Effects of Gravels, Cobbles, and Boulders on Capacity of Large Diameter Shafts at the Cypress Avenue Bridge, Redding, California

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Abstract:
Drilled shaft capacity computations are routinely performed by geotechnical professionals using the Alpha and Beta methods outlined in the Federal Highway Administration (FHWA) Publication FHWA-IF-99-025. For cohesionless intermediate geomaterials, the analysis relies heavily upon the Standard Penetration Test sampler blowcounts obtained from exploratory borings. In the presence of alluvial gravel, cobble and boulder deposits, such evaluation cannot be reliably performed. This paper presents information obtained from Osterberg Cell load testing of 98-inch (2.5 meter) diameter shafts advanced using the casing oscillator method at the Cypress Avenue Bridge over the Sacramento River in Redding, California. The relationships between side shear transfer, end bearing, and settlement in cobble and boulder rich deposits with various matrix materials and degrees of weathering are demonstrated. The effects of particle dilation in the development of shaft side resistance and zones of higher stress concentration are also discussed. The results of instrumented pile load testing are compared with the results of drilled shaft capacity estimates made using the FHWA methods in an effort to better correlate field performance with analytical methods.

Project Description

The Cypress Avenue Bridge crosses the Sacramento River in Redding, California, as the main arterial into the downtown area. The existing bridge consists of two steel girder, eight-span bridge structures, the downstream one built in 1947 and the upstream one built in 1967. The bridge piers and abutments are supported on driven steel H piles. Each bridge supports two lanes of traffic. Due to traffic impacts, the City desires to replace the existing bridges with a new six-lane reinforced concrete box girder bridge. The new bridge will have five spans with four piers in the active river channel. Each pier will be supported by a row of seven, 98-inch (2.5 meter) diameter drilled shafts extending up to 65 feet (20 meters) below the design scour elevation.

Design of the project was performed by T.Y. Lin International in general accordance with California Department of Transportation (Caltrans) guidelines. Design of drilled shaft foundations was performed by Kleinfelder based on the FHWA design approach using the Beta method for Cohesionless Intermediate Geomaterials (IGM's). Kiewit Pacific Corporation was selected to construct the project and retained Malcolm Drilling of Hayward, California to construct the drilled shafts. The contract incorporated a partnering approach to contract management with construction management being provided by PB Americas transportation group.

Site Geology and Subsurface Conditions

Redding is located in the northern end of the Great Valley of California near Mount Shasta and the Trinity Mountains. This reach of the Sacramento River cuts through volcanic rocks, river channel deposits, and various interlayered alluvial fan and river terrace deposits. Located just below Keswick and Shasta Dams, flows are regulated and can vary greatly depending on storage and seasonal considerations. The bridge site is within a high energy portion of the river and thus the river bed is composed of coarse gravel cobbles and boulders with a sand matrix. The channel deposits in the area are underlain by various alluvial and volcanic deposits of Quaternary and Tertiary age.

Due to the anticipated presence of cobbles and boulders, explorations performed for the geotechnical investigation (Kleinfelder, 2006) utilized an air percussion and casing drill rig equipped for Standard Penetration Test (SPT) sampling in accordance with ASTM D 1586. Sampling was performed at roughly five foot intervals in each boring to obtain SPT blowcounts (N values) for use in engineering analyses. The geotechnical report depicts the subsurface conditions at the site as recent channel deposits underlain at depth by older, consolidated and weathered channel and alluvial fan deposits. The recent channel deposits generally consist of poorly graded gravel with sand and abundant hard cobbles and boulders up to about 18 inches in maximum dimension. The channel deposits extend to a depth of about 40 feet below the river bed. The underlying older alluvium generally exhibited a firm to hard clay matrix. The cobble and boulder clasts included volcanic and metamorphic rocks that were slightly to moderately weathered and generally softer than the near-surface clasts.

Based on the subsurface conditions, environmental constraints, river construction considerations, and proposed bridge configuration, large diameter shafts were selected for support of the piers. Driven steel H piles were selected for the abutments and retaining walls. Since drilled shaft construction would require use of a steel casing to advance the hole, conventional auger drilling and casing systems were considered problematic due to the risk of excessive disturbance and caving of the formation and the potential for problems with installation of steel casings due to the presence of cobbles and boulders in a cohesionless matrix. A casing oscillator approach was selected for the drilled shafts in the river. Due to the subsurface conditions and number of drilled shafts required, a full scale load test program was recommended and later incorporated in the project specification. The results of that load testing program, subsequent engineering analyses, and comparisons with FHWA design methods follow.

