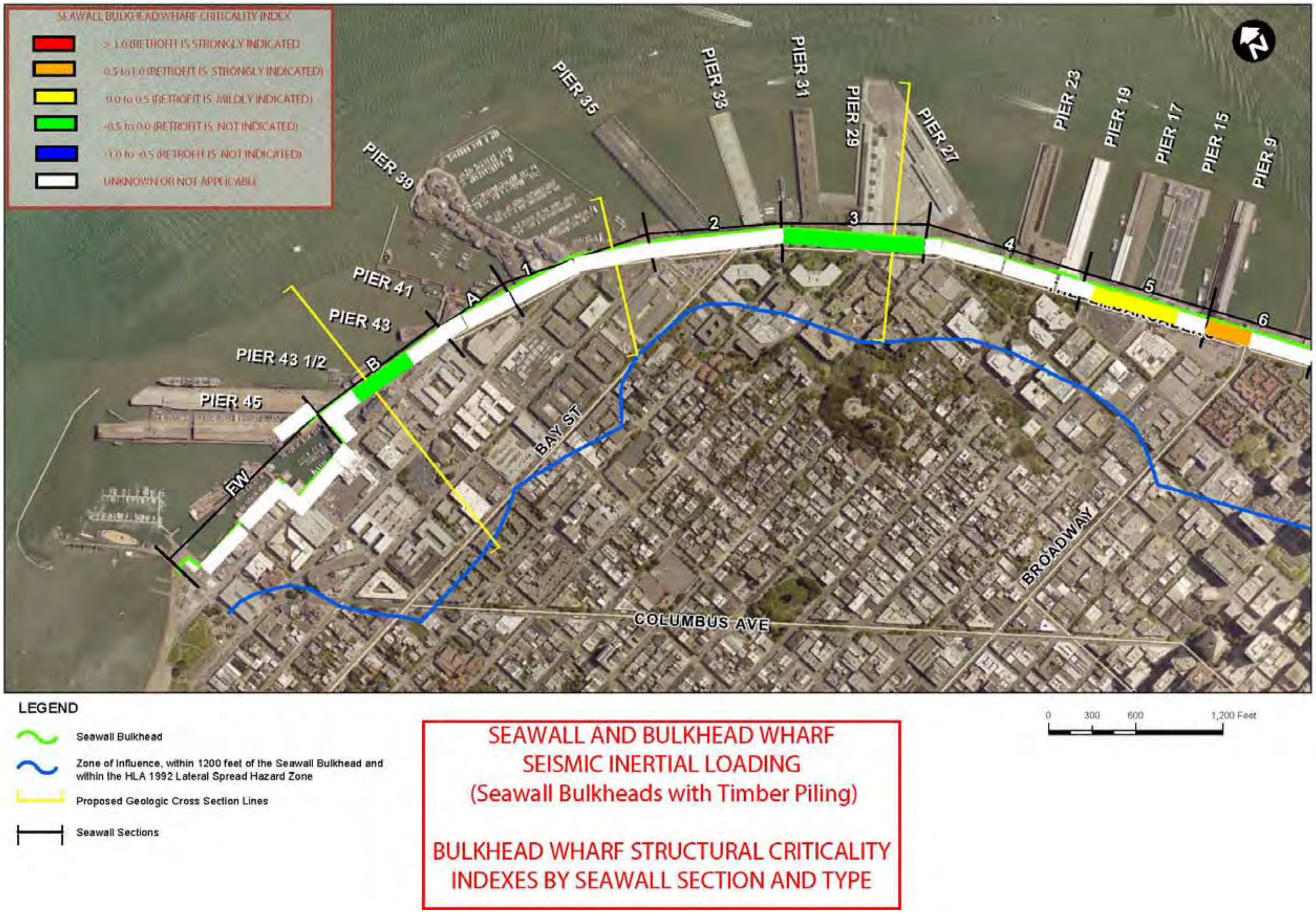


Figure 4-61: Seawall and Bulkhead Wharf Seismic Inertial Loading Summary (1 of 2)

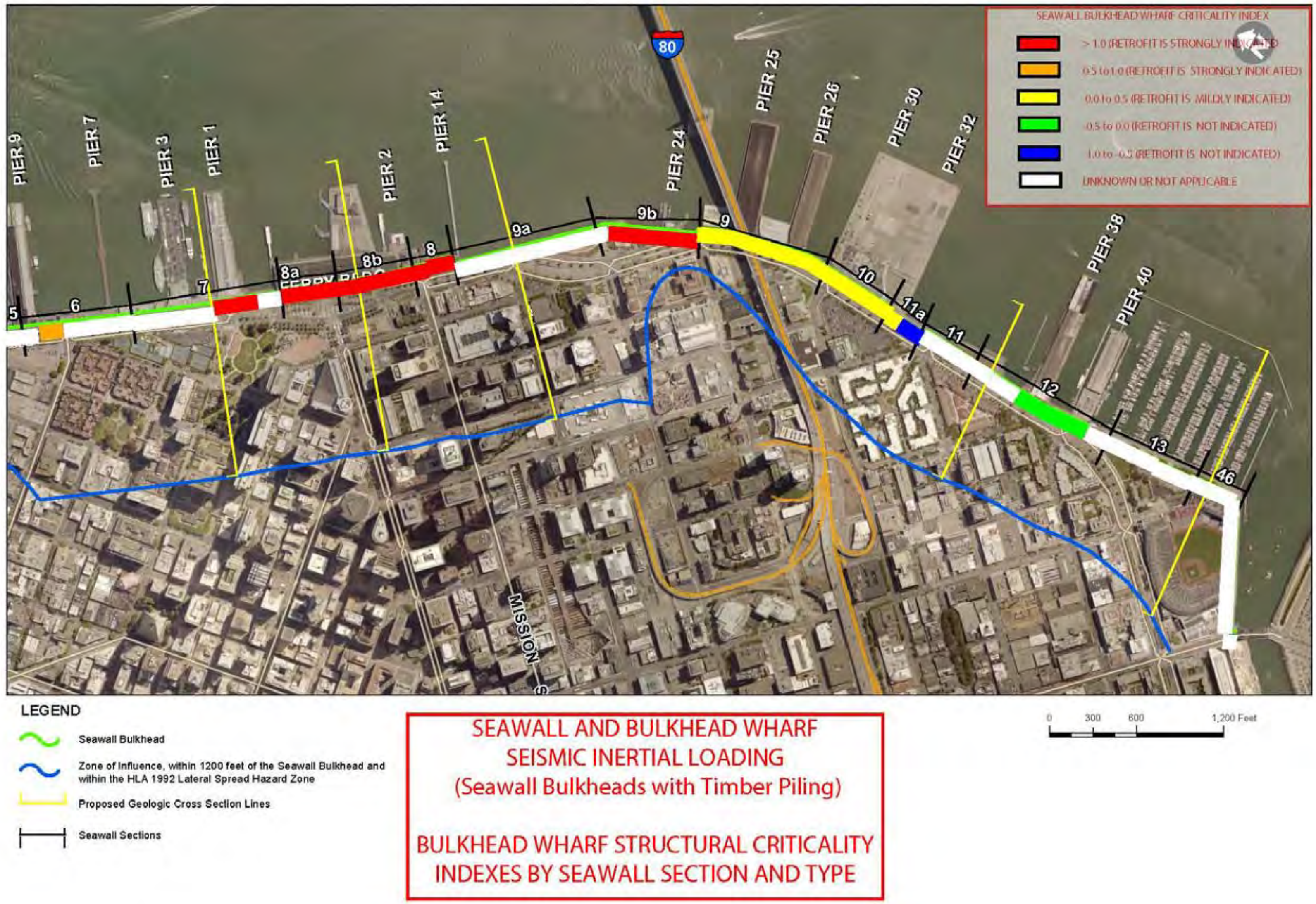


Figure 4-62: Seawall and Bulkhead Wharf Seismic Inertial Loading Summary (2 of 2)

Figure 4-63: Seawall and Bulkhead Wharf Seismic Soil Lateral Sliding Summary (1 of 2)

Figure 4-64: Seawall and Bulkhead Wharf Seismic Soil Lateral Sliding Summary (2 of 2)

Exceptions

No assessment of the seismic vulnerability of buildings or other structures located on the bulkhead wharfs has been performed.

Bulkhead wharf data for the Ferry Plaza, Pier 1, and Pier 15-17 rehabilitation are not available. Bulkhead wharf deck area earthquake damage for these sections have been estimated based upon the approximate thickness of the soft mud layer, available record drawings and assumed wharf construction.

Bulkhead wharf structures at Fisherman's Wharf have not been assessed due to lack of structural data.

Overall Conclusions

Timber piles supporting seawall bulkheads are insufficient for seismic inertial load.

Bulkhead wharf beam connections to seawalls should be inspected and assessed for structural adequacy. Such connections should be retrofitted to accommodate the expected seismic loads and/or relative displacements.

All bulkhead wharfs are susceptible to soil lateral sliding. The percentage of bulkhead wharf damage that may be expected due to soil lateral sliding is a function of the amount of pile skin friction available (ie, rock dike material) to a specific pile above the location of pile failure. Piles located nearer the rock dike toe or outside of the rock dike location are likely to have insufficient axial load capacity.

4.9. Phase 3 Work

The seawall sections and seawall types identified as having some sort of structural deficiency will be evaluated for potential seismic or other retrofit during Phase 3 of this study. The candidate seawall sections will be determined during discussions with the Port of San Francisco. Where needed, we will attempt to obtain additional as-built data. Retrofit alternatives along with associated costs will be developed during this Phase 3 work.

5. Utilities

5.1. General

The infrastructure utility systems study consists of compiling existing utility information within the zone of influence (Figure 1-1), identifying critical utilities and studying their vulnerability due to earthquake, settlement and flooding as defined by the project.

Through the Notice of Intent (NOI) process and the Lifeline Council, we have gathered existing utility system maps from some of the participating utility agencies.

Some of the system maps are in GIS format and contain useful GIS data. We created AutoCAD drawings for the system maps that are received in pdf format. Data are grouped and color coded for each utility system in individual exhibits.

We met with individual utility agencies to discuss and further understand and identify the critical elements of their system. The team presented the findings for the studied events to some of the utility agencies to understand the impact and vulnerability to their critical utility and structures.

5.2. Data Obtained

Some of the existing utility system maps received includes:

- Exhibit X_SFDPW – BOE Hydraulics (SD, SS, CS)
- Exhibit X_SFPUC – Water
- Exhibit X_SFPUC – AWSS
- Exhibit X_SFPUC – Street Lights
- Exhibit X_PGE - Gas
- Exhibit X_ATT
- Exhibit X_Verizon
- Exhibit X_Comcast
- Exhibit X_XO
- Exhibit X_Century Link
- Exhibit X_TCA

Following is some of the information gathered during the meetings with individual utility agency:

1. 5-4-15_SFPUC Hydraulic Department
    a. Hydraulic Department has an Asset Management Program name Seasam.
    b. The program generates ratings (1-100) of each segment of the piping system. 100 being worst condition.
    c. It evaluates two criteria – likelihood of structural failure and consequence of failure. Each criterion can have a rating up to 10. For example, if structural failure likelihood is 10, and the consequence of failure is 10, the rating for that segment = 10 x 10 = 100.
    d. The likelihood of structural failure input data include information provided by:
        i. TV inspection of condition
        ii. Age of pipe but no TV inspection

e. There is a lot of overlapping within the liquefication area provided by SFDPW and the seawall failure zone of influence. Suggestion is to overlay both maps to identify additional areas to study.
f. Preference to study for failure is by events.
g. Treatment Plant and Pump Stations are not part of the Seasam study. And they are within our Seawall zone of influence.

2 6-9-15_Sprint (10am – 11am)
a. Russ – representative from Sprint attended the meeting
b. Restoration of business accounts is a high priority for Sprint
c. The wireless system consists of a network of cell sites and communication lines. All data transmitted and received via cell sites still relays through a network of in-ground communication lines (Back Haul) back to the major communication hub, then is distributed.
d. Most critical backup for cell site to function, other than sever communication lines, is power to the cell sites.
e. Sprint does share communication conduits from other carriers.
f. There is a major transbay conduits duct bank within the seawall zone of influence.

3 6-9-15_Verizon Wireless (11am – noon)
a. Macros are stand alone cluster of 9 antennas. IDAS are in-building antennas. Small are antennas on share poles with MTA, PUC, PGE brown poles.
b. Restoration of services within the Golden sites are high priority. Financial District is one of their golden sites.
c. Verizon does serve some of the City's emergency response agencies and the reliability of their services is critical.
d. Verizon wireless system runs similar to the one discussed for Sprint. Their in-ground communication lines (Back Haul) relay information through the Major Hub building located in Santa Clara. There are many built-in redundancies within the Verizon system.
e. The similar issue will be loss of power during emergency. Permanent generator is fueled by natural gas. During an emergency, portable electrical generator may be required to get service back on line. But individual permitting is very difficult within the city.
f. Suggestion is to work with the City to create a blanket permit that allows easier deployment of emergency generators for critical situations.

4 6-11-15_SFMTA
a. Metro East Rail Yard and the Kirkland Yard for diesel buses are within the Seawall zone of influence.
b. Folsom Street Portal is another major SFMTA yard.
c. SFMTA has 2 emergency response plans. Evacuation plan is a critical element. Their response time varies between 72 hours to 30 days.
d. More concerns of seawater intrusion to the subway tunnels. Any causes from an emergency seawall failure, event triggered Tsunami or from the sea level rise. As this will affect their evacuation plan. Very interested in further discussion about this issue.

5.3. Data Gaps and Assumptions

Some files only include the downtown area and the file data do not extend the information to include the remaining area that is within the Seawall study zone of influence. Data for other agencies, such as PGE - Electrical, SFMTA are still missing.

6. Flooding Vulnerability

Environmental Science Associates (ESA) performed flood exposure mapping and hazard vulnerability during Phase 2 of the study. These results are presented in this section.

6.1. Introduction

The vulnerability of the San Francisco waterfront to flooding and inundation was evaluated for different seismic scenarios that could occur for both existing and future conditions with sea level rise. The assessment utilized prior studies completed for the Port and the City and County of San Francisco, as well as recently adopted guidance for incorporating sea level rise into planning in San Francisco, to define the existing and future flood elevations, extents and pathways. Refinements to these data were performed to best represent the potential impacts of different combinations of sea level rise, storms, and intact or deformed seawall and its zone of influence. Flood vulnerability was measured on a semi-quantitative basis using criteria that was developed during the study in collaboration with the project team and the Port of San Francisco.

The approach to evaluating the flood vulnerability along the San Francisco waterfront comprised selecting flood elevations and estimating the approximate extents of flooding for the conditions of an intact seawall and a damaged seawall associated with seismic scenarios. The increase in risk over time was assessed by considering sea level rise amounts consistent with City guidance at 2050 and 2100. Still water level (SWL) elevations and wave runup heights along the study area were derived using the SFPUC (2014) and URS and AGS (2012) mapping and tabulations of values.

The following sections are organized as follows:

- Section 6.2 Key Terminology, Datums, and Extreme Values: presents a summary of the tidal elevations and extreme water levels along the San Francisco waterfront, as well as defining terminology that is used in coastal flooding and vulnerability assessments;
- Section 6.3 Jurisdiction, Policy, and Sea Level Rise Guidance: presents a description of pertinent policies and guidance for incorporating sea level rise into planning, sea level rise projections, and defines vulnerability and risk terminology;
- Section 6.4 Available Maps and Data Products: summarizes available coastal flood maps and data for existing and future conditions with sea level rise along the San Francisco waterfront;
- Section 6.5 Approach to Assessing Flooding Vulnerability: describes the approach that was used to evaluate the vulnerability of the San Francisco waterfront to flooding for existing and future conditions with sea level rise.

6.2. Key Terminology, Datums, and Extreme Values

This section presents a description of the terminology used in coastal flooding analysis, tidal datums and elevations used along the San Francisco waterfront, and extreme values of water levels and wave runup elevations.

Coastal Flooding Terminology

Coastal flooding is caused by a combination of tides, storm surge, and the effects of waves, including wave setup and wave runup (Figure 6-1). These physical processes are derived from measurements of water levels and waves and from hydrodynamic models. Flood elevations are typically reported using the following terminology (FEMA 2005):

- The still water level (SWL) is the elevation of the free surface in the absence of waves and wave effects, and includes the astronomical tide, El Nino, and surge due to wind effects
- Wave setup is the additional elevation of the water level due to the effects of transferring wave-related momentum to the surf zone
- Wave runup is the the vertical extent of wave uprush on the shore or a structure
- The total water level (TWL) is the sum of the SWL, the wave setup, and wave runup

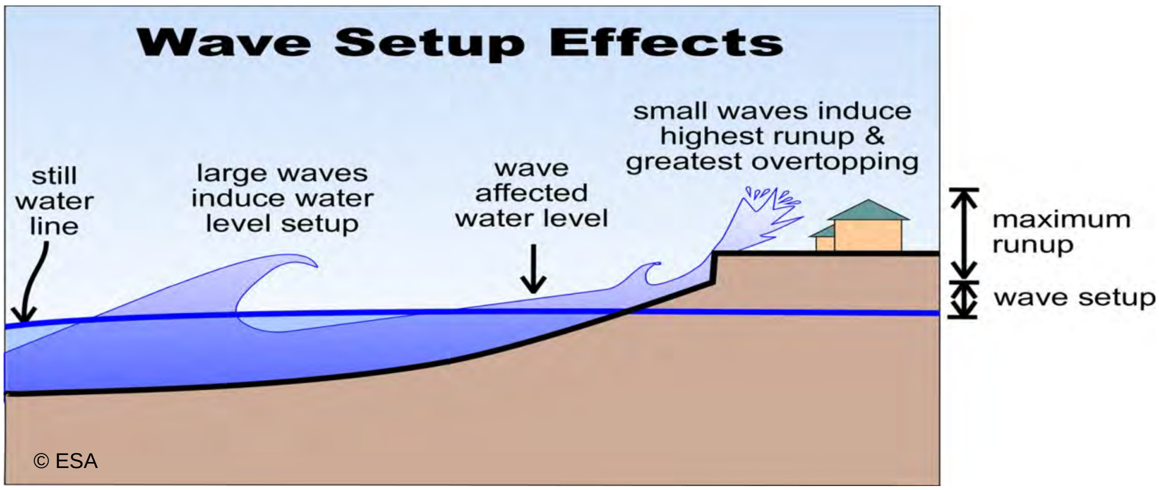


Figure 6-1: Diagram illustrating the still water level (SWL), wave setup, and wave runup: The total water level (TWL) is the elevation of the maximum wave runup

Recurrence frequencies are commonly used to describe the probability of an extreme event occurring within a given time period. The return period, or recurrence interval, is an estimate of the likelihood of an event and is based on the probability that the given event will be equaled or exceeded in any given year. For example, the 100-year SWL is the flood level that has a 1% chance of being equaled or exceeded in any given year. Similarly, the 100-year TWL can be calculated, although the 1% TWL does not correspond to any single physical event. Rather, it is an extrapolation of the TWL conditions from the largest events because of the limited duration of the available data (FEMA 2005). Wave overtopping occurs if the TWL exceeds the backshore elevation. The TWL primarily depends on the water level, wave conditions, and the beach face or structure slope.

Datums

Water levels are commonly referenced to two datums along the San Francisco waterfront: NAVD88 and the San Francisco City Datum. Published tidal datums derived from water level of measurements at the San Francisco Presidio tide gage (NOAA NOS Station 9414290) can be converted from NAVD88 to the San Francisco City datum by subtracting 11.326 feet (Table 6-1). This report presents existing and future water surface elevations in feet relative to NAVD88.

Table 6-1: Summary of tidal datums from the San Francisco Presidio tide gage, NOAA NOS Station 9414290, relative to NAVD88 and the San Francisco City Datum

Datum	NAVD88 (feet)	SF City Datum (feet)**
SF City Datum	11.326	0
Highest Observed Water Level (1/27/83)	8.72	-2.606
Mean Higher High Water (MHHW)	5.92	-5.406
Mean High Water (MHW)	5.31	-6.016
Mean Tide Level (MTL)	3.26	-8.066
Mean Sea Level (MSL)	3.20	-8.126
NGVD29	2.72	-8.606
Mean Low Water (MLW)	1.22	-10.106
Mean Lower Low Water (MLLW)	0.08	-11.246
NAVD88	0	-11.326
Lowest Observed Water Level	-2.82	-14.146

*** Conversion from NAVD88 to SF City Datum based on: SF City Datum = 11.326 feet NAVD88*

Extreme Values

Several studies have estimated extreme values of water levels in San Francisco Bay (USACE 1984a; PWA 2007; DHI 2011; URS and AGS 2012; SFPUC 2014). Although these studies rely on measurements at the Presidio tide gage, the extreme values differ due to differences in the methods used:

- Length of time series: studies for FEMA and SFPUC used shorter time series of 30 years (DHI 2011), whereas studies for the Port and the State considered the full record extending to 1901
- Extreme value distribution: URS and AGS (2012) fit a Weibull distribution to the data and DHI (2011) fit a GEV distribution to the shorter time series, which gives higher values.

