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The Effect of Construction Technique on Axial Capacity of Drilled Foundations

Prepared by

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ABSTRACT: A program of field loading tests was conducted to measure the axial response of drilled foundations constructed using a variety of different drilling techniques. The research was performed at the Auburn University National Geotechnical Experimentation Site at Spring Villa, Alabama in Piedmont geology composed of silty soils formed by weathering of parent metamorphic rocks. A total of 10 drilled shafts (0.9 m diameter by 11 m deep) were constructed using techniques including dry construction with casing advanced ahead of the hole and with drilling slurry composed of polymer fluids and mineral (bentonite) fluids. The results demonstrate the great potential influence that differing construction techniques may have on the load transfer in side shear of drilled foundations. The mineral slurry resulted in significantly lower side shear relative to the other techniques.

INTRODUCTION

The axial capacity of drilled foundations is affected by the conditions at the soil/concrete interface immediately adjacent to the shaft. O'Neill (2001) has shown that the effect of details related to construction can be significant and includes such factors as stress relief, moisture migration from the concrete to the geomaterial, borehole roughness, and borehole smear. The relative importance of various details are dependent upon geological conditions; for instance, soft cohesive rock appears to be particularly sensitive to construction techniques.

The influence of construction drilling fluids on the resulting axial capacity of drilled shaft foundations is reviewed by O'Neill and Reese (1999). Some case histories suggest that the buildup of an excessive filter cake from bentonite mineral slurry is detrimental to capacity (Holden, 1984; O'Neill and Hassan, 1994). However, it is commonly accepted in the industry that limited exposure times (on the order of 4 hours or less) with a properly controlled bentonite slurry leaves only a thin filter cake which does not have a significant effect on axial capacity. Some data in clayey soils in which a substantial filter cake would not be expected to form due to the low fluid loss (Cooke, 1979; Camp et al, 2002) suggests that shafts constructed under bentonite suffer no detrimental effects compared to similar shafts constructed in dry holes.

Synthetic polymer drilling fluids (most commonly composed of partially hydrolyzed polyacrylamide, or PHPA) are long chain polymer molecules which tend to increase the viscosity of the fluid but do not tend to build a filter cake. The "slippery" texture of this material would appear to suggest that this slurry could lubricate the interface between the concrete and soil. However, the lime in the concrete produces a high pH which is thought to break down the polymer and eliminate any such concerns. It seems plausible that some polymer strands could penetrate into the pores of the surrounding soil with unknown effects. Numerous case histories of load test data on shafts constructed using polymer slurry suggest that these materials do not produce any systematic reduction in axial capacity (e.g., O'Neill and Hassan, 1994) and some comparative test data suggest that axial capacity of shafts in granular soils constructed with polymer fluids may compare favorably with similar shafts constructed using bentonite (Majano, et al, 1994; Meyers, 1996). Recent comparative tests in the Charleston, SC area reported by Camp, et al (2002) showed no significant differences between shafts constructed with polymer and bentonite fluids in a calcareous marl formation.

The influence of temporary casing on drilled shaft performance has also shown mixed results. Model shafts constructed under closely controlled conditions (Majano et al, 1994) exhibited greater capacity from shafts constructed in granular soils with casing advanced ahead of the shaft excavation compared with those constructed in a slurry filled hole. Possible mechanisms for such behavior include the tendency for higher *in-situ* lateral stresses and/or densification from installation of the casing in advance of excavation. However, Camp et al (2002) report significantly reduced side shear capacity for shafts constructed using casing advanced ahead of the excavation in a cohesive soil, apparently due to a reduction in sidewall roughness compared to similar shafts excavated under slurry.

The research described in this paper investigates the effects of some aspects of construction details relating to drilling techniques in Piedmont residual soils. The residual soils of the Piedmont geology are derived from chemical weathering of in-place metamorphic rocks, predominantly gneisses and schists of early Paleozoic Age or older (Sowers and Richardson, 1985). The Piedmont covers a wide area of the southeastern United States between the Atlantic Coastal Plains and the Blue Ridge Mountains, extending from Alabama to Pennsylvania. Piedmont soils typically have high silt content, often classifying as ML-CL or ML-SM and frequently contain mica, feldspar, and other non-quartz minerals.

Drilled foundations are a common foundation type in the Piedmont geology. Gardner (1987) describes typical design procedures for drilled shafts in decomposed rocks of the Piedmont, and Mayne et al discuss design using *in-situ* techniques (1999, 2001) in Piedmont residual soils. Harris and Mayne (1994) report measured side resistance of around 70 kpa from tests on two drilled shafts constructed using casing (and dry holes) in Piedmont soils in Atlanta. Burke (2001) reports average measured side resistance of around 120 kpa from a series of tests on drilled shafts using a variety of construction techniques in Virginia Piedmont soils.

The objective of the research described in this paper was to investigate the influence of drilled shaft construction techniques on the axial response of drilled foundations in Piedmont soils. The approach used was to construct a number of full size drilled shafts using a variety of possible construction techniques and to conduct axial static loading tests on these test foundations.

SITE CONDITIONS

The location of the field testing program was the Auburn University National Geotechnical Experimentation Site (NGES) at Spring Villa, Alabama. This site has been well documented and the subject of a wide range of geotechnical investigation techniques (Brown and Vinson, 1998; Mayne, et al;, 2000). Within the upper 15 m which is the subject of this study, the soils are relatively consistent across the site on a large scale. On a small scale, there is significant spatial variability as the soils retain the foliation and structural features of the parent rock.

The soils typically classify as micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams. Average soil classification data are as follows: water content = 34%; grain size: 47% sand, 33% silt, 10% clay; LL=46, PI=10. Clay content typically is somewhat higher in the upper 2 m due to a more advanced state of weathering at shallow depths. Standard penetration test values were typically 8 to 14 blows/30cm over the depth range of 2 to 15 m with a mean of 12 b/30cm. CPT tip resistance was typically around 3 to 4 MPa, with a friction ratio of around 4 to 6%. Piezocone CPT tests indicated that excess pore pressures dissipated back to static levels within a couple of minutes. Laboratory test data from a large number of CU (and a few CD) triaxial tests suggest an effective cohesion intercept, c' = 17 kPa and an effective friction angle of 32 degrees. Undrained shear strength varied widely in the upper few meters but averaged around 92 kPa. Groundwater fluctuates somewhat but was generally encountered at a depth of around 4 to 5 m at the time of testing.

