Case Study

Ground Improvement Design and Construction for Seattle’s Elliott Bay Seawall Replacement and Retrofit

William Perkins\textsuperscript{1}\textsuperscript{*} and Andrew Malinak\textsuperscript{2}

Abstract: The Elliott Bay Seawall in Seattle, Washington, was constructed in the early 1900s over soft/loose non-engineered and liquefaction susceptible fill, estuary, and beach deposits. The fill includes wood from historic waterfront sawmills and debris from the 1889 Great Seattle Fire. After the 2001 Nisqually earthquake, an evaluation of the seawall condition and seismic vulnerability determined that it had undergone significant deterioration and was susceptible to collapse for a 100-year earthquake. This evaluation led to design and replacement/retrofit of 1,130 meters (3,700 feet) of seawall. The new seawall includes an improved soil mass constructed of a cellular arrangement of jet-grout columns that supports a seawall superstructure and provides all seismic lateral restraint. The improved soil mass seismic performance criteria are based on allowable seawall displacement for three earthquake ground motion levels. Final improved soil mass design utilized non-linear dynamic soil-structure interaction analyses. To meet performance criteria, improved soil mass widths range between 7.9 and 18.3 meters (26 and 60 feet), ground improvement area replacement ranges between 50 and 64 percent, and jet-grout soil-cement unconfined compressive strength ranges between 0.86 and 2.76 MPa (125 and 400 psi), depending on the soil type. Improved soil mass construction issues included equipment selection, limited space, spoils handling, wood debris, and obstructions (e.g., buried utilities, piles, and temporary shoring). Lessons learned included: (1) jet grouting was the best construction method given the utilities and thousands of piles beneath the site, (2) early obstructions identification and contingency plans are critical to maintain production, and (3) an understanding of space requirements for all construction activities is required for safe and productive working conditions.

Keywords: ground improvement, jet grouting

Introduction

Seattle’s Elliott Bay Seawall protects the 2.4 kilometers (1½ miles) of downtown waterfront between South Washington and Broad streets. The seawall supports the urban waterfront and infrastructure, including the Alaskan Way Viaduct and Alaskan Way, access to historic waterfront piers, numerous essential buried utilities, and provides lateral confinement for structures founded in the non-engineered fill prevalent behind the wall. Most of the original seawall was constructed between 1916 and 1936. Located up to 152 meters (500 feet) seaward of the original shoreline, the seawall retains predominantly loose, liquefaction-susceptible, non-engineered fills.

Because of the effects of the 2001 Nisqually earthquake on the waterfront, the City of Seattle (City) embarked on an evaluation of the existing seawall, including a condition and seismic vulnerability assessment. This evaluation determined that the seawall and the infrastructure it supports were at significant risk from even a moderate seismic event. This conclusion led to design and construction of nearly 975 meters (3,200 feet) of replacement seawall and retrofit of an additional 152 meters (500 feet). Several seawall replacement/retrofit designs were considered. The final design utilizes an improved soil mass (ISM). The ISM supports a seawall superstructure and provides all seismic lateral restraint. The ISM consists of cellular arrangements of jet-grouted soil-cement columns. Jet grouting was selected for ISM construction because of the variable soil conditions, debris, thousands of potential obstructions posed by the 8,300 original seawall piles and other undocumented trestle piles, and physical constraints imposed by the dense urban waterfront environs. With approximately 6,000 soil-cement columns up to 28.3 meters (93 feet) deep with a total volume of approximately 133,800 cubic meters (175,000 cubic yards), the Elliott Bay Seawall Project is one of the largest jet-grout projects ever constructed. The City provided jet-grout plans and specifications to ground improvement subcontractors and ultimately
awarded the subcontract to Hayward Baker, Inc., who installed ~6,000 jet-grout columns in three construction seasons. Column consistency and quality was exhibited through early demonstration programs and throughout construction by unconfined compressive strength tests and coring.

This paper briefly describes the original seawall, the glacial and post-glacial geologic and subsurface units in which it was constructed, and an assessment of the original seawall. The paper then presents the ISM design, including performance-based criteria and non-linear time-history analyses. It then discusses construction aspects, including installation, QA/QC, and spoils management. Lastly, this paper presents design and construction considerations, challenges, and lessons learned.

