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Stephen A. Bucher
STS Consultants, Ltd., Northbrook, Illinois

Raymond J. Krizek
Northwestern University, Evanston, Illinois

Dimitrios K. Atmatzidis
University of Patras, Patras, Greece

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Stephen A. Bucher
Project Engineer, STS Consultants, Ltd., Northbrook, Illinois

Raymond J. Krizek
Professor and Chairman, Department of Civil Engineering, Northwestern University, Evanston, Illinois

Dimitrios K. Atmatzidis
Professor, Department of Civil Engineering, University of Patras, Patras, Greece

SYNOPSIS: The results of a full-scale load test on a belled caisson bearing on hardpan in the downtown Chicago area are presented herein and are discussed in terms of current design practice and the results of other pertinent full-scale tests and a small-scale model test. Current specifications for allowable bearing pressures are shown to be conservative, and previously established settlement limits required to mobilize side resistance are reconfirmed. The settlement measured during the test is in good agreement with that predicted by use of pressuremeter test data. The confinement of the bell in a hard clay layer appears to be beneficial in that it serves to limit the development of major cracking at the base.

INTRODUCTION

Drilled piers or caissons bearing on very dense glacial till (hardpan) are a common type of foundation for high rise structures in the Chicago area. The local building code specifies a maximum bearing pressure of 12 ksf, but higher pressures are allowed if adequate testing and supporting data are provided. As a result of accumulated experience and increased confidence in the use of in-situ testing, such as the pressuremeter test, design bearing pressures of 20 ksf to 25 ksf are now used commonly. Although full-scale caisson load tests can provide valuable information to validate or improve the bearing capacity and settlement theories used in design, actual load tests are rarely conducted because of the required high reaction loads and the associated expense and inconvenience. Results from two full-scale load tests on caissons in the Chicago area have been reported; one series of tests was conducted during the construction of the Chicago Union Station (D'Esposito, 1924), while the other was completed during foundation construction for the Cummings Biological Research Center at the University of Chicago (Holtz and Baker, 1972).

Presented and discussed herein is information obtained from a full-scale load test on a caisson bearing on hardpan in the downtown Chicago area. The caisson had a shaft diameter of 2.5 feet, a bell diameter of 6.33 feet, and a total length of 60.65 feet; it was instrumented with load cells and strain gauges and tested to a maximum load of 1100 tons, which approached the estimated ultimate bearing capacity. The anticipated performance of this caisson during the load test was determined on the basis of current design practice in terms of settlement, bearing capacity, and side resistance or skin friction. The observed performance of the caisson is presented and discussed both in terms of current design practice and in comparison with available results from full-scale tests in similar soils.

SOIL CONDITIONS

The subsurface conditions at the project site are representative of the typical downtown Chicago soil profile, which has been presented in detail by Bretz (1939) and Peck and Reed (1954). Surficial deposits of fill materials are typically encountered over layers of beach sands. Underlying these soils are glacial deposits (consisting of lacustrine clay and stratified clayey till sheets of varying strengths and water contents) of the Wisconsin Glacial era; these deposits vary with depth from soft to hard silty clays and extremely dense silt, sandy silt, or gravel zones overlying the bedrock.

At the site of the full-scale load test, soft to medium silty clays were encountered to an elevation of approximately -48 feet CCD (Chicago City Datum). Below these soils, very stiff to hard silty clays were encountered to an elevation of -63 CCD. Then, alternate layers of hard sandy and silty clay (hardpan) and extremely dense silt or sandy silt were encountered to the top of bedrock at elevation -105 CCD. Information about the typical subsurface profile and soil properties at the test site is presented in Figure 1. Pressuremeter tests were conducted between elevations -66 CCD and -81 CCD, and the values of the parameters obtained are summarized in Table 1. The soils tested exhibited pressuremeter parameter values that are comparable to a large number of available values for soils in the same general area of Chicago (Lucas and deBussy, 1976).

