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Caisson Design by Instrumented Load Test

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SYNOPSIS Three instrumented axial load tests were performed on 42 inch diameter caissons (drilled piers). These caissons were installed in marine sediments of dense sand overlain by soft sand-clay mixtures. Correlations were made with the Standard Penetration Test to develop design relationships. Production caissons were then designed based on these relationships. Test caissons were approximately 54 feet long and installed by the slurry displacement method. Test loads were carried to 1,000 tons. Mustran cells were used to determine loads in the caissons at different depths. Resulting data is presented graphically as load versus settlement, load versus depth (load distribution), and side friction and end bearing versus both applied load and displacement (load transfer). Special construction considerations and caisson integrity as observed after excavation are presented.

INTRODUCTION

Three bridges were designed for highways to cross a proposed canal near St. Stephen, South Carolina. Design and construction documents were provided by the U.S. Army Corps of Engineers, Savannah District. Though the canal excavation preceded bridge construction, adequate "plugs" were left at each highway location for the bridges to be constructed in the dry. Excavations were then made beneath the bridges to complete the canals.

Originally, each bridge pier was to be supported on a pile cap connected to a group of driven H-piles. Preliminary pile design was made using the Meyerhof (1976) procedures for driven piles. Special excavation and dewatering would be required in order to install these piles and their caps. A load test for each bridge site was required to prove the design capacities determined per Meyerhof, and to correlate them with the blows per foot from the Standard Penetration Test (SPT) obtained during subsurface investigations. If necessary, tip elevations furnished for bidding lengths would be adjusted to obtain the required pile capacities. Provisions were made for additional load tests should a trial and error procedure be required.

Contractors for the bridges submitted a value engineering proposal to delete the pile caps and driven piles and most all of the dewatering and special excavation required to install the government designed foundations. They proposed caissons (drilled piers) be installed from the existing ground surface, i.e. before any excavation. The caissons would be the same diameter as the bridge piers (42 inches typical, 24 inches end bent). The piers would be formed as columns above the top of the caissons. After canal excavation, a portion of the

caissons would be exposed and would act as columns. Significant savings would be obtained by the elimination of dewatering. Standards and criteria of the original design were maintained, while at the same time providing the contractor flexibility and the government quality assurance. The revised specifications required the contractor, at each bridge site, to provide an instrumented load test, develop load transfer data, develop design relationships between these load transfer data and SPT data, and design the production caissons based upon the results of the instrumented load test. Tip elevation was the variable for design. A minimum depth of 25 feet below the final excavated grade was required for lateral load considerations. The original safety factor of 2.5 was maintained. The contractor was required to provide an independent specialist who would interpret the data and develop the design relationships. This paper presents the test results, design relationships, and the more pertinent construction considerations required to provide both the test and production caissons.

SITE CONDITIONS

Geology/Physiography

The bridge sites are located in the Atlantic Coastal Plain physiographic province, consisting of a band of loose to indurated sands, silts, and clays with some limestones and sandstones. These marine sediments are of Upper Cretaceous and tertiary age. The exposed formation is most probably the Santee Limestone.

Topography and Subsurface Conditions

The bridge sites are on relatively flat uplands. Only slight drainage relief is

provided to nearby lowlands. Ground water is typically within a few feet of the ground surface.

Stratigraphy at all three sites is very similar and can be broken into two basic zones as shown in figure 1. The approximate upper 25 feet of material consist of soft sand-clay mixtures, with SPT results in the range of 10-20 blows per foot. (There is abundant SPT data; at least one boring at each bridge bent). Below this lies a much thicker layer of very dense interbedded silty sands, with thin lenses of silty clays and occasional limestone. SPT readings record well in excess of 100 blows per foot in general and never below 50 blows per foot in this zone. The soft upper layer was to be removed by the canal excavation, except for the end bents. Minor subsurface variations between the sites are discussed later.

