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Micropiles in Karst: A Case History of Difficulties and Success

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Abstract

The expansion of Exton Square Mall, a large regional retail shopping mall, consisted of adding a second floor over the existing mall, as well as three new anchor stores and two parking garages. The original structure was built around 1970 and is founded on spread footings. The site is underlain with karstic limestone, including a quarry that was backfilled with random fill, and the site conditions varied greatly. Furthermore, the structural loading conditions for the new four-story garage and the overbuild resulted in 2670 to 5340 kN (300 to 600 ton) column load requirements. The foundation system selected for these high column loads and irregular subsurface conditions consisted of 1335 kN (150-ton) allowable capacity micropiles. Of the nearly 900 piles required, about one half were installed inside the mall with low head clearance and tight access schedules.

The micropile construction involved challenges with drilling variable karstic rock to depths of between 6.1 and 54.9 m (20 and 180 ft) to form a suitable bond zone. The piles consisted of 178 mm (7 inch) steel casing filled with grout. Two load transfer systems were tested to facilitate construction in the difficult geology, including sleeve ports in the casing within the bond zone and an open hole with bundled bars grouted in the rock to transfer load to the rock socket. Eleven load tests of up to 3740 kN (420 tons) were completed prior to production pile construction. Due to tight access constraints, many of the pile caps utilized center to center spacing of about 508 mm (20 inches), which was also duplicated in the load test program. This project provided many lessons and confirmed the viability of high capacity micropiles even in karstic terrain.

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Introduction

The Exton Square Mall is located on the northeast corner of the intersection of State Routes 100 and 30 in West Whiteland, Chester County, Pennsylvania. Prior to a $125 million renovation, the site consisted of the Exton Square Mall, the West Whiteland Post Office, associated paved roads and parking lots, and several old farmhouses that had been converted into office buildings. The mall contained only one anchor store, which was two stories, and was surrounded on all sides by smaller retail stores. The surrounding mall was bilevel: the south side of the mall was the same level as the anchor store's first floor, EL 95.7 (EL 314 (ft)), and the north side of the mall was the same as the anchor store's second floor, EL101.5 (EL 333).

The expansion nearly tripled the size of the existing mall, from 40,000 to 111,500 square meters (436,000 to 1.2 million square feet) and consisted of the addition of three department stores, a food court, and a second story overbuild for the southern half of the mall. The maximum column and wall loads were on the order of 667 kN (150 kips) and 73 kN/m (5 kips/lf) for the anchor stores, and 4000 kN (900 kips) for the overbuild columns. In addition, two parking garages were planned adjacent to the eastern and westernmost portions of the existing mall. The maximum column loads for the west parking garage were on the order of 1334 kN (300 kips), whereas the maximum column loads for the east parking garage were on the order of 4450 kN (1000 kips).

The construction limitations of this project required both the engineers and contractors to provide innovative means for completing the work with minimal inconvenience to the shoppers and store owners. Restrictions included performing the work while most, if not all, of the stores remained open, and performing all the interior work at night when the mall was closed.

One of the most significant potential difficulties involved the subsurface conditions for the facility. The site is underlain by dolomitic limestone, which is defined by the Pennsylvania Department of Environmental Resources as a hazardous rock due to its susceptibility to solutioning and sinkhole development. In order to support the 4000 kN (900 kip) overbuild and 4450 kN (1000 kip) east parking garage column loads, deep foundations had to be used. Several alternatives were investigated including drilled shaft, driven piles, and micropiles. Micropiles proved best suited to the tight access limitations, highly variable bearing grades and irregular rock qualities, and were capable of carrying 1334 kN (300 kips) per pile, well in excess of traditional driven pile systems.

Geologic Conditions

The structure is located within the Chester Valley Sequence of the Piedmont Physiographic Province in Pennsylvania. The Piedmont Physiographic Province is composed of folded and faulted sedimentary, metamorphic and igneous rock. The Chester Valley Sequence comprises carbonate geologies within this province (Geyer, 1982).

According to the Pennsylvania Geological Survey (PAGS) mapping, the Mall is underlain by the Ledger Formation, a medium to coarsely crystalline dolomite containing oolitic, siliceous and cherty beds. Two geologic contacts were observed near the site; each at about 0.16 km (0.1 mile) to both the north and the south of the site. To the north, there is a geologic contact with the Chickies Formation, which comprises quartzite and schist. The southern geologic contact is against the Conestoga Formation, an impure limestone, phyllite and dolomite. Both contacts trend east-northeast to west-southwest (Socolow, 1978).