Table 1. Pressuremeter Parameters

Elevation(CCD) (feet)	Pressure (tsf)			Modulus
	Horizontal At Rest P_0	Creep P_f	Limit P_p	E_d (tsf)
-66.0 to -68.5	4.0	15	29	156
-68.5 to -71.0	4.0	10	20	111
-71.0 to -73.5	5.0	16	32	198
-73.5 to -76.0	4.5	20	40	297
-76.0 to -78.5	4.5	15	29	267
-78.5 to -81.0	8.0	-	-	1043

TEST CAISSON

The test caisson had a shaft diameter of 2.50 feet and a bell diameter of 6.33 feet; it was constructed by using the typical procedures employed for production caissons, although it was not part of the load-carrying grid of caissons for the new structure. The shaft was auger-drilled at a diameter slightly larger than designed to a

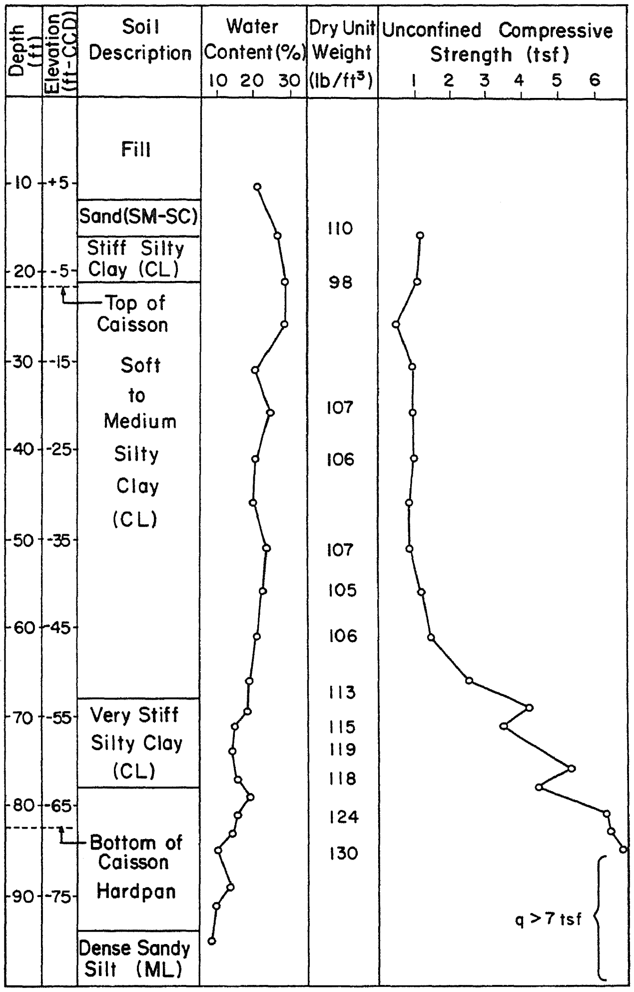

Figure 1. Soil Profile

depth of about 15 feet below existing grade, and a temporary steel casing was inserted through the fill and sand into the underlying silty clay. The shaft was then advanced by augering at the design diameter to a depth of 73 feet (-67.4 CCD), where a suitable hardpan layer was encountered.

The base of the shaft was enlarged at this level by means of a 60^{0} belling bucket. The bell angle was then reduced to about 50^{0} by hand excavation to obtain a geometry similar to that used by Reese and Farr (1980) so that results could be compared in terms of the development of cracks at the base of the bell. The thickness at the perimeter and at the center of the base pad of the bell was 1 foot and 2 feet, respectively. The dimensions of the test caisson, as measured in the field, are shown in Figure 2.

