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SYNOPSIS: The Sohio Corporate Headquarters building foundations in Cleveland, Ohio are relatively unique, involving as they do some of the deepest caissons on record, combined with a socket friction design.

This paper reports the performance of a full-scale load test and the results of instrumentation programs performed to evaluate the design and performance of 240-ft (73 m) deep rock socket caissons at the Sohio Corporate Headquarters building project. The load test was carried out to 2.5 times the theoretical design capcacity and the results are reviewed in terms of both total capacity and the individual design parameters, such as socket friction. Details of the instrumentation program used to evaluate concrete strain and corresponding load transference as a function of applied load, caisson depth, and time are also presented. In addition to the load test, the installation details and results of a production caisson instrumentation program to permit long-term monitoring of concrete stress and strain levels are reviewed.

INTRODUCTION

Construction of the Sohio Corporate Headquarters building in Cleveland, Ohio began in early spring of 1983 and was completed in the spring of 1985. Due to the subsurface conditions at the site, caisson (drilled pier) type foundations were required to support the 46-story tower section of the building. Due to the known gas conditions in and over the shale bedrock (the anticipated bearing stratum), it was anticipated that hand clean-up and physical bottom inspection of the caissons would not be practical and that it might be necessary to construct the caissons under water. For this reason, a design based on extending sockets into the shale sufficiently to carry a major portion of the load in socket friction was developed. The loads on the caissons range from 3,000 to 14,000 kips (13 MN to 62.3 MN) including wind loads, resulting in caissons extending to a depth of up to 250 ft (76 m) below street level with shaft diameters of 3.5 to 7 ft (1.1 to 2.1 m) at the socket.

To substantiate the design, a full scale caisson load test with a planned test load at the socket of 2.5 times the design load was performed. To obtain the required loads, the test setup required a reaction load of 1250 tons (11.1 MN). The purpose of the full scale caisson load test was to determine how the load would be carried by the caisson and socket, and to confirm the design capacity, both total capacity and the individual design parameters, such as socket friction. To further evaluate the design, one of the major production caissons was fully instrumented to permit long-term monitoring of stress levels along the full depth of the caisson, both during construction and after completion of the building.

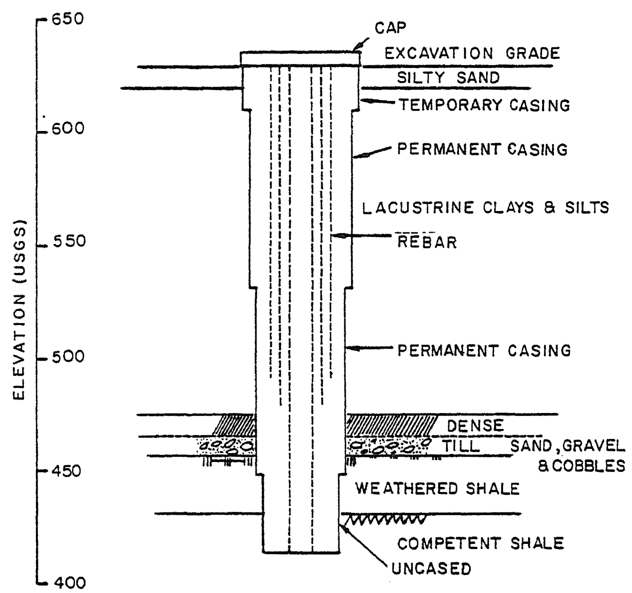
In this paper, a detailed description of the caisson load test is presented, including the physical setup and instrumentation. The results obtained are reviewed and conclusions resulting from the analysis are presented. Also included are details of the instrumentation program for a production caisson and an analysis of measurements taken as of this writing. The results of the load transfer measured in both the load test caisson and the production caisson are then compared.

PROJECT DESCRIPTION

Subsurface Conditions: The subsurface profile at the site is shown in Figure 1. As the figure shows, the subsurface conditions consisted of silty sand to a depth of approximately 30 to 40 ft (9 to 12 m), lacustrine clays and silts to a depth of about 170 ft (52 m), glacial till overlying silty sand, gravel and cobbles to a weathered shale at a depth of 190 ft (58 m), with competent shale at a depth of 220 ft (67 m). The surface water table was located at a depth of approximately 20 ft (6 m) with a deep water table in and over the weathered shale at a depth of approximately 70 to 90 ft (21 to 27 m). To develop sufficient socket friction, the caissons were designed to extend from 1 to 2 diameters into the competent shale layer.

