Seawall Movement and Piling Design at the San Francisco Waterfront

Di Liu, PE[1]; Erik Soderberg, SE[2]

[1]Liftech Consultants Inc., 344 – 20th Street, Suite 360, Oakland, CA 94612-3593; email: DLiu@Liftech.net

[2]Liftech Consultants Inc., 344 – 20th Street, Suite 360, Oakland, CA 94612-3593; email: ESoderberg@Liftech.net

ABSTRACT

The San Francisco Fire Department is acquiring a new floating fire station. A new pile-supported pier structure will connect to the fire station with an access ramp. The new pier piles will extend down through the existing 100+ year old seawall rock embankment, through a weak and thin, 10 ft (3 m), layer of consolidated bay mud below the embankment, into a stiffer alluvial layer and bedrock below.

This paper discusses methods and findings for this project, including challenges encountered and suggestions for future projects. Topics include:

> Soil dynamic shear strength obtained from testing
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> Estimated seawall seismic movement
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> Piling response
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> Comments on ASCE 61 design requirements
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> Methods and findings for evaluating the pile pinning strength
>
> Seawall bulkhead arching across the site due to adjacent larger seawall movements

INTRODUCTION

The San Francisco Fire Department is acquiring a new floating fire station. A new steel pier structure will connect to the fire station with an 80 ft (24.4 m) long access ramp.

The pier superstructure is about 60 ft (18 m) long in the north-south direction and 33 ft (10 m) wide at the south end in the east-west direction. The new pier will be supported on four 90 ft (27 m) long, hollow steel pipe piles with 36 in (900 mm) outside diameter by 1 in (25 mm) thick wall. Piles are spaced 30 ft (9 m) in the north-south direction and 17 ft and 8 ft (5 m and 2.4 m) in the east-west direction for the southern and northern piles, respectively. See Figure 1.

The piles will extend through about 50 ft (15 m) of rock fill embankment, a 10 ft (3 m) layer of young bay mud, into a strong and stiff alluvial layer of sand and clay, and then into Franciscan complex rock layer as shown in Figure 2.
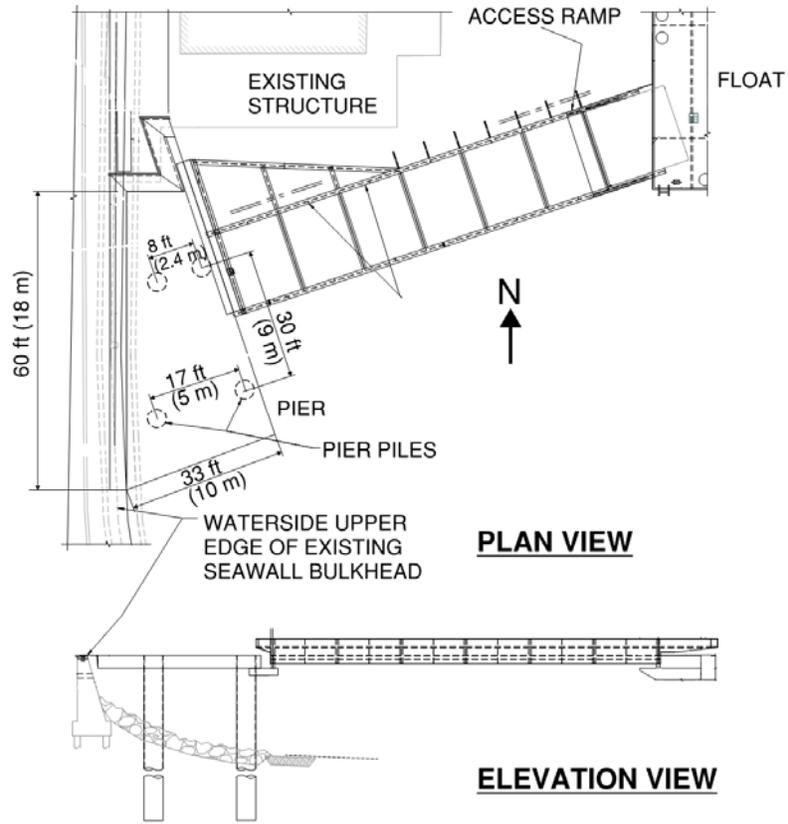
Figure 1. Arrangement of new pier and access ramp

