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EXPANSION AND REHABILITATION OF THE STATE FISH PIER IN GLOUCESTER, MASSACHUSETTS

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ABSTRACT

The State Fish Pier in Gloucester, Massachusetts, has been expanded and rehabilitated to provide an upgraded facility to support the local fishing industry. Expansion consisted of a new Finger Pier and solid fill extension of the existing pier. Rehabilitation consisted of replacing a deteriorated wharf with a new higher load carrying wharf. Subsurface conditions ranged from rock outcrops exposed at low tide at some locations to thick marine deposits overlying rock at other locations. Foundation support for the new Finger Pier and rehabilitated wharf consisted of concrete filled steel pipe piles, a portion of which had to be socketed into bedrock due to lack of soil overburden. Compression and tension load tests were performed to verify the pile design capacities.

KEYWORDS

wharf, pier, pipe piles, rock socket, load tests, containment dike

INTRODUCTION

The State Fish Pier in the coastal city of Gloucester, Massachusetts, is an approximately 300 ft (90 m) wide (north-south) and 850 ft (260 m) long (east-west) earth fill structure that was constructed in 1938 to serve the regional fishing industry for ship berthing as well as for fish processing facilities (Fig. 1). During 1989 through 1995, the existing pier facility underwent expansion and rehabilitation, performed in two phases, to upgrade the deteriorated Pier as a full-service facility.

The first phase (Phase I) consisted of a 280 ft (85 m) by 110 ft (34 m) maximum seaward expansion of the Pier by filling at the west end, and construction of a new 650 ft (200 m) long, 26 ft (8 m) wide, concrete filled steel pipe pile supported Finger Pier extending from the southeast corner of the extended Pier (Fig. 1). This work was completed during 1989 through 1993.

The second phase (Phase II) included demolition of the existing timber pile supported North Wharf and reconstruction of an 800 ft (240 m) long, 43 ft (13 m) wide, concrete filled steel pipe pile supported wharf in the same location (Fig. 1). Construction occurred from 1995 through 1996.

The paper presents a summary of the project geologic setting and geotechnical foundation engineering efforts to meet the design requirements for pier expansion and rehabilitation while accommodating the challenging subsurface conditions. Also included are the results of the compression and tension pile load tests which were used to develop installation criteria for the production piles.

SITE AND SUBSURFACE CONDITIONS

Existing Site Conditions

The Gloucester State Fish Pier is an approximately 850 ft (260 m) long and 300 ft (90 m) wide earth fill structure that provides berthing and fish processing facilities. The pier area was originally filled to approximately El. 14 (4.3 m) to El. 16 (4.9 m). Mean Low Water (MLW) datum. The fill side slopes are at approximately 5H:1V at the Finger Pier (south side), and...
2H:1V along the North Wharf. The topography seaward of the existing pier reflects the effect of previous dredging. Dredging along the berthing portions of the pier typically extends to El. -20 ft. However, at tide level of MLW (El. 0), small to large size rock outcrops become visible in the flatter tidal zone beyond the toe of the existing fill at the western end of the pier where no dredging has occurred and no ship berthing takes place (Fig. 1).

The North Wharf of the Gloucester State Fish Pier is located along the North Channel of Gloucester Inner Harbor and was originally constructed in 1938 when the Gloucester State Fish Pier was built (Fig. 1). The wharf is approximately 800 ft (240 m) long by 43 ft (13 m) wide. The original deck consisted of a reinforced concrete slab and beams, supported on 81 timber pile bents spaced at 10 ft (3 m) intervals. The land side of the wharf included a granite quaywall. An existing building, located adjacent to the wharf, was demolished as part of the present rehabilitation. In recent years, the allowable wharf load had been reduced such that vehicle traffic was limited, and the structure was not suitable for accommodating the fishing boats. The new wharf is capable of supporting a 300 psf (14 kPa) uniform load as well as an HS-20 vehicle loading.