Preliminary Drilled Shaft Design

Preliminary design of the drilled shafts was performed using the FHWA method for cohesionless IGMs. This method uses SPT N values to derive side friction and end bearing capacities. Corrected SPT N values obtained from the boring nearest the test shaft ranged from 162 to 212 in the poorly and well graded gravels and 49 to 209 in the clayey gravels. The lower blow counts in the clayey gravels are thought to represent zones containing less cobbles and boulders and likely represent the approximate stiffness of the matrix materials within the clayey gravels.

Since many of the SPT sampling attempts bounced on a cobble or boulder, the highest values were disregarded. The lower SPT N values were thought to represent finer grained interstitial layers and, therefore, were thought to be representative of the matrix materials in the gravel deposits. On that basis, a design N value of 70 was selected.

This project was designed using allowable stress design methods (prior to the adoption of the LRFD program in California). Drilled shafts to support the bridge piers located in the river were designed for nominal loads of 5,300 to 6,250 kips in compression, and 250 to 1,050 kips in tension. A factor of safety of two against side shear failure was used for design. End bearing was not considered since the shafts would be constructed below groundwater. Caltrans standard practice does not allow consideration of end bearing under such conditions due to uncertainties in the quality of the bottom portion of the shaft. The design shaft lengths ranged from about 66 to 82 feet below the scour elevation. The design shaft diameters were 98 inches (2.5 meters). For the bridge piers located in the river, each pier was supported on seven drilled shafts.

Casing Oscillator Method

The casing oscillator system utilizes a large diameter steel casing fitted with flat carbide teeth on the lead section that is advanced with a hydraulic oscillator. The casing is advanced by twisting it back and forth under the weight of the casing and the oscillator turntable. As the casing advances, a crane-operated clamshell is used to excavate the material from within the casing. To prevent heave of the hole bottom during excavation, the casing is filled with water to near or above the groundwater level. To prevent over-mining or relaxation of the formation, a plug of soil is maintained within and above the tip of the casing. Once the casing advancement and excavation is completed the fine soils suspended in the water within the casing must be removed. These materials are typically allowed to settle out and later removed using an air lift suction system. A venturi nozzle is used to vacuum out the sediments and loose materials from the bottom of the excavation and help flatten it since the clasmshell bucket is rounded and cannot excavate a flat bottom. Complete exchange of the drill water with fresh water must be performed prior to concrete placement.

Reinforcing steel is then placed and concrete is pumped through a tremie tube extending to the bottom of the excavation. The casing is withdrawn in sections as the concrete is placed such that the fluid concrete engages the surrounding materials.

Osterberg Cell Load Testing

Osterberg Cell (O-Cell) load testing was conducted by Load Test Inc. of Gainesville, Florida on a test shaft drilled near the west abutment (Abutment 1). The test site was selected based on its accessibility and the presence of a deep boring drilled during the geotechnical investigation. O-Cell testing has been performed on many large diameter shafts throughout the world and provides a means of testing drilled shafts with high axial loads (Osterberg, 1984, 1995). In this case, a set of three O-Cells were placed at an appropriate depth in the shaft in an attempt to balance the upward and downward loads as well as obtain side shear data in different soil layers. Since the downward loads eventually engage end bearing in addition to side shear resistance, the load cell was placed about ⅔ of the way down the shaft. Since the vertical deformation needed to engage end bearing can be between five and ten times the deformation required to engage side shear resistance, this needs to be taken into account when selecting the O-Cell location within the test shaft. The layer boundary between younger river channel deposits and older alluvium with cohesive matrix materials was a major consideration in selection of the O-Cell and strain gauge positions. Since axial extension of the O-Cells was limited to about 6 inches and mobilization of end bearing was expected to require about 3 to 4 inches of axial movement, it was expected that upward movement would be sufficient to exceed the design settlement criteria of ½ inch. This is the point where the experience of the O-Cell testing technician becomes very important in selecting the O-Cell position in the test shaft.

The O-Cells were positioned 16 feet (4.87m) above the bottom of the shaft. Strain gauge levels were 3 (0.91m), 11 (3.35 m), 27 (8.22m), 37 (11.27m), and 60 (18.28m) feet above the bottom of the shaft. Figure 1 shows the locations of the strain gauges and O-cells for the test shaft.