Both the Ledger Formation and the Conestoga Formation are susceptible to solutioning and are often associated with karstic conditions including sinkholes, subsurface cavities and bedrock pinnacles. A review of the sinkhole maps produced by the PAGS for the region indicated that there were three sinkholes mapped on the site, and a quarry had previously been located on the west side of the site (Kochanov, 1993).

The subsurface exploration consisted of standard test borings and test pits that revealed fill to depths of up to 8.1 meters (26.5 ft). It is likely that areas with significant amounts of fill are due to the backfilling operations of the former quarry. Natural alluvial soils were encountered to limited extent to depths of 1.8 to 4.3 m (6 to 14 ft). These soils are likely due to the periodic flooding and meandering of Valley Creek along the southwestern side of the site. The remainder of the soils encountered were typical of residual soil derived from carbonate rock and extended to depths of up to 24.7 m (81 ft). The residual soils were generally silts and clays with varying amounts of sand and rock fragments.

Deep Foundation Option

Given the highly variable subsurface conditions, the high structural loading conditions and the limited access within the existing structure, micropiles were considered to be the most appropriate foundation system for this project. Although drilled shafts have been used in karstic geology, the potential installation difficulties and cost overruns precluded their use on this site. Driven piles were also considered. However, establishing a bearing grade would be difficult and the tips would likely become damaged during installation. Furthermore, boulders or thin rock ledges and pinnacles may appear to provide

suitable bearing during driving, but may prove unstable for the long term performance of the foundation.

Micropiles afforded the opportunity to carry large loads in a rock socket that could be verified through drilling response and were extremely adaptable to variable site conditions including length, groundwater flow, poor soils and voids expected on this site. Furthermore, the tight access and low headroom could be easily handled with the small micropile drills available.

Micropile Details

The micropiles at Exton Square Mall were Case 1, Type A and D (FHWA 1997), as shown on Figure 1. For discussion, the piles have been divided into two categories, overbuild and east parking garage piles.

Overbuild Piles

A total of 405 micropiles was required for the construction of the mall overbuild: 294 on the interior of the mall, and 111 on the mall perimeter. The corridor pile caps each had three piles, although four piles per cap were typically used within the stores where the number of columns was minimized and bay widths maximized. The lengths of the piles varied from 6.1 to 45.7 m (20 ft to 150 ft) below the existing slab elevation, with an average length of 10.4 m (34 ft).

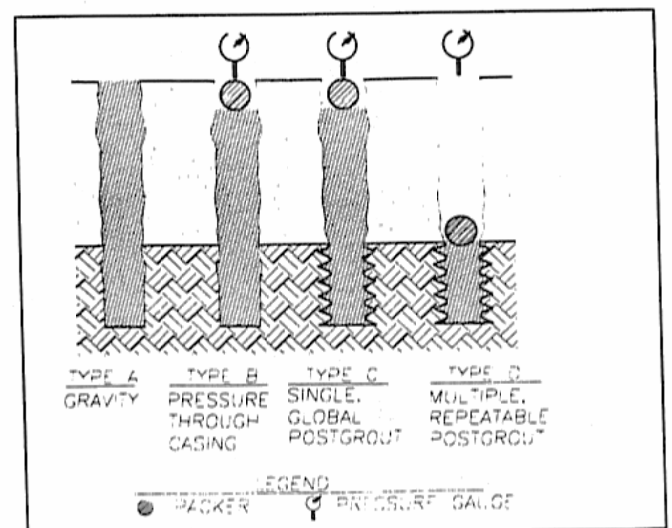

Figure 1. FHWA Micropile Classification Based on Grouting Method (Bruce, 1997)

East Parking Garage Piles

Three hundred and fifty-five piles were required for the parking garage foundation system. The pile caps varied from three to 20 piles. The larger caps were constructed at locations where the garage shear walls were designed. The average pile length for the garage was approximately 13.1 m (43 ft), with the lengths ranging from 7.6 to 54.9 m (25 ft to 180 ft).

The piles were comprised of 178 mm (7 in) outer diameter grout filled N80 (f_y ≥550 MPa (80 ksi)) steel casing, with a wall thickness of 12.2 mm (0.48 inch). This configuration was used to carry the design loads through the overburden and the fractured rock. Two methods were considered as being able to transfer the pile load to the rock in the bond zone as shown in Figure 2.