Load Test Setup: The physical load test arrangement is depicted in Figure 2. The plan for the load test consisted of constructing a 3 ft (0.9 m) diameter load test caisson, a non-production caisson, between two production caissons which serve as anchor caissons. The load test caisson was designed to transfer all of the applied load directly to the rock socket by isolating the caisson shaft from the surrounding earth all the way down into the shale socket. This was achieved by placing a 3 ft diameter casing inside the normal top, intermediate and bottom casings required to construct a normal caisson. The 3 ft diameter inner-casing was braced at the top to minimize lateral movement and at the third points to protect against buckling.

FIGURE 1 SUBSURFACE PROFILE AND CAISSON PLAN



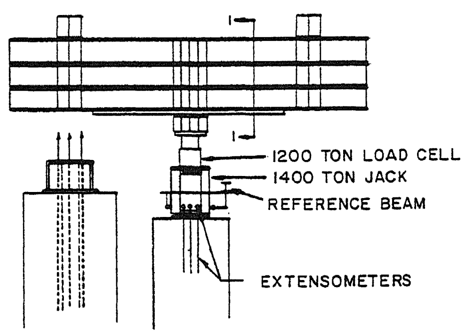


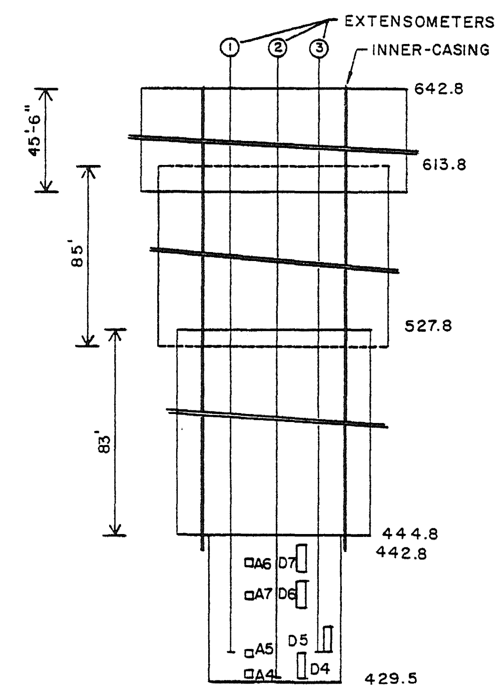
FIGURE 2. CAISSON LOAD TEST SET-UP

Several problems occurred during construction that had an influence on interpretation of the test results. A leak developed beneath the top casing which inadvertently resulted in sand and silt in the annular space between the intermediate casing and the inner-casing. Bottom cleaning and sounding, although performed, were hindered by the close steel cage and concern for instrumentation damage. Finally, an overrun in concrete yardage by 6 yards^3 (4.6 m^3) indicated the possibility that a tight seal was not achieved between the inner-casing and the shale socket so that some concrete leakage could have occurred underneath the casing into the annular space outside the inner-casing. It is also possible that the socket drilled in the shale was larger in diameter than assumed because of wobble in the drill auger as the hole is drilled. These possibilities become very important later when analyzing load transfer to the socket.

INSTRUMENTATION SETUP

The instrumentation setup is depicted on Figure 3 and consisted of; two (2) sets of Carlson strain gauges placed at four different levels in the rock socket, wire extensometers, access casing for non-destructive testing, and a seismic pulse transducer (G-Cell). The Carlson gauges are referenced as either D-gauges or A-gauges. The D-gauges are approximately 30 inches (760 mm) in length. As strains are averaged out over the full length of the gauge, they are more representative of average conditions. The A-gauges are 8 inches (200 mm) in length and while more sensitive, can be misleading because they measure strain over a very short distance and may indicate abnormalities rather than average conditions. The gauges were wired to the cage prior to placement

FIGURE 3. INSTRUMENTATION ARRANGEMENT



In order to be able to monitor the tip movement of the caisson during loading, special tell-tales or wire extensometers were installed as shown in Figure 3.

The movement of the caisson bottom was monitored by measuring the movement of the wire cable attached to a plate at the bottom of an outer protective pipe weight pulling on a wire cable.