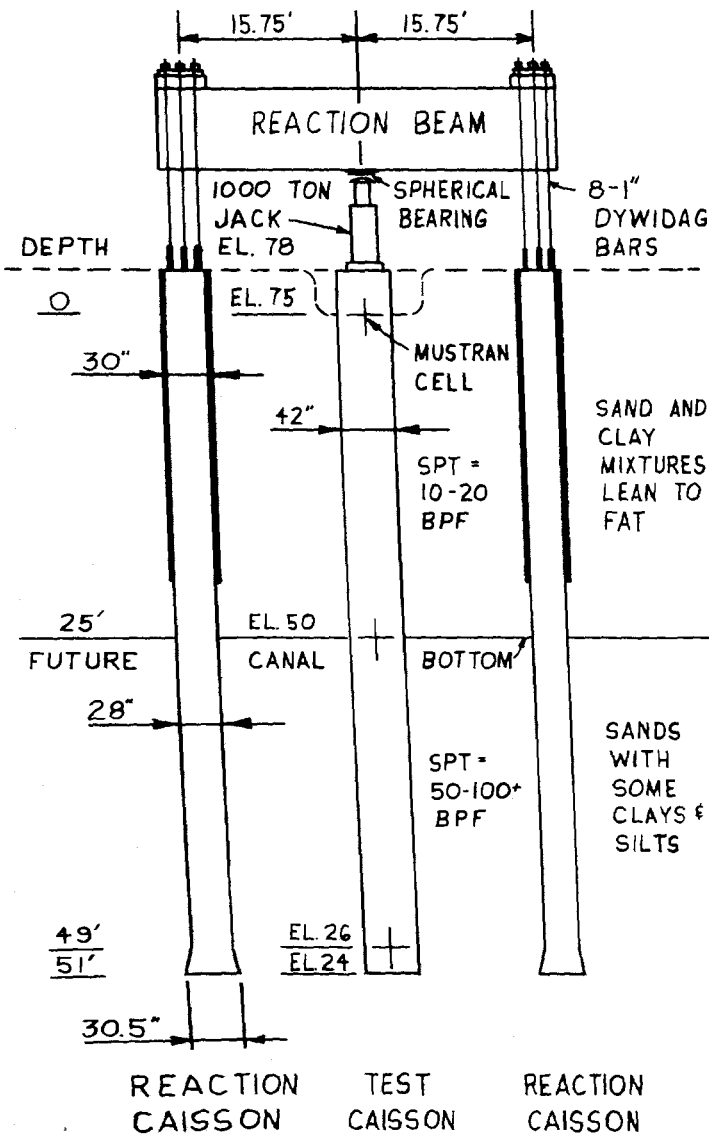


Fig. 1. Soil Strata and Loading Apparatus.

INSTALLATION

All caissons were to be installed by the displacement method because of high ground water conditions and caving soils. Loose of the bottom of the hole due to "boiling" upheaval was of special concern, as this affect end bearing. Research and construction experience indicated a specific procedure preferred for slurry displacement caisson. Installation of the test and production caissons was accomplished as follows:

1. A hole was augered in the upper soft materials to a depth of approximately 15 or 20 feet. A 10 to 15 foot long casing was then installed flush with the top of the hole.
2. As drilling continued, bentonite slurry was premixed and introduced into the hole.
3. When the required depth was reached, the bottom was cleaned out with special bucket augers.
4. The reinforcing cage was then introduced into the hole and secured to the top casing, centered at its proper elevation.
5. Immediately before concreting, the slurry at the bottom of the hole was sampled and checked for density. If too high, the slurry in the hole was agitated and/or water was added.
6. The tremie pipe was then inserted and concrete was introduced into it to the bottom of the hole. As the concrete exited the bottom of the tremie pipe, slurry was displaced out the top of the hole and returned to storage tanks.
7. When the concrete reached the top, the casing was raised slightly. Additional concrete was introduced through the tremie pipe as the casing was slowly lifted. This provided a positive head of concrete to fill in any voids left by the rising casing. A sudden rush of concrete occurred when the casing was clear of the top of the hole.
8. Finishing was accomplished by hand shovels to remove excess concrete and trim the top of the caisson to its proposed dimension.

The reinforcing was somewhat less for the caissons than for the production caissons. Otherwise, the contractor was required to install the production caissons with essentially the same procedures and equipment as test caissons. This was to eliminate differences which could otherwise result from changes in construction techniques or equipment.

INSTRUMENTED LOAD TESTS

Test Apparatus

In the test caissons, Mustran cells were installed at the top, bottom and approximately the middle of the caisson (the point where the soil conditions changed from the upper soft layer to the lower dense sand). See fig. 1. The Mustran cells at the top (where the caisson was isolated from the surrounding ground to prevent load transfer) experienced the full applied load to the caisson. When corrected for differences in shaft stiffness, these readings were correlated to the other cells to determine loads at the other levels. In addition to these cell readings, conventional axial settlement and lateral deflection of the caisson head were measured. Measurements were made by a dual system of wire/scale/mirror and dial gages. Typical loading apparatus is shown in figure 1. A large single ram jack was used to apply the load and an air-driven oil pump was used to provide the load to the jack. Calibration of the jack was used to determine the jack load at each increment. According to the contractor, a load cell that would accommodate a 1,000 ton jack load was not available at the time of this test. The testing frame, reaction caissons, and deflection measurement apparatus conformed to ASTM D1143.

Test Method

The caisson was loaded in accordance with the quick load test method in ASTM D1143. This method provides small increments of load held for short time periods. The time required to hold each load (2 to 3 minutes) is essentially the time required to read all the instrumentation and prepare the jack pump for the next load increment. Increments were typically 20 tons up to a load of 640 tons, then 40 ton increments to maximum load. After holding the maximum load for approximately 10 minutes, the caisson was unloaded in 240 ton decrements. In two of the tests the maximum load was carried to 1,000 tons which represents the capacity of the jack and jacking system. In the third test the maximum load was approximately 800 tons. The test was stopped at this point because of large deflections approaching the limit of the jack ram.

Summary of Results

As previously mentioned, Mustran cell readings were used to determine the load in the caisson at different levels. Many variables are introduced in this system, including unknown caisson diameter and concrete modulus, bending of the caisson, strain gage error and anomalies in the concrete immediately around the Mustran cells. Nevertheless, experience has shown that when proper judgement is applied to the cell readings, satisfactory data can be obtained. Further discussion of the reduction of the data is not made here but can be found in Barker and Reese (1969).

Load versus top deflection (settlement) for the three test caissons differed considerably over the test ranges as shown in figure 2.

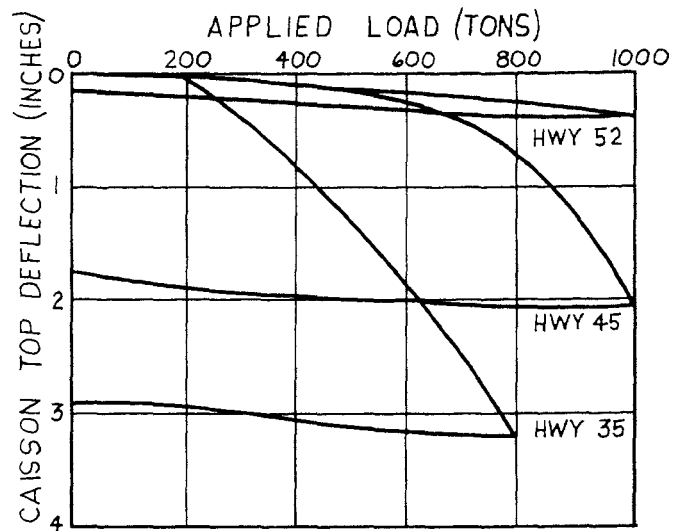


Fig. 2. Load-Deflection Curves.

