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LONG TERM LOAD TRANSFER IN DRILLED SHAFTS

LE TRANSFERT DE CHARGE 'A LONG TERME DANS DES PIEUX FORE'S

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SYNOPSIS: Results are presented from five projects where instrumented drilled shafts were constructed. The structures vary in height from 4 to 66 stories. The shaft types include straight shafts from 1.6 to 3.3 meters in diameter and a 1.1 meter diameter shaft with 1.9 meter diameter bell. The shafts terminate in a variety of soil and rock formations. Electronic instruments placed at varying levels in each shaft at the time of construction have been recorded from 3 to 9 years following construction. Among the five cases, from 0 to as much as 36% of the load at the cutoff has reached the gages near the tip of the shafts, which represents 0% to 49% of the design end bearing stress. Other minor and unique trends in the data are discussed for each case.

INTRODUCTION

To assist with the evaluation of load transfer, one production shaft from each of five different projects was instrumented during construction. The instrumentation programs were either sponsored solely by the geotechnical engineer or jointly with the project owner. As such, the instrumentation installed was the minimum necessary to provide some load transfer information, rather than a more elaborate program which would be desired for a thorough research study.

In each case, the instruments used were Carlson Reinforced Concrete (RC) gages or miniature strain gages (A gages). The gages were installed in pairs for redundancy, near the top of the shaft (cutoff) and at varying depths along the shaft, most notably near the tip. The cables from the gages were protected during construction and routed to an accessible place, usually in the lowest basement level of the completed structure.

Method of Data Reduction

Using superposition, the axial load, P , at any depth is equal to the contribution from the steel and concrete, as indicated in equation 1.

$$P = \sigma_S A_S + \sigma_C A_C \quad (1)$$

where σ_S, σ_C = stresses in steel and concrete.
 A_S, A_C = cross sectional areas of steel and concrete.

Substituting Hooke's law for the stress σ , and assuming that the vertical strains in the steel and concrete are equal, the following equation for shaft stiffness results;

$$P/\epsilon = E_S A_S + E_C A_C \quad (2)$$

where E_S, E_C = Young's Moduli for steel and concrete,
 A_S, A_C are as defined previously.

By recording strains at various levels during construction, and multiplying by the respective stiffness computed in equation (2), the axial load at any depth and at any time is computed.

Limitations of Measurements

Strain gages embedded in drilled shafts necessarily respond to strain from all sources; namely from applied loads, temperature changes, shrinkage and creep. Equations (1) and (2) apply for axial loads and thus do not account for any strains due to bending. In the case of bending, placement of three gages at 120 degrees apart at any level will cancel the bending effects when the strain readings are averaged. Since two gages placed 180° apart were averaged on the projects reported here, bending influence has only been reduced rather than eliminated.

Creep is defined as strain under conditions of constant stress; thus, separation of the applied load component from the creep component during construction requires detailed knowledge of the load being transferred into the particular shaft at the time of gage reading. This was not possible for the case studies included here, although the data to be shown subsequently indicate small increases in strain after final fitting out of the structures which is roughly akin to a constant stress situation.

Creep is dependent on concrete age at the time of loading. In many cases, 1 to 2 years may pass between shaft construction and the buildup of appreciable load throughout the shaft length. Data by Russell and Corley (1977) indicate relatively small strains in laboratory creep tests of concrete aged 360 to 720 days before loading.

In all but one case, the shafts were constructed and several months passed during which time the basement was excavated and grade beams were constructed. During that time, the thermocouples in the gages registered a rapid rise in temperature of the concrete in the first week, followed by a gradual return to ambient ground

temperatures (13 to 16 degrees celsius) before the shafts were uncovered and the structure begun. For the data reduction, the initial strain reading was taken at that time and thus canceled the effect of temperature rise with hydration.

Strains due to shrinkage in the mass concrete were not accounted for in this study.

RESULTS

Soil profiles and load versus time curves are presented in the following section for each of the five projects. In the soil profiles, the instrumentation levels are shown with a cross (+) symbol. Elevations are referenced to USGS datum or local city datum according to the particular project.