The 100-year SWL of 9.2 to 9.3 feet NAVD was reported by the study for the Port (URS and AGS 2012). In the study for FEMA, DHI (2011) reported 100-year SWL of 9.6 to 9.8 feet NAVD, approximately 0.5 feet higher than the value developed using the longer time series and less conservative extreme value distribution. The future extreme SWL is typically calculated by adding the sea level rise amount to the extreme still water level for existing conditions and is described further in Section 6.4.3.

The existing 100-year TWL was estimated along the waterfront for the Port (URS and AGS 2012). TWL values up to 13.2 feet NAVD were reported in areas exposed to longer fetches and predominant wind directions. The mapping products and future TWL is described further in Section 6.4.3.

6.3. Jurisdiction, Policy, and Sea Level Rise Guidance

Guidance for assessing the risks of sea level rise has been issued by the State of California as well as the City and County of San Francisco. The guidance generally presents projections of sea level rise through 2100, and describes recommended methods for evaluating risk and incorporating sea level rise into planning. Summaries of guidance recently adopted by the City and County of San Francisco and issued by the State of California are presented below.

Guidance for Incorporating SLR into Capital Planning: OneSF (CCSF 2014)

As part of the City and County of San Francisco's *OneSF*[1] program, new sea level rise guidance was adopted by the City to require that sea level rise is incorporated into the capital planning and projects that could be impacted by sea level rise. The program recommends using sea level rise values of 1 foot and 3 feet for the years 2050 and 2100, respectively, which are considered mid-range but likely sea level rise projections in NRC (2012) and which comply with guidance established by the State (OPC 2013). These projections were adopted by the CCSF (2014) to be used for consistency in project planning and vulnerability assessments. The program recommends using the SFPUC (2014) inundation mapping to evaluate the impacts to projects and established guidelines for assessing the risk of sea level rise largely consistent with requirements of the San Francisco Bay Conservation and Development Commission (BCDC) and the California Coastal Commission (CCC).

State of California Sea Level Rise Guidance Document (OPC 2013)

On March 15, 2013, the Ocean Protection Council (OPC) staff presented an update to the *State of California Sea-Level Rise Interim Guidance Document*. The purpose of the document remained the same, to help state agencies incorporate future sea-level rise impacts into planning decisions, and was updated to include the best available science from the National Research Council: Sea-Level Rise for the Coasts of California, Oregon, and Washington (NRC 2012). The guidance document seeks to enhance consistency across agencies as each develops its respective approach to planning for sea level rise. Table 6-2 summarizes the NRC (2012) recommended sea level rise projections for use along the coast of California south of Cape Mendocino. Note that the OPC (2013) presents only the ranges of sea level rise (second column), while CCSF (2014) adopted the mid-range "projection" (third column) presented in NRC (2012).

Table 6-2: Recommended Sea Level Rise Projections by NRC (2012)

Time Period	Sea Level Rise Ranges	Mid-level Projection**
2000-2030	2 - 12 inches	6 ± 2 inches
2000-2050	5 - 24 inches	11 ± 4 inches
2000-2100	17 - 66 inches	36 ± 10 inches

*** The mid level curve is referred to as a "projection" in some parts of the NRC (2012) report but is not referred to as such in the OPC (2013) State guidance adopting the NRC (2012) report. OneSF emphasizes the mid-level as a projection. However, the USACE, State, BCDC and CCC have not yet adopted this distinction and have maintained a range.*

The OPC's 2013 *California Sea Level Rise Guidance Document* contains seven recommendations for incorporating sea level rise into project planning:

1. Use the ranges of sea level rise presented in NRC (2012) as a starting place and select sea level rise values based on agency- and context-specific considerations of risk tolerance and adaptive capacity;
2. Consider timeframes, adaptive capacity, and risk tolerance when selecting estimates of sea level rise;

[1] City and County of San Francisco's OneSF program: http://onesanfrancisco.org/

3. Coordinate with other state agencies when selecting sea level rise projections, and use the same projections, where feasible;
4. Do not base future sea level rise projections on linear extrapolation of historic sea level observations;
5. Consider trends in relative local mean sea level;
6. Consider storms and extreme events; and
7. Consider changing shorelines.

The guidance document is expected to be updated regularly, to keep pace with scientific advances associated with sea level rise. This guidance is generally considered to be based on the best scientific data available as of the date of this summary, and is used by BCDC when reviewing projects planned for the shoreline within BCDC's jurisdiction.

6.4. Available Maps and Data Products

Several studies have been conducted that evaluate the existing and future flood risk along the San Francisco Waterfront. The sections below present summaries of available coastal flood and sea level rise maps and data products developed for the following:

- FEMA preliminary flood maps and San Francisco Interim Flood Plain Maps
- SFPUC SSIP Sea Level Rise Inundation Mapping (SFPUC 2014)
- Sea level rise mapping for the Port of San Francisco (URS and AGS 2012)

100-year Flood Zones from San Francisco Interim Flood Plain Maps

The Federal Emergency Management Agency (FEMA) released a preliminary Flood Insurance Rate Map (FIRM) for the City and County of San Francisco on September 21, 2007, which includes approximate designations of property in coastal flood hazard zones for existing conditions. The preliminary FIRM shows Special Flood Hazard Areas within the City as:

- Zone A: areas of coastal flooding with no wave hazard; or waves less than three feet in height; and
- Zone V: areas of coastal flooding subject to the additional hazards associated with wave action.

The preliminary FIRM does not associate the Special Flood Hazard Areas with elevations, but it does provide an indication of the approximate extents of the 100-year coastal flood zone.

Based on the preliminary FIRM, the City created the "Interim Floodplain Map" to support the implementation of the Floodplain Management Ordinance.[2] The Interim Floodplain Map shows that limited areas along the waterfront are within the extents of the Special Flood Hazard Area (Figure 6-2).

FEMA is in the process of completing coastal engineering analyses and mapping of the San Francisco Bay shoreline to provide flood and wave data for the City and County of San Francisco's Flood Insurance Study (FIS) report and Flood Insurance Rate Map (FIRM) panels. Although revised preliminary maps have been prepared, the CCSF and FEMA are currently finalizing the mapping products prior to public release. However, a KMZ file is available on the FEMA website that contains Google Earth layers of designated special flood hazard zones and their elevations, which we reviewed. However, the Port of San Francisco is still reviewing the revised provisional flood hazard maps and expects them to change, and therefore does not want them used in this study.

[2] Floodplain Management Ordinance for the City and County of San Francisco can be accessed online: http://sfgsa.org/Modules/ShowDocument.aspx?documentid=7520

The effects of sea level rise are not included in the FEMA data shown in Figure 6-2. The future flood limits with SLR can be calculated by adjusting the FEMA map and reevaluating the wave contributions to flooding.

Figure 6-2: 2007 Preliminary FEMA Map (left) and 2008 San Francisco Interim Flood Plain Map (right)

SFPUC SSIP Sea Level Rise Inundation Mapping

The San Francisco Public Utilities Commission (SFPUC) recently developed SLR inundation maps for the shore of San Francisco to inform the Sewer System Improvement Program (SSIP). The maps prepared for the "Bayside" show inundation resulting from an increase in the MHHW by fixed amounts of sea level rise, and do not include the effects of wind or waves which would tend to increase the flood hazard (SFPUC 2014). The description of the analysis performed indicates that the tidal modeling using MIKE21 performed by DHI (2011) for FEMA were used to estimate tidal statistics (e.g. MHHW and extreme recurrences of the SWL). For each of the several points along the shore, tidal water levels were projected into the future by adding fixed amounts of SLR to the calculated MHHW elevation. The "future" water levels were then projected landward along transects to map the extents and depths of inundation for areas that are hydraulically connected to the Bay water surface elevation with defined flow paths. Areas hydraulically disconnected without a direct flow path were also mapped as low-lying areas with ponding potential. This process is known as a "bathtub" model, dependent on several assumptions including:

- The effects of waves and wind are not included, which tends to understate the hazard; and
- The mechanisms of flooding and drainage are simplified to allow the site to fill instantaneously, which tends to overstate the hazard.

Several model extraction points are located over the length of the study boundaries. The range in daily tidal elevation and extreme (2-, 50-, and 100-year) SWL for existing conditions reported in SFPUC (2014) are:

- Existing MHHW = 6.1 to 6.3 feet NAVD;
- Existing 2-year SWL = 7.6 to 7.9 feet NAVD;
- Existing 50-year SWL = 9.1 to 9.3 feet NAVD; and
- Existing 100-year SWL = 9.6 to 9.8 feet NAVD.

Note the 100-year SWL calculated by DHI (2011) for FEMA is approximately 0.5 foot higher than several other studies that report a 100-year SWL of 9.2 feet NAVD (USACE 1984a; PWA 2007; URS and AGS

2012). This difference resulted from the statistical analysis methods used: DHI (2011) used a shorter time series of 30 years and fit a GEV distribution, which gives higher numbers, whereas the other studies used a longer data set from the Presidio tide gage and used a different extreme value distribution, yielding the 100-year SWL of 9.2 feet NAVD that is widely used and referenced.

The PUC produced maps showing inundation for different water levels. The water levels are listed in terms of Mean Higher High Water (MHHW) plus additional heights of water that represent several combinations of sea level rise and storm surge contributions. Each map produced is representative of several different sea level rise and storm surge scenarios. Table 4 of SFPUC (2014) presents a summary of the available maps and scenarios as a function of sea level rise and extreme storm surge. In this way, we can use the maps to represent extreme water levels and sea level rise amounts (Table 6-3).

Table 6-3: Typical and extreme still water levels extracted along the study site from the SFPUC (2014) and adjusted with sea level rise

Time Horizon	Sea Level Rise Amount (feet)	Mean Higher High Water (feet NAVD)	5-year SWL (feet NAVD)	**50-year SWL (feet NAVD)**	**100-year SWL (feet NAVD)**
Existing	0	6.1 - 6.3	8.0 – 8.3	9.1 - 9.3	9.6 - 9.7
2050	1	7.1 - 7.3	9.0 – 9.3	10.1 - 10.3	10.6 - 10.7
2100	3	9.1 - 9.3	11.0 – 11.3	12.1 - 12.3	12.6 - 12.7

As an example, Figures 6-2 and 6-3 present the inundation caused by the existing MHHW plus 52” and 77” of sea level rise, respectively, which are close to the calculated future SWL for the future 100-year events.

Due to the differences in mapping approaches and methods, the future elevations estimated using the inundation mapping can not be compared to the FEMA draft FIRM coastal flood elevation. However, the accommodation of sea level rise and other effects such as waves, or freeboard, can be approximated by subtracting the extreme 100-year SWL for existing conditions. This yields a freeboard of 1 foot and 3 feet for the 52” and 77” SLR scenarios, respectively.

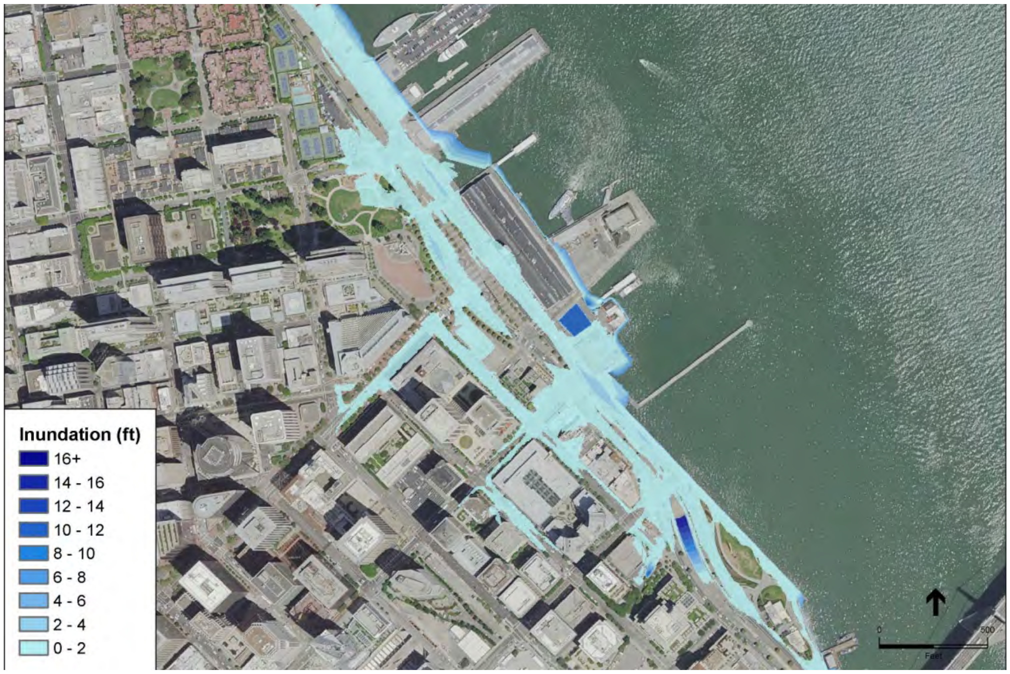


Figure 6-3: Flood inundation mapping for the existing MHHW + 52”: representative of the 100-year SWL at year 2050 (SFPUC 2014)

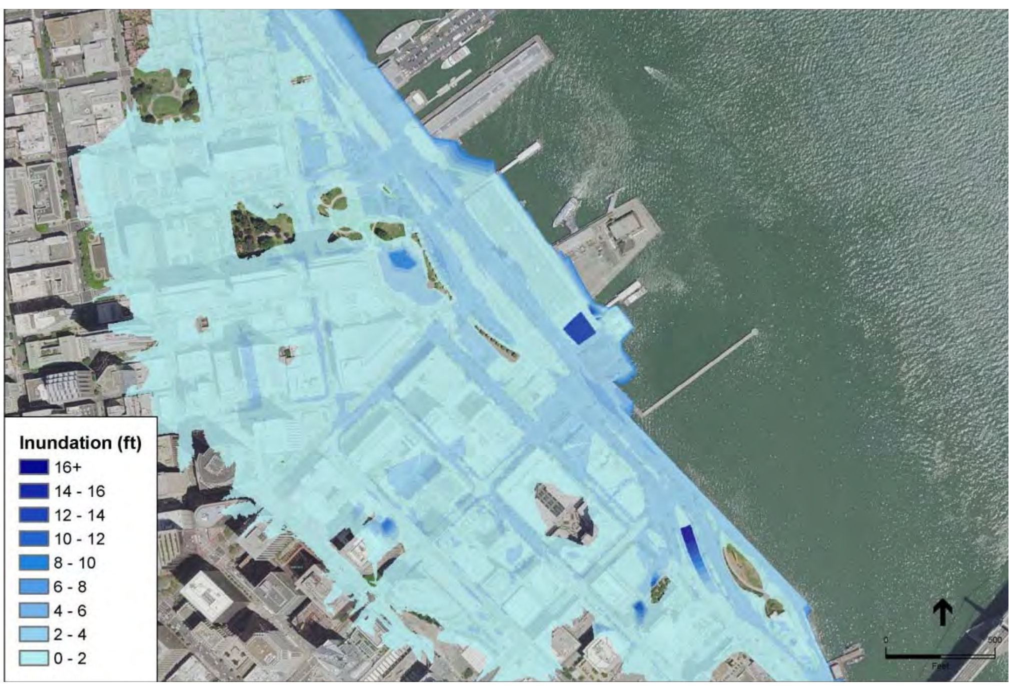


Figure 6-4: Flood inundation mapping for the existing MHHW + 77”: representative of the 100-year SWL at year 2100 (SFPUC 2014)

Sea level rise mapping for the Port of San Francisco

Sea level rise and coastal flood mapping prepared for the Port of San Francisco shows the inundation of existing (2010) and future (2050 and 2100) 100-year SWL, and estimates of the 100-year total water level (TWL) along the waterfront (URS & AGS 2012). The Port of San Francisco maps differ in that the fixed amounts of SLR are tied to planning horizons and are added to the existing extreme 100-year SWL, rather than add a fixed amount of SLR to the MHHW or daily tidal inundation elevation. In addition, the contribution of extreme wave runup was estimated along the San Francisco waterfront for existing and future conditions, although only the extreme SWL was mapped.

Mapping of the future water levels were completed by adding 15" and 55" of SLR to the existing 100-year SWL for 2050 and 2100, respectively (Figure 6-5). The UGS & AGS (2012) study estimated the 100-year SWL to be approximately 9.2 feet NAVD along the waterfront using a MIKE21 model (different from the DHI 2011 model), more than 0.5 lower than that estimated for the FEMA study. The mapped limits shown in Figure 6-5 represent a "bathtub" projection the existing and future 100-year SWL without the effects of waves and wind. This is similar to the SFPUC (2014) study, except that an extreme event is mapped, and different values of SLR are used.

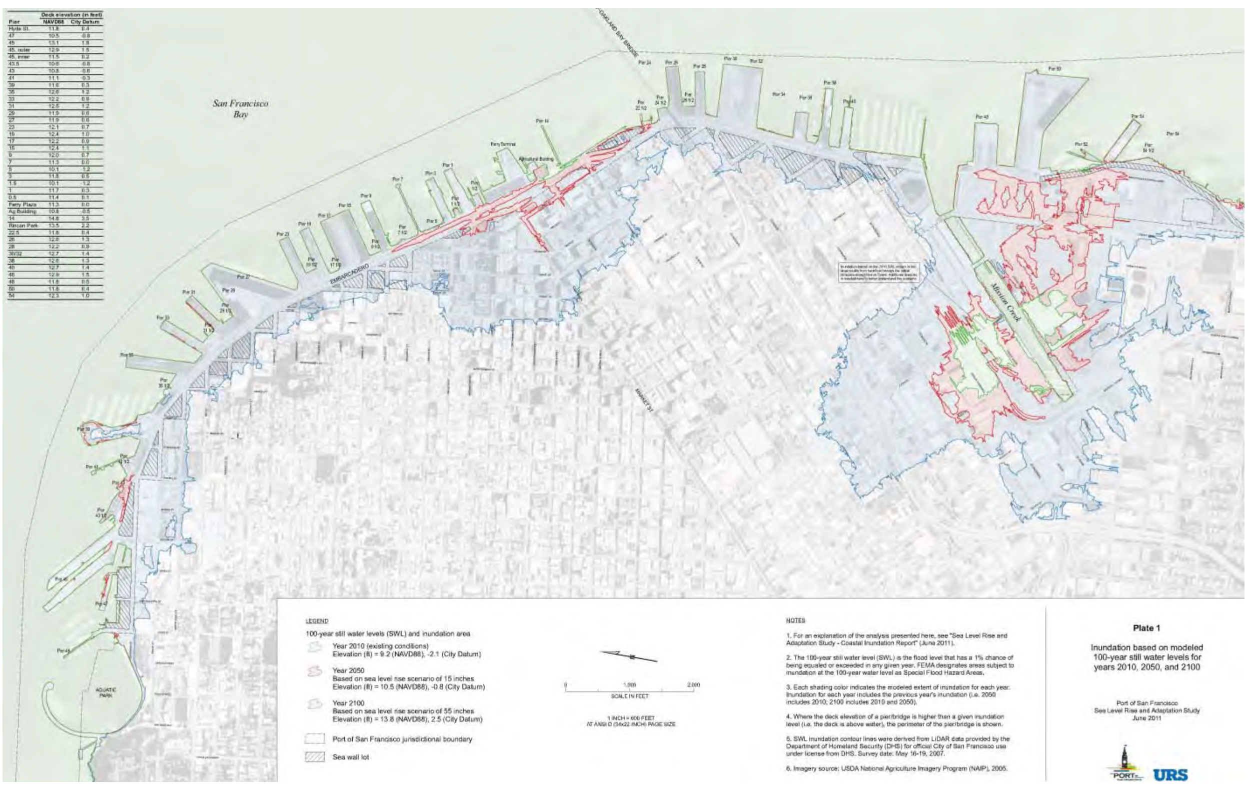


Figure 6-5: Mapping of the 100-year SWL for years 2010, 2050, and 2100 (URS & AGS 2012)

The 100-year total water levels along the waterfront were estimated for existing and future conditions using a time series methodology at several analysis points. Figure 6-6 shows the results of the TWL analysis completed for 2100. The results show that although the 100-year SWL increases by the SLR projection, the 100-year TWL increases non-linearly due to changes in the depths and resulting wave runup heights.