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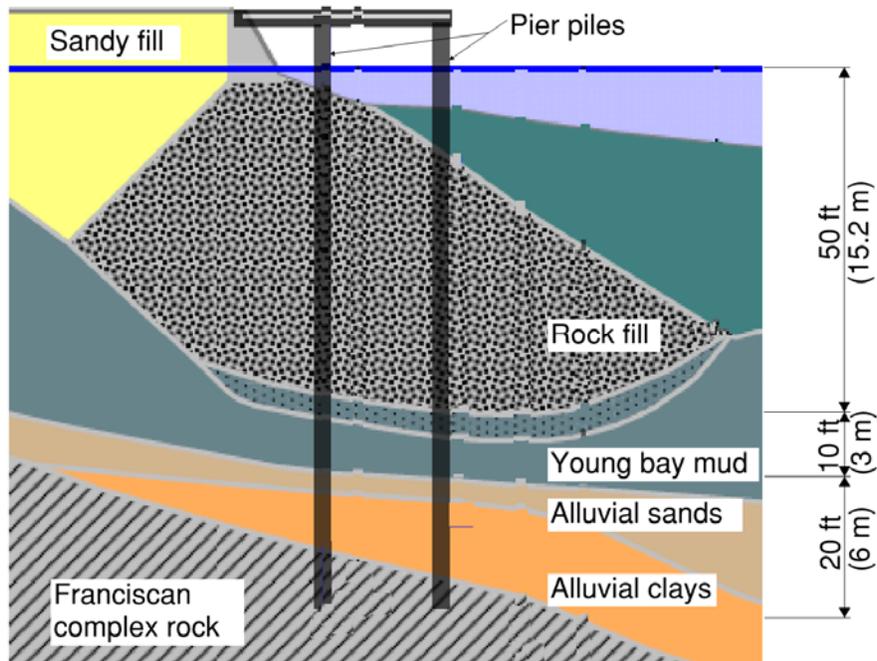
Figure 2. Soil profile and pier piles

The pier structure is designed as an essential facility and to be operational for the considered earthquake levels: with zero to minimal damage after the Operating Level Earthquake (OLE); minimal damage after the Contingency Level Earthquake (CLE); and controlled and repairable damage after the Design Earthquake (DE), with OLE, CLE, and DE as defined in ASCE 61-14 (ASCE 2014).

The primary geotechnical design issue is the loading on the pier from the seawall (bulkhead and rock embankment) moving towards the bay in an earthquake. This type of loading is a significant concern to structures along much of the San Francisco waterfront due to the liquefiable fill landside of the seawall, and the weak bay mud layers under the rock embankment (Rudolph et al 2018).

This paper presents some of the key design considerations for the pier piling.

SOIL DYNAMIC SHEAR STRENGTH

Testing the dynamic strength of the bay mud below the rock embankment was worthwhile for the geotechnical analysis, but more difficult than expected. Collecting test samples was complicated by the thin layer of bay mud, combined with intermixing of the embankment rock with the mud, and required unusual methods. The strength of the Young Bay Mud was evaluated for static shearing resistance, and for variations due to cyclic loading (i.e., rate effects) and large-strain conditions. The measured shearing resistance was greater than used in some previous seawall studies. Refer to the companion paper (Serna et al 2019) and (GHD-GTC 2016).

ESTIMATED SEAWALL MOVEMENT

Site specific design earthquake ground motions were developed by the geotechnical engineer. Horizontal free-field permanent design ground deformations (PGDh) of the rock embankment for different peak ground accelerations are shown in Figure 3 (Rudolph et al 2018). The controlling permanent horizontal embankment design deformation was for the contingency level earthquake and was estimated as 11 in (280 mm) for mean level and 22 in (560 mm) for mean plus one standard deviation.