Deflections include elastic compression of the caisson. The variation noted in the curves is partly due to variation in local subsurface conditions. In addition, Hwy 35 results are likely affected by a construction defect. Hwys 45 and 52 had very small settlements of 0.039 and 0.051 inches respectively at working load of 250 tons. Hwy 35 settlement of 0.246 inches was somewhat larger, but still much less than the one inch allowed (at working load).

Load versus depth data were obtained from the load readings at the three levels where the Mustran cells were installed. This is shown graphically in figure 3. In general, very little load transfer (friction) was developed in the upper layer as noted by the near vertical lines on the graphs. The bottom layer developed a very large amount of frictional capacity as indicated by the slopes of the lines in this layer. Very little end bearing was developed until a load of approximately 250 to 300 tons was applied. Thus, at design load of 250 tons, only a fraction of capacity was due to end bearing. The predominant working capacity of the caisson was developed as side friction. At higher capacities, 30-60% of the applied load was transferred to the tip.

Interpretation

Analytical techniques applied to the load-depth (load distribution) curves presented above provided load transfer data for the upper and lower layers (soil strata). For each applied load, the slope of the load distribution curve is the rate at which load is transferred to the soil. Since only two layers were monitored by the instrumentation, only two rates of load transfer were obtained for each applied load. The rates represent the average interval skin friction of the two layers. The extension of the lower load distribution curve to the bottom of the caisson was used to determine end bearing load. The average skin friction values and end bearing values were plotted versus the applied loads, producing load transfer curves

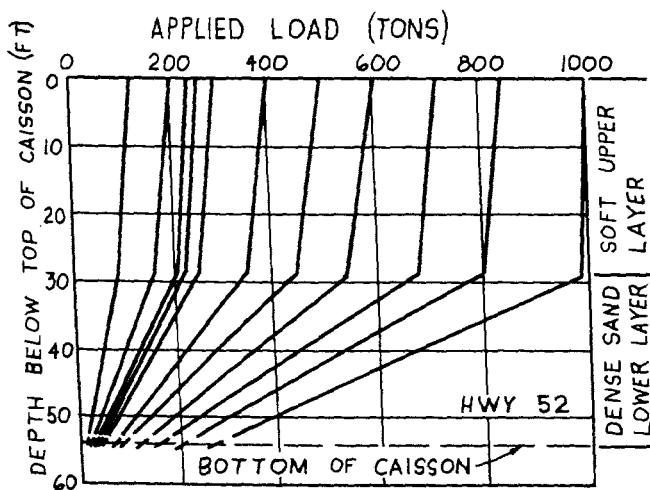
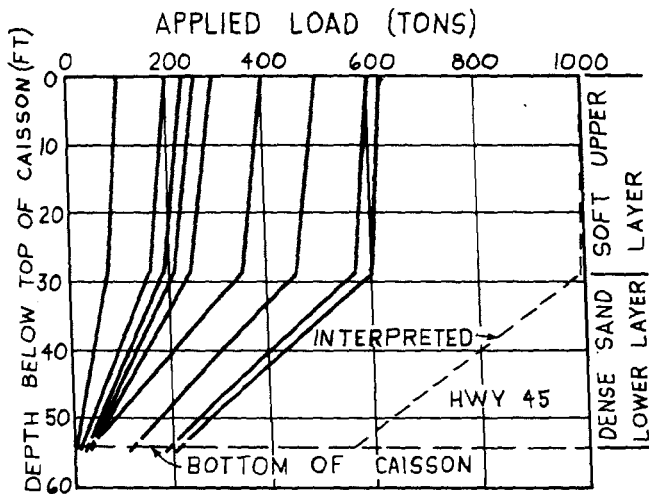
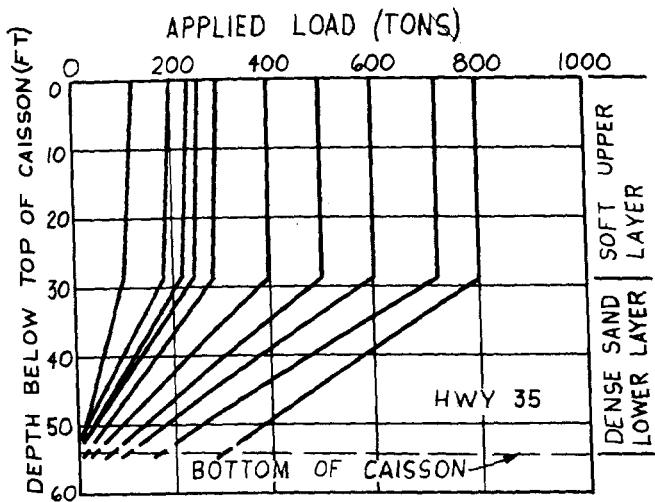


Fig. 3. Load-Depth Curves.

as shown in figures 4 and 5 respectively. expected, load transfer is quite small in the upper layers (less than 0.2 TSF) and very high for the lower layers (1.5 to 2.5 TSF). At 45 and especially 52, peak values for the upper layers were not obtained. At Hwy 35, it appears that the peak load transfer of 1.95 was obtained at an applied load of 750 tons. However, due to anomalies in cell readings, it is much more likely that the average interval skin friction peaked at about 500 tons applied load, being 1.55 TSF. This is further discussed later. As expected for sandy soils, bearing capacity increases indefinitely with the applied load.

