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Pressuremeter Testing in Stiff to Hard Cohesive Soils

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ABSTRACT

The pressuremeter which was introduced into the U.S. in about 1970 provides the geotechnical engineer with a new tool for measuring soil properties. It is conducted in-situ thereby minimizing sample disturbance. It also tests the soil under the prevailing stress conditions in the soil mass. Measurements of the failure pressure and modulus of the soil are used to predict bearing capacity and settlement. This paper discusses a few commercial projects plus sites where load tests were performed and settlement measurements were obtained for comparison to predictions based upon pressuremeter parameters.

HISTORICAL BACKGROUND

Prior to approximately 1970 geotechnical engineers relied upon a number of sampling and testing tools from which calculations and foundation recommendations were made. This included the Standard Penetration Test, Unconfined Compression, Triaxial, Vane Shear, Consolidation, Cone Penetrometer, and other tests. Even with these sampling and testing techniques, certain deposits such as overconsolidated clayey soils could not be sufficiently characterized. The pressuremeter became available in about 1970 and fulfills this need. Soil Testing Services, Inc. obtained a pressuremeter in 1972 and began to use the data obtained from this test for justifying higher bearing pressures and for more accurate settlement predictions. In the early days of pressuremeter testing, some of the initial pressuremeter data was suspect because of poor borehole preparation. However, with experience, reliable test data was obtained that allowed for more reliable settlement predictions.

TYPICAL PRESSUREMETER TEST PLOT

A typical pressuremeter test result performed in a hardpan deposit in downtown Chicago is shown as Figure 1. The pressure where the pressuremeter engages the side of the pre-drilled borehole is labeled as P_0. The creep pressure is labeled P_f. This pressure has been correlated by Lukas and deBussy (1976) to the pre-consolidation pressure of cohesive soils. The modulus of the soil between P_0 and P_f is shown on the

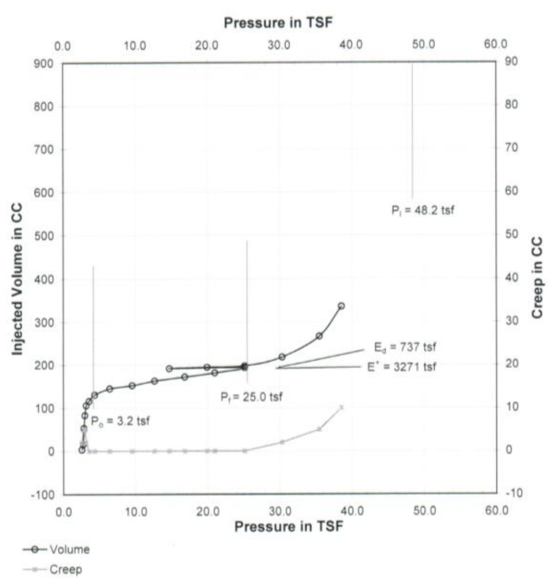

Figure 1. Pressuremeter Data Reduction, 76-foot Depth

pressuremeter plot. Frequently, there is an unload and reload modulus similar to that which is usually done during a normal consolidation test. Beyond the creep pressure, the soil no longer behaves elastically and the failure pressure, which is called the limit pressure, P_l is determined. Generally, P_l is not reached in the test during loading because of the high pressures that would be required. However, there is a prescribed method for extrapolating the data to determine the limit pressure.

DEPOSITS SUITABLE FOR PRESSUREMETER TESTING

The pressuremeter test generally takes on the order of 15 to 20 minutes to perform. The test results measure the undrained properties of the soil. If settlement predictions are to be made, the pressuremeter should be used in deposits where deflections upon loading are not time dependent. In low strength clayey soils that are normally consolidated or in other very soft deposits, more conventional testing such as a vane shear test to determine the shear strength or a consolidation test for prediction of settlement should be used.

