number of loading cycles. The maximum accumulated shear strain was used as a proxy for the following project-specific reasons; (i) the number of loading cycles of interest for the seismic deformation analyses is constant (30 cycles), and (ii) the Newmark-type analyses for bracketing the likely range of the slope deformations tracks the cyclic accumulation of displacement from which shear strain can be approximated. More sophisticated methods of nonlinear deformation analysis (e.g., FLAC, PLAXIS, LS-Dyna) can be used to simulate the cyclic accumulation of shear strain, after element calibration, using an appropriate laboratory data set such as that obtained for this project.

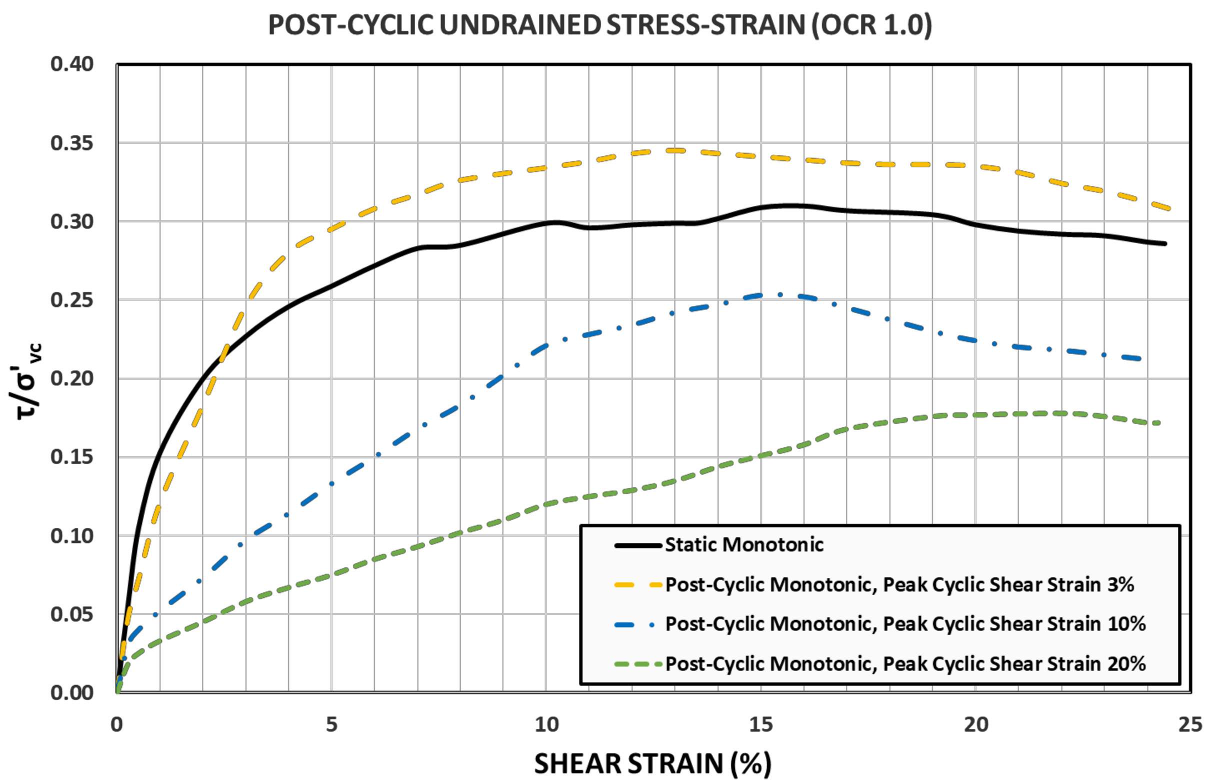

Figure 6. Static Monotonic and Post-Cyclic Monotonic Stress-Strain Behavior of Normally Consolidated YBM Specimens.

Peak dynamic and peak static undrained strengths were incorporated into generalized strain-dependent strength models developed for the YBM based on relationships for the ratio of post-cyclic to static undrained shear strength as a function of maximum cyclic shear strain. The strain-dependent model considers the transition of cyclic and post-cyclic shearing resistance of the YBM across a large range of cyclic shear strains. This includes moderate strains associated with peak dynamic strength transitioning to intermediate strains at softened or "residual" strengths, representative of laboratory DSS and triaxial tests results. Intermediate to large strains representing the transition to fully-remolded strength are associated with field vane shear test results after multiple rotations of the vane per ASTM standards or sleeve friction from the CPT. The fully- remolded shearing resistance is routinely used to define the sensitivity of the soil, which has been found for local deposits of YBM to range from roughly 3.0 to 5.0. The trend of strain-dependent strength and generalized zones of stress-strain behavior for the YBM underlying the rock dike, including the range of peak static strength, are illustrated on Figure 7.