Geology (Phase I)

The subsurface conditions along the 650 ft (200 m) long L-shaped Finger Pier (Fig. 1) consist of (from mudline down) recent organic deposits, silty clay, glacial till, and bedrock. The thickness of the competent soil overburden (i.e., medium stiff to stiff silty clay and dense glacial till) increases from zero to about 15 ft (4.6 m) away from the pier line. Bedrock generally consists of hard to very hard granite below a thin weathered zone. The weathered zone ranges from none to 12 ft (3.7 m). Rock quality designation (RQD) values range from 36 to 98 percent. Figure 2 is a cross-section showing a soil and rock profile at the Finger Pier along with a cross-section of the Finger Pier structure itself. Along the new Pier, not all soil deposits were encountered.

The ground surface at the southwest end, along the expanded solid fill pier portion of the solid fill pier, varies between approximately El. 14 and El. 16 with the fill sloping toward the sea at a slope of approximately 5H:1V. The original fill consists of sand, gravel, silt, building rubble, wood, brick, paper, asphalt, glass, cobbles and boulders, concrete, and granite blocks that was apparently randomly dumped. Below the fill and in the tidal zone are deposits of organic silt, silty fine sand, and glacial till. These deposits range in overall
The subsurface conditions at the site of the 800 ft (240 m) long, 43 ft (13 m) wide North Wharf are similar to the south side except that bedrock was encountered at shallower depths at the three most westerly bents of the new wharf. Subsurface conditions along the wharf consisted of (from mudline down) recent organic deposits, fine sand, organic silt, marine deposits, glacial deposits, and bedrock. Overlying the entire wharf area was a layer of riprap consisting of granite block boulders. The bedrock surface is highly erratic. Figure 3 is a cross-section showing average soil and rock conditions in the North Wharf area along with a cross-section of the new wharf structure.

DESIGN REQUIREMENTS AND SOLUTIONS

Phase I

The 110 ft (34 m) maximum by 280 ft (85 m) extension of the existing pier beyond its west end was designed as solid fill, contained by a perimeter dike (Fig. 4). Extension of the Pier by means of a pile supported deck was considered. However, the shallow depth to rock and in some cases rock outcrops would have required that many of the piles be rock socketed to provide adequate lateral support capacity. The cost of rock sockets made the pile supported deck option too expensive, thus it was not selected and a perimeter earth dike system was adopted. Once the containment dikes were constructed, filter fabric was installed, and the space behind the dike was filled with granular soil. The expanded zone was paved and currently provides access to the Finger Pier (Fig. 1). Riprap protection was provided.

The new Finger Pier structure consists of a pile-supported concrete deck and fender system. The Finger Pier was designed to be supported by 25 bents spaced at 30 ft (9 m) intervals. The bents include both vertical and battered piles with design loads of 130 kip (578 kN) compression (vertical piles), and 180 kip (800 kN) compression, 8 kip (35.5 kN) tension (battered piles). Closed-end, concrete filled, end-bearing, steel pipe piles with 14-in (356 mm) diameter, 0.5-in (13 mm) wall, and 2-in (51 mm) bottom plate were selected.
Adequate draft had to be provided for berthing of the fishing vessels along the Finger Pier. Dredging to 17 ft (5 m) below MLW was implemented within a 100 ft (30 m) width on both sides of the Finger Pier. Soil was dredged using normal procedures. Drilling and blasting was used to remove the bedrock above the dredge level.

Bedrock was present either at the finished dredge level or within a few feet of the dredge level along the 300 ft (91 m) north westerly portion of the Finger Pier. Consequently piles, in this area of limited soil overburden, had to be socketed into rock in order to provide lateral stability for the Pier against boat impact and mooring forces. Rather than the closed-end driven piles, for the 10 bents closest to the existing solid fill pier, open-end vertical pipe piles were socketed 3 ft (0.9 m) and battered piles 6 ft (1.8 m) into sound rock. A typical rock socket detail is shown in Fig. 5. Sockets were excavated with a downhole hammer. Rock socketed piles were expected to develop end fixity. However, the driven piles, due to the possible limited thickness of competent soil overburden before reaching end bearing were analyzed as having a pin connection at the tip. Special project conditions, boat impact and mooring forces along with compression and tensions loads, required that all piles be analyzed as free standing columns with respect to buckling without any allowance for lateral restraint from the soil.

In accordance with the Massachusetts State Building Code, piles designed for loads above 100 kip (50 tons, 445 kN) must be confirmed with a load test. Both compression and tension load tests were performed on a pile driven at a service pile location.