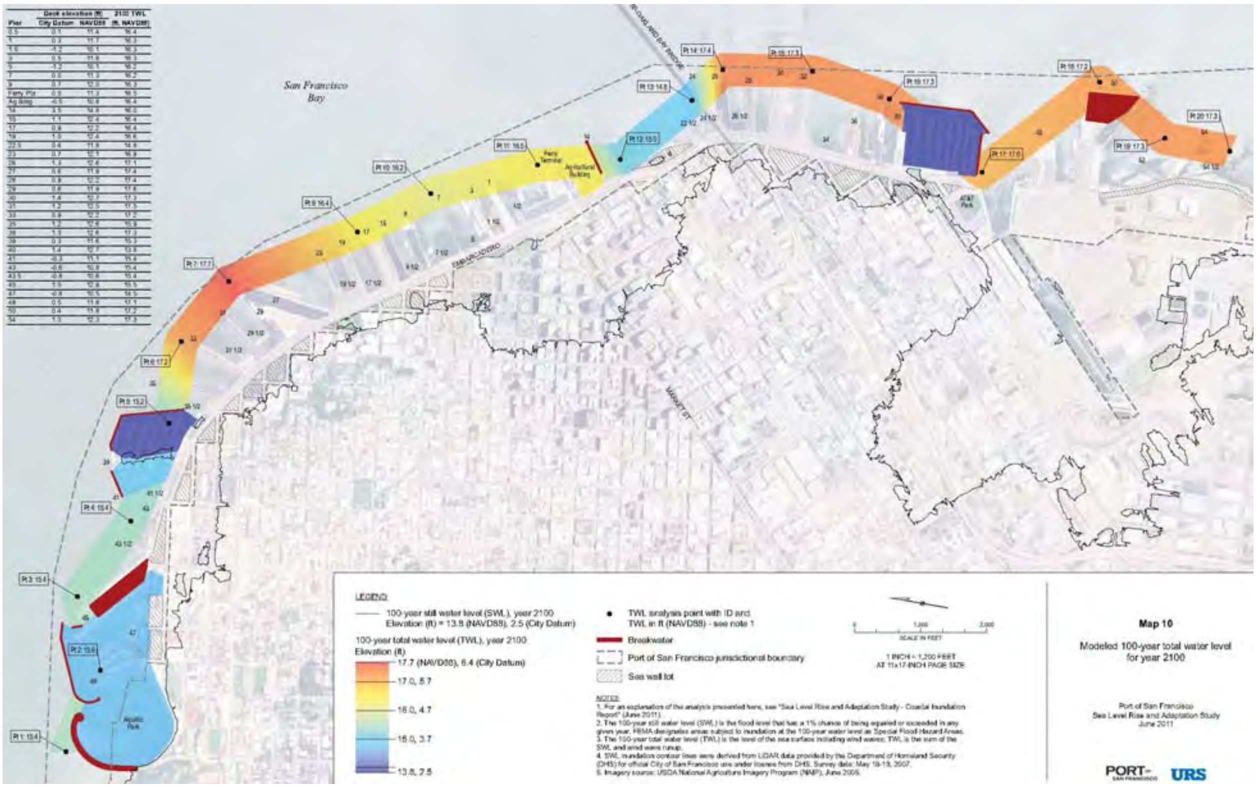


Figure 6-6: 100-year Total Water Level for years 2100 (URS & AGS 2012)

6.5. Flood Exposure Mapping

This section describes the technical analysis used to assess the flood hazard for still water level inundation and wave runup hazards along the waterfront for intact and damaged seawall conditions. The analysis is based on selecting a still water level that could occur within a reasonable risk threshold of about 40%, and converting the runup elevations presented by URS and AGS (2012) to approximate landward extents of wave runup. At this point, mapping has been completed only for the intact seawall conditions, but mapping will be completed for damaged conditions now that vertical and lateral land deformation calculations have recently been completed.

Selection of Still Water Level Recurrence Interval

The 5-year SWL elevation was selected as a reasonable extreme event that has a risk of occurrence of approximately 40% over a two year timeframe, which is assumed to represent the maximum amount of time that damaged conditions would persist. Based on Table 4 of SFPUC (2014), the following SWL maps were selected to represent the project time horizons:

- Existing (2010): MHHW + 24” ≈ 8 feet NAVD
- Future (2050): MHHW + 36” ≈ 9 feet NAVD
- Future (2100): MHHW + 66”: ≈ 11 feet NAVD

Wave Runup Adjustment and Landward Extents

Total water levels (TWLs) were leveraged from the URS and AGS (2012) report. These values were originally calculated for existing and future conditions with sea level rise amounts of 15” and 55” by 2050 and 2100, respectively. ESA linear interpolated the 100-year TWL values over relative sea level rise amounts to yield the 100-year TWL associated with 12” and 36” of SLR. The linear interpolation was conducted at each analysis point along the SF waterfront in the project limits. Ground elevations for each seawall reach along the project extents were selected after reviewing publically available LiDAR datasets collected in 2010 by NOAA and the USGS.[3]

The landward extent of wave runup was estimated for each seawall reach within the project limits using a composite slope method described in the *Shore Protection Manual* (USACE 1984b). A wave height and water level combination was derived by assuming a 5-year recurrence still water level and backing out an incident wave height that would result in the URS and AGS (2012) tabulated 100-year TWL. A wave runup height to wave height ratio of 1.5 was selected for typical conditions in SF Bay from Figure 7-14 of USACE (1984b). The TAW method (van der Meer 2002) was used to iteratively calculate the wave runup for various slopes and converging when the resulting wave runup equaled the existing ground surface elevation. The slope projection extended landward from an elevation one wave height below the still water level to the existing ground surface. This analysis resulted in landward runup extents between 2 feet and over 50 feet.

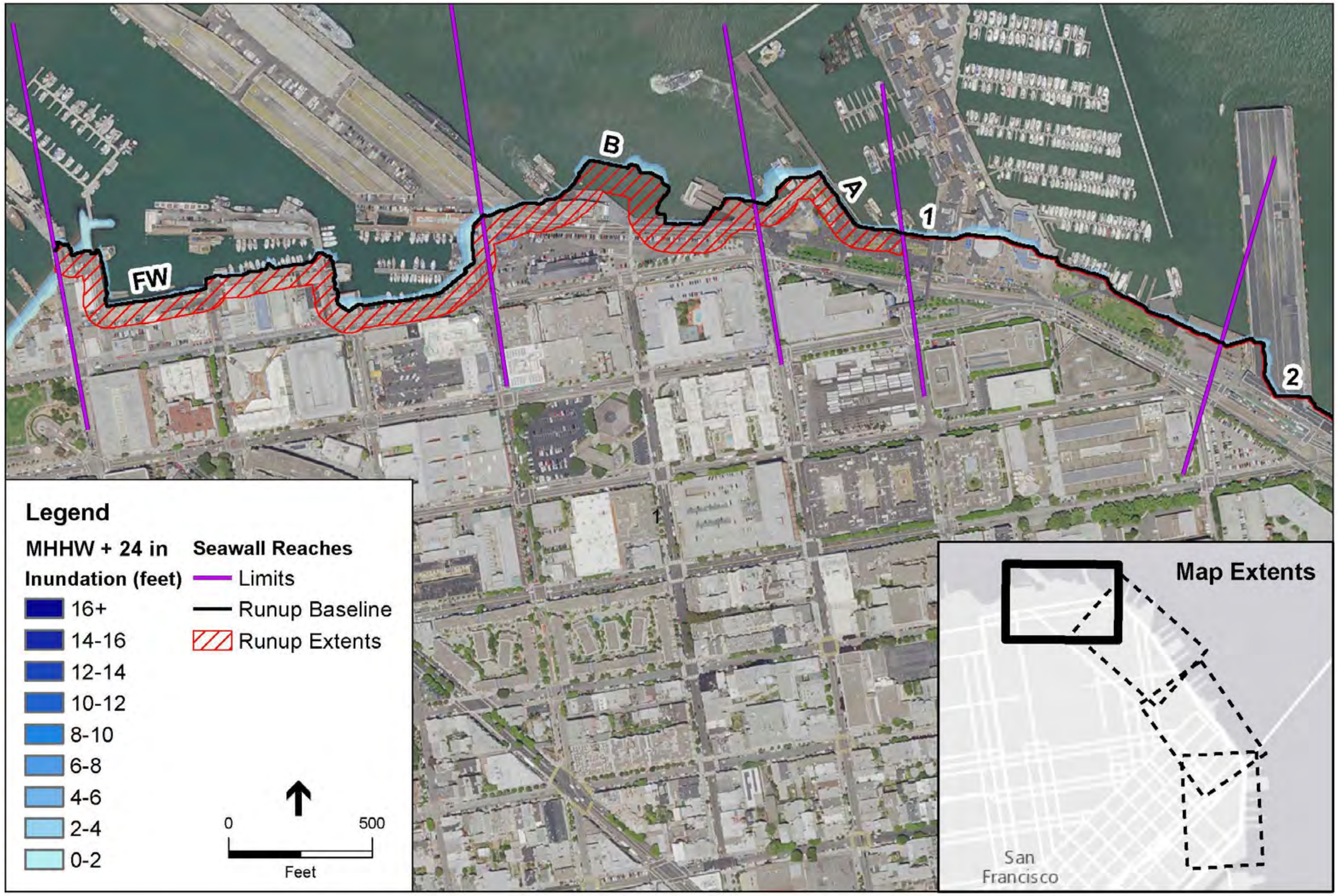
The results of the composite slope method yielded greater runup extents than other methods investigated for this study. Generally, the results were approximately 5 times greater than the extents calculated using the Cox and Machemehl (1986) equation for overland bore propagation. However, in reaches where the 5-year still water level exceeded the ground elevation, the Cox and Machemehl (1986) equation was used and multiplied by a factor of 5.

Flood Hazard Maps for Non-Damaged Conditions

Figures 6-7, 6-8, and 6-9 present flood hazard maps for years 2010, 2050 and 2100, respectively, for non-damaged conditions. Inundation resulting from a 5-year still water level is shown in varying shades of blue, and was leveraged directly from SFPUC (2014) project data. The hatched red areas represent wave hazard zones resulting from wave overtopping at the Bayward edge of development (shown as black line in the figures).

The maps indicate an increase in vulnerability with time, due to sea level rise effects on still water inundation and the increase in wave hazard exposure. The wave hazard limits are approximate and represent a worst case scenario in which piers, breakwaters and other structures do not affect or limit the wave exposure on the developed shoreline. Additional refinements to the wave hazard limits are warranted, and these results should be considered a first-cut draft and approximate potential.

[3] LiDAR Data Accessed on NOAA’s Digital Coast Data Viewer: http://coast.noaa.gov/dataviewer/



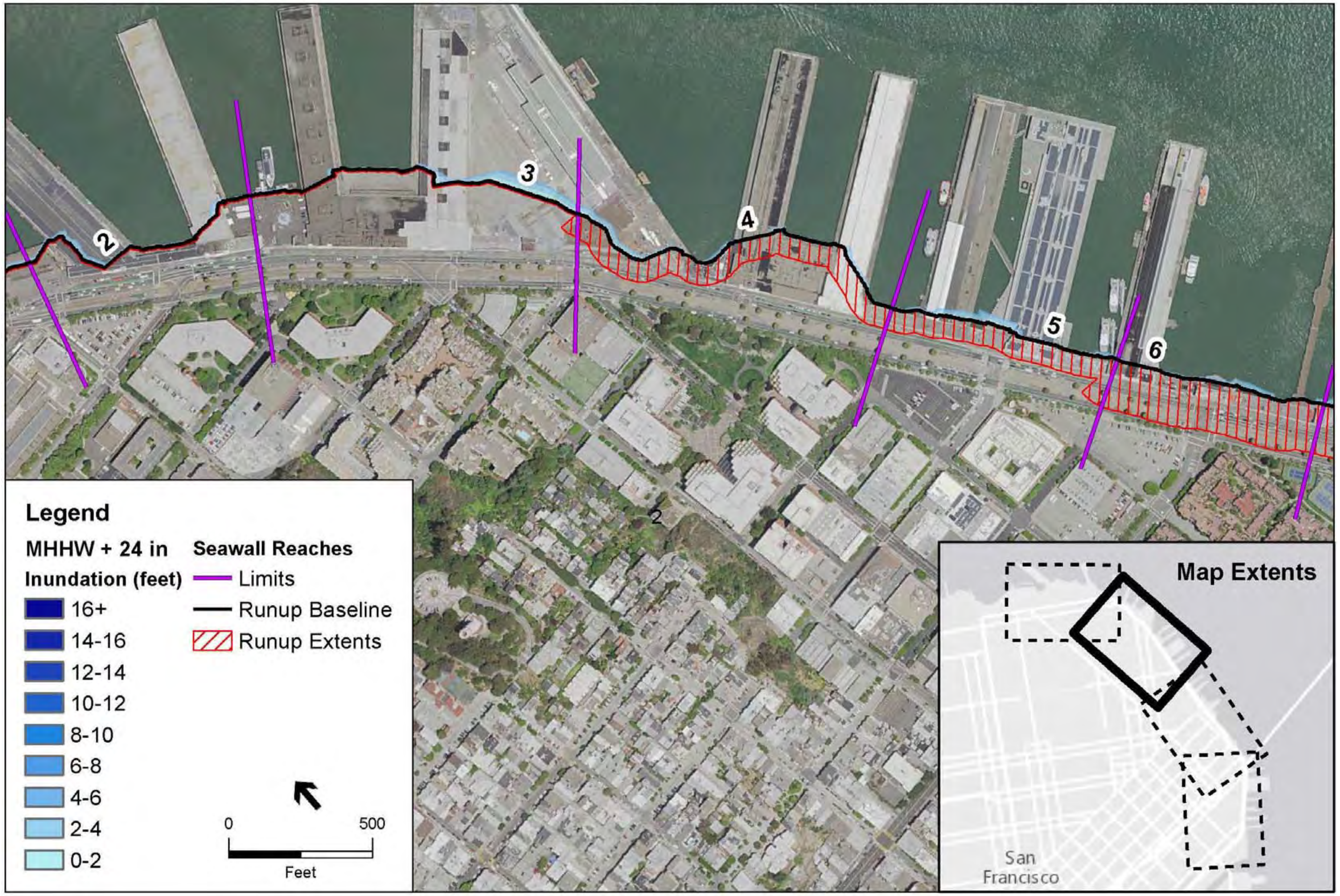
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-7a

5-Year SWL Inundation and Potential Wave Hazard Zone in 2010
Seawall Reaches FW-1

Figure 6-7a: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2010 (Seawall Reaches FW-1)



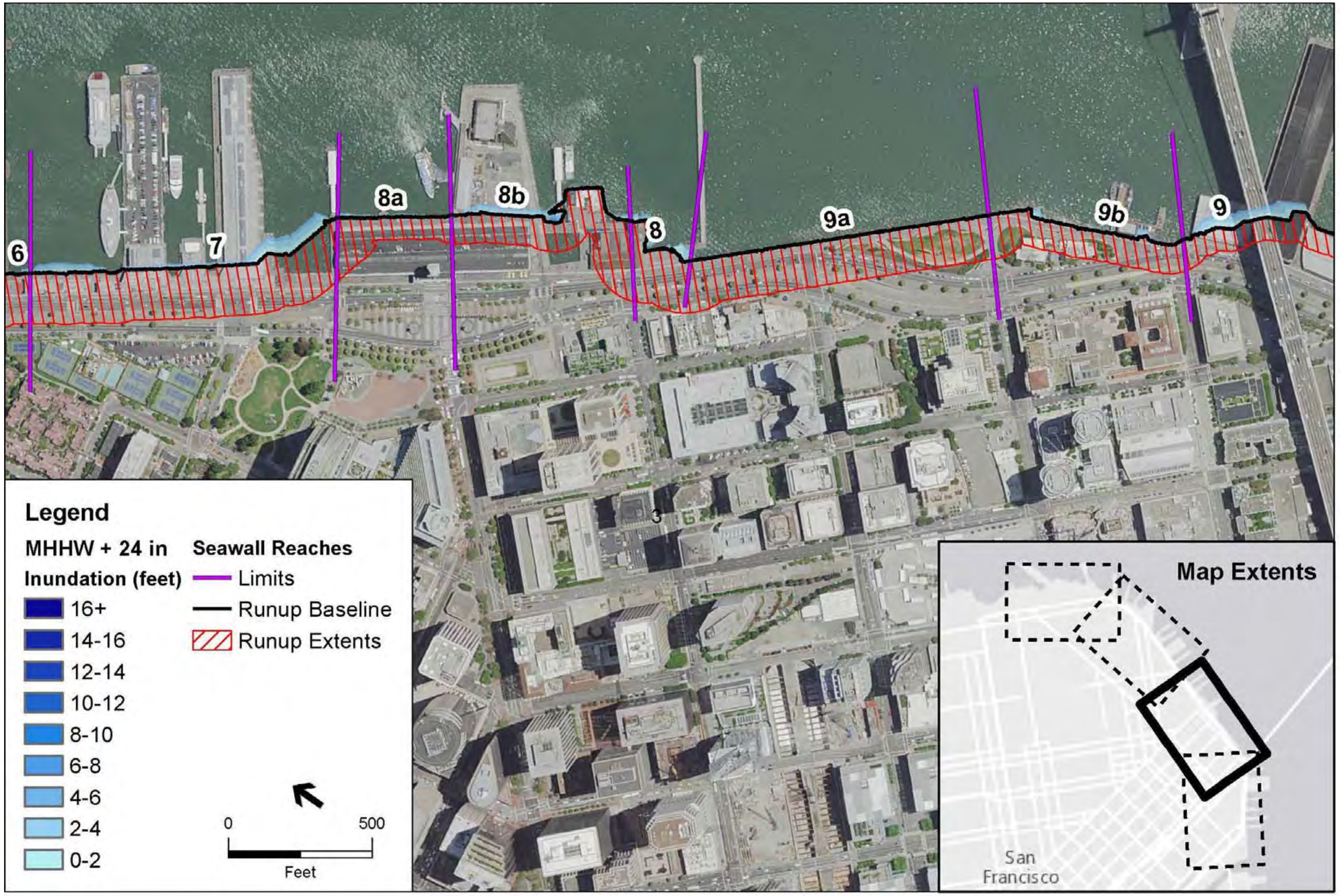
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-7b

5-Year SWL Inundation and Potential Wave Hazard Zone in 2010
Seawall Reaches 2-6

Figure 6-8b: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2010 (Seawall Reaches 2-6)



SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-7c

5-Year SWL Inundation and Potential Wave Hazard Zone in 2010
Seawall Reaches 7-9b

Figure 6-9c: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2010 (Seawall Reaches 7-9b)

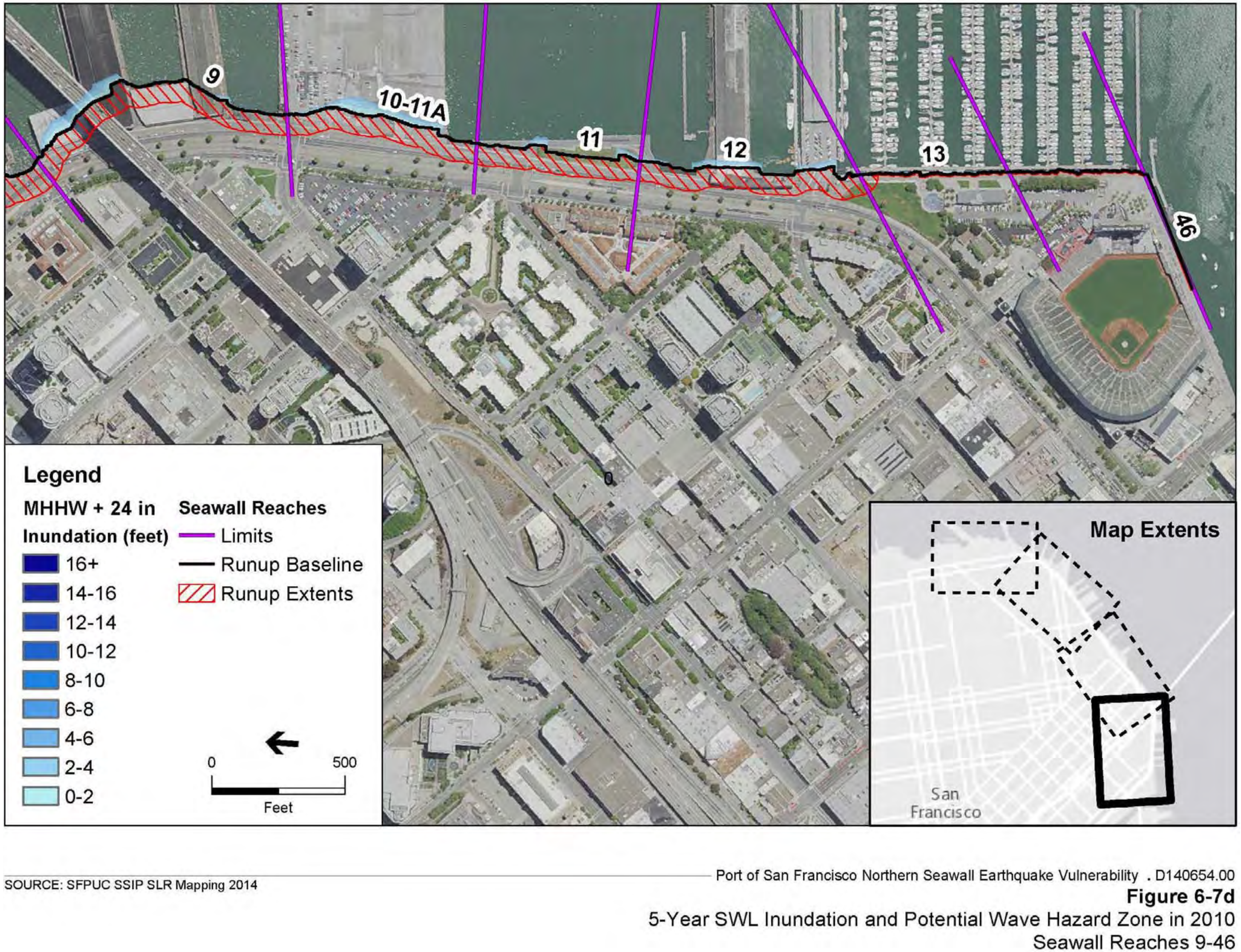
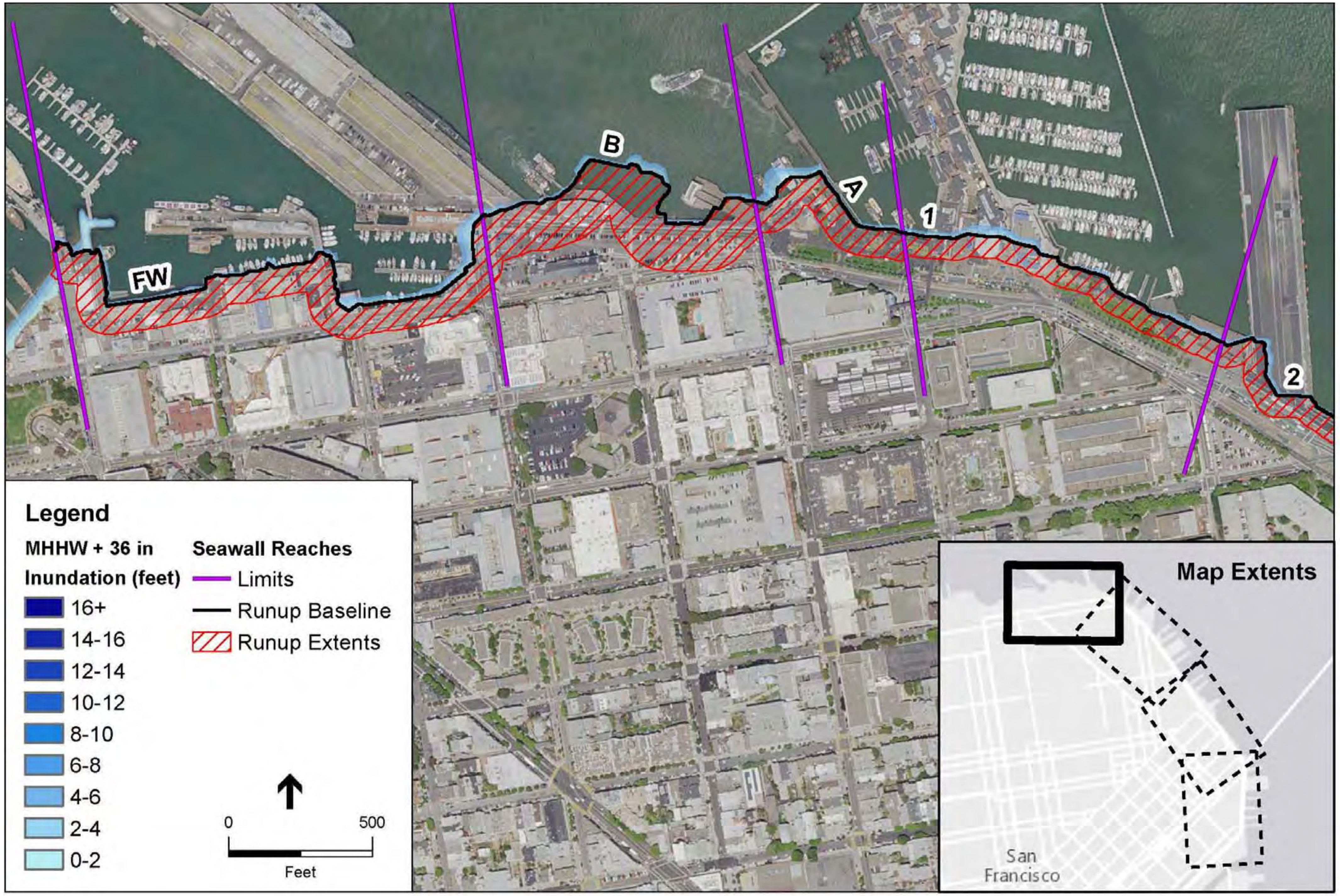


