

DFI Featured Technical Article

Geotechnical Challenges of Pile Driving in a Marginally Stable Slope

TERMINAL 46 APRON UPGRADE, PORT OF SEATTLE, WA

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INTRODUCTION

Terminal 46 is located just south of downtown Seattle, and is one of the Port of Seattle's busiest container terminals. The Terminal 46 Redevelopment Project was undertaken to replace the existing 15-m-wide (50 ft) cranes along the apron with 30-m-wide (100 ft) cranes, which can unload larger, post-Panamax ships. The project included upgrading the existing waterside crane rail with new piles and constructing a new land-side rail behind the sheetpile bulkhead. The new piles had to be installed adjacent to the existing apron without causing damage, which was a significant geotechnical challenge due to the soft soil conditions at the site. An instrumentation program performed during the test pile program evaluated this challenge and allowed for the development of specific production pile installation procedures that had the least possible effect on construction costs and schedule.



Figure 1 – Photograph of site

SITE HISTORY

The site lies along the west side of Elliott Bay, north of the mouth of the Duwamish Waterway. Prior to development, this area was part of the tide flats where the Duwamish River drained into Elliott Bay. Prior to 1970, there were several existing timber piers and structures in the area. In the 1970s, the Port of Seattle constructed a 900-m-long (3,000 ft), 30-m-wide (100 ft) apron to support five, 15-m-wide (50 ft) container cranes. The contained backland area was then filled with dredged spoils (sand and silt with organics) and the upper 5 to 20 m (16 to 65 ft) backfilled with sand and gravel.

The existing apron was constructed using 0.4 m (16 in) octagonal prestressed concrete piles to support the precast concrete deck and crane rails. The pile-supported bents were spaced at 6.1 m (20 ft), center-to-center, along the longitudinal direction. Pile lengths typically varied from about 20 to 40 m (65 to 130 ft) with 15 to 26 m penetration below mudline (50 to 85 ft penetration). The slope below the apron was constructed at about 1.75 horizontal to 1 vertical to allow for sufficient water depth for cargo ships. In most areas, a riprap buttress was placed at the toe of the slope. A typical cross section of the apron is shown on Figure 2.

Several stability, settlement and pile driving issues arose during the original construction of T-46 (Pita, 1983). Reportedly, movement of the submarine slope occurred during pile driving, resulting in settlement behind the bulkhead. Pile driving reportedly caused settlement of adjacent piles. In addition, some piles were “running” (dropping quickly under one hammer blow) during installation.

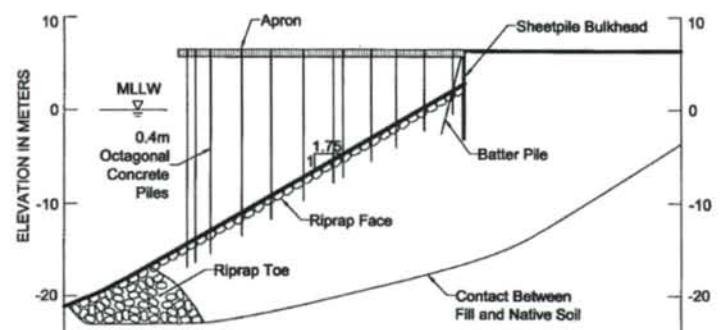


Figure 2 - Existing Apron Cross Section

Over the apron's 25-year life, additional movement of the submarine slope below the apron and settlement of the crane rails has occurred. Surveys indicate that the existing sheetpile bulkhead has lost up to 3 m (10 ft) of soil support due to settlement of the submarine slope. Crane rail surveys also indicated that settlement and lateral movement on the order of 0.3 to 0.6 m (1 to 2 ft) have occurred, primarily in areas where the existing piles do not extend to dense bearing soils.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

The site subsurface profile consists of Quaternary glacial deposits overlain by Holocene beach, estuarine and alluvial deposits of the Duwamish delta, which are, in turn, overlain by recent fill deposits. A schematic of the subsurface profile along the apron is presented in Figure 3. As shown, the depth to glacial deposits increases from about elevation -30 m (-100 ft) along the apron's northern half to elevations greater than -80 m (-260 ft) at the apron's south end. The apron foundations along the south half bear in medium dense to dense alluvial soils, and the foundations along the north half bear in very dense glacial deposits.