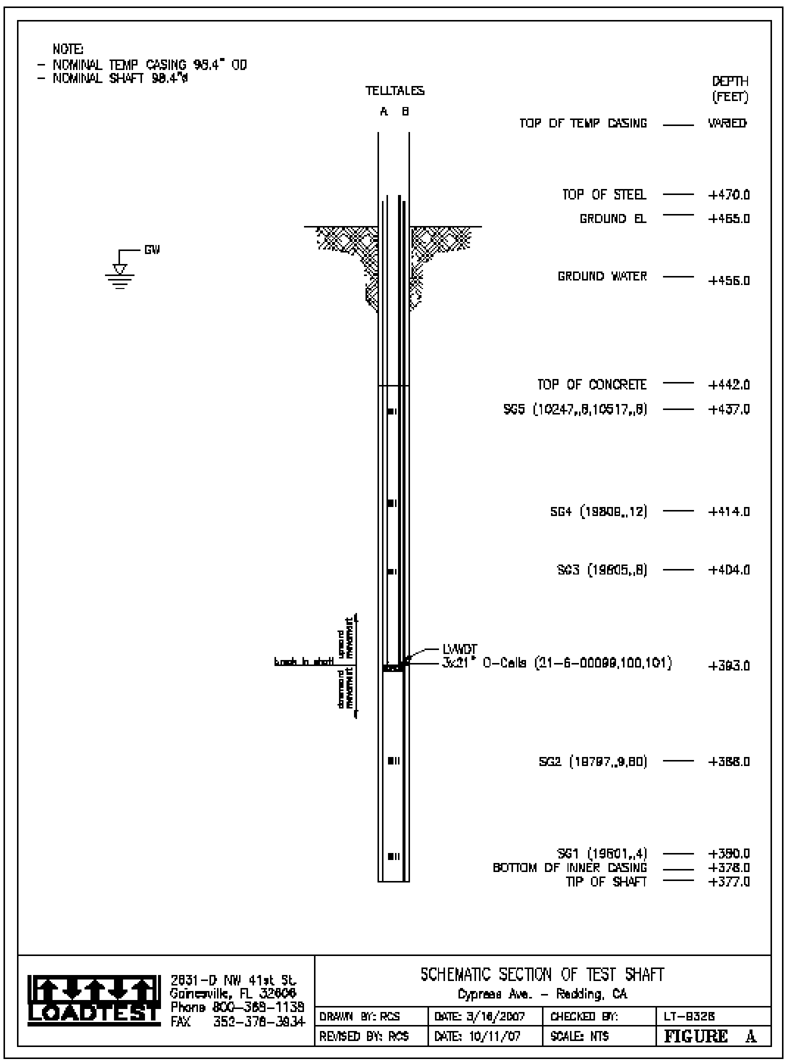
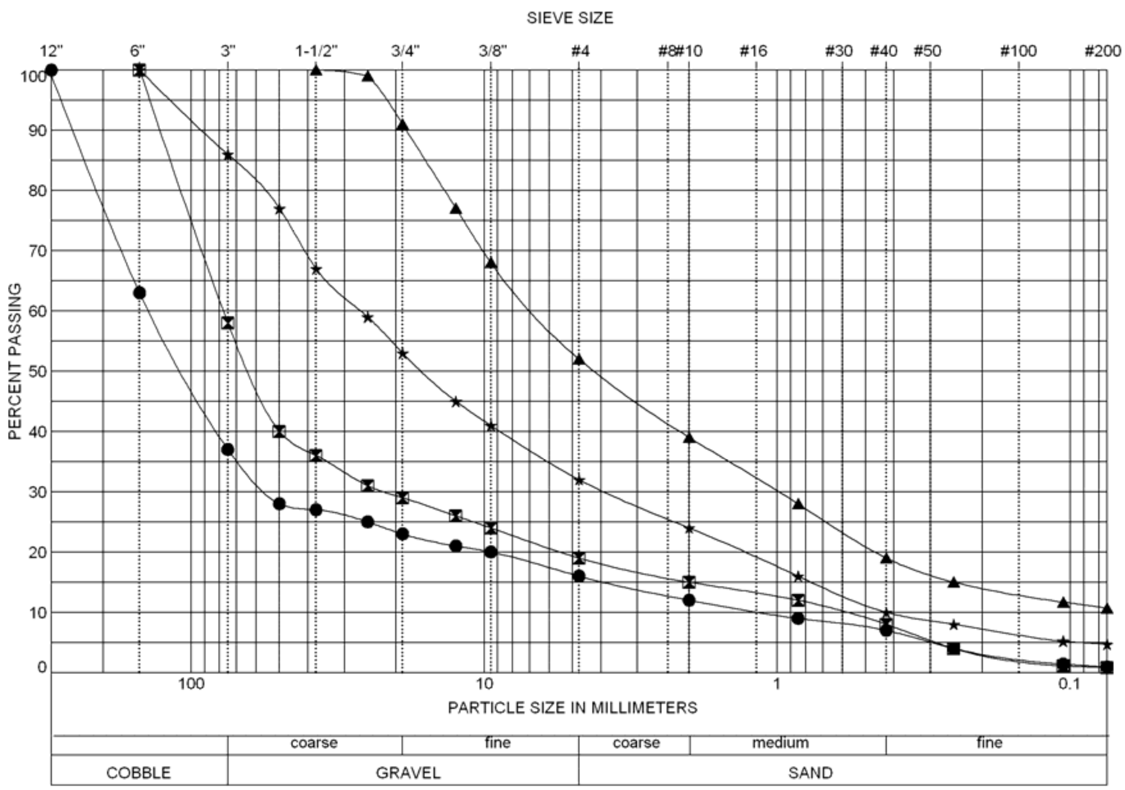