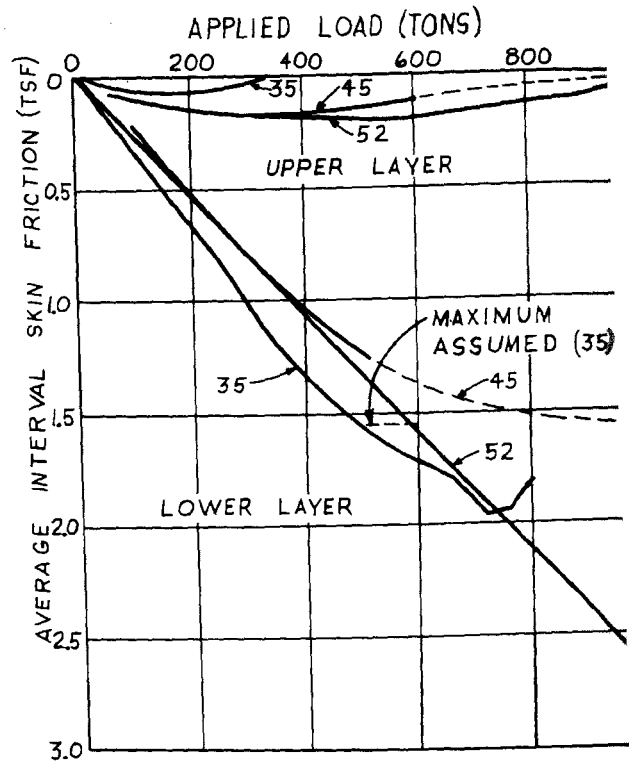


Fig. 4. Load-Skin Friction Curves.

It must be emphasized that the skin friction values discussed above represented an average throughout the layer between two sets of Mustrand cells. Within this layer, zones of higher and lower load transfer stresses no doubt occurred and ultimate values within a layer did not occur simultaneously.

At appropriate points along the caisson, the elastic shortening of the pier above that point was subtracted from the downward deflection at the top to obtain the net downward displacement of that point. From this, load transfer in side friction and end bearing was plotted against displacement and are shown graphically in figures 6, 7, and 8. Upper layer side friction load transfer curves peaked at very small displacements with much lower residual

strengths, typical of soft, cohesive materials. Lower layer friction curves indicated much greater load transfer and at greater displacements. End bearing curves showed a wide variation in tip capacities.

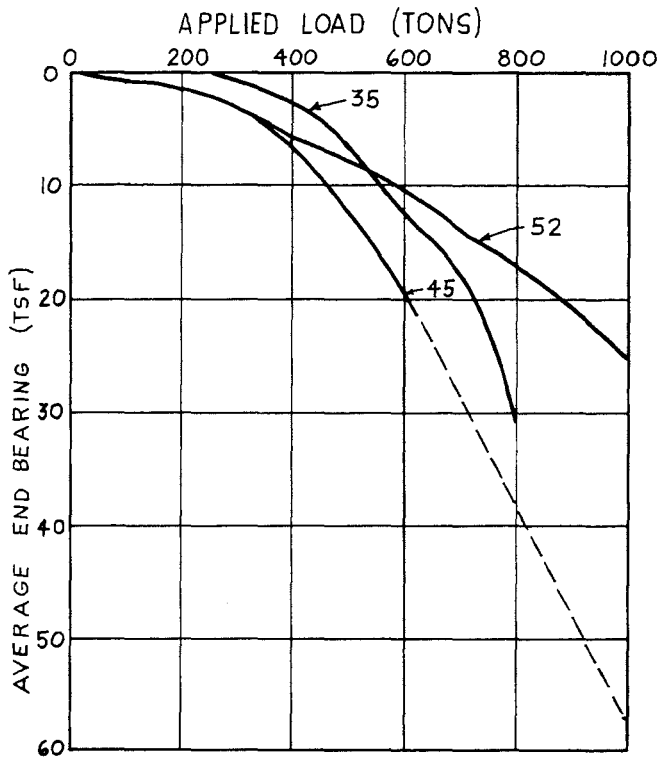


Fig 5. Load-End Bearing Curves.

Discussion

If the load distribution and load transfer curves are examined together, some general observations can be made.

Results from the Hwy 45 test probably best typified expected results (based on the literature and judgement), even though the top of the caisson cracked during the load test (at 640 tons). Upper cell readings required interpretation at higher loads. The interpretation is shown by dashed lines on figures 3 through 8. The average interval skin friction increased with increasing applied load to a peak stress of about 1.55 TSF in the lower layer. Net maximum displacement in the lower layer was about 1.8 inches, 4% of the caisson diameter. However, linear displacement ceased at about 0.2 inch, only one-half percent of the caisson diameter, at a load transfer of about 1.4 TSF. Average end bearing increased with applied load to 58 TSF at 1000 tons. This occurred at a tip deflection of 1.75 inches, a little less than 5% of the base diameter. The slope of the curve indicates additional base capacity could be developed at higher applied loads. At 1000 tons, over half the load was being distributed to the tip. SPT data in the lower layer averaged 68 bpf, and at the tip were 90 bpf.