Unfortunately, in cutting the caisson, the wires to the G cell were destroyed, rendering it inoperable

LOAD TEST RESULTS

The load test procedure consisted of loading the caisson on the first load cycle in increments of 100 tons (890 kN) up to 1,000 tons (8.90 MN) and in increments of 50 tons (450 kN) above 1,000 tons (8.90 MN) to the planned maximum of 1,200 tons (10.7 MN) and then unloading the caisson in three equal increments of 400 tons (3.56 MN). The load vs. deflection was recorded with time using two dial gauges attached to a reference beam with gauges located on opposite sides of the caissons. The dial gauge readings were checked using a wire scale and mirror arrangement with the wire attached to a separate reference from the dial gauges. The load was increased at one (1) hour increments or when the load vs. deflection tended to level off if it occurred in less than one (1) hour.

On unloading from the first load cycle, a small seating load of 70 tons was maintained on the caisson until commencing the second load cycle the next day. On the second load cycle, the first load increment was to 200 tons (1.78 MN) and then each load increment thereafter was 200 tons (1.78 MN) up to 1,200 tons (10.8 MN). With approval of the contractor's engineer, who designed the reaction frame, an additional 50 tons (450 kN) was applied making the maximum load on the second load cycle 1,250 tons (11.1 MN). The unloading sequence was to 800 tons (7.12 MN), 400 tons (3.56 MN) and 0 () MN) tons.

LOAD TEST RESULTS AND ANALYSIS

Load Test: The load test results are summarized on Figure 4 and show the observed deflection of the top of the caisson versus load. Also plotted on the curve are two elastic lines for the concrete. The lower elastic line assumes that all the load is carried from the top of the concrete shaft to the bottom of the concrete with no load dissipation and no deflection at the tip. The upper elastic line assumes full load carried in the concrete shaft to a depth of 20 ft (6.1 m) without dissipation and then gradual linear dissipation of the load to the bottom of the caisson. The modulus of elasticity to develop the elastic lines was obtained by performing laboratory tests on concrete cylinders that were cast at the time of placement of the concrete in the caisson. Allowing for the confinement effect of the steel casing and reinforcement, a modulus of elasticity for the concrete in the caissons of 3.2 million psi (22,000 MPa) was utilized.

It is evident from the load deflection plot that the points plot way above the bottom elastic line. This would indicate that load is being taken out in friction very quickly well above the shale socket where the load was attempted to be transmitted.

In spite of the load apparently carried by friction, at 1,200 tons (10.7 MN) the top deflection falls significantly below the lower elastic line indicating movement of the tip. On unload, a net deflection of 0.8 inches (23 mm) was recorded and the slope of the unload curve is much flatter than the lower elastic line indicating significant locked in friction. This will be discussed further in later sections.

Extensometer Results: The caisson bottom or tip movement as indicated by the tell-tale or extensometer data, is shown on the bottom half of Figure 4. On the initial loading cycle, it appeared that not all of the slack was taken out of the wire lines and that a certain amount of

