EVALUATION OF AND DESIGN CONSIDERATIONS FOR DRILLED SHAFTS SOCKETED INTO CORAL AND COQUINA LIMESTONES

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This thesis is dedicated to the two people to whom the author is most grateful, his parents, Michael and Ruth Semeraro, whose constant support and encouragement never fade.

ABSTRACT

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The load carrying capabilities and parameters associated with seven rock socketed drilled shafts located at four different South Florida sites are evaluated and presented. The founding rocks are coral and coquina limestones. These limestones are very porous and in places riddled with solution channels. Both side friction and end bearing evaluations are performed and applicable factors of safety for each are discussed.

Present load test configurations are also discussed, and suggested modifications are made for load tests in the South Florida area.

Thesis Supervisor: Dr. W. Allen Marr

Title: Research Associate

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LIST OF PRINCIPAL NOTATION

A	-	Area
E_c	-	Modulus of concrete
E_{eq}	-	Equivalent shaft modulus
E_i	-	Modulus of the intact rock
E_m	-	Modulus of in-situ rock mass
E_p E_{pile}	-	Modulus of shaft
E_R E_{rock}	-	In-situ modulus of rock
E_s	-	Modulus of steel
F	-	Force exerted on the top of the rock socket
F_{base}	-	Force transmitted to the shafts base
f_{su}	-	Maximum Frictional load transfer
j	-	Mass factor = E_m / E_i
K	-	Ratio of shaft modulus to rock modulus
P_t ; P_b	-	Load at top of rock sockets; load at base of socket
ΔQ	-	Change in load between two stated depths
q_u	-	Unconfined compression strength of the intact rock
% REC	-	Percentage of rock core recovered
RQD	-	Rock quality designation, percentage of recovered rock core in excess of 4 inches.
α	-	Side resistance reduction factor - f_{su} / q_u
β	-	Side resistance reduction factor - q_u/f_{su}
ρ ; $\Delta\rho$	-	Settlement; change in settlement

1. SUMMARY

Seven rock socketed drilled shaft axial load tests were evaluated. The load tests were performed on four different South Florida sites as shown in Figure 3-1. The founding rock varied from coral to coquina limestones and had RQD values ranging from 30 to 100. The load tests have shown the applicability of high capacity deep drilled shafts in the South Florida Area.

The tests as analysed produced load transfer curves for side resistance and base resistance. The base resistance curves show that base resistance in the Florida Limestones can carry large loads. Site L has shown that large quantities of stress, (over 80 tsf), can be elastically transferred through end resistance with tip strain being less than 0.9 percent. This implys that future drilled shafts in the South Florida area can be designed to resist some portion of their load in end bearing. Present practice in South Florida uses frictional resistance only; thus these results allow engineers to design higher capacity more economical shafts.

Side resistance reduction values, α and β, which correlate the peak mobilized side resistance to the unconfined compression strength of the intact rock and RQD values were estimated for the field tests, (Figure 5.6-1). These six values (three of which are lower bounds) are shown with existing data in Figure 5.6-2 and can be used in preliminary design studies. Peak side resistance values in the three shafts which were loaded to failure occurred with side movements between 0.05 and 0.25 inches. No residual side resistance values were observed with increased movements. Percentages of unconfined compressive strength to peak side resistance, ($\alpha * \beta$), varied from 1.2% for a RQD of 35 to 19.6% for a RQD approximated as 100. Two load tests which did not fail are shown to suggest percentages a multiple of those given above.

Instrumentation for the tests evaluated consisted of telltales and load cells. Load cells, particularly the mustran cell from the University of Texas at Austin,

along with a tip telltale are suggested instrumentation for future frictional load tests. Future frictional load tests are also suggested to employ socket lengths of 3 to 4 times the socket diameter, to employ a method to prevent end bearing, and to load the shaft to failure. End bearing tests are suggested to be cased to the base elevation being tested.

A minimum factor of safety of three is suggested to be used for side resistance values obtained through load tests due to the variability of the limerock in South Florida. For end bearing the factor of safety is suggested to exceed 3 and should be decided on a site specific basis considering the variability of the site limerock.

2. INTRODUCTION

There exists two major types of support for use in deep foundations. They are piles and drilled shafts. Piles displace soil by being driven into place. They are typically driven by hammering, although they can also be driven by vibration or jetting. Drilled shafts are non-displacement elements, in which a hole is augered, reinforcement steel is placed and the hole filled with concrete.

Due to the accelerated development of South Florida, a demand for tall buildings and bridges has developed. Because of the demand for taller and heavier structures, compounded with 120 mph hurricane wind loadings, a need for reliable high capacity foundation elements appeared. To fulfill this need, drilled shafts which have worked well in other parts of the Country were first tested in South Florida during 1977. Although additional load tests have been performed, at other sites since 1977, these data have not been published and are not readily available to the public.

Drilled shafts are now being specified for many sites where heavy loads are present. Individual piles in the South Florida area are designed for capacities of 50 to 100 tons, whereas drilled shafts in the same area are now being designed to carry vertical loads in excess of 800 tons.

Present practice in South Florida for the use of drilled shafts considers side friction only. End Bearing in the South Florida area for drilled shafts is normally neglected thus leading to a conservative design. The major reasons end bearing is not presently included in the drilled shaft design include: a) the desire to keep the design conservative since this type of foundation is new to this area, b) the inability to check the bearing strata because it is not practical to dewater the shafts, and c) the inability to determine the extent and location of sand vugs in some of the limestone strata (Fort Thompson Formation in particular). Through this analysis, the applicability of employing end bearing into the drilled shaft design will be shown and discussed.

Specific parameters will be computed and evaluated to relate the frictional load carrying capabilities of the drilled shafts to the physical properties of the founding limerocks. During this evaluation, any field procedure used in drilled shaft construction which makes the evaluation of the test results more difficult or less reliable will be noted and discussed. Recommendations with respect to construction procedures used and factors of safety for prodution shafts will be suggested.

This thesis incorporates load tests from 4 South Florida sites where full-scale drilled shaft load tests have been performed. The four sites to be evaluated, (noted as Sites L, SE, S, and I) are located on the geologic map in Figure 3-1. The load tests employ rock socketed shafts. Maximum loads on these shafts range from 1000 to 1260 tons. Site L in which 3 load tests are evaluated, and the load test of site SE, are founded in the Fort Thompson Formation (Miami, Florida). The site S load test is located in Palm Beach County's Anastasia formation. The fourth site is located in the Florida Keys, founded in the Key Largo limestone and contains two load tests.

The remainder of this thesis is outlined as follows:

Chapter 3 describes the geologic conditions of the formations mentioned above. The theory and available evaluation procedures for drilled shaft load tests are reviewed in Chapter 4. A typical soil profile for each load test is given along with the shaft geometry, load test data, and evaluation in Chapter 5. Chapter 6 summarizes suggested factors of safety and Chapter 7 mentions concerns about group interaction of drilled shafts. For the interested reader, load test instrumentation, load test variations, effects on rock parameters derived from laboratory tests, and drilled shaft construction are reviewed in the Appendicies.

3. GEOLOGIC CONDITIONS

Rocks which are composed of magnesium and calcium carbonates are collectively called limestones in engineering practice. Limestone ingredients can include shell fragments, coral lime muds, sand size quartz, and plant and animal remains which can vary from sand to silt-sized. Coquina is a term commonly used for limestone composed of cemented quartz sand and shell fragments.

The Florida limestones are soluable in acidic water and past solution activity has occurred in some areas. The limestone solution stops once the water is saturated with dissolved carbonates (Ph of water becomes neutral or basic). The voids left in the rock after stabilization occurs typically become infilled with uncemented shell fragments, lime sands and muds. This past solutioning is readily found in the form of sand vugs in the Fort Thompson Formation. Figure 3-1 shows a geologic map of South Florida. In Figure 3-1, the major limerock formations are denoted and the approximate locations of the 4 test sites evaluated in this report are shown. The formations in which the load tests are founded are: The Anastasia Formation, The Fort Thompson Formation, and the Key Largo Formation. Typical Profiles for each test site are given in Chapter 5. A general description of the formations encountered are below.

Anastasia Formation

This formation has a maximum thickness of 60 feet occurring near the coast line. It is a wedge shaped body which abruptly terminates in the easterly direction and tapered westerly. The Anastasia formation is a white, gray, or tan colored coquina (shell fragment limestone). The formation is a good aquifer and highly permeable.

Fort Thompson Formation

The Fort Thompson Formation is a complex wedge-shaped formation with a variable thickness. The thickness in the Miami area is approximately 60

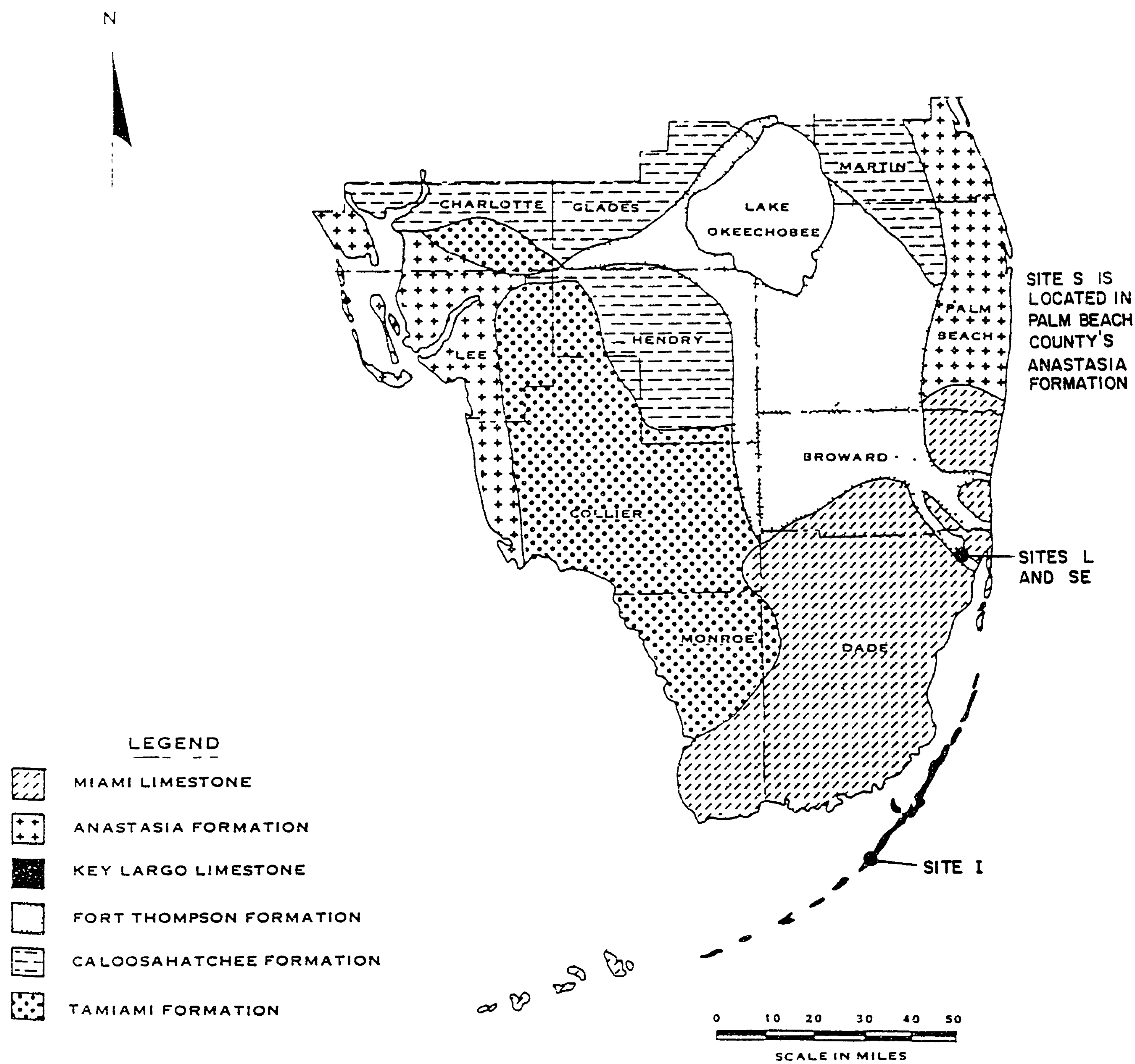


FIGURE 3-1 GEOLOGIC MAP OF SOUTH FLORIDA AND APPROXIMATE TEST SITE LOCATIONS
(AFTER LAW 1978, ORIGIONALLY ADAPTED FROM PURI AND VERNON 1964)

feet. The layers found in the Fort Thompson formation (which are not all continuous), consist of freshwater limestone, sandy limestone, quartz sandstone, quartz sand and coralline limestone.

The freshwater limestone is not continuous throughout the formation and exists in thicknesses varying from 6 feet to being non-existant. It is a dark brown to brown moderately hard, very fine grained freshwater limestone containing some small gastropod fossils, (Law, 1978).

Sandy limestone is the major component of the Fort Thompson formation with continuous moderately hard layers extending to 20 feet in thickness. This tan fossiliferous sandy limestone also contains friable uncemented zones. In places this limestone is known to contain quartz sand vugs resulting from past erosion.

Calcarious quartz sandstone is interspersed with the sandy limestone described above. This calcerious sandstone as noted by Kaderabek (1981) and Law (1978), has not been found, or is noted as a limestone or limerock by others, (Gupton et al 1982 and O'Brien and Logan 1981). To distinguish between Quartz Sandstone and Sandy Limestone as found in Florida, is very difficult. In this analysis with the exception of profiles taken from published references the sandy limestone and calcerious sandstone are referred to as limerock.

Coralline limestone in discontinuous layers of two to five feet in thickness can also be observed in parts of the Fort Thompson formation. This coralline limestone is composed of intact coral skeletal remains, primarily Montastrea sp and Diploria sp (Law 1978).

A fine grained quartz sand layer, very variable in thickness is usually encountered in thicknesses of 5 to 25 feet. This sand, light gray to brown in color is unfossiliferous.

The Miami limestone formation, which is a soft to medium white to light tan oolitic packstone is found to overlie the Fort Thompson formation as shown on the site specific profiles, (Figures 5.2-2, 5.2-4, 5.2-6, and 5.4-1). This oolite is at places sandy and fossiliferous. It has been completely removed through solutioning in some areas.

Key Largo Limestone

This formation, attaining a maximum thickness of about 60 feet is composed of coralline limestone containing large intact coralline forms.

A typical profile for each site is shown adjacent to the particular test shaft in Section 5. For a more detailed geological review of coral and coquina limestones, an interested reader is referred to Sowers (1975), Parsons (1971), and Gupton et al (1982). It should be noted that although these rocks classify as limestones, they are extrememly porous at places riddled with solution channels, and soluable in nature.

4. AVAILABLE PROCEDURES TO EVALUATE LOAD TESTS

4.1 General

Methods to evaluate load tests vary from employing a finite element program to model the test shaft behavior, to evaluating the load transfer of the shaft through the use of elastic formulas and load cell or telltale data. Finite element models are used to evaluate and predict pier behavior, some of the more recent being Bauer (1980), Donald et. al. (1980), Gill (1980, 1970), Pells and Turner (1980), Poulos and Davis (1981), and Rowe and Pells (1980). Prior to the extensive use of finite element models and still popular today, are various numerical and elastic model approximations. Methods that employ the theory of elasticity have the major limitations of assuming that the modulus of the rock equals that of the concrete, and that the system is totally elastic. Finite element analysis overcomes these problems by allowing; a) a change of any soil parameter in space, b) incorporating a Mohr-Coulomb failure criterion or any other plastic or elastic response, and c) by solving the continuity equations (within a given tolerance), with various boundary conditions. Many researchers have found that elastic finite element solutions, although approximate, typically give values within 5% of the exact value evaluated through complex differential equations. Through use of finite element models one could test many variations of concrete and rock parameters as well as modeling layered systems, thereby obtaining approximate elastic parameters for rock systems whose differential equations would otherwise be too complicated to solve by hand. Typical finite element elastic curves useful in the analysis and design of drilled shafts are shown in the above sited references, as well as in Koutsoftas (1981), Pells and Turner (1979), and Poulos and Davis (1974).

Solutions from finite element analyses are employed in predicting expected behavior of a test shaft. The elastic analysis by Pells and Turner (1979) as shown on Figure 4.1-1, developed charts which aid in separating end bearing resistance from the side adhesion of a drilled shaft.

When using elastic analysis one must remember that side friction and ultimate end bearing are not elastically compatible. Shaft movement necessary to develop peak side adhesion is much less than that for end bearing. Thus, when utilizing both end bearing and side adhesion a combination side resistance end bearing influence curve should be employed.

4.2 Data Evaluation

The common analysis of test shafts result in empirical factors used in the design of drilled shafts for that specific site. Common load test data consist of telltale values or load cell readings with calibration curves, refer to Figure 4.2-1, and Appendix 4. The use of Mustran cells, (or similar load cells), give direct load distribution plots for the shafts as shown in Figure 6.1-4. When telltales are utilized, the telltale data, (load settlement curves) must be transformed into load distribution plots. The theory of elastic deformation is used to transform the load settlement curves into load distribution curves and should be done for several top load increments. Elastic deformation equations for the various load distributions encountered during this transformation are derived in Figure 4.2-2.

To calculate the load distribution curve one must start from a point of known load. If there is no end bearing, (or if the end bearing is known), one could start from the tip of the shaft where the load is zero, (or the known value). Alternately one may start the load distribution derivation from the top of the shaft where the known load is the applied load. Through this analysis, as shown in Figure 4.2-3, one constructs the load distribution curve by repetition of this method from one end of the shaft to the other, If end bearing is present, by evaluating the load distribution curve from the top of the shaft, one could also develop an end bearing tip settlement curve.

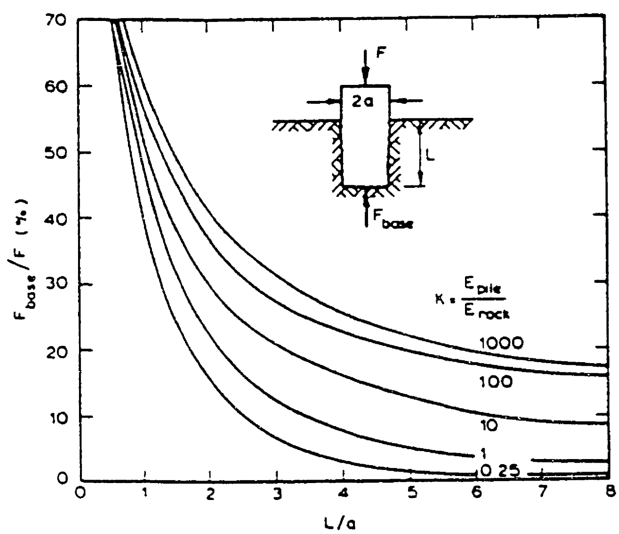
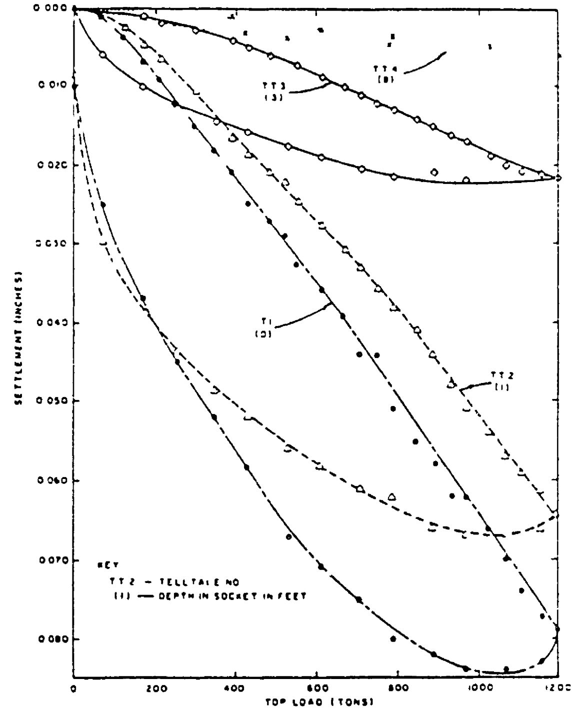
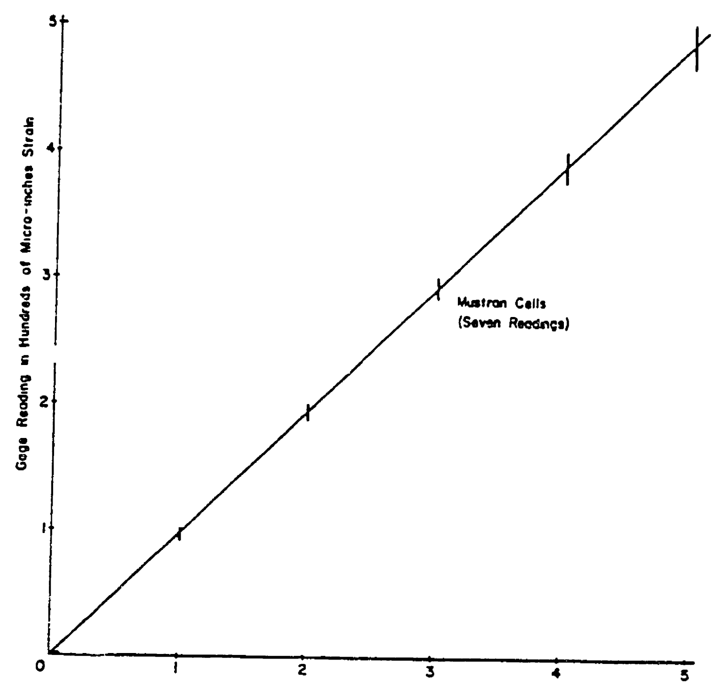


FIGURE 4.1-1 PERCENTAGE OF TOP LOAD TRANSFERED
TO BASE OF ROCK SOCKET
(AFTER PELLS AND TURNER 1979)



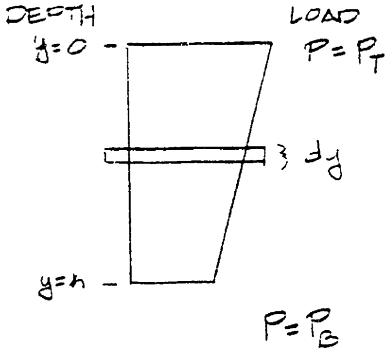
a) TELLTALE MOVEMENT vs TOP LOAD
(AFTER GUPTON et. al. 1981)



b) CALIBRATION CURVE FOR MUSTRAN CELL
(AFTER BARKER AND REESE 1969)

FIGURE 4.2-1 a) TELLTALE DATA
b) CALIBRATION CURVE FOR MUSTRAN CELL
DATA REDUCTION

USING A TRAPEZOIDAL LOAD DISTRIBUTION



WHERE,

y = DEPTH

L = LENGTH BETWEEN KNOWN LOAD POINTS

P = LOAD

P_T = LOAD OF TOP LOAD POINT

P_B = LOAD OF BOTTOM LOAD POINT

dy = CHANGE IN DEPTH

FROM ELASTICITY WE KNOW:

$$\rho = \frac{PL}{AE}$$

WHERE:

P, L = DEFINED ABOVE

A = CROSS SECTIONAL AREA

E = AVERAGE MODULUS OF ELASTICITY

ρ = ELASTIC DEFORMATION

GIVEN: TRAPEZOIDAL LOAD DISTRIBUTION

SOLVE FOR: ELASTIC DEFORMATION

$$\rho = \frac{PL}{AE} = \frac{1}{EA}\int P\,dy$$

@ $y=0$ $P=P_T$ AND @ $y=h$ $P=P_B$

$$\therefore P = P_T - (P_T - P_B)\frac{y}{L}$$

$$\rho = \frac{1}{EA}\int (P_T - (P_T - P_B)\,dy$$

$$\rho = \frac{1}{EA}\left(\int P_T\,dy - \int (P_T - P_B)\frac{y}{L}\,dy\right.$$

$$\rho = \frac{P_T}{EA}\int_L^0 dy - \frac{P_T - P_B}{EAL}\int_L^0 y\,dy$$

$$\rho = \frac{(P_T - P_B)L^2}{2EAL} - \frac{P_T L}{EA}$$

$$\rho = \frac{(P_T - P_B)L}{2EA} - \frac{P_T L}{EA}$$

$$\rho = \frac{L}{2EA}(P_T - 2P_T - P_B);\quad \rho = \frac{L(-P_B - P_T)}{2EA},\quad P_B\ \frac{2EA\Delta\rho}{L} - P_T$$

NOTE: CHANGED SIGN SO COMPRESSION IS POSITIVE SETTLEMENT AS IS USED IN PRACTICE

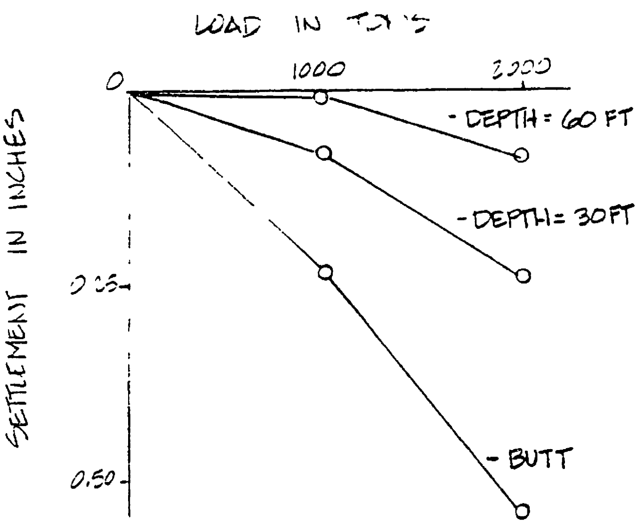
FOR A RECTANGULAR LOAD DISTRIBUTION, $P_B = P_T$

$$P = \frac{EA\Delta\rho}{L}$$

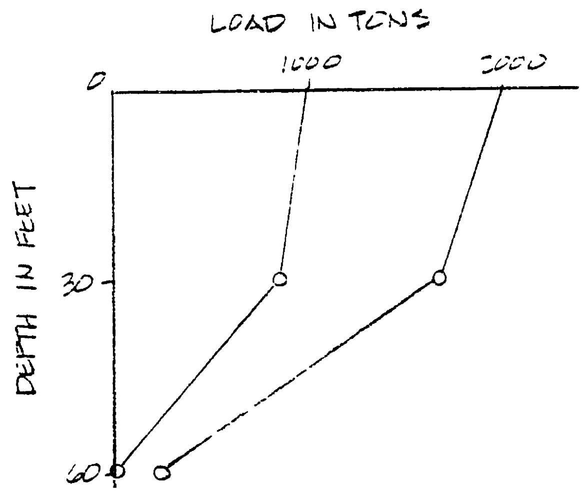
FOR A TRIANGULAR LOAD DISTRIBUTION, $P_B = 0$

$$P_T = \frac{2EA\Delta\rho}{L}$$

FIGURE 4.2-2 DERIVATION OF ELASTIC DEFORMATION FORMULA FOR VARIOUS LOAD DISTRIBUTIONS



TELLTALE SETTLEMENT CURVE
0-30 FT. SOIL 30-60 FT ROCK



DERIVED LOAD DISTRIBUTION CURVE

AREA OF SHAFT = 7.07 FT^2

E_{AVE} = 3.024 × 10^5 TSF

$$P_B = \frac{2EA\Delta\rho}{L} - P_T$$

DERIVING LOAD DISTRIBUTION CURVE FROM SHAFT BUTT

LOAD TONS	DEPTH FT.	LENGTH IN.	TOP ρ IN	BOTTOM ρ IN	CHANGE IN ρ IN.	TOP LOAD TONS	BOTTOM LOAD TONS
2000	0-30	360	0.550	0.240	0.310	2000	1682
2000	30-60	360	0.240	0.080	0.160	1682	218

DERIVING LOAD DISTRIBUTION CURVE FROM SHAFT TIP

LOAD TONS	DEPTH FT.	LENGTH IN.	TOP ρ IN	BOTTOM ρ IN.	CHANGE IN ρ IN	BOTTOM LOAD TONS	TOP LOAD TONS
1000	60-30	360	0.080	0.010	0.070	0	831
1000	30-0	360	0.234	0.080	0.154	831	998 ~ 1000

NOTE: THIS DATA IS
FOR ILLUSTRATIVE PURPOSES ONLY

FIGURE 4.2-3 EVALUATING THE LOAD DISTRIBUTION CURVE

4.3 Evaluation of Frictional Resistance

Once the load distribution curves are calculated from telltale data, or interpolated from Mustran cell readings, load transfer curves are drawn. A load transfer curve is a plot for a specific depth, of load transfer (TSF) with respect to side movement (inches). The socket side movement can be taken directly from telltale measurements, or from computing elastic deformation between the butt, (or another known settlement point), to the depth being evaluated. This elastic deformation is subtracted from the known point of settlement to give the side movement. This process should be done for each load transfer point. Load transfer is the change in load per area of the shaft and is equivalent to the slope of the load distribution curve divided by the shaft perimeter at that depth. The load transfer value could be integrated manually by taking a representative section of tangent from the load distribution curve at the depth in question for each load distribution curve. Barker and Reese (1969), found that fitting a 4th or 5th order curve to the load distribution data then analytically calculating the load transfer worked well. The process of deriving the load transfer curve through manual integration is shown in Figure 4.3-1. Once an average side resistance value is determined applicable for a particular site from load tests, the approximate design side friction for the rock strata is determined (employing the desired factor of safety). Production shafts for the site are then sized. A non-numerical schematic of this method is shown in Figure 4.3-2.

Peak frictional side resistance is less than the unconfined compressive strength of the rock. This is due to the different loading mechanisms and rock mass properties varying from those of an intact core sample. Because of these variations two different side resistance reduction factors are introduced. An α factor has been used by many authors, Williams (1980), Matich and Kozicki (1967), Wilson (1976) and Williams and Pells (1980), to correlate the unconfined

$$\delta_e = \frac{(P_T + P_B)L}{2EA}$$

$E_{EQ} = 3.024 \times 10^5$ TSF

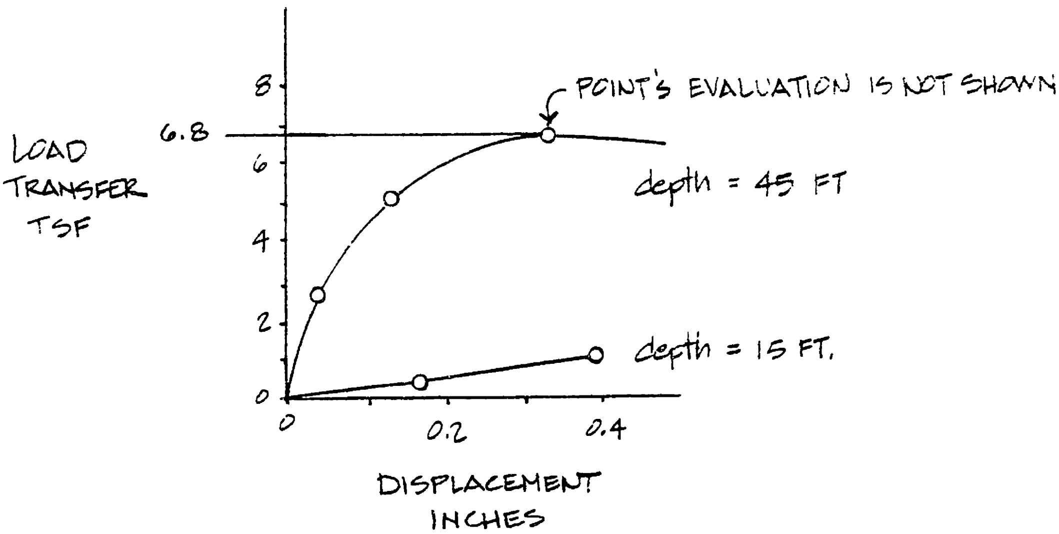
$A = 7.07\ FT^2$

PERIMETER = $\pi D = 9.4$ FT

$\therefore$ LOAD TRANSFER $= \dfrac{\Delta Q}{9.4 \Delta x}$

CALCULATING THE LOAD TRANSFER CURVE

DEPTH FT.	TOP LOAD TONS	ΔQ TONS	Δx FT	TSF	REFERENCE SETTLEMENT FROM	REFERENCE SETTLEMENT INCHES	ELASTIC SETTLEMENT INCHES	DISPLACEMENT INCHES
15	1000	169	30	0.60	BUTT	0.234	0.081	0.153
15	2000	318	30	1.13	BUTT	0.550	0.162	0.388
45	1000	831	30	2.95	30 FT	0.080	0.052	0.028
45	2000	1464	30	5.19	30 FT	0.240	0.111	0.129



LOAD TRANSFER CURVE

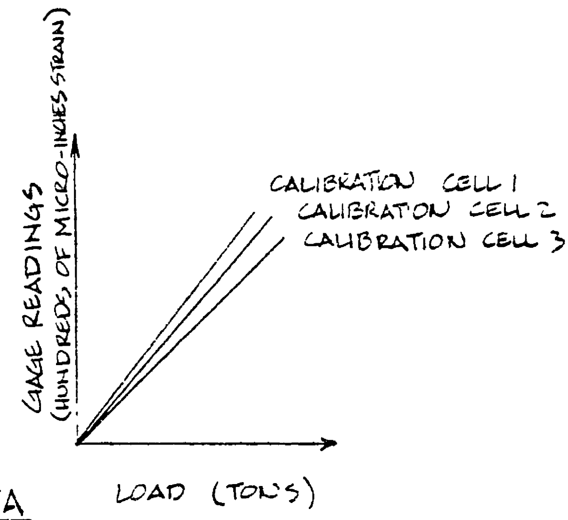
MAX. LOAD TRANSFER = f_{su} FOR DEPTH = 45 FT. $\bar{f}_{su} = 6.8$ TSF

SIDE RESISTANCE REDUCTION FACTOR = $\alpha = f_{su}/q_u$ USING $q_u = 20.9$ TSF, $\alpha = 0.32$

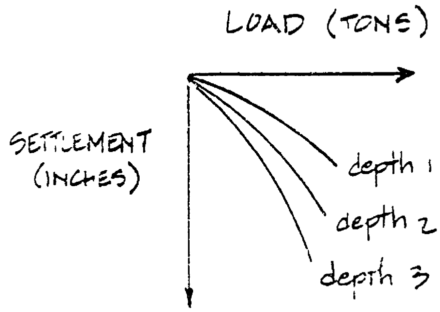
NOTE: THIS DATA IS FOR ILLUSTRATION PURPOSES ONLY.

FIGURE 4.3-1 EVALUATING THE LOAD TRANSFER CURVE

TOP LOAD TONS	CELL 1 MICRO-INCHES STRAIN	CELL 2 MICRO-INCHES STRAIN	CELL 3 MICRO-INCHES STRAIN
100	100	163	208
200	150	215	267
300	173	232	276
400	208	256	312
500	268	322	375

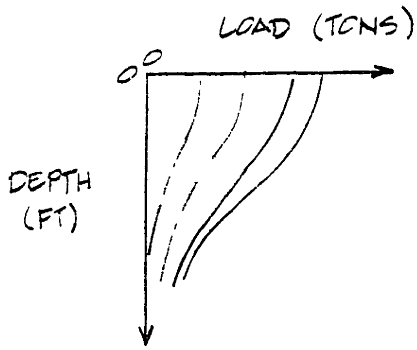


STRAIN CELL DATA

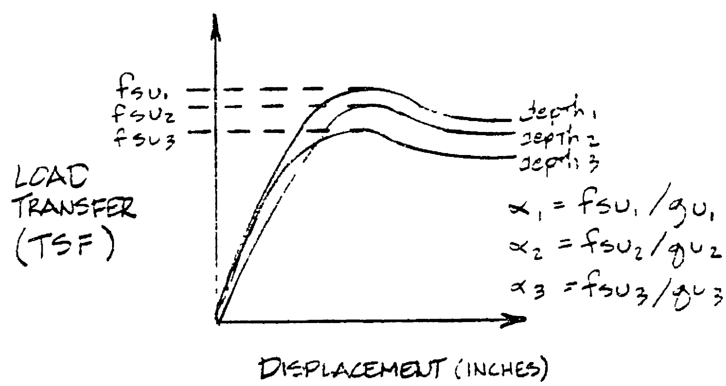


ELASTIC DEFORMATION EQUATIONS (SEE FIGURE 4.2-2)

a) TELLTALE DATA



b) LOAD DISTRIBUTION CURVE



c) LOAD TRANSFER CURVE

d) DESIGN OF SHAFT:

LOAD = L

DIAMETER = D, PERIMETER = P

FACTOR OF SAFETY = 3

$f_{s_1} = \frac{1}{3}\alpha_1 q_{u_1}$, $f_{s_2} = \frac{1}{3}\alpha_2 q_{u_2}$, $f_{s_3} = \frac{1}{3}\alpha_3 q_{u_3}$

THICKNESS OF ADHESION LAYERS; ℓ_1 = Thickness f_{s_1} Applies $\ell_2 \to f_{s_2}$ $\ell_3 \to f_{s_3}$

$\therefore$ LENGTH = MAX$\left[\min.\left(L/(P \cdot f_{s_1}), \ell_1\right), \mathrm{MIN}\left[\left(\ell_1 + (L - \ell_1 f_{s_1} P)/(P f_{s_2})\right), \ell_1 + \ell_2\right], \left(\ell_1 + \ell_2 + (L - \ell_1 f_{s_1} P - \ell_2 f_{s_2} P)/P f_{s_3}\right)\right]$

IF LENGTH $> \ell_1 + \ell_2 + \ell_3$ THEN INCREASE DIAMETER AND REPEAT IF $L - \ell_1 f_{s_1} P - \ell_2 f_{s_2} P < 0$ THEN DISREGARD LAST EXPRESSION IN ABOVE EQUATION

FIGURE 4.3-2 TYPICAL PROCESS OF EMPERICAL DESIGN compressive strength of the intact rock cores to the peak mobilized side resistance. α is equal to the peak side resistance divided by the unconfined compressive strength of the rock. This correlation as shown in Figure 4.3-3 represents the variations of the intact strength of the sample tested. The α value is only representative of the entire rock continuum when the % recovery and RQD from field borings = 100%. Weak rocks, as found in Florida's limerock, can have RQD values much lower than 100%.

The second side resistance reduction factor incorporates the variations between the rock mass and the tested intact rock sample. Williams (1980) has argued that this additional side resistance can be correlated to the rock modulus mass factor j . The mass factor is equal to the rock mass modulus divided by the intact rock modulus. The mass modulus can be obtained from pressure meter results (Schmertman 1970) or from RQD values (Deere et al 1967). Intact modulus can be obtained through testing intact rock cores. Williams (1980) has called this second side resistance reduction Factor β . β is equal to the field peak side resistance divided by the product of the side resistance reduction factor α, and the unconfined compressive strength of the intact rock. At mass Factor $j = 1.0$, $\beta = 1.0$ by definition. The β values determined for Melbourne Mudstone are shown in figure 4.3-4. The β values shown should not be directly applied to coral limerock, but through knowing the peak side resistance, unconfined compression strength and assuming Figure 4.3-3 as an estimate of the appropriate α , β can be backsolved for the Limerock through load test results and compared to the published values. The above analysis using α factors has been successfully employed for individual sites in South Florida, Nyman (1980), and on many other sites throughout the world.

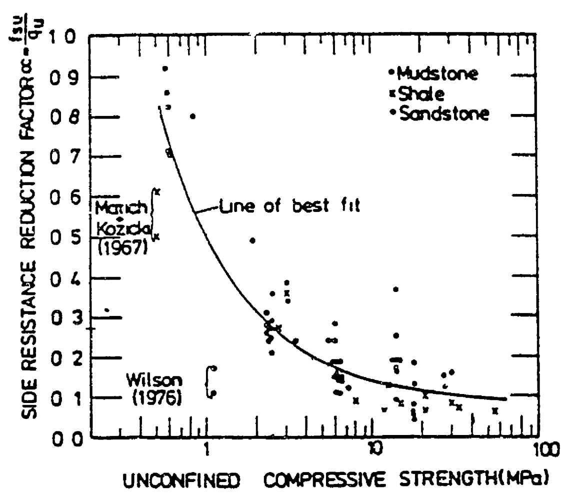


FIGURE 4.3-3 SIDE RESISTANCE REDUCTION FACTOR α
(AFTER WILLIAMS AND PELLS 1981)
1 MPa=10.44 TSF

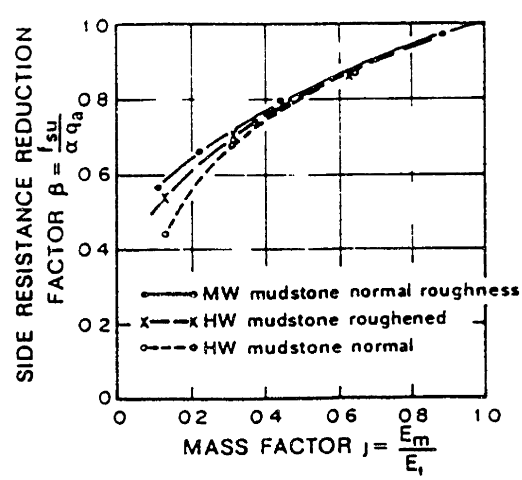


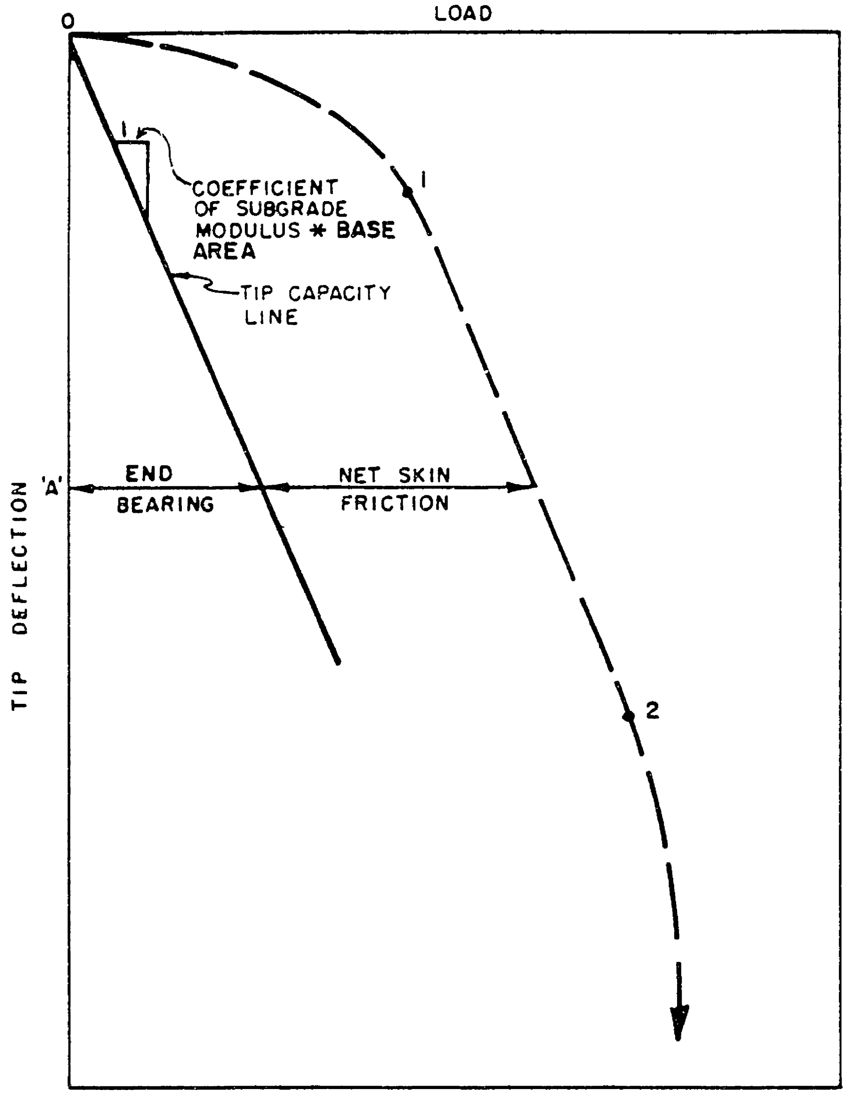
FIGURE 4.3-4 SIDE RESISTANCE REDUCTION FACTOR β
(AFTER WILLIAMS et.al.1980)

4.4 Evaluation of Base Resistance

If end bearing is to be incorporated into the design, base load tests should be performed to determine the load settlement characteristics at the particular site. Base load tests to the authors knowledge have not been performed in the South Florida area. Van Weele (1957) and Brierley et al (1979) have shown that tip load - base movement curves can also be derived from top load tip deformation plots if enough tip movement can be obtained. Typically 1.5 to 2.0 inches of tip movement defines a nice load - base movement curve. The separation of friction and base resistance as suggested by Van Weele and Brierley is shown in Figure 4.4-1. The method as shown in Figure 4.4-1 makes use of the elastic modulus of subgrade reaction, (units of base stress per length of base movement). In the elastic range of base movements this characteristic modulus of subgrade reaction can be used to predict base movements. Base resistance plots are often plotted as bearing stress-strain plots. The strain is evaluated by employing a zone of influence extending a depth of two times that of the shaft diameter (Schmertmann 1970, Skempton 1951). With an average bearing stress-strain plot, (Figure 4.4-2), or subgrade reaction modulus for elastic loadings, one designs the end bearing resistance employing a suitable factor of safety.

4.5 Discussion

With the procedures as outlined above, data from field load tests can be properly divided into frictional and end bearing components. End bearing results can be represented through stress strain plots similar to Figure 4.4-2, and the coefficient of subgrade modulus can be determined. Frictional resistance is presented through the use of load transfer curves as shown in Figure 4.3-2. The peak frictional resistance can be correlated to the unconfined compressive strength of the rock and the mass factor through appropriate α and β values.



SEGMENT 0-1 - THE SHAFT IS PREDOMINATELY BEING SUPPORTED BY SIDE FRICTION. THE TIP IS DEFLECTING DUE TO AN UNKNOWN LOAD APPLICATION.

POINT 1 - SIDE FRICTION IS MAXIMIZED.

SEGMENT 1-2 - SIDE RESISTANCE IS MAXIMIZED AND ALL ADDITIONAL LOAD IS BEING APPLIED DIRECTLY TO THE SHAFTS BASE. THIS LINEAR PORTION DEFINES THE ELASTIC RESPONSE OF THE SUBGRADE.

POINT 2 - THE SUBGRADE DEVIATES FROM ELASTIC BEHAVIOR AND PLASTIC DEFORMATION BEGINS. WITH CONTINUED PLASTIC DEFORMATION A BEARING CAPACITY FAILURE WILL OCCUR.