Prudential Plaza, Chicago, Illinois

As shown in Figure 1, a 3.0 to 3.3 meter straight shaft was instrumented for strain measurements near the cutoff and at the 75% depth level in the rock socket. Since the gages are slender and approximately 0.8m long, the center of the gage cannot be placed at the very bottom of the shaft. The 2.2 meter long socket terminated in dolomitic limestone and was designed in end-bearing at 19.2 MPa.

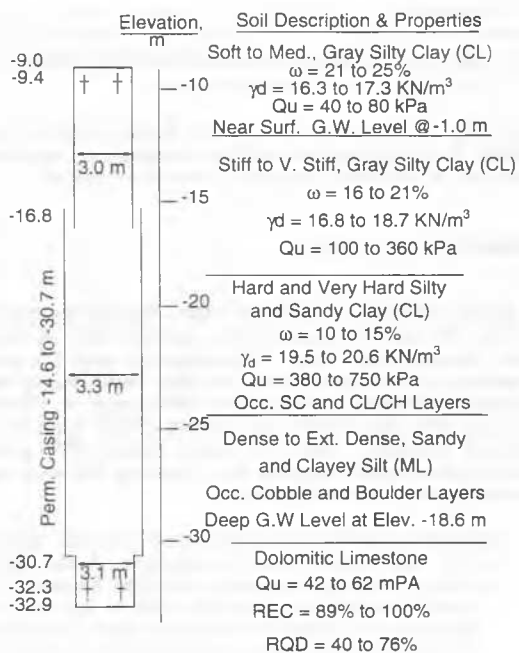


Figure 1. Soil Profile at Prudential Plaza.

The load versus time data in Figure 2 show the expected increase in load throughout the construction phase (building topout with perimeter cladding at approximately 30 months). Additional fitting out of the interior floors resulted in a modest load increase through 40 months, and thereafter the load has remained essentially constant at both the cutoff and near the bottom of the rock socket. This corresponds to an applied stress of 8.3 MPa at the lowest gage currently, or 43% of the design bearing pressure.

It is noted that the computed stress from the top gages is 23.5 MPa at present. This is 22% above the design bearing pressure.

Since the caisson is under the rigid core of the structure, it is possible that small variations in concrete modulus and rock seams from caisson to caisson could result in local hard spots that attract load. Evaluation of this possibility would have required instrumentation of several core caissons. It is possible also that part of the increase represents slight long term creep.

Assuming the total design load is carried in the rock socket, the extrapolated bearing pressure at the base of the socket would be only 4.0 MPa, or 21% of the design bearing pressure. This is not an overly conservative assumption in this case. Although significant hard clay and dense silt strata exist from 19 meters to the top of rock, the shaft was constructed with a permanent casing through these strata and thus less load would be shed in friction than in an uncased shaft. However, some load is undoubtedly resisted at the shaft/socket transition, where the casing is screwed into the rock surface.

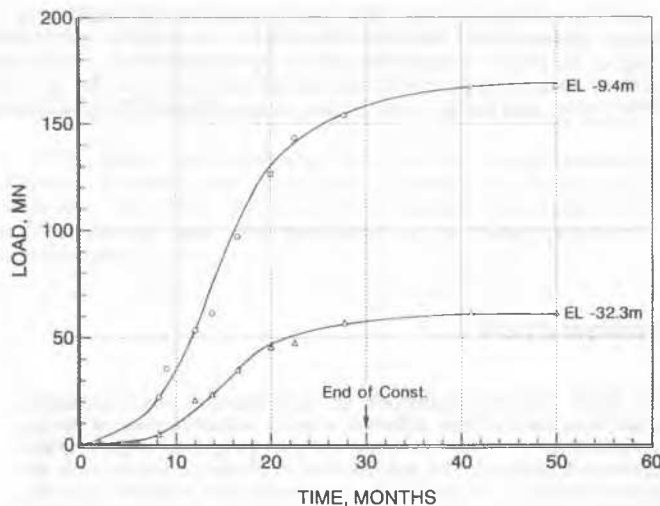


Figure 2. Load vs. Time at Prudential Plaza.

