LONG-TERM OVERTURNING LOADS
ON DRILLED SHAFT FOOTINGS

by

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IMPLEMENTATION STATEMENT

The results of this research have now reached the point at which full implementation can be achieved. The study was begun because of the belief among some Texas Highway Department engineers that the present method of designing drilled shaft footings to resist overturning loads was overly conservative. This belief has now been confirmed.

Research Report 105-3, the last of the reports dealing exclusively with the effects of relatively short term, static overturning loads, compares the theory developed and correlated in Research Reports 105-1 and 105-2 with full-scale tests of drilled shaft footings. A new design procedure is presented which is extremely easy to apply, since it is based on the use of design curves rather than the cumbersome application of equations. The "Tentative Design Procedure" is given in Research Report 105-3 on pages 45 and 46 and an example problem is worked on page 49. The effects of both dynamic and long-term sustained loads are considered in Reports 105-4 and 105-5F, respectively.

The design curves included in 105-3 allow the selection of a particular size footing as a function of the loads acting on the footing and the characteristics of the soil. The methods of determining the desired soil parameters limit to some degree the full application of the design curves by some Texas Highway Department Districts. The design must be based on an estimate of the cohesion and angle of shear resistance of the soil. The most desirable way of determining these parameters is by use of the triaxial test. Since only a few THD Districts make wide use of this test method, a section of Research Report 105-3 has been devoted to more approximate methods which include THD and Standard penetrometer tests. The use of these tests will probably dictate some conservatism on the part of the designer, but will still represent a considerable improvement in our current methods.

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ABSTRACT

A theory which will predict the ultimate resistance of a drilled shaft footing to overturning loads was presented in Research Report 105-1 and was correlated with model tests reported in Research Report 105-2. The results of full-scale tests on drilled shaft footings were presented and compared to a "Tentative Design Procedure" in Research Report 105-3. In the research presented herein, long-term constant loads were applied to model and full-scale footings placed in soils which ranged from soft clay to sands. The purpose was to determine if the application of sustained overturning loads might cause excessive time-dependent footing movement. The results of these tests are presented in graphs of footing rotation as a function of time and are compared to results obtained from creep tests on triaxial specimens. The long-term loads are compared to the "pull-over" static loads which were reported previously, in Research Report 105-3. Suggestions are made regarding design soil strengths for use in limiting footing movement resulting from sustained overturning loads.

INTRODUCTION

As part of a three-year study to develop a usable design procedure for drilled shaft footings subjected to all types of overturning loads, a number of model and full-scale drilled shaft footings were subjected to constant long-term overturning loads. This is the fifth in the series of papers documenting the complete study. A theory which predicted the ultimate resistance of a drilled shaft footing to static overturning loads was presented in Research Report 105-1.[1*] This theory was correlated with model tests which were reported in Research Report 105-2[2] and was used to develop a tentative design procedure which was reported and compared with full-scale tests in Research Report 105-3.[3]

All work reported to this time has dealt with the displacements of drilled shaft footings under gradually increasing, short-term static loads. In the research reported herein, long-term constant loads were applied to model and full-scale footings placed in soils which ranged from very soft clays through stiff sandy clays to clean sands. The purpose of this phase of the study was to determine if the application of sustained overturning loads might result in excessive time-dependent deformations. The results of these tests are presented in graphs of footing rotation as a function of time and are compared to laboratory creep tests on triaxial specimens.

The long-term loads are compared to the "pull-over" static load tests which were reported previously.

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*Superscript numbers refer to corresponding numbers in the Selected References found at the end of this report.

TEST PROCEDURES

Model Tests

The seven model footings which were tested were four inches in diameter by twelve inches in depth, with the load applied as shown in Figure 1. The rotation of the footing was measured by means of dial gages accurate to 1/1000 of an inch placed at two points on the loading arm. The positioning of these dial gages is shown by Figure 1. Readings were made at various time intervals, as dictated by the rate of movement of the test footing. A redundant measurement of displacement was achieved by the measurement of the change in height of a "target" fixed to the loading cable. A cathetometer accurate to 1/100 of a centimeter was used to measure the target height. This measurement was used in those cases when the top dial gage lost contact with the loading arm.

The procedure used in placing the footings and soils for the model tests was the same as that given in Research Report 105-2. The two soils tested were 20-30 mesh Ottawa Sand and a laboratory clayey sand produced using, by weight, a mixture of 33% Trinity Clay and 67% concrete sand. Physical characteristics of these soils and other details of individual tests are given in Table 1.

Full-Scale Tests

Six full-scale footings were tested, all of which were two feet in diameter by six feet in depth. A 12 WF 120 column was bolted to the top of the footing, and the load was applied using a dead-load method as shown in Figure 2. The footings were placed using typical drilled shaft procedures: the holes were drilled in the earth using a 24-inch auger,