Corrosion protection was provided by coating the piles (tip to butt) with a coal tar epoxy coating. The bottom 6 ft (1.8 m) of piles designed for tension loads were not coated so as to mobilize adequate adhesion against pull out. Additional protection against corrosion in the splash zone was provided by a plastic pile jacket consisting of an 18-inch (458 mm) diameter oversized sleeve extending from the pile butt to 2 ft (0.6 m) below MLW, with the annulus between the pile and the jacket filled with concrete.
Phase II

At one side the original wharf was supported on the existing granite quaywall (Fig. 3). The quaywall consists of a concrete gravity wall with a granite block face (seaward face). It is supported on timber pile bents spaced at 5 ft (1.5 m) on center, and is anchored to a deadman located within the solid fill pier. Reuse of the quaywall for partial vertical support of the new wharf was considered but the wall could not be used because the load capacity of the piles was unknown. The quaywall, however, was evaluated and considered adequate for continued use as an earth retaining structure.

The timber piles supporting the original North Wharf were deteriorated. Although the pile may have been adequate to support the wharf, it was considered unlikely that the 50 year old piles provide adequate service life and were not considered for reuse.

Bents for support of the new wharf were spaced at 30 ft (9 m) on center and arranged to avoid conflicts with the old timber pile bents, thus minimizing the potential for obstructions. The new bents consist of five piles each; one battered pile and four vertical piles. Design pile loadings are up to 200 kip (890 kN) in compression and the land side row of vertical piles will receive tension loads of up to 8 kip (35.5 kN). Batter piles will resist a maximum load of 120 kip (534 kN) in compression. The piles are 14-in (356 mm) diameter, closed -concrete filled steel pipe. Pile wall thickness is 0.5 in (13 mm) with a 2-in (51 mm) bottom plate.

An important factor in pile selection was the highly erratic nature of the bedrock surface. For example, test borings disclosed that the bedrock surface varied by as much as 35 ft (10.5 m) over the width of the wharf. Such variable pile lengths can be readily accommodated with steel pipe piles. This pile type was selected over precast prestressed concrete piles.

Soil overburden at the west end of the North Wharf is relatively thin and consists primarily of loose to medium dense fine sands. As a result, the piles along the three western most bents (90 ft of the wharf) were socketed into bedrock as discussed for the Finger Pier.

At the remaining bents, competent soil overburden, consisting of marine and glacial soil deposits, was adequate for pile support. The piles were designed to be driven to end bearing in either the glacial deposits or to bedrock. The required 8 kip (35.5 kN) tension capacity was mobilized through the silty clay and glacial deposits. In addition, the pile penetration into the silty clay and glacial deposits provided adequate resistance to nominal horizontal loading.

Corrosion protection was provided as described for the Finger Pier.

The riprap which had been placed at the time of the original construction was dredged at the locations of the new bents prior to pile installation to avoid potential obstructions. The riprap was then replaced after pile driving.

LOAD TESTS AND CONSTRUCTION

Phase I

At the beginning of construction for the Finger Pier, three vertical piles, at selected representative locations, were driven as indicator piles to a penetration resistance determined by dynamic wave equation analyses. The piles were installed using an ICE 40S open end diesel hammer (40,000 ft-lb rated energy) to final driving resistance of 14 blows per inch for the last three inches of driving. The three indicator piles were driven to final driving resistance into the glacial deposit or weathered bedrock. A steel template was used to position the piles over open water. The piles were secured against lateral movement (swaying) during the waiting period before and between load tests.

One of the indicator piles, driven to final resistance in the glacial deposits, was then load tested; first to 360 kip (1600 kN) compression and then to 16 kip (70 kN) tension (pull out). The compression load test was conducted first followed by the tension test. The pile load test data are presented in Fig. 6 and Fig. 7 for the compression and tension tests, respectively. These data were utilized in finalizing the driving criteria for the production piles. All three indicator piles were concreted shortly after completion of the load tests.

The production piles were installed using the same driving equipment. Tension piles were driven to the required resistance and through a minimum of 8 ft (2.4 m) of competent soil overburden. Where 8 ft of competent soil overburden was not present, the pile was removed and reinstalled in a rock socket. There were approximately 6 such piles.