The Design Earthquake seawall movement is 2/3 that of the MCE$_R$. The Risk-Targeted maximum Considered Earthquake (MCE$_R$) is the lesser of the probabilistic ground motions (1% exceedance in 50 years) and deterministic ground motions (84 percentile 5% damped spectral response in the direction of maximum response). Therefore, it is not necessary that the DE be greater than the CLE. For this project, the CLE PGA is greater than the DE PGA.
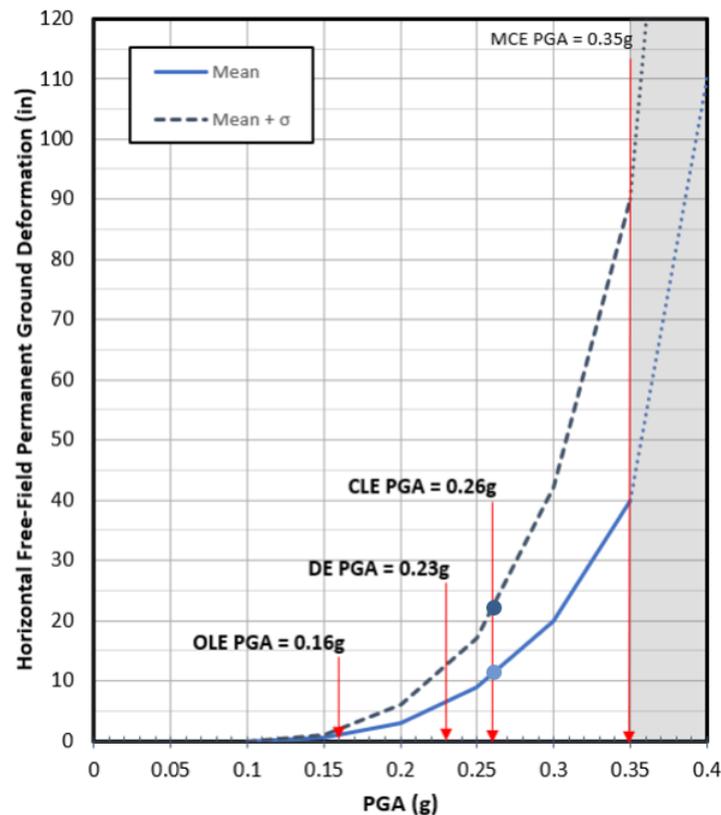
Figure 3. Estimated seawall earthquake-induced PGDh

PILING EVALUATION METHOD

Piling behavior was evaluated using LPile and SAP2000 analyses. In SAP2000, the piles were modeled using beam elements, and soil-pile interaction was modeled using two non-linear springs in two orthogonal directions every 5 ft (1.5 m) along the pile, as shown in Figure 4. A trilinear load-deformation soil spring was used.

(a) Pier elevation   (b) Soil spring model
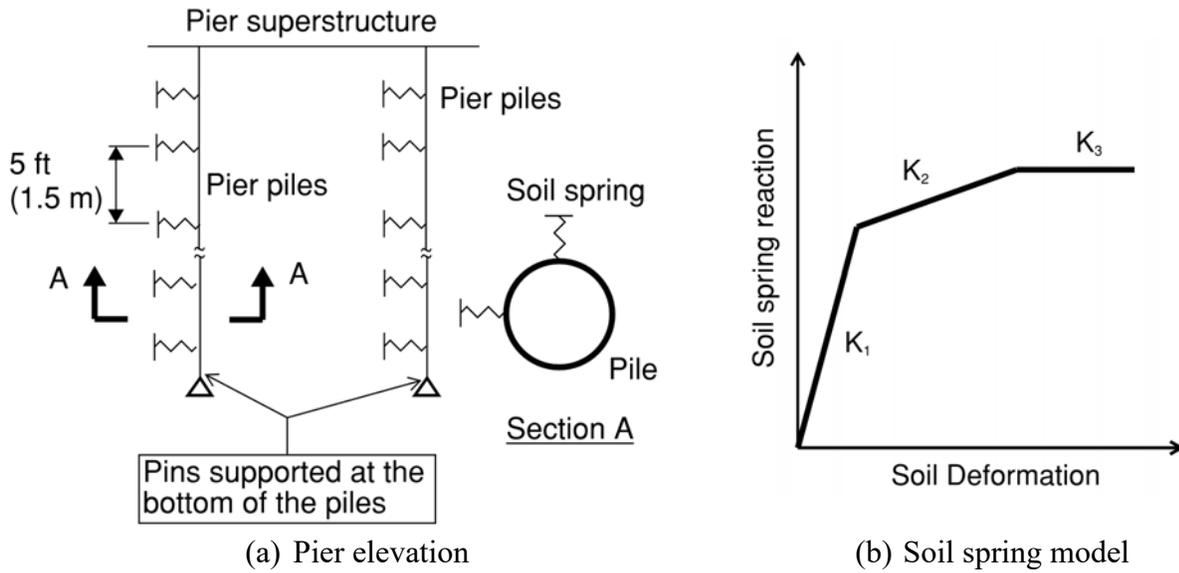
Figure 4. Pier elevation and soil spring