Figure 1

Excavated Material Properties at Test Shaft

During excavation of the load test shaft, the materials encountered were observed and logged in accordance with the Unified Soil Classification System. Samples of the materials encountered were also obtained for laboratory sieve analyses. Results of those sieve analyses are presented on Figure 2 below.

Laboratory Sieve Analyses
Cypress Avenue Bridge - Test Shaft TS-1



LEGEND:	SOURCE	DEPTH (ft)	COBBLE (%)	GRAVEL (%)	SAND (%)	FINES (%)	D60 (mm)	D10 (mm)	Cu	Cc	DESCRIPTION
●	TS-1	36.0	62	21	15	1	138.47	1.13	122	19	POORLY GRADED GRAVEL with SAND(GP)
⧗	TS-1	52.0	42	39	18	1	77.52	0.6	129	10	POORLY GRADED GRAVEL with SAND(GP)
▲	TS-1	60.0	0	48	41	11	6.72		114	2	Poorly Graded GRAVEL with clay and sand (GP-GC)
★	TS-1	72.0	14	54	27	5	26.3	0.42	62	1	Poorly Graded GRAVEL with clay and sand (GP-GC)

Figure 2

Due to the abundance of coarse particles in the gravel and cobble deposits sampled, the matrix materials (taken as the percent passing the No. 4 US sieve) in the river channel deposits comprised about 16 to 19 percent of the material by weight. In the underlying older alluvium, the matrix materials comprised about 32 to 52 percent of the material by weight. The fines content (percent passing the No. 200 US sieve) was about 1 percent in the river channel deposits and about 5 to 11 percent in the older alluvium. These relatively small variations in fines content represented significant differences in the behavior of the matrix materials. The sandy gravel river channel deposits behaved as cohesionless material whereas the older alluvium behaved as clayey gravel with significant cohesion and occasionally exhibited some weak cementation.

Instrumentation

The O-cells were welded between two steel plates. To measure axial movement of the O-Cells, six Linear Vibrating Wire Displacement Transducers (LVWDT's) were installed between the upper and lower plates. In addition, two telltales opposite each other were mounted to the reinforcing steel cage. The telltales allowed for measurement of the compression of the shaft between the O-cell top plate and the top of the shaft.

Vibrating wire strain gages with sister bars were mounted at selected intervals along the reinforcing steel cage to permit measurement of side shear transfer along the shaft. The strain gauge locations were selected based on the geotechnical information obtained from the nearby exploratory boring to obtain side shear data for the upper channel deposits and the lower alluvium. Three levels of strain gauges were installed above the O-Cells and two levels were installed below them. Each level included four gauges at 90 degree intervals around the reinforcing steel cage. Gauge sets of two are typically mounted 180 degrees opposed to each other so that if the shaft bends during testing, the true axial deformation can be obtained by averaging the two gauge readings. Using four gauges allowed for redundancy in case a gauge or cable was damaged during installation.

Monitoring of the shaft top movement was performed using two automated digital survey levels placed at separate locations about 37 feet away from the test shaft. The two survey levels were used to provide redundancy in case one level was moved or inoperable, and as a data check.

The strain gauges and LVWDT's were connected to a data logger and a laptop computer. The digital survey levels were also connected to the laptop computer. During testing the instruments were monitored and the data recorded on the laptop computer. For each test load interval the pressure applied to the O-cell was measured using both a vibrating wire pressure transducer and a Bourdon pressure gage. The transducer provided a check on the Bourdon pressure gage and allowed for real-time data plotting during the test

Load testing was conducted using the Quick Load Test Method (ASTM D 1143 Standard Test Method for Piles Under Static Axial Load). Each load increment was held for eight minutes prior to loading to the next increment. Additional load increments were applied until the maximum O-cell capacity was reached. Unloading was conducted in four decrements and the test was completed.