ANTICIPATED PERFORMANCE OF TEST CAISSON

The test caisson was designed to transfer load to the foundation soils primarily through end bearing, but some amount of side resistance or skin friction was also expected to develop. During testing, the top of the caisson was expected to settle by an amount equal to the sum of the elastic compression of the shaft and the settlement at the base. Accordingly, estimates of bearing capacity, skin friction, base settlement, and elastic compression were made on the basis of available data. The ultimate end bearing capacity, Q_b, of deep circular foundations in cohesive soils can be computed according to Skempton (1959) as

$$Q_b + W = A_b(Nc_b + \gamma H) \qquad (1)$$

where W is the weight of the caisson, A_b is the cross-sectional area of the base, N is a bearing capacity factor, c_b is the average shear strength of the soils within a depth of two-thirds the base diameter from the base, and γ is the average total unit weight of the soil for the total length, H, of the caisson. If it is assumed that $W = A_b \gamma H$ and N is set equal to 9 for saturated cohesive soils (Skempton, 1951), Equation (1) can be reduced to

$$Q_b = 9c_b A_b \qquad (2)$$

The undrained shear strength of cohesive soils can be calculated from pressuremeter data according to the

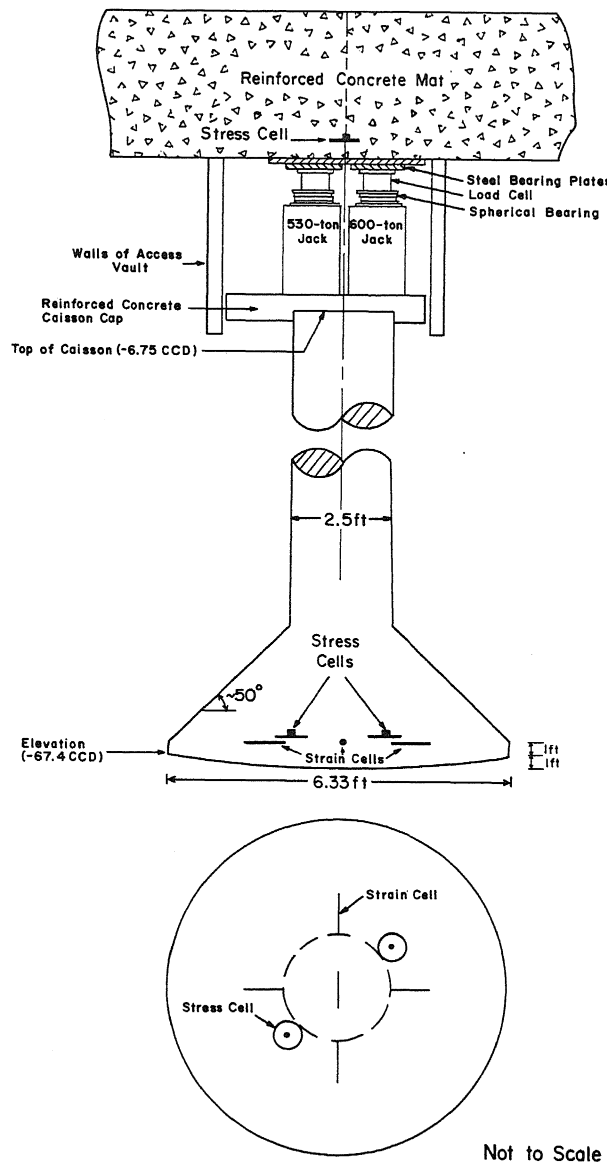

Figure 2. Details of Instrumented Caisson

following relationship advanced by Menard (1965, 1975):

$$c = \frac{p_\ell - p_o}{2K_b} \qquad (3)$$

where c is the cohesion, p_ℓ and p_o are the limit pressure and horizontal earth pressure at rest, respectively, at the pressuremeter test level, and K_b is a coefficient which, for typical Chicago area soils, has a value of about 2.7 (Lucas and deBussy, 1976). Using average values (obtained from Table 1) of 27 tsf and 4.3 tsf, for p_ℓ and p_o, respectively, Equation (3) yields a cohesion of about 4.2 tsf. The available unconfined compression data shown in Figure 1 indicate a minimum cohesion value for the soils under the base of the caisson of about 3.5 tsf. Accordingly, the net end bearing capacity of the caisson was computed to be about 1100 tons. The appropriateness of using Equation (1) to compute the end bearing capacity of caissons on Chicago hardpan has been confirmed by Holtz and Baker (1972).