FIGURE 4.4-1 PROCESS OF SEPARATING END BEARING FROM TIP DEFLECTION (ADAPTED FROM BRIERLY et.al. 1979)

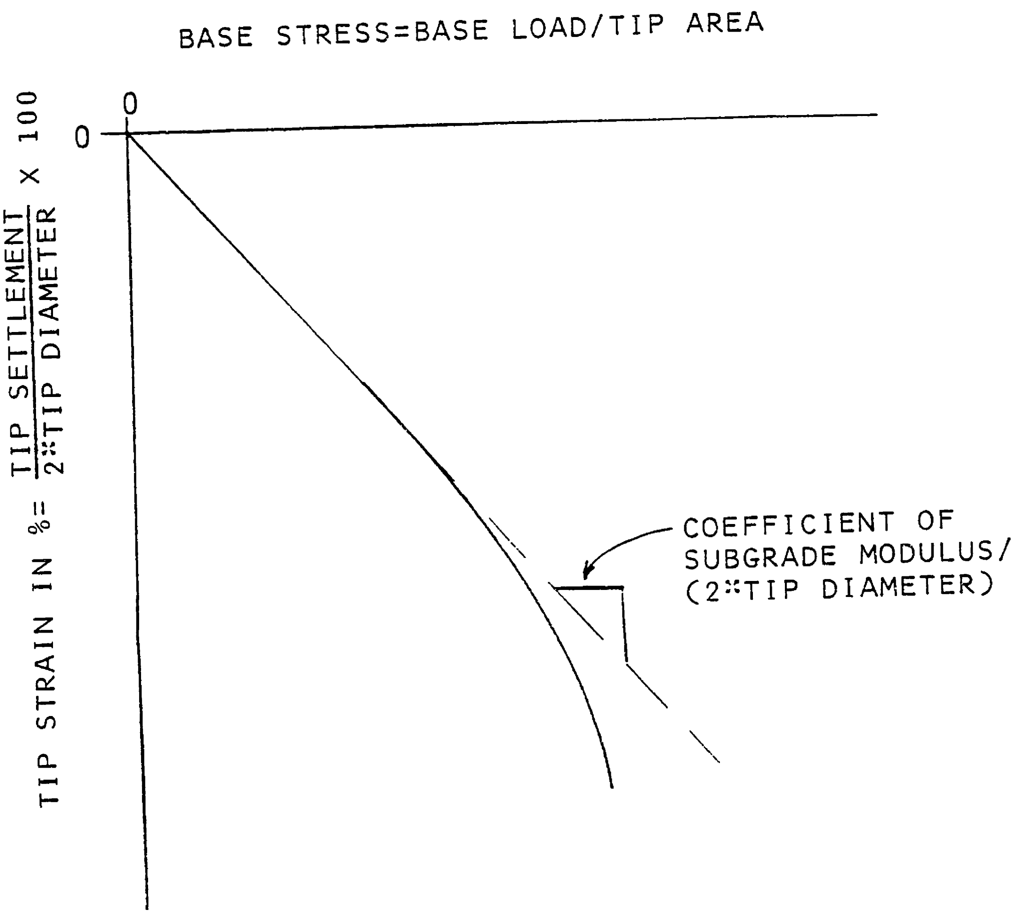


FIGURE 4.4-2 TYPICAL END BEARING STRESS-STRAIN PLOT

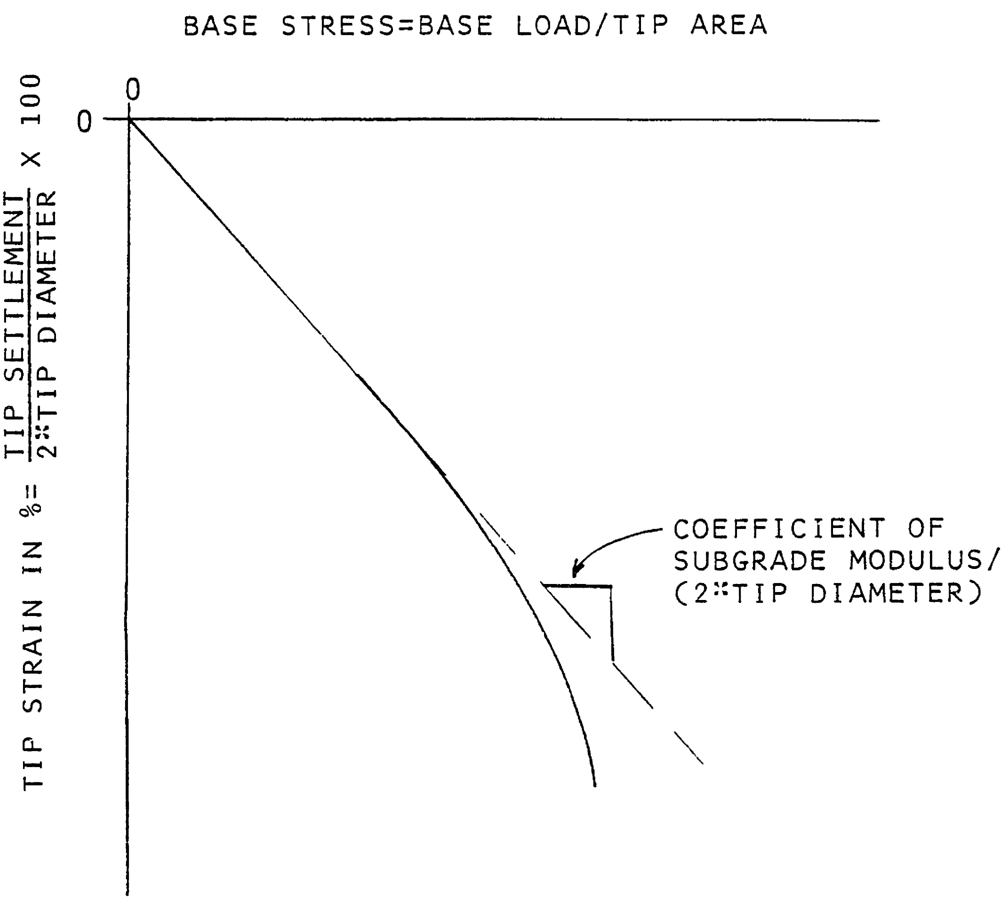


FIGURE 4.4-2 TYPICAL END BEARING STRESS-STRAIN PLOT

5. EVALUATION OF LOAD TESTS ON CORAL AND COQUINA LIMEROCKS

5.1 General

The four load test sites are evaluated in turn. For each load test a soil-rock profile, rock properties and telltale displacements or load cell evaluated load distribution curves are presented. General descriptions of the limerock formations described are given in Chapter 3, the location of these sites are shown in Figure 3-1.

Load tests for site L were combination end bearing and side friction shafts while sites S, SE and I were predominately side friction. Design curves developed from each test are explained and any possible causes of concern in the analysis are identified.

5.2 Evaluation of Load Tests at Site L

At Site L four compression load tests were performed. The instrumentation for load tests L-1 and L-2 consisted of 16 load cells and two telltales, (one at the top of the rock socket and one toward the base of the socket). Load tests L-3 and L-4 employed 6 telltales each.

The load cells employed in Tests L-1 and L-2 were obtained through James N. Anagnas and were similar to the Mustran type cell manufactured by the University of Texas at Austin, (See Appendix 4). Two Vishay, (Model SB-1), Switch and Balance units and two Vishay, (Model P-350A) strain indicator units were used to measure the internal stresses of the load cells during the load testing. The load cells in both tests gave contradicting results as shown in Figure 5.2-3 and 5.2-5. The load cell data reduction as plotted shows shaft and socket load in excess of the applied top loading. Since an increase in load with depth is not reasonable the load cell distribution curves were not used in the following analysis.

The telltales employed in all Site L load tests were bent at right angles toward the top of the shaft and extended horizontally outside the shaft's butt to supported dial gages. Telltales and a typical schematic of the reference frame are shown in Figures A4.4-1 and A5.2.2-1. The ½" (hollow center) steel telltales were cased with 3/4" PVC tubing. At their base the telltales were exposed to the concrete for a distance of approximately 12 inches. The end of the telltale was crimped closed with a pair of pliers as shown in Figure 5.2-1. The entrance of the telltale into the 3/4" PVC casing was taped with a heavy silver "Duct" tape.

The socket diameters used in the analysis were evaluated from an estimate of the placed concrete volume and the known area of the cased portion of the shaft. Shaft L-2 was cased directly to the rock socket. With the information from the installation of test shaft L-2 the socket diameter was evaluated to be 15% larger than design. This 15% increase in diameter was used as an estimate of the increase in socket size for shafts L-1 and L-4. The remaining excess concrete for shafts L-1 and L-4 is assumed to be a result of caving of an uncased sand strata found above the lower limerock.

Test Shaft L-3 failed at a very low load (200 tons). A subsequent coring investigation on Test Shaft L-3 showed a discontinuity existed between the depths of 43 to 47 feet. Due to this discontinuity this test is not considered here.

<u>Evaluation of Load Test L-1</u> - The total settlement of test shaft L-1 is less than one half that found from shafts L-2 and L-4. Through a comparison of the load deflection curves for these 3 load tests, (shown in Figures 5.2-2, 5.2-4, and 5.2-6) one notices that Shaft L-1 behaves differently than Shafts L-2 and L-4.

The deep telltale in Shaft L-1 gave fluctuating readings from 0.045 inch in compression to 0.04 inch in tension. Actual movement of the shaft's base is not expected to vary from compression to tension. This fluctuation in base movement could have been caused by any combination of temperature fluctuations, actual

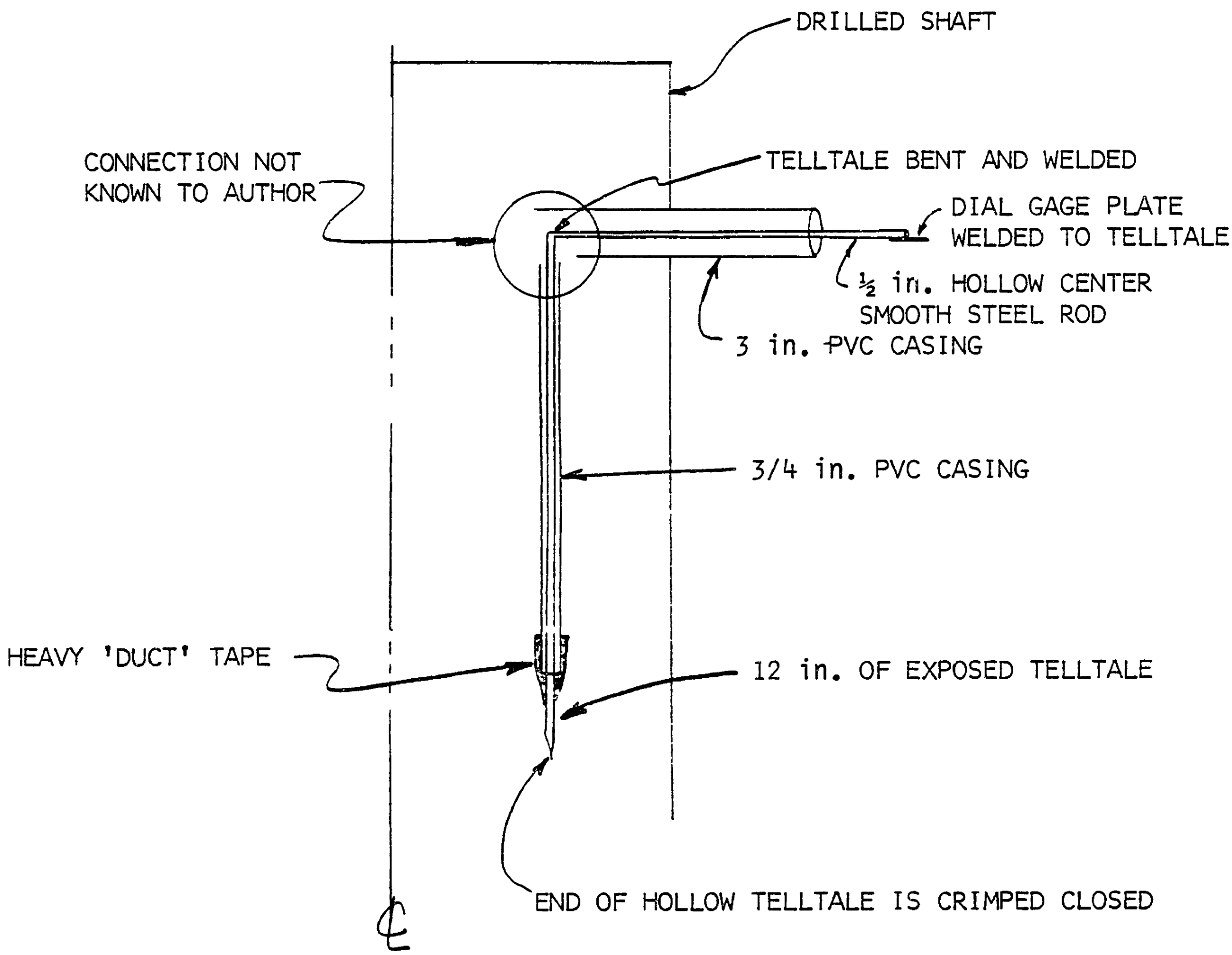


FIGURE 5.2-1 SIDE VIEW OF 90° TELLTALE

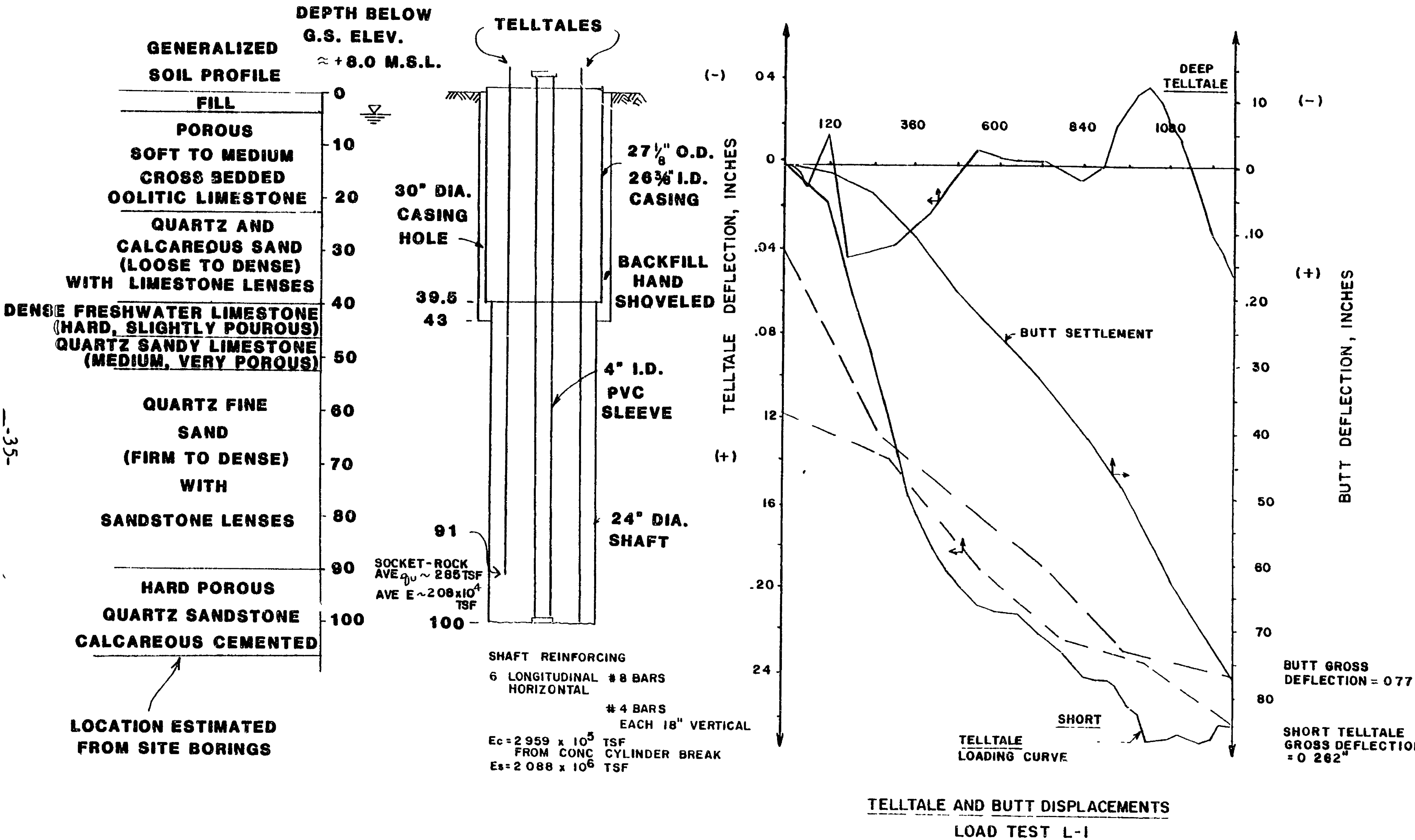


FIGURE 5.2-2 TEST SHAFT L-1
(DATA AFTER KADERABEK 1981)

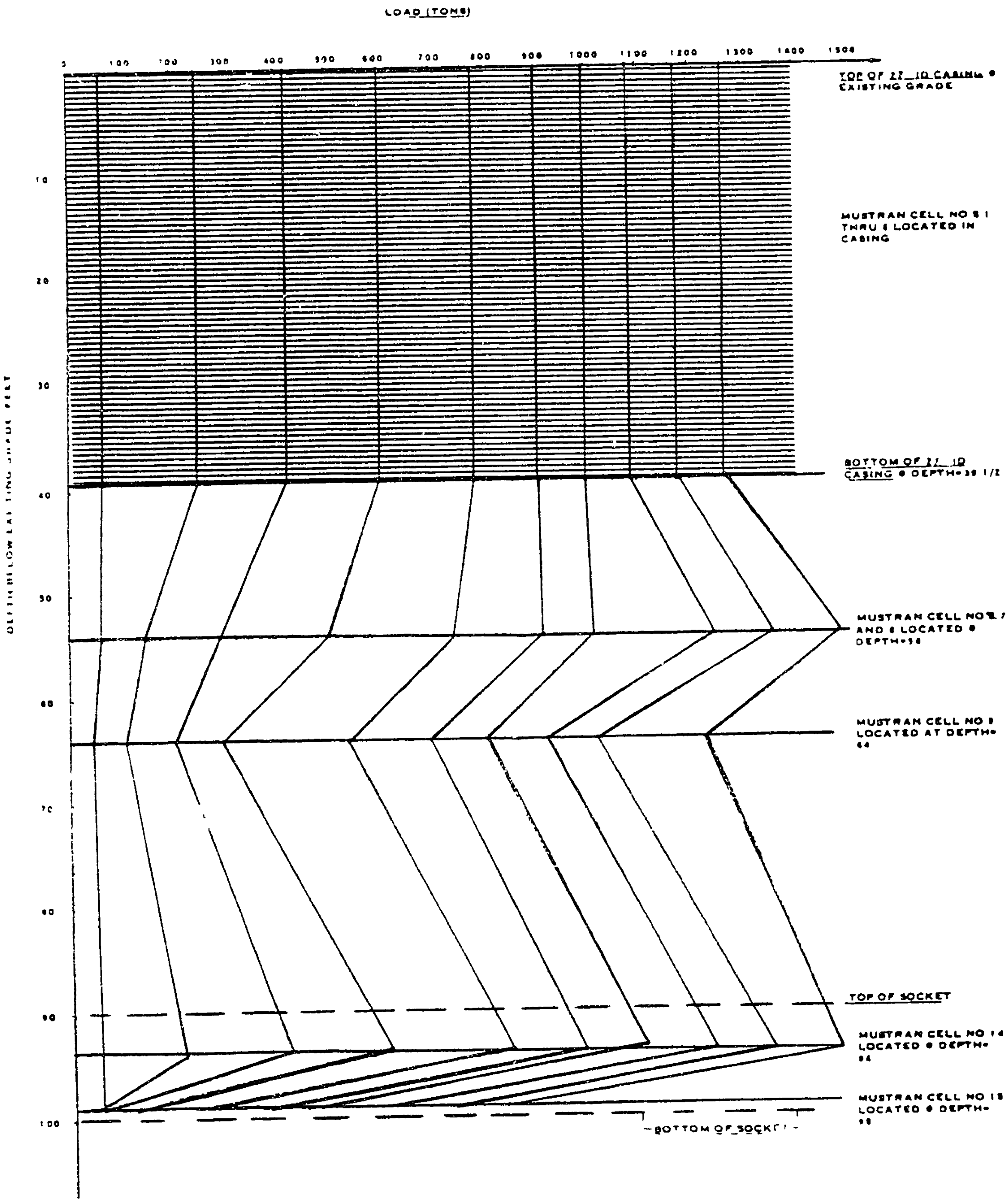


FIGURE 5.2-3 LOAD TEST L-1, MUSTRAN CELL DERIVED LOAD DISTRIBUTION
(AFTER KADERABEK 1981)

movement, and faulty instrumentation (mechanical difficulties). Temperature effects do not by themselves account for such erratic behavior. Using the coefficient of thermal expansion of steel = 6.6×10^{-6} in/in/F^{o}, and a length of 100 feet, a net change of 10^{o} F along the entire length of the telltale would be required to cause this movement. Many authors, as stated in Appendix 4, have found that the shaft temperature essentially remains constant below a depth of 10 ft. This unexplained deep telltale behavior implies that there is a problem with the telltale instrumentation for Load Test L-1. Telltale movements for the short telltale of Load Test L-1 gives further support to believing the telltale movements of Shaft L-1 are in error. If the fluctuations in the deep telltale (base of socket) was taken to represent no tip movement, then the elastic deformation of the rock socket, (being the difference in telltale movements), would equal the telltale movement of the short telltale (located at the top of the rock socket). The load required to cause this elastic deformation can be evaluated using elastic theory.

Using elastic theory similar to that shown in Figure 4.2-2 the evaluated load to cause the elastic deformation described above exceeds the top applied load by a factor of 3 to 4 throughout the loadings applied. This excessive deformation may be due to contaminated concrete or reduced socket diameter. Because of the small butt settlement compared to shafts L-2 and L-4, it appears that the shaft size and concrete were of adequate quality.

The odd concaveness of the short telltale shown in Figure 5.2-2 also suggests instrumentation problems. The net deflection of the short telltale is less than the butt movement, but the total deflection exceeds that of the butt for top loads in the range of 150 to 550 tons. The magnitude of the telltale deviations from expected behavior suggest a mechanical problem with the instrumentation. This problem could occur from excessive friction in the telltale casing. (Barker and Reese 1969), or from improper installation. Due to the difficulties with the two telltales in Shaft L-1, any use of their readings would result in too many speculations. Telltale readings for Test Shaft L-1 were not used in the analysis of the site.

Because of the telltale problems Load Test L-1 was not analytically reduced. This load test, (L-1) does show that in places the lower Fort Thompson formation can support 1260 tons of load using a 10 foot long 28 inch diameter rock socket with less than 0.8 inch of butt settlement. Employing elastic theory, 1260 tons applied over 90 feet depth, (to the top of socket), can account for up to 0.76 inches of settlement with the given test shaft geometry. This shows the 1260 tons is supported with essentially no tip movement.

Evaluation of Load Tests L-2 and L-4 - Load Tests L-2 and L-4 are similar in behavior and are evaluated together (Figures 5.2-4 and 5.2-6). Since borings were not done at the location of the test shafts, rock properties were estimated from the 6 borings which entered the lower Fort Thompson Formation.

Figure 5.2-6 gives a plot of telltale movement with depth of shaft for Load Test L-4 for various load increments. Figure 5.2-7 shows some of the telltale information for Shaft L-4 is somewhat contradictory. The rock socket in test Shaft L-4 is 12 feet long. By using the calculated socket diameter and concrete modulus, one would expect an elastic socket deformation of 0.04 inch (for an average socket load of 520 tons) to 0.07 inch (for an average socket load of 900 tons). Knowing this expected degree of elastic socket deformation, telltale No. 2 (depth = 90.25 feet, mid socket telltale) is considered in error. Telltale No. 4, (depth = 70.33 feet, mid sand strata telltale) is also considered in error since it insinuates the center of Shaft L-4 is moving more than the top or base of the shaft.

Telltale No. 6 located 5.17 feet into Shaft L-4 shows more settlement than the butt. Since the butt settlement is also less than most readings from telltale No. 5 (depth = 43 ft), it is felt that telltale No. 6 is more representative of telltale movement than the butt movement which is recorded from the steel casing. The discrepancy between the butt readings and the telltale readings in Shaft L-4 may be recording error or may result from breaking of the concrete-steel bond. Breaking of the concrete-steel bond may occur due to concrete - steel eccentricities at the top of the shaft, (not being level), thereby overstressing one material with respect to the other. Regardless of the reason for this settlement difference near the top of Shaft L-4, the load is expected to be uniformally distributed through the drilled shaft prior to the end of the casing, and is distributed to the rock socket.

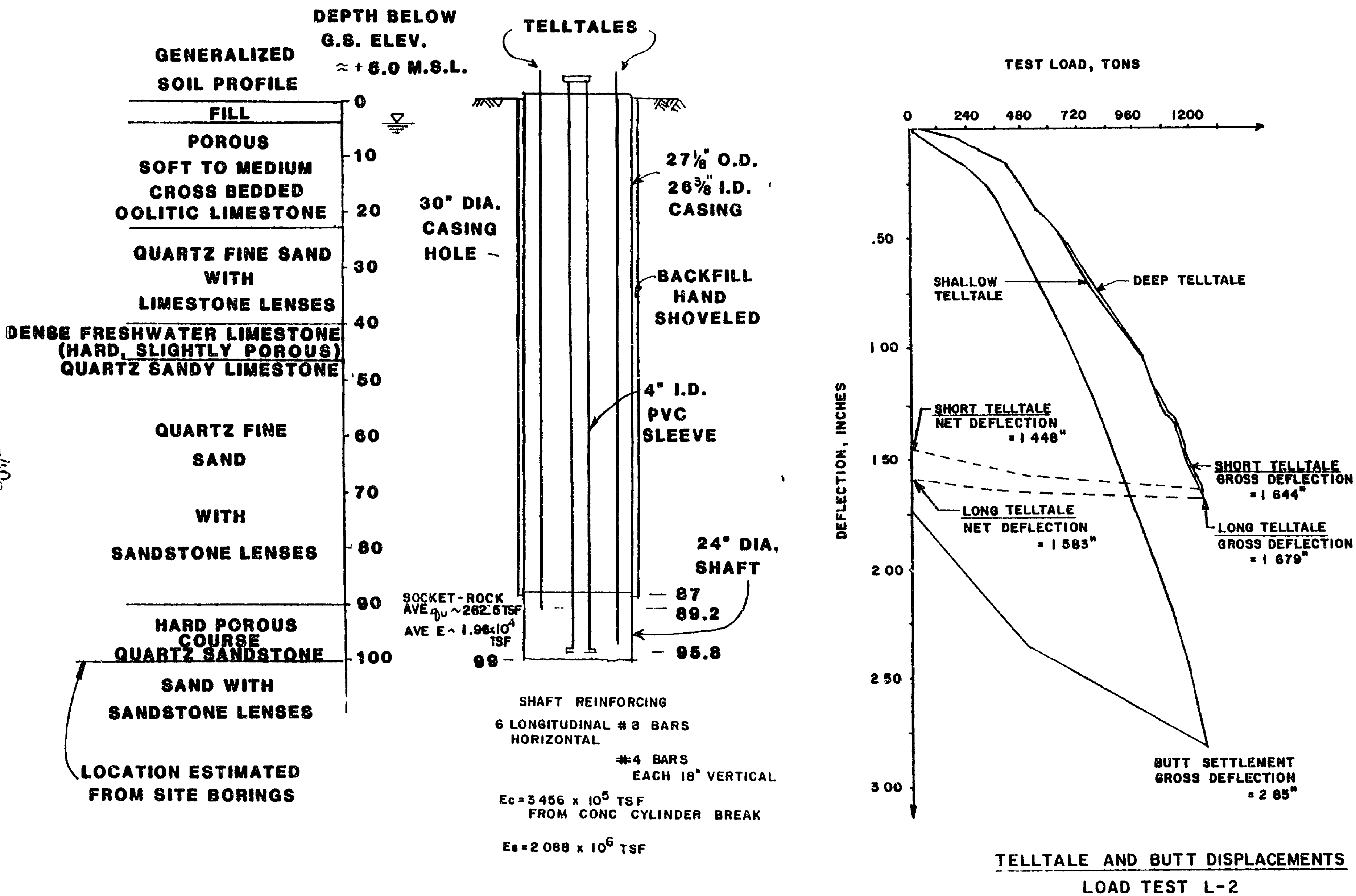


FIGURE 5.2-4 TEST SHAFT L-2
(DATA AFTER KADERABEK 1981)

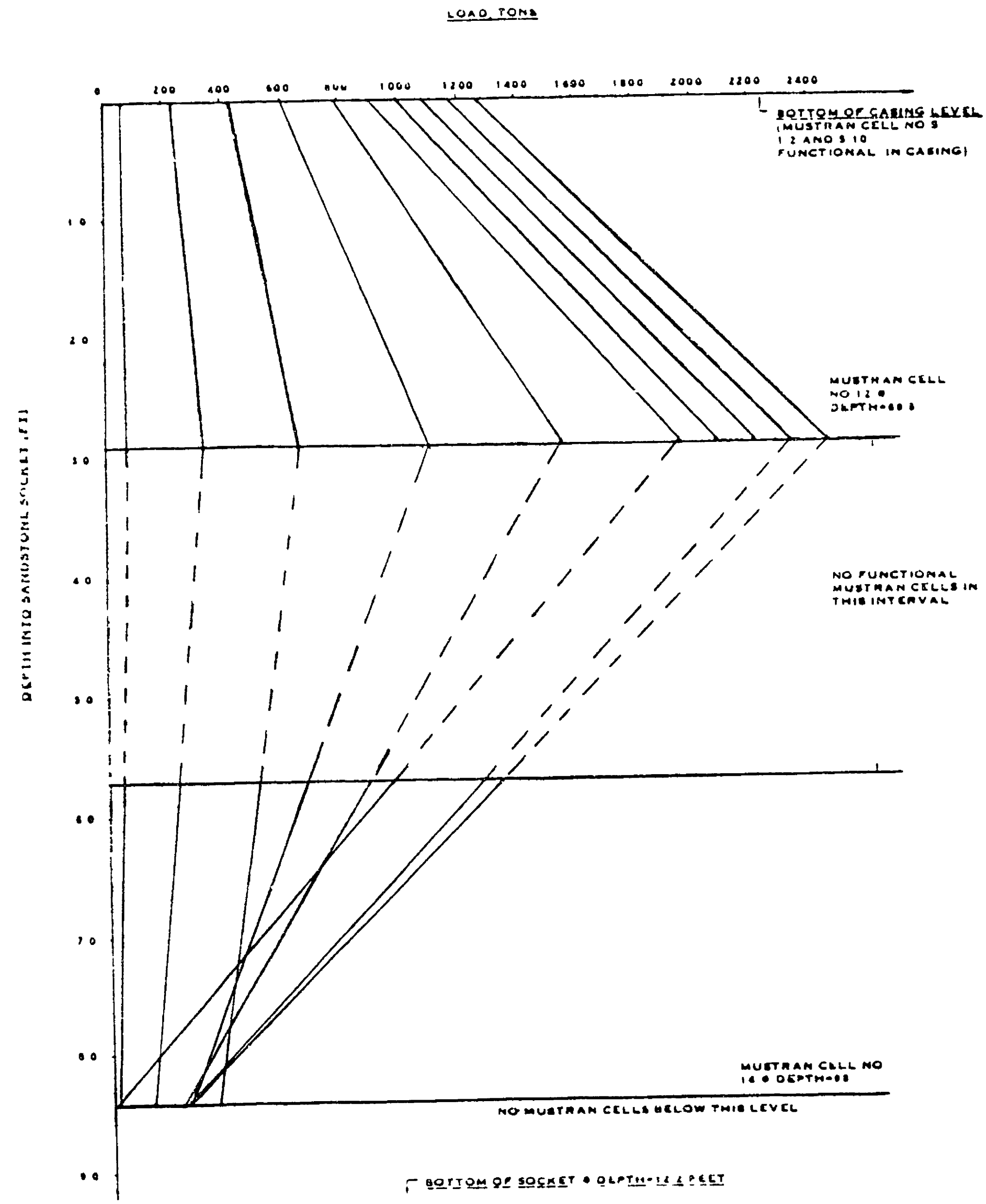


FIGURE 5.2-5 LOAD TEST L-2, MUSTRAN CELL DERIVED SOCKET LOAD DISTRIBUTION (AFTER KADERABEK 1981)

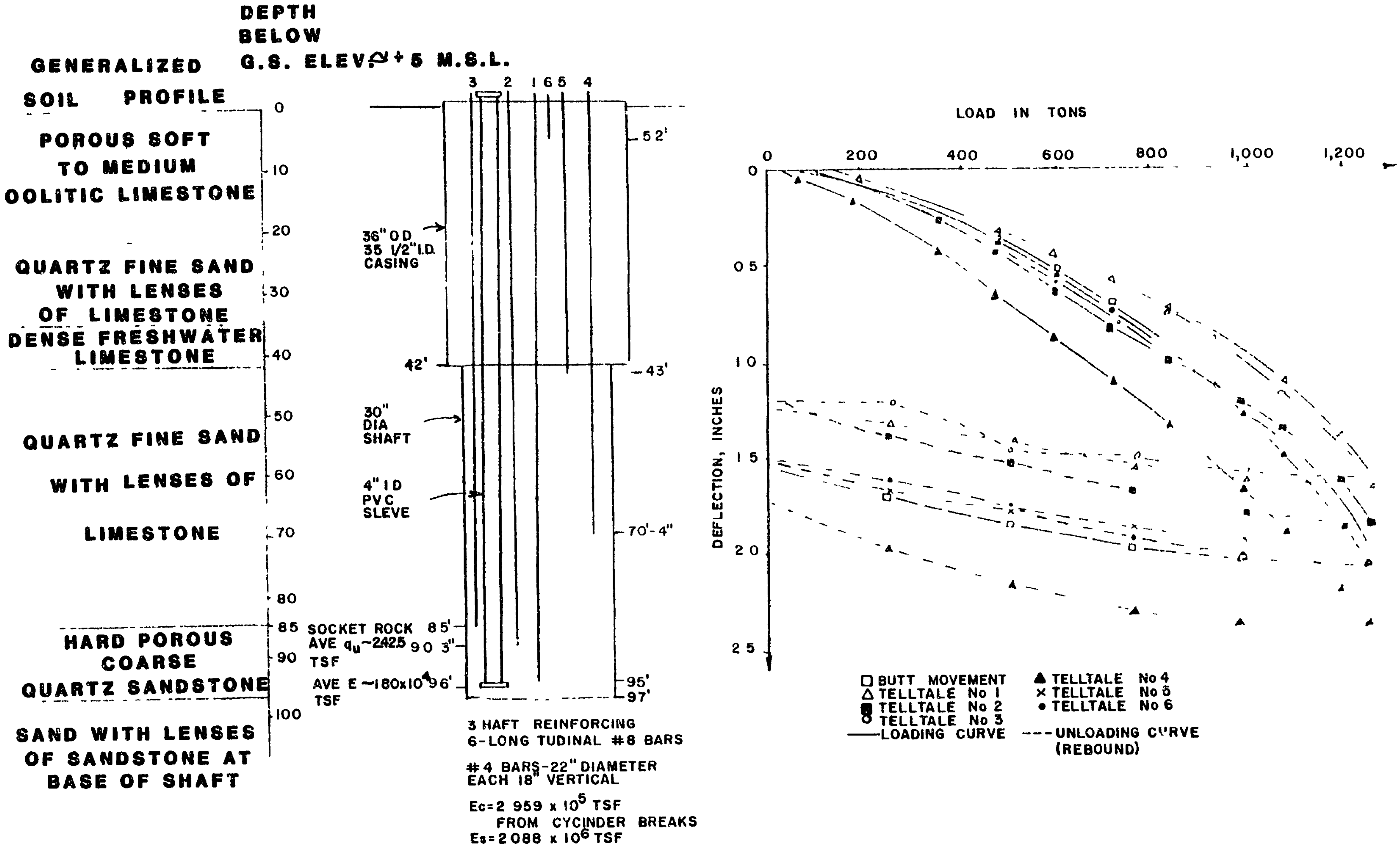


FIGURE 5.2-6 TEST SHAFT L-4
(DATA AFTER KADERABEK 1981)

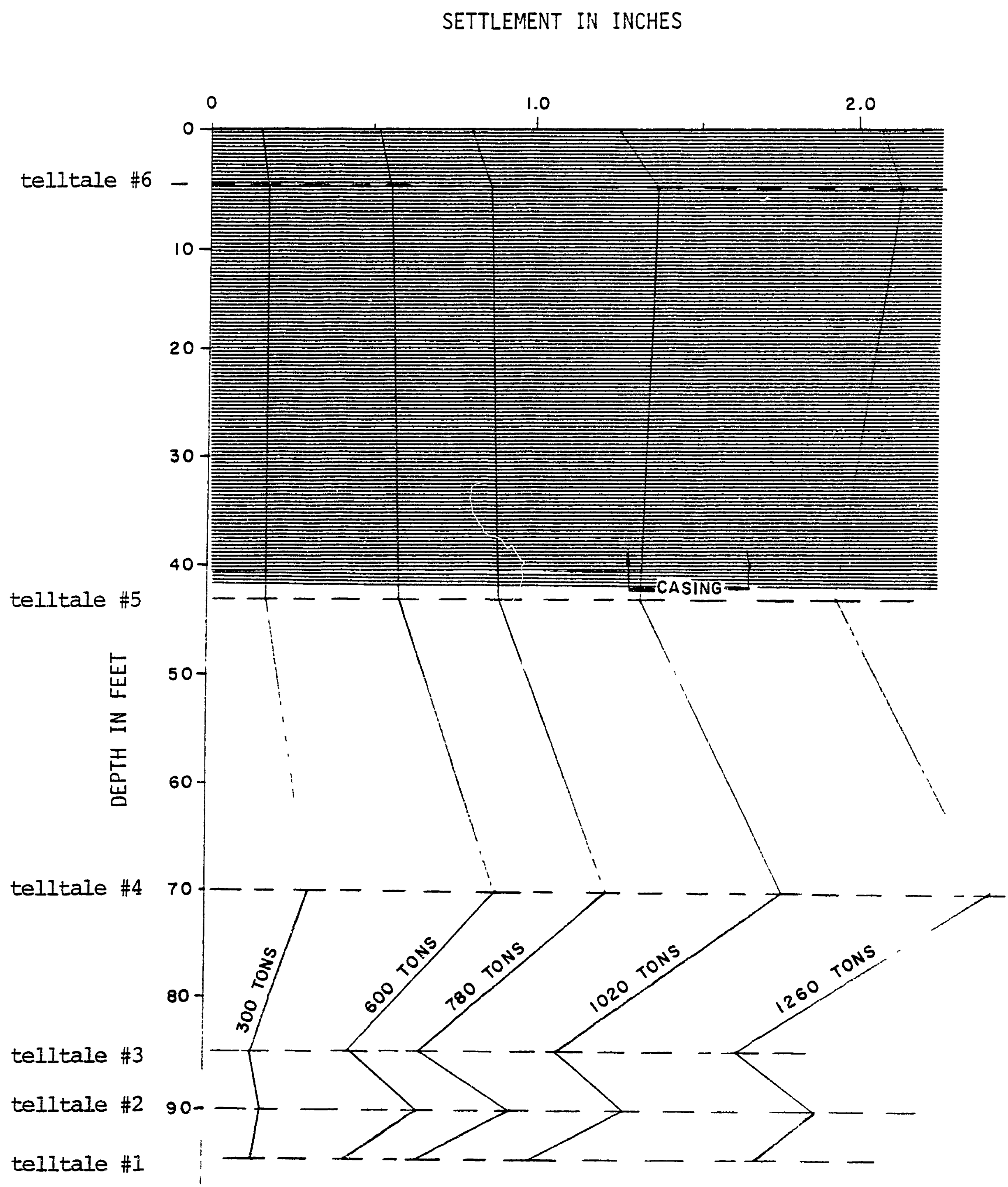


FIGURE 5.2-7 TELLTALE DATA LOAD TEST L-4

The evaluation procedure used on test shafts L-2 and L-4 considered tip movement. Elastic theory would predict an elastic deformation between the socket telltales to be less than 0.04 inch for a top load of 500 tons and less than 0.10 inch for a top load of 1260 tons. In both shafts L-2 and L-4, the telltale movements on the top and base of the rock socket give similar results, (as expected), and thus give credibility to the tip evaluation method used. It should be noted that due to the small elastic deformation expected through a rock socket of this size, (indicated above) and the length of the telltales used for shafts L-2 and L-4, the values obtained for base movement can be in error as much as the expected socket elastic deformation. Error, as indicated before, can consist of a combination of temperature effects, instrumentation problems (possible friction between telltale and telltale casing), and can also result from poor dial gauge monitoring (time delays or inaccurate recording when taking dial readings). Due to the poor quality of the telltale readings, (Figures 5.2-4 and 5.2-6), integrating an evaluated load between telltales is not an appropriate method to evaluate Load Tests L-2 or L-4. A tip movement evaluation as explained in Section 4.4 which employs tip movement rather than relative movements between telltales is used in analysing the data from Load Tests L-2 and L-4.

Through analyzing telltale settlements for load tests L-2 and L-4, it is reasonable to assume the total load is distributed down to the rock socket.

Plotting the load-tip settlement curves for shafts L-2 and L-4, one notes a linear portion in each curve. The settlement beneath the drilled shaft's tip is composed of elastic and eventually plastic straining of the substrata beneath the shaft. The load-settlement curves as shown in Figures 5.2-8 and 5.2-9 are broken into 3 sections as suggested by Van Weele (1957) and Brierley et al (1967). The first portion being delineated by the section from the origin to point 1 represents

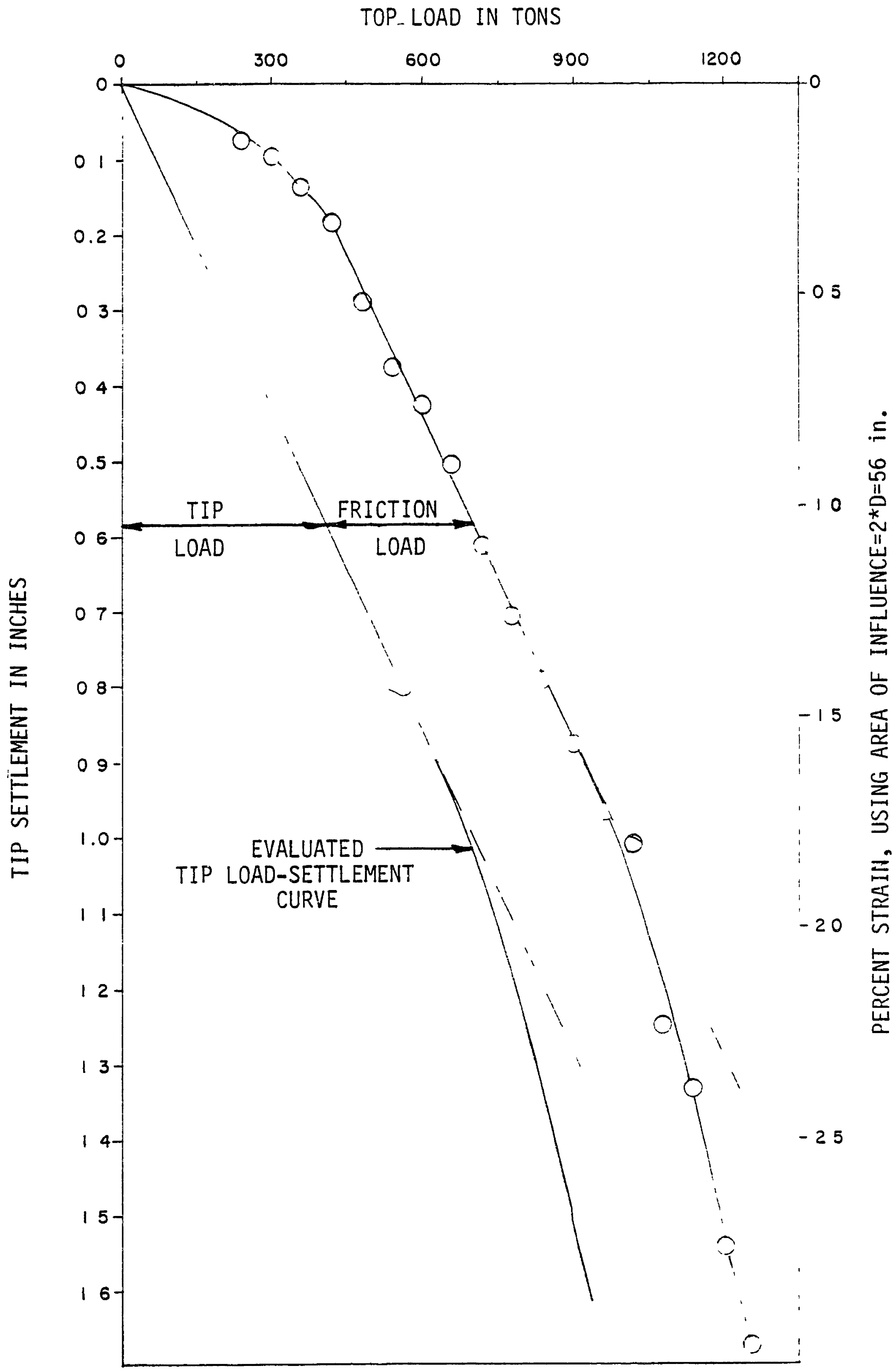


FIGURE 5.2-8 TIP SETTLEMENT CURVE AND DERIVED TIP LOAD SETTLEMENT CURVE FOR LOAD TEST L-2

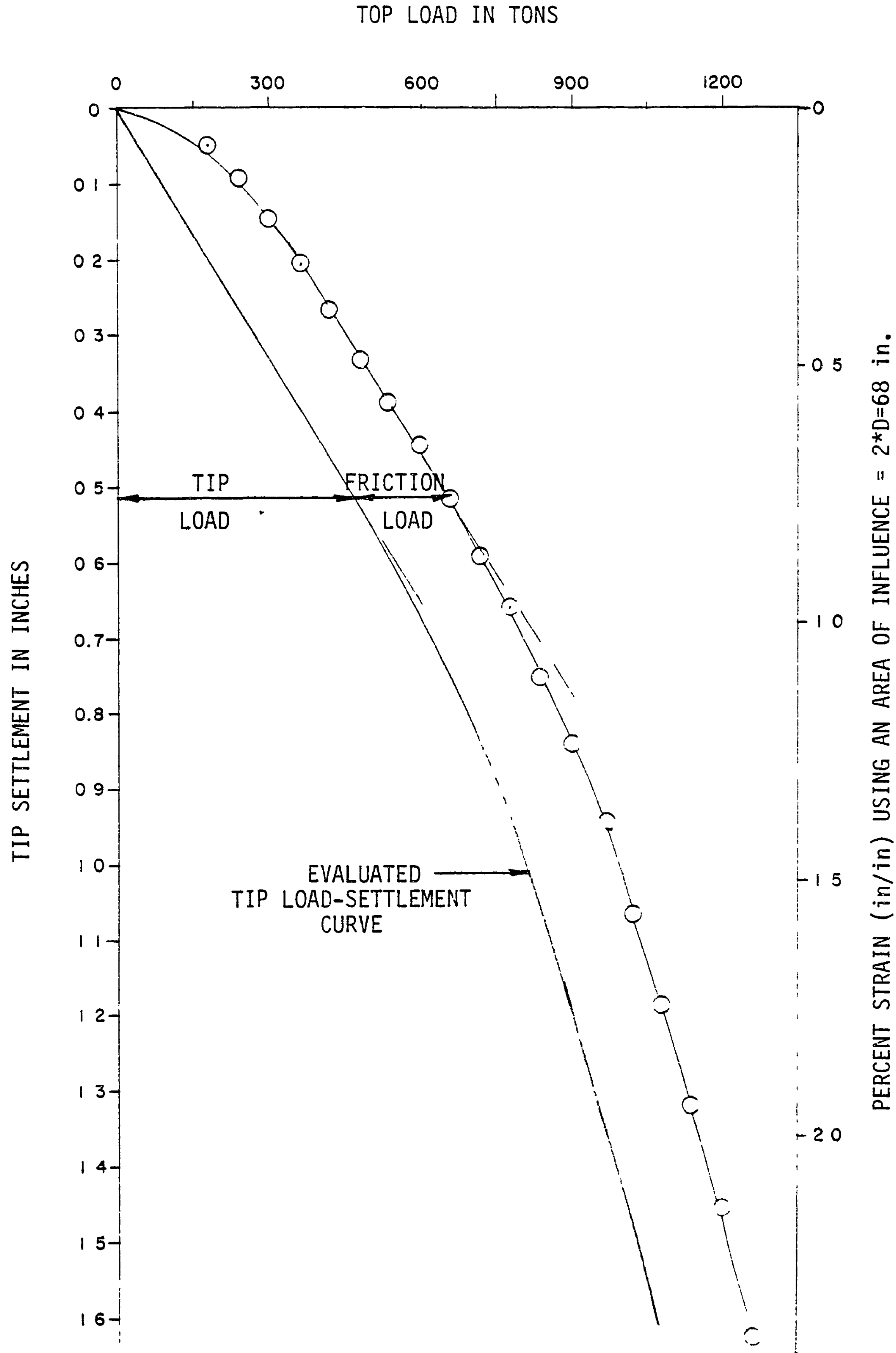


FIGURE 5.2-9 TIP SETTLEMENT CURVE AND DERIVED TIP LOAD SETTLEMENT CURVE FOR LOAD TEST L-4

skin frictional resistance and a small unknown amount of base resistance. After Point 1, skin friction is maximized and any additional load is supported through end bearing. From Point 1 to Point 2 one finds a straight portion of the load deflection curve. This straight portion defines the coefficient of subgrade modulus and is a function of the media beneath the pier. Beyond point 2 the subgrade is no longer acting elastically and plastic deformation is beginning, which if continued, would eventually lead to bearing capacity failure. As shown in Figures 5.2-8 and 5.2-9, deviation from elastic behavior occurs at strains of 1.6% for Load Test L-2 and 0.8% for Load Test L-4. Shaft L-4 is known to have punctured the lower limerock, (noted during installation), and rests on the quartz sand containing rock lenses which underlies the Fort Thompson Formation.

Figure 5.2-10 which superimposes the tip movements for Shafts L-2 and L-4, implies that the difference in bearing materials appears to be offset by the difference in diameter for the loads employed. To show the effects of the different diameters a plot of load resistance against % strain will be compared for Load Tests L-2 and L-4.

All strain analysis for end bearing strain are evaluated using a zone of influence equal to two times the shaft diameter. Thus strain = tip settlement/two times the shaft diameter.

The base resistance - strain plot is shown in Figure 5.2-11. Shaft L-4 is known to be seated on top of the sand stratum which exists below the lower Fort Thompson Formation. This sand stratum contains lenses of calcareous rock. Shaft L-2 is believed to have a thin portion of the Lower Fort Thompson Formation between the shaft and the lower sand-rock lensed stratum. As expected the base resistance of Shaft L-2 for high strains is higher than that of Shaft L-4 which has no rock base support.