SOHIO Corporate Headquarters, Cleveland, Ohio

Figure 3 shows the caisson instrumentation and soil profile. A 2.1 to 2.4 meter diameter straight shaft with 2.0 meter diameter rock socket tip was instrumented. In the original design, the casing shown between elevation 138.4 and 131.1 meters was not required but was added during construction due to some deep, caving sands and gravels. Thus, two levels of the rock socket instrumentation were cast within a 2.1 meter diameter steel casing. The design bearing pressure was 2.4 MPa, and the design friction was 1.1 MPa in the competent shale.

The results in Figure 4 indicate approximately 44 and 39 MN, respectively near the top of the shaft (a cold joint at elevation 182.9 meters) and in the weathered shale at elevation 132.9 meters. An intermediate set of instrumentation near the top of the dense till at elevation 144.5 meters malfunctioned approximately one year after installation, as did a pair of gages at elevation 131.1 meters.

The remaining data indicates nearly all of the load reached the mid-depth of the weathered shale layer near the bottom of the permanently cased portion of the shaft (where the casing had been inserted in a 6 inch oversized hole drilled open or under slurry), and was shed rapidly thereafter in the remainder of the rock socket within the competent shale. At both 128.6 meters and 126.2 meters, the readings were negative during early loading. However, at elevation 128.6 meters, approximately zero load has

been measured from the end of construction to the present. At 126.2 meters, the readings remain negative.

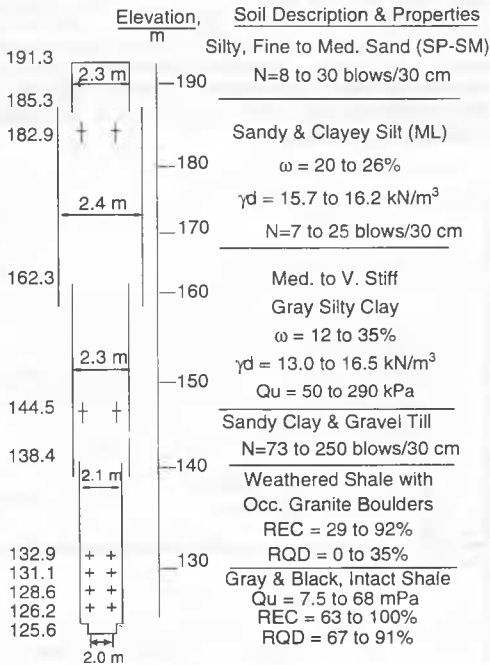


Figure 3. Soil Profile at SOHIO Corp. Headquarters.

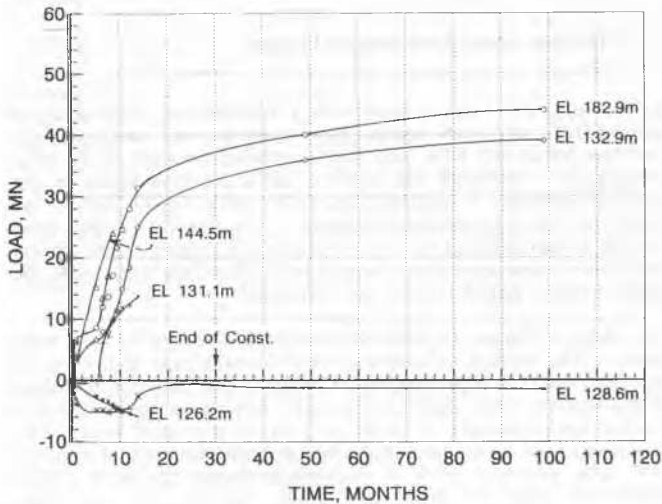


Figure 4. Load vs. Time at SOHIO Corp. Headquarters.