The more traditional approach featured withdrawing the casing to the top of the bond zone (3.1 meters) and installing central reinforcing bars. In this case, the bond zone was reinforced with two 4.6 m (15 ft) long, 57 mm (#18) threaded steel bars (f_y = 517 MPa (75 ksi)) inserted into the bond zone and extending 1.5 m (5 ft) into the upper steel casing. The alternative method tested was to extend the casing through the full depth of the pile and provide sleeve ports similar to a tube-à-manchette system within the bond zone. This would allow the piles to be regrouted within the highly fractured rock, and so it would facilitate shorter overall pile lengths and provide the ability to handle difficult karst conditions

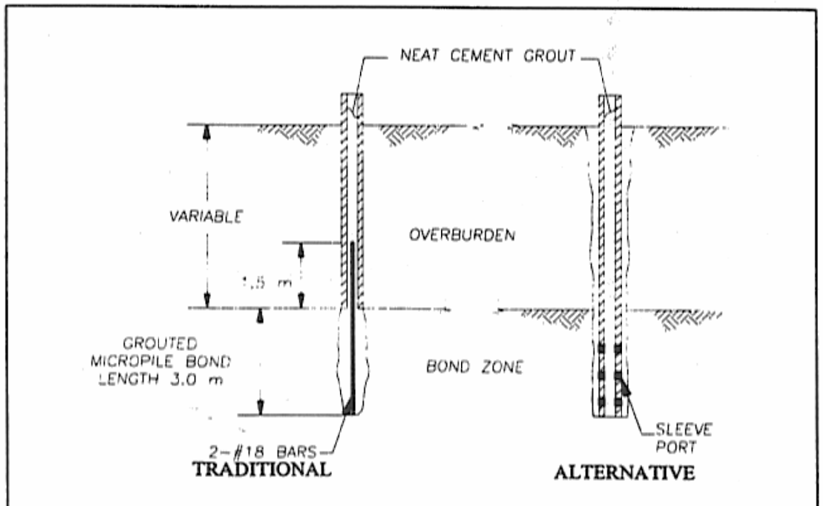

Figure 2. Schematic of Micropile Options

without the more typical (consolidation) void grouting and redrilling. Both of these systems were suitable for the structural requirements of the project.

The pile grout consisted of Type I Portland cement with a water/cement ratio of 0.45 and a 28 day design strength of at least 27.6 MPa (4,000 psi).

Pile Construction

The pile construction began with the installation of test piles within the mall area in an open retail space, to evaluate subsurface conditions, drilling methods, and performance. Concurrently, test micropiles were installed in the east parking garage area with larger drilling equipment. Eight control piles were located around the site to evaluate the variability of the drilling conditions. Each of these early control piles was installed using the sleeve port alternative. While drilling one of the control piles, a clean angular sand was encountered which resulted in running soil conditions below the water table making grouting operations difficult, and did not allow regrouting of the ports, likely due to sand packing around the sleeves. This pile was selected for testing as discussed below. These sands were also encountered during installation of the reaction anchors and were likely related to a weakly cemented carbonate sand zone that broke down during drilling. Grouting of the piles was completed by placing either a single or double packer into the casing and injecting grout at pressures of up to 8.3 MPa (1200 psi) to open the sleeves and allow grout to flow into the annular space around the casing. Where grout flow to the surface was not observed, or high grout volumes and low pressures were encountered, regrouting of the ports was performed.

Three of these control piles were selected for load testing. The locations of the piles load tested are shown in Figure 3. Due to problems with several of the load tests, two additional piles were tested which also provided verification

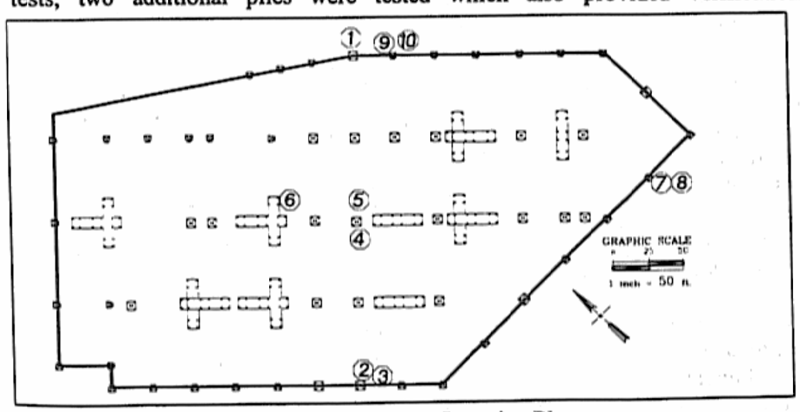

Figure 3. Load Test Location Plan

information on the effects of piles installed at a spacing of 533 mm (21 inches) center to center.