Load Test Results

Result of the load test are shown on the following plots. Figure 3 shows the shaft movements above and below the O-cells. Figure 4 shows an equivalent top down load test result.

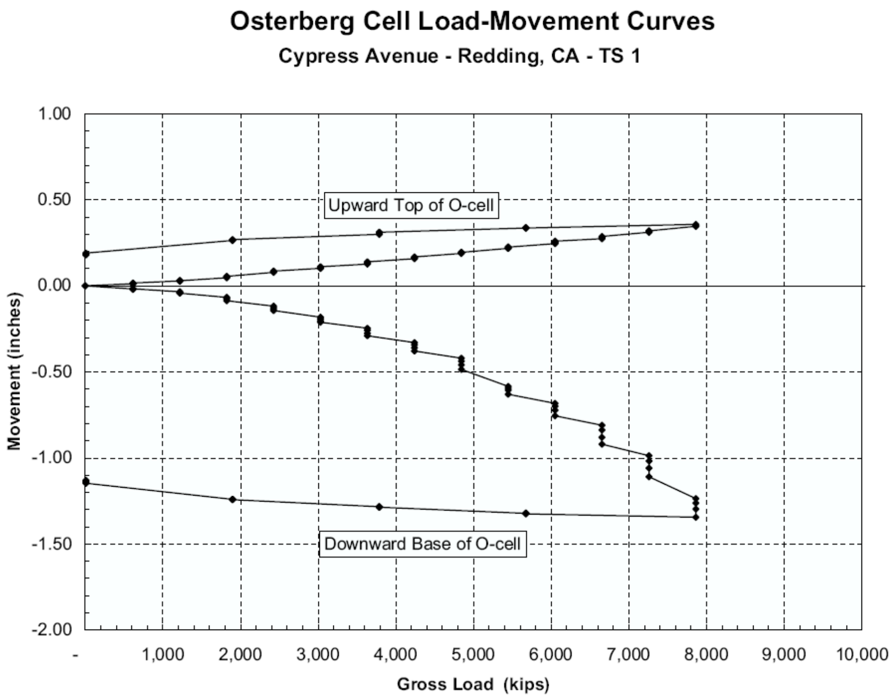


Figure 3

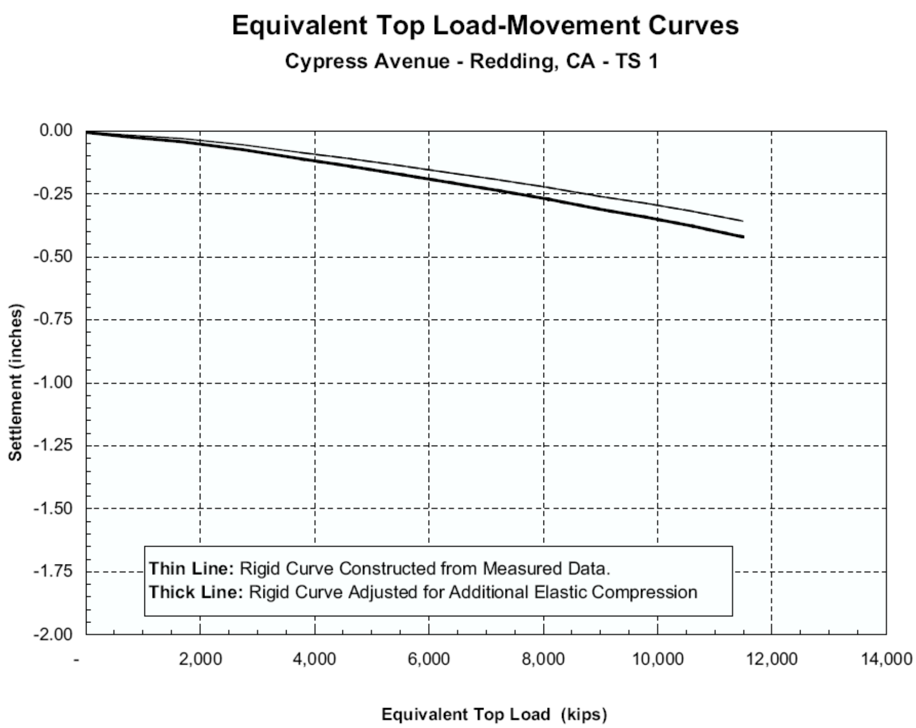


Figure 4

As can be seen in Figure 4, the upward deformation was not sufficient to reach a failure condition due to the limited capacity of the O-cell. However, the deformation was about ⅜-inch or near the specified settlement limit of ½ inch. In the downward direction, the shaft movement exceeded the specified settlement limit of ½ inch at a total load of about 5,000 kips over 16 feet of shaft length.