The side resistance or "skin friction" of the test caisson was estimated on the basis of the available undrained shear strength values for the various soil layers. To facilitate the computations, the bell was neglected and the shaft of the caisson was separated into two parts with lengths L_1 = 41.25 feet (-6.75 CCD to -48 CCD) and L_2 = 17 feet (-48 CCD to -65 CCD). The average cohesion, c_1 and c_2, for each section was estimated to be 0.48 tsf and 2.05 tsf, respectively. To obtain estimates of the adhesion between the soil and the caisson, these cohesion values were multiplied by 0.8 and 0.4, respectively (Department of the Navy, 1982), to give:

$$Q_S = \pi D(0.8\, L_1\, c_1 + 0.4\, L_2\, c_2) \qquad (4)$$

where D is the diameter of the caisson shaft. Accordingly, the total skin friction was found to be about 230 tons.

A method for using pressuremeter data to estimate the settlement, w, at the base of foundations has been presented by Menard (1965, 1975); a general form of the resulting equation (Lucas and deBussy, 1976) is

$$w = \frac{1+\nu}{3E_d} PR_o(\lambda_2 \frac{r}{R_o})^\alpha + \frac{\alpha\lambda_3}{4.5\, E_d}\, pr \qquad (5)$$

where E_d is the pressuremeter or deviatoric modulus, ν is Poisson's ratio and is set equal to 0.33 because the value of E_d is computed from pressuremeter data on the hypothesis that ν = 0.33, R_o is an empirical coefficient equal to 30 cm, r is the radius (expressed in cm) at the base of the foundation, p is the uniform pressure on the foundation, λ_2 and λ_3 are empirical coefficients that are functions of the shape of the foundation, and α is an empirical coefficient depending on the type and structure of the soil. By setting $\lambda_2 = \lambda_3 = 1$, $\alpha = 2/3$, p = 35 tsf, and E_d equal to 200 tsf for a depth equal to the foundation radius below the base of the foundation and 270 tsf for larger depths, the anticipated settlement was computed according to Equation (5) to be about 2.4 inches for the maximum applied load of 35 tsf.

The theoretical elastic compression, ΔL, of the test caisson was computed according to the relationship

$$\Delta L = \frac{PL}{AE} \qquad (6)$$

where P is the axial load, L is the length of the caisson, A is the cross-sectional area of the caisson, and E is the elastic modulus of the concrete used to construct the caisson. Samples of the test caisson concrete were obtained by coring the top of the caisson to a depth of approximately 5 feet two weeks after the load test was completed. This limited portion of the shaft was assumed to be representative of the entire caisson and yielded values of 3.4 x 10^6 psi for the elastic modulus and 0.28 for Poisson's ratio. Because typical values of Poisson's ratio for normal strength concrete are on the order of 0.15 to 0.20, a value of 0.25 was selected for further computations.

INSTRUMENTATION AND TESTING

The instrumentation for the test caisson consisted of five strain cells and two stress cells installed at the base and a single stress cell mounted directly over the caisson within the reinforced concrete mat. All cells were stock items manufactured by the Carlson Instrument Company in Campbell, California. The locations of the instruments within the caisson are indicated in Figure 2. The strain cells were mounted horizontally at the base of the caisson and were intended to monitor lateral strains at the base of the bell. All instruments at the base of the bell were carefully embedded in fresh concrete and their positions were fixed by allowing the concrete to harden overnight. Placement of the remaining caisson concrete was completed the following day.