In early stages of loading before appreciable compressive load reaches the shaft, slightly negative readings are common and are thought to result from thermal expansion in the massive concrete during hydration. Typically even the deepest gages will eventually register positive readings during and following construction. In this case, the continued negative readings from the base of the rock socket may indicate that negligible load has reached the base to overcome those stresses generated in the socket concrete during hydration. For those gages still functioning, the long term data indicate slightly increasing loads, with little variation in load shedding. Apparently the load is carried at the socket lip and in

socket friction above the base. The slight increase in load from the 50 month reading to the 100 month reading may be a measure of small concrete creep rather than structural load increase.

Newport Office Tower, Jersey City, New Jersey

At this site, a straight shaft core caisson 2.0 to 2.3 meters in diameter was constructed under full bentonite slurry, and instrumented prior to concreting (Figure 5). The shaft was designed for end bearing in the siliceous shale at 3.8 MPa, and for friction in the soil and rock materials below the organic silt at -5 meters. Primary frictional load transfer is in the decomposed shale from approximately -17 to -24 meters (27.6 MPa design), and in the reddish-brown siliceous shale below -24 meters (110.3 MPa design). Instrumentation was installed near the shaft cutoff, near the top of the decomposed shale, and near the bottom of the rock socket.

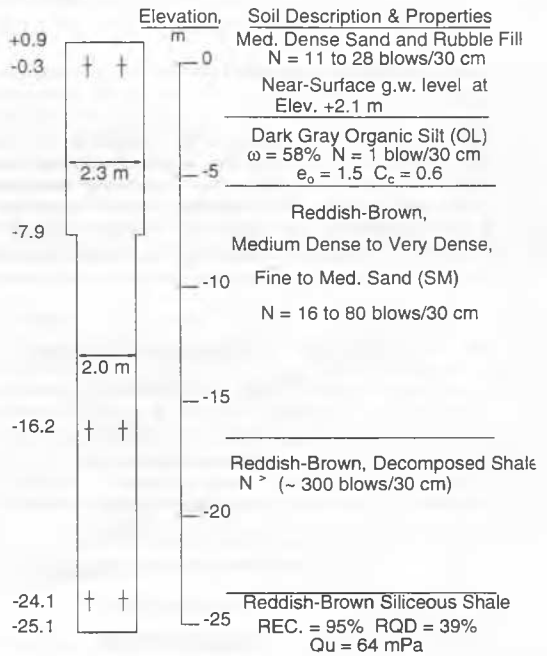


Figure 5. Soil Profile at Newport Tower.

The load versus time data in Figure 6 indicate approximately 23.7 MN has reached the top of the shaft, which corresponds very closely with the structural design (which assumed dead load equal to 70% of the total load, or approximately 23.4 MN). Near the base of the socket, approximately 1.9 MN has been recorded, or approximately 8% of the load entering the top of the shaft. The design was based on a tip capacity equal to 50% of the dead load, or 11.7 MN. This indicates approximately 92% of the measured load has been taken out in shaft friction, vs. 50% in the design. An applied pressure of 630 KPa is computed from the lowest gages, which is 16% of the design bearing pressure.

The strain data from the intermediate gages near the top of the decomposed shale layer have been erratic since the middle of the construction period, although the data up to 15 months indicate little shedding of load in the upper 16 meters of the soil profile.

Chicago Place, Chicago, Illinois

Chicago Place is a 9 story portion of an attached 55 story tower.