We developed similar strain-dependent relationships for the YBM on the landside of the rock fill dike, YBM on the bayside of the rock fill dike, and Recent Bay Mud (RBM), the latter representing re-deposited Bay Mud (i.e., siltation in areas with no recent maintenance dredging). The strain-dependent strength models were applied to evaluate the potential for large-scale earthquake-induced ground deformations.

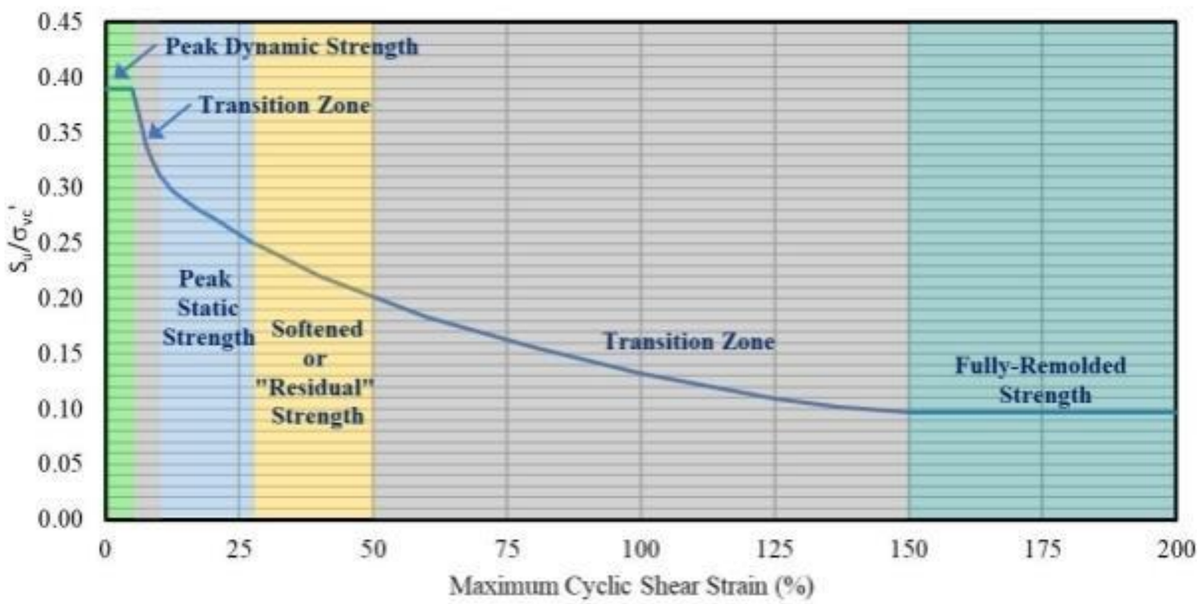

Figure 7. Peak Undrained Shear Strength Ratio vs. Maximum Cyclic Shear Strain for Normally Consolidated YBM Under Rock Fill Dike.

EARTHQUAKE GROUND MOTIONS

For seismic design of the proposed pier piles, site-specific ground motions were developed in accordance with ASCE 61-14 per the project seismic design criteria. The proposed pier structure is assigned a design classification of “high” in accordance with ASCE 61-14 design classifications. Therefore, ground motions corresponding to the Operating Level Earthquake (OLE), Contingency Level Earthquake (CLE), and Design Earthquake (DE) defined in Table 1 must be considered as the minimum seismic hazard and performance level requirements. Design acceleration response spectra and ground motions within the rock fill (not at the mudline) were developed for structural design of the pier based on one-dimensional non-linear site-specific site response analyses performed using DEEPSOIL v6.1 (Hashash, Y.M.A. et al. 2016). Earthquake ground motion parameters for the proposed pier are summarized in Table 1. These represent averaged motions within the rock fill and were used in the assessment of seismically induced-ground deformations, which is discussed in more detail in subsequent sections of this paper. In support of the seismic design of the proposed guide piles, time histories for the DE were also developed at the equivalent pile depth to fixity for nonlinear dynamic analyses of the float piles performed by the structural engineer.

