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A system of micropiles installed in an A-Frame configuration along with multiple 12 strand anchors were designed and installed to stabilize the Fountain Slide located about 10 mi (16 km) northeast of Lillooet, British Columbia, Canada, within the Coastal Mountain Range. The author describes the project constraints, design methodology and construction approach used to arrest the movements of this unstable soil mass.

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The authors discuss past and current methodology for the design of drilled shafts used in transportation projects. In addition, a case study is presented that uses the results of full-scale load testing to compare predicted and measured resistances of side friction and end bearing.

89 High Mobility Grouting Prevents Historic Fountain from Sinking

Brian M. Fraley

High mobility grout (HMG) was injected beneath the historic statues, marble fountain and granite steps that were in "imminent danger" of collapsing due to open voids in the subsurface that were causing ongoing settlement of the fountain. HMG was used to fill the voids in the rubble fill and to prevent future raveling of the subbase material.

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Production shaft drilling

Predicting Geotechnical Drilled Shaft Capacity: Are We Close?

Recently modified design equations used in the transportation industry to calculate side friction and end bearing capacity for drilled shafts are now providing more realistic estimations of capacities than did previous methods. Using multiple case studies and test results from various projects, a more realistic design approach was formulated by the Federal Highway Administration (FHWA), which resulted in greater values of ultimate capacity for side friction and end bearing and in a more efficient design overall.

In 2010, the FHWA published *GEC-10 – Drilled Shafts: Construction Procedures and LRFD Design Methods*, which illustrates a different method of calculating side friction and end bearing resistance and results in greater values for design. In 2014, the Association of State Highway and Transportation Officials (AASHTO) adopted the method put forth by FHWA, which was included in its *LRFD Bridge Design Specifications, 7th Edition*. The Pennsylvania Department of Transportation (PennDOT) recently adopted and incorporated the similar methodology as AASHTO for calculating side friction and end bearing for drilled shafts in rock, and these changes are reflected in the 2015 edition of the PennDOT *Design Manual, Part 4 (DM-4)*. This article discusses the past and current design methodology along with a project case study with results from Osterberg Cell (O-cell) load testing, which presents a comparison between the design resistances of ultimate side friction and end bearing and the measured capacities at failure.

The Old and the New

Editions of the *AASHTO LRFD Bridge Design Manual* from 2012 and earlier estimated ultimate side resistance, q_s, in rock using the following equation (modified from Horvath and Kenney, 1979):

$$q_s = 0.65\alpha_E p_a (q_u/p_a)^{0.5} < 7.8 p_a (f'_c/p_a)^{0.5}$$

where: q_u is the uniaxial compressive strength of the rock in ksf; p_a is the atmospheric pressure in ksf; α_E is the reduction factor to account for jointing in rock; and f'_c is the compressive strength of the concrete in ksi. For intact rock, previous versions of the AASHTO manual required that the ultimate end bearing resistance, q_p, was calculated using the following equation: $q_p = 2.5q_u$

Editions of PennDOT *DM-4* from 2012 and earlier adhered to the same methodology as AASHTO, with slight modifications for calculating the value of α_E, based on the rock quality designation (RQD) and open or closed joints.

In editions of the AASHTO manual prior to 2014, the method for determining the interaction between side friction and end bearing resistance was not practical and often resulted in conservatively neglecting one of these two resistance components. Moreover, the AASHTO manual did not provide a clear guideline for calculating side friction and end bearing resistance when both components were considered:

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"Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation,..."

In addition, AASHTO does not provide a clear guideline to determining what the peak shear deformation is and how to calculate the reduced side (skin) friction.

The 2014 version of AASHTO provides that the ultimate side friction resistance for intact rock is to be calculated using the following equation (Kulhawy et al., 2005):

$$q_s/p_a = C\sqrt{q_u/p_a}$$

where: q_u should be the lowest between the unconfined compression strength of the rock and concrete, and C = 1.0. The ultimate end bearing resistance is calculated as shown in the prior edition of the manual: $q_p = 2.5q_u$.

The 2015 version of *DM-4* requires that the side friction resistance is calculated similar to that shown in AASHTO 2014, yet goes further to evaluate the load transfer behavior of drilled shafts in intact rock. In rock, Turner (2006) and Brown et al. (2010) state that a displacement of 0.4 to 0.6 in (10 to 15 mm) is needed to fully mobilize the side friction resistance and a displacement of approximately 4% of shaft diameter is needed to fully mobilize the base resistance. Based on the findings described by Turner (2006) and Brown et al (2010), *DM-4* recommends that the ultimate end bearing resistance is a function of the rock socket diameter, and is to be computed using the following:

$$q_p = [0.5''/(0.04D_{rock\ socket})](2.5q_u)$$

This end bearing equation provides a direct proportion of ultimate end bearing resistance based on the deformation experienced at the peak strength of side friction. Due to space limitations, additional details and discussion regarding each of the equations can be found in the references mentioned. Nonetheless, the equations from the newer versions of the AASHTO manual and PennDOT *DM-4* have allowed for a more realistic estimate of the ultimate capacity of drilled shafts.