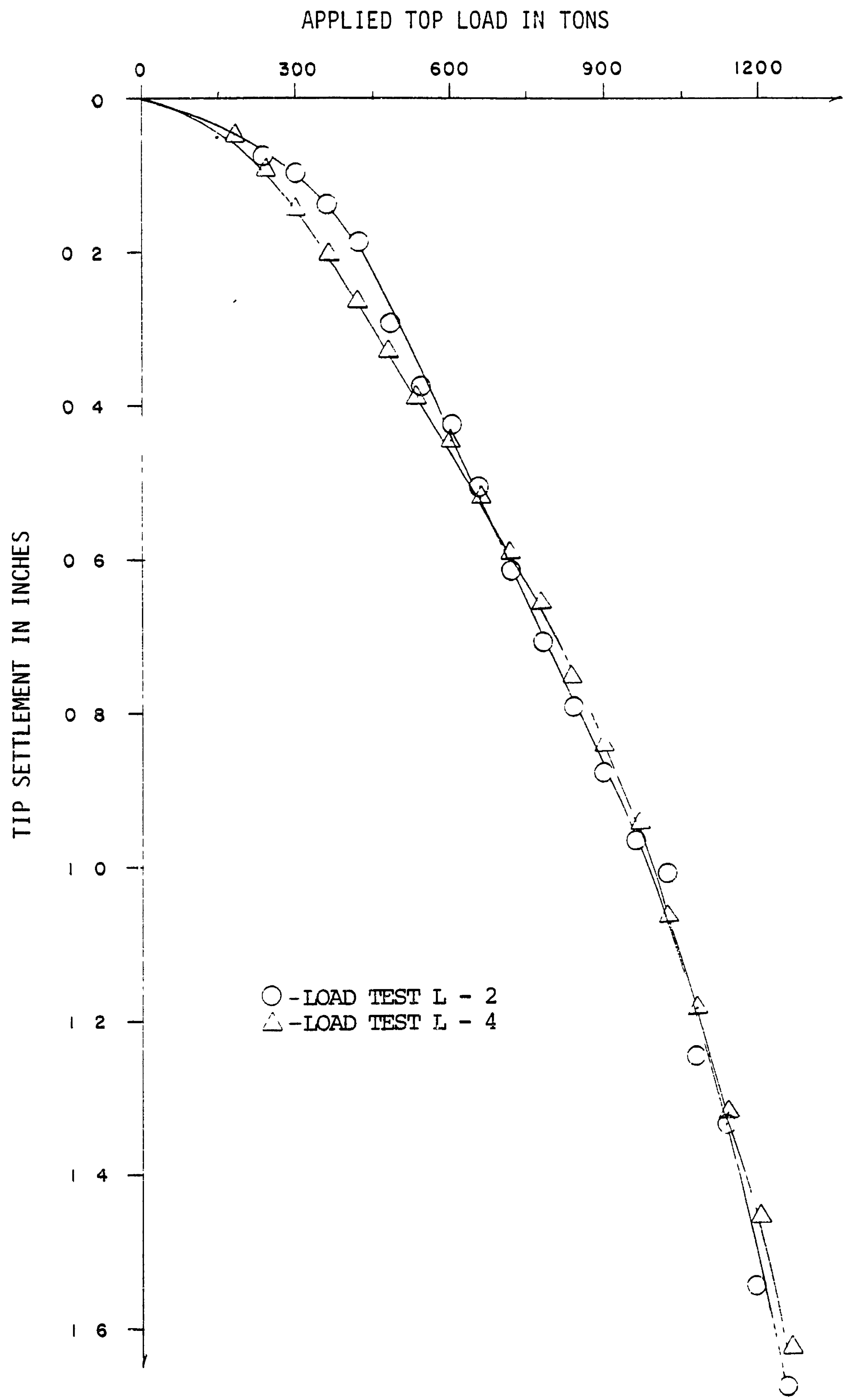


FIGURE 5.2-10 SUPERIMPOSED TOP LOAD vs TIP SETTLEMENT CURVES FOR LOAD TESTS L-2 AND L-4

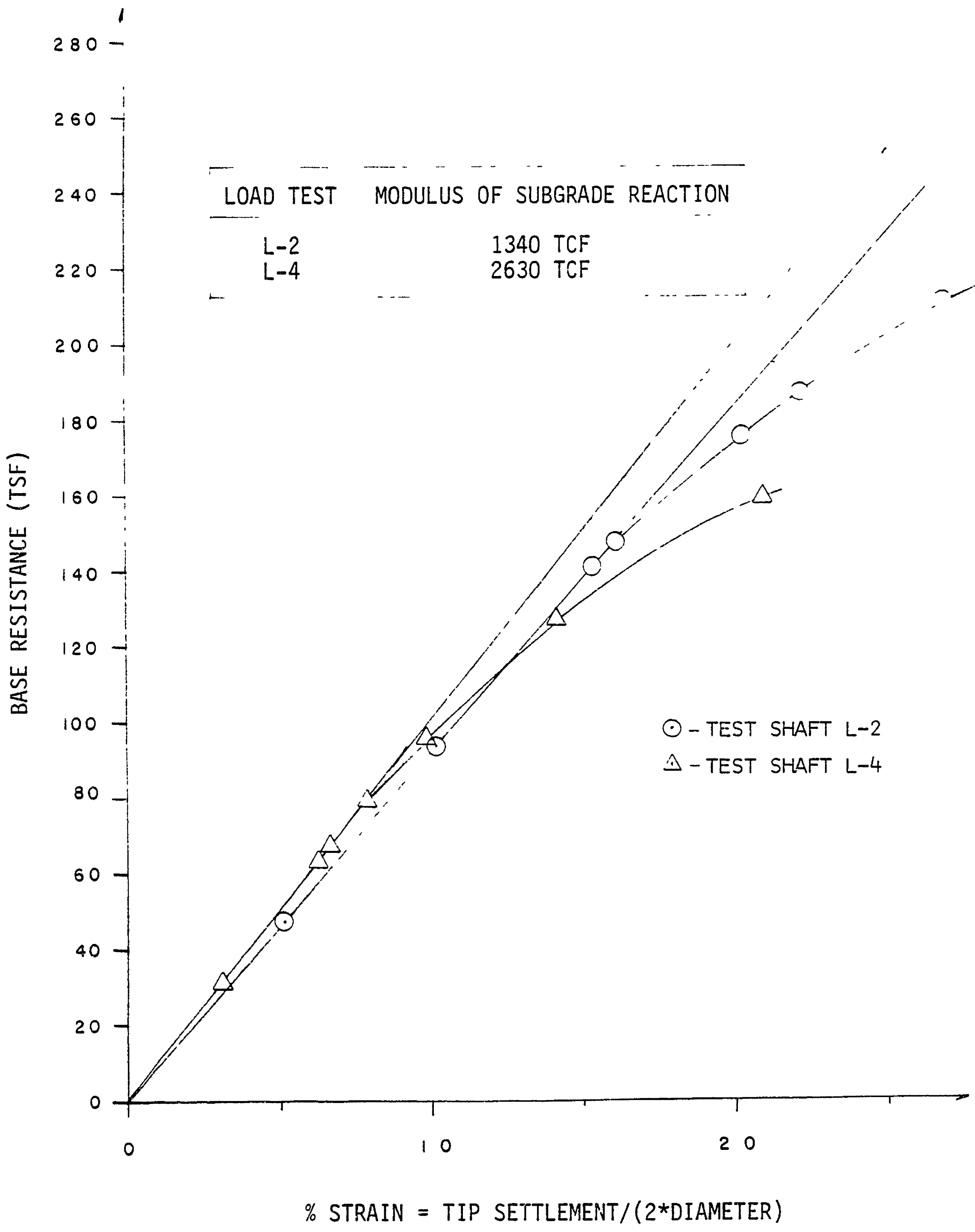


FIGURE 5.2-11 EVALUATED TIP SETTLEMENT - BASE RESISTANCE CURVES FOR TEST SHAFTS L-2 AND L-4

The frictional load curve with respect to side movement can be estimated from total load-tip settlement curves. This is done by subtracting the estimate of end bearing made earlier. Side friction-displacement curves for shafts L-2 and L-4 are shown in Figure 5.2-12.

Load transfer curves for the two shafts could then be developed as outlined in Section 4.3. The load transfer curves for load tests L2 and L-4 are found in Figure 5.2-13.

<u>Side Resistance Correlations</u> - The peak side resistance can be compared to other sites with the use of the estimated rock properties. In Figure 5.2-14 Williams and Pells (1981) shows the α correlation between the peak side resistance and the unconfined compressive strength of the intact rock. Employing Figure 5.2-14 α values for Load Tests L-2 and L-4 would be 0.11 and 0.115 respectfully. Williams (1980) also suggested an additional side resistance reduction factor β (See Figure 5.2-15). β is defined as the peak field side resistance / (α * The intact unconfined compressive strength of the rock).

β reflects the condition of the rock mass through the use of the mass factor, (Mass factor = j = Mass modulus of rock/intact rock modulus of the rock). The intact rock modulus is normally obtained through testing rock cores. The mass modulus can be estimated through pressure meter tests (Schmertmann 1970), standard penetration resistance results (Schmertmann 1970), or through correlations with RQD values (Deere et al 1967).

Deere et al (1967), has compiled information showing a trend between rock quality, (RQD or squared wave ratio), and the mass factor j (see Figure 5.2-16). As seen in Figure 5.2-16, the data is scattered. Pressure meter results would be more desireable if available. Since pressure meter results were not available, RQD values obtained from the lower Fort Thompson Formation at Site L are shown in Figure 5.2-17. Using the mean RQD of 35% one would employ a mass factor of

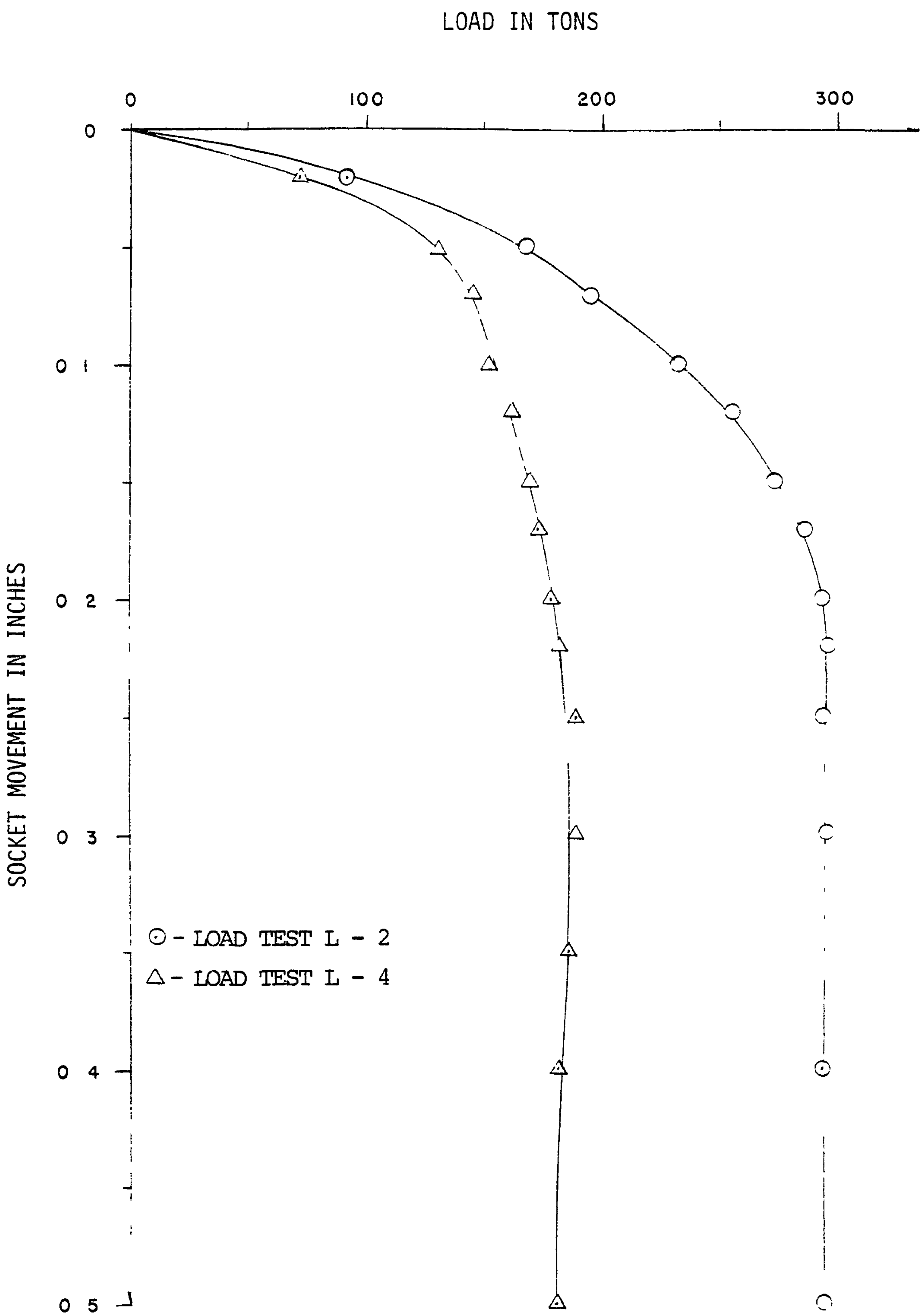


FIGURE 5.2-12 FRICTION LOAD TRANSFER CURVES FOR SHAFTS L-2 AND L-4

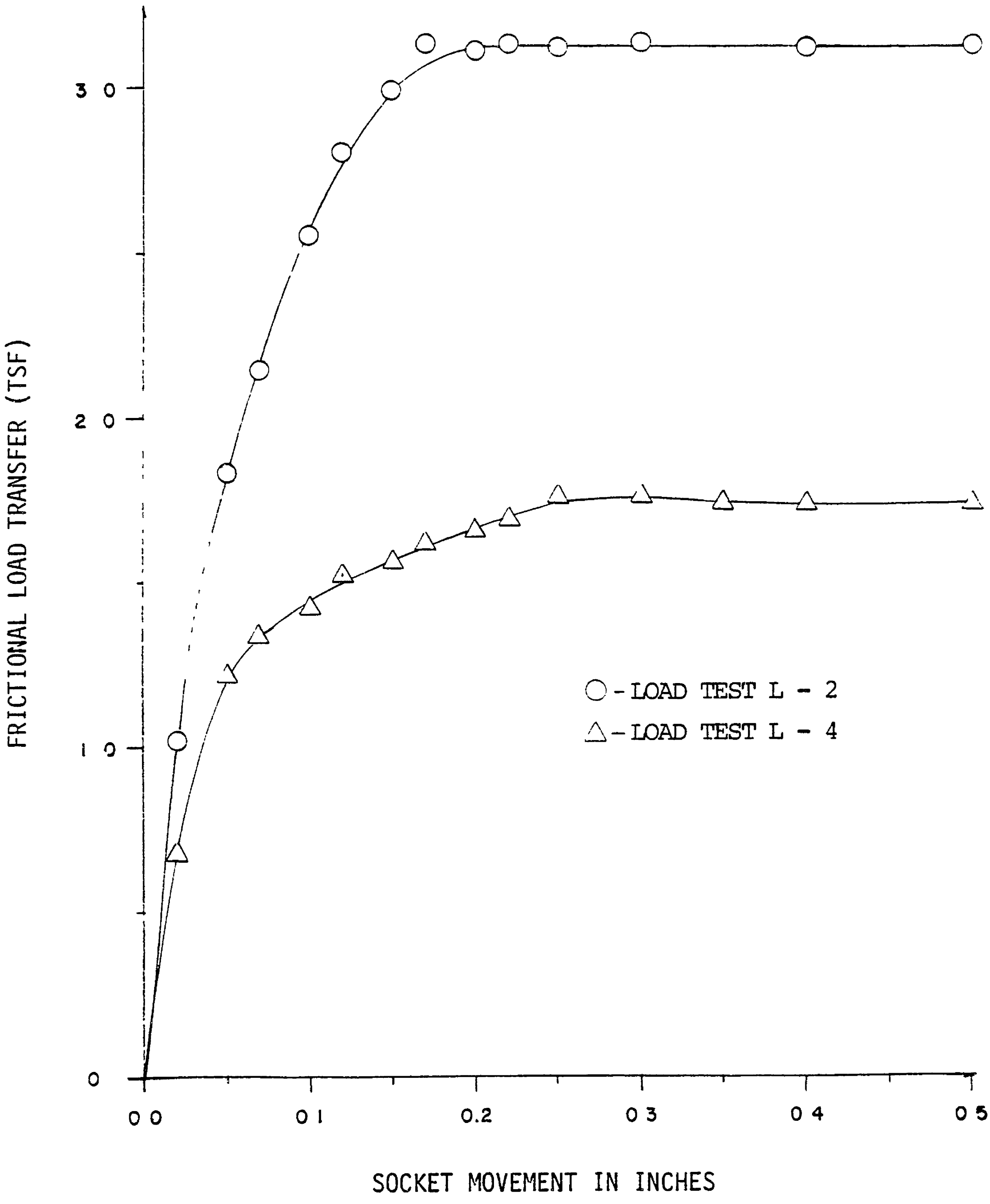
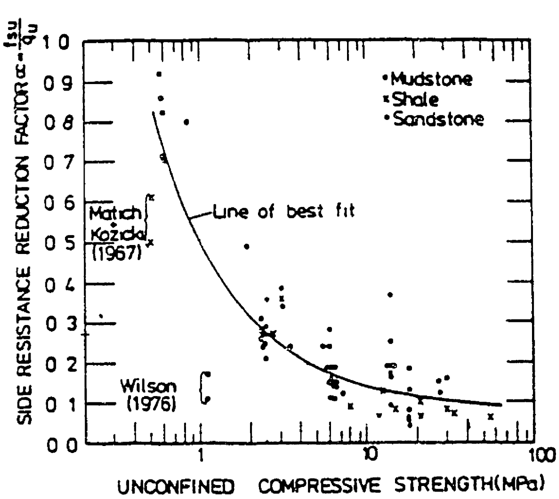


FIGURE 5.2-13 LOAD TRANSFER CURVES FOR SHAFTS L-2 AND L-4



1MPa=10.44TSF

FIGURE 5.2-14 α SIDE RESISTANCE FACTOR (AFTER WILLIAMS AND PELLS 1981)

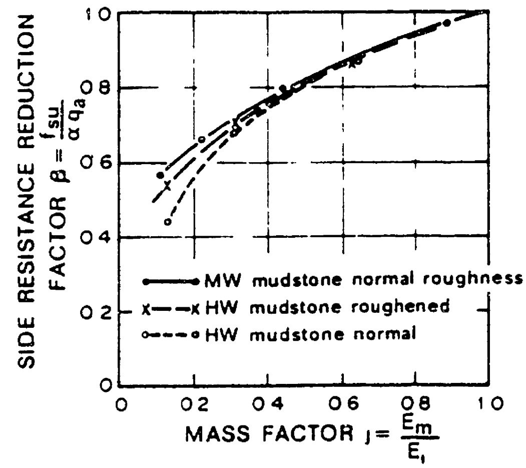


FIGURE 5.2-15 β SIDE RESISTANCE FACTOR (AFTER WILLIAMS et. al. 1980)

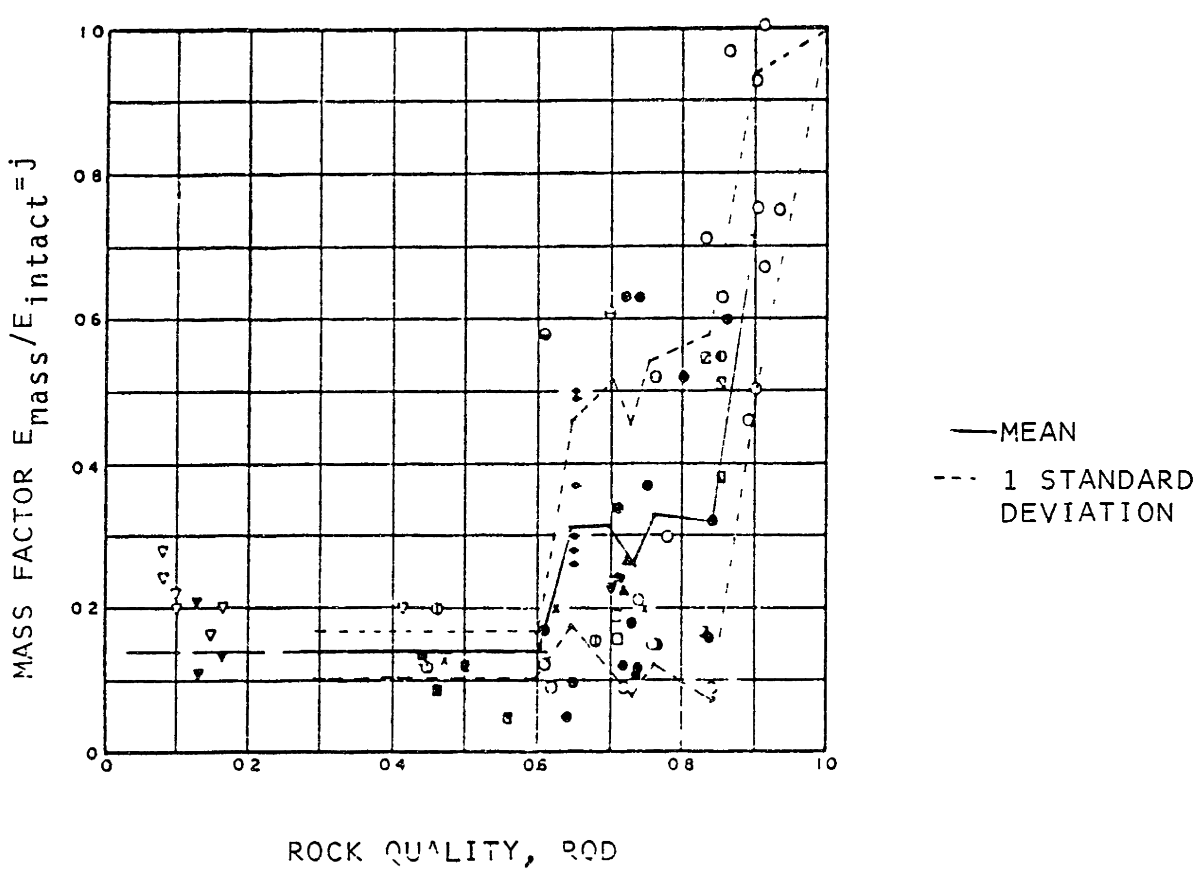
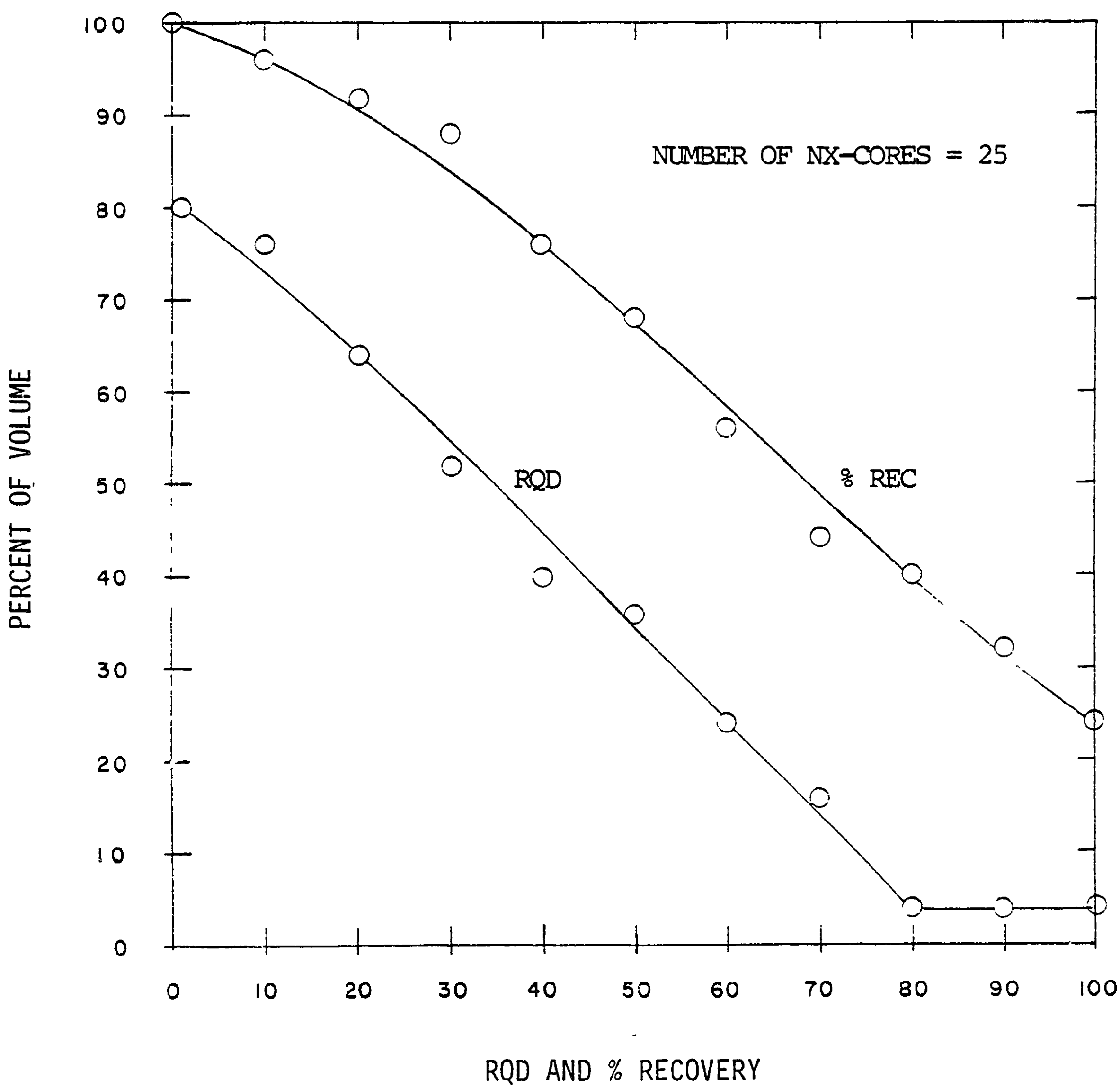


FIGURE 5.2-16 MASS FACTOR-RQD CORRELATION (AFTER DEERE et. al. 1967)



% RECOVERY - MEAN = 69, AVERAGE = 65
RQD - MEAN = 35, AVERAGE = 34.8

FIGURE 5.2-17 RQD AND % RECOVERY VALUES FROM NX-CORES AT SITE L
Data is for lower limerock (depth=88-108 ft.)

0.14 (Figure 5.2-16) with the unconfined compressive strength as estimated from the lab tests, and employing Figure 5.2-14, β values can be developed for tests L-2 and L-4. β for test L-2 was evaluated as 0.11 and for Test L-4, β = 0.06. The β values just determined are for a mass factor of 0.14. The mass factor is estimated as 0.14 for all RQD values less than 60 (Figure 5.2-16). Because the β values for RQD values between 0 and 60 would be evaluated in the range of 0 to approximately 0.5 (Figure 5.2-15), representation on the β side resistance reduction factor correlation, (Figure 5.2-15) has little meaning for mass factors of 0.14.

Figure 5.2-18 shows the range of β values if the β reduction factor was correlated to RQD instead of mass factor. (Figure 5.2-18 used the correlation of Figure 5.2-16 and the center β curve of Figure 5.2-15). As shown, a plot of the β side resistance reduction factor against RQD gives much more scatter but gives a larger range for rock quality. The scatter is primarily due to the range of values shown in Figure 5.2-16. With more data, (along with using RQD - mass factor correlations for the particular rock being analysed), a better defined trend would be substantiated.

The β values as derived here by themselves have credibility but due to the limited data, extension of this data to other sites may be difficult. It should be noted that Williams' data is for mudstone and may not be applicable to coquina limerocks. As with the end bearing data the analysis for this frictional data should be used for this particular site. The frictional data should not be directly applied to other sites unless the site conditions are very similar. Even with similar rock conditions additional field data would be recommended.

Peak side shear was found to occur at 0.2 inches for Load Test L-2 and at 0.25 inches for Test L-4 which are consistant with published movements to develop peak shear.

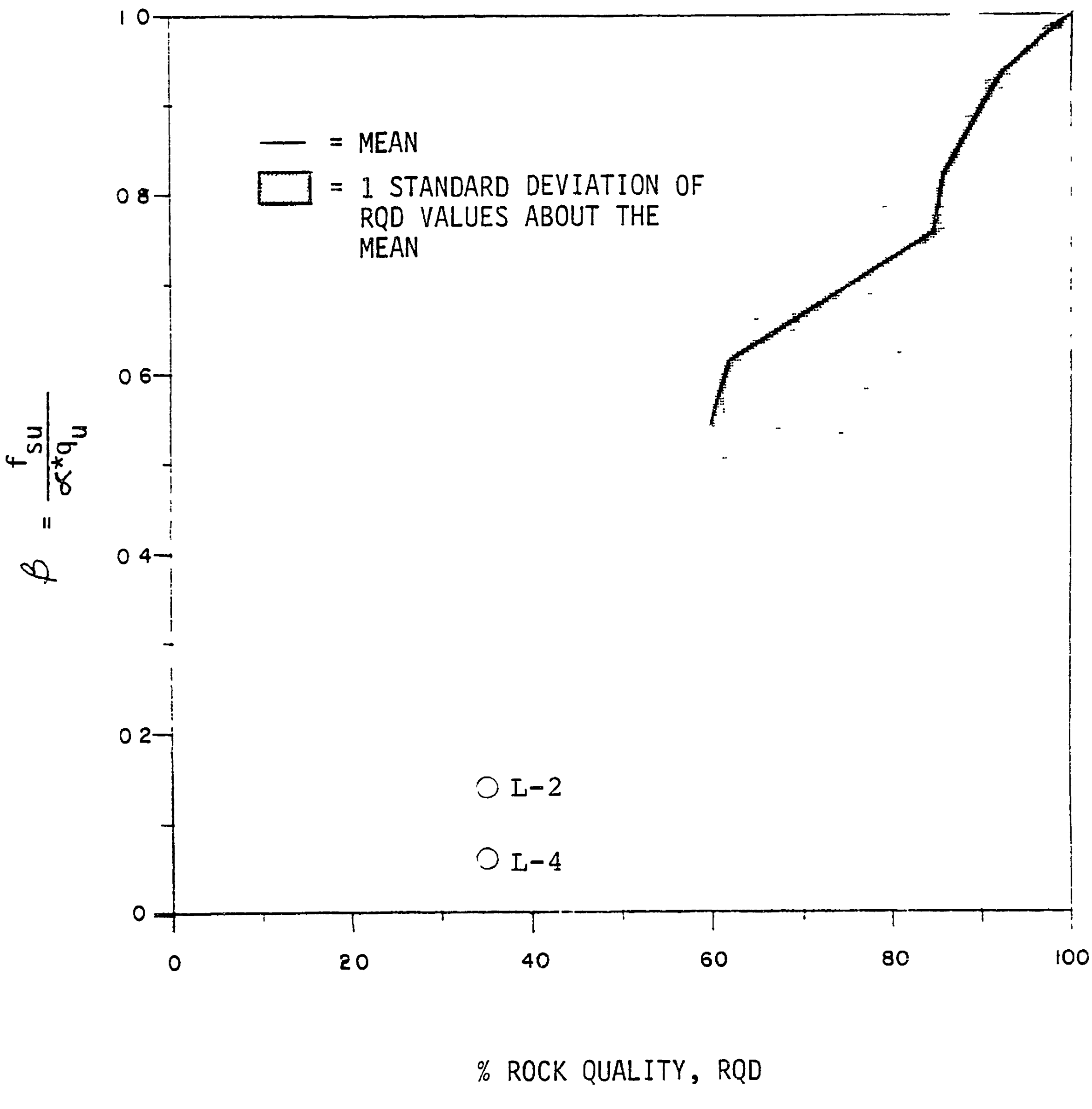


FIGURE 5.2-18 β REDUCTION FACTOR - RQD CORRELATION (DATA ADAPTED FROM DEERE et.al. 1967, AND WILLIAMS AND PELLS 1981)

Conclusions for Site L - The drilled shafts of Site L show the large end bearing load carrying ability of drilled shafts in South Florida. At the maximum applied load of 1260 tons Test Shaft L-2 carried approximately 76% of the load in end bearing. Test Shaft L-4 carried approximately 85% of the load in end bearing. Behavior of the subgrade became plastic for top loads of 920 tons and 675 tons respectively. From the evaluation it is shown that 80 tsf can be resisted through elastic deformation in end bearing with less than 0.9% of tip strain.

Test shaft L-1 has shown this same site can carry 1260 tons with virtually no tip movement. This variability in rock characteristics as shown through the range of RQD values in Figure 5.2-17 is very important with respect to differential settlement between shafts. Shaft L-1 implys that differential settlement can equal the tip settlement developed from other shafts to support their design load. This variable behavior may limit the design settlement of any one pier to the maximum differential settlement allowed for the foundation.

With the curves shown in Figure 5.2-11 and Figure 5.2-13 production shafts can be designed. The tip deflection must be larger than 0.25 inch to mobilize the peak side resistance as shown in Figure 5.2-13. The desired factor of safety used for end bearing and side friction should reflect the variability of rock quality Figure 5.2-17 and are suggested in Chapter 6.

For sites of this nature, borings at each shaft location may help one substantiate a lower factor of safety. The cost effectiveness of this would have to be evaluated. The cost of boring 100 ft would be compared to the additional socket length required for a higher factor of safety.

5.3 Evaluation of Load Test S-1

Test S-1 was founded in the Anastasia Formation as shown in Figure 5.3-1. Six telltales were employed for instrumentation and were similar in construction to those suggested by Reese and Hudson in Appendix A-4. The calipered socket diameter is 30.5 inches and the shaft evaluated concrete modulus is 2.84×10^5 tsf. The evaluated concrete modulus is derived and expained in Figures 5.3-2 and 5.3-3. Figure 5.3-3 gave an excellent elastic response and its value for the concrete modulus is used in this analysis instead of the cylinder breaks. This evaluated modulus is 81.4% that of the cylinder breaks. Authors have published results for in-situ modulus averaging 80% that of cylinder breaks (Holtz and Baker 1972), and results for insitu concrete modulus being larger than cylinder breaks (Pells et al 1980).

The load distribution curves were evaluated from the telltale data. Since no movement was observed at telltale No. 5, this load was taken as 0 tons and the curves shown in Figure 5.3-4 were evaluated using elastic theory as expained in Section 4.3

From the evaluated load distribution curves, load transfer curves were derived as shown in Figure 5.3-5. As shown in Figure 5.3-5, the load transfer curve for a depth of 12.75 ft develops a large load transfer (in excess of 25 tsf). This curve may not be a true representation of side resistance. Since the casing is 5.5 inches larger in diameter than the shaft, it is not clear if end bearing is present. If end bearing at the base of the casing was present, this would be incorporated in the frictional resisted load. The low values of side movement gives creadence to this being side resistance, although one must also consider that the RQD is 90 to 95. The other two depths shown are solely side resistance and give additional credibility to the load distribution curve at 12.75 feet.

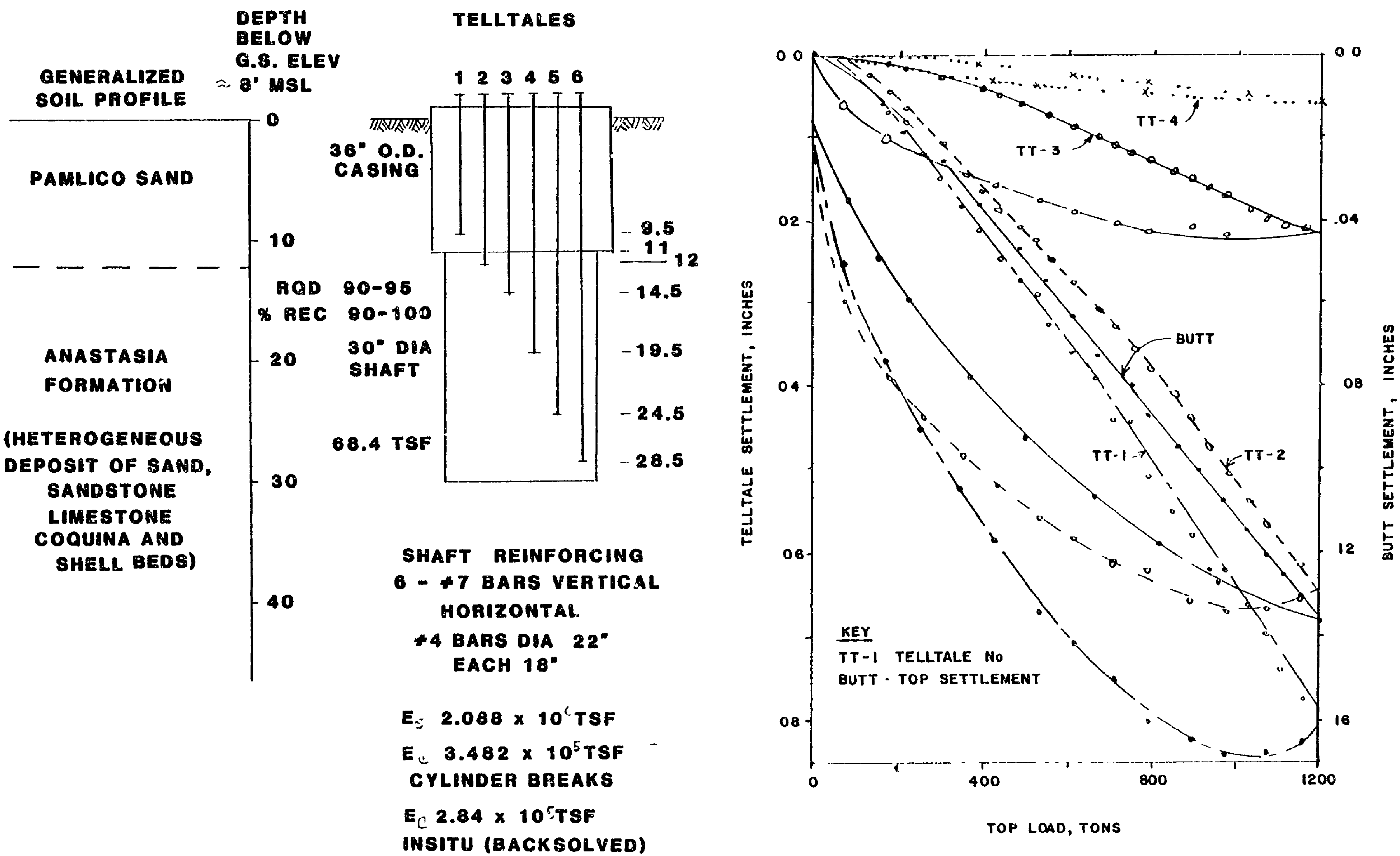


FIGURE 5.3-1 TEST SHAFT S-1
(DATA AFTER GUPTON et. al. 1982)

As found through analysing cased shafts, frictional resistance for the cased portion, (when the hole is overdrilled or the casing is lubricated with slurry), is almost neglible particularly with the larger loads. The estimate of the in-situ concrete modulus can be evaluated from telltale data within the cased portion. This is completed through knowing the steel and concrete cross-sectional area, steel modulus, load applied at the butt, and the difference in butt-telltale settlement. Having this information one then solves for the concrete modulus using elastic theory. The concrete strain is then estimated and a stress strain curve can be drawn as shown in Figure 5.3-3. Since the butt load is assumed as a constant load throughout the cased length, this procedure would give an upper bound for the concrete modulus.

From Elastic Theory:

$$E_c = \left(\frac{PL}{\delta e} - A_s E_s\right) / A_c$$

Also through elastic theory the load on the concrete can be solved by:

$$P_c = \left(\frac{A_c E_c}{A_s E_s} P_t\right) / \left(1 + \frac{A_c E_c}{A_s E_s}\right)$$

Figure 5.3-2: Process of Evaluating In-Situ Concrete Modulus

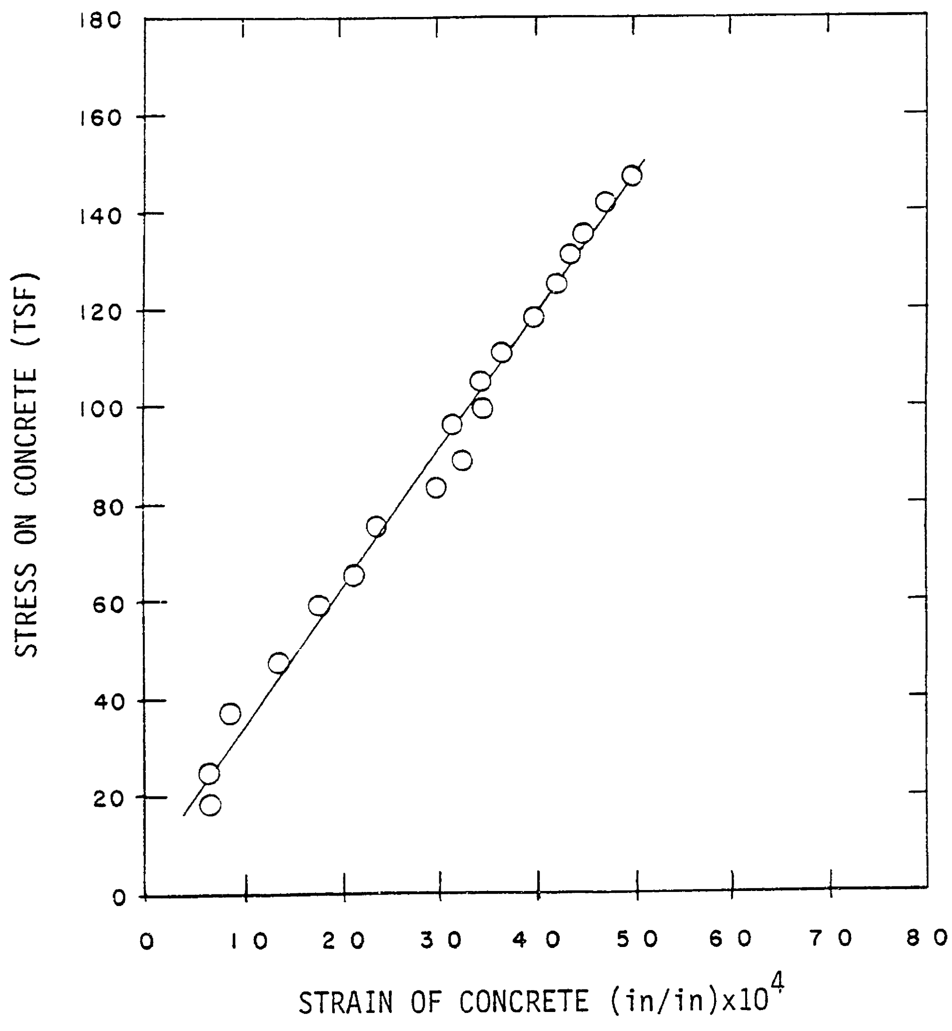


FIGURE 5.3-3 IN-SITU CONCRETE STRESS - STRAIN CURVE FOR LOAD TEST S-1

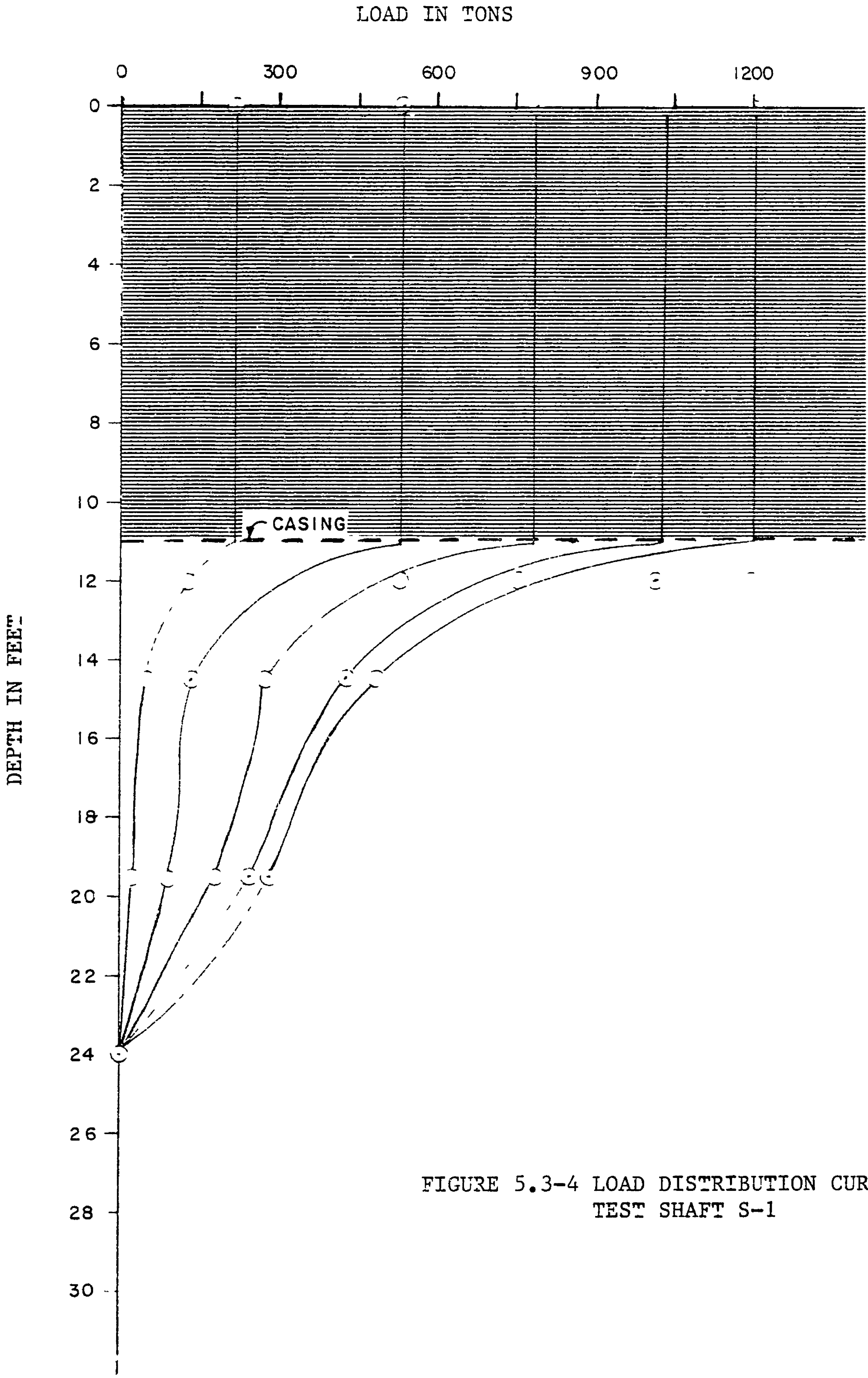


FIGURE 5.3-4 LOAD DISTRIBUTION CURVES FOR TEST SHAFT S-1

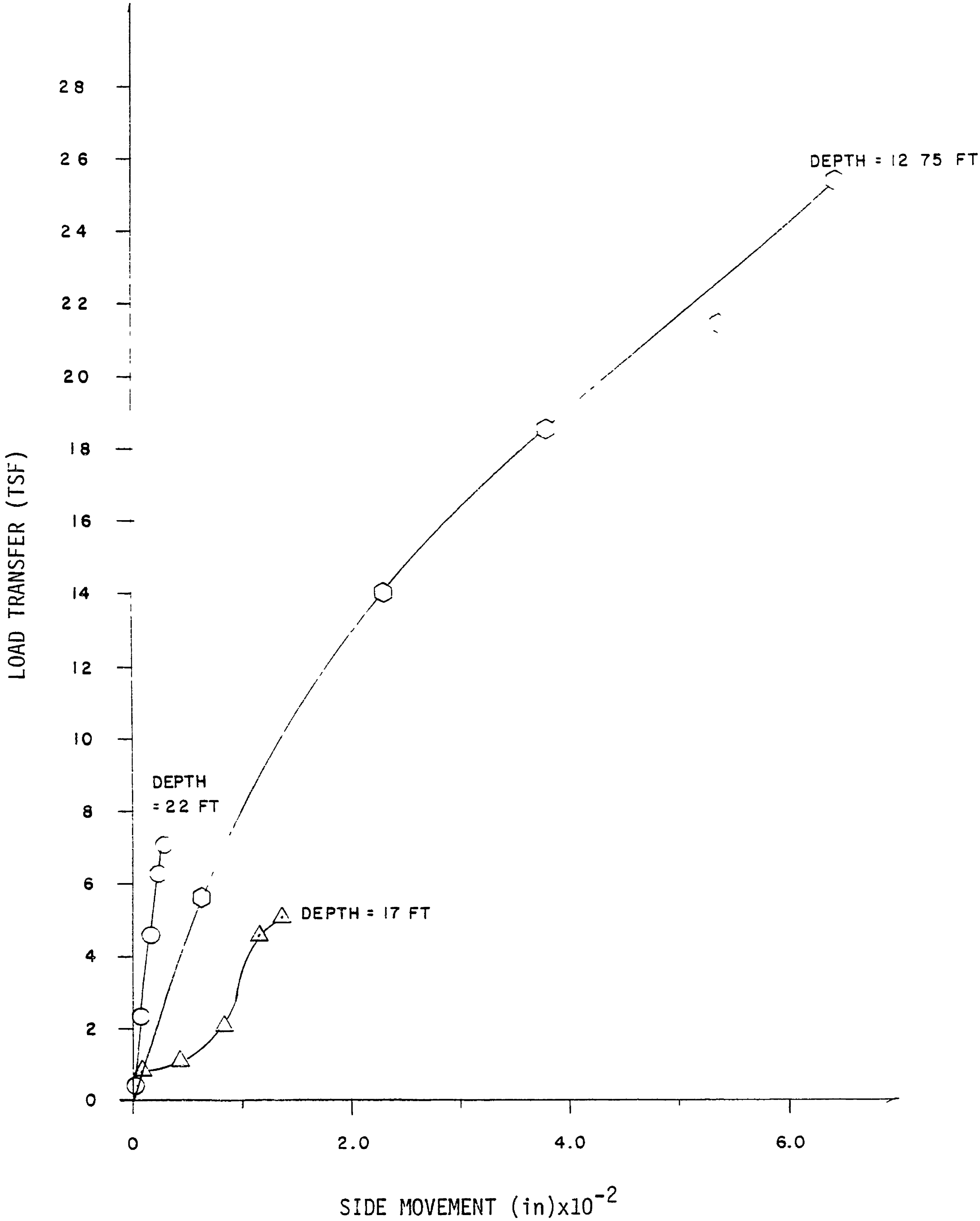


FIGURE 5.3-5 LOAD TRANSFER CURVES FOR LOAD TEST S-1

Using Williams and Pells (1981) α side resistance factor ($\alpha = 0.16$) peak side resistance using ($\beta = 1.0$ which is applicable for RQD = 95), would give a peak side resistance of 10.9 tsf. This evaluated value is much lower than shown at the 12.75 depth load transfer. The precentage of possible end bearing in the load transfer curve could not be determined from the data shown.