The soil types where the pressuremeter has been used includes:

1. Hardpan deposits – The hardpan in Chicago is so high in strength that it is not possible to push a Shelby Tube so as to get an undisturbed sample for testing. The bearing pressures that are used on hardpan are kept below the creep pressure because the modulus used in the settlement predictions is only appropriate up to the creep pressure.
2. Sandy and silty soils – It is virtually impossible to obtain an undisturbed sandy or silty soil for laboratory testing. The standard penetration test is generally used to indicate the relative density of the soil from which bearing capacity and settlement predictions are made. The pressuremeter in the sandy and silty soils will have a similar shape as for clayey soils with a modulus and a creep pressure that can be used for predicting settlements more accurately. Most pressuremeter tests in these deposits are performed above the water table because it is difficult to maintain an open borehole even when using drill mud to prepare the proper diameter borehole for pressuremeter testing below the water table.
3. Fill deposits after soil improvement – Dynamic compaction has been used to densify many miscellaneous fill deposits. This improvement can be measured by pressuremeter testing and thus allowable bearing capacity and settlement predictions can be made. This is a more appropriate type of soil property measurement than the standard penetration or cone penetration test because refusal of large chunks of debris within the fill may cause refusal with these sampling techniques.
4. Residual soils –Along the eastern sea board of the United States the pressuremeter has been used to test residual soil deposits. The classification of these soils varies considerably from a soil to a soil and rock mixture to a weathered rock. Conventional sampling often does not produce significant data to allow for bearing capacity and settlement predictions.

PRESSUREMETER TESTING IN DENSE COHESIVE SOILS

In the 1920 decade, the highest buildings that were constructed within the downtown area of Chicago were on the order of 10 to 20 stories. Many of these buildings were constructed on shallow foundations situated just below basement level. Significant settlement followed as a result of consolidation of a low strength clayey soil that is generally present within a depth range of 15 to 50 feet below grade in the Chicago area. The typical soil profile in the business area of Chicago is shown in Figure 2. Note the hard clay below

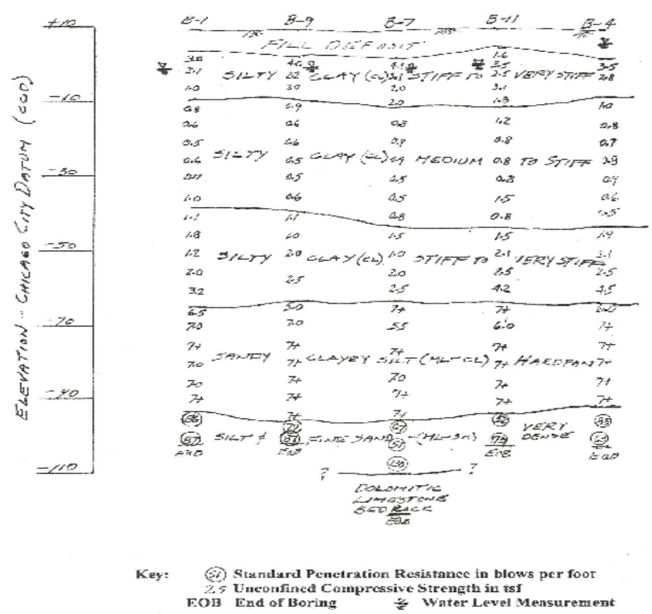

Figure 2. Typical Soil Profile – Downtown Chicago

elevation -85 Chicago City Datum. The natural water content is on the order of 11 to13 % and the unconfined compressive strength exceeds 6 tsf. This deposit is commonly called hardpan.

As the number of stories started to increase subsequent to 1920, deep foundations became necessary to support the heavier column loads. This included pile foundations and drilled piers extended to hardpan or to rock. An allowable bearing pressure of 12,000 psf was considered acceptable by the Chicago Building Code for piers extended to hardpan. This bearing pressure remained as the normal pressure for hardpan piers for a long period of time. Most geotechnical engineers knew that the allowable bearing pressure could be increased, but confirmation was lacking. It was not possible to push a Shelby Tube into the hardpan because of the high resistance. Standard penetration testing frequently resulted in disturbed samples with very high blow counts. Attempts were made to core the hardpan with both a standard core barrel or a Denison sampler but full recovery was rarely obtained. Gradually higher bearing pressures on the order of 20 ksf to 25 ksf were used based upon judgment plus performance of buildings where higher pressures were used. Baker [1984]. In more recent time, the pressuremeter data was used to justify bearing pressures as high as 40 to 50 ksf. At this pressure settlements can range from 1 to 2 inches but the structural engineers can design the superstructure to accommodate this movement.