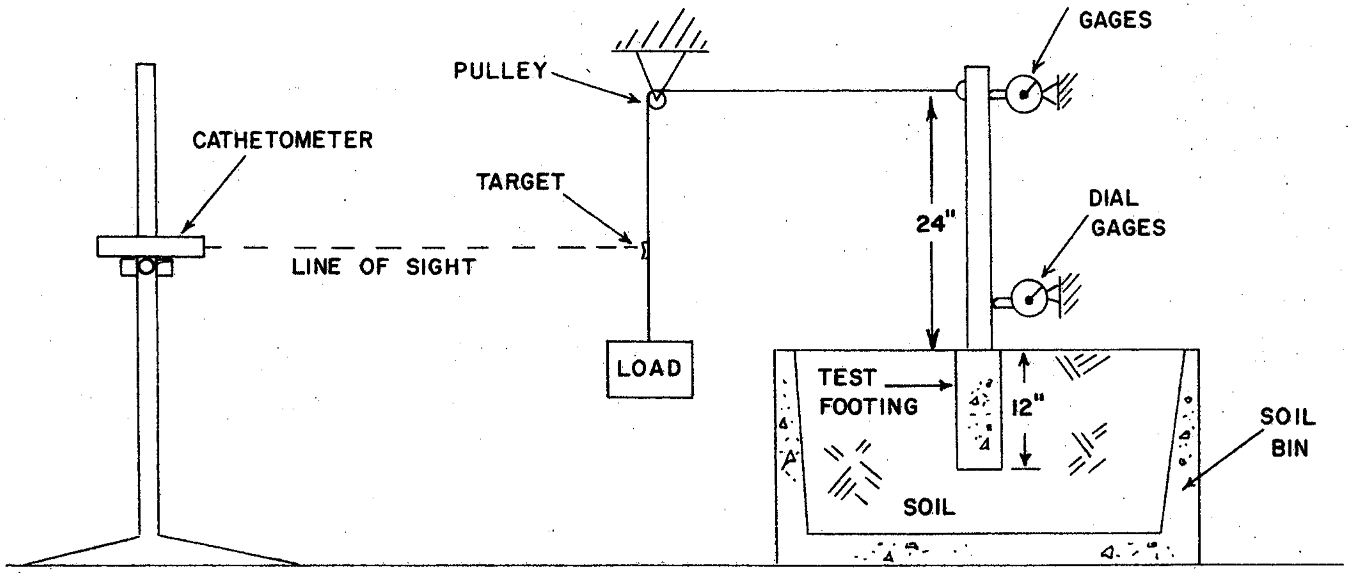


FIGURE 1, MODEL TEST LOADING AND RECORDING SYSTEM

TABLE 1. SUMMARY OF TESTS

TEST NUMBER	SOIL	FOOTING SIZE	HORIZONTAL LOAD (lbs)	DURATION OF LOADING (days)	ROTATION		5° ROTATION LOAD (lbs)	SOIL PARAMETERS		
					AFTER 1 DAY	FINAL		c (psf)	ϕ (deg)	γ (pcf)
S1	Ottawa Sand (20-30)	4- X 12-in.	22.5	203	0°51'	1°28'	30	0	37	109
S1	Ottawa Sand (20-30)	4- X 12-in.	22.5	42*	1°10'	1°26'	30	0	37	109
S2	Ottawa Sand (20-30)	4- X 12-in.	15.0	319	0°34'	1°43'	30	0	37	109
S3	Ottawa Sand (20-30)	4- X 12-in.	7.5	60	0°04'	0°04' (Stable)	30	0	37	109
C1	33% Trinity Clay 67% Concrete Sand	4- X 12-in.	50.0	8**	8°	15°33'	91.7	673	5.25	138.5
C2	33% Trinity Clay 67% Concrete Sand	4- X 12-in.	25.0	314	2°30'	2°51'	91.7	673	5.25	138.5
C3	33% Trinity Clay 67% Concrete Sand	4- X 12-in.	15.0	210	0°20'	0°33'	91.7	673	5.25	138.5
NAV 1	Sand	2- X 6-ft.	4,600	320	0°40'	0°54' (Stable)	9,200	See Appendix		
BRY 1	Sandy-Clay	2- X 6-ft.	6,200	290	0°40'	2°29'	12,400	"		
BRY 2	Sandy-Clay	2- X 6-ft.	6,200	180	0°53'	2°00'	12,400	"		
GAL 1	Soft Clay	2- X 6-ft.	4,200***	180	---	3°13'	5,500	"		
GAL 2	Soft Clay	2- X 6-ft.	2,800	273	0°54'	1°32'	5,500	"		
GAL 3	Soft Clay	2- X 6-ft.	1,400	93	0°07'	0°11' (Stable)	5,500	"		

*Test disturbed.
**Test stopped due to excessive rotation.
***4,200 lb. load was not maintained.