This possible end bearing ambiguity at the base of the casing might be avoided in future load tests by attaching a doughnut shaped polystyrene to the reinforcing bars during construction. This could be intended to rest on the inside of the casing where end bearing might develop.

Conclusions - Since the load transfer curves did not develop a peak side resistance, the peak side resistance could not be determined. Shafts would be designed using a derived "average" load transfer curve from those developed. An appropriate factor of safety to apply to this "average" curve would have to be determined taking all aspects into consideration. Any factor of safety would be a lower bound since the true ultimate side resistance was not evaluated through this load test.

5.4 Evaluation of Load Test SE-1

Test Shaft SE-1 was located in the upper Fort Thompson Formation as shown in Figure 5.4-1. This shaft was monitored by Mustran Cells (Appendix A-4), obtained through the University of Texas at Austin. The cells worked properly and the load distribution curves as shown in Figure 5.4-2 were developed from the load cell data. In shaft calibration of these cells prevents any variations due to different estimates of concrete modulus.

The socket diameter was calipered and was determined to be 33.5 inches in diameter. With this, and the load distribution curves, load transfer curves were developed as shown in Figure 5.4-3.

As shown in Figure 5.4-3 the shaft was not loaded to failure. Using the α side reduction factor of 0.17, Figure 5.2-14 and an unrealistic β of 1.0 (RQD 57% from 10" O.D. cores) peak side resistance is estimated as 10.7 tsf. Since the load transfer at a depth of 41.5 ft may include end bearing effects due to the casing being larger than the rock socket, the 44 ft depth curve which is not expected to include any end bearing will be used for comparison. The 44 ft load transfer exceeds 11.5 tsf and has not reached its peak value. This suggests an α side reduction factor exceeding 0.19. The scatter in α as shown in Figure 5.2-14 indicates that an α exceeding 0.19 does fall inside the existing scatter. Figure 5.2-14 does not include coquina limestone. Using this figure one assumed the trend is similar in the rock being used.

Conclusion - This test, since it has not reached its peak side resistance, cannot be directly correlated with other results. The curves as shown in Figure 5.4- 3 give a good correlation for one to select a site specific design curve for friction only shafts.

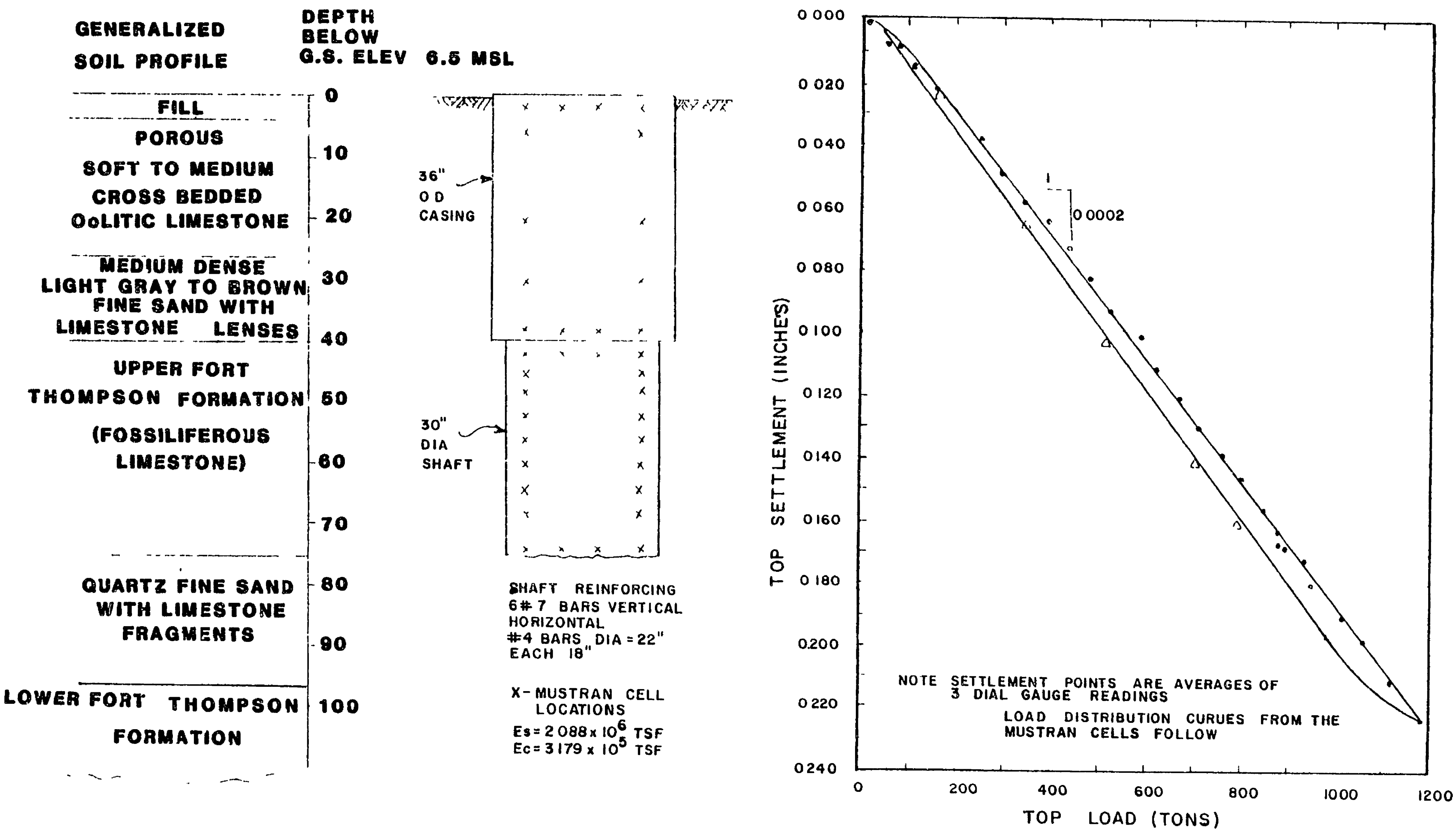


FIGURE 5.4-1 TEST SHAFT SE-1
(DATA AFTER O'BRIEN AND LOGAN 1981)

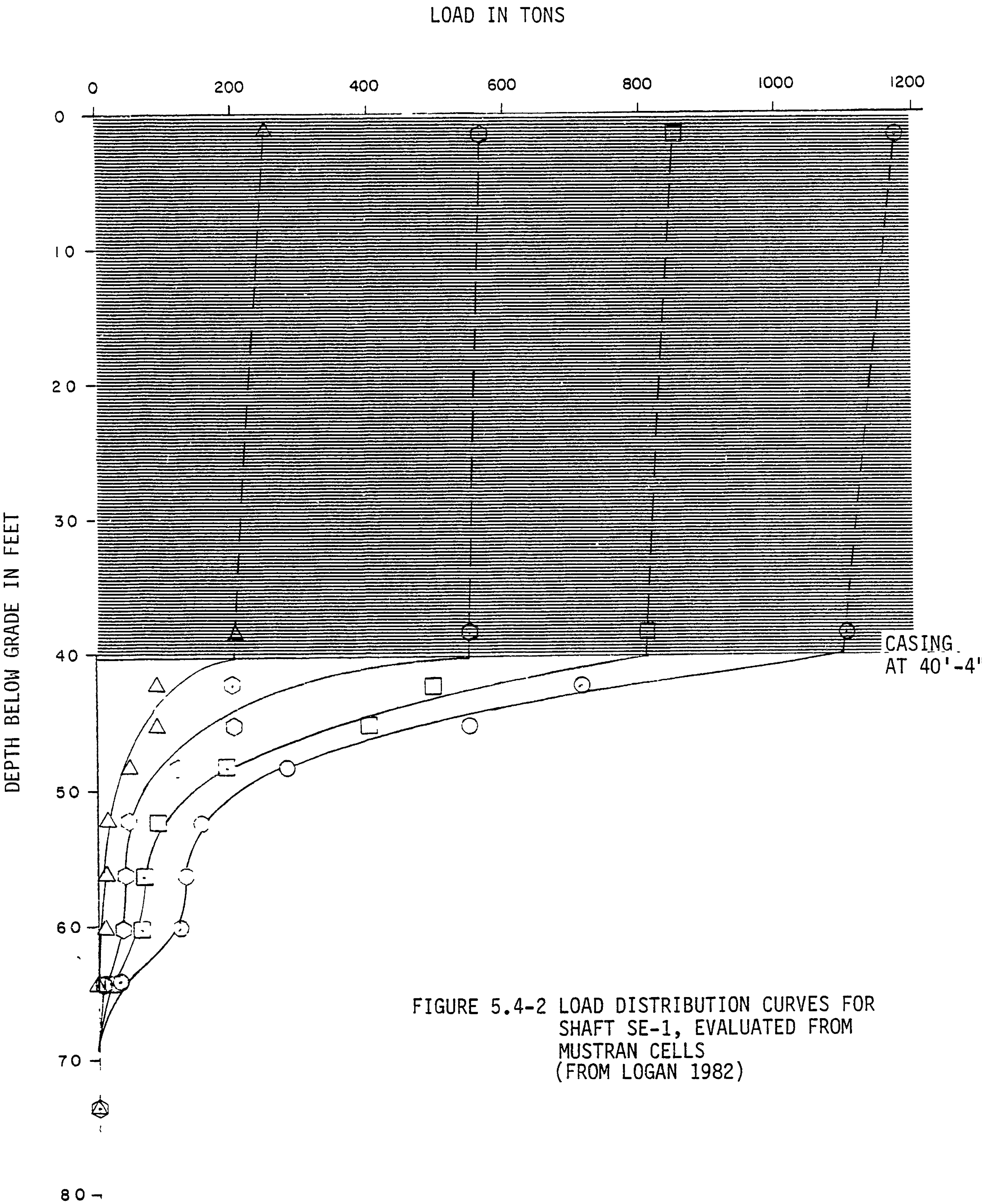


FIGURE 5.4-2 LOAD DISTRIBUTION CURVES FOR SHAFT SE-1, EVALUATED FROM MUSTRAN CELLS (FROM LOGAN 1982)

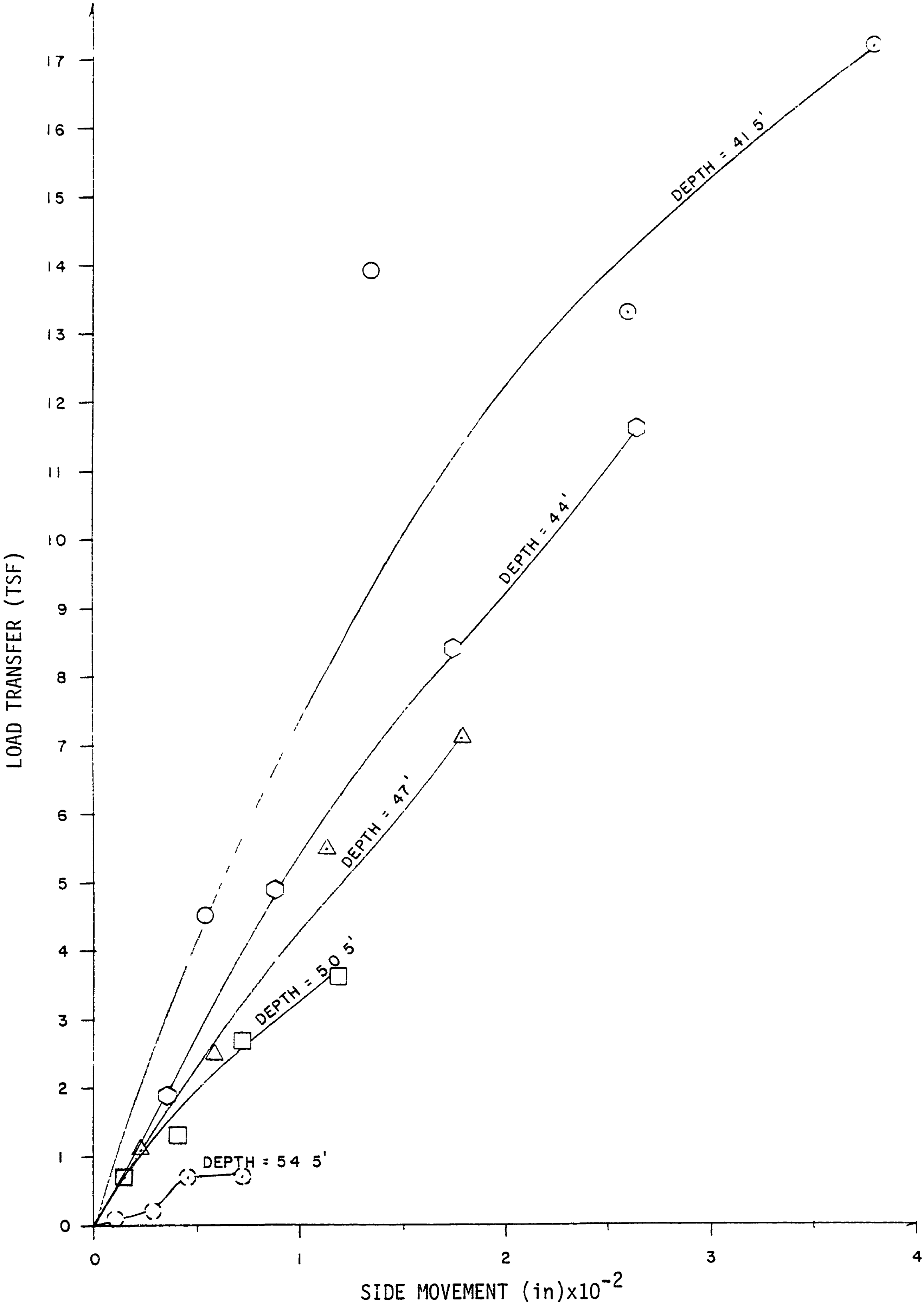


FIGURE 5.4-3 LOAD TRANSFER CURVES FOR SHAFT SE-1

5.5 Load Test Evaluations for Site I

As shown in Figures 5.5-1 and 5.5-4 load tests I-1 and I-2 were instrumented with Mustran cells, (from the University of Texas at Austin), and the shafts were not cased. The evaluated load distributions and load transfer curves are shown in Figures 5.5-2, 5.5-3 and Figures 5.5-5 through 5.5-7.

Load Test I-1 did not fail so no ultimate side friction value can be derived. The load transfer curve at 1.5 ft below the limestone for Test I-1 does show a peak value. Since 1.5 ft is only half the shaft diameter into the socket, and a small distance into the rock it is not considered representative of Shaft I-1. Because of this shaft being over designed for the loading system (shaft did not fail), a smaller socket length was employed in test shaft I-2.

The load transfer curves in test I-2 (Figures 5.5-6 and 5.5-7, after Nyman 1980) do show peak frictional values at 0.05 inches of side movement. A superposition of all these curves and the selection of an average curve is shown in Figure 5.5-8. The peak mobilized friction was determined to be 13.5 tsf for a side movement of 0.05 inches. Since 6 inch rock cores were recovered from the site for unconfined compression tests it is reasonable to take $\beta = 1.0$. Using the evaluated unconfined compression results of 69 tsf, α is backsolved as $f_{su}/(q_u * \beta)$; α = 0.196 which is well within the range of scatter shown previously in Figure 5.2-14.

Conclusion - The average curve once derived could be used, with the desired factor of safety to evaluate the size of friction only shafts to support the desired foundation loads. A small amount of end bearing was observed in test shaft I-2. Since the butt movement was very small and the tip movement not measured, no estimation of end bearing could accurately be estimated.

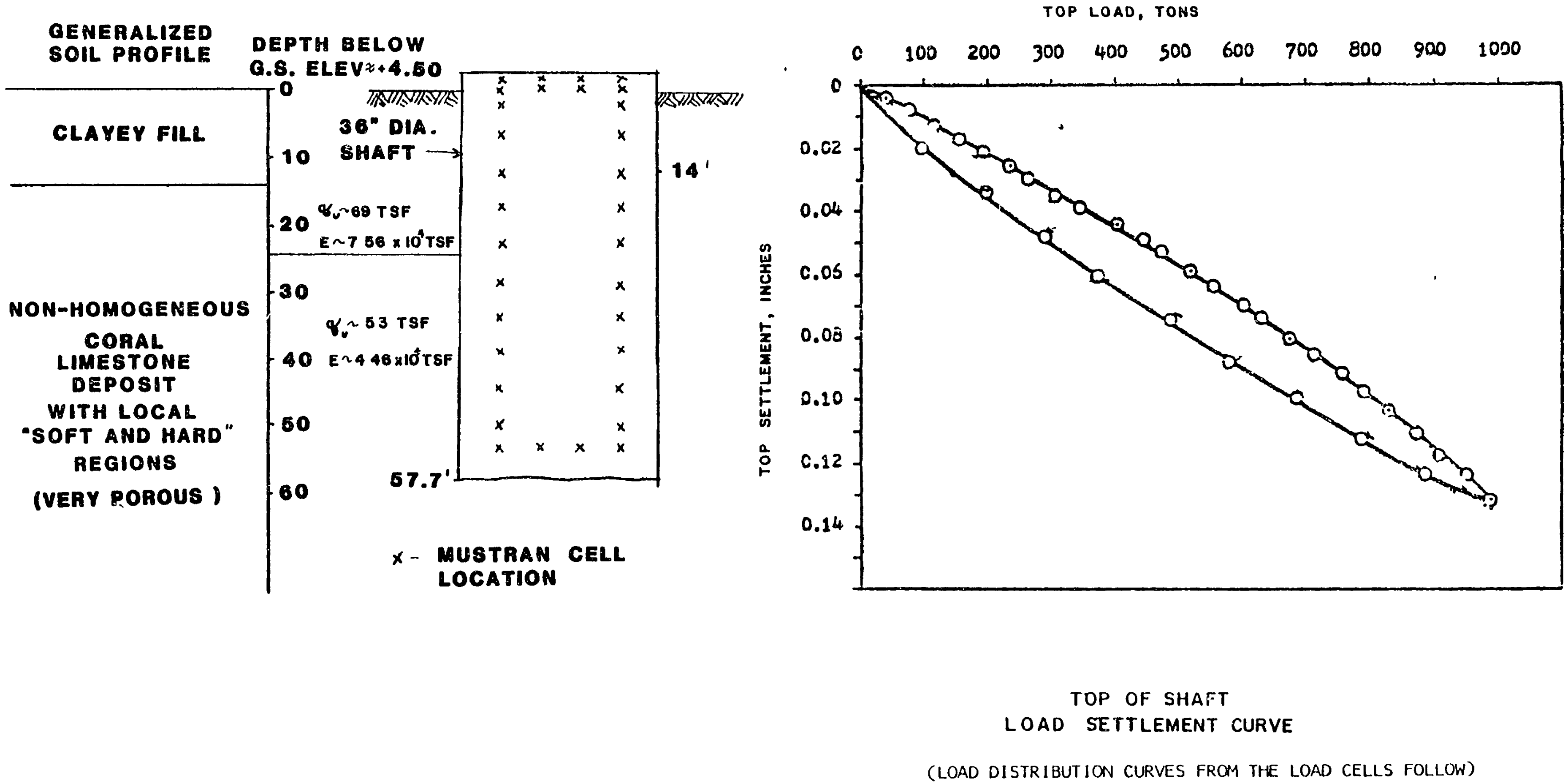


FIGURE 5.5-1 TEST SHAFT I-1
(DATA AFTER NYMAN 1980)

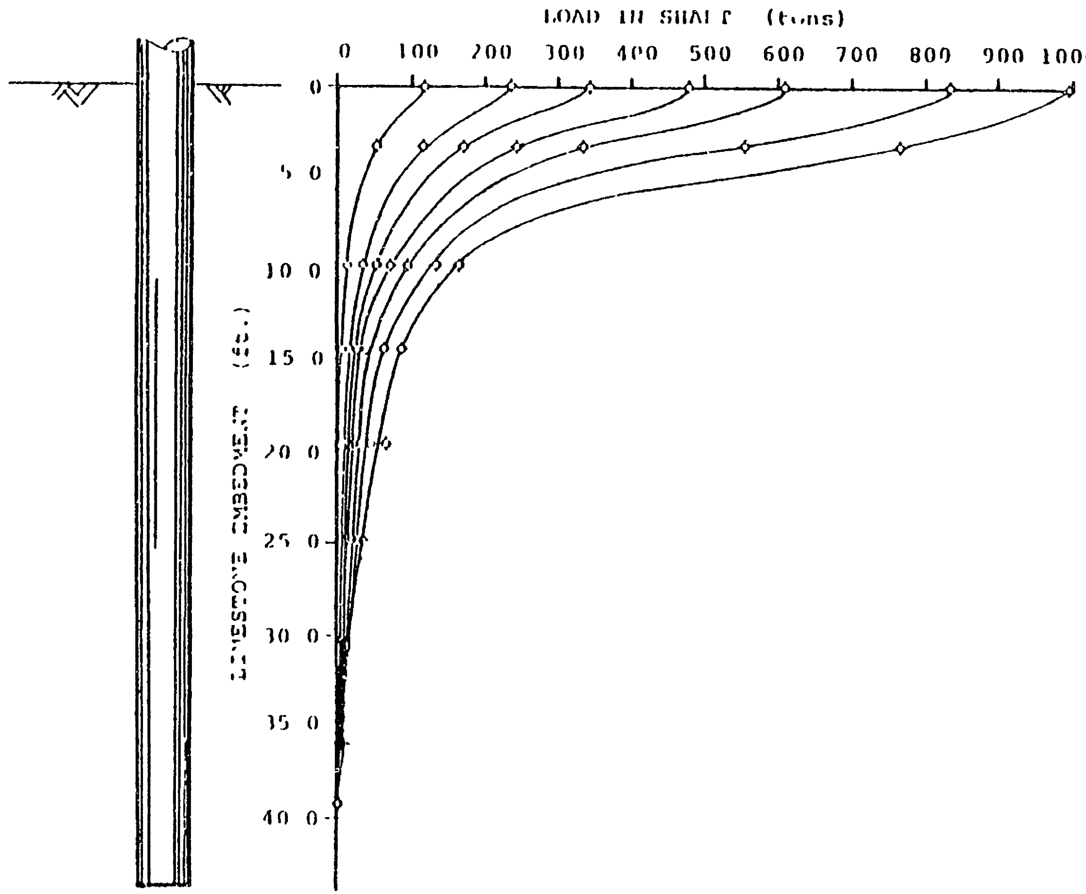


FIGURE 5.5-2 LOAD DISTRIBUTION CURVES FOR TEST SHAFT I-1
(AFTER NYMAN 1980)

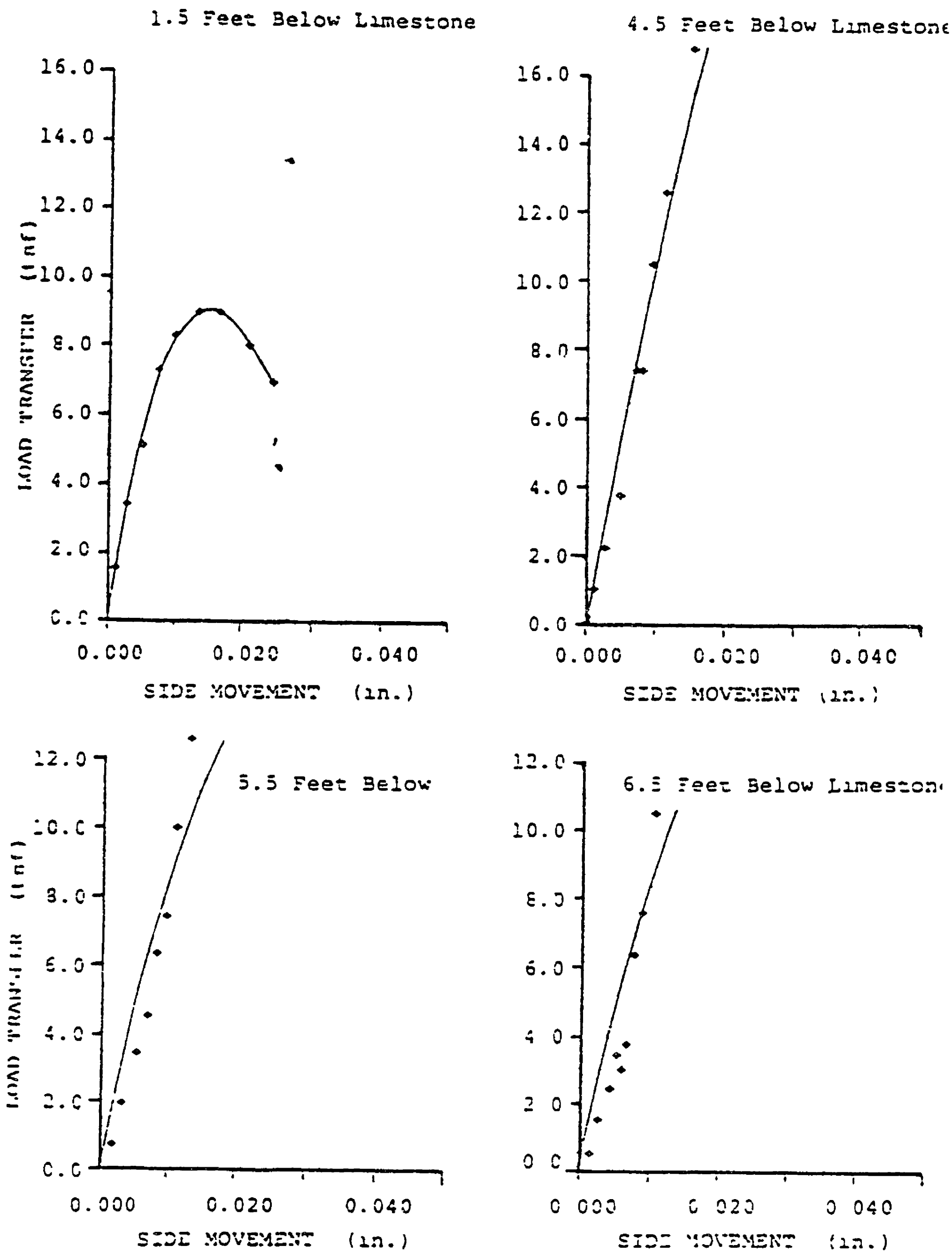
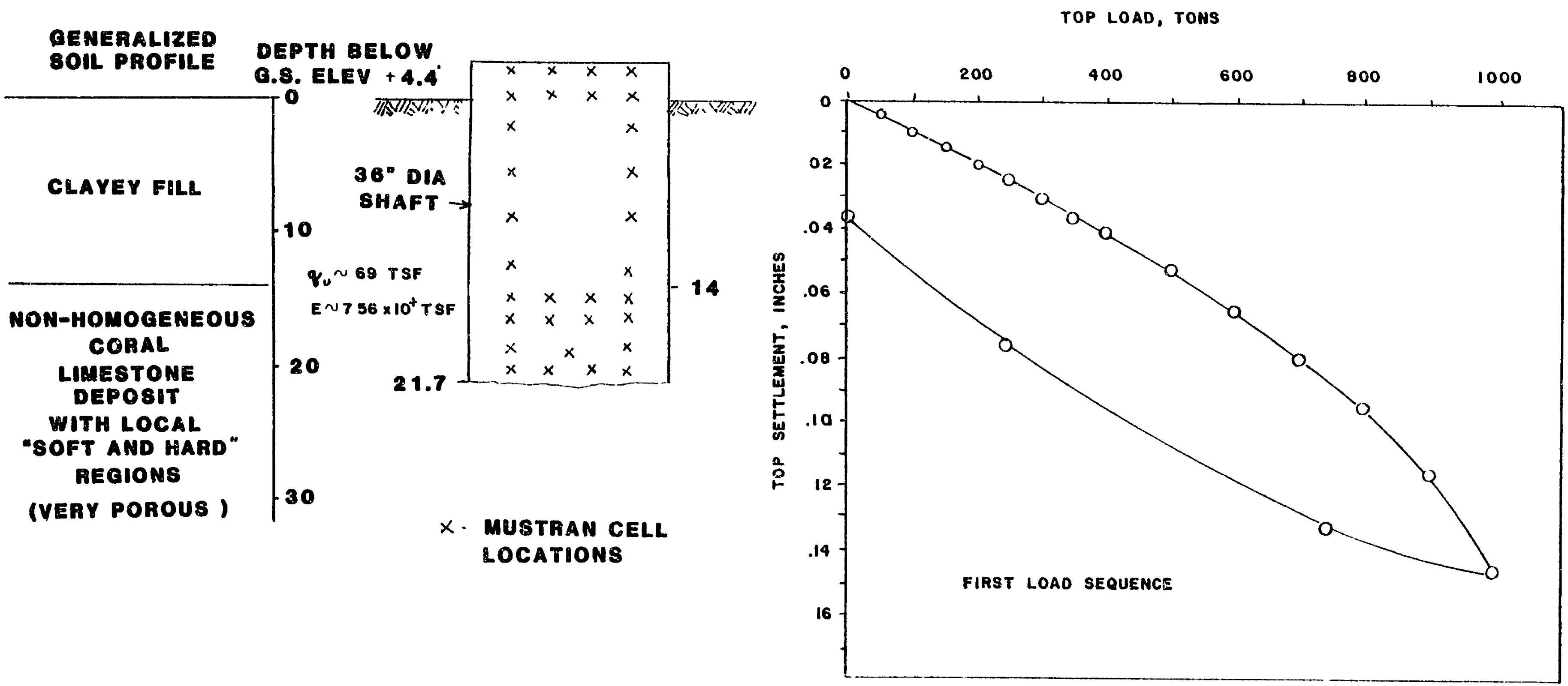


FIGURE 5.5-3 LOAD TRANSFER CURVES FOR TEST SHAFT I-1
(AFTER NYMAN 1980)



TOP OF SHAFT
LOAD SETTLEMENT CURVE

(LOAD DISTRIBUTION CURVES FROM THE LOAD CELLS FOLLOW)

FIGURE 5.5-4 TEST SHAFT I-2
(DATA AFTER NYMAN 1980)

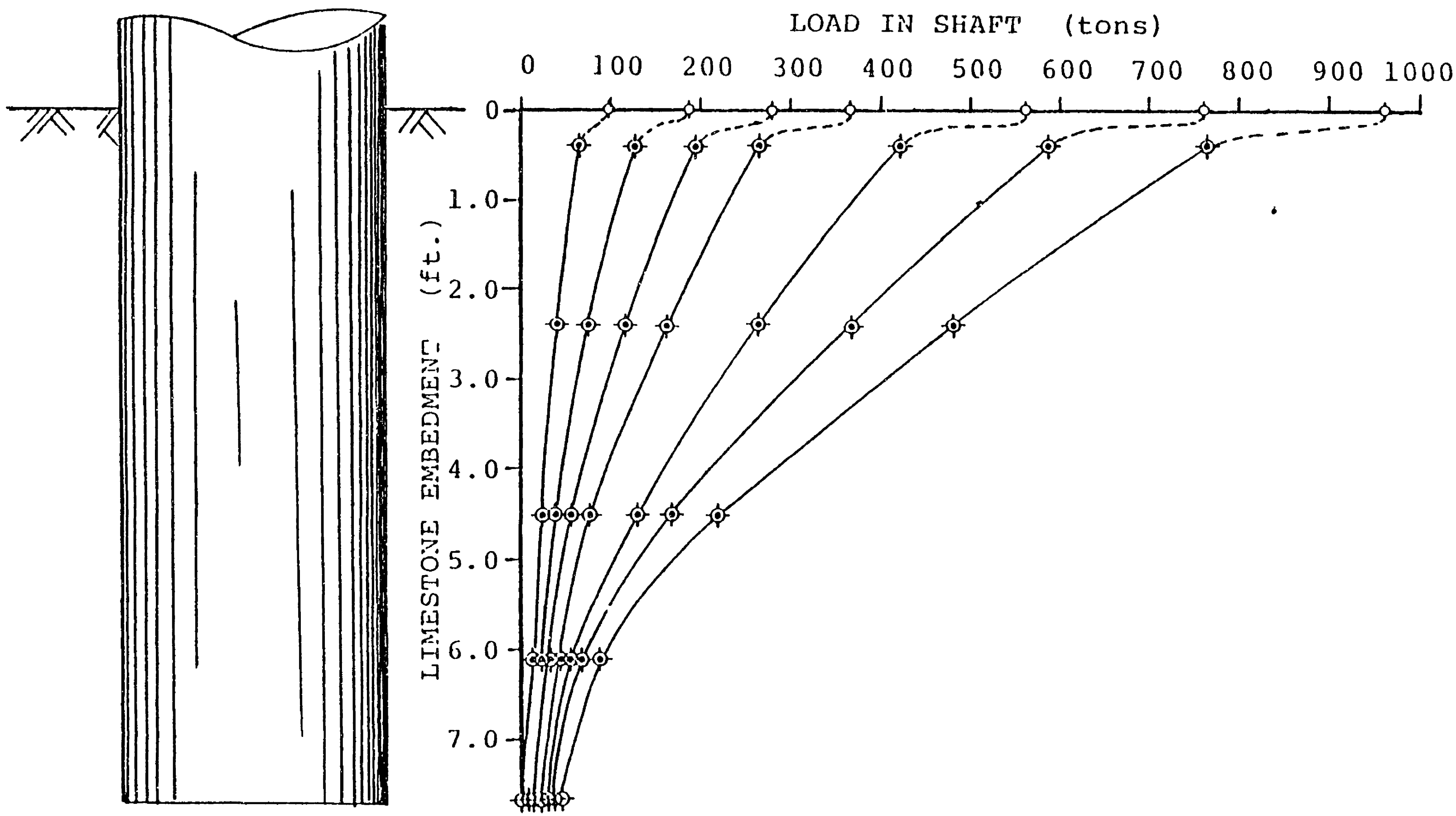


FIGURE 5.5-5 LOAD DISTRIBUTION CURVES FOR TEST SHAFT I-2
(AFTER NYMAN 1980)

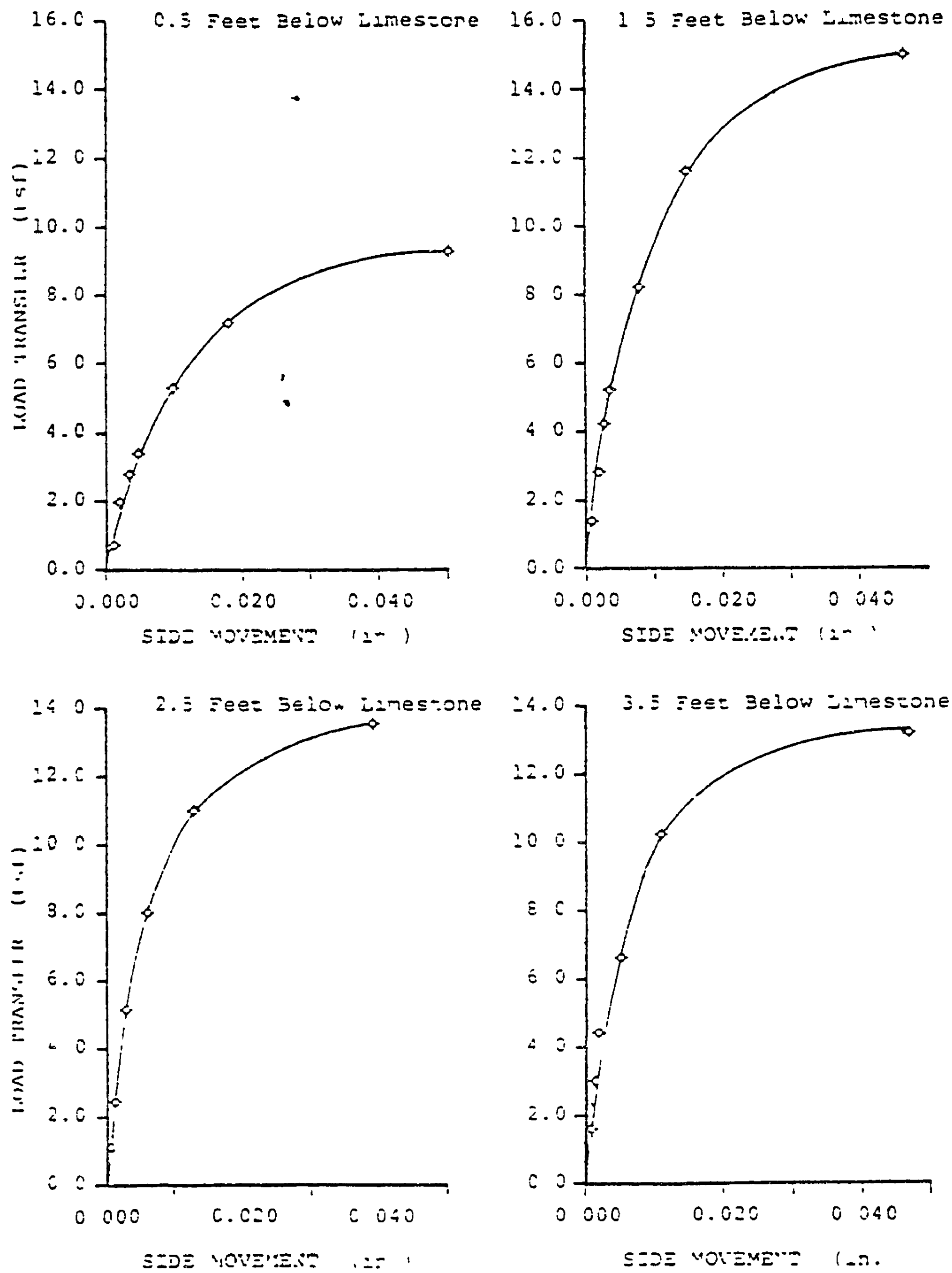


FIGURE 5.5-6 LOAD TRANSFER CURVES FOR TEST SHAFT I-2
(AFTER NYMAN 1980)

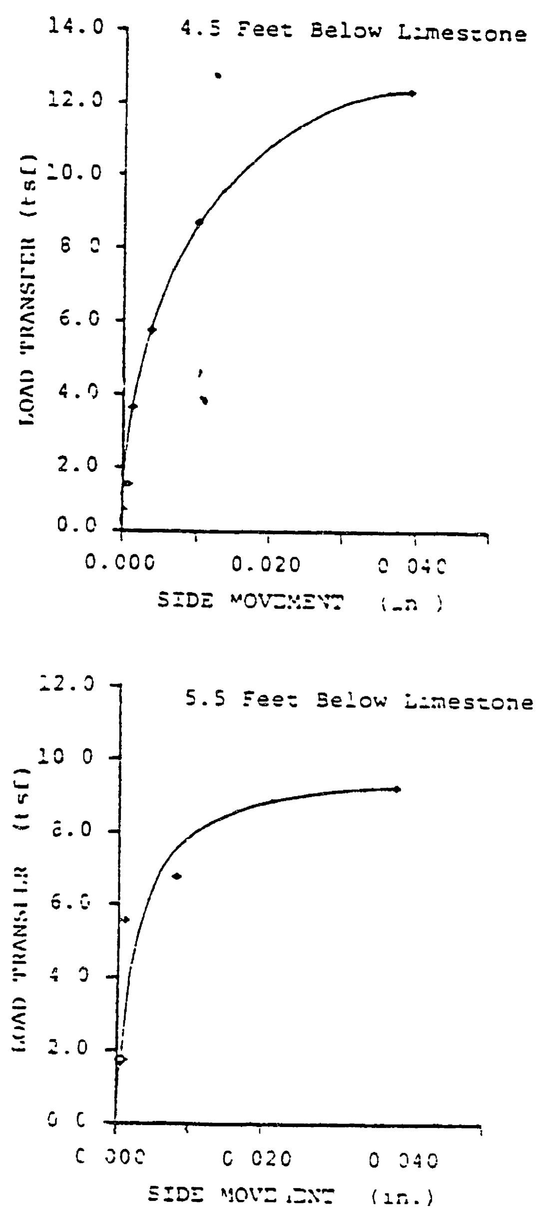


FIGURE 5.5-7 LOAD TRANSFER CURVES FOR TEST SHAFT I-2
(AFTER NYMAN 1980)

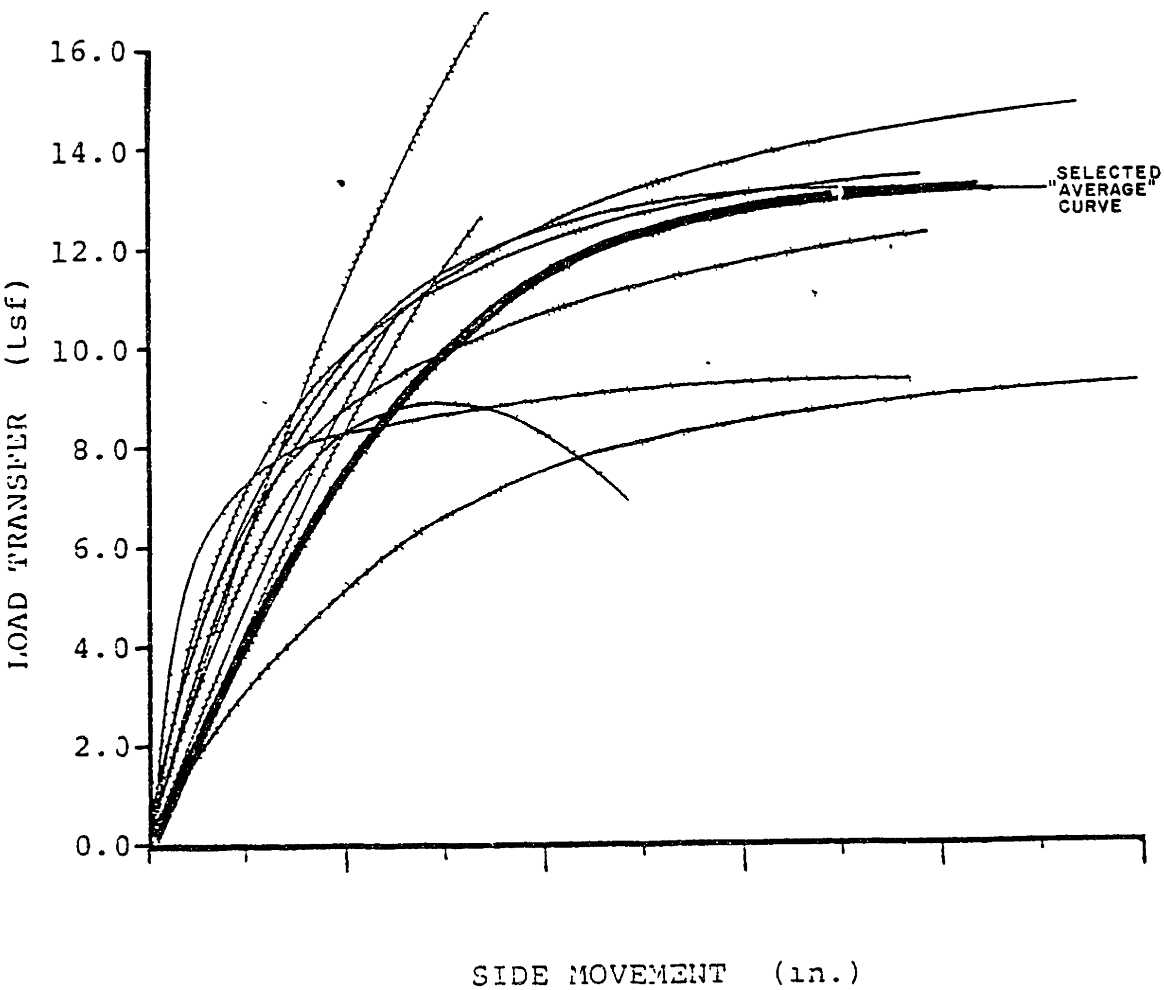


FIGURE 5.5-8 COMPARISON OF LOAD TRANSFER CURVES FOR TEST SHAFTS I-1 AND I-2 (AFTER NYMAN 1980)

5.6 Discussion

As shown through these 4 test sites drilled shafts can be used to successfully support heavy foundation load in The South Florida limerock. Site L has shown the advantages of employing end bearing along with frictional support. Tests S, SE and I developed primarily frictional resistance and from the tests as performed a base resistance curve could not be properly evaluated.

Separate side friction and base load tests can be performed as suggested in Appendix A-5. With separate load tests one could design a combination shaft which may be more cost efficient than friction only shafts. A summary of the field test parameters from these 4 sites are given in Figure 5.6-1. Values shown in Figure 5.6-1 which are not peak values are given in greater than terms.

Figure 5.6-2 shows the frictional results (a reduction factor) for the tests evaluated and compares these to published results from other rocks. The frictional parameters developed in Figure 5.6-2 show a large range of β values. Values from Figure 5.6-2 can be used in a preliminary design along with Figure 5.2-14 to compare costs of possible foundation designs.

Coefficients of subgrade modulus were not found in the literature. Values from load tests L-2 and L-4 indicate moduli in the range of 1300 tcf to 2600 tcf. For site L the peak elastic base strain was 0.9% occurring at a base resistance of 80 tsf.

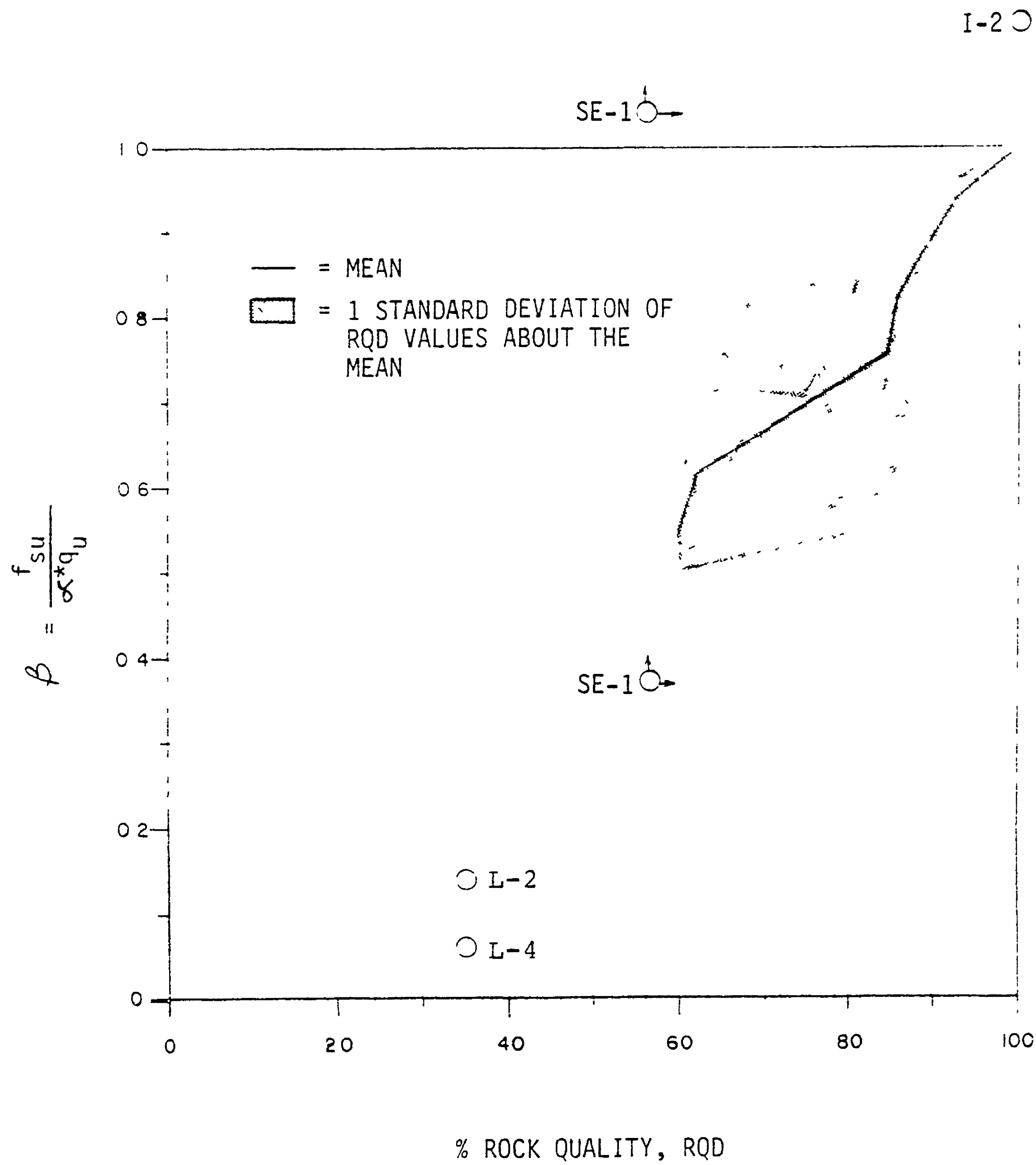
It is important to note that with the performed analysis it was assumed that no adverse construction methods affected the test results. Slurry was used at sites L and I. Due to the natural porosity of the Florida limerock it was reasoned, as explained in Appendix A-1, that the sockets were rough enough so that no frictional reduction would result. Also, in this analysis, it is believed no accumulation of slurry affected end bearing at Site L. The load tests as performed

Test	f_{su} tsf	q_u tsf	α Figure 5.2-14	f_{su}/q_u	β Evaluated	RQD
L-2	3.23	262	0.110	0.012	0.11	35
L-4	1.76	242	0.115	0.0073	0.06	35
S-1	>25.4?	68	0.165	>0.371	>2.25	93
SE-1	>11.6	62	0.187	>0.187	>1.04	>57
SE-1	> 3.6	49	0.074	>0.073	>0.37	>57
I-2	13.5	69	0.170	0.196	1.15	100

COEFFICIENTS OF SUBGRADE MODULUS

TEST L-2 1340 TCF
TEST L-4 2630 TCF

Figure 5.6-1: Summary of Field Test Parameters



$f_{su} = \beta \alpha \cdot q_u$ α - FROM FIGURE 5.2-14

FIGURE 5.6-2 EVALUATED β VALUES FROM LOAD TESTS (ADDITIONAL DATA ADAPTED FROM DEERE et. al., 1967, AND WILLIAMS AND PELLS 1981) serve the function of aiding the geotechnical engineer in designing an adequate drilled shaft foundation. It is hoped that through continued load tests enough data will be collected so that α and β curves, especially for South Florida limerock, can be better defined. Once enough data is accumulated one can look back at the values in Figure 5.6-1 and possibly comment on construction methods (slurry, casing etc.) affecting developed side friction in The South Florida limerock.