**Project Background**

Geology and Soils

The downtown Seattle Waterfront is located on Elliott Bay at the end of the Duwamish River delta. Troost et al., 2005, map the area as filled tidal flats. The bay and tide flats are located within a several-hundred-foot-deep glacial trough that was carved into pre-existing, glacially overridden soils. During the most recent glacial episode in central Puget Lowland (Vashon Stage of Fraser Glaciation), about 16,500 to 13,500 years ago, the ice is estimated to have been about 900 meters (3,000 feet) thick in the Seattle area, resulting in all prior soil deposits being highly overconsolidated (Porter and Swanson, 1988; Booth, 1987). When the ice sheet receded about 13,500 years ago, it left a landscape sculpted into a series of north-trending ridges and troughs with a relief of several hundred feet. With the retreat of the ice sheet, sea level rose into the Elliott Bay/Duwamish River trough, reaching equilibrium at present levels about 5,000 years ago. The trough has been subsequently filled with Holocene beach, estuarine, and alluvial sediments deposited as sea level rose and as the Duwamish River delta advanced northward in the trough toward Elliott Bay. Beneath the seawall, the depth to the glacially overridden soils that form the trough ranges between about 9.1 and 27.4 meters (30 and 90 feet). These soils are typically very dense or hard and are over 900 meters (3,000 feet) thick (Yount et al., 1985).

Beach deposits overlie the Pleistocene glacially overridden soils. The beach deposits mantle the sides of the trough, rising in elevation as sea level rose within the trough. The beach deposits range from loose to very dense sand with varying silt and gravel content, and range between about 0.0 and 10.7 meters (0 and 35) feet thick beneath the seawall. Scattered Holocene landslide and reworked glacial deposits are also present directly on the glacially overridden Pleistocene soils.

Estuarine soils typically overlie the beach deposits and were deposited in the relatively quiet waters of the Duwamish River estuary and Elliott Bay. Beneath the seawall, these estuarine sediments range from very soft silty clay to loose to medium dense silty fine sand, and range between 0.0 and 4.6 meters (0 and 15) feet thick.

Non-engineered fill overlies the beach and estuarine deposits. Much of the fill behind the seawall is a sand and gravel backfill placed hydraulically during seawall construction in the 1930s. Fill placed prior to the 1930s includes debris from historic waterfront sawmills (sawdust, wood chips, mill ends), debris from the 1889 Great Seattle Fire, and other non-engineered fill and debris. The fill thickness ranges between 7.6 and 15.2 meters (25 and 50 feet).

Original Seawall

Most of the original seawall north of Madison Street was built in 1934 and 1935 and consisted of a pile-supported timber relieving platform with a front concrete face panel and steel sheet. The pile-supported relieving platform walls were designated as Type A and B, depending on the width of the platform (12.8 meters [42 feet] for Type A, 18.9 meters [62 feet] for Type B) and exposed wall height (typically 6.1 to 15.2 meters [20 to 25 feet] for Type A, up to 12.2 meters [40 feet] for Type B). The original design drawings wall plan and cross section for the Type B wall are shown in Figure 1. The maximum pile spacings supporting the platforms are between 0.8 and 1.6 meters (2.7 and 5.4 feet), resulting in a forest of several thousand piles beneath these walls.

The 365.8 meters (1,200 feet) of original seawall south of Madison Street consisted primarily of two types: a 1915 timber-pile-supported, unreinforced concrete gravity wall; and a 1964/1987 pile-supported, reinforced concrete sidewalk frame.

As a result of seawall movements during the 2001 magnitude 6.8 Nisqually earthquake, the City commissioned a study of the existing relieving platform seawall condition and seismic vulnerability. To evaluate the existing seawall condition, seven test pits were excavated to depths of up to 5.5 meters (18 feet) to visually inspect the platform, pile tops, and connections to the concrete face panels. Severe to significant marine borer damage of the timber platform beams at their connection to the concrete face panel was observed in three of the seven test pits. The damage was typically within about 1.5 meters (5 feet) of the connection, and the severe damage was characterized by near complete deterioration of the 30.5×40.6 cm (12”×16”) timber cab beam (e.g., original 30.5×40.6 cm [12”×16”] cap beam cross section eaten away to 6.4×7.6 cm [2½”×3”]) and complete deterioration of the overlying 10.2×30.5 cm (4”×12”) platform planks. To extrapolate the test pit observations along the entire 2,042 meters (6,700 feet) of relieving platform, 104 geoprobe soundings were made. The soundings targeted the connection between the timber platform and the concrete face panel. The results of the geoprobe soundings coupled with the test pit observations suggested that about 50 to 60 percent of the platform connection had significant deterioration and 15 to 20 percent was severely deteriorated. The liquefaction susceptibility and impact on the existing seawall was also evaluated in this study. It was determined that the ground shaking and extent of liquefaction for ground motions with a 50 percent probability of exceedance in 50 years...
(108-year return period) would likely result in large sections of the wall failing.

