

Empirical Assessment of Base Material for a Drilled Pipe Pile

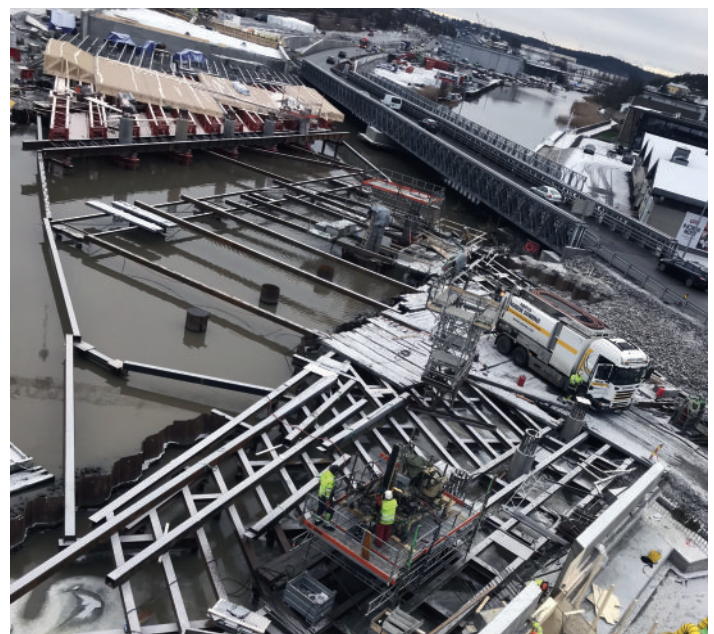


FEATURE ARTICLE

Drilled pipe piles are considered one of the common types of deep foundation used in Norway to support bridge structures. The standard installation practice for these foundation elements is the use of down-the-hole hammers (DTH). Norway's complex geology and difficult subsoil conditions have resulted in a highly variable layering of soil and rock, which, in turn, makes for differences between actual subsurface conditions and those expected from geotechnical exploratory borings. This situation is strongly noted with sloping rock layers. Special testing procedures are required to assess the base material at the tip of drilled piles and to evaluate the rock-foundation contact surface.

Project Description

The project is related to the foundation system of a vehicular bridge crossing the Seutelva River, located in Fredrikstad, Norway. According to the project information, the proposed bridge will be supported by 25 ea, 1,016 mm (40 in) diameter open-ended pipe piles advanced to the competent rock layer. Each bridge bent consists of five open-ended pipe piles, which will be filled with concrete after reaching the final base elevation. These piles were installed using a DTH hammer system, which uses the rotary-percussion drilling method with a button bit with its outer diameter being the same as the inner diameter of the pipe pile. The



Aerial view of the project site

drilling process is completed by using a first pile section with the button driving ring and a button-bit hammer fitted through the pile using a drill string.

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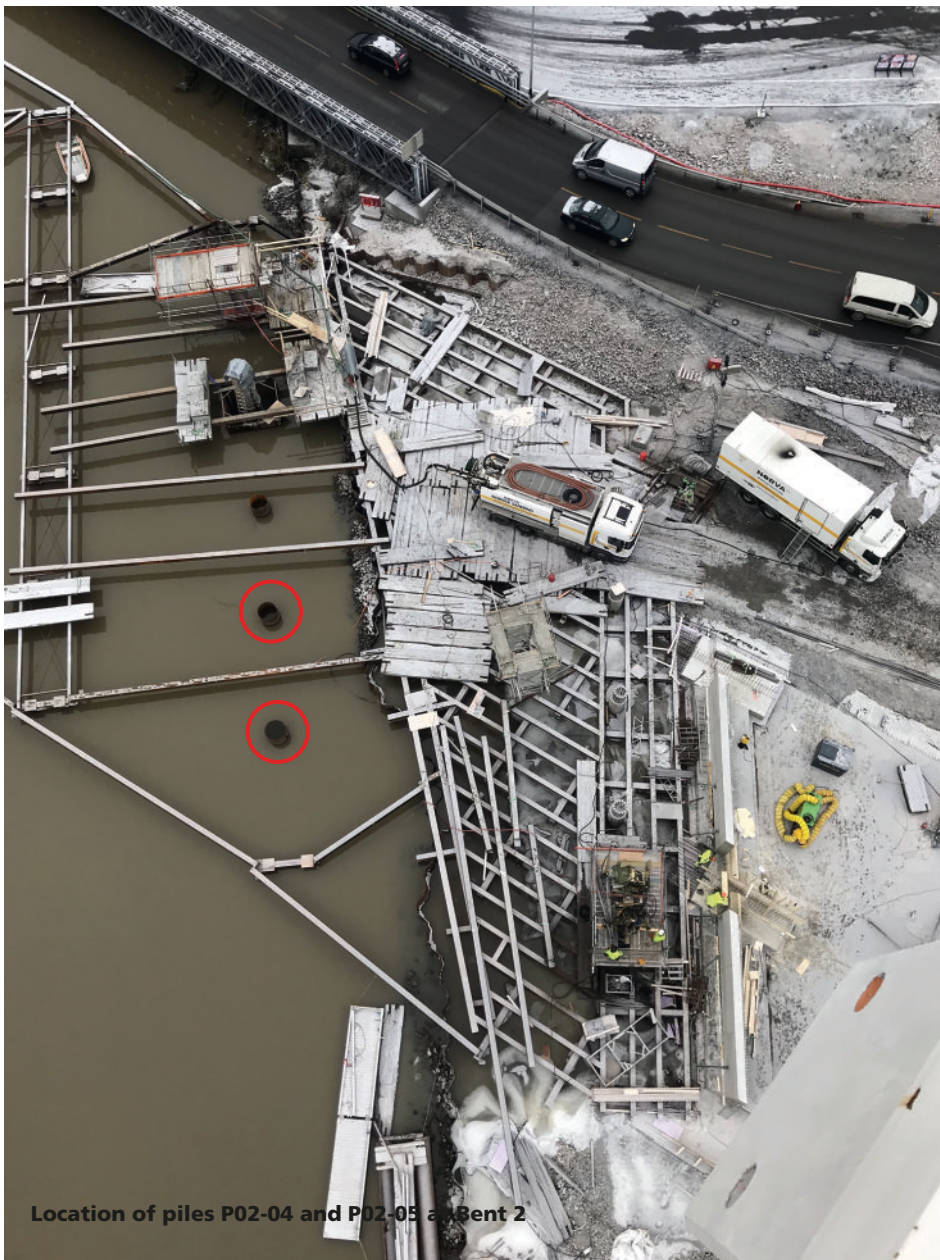
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foundation was not completely flat. Furthermore, according to the plotted results, the average stress and displacement for the material at the base of the foundation were determined to be 35 MPa (5.1 ksi) and 7 mm (0.28 in), respectively. These values were within the established resistance and displacement thresholds for the competent material at the base of the foundation.