CONSTRUCTION OF THE TEST SHAFTS

A total of 10 drilled shafts were constructed for this project, at the locations shown on Figure 1. In addition to these shafts, a single displacement-type continuous flight auger (CFA) pile was also included for comparison because of the different construction methodology with respect to conventional drilled shaft installation. All the shafts were 0.9 m diameter by 11 m deep, and the CFA pile was 0.45 m diameter by 11 m deep. Two of the shafts were constructed using bentonite slurry, four were constructed using polymer slurry, and four were constructed using temporary casing advanced ahead of the shaft excavation. All shafts were constructed with a concrete mix with 13 mm (½ inch) maximum size gravel aggregate, a slump of 175 to 225 mm (7 to 9 inch), and a design compressive strength of 28 Mpa (4 ksi) . Reinforcing steel was composed of ten

#9 (29 mm) longitudinal bars extending full length and #4 (13 mm) hoops on 0.3 m spacing.

Slurry Shafts

The slurry shafts were constructed using a truck-mounted Texoma drilling rig with a conventional soil auger. A short piece of surface casing was used in the upper 1 to 2 m and extending approximately 1 m above ground to facilitate testing. The bottom of each shaft was cleaned successively using a clean-out bucket and an air-lift pipe. The shaft base was inspected by feel using a weighted tape with a short (approx. ½ m) piece of #18 (58 mm) rebar to verify that loose material was removed and a firm feel to the sounding tool was present. Samples of slurry for testing were obtained prior to concrete placement from near the base of the shaft.

All the slurry materials were obtained from the same supplier, Bariod, of Houston, Texas. The shafts designated with a "B" were constructed using a high grade Wyoming commercial bentonite. The shafts designated with "DP" were constructed using a polymer in a dry pellet form, and those designated with "LP" were constructed using a polymer in a liquid form. Both polymers were PHPA, but the liquid form includes an emulsifying agent for ease of mixing. Each of the three slurry materials was mixed in a separate tank at least 24 hours prior to use and added to the hole prior after drilling dry through the upper few meters and prior to encountering groundwater. Excavation of the shafts took only around 2 hours. The shafts are designated with "1" or with "24" to indicate the exposure time to the slurry after completion of excavation and prior to concrete placement. The "1" designation indicates approximately one hour of exposure, which was generally between 1 and 2 hours. The "24" designation indicates an overnight exposure which ranged from 18 to 24 hours. Slurry levels in the holes were not observed to fall significantly during the exposure periods. Concrete was placed via a bucket through a 0.25 m diameter tremie pipe. Construction details of the slurry shafts are provided on Table 1.

Cased Ahead Shafts

Four of the shafts (all designated with a "C") were constructed using temporary casing advanced ahead of the excavation. The casing was a segmental double walled, heavy steel casing with cutting teeth on the bottom and which could be rotated independently of the auger inside. The cutting teeth made a hole which was a few mm larger than the outside diameter of the casing. The casing was advanced (by rotating) for several meters, then the soil inside was drilled out, always maintaining at least 2 m of soil inside the casing until the bottom of the shaft was reached. At the prescribed depth of the bottom of the shaft, the auger excavated the remaining soil to the bottom of the casing. Two of the cased-ahead shafts were constructed using the overnight exposure in which the casing was left in place and concrete placed the next day. In these shafts (designated with a "24"), a 2 to 3 m soil plug was left inside the casing to be removed shortly before completion. Groundwater was observed to seep into the hole (through this soil plug) overnight, and was at a depth of around 6 m the next day (deeper than the groundwater level outside the shaft excavation). After removal of the soil plug, a very small amount of seepage (less than 50 mm in the bottom of the hole) was noted in the 1 hour or so until concrete placement on either the overnight shafts or the shafts completed immediately.

Two of the shafts were constructed with intentional soil inclusions. These inclusions were formed by filling sandbags with soil from the excavation and tying these bags to the rebar cage. The two shafts with soil inclusions are designated "Def", as in "1CDef" for a shaft constructed using casing with soil defects and a 1 hour exposure time prior to concrete placement. Each of the two defect shafts had two soil inclusions. Each soil inclusion was approximately 0.6 m in height and had a cross sectional area of either 10% or 20% of the cross sectional area of the shaft. Shaft "1CDef" had a 20% inclusion defect at a depth of approximately 4 m and a 10% defect at a depth of approximately 8 m. Shaft "24CDef" had similar inclusion defects but with the smaller at the shallower depth. The defects were placed towards the edge of the shaft extending outside the rebar cage. Several consultants performed non-destructive integrity tests on the shafts at this site and were all very successful in detecting these soil inclusions using crosshole techniques but unsuccessful using sonic echo (top of shaft) techniques.

Concrete was placed using free fall into the dry holes. After the excavation was filled to the top with concrete, the casing was withdrawn by the rig, with a back and forth twisting motion through about 30° of rotation while pulling. After withdrawal of the casing, a short surface form was placed and filled with concrete to extend the shafts approximately 1 m above grade. Concrete properties of the cased-ahead shafts are given in Table 2.

Displacement CFA Pile

The one pile noted "CFA" was a 0.45 m diameter displacement pile constructed using continuous flight augers. This pile was constructed as a part of a study reported by

Brown and Drew (2000) and is included in this paper for comparison with the drilled shafts. The pile was constructed by twisting the hollow stem augers into the ground and filling the 250 mm (10 inch) hollow center with concrete during withdrawal. The concrete mix used for these piles was similar to that of the drilled shaft concrete. A sacrificial steel shoe is placed over the bottom to prevent soil from entering the hollow center. The augers used were of a special type with (trade name "DeWaal") and designed to displace the soil laterally rather than withdraw the soil from the hole. The auger string extended only for the lower 2 m or so and above this level is a steel bulge which forces the soil laterally. Above this bulge are auger flights in reverse, designed to force any soil downward rather than up towards the surface. This pile provides an interesting comparison with conventional cast-in-place shafts at this site, because it is effectively a cast-in-place displacement pile. After full depth was achieved, the concrete was pumped to a hopper atop the rig and placed into the pile by free fall. Augers were withdrawn after a full head of concrete was developed within the auger string, and additional concrete added as the auger string was withdrawn. Only a nominal amount of soil cuttings (less than ½ m^3) was withdrawn from this hole upon completion. The CFA pile was reinforced with a single #11 (35 mm) bar through the center extending full length and a rebar cage of six #6 (19 mm) bars in the upper 5 m of the pile. The center bar was placed through the augers prior to concrete placement and the rebar cage placed into the fluid concrete after the augers had been removed. Properties of the concrete for this pile are included in Table 2.