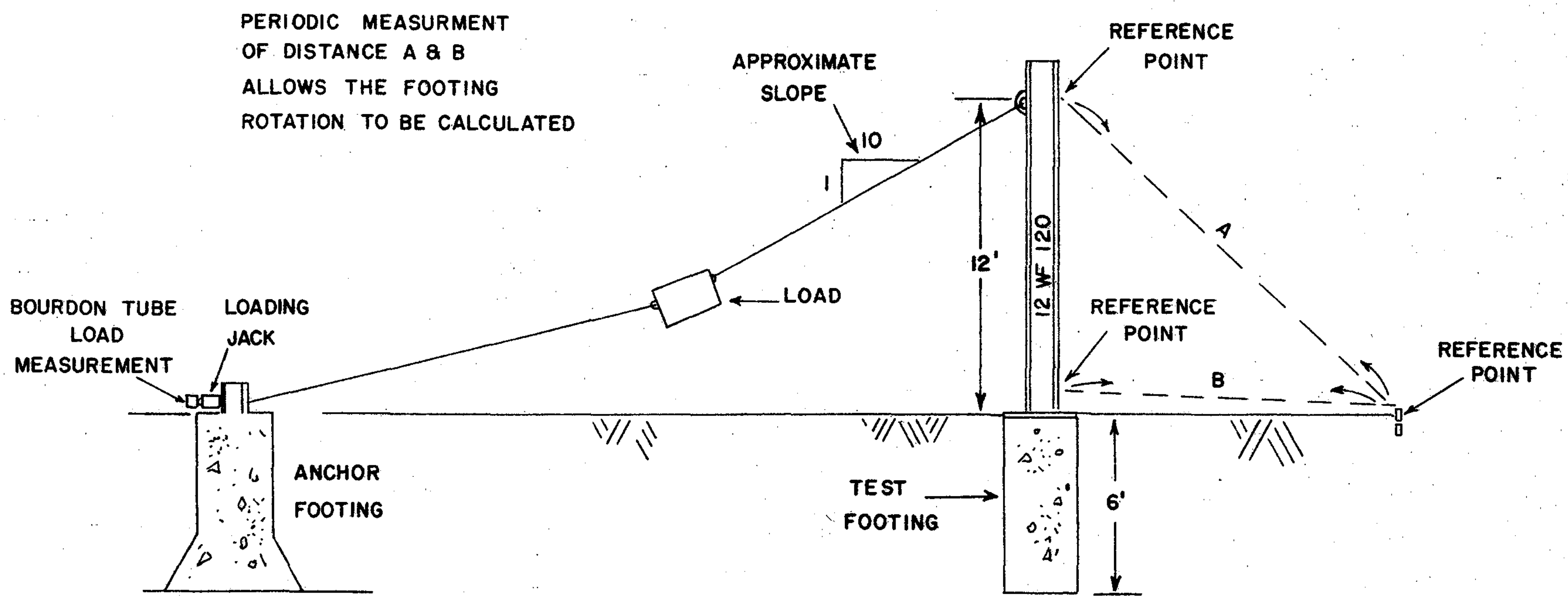


FIGURE 2, FULL-SCALE TEST LOADING AND RECORDING SYSTEM the footing cage was positioned in the hole, and the concrete was placed. Full-scale footings were placed at three test sites. The soils at these sites were a fine, clean sand; a stiff, sandy clay; and a soft clay. They are designated: Navasota Sand, Bryan Sandy Clay, and Galveston Clay, respectively. Physical characteristics of these soils are given by Figures 16 through 18 in the Appendix. The initial load was placed on the footing by means of a prestressing jack on the anchor end of the loading cable. By determining the angle that the load cable made with the horizontal, at the point it connects to the WF column, the horizontal load on the support may be calculated. By determining the height of the dead load when the horizontal load has reached a specified value, the load can be maintained by keeping the height of the dead load constant. Periodic checks of cable load were made using the prestressing jack with a bourdon-tube gage. The rotation of the footing with time was determined by periodically measuring the distance between two points on the loading arm and a reference point to the rear of the footing. By observing the variation in these two distances (A and B in Figure 2), the rotation of the footing was calculated. To guard against possible destruction of the ground reference point, an additional reference point was carefully hidden at some known position behind the primary reference.

The variation in the soil parameters of cohesion, angle of shear resistance, and unit weight were determined when the footings were placed. These footings were placed near the pullover tests reported in Research Report 105-3.

TEST RESULTS

Model Tests

The results of the model tests are presented in Figures 3 and 4. The top graph gives footing rotation versus elapsed time in hours for the first 25 hours of load, and the bottom graph gives footing rotation as a function of the elapsed time in days on a logarithmic scale.

For the Ottawa Sand tests, the loads applied were 75%, 75%, 50%, and 25% of the static overturning load. As seen in the lower graph, the 25% load became stable after the first few hours of loading, while the footing rotation due to a 50% load was still increasing after 330 days. At the time the test was discontinued, the footing rotation had reached a little over 1 1/2°. Under the 75% load, which was placed on Tests S1 and S1', the footing rotated approximately 1° at the beginning of the test and then gradually increased up to a total rotation of about 1 1/2° after 30 days of loading. At that time, S1' may have been disturbed: the rotation increased abruptly during a three-day period, and at some time during this period contact with the dial gages was lost. Test S1 was discontinued after 200 days of loading and appeared to be very stable between 100 and 200 days.

Three tests were conducted on the two laboratory clayey sand bins. It was intended to place 50% and 25% of the 5° rotation load, respectively, on two footings, based on the 5° rotation load of 102 lbs. which was obtained on this soil in earlier tests[2]. However, when a load of 50 lbs. was applied to the first footing (Test C1), it rotated nearly 15°, achieving 5° rotation in about 12 minutes elapsed time. Subsequently, a new footing (C2) was installed in the bin, and a load of