SEISMICALLY-INDUCED GROUND DEFORMATIONS

The assessment of seismically-induced permanent ground deformation incorporated a practical, yet adequate, correlation between accumulated shear strain throughout the ground motion time history and shear strength of the YBM. The proposed structures were designed considering kinematic loading (displacement demand) from seismically-induced horizontal free-field permanent ground deformations; therefore, pile pinning effects on slope deformation were not evaluated.

Table 1. Summary of Earthquake Ground Motions for Pier (Within Rock Fill).

Seismic Hazard Level	Ground Motion Probability of Exceedance	Peak Ground Acceleration (PGA)	Earthquake Magnitude (Mw)
Operating Level Earthquake (OLE)	50% in 50 years (72-year return period)	0.16g	7.5
Contingency Level Earthquake (CLE)	10% in 50 years (475-year return period)	0.26g	7.9
Design Earthquake (DE) per ASCE 7 (2005)	Not Applicable	0.23g	7.9

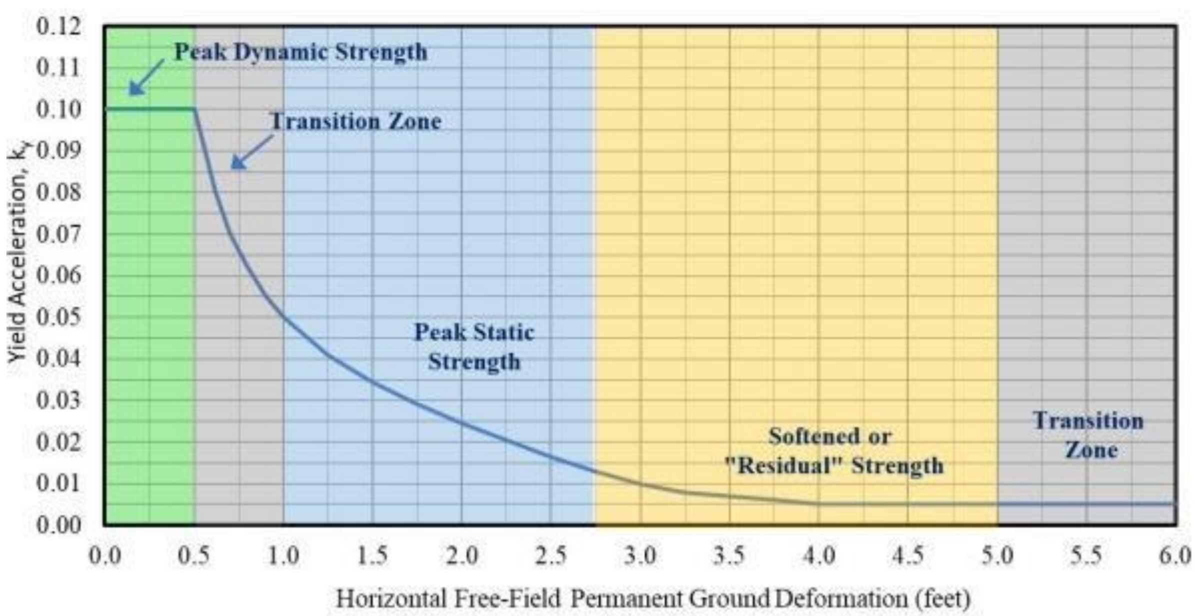

Figure 8. Yield Acceleration vs. Horizontal Free-Field Permanent Ground Deformation.

Strain-Dependent Seismic Slope Stability Analysis: Pseudo-static slope stability analyses of Subsurface Cross Section A-A', generally depicted on Figure 2, were performed using the computer program SLIDE and Spencer's method of analysis to evaluate seismic stability (i.e., compute the yield acceleration, ky, the horizontal seismic coefficient corresponding to a slope stability factor of safety of 1.0). The strain-dependent strength models were developed based on relationships for the ratio of post-cyclic to static undrained shear strength as a function of maximum cyclic shear strain defined considering the results presented on Figure 6. The strain-defined undrained shear strength ratios, shown on Figure 7, were used to simulate the variation in strength considering rate of loading effects and cyclic degradation for a range of maximum cyclic shear strains or estimated slope deformations (referred to as horizontal permanent Peak Ground Displacement, PGDh). A residual shear strength ratio of 0.1 was used to model the postliquefaction strength of the potentially liquefiable sandy fill to account for liquefaction potential at all considered seismic hazard levels.