Although three of the load tests on the alternate sleeve port design proved to be successful, concern with the grouting process and the verification of these piles during production resulted in modification of the design to utilize the more traditional micropile configuration. Three additional test piles were then installed at selected locations across the site. These piles were drilled to advance the steel casing to a depth where at least 3.1 m (10 ft) of relatively competent rock had been penetrated. The casing was then lifted 3.1 m (10 ft) to the top of the design bond zone and grout was placed into the hole using a tremie tube. If the bond zone proved capable of maintaining the grout level inside the casing, the reinforcing bars were lowered to the bottom of the hole to complete the pile. Where the grout level dropped inside the casing during construction, indicating that the bond zone was not a tight rock formation, the grout was allowed to set and the hole was redrilled and regrouted before the central bars were installed in the bond zone.

Three more piles were again selected and load tested as discussed below. A pair of piles was subsequently selected, in an area of the site known to have highly fractured rock, for testing to determine the effects of poor quality rock on a 3.1 and 4.6 m (10 and 15 ft) bond zone, respectively.

Following completion of the testing program, the remainder of the piles was installed essentially without incident. At one location within the mall overbuild area, installation of a pile encountered a large void within the overburden soils directly beneath an existing foundation. Grouting was immediately initiated in an effort to stabilize the area. However, significant settlement of the old foundation occurred and compaction grouting was required to stabilize the area before further piles could be completed.

Pile installation was completed with large track-mounted drill rigs, including a Casagrande C-12 and a Klemm 806 dual head drill in the parking garage and overbuild exterior piles. A Davey Kent DK-50 and several Klemm 704 electrical/hydraulic drill rigs were used for the interior overbuild interior pile installation. All rigs employed eccentric rotary percussion duplex drilling techniques (Bruce 1989). Compressed air was used as the drilling fluid, and was circulated through the inner rod and returned through the annular space between the rod and outer casing.

The garage and exterior overbuild piles were advanced in 3 m (10 ft) cased sections. Grouting was conducted using 3.8 cm (1½ inch) diameter tremie tubes. The bundled bars were then installed once the grout elevation stabilized. However, installing micropile foundations in the existing mall with limited

headroom involved changing the drilling procedures to accommodate tight areas with limited access and required finding a way to place the 4.6 m (15 ft) long reinforcing steel bars.

The interior overbuild piles used 1.5 m (5 ft) sections of drilling rod and pile casings to accommodate the limited ceiling height. The use of these small casings significantly affected the rate of progress. To deal with the drill cutting spoils, delivery of grout and bar placement, a hole was cut in the roof of the mall above each pile cap location. The grout batch plant was located outside the building and grout was then pumped up to the roof and flowed by gravity into the piles.

A major concern to the storeowners and challenge to the contractors was how to keep the store inventory from being damaged by the drilling operations. In order to address this concern, the contractors had plywood boxes built to surround the drill at each pile cap location. In addition to the boxes, the contractors provided a diverter mounted to the floor around the drill casing to catch the cuttings. The drilling air was sufficient to push the cuttings through a discharge hose that extended through the hole in the roof to a dumpster used as a sedimentation trap.

Load Testing

Ten load tests were performed on this project as shown in Figures 3 and 4. The first test, Test No. 1, was unsuccessful due to problems encountered with running sand during drilling and grouting, as discussed above. This test failed well below the design load indicating that inadequate bond had been created between the pile and the weathered bedrock penetrated during drilling. This test has not been included in the remainder of this discussion.