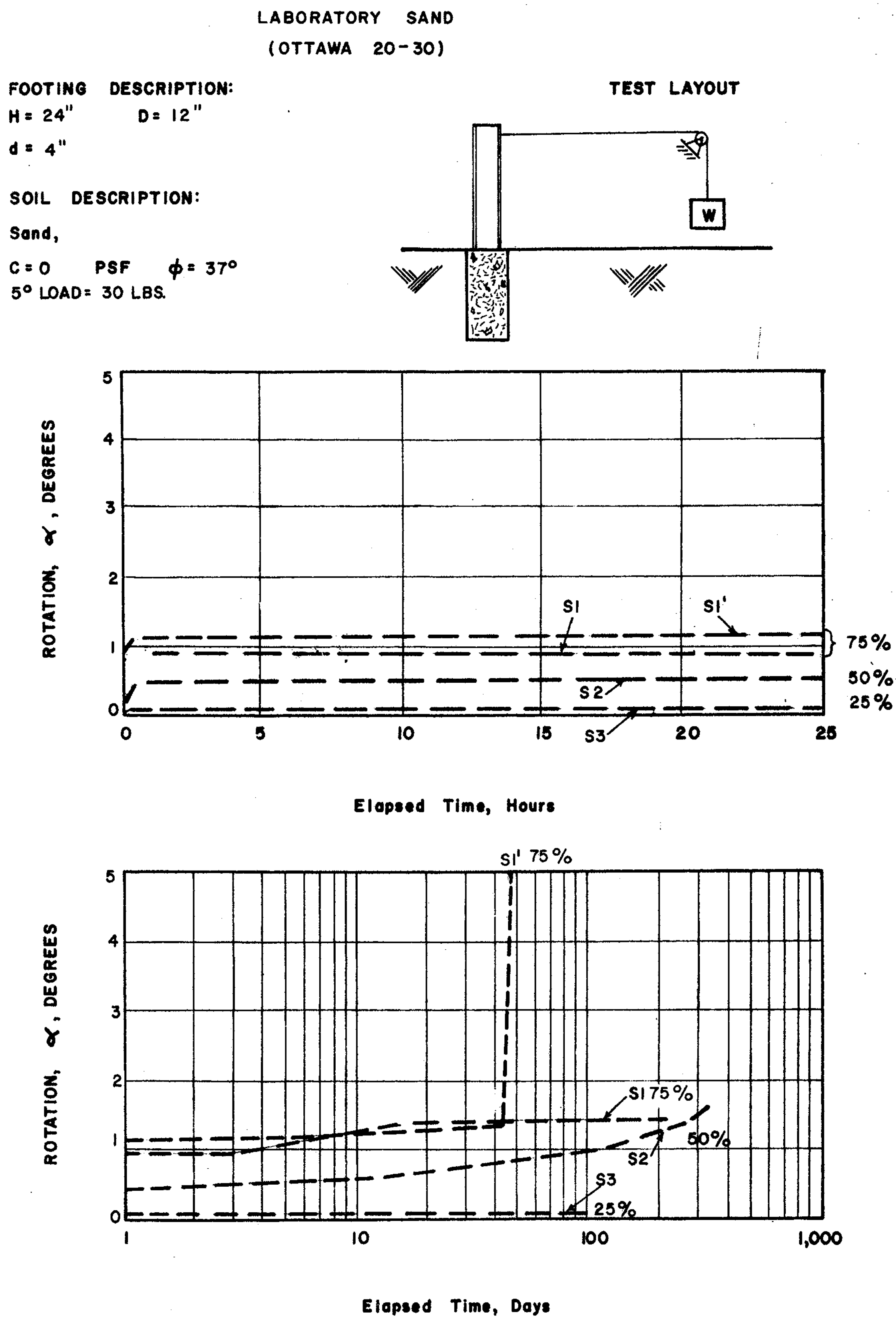


Figure 3, Results of Model Tests in Laboratory Sand

LABORATORY CLAYEY SAND

FOOTING DESCRIPTION:

H = 24" D = 12"

d = 4"

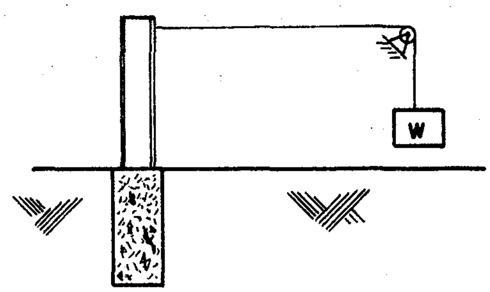
SOIL DESCRIPTION:

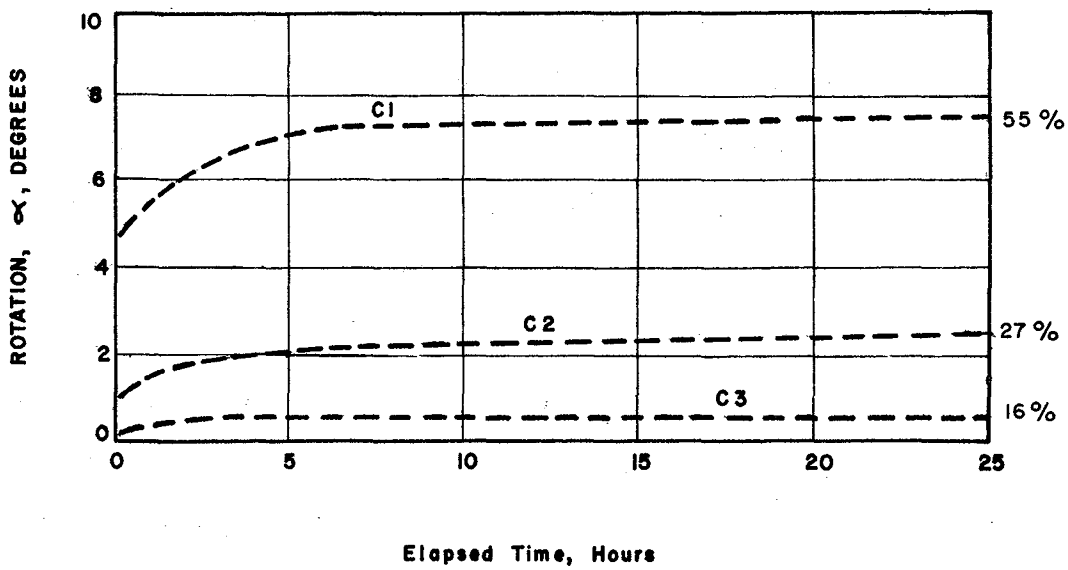
Clayey Sand,

C = 673 PSF ϕ = 5.25°

5° LOAD = 91.7 LBS.

TEST LAYOUT





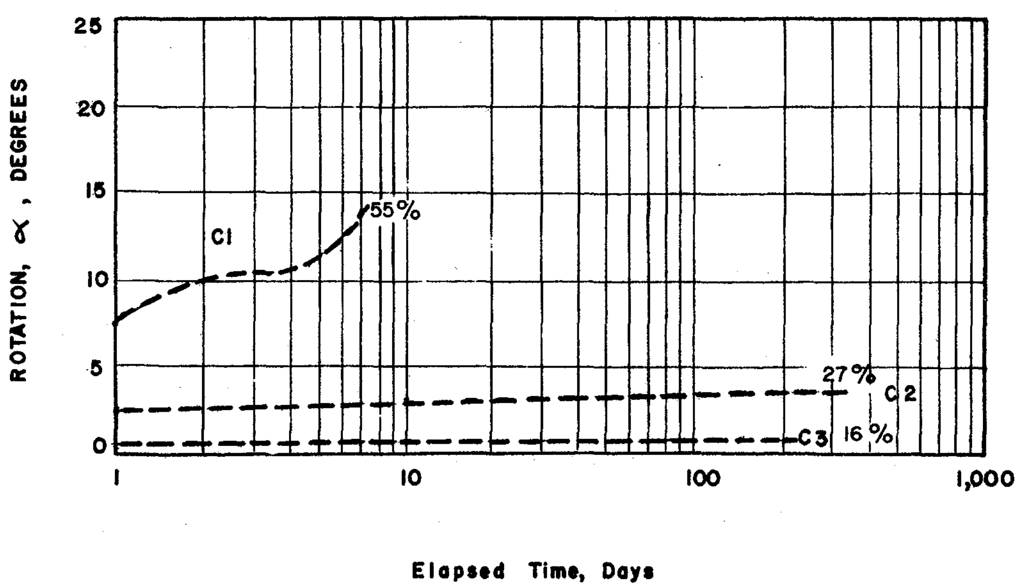


Figure 4, Results of Model Tests in Laboratory Clayey Sand

25 lbs. was applied. A 15 lb. load was applied to the other footing (C3). At a later date, unconsolidated, undrained triaxial tests were performed on specimens obtained from the test bins with the following results:

$c = 673$ psf

$\phi = 5.25°$

On this basis, the C1 test was loaded to approximately 55% of its 5° rotation load, while the C2 and C3 tests were loaded to 27% and 16%, respectively, of their 5° rotation loads. Under these loads, the C2 test rotated gradually up to 100 days and apparently became stable at approximately 130 days of elapsed time. The C3 test became very stable after the first few days of loading and reached a maximum rotation of 33 minutes after 200 days.

The reason for the failure of the C1 footing at a load considerably less than the predicted failure load is not explainable at the present time. However, it should be considered that the 5° rotation load was based partly on tests in which the load was applied slowly, in comparison to the model creep tests where a significant load of 50 lbs. was applied in a period of less than ten seconds.

Full-Scale Tests

The results of the full-scale, long-term tests are presented in Figures 5, 6, and 7. As in the presentation of the model tests, the top graph gives footing rotation versus elapsed time in hours for the first 25 hours of load, and the bottom graph gives footing rotation versus the elapsed time in days on a logarithmic scale.