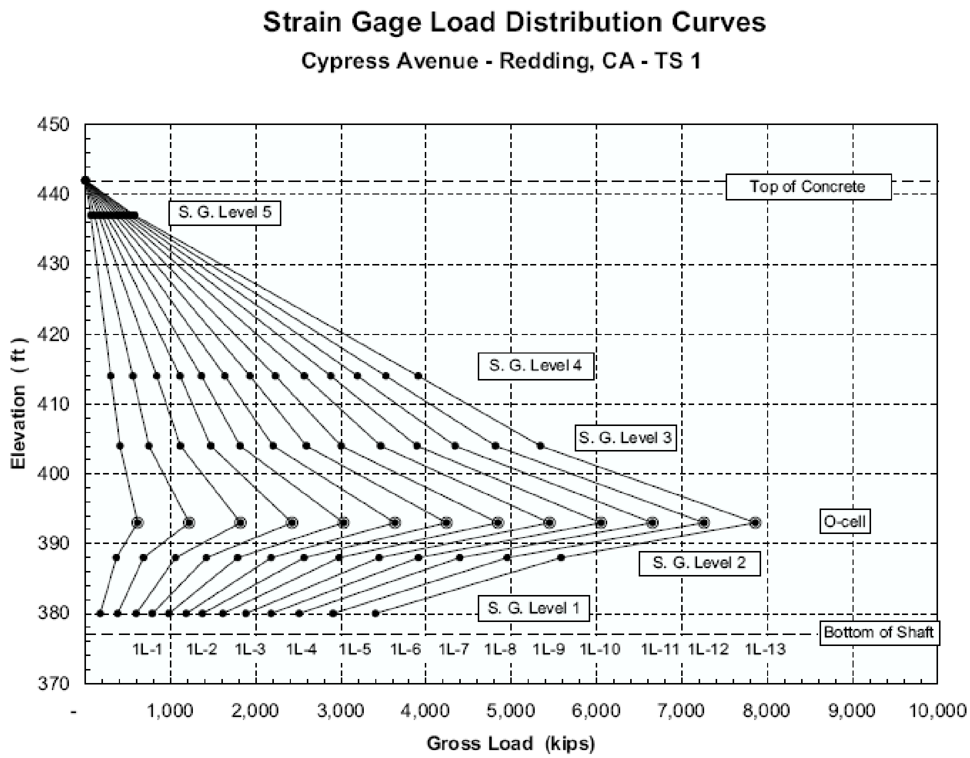


Figure 5

Figure 5 above shows the strain gauge load distribution curves. This data was used to evaluate the side shear transfer along the various intervals of the shaft. The side shear data derived from the load test is presented in Table 1 below.

Table 1
Unit Side Shear Data

Shaft Interval	Soil Condition	Depth Below Surface (feet)	Unit Side Shear (ksf)
Top of Shaft to Strain Gage Level 5	Sandy gravel river channel deposits	23 to 28	4.31
Strain Gage Level 5 to Strain Gage Level 4	Sandy gravel river channel deposits	28 to 51	5.43
Strain Gage Level 4 to Strain Gage Level 3	Sandy gravel river channel deposits	51 to 61	5.38
Strain Gage Level 3 to O-cell	Clayey gravel older alluvium	61 to 72	8.72

O-cell to Strain Gage Level 2	Clayey gravel older alluvium, weakly cemented	73 to 77	17.70
Strain Gage Level 2 to Strain Gage Level 1	Clayey gravel older alluvium	77 to 85	10.56
Note: Net unit shear values derived from the strain gages may not be ultimate values.			

In order to evaluate side shear values at similar levels of axial displacement, the side shear data from the stain gauges located below the O-cell were taken at a displacement equal to that of the upper portion of the shaft. Since the O-cells were not capable of displacing the test shaft upward an amount equal to the design settlement limit of 1/2 inch, the side shear values presented in Table 2 below were evaluated at the maximum axial displacement of about 3/8 inch.