Limit equilibrium analysis in SLIDE was used to define the trend of ky with strain-dependent shear strength for a range of maximum cyclic shear strains. Neglecting any effect of pile pinning, we computed ky values for circular and block failure surfaces and selected the critical value. Based on the results of the limit equilibrium analyses, we developed a trend of yield acceleration with permanent slope displacement as shown in Figure 8. This was used as key input for our Newmark analyses performed to evaluate seismically-induced PGDh. In order to estimate the cyclic and post-cyclic shearing resistance of the YBM, the accumulated slope deformation computed at each time-step in the ground motion was divided by the 10-foot thickness of the YBM underlying the rock dike to provide the time-dependent shear strain during cyclic loading. This approximation is based on a simplified assumption that the shear strains are uniformly mobilized throughout the YBM under the rock dike. This approximation is supported by observations made of an instrumented test fill that was loaded to static failure in deposits of YBM at the Port of San Francisco (Taylor and Buchignani 1972).

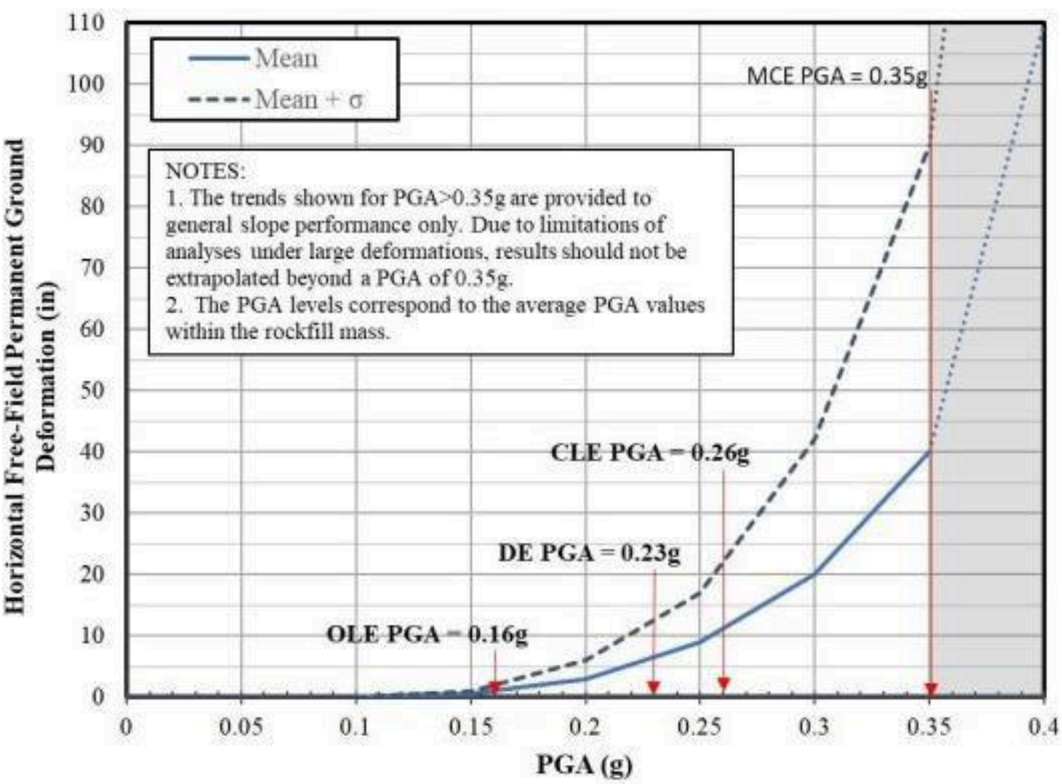

Figure 9. Estimated Earthquake-Induced Horizontal Free-Field Permanent Ground Deformation.

Seismically-Induced Ground Deformation: Estimates of seismically-induced PGDh were provided for the waterfront and rock fill slope at the location of the proposed pier and float. The program SLAMMER (Jibson et al. 2013) was used to perform Newmark (sliding-block) analyses. The program is designed to facilitate the analysis of permanent slope deformation using a user- defined relationship for ky as a function of slope deformation and selected acceleration time histories scaled to be representative of the design-level ground motions at the depth of interest. Two suites of acceleration time histories were used in the analyses; (i) 6 pairs of horizontal motions obtained from our site-specific site response analyses within the rock fill and (ii) an additional 15 pairs of ground motions, selected based on similar characteristics that are important to sliding- block deformation analyses (e.g., significant duration and Arias Intensity) and scaled to the representative PGA.