When the seawall moves towards the water the piling moves with it. Figure 5a shows the calculated deflected shape with two plastic hinges forming in the pile based on the rock within the blue dashed line providing enough lateral support to plastically bend the pile. If the rock embankment is not strong enough, the upper plastic hinge may not form, as shown in Figure 5b. For our analysis, the rock embankment was determined to be strong enough to plastically bend the upper portion of the pile, i.e., two plastic hinges form.
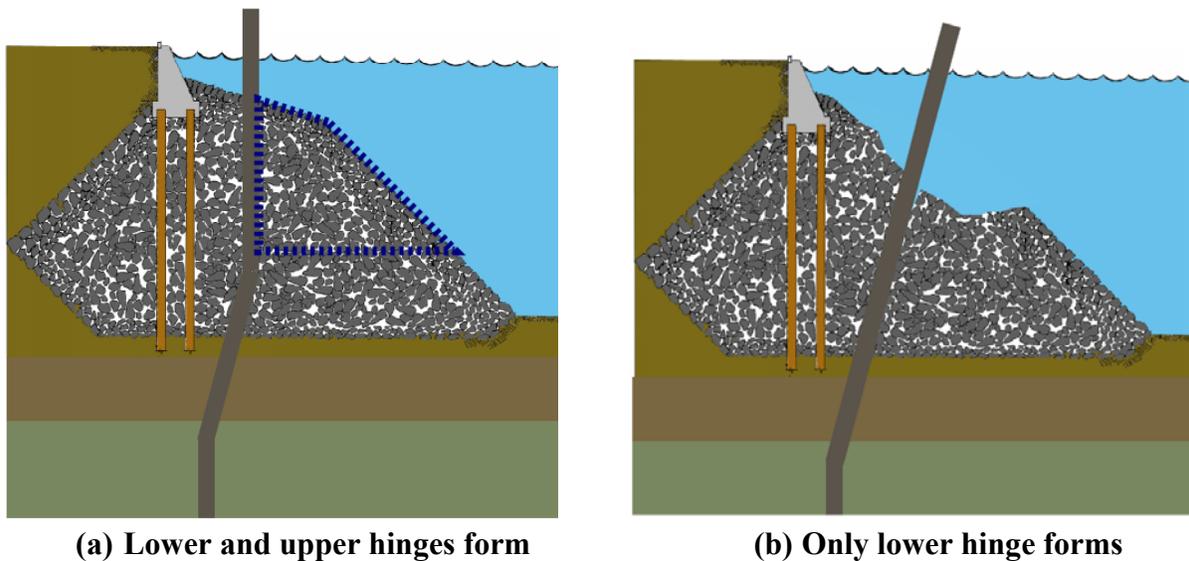
(a) Lower and upper hinges form   **(b) Only lower hinge forms**
Figure 5. Pile deflected shape

PILING RESPONSE

Inertial and Kinematic Interaction

The soil stiffness varies significantly between soil layers. Table 1 shows the normalized soil spring stiffnesses (K_1 in Figure 4) at different locations along the pile. The normalized soil spring stiffness is the spring stiffness at each location divided by the smallest spring stiffness along the pile. The young bay mud layer is the most flexible layer. The alluvial layer stiffness is about five times stiffer than the young bay mud, the embankment rock between 11 and 83, and the bed rock layer 1900 times stiffer.