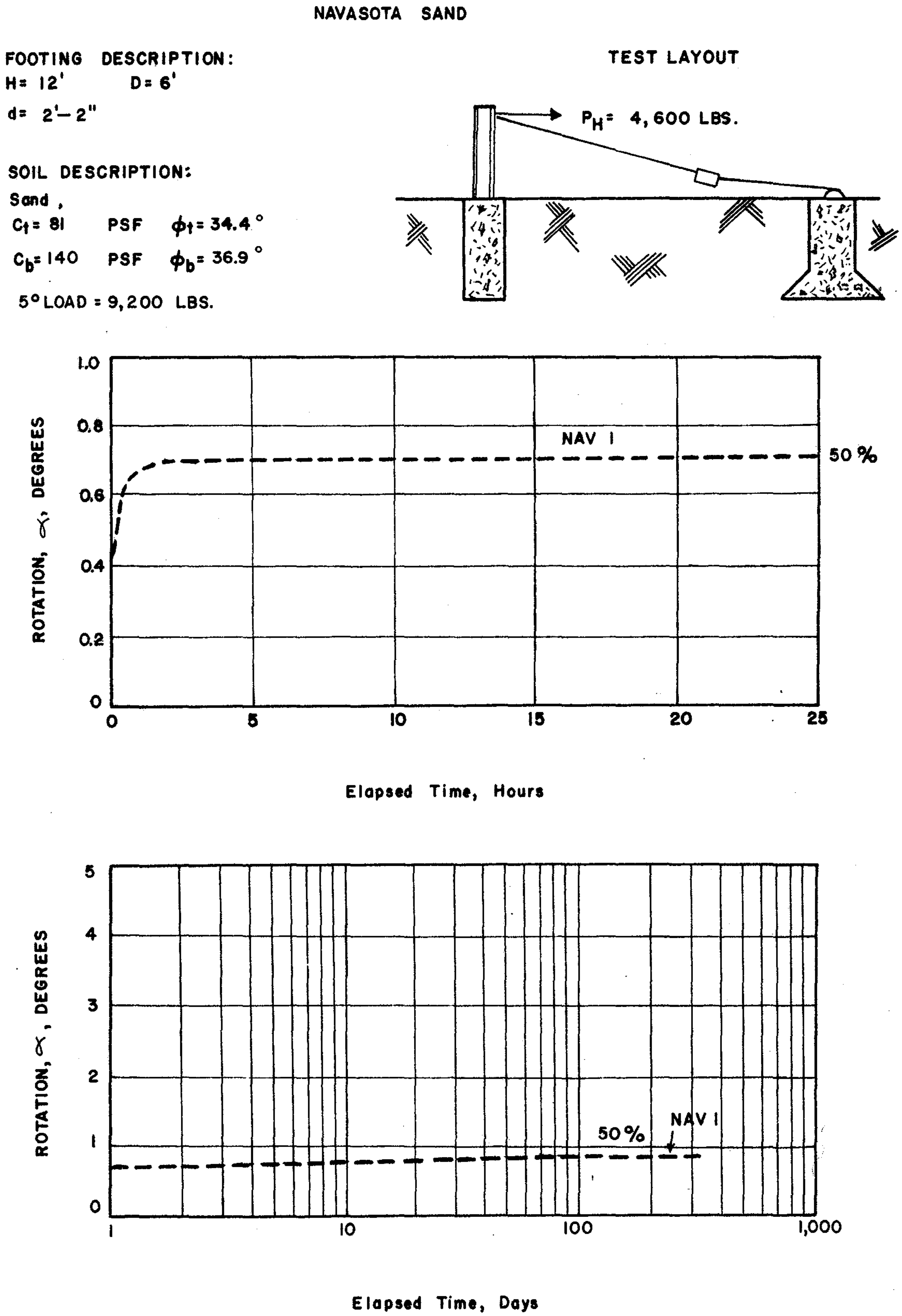


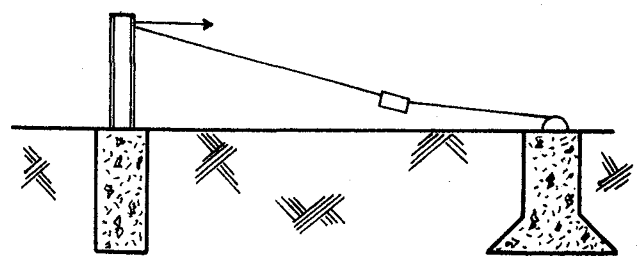
Figure 5, Full-Scale Test in Navasota Sand

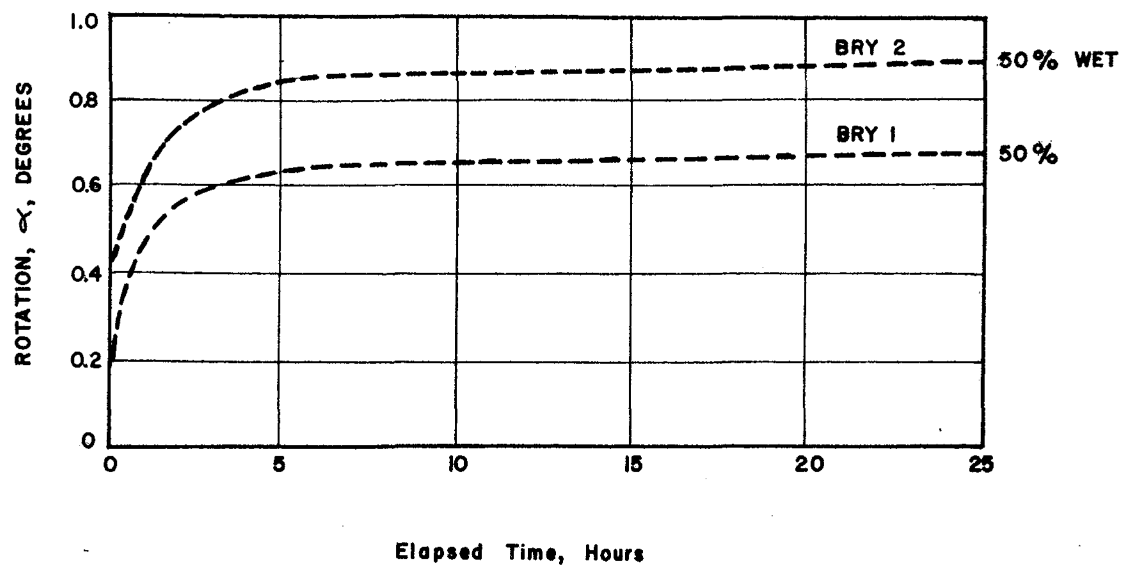
BRYAN SANDY CLAY

FOOTING DESCRIPTION:
H= 12' D= 6'
d= 2'-2"

SOIL DESCRIPTION:
Sandy Clay,
C_t= 2350 PSF ϕ_t= 3.0°
C_b= 2590 PSF ϕ_b= 20.6°
5° LOAD = 12,400 LBS.