MEASURED AXIAL PERFORMANCE AND COMPARISONS BETWEEN CONSTRUCTION TECHNIQUES

Static axial compression load tests of all test shafts were conducted in a similar manner. The test shaft was loaded in compression by hydraulic jacking against a reaction frame. An electric pump was used to supply pressure to the jack. The reaction frame utilized 4 CFA piles acting in tension, each at a radial distance of 4.6 m from the test shaft. The load was applied in increments of approximately 200 to 300 kN, and each load increment was held for a period of 5 minutes. Total testing time was generally around one hour. An electronic data acquisition system (Megadac, from Optim Electronics) recorded data at 10 second intervals during the entire test. The measurements included load from a load cell, displacement from two linear potentiometers at the top of the test shaft, and up to 12 strain gauges within the test shaft. Manual recordings were taken for backup of hydraulic pressure on the calibrated jack and dial indicators on the test shaft and reaction piles.

Load vs displacement plots for the shafts are provided on Figures 2 through 4. All the shafts were loaded to what appears to be a plunging type failure. The polymer shafts, shown on Figure 2, appear to exhibit a notable strain softening response. Shaft 24DP in particular shows a significant reduction in load transfer at displacements exceeding about 10 mm. These data may appear unusual in comparison to routine load test measurements in which manual observation of dial indicators is recorded; however, the data acquisition system captures the true load deformation response of the shaft as plunging failure initiates. The plot shown represents all the data measured, and not just the points after a 5 minute hold period. The peak loads achieved for shaft 24DP occurred during the transient loading after a 5 minute hold at 2000 kN; the shaft was not capable of reaching the next planned hold period, and when plunging failure occurred the residual load dropped to somewhat less than the 2000 kN which had been sustained. Other polymer shafts exhibit smaller amounts of strain softening.

The bentonite shafts are characterized by a smaller load capacity than the polymer shafts and a rather abrupt change in the slope of the load – deflection plot at around 5 mm of displacement. These shafts did not exhibit strain softening, but rather gained a small amount of resistance at large displacement.

The cased shafts exhibit a significant increase in resistance with increasing displacement, notably different from the slurry shafts. There does not appear to be a significant difference between the shafts with soil inclusions and the two which are free of defects. In fact, the two shafts with soil inclusions are slightly stronger (but probably within the range of normal site variability).

The shafts were instrumented with strain gauges, which were mounted on 1.2 m long #4 (13 mm) bars and tied into the rebar cage. These gauges were installed in pairs at each of 6 levels, but were concentrated toward the top of the shafts for subsequent lateral load testing. Although a few gauges were somewhat erratic, the general trend is typified by the measurements illustrated on Figure 5. These measurements suggest that most of the load was carried in side shear, with only modest loads reaching the base of the shaft.

The strain gauge data for the test shafts were evaluated at selected load intervals in order to estimate the distribution of soil resistance. Because of uncertainties relating to strain measurement errors, variations in shaft modulus, limited data vs depth, etc., the strain measurements vs depth were fitted with a best fit linear strain vs depth relationship.

This best fit was then used to estimate the average unit side shear along the length of the shaft and the soil resistance in end bearing from the projection of this best fit to the shaft toe. The resulting unit side shear values were not adjusted for any "shadowing" effect above the toe or for any reduction near the surface, but rather represent the total estimated side shear load divided by the circumference times the length.

The computed unit side shear vs displacement is shown on Figure 6, where shafts with similar construction techniques are averaged to facilitate comparisons. Also shown is the response of the displacement CFA pile. This pile did not have the level of instrumentation of the test shafts, and so the tip resistance is estimated to be 10% of the total resistance, assumed to be mobilized at a displacement equal to 5% of the pile diameter (about 23 mm in this case). In this manner, the unit side shear for the CFA pile is estimated for comparison with the test shafts.

The data shown on Figure 6 demonstrate the substantial differences in the soil resistance in side shear relating to construction technique. The polymer shafts exhibit the largest peak side resistance, with a somewhat strain softening response. The liquid polymer achieved somewhat higher resistance than did the dry polymer, a result which is surprising because the two polymers appear similar after mixing. The cased – ahead shafts exhibit a similar response to the polymer slurry shafts initially and at large displacements, but without the higher peak values. The single displacement CFA pile had slightly lower side shear resistance, but probably comparable to the cased ahead shaft. It is interesting to note that the benefit one might expect from displacing the soil laterally in the displacement CFA pile did not materialize in this case. See Brown and Drew (2000) for additional discussion of CFA pile testing at this site). The bentonite shafts had significantly reduced side shear capacity relative to other construction techniques.

Shown on Figure 7 are the estimated unit end bearing values as a function of displacement for the shafts. The CFA pile is not shown, since this tip response was not determined from analysis of strain measurements for that pile. All the shafts had increasing tip resistance up to the maximum measured values at around 5% of the shaft diameter (which corresponds to .045 m in this case). The bentonite shafts had somewhat lesser end bearing resistance; note that although the line shown is an average of the two bentonite shafts, these two shafts were virtually identical in their end bearing response. It might be speculated that bottom hole cleanliness could have affected these results, although the sounding inspection used during construction did not suggest that these shafts were any different than the polymer shafts.

POST MORTEM INSPECTION

A few months after completion of testing, a backhoe was used to excavate to a depth of around 3 to 3.5 m alongside several of the shafts. This afforded an opportunity for the author to inspect the conditions at the shaft/soil interface which resulted from the different construction techniques. The differences observed were striking.

The shaft installed using bentonite (24B) had a thin seam of easily identifiable bentonite separating the native residual soil from the shaft sidewall. The shaft itself was straight, in good structural condition, and fairly rough. The native residual soil has thin seams of differing mineralogical composition which are folded and maintain the appearance of the parent rock from which it is derived and is thus quite distinctly different from the bentonite film at the concrete interface. The soil was easily dislodged from the face of the concrete with a small shovel to reveal the concrete surface. The bentonite filter cake appeared to be approximately 1 to 3 mm thick.

Inspection of the shaft installed using the polymer slurry (24DP) revealed no distinct film at the interface between concrete and soil. In fact, it appeared that the cement paste had penetrated the pores within the soil so as to render the interface between soil and concrete indistinct. The author scraped the concrete with a sharp tool in an attempt to remove soil, but it was not easy to delineate the concrete surface.