In conjunction with the construction of the heavily reinforced concrete foundation mat, a concrete vault was built below the mat to provide access to the top of the test caisson once construction of the high rise structure had proceeded above the foundation level. Foil strain gauges were bonded to the upper and lower surfaces of the mat to monitor strains during the load test. Gauges on the lower mat surface were damaged during the set-up of the jacking system and water seepage later rendered them useless. However, the top gauges functioned throughout the test.

The theoretical failure load of the test caisson was estimated to be 1100 tons, but a single hydraulic jack with that capacity was not available. Therefore, a special reinforced concrete caisson cap was constructed to accomodate two smaller jacks which would provide this capacity. The concrete mat, together with about 10 stories of the newly constructed concrete structure, provided a sufficiently large reaction to perform the load test, which was conducted according to the Standard Method of Load Testing for Piles under Axial Compressive Load as desribed in ASTM Specification D-1143-74. Two dial gauges were mounted at diametrically opposite locations over the top of the caisson to measure the settlement during loading. However, the length of time that each load was maintained on the caisson was different from that specified by ASTM so that the test could be completed within a reasonable time. Loading the caisson incrementally to the maximum capacity of the jacks was the primary criterion, since loading to failure would not likely be possible with the equipment available. The actual loading sequence used during the test is shown in Figure 3.

Although no cycling of the load test was originally planned, a temporary malfunction in one of the jacks necessitated an unloading to just below 500 tons. The caisson was then reloaded from this point to the maximum load of 1100 tons. Each load increment was maintained for at least one hour. A maximum load of 1060 tons was applied for a period of 6 hours. This load was increased to 1100 tons by taking both jacks to their full capacity. Due to difficulties in controlling the release of the hydraulic pressure, the unloading proceeded directly to zero load without intermediate steps.

Prior to unloading the caisson, specially fabricated steel shims were placed between the caisson cap and the concrete mat to transfer as much of the jack load as possible to the caisson. Since the load cell readings indicated that very little load had been transferred, the test caisson was reloaded several days later to a maximum of 1040 tons and additional shims were placed. Each of the instruments was monitored during these procedures, as well as over the following 15 month period.

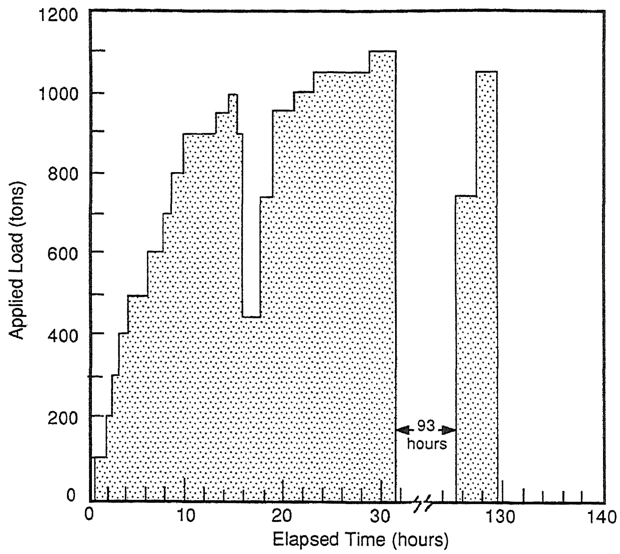

Figure 3. Summary of Test Load Sequence

RESULTS AND OBSERVATIONS

The measured settlement at the top of the caisson as a function of the applied load is shown in Figure 4, together with two lines indicating the computed elastic compression of the test caisson. The elastic compression represented by the lower line was computed by assuming that the entire difference between the applied load at the top of the caisson and the load reaching the bottom is carried in the concrete shaft with no load dissipation along the shaft and no settlement of the base of the caisson. The upper elastic line is obtained by assuming a linear dissipation of the actual load in the shaft (that is, the difference between the applied load and the load reaching the bottom) beginning at the top of the shaft and continuing to the base. The latter line seems more realistic, since it is apparent that some load is carried in side friction and considerable settlement of the base has occurred.