Table 2
Comparison of Side Shear Values

Soil Type	**Axial Displacement (inches)**	**Low Unit Side Shear (ksf)**	**High Unit Side Shear (ksf)**	**Average Unit Side Shear (ksf)**
Sandy Gravel with cobbles and small boulders (GP)	3/8	4.3	5.4	5.3
Clayey gravel with cobbles and small boulders, some weak cementation (GP-GC)	3/8	5.0	12.0	8.2

As can be seen in Table 2 above, the side shear values in the lower older alluvium exhibited significantly higher side shear resistance than the upper river channel deposits. This is attributed to the higher degree of weathering, consolidation, and cohesive matrix materials present in the lower alluvium compared to the essentially cohesionless and unweathered upper river channel deposits. The presence of stiff clay matrix material in the older alluvium likely provided much higher resistance to dilation and/or rotation of the coarse particles. The presence of weak cementation further exacerbates the effect resulting in a material resembling a weak conglomerate. A much lesser degree of resistance to dilation and/or rotation of coarse particles appears to exist for the relatively clean sand matrix materials present in the river channel deposits. The difference in dilation tendencies of the cohesive and cohesionless matrix materials appears to have resulted in the concentration of stress within the cohesive matrix materials at similar levels of axial shaft displacement.

The lower side shear values obtained in the older alluvium were similar to the higher values obtained in the river channel deposits. However, the higher values in the older alluvium were more than double the values obtained in the river channel deposits. This is attributed to the presence of a weakly cemented layer at the interval between the O-cell and Strain Gauge level 1. On average, the cohesive older alluvium exhibited about 55 percent higher side shear values than the cohesionless river channel deposits. Ignoring the cemented layer, the clayey alluvium exhibited about 35 percent higher side shear values that the cohesionless river channel deposits.

Comparison of Load Test Results and FHWA Drilled Shaft Design Method

The results of Osterberg load testing revealed higher side shear transfer than anticipated based on the FHWA design method. This is attributed to the inherent difficulties of obtaining representative SPT N_{60} values in coarse grained materials containing cobbles and boulders and the conservative judgment used in selecting the appropriate SPT N_{60} values for preliminary design. For economic reasons, it is important that the shaft lengths determined in preliminary design are not shorter that the lengths eventually determined by full-scale load testing so that the funding procured for the project is not exceeded due to unconservative estimates. A discussion of the SPT N_{60} values selected for preliminary design along with back-calculated N_{60} values derived from the load test is presented in the following section of this paper. Conclusions regarding the selection of appropriate preliminary design values for use in the FHWA procedure and a discussion of appropriate subsurface exploration methods for coarse grained materials are also provided.

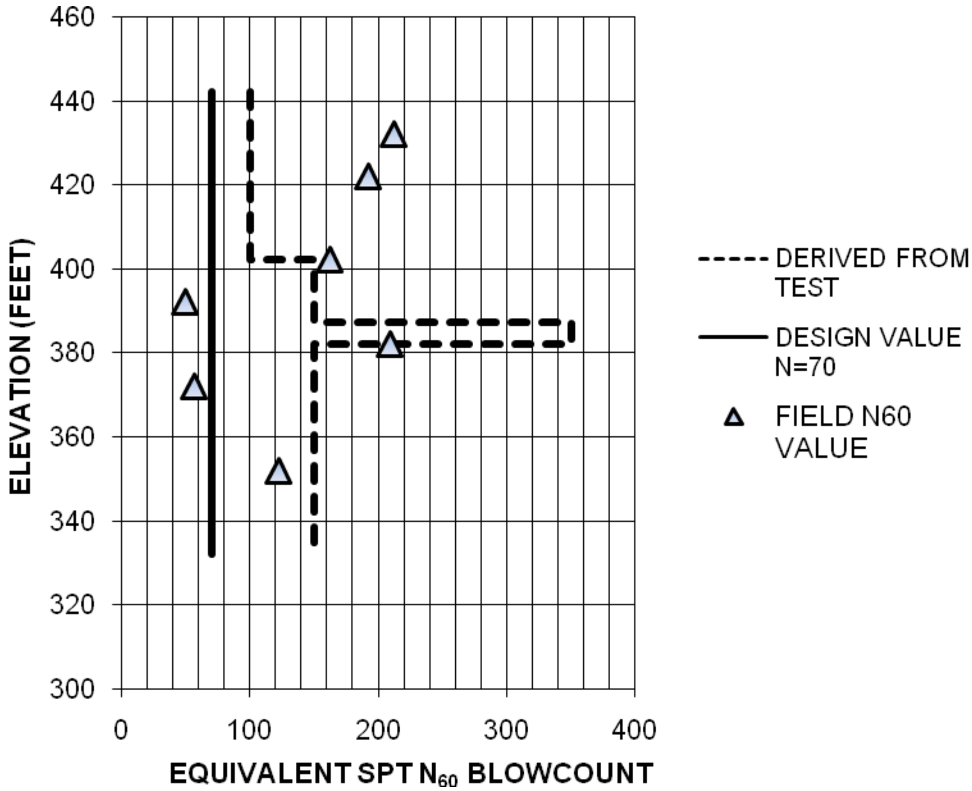


Figure 6

Figure 6 shows the SPT N_{60} values from field exploration, the selected values for preliminary design, and the values derived from the load test. The load test N_{60} values are averages between the various strain gauge levels within the test shaft. As can be seen in Figure 6, there is not good correlation between the field N_{60} values and the values derived from the load test. This is attributed to the relatively small SPT sampler bouncing on cobbles and small boulders rather than wedging between them and penetrating the matrix. Most of the SPT blowcounts shown above were within the general range of values interpreted from the load test.