Cased-ahead shaft "1Cdef" was examined in a similar manner, although the excavation did not penetrate deep enough to reveal the soil inclusion. The soil was removable from the concrete face, although there was no distinct film as with the bentonite. The concrete surface was smooth on a small scale, but with a superimposed herringbone pattern left by the cutting teeth as the casing was twisted back and forth during withdrawal. This pattern on the concrete surface suggests that a rough side shear condition existed during load testing. Soil disturbance and remolding was notable in the near field within about 5 to 15 mm adjacent to the shaft concrete. The soil coloration and structure were noticeably sheared from the rotation of the casing during installation, leaving this remolded zone of soil at the soil/concrete interface.

COMPARISONS WITH DESIGN GUIDELINES

It is instructive to compare the results with the recommendations used for routine design of drilled shafts in soil, as the Piedmont residual soils at this site are not easily characterized as either sands or clays (the only soils recognized in many textbooks).

Because Piedmont silty soils are intermediate between clay and sand, in practice design may include both a total stress analysis and an effective stress analysis. The computed values of axial unit side shear and tip resistance for drilled shafts using the FHWA guidelines (O'Neill and Reese, 1999) are provided in table 3. For side shear in cohesive soil, the current FHWA guidelines suggest a unit side shear, $f_{max} = \alpha S_u$, where $\alpha = 0.55$ and S_u = average undrained shear strength, = 92 kPa in this case. Computed unit end bearing for cohesive soils is taken as $9S_u$. For cohesionless soil, the current FHWA guidelines suggest a unit side shear, $f_{max} = \beta\sigma'_v$, where $\beta = 1.5 - 0.245[z]^{1/2}$, z is the mid height depth in meters, and σ'_v is the effective vertical stress at mid height. Computed unit end bearing for cohesionless soils is taken as $57.5N_{SPT}$, kPa, where N_{SPT} is the standard penetration test resistance (blows/0.3m) within 2 diameters below the base (around 14 b/0.3m in this case). These guidelines are based on the computed unit end bearing mobilized at a displacement of approximately 5% of the base diameter.

There are a great many predictive methods which could be evaluated, including the use of cone penetration, pressuremeter, alternative effective stress methods, and others, but such an exercise is not the primary objective of this paper. The data presented in table 3 serve to illustrate that the measured values in these Piedmont soils are reasonably consistent with expected values for other soils with similar engineering properties.

SUMMARY AND CONCLUSIONS

A series of axial load tests on drilled shafts in a Piedmont residual soil are reported. Varying construction techniques were used to install the shafts in an attempt to identify the significance of installation technique on performance under axial load. Excavation and visual inspection of the concrete/soil interface provided additional insight into the effects of installation technique on side shear resistance. For soil conditions similar to this site, the following conclusions are suggested by this research:

- In these fine grained silty soils, the shafts installed using bentonite slurry had a reduced capacity compared to other installation techniques. This effect appears to be largely related to the presence of a thin film of bentonite left at the concrete/soil interface as a result of filter cake formation during drilling. This observation seems consistent with several other limited studies in granular soils, but the surprising finding at this site was that the effect seemed to be pronounced even for shafts with very limited exposure times. This effect may not extend to soils of lower hydraulic conductivity, which may reduce the tendency for filter cake formation.
- The polymer slurry materials appeared to promote an excellent bond between the concrete and soil. There was a distinct tendency for shafts constructed using these materials to exhibit strain softening behavior, although the mechanism for this effect is unclear. The strain softening may be related to dilatant behavior of the relatively undisturbed Piedmont residual soil in the near field around the shaft during shear.
- The use of casing advanced ahead of the shaft excavation resulted in axial shaft capacity which was comparable to that of the polymer shafts, but without the strain softening tendency. The rotation of the casing during

installation was observed to remold and distort the soil in the near field around the shaft and the casing produced a smooth concrete/soil interface. However, the twisting of the casing during extraction and the cutting teeth on the bottom of the casing left a rough macro-texture on the sidewall. It may be speculated that the effects of casing installation might be more severe in a soil with higher clay content in which the effects of remolding may be more pronounced, but less so in a more granular soil. The surface texture left by the cutting teeth appeared to be beneficial; a smooth steel casing which might be extracted by a vibro-hammer could produce less desirable side shear capacity, particularly in cohesive soils (see Camp, et al, 2001).

- The occurrence of soil inclusions with cross sectional area of up to 20% of the shaft cross section had no effect on the short term performance under axial load. It should be noted, however, that the maximum load divided by the gross cross sectional area for these shafts was only around 4300 kPa, and so structural failure of the shaft column was not an issue at these relatively low stresses.

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REFERENCES

Brown, D.A. and Vinson, J. (1998). "Comparison of strength and stiffness parameters for a Piedmont residual soil," *Geotechnical Site Characterization,* ISC'98, P.K. Robertson and P.W. Mayne, eds, Vol. 2, Balkema, Rotterdam, 1229-1234.

Brown, D.A. and Drew, C. (2000). "Investigation of Axial Capacity of Augered Displacement Piles at the Auburn University Test Site" *New Technological and Design Developments in Deep Foundations, GSP 100,* N.D. Dennis, R. Castelli, and M.W. O'Neill, eds., ASCE, New York, N.Y., pp. 397-403.

Burke, E.C. (2001). "Drilled Shafts in the Virginia Piedmont" *Foundations and Ground Improvement, Geotech. Spec. Publ. 113,* T. L. Brandon, ed., ASCE, Reston, VA, pp. 175-189.

Camp, W.M., Brown, D.A., and Mayne, P.W. (2002). "Construction Methods Effects on Drilled Shaft Axial Performance," *Deep Foundations 2002, Geotech Spec. Publ. 116,* M.W. O'Neill and F. C. Townsend, eds., ASCE, Reston, VA, pp. 193-208.

Cooke, R.W. (1979). "Load Transfer from Bored, Cast-*in-Situ* Piles in London Clay," *Behavior of Deep Foundations*, STP 670, ASTM, Philadelphia, PA, pp. 250-263.

Harris, D.E., & Mayne, P.W. (1994). "Axial compression behavior of two drilled shafts in Piedmont residual soils," *Proceedings, International Conference on Design and Construction of Deep Foundations, Vol. 2*, Federal Highway Administration, Washington, D.C., 352-367.

Holden, J.C. (1984). "Construction of Bored Piles in Weathered Rock, Part 4: Bentonite Construction Procedures," *Technical Report No. 6*, Road Construction Authority of Victoria, Australia.

Majano, R.E., O'Neill, M.W., and Hassan, K.M. (1994). "Perimeter Load Transfer in Model Drilled Shafts Formed Under Slurry," *Journal of Geotechnical Engineering,* ASCE, Vol. 120, No. 12, pp. 2136-2154.