**Seawall Design**

With an understanding of the original seawall condition and vulnerability, the City commissioned design of a replacement seawall for the most vulnerable 1,130 meters (3,700 feet), from South Washington Street to about 91 meters (300 feet) north of Pine Street. Earth pressure from earthquake-induced liquefaction was the controlling load case for global stability of the seawall. For this loading case, structural, ground improvement, and hybrid structure/ground improvement options were explored. Ground improvement was selected to serve as the primary means to mitigate liquefaction effects because of issues with post-earthquake inspection, condition assessment, and repair of structural options (e.g., drilled shafts, tie-backs).
along the waterfront. While several ground improvement methods were considered by different design teams and peer reviewers, jet grouting was selected because of the congested urban environment and the virtual forest of timber piles from the original seawall and network of historic trestles.

The selected ground improvement or ISM consists of a cellular arrangement of jet-grouted soil-cement columns that extends vertically from about 3.0 to 4.6 meters (10 to 15 feet) below the street level (typically the bottom of the original seawall relieving platform) down to the top of the glacially overridden Pleistocene soils. The cellular structure consists of a seaward ISM front panel, landward back panel, and a connecting center panel constructed of overlapping jet-grout columns. The panels and overlapping jet-grout columns are shown schematically in cross section and plan in Figure 2. The top of the completed ISM is shown in Figure 3.

The ISM was designed to restrain the unimproved liquefiable Holocene soils behind the wall and provide vertical support to the seawall superstructure. The superstructure includes a concrete support slab cast directly on top of the ISM, precast seawall face panels, precast cantilever Zee beams placed on top of the support slab and over the face panels, precast light-penetrating sidewalk panels placed on the cantilevers, and other cast-in-place and precast elements to tie these together. The seawall ISM and superstructure are shown in Figure 4.

**Performance Criteria**

The overall seawall performance objectives and explicit ISM displacement and stress criteria were defined for three seismic ground motion levels and are summarized in Table 1. The ISM design criteria were developed to ensure that the ISM failure mode would be sliding along the base; toppling or shear through the ISM would be unacceptable.

To improve the marine environment and migration of juvenile salmon along the downtown waterfront, the face of the original seawall was moved landward up to 4.3 meters (14 feet), and a shallow water habitat bench was constructed in front of the new seawall face (“Fish Habitat” on Figure 3). The seawall superstructure Zee beams were used to cantilever the light-penetrating sidewalk over the habitat bench to the existing piers. Consequently, the ISM also had to have sufficient shear strength at its seaward edge to support the concentrated vertical loads from the cantilevered sidewalk.

**Method**

Because of the performance-based nature of the criteria, the ISM design was based primarily on finite difference, two-dimensional dynamic soil-structure interaction (DSSI) time-history analyses. These analyses were used to develop the ISM widths, soil-cement strength, and area replacement ratio. FLAC (Itasca Consulting Group, 2010) was used to perform the DSSI analyses to explicitly model the liquefaction behavior of the soil and its interaction with the ISM.

An important aspect in meeting the seismic performance criteria is the selection of earthquake time histories used in the DSSI analyses. Earthquake time histories used for 35 percent design were developed in 2004 and were matched to uniform hazard spectra (UHS). However, at this site, the UHS has significant contributions from widely different seismic
sources, and time histories match to the UHS at this site are not representative of an actual earthquake from any one of the seismic sources. Consequently, time histories match to the UHS may yield unrealistic and overly conservative results. For final design in 2012-2013, this issue was addressed by using conditional mean spectra (CMS) (Baker, 2011) for RE and MCE ground motions. Seismic sources that contribute significantly to the seismic ground motions at the site include the Cascadia Subduction Zone interplate and intraplate and local shallow crustal faults such as the Seattle Fault (located about ¼ mile south of the seawall). CMS were developed for each of these sources and conditioned at periods of 0.1, 0.5, and 3.0 seconds. A suite of seven ground motions representative of the different earthquake sources were then selected and modified to be compatible with their respective target CMS. The number of earthquakes representative of a specific