TEST LAYOUT





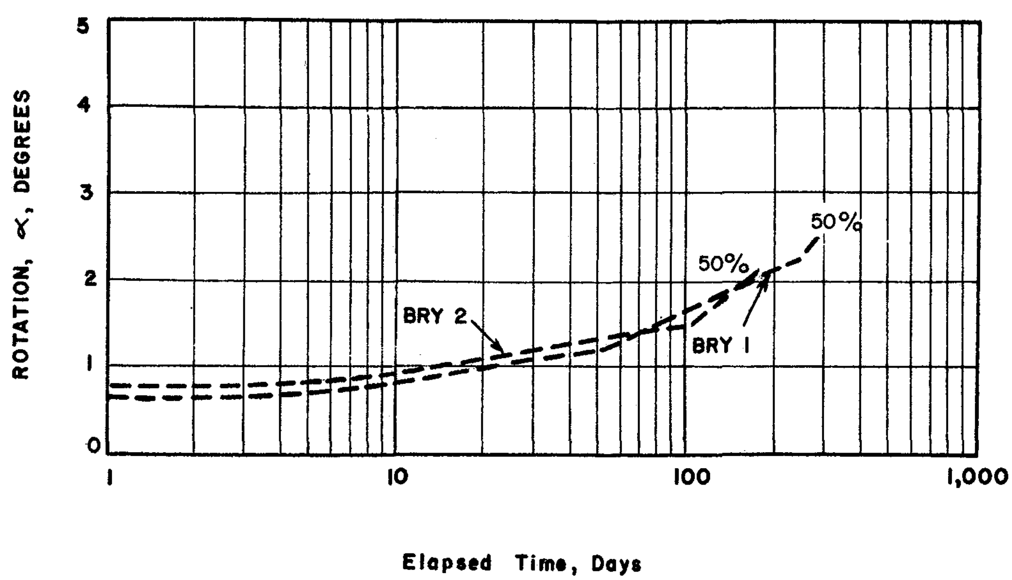


Figure 6, Full-Scale Test in Bryan Sandy Clay

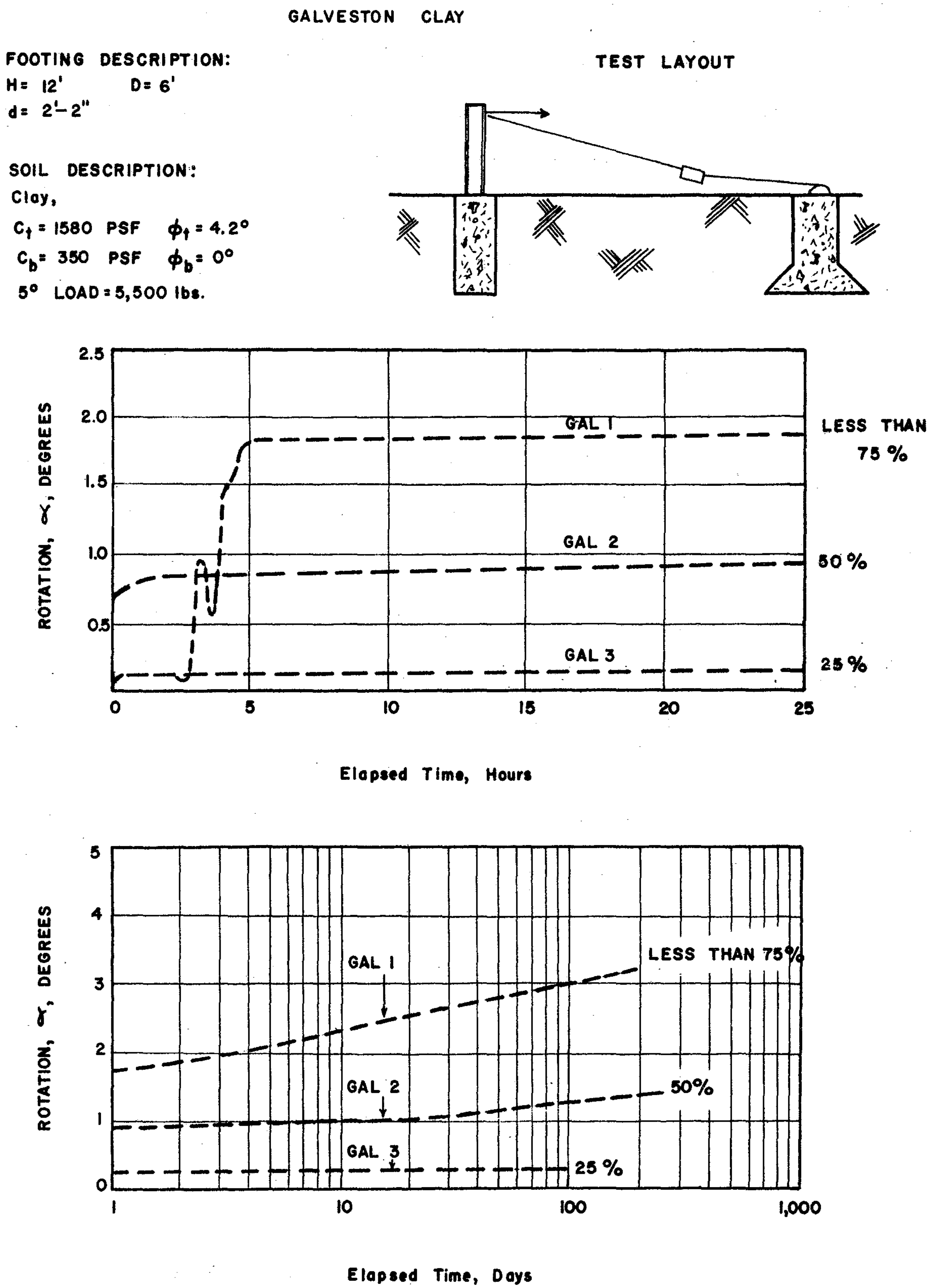


Figure 7, Full-Scale Test in Galveston Clay

A single test was conducted in the Navasota Sand. The long-term, horizontal load was maintained at 50% of the load, which produced a 5° rotation for the overturning tests reported in Research Report 105-3. As shown in Figure 5, the footing rotated initially through 40 minutes or approximately 0.7°; it then became relatively stable, gradually increasing to a rotation of 53 minutes. It remained stable at 53 minutes of rotation from the 70th to the 320th day of loading, when the test was discontinued.

Two tests were conducted in the Bryan Sandy Clay, each at a load which was 50% of the 5° rotation load. The difference in these two tests is that the soil in test Bry-1 was allowed to vary in moisture content as dictated by atmospheric conditions. In the case of Bry-2, a small dike, approximately 12 inches high, was erected surrounding the footing and the surface soil was kept moist throughout the 180 days of loading, which included the hot summer months. Although deep cracking of the soil surrounding the footings due to drying shrinkage has concerned some engineers, this condition did not occur in the Bryan tests. Some minor surface cracking was noted during the summer in the test area which was not kept wet (Bry-1), but the effect on the rotation-time characteristics of the footings was not significant. Also the effect of keeping the surface wet around the Bry-2 test footing was apparently negligible. When initially loaded, the Bry-1 footing rotated 40 minutes during the first day and then continued a gradual rotation until it reached a value of 2 1/2° at the end of 290 days. The Bry-2 footing rotated slightly more initially to a value of 53 minutes after the first 5 hours and then gradually increased to 2° after 180 days. The lower graph in Figure 6 shows that the rotation-time curves of these two footings are similar.

Three load tests were conducted on two footings in the Galveston Clay. The site location was on Pelican Island across the ship channel from Galveston. The two footings were initially loaded to 25% and 50% of the 5° rotation load. These tests are designated Gal-3 and Gal-2, respectively. The Gal-2 test rotated nearly 1° within one day after the initial loading. It continued to rotate slightly and after 239 days appeared to remain constant at a rotation of approximately 1 1/2°. After the first 50 days, the Gal-3 Test was stable at a rotation of 11 minutes and remained stable for the next 32 days, when the test was discontinued. When this test became stable, it was decided to load the same footing