6. FACTOR OF SAFETY

The factor of safety is a number which the calculated maximum load is divided by to obtain the design load for the production shafts. The factor of safety is determined using experience and engineering judgement. The factors which one must consider when choosing an appropriate factor of safety are:

1. The variability of the rock found throughout the site.
2. Drilled shaft construction procedures, contractor expertise, and inspection procedures.
3. Method of obtaining and evaluating the design parameters.
4. Any simplifying assumptions made in the analysis.

Williams et al (1980), suggests for sound rock to use a factor of safety of 2 with respect to the design settlement (different definition of factor of safety = settlement allowable/settlement predicted), and a factor of safety of 3 with respect to total capacity (bearing capacity).

Gill (1980), suggests to use a factor of safety of 3 if employing end bearing resistance only, and a factor of safety of 2 to 2.5 if both side friction and end bearing are used in the analysis.

Webb and Davis (1980), suggests a varying factor of safety. They suggest using a factor of safety of 10 for shafts whose rock socket has a length to diameter ratio of 1. This suggested factor of safety is proportionately reduced to 3 for shafts whose length to diameter ratio exceeds 3.

Nyman (1980) suggests employing a factor of safety of 3 to side friction shafts in coral limerock.

Due to the variability in some of the limerock formations, (particularly the Fort Thompson Formation), it is suggested through this study to use a minimum factor of safety of 3 on the determined average peak side resistance. Factor of safety in excess if 3 is suggested for end bearing settlement where RQD values are less than 100%. The reason for suggesting a higher factor of safety for end bearing is due to the solution nature of the limerock. If the base of a shaft is probed to insure competency, a lower factor of safety of 3 may be justified. Depending on possible group effects (Chapter 7) a higher factor of safety or a lower design settlement might be chosen.

The factors of safety as suggested are to relate representative load tests to production shafts for the same site. As shown in Site L rock quality within the foundation limits can vary. The variability of the rock can ususally be observed through borings (RQD values). If numerous borings are taken within the foundatin area and all show the same range of RQD values, a single representative load test may be applicable. If the borings show the founding rock varies, as found in Site L, more than 1 load test should be performed and a higher factor of safety should be considered. The load test, where possible, should be founded in the poorest rock that will be used for the foundation.

7. GROUP EFFECTS

Shaft group effects should definitely be considered and estimated for a completed design. The added effect of having sand layers beneath the founding rock layer of the drilled shaft at some of the sites in Florida, (The Fort Thompson Formation in particular), necessitates an engineer to consider group action. Group effects on drilled shafts have not been documented in the literature but, group effects with respect to piles have been well documented and can be extended to drilled shafts. Pile group behavior is suggested to be extended to drilled shafts when the behavior is derived using elastic theory.

The process of group interaction using elastic theory employs shaft interaction factors. These interaction factors defined by the adjacent shaft size, length, load, founding and shaft materials, are used to estimate the added settlement due to group behavior. Poulos and Davis (1980) have stated that the method of super-position is applicable with these individual interaction factors, but have noted that as the number of shafts increase the larger proportion of the design load is transferred to the shafts base.

The various possibilities of group geometry along with different shaft sizes prevents a simple summation of group behavior for the shafts presented in this report. Poulos and Davis (1980), in their text give an excellent presentation on using elastic theory to predict group effeciency. Their analysis presents many design curves and charts developed through employing elastic theory, and also gives individual interaction factors to estimate a non-symmetrical group efficiency using the principle of super-position. Some assumptions with respect to an equivalent modulus would have to be made when applying Poulos and Davis solutions to the Florida substratum. The reader is referred to Chapter 6 in their text, (Poulos and

Davis, 1980), for a complete presentation. This elastic analysis would only be an estimate in absence of drilled shaft group load tests, which are normally cost prohibited.

8. CONCLUSIONS

1. Through the load tests presented, high capacity drilled shafts are shown to be applicable and economic in the support of heavy foundations for the South Florida area.

2. End bearing can be incorporated into the drilled shaft design. Load tests L-2 and L-4 show that significant end bearing can develop in Floridian Limestones. Due to possible cavities in the limestones beneath the drilled shaft the decision to incorporate end bearing would be site specific, and decided by the Geotechnical Engineer after careful examination of the boring logs.

3. If the load test is taken to failure, average emperical load transfer factors can be evaluated for the site. With these parameters an estimated factor of safety can be stated for a given design. It is hoped once enough of these α and β side resistance reduction factors are correlated to the physical limerock parameters, (q_u, E, and RQD), as shown in Figure 5.6-1, a trend will develop and future field load testing minimized.

   For a first estimate of the load carrying capabilities of drilled shafts in Florida Limestone the frictional and end bearing parameters developed in Figures 5.6-1 and 5.6-2 can be used as mentioned in Section 5.6.

4. No residual side resistance was found in any of the load tests performed. The peak side resistance remained relatively constant through side movements exceeding 1% of the shaft diameter. This was shown in load tests L-2 and L-4, a similar trend was shown in test I-2.

5. Telltales and load cells have both accurately monitored field load tests. Load cells were found to be more applicable than telltales for short sockets where small elastic deformations must be measured.

6. Depending on the foundation layout, group effects can effect total settlement and should be considered.

9. RECOMMENDATIONS

1. Whenever possible load test each drilled shaft to failure. Osterberg and Gill (1973) have shown through elastic analysis, that diffusion of frictional load shafts typically occur within 1.5 to 3 diameters in length from the top of the rock socket. This should be kept in mind during the design of the test shaft. Separate end bearing and side friction tests are suggested where economics permit. If side friction only shafts are to be tested a method to ensure no end bearing at the base of any casing or at the base of the shaft should be employed. Possible styrofoam or polystyrene plugs to prevent end bearing have been described in Sections 5.3 and A5.2.3. A base plug is suggested to ensure side resistance failure and thereby determine the peak side resistance for the rock being tested. If a base plug is not employed a method to measure the base resistance should be used, (the bottom hole cell is suggested as described in Appendix 4).
2. Employ reliable pre-tested load cells. It is also suggested to use at least 1 vertical telltale extended to the tip of the shaft. The telltale can give valuable results for the load test even if the load cells do not function properly.
3. If telltales are used, telltales extending vertically to supported dial gages are suggested and shown in Figure A4.4.-1. The telltales which are bent horizontally out of the shaft, as shown in Figure A4.4-2, should be avoided due to past conflicting data (Figure 5.2-2, 5.2-4 and 5.2-6).
4. End bearing should be incorpoated into the shaft, the factor of safety for end bearing should exceed 3 and should reflect the soundness of the strata beneath the shaft. Because of the high factors of safety for end bearing, and separate end bearing load tests being required to obtain

good end bearing tip movement curves, ignoring end bearing for small projects at the present time may be more economical.

5. When side friction is evaluated, an average load transfer curve from site load tests along with a factor of safety of 3 is recommended. For very solutioned limerock formations it is suggested to use a higher factor of safety.

6. From all frictional load tests performed evaluate reliable α and β, values. These values should be made publically available so that this rational method for evaluating frictional side resistance can be further refined for the South Florida limestone. Similarly, coefficients of subgrade moduli should be evaluated and published when end bearing tests are performed.

7. Along with load testing, the following lab test results, shaft dimensions, and boring results should be obtained. This additional data aids the engineer in properly extrapolating the load test results to shafts of different dimensions or slightly different ground conditions throughout the site.

   a. Unconfined compression strength modulus and poisson's ratio of the intact rock.

   b. Diameter of the shaft.

   c. RQD and % recovery of the rock tested, (or other suitable in-situ testing parameters).

   d. Length of shaft and rock socket.

   e. Modulus and poisson's ratio of the concrete.

   f. Stress on the shaft.

8. Appropriate borings to ensure the Geotechnical Engineer that no large cavities exist near or beneath the drilled shaft should be performed.

Borings should be performed at each test shaft location and are also suggested to be performed at each production shaft location.

9. Proper construction techniques and constant inspection should be supplied (as described in Appendix 2). If slurry is used, constant monitoring of viscosity and density as well as complete recirculation prior to concreting is suggested.

TABLE OF CONTENTS FOR APPENDICIES

APPENDIX 1

FACTORS AFFECTING SHAFT FRICTION

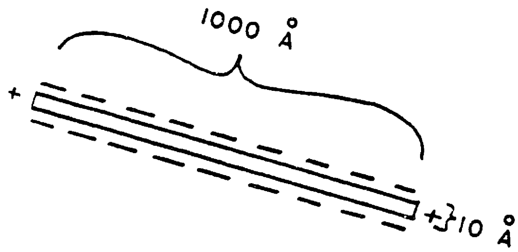
A1.1 THE SLURRY DISPLACEMENT METHOD

General

The effect of drilling fluids on bored drilled shafts has been studied by many investigators; Barker and Reese (1970), Boyes (1972), Reese and Tuma (1972), Tuma and Reese (1972), Pells et. al. (1980), and Williams (1980B, 1981). Slurry, or drilling mud is a three phase water base mud, consisting of a liquid phase, a colloidal fraction, and an inert fraction. The colloidal fraction is the reactive portion of the slurry, and the inert fraction contains weighting material which consists of sands, rock cuttings, and other material.

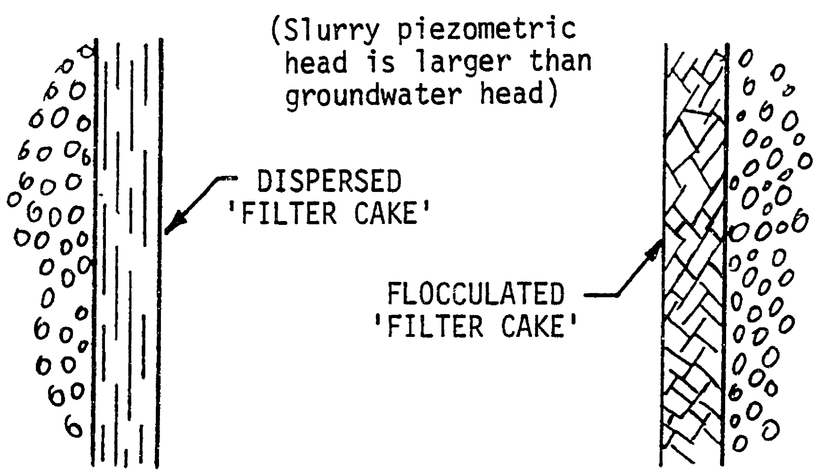
The most common slurry is a bentonite mud. Bentonite is predominately composed of the clay mineral Montmorillonite. As shown in Figure A1.1-1, montmorillonite is a negatively charged particle. Sodium montmorillonite which employs sodium as the cation to bond the flat mica-like layers of the montmorillonite is the most common form of bentonite in use today. Calcium montmorillonite can also be used as a drilling mud. In calcium montmorillonite calcium cations (which are twice as positively charged as sodium), bond the particles in a tighter manner. More water can enter between sodium-bond layers and be absorbed, than with calcium-bond layers. Calcium montmorillonite is called a sub-bentonite.

Sodium montmorillonite forms a stable suspension in fresh water with low solids concentrations. Bentonite is highly thixotropic with respect to applied shear stress and provides a "filter cake" of low permeability at the interface of the bored hole and the soil-rock continuum.



PARTICLE OF MONTMORILLONITE
(a)

(b)



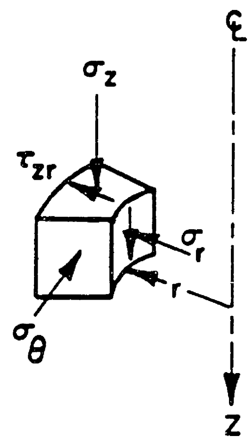
(c)

FIGURE A1.1-1 DISPERSED AND FLOCCULATED SLURRY
(ADAPTED FROM TOUMA AND REESE 1972)

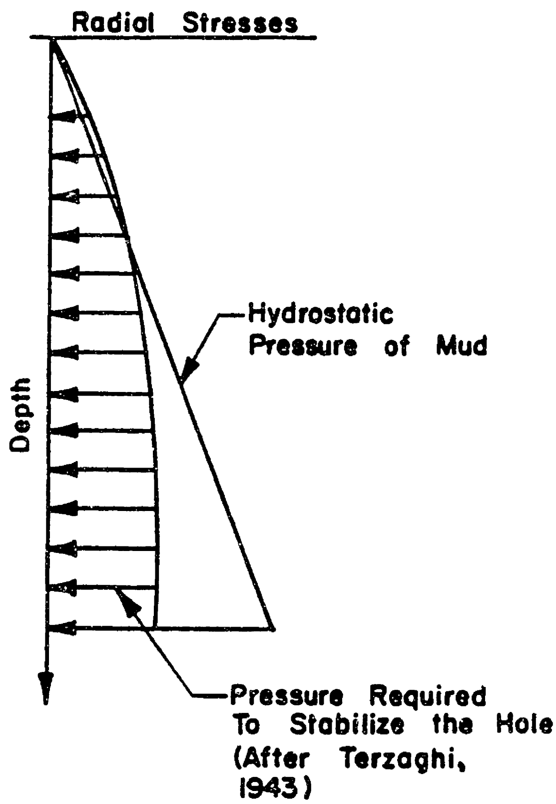
Factors which affect the nature of the "filter cake" are dependent on the slurry composition (type of bentonite, additives, contaminants, and drilled solids), the rock mass properties (permeability and groundwater chemistry), and on construction influences (time, slurry head with respect to groundwater head, and agitation). The purpose of the drilling fluid in drilled shaft construction, is to line the walls of the drilled shaft with a low permeability, dispersed "filter cake", which counters the groundwater piezometric head and soil pressure limiting any inflow of groundwater or soils, thereby stabilizing the shaft. To counter wall pressures, the height of slurry in the bored hole is typically kept above the hydrostatic head of the groundwater, creating an outward gradient for the slurry in the drilled shaft. Relative pressure diagrams are shown in Figure A1.1-2. Pollutants, low Ph contaminants, and salt water can disrupt the charge balance of the bentonite and cause the mud to flocculate creating a bulky pervious wall cake, Figure A1.1-1. Chemical additives can be added in these instances to disperse the mud, but another drilling fluid may be more economical.

Attapulgite clays are very useful and will hydrate to form stable suspensions in salt water. If the salt contamination is not excessive, Touma and Reese (1972) have found that premixing the bentonite in fresh water attains favorable results. Many investigators suggest the Wyoming bentonite, which is a sodium montmorillonite as the best available. Palmer and Holland (1966) suggest using approximately 30 pounds of bentonite per cubic yard of material. This suggested value would fluctuate depending on the past experience of contractors who have worked in the project area.

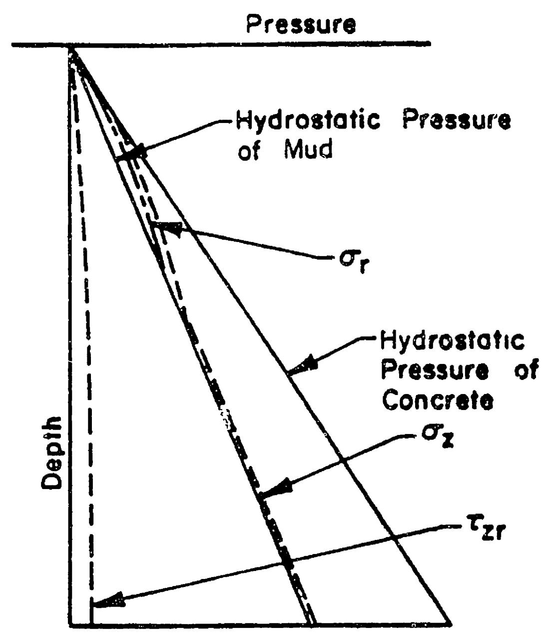
The characteristic of the slurry is to put particles of granular soil, that are not removed with the augers or mud buckets, into suspension so that they can be removed. As the slurry becomes contaminated with soil, it must be replaced or recycled by removing the excess granular material. A contaminated thick, or



a. Stresses on a Radial Element



b. Profile of Radial Stresses



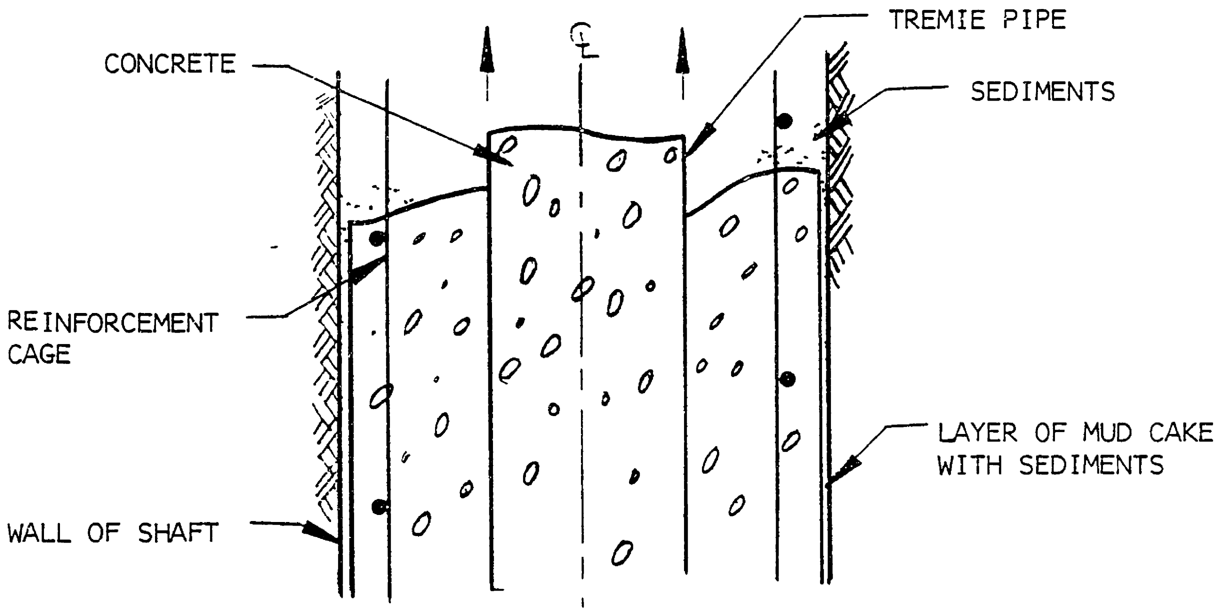
c. Stresses at the Surface of the Shaft when Concreting by Slurry Displacement

FIGURE A1.1-2 STRESSES AT THE RADIUS OF A DRILLED SHAFT EMPLOYING DRILLING MUD (AFTER TOUMA AND REESE 1972) flocculated slurry increases the danger of causing negative pore pressures, (suction), due to auger or bucket withdrawal and possible collapsing of a sand or silt strata into the shaft. All drilling tools employed should allow sufficient space for free flow of slurry as the tool is introduced and withdrawn from the shaft.

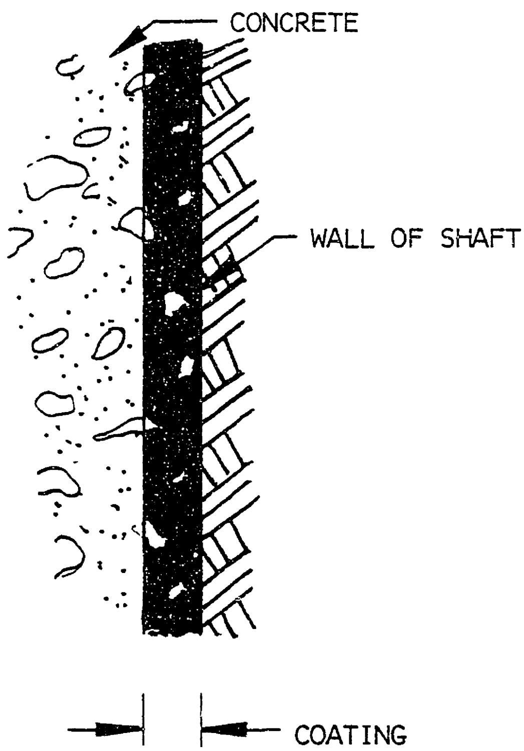
Frictional Aspects

The following 5 step process employing both slurry and casing in the drilled shaft construction is not recommended for friction shafts. 1) Drill the slurry shaft. 2) Enter casing down through the entire length of the shaft. 3) Seal the casing at the base with a plug of tremie concrete. 4) Pump the casing dry. 5) Pour the shaft in the dry while vibrating out the casing (the casing is vibrated out once enough concrte head is in the shaft to balance the base water pressure). From the study of Barker and Reese (1970), this method was determined to have a higher probability of slurry inclusion in the side of the shaft than tremie concrete under slurry. Their study showed an inclusion of slurry when using 1.5 inch clearing between the casing and the soil-continuum. From this study, it was also found that during excavation of the test shaft for inspection, chunks were easily broken from the shaft. These chunks of concrete broke along cleavage planes, leaving a cylinder shape equal in diameter to that of the casing used. This suggested the concrete outside the casing was contaminated.

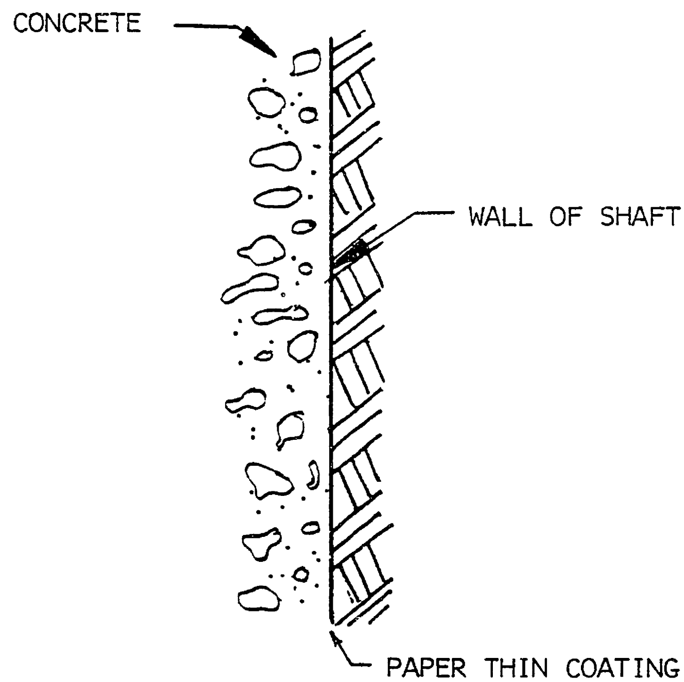
When proper construction techniques are employed drilling mud in connection with tremied concrete will provide adequate load carrying characteristics for drilled shafts (with the exception of smooth rock sockets which are discussed later). The trimie used should be the largest diameter available (usually 6 to 12 inches), so as to create a vigorous scour by wet concrete on the walls of the drilled shaft. A vigorous scour by wet concrete helps prevent mud and sediment entrapment, see Figure A1.1-3. Reese and Touma (1972) suggest a minimum of 5



A. FORMATION OF COATING



B. WITH MUD ENTRAPMENT & SEDIMENT



C. WITHOUT ENTRAPMENT

FIGURE A1.1-3 CONCRETE SOIL INTERFACE

feet of tremie embedment for a 30 inch shaft using a 12 inch tremie. The concrete should be kept as fluid as possible, typical slumps range from 7 to 9 inches. Barker and Reese (1970) determined slurry has no detrimental effects (if employed properly) and may actually be helpful in developing frictional resistance in sands and silts. This was shown through field load tests and lab tests. Reese and Tuma (1972) found the thin coating on the walls of the shaft did not coincide with the failure plane of the test shafts, but rather, the failure surface was found further into the soil continuum. Many authors agree that using drilling buckets instead of flight augers has attained higher side adhesion values.

In rock socketed drilled piers of normal roughness (rock auger excavation), Williams 1980B, (1981), Pells, et. al. (1980), and Fleming and Slwinski (1977), have found bentonite has not significantly affected side resistance. Both Pells and Williams, through numerous field load tests stated on smooth rock sockets, (sandstone and mudstone were tested), one can expect reductions of up to 25% of the adhesion expected from a clean socket. Williams (1981), defined the desired minimum roughness as asperities on the socket wall of 0.4 inch deep by 0.8 inch wide on 4 inch centers. If one has a smoother socket than just suggested, possibly from rock coring, a reduction in side resistance due to bentonite can be expected.

Tremie Process

Concrete quality tremied under bentonite reported from all authors was excellent unless the bentonite was contaminated. Slurry or soil inclusions in the concrete occur due to improper thinning and cleaning of slurry prior to concreting, improper construction or debris falling into the shaft while concreting, or improper use of the slurry. It is suggested to place a small piece of casing or sono-tube in the top portion of the hole to prevent debris from the surface from entering the shaft while the concrete is being tremied. It is also suggested to waste the first portion of concrete coming up over the top of the shaft, for that is the original interface between the concete and slurry, and usually contains sediments from the mud. See Figure A1.1-3.

With the use of slurries, construction supervision provided by the engineer or owner must be continuous, competent, and vigilant. Side friction values with proper surveillance and construction techniques are not adversely affected due to the use of drilling muds. End bearing values determined from field test results have been lower than without slurry but with proper cleaning of the shaft and proper recycling and thinning of the slurry just prior to concreting, end bearing resistance can be incorporated into the design. As stated in Barker and Reese (1970), page 5, "The use of drilling fluids in the construction of drilled shafts is not a science but an art based on experience", and as such one should only engage an experienced contractor in the installation of slurry drilled shaft.

A1.2 SOCKET ROUGHNESS

Frictional Socket Parameters

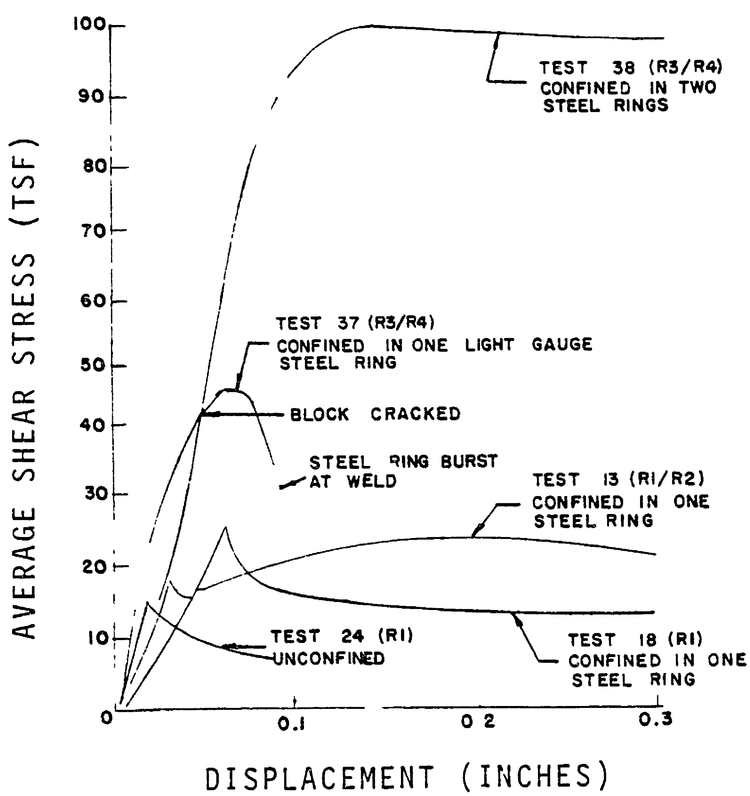
The frictional behavior of the shaft-rock interface is similar to that of a natural rock joint. The major factors which affect the mobilized peak side adhesion of frictional rock sockets are:

1. Strength and deformation properties of the concrete and rock.
2. Normal stress acting on the shaft-rock interface (normal stress is composed of the poisson's effect and any dilatancy effects).
3. Socket roughness and shape of the asperities.
4. Bond strength between the concrete and the rock mass.

Roughness and Confinement

Pells et. al, (1980) performed several lab and field tests on drilled shafts in sandstone. Laboratory model socket tests were performed to analyse any effects due to lateral confinement and socket wall roughness. The results are shown in Figure A1.2-1. As shown, the degree of confinement greatly affects the shear stress mobilized. The confinement of a rock socket is typically not a concern even in the porous coral limestone unless one suspects an open shaft or cavity within the radius of influence of the shaft. Figure A1.2-1 shows a trend that rough sockets are able to attain higher peak and residual shear stresses than smooth sockets. These higher stresses being developed at the concrete rock interface is of great importance and the field testing (Pells et al 1980), verified these laboratory trends. The field program consisted of several bored piles of different diameters and different degrees of roughness. Figure A1.2-2 shows field test results with three different degrees of roughness. Additional field tests from different studies reflect the same trend and are shown in Figure A1.2-3.

Smooth sockets exhibit brittle behavior consisting of a peak, (which is lower than comparable rough sockets), followed by a substantial drop in adhesion to a residual value. This is explained by noting that smooth sockets develop their shear resistance due to the bond between the concrete and rock surface and the interfacial normal stress with the rock friction angle. Once a smooth socket reaches its peak shear stress the concrete-rock bond, which often amounts to a large percentage of the peak shear stress, is broken and lost. The socket then relies on the interfacial normal stress with the residual friction angle of the rock to develop the residual shear stress shown. Pells et., al. (1980), concluded a side roughness of asperities being 0.08 to 0.12 inch deep, 0.2 to 0.4 inches wide at 4 inch spacing is sufficient to prevent the brittle behavior of smooth walled drilled shafts. Figure A1.2-4 shows that auger smear or contaminated bentonite smear can



ROUGHNESS CLASSIFICATION

ROUGHNESS CLASS	DESCRIPTION
R1	STRAIGHT, SMOOTH SIDED SOCKET, GROOVES OR INDENTATIONS LESS THAN 0.04in DEEP.
R2	GROOVES OF DEPTH 0.04-0.16 in, WIDTH GREATER THAN 0.08 in, AT SPACING 1.9-7.9 in.
R3	GROOVES OF DEPTH 0.16-0.39 in, WIDTH GREATER THAN 0.20 in, AT SPACING 1.9-7.9 in.
R4	GROOVES OR UNDULATIONS OF DEPTH 0.39in, WIDTH 0.39in AT SPACING 1.9-7.9 in.

FIGURE A1.2-1 LABORATORY MODEL SOCKET RESULTS SHOWING EFFECTS OF SOCKET WALL ROUGHNESS AND DEGREE OF LATERAL CONFINEMENT (ADAPTED AFTER PELLS et. al. 1980)

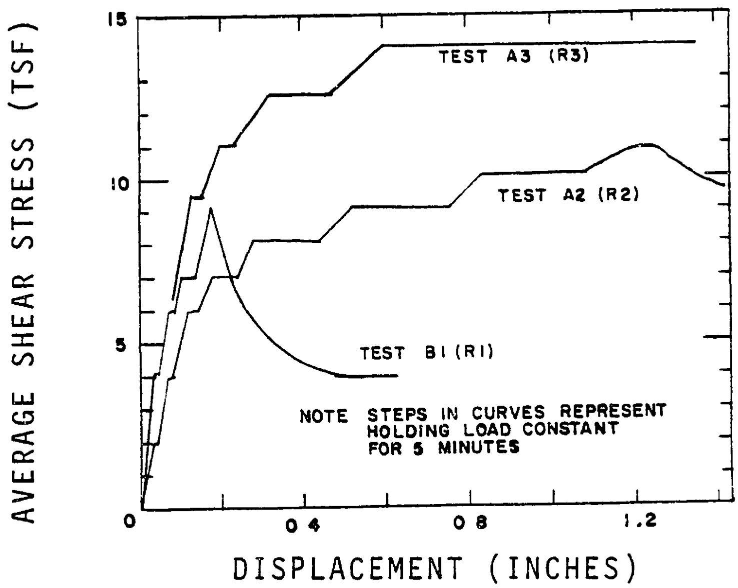


FIGURE A1.2-2 TEST RESULTS FOR CLEAN SOCKETS WITH DIFFERENT DEGREES OF ROUGHNESS (ADAPTED FROM PELLS et. al. 1980)

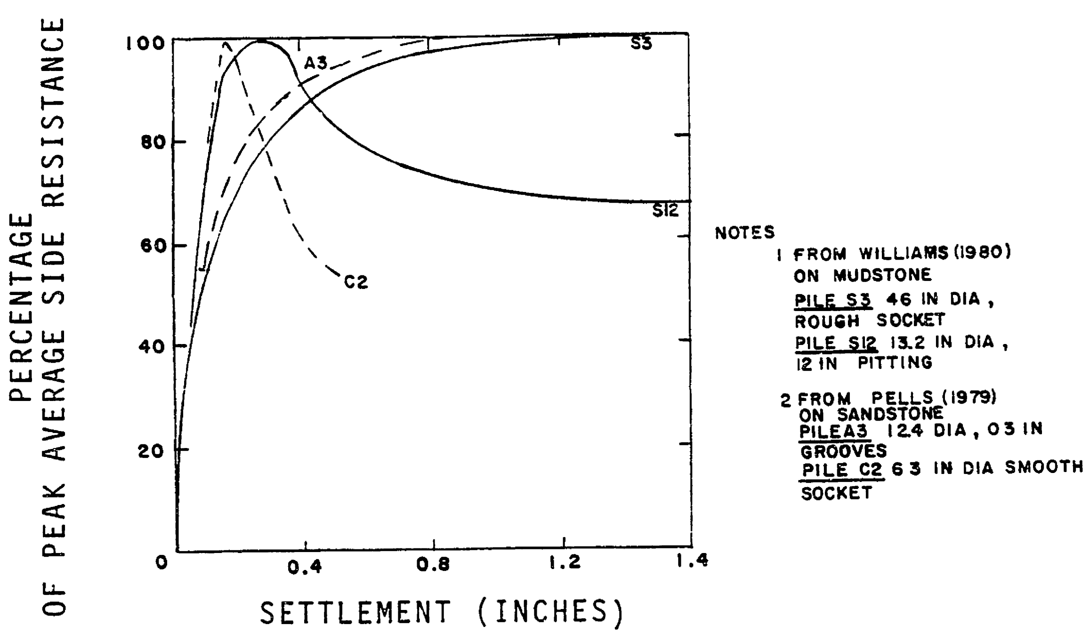


FIGURE A1.2-3 SOCKET ROUGHNESS EFFECT ON SIDE RESISTANCE

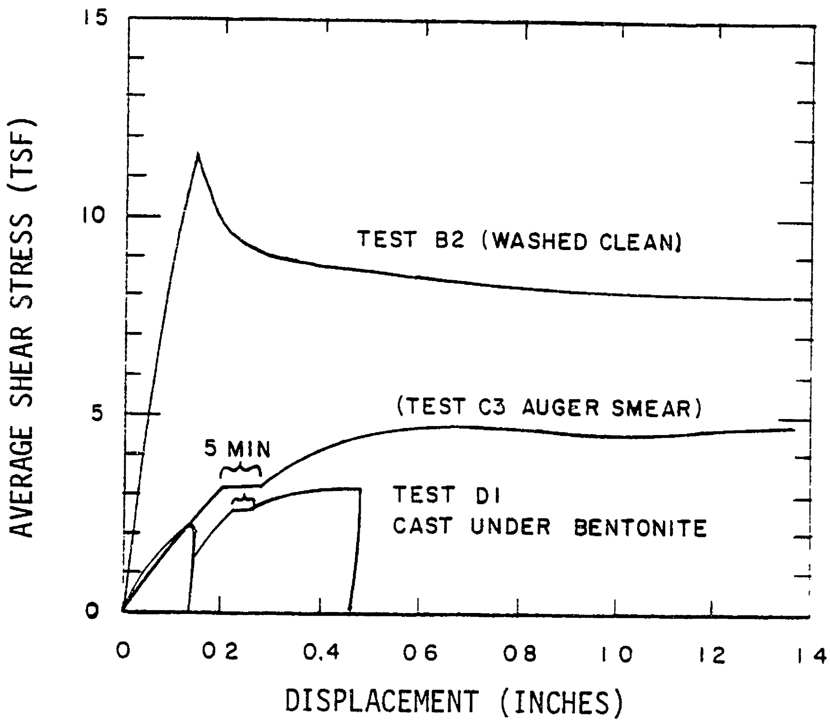


FIGURE A1.2-4 TEST RESULTS FOR SMOOTH SOCKETS WITH DIFFERENT DEGREES OF SIDEWALL CLEANLINESS (AFTER PELLS et. al. 1980) prevent the additional shear stress the concrete-rock bonding provides in smooth sockets.

Rough sockets have been observed to have a "work hardening" behavior. Rough sockets can attain higher radial normal stresses than smooth sockets through socket dilatation occurring during side movement. With these higher normal stresses rough sockets can attain residual side resistance values which are often equal to the peak shear stress. Williams found that a roughness containing asperities 0.4 inches deep at 4 inch centers can obtain this roughness behavior desired in socketed drilled shafts. Kenney (1977), found with very rough sockets that a lubrication of the concrete-rock interface actually increases the shear resistance due to this wedge type action. Williams and Pells (1981), employing vertical and radial strain gages in field tests were able to measure the normal stress due to dilatation in the residual portion of the load test. They found the calculated residual shear stress using the residual friction angle:

$$f_{sr} = \sigma_n \ \mathrm{Tan}\ \phi_r,$$

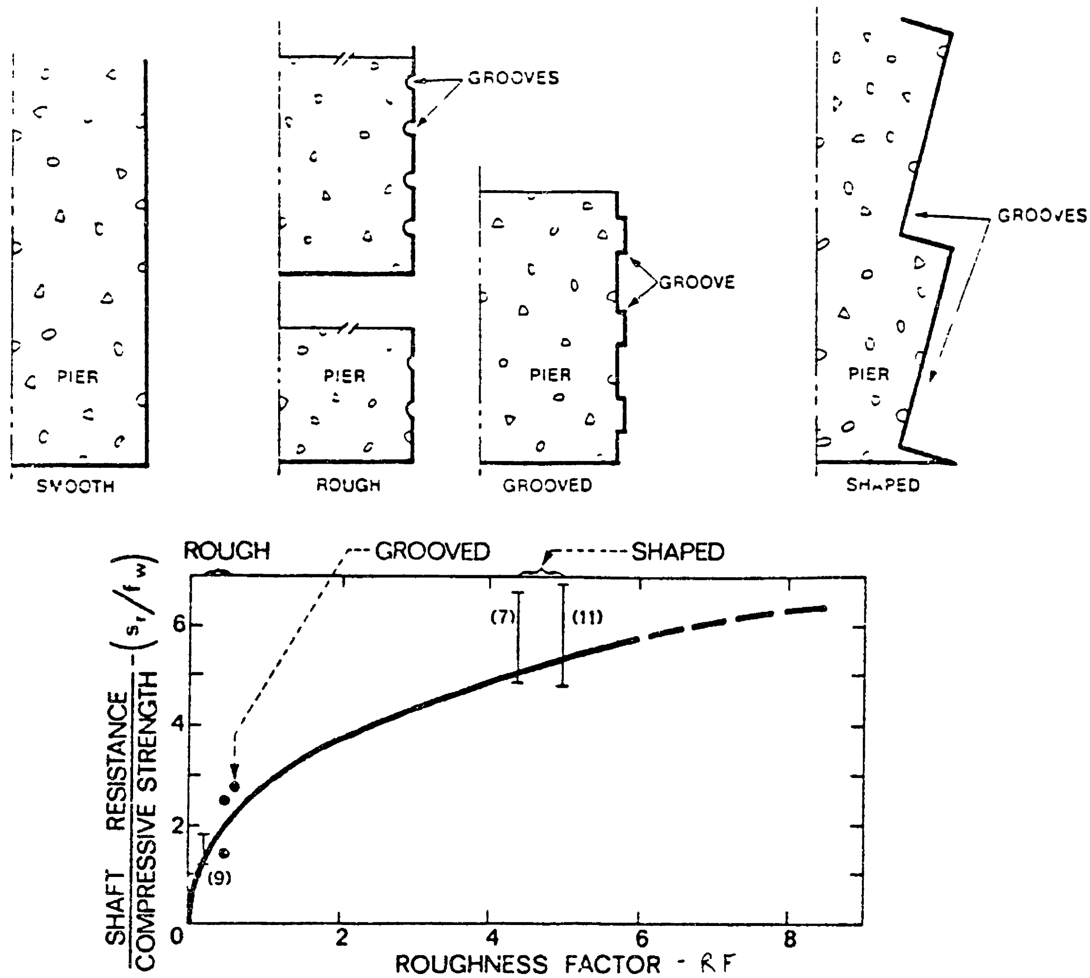
Compared to within 12% of the evaluated field test results, they also found that the normal stress basically remains constant despite displacements over 5 times that required to attain the peak shear stress. It was also shown that this normal stress due to dilatation effects remained "locked in" after unloading the shaft. This "locked in" effect suggests, as has been shown by others, that preloading a rough rock socket can decrease the side movement required to attain peak side resistance.

Due to the large porosity of the Florida limestone most all drilled sockets would be classified as rough or grooved and not as smooth. It is important to note that coring sockets in what is sometimes called calcareous sandstone, which is a less porous sedimentary rock of coral extraction, may cause a smooth type socket if the rock is tightly sedimented or smeared during construction. Proper construction methods and continuous inpection can greatly influence the cleanliness of the rock socket surface, and as shown a socket with auger smear can be detrimental to the development of the designed shear stress.

There is economy to shaping a socket wall. Horvath and Kenney (1979) and Horvath et. al., (1980) have done a limited number of model tests on smooth, rough, and shaped sockets. These show by shaping a rock socket one can increase shaft resistance by a factor of at least 2. Their results along with a roughness factor is defined in Figure A1.2-5. With these results it may be economical to add roughness to rock sockets. William and Pells (1981) suggest adding extra teeth to rock augers or drilling buckets to insure the desired roughness.

A1.3 DIAMETER AND LENGTH EFFECTS

Most all investigations are in agreement that variations in peak side resistance due to different rock socket diameters for drilled shafts are neglible in the range of production shafts used. Havorath and Kenney (1979) completed various field tests and developed the curve in Figure A1.3-1. This shows no change in the strength ratio of drilled shafts of diameters from 16 inches to 40 inches. The tests shown in Figure A1.3-1 were performed in shale, clay-shale, siltstone, and mudstone. Since the same tendency of diameter effects have been noted by other authors, it is assumed to hold true with the Florida limestone. Pells et. al (1980) show the effect of the L/D ratio on the peak average shear developed was not significant. Their field load tests were performed on a massive sandstone, the results are shown in Figure A1.3-2.



$$RF = \left(1 - \frac{r_{min}}{r_{max}}\right) \frac{L_T}{L_C}$$

Where: r_{min} is minimum radius of the socket

r_{max} is maximum radius of the socket

L_T is total travel length along socket wall profile (for smooth shaft $L_T = L_S$, for rough pier $L_T > L_S$)

L_S is vertical length of socket

L_C is closure length

$$= \sqrt{(2r_{max})^2 + (L_S)^2}$$

FIGURE A1.2-5 SOCKET ROUGHNESS EFFECT ON SHAFT RESISTANCE
(AFTER HORVATH et. al. 1980)

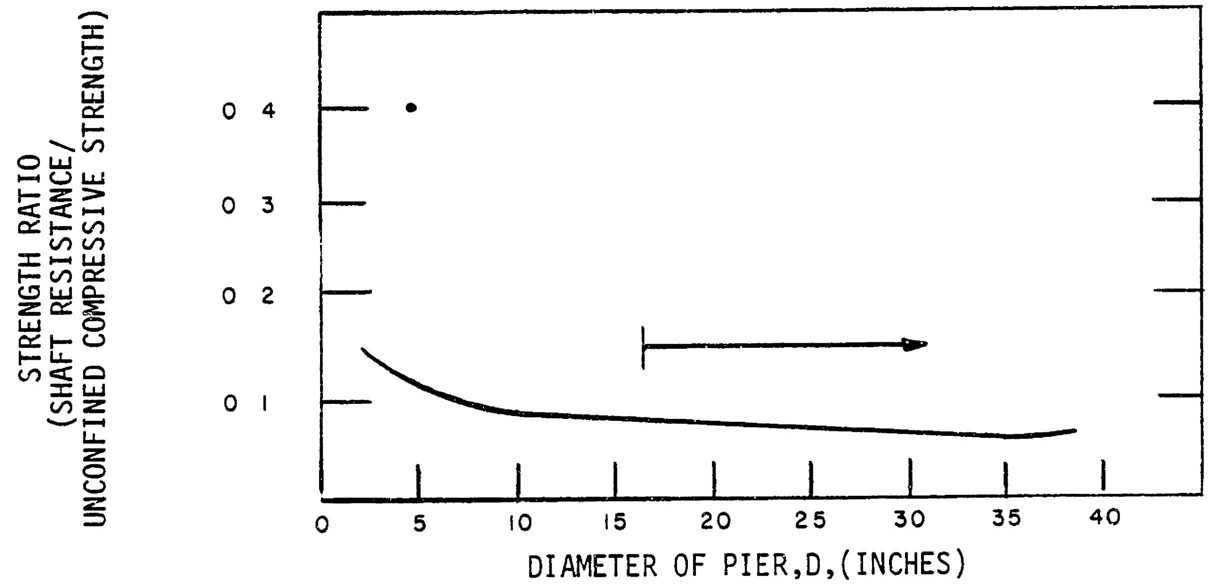


FIGURE A1.3-1 AVERAGE STRENGTH RATIO, R_s, FOR PIERS OF VARIOUS DIAMETERS
(AFTER HAVORATH AND KENNEY 1979)

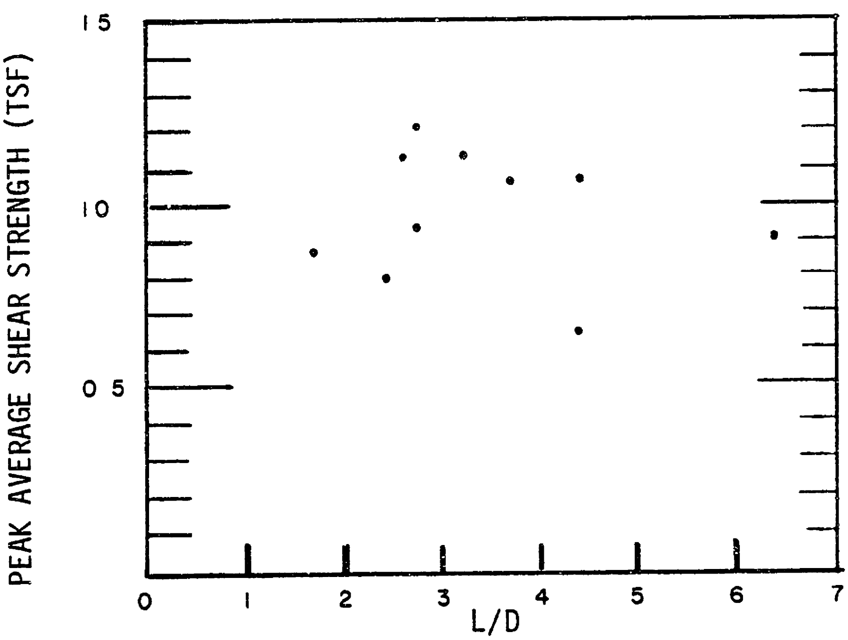


FIGURE A1.3-2 EFFECT OF L/D RATIO ON THE PEAK AVERAGE SHEAR STRESS
(AFTER PELLS et.al. 1980)

APPENDIX 2

DRILLED SHAFT CONSTRUCTION, INSPECTION AND REPAIR

A2.1 CLASSIFICATIONS AND PROCEDURES

The three common techniques employed in the routine construction of drilled shafts are the dry method, the casing method, and the slurry displacement method. Any of these methods can be employed by itself for the construction of drilled shafts or combined with any of the other two.

The Dry Method

The dry method of drilled shaft construction is applicable when the sub-stratum involved has sufficient cohesion that it will not cave or slump, and the designed base of the drilled shaft is above the ground water table. Since the groundwater table in South Florida is typically near to the surface, and the soil strata consists of sands and porous rock, this method is clearly not applicable by itself.