Rock socketed piles were installed for support of 300 ft (91 m) of the new Finger Pier. The 17.5-inch (445 mm) diameter sockets were drilled from inside a 20-inch (509 mm) diameter cased hole using a downhole hammer. Once the socket was cleaned of cuttings, it was tremie filled with high slump concrete. The open-end pipe pile was inserted into the socket. Subsequently, concrete was tremied into the remaining portion of the pipe pile.

The Pier extension and Finger Pier constructed in 1993 has performed satisfactorily through the present (June 1997).

Phase II

Prior to installing any piles at the North Wharf, the existing riprap was removed from the channel slope at the locations of the new pile bents. The relatively steep slope (2H:1V) and the soft organic deposits made riprap removal below the water level difficult.
The timber piles supporting the existing North Wharf were broken off and the embedded lengths left in place. It is believed that disturbance from pulling the piles would have jeopardized the stability of the existing underwater slope.

Rock socketed piles at the three western most bents were installed similar to those installed for Phase I. A total of 15 piles were installed open ended in drilled rock sockets.

After the rock socketed piles were installed, five vertical piles, at selected representative locations, were driven as indicator piles to a penetration resistance determined by dynamic wave equation analyses. The piles were installed using the same hammer used at the Finger Pier (ICE 40S open ended diesel hammer). A steel template was used to position the piles over open water. The piles were secured against lateral movement (swaying) during the waiting period before and between load tests.

During installation of the indicator piles, the hammer did not perform consistently, with the measured ram stroke ranging from 7 to 9 ft (2.1 to 3 m) during final driving for the five piles. This variation in ram stroke, which results in varied hammer energy, impacted both selection of a pile for load testing and determination of production pile driving criteria. As a result, the hammer was overhauled and two additional indicator piles were installed. The ram stroke and hammer speed were measured closely for the additional indicator piles. One of these indicator piles was then selected for load testing.

Although the same pile and hammer was used at the North Wharf as the Finger Pier, a second pile load test was required due to differences in geology (i.e., depth to bedrock and thickness of competent overburden soils) and maximum design load. The indicator pile which had been driven to final driving resistance in the glacial deposits was load tested first to 400 kip (1780 kN) compression and then to 20 kip (89 kN) tension (pull out). The load tests were conducted approximately three weeks after pile installation. The pile load test data are presented in Fig. 6 and Fig. 7 for the compression and tension tests, respectively. This data, together with the ram stroke measurements taken during driving of the indicator piles, was utilized in finalizing the driving criteria for the production piles.

The existing quaywall along the land side of the North Wharf is supported on timber piles, some of which are battered seaward (Fig. 3). As a result, there was a potential for interference between the new wharf piles closest to the wall and the existing battered timber piles. Since no as-built plans were available for the wall and the wall extended below MLW, any interference could not be determined prior to construction. Therefore, the Contractor was required to perform an exploratory 8-inch (203 mm) diameter probe at the location of
the piles closest to the wall. Five probes encountered obstructions and the respective piles were driven at an out-of-plane batter.

The exploration program conducted prior to construction revealed that the underlying glacial deposits contained cobbles and boulders which could, if encountered during driving, cause the pile to move out of location, damage the pile, or even prevent the pile from reaching the required depth. Six percent of the new piles encountered some form of obstruction (cobbles and boulders) in the glacial deposits during the driving. However, no additional piles were required.

The wharf has performed satisfactorily through the present (June 1997).

**CLOSURE**

The design and construction of a rehabilitation project at the waterfront is documented. The work included extension of an existing solid fill pier, construction of a new finger pier and reconstruction of a dilapidated wharf structure. The pier and wharf units are supported on concrete filled steel pipe piles. A portion of the piles had to be socketed into shallow rock where adequate soil overburden was not present to provide lateral pile stability. The remaining majority of the production piles were driven closed-end to end-bearing. Both compression and tension (pull out) pile load tests were performed to verify the driving criteria developed by dynamic wave equation analyses.

It is demonstrated that geologic conditions dictate the design and construction of the waterfront structures to a great extent. For example, the selection of pile type to be used for the project was strongly influenced by the erratic nature of the top of bedrock surface. Also, the presence of bedrock at finished dredge level resulted in the design and installation of rock socketed piles over a length of each structure.

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