Central Susquehanna Valley Transportation Project

Project Background — The Central Susquehanna Valley Transportation (CSVT) project is part of PennDOT's effort to improve mobility in the area between Selinsgrove Bypass (PA Route 11/15) and PA Route 147 in Northumberland County, Pa. Part of the overall project includes the construction of a new major bridge, which crosses the Susquehanna River with a 15 span steel structure carrying traffic on two lanes in each direction. The overall length of the proposed bridge structure is approximately 4,545 ft (1,385 m).

Of the 14 piers, each of the 5 river piers were designed to have a group of 8 drilled shafts that were 60 in (1.5 m) in diameter and that relied on side friction and end bearing resistance to provide the necessary design capacity to support the superstructure. These drilled shafts were designed primarily to bear on intact shale or shale with interbedded sandstone and/or siltstone.

Test Setup — A demonstration and load test shaft was constructed to confirm the design values by performing a bi-directional load test. The shaft was drilled with a 60 in (1.5 m) diameter rock socket that was about 25 ft (7.6 m) in length, and

Construction of bridge substructures

Exposed rock socket

the O-cell was located approximately 7 ft (2.1 m) above the bottom of the shaft. The O-cell was positioned such that the lengths of the rocket socket above and below the O-cell would provide ultimate geotechnical capacities of the shaft (i.e., designed such that the geotechnical resistance would be less than the rated capacity of the O-cell: 9,300 kips [41.4 MN]). That is, if a load of 9,300 kips were to be applied by the O-cell, the testing would confirm the ultimate geotechnical resistance for side friction and end bearing resistance even though failure may not be observed.

Once the drilling of the test shaft was completed, a miniature drilled shaft inspection device (miniSID) was used to evaluate the conditions at the bottom of the shaft and to ensure the specification requirements were satisfied. The project specifications mandated that not more than 50% of the area of the base had more than ¼ in (6 mm) of sediment and that the maximum thickness of sediment in any location was not greater than ½ in (12.5 mm). These criteria are half of the typical QC/QA requirements for this type of drilled shaft construction. Because this project was one of the first PennDOT projects to incorporate both side friction and end bearing resistance, the design team decided to use the more strict criteria. A downhole camera and a sonic caliper were also used to observe the sidewall of the rock socket and to verify the socket diameter.