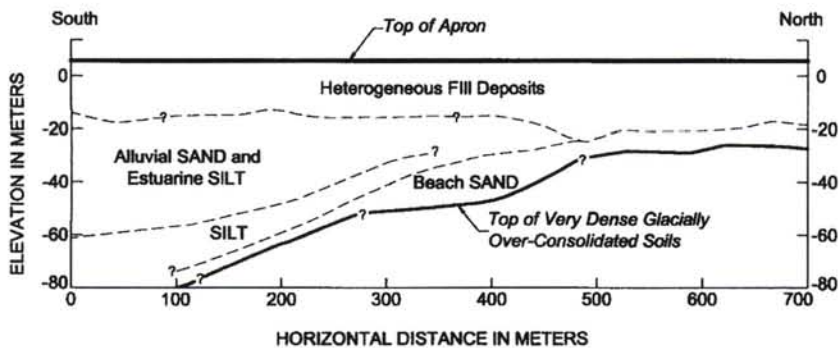


Figure 3 - Generalized subsurface profile along apron (behind bulkhead)

The glacial and beach deposits are overlain by alluvial and estuarine deposits consisting of interbedded sandy silt and silty fine sand. These materials are loose to dense, and are generally liquefiable during a design earthquake with a 475 year return period.

NEW CRANE RAIL FOUNDATION DESIGN

The structural design of the apron upgrade, performed by Moffatt & Nichol Engineers, required additional piles to support the waterside crane rail. Also, a new row of piles was required to support the land-side crane rail. Along the waterside, the number of additional piles had to be kept to a minimum to maintain clearances under the apron. For the land-side crane rail, these limitations were not required, and steel H-piles were selected for foundation support. The pile design and construction of the waterside crane rail posed the greatest geotechnical challenges in that installing new piles could impact the stability of the marginally stable submarine slope or damage the apron, bulkhead and adjacent facilities.

Selecting appropriate deep foundation elements for the crane rails was based on subsurface conditions, design loads, and the requirement that the existing apron structure be protected from damage during pile driving. For most of the Port of Seattle facilities, octagonal prestressed concrete piles are a preferred foundation pile type. This type of piles is defined as a "displacement" pile because, during driving, the soil through which the pile is being driven is displaced by the volume of the pile. Installing displacement piles can cause soil densification, pore pressure generation and vibrations. Because of the previously observed movement of the crane rails, submarine slope and existing piles during and following the original apron construction, the designers determined that there was a potential for additional lateral movement or settlement to occur during driving of the new piles. Therefore, open-ended steel pipe piles (defined as "nondisplacement" piles) were also considered. Based on the design loads, 0.6 m (24 in) octagonal prestressed concrete piles and 0.6 m (24 in) diameter, open-ended, steel pipe piles were selected for design consideration.

Analyses included evaluating axial capacity, spring constants and pile driving criteria for the new piles. The capacities of the existing apron piles were also evaluated. In many areas, the estimated factors of safety under vertical loading for the existing piles were less than 2.0 based on the original design loads. The piles with low factors-of-safety generally corresponded to areas where larger settlements had been observed. Therefore, the design approach was to accommodate all of the additional load of the new cranes on the new piles.

The static capacity analyses indicated that, along the southern half of the apron, the new waterside piles would obtain most of their resistance from side friction, and lengths for octagonal prestressed concrete piles would have to be on the order of 55 m (180 ft) to achieve the required design capacity of 2,135 kN (240 t). For the open-ended, steel pipe piles, longer lengths would be required due to lower skin friction, less displacement during driving and lower end bearing. Along the northern half of the apron, the new piles would bear in very dense glacial soils and therefore would obtain significant support from end bearing, resulting in pile lengths on the order of 35 to 40 m (115 to 130 ft).

INDICATOR PILE/INSTRUMENTATION PROGRAM

The design team chose to undertake an indicator pile program for the waterside rail along southern half of the apron prior to the award of the construction contract to obtain information for selecting pile types and lengths, and evaluate construction methods. An instrumentation program was performed in conjunction with the indicator pile program to evaluate the following:

- The effect of pile driving on the existing apron and submarine slope (i.e., settlement and lateral movement).
 - Displacement piles versus non-displacement piles
 - Single pile versus a “cluster” of three piles
- The handling of a 55-m-long (180 ft) concrete pile versus splicing two shorter sections of a concrete pile.
- The drivability of the different piles under the existing subsurface conditions.
- Possibility of tension cracking in concrete piles when driven through soft soil.
- The ultimate axial capacities and load distribution of concrete versus steel piles at various depth intervals.
- The pile drivability, noise and vibration experienced during pile driving when using a hydraulic hammer versus a diesel hammer.