The Casing Method

This method as employed in South Florida is shown in Figure A2.1-1. Steel casing, normally with cutting teeth is vibrated, or screwed into the ground to the bottom of the low cohesion strata susceptible to caving or deforming. If the cohesionless material is below a hard rock layer the hole can be advanced to the top of the cohesionless layer prior to introducing the casing. Once the casing is vibrated into place, the inside is cleaned out with augers or cleanout buckets. If the stratum below the casing is now rock, the rock (or cohesion material) could be drilled without shaft wall caving from the above soil layers.

The casing can either be vibrated out while concreting, (keeping sufficient concrete head above the base of the concrete to prevent soil inclusions), or concreted in place permanently. Concrete would be introduced by tremie. It

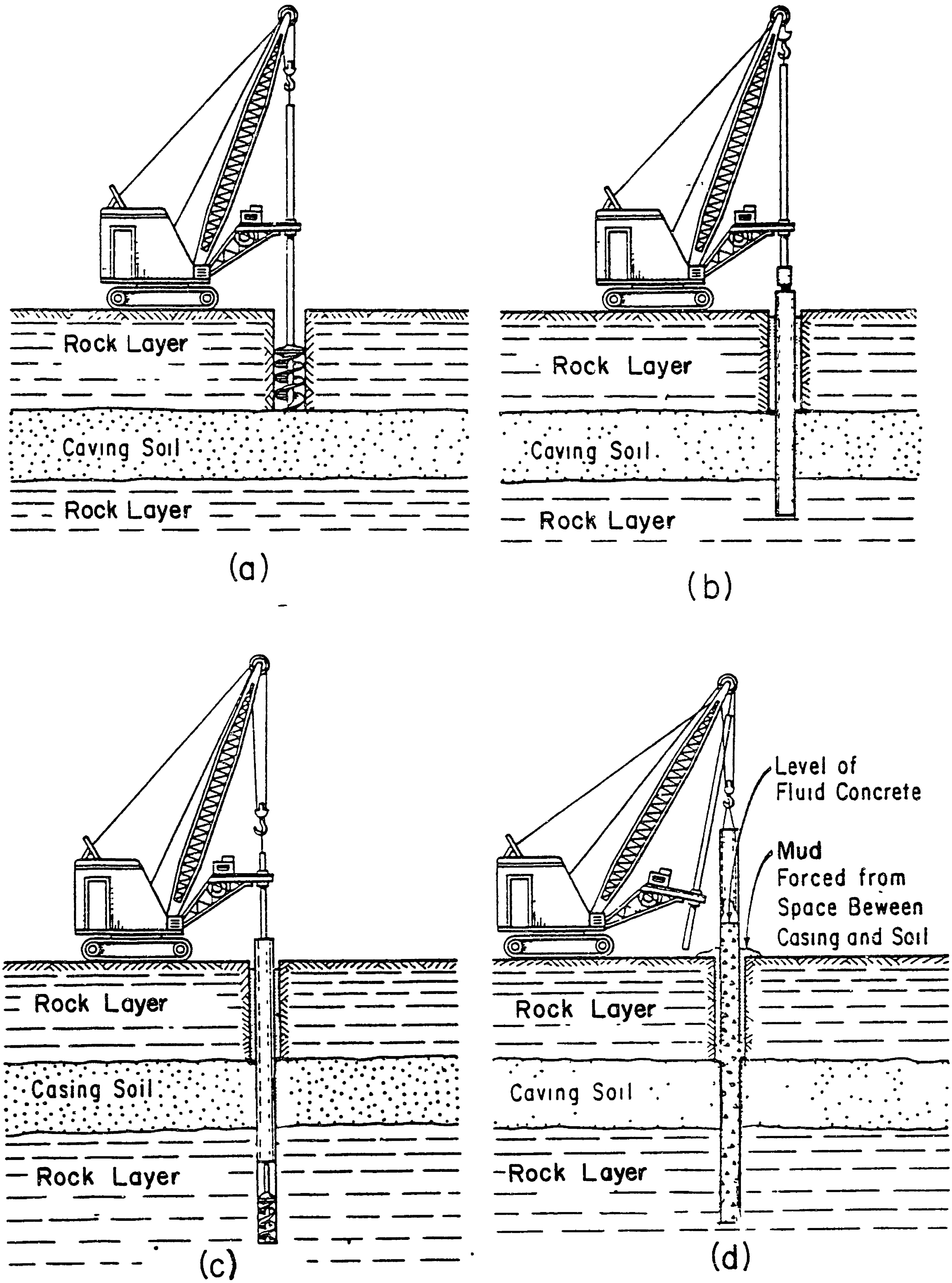


FIGURE A2.1-1 CASING METHOD OF CONSTRUCTION a) INITIATING DRILLING;b) CASING IS ENTERED INTO THE HOLE AND VIBRATED OR SCREWED TO THE BOTTOM OF THE COHESIONLESS LAYER;c)DRILLING BELOW CASING;d) REMOVING CASING.

(ADAPTED FROM REESE AND WRIGHT 1977)

would not be economical or practical to try to reduce the permeability of the limerock to permit concreting in the dry. The casing method is applicable to the South Florida region and has been employed satisfactorily (Nyman 1980), Gupton et al 1982). It has been noted while using this method and augering through the lower Florida limestone that small amounts of sand was seeping in the hole from the sand vugs in the rock. If substantial solids flow into the hole, this may affect the concete and lead to lower values of adhesion and end bearing than expected. Casing can also be introduced into a slurry stabilized hole for thinning or removal of the slurry prior to concreting, this is discussed below and in Section A1.1.

The Slurry Displacement Method

The slurry displacement method as shown in Figure A2.1-2 is also applicable to the South Florida stratigraphy. The Contractor drills through the rock and sands while introducing drilling mud to prevent hole distortion. Drilling mud is introduced immediately upon reaching the ground water table and its surface is kept near ground level during the operation. The minimum height of slurry is 3 feet above the ground water table. This is to keep a positive pressure head on the walls of the cohesionless soils. During the drilling process the slurry should be replaced or recycled whenever contaminated with soil and prohibiting free flow of slurry through the drilling tools. Once the hole is drilled and cleaned to its design depth, the slurry should be thinned and preferably totally recycled. The hole is then filled with concrete by tremie displacing the drilling mud.

The drilling tools, employed in the casing and particularly the slurry displacement method, should be fabricated with sufficient space, so that the ground water or slurry can freely flow through the tools as they are introduced and extracted from the shaft during the drilling process. If sufficient flow is not available, or the slurry becomes too thick due to contamination with soil, the

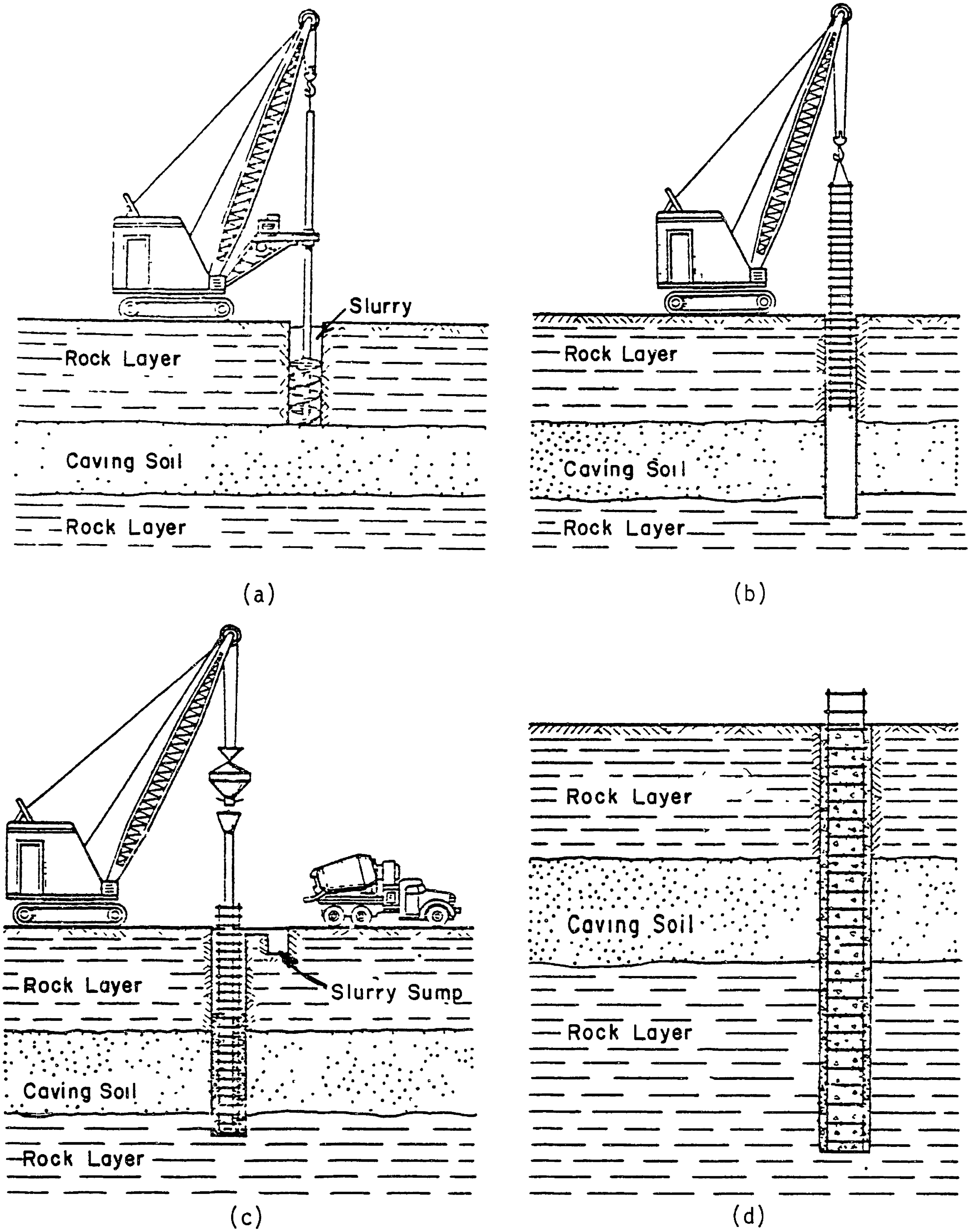


FIGURE A2.1-2 SLURRY METHOD OF CONSTRUCTION a) DRILL WITH SLURRY; b) PLACING REBAR CAGE; c) PLACING CONCRETE; d) COMPLETED SHAFT (AFTER REESE AND WRIGHT 1977) movement of the drilling tools will create negative pore pressures which will cause deformation or collapse of the sand layers.

Typical Drilling Procedures

1. If using casing vibrate casing to desired depth.
2. Drill hole dry until reaching the groundwater table.
3. If employing slurry, add slurry and keep slurry head at least 3 feet above the groundwater table. Replace slurry as required due to soil contamination.
4. Drill under slurry or clean out casing. Shaft diameter is typically drilled 6 inches smaller beneath the bottom of any casing used.
5. Drill to desired elevation and clean bottom of hole. Clean and thin slurry.
6. Place reinforcement cage. Suspend cage or add concrete blocks to its base to prevent steel contact with rock.
7. Insert sealed tremie.
8. Fill tremie with concrete then unplug.
9. Place concrete, keeping tremie base embedded in fluid concrete.
10. If casing is to be vibrated out, vibrate once adequate head is above the casing base.
11. Waste the first portion of concrete which may be contaminated with sediment.

A2.2 CONSTRUCTION PROBLEMS

In South Florida due to the porosity of the Florida limerock concreting using the dry method is impractical. A method whereby one employs slurry, then introduces casing, seals the bottom of the casing, pumps the casing dry and then pours concrete in the dry while vibrating out the casing is not recommended for

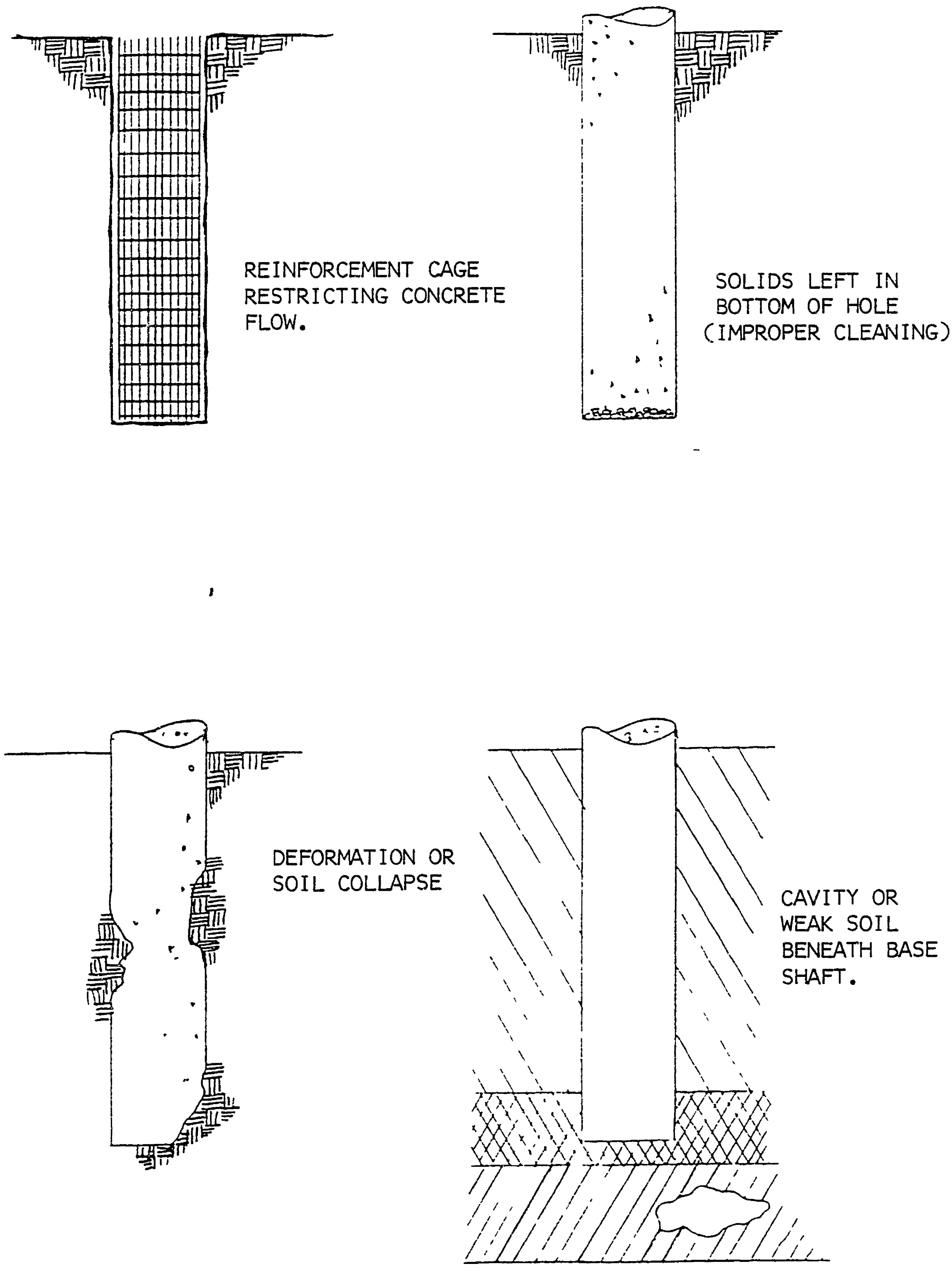


FIGURE A2.2-1 CONSTRUCTION PROBLEMS
(AFTER WINTERKORN AND FANG 1975)

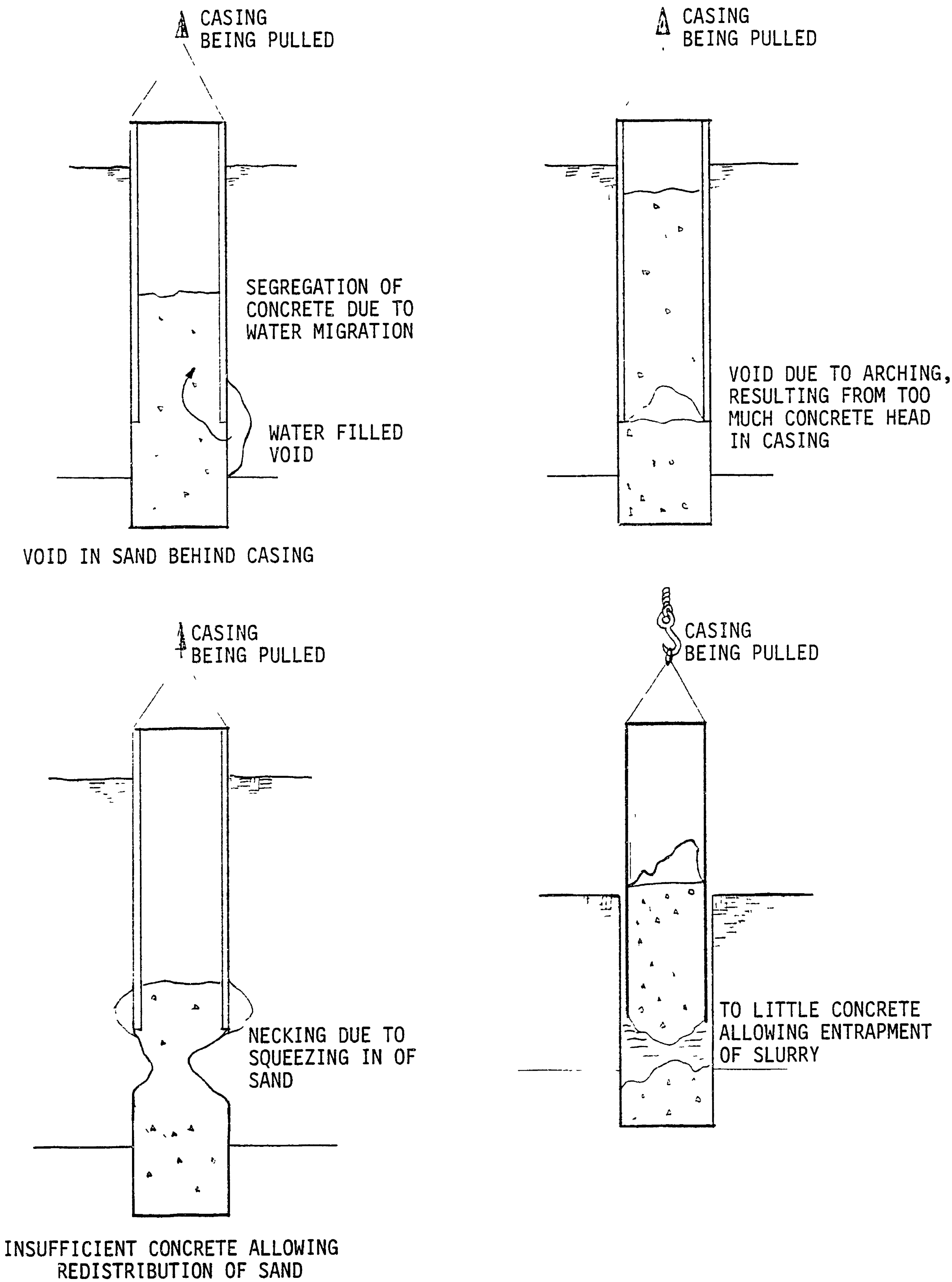


FIGURE A2.2-2 CONSTRUCTION PROBLEMS
(ADAPTED FROM WINTERKORN AND FANG 1975)

frictional reasons as specified in Section A1.1. The casing method and slurry displacement method wich were described earlier are presently being employed in the area. With proper supervision and construction procedures, these methods provide desireable results. Possible problems associated w h casing and slurry construction are described below along with suggestions to minimize the possibility of these problems.

FACTORS TO BE CONSIDERED

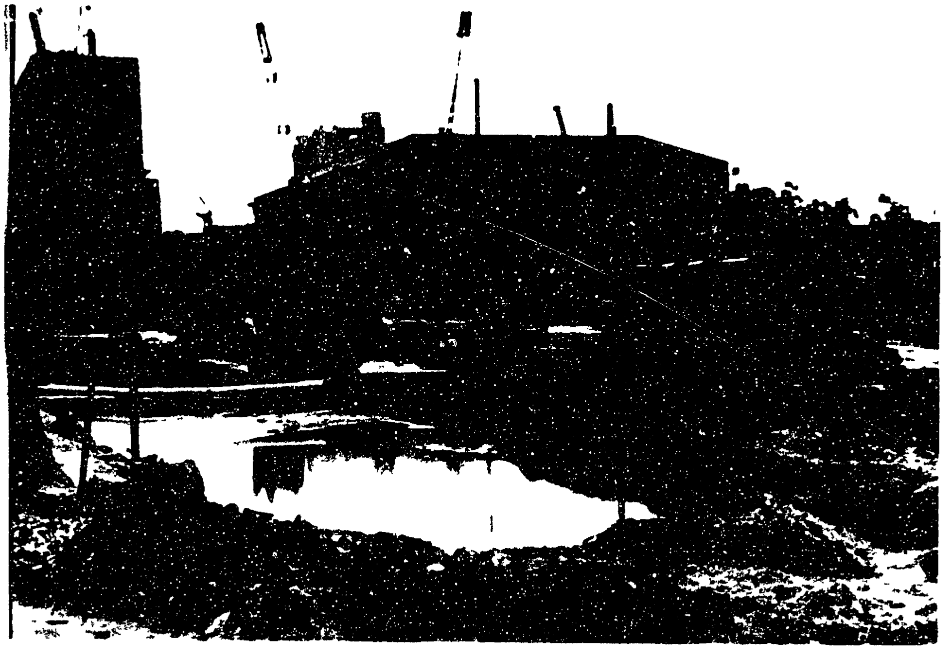
Destabilization of Slurry - As mentioned in Section A.1.1, many factors attribute to the proper dispersion of slurry, and the development of the desired thin "wall cake". If the slurry is not properly dispersed or if shear stresses are applied to the "wall cake", the cohesionless soils will destabilize and cave into the shaft. Greenwood stated a case where the driving of piles, with a steam hammer pile driver, at the same site as drilling slurry shafts, destablized the thixotropic bentonite slurry and as a result the shaft walls caved. Because the slurry is thixotropic, if the level of the slurry is kept at or near the level of the caving stratum the scouring action due the fluctuation of the slurry's surface, (with the introduction and withdrawal of drilling tools), will cause destablization and caving. In hole mixing of slurry could add to this scour. It is recommended to keep the level of slurry at least 3 to 5 ft above the ground water table (preferrably at the ground surface), to keep a net exit gradient head on the slurry in the hole.

Slurry Slime - The drilling mud or slurry develops an impervious thixotrpic cake around the walls of the cohesionless strata as shown and described in Figure A1.1-1 in Section A1.1. Improper cleaning of the walls and bottom of the drilled shaft just prior to concreting can leave over 16 inches of slime under the tip and on the sides of the shaft (Endo 1977). Improper recycling of the slurry to rid the slurry of soil contaminants or improper thinning of the slurry, (which reduces the density of the displacement mixture), prior to concrete placement by tremie, also increases the chances of contaminating the concrete. When large qantities of slurry are desired and land available recycling the slurry with sedimentation ponds and use of mechanical equipment become very economical (See Figure A2.2-3).

Shaft Reinforcement - The design of the reinforcement steel must be acceptable for the designed axial and horizontal loads, as well as the excessive stresses due to its being picked up and placed in the drilled hole. It is recommended to keep the reinforcement suspended from the ground surface to limit the possibility of buckling and distortion during concreting. The reinforcing bar spacing in all directions should be such that allow the fresh concrete to flow readily through the bars. Reese and Wright (1977), suggests a minimum clear spacing of at least three times the size of the largest aggregate between all reinforcing. Special mixes where small sized coarse aggregates are used are suggested and have been successfully used in drilled shaft construction.

Prolonged Pumping - Excessive pumping due to air lifting solids from the shaft base for long periods of time, pumping to pour drilled shaft caps, or foundation slabs, and even recycling a polluted ineffective drilling mud may remove sufficient fines to cause settlement of adjacent structures. Gill (1980), observed settlements of 6 to 8 inches, 60 to 80 feet from the drilled shaft attributed to prolonged pumping. For this reason, an exit gradient with respect to the head of slurry or water within the drilled shaft is always suggested. Discharges from pumps used to lower the ground water prior to pouring drilled shaft caps or floor slabs should be checked for solids content.

Spudding - Spudding, which is sometimes used when the rate of drilling the rock with conventional augers and core barrels becomes too slow is not suggested for small diameter holes. Spudding consists of dropping a 20 to 25 foot long I beam 20 to 30 feet onto the rock. This impact is repeated as often as necessary to



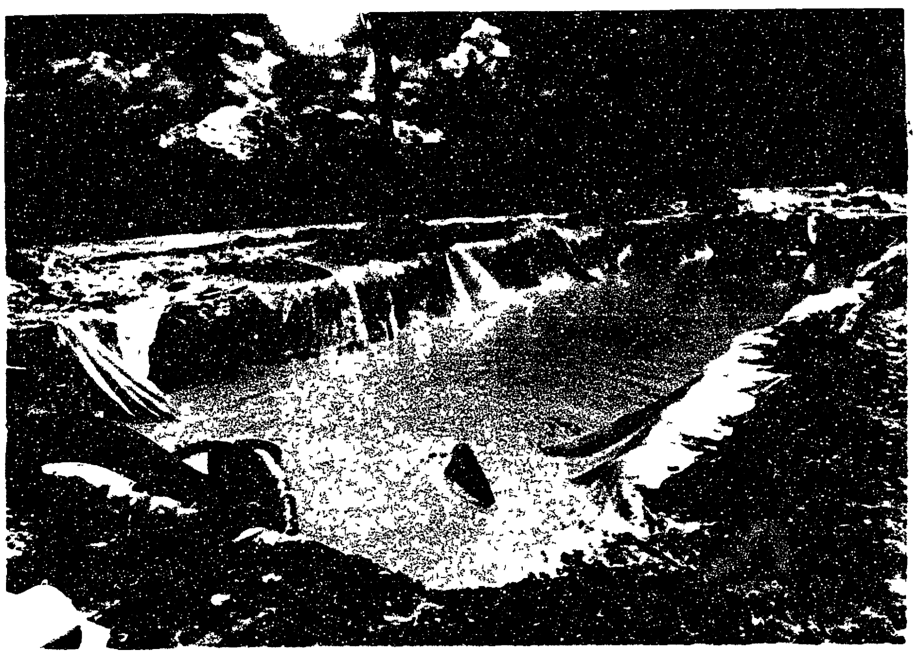


FIGURE A2.2-3 - SLURRY SEDIMENTATION PITS USED FOR RECYCLING SLURRY

fracture the rock until it can conveniently be removed with rock augers. The problem with this method is possible fracturing of the limestone beyond the desired diameter. This extended fracturing would reduce the frictional resistance along the rock socket walls.

Weak Soil or Cavity Under Base of Shaft- If end bearing is being considered and carries a large percentage of the design load, the base of the shaft should be probed to check for a weaker stratum or cavity. It is suggested to probe a minimum of one times the shaft diameter if in rock, and to a depth of 8 feet or 1½ times the shaft diameter whichever is larger, if the shaft terminates in soil, (Gill 1980). This probing can be done with a small auger. In the Florida limestone, this probing depth would be left to the discretion of the engineer based on the consistency of the borings for the site.

CONSTRUCTION DEFECTS

Casing Collapse - Since casings, when used in South Florida are normally always filled with water, the buckling pressures exerted on the casing are less than if the shaft was pumped dry. Even so, because of the use of air lifts to clean the hole bottoms, and with rapid extraction of the construction tools, it is suggested to design the casing based on hydrostatic forces (normally larger than soil forces), on the exterior and no interior forces. Gill (1980), suggested this design and states a factor of safety of 1.33 should be used. This suggestion was made after the casing collapse in a 10 ft diameter, 135 ft deep rock caisson for the John Hancock Building in Chicago and similar failures. The critical condition was determined to be when the casing was not in contact with the soil and hydrostatic forces were applied on the outside of the casing while the inside water level was being lowered. The required buckling analysis is common and should be found in any strength and materials text, Timoshenko is suggested. It should be noted small eccentricities in the roundness of the casing can greatly increase the possibility of collapse. Collapsing of a casing can normally be detected through observation.

Poor Concrete - The wrong strength or aggregate concrete can cause failure of a properly constructed shaft. For this reason, concrete cylinders should be taken from each truck and tested to check the quality of concrete delivered to the site.

The two most important aspects of using tremie concrete are the plug which separates the concrete as it enters the tremie from the water or slurry in the shaft, and the tremie embedment. Many types of plugs are used and any type which has successfully proven itself in the past is sufficient. Proper tremie embedment in fresh concrete is essential to prevent soil entrapment. Tremie embedments from 5 to 15 feet have been suggested. The size of the tremie pipe should be as large as possible to create as much scouring of the sidewalls with the fresh concrete as possible. Touma and Reese (1972) determined the optimum tremie size was when the ratio of the effective diameter of the hole (taking reinforcing steel into account) to the diameter of the tremie was 1.6. This would lead to excessive impractical diameters for tremies; but it does show that the largest tremie possible should be employed, typical diameters range from 6 to 12 inches.

The concrete should be as fluid as possible to allow easy flow through the reinforcing cage, and provide a proper scour on the shaft walls. Concrete slumps of 7 inches ± 1 inch have provided good results. The rate of rise of concrete in the shaft should always equal or exceed that of any test shaft used in analyzing the substratum. Reese (1978) suggests to avoid vibrating the concrete due to possible collapse of weak surface soils. Careful rodding once the shaft is completed may be employed.

Improper Excavation - The contractor may drill the hole in the wrong location, or may drill a different size than designed. The shaft can be drilled out of plumb or to the wrong elevation. Typical drilled shaft specifications call for center placement within 3 inches of plan location, within 2% of being plumb from top to bottom, and for the top of the concrete to be no more than 1 inch above or 3 inches below the desired plan elevation. If end bearing is to be employed the contractor must pay strict attention to obtaining the cuttings from the bottom of the shaft. For proper end bearing, any leftover spill should be thin and cover less than 5 percent of the base diameter.

Soil Inclusion - This can develop from improper use of slury as already described, from workmen walking too close to the top of the shaft or a spill pile too close to the shaft, and from caving as casing is pulled. Improper use of slurry can be detected and corrected by a seasoned inspector on the job site. It is suggested to have sufficient casing stick up to prevent surface soils from falling into the shaft while concreting. In cases of using drilling mud with no casing, a large clear area at the top along with a short piece of sonotube placed over the top few feet and sticking out of the ground gives satisfactory results. Adequate concrete head in the casing at the time of pulling prevents inclusions due to casing removal, the head should be checked physically by sounding with a wire line.

Voids - Voids occur in the concrete when pulling a casing out of the hole. They can be created due to temporary arching of the concrete or hanging up in the casing due to an initial concrete set. Voids typically occur due to inadequate vibration or extracting the casing too fast. The use of a retarder and small aggregates in the concrete improves the flow characteristics and retards setting. This allows the casing to be pulled away with a lower probability of arching. Careful monitoring of the concrete head may not always detect these occurrances, one must rely on the quality and experience of the contractor.

A2.3 METHODS OF DETERMINING DEFECTS

Coring

The most common method for checking an intact drilled shaft for discontinuities is by making a core run through the entire depth of the drilled shaft. Baker and Khan (1971), notes with a 1" aggregate the larger the diameter core, the better recovery. Small scale core diameters could vary from 2 to 6 inches. When choosing a drilling contractor for the coring, he should be the best available and hopefully not connected in the installation portion of the project. The quality of the core driller will affect the quality of the recovered samples.

Caliper Logging

Seismograph Service Corporation has developed a logging tool which when lowered into the hole, gives the diameter of the core hole with depth, See Figure A2.3-1. Any increase in diameter may indicate weak concrete, voids, or seams, thus giving the engineer information which would lead to further investigation.

Stress Wave Propigation Methods

Methods of setting off a shock wave at the top or base of the drilled shaft and measuring the time for it to travel through the shaft or be reflected from the base or any discontinuities have been performed. The object is to compare the wave velocity of concrete with comparable reinforcement to that found in the field. Through this comparison, one can detect voids or poor quality concrete by variations in this wave velocity. These types of tests performed by Baker and Khan (1971), were not conclusive. Steinbach and Vey (1975) have performed field and lab tests on this method and have determined decipherable results but add that one cannot obtain results below a discontinuity and that analysing the results is difficult.

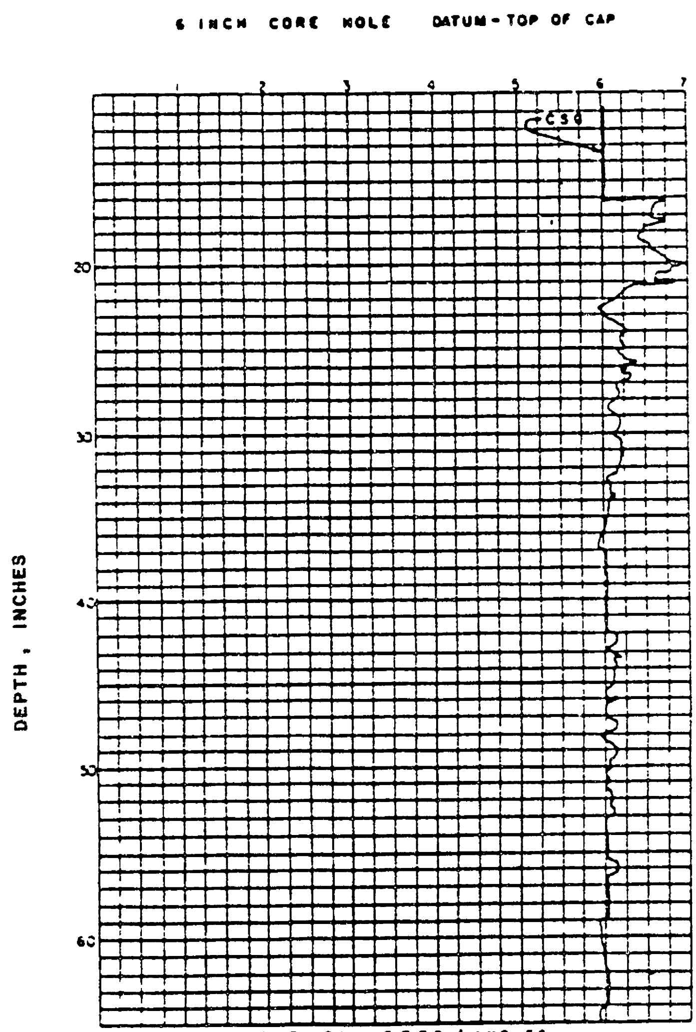


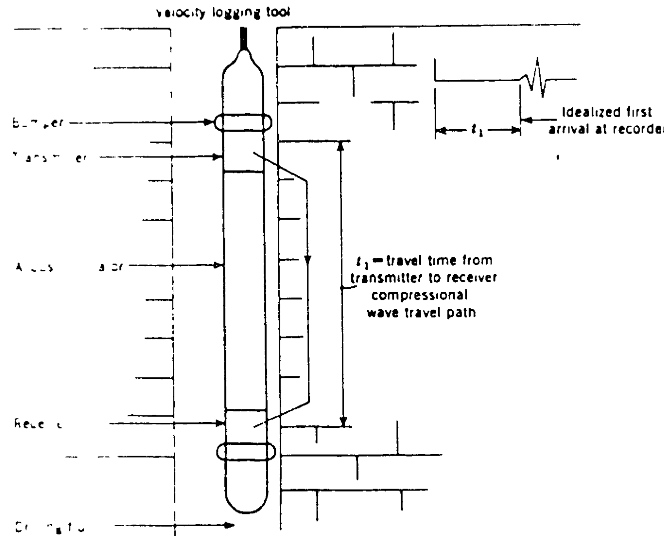
FIGURE A2.3-1 TYPICAL CALIPER LOG
(AFTER BAKER AND KHAN 1971)

Gamma Ray Logging

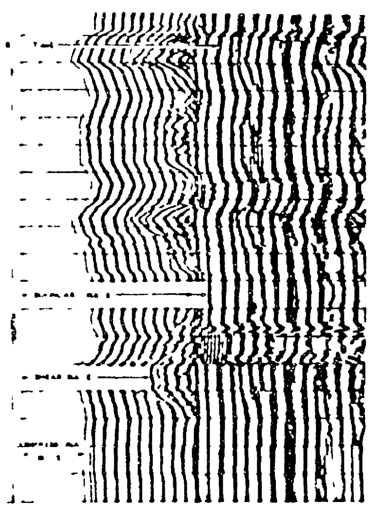
Insertion of a Gamma Ray Logger which logs the density of the concrete around the core hole it is sent down has yielded interpretable results. The results show pronounced changes for poor quality concrete. With these results, one could tell if the low quality recovery of the cores are from poor coring technique or from poor concrete.

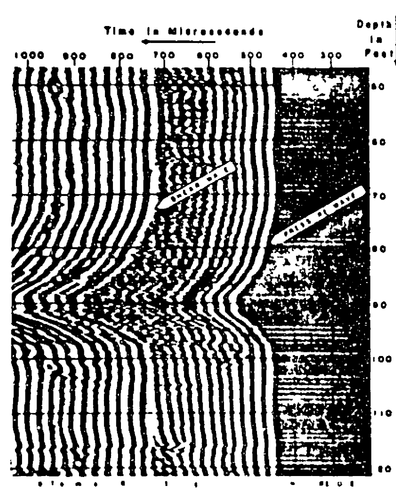
3-D Logging

3-D Logging is a by product of oil industry technology and is used extensively by that industry. In respect to drilled shaft inspection 3-D logging can be monitored through a single hole and a velocity logging tool, or with a sonic 3-D logger employing two holes. See Figures A2.3-2 and A2.3-3. The velocity logging tool consists of a transmitter and a receiver with an acoustic insulator betwen the two, Figure A2.3-2A. The transmitter consists of a magnet device which when pulsed (approximately 15 times a second) creates shear, compressional and boundary wave modes. The crystal receiver receives these signals and by employing film with an oscilloscope the intensity of the signals creates a log as shown in Figure A2.3-2B and C. As displayed in these logs, a drop in wave velocity detects a change in material properties which would be a defect in a proposed constant section drilled shaft. The cross hole logging evaluates the quality of concrete through the shafts cross sections. In cross hole logging, the receiver is lowered in one hole and the transmitter is simultaneously lowered into the adjacent hole. The velocity signal travels between the two core holes as shown in Figure A2.3-3. 3-D logging obtains considerable information on the quality of the drilled shaft and has performed well in numerous cases. Cases where reinforcing steel was too close to the core holes have presented interpretation problems and should be considered when deciding to use either single or cross hole signaling.



a) 3-D VELOCITY LOGGING APPARATUS

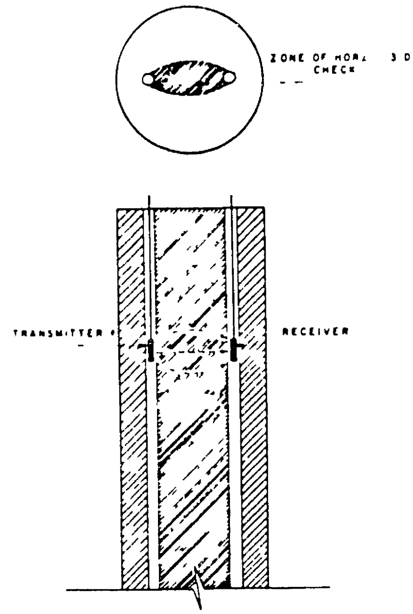




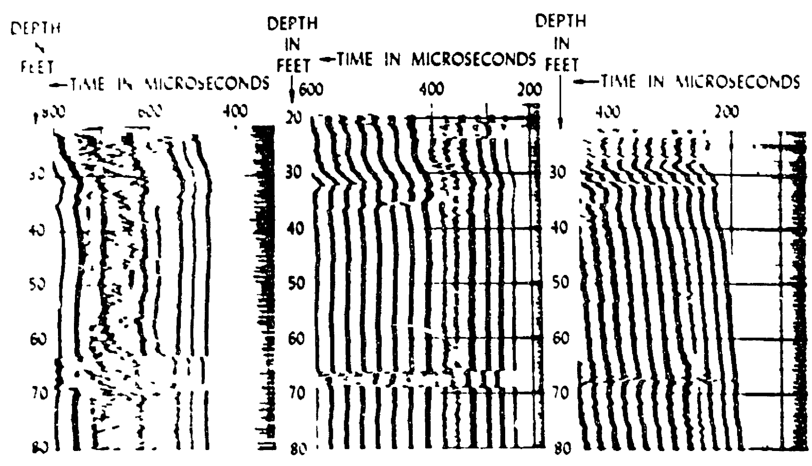
b) 3-D VELOCITY LOG

c) EXAMPLE LOG FROM SHAFT
WITH OBVIOUS DEFECT

FIGURE A2.3-2 3-D LOGGER
(AFTER BAKER AND KHAN 1971)



a) SONIC 3-D LOGGER IN OPERATION



b) EXAMPLE LOG FROM 3 CORE HOLES

FIGURE A2.3-3 3-D CROSS-HOLE LOGGER
(AFTER BAKER AND KHAN 1971)

A2.4 REPAIR OF DEFECTIVE DRILLED SHAFTS

Voids

Voids found through core holes can be grouted with a high pressure grout. The proper use would entail 2 corings through the void and injecting a grout into one core hole until it physically displaces the air or slurry, fills the void and exits the shaft though the adjacent core hole placed at the other side of the void. Grouting is not recommended unless the full extent of voids are known and proper remedial construction procedures are carefully monitored.

Weak Concrete

If a zone of weak concrete is found, the addition of grouted heavy reinforcement through core holes could transmit the load to sound concrete, See Figure A2.4-1. This procedure has been found to be successful and economical.

Extensive Defects

Removal of drilled shafts are typically prohibitively expensive. If grouting, adding reinforcement or partial excavation through coring of the shaft cannot provide suitable bearing, the use of new drilled shafts on either side of the existing shaft with transfer girders would be required, see Figure A2.4-2. Excavation of the shaft above the ground water table to repair shallow defects would be possible. Due to the high permeability of the sands and Florida limestone, test pits adjacent to a shaft to repair defects more than a few feet below the water table is not practical and the use of drilled shafts and transfer girders would be required.

A2.5 INSPECTION PROCEDURES

All the conditions and problems met in the field during construction cannot be anticipated by plans and job specifications. Due to this unpredictability of construction problems, constant, vigilant inspection by an engineer who has the

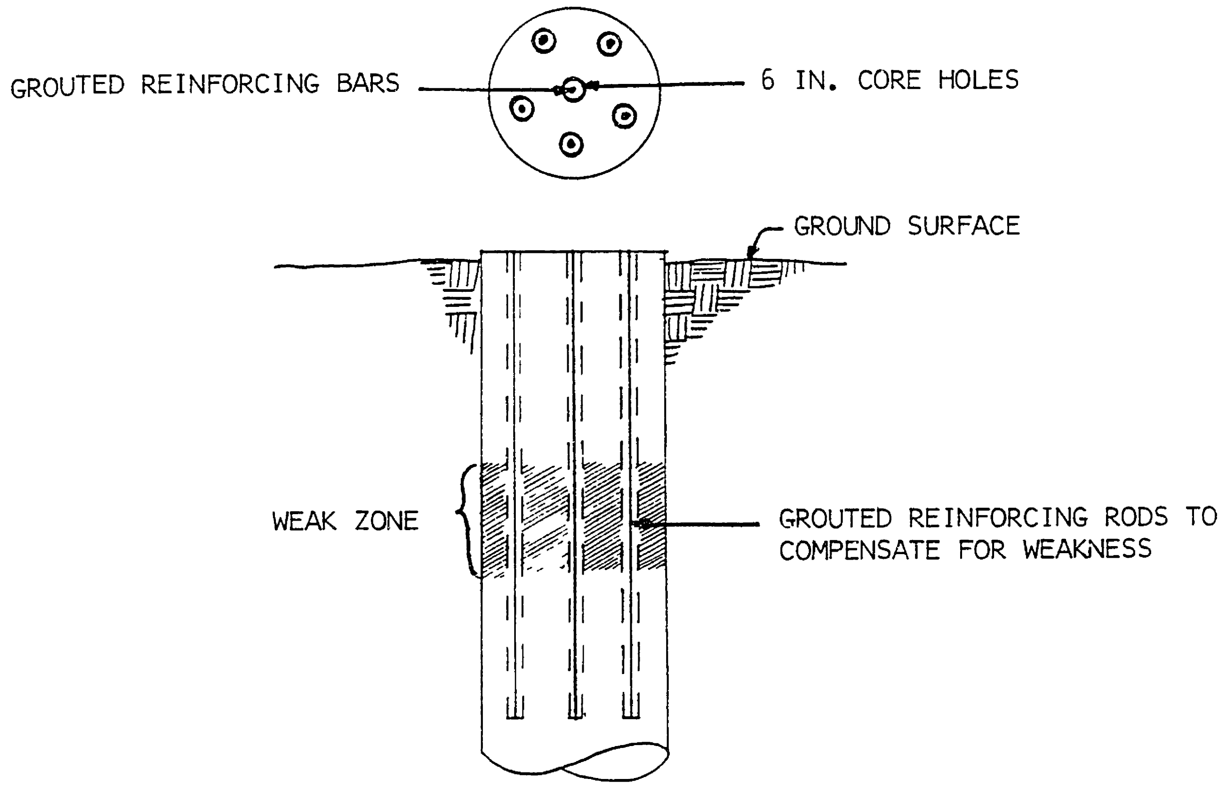


FIGURE A2.4-1 ADDITION OF REINFORCING BARS
TO BRIDGE OVER WEAK ZONE
(ADAPTED FROM BAKER AND KHAN 1971)

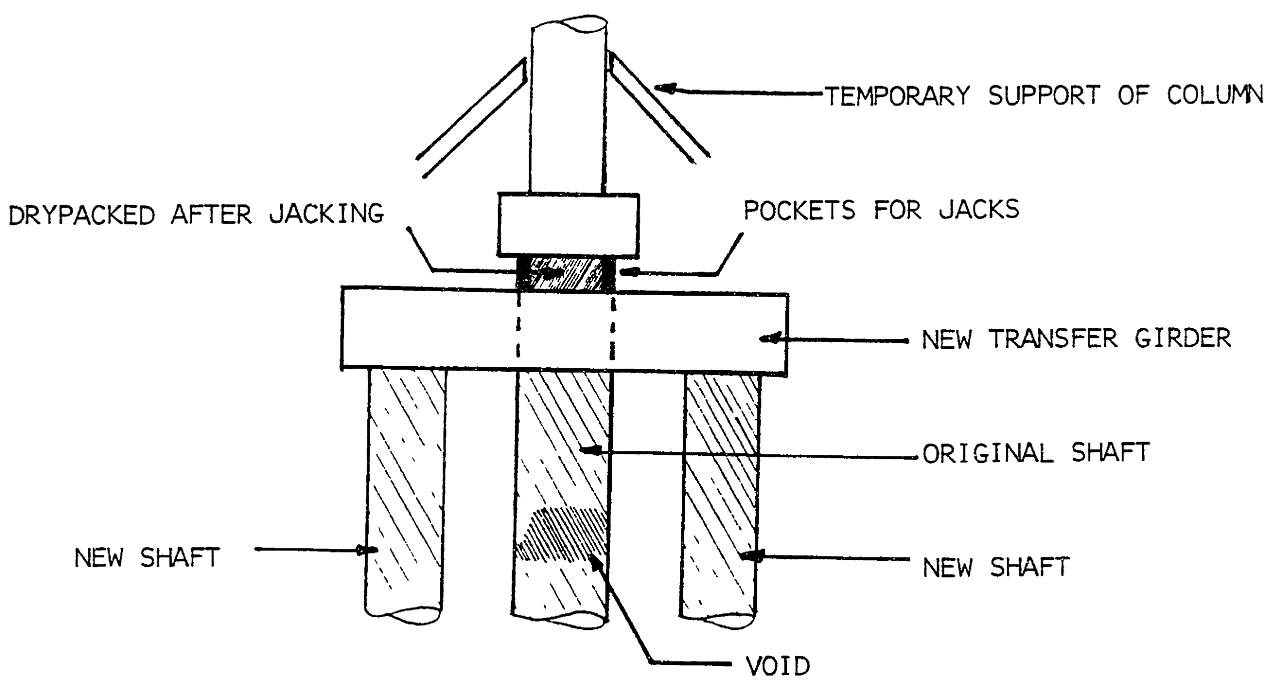


FIGURE A2.4-2 USE OF NEW SHAFTS AND TRANSFER GIRDERS TO
REPLACE A DEFECTIVE SHAFT
(ADAPTED FROM BAKER AND KHAN 1971)

authority and experience necessary to make decisions based on general geotechnical principles and intuition is required. The success of any drilled shaft is dependant on the experience and expertice of the contractor and hence, construction. The purpose of the inspector is to verify the proper execution of all construction aspects in the drilled shaft operation.