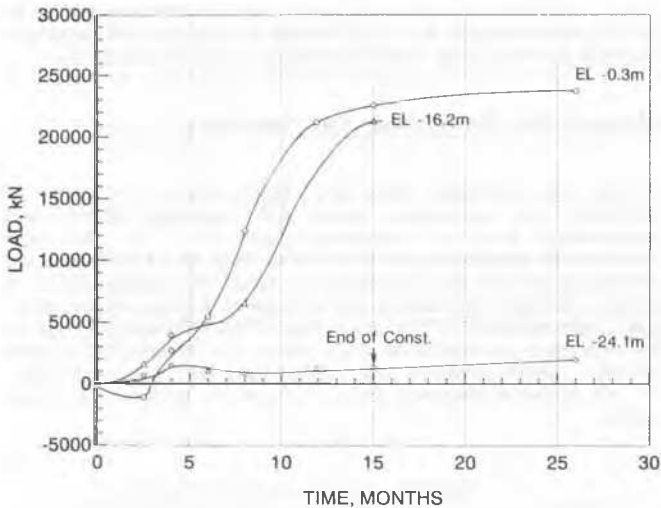


Figure 6. Load vs. Time at Newport Tower.

Straight shafts 1.6 meters in diameter were designed for friction (120 KPa) and end bearing (1,920 KPa) to avoid obstruction with belled caissons from a previous structure. The shafts terminate in very hard, silty and sandy clay hardpan which begins at an elevation of approximately -20 meters. Instrumentation was installed near the top of the shaft, and near the top and the bottom of the hardpan layer (Figure 7).

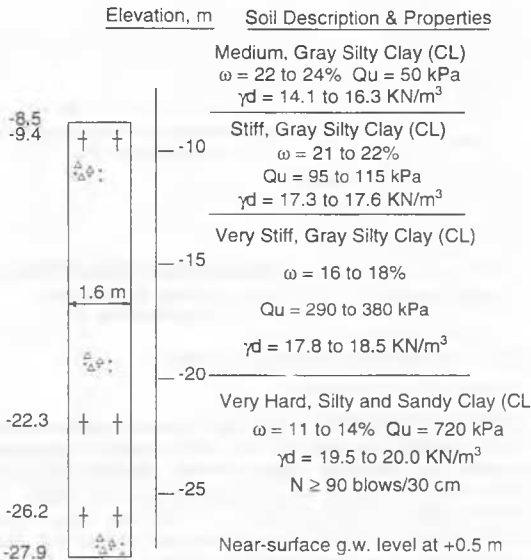


Figure 7. Soil Profile at Chicago Place.

The load versus time data in Figure 8 show consistency with respect to the proportion of load with depth, although the individual curves are somewhat variable. In particular, two sets of intermediate readings were somewhat lower than previous sets of readings at each level. It is not known whether this was an isolated problem related to the readout box or has some other explanation.

The most recent data indicate increasing load at the top of the shaft and relatively constant load near the top of the hardpan, with a significant decrease in load near the bottom of the shaft. A

characteristic trend in all three instrument pairs was the negative strain readings early in construction, before appreciable load was framed into the shaft. The maximum stress computed from the lowest gage data is 330 KPa, or 17% of the design bearing pressure. The load at the top of the shaft is 3,185 kN which is very close to the design dead load (3,170 kN). It is possible that the behavior of the gages is somehow influenced by the concurrent construction of the much more heavily loaded, adjacent 55 story tower. The resulting deep seated compression in the hardpan is in effect unloading the lower gages.

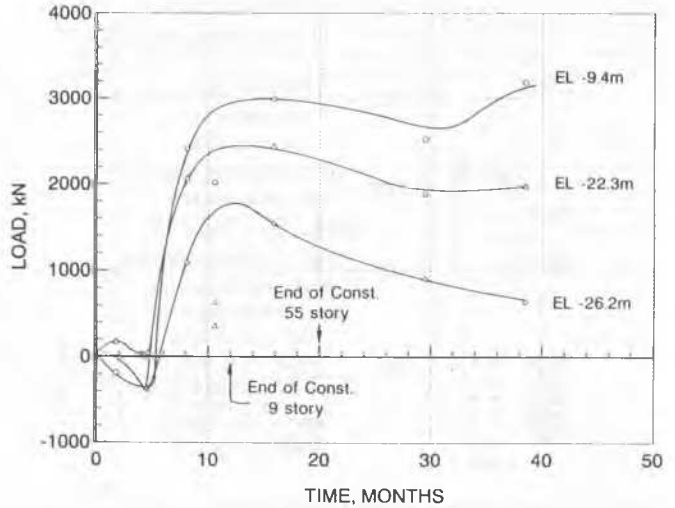


Figure 8. Load vs. Time at Chicago Place.