The indicator pile program included driving six, 0.6 m (24 in) octagonal prestressed concrete piles and two, 0.6 m (24-in) diameter, open-ended, steel pipe piles. Three of the concrete piles were installed in a “cluster,” at a spacing approximating the proposed production pile spacing of 1.8 m (6 ft), center-to-center, to evaluate the effect of driving piles in close time and proximity on the existing apron and submarine slope.

The instrumentation program consisted of surface surveys to monitor potential apron movement, inclinometers to measure lateral movement, probe extensometers to measure subsurface settlement, and vibrating wire piezometers (VWPs) to monitor pore pressure generation and dissipation during and after pile driving. The instruments were installed in borings at various distances from the indicator piles. VWPs were also installed inside the concrete piles to monitor pore pressures at the soil/pile interface.

ANALYSES AND RESULTS OF TEST PROGRAM

The data collected was analyzed to evaluate pile capacity, load distribution, pile driving stresses, hammer performance, pile handling and the effect of pile driving on the existing apron and slope.

Dynamic testing using a Pile Driving Analyzer® (PDA) was performed on all of the indicator piles during continuous driving and restrike. The dynamic measurements were evaluated based on a closed-form solution of the wave equation for piles using a program called Case Pile Wave Analysis Program, or CAPWAP® (Rausche et al., 1972). The results of the analyses indicated that the 0.6 m (24 in) octagonal prestressed concrete piles would have to penetrate at least 34 m (110 ft) below the mudline (total pile length of 52 m or 170 ft) to achieve the required design load (2,135 kN or 240 t) within the alluvial sand soils. The results also indicated that, for the 0.6-m-diameter, open-ended, steel pipe piles, a total penetration of about 44 m (145 ft) would be required (total pile length of 64 m or 210 ft). These results were consistent with lengths estimated by static capacity analyses for these pile types.

The indicator pile program was also directed towards evaluating whether or not local contractors could handle 55-m-long, (180 ft) concrete piles. The selected contractor was capable of handling, lifting, and positioning the long piles. The indicator pile installed with a splice was also done without difficulty. Four pick-up points were used to lift each of the long concrete piles as shown on the photo in Figure 4.



Figure 4 - Handling of 55-m-long concrete pile

One of the primary concerns for driving the proposed octagonal concrete piles at this site was the potential for apron settlement and submarine slope movement. The location with the highest potential for being affected by pile driving would be at the site of the octagonal concrete pile cluster. Three octagonal concrete piles were consecutively installed at 1.8 m (6 ft) centers. The area with the least potential for being affected by pile driving was expected to be at the site where an open-ended (nondisplacement) steel pipe pile would be driven. The instrumentation program was implemented at these two locations (near the cluster and near an open-ended pipe pile). Settlement, lateral movement, and pore pressures were monitored and analyzed as follows:

- **Settlement:** Settlement was monitored using probe extensometers installed in boreholes, settlement plates on the submarine slope, and optical surveys of the apron deck. In the area of the pile cluster, up to 2 cm (0.8 in) of settlement occurred at the mudline, at a distance of approximately 4.6 m (15 ft) from the pile cluster. Most of this movement appears to have occurred after driving the second pile in the cluster. This observation would indicate that driving adjacent piles in succession during construction could cause settlement of the subsurface soils, depending on the speed (time interval between piles) at which adjacent piles were installed. The settlement was estimated to be due to pile driving vibrations causing settlement of the fill soils, which resulted in small movements of the submarine slope during pile driving. Loss of soil strength caused by elevated pore pressures during pile driving may have also contributed to the movement.

- Lateral Movement:** The inclinometer data indicated that no lateral movement occurred adjacent to the open-ended steel pipe pile. Adjacent to the octagonal concrete pile cluster, however, approximately 1.5 cm (0.6 in) of lateral movement occurred at a distance of about 4.6 m (15 ft) from the cluster. The movement occurred primarily in the upper sand and gravel fill soils that make up the submarine slope. This lateral movement also occurred upslope, where lateral movement of about 0.8 cm (0.3 in) was observed about 12.2 m (40 ft) from the cluster. This indicated that the movement did not extend behind the existing sheetpile bulkhead.
- Pore Pressures:** Vibrating wire piezometers (VWPs) were installed at various depths in several boreholes and also at various locations inside two of the concrete indicator piles in the cluster. These VWPs were used to monitor pore pressures in the upper alluvial sand and estuarine silt deposits during pile driving, because elevated pore pressures can result in a loss of soil strength. Figure 5 shows data obtained from VWPs inside one of the 0.6 m (24 in) octagonal concrete piles during driving of an adjacent pile 1.8 m (6 ft) away. The VWP data was used to evaluate the peak excess pore pressure generated during pile driving, and the time required for 90 % dissipation of the peak pore pressure. In general, the peak pore pressure occurred when the tip of the pile being driven was at the same elevation as the VWP. As the pile passed the VWP elevation, the pore pressures began to dissipate. After pile driving stopped, pore pressures dissipated over time. Although there was a wide scatter in the data, it appears that the time for 90 % pore pressure dissipation in the alluvial sand soils varied from about 5 to 50 minutes, and the time for dissipation in the estuarine silt soils varied from about 50 to 100 minutes. Based on interpolating the VWP data, localized liquefaction likely occurred within about 0.9 m (3 ft) around the concrete piles when pile “running” conditions were observed.