Table 1. Normalized Soil Spring Stiffness along Pile

Distance below Pile Top ft (m)		Layer	Normalized Soil Spring Stiffness
-15	(-4.6)	Rock Fill Layer	11
-20	(-6.1)		20
-30	(-9.1)		38
-40	(-12.2)		56
-50	(-15.2)		74
-55	(-16.8)		83
-60	(-18.3)	Young Bay Mud	1
-65	(-19.8)		1
-70	(-21.3)	Alluvium	5
-75	(-22.9)		5
-80	(-24.4)	Bed Rock	1900

The bending from inertial and kinematic loadings is at significantly different locations along the pile (Figure 6). The rock embankment anchors the upper section of piling for the design inertial loading, with bending becoming negligible below 40 ft from the pile top (12.2 m). Under the design kinematic loading, the rock embankment moves as a rigid body relative to the young bay mud underneath. The first plastic hinge forms at 55 ft (16.8 m) below the pile top and the bending above 40 ft (12.2 m) is negligible.
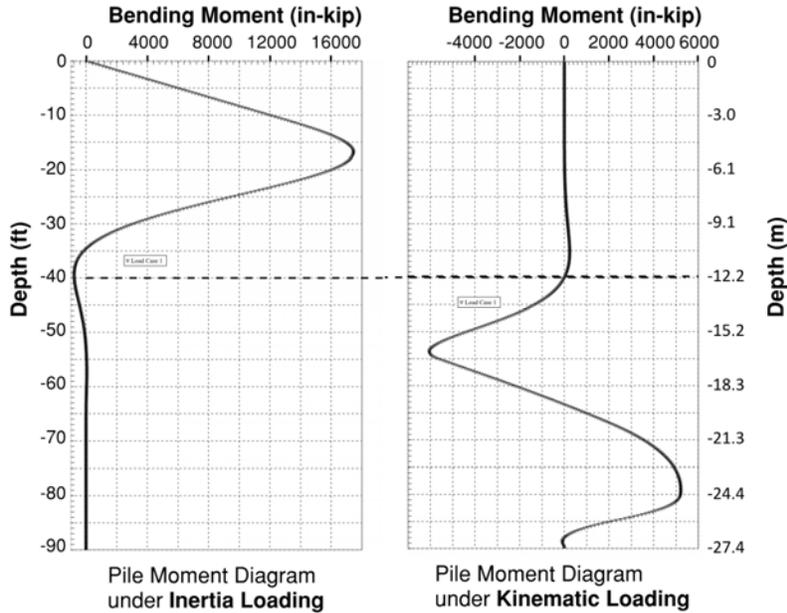
Figure 6. Pile bending moments - inertial and kinematic loading

Kinematic Loading

The allowable pile deflection is calculated by combining the elastic deflection and the plastic deflection corresponding to the strain limits for each level of earthquake as specified in ASCE 61-14, 0.01 for OLE and CLE, and 0.025 for DE. The plastic deflection equals the plastic hinge rotation times the distance between hinges (Figure 7). The plastic hinge rotation equals the product of the plastic hinge length and plastic curvature for the OLE, CLE, or DE allowable strain limit. See ASCE 61 (ASCE 2014).
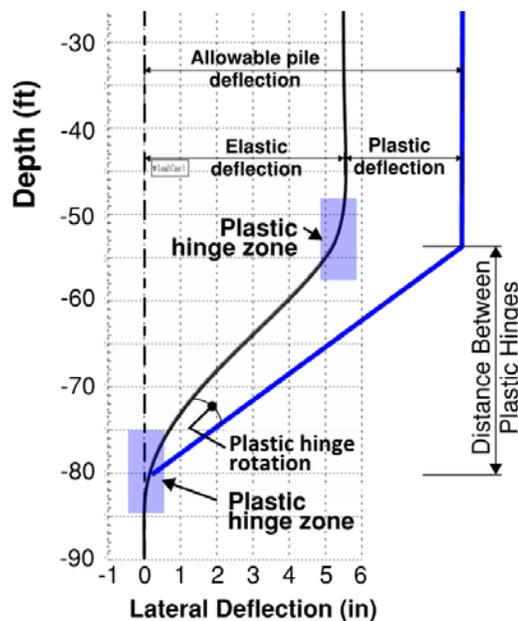
Figure 7. Calculation of allowable pile deflection

The pile deflections were calculated using LPile and SAP2000 programs considering nonlinear behavior of both the soil and pile structure.