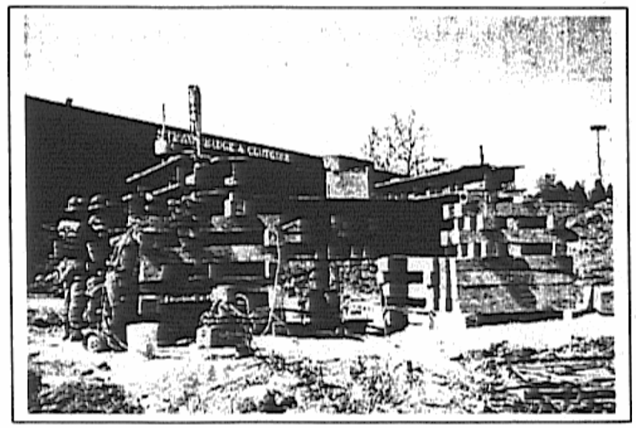

Figure 4. Load Tests: Numbers 4 and 5

Four additional load tests were performed on piles installed using the alternate sleeve port casing system (Test Nos. 2 through 5). Port locations, covering system and grouting process were adjusted slightly through the testing program to improve the reliability of the grout coverage. In general, the bond zone included three 13 mm (½ inch) diameter ports evenly spaced around the steel casing at 0.8 m (2.5 ft) intervals. The latter piles installed also had a port located in the bottom cover plate of the pile casing to improve grout coverage of the tip of the pile. Single and double pneumatic packers were used to allow the casing to be pressurized and the ports to be opened multiple times. Grouting was continued until the grout was observed at the ground surface around the pile annular space. If grout flow was not achieved around the pile, the packer systems allowed individual ports to be isolated to ensure focused grouting at the sleeve levels. As would be expected where solid rock was present, the initial grouting was sufficient to fill the annular space and regrouting efforts were unsuccessful at opening the ports in the pile due to high lateral restraint. Where highly fractured zones were present, pressures of 1.4 to 8.3 MPa (200 to 1200 psi) were required to open the ports and initiate subsequent grout flow.

The results of these load tests are shown on Figure 5 in the form of plots depicting the elastic movement of the pile and the residual movements. Test No. 2 failed at about 50 percent of the design load (667 kN) catastrophically due to buckling at the first threaded joint below the ground surface.

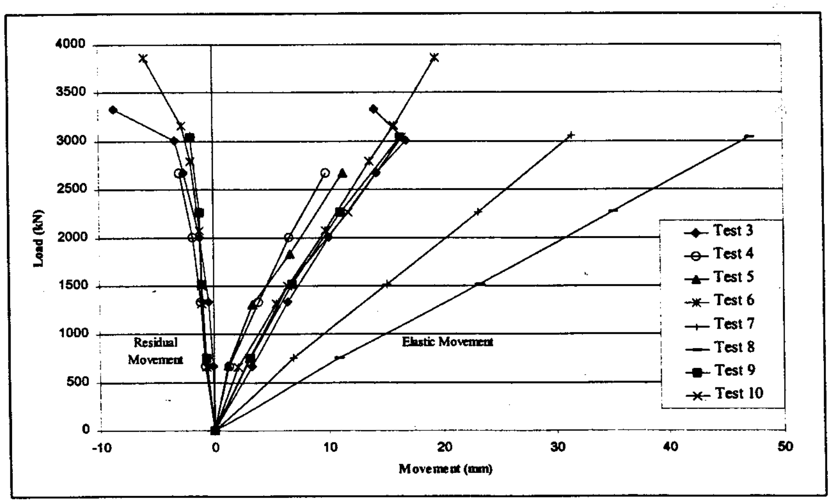

Figure 5. Load Movement Curves

The remaining three tests on this pile configuration generally met the requirements of the test. The third test was loaded to 125 percent of design working load where a significant movement was observed. Calculations show that about 2669 kN (300 tons) were carried by the first 2.4 m (8 ft) of the bond zone. The rapid movement depicted in Figure 5 for this pile is believed to be the result of incomplete coverage of grout in the bottom 0.6 m (2 ft) of the pile. This lack of coverage resulted in movement of the pile until end bearing was developed, at which point the pile began to carry additional load. Test Nos. 4 and 5 were performed on adjacent piles with variable bond zone lengths.

Although the final three tests resulted in successful performance, the pile design was modified and a more traditional pile was installed, which required drilling to more competent rock that could be tremie grouted. These tests are also depicted on Figure 5 for comparison. Table 1 provides a summary of basic pile configuration and load test results for comparison.

The tests completed for this project show that a properly installed micropile of either configuration described in this paper successfully carried up to 3638 kN (420 tons). At these loads, the pile carried the load without sign of geotechnical or structural failure.