PRESSUREMETER TESTING IN STIFF COHESIVE SOIL

At a site in the northern suburbs of Chicago, a 5 story building was planned to be supported on shallow foundations. Column loads ranged from 1,000 to 1,500 kips within the center core of the building and 200 to 500 kips along the perimeter. The initial subsurface exploration indicated that a 25 foot thick deposit of silty clay soils [Liquid Limit= 25, Plastic Limit= 13] was present at about 5 foot distance below proposed foundation level. The unconfined compressive strengths based upon hand penetrometer tests obtained from the split-barrel sampling procedures indicated the compressive strength to be only on the order of 1 to 1.25 tsf.with an average of 1.11 tsf. The average unconfined compressive strength of the clayey soils based upon 3 inch Shelby Tube sampling was found to be on the order of 1.5 to 1.75 tsf. Vane shear test in this deposit indicated an average shear strength of 2.9 ksf. Using an approximate procedure for estimating the shear strength based upon pressuremeter measurements, Lukas [2005], the shear strength calculates to be 2.7 ksf. The shear strength measured by these various procedures and the resulting allowable bearing pressures for a footing supported at a depth of 3.5 feet are shown in Table 1. An allowable soil bearing pressure of 2 to 3ksf would be predicted by the conventional sampling and laboratory tests. Based upon the vane shear and pressuremeter tests, an allowable bearing pressure of about 6

Table 1

Measurement	Hand Penetrometer	Unconfined Compression	Vane Shear	Pressuremeter
Undrained Shear Strength	1.11ksf	1.5 to 1.75 ksf	2.91 ksf	2.70 ksf
Allowable Bearing Capacity	2.19 ksf	3.1 ksf	5.51 ksf	5.80 ksf

The allowable bearing capacity was calculated for a footing at a depth of 3 feet below grade. The pressuremeter bearing capacity was based upon the pressuremeter prediction procedure.

ksf could be used. The high silt content likely contributed to the low shear strength measurements based upon the conventional sampling and laboratory testing. A CU triaxial test was performed on one sample and the shear strength was determined to be 2.4 ksf. This is in reasonable agreement with the vane shear and pressuremeter predicted shear strength.

ADVANTAGES OF USING PRESSUREMETER DATA

Using the data generated by the pressuremeter test generally results in a higher allowable bearing capacity and a better settlement prediction than by the parameters obtained from conventional soil testing. There are a number of reasons for this, and a few are listed below.

1. Undisturbed testing – With proper borehole preparation, the pressuremeter tests the soil deposit in an undisturbed condition. With conventional sampling such as Standard Penetration Testing or 3 inch Shelby Tube sampling, there is always some disturbance produced by procuring, handling ,and transporting of the soils plus stress relief associated with removal of the sample from the ground or from the sampling process itself. As part of testing for the Chicago Subway System, Peck and Reed (1954) determined that unconfined compressive strengths obtained from 2 inch diameter Shelby Tube Samples was about 75% of the strength obtained from very carefully hand carved specimens from subway headings. Ladd and Lambe [1963] stated that for a wide variety of clays, the strength values from UU tests are only 40 to 60% of the values from CU tests.
2. Lateral stress in the ground – When the soil sample is removed from the ground by conventional sampling procedures, the lateral stress is removed. During the pressuremeter test, that lateral stress and even the stress history of the deposit is maintained during the testing.