The fully-coupled method of analysis was used for the rockfill slope and incorporated the deformation-dependent ky trend, shown on Figure 9, into the program SLAMMER based on the results of seismic slope stability analyses summarized above. Estimates of PGDh based on trends of median, mean, and mean plus one standard deviation (mean + σ) were developed as part of the analyses. The trends of mean and mean + σ PGDh versus PGA are provided in Figure 9 to demonstrate the variability in these estimates due to the characteristics of the ground motions. This includes PGDh values for the OLE, CLE, and DE seismic hazard levels. Although the project design criteria do not require an evaluation of earthquake-induced PGDh considering MCE ground motions, the trends shown below include estimates considering the MCE PGA for reference. It is worth noting that the mean and mean + σ estimates of PGDh are considered reasonable estimates based on observed displacements along portions of the waterfront following the 1989 Mw 6.9 Loma Prieta Earthquake and Mw 8.0 San Francisco Earthquake.

KINEMATIC LOAD CONSIDERATIONS ON STRUCTURAL RESPONSE

The effects of kinematic loading from seismically-induced PGDh on the piles were key considerations for the seismic response of the pier structure. The estimates of PGDh developed were incorporated into the structural models developed by the structural engineer to evaluate this dynamic soil-foundation-structure-interaction.

Kinematic Loading on Pier Piles: The proposed pier piles were designed for the effects of kinematic loading from seismically-induced PGDh. While the seat of the movement is within the relatively weak YBM, the overlying rock fill, seawall, and hydraulically-placed sandy fill will move with the upper portion of the YBM. To evaluate this, various magnitudes of PGDh were applied to the piles corresponding to the selected seismic hazard levels (OLE, CLE, and DE) shown in Figure 9. P-y soil springs were used to model the PGDh in the structural modeling as part of the structural engineer's assessment of impacts on structural response.

Kinematic loading can occur both during and after strong earthquake ground motions. Simultaneous application of inertial loading and kinematic loading needs to consider phasing and location of the load application, per ASCE 61-14. The peak inertial loading and peak kinematic loading of a pile generally do not occur at the same time or at the same location along the pile. The structural design of the proposed pier piles considered the simultaneous application of these loading conditions.

CONCLUDING REMARKS

The seismic performance evaluation of the proposed waterfront fire station and fire boat mooring highlighted the importance of characterizing the static, cyclic, and post-cyclic behavior of deposits of the soft Young Bay Mud at the site. The primary practical insights gained on this project included the following.

1. Having flexibility when drilling in difficult subsurface conditions (rock fill) in order to obtain high-quality samples of the underlying clay for laboratory testing.

2. Conducting comprehensive laboratory testing to assess cyclic strength degradation and static strain-dependent strength for soils such as YBM.
3. The significant influence of permanent ground deformation on yield accelerations used in practice-oriented Newmark analysis to assess slope deformation due to cyclic strength degradation, earthquake duration, and strain-dependent soil strengths.

The in-depth characterization of the cyclic behavior of the soft clay on the seismic design of pile foundations for the pier and float structures provides considerations for practitioners addressing similar situations involving pier and wharf design in regions of high seismicity. Structural aspects of the assessment are presented in a companion paper by Soderberg and Liu as part of the Proceedings of the Ports '19 Conference.

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Investigation and Initial Stability Analysis of a Wharf on Severely Deteriorated Steel H-Piles

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ABSTRACT

The wharf at the Fore River Shipyard was built for shipbuilding circa 1960; recently, it was home to the USS Salem, a permanently-moored retired U.S. Navy heavy cruiser that served as a museum. True to its intended purpose, the wharf is a robust structure comprised of a thick reinforced concrete superstructure supported by many steel H-piles. In 2013, the project team performed a condition assessment of the wharf and discovered that many of the H-piles were significantly deteriorated, some with full thickness section loss at the low-tide elevation. This discovery forced the team to change the investigative approach for fear of collapse, only collecting as much data as necessary to understand the stability of the wharf. Meanwhile, the team performed three-dimensional, non-linear analyses that consider non-uniform pile deterioration to assess the likelihood of collapse and possible collapse mechanisms.