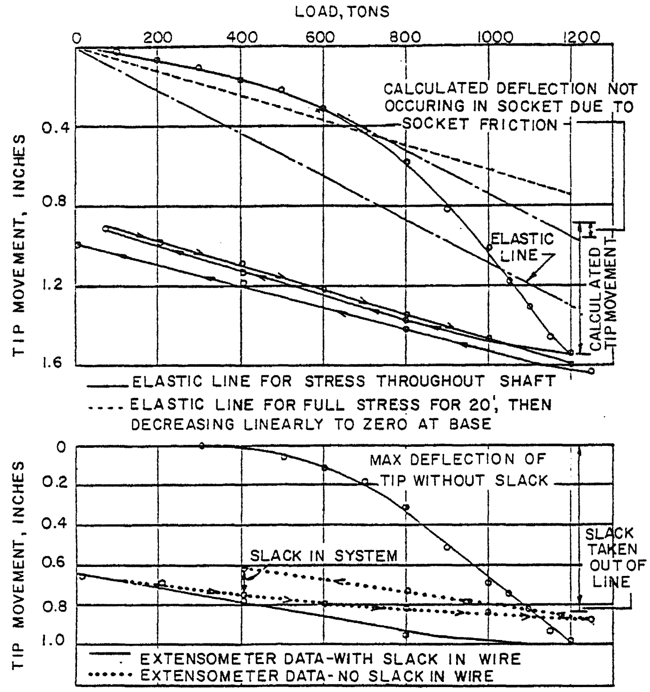


FIGURE 4. CAISSON LOAD TEST RESULTS - LOAD VERSUS SETTLEMENT CURVE

this slack gradually was pulled out as the test progressed. Apparently, kinks in the line that developed during the wire unwinding in installation were not adequately pulled out by the weights that maintained tension in the lines. As this was discovered during the progress of the tests, greater effort was put into pulling the slack out of the line before taking readings for the second load cycle. Thus, the first load cycle is believed to over-indicate the amount of tip movement. Since the measured top movement in the second load cycle went almost to the exact deflection under maximum load as the first load cycle, there could not have been significant increases in the tip deflection of the second load cycle. Thus, the difference in tip deflection measured on the second load cycle using the tell-tales was an indication of slack taken out of the tell-tale system. It is even possible that not all of the slack was yet taken out so that the measured maximum tip deflection of 0.8 inches (20 mm) could still be on the high side. This compares with a calculated tip movement using elastic line analysis and top measured deflection of slightly less than 0.7 inch (18 mm).

Strain Gauge Results: The presumed concrete modulus of 3.2 million psi (22,000 MPa) was also used to calculate the stress level in the concrete at the strain gauge locations. These results are shown in Figure 5.

The stress levels were then multiplied by the transformed area of the caisson shaft at the strain gauge location and the load distribution curves plotted as depicted in Figure 6. The A-strain gauges and the D-strain gauges agree reasonably well for the top 2 strain gauge locations. The second gauges from the bottom appear to not be functioning properly at either

FIGURE 5 . STRESS TRANSFER CURVE

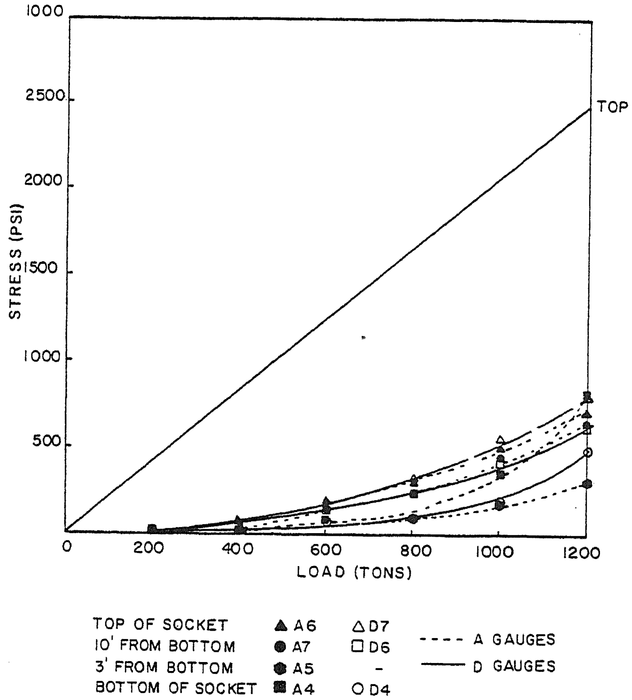
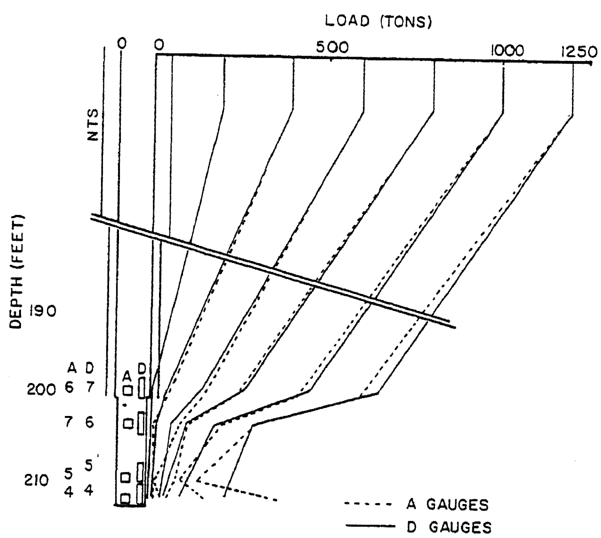


FIGURE 6. LOAD TRANSFER



the A or D locations since negligible changes in readings occurred throughout the loading sequence. At the bottom gauge locations, the A-gauge and D-gauge are markedly different with the A-gauge indicating an illogical increase in loading as compared to upper gauges. The D-gauge indicates a reasonable distribution. A possible explanation for the large strain observed in the A-gauge could be contamination of the concrete in the area of the gauge resulting in a much lower modulus than actually used to calculate the stress. Since the D-gauge is larger and averages more concrete, it is believed to be more representative of the conditions and forms the basis of our subsequent analysis on socket friction. Subsequent non-destructive testing with a nuclear gamma logger supported the possibility of bottom contamination.