Each test was performed in general accordance with the guidelines of ASTM D1143, except that a cyclic loading pattern was used similar to the Post Tensioning Institute Recommendations for Soil and Rock Anchors (PTI, 1996). This was performed to allow interpretation of load transfer down the pile length. By subtracting the total deflection from the residual deflection at each load cycle, the elastic movement of the pile is calculated and compared to the load applied. Using a composite modulus for the grout filled casing, the length of the pile stressed by the test is calculated. The distance which the stress extends into the bond zone is referred to as the debonded length.

Table 1
Load Test Data

Load Test	Total Casing Length (meters)	Bond Zone (meters)	Grout Zone Diameter (mm)	Max. Load (kN)	Max. Total Movement (mm)
1	6.1	None	218	667	-----
2	6.1	3	218	2,669	114
3	10.6	3	218	3,336	23
4	8.8	3	218	2,669	13
5	10.4	4.6	218	2,669	13
6	11.9	3	188	3,638	23
7	30.5	3.2	188	2,669	33
8	21.5	3.2	188	2,669	48
9	10.7	3	188	2,669	19
10	12.2	4.6	188	2,669	33

Once the debonded length is calculated, the bond stress can be calculated for each load cycle. The early load cycles are believed to be impacted by the adhesion to the surrounding soils above the bond zone. As the loads are increased, the average bond values generally become more consistent, which is believed to be more representative of the actual rock to grout (or steel to grout) bond value achieved in the pile construction. For the purposes of this analysis, it was assumed that the failure occurred at the grout to rock bond. Figure 6 shows the bond values calculated for the piles once the applied load exceeded 890 kN (100 tons) (about 66% of the design load). As can be seen, the average bond values have significant scatter between about 1 to 2 MPa (150 psi and 300 psi). An average bond value of about 1.4 to 1.7 MPa (200 to 250 psi) was considered representative for this site.

Conclusions

Nearly 900 micropiles were successfully installed in highly variable karstic limestone conditions. These piles had an allowable design capacity of 1334 kN (150 tons) and were initially tested to loads of over 3736 kN (420 tons). Given the results of nine successful load tests, a rock bond value of about 1379 to 1723 kPa (200 to 250 psi) was verified for the piles installed.

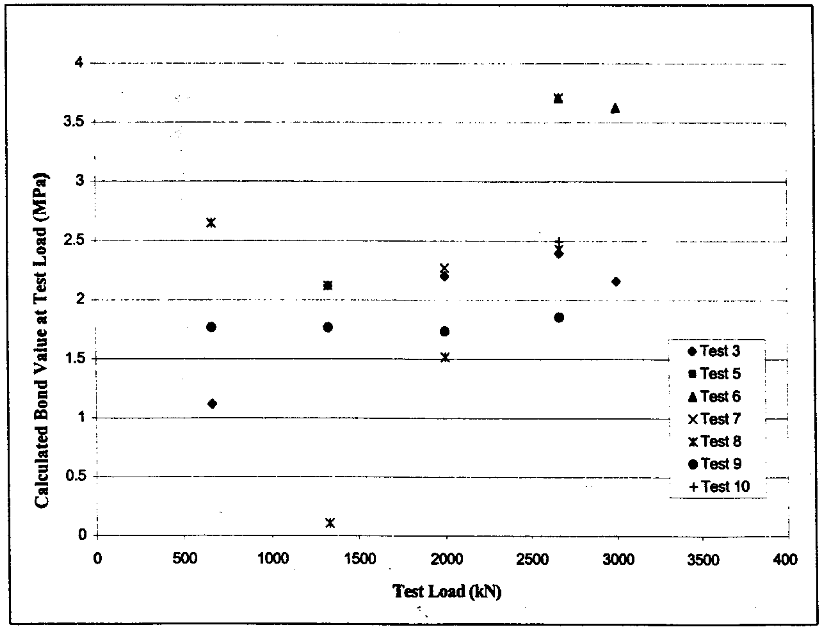

Figure 6. Calculated Bond Stress for Rock Socket

Difficulties were encountered during drilling and grouting that required care by the contractor to ensure consistent installation quality control of each pile. Full time observation and testing by experienced engineering inspectors were also imperative on this project to ensure that anomalous conditions were identified and that suitable steps were taken to establish confidence in each pile.

Acknowledgement

We would like to thank The Rouse Company and The Lathrop Company for their assistance during the pile load testing and production pile installation phases of this project. Difficult site conditions, poor weather and an aggressive construction schedule required close cooperation of all parties to complete this project successfully.

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