INTRODUCTION

On 7 November 2012, we set off to brave an oncoming nor'easter to get a first look at the condition of the steel piles supporting the reinforced concrete wharf at the Fore River Shipyard in Quincy, MA. The investigation team had just completed an assessment of the adjacent bulkhead wall that moved outward 18 in. because its steel battered piles were severely corroded. Bethlehem Steel built both structures around 1960 and used them for shipbuilding at their facility in the Fore River Shipyard. The wharf was originally designed to support large cranes, including a 1,200-ton rail-mounted portal crane for heavy-lift operations, so the concrete superstructure is very strong and stiff, and the substructure is redundant with many steel H-piles.

At the time of the inspection, the wharf superstructure was used for vehicular parking and as a Type IV permanent mooring (UFC 4-159-03, 2016) for the USS Salem, a non-operational, former US Navy heavy cruiser that has been converted into a museum. The USS Salem has since been moved to another location in the Fore River Shipyard. The ship hosted large groups of students, scouts, paranormal groups, and many other groups on extended overnight stays. With the safety of the ship's occupants in mind, along with the poor condition of the adjacent, failing retaining wall and the impending hurricane season, the owner of the wharf asked the engineering assessment team to investigate the condition of the wharf and recommend repairs.

Once our investigation began, we discovered that many of the steel H-piles were severely corroded. We shifted our focus to understanding the stability of the wharf and risk of its collapse, while mitigating the hazards posed to the engineer-divers collecting measurements under the wharf. This paper describes the investigation, from the planning stages through the initial analysis of the stability of the wharf, and outlines the importance of having an investigative plan, constantly evaluating the validity of the initial plan, and making changes to the plan as data are collected and as safety warrants. The paper also describes the simplified non-linear failure analysis performed to quickly evaluate the potential for collapse of the wharf.

DESCRIPTION OF SITE AND STRUCTURE

When the USS Salem was present, it was oriented east/west; such that the bow of the ship faced west. The USS Salem was located to the south of the wharf. The only access to the ship was a gangway located near its stern. The wharf and USS Salem are shown in Figure 1.

Figure 1: Overall View of USS Salem and Wharf (© 2019 Pictometry Reproduced with Permission)

The wharf is approximately 710 ft long by 45 ft wide. The concrete superstructure of the wharf consists of a slab, which spans north-south, supported by beams, which span east-west. The beam ends are supported on concrete bent caps. Each bent cap is supported on seven vertical steel piles (Rows A-G). There are 40 bents, each spaced at approximately 18 ft on center (Bents 1-40). The northern edge of the wharf is supported on a battered pile bulkhead wall. The bent caps are integral with the cap beam of the bulkhead wall.

Attached to the southern edge of the wharf slab and bent caps is a fender system that consists of steel frames with wooden rubstrips. Steel mooring bollards are located approximately at every fourth bent along the southern edge of the wharf. Mooring ropes and chains from the USS Salem were tied to several of these bollards.

The southern and eastern edges of the wharf are open to the Fore River. The wharf is located near the mouth of the river at the Massachusetts Bay. The tidal fluctuation is about 10 ft. The mean high water elevation is near the bottom of the concrete beams and the mean lower low water elevation is about three feet below the bottom of the concrete bents exposing the top of the steel H-piles within the tidal zone.

An active cathodic protection system is connected to the vertical piles, battered piles, and sheet piling of the wharf and bulkhead wall structures. The system was turned off and not functioning.

ORIGINAL DESIGN DOCUMENTATION

An archive document, Commonwealth of Massachusetts Form WD 54 No. 4208, dated 27 April 1959, was found that authorized Bethlehem Steel Company to construct a pier, sheet pile bulkhead, and mooring basin in Weymouth Fore River at its property in Quincy. This document includes a written description of the work and drawings depicting the basic geometry and layout of the wharf and bulkhead wall. The written description and drawings indicate:

- Overall plan dimensions of the structure.
- Battered pile size, HP14x89.
- Vertical pile size, HP 14x73.
- Design mudline elevations; 10 ft below mean lower low water at the northern edge of the wharf; 25 ft below mean lower low water at the southern edge of the wharf.

The design documents do not include the pile embedment depth into soil, reinforcement layout of concrete members, soil properties, steel grade, concrete mix design, design intent, and design loads.