Measurements during the load test indicated that sand and silt had flowed in all the way up to approximately 20 ft (6.1 m) from the top of the caisson. In order to see if the observed deflections could be theoretically calculated, based on reasonable soil resistance factors, an analysis was performed.

Since the observed deflection was even flatter than the upper elastic line which assumes gradually increasing soil resistance, it is concluded that some load must be taken by the bracing system used between the inner casing and outer casing to avoid lateral deflections and to protect against buckling. By assuming 100 kips (450 kN) load carried in the braces (obtained by straight line extension of the initial points back to 0) and assuming reasonable soil friction parameters of 30 degrees for friction angle and 0.45 for earth pressure at rest, and by further assuming that a maximum friction value is reached at approximately 20 caisson diameters (60 ft (18.3 m) of soil surface or 80 ft (24.4 m) below the top of caisson) (STS Consultants, Ltd., 1983) a reasonable check was made. The calculated deflection is shown by an "X" plotted on the load deflection curve in Figure 4. The calculated deflection almost plots exactly on the curve. This indicates a maximum load being taken in soil friction and bracing friction of 1180 kips (5270 kN) leaving 1220 kips (5410 kN) of load reaching the socket at the point of maximum loading. This agrees reasonably well with the maximum load indicated by the strain gauges of 1190 kips (5310 kN).

Socket Friction Analysis: In order to confirm the design basis for the caissons, a socket friction analysis was made. Calculations indicated that the 1.3 ft (0.40 m) of competent shale above the bottom gauges carries an average friction of 190 psi (1300 KPa) or well above the design assumption of 160 psi (1100 KPa). If this same friction is assumed to continue for the next 1 ft (0.3 m) of competent shale socket, a load of only 162 kips (720 kn) is left remaining for the bottom 1 ft (0.3 m). Theoretically, this should all be carried in the bottom 1 ft (0.3 m) and there should be no tip movement. Since an observed and calculated tip movement on the order of 0.7 inch (18 mm) was believed to have occurred, the data indicates a soft bottom. One possible explanation is that several inches of sand leaked into the bottom underneath the casing prior to concrete placement (as previously indicated). Such a possible sand bottom would be consistent with observed data, particularly with regard to the second load cycle performance and the reaction of the A-gauge at the bottom of the caisson. In the second load cycle, the compressed bottom appears to behave almost elastically and similar to concrete. This is the way confined sand would behave as increasing load were applied. If the A-gauge were partially or entirely embedded in sand, the sand modulus would be much lower than the concrete modulus used to calculate the stress of the gauge. Thus a much lower stress similar to that obtained from the D-gauge would be obtained.

Another interpretation of the data could have a major part of the load on the socket carried at the top of the socket because of the fact that the casing is 3 ft (0.9 m) in diameter and the theoretical socket diameter is 30 inches (760 mm). The gauge reading 3 ft (0.9 m) below the casing appears to confirm a major load transfer occurring in the top 3 ft (0.9 m) of the socket.

CORRELATION OF CAISSON LOAD TEST TO PRODUCTION CAISSONS

Concerning the data and analysis from the load test, the interpretations presented appear to be reasonable on the basis of the observed data. Other assumptions might vary the load distribution calculated, but would not effect the ultimate fact that the caisson was successfully loaded to 2.5 times it theoretical design capacity and that at maximum load, the total system was behaving almost elastically with not even the first signs of approaching capacity limits. Further, whether a disproportionate amount of load is taken out at the top of the socket or whether it is averaged over the thickness of the socket is academic as far as the design of the production caisson is concerned, since the production caissons have the same general geometry with regard to the casing diameter being 6 inches (150 mm) larger than the socket diameter. However, to provide a clearer picture of the actual load transfer mechanism, a production caisson was fully instrumented.

Instrumentation Program: For the instrumentation program, one of the large caissons which was required to carry the largest loads and would involve the most significant change in loading condition under high wind loads was selected. The particular caisson extended to a depth of approximately 245 ft (74.7 m) below street level with a 7 ft (2.1 m) diameter rock socket extending 17 ft (5.2 m) into competent shale.