   The importance of lateral stresses in determining bearing capacity, stress distribution within the soil mass, pile/shaft skin friction, and settlement is discussed in the paper by Schmertmann (1985). Plate load tests were performed within a sandy soil deposit where the lateral confining pressure could be increased by inflating airbags all around the test cell (See Figure 3). At a lateral confining pressure

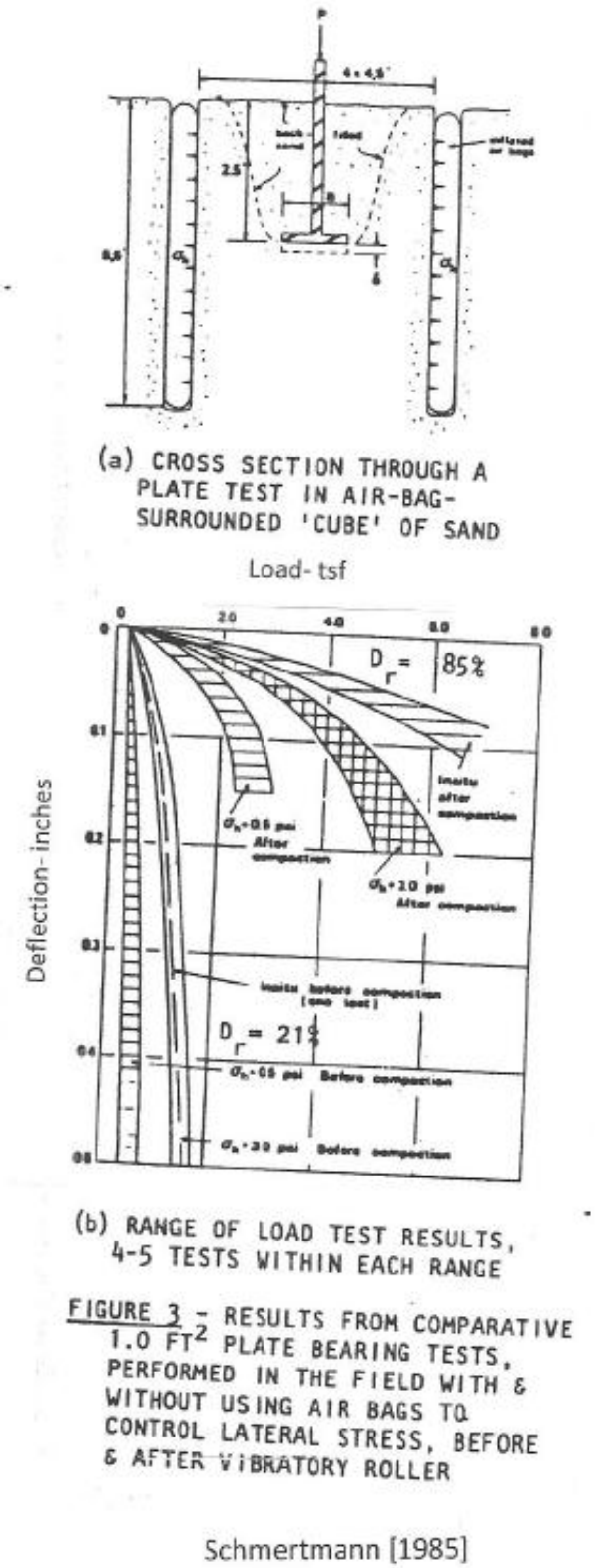

Figure 3. Lateral Stress on Vertical Load Support

of 0.5 psi, an allowable bearing pressure of 2.5 tsf could be applied to the plate before excessive deflection. If the lateral confining pressure was increased to 3 psi, a load of 5 tsf could be applied to the plate.

In the manual entitled " Micropile pile design and construction guidelines" (FHWA 2000), the grout to soil skin friction increases significantly as a result of post-grouting. The increase in the grout to soil strength as a result of post-grouting is frequently on the order of 1.5.

3. Influence of overburden pressure – At a site located to the south of downtown Chicago, pressuremeter tests were performed within a hard clay deposit classified as a hardpan soil. The natural water content of this deposit ranges from 12 to 13 percent and the unconfined compressive strengths are in excess of 7 tsf. The initial pressuremeter test was performed at a depth of 53 to 55 feet below original grade before excavation of a deep basement started. A second pressuremeter test was performed after the excavation extended to a level of approximately 54 feet below ground surface. A hand auger was used to prepare the borehole in the hardpan and the second presuremeter test was performed at a depth of 54 to 57 feet below ground surface. The results are plotted in Figure 4.