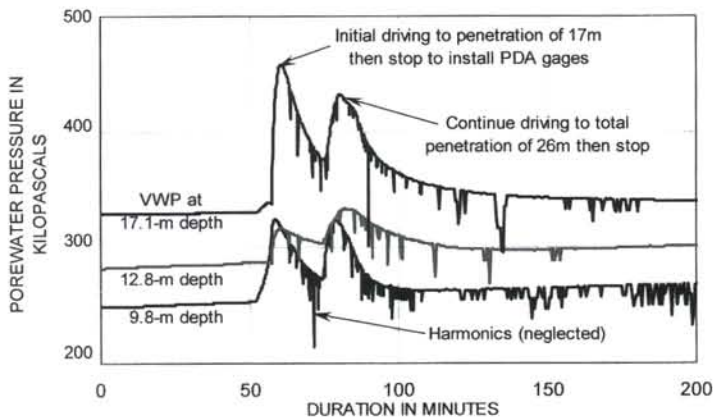


Figure 5 - Raw pore pressure measurement data

PRODUCTION PILE RECOMMENDATIONS

Based on the results of the indicator pile program, 0.6 m (24 in) octagonal prestressed concrete piles were recommended for the waterside crane rail foundations because they are more

economical than steel pipe piles. The indicator pile program showed that driving adjacent displacement piles (piles within 1.8 m or 6 ft of each other) could not be performed in immediate succession without causing movement of the adjacent submarine slope. The indicator pile driving data suggested that successive piles be spaced at least 5.5 m (18 ft) apart, or be driven with a waiting period of at least 2 hours between each pile to minimize submarine slope movements. Since these spacing and waiting period recommendations had the potential to affect construction schedule and cost, criteria were developed based on the approximate amount of slope movement anticipated. This allowed the Port of Seattle to decide what criteria to impose on the contractor depending on the magnitude of slope movement that would not damage the existing apron. These criteria are summarized as follows:

- For successive piles spaced at least 5.5 m (18 ft) apart (every third pile) or a waiting period of at least 2 hours between successive piles, estimated settlement and lateral movement would be less than 1.3 cm (0.5 in).
- For successive piles spaced about 3.7 m (12 ft) apart (every second pile) or a waiting period of at least 1 hour between successive piles, estimated settlement and lateral movement would be less than about 5 cm (2 in). The contractor selected this option.
- If adjacent piles are installed successively (at a 1.8-m or 6 ft spacing), greater than 15 cm (6 in) of movement could occur. This could cause distress to the existing apron, sheetpile bulkhead and adjacent structures.

SUMMARY AND CONCLUSIONS

During construction, the contractors installed the octagonal prestressed concrete piles at a rate of about 6 piles per day using the criteria developed from the indicator pile program. Surveys of the apron deck and monitoring of the instruments during construction indicated that less than 2 cm (0.8 in) of settlement or lateral movement occurred on the submarine slope below the apron due to production pile driving. No distress was observed on the apron deck or behind the bulkhead.

The indicator pile and instrumentation program performed for the apron upgrade allowed the Port of Seattle to install the preferred pile option (concrete piles) within budget and without causing damage to the existing apron or submarine slope. The cost for the instrumentation program (not including furnishing and installing the indicator piles) was about \$300,000. The cost for furnishing the production piles for the overall apron upgrade was approximately \$1.7 million. If non-displacement piles such as open-end steel pipe piles had been selected because of concerns over damaging the existing apron, the piles along the south half of the apron would have had to be about 50% longer, and long-term corrosion protection measures would have been required. The Port saved about \$2.5 million overall by implementing the instrumentation during driving of the test piles to develop a construction approach that would not damage the apron.