Figure 7 depicts the instrumentation setup. Strain cells were located at six different levels in the caisson; namely, near the top of the caisson, near the top of the deep dense till layer, at the bottom of the bottom casing, at the top of the socket penetration into the competent shale, at the middle of the socket, and at the bottom of the socket. In addition, a load cell was located near the top of the caisson. For redundancy and checking purposes, three (3) different strain gages were located at each level. The same short and long Carlson gauges (8" long A-gauges and 30" long D-gauges) used in the load test were selected along with Geokon vibrating wire embedment strain gauges. One of each type of gauge was installed at each level. A Geokon vibrating wire total pressure cell was installed at the cold joint, approximately 20 ft below the top of the caisson. This allowed for careful hand placement of the gauge and the embedding it in non-shrinking grout.

Instrumentation Results: Figure 8 and Figure 9 show the results to date (over two years after completion) of the calculated stress levels in the concrete at the strain gauge locations. These results were calculated using a presumed

FIGURE 7 SCHEMATIC OF INSTRUMENTATION FOR PRODUCTION CAISSON

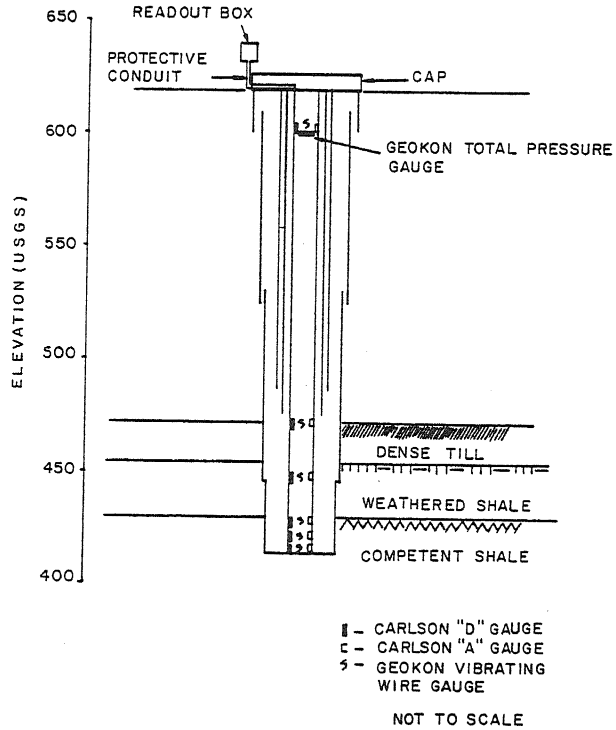


FIGURE 8. A-GAUGES

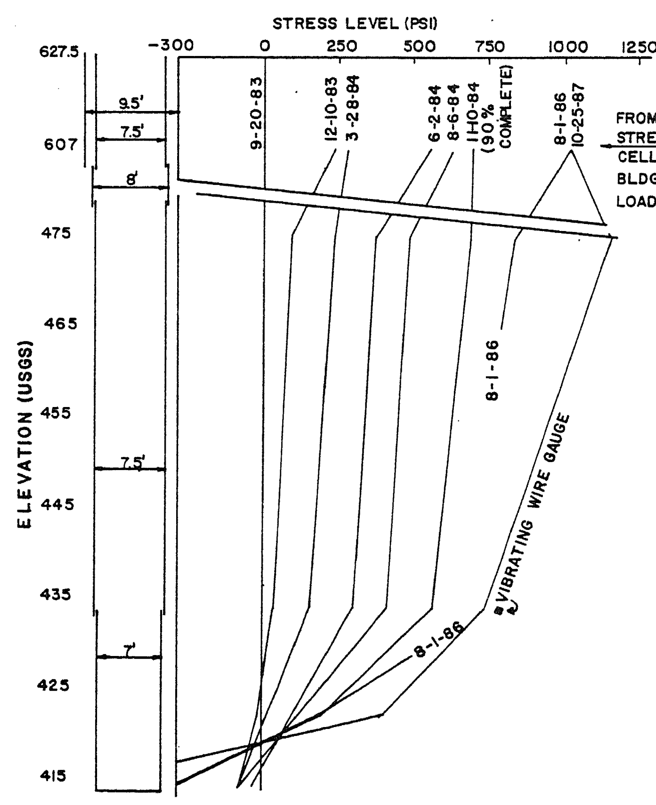
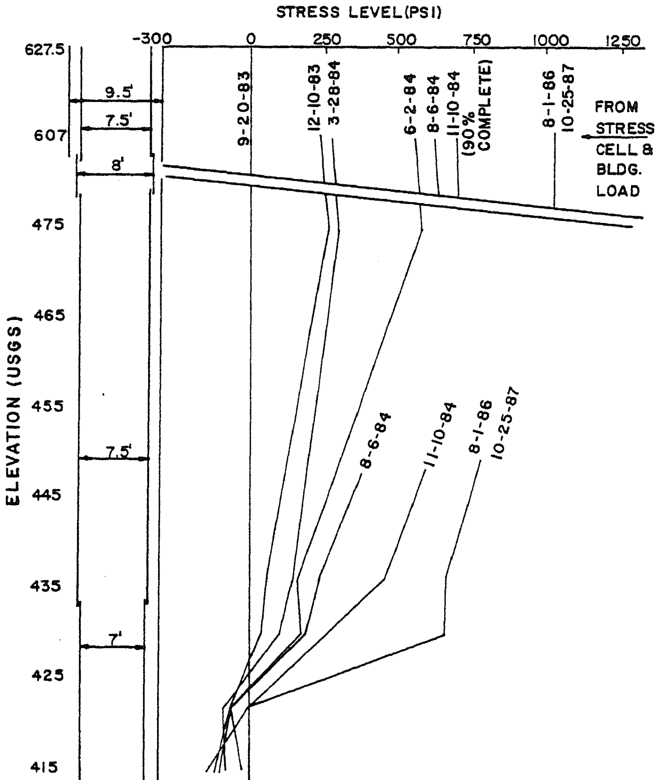


FIGURE 9 . D-GAUGES



concrete modulus of 3.2 million psi (22,000 MPa). Consideration for creep effects and modulus increase due to age were not included due to evaluation difficulties. Creep under a sustained load would result in an apparent increase in strain while increase in modulus would cause an actual decrease in measured strain. As the influences are somewhat offsetting, the relative trend of stress transfer should not have been highly influenced.

The data shown in the figures indicates comparable stress transfer obtained from both gauges. One obvious anomaly is the negative stress results obtained from the bottom socket gauges. The data shown was calculated from a presumed 0 stress level prior to load application. However, internal stresses can be built into the concrete and gauges during thermal changes in the concrete. If a high residual stress were built into the caisson during thermal expansion, this stress may not be relieved as rapidly as upper level stresses since the socket portion of the caisson is confined by relatively