Figure 6-10d: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2010 (Seawall Reaches 9-46)



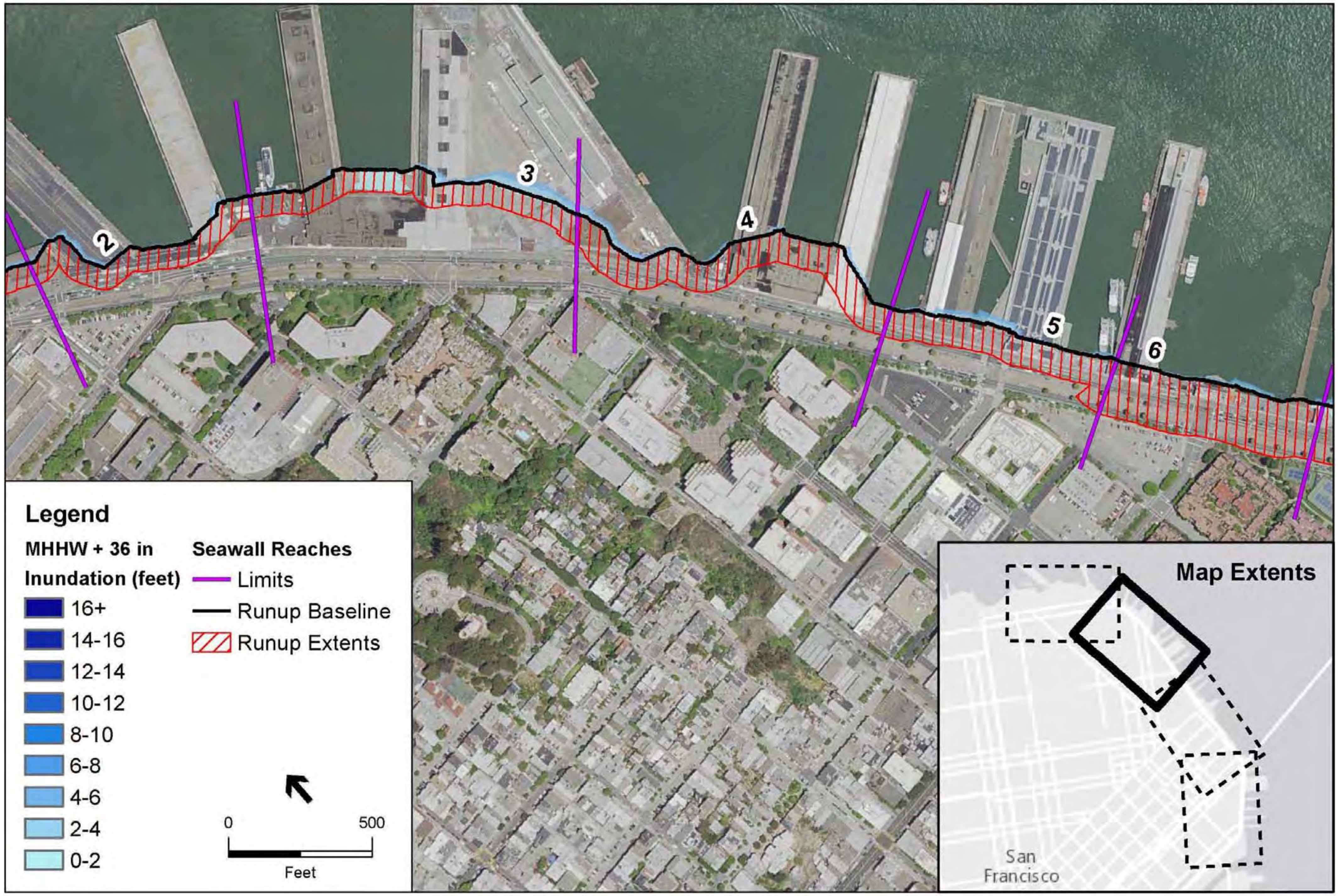
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-8a

5-Year SWL Inundation and Potential Wave Hazard Zone in 2050
Seawall Reaches FW-1

Figure 6-11a: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2050 (Seawall Reaches FW-1)



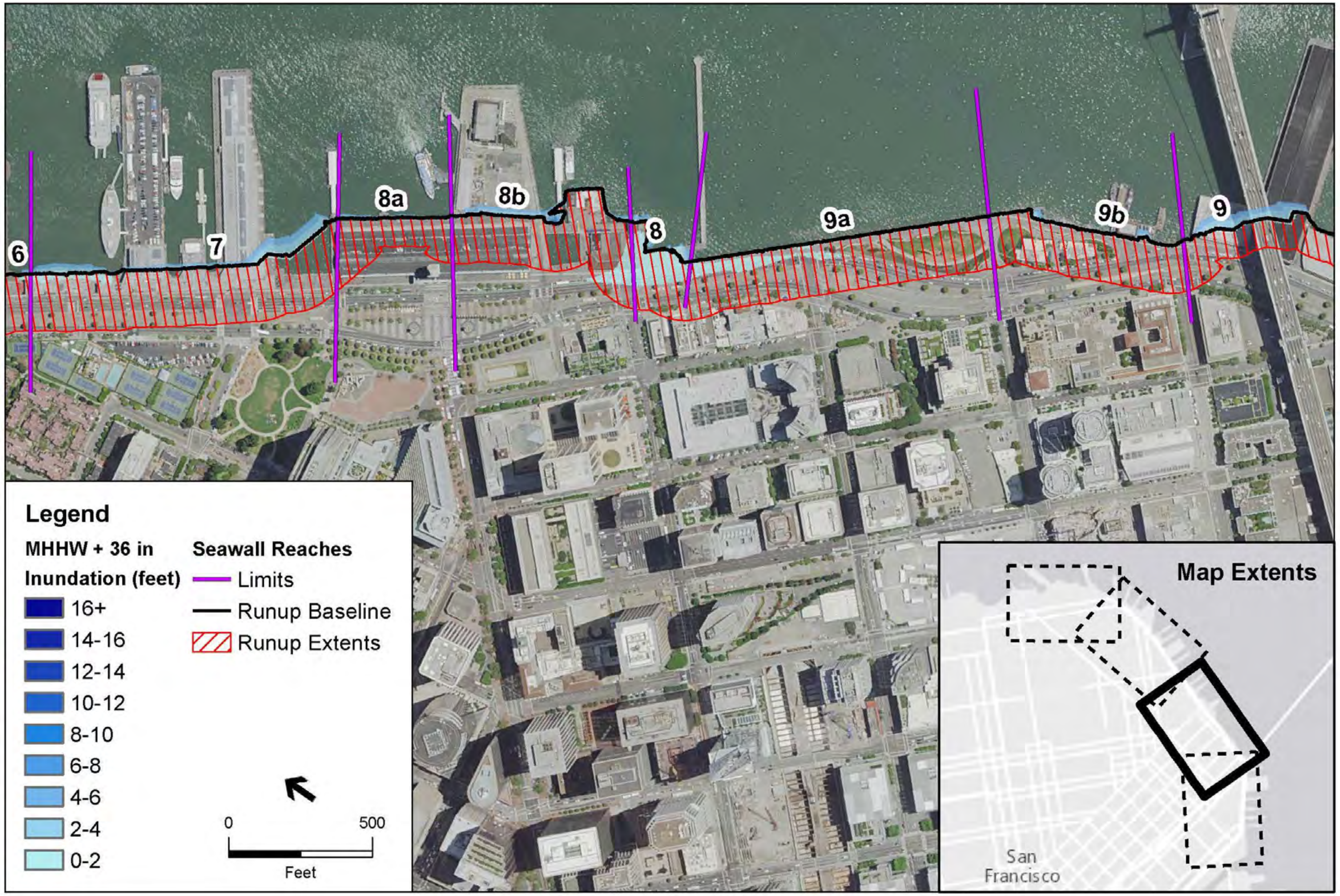
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-8b

5-Year SWL Inundation and Potential Wave Hazard Zone in 2050
Seawall Reaches 2-6

Figure 6-12b: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2050 (Seawall Reaches 2-6)



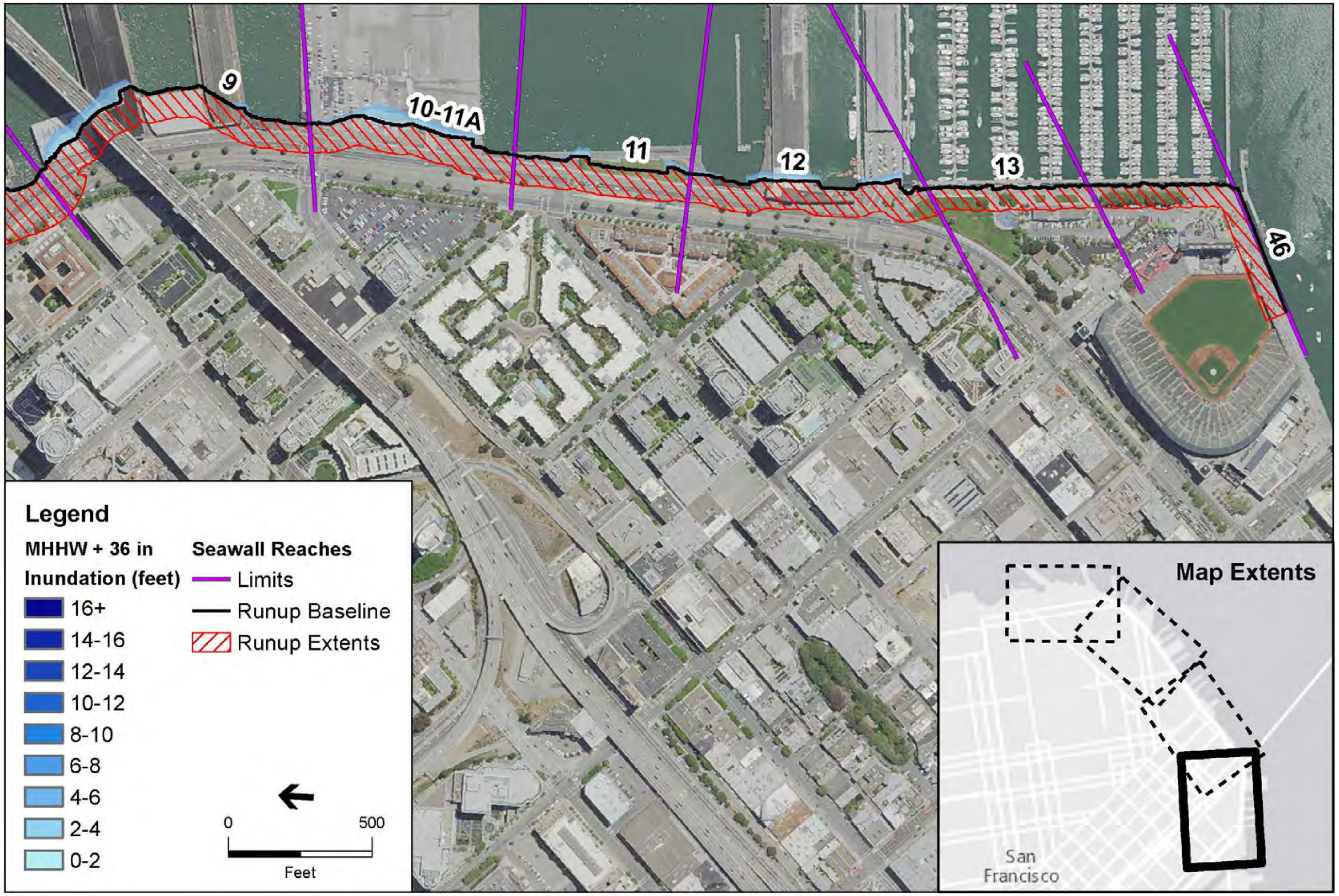
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-8c

5-Year SWL Inundation and Potential Wave Hazard Zone in 2050
Seawall Reaches 7-9b

Figure 6-13c: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2050 (Seawall Reaches 7-9b)



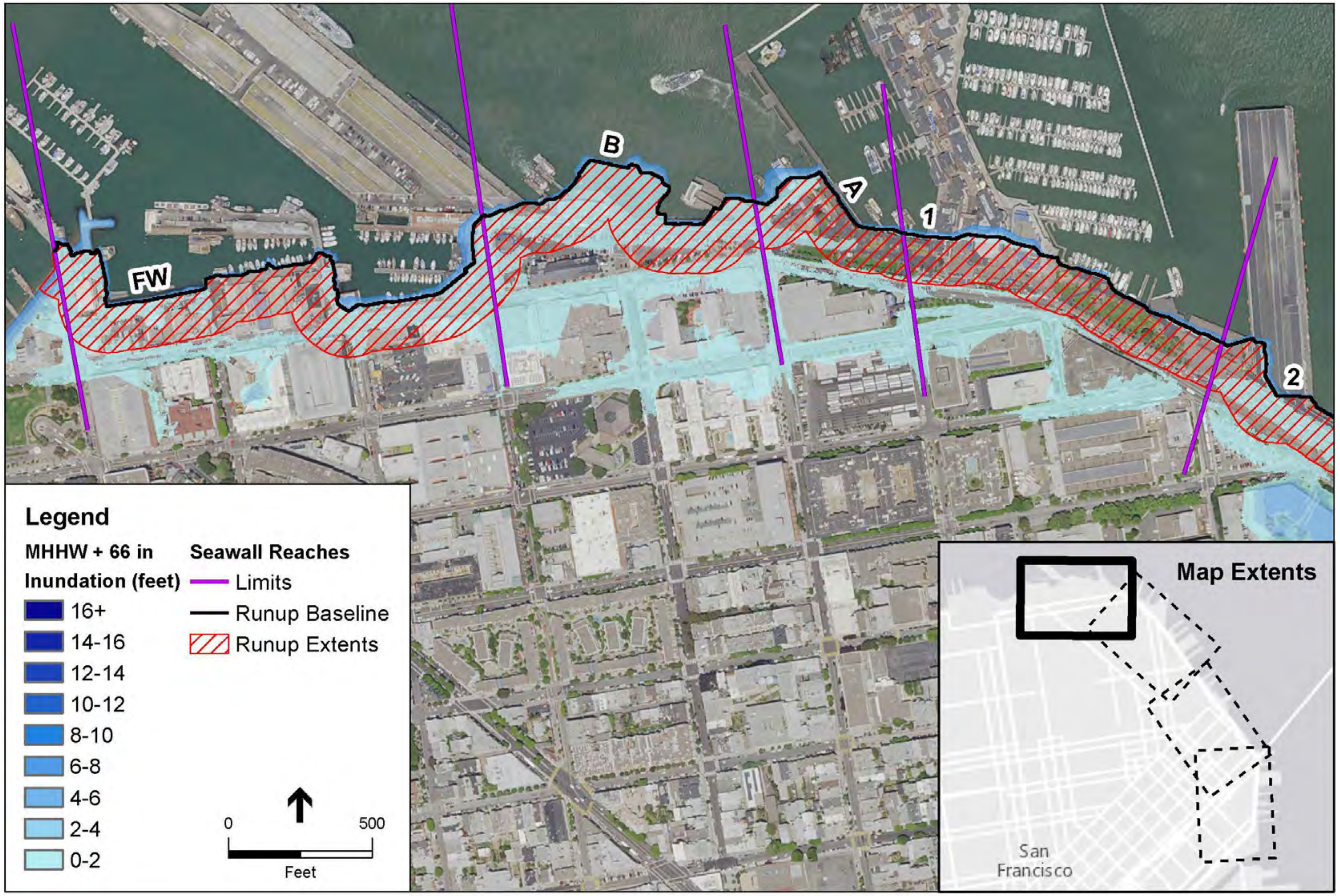
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-8d

5-Year SWL Inundation and Potential Wave Hazard Zone in 2050
Seawall Reaches 9-46

Figure 6-14d: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2050 (Seawall Reaches 9-46)



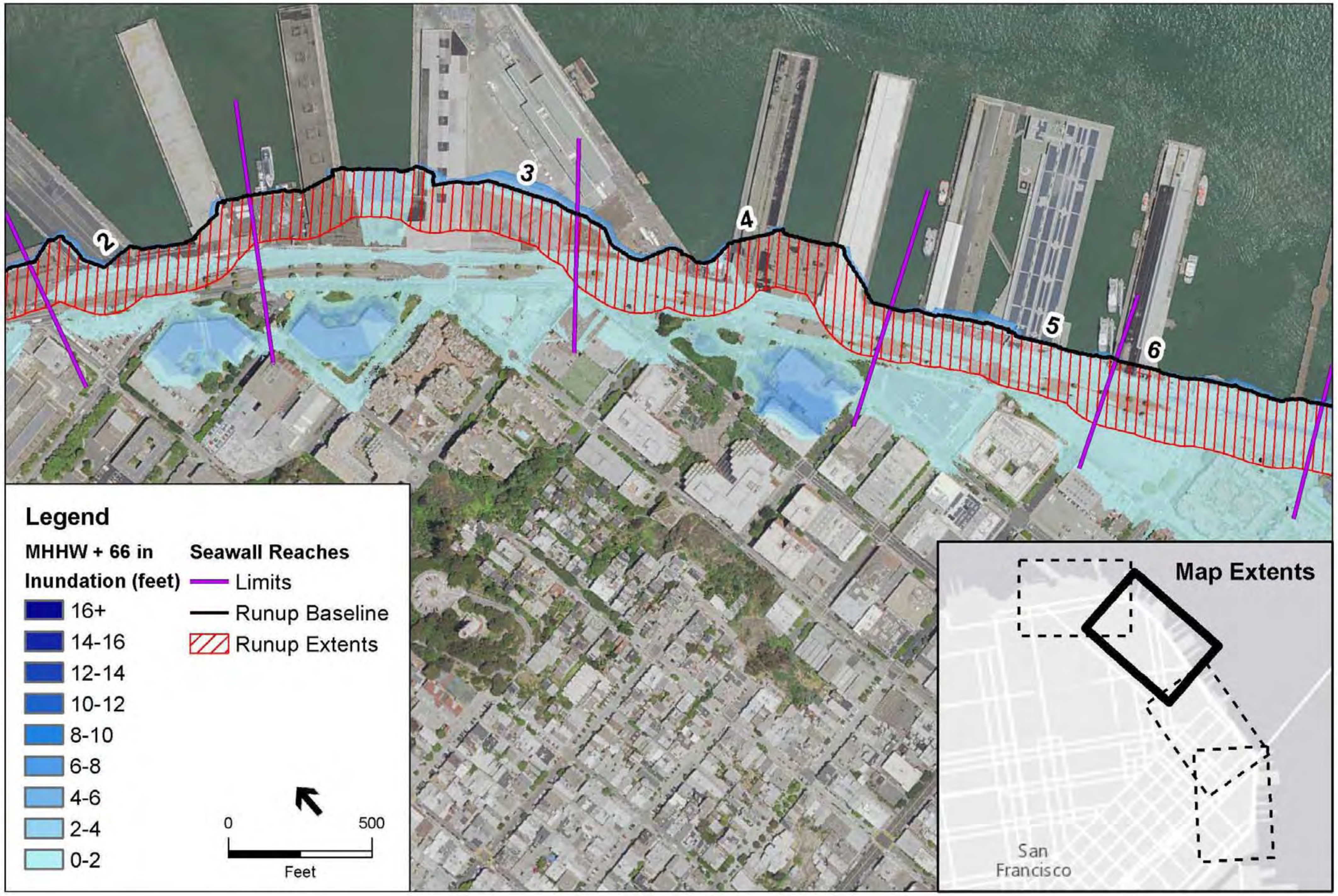
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-9a