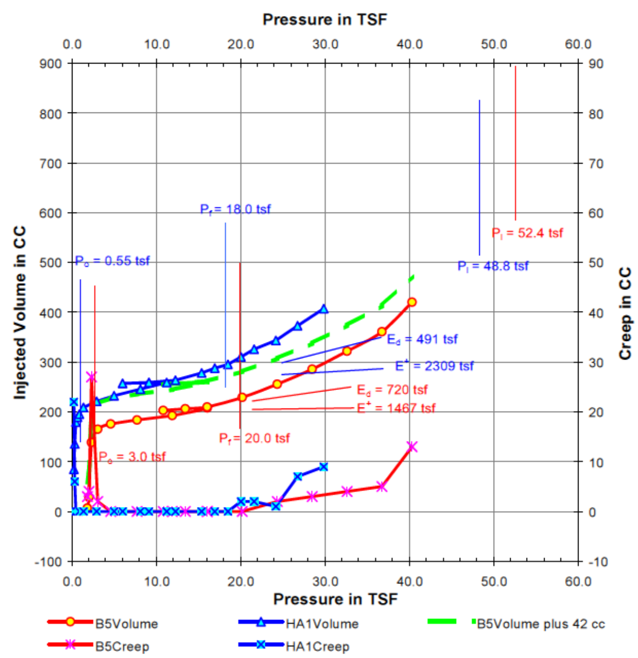

Figure 4. Pressuremeter Before and After Excavation

The limit pressure has been decreased by approximately 3.9 tsf which corresponds approximately to the overburden pressure of 3.6 tsf at a depth of 55 feet below grade. This demonstrates that the overburden pressure has a pronounced influence on the limit pressure. The difference between the limit pressure and the pressure at rest is also slightly reduced after the excavation. The creep pressure is approximately the same before and after excavation although there is a slight reduction. The greatest difference occurs with the pressuremeter modulus. Before the excavation was made, the modulus was determined to be 720 tsf, whereas, after excavation of the modulus was only 491 tsf. The calculation for the shear strength using the expression developed by Lukas [2005] results in approximately the same predicted shear strength of the hardpan soils. Before excavation, this value was predicted to be 7.34 tsf and after excavation, the shear strength is predicted to be 7.65 tsf.

It can be seen that the overburden pressure has an effect on the limit pressure, creep pressure, and earth pressure at rest, that is somewhat related to the overburden pressure. However, the modulus is greatly affected by the removal of the confinement.

CASE HISTORIES

High Rise Building on Drilled Piers One of the earliest projects where the pressuremeter was used for predicting settlement of a high rise building in Chicago supported on drilled piers extended to hardpan is discussed in Lukas [1986]. Figure 5 depicts the soil profile and the location of the drilled

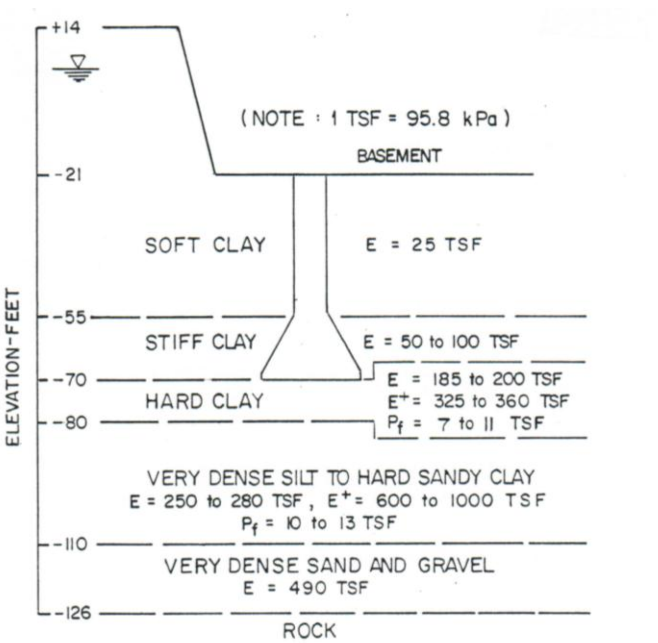

Figure 5. 75-Story Apartment Building

piers. The column loads ranged from 15,000 kips for an interior pier to 9,800 kips for an exterior pier. The predicted settlements ranged from 1.9 inches for an interior column to 1.3 inches for an exterior column supported on drilled piers. The predicted settlements were for compression within the soil deposits. Based upon survey readings with the elastic compression of the shaft subtracted from the measurements the resulting net settlement of the soil below the base of the drilled pier was determined. As shown in Figure 6, the predicted settlements are in close agreement with the measured settlement.