<table>
<thead>
<tr>
<th>Performance Criteria</th>
<th>Expected Earthquake (EE)</th>
<th>Rare Earthquake (RE)</th>
<th>Maximum Considered Earthquake (MCE)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50% probability of exceedance in 75 years (~108-year return period)</td>
<td>7% probability of exceedance in 75 years (~1,000-year return period)</td>
<td>3% probability of exceedance in 75 years (~2,500-year return period)</td>
</tr>
<tr>
<td>Overall Seawall Performance</td>
<td>• Safe for public use and no service disruption. • Elastic response. • Easily repairable cosmetic damage.</td>
<td>• Serviceability maintained. • Some permanent displacement and repair acceptable.</td>
<td>• No collapse. • Life safety maintained. • Large displacements acceptable.</td>
</tr>
<tr>
<td>ISM Horizontal Displacement</td>
<td>&lt;15 cm (&lt;½ foot)</td>
<td>&lt;30 to 45 cm (&lt;1 to 1½ feet)</td>
<td>No criterion</td>
</tr>
<tr>
<td>ISM Rotation</td>
<td>&lt;0.5 degrees</td>
<td>&lt;2.0 degrees</td>
<td>No overturning</td>
</tr>
<tr>
<td>Peak Shear Stress in ISM Soil-Cement</td>
<td>Always, &lt;Soil-cement shear strength</td>
<td>Typically, &lt; Soil-cement shear strength</td>
<td>No shear failure through the ISM</td>
</tr>
</tbody>
</table>
source and matched to a given CMS was proportional to the hazard posed by that source at the CMS conditioning period. The ISM width-to-height ratio to meet design requirements at 35 percent design with the UHS-compatible time histories was 0.95; for final design with the CMS-compatible time histories, the ratio was reduced to 0.7. This reduction in height to width ratio resulted in about a 26 percent reduction in the improved ground volume and reduced ground improvement cost.

Static, three-dimensional pushover-type analyses of individual ISM cells were also used to evaluate the sensitivity of the ISM to imperfections in the cellular structure. For example, an existing timber pile could inadvertently create a zone of unimproved soil or "shadow zone" within a jet-grout column, cause an undersized column, and/or be encapsulated within a row of jet-grout columns (i.e., become a timber pile inclusion). These three-dimensional analyses ensured the cellular structure was sufficiently redundant so that overall the ISM performed as intended.

The seawall superstructure was designed as a gravity structure sitting atop the ISM with the support slab and soil above sized to counterbalance the cantilevered sidewalk and pedestrian and vehicle traffic on the sidewalk. The superstructure elements were designed to be deformation compatible with the ISM, to resist seismic loads imposed on the elements from inertia and adjacent soils, and to undergo acceptable inelastic behavior under RE ground motion levels.

Results
The designed ISM width ranged between approximately 7.9 and 18.3 meters (26 and 60 feet), with the width varying proportionally to the thickness of the Holocene soils (i.e., the thicker the Holocene soils, the wider the ISM). The percent of soils within the ISM required to be improved varied between 50 and 64 percent to provide sufficient shear resistance. Because of the numerous piles from the original seawall, the overlying utilities, and minimum width of the seaward side of the cell to support the seawall superstructure, the City’s geotechnical engineering consultant prepared design drawings of the location and diameter of each of the approximately 6,000 jet-grout columns. Because of the close timber pile spacings of the original seawall, the design column diameters ranged between 0.9 and 1.8 meters (3 and 6 feet); larger columns would not be as efficient due to pile shadowing.

The average design unconfined compressive strength (UCS) for the jet grout improved ground was 2.76 MPa (400 psi) in the granular fills and 0.86 MPa (125 psi) in the fine-grained estuarine and organic (sawdust) deposits. To achieve these strengths, the project specifications required that at least 50 percent of the tested granular soils have a UCS of 3.03 MPa (440 psi) or greater with a minimum of 90 percent greater than 1.10 MPa (160 psi); for the fine-grained soils, the requirements were 50 percent greater than 1.10 MPa (160 psi) with a minimum of 90 percent greater than 0.41 MPa (60 psi). Spatial requirements for calculating UCS percentages were also included in the specifications to ensure that the required strengths were achieved throughout the ISM.

Extensive wood debris is present over an approximately 107-meter (350-foot)-long section of the seawall face near Yesler Way (the location of Yesler’s historic sawmill). The strength and bearing capacity of the jet-grouted sawdust, wood chips, and debris for the new seawall superstructure was a concern in these materials. To address this concern, 0.9-meter (3-foot)-diameter drilled shafts were installed 2.4 meters (8 feet) on center along the seaward edge of the ISM in that area to provide additional vertical support. The shafts were not extended beyond the base of the ISM to ensure that the ISM failure mode would remain sliding along its base.