111 Pfingsten Road, Northbrook, Illinois

At this site, a 1.1 meter shaft with 1.9 meter bell terminating in very hard, silty and sandy clay hardpan was designed for end-bearing at 960 KPa, and friction (above the bell) at 24 KPa. (Figure 9). Although this shaft is an active foundation in the existing building, it was installed as the only drilled shaft in an otherwise footing-supported structure. The general bearing level for the spread footings was near elevation 197 meters. The drilled shaft was instrumented at several levels, extending from near the shaft cutoff to just above the base of the bell.

The data in Figure 10 indicate consistent shedding of load with depth. The average of a stress cell and strain gage at the top of the shaft indicate approximately 6.1 MN, while stress cells placed at the bottom of the shaft have recorded strains corresponding to a load of approximately 1.3 MN, or 21% of the cutoff load. The computed load at the base of the bell indicates an applied stress of 470 KPa, which is 49% of the design bearing pressure. (The reinforced concrete gages just above the base of the shaft malfunctioned after approximately one year. Therefore, the data for the base of the shaft were taken from two flat-plate stress cells also manufactured by Carlson Instruments).

One interesting observation from this data is that the design load for this shaft was approximately 4.5 MN. The top-most set of gages indicates a load of 6.1 MN, approximately 60% greater than the design load. Mechanically, this may be caused by an unknown shift in the gages at this level, although both gages are registering higher than expected strain readings. A physical explanation is that the drilled shaft is attracting more than its share of load. The bearing strata for the drilled shaft is harder and has a higher modulus (indicating lesser potential settlement) than the hard, near-surface clay in which the footings are supported.

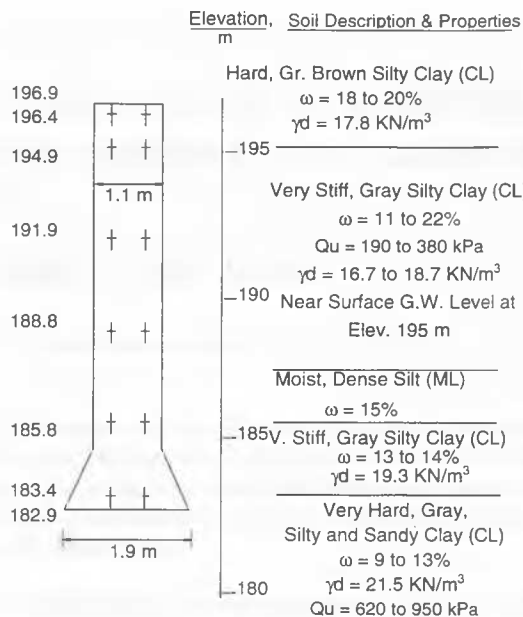


Figure 9. Soil Profile at 111 Pfungsten Road.

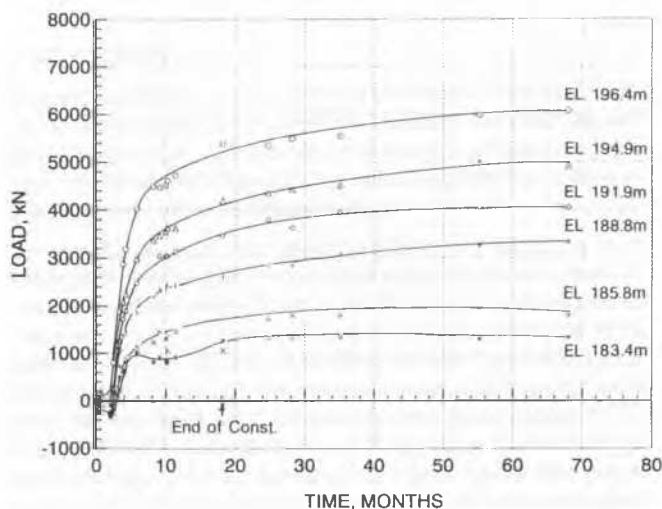


Figure 10. Load vs. Time at 111 Pfungsten Road.