LOADING INFORMATION AND SETTLEMENTS

	Interior Pier	Exterior Pier
Bell Size	27 feet	25 feet
Soil Pressure	13.5 tsf	10 tsf
Predicted Settlement	1.9 inches	1.3 inches
Measured Settlement	1.6 - 2.1 inches	1.0 to 1.6 inches

Note: 1 inch = 25.4 mm, 1 ft = .305 m

Figure 6. 75-Story Apartment Building

Mat foundation on hardpan type soil. A 65 story building with a basement extending to 66 feet below grade was originally planned to be supported on bored piles. The soil below the

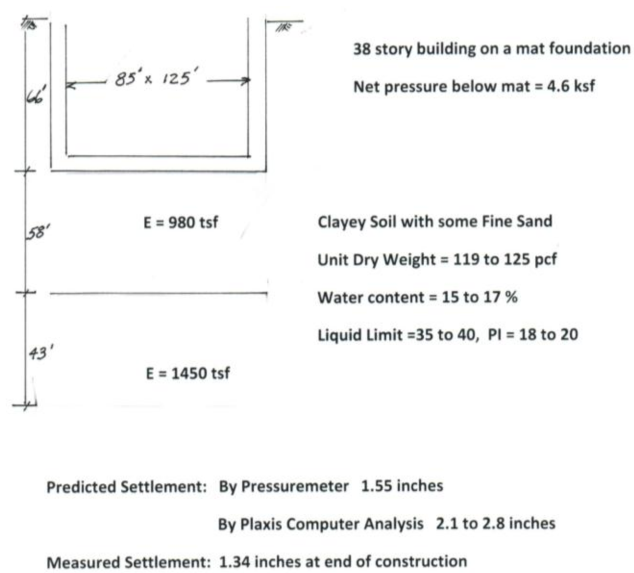

Figure 7. Repsol/Casa Madrid Settlement

basement consists of a hard sandy silty clay. As shown in Figure 7 the natural water content of the formation was on the order of 15 to 17 percent and the net contact pressure beneath the mat was on the order of 4.6 ksf. Based upon the modulus values obtained from pressuremeter testing, the settlement was predicted to be 2 inches. The structural engineer was confident the building could tolerate this movement. The design/build contractor for the project completed a separate set of calculations based upon elastic theory and the Plaxis Computer Method for calculating settlements. These calculations resulted in a settlement prediction of 2.5 inches. Settlement plates were installed within the hard clay deposit at basement level and measurements taken as the structure rose to its full height of 65 stories. These measurements indicated that the ground heaved on the order of 0.5 inches as the excavation was made to the final depth and this was then

Figure 8. Repsol/Casa Madrid – Mat Foundation

followed by an additional two inches of compression. Figure 8 is a picture of the completed structure.

Load tests – It is very expensive to perform load tests on a drilled pier foundation because of the extremely high reaction loads that are required. However, there are at least 3 known load tests performed on drilled piers that were extended to hard clayey soil deposits. This includes:

1. Union Station – Both a straight shaft and an enlarged base drilled pier that were extended to hard clayey soils were load tested, D'Esposito [1922]. A shaft with an enlarged base of 8.2 feet diameter was loaded to a maximum of 970 tons. After subtracting the skin friction load of 250 tons, the resulting bearing pressure on top of the hardpan was 13 tsf. Settlement at one half the maximum load was 0.3 inches. Using pressuremeter tests that were performed on a more recent project at this site, the predicted settlement was 0.33 inches. Figure 9 shows the load test set up. Figure 10 lists the predicted and measured settlement