Mayne, P.W., Brown, D.A., Vinson, J., Schneider, J.A. and Finke, K.A. (2000). Site characterization of Piedmont residual soils at Opelika, Alabama. *National Geotechnical Experimentation Sites, Geotech. Spec. Publ. 93*, ASCE, Reston, VA, pp. 160-185.

Mayne, P.W., Martin, G.K. and Schneider, J.A. (1999). "Flat dilatometer modulus applied to drilled shaft foundations in Piedmont residuum," *Behavioral Characteristics of Residual Soils Geotech. Spec. Publ. 92*, ASCE, Reston, VA, pp. 101-112.

Mayne, P.W. & Schneider, J.A. (2001). Evaluating axial drilled shaft response by seismic cone. *Foundations & Ground Improvement, Geotech. Spec. Publ. 113*, ASCE, Reston, VA, pp. 655-669.

Meyers, B. (1996). "A Comparison of Two Shafts: Between Polymer and Bentonite Slurry Construction and Between Conventional and Osterberg Cell Load Testing," Paper Presented at the Southwest Regional FHWA Geotechnical Conf., Little Rock, AR, April.

O'Neill, M.W. (2001) "Side Resistance in Piles and Drilled Shafts," the 34th Terzaghi Lecture, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 1, pp. 1-16.

O'Neill, M.W. and Hassan, K.H. (1994). "Drilled Shafts: Effects of Construction on Performance and Design Criteria," *Proceedings*, Int'l Conf. on Design and Construction of Deep Foundations, U.S. Federal Highway Admin., Washington, D.C., Vol. 1, pp. 137-187.

O'Neill, M.W. and Reese, L.C. (1999). "Drilled Shafts: Construction Procedures and Design Methods," Publication No. FHWA-IF-99-025, U.S. Dept. of Transportation, Federal Highway Admin., Washington, D.C., 535p.

Sowers, G.F. & Richardson, T.L. (1985). Residual soils of the Piedmont and Blue Ridge, Transportation Research Record No. 919, National Academy Press, Washington, D.C., 10-16.

		1B (Bentonite)	24B (Bentonite)	1DP (Polymer)	24DP (Polymer)	1LP (Polymer)	24LP (Polymer)
Slurry	Density (g/cc)	1.04	1.06	1.05	1.01	1.01	N/A
	Marsh Funnel Viscosity (seconds/quart)	52	52	57	44	46	47
	Sand Content	2%	1.5%	1%	1%	0.25%	N/A
	PH	10	10	9	10	10	10
Concrete	Age at Load Test	43 d	47 d	42 d	41 d	47 d	43 d
	Compressive Strength	40 MPa (5.9 ksi)	35 MPa (5.1 ksi)	45 MPa (6.6 ksi)	36 MPa (5.2 ksi)	32 MPa (4.8 ksi)	34 MPa (5.0 ksi)

Table 1 Slurry Shaft Properties

		1C	24C	1CDef	24CDef	CFA
Concrete	Age at Load Test	29 d	33 d	36 d	34 d	25 d
	Compressive Strength	33 MPa (4.8 ksi)	37 MPa (5.3 ksi)	36 MPa (5.3 ksi)	38 MPa (5.5 ksi)	20 MPa (2.9 ksi)

Table 2 Cased-ahead Shaft and Displacement CFA Pile Properties

Method	Unit Side Shear, kPa	Unit End Bearing, kPa
Measured average (excluding bentonite)	55	850
FHWA 1999 guidelines, cohesive soils	51	830
FHWA 1999 guidelines, cohesionless soils	82	800

Table 3 Measured and Computed Unit Load Transfer Values

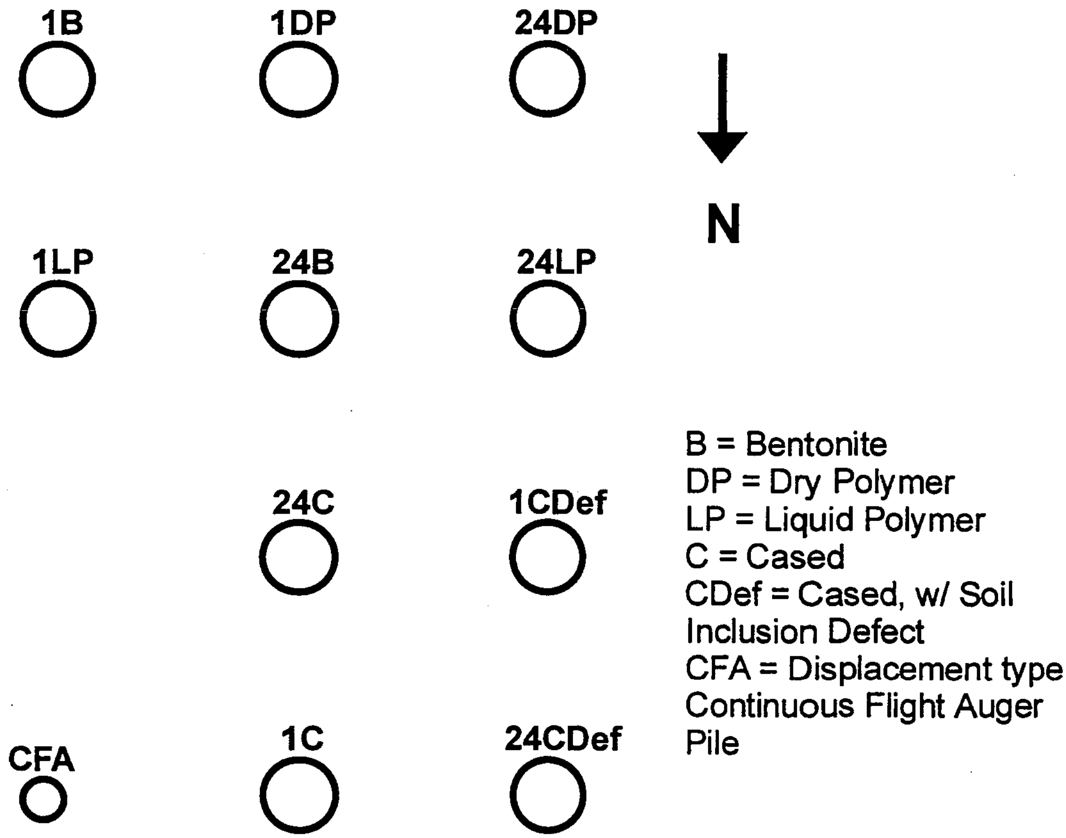

Figure 1 Test Site Layout (no scale)