5-Year SWL Inundation and Potential Wave Hazard Zone in 2100
Seawall Reaches FW-1

Figure 6-15a: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2100 (Seawall Reaches FW-1)



SOURCE: SFPUC SSIP SLR Mapping 2014

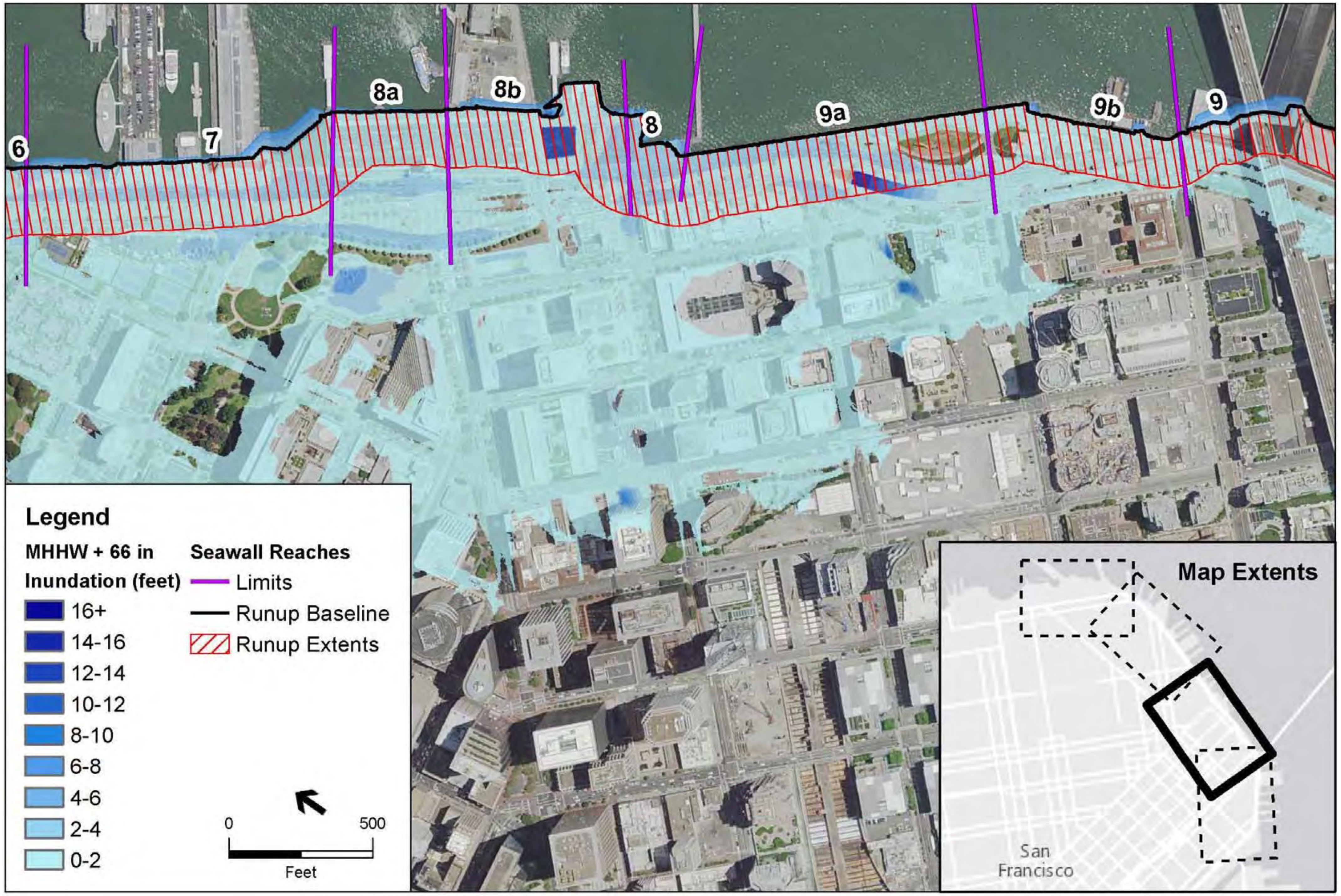
Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-9b

5-Year SWL Inundation and Potential Wave Hazard Zone in 2100

Seawall Reaches 2-6

Figure 6-16b: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2100 (Seawall Reaches 2-6)



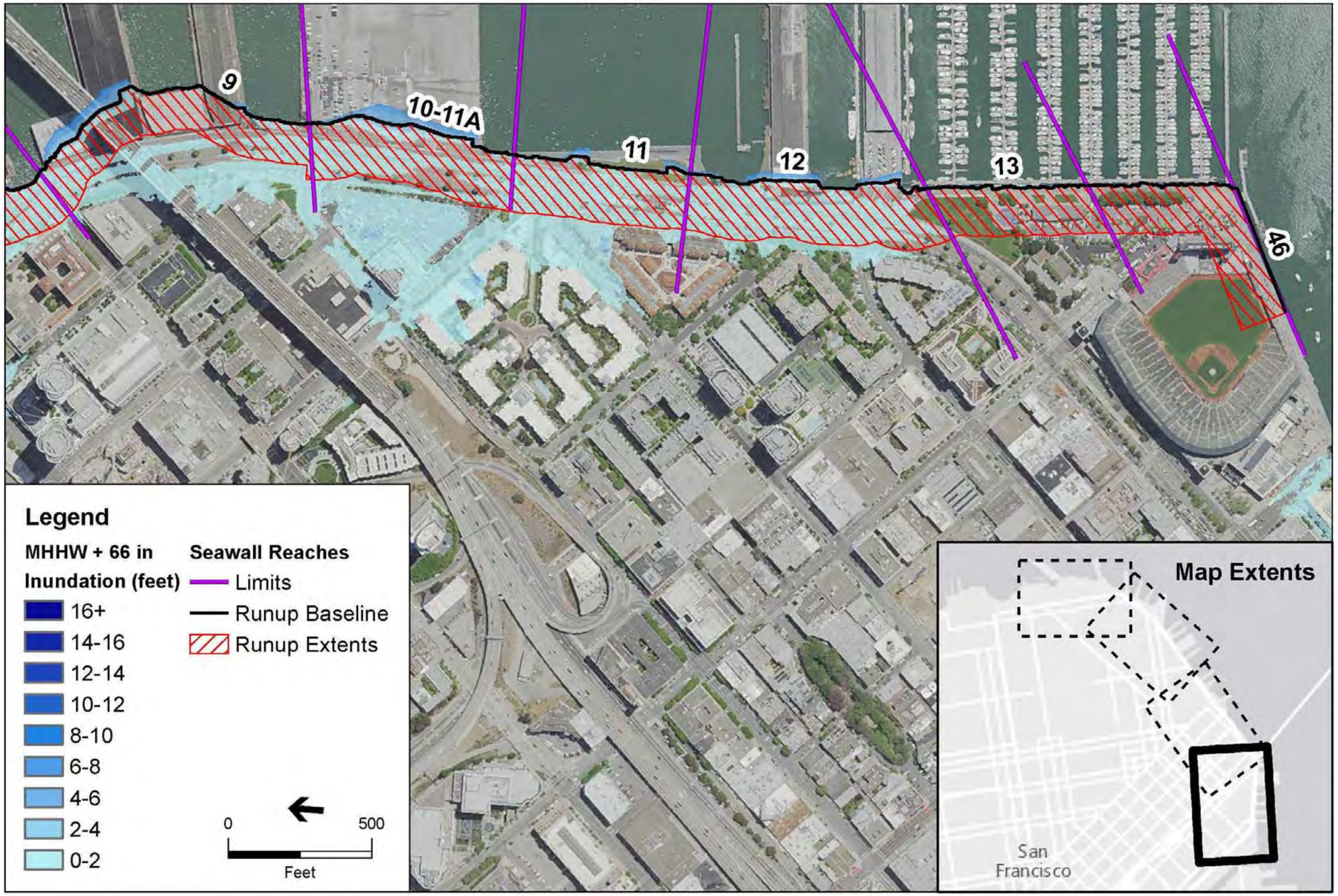
SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-9c

5-Year SWL Inundation and Potential Wave Hazard Zone in 2100
Seawall Reaches 7-9b

Figure 6-17c: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2100 (Seawall Reaches 7-9b)



SOURCE: SFPUC SSIP SLR Mapping 2014

Port of San Francisco Northern Seawall Earthquake Vulnerability . D140654.00

Figure 6-9d

5-Year SWL Inundation and Potential Wave Hazard Zone in 2100

Seawall Reaches 9-46

Figure 6-18d: 5-Year SWL Inundation and Potential Wave Hazard Zone in 2100 (Seawall Reaches 9-46)

6.6. References

The following documents were reviewed during the flooding vulnerability research, data collection and synthesis and analysis phases of this study:

City and County of San Francisco (CCSF), 2014, Guidance for Incorporating Sea Level Rise into Capital Planning in San Francisco: Assessing Vulnerability and Risk to Support Adaptation, Prepared by the City and County of San Francisco Sea Level Rise Committee for the San Francisco Capital Planning Committee, Adopted by the Capital Planning Committee, September 22, 2014.

Cox, J.C., and Machemehl, J., 1986, Overland Bore Propagation due to an Overtopping Wave, *Technical Note, Journal of Waterway, Port, Coastal and Ocean Engineering*, Vol. 112, No. 1, pp. 161-163.

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U.S. Army Corps of Engineers (USACE), 1984a, San Francisco Bay Tidal Stage vs. Frequency Study, San Francisco District U.S. Army Corps of Engineers, October 1984.

U.S. Army Corps of Engineers (USACE), 1984b, *Shore Protection Manual*, 4th ed., 2 Vol., U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, D.C., 1,088 pp.

van der Meer, J.W., 2002, Technical Report on Wave Run-up and Wave Overtopping at Dikes, TAW, Technical Advisory Committee on Flood Defenses, Netherlands.

7. Economics

7.1. General

This memorandum summarizes the approach and methodology adopted in assessing the economic impacts of potential damage to the San Francisco seawall in the event of a major seismic event. The data may be adapted for use with the HAZUS modeling framework in a subsequent phase of work.

7.2. Port Asset Data

We have obtained several important documents from the Port that help to frame the analysis. The first is detailed information on all of the tenants in Port facilities. While far more data than is needed the file contains key metrics such as gross leasable square footage by tenant name, monthly fixed rent amounts, and where appropriate, percentage rent amounts. The data file obtained is for January of 2015. We have extrapolated monthly rents to approximate annual values, but it may be possible to obtain annual totals in a subsequent report. The two exceptions to the data are detailed information for tenants in the Ferry Building and on Pier 39. These properties have master leases: We have an approximate breakdown of the spaces by type of business for the Ferry Building, but have not obtained similar data for Pier 39. Barring the ability to obtain the information from the leasing agent we will perform a physical audit and approximate the square footage distribution. While not as important in assessing the potential business loss to the Port, it will be important to have the data in support of the HAZUS modeling, which does require inputs for business type, operating hours, and employee count as key assumptions for measuring physical damage and social impacts. **Exhibit 1** shows a sample of the information contained in the file. Within the study area there are almost 340 separate tenants, excluding the master leases.

Table 7-1: Exhibit 1: Sample of Tenant Rental Income Data

Seawall Segment	Facility	Company Name	Commencement	Expiration	Revenue Type	GLA	Min Monthly Rent	Jan % Rent	Jan Total Rent
1	1390	Pier 39 Limited Partnership	8/3/1977	12/31/2042	PERC	1,236,852	$ 41,666.67	$ 1,140.00	$ 2,553.39
1	1390	The Bay Institute Aquarium Foundation	3/31/1994	3/31/2019	PERC	-	$ 7,949.55	$ 7,950.00	$ -
1	3130	Embarcadero Triangle Associates	11/14/1972	11/13/2038	CRNT	47,277	$ 49,666.67		$ 3,919.02
2	1330	Alcatraz Enterprises, Inc.	3/1/2014	2/28/2019	CRNT	1,230	$ 1,180.80		$ 10.42
2	1330	Andre-Boudin Bakeries, Inc.	9/1/2014	8/31/2016	CRNT	1,230	$ 1,230.00		$ 20.00
2	1330	Art & Glass, Inc.	11/1/2014	10/31/2017	CRNT	1,230	$ 1,537.50		$ 793.55
2	1330	Bobier, Richard A.	6/1/2014	5/31/2019	CRNT	2,460	$ 2,533.80		$ 240.00
2	1330	E.A.N. Corporation	7/1/2014	6/30/2019	CRNT	10,994	$ 10,994.00		$ 20.00
2	1330	Isis Imports Ltd.	2/1/2014	1/31/2019	CRNT	9,055	$ 8,873.90		$ 111,795.25
2	1330	M.F.M. Seafood, Inc.	11/1/2014	10/31/2019	FFTY	10,417	$ 10,455.10		$ -
2	1330	MetroPCS California, LLC	12/4/2007	12/3/2016	CRNT	-	$ 6,804.00		$ 1,500.00
2	1330	Osprey Seafood of California, Inc.	1/1/2014	12/31/2018	FFTY	3,690	$ 3,477.70		$ 21,923.28
2	1330	P & T Flannery Seafoods, Inc.	9/23/2003		WHFF	-	$ 113.00		$ 4,805.74
2	1330	Pacific Bell Wireless, LLC	12/2/2003	12/1/2012	CRNT	416	$ 4,589.45		$ -
2	1330	Pier 39 Limited Partnership	4/1/2011	3/31/2016	CRNT	4,920	$ 4,454.70		$ 1.00
2	1330	SFCC Public Utilities Commission	5/1/2010	4/30/2015	CRNT	3,600	$ 3,722.40		$ -
2	1330	Seafood Suppliers, Inc	7/1/2010	6/30/2020	FFTY	8,470	$ 5,736.04		$ 68,067.31
2	1330	Simco Restaurants, Inc.	12/1/2013	11/30/2018	CRNT	1,230	$ 1,217.70		$ 50,437.67
2	1330	Priority Parking-CA	5/1/2011	12/31/2012	PRNT	-	$ -	$ 2,037.00	$ 1,233.20
2	1330	San Francisco Pier 33, LLC	6/13/1984	6/12/2014	PERC	4,300	$ 6,596.64	$ 156.00	$ 26,813.00
2	1335	Northern California World Trade Center	1/30/2014	1/29/2016	CRNT	970	$ 3,540.50		$ 2,728.34
2	1335	RGN Corporation	11/1/1997	11/19/2012	PERC	6,772	$ 8,348.90	$ 8,349.00	$ 1,601.76
2	1351	Barulich, Jerome M.	1/10/2013	3/31/2016	CRNT	797	$ 1,284.06		$ 19,821.28
2	1351	California Foundation on the Environment & Econ	6/1/2013	5/31/2018	CRNT	2,902	$ 8,967.18		$ -
2	1351	Herman, Steven H.	7/1/2014	6/30/2019	CRNT	1,553	$ 4,270.75		$ 2,502.00
2	1351	Pier 39 Limited Partnership	4/1/2011	3/31/2016	CRNT	1,896	$ 1,649.52		$ 7,616.25
2	1351	The Bay Institute Aquarium Foundation	5/1/2013	6/30/2015	CRNT	3,386	$ 6,964.90		$ 38,311.32
2	3140	Parking							
2	3150	I&G Waterfront Plaza, Inc	6/28/1974	6/27/2040	CRNT	153,357	$ 62,625.00		$ 16,020.12
2	4015	Hillstone Restaurant Group	12/15/1997		CRNT	-	$ 127.50		$ 3,963.30

The second file obtained contains descriptions of the physical assets and all of the systems contained in each building. This file apparently is used for long term capital planning for it contains the year of construction for the underlying pier structure and the physical buildings contained thereon, as well as the expected life of the asset. Importantly, it shows the gross square footage of each building and the underlying pier structure. Thus, it is possible to assign replacement cost values to each asset based on type of construction, as well as to treat the pier structures as a separate entity as required by HAZUS. **Exhibit 2** shows a sample extracted from this file. You will note the type of building classifications assigned, which will enable summery level estimates of replacement costs in each seawall section based on the type of construction. For instance the sheds on the pier are listed as 'simple'. Those that have been renovated, such as Pier 3, are listed as 'basic'.

Table 7-2: Exhibit 2: Sample Building Information

BldgNo	BuildingName	BuildingID	Year Built	GSF	TypeName	SubSystem	Floor	Life	Sum of 10 Year Total
1245	Pier 24 1/2 -Bulkhead/Shed Building	1000	1936	28,000	SIMPLE	f.2. Electrical Rough-in	1	70	62
1245	Pier 24 1/2 -Bulkhead/Shed Building	1000	1936	28,000	SIMPLE	i.1. Fire Protection Systems	1	40	49
1245	Pier 24 1/2 -Bulkhead/Shed Building	1000	1936	28,000	SIMPLE	i.2. Fire Detection Systems	1	20	61
1245	Pier 24 1/2 -Bulkhead/Shed Building Total								589
1245 Total									4,893
1260	Pier 26	770	1912	156,589	Piers	Substructure	1	75	14,716
1260	Pier 26 Total								14,716
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	b.2. Building Exteriors (Soft)	1	20	494
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	d.1. HVAC - Equipment	1	25	218
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	d.2. HVAC - Controls	1	20	99
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	e.1. HVAC - Distribution Systems	1	50	816
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	f.1. Electrical Equipment	1	30	468
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	f.2. Electrical Rough-in	1	70	284
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	g.1. Plumbing Fixtures	1	25	112
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	g.2. Plumbing Rough-in	1	50	335
1260	Pier 26 - Bulkhead/Shed	956	1912	128,834	SIMPLE	l.2. Interior Finishes	1	15	321
1260	Pier 26 - Bulkhead/Shed Total								3,147
1260 Total									17,862
1265	Pier 26 1/2	771	1927	31,400	Piers	Substructure	1	75	3,243
1265	Pier 26 1/2 Total								3,243
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	a.3. Roofing - Mmbrn,Built-up,Sh	1	25	346
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	f.1. Electrical Equipment	1	30	496
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	f.2. Electrical Rough-in	1	70	867
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	g.1. Plumbing Fixtures	1	25	213
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	g.2. Plumbing Rough-in	1	50	616
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	l.2. Interior Finishes	1	15	453
1265	Pier 26.5 - Bulkhead	998	1927	27,300	BASIC	NEW Fire Detection	1	(blank)	0
1265	Pier 26.5 - Bulkhead Total								2,991
1265 Total									6,234
1270	Pier 27	772	1907	420,690	Piers	Fenders	1	12	426
1270	Pier 27	772	1907	420,690	Piers	Substructure	1	75	494
1270	Pier 27 Total								920
1270	Pier 27 - Office Annex	928	1909	3,552	SMALL	m.1. All Renewal - SMALL 1	1	25	536
1270	Pier 27 - Office Annex Total								536
1270 Total									1,456
1280	Pier 28	773	1912	104,896	Piers	Substructure	1	75	9,451
1280	Pier 28 Total								9,451
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	a.3. Roofing - Mmbrn,Built-up,Sh	1	25	977
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	b.1. Building Exteriors (Hard)	1	50	258
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	b.2. Building Exteriors (Soft)	1	20	148
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	d.1. HVAC - Equipment	1	25	130
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	d.2. HVAC - Controls	1	20	59
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	f.1. Electrical Equipment	1	30	280
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	f.2. Electrical Rough-in	1	70	170
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	g.1. Plumbing Fixtures	1	25	67
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	g.2. Plumbing Rough-in	1	50	201
1280	Pier 28 - Bulkhead/Shed Building	1042	1912	77,088	SIMPLE	l.2. Interior Finishes	1	15	192
1280	Pier 28 - Bulkhead/Shed Building Total								2,482
1280 Total									11,934
1285	Pier 28 1/2	774	1912	4,738	Piers	Substructure	1	75	644
1285	Pier 28 1/2 Total								644
1285	Pier 28 1/2 - Hidive Restaurant	898	1912	1,307	SMALL	m.1. All Renewal - SMALL 1	1	25	197
1285	Pier 28 1/2 - Hidive Restaurant Total								197
1285 Total									841
1290	Pier 29	775	1916	225,374	Piers	Substructure	1	75	15,742
1290	Pier 29 Total								15,742
1290	Pier 29 - *Bulkhead/Shed Building	947	1917	155,279	SIMPLE	a.3. Roofing - Mmbrn,Built-up,Sh	1	25	1,968
1290	Pier 29 - *Bulkhead/Shed Building	947	1917	155,279	SIMPLE	f.1. Electrical Equipment	1	30	564
1290	Pier 29 - *Bulkhead/Shed Building	947	1917	155,279	SIMPLE	l.2. Interior Finishes	1	15	387
1290	Pier 29 - *Bulkhead/Shed Building Total								2,918
1290 Total									18,660
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	a.3. Roofing - Mmbrn,Built-up,Sh	1	25	667
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	b.2. Building Exteriors (Soft)	1	20	202
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	f.1. Electrical Equipment	1	30	191
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	g.1. Plumbing Fixtures	1	25	46
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	g.2. Plumbing Rough-in	1	50	137
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	i.1. Fire Protection Systems	1	40	91
1295	Pier 29 1/2 - Bulkhead Building	1043	1917	52,650	SIMPLE	l.2. Interior Finishes	1	15	131
1295	Pier 29 1/2 - Bulkhead Building Total								1,465
1295 Total									1,465
1310	Pier 31	781	1917	128,785	Piers	Substructure	1	75	11,146
1310	Pier 31 Total								11,146

A third file, critical to accurately listing the assets contained in each of the seawall sections provides maps with a unique four digit identifier for each parcel or pier. Thus, it is possible to compare the location of each tenant to the appropriate map and allocate it to the proper seawall section. **Exhibit 3** shows one of the maps for the area from Pier 9 to Pier 35.