The inspector should be aware of the following elements of each drilled shaft:

1. Constantly aware of the construction process, and looking for visible problems.
2. Logging the drilling and construction of the shaft. Rates of drilling and times of various processes, along with estimates of rock and sand strata depths.
3. If slurry is employed:
   - A. Monitoring specific gravity and viscosity.
   - B. Checking for proper cleaning and thinning of the slurry prior to concreting.
4. Proper location, plumbness, diameter, base elevation and cleaning of the proposed shaft.
5. If casing is used:
   - A. Correct casing diameter, thickness and placement elevation.
   - B. If casing is extracted, that the proper construction procedure is used.
6. Proper reinforcing steel and spacing is used.
7. Proper placement of reinforcement steel in hole.
8. Concrete cylinders are taken from each truck.
9. Concrete falls within the specified temperature and slump range.

10. When concreting by tremie:

    A. Proper placement of tremie and adequate plug between the concrete and the fluid.

    B. Sound concrete level periodically to ensure proper embedment of tremie.

11. To allow an overflow of concrete over the top of the hole until no contaminants are seen on the concrete surface.

12. Estimate volume of concrete used in pouring the drilled shaft.

APPENDIX 3

FACTORS AFFECTING LABORATORY AND MODEL TESTS

A3.1 LABORATORY TESTING

General - Laboratory investigations are performed to obtain a range of the strength and deformation properties of the intact rock found at the test site. The intact samples used for laboratory tests are small in size and are often from the best rock cores obtained from the field. Since these laboratory results do not represent the average in-situ rock properties, laboratory test values must be correlated to field test results through the use of RQD, % recovery, standard penetration resistances and cone probes. Through these correlations, estimates of in-situ properties of rock are made.

The limestones in some of the Florida formations are riddled with solution channels and are poorly cemented. This causes a problem with testing specimens by making it virtually impossible to eliminate irregularities which appear on the sides and ends of the test samples. These irregularities cause problems in strictly adhering to the ASTM standards for rock testing, modifications, typically taken from the ASTM procedures for testing concrete, have been used.

Pells, et al (1980) worked on a sandstone which had the characteristics of having a much lower strength when wet than dry. Because of the solution possibilities of the Florida limestone, it is important to perform all lab tests in an environment equal to that of the field. It is suggested to keep the samples saturated with groundwater from field sampling through testing to minimize any environmental effects. Groundwater salinity and pollutants change with depth, and it is important not to change the characteristics of the groundwater in contact with the sample due to the solution nature of the limerock, Chapter 3.

Unconfined Compression Tests

The end result of empirical analysis is to determine side friction as a percentage of unconfined compression strength (see Section 4.3). Samples which are used in lab tests are typically taken from rock cores performed in the preliminary field investigation and from additional rock cores from a more detailed follow up investigation. Theoretical studies have shown that end effects due to the rigid end plattens used on the sample extremes effect (stiffen) the internal stress field to a distance equal to one half the sample diameter from each end, Hawkes and Mellor (1970). It is therefore important to use a length to diameter ratio of at least two.

Prior to placing the sample in the testing apparatus, the ends of the test sample must be smoothed to help prevent stress concentrations which would otherwise develop at the irregularities present at the sample ends. Nyman (1980), utilized a thin layer of gypsum cement spread over the ends of the sample to fill the small discontinuities present. Other engineers have used end caps to create smooth surfaces at the testing machine plattens.

Tensile Tests

This test, (the Brasilian Tensil Test), being adapted from ASTM "Splitting Tensile Strenth of Cylindrical Concrete Specimens", uses the same testing machine as the unconfined compression test. The sample is 3 to 6 inches long, and no length to diameter ratio is specified. The sample is placed in the loading apparatus length-wise with thin wood strips (1/16 to 1/8 inch thick) being placed lengthwise between the sample and the testing machine. The strips of wood cushion and distributes the load to the irregular longitudinal surface of the sample. After application of a seating load, the load is increased until failure. The rate of loading is suggested to be such to cause failure in approximately 10 minutes. The evaluation of results and specifications for the test are shown in ASTM C45-71.

Triaxial Rock Shear Strength Tests

Triaxial tests should be performed in accordance with ASTM specifications. These tests are typically employed when the mohr-coulomb failure criteria for the intact rock is desired.

Direct Shear Strength Tests

Direct shear tests could also be used to help define the mohr-coulomb failure criteria and should be performed in accordance with ASTM specifications. A high-strength gypsum plaster has been successfully used in securing the sample in the testing apparatus, and is normally required due to the rocks irregular surface.

A3.2 MODEL PLUG TESTS

Different types of model plug tests have been performed by Kaderabeck (1981) and Gupton et al (1982) to estimate the side friction of drilled shafts. Careful testing and handling of the materials should be employed, and the following factors considered in model tests.

1. If the rock is the actual field rock, the pores in the simulated shaft are the actual size found in the field. This should be compared to the size of grout related to the concrete used in the field.
2. Limestone properties may degrade due to changes in the environment. For this reason, the model environment which consists of temperature, groundwater, rock confining pressure, and shaft concrete head, should be as similar as possible to field conditions. Arching occurs which prevents the full gravitational confining force from acting on the pier, (Ladanyi 1980), this should be considered when designing a model test.

Gupton et al (1982) used 8 inch core samples in which a 3 inch core hole was drilled. The 3 inch core was tested in unconfined compression. The 8 inch core with 3 inch core hole was then used as a model plug test. A non-metallic nonshrink, high early strength grout plug was cast into the donut-shaped core with foam rubber at the base to prevent end bearing. The grout plug was then sealed at the top and various loads were applied to simulate the concrete head in the drilled shaft. After two days of curing under the weights, the weights were removed. The model was failed by applying an axial load to the grout plug. The results from 5 such tests show a peak side friction value ranging from 40 to 50% of the ultimate unconfined compressive strength of the intact rock. These values seem high and are over twice those predicted using correlations based on unconfined compressive strength values reported by Williams and Pells (1981).

Kaderabeck (1981) employed cylindrical rock cores being vertically cast in reinforced concrete. The rock core was prevented from allowing end bearing and subsequently failed by axial loading. Tests of this type were called "Skin Friction Tests" and are plotted with other laboratory results shown on Figure A.3-2-1.

TESTS PERFORMED

12 SPLITTING TENSILE TESTS
12 UNCONFINED COMPRESSION TESTS
4 ROCK TRIAXIAL TESTS
3 SKIN FRICTION TESTS
2 ROCK DIRECT SHEAR TESTS

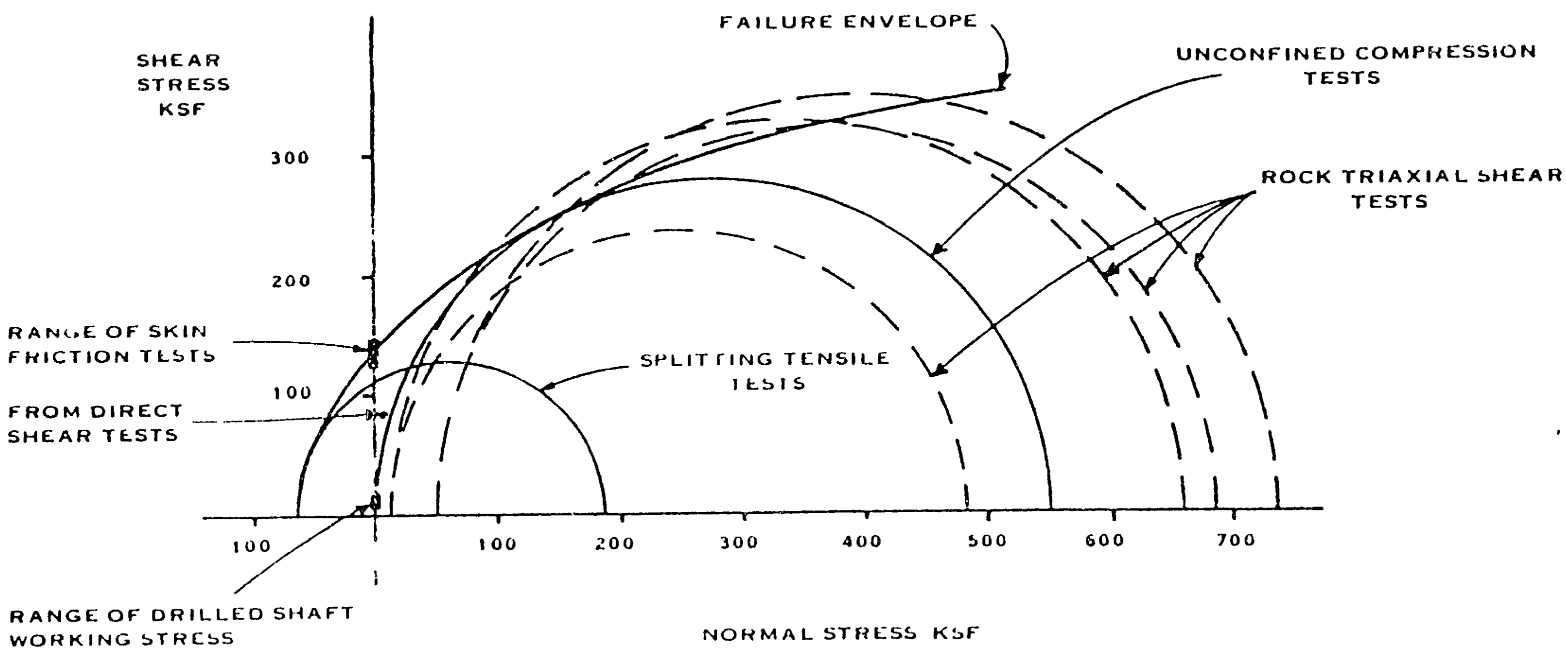


FIGURE A3.2-1 MOHR ENVELOPE OF LOWER FT. THOMPSON SANDSTONE STRENGTH (AFTER KADERABEK 1981)

APPENDIX 4

TYPICAL FIELD LOAD TEST INSTRUMENTATION

A4.1 LOCATION

Williams et al (1980) performed a variety of finite element analyses on the vertical stress distribution across the diameter of drilled shafts. They concluded the distribution of vertical stress throughout the diameter of the tested drilled shaft can be assumed uniform from below D/2 from the top of the shaft to above D/2 from the base, (D being the shaft diameter). This analysis was confirmed with actual field test data. This implies that instrumentation should not be placed within D/2 of the drilled shaft extremes. They also determined that the error in neglecting radial and circumferential stresses in deformation analysis will amount to an error less than 5% and suggest they be neglected for practical work. Thus, only vertical stress indicators are necessary to determine vertical deformation of the drilled shaft. Keeping this in mind, all instrumentation should be placed where specified by the engineer and will vary depending on the sub-strata profile.

A4.2 TYPES OF INSTRUMENTATION

There are mechanical and electrical types of instrumentation being used to monitor the movement and stress changes in test shafts. The most often used mechanical measure is a telltale. Electrical systems consist of strain gauges, used to measure shaft stress and end bearing stress. Mechanical systems are rugged and more suitable over the long run, but do not have the accuracy or automatic readout capabilities that electrical systems have. Electrical systems are more accurate giving instantaneous discrete readings which could be automatically recorded throughout the test. Since the systems are electrical they increase their accuracy through use of signal amplification. The major drawback in electrical systems is that they are very delicate and can be damaged through handling or through the installation procedure used. They must be tested prior to use and at times have been found to be unreliable. Typical electrical systems are not recommended for long term loadings, their calibration curves have been found to vary over time (Barker and Reese 1969).

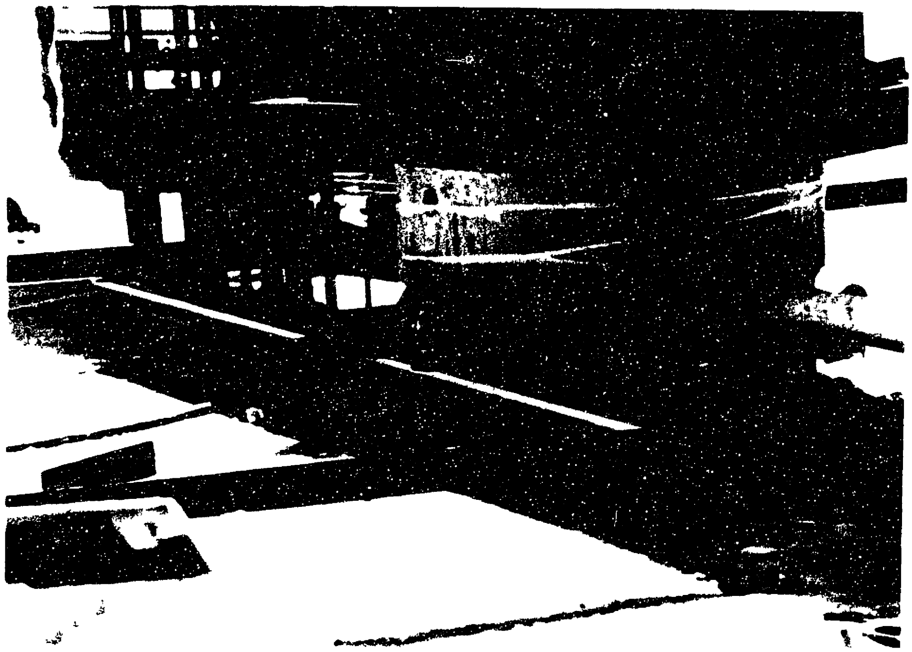
A4.3 MONITORING BUTT MOVEMENT

Butt movement is typically monitored using dial gauges and also with a wire and mirror. When dial gauges are used, 3 pieces of angle iron or similar metal is welded to the butt of the shaft at 120° centers, see Figure A4.3-1. Dial gauges are then supported from the reference beam and rest on the flat portion of the angle irons. Results from the 3 dial gauges are averaged to attain the butt settlement. A wire and mirror system consists of a wire supported on reinforcing bars, (or equal), set into the ground at least 10 ft from the test shaft center and a couple of inches from the casing at the butt. A spring is used to keep a constant tension on the wire so that temperature variations would not affect the readings. A mirror with a scale attached in the center is then bonded to the casing behind the wire. By lining up the wire with its image shown on the mirror a reading is taken from the scale.

A4.4 TELLTALES

Telltales once embedded into the shaft give deformation between the embedded point and the reference point with the use of dial gauges. The reference point could be a reference beam or the top of the shaft. The telltale as suggested by Reese and Hudson (1968) is shown in Figure A4.4-1. Telltales extending vertical to the dial gauge are suggested. To attain the high loadings for the drilled shafts in the South Florida area some projects utilized two hydraulic jacks which are centered on the top of the shaft. Telltales were then bent 90° and cased extending

INTENTIONAL DUPLICATE EXPOSURE



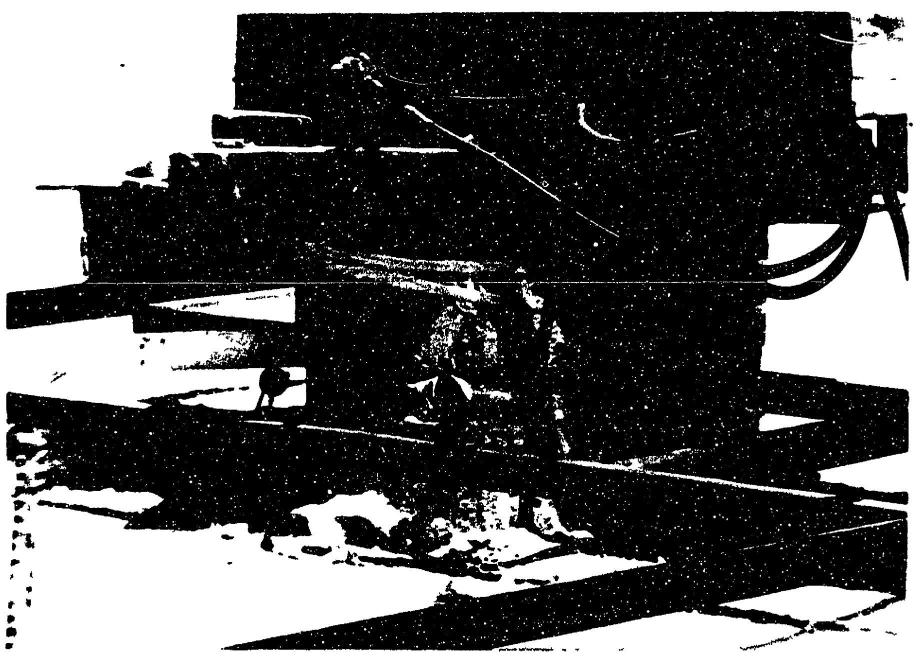
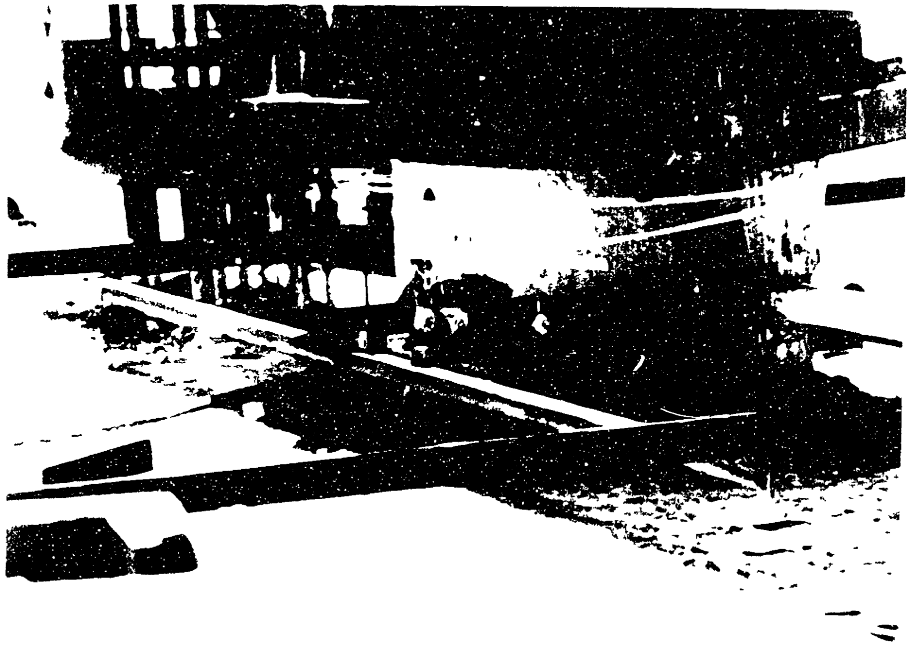
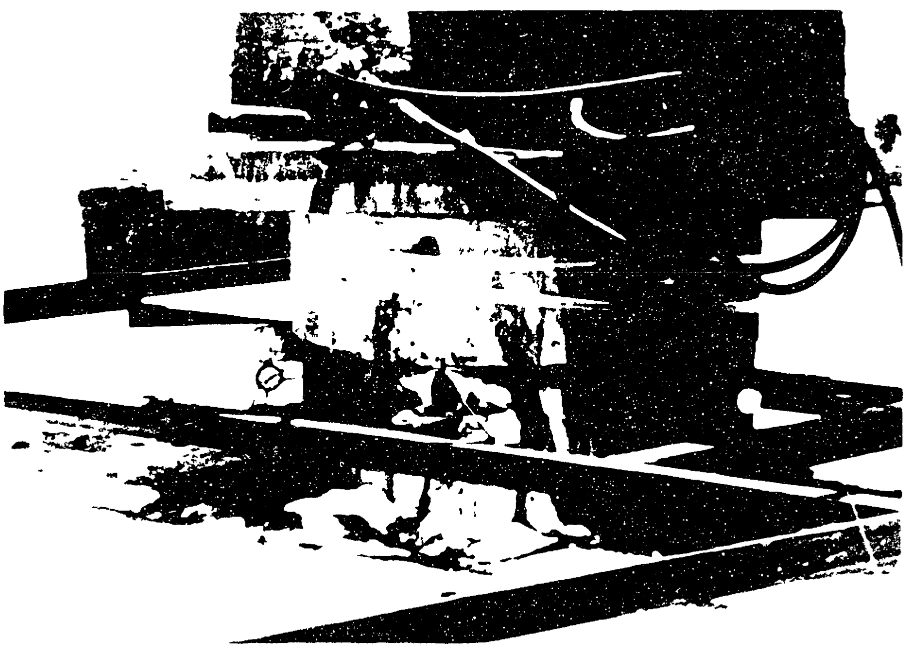


FIGURE A4.3-1 - BUTT INSTRUMENTATION





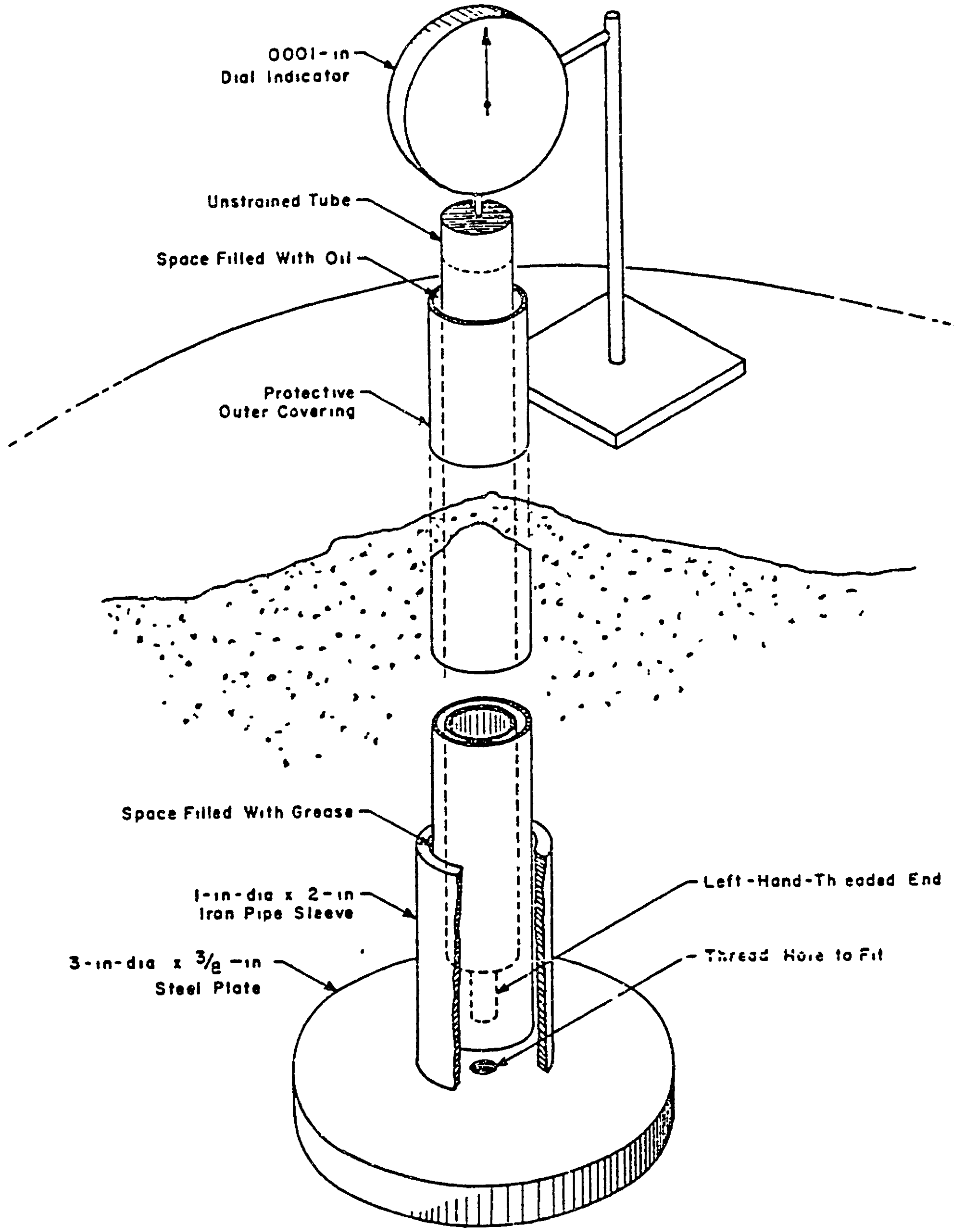


FIGURE A4.4-1 DETAILS OF THE ASSEMBLY OF A TELLTALE
(AFTER REESE AND HUDSON)

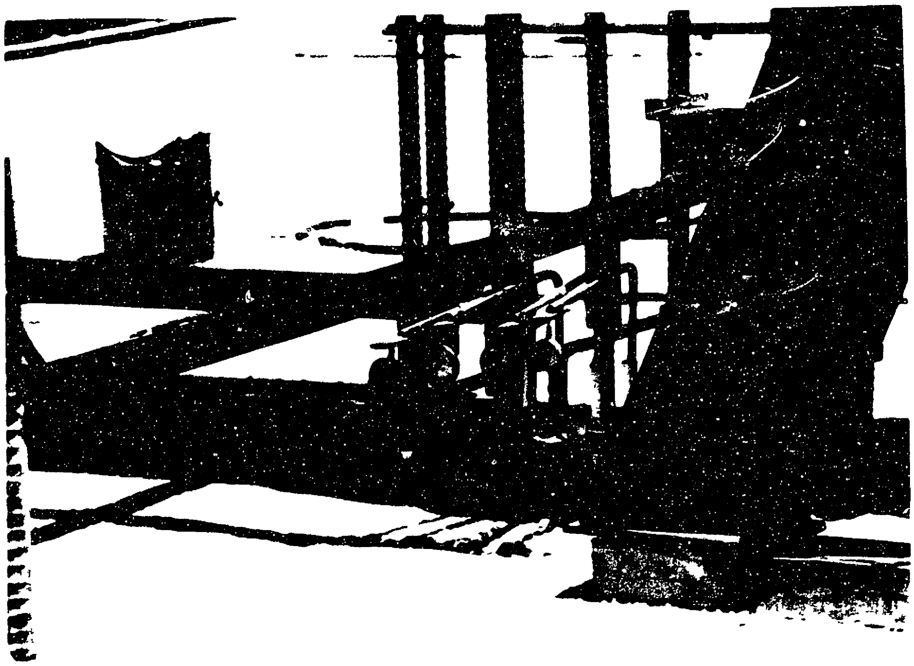
horizontally from the shaft. The deflections, when telltales are bent 90°, are taken from a flat plate welded to their end, See Figure A4.4-2.

Touma and Reese (1972) found that the temperature of the shaft drops from the high temperature it achieves during curing to that of the ambient medium in about two days. They found after that normalization the temerature, except for the top 10 feet of the shaft, is relatively constant. The temperature in the top 10 feet was found to oscillate with the cycles of air temperatures. It should be evident that temperature fluctuations which affect the top 10 feet of the drilled shaft can affect the steel being used for telltales. Barker and Reese (1969) state the largest source of error when using telltales, results from friction developing between the casing and the rod. These difficulties suggest that telltales are not adequate when a great degree of accuracy is required or if the expected deflections or expected relative deflections between telltales are small. When telltales are used, Barker and Reese (1969) suggests to employ 2 telltales for each level at opposite ends of the shaft to compensate for any possible shaft bending and to use telltales extending vertical to the dial gages.

A4.5 SHAFT LOAD CELLS

The most common load cell being used in South Florida is the Mustran cell, which is a multi-strain cell, See Figure A4.5-1. The Mustran cell, which was developed at the University of Texas at Austin is designed specifically for the analysis of drilled shafts being subjected to axial loading. The development and theory of the Mustran cell is documented and described in Barker and Reese (1969). The function of the cell is to measure the axial strain between its two end caps through the use of strained bars instrumented with foil strain gages, Figure A4.5-2. The cell column is adjusted such that the modulus of elasticity of the gauge equals that of the displaced concrete, See Figure A4.5-3. The major difference between the type 1 and type 2 Mustran cells as shown in Figure A4.5-3 is the range

INTENTIONAL DUPLICATE EXPOSURE



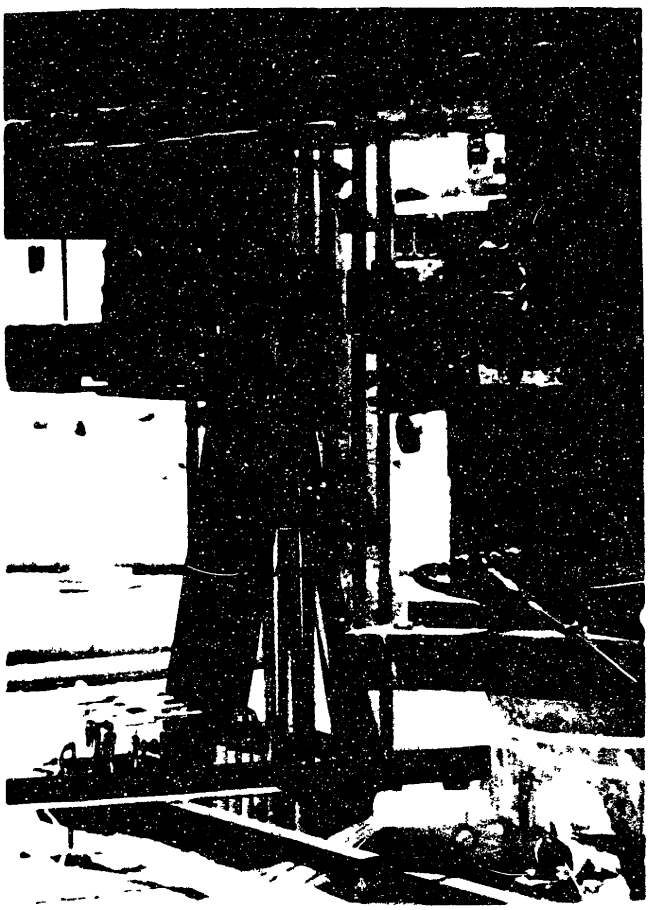


FIGURE A4.4-2 MONITORING TELLTALES WITH 90° BEND

INTENTIONAL DUPLICATE EXPOSURE

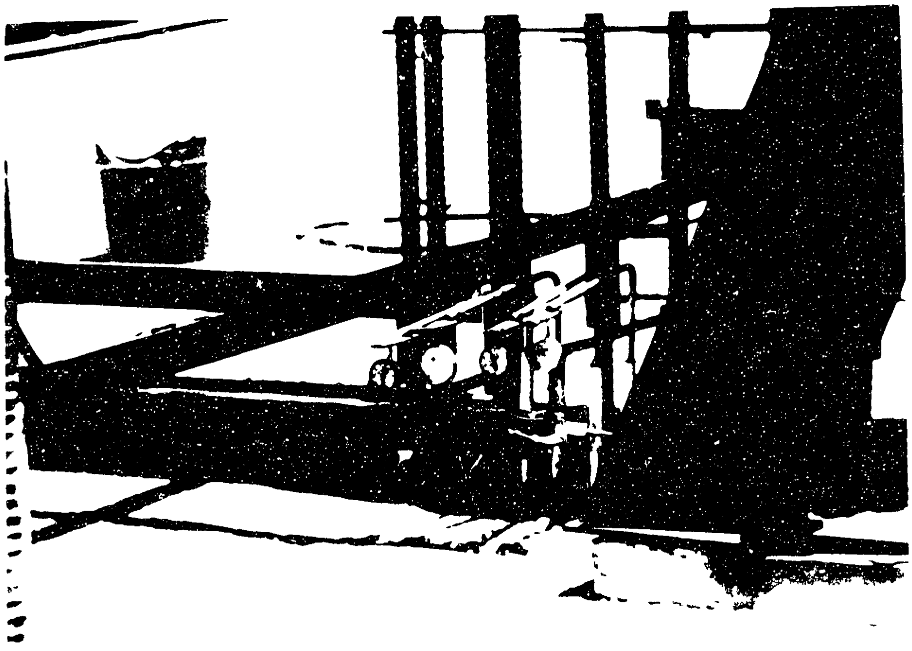


FIGURE V4.4-2 MONITORING TELLTALES WITH 90° BEND

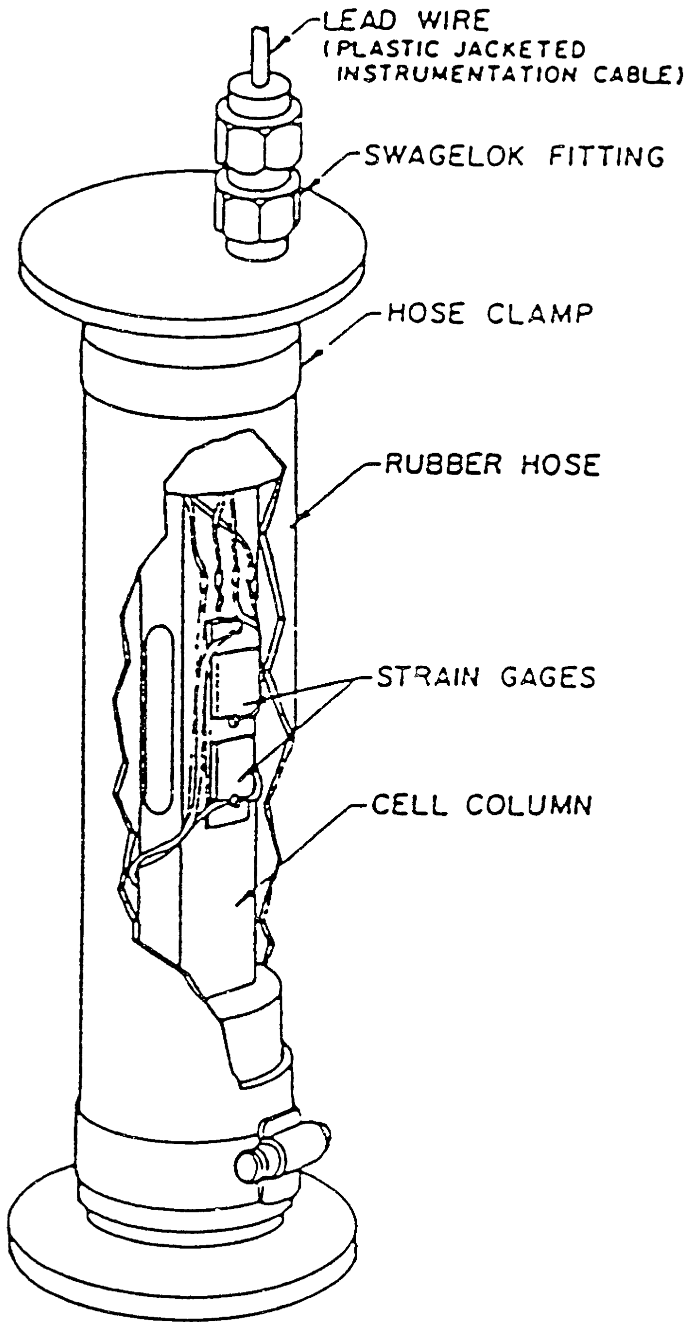


FIGURE A4.5-1 MUSTRAN CELL
(AFTER REESE 1979)

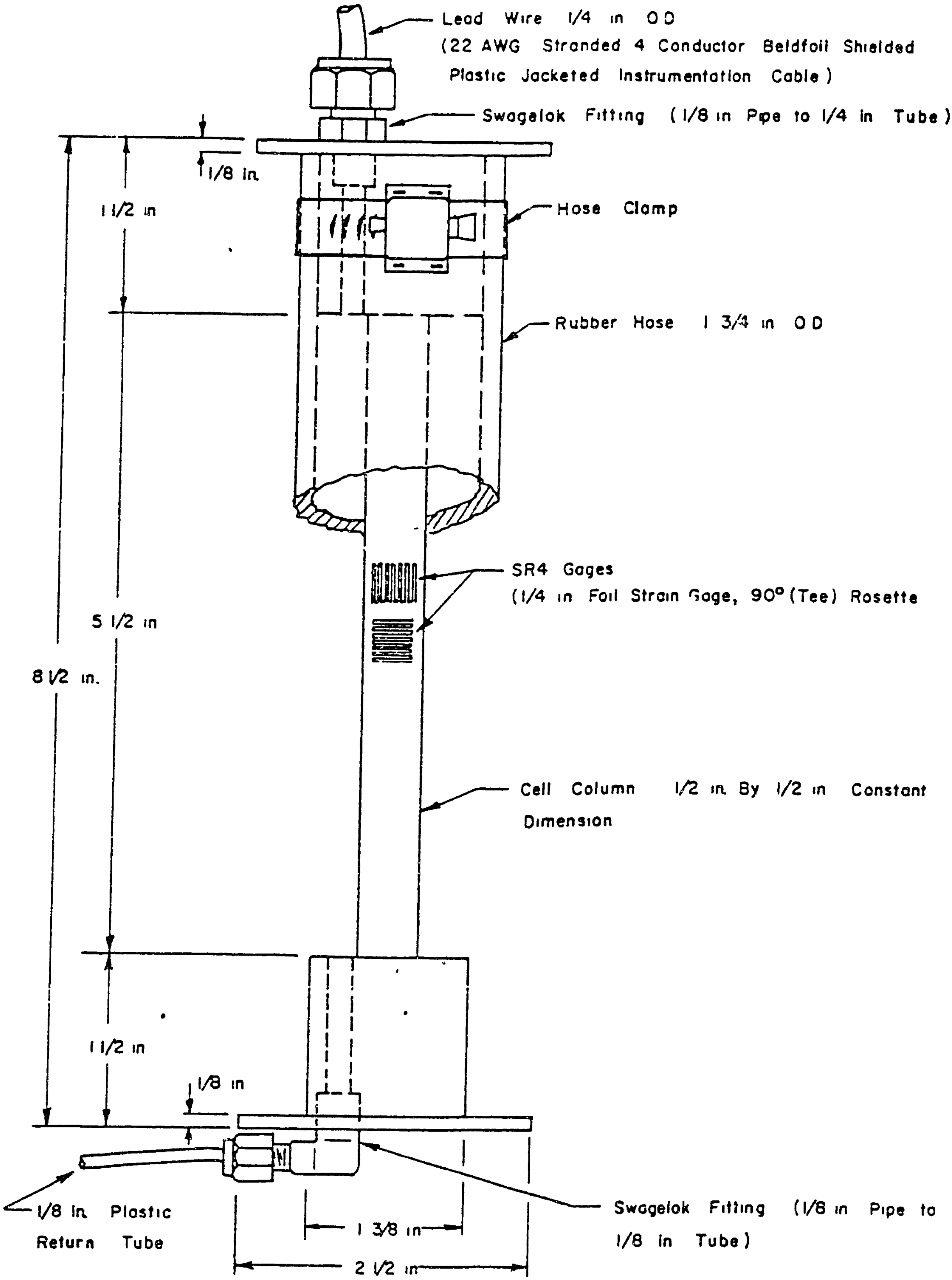


FIGURE A4.5-2 MUSTRAN CELL, TYPE 2
(AFTER BARKER AND REESE 1969)

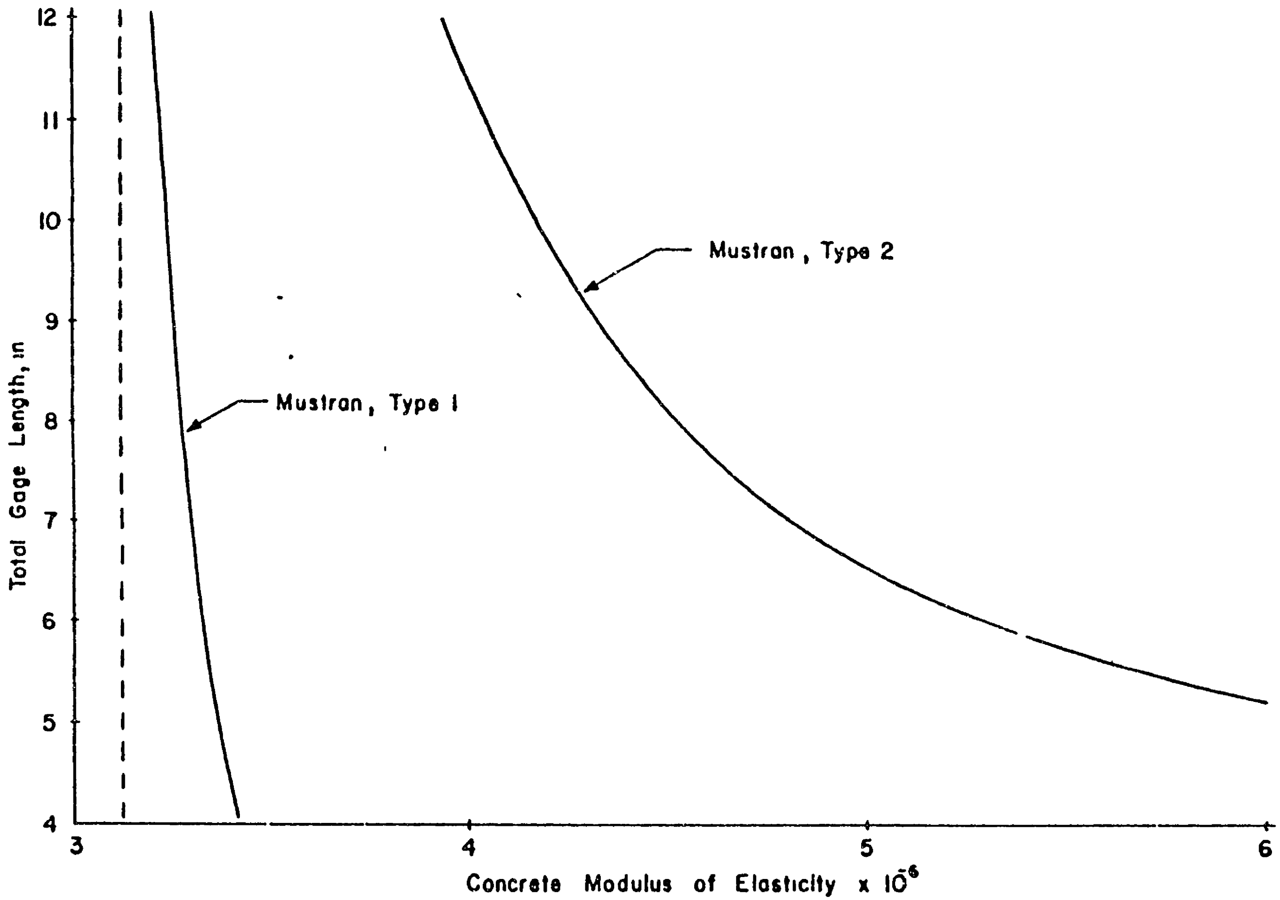


FIGURE A4.5-3 DESIGN CURVE FOR MUSTRAN CELLS

(AFTER BARKER AND REESE 1969)

of modulus. Construction details for the type 1 cell are shown in Barker and Reese (1969). The cell as developed provides temperature compensation, bending strain elimination and output amplification.

Waterproofing consists of a 1 5/16 inch ID rubber hose clamped to the end caps. For added protection against dampness, and collapse of the cell during concreting, the inside of the cells should contain a desiccant and be pressurized with nitrogen gas. The pressurization used is typically 20 psi during installation. For those cells not within the cased portion of the drilled test shaft, the pressure is raised to 40 psi just prior to concreting. This pressurization is to help prevent damage during concrete pouring and curing, and should remain pressurized for approximately 3 days after the shaft is poured. Since the calibration curves for the cells have been found to change with time, (Barker and Reese 1969), in shaft calibration is normally completed using the upper cells in the cased portion of the shaft. Employing cell pressurization, a desiccant and in-shaft calibration, along with prior testing of all cells prior to installation has yielded satisfactory results from Mustran cells in South Florida as well as many other areas throughout the states.

A4.6 BOTTOM HOLE CELL

The bottom hole cell is an apparatus which measures end bearing, and was also developed in the University of Texas at Austin. The bottom hole cell has performed well in field tests, Barker and Reese (1969), but it is expensive and can be hard to install. The cell as shown in Figures A4.6-1 and A4.6-2 employs three load cells 120° apart. The load cell output can be connected in series or separately. Barker and Reese (1969) suggest using separate load readings although in their tests the 3 cells were connected in series. With the proper use of the bottom hole cell one can easily and accurately differentiate between end bearing and side friction. A typical calibration and loading curve are shown in Figure A4.6-3.