At the northern 152 meters (500 feet) of the project, the existing mudline met the elevation criterion for the habitat bench, and a light-penetrating sidewalk could be incorporated into the reconstruction of the adjacent pier. Consequently, it was not necessary to move the front of the seawall back from its original location, so to save costs, the existing seawall was retrofitted. The vertical support for the existing seawall concrete face panel was provided by moving the seaward edge of the ISM directly beneath the existing seawall face panel. Steel shear pins were installed through the heel of the face panels into the underlying ISM front panel to tie the face panel to the ISM and provide lateral restraint. To reduce lateral seismic earth pressures on the seawall face panels, the existing fill above the ISM (i.e., above the left-in-place relieving platform) was improved via permeation grouting. Permeation grouting extended from immediately behind the concrete face panel landward 3.0 meters (10 feet) and was designed with a minimum UCS of 0.86 MPa (125 psi).

Seawall Construction
The remainder of this paper discusses the process by which the years of study and design addressed above were turned into the final product.

Bid Process
At bid time, potential ground improvement contractors were presented with a plans-and-specifications jet-grout project, complete with a jet-grout ISM design, a set of specifications, volumes of geotechnical data, and additional requirements for public works contracting in the City. The estimated value of the specified work was large enough to attract five competitive jet-grout contractors from across North America.

By the time the jet-grout bid package was released, the general contractor/construction manager (GC/CM) had already been selected. Jet-grout bids were solicited by the GC/CM.

The City of Seattle and the GC/CM were able to provide clear explanations and thoughtful answers to the questions asked, which allowed contractors to provide increasingly responsive bids. The bid questions and answers, which were incorporated into the contract documents by addenda, formed a record of assumptions and expectations held by the various parties at bid time, and in many cases set the baseline as situations arose during construction.

Pricing of the work came down to a few major items. The largest and perhaps most obvious was how to safely construct
jet-grout columns meeting the specification requirements as quickly as possible. This meant maximizing the amount of time a jet-grout rig is constructing jet-grout columns. At the bid stage, the jet-grout contractor selected to perform the work planned to use equipment capable of drilling to full depth in a single stroke to avoid delays from adding and subtracting drill rods and to predrill holes to minimize set-up time between locations and time getting to depth.

The final bid included a base bid for installing jet grout, plus allowances for abandoning holes due to obstructions, added columns, additional testing, and deletion of spoils-handling scope, as well as a score for guaranteed use of disadvantaged businesses.

**Early Fieldwork and Testing**

**Bench-Scale Testing**

Prior to beginning work, HBI drilled additional investigation borings to collect soils for bench-scale testing. This process was conducted with soils from the southern portion of the site, including existing fill, estuarine and beach deposits, and fills containing high organics (e.g., sawdust, mill ends). Each was lab tested with various binder types and dosing rates to reduce risk of the ISM not attaining the specified strengths. Through this process, it was identified that a 90/10 cement/slag blend would produce the desired results across all anticipated soil types along the seawall alignment.

**Demonstration Program**

From the time the jet-grouting work began on site, there was limited time to do upfront testing and verification while maintaining the overall project schedule. As a result, a true test program was not conducted. Where a test program would typically compare several parameters, including pull rates, rotations, grout flows, and air pressures to find the most efficient way to create a column of a desired size, a demonstration program is used to prove a single set of parameters that are typically more conservative.

On this project, four demonstration programs were conducted. One series of tests was conducted for each major soil type planned for jet-grout improvement. The demonstration programs were used to prove that the contractor-selected jet-grouting parameters would result in adequate column diameters.

For each demonstration program, a series of three overlapping columns was installed for each diameter anticipated. Season 1 work included both full-diameter and half-sector demonstration columns to prove parameters used for both methods of construction.

Once columns were constructed, cores were drilled at the column intersections and 75 percent of the radius to verify diameters.

**Project Sequencing and Schedule**

The project schedule and sequence were set in large part by the GC/CM before bid time. Work was sequenced into three construction seasons and subdivided spatially into ten boxes.

Season 1 began in summer 2014 and consisted of the southernmost portion of the project south of Yesler Street’s busy Colman Dock ferry entry. Season 1 included the area with the highest amount of organic soils as well as portions of the historic 1915 gravity wall and 1964/1987 pile-supported sidewalk frame that were to remain in place and largely covered by an offshore intertidal marine habit (i.e., beach) to be built offshore in a later project. This area was completed first to precede construction of the State Route 99 tunnel that passes beneath this area.

Season 2 stretched from Yesler Street to the Seattle Aquarium and consisted of about 80 percent of the project. Beginning in October 2014, the work coincided with the tourist off season. This allowed for the majority of the adjacent piers to be closed, something necessary to allow construction to progress more efficiently. During the height of Season 2, HBI employed nearly 100 people on site working around the clock with four jet-grout rigs and two batch plants to meet the project schedule.