It is important to note that FD1 plot slightly exceeds the displacement threshold limit of 8 mm (0.32 in) with a stress value of about 24 MPa (3.5 ksi) and a maximum stress of 34 MPa (5 ksi). However, the other two plots (FD2 and FD3) are within the established threshold values. Another important observation is



Attachment of the device to the end of a drill string

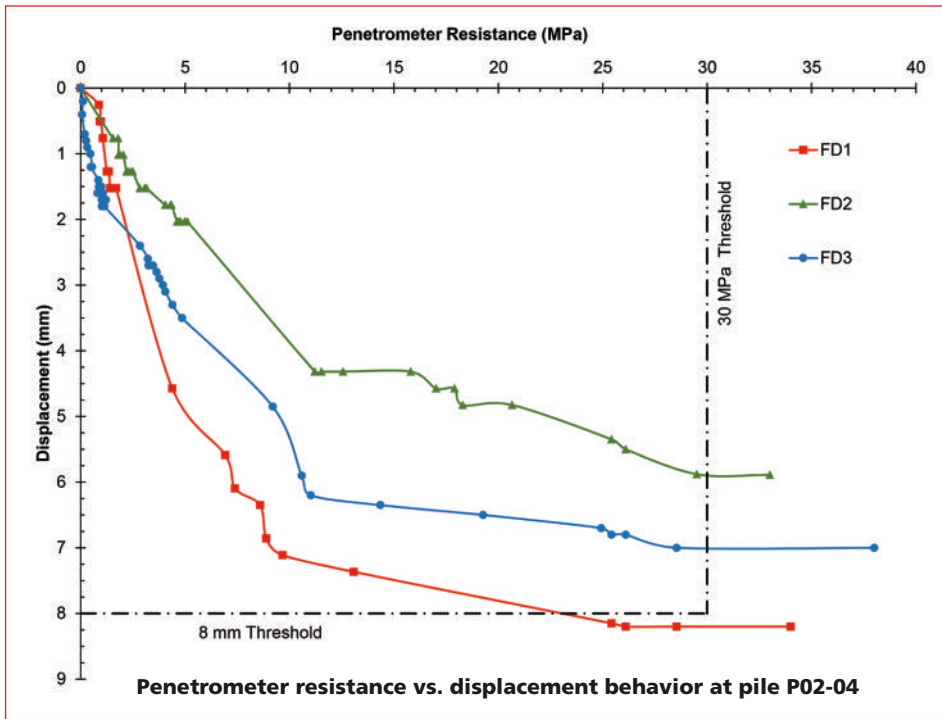


Location of piles P02-04 and P02-05 at Bent 2

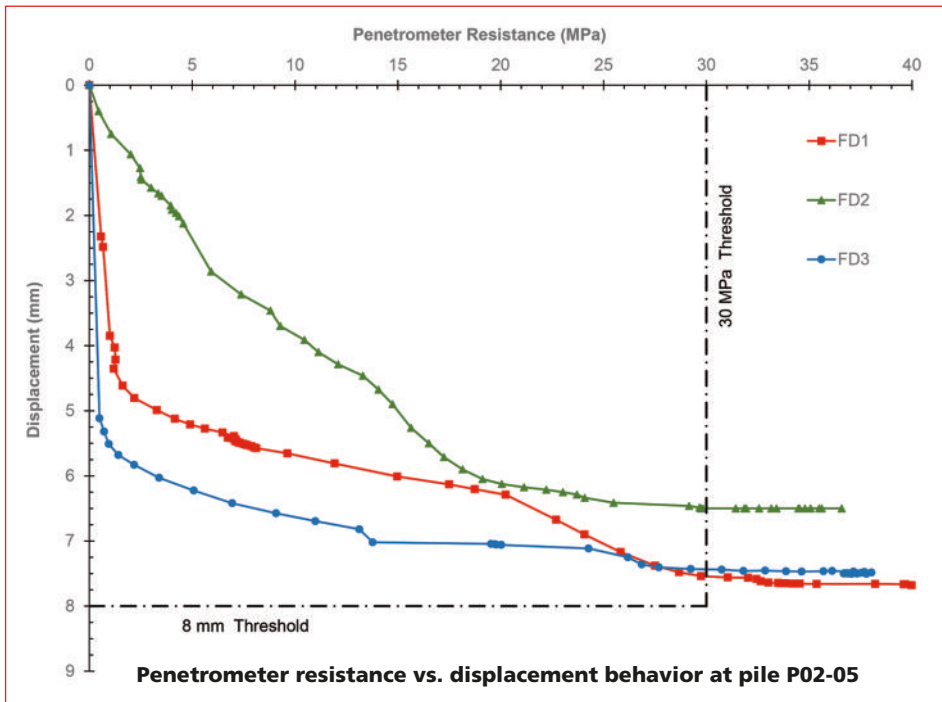
the similarity in the shape of the relationships from the start of the test up to approximately 10 MPa (1.45 ksi) where a steady increasing slope is observed. Beyond this point, the relationships transition to a more constant and gentler slope with increasing stress, which may indicate an abrupt change in material property possibly indicating going from softer rock to more competent rock. However, a visual inspection using a down-hole camera, as discussed above, did not reveal softer material accumulated near the base of the foundation. Therefore, the existence of a softer rock versus a more competent rock is more likely to be the case at this location.

Drilled Pile P02-05

Similar to those performed at pile P02-04, a total of 13 runs were performed at the base of pile P02-05 to obtain the data to determine the force-displacement relationships. The graphs of the stress-displacement relationships were analyzed to evaluate the behavior of the material encountered at the base of the foundation. The results indicated that each of the three penetrometers follow a similar trend, especially toward the end of the test. Similar to the tests at pile P02-04, a nonuniform material at the base of the foundation was detected, which was evident from the plots as the individual graphs were not overlapping. Considering the maximum stress measured by each



Penetrometer resistance vs. displacement behavior at pile P02-04



Penetrometer resistance vs. displacement behavior at pile P02-05

penetrometer and the corresponding displacements, the average stress and displacement for the material encountered at the base of the foundation were determined to be about 38 MPa (5.5 ksi) and 7.2 mm (0.28 in), respectively.

It is important to note that trend of FD2 is different compared to FD1 and FD3. The relationship for FD2 indicates a linear behavior with a constant slope up to a stress value of approximately 20 MPa (2.9 ksi) when the abrupt change of slope occurs. From this point forward, the relationship