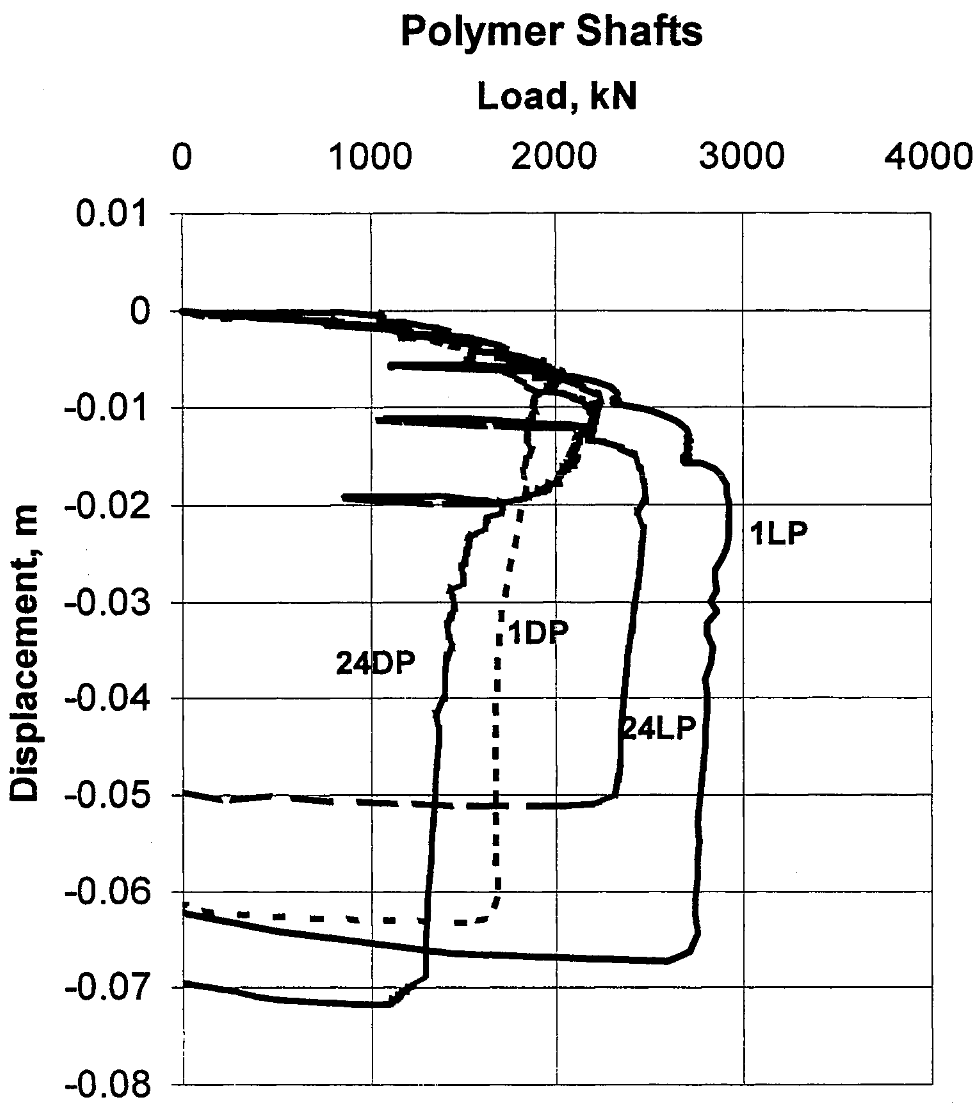

Figure 2 Load – Deflection Response of Polymer Shafts

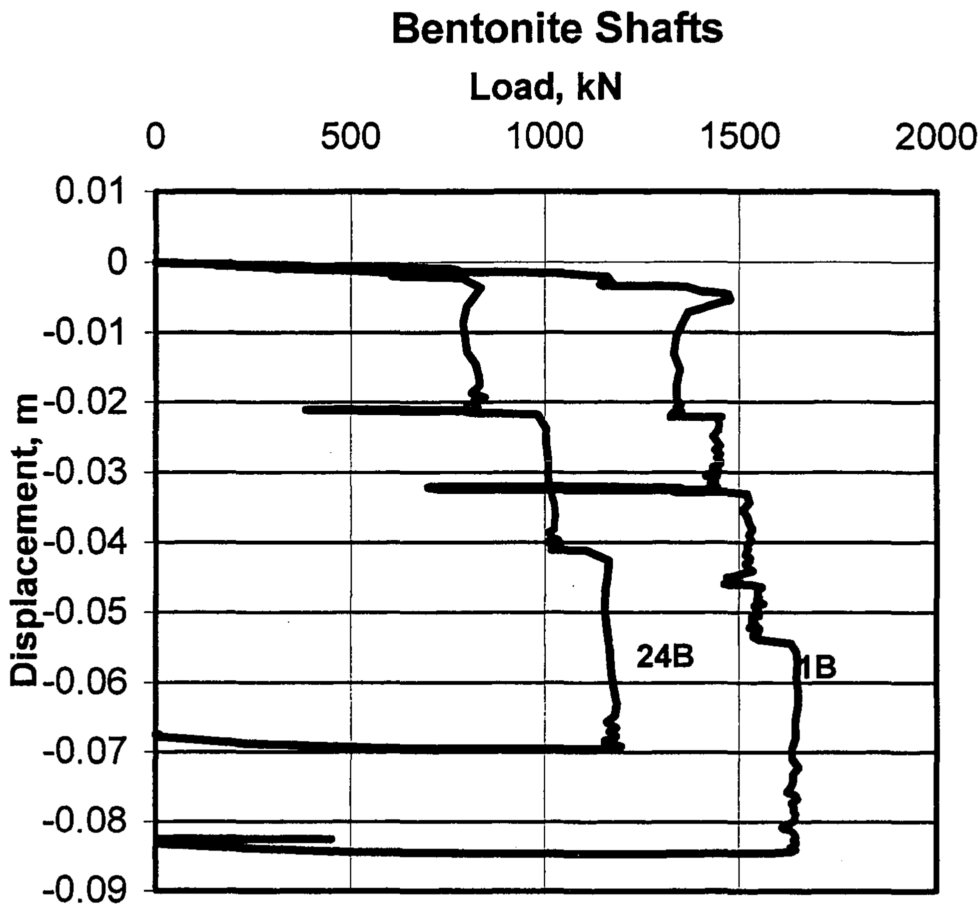

Figure 3 Load vs Deflection Response of Bentonite Shafts