By June 2015, work was complete to Madison Street. The remaining stretch of work, about two city blocks, was sequenced between July 2015 and February 2016. This area included working around and beneath an active pedestrian overpass, sequenced work at the main Colman Dock ferry exit, and working around a 4.6-meter (15-foot)-deep combined sewer outfall structure (see Figure 5).

Later in Spring 2016, HBI mobilized back to the front door of the Seattle Aquarium to work north in Boxes 2 and 3. A final mobilization was completed in Spring 2017 to complete the northernmost box and in advance of the future redevelopment of the adjacent Pier 62/63 by Seattle Parks. Jet grouting concluded in August 2017 after three years on site.

**Equipment Selection**

Jet grouting is a specialty geotechnical construction technique that involves a significant amount of specialty and custom equipment. To meet the project demands, HBI pulled resources from across the country and deployed new equip-

Figure 5. Crews installing jet grout ISM in Nov. 2015
ment as needed. This equipment included drills, pumps, and batching equipment.

As previously stated, to maximize the amount of time the jet-grout drill rigs were jetting, they needed to be large enough to drill up to 29.3 meters (96 feet) in some areas, without stopping to add drill rods. They also needed to be capable of jetting 180-degree sector columns and be wired for recording and control by the computerized control and data acquisition systems required by the specification. HBI used a variety of jet-grout drills on this project, each selected for their ability to meet the above requirements and to meet other access constraints of the site as encountered.

For batching, HBI employed both a traditional silo-type batching system and a trailer-mounted pneumatic system. On many projects, cement is batched from vertical silos because of the ease of operation and lower costs to run such a system. However, with space at a premium on this urban project, one of the Season 2 batch plants had to be set up beneath the Alaskan Way Viaduct. For this, HBI used modified oilfield equipment to move and batch cement horizontally.

HBI also used a fleet of non-specialty drills to complete this work, including drills to install casing (discussed below), drills to pre-drill to depth in advance of jetting, and coring rigs to verify production columns.

**Challenges and Obstructions**

As with most construction projects, minimizing cost and schedule were important. For the jet-grouting scope, the best way to accomplish these objectives was to install jet grout as quickly as possible. This was, however, complicated by several challenges, including known and unknown obstructions, a narrow linear work area created by the excavated portion of the site, numerous utilities, and overall site access constraints.

After a quick review of the site history and current configuration – a historic trestled working waterfront turned urban corridor – it came as no surprise to anyone that any underground work at this project would encounter obstructions of some sort, e.g., timber, utilities, or otherwise. Each of these are described briefly below.

**Timber**

Timber piles were known to exist and were a key factor in the selection of jet grout for ground improvement on this project. Nonetheless, the exact location of each timber pile was unknown, and HBI had to construct each jet-grout column as designed, regardless of whether timber was encountered or not.

Drilling around, not through, the timber piles was required not only to reduce drilling time and equipment wear, but in order to construct the ISM as designed.

With that in mind, when it was possible to map the timber piles, the project team did just that. During the excavation to remove the existing timber-relieving platform, the timber piles were exposed for a brief period of time. During that time, HBI was able to conduct a sample survey of timber pile tops, most of which were not only visible, but were clearly still solid trunks of old-growth timber (see Figure 6). With this survey data in hand, HBI and the City’s geotechnical engineering consultant were able to compare the assumed timber pile locations to the actual surveyed location at several locations along the seawall alignment. Where the assumed locations differed from the actual, a simple shift in the jet-grout column layout was a very easy change to make at this stage. With the as-built pile locations provided by HBI, the engineer was able to modify the jet-grout layout such that piles were encapsulated within the ISM with minimal shadowing (see Figure 7).

**Utilities**

Alaskan Way is a part of Seattle’s urban downtown. Unsurprisingly, a large number of utilities exist below the site. Most notable among them is a 115kV electrical duct bank and associated vaults. Less significant, but still very important, were a multitude of services connecting the piers to land, such as sewer, water, gas, steam, and communications.

![Figure 6. As-built surveying of historic timber piles](image1)

![Figure 7. Timber piles encapsulated by jet-grout columns in ISM front panel. Piles sheared off during preparation of ISM surface with no observed damage to ISM](image2)
Location, protection, and monitoring of utilities throughout the jet-grouting process was one of the major challenges on the project and are detailed later in this paper.

Upland vs. Excavation

For the majority of the project, the site was split in two: one area where a shoring wall protecting a duct bank created an at-grade upland area with another area to the west where a 3-meter (10-foot) excavation created a trench in which the jet-grout work platform was built. This excavation allowed removal of all belowground utilities in the ISM zone, but created some additional challenges, including access for crews and equipment, access for the public to adjacent piers, and a constrained space in which a large amount of construction was performed.