flattens to an almost zero slope with increasing stress along with a very small change in displacement. The relationships for FD1 and FD3, on the other hand, follow a similar shape to each other and can be divided into three segments: (1) a steep slope from a stress of 0 to about 1.7 MPa (0.25 ksi), then (2) a small slope between 1.7 MPa (0.25 ksi) and 29 MPa (4.2 ksi), and then (3) a near zero slope after 29 MPa (4.2 ksi). The first portion of these plots could correspond to a very soft material accumulated at the base of the foundation,

which is then followed by a material with higher strength (e.g., weak or weathered rock). The third segment is where the abrupt change occurs similar to FD2 and the graphical relationship (similar to pile P02-04) indicates the existence of competent material or rock that satisfies the established criteria.

Conclusions

The foundation system for the project was designed for end-bearing resistance only, where all of the nominal loads were to be supported by the foundation's base. However, due to difficult subsurface conditions and sloping rock, each drilled pile was terminated at a different base elevation. One of the assessment criteria for the material at the base of the foundation was to obtain the material strength and perform the assessment based on the applied stress and measured deformations. To obtain such data, the SQUID, along with its associated testing procedure, was utilized to perform penetration tests at two pile locations, P02-04 and P02-05. Results from the testing indicated an average displacement of 7 mm (0.28 in) and 7.2 mm (0.28 in) corresponding to a stress of about 35 MPa (5.1 ksi) and 38 MPa (5.5 ksi) for Piles P02-04 and P02-05, respectively.

The resistance to penetration determined from SQUID testing is not a correlation to other strength properties defined for soil, weak rock or rock. However, in circumstances where access to the pile base is limited and a sampling process significantly impacts the construction schedule, results such as those obtained from this testing device could be qualitatively and quantitatively assessed by a qualified geotechnical engineer to further determine the suitability of the tested material.

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