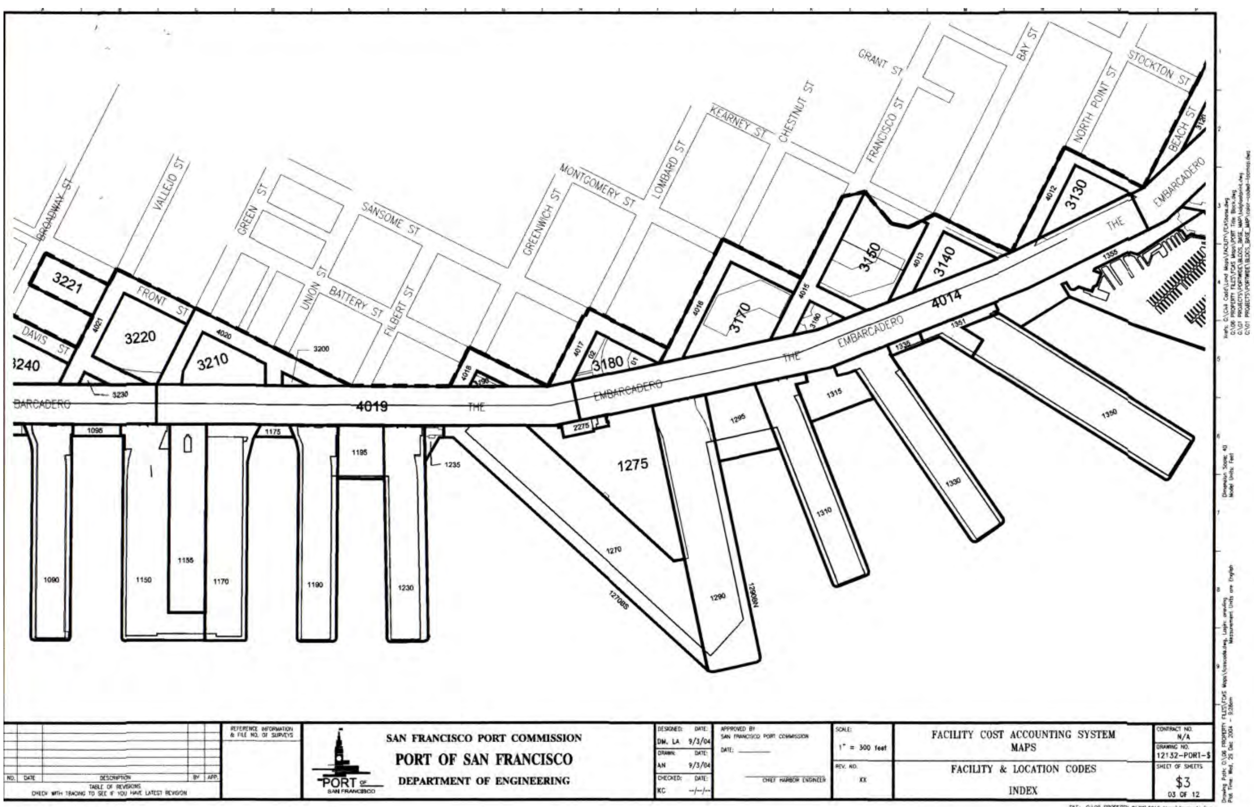

Figure 7-1: Exhibit 3: Example of Numeric Designations for Port Owned Properties

A fourth file provides a cross check on the above because it provides the identity of each tenant and the detailed characteristics of the lease, i.e. total leased area, size of the building footprint, associated deck space, etc. This is also important to distinguishing between, for instance, a tenant who may occupy a space that is mostly located on a pier, but may also have an occupied space on the marginal seawall. **Exhibit 4** shows a portion of one of the maps that displays information for the Waterfront Restaurant, adjacent Exploratorium, and related parking.

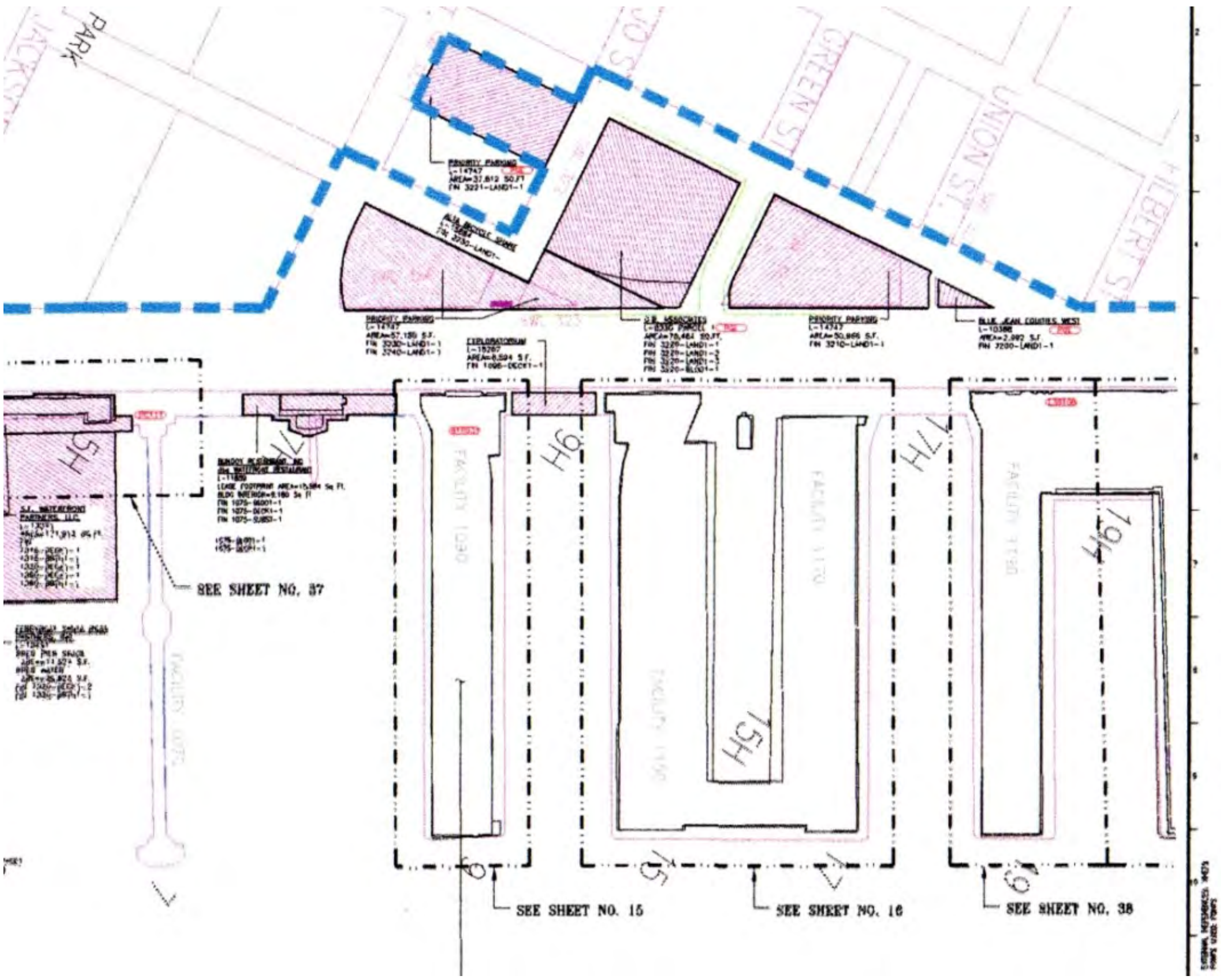

Figure 7-2: Exhibit 4: Detailed Lease Information on Port Properties

7.3. Assessment Delineation by Seawall Section

The Port utilizes a unique four digit numerical identifier. Assets whose numbers begin with a '1' are categorized as piers. Those whose numbers beginning with a "2" are tenants who are on the seawall fronting onto the Embarcadero. Those beginning with a "3" are assets on the opposite side of the Embarcadero, and are either buildings or parking lots. Assets starting with a "4" are tenants/buildings fronting on one of the adjacent streets, but not on the Embarcadero. Numbers that start with the number "5" relate to water assets such as the marina areas adjacent to Pier 39 and South Beach Harbor. There are additional classifications, but none of those assets are within the study area. Thus, for example Pier 3 carries a unique identifier of 1030. Part of the Pier 23 restaurant is identified as 1235 (pier 23 and a half). The Fog City Diner bears the identifier of 3190, also shared with the western part of the Levi office complex. The Raintree Forest Café bears the identifier of 4007.

Because the various structures and tenants do not align completely with the breaks in the seawall we have elected to modify the section identification somewhat to facilitate the economic analysis. **Exhibit 5** shows a map that defines the sections and shows the boundaries we will use. We believe any impact on the analysis will be minimal unless we are told by you that the failure in a seawall will occur precisely at the juncture of two different sections. Thus, as shown in the map the entirety of Pier 27 is located in Section 3, and the entirety of the Ferry building has been allocated to Section 8A. All of Pier 9 has been allocated to Section 6, and both piers 38 and 40 belong in Section 12. Similar small changes have been made in our definition of the other sections, and should be compared to your mapping to see if any changes are necessary.

Figure 7-3: Exhibit 5: Defined Seawall Sections for Economic Analysis Purposes

Exhibit 6 shows a 'roll up' of revenues by seawall section, and by location (pier, seawall, or on the Embarcadero. These should be taken as examples only for some of the base data still requires clarification with the Port, and we are still trying to allocate space on the seawall section rather than by the more generic four digit identifier.

Table 7-3: Exhibit 6: Sample 'Roll Up' Revenue By Seawall Section and Orientation

SEAWALL SEGMENT #	Description	GLA	Fixed Rent	Monthly Rent (1)	Total Rent
FW	**Hyde Stree to Jones**				
	Pier	264,947	$ 2,701,355	$ 1,058,436	$ 3,759,791
	Marginal Wharf				
	Embarcadero	131,647	$ 1,500,035	$ 987,384	$ 2,487,419
	Marine Leases				
B	**Jones to Powell**				
	Pier	35,772	$ 594,011	$ 306,924	$ 900,935
	Marginal Wharf				
	Embarcadero	122,647	$ 2,603,439	$ 1,008,096	$ 3,611,535
A	**Powell to Stockton**				
	Pier	60,300	$ 347,536	$ 347,532	$ 695,068
	Marginal Wharf				
	Embarcadero				
1	**Stockton to Kearny**				
	Pier	1,236,852	$ 595,395	$ 109,080	$ 704,475
	Marginal Wharf				
	Embarcadero	47,277	$ 49,667		$ 49,667
	Marina Leases	3,620	$ 6,792	$ -	$ 6,792
2	**Kearny to Pier 31 and a half**				
	Pier	81,518	$ 1,302,510	$ 126,504	$ 1,429,014
	Marginal Wharf				
	Embarcadero	153,357	$ 753,030	$ -	$ 753,030
3	**Pier 31 and a Half inclusive of Pier 27**				
	Pier	357,313	$ 579,122	$ 1,172,628	$ 1,751,750
	Marginal Wharf				
	Embarcadero	47,836	$ 1,164,148	$ -	$ 1,164,148
4	**Pier 23 and a half to include Pier 19**				
	Pier	12,624	$ 411,268	$ 400,836	$ 812,104
	Marginal Wharf				
	Embarcadero	13098	$ 433,044	$ 221,520	$ 654,564
5	**Pier 17 inclusive of Pier 9**				
	Pier	606,850	$ 1,167,887	$ -	$ 1,167,887
	Marginal Wharf				
	Embarcadero	223,788	$ 3,104,952	$ -	$ 3,104,952
6	**Pier 7 and a half and Pier 7**				
	Pier	206,574	$ 2,803,221	$ 201,756	$ 3,004,977
	Marginal Wharf				
	Embarcadero				
7	**Pier 3 and Pier 1 to Ferry Plaza edge**				
	Pier	415,941	$ 1,674,792	$ 1,146,492	$ 2,821,284
	Marginal Wharf				
	Embarcadero				

Exhibit 7 shows an example of how the data is being amalgamated for purposes of the HAZUS analysis. Rather than presenting financial data it concentrates on gaining an understanding of the physical properties of the assets. Both gross and gross leasable square footages are shown for the piers and the buildings thereon, again showing tenant details. For the businesses housed in the pier sheds we have used professional judgement and increased the leasable square footages by 20 percent to reflect circulation, etc., space perhaps not included in lease rates. This will allow categorizing the businesses by type (office, retail, food and beverage, storage, etc.) which are critical inputs to HAZUS. The example shown is for Section 2 which has a single shed building on Pier 33, a building on the bulkhead at Pier 33 and a half, a restaurant, and an office building. Most of the shed building is used for storage and Priority Parking leases the interior central core. Once these distinctions are made for all the buildings it will be possible to assign estimates for operating hours, number of employees, and in the case of restaurants estimates of peak and off peak visitation.

Table 7-4: Exhibit 7: Sample Classification of Properties for HAZUS

Seawall Segment	Facility	Company Name	GLA	Gross Square Footage					Usage (Sq Ft)				
				Pier	Parking	Simple	Basic	Small	Office	Public	Retail	Storage	F & B
2	1330	Pier 33		136,810									
2	1330	Shed				112,840							
2	1330	Alcatraz Enterprises, Inc.	1,230									1,476	
2	1330	Andre-Boudin Bakeries, Inc.	1,230									1,476	
2	1330	Art & Glass, Inc.	1,230									1,476	
2	1330	Bobier, Richard A.	2,460									2,952	
2	1330	E.A.N. Corporation	10,994									13,193	
2	1330	Isis Imports Ltd.	9,055									10,866	
2	1330	M.F.M. Seafood, Inc.	10,417									12,500	
2	1330	MetroPCS California, LLC	-									-	
2	1330	Osprey Seafood of California, Inc.	3,690									4,428	
2	1330	P & T Flannery Seafoods, Inc.	-									-	
2	1330	Pacific Bell Wireless, LLC	416									499	
2	1330	Pier 39 Limited Partnership	4,920									5,904	
2	1330	SFCC Public Utilities Commission	3,600									4,320	
2	1330	Seafood Suppliers, Inc	8,470									10,164	
2	1330	Simco Restaurants, Inc.	1,230									1,476	
2	1330	Priority Parking-CA	-									-	
2	1330	San Francisco Pier 33, LLC	4,300									5,160	
2	1335	Northern California World Trade Center	970				16,452		1,164				
2	1335	RGN Corporation	6,772						8,126				
2	1351	Barulich, Jerome M.	797						956				
2	1351	California Foundation on the Environment & Economy	2,902						3,482				
2	1351	Herman, Steven H.	1,553						1,864				
2	1351	Pier 39 Limited Partnership	1,896						2,275				
2	1351	The Bay Institute Aquarium Foundation	3,386						4,063				
	3140	Parking											
2	3150	I&G Waterfront Plaza, Inc	153,357				132,316		153,357				
2	4015	Hillstone Restaurant Group					12,622						12,622

Appendix A

Report Comment Log

Port of San Francisco Seawall Earthquake Vulnerability Study - Peer Review Comments

Date Modified: 11-30-15
Peer Review Status - In Progress

GTC Responses: thru 1/19/2016
GHD Responses: thru 1/21/2016
New review comments added 12/15/2015

General Notes and Exclusions on Peer Review:

Scope Of Services	Document	Review Item				
		Item No.	Peer Reviewer	Comments	GHD/GTC Response	Path Forward
	Phase 1 Draft Report 30 March 2015	1.1	Langan Treadwell Rollo	There are two figures titled Figure 3-4. The figure presenting the elevation of top of bedrock on page 64 should be Figure 3-5?	We will update in Final Phase 1 Report.	
	Phase 1 Draft Report 30 March 2015	1.2	Langan Treadwell Rollo	Table 3-3. Table 2 - Historical Geotechnical Data within the Seawall Zone of Influence Earthquake Vulnerability Study of the Northern Waterfront Seawall: It is difficult to correlate the borings listed to the borings shown on the contour Figures 3-1 through 3-4 and the sections.	The borings were also input into GIS where the data for each boring location can be viewed.	
	Phase 2 Draft Report 16 July 2015	1.3	Langan Treadwell Rollo	Spot check of sections and reported boring logs show some large discrepancies. For example: 1) Section B - thickness of upper layered sediments appear to be thicker by approx. 50 feet when compared with boring TR-1 (T&R). 2) Section 1 - thickness of Silt and Clay Marine Deposit appears to be thicker by approx. 10 feet when compared with boring B-5 (HMLA). The thickness of the Upper Layered Sediments appears to be thinner by approx. 17 feet when compared with boring B-5 (HMLA). 3) Section 3 - thickness of the Young Bay Mud appears to be thicker by approx. 11 feet when compared with boring B-3L (D&M). The Lower Layered Sediments and Old Bay Clays appear to be thinner by approx. 10 and 15 feet, respectively, when compared with boring B-3L (D&M). 4) Section 8b - thickness of Sand Fill beneath seawall appears to be thinner by approx. 20 feet when compared with boring PIB-8 (PBTB). 5) Section 9a - thickness of the Young Bay Mud appears to be thinner by approx. 12 feet when compared with boring B-20 (WCC). 6) Section 12 - thickness of the Lower Layered Sediments appear to be thicker by approx. 10 feet when compared with CPT-2 (GTC). 7) Section 46 - thickness of the Upper Layered Sediments appear to be thinner by approx. 17 feet when compared with Boring B-6 (HLA).	First, some discrepancies should be expected since we are idealizing the geologic profile based on data from several boring logs. Idealizing the profile is more appropriate since we are projecting data onto a single line and large changes in unit thicknesses over short horizontal distances is unlikely given the geologic history of the area. We, however, will review the cross sections versus the contour maps and correct any significant discrepancies since some data was added to the GIS database (mostly bedrock elevations) just prior to submitting the draft Phase 2 report. 1) Should be labeled TR-2. Will correct label. 2) B-5 (HMLA): Silt and Clay, boring log 8.5', cross section 17'; ULS, boring log 25', cross section 18'. Due to averaging thicknesses and projecting onto cross section line. 3) Cross section will be edited accordingly. 4) PIB-8 (PBTB): Sand fill, boring log 18', cross section 14'. 5) B-20 (WCC): Young Bay Mud, boring log 81', cross section 71'. Bay mud will be thickened on cross section. 6) Minor difference in interpretation of CPT data and would not affect results. 7) Boring B-6 (HLA): Upper Layered Sediments, boring log 16', cross section 16'.	

