INTRODUCTION

The West Dock Causeway Bridge (WDCB) is located along a 3650±m long gravel causeway, which extends into the Beaufort Sea near Prudhoe Bay, Alaska. The gravel causeway was constructed in 1975 with two levels, the lower level at +1.5 m Mean Lower Low Water (MLLW) and the upper level at +2.4 m MLLW. The existing seafloor in the breach area is -1.8 m MLLW. The main bridge is a 213-m 4-span crossing, with 24-m long box girder approach trestles at each end, for an overall length of 262 m. The main bridge spans a 198-m breach made in the causeway to accommodate fish passage and water circulation, while the trestle bridges allow multiple pipeline transitions from below and above ground onto the bridge. The bridge deck elevation is approximately +7.0 m MLLW. Figure 1 shows a typical bridge pier.

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Soil conditions consist of frozen and unfrozen silts, sands and gravels, and ice lenses. In the region down to approximately -15 m MLLW, soils were frozen due to the permafrost shadow caused by the causeway. Between -15 m and -61 m MLLW, the soils ranged from thawed to marginally frozen, and below this elevation permafrost conditions were encountered. Special pile installation techniques developed during 15 years of arctic pile driving in permafrost conditions were used to ensure reliable pile placements (Nottingham and Christopherson, 1983). Cell cofferdam construction was used to allow pile installation and later pier cap welding to piles. Piles were driven with a large diesel impact hammer.

The main span is supported by three ice breaking piers (Figure 1). The Beaufort Sea in this area freezes annually and is subject to large onshore ice advances. The piers consist of a vertical steel column extending from the pier cap to a pile cap. Each pile cap is welded to eight foundation piles installed on a 2:1 (V:H) batter.

DESIGN CONCEPTS AND CONSIDERATIONS

In order to evaluate cost, long-term performance and constructibility issues, the Owner requested evaluation
of several different foundation systems during concept design. These foundation concepts included gravity structures, steel pile bents, steel and concrete caissons, offshore oil platform technology, sheet pile cells and other similar structures. These various structures were evaluated utilizing Owner-supplied design criteria.

Due to soil conditions, design scour and ice loads, it was quickly shown that the only viable foundation solutions were drilled or driven steel piles. Available geotechnical information indicated to Peratrovich, Nottingham & Drage, Inc. (PN&D) that pile driving was possible.

As the design progressed, ice load and scour criteria became more refined. It was shown by concept design and cost analysis that large diameter vertical foundation structures to resist ice forces would be very expensive to construct. The design process determined that ice forces on foundations could be greatly reduced by using sloping piers to cause ice impacting the pier to fail in bending rather than compression. It was concluded that a series of batter piles supporting a sloped pier would be extremely stable against ice attack from any direction. Due to design accommodations with the Owner and other subconsultants, slopped piers were not used. Rather a pier cap and slender pier column were designed to be compact yet allow transfer of large ice forces to the piles. The development of this solution follows.

**Pier Final Design and Installation**

The final in-water pier design consisted of a vertical 1.83-m diameter steel pier column extending from the pier cap to a pile cap (Figure 1). Each pile cap is a fabricated plate steel weldment approximately 4.3 m in diameter, 1.5 m deep, with a mass of 32,000 kg. Each of the three pile caps were welded to eight 914-mm diameter, API 5LX-52 piles installed on a 2:1 (V:H) batter which were driven to approximately -58 m MLLW. Each pier pile has ten radial plate steel spin-fins inclined in a counter-clockwise direction at their tips for increased tension capacity (Figure 2) (Christopherson and Nottingham, 1990; Christopherson et. al., 1987; Nottingham, 1994).

The piles were tipped with outside flange cutting shoes to assist in driving. The spin-fin piles were driven with a Delmag D100-13 diesel pile hammer operating at the maximum energy rating of 400,000 N•m.

Due to operational constraints and to facilitate pile installation the gravel causeway was left in place during foundation construction. However, the pier cap bottom design elevation was -3.5 m MLLW, thus requiring a 7.3-m x 7.3-m x 6.7-m deep sheet pile cofferdam for pile driving and pier installation (Figure 3).

Initially the piles were started by over-drilling (augering) a 1-m diameter starter hole past the top permafrost layer, inserting the pile into the augered hole, and driving the pile to depth. In augering the starter hole, the auger was initially supported by a pipe half-shell inclined at a 2:1 (V:H) batter welded between the two horizontal frameworks. In the process of drilling the auger tended to wander downwards due to gravity.
Additionally, the holes filled with water and sloughed. This process created a fluidized mud, which the auger could not remove.