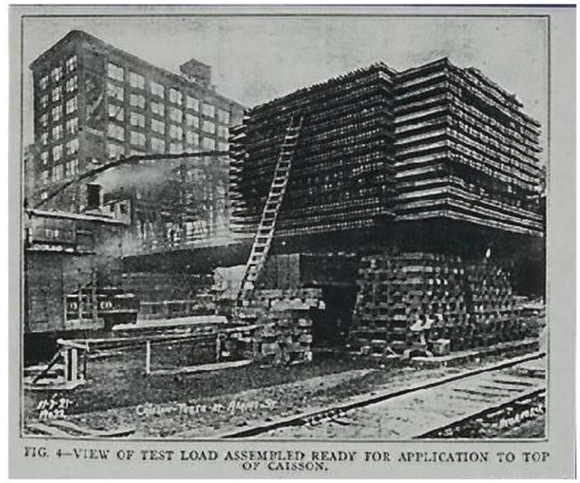

FIG. 4—VIEW OF TEST LOAD ASSEMBLED READY FOR APPLICATION TO TOP OF CAISSON.

Figure 9. D'Esposito Load Test

Test Location	Caisson Diameter	Caisson Elevation	Maximum Test Load Bearing Pressure	Observed* Settlement of Base	Observed* Settlement @ ½ Max. Bearing Pressure	Average Pressure-meter Modulus in TSF E_a	Average Pressure-meter Modulus in TSF E_b	Pressure-meter** Settlement at ½ Max. Load Bearing Pressure	Ultimate Capacity On: Pressurem eter	Ultimate Capacity On: 9 x C
Union Station 1	8.2'	-60.0±	18.4 tsf	0.75"	0.3"	335	335	0.33"		
Union Station 2	4.2'	-60.0±	61.0 tsf	2.0"	0.9"	335	335	0.88"	85.0 tsf	36 tsf
One Park Place	6.3'	-67.4±	24.0 tsf	1.4"	0.4"	247	320	0.55"	54.4 tsf	27 tsf
Univ. of Chicago	2.5'	-38.0±	50.0 tsf	2.2"	0.45"	460	460	0.41"	48.6 tsf	52 tsf

Conversion Key: 1 Ton Per Square Foot (tsf) = 95.8 kilopascals (kPa)
1 inch (IN) = 25.4 Millimeters (mm)

*First Load Only
**Based on Menard Rules and using $\alpha = +0.5$

Figure 10. Caisson Load Test Comparison

2. A straight shaft that was 4.2 feet in diameter was extended to the same depth as the enlarged base pier. A load of 1200 tons was applied at the surface. After subtracting the skin friction load of 250 tons, the resulting soil bearing pressure was 61 tsf. The measured settlement at one half the maximum pressure was 0.9 inches. The predicted settlement from pressuremeter data was 0.88 inches after the load test was completed an access shaft was excavated approximately 40 feet away from the test pier and a tunnel extended horizontally to the bottom of the shaft to permit removal of the soil from below the base of the straight shaft. Upon reloading, the maximum value of side friction was reached at a load of 250 tons and the corresponding settlement was less than ½ inch. The corresponding unit skin friction was calculated to be approximately 650 pounds per square foot. See Figure 11.

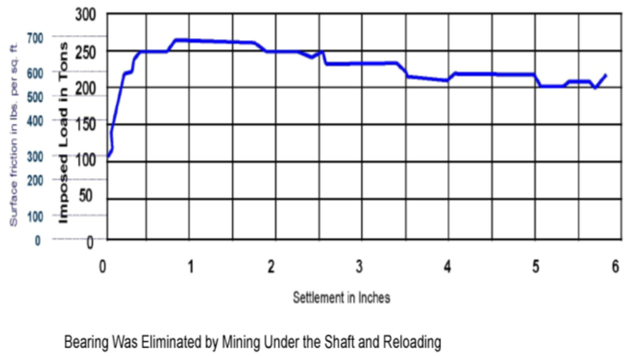

Figure 11. Union Station Friction Load Test

3. University of Chicago load test – The University of Chicago is located approximately 8 miles south of the downtown area of Chicago. A hard silty clay with an unconfined compressive strength of 6 tsf is present below depths of 30 feet below ground surface. Three drilled piers were installed to a depth of 40 feet and were load tested. The results of these tests are described in an article by Holtz and Baker (1972). Figure 12 shows the load test set arrangement.