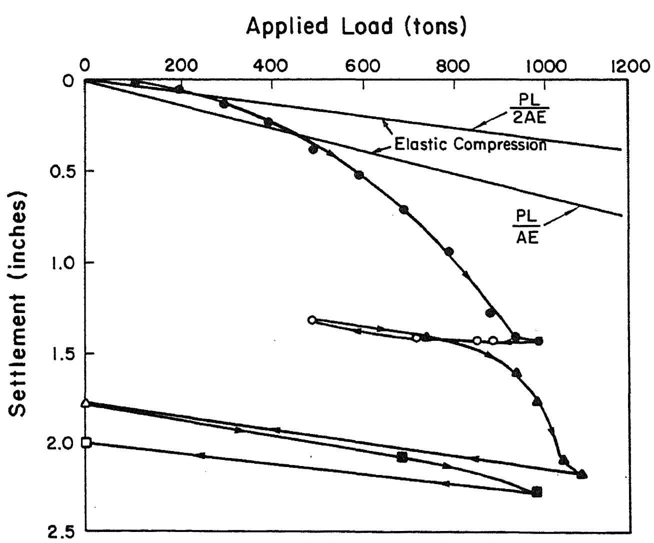

Figure 4. Measured Settlement of Top of Caisson as a Function of Applied Load

According to previous investigators (Burland et al, 1966; Whitaker and Cooke, 1966; Holtz and Baker, 1972; and Reese et al, 1976), a movement of up to 0.5% to 1.0% of the shaft diameter, or a maximum of 0.25 inch, is required to mobilize the full side resistance. The upper elastic compression line and the load test curve intersect at a settlement of about 0.1 inch or 0.33% of the shaft diameter and indicate a maximum side resistance of about 245 tons, which is in very good agreement with the value of 230 tons computed by using conventional procedures.

The average adhesion factor calculated according to the indicated side resistance of the test caisson is on the order of 0.65 to 0.70 and corresponds well with the range of 0.5 to 0.8 reported by Holtz and Baker (1972) for friction caissons in typical Chicago clayey soils. However, when compared with adhesion factors in the range of 0.4 to 0.7 for overconsolidated Texas plastic clays (Reese and O'Neill, 1969) and London clay (Skempton, 1959), the factors obtained are somewhat high. This could be attributed to the lower plasticity and lower sensitivity of the Chicago silty clays.

If the elastic compression of the shaft is subtracted from the settlement measured at the top of the caisson, the settlement of the bottom can be obtained. Accordingly, it can be observed that the bottom of the caisson settled by about 1.85 inches under the maximum applied load of about 35 tsf. According to Equation (5) and for a load at the base equal to about 26 tsf (maximum applied load adjusted for load supported by side resistance), the anticipated settlement at the base, computed according to pressuremeter data, is about 1.90 inches, which is in very good agreement with the measured settlement. Caissons bearing on hardpan were loaded during the Chicago Union Station tests (D'Esposito, 1924) and settled by 0.9 inches and 2.0 inches under maximum applied loads of 18.4 tsf and 87.5 tsf, respectively. The hardpan bearing caisson loaded during the University of Chicago tests (Holtz and Baker, 1972) settled about 2.5 inches under a maximum load of 53 tsf. Accordingly, the settlement at the bottom, as a percent of the diameter of the loaded area, was about 1% to 4% for the Union Station caissons, about 1% for the University of Chicago test caisson, and about 2.6% for the test reported herein. Whitaker and Cooke (1966) found that full mobilization of the base resistance in London clay did not occur until settlments were between 10% and 20% of the base diameter. The load-settlement curve shown in Figure 4, as well as those reported by Holtz and Baker (1972), do not show a sharp break, and it is therefore not clear if these caissons were actually loaded to their maximum capacity. This observation is further reinforced for the case reported herein if it is considered that the maximum applied load was about equal to the computed ultimate bearing capacity, but about 20% of that load was supported by side resistance.