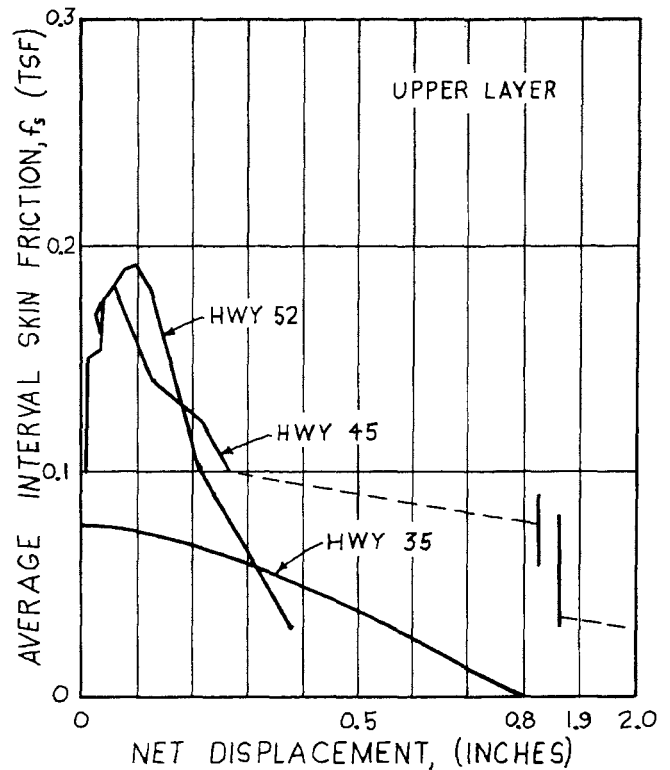


Fig 6. Skin Friction-Displacement Curves.

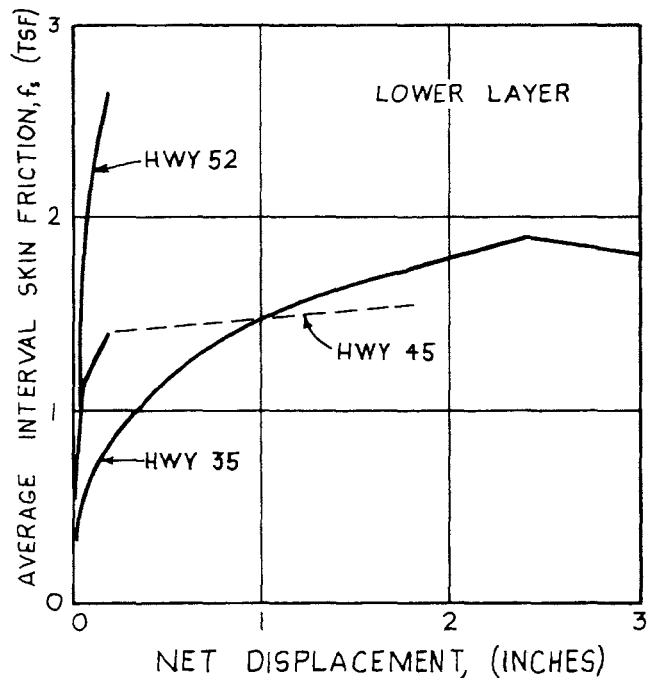


Fig 7. Skin Friction-Displacement Curves.

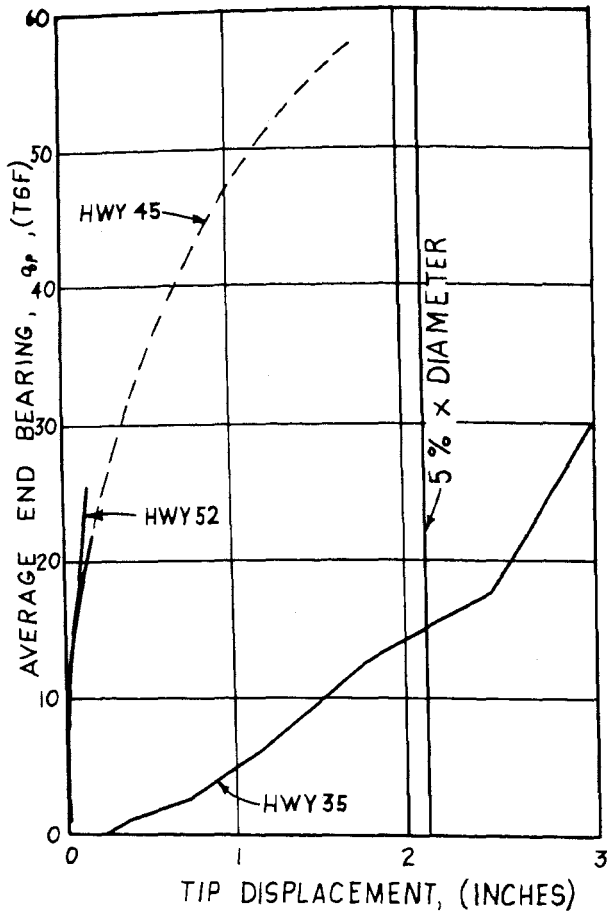


Fig. 8. End Bearing-Displacement Curves.

Hwy 35 load test was in somewhat less stiff (lower layer) soils, and possibly had a defective tip. This latter issue arose from observations during installation and a core boring made down the center of the caisson after the load test. The boring indicated some loose gravel and slurry above the excavated tip elevation. The average end bearing-displacement curve is reversed from the other curves, i.e., end bearing capacity is increasing with less displacements, rather than with greater displacements. This was expected for a poorly developed tip. However, at the maximum load of 800 tons, 40% of the applied load was carried by the tip, with a tip deflection of 3 inches (7% of base diameter). The lower layer skin friction developed a peak greater than at Hwy 45, but at a displacement of 2.5 inches (6% of caisson diameter). As mentioned previously, erratic cell readings beyond 500 tons challenges the data beyond this load. At 500 tons applied load, displacement was approximately 3% of the caisson diameter, and the maximum interval skin friction was assumed there (1.55 TSF). The ultimate end bearing was assumed to be that at a deflection of 5% base diameter (2.1 inches), resulting in 16 TSF. SPT data in the lower layer averaged 47 bpf, and at the tip were 55 bpf.