COMMENTS ON ASCE 61-14

Strain Limits

ASCE 61 strain limits are constant and are not related with section compactness. This is being addressed by the ASCE 61 committee for the next version. The ASCE 61 preliminary strain limit revisions as of July 2018 were also considered in our design. The planned revised limits were within 10% of the strains estimated for this project.

A 1 in (25 mm) wall thickness was selected to facilitate installation through the rock embankment and into the Franciscan complex rock layer, and to provide sacrificial steel for corrosion at the waterline.

In-Ground Strain Limits – Required Performance

ASCE 61 requires strain limits regardless of the effect on pile performance, i.e., fitness for purpose. For this project, we found that if the plastic hinges deteriorate to carry no bending moment, the pile axial capacity will be unaffected, and the piles will still be acceptable. The eccentricity that develops would be resisted by the lateral soil pressures (Figure 8).

> Example: If the plastic hinges deteriorate and carry no moment, and if there was a 4 ft (1.2 m) eccentricity on the 200-kip (91-t) design loading, only 40 kip (18 t) lateral soil resultants would be required if the resultants are 20 ft (6 m) apart, or 32 kip (14.5 t) if 25 ft (7.6 m) apart.
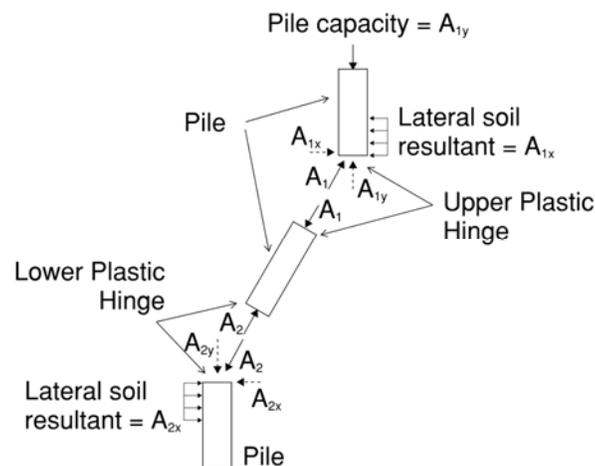
Figure 8. Free body diagram if plastic hinges have no bending capacity

We recommend that ASCE 61 include a provision that allows deviation from the specified strain limits if the piling can perform as required.

METHODS AND FINDINGS FOR EVALUATING THE PILE PINNING STRENGTH

The effect of pile pinning was initially considered for the embankment movements, but it was determined that the effect did not need to be considered to comply with the project criteria, i.e., the free field embankment design movements were less than the allowed pile deformations.

Initial evaluations, comments, and suggested considerations for future work follow:

1. Large steel pile can provide significant shear resistance, particularly if the bay mud layer is not thick and two hinges can form in the piling.

2. If plastic hinges form and large soil movements are expected, it will be critical to use a compact pile section to help ensure moment capacities and shear resistance does not deteriorate after limited seawall movements.

3. Piles with non-compact sections may be practical if seawall movements can be limited, i.e., no or limited plastic deformation.

4. If strong piling is considered for pinning, evaluate the ability of the anchoring layer, e.g., the rock embankment, to anchor the pile and provide the required lateral reaction to the pile. At some strength the pile will rotate through the embankment.

SEAWALL BULKHEAD ARCHING ACROSS THE SITE DUE TO ADJACENT LARGER SEAWALL MOVEMENTS

The seawall concrete bulkhead is a large concrete structure and has significant bending strength (Figure 9).
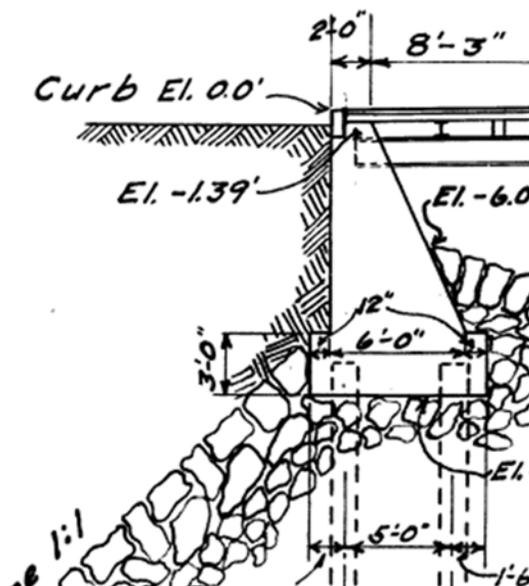
Figure 9. Concrete bulkhead section (Board of State Harbor Commissioners 1912)

For our design, we made preliminary estimates of the bulkhead strength based on the arching moment capacity. The moment capacity was estimated to equal the compression reactions on

either end of a bulkhead length ("C" in Figure 10) times the eccentricity ("e" in figure 10) between the reactions.