Finally, the unloading curves shown in Figure 4 indicate that most of the measured settlement is nonrecoverable. Tests in London clay (Whitaker and Cooke, 1966; Ellison et al, 1971) have also indicated that most of the vertical movement, which occurs after the ultimate adhesion between the shaft and the surrounding soil is reached, is nonrecoverable and that significant rebound should not be anticipated. Although the conditions of the load test reported herein are different from those of the test conducted in the London clay (primarily an end bearing caisson on hardpan compared to primarily a friction pier in stiff fissured clay), slippage along the shaft is still nonrecoverable and any elastic rebound from the base of the caisson greater than about 0.1 inch would be resisted by negative friction along the shaft of the caisson. This observation is in good agreement with information reported by D'Esposito (1924) for the Union

Station tests and Holtz and Baker (1972) for the University of Chicago tests where, upon unloading, the rebound of caissons bearing on Chicago hardpan was not more than 0.1 inch after accounting for the elastic rebound of the concrete shaft.

Based on the load test curve shown in Figure 4 and the foregoing observations, it can safely be concluded that a large portion of the final applied load (about 820 tons) would reach the bottom of the caisson and would be transferred to the soil through the base of the bell. Unfortunately, computations of the load at the base of the bell, made on the basis of the stresses measured by the two stress cells installed at the base of the bell, yielded only a small fraction of this anticipated load. The average computed load, based on stress measurements, was only about 200 tons for an applied load of about 1060 tons and should not be considered indicative of the actual load transferred to the base of the caisson. It is likely that some "honeycombing" of the concrete below the cells may have been caused during installation and, consequently, relatively softer zones of concrete may have existed below the cells. This would have the effect of significantly reducing the modulus of elasticity of the concrete below the cells and could account for the low measured stresses. Furthermore, arching in the concrete above the cells may have occurred upon initial deflection of the cell face and additional stresses may not have been directly transferred to the cell. Finally, stress cell calibration may change due to a number of reasons, which include the development of stress concentrations and/or relief zones during installation procedures.

Alternatively, stresses and loads at the base of the caisson were computed on the basis of strains measured by the five horizontally oriented strain gauges, together with the assumption that, in the absence of applied horizontal stresses, the measured horizontal strains are attributable primarily to the Poisson effect. By assuming elastic behavior for the concrete and using laboratory test values of 3.4×10^6 psi and 0.25 for Young's modulus, E, and Poisson's ratio, ν, respectively, the vertical stress, σ_v, was computed in terms of the measured lateral (horizontal) strain, ε_h, as

$$\sigma_v = E \frac{\varepsilon_h}{\nu} \qquad (7)$$

Using an average value for the computed vertical stress, the load transferred to the base was computed and the results are shown in Figure 5. Also shown in Figure 5 is a line indicating the anticipated relationship between the applied load and the load transferred to the base. This line was obtained by considering that about 230 tons of the applied load are supported by side resistance. It can be observed that the results of the strain cell measurements are in very good agreement with predictions based on conventional procedures, as well as with the actual load test curve presented in Figure 4 for an applied load of up to about 900 tons. For higher loads, the measured strains increased disproportionately with the increases in load.

It can also be observed that, upon unloading, a large percentage of the stress or load remained "locked in", as indicated in Figure 5 by the significant shift of the data above the theoretical line. This apparent "locked in" stress may very well indicate the development of minor cracks at the base of the bell. Furthermore, the computed stress is observed to increase with time under the maximum load of 1060 tons, which was held constant for six hours, and this may be due to the propagation of microcracks or creep under constant load. Lateral strains in excess of 30 x 10^{-6} inches per inch were measured by four of the five strain cells, while strains