Hwy 52 skin friction in the lower layer was nearly linear up to the maximum value, which was much higher than expected. The maximum value recorded was 2.55 TSF. Both drillers observers independently recorded a hard rock layer about elevation 30 to 35. This would explain the behavior shown by the curve. T loads were less than half of those at Hwy 4. Only 32% of the applied load was transferred the tip at 1000 tons. Not only was the frictional load transfer high, it became hi at very low displacements (less than 0.2 in 1/2% caisson diameter), and showed no signs peaking. The end bearing-displacement curve was very similar to Hwy 45 up to its maximum except tip displacement was small at 1,000 applied load (0.1 inch, 0.2% base diameter). The ultimate end bearing capacity was likely greater than that developed in this test. However, it was not extrapolated, since the skin friction values were not reduced. SPT data in the lower layer averaged about 56 b and at the tip were about 90 bpf.

All of the bents except the end bents were have the top 25 feet of material removed. conventional theory, the lower confining stresses associated with this removal could result in lower capacities than those demonstrated by the load tests. Procedures for computing appropriate reduction factors are offered in the literature. However, it is writer's belief that because the subsurface soils are layered, preconsolidated, somewhat cemented, and have high SPT values (100+ bpf) no reduction is warranted.

Observed total settlements at all three caissons were acceptable for the specified design criteria (one inch at working load). Since some test load was transferred in the top 2 feet of the caisson, the settlement occurred at a load of 250 tons at elevation 50 (bottom of canal) would be a more appropriate indication of expected settlement. This was deduced from the load-depth and load-deflection curve and was found to be 0.321, 0.051, and 0.058 inches for Hwys 35, 45, and 52, respectively. Since conditions at the sites were relatively uniform, no settlement problems were anticipated from caissons designed to an ultimate capacity of 2.5 times the working load.

CAISSON DESIGN

Background

Ultimate limit state design procedures were considered appropriate. Where the test results did not furnish clear ultimate values of load transfer, the maximum values obtained, with some judgment, were considered ultimate. In the absence of more direct soils information SPT data were considered the index property soil at the sites.

SPT - Load Transfer Correlations

Available SPT data near each test caisson was assumed to represent the consistency of the material at the applicable test caisson. Test data were presented in the discussion of the

instrumented load tests. According to Meyerhof (1976), average ultimate interval skin friction and ultimate end bearing are approximately proportional to the average soil property expressed by SPT (blows per foot) for a given layer.

By comparing appropriate load transfer values with the SPT data, a proportional relationship was established at each test site between skin friction along the shaft and SPT along the shaft (in the lower layer), and between end bearing and SPT at the base of the shaft. The proportional relationships from the subject tests were as follows:

Table I

(Lower Layer)

Side Friction	Hwy 35	$f_s = N/30$	< 1.55 TSF	(1)
	Hwy 45	$f_s = N/44$	< 1.55 TSF	(2)
	Hwy 52	$f_s = N/22$	< 2.55 TSF	(3)
End Bearing	Hwy 35	$q_p = N/3.4$	< 16 TSF	(4)
	Hwy 45	$q_p = N/1.5$	< 60 TSF	(5)
	Hwy 52	$q_p = N/3.6$	< 25 TSF	(6)

where f_s is the ultimate developed skin friction along the side of the caisson in the dense sand layer, TSF

q_p is the ultimate developed end bearing at the base of the caisson in the dense sand layer, TSF

N is the SPT average in the layer or at the base, bfp

These relationships are shown graphically in figures 9 and 10. Also shown in these figures are the recommendations by Meyerhof (1976, 1983) and Reese et al. (1977) based upon their independent studies. Limit values were based upon the maximum load transfer values that occurred. As described previously, some adjustments to the maximum test values were required, based upon local conditions, construction anomalies, and judgment. Limit values for the upper layer are much lower. From fig. 9, the lower layer side friction proportional relationships and limit values obtained for Hwys 35 and 45 are reasonable and correspond very closely to Reese. The higher proportion and limit value for Hwy 52 were explained previously by the presence of a hard rock lense in the lower layer. The Meyerhof (1983) range of proportions ($N/50$), though somewhat conservativ conser, ie an improvement over his 1976 expression for bored piles of $N/100$. This improvement results from consideration of scale effects.

Fig. 10 indicates Hwy 45 end bearing correlates very closely to Reese, except a much higher limit value (60 TSF) is obtained. The Meyerhof (1983) relationship is only slightly more conservative. The reduction factor for large piles produced this proportion. The proportion