Scope Of Services	Document	Review Item				
		Item No.	Peer Reviewer	Comments	GHD/GTC Response	Path Forward
	Phase 2 Draft Report 16 July 2015	1.4	Langan Treadwell Rollo	Spot check of Figures 3-1 through 3-4 and sections show some large discrepancies. For example: 1) Sections B and 1 - the sand layers within the Marine Deposits appear to be included in the Young Bay Mud thickness shown on Figure 3-2. 2) Section B - the top of bedrock elevations do not appear to be consistent with Figure 3-4 on page 64. 3) Section 1 - the elevations of the top and bottom of the Marine Deposits/Young Bay Mud do not appear to be consistent with the elevations shown on Figures 3-3 and 3-4. 4) Section 3 - the elevations of the bottom of bay mud shown on the section appear to be shallower by approx. 30 feet when compared to Figure 3-4 on page 63. 5) Section 7 - thickness of Artificial Fill shown on the section appears to be thinner by approx. 10 to 25 feet when compared with Figure 3-1. 6) Section 9a - thickness of Young Bay Mud beneath the seawall shown on the section appears to be thinner by approx. 30 feet when compared to Figure 3-2. The elevation of the top of Young Bay Mud beneath the seawall appears to be shallower by approx. 20 feet when compared with Figure 3-3. The elevation of the top of bedrock appears to be shallower by approx. 40 feet when compared to Figure 3-4 on page 64. 7) Section 12 - thickness of the Young Bay Mud beneath the seawall shown on the section appears to be thinner by approx. 15 to 20 feet when compared with Figure 3-2. The elevation of the top of Young Bay Mud shown on the section appears to be deeper by approx. 10 to 15 feet when compared to Figure 3-3. The elevation of the top of bedrock shown on the section appears to be deeper by approx. 80 to 100 feet when compared with Figure 3-4 on page 64. 8) Section 46 - thickness of the Young Bay Mud beneath the seawall shown on the section appears to be thinner by approx. 20 feet when compared with Figure 3-2. The elevation of the top of bedrock shown on the section appears to be deeper by approx. 10 to 15 feet when compared with Figure 3-4 on page 64.	1) True. Marine Deposits were lumped into the "Young Bay Mud" as a geologically young bay deposit for purposes of contouring unit thicknesses. 2) Bedrock contact in cross section will be revised per updated contours. 3) Contacts at bottom of marine deposits will be made consistent with cross section. Also corrected datum for some borings which will revise contours. 4) Sand unit in young bay mud previously grouped with underlying upper layered sediments in cross section. The cross section will be revised to be consistent with contours. 5) The contours reflect some borings where rock dike was encountered to up to 25 feet deeper than the sandy backfill soils. 6) YBM: thickness, section 71' (will be thickened to 81' as noted above), contours 82'; top of YBM, section El. -16', contours El. -18'; Bedrock, section 125', contours 130'. 7) The section uses specific data from borings drilled directly through the rock dike. Agreed that the contours need to be revised in this area. 8) All of the bay mud is assumed to be removed at the seawall location. This is not necessarily captured in contours since it is over a large area and uses data from boring data points. Top of bedrock section and contours generally within 5 to 10 feet and considered within reasonably close agreement.	
	Phase 2 Draft Report 16 July 2015	1.5	Langan Treadwell Rollo	It would be beneficial to shown the section locations on the contour Figures 3-1 through 3-5.	We will add the section locations in the Final Phase 2 Report.	
		1.6				
		1.7				
2. Geotechnical Sections	Phase 2 Draft Report 16 July 2015	2.1	Langan Treadwell Rollo	We have started reviewing the selected locations, number, and appropriateness of the subsurface profiles		
		2.2				
		2.3				

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Scope Of Services	Document	Review Item				
		Item No.	Peer Reviewer	Comments	GHD/GTC Response	Path Forward
3. Seismic Ground Motions	Phase 2 Draft Report 16 July 2015	3.1	Langan Treadwell Rollo	Phase 2 Report - Section 1.2 - Considering that the performance of the port facilities during the Kobe earthquake was in the very near-field region, how does this performance correlate to the Port of SF seawall?	The trends in horizontal ground displacement (PGDh) versus distance from the waterfront developed from data collected after the 1995 Kobe EQ were normalized as (PGDh)x=L/(PGDh)x=0, where x is the distance from the waterfront. The absolute displacements from Kobe were not applied directly for the PoSF seawall application. Instead, (PGDh)x=0, defined as (PGDh)max, was computed at each section along the Embarcadero then the reduction in PGDh with x was estimated using the normailized trend from Kobe. Note that several additional data points have been added for the sake of comparison, most appropriately, trends from the 1989 Loma Prieta EQ at two sites in SF that support the proposed relationship. Refer to Figure 3-56 in the report.	
	Phase 2 Draft Report 16 July 2015	3.2	Langan Treadwell Rollo	Phase 2 Report - Section 3.5.1 - What was the basis of using USGS maps for rock spectral values instead of performing site-specific determination? Are the values geometric mean or max. direction?	The 2008 USGS PSHA (with comparion to the 2014 edition) was used as the basis for defining the seismic ground motions on weak bedrock in this screening evaluation for the sake of efficiency. The level of refinement in the ground motion estimates that might accrue from the use of a site-specific PSHA evaluation using standard of practice tools (EZFRISK, HAZ38, HAZ42) was considered small relative to other uncertainties and approximations applied in this broad investigation. The site-specific models would be recommended for site-specific investigations leading to design; however, that is not the intent of this investigation. The GMPE's used in the 2008 edition of the USGS PSHA provide geomean ground motions.	
	Phase 2 Draft Report 16 July 2015	3.3	Langan Treadwell Rollo	Phase 2 Report - Section 3.5.1 Dynamic Soil Response and Site Effects - The section details of the approach however, no results have been presented to substantiate the amplification ratios presented on Figures 3-15 thru 3-18. Please provide the results of the various analyses for our review.	Provided response in letter dated 12/1/2015.	
	Phase 2 Draft Report 16 July 2015	3.4	Langan Treadwell Rollo	Please provide further information regarding the Loma Prieta results presented on Figures 3-15 through 3-18. Were these computed or recorded?	The SAR values and ranges for the 1989 Loma Prieta EQ were computed using recorded ground surface records at ten sites located along the margins of the SF Bay and local rock outcrop motions. The ten sites can be generally characterized as deep, cohesive soil sites with varying thicknesses of both young bay mud and old bay clay.	
		3.5				
4. Seismic Induced Ground Deformations	Phase 2 Draft Report 16 July 2015	4.1	Langan Treadwell Rollo	Table 3-3 presents the parameters used for the various materials considered in the slope stability calculations. Considering the large study area, significant variations in thicknesses and stress histories of the various near surface strata, we would expect similar variations in the strength and engineering characteristics of these materials. Please provide the basis for the assigned material characteristics.	We did not create summary plots. As a first step, we evaluated the strength and engineering characteristics of the soils from existing borings and lab test results along each cross section line and assigned average geotechnical parameters for each soil unit. This is represented by the range in parameters for some of the deeper soil units in Table 3-3. Rather than assigning local variations in strengths for the artificial fill, rock dike and young bay mud units, we assigned average parameters across the entire waterfront for these units since the blow counts, strengths, etc. are often wide-ranging even within borings in close proximity. For a screening-level study, using average parameters seemed more appropriate as to not influence the analysis outcome based on limited subsurface information.	Geotechnical parameters used at each slope stability section will be provided to the peer review team.

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Scope Of Services	Document	Review Item				
		Item No.	Peer Reviewer	Comments	GHD/GTC Response	Path Forward
	Phase 2 Draft Report, 16 July 2015 & DRAFT Reply to External Peer Review Comments, 23 November 2015.	4.2	Langan Treadwell Rollo	Section 3.7.1 - Item 4: The Phase 2 report indicates that multiple methods were used to develop displacement estimates with yield acceleration and Average Return Period. However, the DRAFT Reply to External Peer Review Questions, appears to state that the displacement estimated presented in the Phase 2 report are based on the estimated slope displacements from SLAMMER. Is this the case?	A two-step procedure was used before selecting the preferred method of evaluation. First, four simplified, Newmark methods were applied using a constant value of yield acceleration (Ky). A comparison was made between the software SLAMMER and the other procedures. The SLAMMER results provided trends in PGDh that were comparable to the average of the four methods. SLAMMER provides the added benefit of allowing for the use of a displacement-varying value of Ky, which has been determined to be an important aspect of Newmark-based slope displacement analysis. The three other procedures do not readily allow for a displacement-based value of Ky. After making this comparison, and performing sensitivity analyses using SLAMMER for ground motions considered reasonably representative for both the 1906 San Francisco EQ and the 1989 Loma Priate EQ, SLAMMER was used for all subsequent analyses at each section. The plots in the Phase 2 report are based entirely on analyses made using SLAMMER with displacement-dependent values of Ky.	
	Phase 2 Draft Report 16 July 2015; & DRAFT Reply to External Peer Review Comments, 23 November 2015.	4.3	Langan Treadwell Rollo	Section 3.7.1 - There appears to be a discrepancy in the estimated slope displacement for Section 8B between the DRAFT Reply to Peer Review Questions, and those shown on Figure 3-55 of the Phase 2 Report. On figure 3-55 of the Phase 2 Report the maximum estimated displacement is approximately 20-inches, and the maximum horizontal displacement shown on the figures in the DRAFT Reply to External Peer Review Questions is between 40 and 50-inches. Please clarify how the estimated displacements were determined.	This difference is due entirely to the influence of the displacement-dependent value of Ky. Figure 3-55 was developed using SLAMMER with a displacement-dependent Ky, as deemed to be the most appropriate approach for this project. The comparison provided in the "DRAFT Reply to Peer Reviewer Questions" was developed using the four simplified methods of displacement analysis and, necessarily, a constant value of Ky (0.055). The constant value of Ky was used in response to the request by peer reviewers to provide a direct comparison of methods.	
	Phase 2 Draft Report 16 July 2015	4.4	Langan Treadwell Rollo	Section 3.7.1 - The text indicates that subjective scaling factors were applied to estimated ground displacements for Zone C2 for annual return periods greater than 975 years due to "saturation and duration effects". How were these factors determined?	The scaling factors were applied due to considerations of ground motion duration with increasing average return period, or seismic hazard level. Note that PSHA deaggregation yields similar magnitude and distance for all ARP's greater than 475 years. It is likely that in addition to increasing ground motion amplitude at longer ARP (modeled with σ in the GMPE's), the duration of the motions should also be increased. The scaling factors were based in part on mean versus mean +1σ values in relevant ground motion intensity measures such as duration (e.g., Silva and Abrahamson) and Arias Intensity.	

Scope Of Services	Document	Review Item				
		Item No.	Peer Reviewer	Comments	GHD/GTC Response	Path Forward
	Phase 2 Draft Report 16 July 2015	4.5	Langan Treadwell Rollo	Section 3.7.1 - Please provide the list of earthquake records used in the Newmark type analyses such as SLAMMER, etc. to compute slope displacements. Were these ground motions scaled or spectrally matched to the levels of shaking considered in the seismic hazard analysis (Section 3.5)?	The ground motions used in SLAMMER were selected from the collection of motions provided with the software. A filtering criteria can be applied in SLAMMER in order to select numerous, reasonably representative motions based on parameters such as; Magnitude, Source-to-Site Distance, Duration, Style of Faulting, Vs30, recorded PGA, Arias Intensity). Once the records to be used in the analyses were selected all of the records were scaled to a defined PGA. No spectral matching was applied to the ground motions. Fifty ground motions were used for each analyses at each seawall section. A list of the specific motions is provided as an attachment. It is was noted that the SLAMMER database of records is not optimal for M > 7.8. The minor defficiency in ground motion characterization due to motions from earthquakes with M 7.3 to 7.6 (specifically shorter average duration than would be associated with the average for M 8.0) is partially offset by the use of computed mean + 1σ slope displacements at longer return periods.	
	Phase 2 Draft Report 16 July 2015	4.6	Langan Treadwell Rollo	Section 3.7.1 - It appears that the yield accelerations were computed considering both peak and residual (post cyclic) strengths (Section 3.6, Table 3-4). However, it is not clear how the cyclic degradation and rate of loading increase in strength, which are compensating effects, were considered in the stability analyses.	We can confirm that yield accelerations were computed for both static (peak) strength and for the situation of residual strength. For liquefiable sand the residual strength (Su_r) was estimated using standard of practice procedures based on in situ penetration measurements (SPT or CPT) and back-calculated valuse of Su_r from field case studies. The strength of the young bay mud and old bay clay was modeled as follows; (i) the static peak strength (Su) was estimated from Su/σ'v ratios from DSS tests on these soils from local projects, (ii) the static strength was increased to account for rate effects using a scaling factor of 1.3 and this strength was applied for seismically-induced shear strains of less than 7.5%, (iii) the large-strain residual strength (Su_r) was assumed to be equal to an undrained strength ratio of 0.22 and this was applied for shear strains greater than 30%. The relationship between shear strain (converted to displacement for use in SLAMMER) was approximated using a stepped transition from peak to residual strength. Four steps were used with the yield acceleration decreasing at strain levels of 0% to 7.5%, 7.5% to 15%, 15% to 30%, and greater than 30%.	
	Phase 2 Draft Report 16 July 2015	4.7	Langan Treadwell Rollo	Section 3.7.1 - Please clarify how the 1906 earthquake shaking was quantified. From the results shown on Figure 3-14 it appears only probabilistic results were developed. Please provide the results of the development of the 1906 earthquake shaking.	Provided response in letter dated 12/1/2015.	
	Phase 2 Draft Report 16 July 2015	4.8	Langan Treadwell Rollo	Section 3.7.1 - How were the slope deformations computed for the liquefied case, i.e. were records from liquefied sites used?	The slope deformations were computed using SLAMMER as previously described. The residual undrained shear strength of the liquefied sand was used in GLE slope stability models to obtain Ky, then Newmark analyses performed. This approximation inherently models the soil as liquefied from the outset of the ground shaking, a procedure that is conservative. The significance of the approximation is a function of the geostatic stresses and in situ void ratio of the sand. Ground surface motions from liquefied sites were not used in the Newmark (SLAMMER) analyses.	

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	Phase 2 Draft Report 16 July 2015	4.9	Langan Treadwell Rollo	Section 3.7.1 - There are a significant number of existing underground improvements present within the study area. These have been neglected in the analyses.	Agreed. The existence of buried utilities, major buried structures, and other underground improvements has been acknowledged by the project team in discussions with the Port. In light of the "advanced screening" level of evaluation that forms the focus of this investigation the decision was made to model free-field conditions. The influence of the underground improvements on seawall and waterfront performance will likely be evaluated for high risk and higher value assets in subsequent phases of the vulnerability investigation.	
	Phase 2 Draft Report 16 July 2015	4.10	Langan Treadwell Rollo	Section 3.7.2 - It appears that the study considers settlements behind the sea wall as a function of the lateral movement of the sea wall. Such settlements are complicated and depend upon many factors. Please provide justification for the approach used	The total seismically-induced settlement behind the seawall was computed as the sum of; (i) 1D volumetric strain following re-consolidation of liquefied sand, and (ii) deviatoric strain associated with lateral deformation of the seawall. The volumetric strain was estimated using standard of practice procedures that have been routinely used in SF. The deviatoric component of settlement was estimated using normalized trends of PGDv with distance from a moveable boundary. This procedure was similar to the method applied for lateral deformations behind the seawall, except that trends of PGDv/PGDh with distance from the waterfront as measured at numerous sites affected by the 1995 Kobe EQ were used. Other empirical procedures that provide settlement profiles behind yielding walls were also used to bracket the likely range of response and develop the recommended trend for the project. An approximation was made to account for the observation that the empirical data has been obtained from sites with rigid, yielding walls (e.g., concrete caissons). It is acknowledged that the seawall would not perform in a similar rigid-body manner, therefore the PGDv trend was adjusted in proximity to the seawall to account for the pattern of deformations across the seawall section. The deformation pattern was assumed based on 2D numerical modeling on rock dikes and other earth structures, and on judgment.	
	Phase 2 Draft Report 16 July 2015	4.11	Langan Treadwell Rollo	Section 3.7.3 - An average volumetric strain of 3 percent was assumed for the sandy fill. Consistent with Tokimatsu and Seed (1987), this volumetric strain corresponds to an $N_{(1)60}$ of about 7; which appears to be inconsistent with the other parameters assigned to the fill in the other aspects of the study.	Seems to be consistent for phi = 32 degrees and residual shear strength of 250 psf.	
	Phase 2 Draft Report 16 July 2015	4.12	Langan Treadwell Rollo	Section 3.8.1 - Recent studies by Sitar et al. have shown the seismic increment of earth pressure to be triangular and increasing with depth. This is contrary to the Mononobe-Okabe (M-O) inverted triangular distribution. Please provide justification in using a uniform pressure distribution for the seismic increment.	There is not consensus within academia that Sitar et al.'s results indicating that the resultant of the seismic earth pressure is at 1/3 H. Recent papers have suggested that the resultant is at 1/2 H or higher. We have adopted a rectangular distribution for this project.	
	Phase 2 Draft Report 16 July 2015	4.13	Langan Treadwell Rollo	Figure 3-82 - The equivalent fluid weight provided for the sand fill above the water table (35 pcf) appears to be inconsistent with the parameters provided.	Seems consistent for loose sandy fill.	
	Phase 2 Draft Report 16 July 2015	4.14	Langan Treadwell Rollo	Figure 3-83 - It appears that the rock dike is considered to liquefy; however, in other sections of the report it does not appear to be considered liquefiable. Is this correct?	No. The rock dike is not considered to liquefy. Figure 3-83 is a simplification, and the artificial fill over some portions of the seawall will be the full depth of the concrete bulkhead. The figure will be corrected to represent the intention.	
	Phase 2 Draft Report 16 July 2015	4.15	Langan Treadwell Rollo	Figure 3-84 - The passive pressure for the Rip Rap does not appear to be considered liquefiable, which appears to be contrary with Figure 3-83. Please clarify.	Figure 3-84 is provided for seawall sections where riprap has been placed in recent years as a mitigation. This material is different than the rock dike.	

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	Phase 2 Draft Report 16 July 2015	4.16	Langan Treadwell Rollo	Section 3.8.2 - The material properties presented in Tables 3-6 through 3-14 appear to be inconsistent with other sections of the report and for similar materials. For example, the strength for Young Bay Mud in Tables 3-9 and 3-10 under 45-feet and 28-feet of overburden soil, respectively, have a strength of 1,500 psf. Furthermore, considering a stress ratio of 0.28 as discussed in Table 3-3, does not result in these strengths. Please provide explanation and justification for the strengths presented in Tables 3-6 through 3-14.	Young bay mud strengths were determined using a stress ratio of 0.28 at the mid-depth of the young bay mud layer.	The strengths of the young bay mud will be defined as linearly increasing with depth rather than averaging the strength across its full depth.
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.17	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Please provide the list of the 1989 Loma Prita Earthquake recording stations from the 10 deep clay sites. Also, please provide the rock recording stations that these 10 stations were compared to.	Deep clay sites with regional bedrock/shallow stiff soil sites (in parentheses): Larkspur Ferry Terminal (Rincon Hill, Pacific Heights, YBI, Richmond City Hall), Treasure Island (YBI), Emeryville (LBL), Oakland Outer Harbor Wharf (YBI and Piedmont Jr. High School), Alameda NAS (YBI and Piedmont Jr. High School), SFIA (Sierra Pt.), Foster City (Woodside Fire Station and Upper Crystal Springs-Pulgas), Redwood City-APEEL 2 (Woodside and Pulgas), Foster City-Redwood Shores-APEEL 1 (Woodside and Pulgas), and Dumbarton Bridge-West (Woodside and Pulgas). The selection and scaling of the weak rock/stiff soils records, dynamic soil response modeling, and development of SAR's was thoroughly reviewed at the time of the work by Prof. Ray Seed, Prof. I.M. Idriss, and Prof. Bruce Bolt. Documentation is provided in "Dynamic Response of Soft and Deep Cohesives Soils during the 1989 Loma Prieta Earthquake of October 17, 1989" by S. Dickenson (PhD dissertation, Civil Engineering, UC Berkeley).	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.18	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Figures 1-1 and 1-2. The SAR plotted results for the Loma Prieta Earthquake evaluations appear to be identical for both the PGA and SA0.2. Are these correct? Similar comments regarding the code site Class D and E SAR. The code SAR for PGA assumes that the Fa values for 0.2 second applies directly to the PGA as well. Is this opinion shared by the GTC/GHD team?	The trend of uniform SAR between periods of 0 and 0.2 second has been demonstrated in numerous empirical (e.g., Borcherdt) and analytical site response investigations. These observations formed the basis for the Fpga and Fa trends found in provisions and codes (NEHRP, ASCE, AASHTO, IBC). Note that SAR is a function of ground motion amplitude on the firm base, and that the PGA and SA0.2 values are not the same at a given period, thus the SAR values are not "identical" with respect to amplitude. They are however identical at the periods of 0 and 0.2 second.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.19	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Figures 1-3 and 1-4. Considering that the majority of the Loma Prieta Recording station sites that include both Bay Mud and Old Bay Clay are from much deeper sites, i.e. rock in excess of 350 feet below the ground surface, these amplification ratios could be much higher than for the site conditions as part of the Port study area. Please provide explanation and justificastion.	This topic was addressed at the peer review meeting held on 12/4/2015. Several pertinent aspects of this issue were discussed; (i) the 1989 LP EQ data has been used along with code-based trends for SAR as a guide at lower values of input ground motion amplitude, (ii) it is acknowledged that the 1989 LP EQ data provides SAR values for input motions that are significantly lower amplitude than the motions of interest for the seismic performance evaluations made in the seawall vulnerability assessment, and (iii) the possible variability in SAR's for the 1989 LP EQ due to the respective soil profiles (i.e., ten deep soil sites and the seawall alignment) will have a very small impact on the SAR values for the ground motion levels of primary interest for the seismic performance assessment. A range of site-specific SAR values representative of the seawall alignment could be made with dynamic response analysis using rock motions from Rincon or Telegraph Hill, however, it is the opinion of the project team that this modeling would have little impact on the SAR trends at ground motion amplitudes of primary interest for the project.	