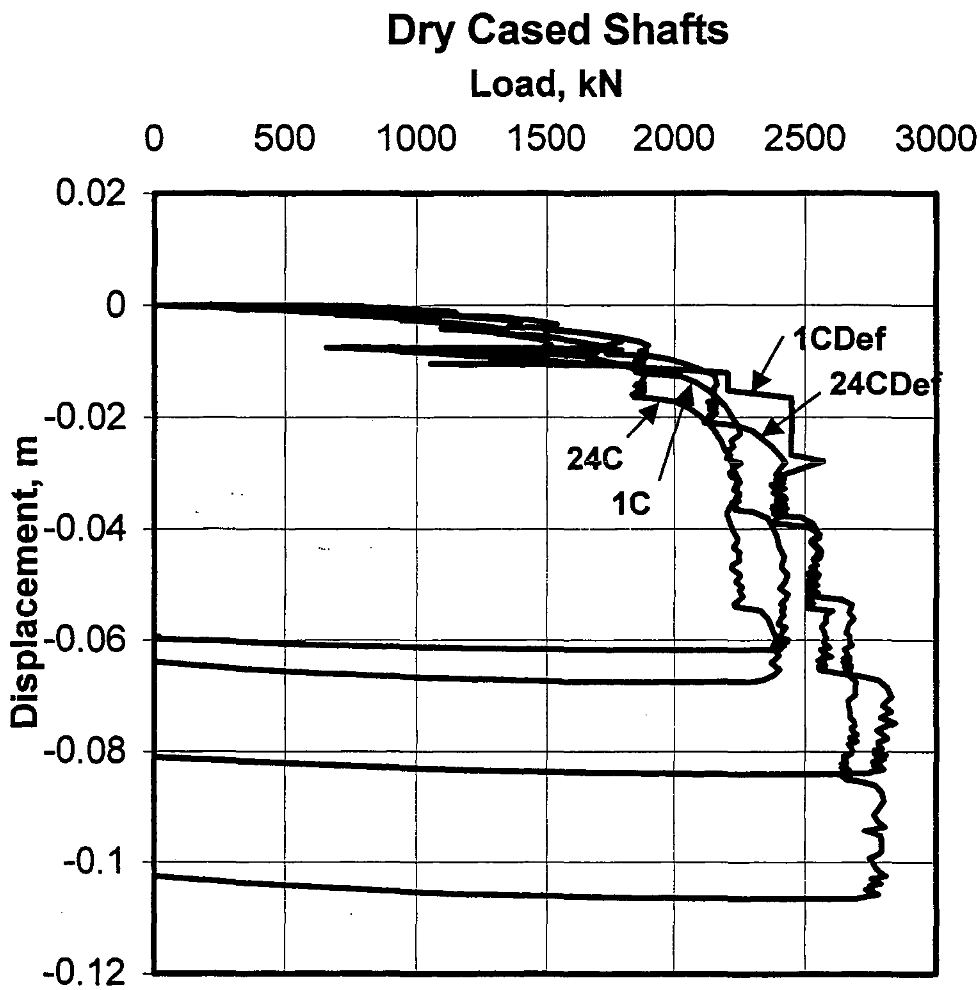

Figure 4 Load vs Deflection Response of Dry Cased Shafts

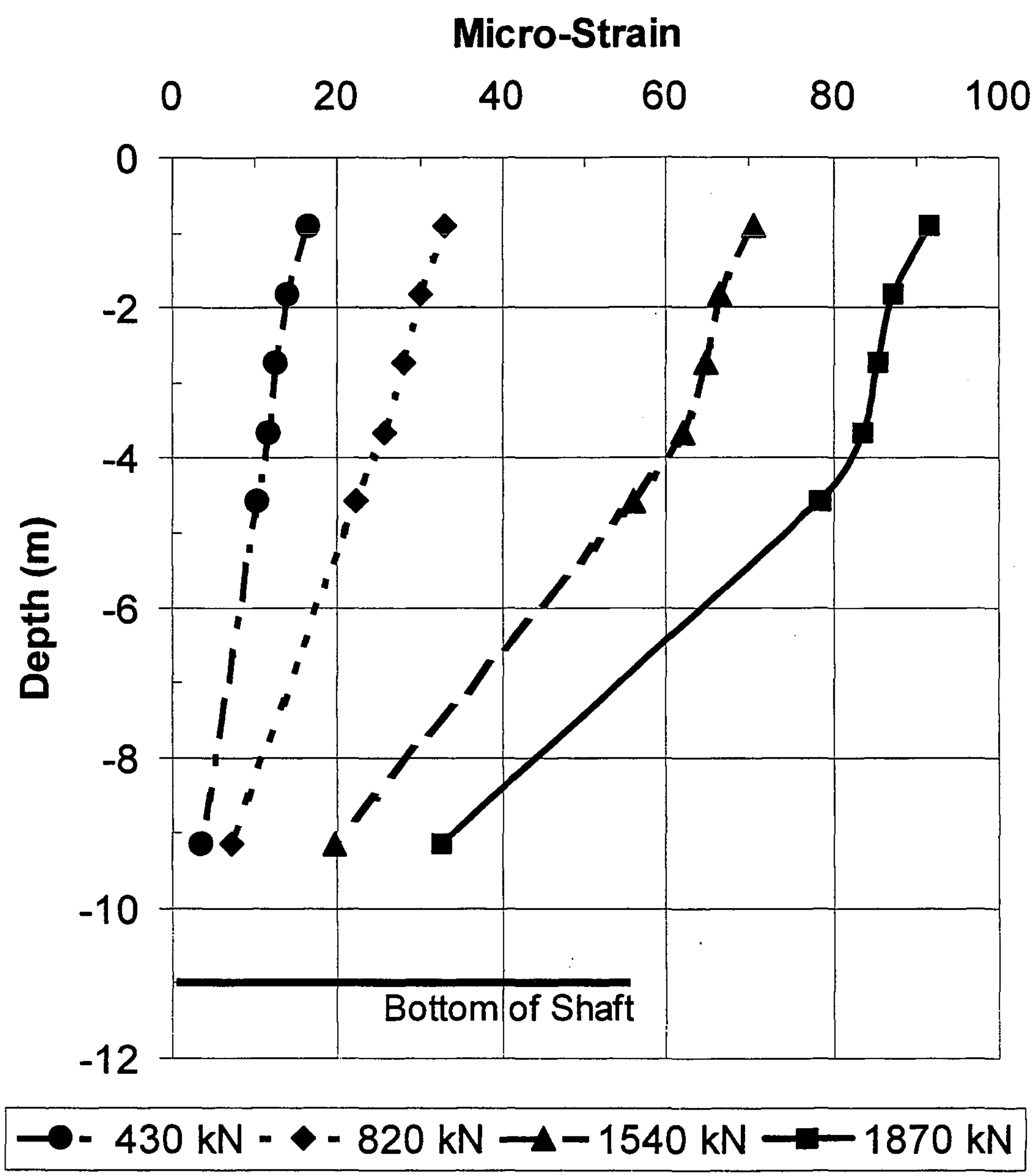