When considering appropriate means for subsurface explorations in coarse materials, available methods could air percussion and casing systems as used here, Becker hammer drilling, and sonic coring. Becker hammer correlations to SPT N_{60} values have been developed by Harder & Seed (1986) and others. However, the correlations may be in error by ½ to 2 times the field values at blowcounts below about 40 and by about ten to twenty percent at higher blow counts. It would appear that the results obtained for this project from SPT samplers driven using air percussion and casing drilling methods, as shown in Figure 6, would provide less accurate N_{60} values than a Becker Penetration Test (BPT) in materials where the BPT blow counts exceed 40. Sonic coring rigs can be equipped with special SPT drive samplers and provide a continuous core of the subsurface materials. This allows for good characterization of cobble and boulder sizes, degree of weathering, and hardness. It can also aid in the interpretation of appropriate SPT N_{60} values for preliminary design analysis. This method would be superior to air percussion and casing methods that pulverize the drill cuttings and, therefore, do not allow for good characterization of cobble and boulder sizes, degree of weathering, and hardness. Where BPT is available and engineers are experienced with its operational characteristics, it could be used in dense gravel and cobble formations to get better estimates of SPT N_{60} values than typical SPT sampling methods.

Following the results of the load test, the production shafts on Piers 3 through 5 were shortened between 14 and 16 feet from the lengths specified in preliminary design. The shafts at Pier 2 could not be shortened due to lateral demands. The effect of the cemented layer within the older alluvium was neglected since lack of deep boring data within the river channel did not allow for confirmation of its presence at each bent location. The majority of the additional side shear capacity was observed within the cohesive older alluvium.

For preliminary design and construction cost estimating, the use of the FHWA cohesionless IGM approach provides reasonable side shear estimates within truly cohesionless materials but, likely underestimates side shear in materials with a cohesive matrix by a factor of about 2. Where cementation is present, the discrepancy can be much greater. Obtaining representative core samples of coarse alluvium with cobbles and boulders is, at best, difficult to perform. In moderately cemented to well cemented materials such as volcanic tuffs and conglomerates, typical rock coring techniques can be successful and provide compressive strength data that is useful in better evaluating side shear transfer using design methods for shafts in rock.

Based on the data obtained during this project, it appears that the use of SPT N_{60} values greater than 100 may result in unconservative estimates in similar cohesionless materials. The use of value over 100 in similar cohesive materials that are not cemented should be approached with caution.

For major bridge projects such as this one that employ multiple large diameter drilled shafts constructed over water, performance of Osterberg cell load testing can be critical to verify shaft performance as well as to optimize construction costs since geotechnical exploration methods cannot precisely estimate the as-constructed conditions. In this case the load test program also served as a constructability evaluation for the casing oscillator method. The construction technique for casing oscillator shafts is known to have a significant effect on shaft capacities. Keeping a positive head of water within the shaft to help prevent bottom heave can be critical. In addition, over-excavation near or beyond the tip of the casing and be detrimental. In this case about 4 feet of a soil plug within the tip of the casing was used to make sure the casing cut into the formation cleanly and did not allow relaxation around the shaft or over mining at the tip. Results of concrete over break analysis (volume of concrete placed versus theoretical volume of the shaft) was used to verify proper shaft construction.

The presence of cemented clayey alluvium was not evident in the exploratory borings drilled for the geotechnical investigation. The cemented soils pushed the selected casing oscillator equipment to its limit during excavation of the load test shaft. Use of larger equipment was precluded since the trestles for the drilling equipment had already been designed and would not be able to support the next size larger oscillator. Through collaboration with the foundation contractor, Malcolm Drilling, a solution to the penetration resistance problem in the weakly cemented soils was developed and the production drilled shafts were constructed to the final tip elevations specified. It is important to note that the test data provided herein is thought to represent a well constructed shaft using the casing oscillator method. Poor shaft construction can and likely will result in substantial reductions in side shear capacity and increases in post construction settlement. In addition, shafts constructed using auger or similar drilling methods in such coarse materials would be expected to greatly disturb coarse particles during drilling resulting in much greater particle dilatency and associated reduced capacity and/or excessive settlement.

Acknowledgements

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