After a few piles had been driven, the over-drilling process was abandoned. Between the water at the bottom of the cofferdams and the covering of the top of the cofferdam, the permafrost shadow layer thawed sufficiently to allow pile driving through this layer without difficulty. The remainder of the piles were driven without starter holes.

**Method of Analysis**

The pier was designed for several load combinations consisting of dead, pipeline, live, wind and ice loads. The main load combinations considered for pier design were:

1. Dead + Pipeline + Live + Wind + Ice
2. Dead + Pipeline + Ice (as discussed below)

Load combination 1 was the more improbable loading condition and was used to design for extreme events. Load combination 2 was the more likely structural loading combination and was used for the ultimate pier analyses and for comparison to the extreme events.

The pier column was analyzed through hand calculations utilizing load case 1 and assuming the column acted as a beam-column fully fixed at the pile cap. The pier piles were designed using a finite element program. The simplified model used is shown in Figure 4. Both load cases were applied to this model.

The original model (Figure 4) was used first to determine the ultimate capacity of the pier. In this analysis, the pile tension capacity was found to control the failure. The maximum calculated ultimate horizontal ice load was 11,000 kN for the scoured condition and 13,000 kN for the no-scour condition.

To further refine the ultimate analysis, a second model was constructed. This model used spring boundary conditions along the pile lengths to simulate an elastic soil response. The spring constants were determined using the pile load test results and approximate published lateral soil spring constants. With this model, the pile tension capacity was again found to control the failure and the same maximum horizontal ice loads as the original model were obtained.

A final, detailed model (Figure 5) was constructed to analyze the stress flow through the pier. This model utilized hybrid elements (triangular and quadrilateral) for modeling plane stress, plate bending, out of plane shear and flat shell behavior. Because the behavior of the piles and the pier column had been previously analyzed, the model was constructed with a coarse mesh for the piles and the pier column. The pile cap was modeled with a finer mesh, particularly around the pile to pile cap connection.
The detailed model again showed the controlling failure mechanism to be pile tension capacity. This model also revealed that the pile cap bottom plate would experience localized yielding of the plate between the stiffeners at the pile to pile cap connection. As a result of this localized yielding, the second mode of failure was determined to be the weld between the piles and pile cap.

All models assumed failure occurred once a single foundation pile reached ultimate elastic tensile capacity. However, due to the spin fin characteristics and pier geometry, ultimate capacity of the piers is larger if more than one pile is allowed to deflect to ultimate capacity. Based on the designer’s experience, approximately twenty-percent additional lateral load capacity could be achieved prior to complete failure of the pier. Because the resulting larger displacements due to these forces could affect the serviceability of the foundation and bridge, the ultimate pier capacity was based upon the tensile capacity of one pile.

Foundation investigations

Two geotechnical investigations and three pile load uplift tests conducted on site. The two geotechnical investigations and the first two pile load uplift tests were conducted prior to construction. The last pile load uplift test was conducted on a production pile at the beginning of construction.

The geotechnical investigations were conducted by others and the pile load tests were conducted by PN&D. The first geotechnical investigation was conducted from June 28 through July 6, 1992, and consisted of four bore holes drilled to approximate depths of 21.9 to 31.4 m below the ground surface along the gravel causeway.

The general findings from this investigation were that a permafrost shadow had formed underneath the gravel causeway to a depth of approximately -17.4 m MLLW with the remaining portion down to -30.5 m MLLW consisting of unfrozen soils with occasional ice lenses. The test hole temperatures ranged from -0.84°C at the surface (+1.5 m MLLW) to -6.25°C at -3.0 m MLLW to -2.27°C at -27.4 m MLLW. The salinity varied from 9 to 75 parts per thousand (ppt), with the higher salinities occurring at the greater depths.

The first vertical pile uplift test was performed on July 30, 1993. A 610-mm diameter, 19-mm thick test pile was driven to a depth of -25.9 m MLLW and subsequently uplift tested to failure at a test load of ±930 kN.

Upon review of the data from the first pile uplift test, the Owner decided to conduct a second test which was performed on August 10, 1993. A second 610-mm diameter, 19-mm thick test pile was driven vertically in a production trestle pile location to -60 m MLLW and subsequently uplift tested to failure at a test load of ±2,000 kN. After testing it was reseated and left for incorporation into the trestle foundations.