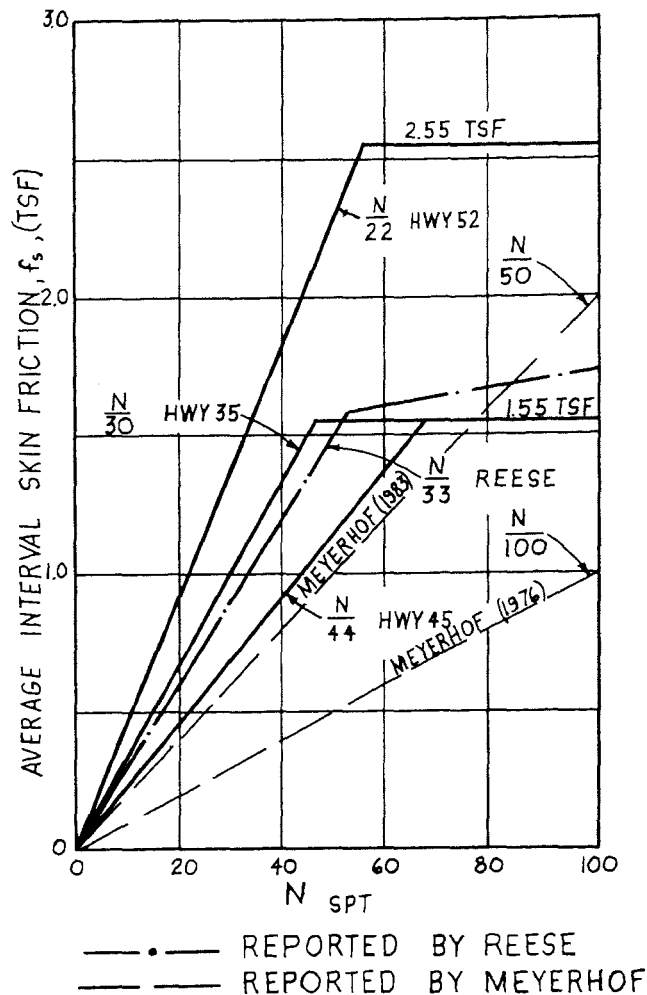


Fig. 9. Skin Friction vs. N .

and limit values for Hwys 35 and 52 were expectedly low and do not correlate well with Reese or Meyerhof. As previously explained, Hwy 35 developed low end bearing because the tip of the caisson was defective. Insufficient load was transmitted to the tip at Hwy 52 to develop high end bearing, due to high skin friction in a rock layer.

Design Procedure

At each proposed bent location, SPT data were available for design. From these SPT data, proportional relationships (equations 1 through 6) were used to determine unit skin friction and end bearing for production caissons at each bent location. The typical design procedure consisted of the following trial and error steps:

1. The minimum depth of 25 feet was assumed for the first trial. The average SPT blow counts were determined separately in the upper and lower layers down to the trial tip elevation for skin friction. Average SPT for a depth from the tip to several diameters below the tip for end bearing was obtained.

2. These averages were used to obtain the unit load transfer values from the appropriate proportional equations.

3. These unit values and the geometry of the caisson were combined in the conventional formula to develop the ultimate capacity:

$$Q_{ult} = q_p A_p + f_s A_s \quad (7)$$

Where A_p , A_s are the cross sectional area of the point and surface area of the side of the caisson, respectively

4. This capacity was then compared to the required ultimate capacity. If too small, then a greater caisson depth was assumed and the entire procedure was repeated until the computed ultimate capacity was obtained.

Note that each time the caisson depth is changed, the average SPT for the lower layer is changed. It is not appropriate to divide this lower layer into sub-layers and average each one. The SPT values would represent localized averages rather than a layer average.

Settlement was not considered for this design procedure per earlier discussion. Also, pier spacing was approximately 25 feet and bent spacing 65 to 70 feet, precluding group effect.

Design Example

An example of the procedure used to design the production caissons is shown in Table II. This design is for a hypothetical bent at Hwy 35. The proportional relationships used are

equations 1 and 4, and the ultimate capacity determined by equation 7. This bent is assumed to be in the slope of the canal, so top of ground is elevation 71. The design load is 150 tons; with F.S. = 2.5, required ultimate capacity is 590 tons.

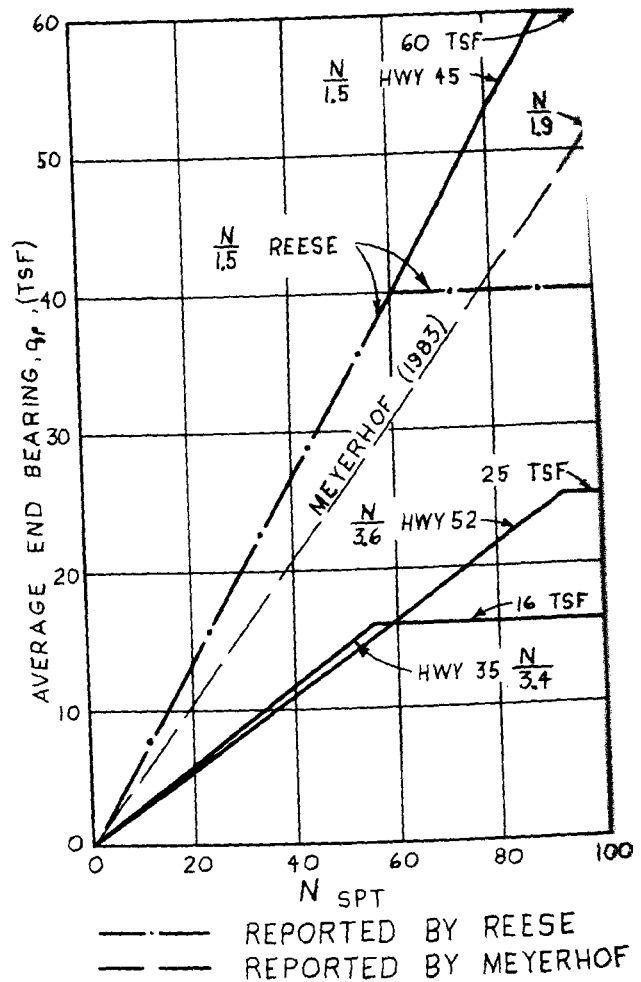


Fig. 10. End Bearing vs. N.

Table II

Interval Elevations (Feet)	SPT (Avg.) Side	f_s (TSF)	SIDE (Tons)	SPT (Avg.) End	q_p (TSF)	END (Tons)	Q_{ult} (Tons)	Accum. Q_{ult} (Tons)	Remarks
71-50	3	0.08*	18	80	16*	154	172	172	
50-25	48	1.55*	426	80	16*	154	580	752	Too large
50-27	46	1.53	387	80	16*	154	541	713	Too large
50-29	42	1.40	323	60	16*	154	477	649	Too large
50-30	37	1.23	271	50	14.7	141	412	584	O.K.