Access and Working Platform

A variety of different jet-grout rigs were used, typically weighing between 23 and 27 metric tons. Sequencing the various phases of construction meant construction of ramps to get rigs in and out of the excavation. With the space constraints of the project, there was no ramp that could be built without interfering with jet-grout column locations, which meant ramps had to be built and removed multiple times in some work areas.

While the old Alaskan Way had no shortage of bumps and potholes, the 20.3-cm (8-inch)-thick concrete roadway had no trouble supporting a jet-grout drill rig. Below the surface was another story. The top 7.6+ meters (25+ feet) of soil was fill (mostly non-engineered) placed over 80 years ago. During mass excavation, once the relieving platform was removed, a working platform consisting of about 1.5 meters (5 feet) of road-base-type material was placed to provide both a stable level working platform and a separation between the top of the jet-grout column and the workers and equipment constructing the columns. This separation is needed to prevent high-velocity grout from spraying along the ground surface, creating safety and environmental challenges.

At various times, the GC/CM chose to use quarry spill for the working platform. Despite the material’s cheaper cost, this created some new issues. First, HBI found that some spills would fall into the hole during drilling and jetting, causing more drilling challenges. Second, jet-grout spills, upon returning to the surface, would find this very permeable layer and migrate laterally across the work area, making spills containment very difficult. Finally, after jet grouting was completed, the excavation contractor found that removing the working platform was very difficult, as it had hardened into a cemented pad.

Spoils Handling

The jet-grout process creates spoils. As a grout stream is introduced into the subsurface and hydraulically mixed with the in situ soils, some of the grout/soil mixture returns to the surface. For every gallon that goes in the ground, one must come out. Handling spoils is easier when there is ample room and time. A trench and pit are often all that are needed to allow the spoils to cure and solidify. Once solidified, the spoils can be handled with an excavator. However, at this project site, the working platform was only about 15.2 meters (50 feet) wide and sandwiched between a shoring wall and Elliott Bay. In other places, work was performed from a concrete road surface with no place to dig a trench. Consequently, there was no room to let spoils cure overnight. Spoils were a major challenge and something that required careful planning to prevent environmental accidents. Since there was not space on site for spoils to cure, they were transported off site while still wet. HBI used a fleet of vacuum trucks to move wet spoils from the drill rig to a transfer facility about a 20-minute drive from the site. From there, the facility operator provided a pit to cure the spoils and arranged for final disposal at a lined landfill.

Protect-in-Place (PIP) Corridors

At four locations over the course of the project, utilities crossed the excavation to maintain services to piers still in operation. To do this, the GC/CM installed PIP corridors, a steel lattice fitted with various water, electrical, gas, and communication services. However, these overhead structures created a challenge for jet grouting as they were often too low to walk a person under, and thus it was not possible to jet grout directly beneath them.

To work around the PIP structures, jet grout was installed from the road surface at the future location of each PIP. Once excavated, jet grouting could proceed freely on either side. Once complete within an area, ramps were constructed to move drills out of the excavation and around the PIP structures.

Overhead Obstructions

In addition to the PIP structures, the Marion Street pedestrian overpass presented a logistical challenge. The pedestrian overpass connects pedestrians arriving at Colman Dock from the Washington State Ferries to downtown and is packed with commuters during the morning and evening rush hours. The GC/CM could close the bridge only when ferries were not running, or between the hours of about 1 a.m. and 5 a.m. With these restrictions, the GC/CM first replaced the 1950s-era steel bridge with a light-weight aluminium-truss bridge. HBI installed jet-grout columns in the area near the bridge during the day, and at night the bridge was disconnected and moved aside by crane to allow access for HBI’s equipment. The bridge was reconnected a few hours later to allow the morning rush hour commuters to cross.

Utilities

The Seattle waterfront is an urban environment and thus has a significant amount of utilities. Running along the length of the seawall alignment is a 115kV underground electrical duct bank. To its west, where most of the jet grouting took place, was a temporary shoring wall (required to construct the seawall superstructure) and excavation in which utilities were
removed prior to jet grouting. To the east was the remaining portion (approximately 20 percent) of the jet-grout work. In this area, described as the upland area, jet grouting was done from the road surface around existing live utilities. To work safely around the live utilities and to protect both employees and utilities, HBI and the project team worked to locate, protect, and monitor the utilities.

Locate
On paper, a live utility map was generated by the project team, including as-builts and survey data for the various known utilities. This was used by the City’s geotechnical engineering consultant to create the jet-grout column layout, aiming to avoid known utilities. This drawing was also reviewed before each work shift with crews working around live utilities.