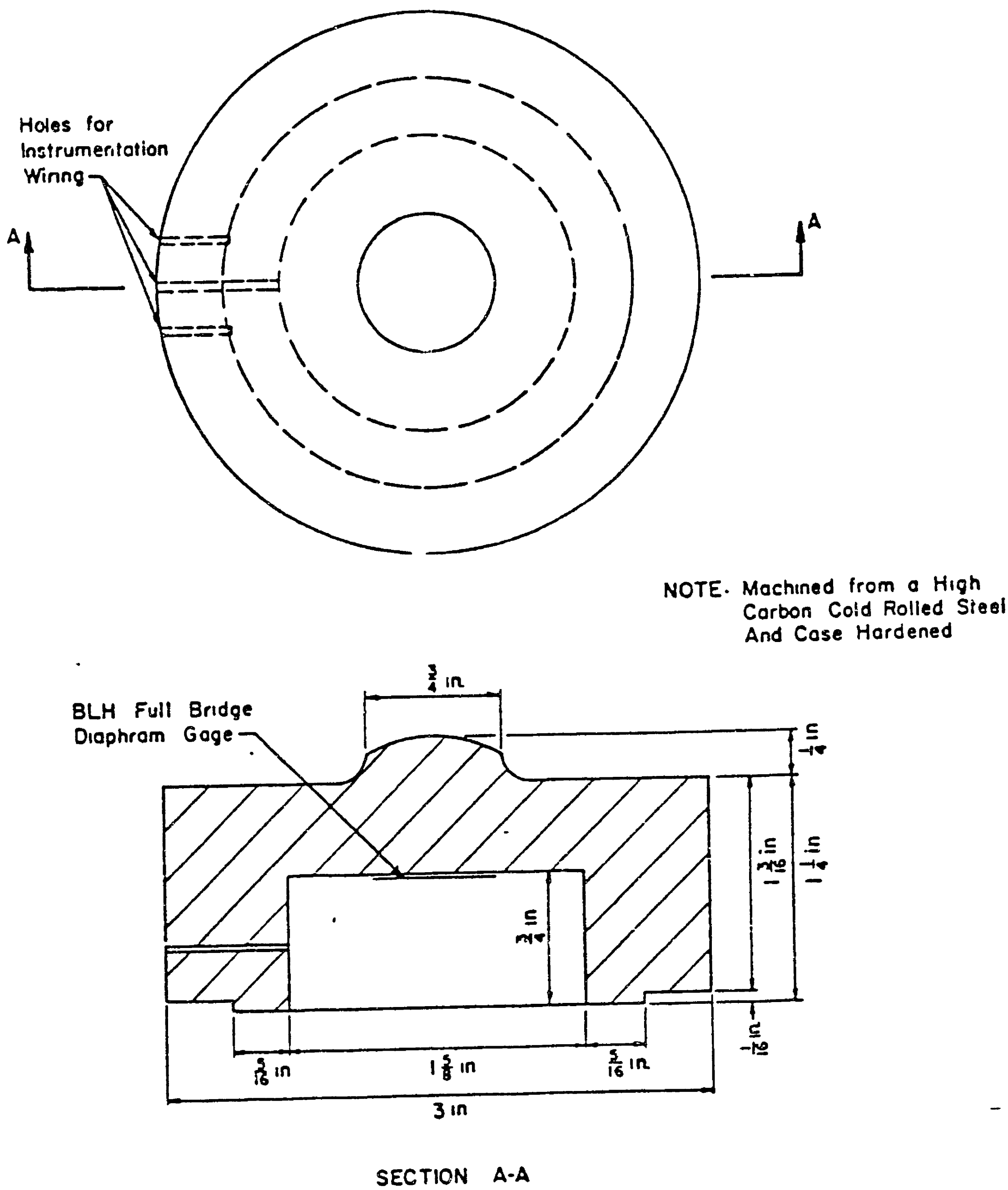
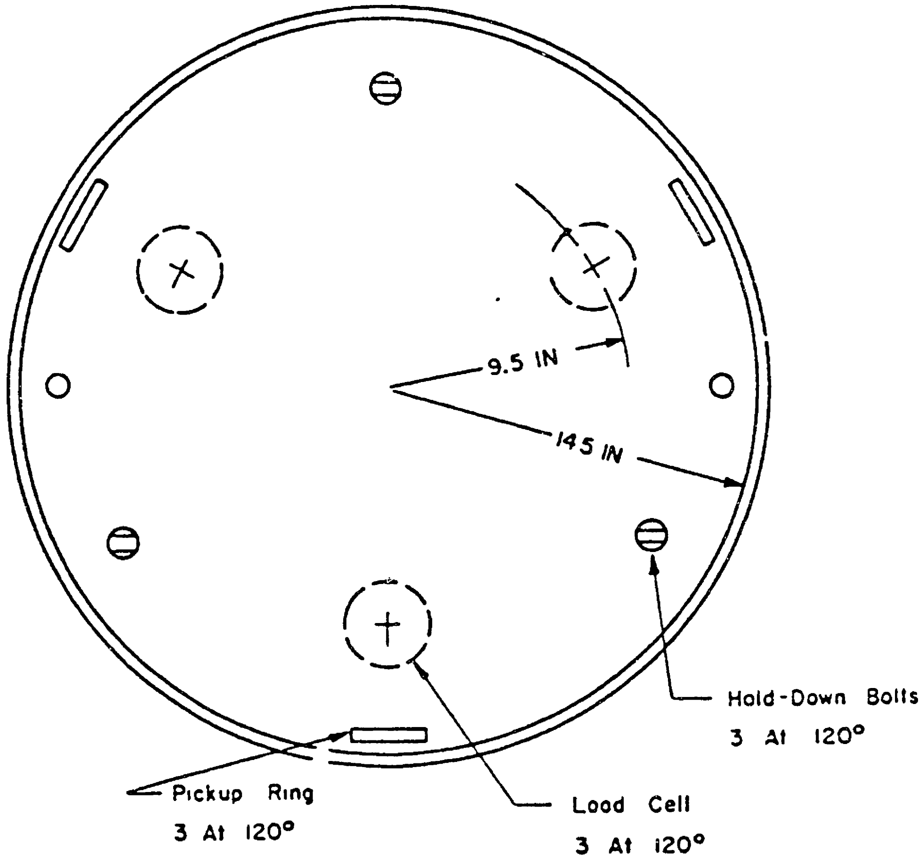


FIGURE A4.6-1 DIAPHRAGM LOAD CELL FOR BOTTOM HOLE CELL

(AFTER BARKER AND REESE 1969)



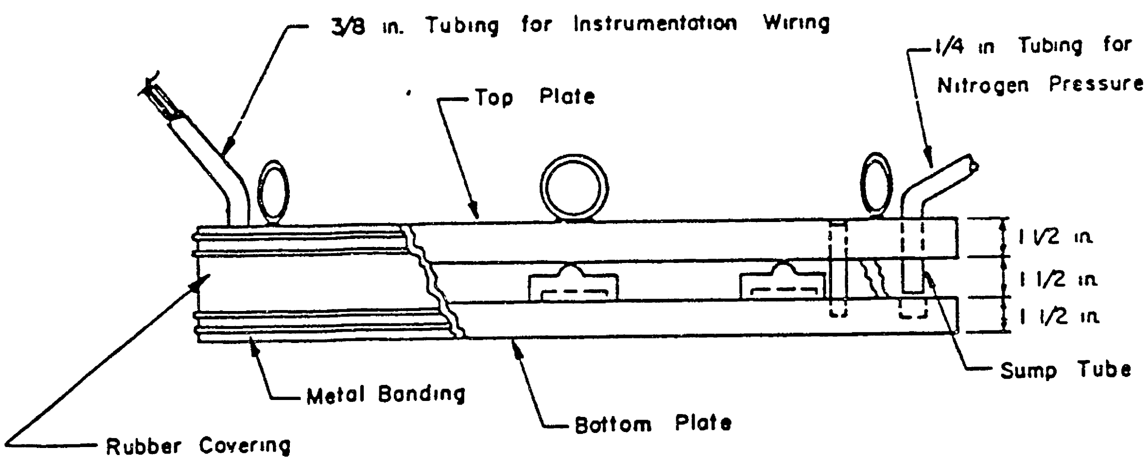
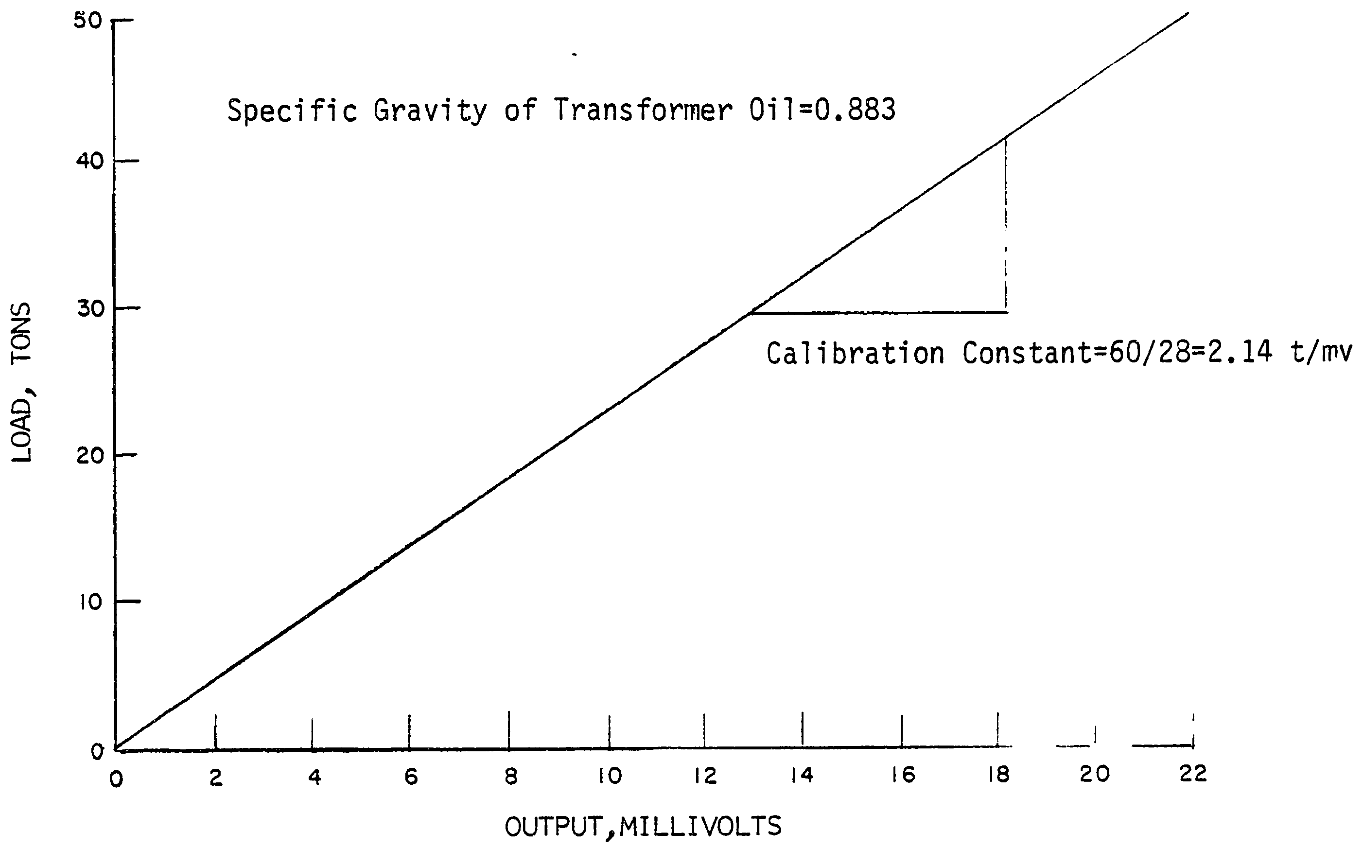
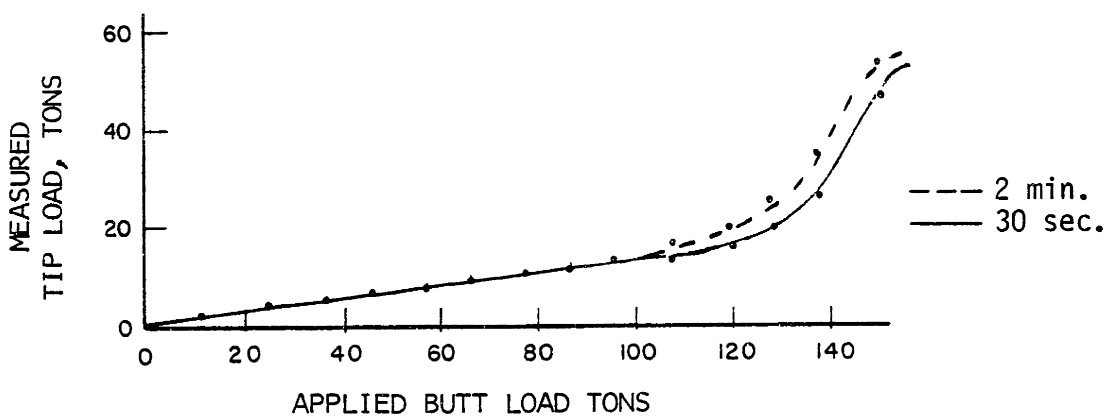


FIGURE A4.6-2 BOTTOMHOLE CELL
(AFTER BARKER AND REESE 1969)



a) CALIBRATION CURVE FOR BOTTOMHOLE CELL



b) LOAD INDICATED AT SHAFT TIP BY BOTTOMHOLE CELL

FIGURE A4.6-3 CALIBRATION AND LOADING CURVES
FOR THE BOTTOMHOLE CELL
(AFTER BARKER AND REESE 1969)

APPENDIX 5

FACTORS AFFECTING FIELD TESTS

The most often used field testing procedures employed in field investigations are subsurface borings. Boring techniques which have been used successfully in the area consist of wash boring; Standard Penetration Testing; rock coring with a variety of diameters, (the most common being a 4 inch diameter and the NX size); static and dynamic cone penetrations; and downhole geophysical logging. The location, depth, and type of method to be employed is determined by the Geotechnical Engineer and is dependent on the information desired.

A5.1 TENSION PLUG TESTS

Field model tests typically consist of tension plug tests. It is well documented in the literature that tension side resistance in rock sockets is less than that for compression, Freeman et al (1972), Horuath and Kenney (1979), and Webb and Davies (1980). The tendency for the shaft diameter to shorten and elongate during a pull out test would result in the interfacial normal stress between the shaft and socket to decrease, thereby decreasing the frictional component of shaft resistance. The dilantancy effect for rough shafts is also expected to be affected, although, to what degree has not been estimated. The bonding between the concrete and the rock would be expected to remain approximately the same in tension as in compression. Unlike compression shafts, Webb, and Davies (1980) show through field results, that the length to diameter ratio of the tension shaft will affect the ultimate unit shaft resistance.

Since tension adhesion is typically less than compression many authors state designing on tension plug tests give conservative results. This has been proven in massive rocks, but where a thin rock layer exists with respect to socket length, and a highly eroded rock or sand is below the rock layer, a possibility of a lower compressive adhesion due to anisotropic rock properties or possible voids should be investigated. When socketing drilled shafts into relatively thin rock strata, underlain by a weaker substrata, additional caution should be used with any assumptions.

With grout plug tests as with all model tests, a scale-size effect should be taken into consideration. Since the size of the pores in the porous rock remain constant, a fine grained grout may penetrate the rock shaft sides further than the coarser aggregate concrete being used in the test and production shafts. From past experience in the South Florida area, it is suggested for future tension plug tests to use a diameter as large as practical, a grout or concrete as close as possible to that of the production concrete, and to apply the same head as would be applied if concrete was above the plug. A suggested scheme is shown in Figure A5.1-1.

Several projects in South Florida have employed tension plug tests. At one project where actual concrete was used, the results obtained for side friction were scattered and low. It was suggested lack of equivalent concrete load prevented the shaft tension socket from forming a good bond with the rock face (no head was applied on top of the grouted plug). At another project, Nyman (1980), obtained relatively high side friction results, 100 -169% of the average ultimate compressive frictional resistance. These tests (Nyman 1980) were employing short (approximately 2 feet long) grouted plug tests, See Figure A 5.1-2.

A5.2 FIELD LOAD TESTS

A5.2.1 GENERAL

Field load tests are performed for any or all of the following three purposes:

1. To prove a design.

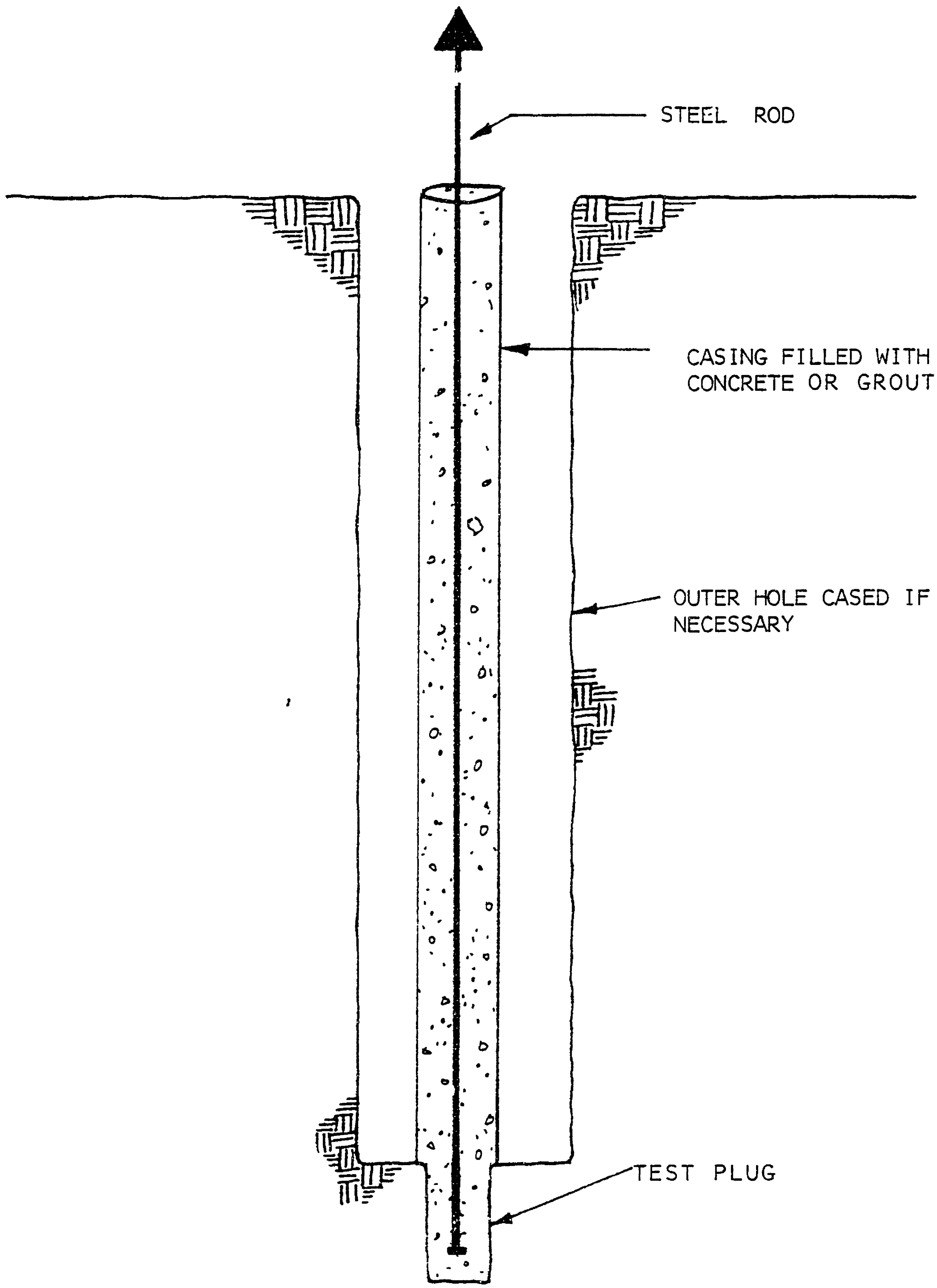


FIGURE A5.1-1 SUGGESTED SCHEMATIC FOR GROUT PLUG TESTS.

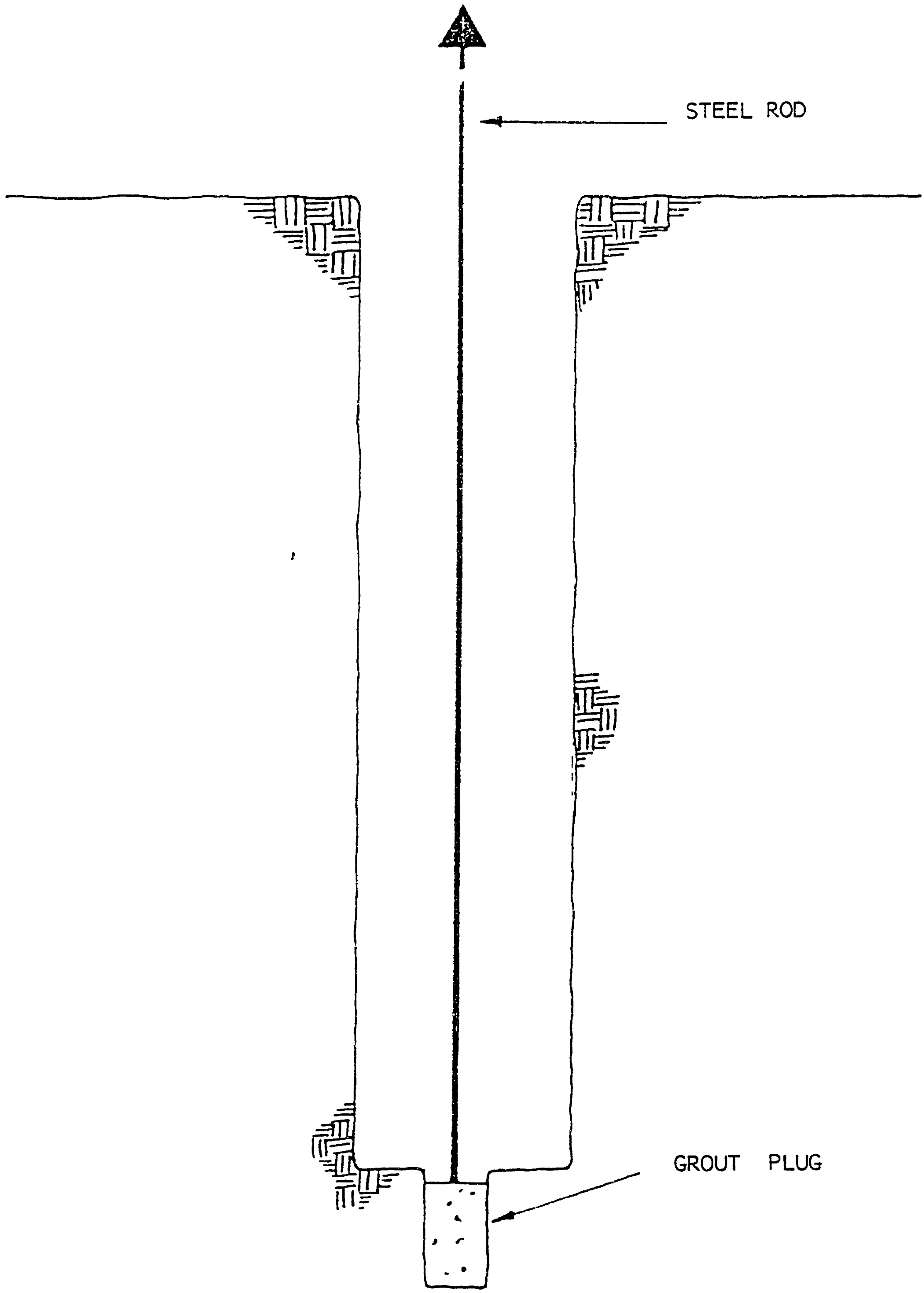


FIGURE A5.1-2 - SCHEMATIC OF THE GROUT PLUG TEST ARRANGEMENT (AFTER NYMAN 1980)

2. To gain information on the rock-shaft load transfer parameters

3. To evaluate installation procedures, and to allow the installation logs to be reviewed by the contractors prior to bidding.

Load testing to prove a design typically does not load the drilled shaft to failure. Loading the drilled shaft to failure allows the engineer to predict the maximum allowable load. This allows him to decide on the amount of conservatism through employing a desired factor of safety to the evaluated ultimate load. Thus, a proven design, unless shown to have a factor of safety approximately equal to that desired, is not necessarily the most economical. Since drilled shaft foundations are new to the South Florida area, a conservative design should be employed. Keeping this in mind the engineer should also consider that soil mechanics does not benefit, nor does the owner realize this optimum utility, if the designer does not know how conservative the design is. Due to the excessive cost of test shafts, and the lack of precedent in the area, the first few test sites have been unable to load the test shafts to failure. It is hoped with the information now available future load tests will be designed with a better estimate of the rock load transfer parameters and thereby be allowed to reach their failure load prior to reaching the load capacity of the testing apparatus.

By allowing a test shaft to fail, and incorporating accurate instrumentation, the load carrying parameters can be backsolved from the test data. With an accurate estimation of the rock parameters the most cost efficient design with the desired amount of conservatism could be designed.

Load testing of rock socketed drilled shafts takes a considerable amount of time. It has been found that load testing the design and letting the contractors review the drilling logs prior to accepting bids results in a more efficient process. Load testing should be scheduled and favorably analysed prior to constructing the production shafts. Failure to have an acceptable design can lead to time delays resulting in higher construction costs for the project.

It should again be noted that because of the excessive cost of drilled shaft load tests, and depending on the size of the project, it may be more economical to proof test a design that is overly conservative, rather than performing a few load tests to develop the most cost efficient design per shaft. An added benefit in proof testing a conservative design is since the shaft has not been failed, it can also be used as a production shaft, and stress readings can be taken over time to monitor actual performance with time.

A5.2.2 FIELD LOAD TESTING

In South Florida, to the authors knowledge, the Quick Load Test Method, (ASTM D1143-74.4.7), has been used in the testing of drilled shafts. This method saves time and has yielded desireable results. Due to the lack of cohesive soils, and high porosity of the rock, creep effects are not expected to play a major role and the quick method is applicable. Creep and rock strength should be studied more carefully if the rock layer where the shaft is placed does not extend several diameters below the shaft tip.

Shown in Figure A5.2.2-1 is the load test plan view of the load test L-4. The testing system can employ 1 jack as shown in Figure A5.2.2-2 or two smaller capacity jacks in parallel, Figure A5.2.2-3. Typical load test specifications have been outlined in the ASTM Code D1143-74 (1981), and in the Drilled Shaft Manuals (1977).

A5.2.3 TYPE OF LOAD TESTING

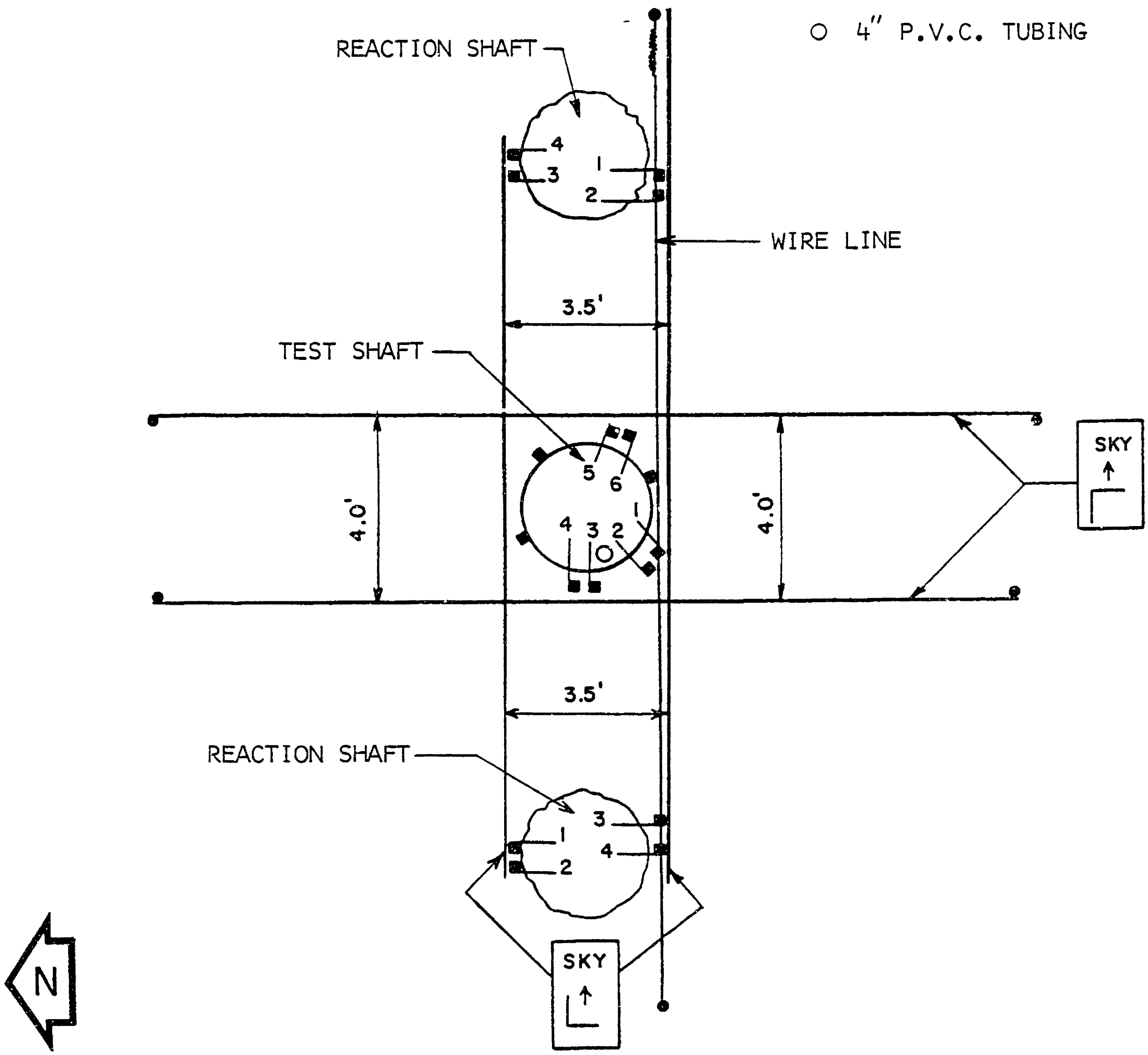
Due to the high water table and porosity of the rock in this area, it is not practical to manually inspect the base of the shaft for suitable end bearing. As a

JACK SYSTEM USED

2-18 800 TON CAPACITY
RICHARD DUDGEON JACKS
(BROOKLYN, NEW YORK)
HYDRAULICALLY JACKED
AGAINST 2 REACTION SHAFTS
NO PROOF LOAD GAGE
WAS USED.

KEY

| TELLTALE NUMBER
TELLTALES
GAGES FOR CASING
REBAR SUPPORTS
4" P.V.C. TUBING



GAGES FOR TEST SHAFTS HAVE
A MAGNETIC ATTACHMENT TO ¼"
THICK ANGLES WELDED TO REBARS
IN GROUND.
REACTION SHAFT GAGES ARE
CLAMPED TO ANGLES WHICH ARE
WELDED TO THE FIXED ANGLES.

FIGURE A5.2.2-1 LOAD TEST PLAN VIEW

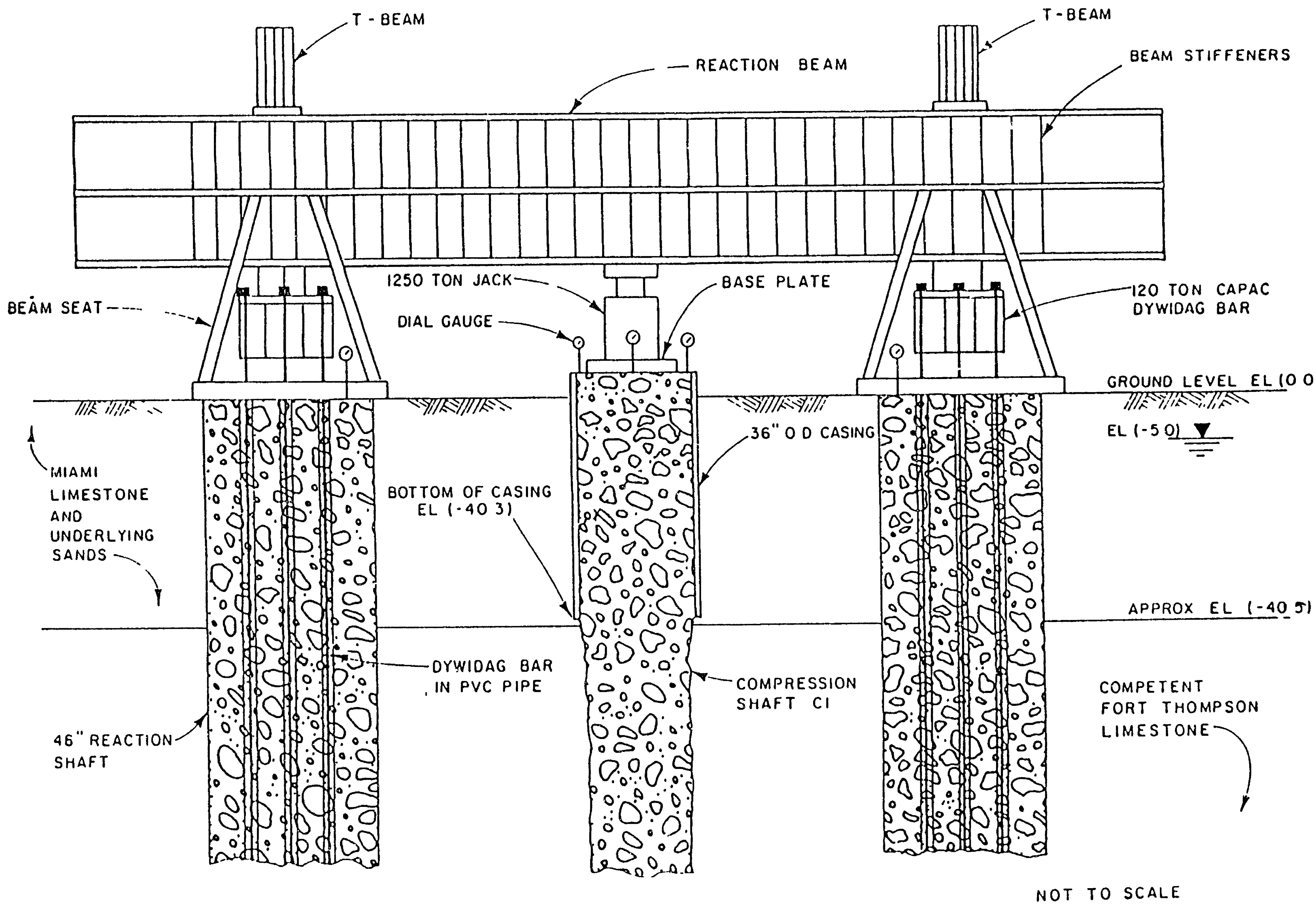


FIGURE A5.2.2-2 LOAD TEST (SIDE VIEW)
(AFTER O'BRIEN AND LOGAN 1981)

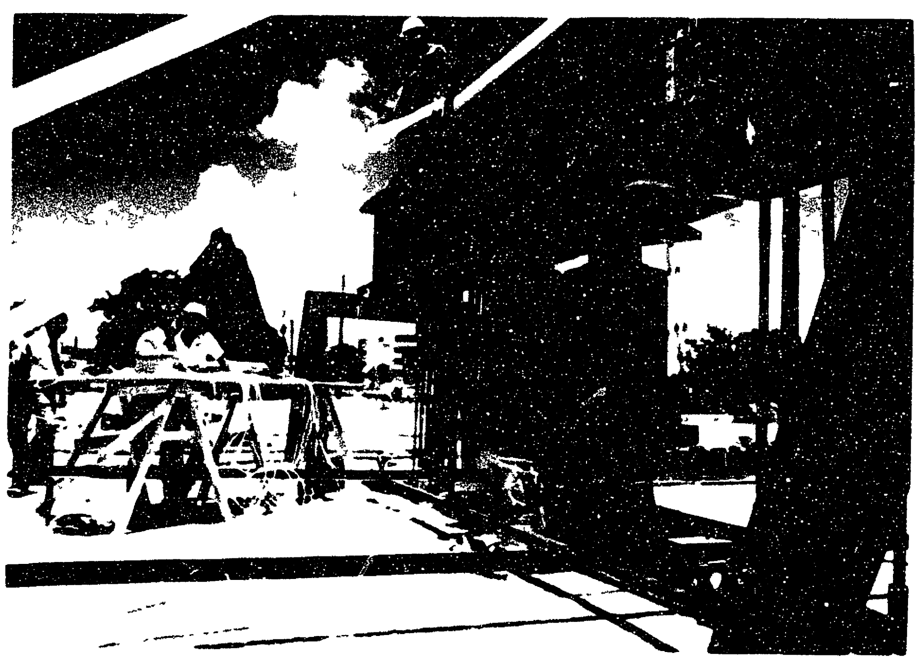
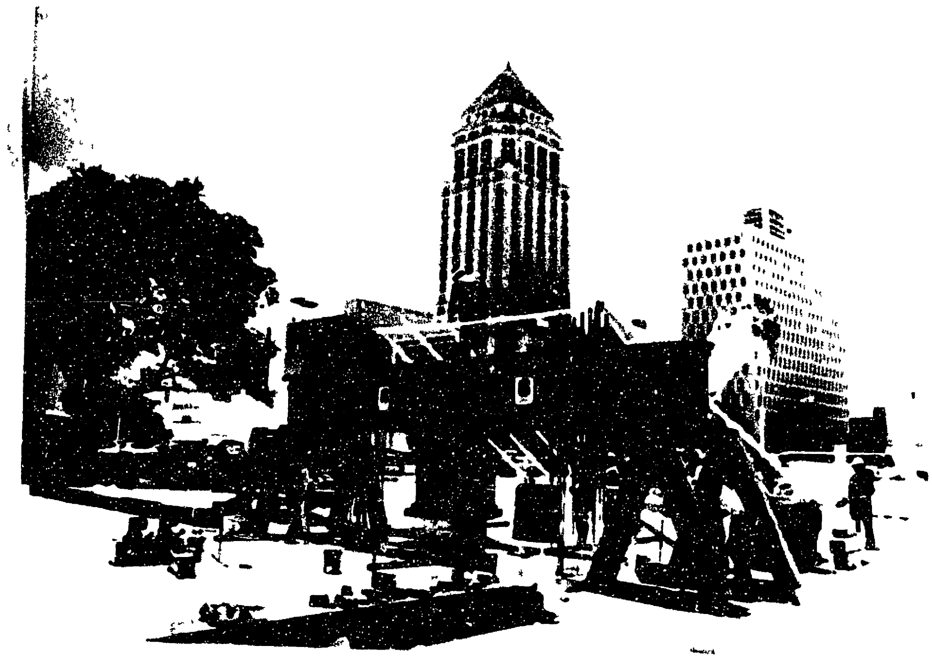


FIGURE A5.2.2-3 LOAD TEST CONFIGURATION

INTENTIONAL DUPLICATE EXPOSURE



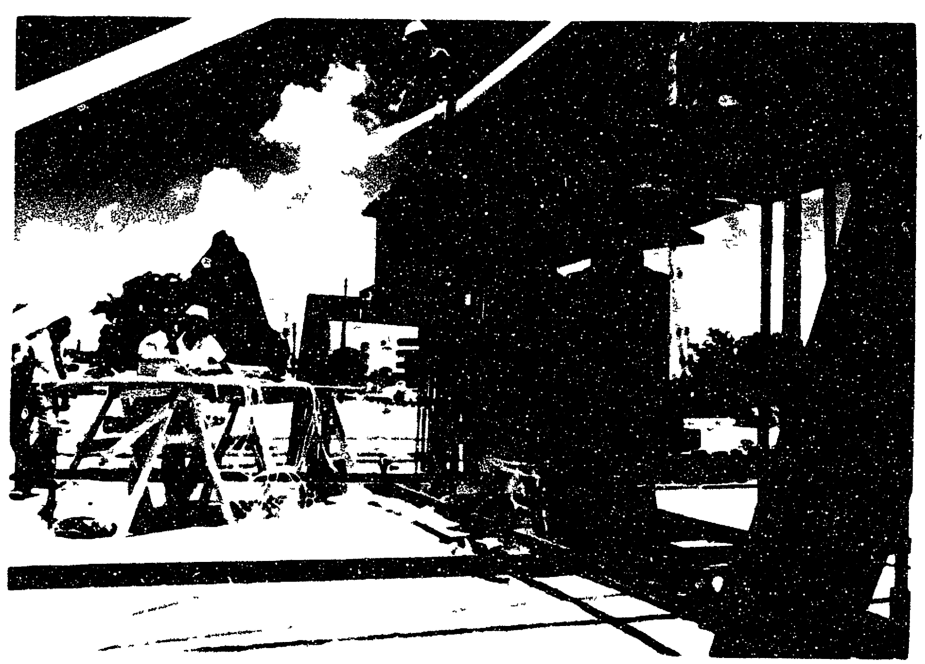
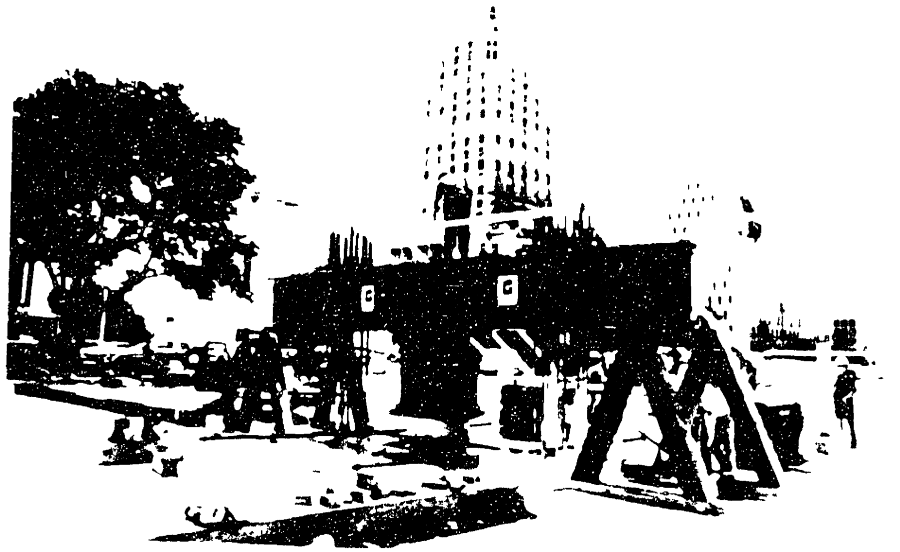


FIGURE A5.2.2-3 LOAD TEST CONFIGURATION



result most shafts are designed to carry their load through side friction. When incorporating end bearing either a separate end bearing load test should be performed, or appropriate instrumentation which could separate end bearing from side adhesion should be incorporated.

Friction Only Load Tests

Friction only load tests either use a method to prevent end bearing or are loaded such that end bearing is negligible. As mentioned, by inducing failure, one could determine the ultimate adhesion capacity, then employ the desired factor of safety to achieve the desired design. Through over designing a frictional shaft (with respect to the loading system) this failure cannot take place. For this reason, it is suggested to case the shaft from the butt to the top elevation desired to be tested, and to employ a method to prevent end bearing below the area to be tested. The casing is used to prevent frictional resistance from the upper rock and/or soils and has worked well in South Florida. The system is then loaded to failure with an appropriate loading apparatus. The required side movement to develop ultimate side friction has been reported by many authors and has typically been found to be less than 0.25 inch.

Various methods to prevent end bearing in test shafts have been used successfully in the past. The most common is using a styrofoam or polystyrene plug, typically 12 inches thick (Horvath et al 1980, Thorne 1980, Pells and Rowe 1980, Pells and Turner 1979). This 12 inch plug would be lowered into the hole prior to setting the reinforcing cage (unless attached in some manner to the base of the cage). The plugs employed are designed to support the weight of the wet concrete with little deformation but to deform substantially with higher loads, see Figure A5.2.3-1. Webb and Davis (1980) used a steel plate which was fixed to the reinforcing bars and suspended above the base of the hole. Watt et al (1969) used a similar plate, resting on a hollow cylinder of polystyrene, which had thin metal

INTENTIONAL DUPLICATE EXPOSURE

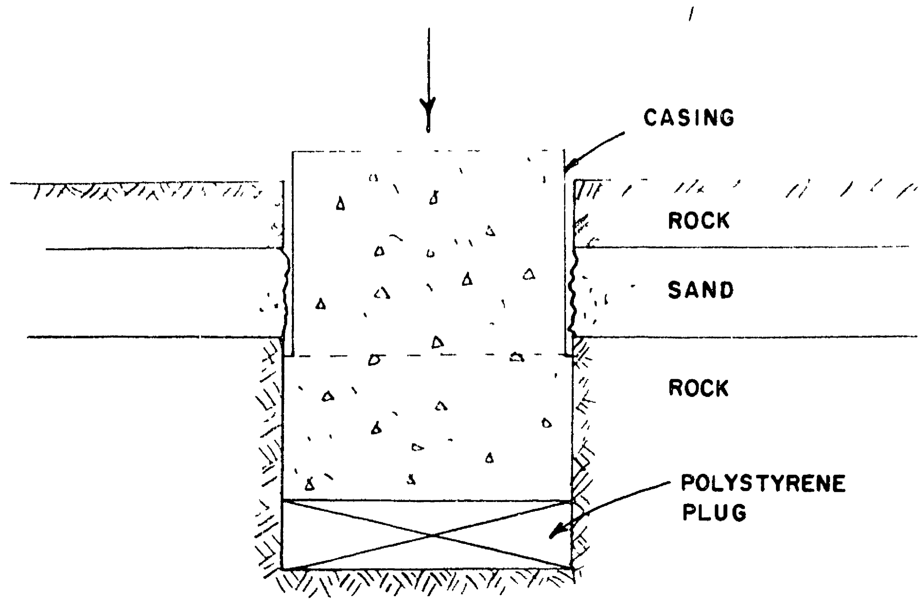


FIGURE A5.2.3-1 SHEAR SOCKET

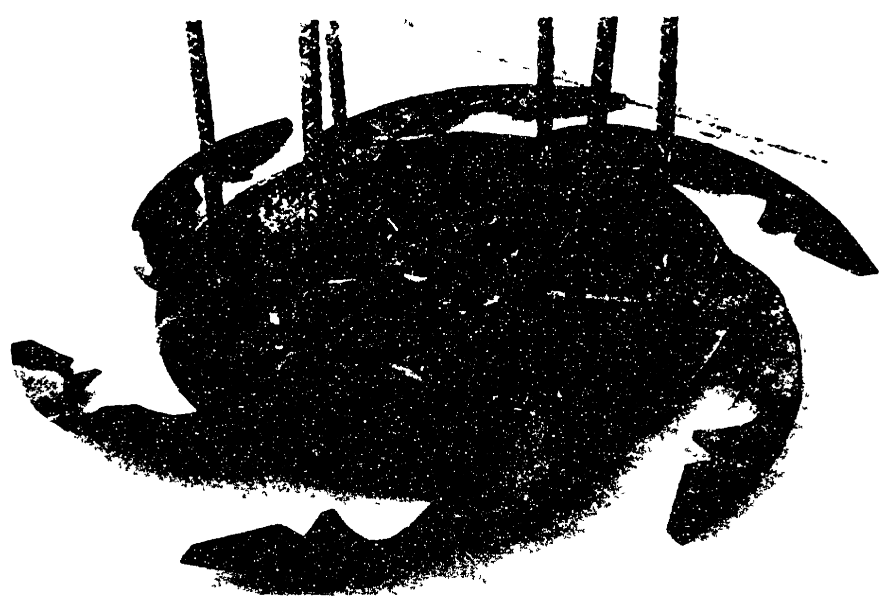
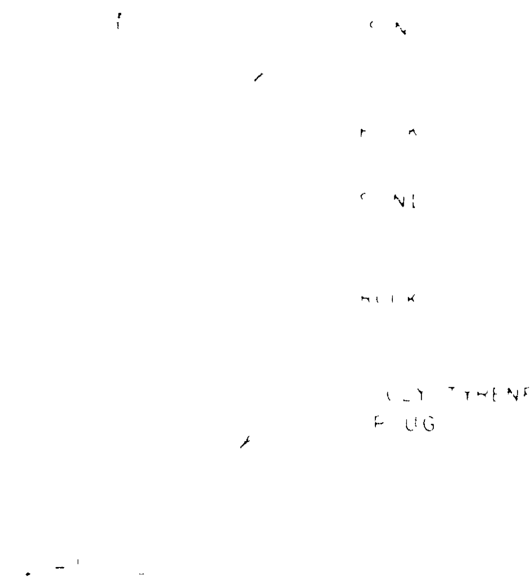


FIGURE A5.2.3-2 PLATE, AT BASE OF SHAFT TO ENSURE
VOID BENEATH SHAFT
(AFTER WATT et. al. 1969)

INTENTIONAL DUPLICATE EXPOSURE



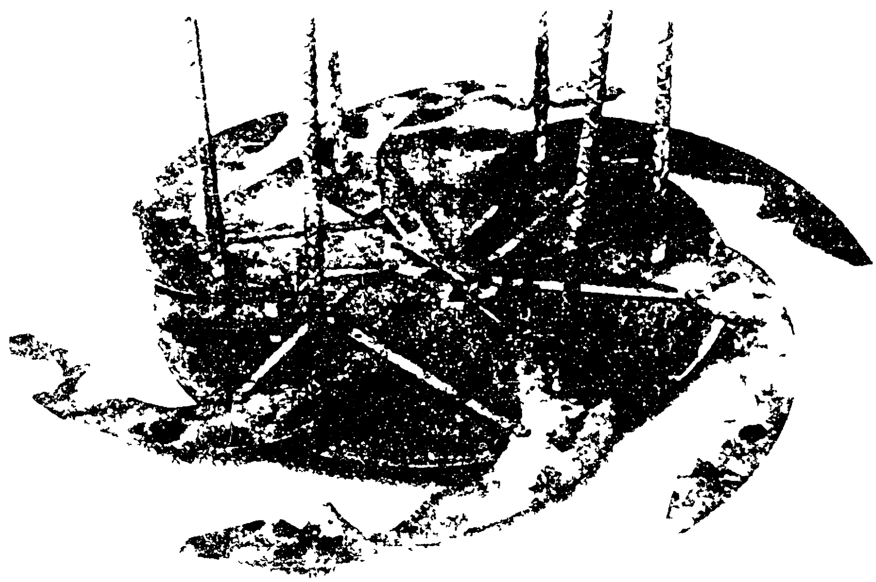


FIGURE A5.2.3-2 PLATE AT BASE OF SHAFT TO ENSURE
VOID BENEATH SHAFT
(AFTER WATT et al. 1969)

shutters that were spring receeded and expanded into the soil at the base (See Figure A5.2.3-2). Moore (1964) designed an isolation plug employing shear pins which sheared at a load in excess of that of the plug and wet concrete poured above the plug, See Figure A5.2.3-3.

Since any plug in South Florida would typically be under water, installation problems would have to be evaluated and dealt with in the field. It is suggested to employ a type of isolation plug whose combined density is larger than the water or drilling mud it will replace. As long as water could freely flow out of the plug, it is believed the rock is sufficently porous to prevent water pressure from developing end resistance.

<u>End Bearing Only Load Tests</u>

End bearing tests are performed by casing the test shaft its entire depth. Side friction on the casing could be prevented either by using double casing in cohesionless soils or by drilling the hole larger than the casing size in soils or rock with sufficient cohesion or apparent cohesion to prevent hole distortion, see Figure A5.2.3-4. Ultimate end bearing typically occurs at settlements of 10 to 15 percent of the diameter. For this reason, a settlement criteria rather than ultimate load is normally the controlling factor.

<u>Combination Load Test (Both End Bearing and Side Friction)</u>

With the instrumentation available any single load test could accurately be divided into side friction and end bearing. The problem with a combination test in rock sockets are the very high loads required to obtain a good settlement end bearing graph. It should be noted that end bearing settlement information is normally only needed for deformations of 1 to 2 inches since a settlement criteria of less than 1 inch normally applies.

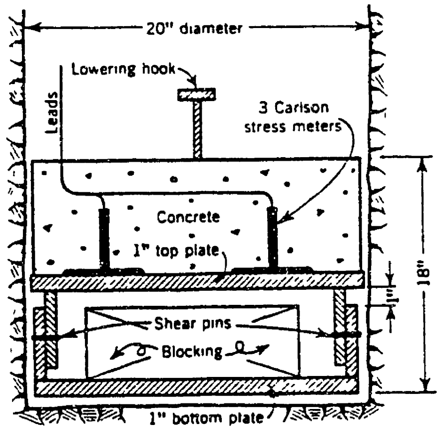


FIGURE A5.2.3-3 ISOLATION PLUG, TO TEST THE LOAD-CARRYING ABILITY IN SIDE FRICTION (AFTER MOORE 1964)

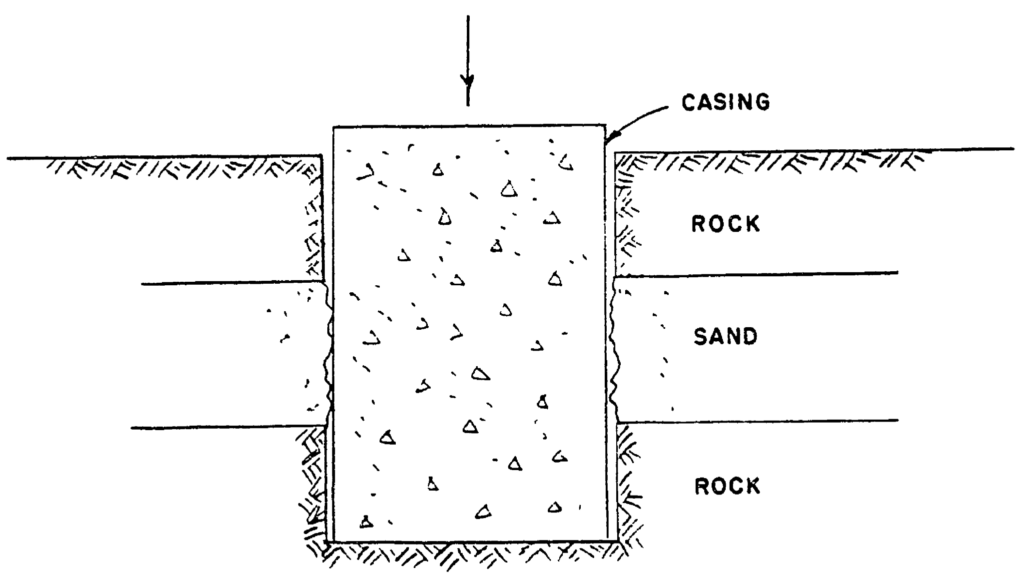


FIGURE A5.2.3-4 END BEARING SOCKET

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