Due to the additional depth reached during the second pile test, the Owner decided to conduct a second geotechnical investigation. On October 10, 1993 a second geotechnical investigation hole was drilled to an approximate depth of -64 m MLLW at a horizontal distance of approximately 3 m from the second pile test (production pile) location. The results from this second investigation supplemented the findings from the first investigation. There was well bonded ice between -4.0 m and -13.7 m MLLW, with poorly bonded ice continuing to -17.4 m MLLW. The profile between -17.7 m and -41.8 m MLLW consisted of unfrozen soils with occasional ice lenses. Well bonded ice began again at -42.1 m to -48.1 m MLLW. Poorly bonded ice again occurred from -48.1 m to -51.5 m MLLW with well bonded ice continuing down to end of boring, -64.3 m MLLW. The test hole temperatures ranged from -2.7°C at -14.9 m MLLW to -3.0°C at -62.6 m MLLW. The salinity varied from 6 to 24 ppt.

The information obtained from the second geotechnical investigation was then correlated with the results from the second pile load test and utilized to establish pile skin friction values and spin fin tip resistance for use during the final design of the piers.

Pier Pile Load Test

The batter pile tension test was conducted on January 31, 1994, and was performed using jacking frames, calibrated jacks and dial gauges. The jacking frame was designed by PN&D and built by the pile installation contractor in Deadhorse, Alaska.

The batter pile selected for the test was on the east side of the north pier. This pile was selected because of its anticipated exposure to the most severe ice forces. Installation of the selected test pile was completed on January 23, 1994 and the pile test was conducted on January 31, 1994. The delay in conducting the test allowed the soil to setup and dissipate most of the soil effects due to driving.

The test pile was driven to refusal with a total embedded length of 53.5 m and a final tip elevation of -47.9 m MLLW. The D-100 diesel pile hammer reached 165-170 blows/300 mm several times as the pile passed through layers of partially bonded frozen soil. The pile hammer reached 150 blows/300 mm and 33-34 blows/minute at refusal. The day prior to the test, steam lances were...
placed down the pile and the soil inside the pile was thawed, then augered out to ensure there was no freeze bonding above -15.2 m MLLW. This was done to emulate the design worst case scenario of 12.2 m of scour and maximum uplift forces.

The tension test was performed according to ASTM 3689-83, using section 7.7 “Quick Load Test Method for Individual Piles”. The test was conducted during the early evening with the ambient temperature about -22°C with an east wind of 48-56 kmph resulting in a wind chill factor of approximately -51°C. There was less than one hour of daylight. However, because the area was enclosed and heated, the ambient conditions were adequate for all of the equipment to function properly.

The tension test was conducted utilizing four 180-ton jacks activated by a single pump acting through a common manifold. Four dial gauges oriented at 90° points around the test pile were used to measure pile movement. The test load was applied in 400 kN load increments and held for 5 minutes at each increment. The test was conducted until pile failure of approximately 100 mm was observed.

Pile failure was defined as that point where constant jacking pressure (from continuous jacking) resulted in continual pile movement. This condition is contrasted to normal jacking procedure where increasing jacking pressure resulted in initial limited pile movement which quickly stabilized once jacking ceased. Load curves were developed from the jacking pressures and dial gauge measurements of movement and are presented in Figure 6.

From the load curves it was determined (constant pile load and continual pile movement) that the batter pile failed the tension test at a gross uplift (tension) force of 5,340± kN. This load matched PN&D’s pile spin fin tip resistance estimate, which was based upon the previous pile load tests and provided a factor of safety of two for design pile pullout. Due to the salinity and temperature measurements from the geotechnical holes and the short time between driving and testing, adfreeze was not a significant factor in the tension test. Based on the load curves, pile driving criteria was then developed for the remaining foundation piles and was not used in design.

**Conclusion**

The ice breaking piers developed for the WDCB bridge incorporated many innovative technological ideas such as spin-fin pile tips for increased pile pullout (tension) capacity, modular pier cap construction for reduced field installation requirements, batter pile grouping for significant lateral structure load capacity, welded pier cap construction to withstand extreme ice loads, and driven pile technology for permafrost conditions.

These foundations have many applications for proposed offshore structures. They are cost effective compared to several other offshore technology foundations evaluated, structurally redundant, and constructible.
with conventional above water equipment. Similar foundations were used in the arctic marine environment for the Endicott causeway breach (1994) and for two offshore dolphins at the Oliktok dock (1983) near Kuparuk. These designs incorporate prefabricated welded caps and driven steel batter piles. These structures have fully demonstrated their high lateral load potential under extreme conditions.

References