At each location in the upland area, the road surface first had to be removed. Initially, the road surface, consisting of about 20.3 cm (8 inches) of lightly reinforced concrete, was cored for drill access. HBI quickly realized that a small core would not provide flexibility for relocation of jet-grout columns should utilities or obstructions be encountered and changed the plan to saw cut and remove trenches along the work area.

From there, HBI undertook extensive vacuum excavation to locate the known utilities in the work area. In some cases, they were found where expected, and in many cases, they were not. Where conflicts existed, surveys were performed in the field to provide to the engineer for redesign. At each jet grout location in the upland area, vacuum excavation to a minimum of 1.8 meters (6 feet) or the bottom of the nearest adjacent located utility was completed to check for unknown utilities or unknown lateral connections.

Protect
Once vacuum excavation was complete, a polyvinyl chloride (PVC) sleeve was installed at the jet-grout location through the top of the relieving platform. The PVC casing served two purposes: first to act as a utility-free conduit for the jet-grout rig to work in, and second to protect adjacent utilities from jet-grout spoils.

Once the PVC was installed through the relieving platform, it was grouted in place to create a seal and prevent the casing from moving later on.

Monitor
When jet grouting began, utilities were monitored for movement. Jet grout can create settlement or heave if not installed carefully. With surveyors in place at critical points such as the surface of utility vault lids or deep monitoring points installed on duct banks and sewer lines, HBI was able to stop jetting if movement in excess of project-specified alert levels was observed. Some surface movement was observed, especially in parts of the area with high wood contents or stiff clays, but with the monitoring plan in place and good communication, the project team was able to avoid damage to utilities.

Verification
The final verification program included coring soil-cement columns, examining cores for zones of untreated soil, and UCS testing of retrieved samples; and collection of in situ wet samples once per shift per rig using a downhole sampler at three depths, placement into cylinders, and UCS testing of the cured cylinder samples. This type of field verification is a standard part of any jet-grouting program.

In addition to rock coring, sonic coring of select columns was also accomplished, with chemical testing of the core to evaluate the effectiveness of the soil mixing and column diameter. While the sonic core samples are disturbed and are not suitable for UCS testing, however, the cores are continuous and allow an assessment of column continuity.

To verify target column geometries were achieved, cores were drilled at column intersections at a rate of 3/100 columns. Core drilling was by both traditional rotary rock drilling and sonic drilling.

Coring of young soil-cement by rotary methods can be difficult, especially when gravels are present in the weak grout matrix. To improve chances of recovery, longer cure times are recommended, but this may mean delays to follow-on work.

Constructability Lessons
Throughout the project, as different challenges arose, it remained abundantly clear that jet grouting was the right technique for this project. Whether working in and around the thousands of driven piles, or trying to delicately weave between the labyrinthine utilities, no other geotechnical construction technique could have matched jet grout’s versatility.

There were many lessons learned throughout the construction of this project and some areas that could have been improved. First, had additional lead time been available, a true test program may have allowed for efficiencies and increased production rates.

Early identification of obstructions was essential to keeping the project running smoothly and a lesson learned many times throughout the project. From identifying the location of timber piles to uncovering stray and misplaced utilities to recognizing conflicts with temporary works, having a plan to rapidly handle changes to the jet-grout design without slowing production was a must.

As with many urban construction projects, space on the Elliott Bay Seawall site was scarce. With limited space and limited time, multiple geotechnical construction contractors with very large equipment were squeezed into a narrow space, potentially creating an unsafe and unproductive environment. To solve this, an understanding by all involved with the project on the space-intensive geotechnical construction requirements is a must for a successful project.

Finally, visiting a site where jet grout being used is highly encouraged for owners, general contractors, and design engineers to get a sense of the challenges jet grouting can pose and simple ways to avoid relearning the same lessons.
Acknowledgements

The authors appreciate the opportunity to work on this interesting and significant project and gratefully acknowledge the Seattle Department of Transportation (owner), Parsons Corp. (design lead), COWI North America, Inc. and Exceltech Consulting Inc. (seawall structural design), Mortenson/Manson JV (GC/CM), and other design and construction team members’ extensive work and contributions to the successful design and completion of this project.

The material presented in this manuscript has been published in part as conference proceedings by the Vancouver Geotechnical Society. The content has been modified and expanded to meet the requirements of the Journal. The project was executed while the second author worked as Project Manager for Hayward Baker, (2010-2019). Hayward Baker worked as one of the main contractors on the project. The author affiliations on Page 1 reflect the current employer.

References