* Limiting Values

Caisson Tip Elevation 30 recommended.

CONSTRUCTION CONSIDERATIONS

Installing a caisson involves a complicated set of operations which must be closely coordinated and monitored to assure that certain measurable field parameters occur within allotted criteria. Some of the more noted parameters and criteria used on these projects are as follows:

Slurry. Density was maintained between 65 and 85 pounds per cubic foot. The minimum density was required to assure the slurry had enough weight to keep the hole from caving in and prevent ground water from entering the excavation. The maximum density limit prevented the slurry from becoming so heavy that it could not be displaced by the tremie concrete. The slurry was sampled immediately before introducing the tremie concrete.

Clean Out. The excavation was carefully cleaned out to remove all loose deposits at the bottom of the hole prior to concreting. This was particularly important since end bearing capacity was required. Clean out was accomplished with a special bucket having very shallow blades on the bottom. This bucket was rotated with slight downward pressure on the bottom of the hole to scoop up any soft material without advancing the hole any further. Several passes of this bucket were required until the bucket no longer obtained any additional material.

Concrete. Probably the most important property of the concrete was its slump. Six to 9 inches was specified for this project. A too high slump would interfere with the design properties of the concrete and invite segregation, but a too low slump would result in a concrete which would not properly flow around the reinforcing, and would not displace the slurry without leaving voids and honeycombing. Time between mixing and placing the concrete and the time between pours in the same caisson were critical. These times were 90 min. and 30 min. respectively for this project.

Tremie. The most important point was to avoid mixing the concrete with the slurry in the hole. The contractor was required to use either a flap valve or a go devil to prevent mixing at the start of tremie. This procedure was also required to restart the tremie if the pipe was inadvertently removed from the concrete while a pour was under way.

Tolerance. Though acceptable tolerances are recommended in the literature, of practical significance is the measurement of plumbness. The contractor used a bucket the same diameter as the hole and placed it on the bottom. The rod was then plumbed and the offset measured.

Surface Quality. Since the canal was to be excavated to a depth of about 25 feet after the installation of these caissons, inspection of the exterior surface of the top 25 feet of the caissons was readily made. In general, the surface quality of the caissons was good. Some bulging was noted near the surface which was presumably due to soft zones or caving of the hole before slurry was introduced. A fairly

neat cylindrical shape was the rule. In a few instances, grooves and pockets in the surface of the caisson were noted. See figure 11. These indicate insufficient displacement of the slurry by the concrete. A minor amount of reinforcing steel was exposed in a very few caissons. See figure 12. This occurred near the top and was probably due to the removal of the casing, and the fact that the pressure of the concrete near the top of the caisson was not sufficient to fully displace the slurry. This perhaps indicated a need for special



Fig. 11. Grooves in outside surface of caisson



Fig 12. Exposed reinforcing in caisson.

attention near the top of the caisson such as rodding. In one instance the caisson had a considerable amount of reinforcing exposed throughout its observed length (top 25 feet). The contractor indicated informally that an inexperienced crew installed this caisson. Though this could have been patched as the other minor problems were, it did not appear prudent to depend totally on this caisson having full capacity. For this reason excavation was made around the caisson to undisturbed dense sands and a footing was poured around dowel rods inserted through the caisson. Conservatism such as this is considered warranted due to the nature of caissons (high loads carried by single elements with no redundancy).

The centers of two test caissons were cored to determine if there were any voids or honeycombing in the concrete. Coring indicated that the caissons were of good quality throughout their depth, with the exception of the base of the caisson at Hwy 35 as previously discussed.

CONCLUSIONS

The advantages of instrumented load tests are well demonstrated at this project. Though the average end bearing and skin friction load transfer values obtained varied somewhat from those suggested by Reese (1977) and Meyerhof (1983), the procedures were valid. Site specific information is clearly preferable to the reported data for a more accurate design procedure.

SPT as an index property seemed to work reasonably well. For best correlation, an additional SPT boring should have been taken immediately adjacent to each test caisson. A small change in SPT can significantly affect the proportional design relationship. It is not anticipated this procedure would be appropriate for sites whose subsurface conditions were not reasonably consistent. However, more varied conditions can be compensated somewhat by more intervals (layers) for which load transfer information is obtained. The skin friction averages would then apply to better-defined, thinner layers representing more different types of materials. For example, at Hwy 52, additional Mustran cells placed to isolate the rock layer would have provided more applicable load transfer values at that site. Also, in dense materials, even higher test loads are required to develop ultimate (limit) values. Ultimate end bearing would probably have been developed at tip displacements of 10% or more of base diameter. No general relationship was found for skin friction-displacement in the lower (dense) layer, except it was much smaller than for end bearing (probably 0.5 to 3% of caisson diameter). In the upper (soft) layer, ultimate skin friction occurred at a displacement of about 0.1% of caisson diameter.

Some concern is felt for the occasional construction anomaly that produces the inferior caisson such as the one discussed earlier. Construction techniques based upon sound experience are required, on every caisson.

ACKNOWLEDGMENTS

The caissons were constructed and load tests under the direction of Mr. J. Steven Farr, foundation drilling contractor. Load tests were recorded, reduced, analysed, and reported by Mr. Robert Smith of Law Engineering Test Company. Mr. Lymon C. Reese, University of Texas at Austin, provided advice and assistance in the design and specification of these caissons. Their contributions to this project are greatly appreciated.

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