SUGGESTIONS FOR FUTURE CAISSON INSTRUMENTATION PROJECTS

Based on the results of these studies and others with which the authors have experience, the following suggestions are offered for future instrumentation related to drilled shafts:

1. Predicting the actual shaft friction continues to be a significant challenge in drilled shaft design. On critical projects where the design involves major load carried in shaft friction, and particularly where construction is under slurry, the authors recommend instrumentation of a production shaft, as well as completion of an instrumented load test at the outset of construction. In the latter case, the results will allow the shaft design to be economized before construction proceeds and thus will provide the most cost-effective foundation for the particular site.

2. Some advantage in strain resolution can be realized by using vibrating wire, sister bar strain gages, instead of the Carlson resistance ratio type sister bar gages. This can be particularly useful in the lower load range of a drilled shaft load test or for smaller capacity shafts where the stress levels are low. However, where production drilled shafts are completed by drop-chuting concrete, some of this concrete will splash and impact the gages attached to the rebar cage. The Carlson gage has an advantage in this case since it is very robust. The authors have used both types of gages in these conditions with comparable success.
3. Where possible, and particularly if there are limited funds for instrumentation redundancy, avoid placing strain gages too close to abrupt changes in shaft geometry since the strain data there can be difficult to interpret. This may be in conflict with the most desirable location for load transfer information. For example, a transition between soil overburden and the top of the rock is a difficult instrumentation location if there is a reduced diameter at the rock socket. If this information is primary, then make certain there are more instrumentation levels immediately above and below the transition, as well as across the transition area.
4. On projects where instrumented load tests are performed, important data on the in-situ concrete modulus (Young's Modulus in Equation 2) can be obtained by placing strain gages near the top of the shaft. By measuring accurately the shaft dimensions and gage locations, the upper section of the shaft acts as a large test cylinder and allows calculation of the elastic modulus. In conjunction with this measurement, it is important to avoid any overbreak in the upper shaft concrete (through the use of surface casing or form-tube) since appreciable load can be shed there.
5. Reliable long term settlement measurements throughout construction would greatly enhance the value and facilitate interpretation of long term strain gage readings. Long term settlement measurements were recommended on each of the projects reported here, although no funding for such measurements was ever obtained. Limited elevations taken by the contractor for his purposes were not sufficiently accurate or precise to be of value. Accessible settlement points and repeatable elevation accuracy to at least 0.5 mm is essential.

CONCLUSIONS

Based on the results presented, the following conclusions and observations are made:

1. In all cases reported, the load reaching instruments near the base of the shafts indicates an applied pressure significantly less than the design bearing pressure, particularly for those shafts socketed in rock. Even those shafts designed in end-bearing only are realizing significant load transfer in friction.
2. In all cases, the load carried in shaft side resistance eventually reached a stable condition with no significant reduction over time.
3. The slight long term measured load increases with time after project completion indicate creep effects are small in concrete that is more than 3 years old.
4. The results show that load transfer in drilled shaft foundations can be monitored successfully for many years. The fact that certain gages stopped functioning in each of the shafts points to the need for redundancy to ensure sufficient working instrumentation for meaningful results. The authors recommend placing gages in pairs at each depth for redundancy, in the event one gage is damaged during concreting or becomes inoperative with time. At the top of

the shaft, three gages at 120 degrees apart or 4 gages at 90 degrees apart are recommended to eliminate bending effects near the top of the shaft.

REFERENCES

Russell, H.G. and Corley, W.G. (1977). Time-Dependent Behavior of Columns in Water Tower Place. Research and Development Bulletin RD 052.01B, Portland Cement Association.

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