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	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.20	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Ground Reponse Analysis - Bedrock motions were scaled to 0.2g, 0.4g, and 0. 6g. How do these values compare to the probabilistic spectra developed for the project?	The PGA scaling values of 0.2g, 0.4g, and 0.6g were selected to span the range of pertinent PGA values, allowing for the development of trends in SAR versus input ground motion amplitude. The PGA values were therefore selected with an appreciation of pertinent ground motions per PSHA, yet the two analyses are uncoupled.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.21	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Ground Reponse Analysis - The connection between the scaled bedrock motions and the GMPE for M 8.0 and R 13 km is unclear. How do the scaled pga's relate the considered scenario earthquake?	For the sake of developing trends in SAR versus ground motion amplitude, it is unnecessary to model specific M-R scenarios. The collection of bedrock motions were all scaled to the prescribed PGA value in order to construct the trend in SAR. Once the trend has been developed, then the applicable SAR value for a given M-R scenario can be estimated from the estimated bedrock ground motion amplitude derived from appropriate GMPE's.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.22	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Table 1-B: Input Rock Motions used in the DEEPSOIL Analyses - The Denali Earthquake "Carlo (temp)" has very low spectral values. Please provide the rationale and justification in selecting this record for the analysis?	The seven bedrock motions were selected from the PEER NGA-West on-line database. The target spectrum was used in the filtering for, and selection of, the seven motions. The average of the seven spectra matched the target very well (refer to Figure 1-9 in the TM), thus this collection of motions was used in the dynamic response analyses. The computed SAR values were compiled as averages of the seven analyses therefore the influence of a single motion has been appropriately reduced as is routine for project-specific analysis.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.23	Langan Treadwell Rollo	Item No. 3.3: Phase 2 Report - Figure 1-14 - is the FLAC results from Soil-Structure-Interaction analysis?	It is our undertanding that the FLAC results represent motions adjacent to the tube and considered "free-field" conditions. This can be pursued if necessary. The comparison from a pertinent, local project was provided for the sake of illustration only. The objective was to demonstrate variability in computed motions that can occur with different models. The most direct comparison would be made between the 1D free-field analyses using SHAKE and TESS.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.24	Langan Treadwell Rollo	Item No. 4.2: Phase 2 Report - Please provide futher explanation and details on the deformation-dependent decrease in the yield acceleration.	This topic was addressed at the peer review meeting held on 12/4/2015. Pertinent aspects of discussion included; (i) The strength of the liquefiable sand and young bay mud changes throughout the cyclic loading, decreasing from static stength to residual, (ii) as the soil strength decreases the FS against slope displacement and the corresponding yield acceleration (Ky) also decreases, (iii) the residual strengths of both soils were estimated and incorporated in GLE analyses to obtain the yield acceleration for fully softened, residual conditions, (iv) the gradual reduction in YBM strength with displacement (or shear strain) was modeled in order to develop the trend in Ky versus slope displacement, and (v) the resulting trend in Ky versus slope displacement varied from the Ky value computed for initial soil strengths and the value associated with post-cyclic residual strength. The nonlinear trend in Ky versus slope displacement was simplified in SLAMMER using a stair-stepped approximation. Four steps were used to transition from initial strengths (i.e., Ky) to residual strengths.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.25	Langan Treadwell Rollo	Item No. 4.2: Phase 2 Report - Please provide the list and pertinent attributes of the records used in the SLAMMER evalutions.	List of the 50 motions provided as a tabulation from SLAMMER (screen shot).	

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	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.26	Langan Treadwell Rollo	Item No. 4.3: Phase 2 Report - Please provide the list and pertinent attributes of the records used in the SLAMMER evalutions for Loma Prieta, M 8.0 San Francisco, and 50 records from the SLAMMER database.	The ten records used for evaluations involving the 1989 Loma Prieta EQ are from the sites listed under Item 4.17.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.27	Langan Treadwell Rollo	Item No. 4.3: Phase 2 Report - What the basis and justification of using the Loma Prieta Eathquake 10 deep site recordings as they relate to the Port study site?	As addressed in Item 4.19, the 1989 LP EQ ground motion records from deep cohesive soil sites were primarily used to highlight the range in dynamic behavior of sites around the margins of the SF Bay. The ground surface recordings provide data that was deemed useful and relevant for the sake of demonstration. This data set supplemented other procedures (code-based site coefficients, site-response analyses) for estimating SAR trends along the seawall alignment.	
	Memorandum - Reply to External Peer Review Comments by New Albion Geotechnical, Inc., 1 December 2015	4.28	Langan Treadwell Rollo	Item No. 4.3: Phase 2 Report - It is not clear if the analyses were performed for three levels of scaled pga (0.2g, 0.4g and 0.6g) as indicated in the previous sections of the memorandum or more (72, 475, 975, 2,475 years). Please provide further explanation?	Site-response analyses were only performed for PGA values of 0.2g, 0.4g, and 0.6g. The trends in SAR versus input ground motion amplitude facilitate subsequent seismic performance analyses at any seismic hazard level or Average Return Period of interest (72 to 2475 years).	
5. Structural Assessment - Bulkhead Walls	Earthquake Vulnerability Study for the Seawall Vulnerability Study at Northern Seawall - San Francisco, California, Phase 2 Report, July 16, 2015	5.1	WHL (COWI)	Section 3.8.2 Lateral Capacities of Driven Piles: Tables 3-6 to 3-14 list preliminary LPILE parameters for lateral pile analysis (used to derived p-y springs). Please verify that upper and lower bounds were considered in the seismic analysis and the liquefaction effects were also considered in the derivation of p-y springs for the liquefied soil layer.	5.1.1 - This work is a study, not a final design so upper and lower bounds were not considered. Only base case soil data was used to develop p-y springs for seismic analysis. Liquifaction effects were not considered in p-y development as the stiffer piles are embedded in non-liquified rock dike materials. 5.1.2 - Active pressures due to liquified backfills were applied to the seawall for seismic inertial load cases where liquifaction was determined to occur.	
	(1) Earthquake Vulnerability Study for the Seawall Vulnerability Study at Northern Seawall - San Francisco, California, Phase 2 Report, July 16, 2015; (2) Bulkhead Wharf Analysis Procedure.doc; (3) BHD wharf - SAP plots.zip	5.2	WHL (COWI)	Pages 87-88 of Ref. (1) concluded Seawall Section 5, Bulkhead Wall Type X (Pier 15 & 17) has sufficient capacity against static and seismic loads; however, Ref. (2) & (3) indicated that Pier 17 bulkhead wharf has insufficient capacity to resist seismic loads (timber piles supporting Seawall fail). Please clarify these inconsistencies.	5.2 Reference 1 covers only the seawall stability and structural strength and does not include the SAP analysis of the adjoining bulkhead wharf. References 2 and 3 cover the bulkhead wharf seismic analysis which indicates the seawall piles will take a greater percentage of lateral load if the seawall-to-wharf connection can transmit the lateral load.	
		5.3				
		5.4				
6. Structural Assessment - Bulkhead Wharves	Bulkhead Wharf Analysis Procedure.doc; BHD wharf - SAP plots.zip	6.1	WHL (COWI)	Pile Hinges - Please provide concrete pile hinge details (developed based on SAP Section Designer and Caltrans moment-curvature model) and how limit states were determined based on ASCE /Copri 61-14 limiting strain values.	GHD/GTC provided the summary calculations for Pier 17. We are reviewing it. (COWI comment) . 6.1.1 Summary calcs for Pier 17 provided to COWI on November 17. Summary calcs for the other four bulkhead wharfs provided to COWI on November 23. 6.1.2 Limit states were determined based on ASCE limit strain values using the SAP Section Designer strain/curvature relationship detail data for each section. Where insufficient strain was obtained by Section Designer for a given limit state, the curvature associated with the maximum strain was used for the limit state.	

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	Bulkhead Wharf Analysis Procedure.doc; BHD wharf - SAP plots.zip	6.2	WHL (COWI)	Inertia Seismic Loads - Please clarify how bulkhead wharf inertia seismic load demands were determined. Did the seismic analysis consider soil liquefaction effects on the p-y springs? upper and lower bounds of p y springs? orthogonal excitation effects? Were concrete piles assumed cracked during the seismic event?	GHD/GTC provided the summary calculations for Pier 17. We are reviewing it. (COWI comment) 6.2.1 The full inertial seismic loads were applied on a linear model and the associated deck displacement demand was determined. A nonlinear pushover analysis was performed to determine when the various limit states were reached; the associated deck displacement capacities were determined for each limit state. 6.2.2 The seismic analysis did not consider soil liquefaction effects on the p-y springs. 6.2.3 Base case p-y springs were used in this study due, in part, to study budget constraints and the study scope. Lower and upper bound springs would be more appropriate for detailed design. 6.2.4 Concrete piles were assumed cracked during the seismic events, see SAP model property modification factors.	
	Bulkhead Wharf Analysis Procedure.doc; BHD wharf - SAP plots.zip	6.3	WHL (COWI)	Please provide Bulkhead Wharf SAP2000 models.	GHD/GTC provided the SAP model for Pier 17. (COWI comment) - The P17 bulkhead wharf SAP2000 model was transmitted to COWI on November 17; the models for the other four bulkhead wharfs were transmitted to COWI on November 23rd.	
	Bulkhead Wharf Analysis Procedure.doc; BHD wharf - SAP plots.zip	6.4	WHL (COWI)	Moment Curvature (per Caltrans) - Moment-curvatures shown for existing concrete piles on pages 10, 11, and 15-17 did not include axial load effects. Please also clarify if the bi-linearization met ASCE/Copri 61-14 criteria. Please provide Moment-curvatures for pile-deck connections.	6.4.1 The zero axial load condition was assumed as a balance between undefined compression and potential tension loads. We also re-assessed P17 with hinges that account for axial-load effects, see additional response in 6.18. 6.4.2 The assumed moment curvatures were developed based on the Caltrans model. The SAP hinge definitions provided in the calculations show this bilinear response. 6.4.3 Moment-curvature relationships input into the SAP analysis are presented in the calculations transmitted to COWI on November 23rd.	
	Bulkhead Wharf Analysis Procedure.doc; BHD wharf - SAP plots.zip	6.5	WHL (COWI)	For Pier 17, Please provide structural drawings for piles and pile-deck connection details.	The structural drawings for piles and pile-deck connections for the five bulkhead wharfs were transmitted to COWI on November 23rd.	
	Bulkhead Wharf Analysis Procedure.doc;	6.6	WHL (COWI)	Moment Curvature (per Caltrans) - Moment-curvatures shown for existing concrete piles on pages 10, 11, and 15-17 did not include axial load effects. Please also clarify if the bi-linearization met ASCE/Copri 61-14 criteria. Please provide Moment-curvatures for pile-deck connections.	Same comment as 6.4, see response to 6.4.	

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	P17-BhdWharf.sdb (SAP2000 Model of Pier 17); POSF BhdWharf P17 Calcs.pdf; Due Diligence Report Pier 17.pdf (Rutherford & Chekene, 2007)	6.7	WHL (COWI)	R&C performed material testing on Pier 17 concrete pile jacket, where it stated in its 2007 Due Diligence Report that the yield strength of the rebar removed from the pile jacket range from 48.4 ksi to 48.5 ksi and the tensile strengths range from 64 ksi to 69 ksi. These test results should be more appropriate to represent the concrete pile reinforcement strength for the Pier 17 Bulkhead-Wharf concrete piles, instead of assuming these concrete pile rebars as Grade 33 steels.	We can re-assess the Pier 17 bulkhead wharf to determine the sensitivity to the reinforcement yield and ultimate strengths. We would expect a 50 percent increase in moment capacity with an increase in displacement capacity for seismic inertial load due to this stronger reinforcement. A similar increase may occur of lateral sliding but the piles will still be deficient for this load condition. See additional response in 6.18.	
	P17-BhdWharf.sdb (SAP2000 Model of Pier 17); POSF BhdWharf P17 Calcs.pdf; Due Diligence Report Pier 17.pdf (Rutherford & Chekene, 2007)	6.8	WHL (COWI)	The plastic hinge length for pile-to-deck connection was calculated based on ASCE/COPRI 61-14 Table 6-1; however, these are defined for steel pipe pile or prestressed concrete pile connections to the deck. For reinforced concrete pile connection to concrete deck or cap beam, we recommend to calculate plastic hinge length base on MOTEMS section 3107F2.5.3 Equation (7-5) or Port of Long Beach Code Section 4.6.6.2.	We can re-assess the Pier 17 bulkhead wharf to determine the sensitivity to the pile-to-deck connection hinge length. POLB Table 4.2 makes no distinction between prestressed and reinforced concrete piles and does give a larger hinge length for concrete dowelled connections. We used ASCE 61-14 Table 6-1 for prestresed dowel connections on the basis that the dowels were the significant contributor to connection hinge length. We reassessed P17 bulkhead wharf for hinge lengths based on POLB Table 4.2, see additional discussion in 6.18.	
	Bulkhead Wharf Analysis Procedure.doc; BHD wharf - SAP plots.zip	6.9	WHL (COWI)	There are some discrepancies when using load combinations from ASCE 7 when calculating seismic load demands and compared to capacity calculated based on ASCE/Copri 61-14. For example, (1) limiting strain values defined in ASCE/Copri 61-14 codes for OLE, CLE and DE earthquake events are different than those used in the ASCE 7 load combinations; and (2) effect of live load on P-Delta effects on piles are significantly larger in ASCE 7 load combinations where 100% Live Loads were included as compared to ASCE/Copri 61-14 where only 0.1LL is required.	Please clarify the intent of this comment. Limiting strain values defined in ASCE 61 were used for the seismic inertial analyses. The ASCE 7 load combinations were used with all phi and load factors equal to 1 for the purposes of assessing seawall stabilty and strength. Similarly, no load factors are applied in the seismic inertial analyses. Full live load was applied in the analyses since building specifics were not specifically known.	
	P17-BhdWharf.sdb (SAP2000 Model of Pier 17); POSF BhdWharf P17 Calcs.pdf;	6.10	WHL (COWI)	Pier 17 Bulkhead-Wharf seismic performance was evaluated assuming wharf deck/beams are integrated with the supporting unreinforced Seawall. The mass (inertia loads) of the wharf contribute significant load effects on supporting timber piles of the bulkhead wall. Please verify this assumption from design drawing.	The true capacity of the beam-to-seawall connection is not specifically known and may be as low as effectively zero. If zero, the lateral resistance will shift from the bulkhead wall piles to the wharf piles. For rehabilitation purposes, a conceptual retrofit connection will be developed to carry this demand loading.	
	POSF BhdWharf P17 Calcs.pdf;	6.11	WHL (COWI)	Plastic hinge properties (moment-curvature curves) for concrete piles should be determined including the axial load effects on the pile. The moment curvature curve should be bilinearized according to ASCE/COPRI 61-14 Section 6.6.2. Please verify moment-curvature curves determined per Caltrans by Xtract sectional analysis.	6.11.1 See response to 6.4.	
	P17-BhdWharf.sdb (SAP2000 Model of Pier 17);	6.12	WHL (COWI)	From the analysis, plastic hinges in the concrete piles always formed right at the first p-y springs. The moment diagram of the pile shows the maximum moment at the first p-y spring (as if the pile is "fixed" at the location of the first p-y spring), whereas the maximum moment is expected to occur somewhere below the mudline elevation (below the first few p-y springs). Please check pile moment diagrams in the SAP2000 model and clarify whether this is a modeling issue/error.	We checked the models for any unintended fixity and did discover this. We revised all models to correct this and reanalyzed. The Phase 2 report will be revised to reflect the revised analysis results. See also additional response in 6.18.	
	P17-BhdWharf.sdb (SAP2000 Model of Pier 17);	6.13	WHL (COWI)	Design drawings shown that the deck is composed of 7" concrete slab with 2" asphalt topping with concrete cap beams, but no buildings on Pier 17 Bulkhead-Wharf. SAP2000 model assumed 12" concrete slab with 7.2" concrete building roof and zero pile mass. Please verify total wharf weight/mass per design drawings.	We will verify but there is no roof structure modeled in the P17 analysis model. The comment implies a difference of 3 inches or about 37.5 psf. This discrepancy falls within the range of uncertainty for this study.	

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	(1) P17-BhdWharf.sdb (SAP2000 Model of Pier 17); (2) POSF BhdWharf P17 Calcs.pdf; (3) Bulkhead Wharf Analysis Procedure.doc	6.14	WHL (COWI)	It was stated in (3) that inertia seismic loads for all models are at PGA except for buildings, where PGA*Sa(0.7) was applied to buildings per geotechnical recommendation. This likely underestimates the design earthquake loads on bulkhead-wharf structures. One may readily estimate the design earthquake spectral acceleration from the Capacity Spectrum Method, which is significantly larger than PGA for Pier 17 bulkhead-wharf structure.	Agreed that the use of the spectral peak may underestimate the design elastic earthquake loads on the bulkhead wharf structure. However, precise details of the building structures and their non-linar response are generally unavailable and are outside the scope of this study. . The CSM is a method that requires a non-linear pushover analysis of the building to determine structural capacity combined with a response spectrum analysis of the building to determine structural demand. This exercise was deemed beyond the current scope of the study.	
	(1) P17-BhdWharf.sdb (SAP2000 Model of Pier 17); (2) Earthquake Vulnerability Study for the Seawall Vulnerability Study at Northern Seawall - San Francisco, California, Phase 2 Report, July 16, 2015;	6.15	WHL (COWI)	When evaluating soil sliding loads on Pier 17, the nonlinear load case "NL-Static(LSP)" used 120 pcf as the liquefied soil weight in combination with "Hydro-Fill-UPR-TRI" and "Hydro-Fill-LWR-RECT" loads. These unbalanced hydrostatic pressures were not shown on Figure 3-83 on page 38 of the Phase 2 Draft Report. In addition, was the "Passive Earth Pressure" shown on Figure 3-82 (Non-Liquefied Soil) and Figure 3-83 (Fill below Groundwater Liquefied) included in the SAP2000 analysis?	6.15.1 The "Hydro-Fill-UPR-TRI" and "Hydro-Fill-LWR-RECT" load patterns are unit load cases to capture the differential hydrostatic pressures behind the wall relative to MLLW. While this case is not shown on Figure 3-83, the figure does show a 3.84 ft differential height between the water table behind the wall and MLLW. 6.15.2 The passive earth pressure at the bottom of the seawall was conservatively neglected in the analysis.	
	(1) P17-BhdWharf.sdb (SAP2000 Model of Pier 17); (2) Earthquake Vulnerability Study for the Seawall Vulnerability Study at Northern Seawall - San Francisco, California, Phase 2 Report, July 16, 2015;	6.16	WHL (COWI)	The SAP2000 analysis (Pier 17) shows that wharf piles formed plastic hinges at the pile-deck connections under "NL-Static(LSP)" load case, mainly due to the active soil pressure assuming fill below groundwater liquefied. Generally speaking, as this soil mass sliding out (120" for the 475-yr EQ), what is the effective active soil pressure against the seawall? Are same p-y springs used for both non-liquefied and liquefied soil cases?	6.16.1 The active soil pressure against the seawall is assumed unchanged as the soil mass slides out. . 6.16.2 The same soil springs were used for both non-liquified and liquified soil cases but gap elements are provided over the pile liquified zone to eliminate any soil resistance through this sliding zone.	
	P17-BhdWharf.sdb (SAP2000 Model of Pier 17)	6.17	WHL (COWI)	For Pier 17 and other bulkhead-wharf structures in general, when the weak soil layer below rock dike liquefies and slides, is it reasonable to assume that everything above the liquefied soil layer to slide the same distance (as liquefied or non-liquefied soil mass)?	The actual response under soil lateral sliding is very complex and is not predictable to any degree of accuracy. The constant mass movement was deemed an appropriate assumption to ascertain the capacity of the piles to resist such movement. The general conclusion of this study is that there is a significant issue regardless of seawall location. If the assumption of mass movement were changed, this conclusion should remain the same.	

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		6.18			The Pier 17 bulkhead wharf model was revised to correct the observed pile fixity issue and assuming the following: revised rebar strength Fy=50 ksi (expected yield of 55 ksi), revised pile moment-curvature hinges based on axial loads ranging from 50 kips tension to 100 kips compression, and revised pile hinge lengths based on POLB WDC. The fixity error is applicable to both linear and non-linear models and resulted in an increase in both seismic demand and capacity. This error was corrected in all pier bulkhead models analyzed and all models were reanalyzed. Comparing the Pier 17 bulkhead wharf models when varying the rebar yield strength and other non-linear parameters, the inertial load and soil lateral sliding analyses resulted in DCRs that were about 5 percent lower and 50 percent lower, respectively. This minimum DCR achieved for all bulkhead wharf lateral sliding analyses is 4.3 with most DCRs significantly greater than this minimum; thus, soil lateral sliding remains a problem. Discussions with the peer review members during the Phase 3 review meeting indicated that the peer review members were satisfied with these results and deemed this variance acceptable since it has no significant qualiftative effect on this vulnerability study.	
7. Retrofit Concepts		7.1				
		7.2				
		7.3				

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