   A 30 diameter shaft was drilled and a 30 inch concrete base was formed at bearing level. A 24 inch diameter casing was installed above this level and filled with concrete. Bentonite slurry was introduced in the annular space between the casing and the surrounding soils thereby removing skin friction so load support was entirely in end bearing on the 30 inch diameter concrete pad.

   Another 24 inch diameter shaft was installed with the concrete poured tight to the soil so as to develop the skin friction. End bearing was eliminated or reduced by creation of a void at the base of the shaft with a plywood board underlain by a doughnut shaped inner tube. Sufficient air pressure was injected into the inner tube to support the fresh concrete that would rest on the plywood. The third shaft was constructed in a conventional manner with combined end bearing and friction. This was also a 24 inch diameter shaft.

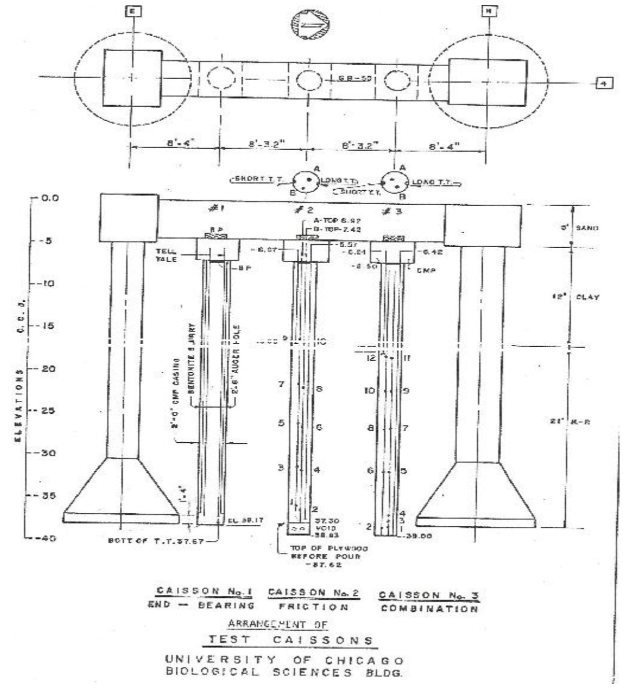

Figure 12. University of Chicago Load Test

   Based upon the load test with only end bearing, the unit contact pressure at failure was 56 tsf. At one half of this pressure, the measured settlement was 0.45 inches. Based upon pressuremeter data, the predicted settlement was 0.46 inches. Figure 10 lists the measured and predicted settlement.

4. High rise building load test – A new building was constructed at a site where existing drilled pier foundations were present from the previous building that was demolished. Drilled piers were installed to support the new structure. A thick concrete mat was constructed over the new and existing drilled piers. The original drilled pier shaft had a diameter of 30 inches and a bell diameter of 6.3 feet. This pier was load tested using the mat to transfer loads from the weight of the super structure as it was rising in height. Details are presented in a master's thesis by Bucher (1986). The maximum load of 1060 tons was applied for 6 hours. After subtracting the load carried in skin friction, the pressure at the base of the bell was on the order of 56 ksf. A settlement of 2 inches was measured under this loading. Elastic settlement under loading was measured by stress

cells and the deflection at the base of the pier was determined to be 1.75 inches. Using pressuremeter test data, the settlement was calculated to be 1.55 inches.

CONCLUSIONS

1. The pressuremeter data and calculation procedure provides a better estimate of bearing capacity because the measurements are obtained in-situ under the lateral and vertical stress conditions that prevail in the soil mass.
2. In cohesive soils with significant silt content, the pressuremeter provides a better measure of bearing capacity than conventional sampling and testing. This is attributed to testing under confined conditions in the ground during pressuremeter testing.
3. For cohesive soils loaded to a pressure less than the creep pressure, the settlement predictions are in good agreement with the measured settlements of constructed buildings and from load tests.
4. The pressuremeter provides a useful geotechnical tool where conventional sampling procedures cannot produce undisturbed samples for laboratory testing.

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