incompressible rock. This stress should be relieved with time and it may be that through relaxation the stress levels are decreasing faster than load is actually being applied. For some as yet unexplained reason, there continues to be an increase in negative values. As it is apparently under no load, the continued increase in negative values at the base may be attributed to minimal concrete shrinkage with age below the level at which load is being carried.

Several gauges are inoperative at this time including most of the vibrating wire strain gauges (apparently damaged during construction), the D-gauge at the top of the till and the D-gauge at the base of the caisson. The A-gauge and the D-gauge at the top of the competent shale rock socket are markedly different with the A-gauge indicating an illogically high stress as compared to the upper gauges and the applied load.

Even with the interpretation problems mentioned above, the data does show certain trends consistent with the previous load test data. It definitely appears that significant load is taken out of the caisson above the socket and that no load is being transferred to the base of the caisson.

CONCLUSIONS

The full scale instrumented test caisson was successfully loaded to 120 tons (11.1 MN) which was 2.5 times the theoretical design capacity, thus confirming the design for the production caissons. However, the actual stress at the base of the caisson was much less than anticipated by design indicating substantial load support through friction in the till and weathered shale layers.

The instrumentation of a production caisson correlates well with the load test results in that negligible load has been recorded at the base of the caisson socket even though the full design building load has been in place for several years. Continued monitoring of the production caisson will certainly reveal more information as to the actual long-term support mechanisms including uplift forces under high wind loading conditions.

The results of this testing may allow for an even greater increase in the allowable bearing capacity of caissons in the Cleveland area with confidence that they will provide the necessary load carrying capacity. Hopefully, these results can also be correlated with other tests and design theories on other projects. Only through such hard physical data can theories be verified or modified.

ACKNOWLEDGMENTS

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