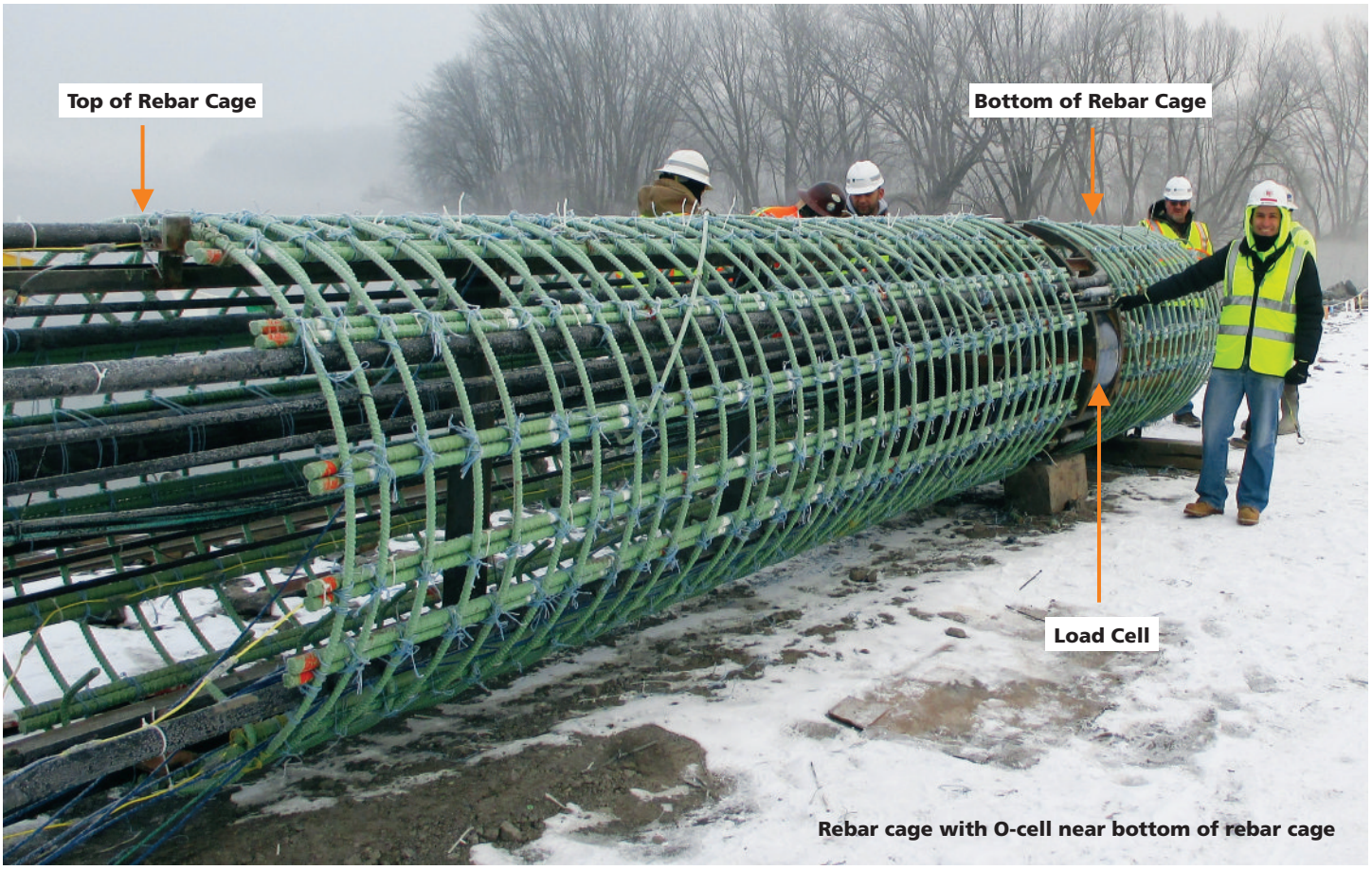
Per the project specifications, other instrumentation and nondestructive testing (NDT) were used to confirm the quality of the concrete during and after curing. Prior to placement in the drilled hole, tubes for crosshole sonic logging (CSL) and thermal integrity profilers (TIP) were installed within the rebar cage. The concrete for the test shaft was placed using the tremie method. To evaluate the compressive strength of the mix, concrete cylinders were tested at 7, 14 and 28 days after placement.

Lowering rebar and O-cell into the hole

A bi-directional load test using the O-cell, which was installed with a frame and was incorporated directly into the rebar cage, was performed after the CSL, TIP and 7-day concrete compressive tests confirmed the integrity of the shaft. An O-cell is a hydraulically driven bi-directional loading device capable of applying load simultaneously in an upward and downward direction within a drilled shaft. The shaft was instrumented with strain gages, telltales and extensometers to measure internal deformation of the shaft during loading. The test was performed 10 days after concrete placement.