INITIAL VISUAL INSPECTION OF PILES

During an initial boat-by on 7 November 2012, the team observed substantial build-up of corrosion by-products on the steel but did not observe major holes or very thin sections, even at locations where we removed corrosion by-product. Unfortunately, the water elevation was just above low-water during the boat-by, and the most deteriorated portions of the steel piles were under the water surface. We did not observe any other signs of deterioration, such as displacement of the wharf or bowing of the H-piles. We knew that the wharf needed repairs, but we assumed that it was not on the verge of collapse. The team then began planning an investigation of the wharf, which began in the spring of 2013.

INITIAL INVESTIGATIVE PLAN

We designed an investigative plan to answer the following questions: *What repairs/remediation do the wharf and integral bulkhead wall need to maintain current operations, and what service life is remaining?* Answering these questions involved the following steps:

- Above and underwater inspection of the H-piles to collect measurements at multiple locations along the height of many vertical and battered piles.
- Physical testing of steel reinforcement, steel sheet piling, and concrete samples to determine material properties.
- Deterioration mapping of the concrete superstructure from the topside and underside.
- Sample openings and ground-penetrating radar scanning to understand the size, spacing, and cover over reinforcing bars.
- Petrographic examination of concrete samples.
- Geotechnical borings performed on land and from a barge.
- Geophysical testing of soil samples from borings.
- Environmental geo-probes and soil testing.
- Groundwater elevation monitoring.
- Sheet piling depth testing using seismic, magnetic, and radar methods.
- Bathymetric survey.

- Mooring survey.

The goal of the H-pile survey was to estimate of the remaining service life of the H-piles. We determined the number of H-pile measurements in accordance with the Naval Civil Engineering Laboratory (NCEL, 1988) technical note titled "Sampling Criteria and Procedures for Underwater Inspection of Waterfront Facilities," which is sponsored by the Naval Facilities Engineering Command (NAVFAC) and ASCE Underwater Investigations: Standard Practice Manual (ASCE MOP 101, 2001). We initially planned for a five percent sample to achieve a desired level of confidence in the expected service life prediction. We then planned to evaluate if an additional five percent sample of measurements was required to achieve the desired level of confidence.

The marine investigation included a bathymetric survey of the river bottom, selective sample openings taken from the sheet pile wall at four locations along the length of the wall, and an above and below water investigation of the sheet pile wall, battered piles and vertical piles.

A Level I inspection (visual/swim-by) on all the components was performed by the engineer-dive team to confirm as-built structural plans and detect obvious major damage or deterioration due to overstress, severe corrosion, or extensive biological growth and attack. At selectively random locations, Level II inspections to remove marine growth and corrosion by-product and Level III investigations to collect ultrasonic steel thickness measurement locations of the piles as well as sample openings of the concrete were completed. Locations were randomly selected from specific regions of the structure to gain a representative sample of the entire structure. For example, the four sample openings were randomly selected within each quadrant of the wharf bulkhead wall. Level II/III survey including ultrasonic thickness measurements at five locations along the height of the steel piles and sheet piles was planned to be performed at sixteen locations or roughly every 50 ft. Thickness profiles including ultrasonic thickness measurements at 1 ft increments along the height of the sheet piles, steel battered piles, and vertical piles were planned to be performed at four locations or roughly 200 ft intervals. Profiles were completed prior to the Level II measurements because the profiles give a better understanding of where the locally high and low levels of section loss occurred on the structure. This information was then used to calibrate the ultrasonic thickness measurements within each of the five measurement regions: below the mudline (hand excavate up to 18 in.), mudline, low water, splash zone, and the location of maximum anticipated stress under typical design conditions. Both web and flange thicknesses would be measured at each height. The engineer-divers measured thicknesses to identify the greatest section loss at a location and thus allow for a conservative estimate for the remaining service life.

The concrete investigation included visual survey of the top and bottom side of the wharf, sounding of the top and bottom side of the wharf in representative areas, GPR scanning of the top and bottom side of the wharf, and destructive openings in the beams, slabs, pile caps, and girders. The GPR scans were used to identify reinforcing bar locations and spacing. The destructive openings were necessary to confirm the bar locations and sizes, calibrate the GPR data, and to observe the condition of the bars. Core samples were taken for both compression testing and petrographic analysis. The petrographic analysis is used to determine the expected service life of the concrete portions of the structure. Obvious areas of concrete deterioration from freeze thaw were noted during visual surveys of the top side of the wharf. Petrographic cores were taken at the perimeter of locations with obvious deterioration to determine if deterioration extended beyond the visually obvious areas. A few samples were also taken at locations with little to no observed deterioration to serve as a point of reference.