If piles pin the embankment, but the pinning effect is only localized, the bulkhead from adjacent unpinned areas could transfer significant shear forces. Based on the arching phenomenon, the shear that can be transferred by the bulkhead from unpinned area equals to Ce/L (Figure 10).

Based on initial evaluations, comments and suggested considerations for future work follow:

1. An analysis considering the seawall adjacent to a project location may be required to understand the effect of the bulkhead movement and seawall movement at the project location.

2. The bulkhead could be weakened at select locations to limit the shear it could transfer. This idea was not received favorably for our project.

3. It may be practical to isolate the bulkhead from the embankment and piling at a pinned area. Ideas include providing compressible material or cavities waterside of the bulkhead so its movement would not result in significant loading on pinned areas.
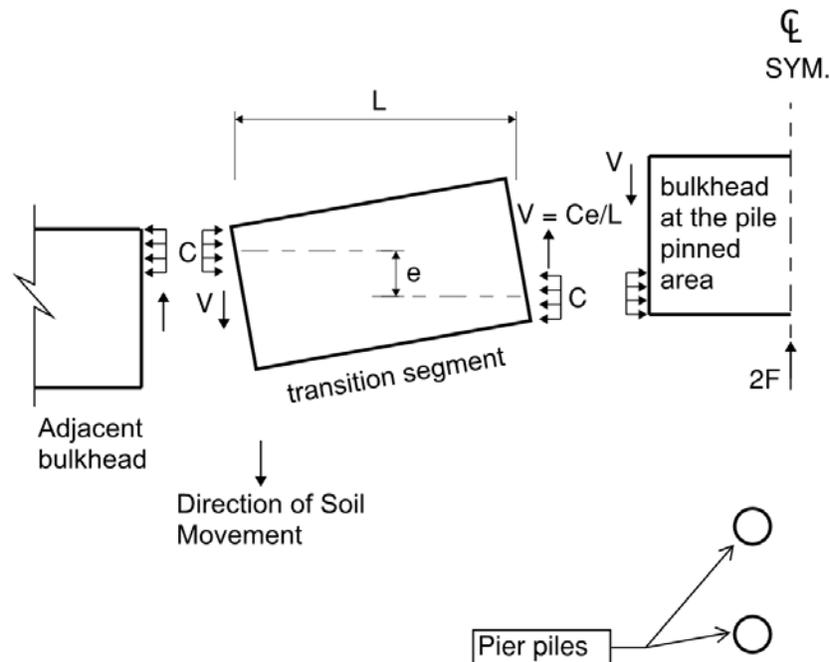
Figure 10. Seawall bulkhead arching

CONCLUSION

A steel pier structure connecting to the future San Francisco floating fire station was designed to resist seismic inertial loads, and kinematic loading caused by permanent ground deformation towards the Bay. The pier structure has four 36 in (915 mm) diameter, 1 in (25 mm) thick wall, 90 ft (27 m) long, hollow steel pipe piles extending through a stiff rock fill embankment, then a

very flexible and weak young bay mud layer, then a strong and stiff alluvial layer of sand and clay, and then into Franciscan complex rock layer. The following are some primary findings:

> The soil stiffness and strength varied significantly between the different soil layers.

> The maximum bending moment from the inertial and kinematic loadings occurred at significantly different locations along the pile.

> The design seawall movement did not result in excessive pile strains.

> Even if the pile strain were to significantly exceed the design limit and hinges carrying no moment developed in the pile, the pile axial capacity would probably still be maintained due to the modest lateral soil reactions required to stabilize the "pinned" pile segment.

The work resulted in several comments and suggestions regarding pile design in or near the San Francisco seawall.

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