O-cell test shaft rebar cage with instrumentation

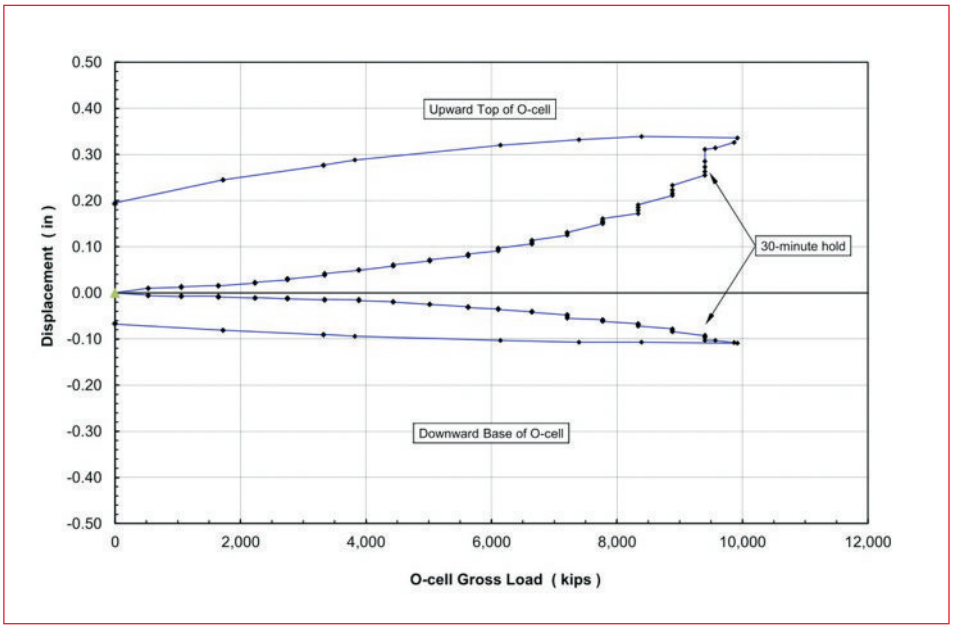
O-cell cage ready for lifting



Rebar cage with O-cell near bottom of rebar cage

Predicted Resistance and Deformation — The test shaft was installed in hard shale that had an unconfined compressive strength of about 465 ksf (22.3 MPa). The design of the drilled shaft was controlled by the strength of the in-situ bedrock instead of the concrete. Per PennDOT *DM-4* (2015), for the 60 in (1.5 m) diameter shaft with 25 ft (m) rock socket length, the calculated ultimate skin friction and end bearing resistance were approximately 31.4 ksf (1.5 MPa) and 242.2 ksf (11.6 MPa), respectively. Applicable LRFD resistance factors were applied to the side friction and end bearing resistances to determine the design values for the test shaft. The intention of the bi-directional load test was to confirm the resistance values used in the design.

As mentioned above, the length of the shaft was determined so the geotechnical ultimate resistance was slightly less than the rated capacity of the O-cell. For a length of about 18 ft (5.5 m) above the cell, an upward load of 9,300 kips (41.4 MN) imposed by the cell was expected to be resisted by the calculated ultimate side resistance (31.4 ksf x π x 5 ft x 18 ft) and the self-weight of the concrete. For a length of about 7 ft (2.1 m) below the cell, a downward load of 9,300 kips imposed by the cell was expected to be resisted by the calculated side resistance (31.4 ksf x π x 5 ft x 7 ft) and the ultimate end bearing resistance (242.2 ksf x 5^2 x $\pi/4$). Regarding deformation, it was anticipated that failure would occur at a displacement of about 0.4 in (10 mm) since this is the minimum expected deformation required to mobilize full side resistance at the instant of side friction failure, as defined by both PennDOT *DM-4* (2015) and the AASHTO manual (2014).



Load test result showing upper and lower portion of drilled shaft resistance

Up-close photo of the O-cell

Comparison of Test Results	Ultimate Capacity		Displacement at Failure	
	Predicted (kips) [MN]	Measured (kips) [MN]	Predicted (in) [mm]	Measured (in) [mm]
Upper portion (above O-cell): skin friction only	9,300 [41.4]	9,371 [41.7]	0.40 [10]	0.31 [7.9]
Lower portion (below O-cell): skin friction + end bearing	9,300 [41.4]	9,402 [41.8] (no failure observed)	0.40 [10]	0.10 [2.6] (no failure observed)

O-cell Test Results — The bi-directional load test was performed in general accordance with the standard Quick Load Test Method for Individual Piles (ASTM D1143) using 15 equal loading increments of about 620 kips (2,758 kN) based on the rated capacity of the O-cell (9,300 kips or 41.4 MN). Each load increment was maintained for approximately 8 minutes, and a creep test was conducted for 30 minutes at the maximum load of 9,300 kips.

It was intended that if failure of the side friction or end bearing was not observed, loading would continue above the rated capacity until reaching the O-cell limit (typically between 1.5 and 2 times the rated capacity of the device). Failure did occur, however, along the length of the shaft above the O-cell, where the geotechnical resistance along this length was controlled by side friction. For side friction along the upper portion, the predicted and measured values for load and deformation at failure were very similar. However, no failure in end bearing was observed. The measured values shown in the table correspond to the resistance and displacement along the lower portion at the instant of failure along the upper portion. Once the upper portion of the shaft failed, the lower portion was not able to sustain additional load. After a review of the test results and confirmation of design values, construction of the production shafts commenced.

Conclusions

The bi-directional load test results for the CSVT project illustrate how realistic design values are now compared to actual test values of ultimate side friction resistance and deformation in competent rock. Although design manuals and codes are allowing a significant contribution of the resistance from end bearing, it is very likely that the ultimate end bearing resistance of a drilled shaft in competent rock is controlled by the structural capacity of the shaft and not the geotechnical capacity. Using bi-directional load test results, such as those described in this article, results from other static and dynamic load tests, and more than 1,600 load tests in the new version of the FHWA *Deep Foundation Load Test Database* (DFLTD v.2.), we are getting closer to predicting the true ultimate geotechnical capacity for drilled shafts.

Acknowledgments

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