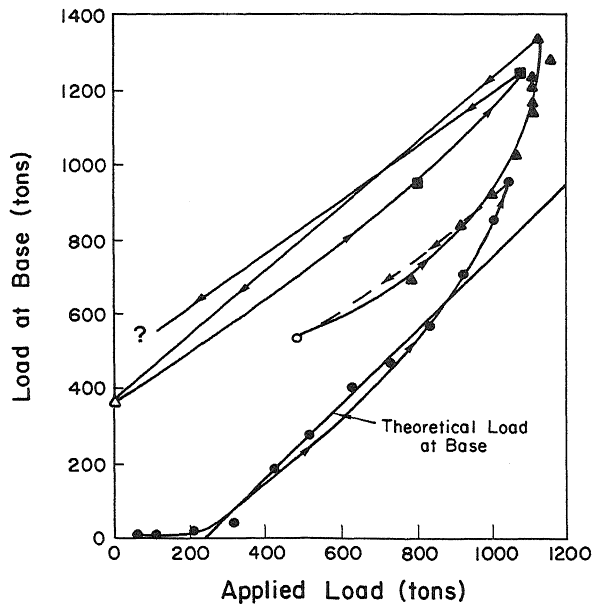

Figure 5. Load Transferred to Base of Caisson as a Function of Applied Load

of up to 50 x 10^{-6} inches per inch were recorded by two strain cells. These levels of lateral strain are significant enough to suspect the development of microcracks at the base of the bell. Reese and Farr (1980) performed unconfined compression tests on small-scale model caissons which were constructed with a variety of bell angles. It was observed that caissons with bell angles of less than 60^o to the horizontal would fail by the formation of a tension crack in the bell and that a 45^o bell would fail at significantly smaller loads than a 60^o bell with the same base area. It appears that the 50^o bell of the test caisson described herein performed better than would have been anticipated on the basis of the conclusions reached by Reese and Farr (1980). Although small lateral strains developed at the base of the bell, it can be stated that the confinement of the bell within the hardpan layer provided an additional factor of safety against failure by major cracking at the base of the caisson.

Monitoring of the instrumentation was continued for a period of 15 months after the end of the test and the final shimming. At the end of this period survey measurements indicated that the finished structure had settled approximately 0.62 inches. Since the settlement which resulted during the load test was on the order of 2 inches, the building settlement may have not been enough to result in complete load transfer through the shims to the test caisson. The average increase in strain cell readings was on the order of 20 x 10^{-6} inches per inch, indicating that about 850 tons of structural load was being transferred to the caisson through the shims and the natural building settlement. However, it could also indicate that there had been an equivalent amount of creep under a much smaller load because of the suspected past microcracking. The cumulative lateral strain measured at the base of the caisson from the start of the load test through the last readings was less than 70 x 10^{-6} inches per inch, and this could indicate that the caisson bell remained essentially intact, aided perhaps by confinement in the very stiff clay and hardpan soils which surrounded it.

CONCLUSIONS

Based on the results of the full-scale load test and the observations and discussion presented herein, the following conclusions, which are primarily applicable to the soil profile encountered in the Chicago area, can be advanced.

1. The high bearing capacity of Chicago hardpan and the accuracy of settlement predictions based on pressuremeter data have been reinforced. The base of the caisson was loaded to about 26 tsf without approaching the bearing capacity of the hardpan and the resulting settlement was about 1.85 inches. Accordingly, increased allowable bearing pressures can be established for caissons on Chicago hardpan, when the anticipated settlements are tolerable.

2. Previously established limits of movement for the mobilization of side resistance are confirmed; movements on the order of 0.5% to 1% of the shaft diameter or up to 0.25 inches are more than adequate to mobilize the full side resistance of caissons in Chicago silty clays.

3. The generated side resistance is found to be in close agreement (about 5% difference) with that computed according to conventional methods. The corresponding average adhesion coefficient of about 0.65 to 0.70 is within the limits established for similar soil profiles.

4. Very small, but nonrecoverable, strains were measured at the base of the caisson bell, indicating probable microcrack development during loading. The confinement of the bell in a hard soil layer is considered beneficial and, although not considered in current practice, it provides a measure of additional safety to current design procedures. The current requirement for 60^{o} bells should be maintained for high bearing pressure caissons.

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