Figure 5 Strain Distribution, Shaft 1DP

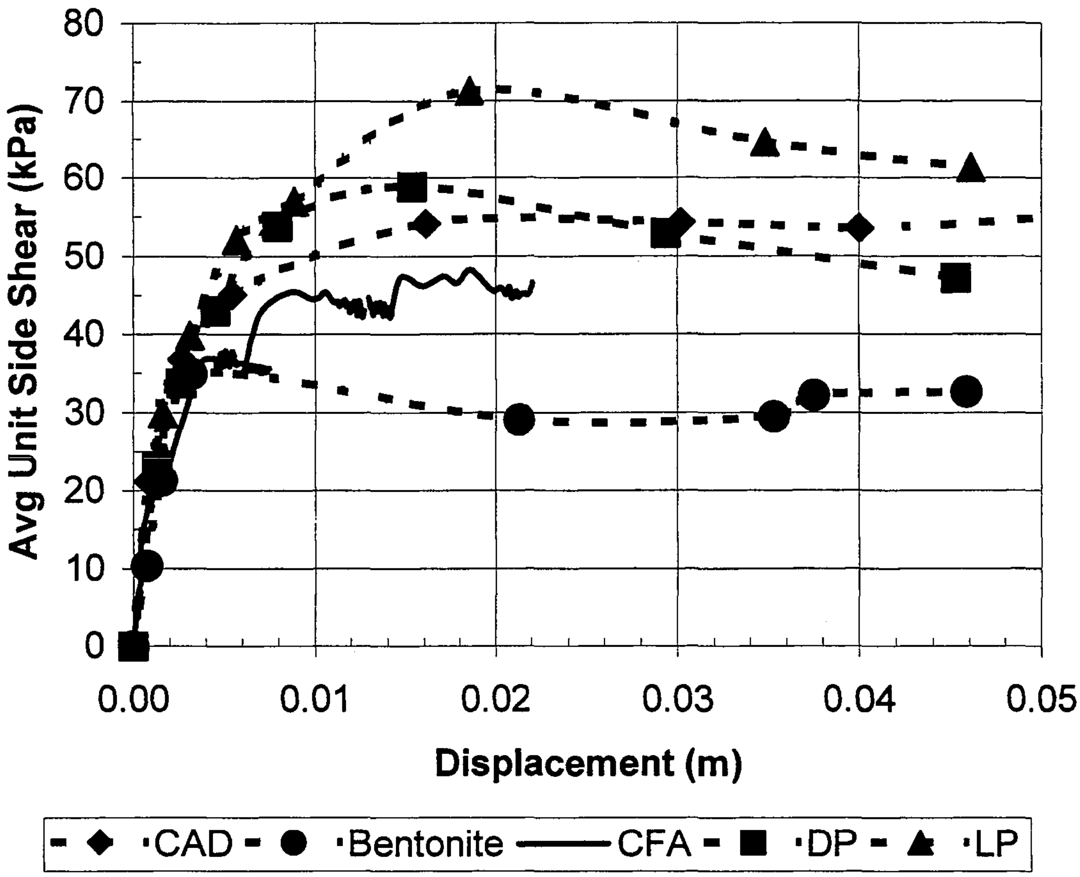

Figure 6 Load Transfer in Side Shear

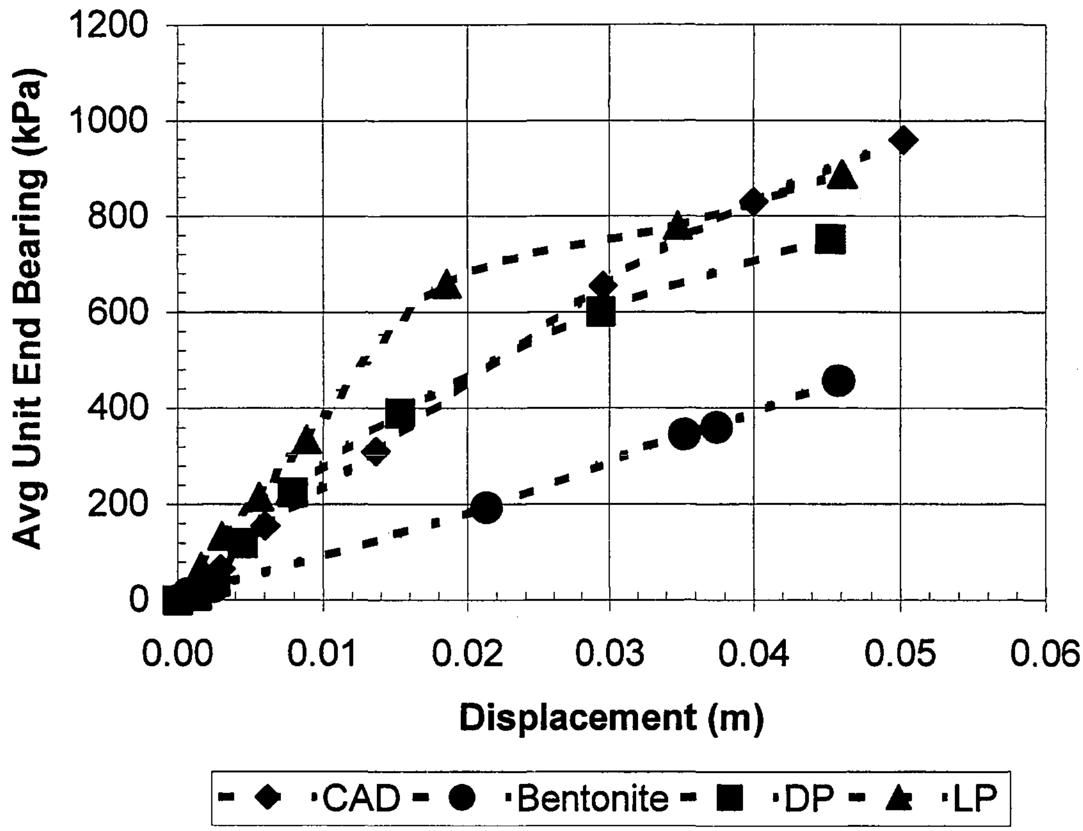

Figure 7 Load Transfer in End Bearing