to 75% of the 5° rotation load. Extreme difficulty was encountered in trying to keep a 75% load on this footing. When the proper load was achieved, deflection would progress at such a rate that the load was quickly reduced. For this reason, a 75% load was never maintained on the Gal-1 Test. When the footings were periodically checked, the load was increased to 75%, but it was difficult to ascertain the variation in load which was on this footing during the 180 days it was loaded. Results of this test are not reliable.

LABORATORY CREEP TESTS

It was desired to develop a laboratory test which would simulate the long-term loading conditions of the soil around the footings or could be used to predict the soil-strength parameters appropriate for use under long-term loading conditions. Two basic criteria were established for these tests:

a) They should utilize equipment and techniques compatible with those prevailing within the Texas Highway Department.

b) They should be of a short enough duration to be economically feasible to perform for use in design of minor service structures.

A standard creep test for soils has not been developed, primarily because the creep phenomenon, although known to exist, is not well understood and has not been subjected to extensive investigation. Much of the previous research has been directed toward studying secondary consolidation, which is a phenomenon not necessarily related to the creep observed under shear strains.

Creep in a soil mass refers to the time-dependent deformation behavior of the soil under a given set of sustained stresses. It is a function of several variables, including soil type, soil structure, and stress history, to name a few. Casagrande and Wilson[4] conducted creep tests on consolidated, undrained triaxial specimens and found that under sustained load some types of undisturbed, brittle clays and clay shales ultimately failed at loads appreciably less than the strength indicated by normal laboratory compression tests. In some partially saturated soils, just the opposite effect was noted. Singh and Mitchell[5] used a generalized stress-strain-time function to study creep potential and creep rupture in soils and proposed empirical formulas to predict creep potential. In addition, they proposed a method of predicting the time needed to develop creep rupture or to reach a certain specified deformation.

Constant stress-level creep tests were performed by Bishop and Lovenbury[6] on triaxial specimens under drained conditions. An overconsolidated and a normally-consolidated clay were tested, the test duration being up to 3 1/2 years. The results show that simple logarithmic or power laws relating strain and time are applicable only for limited periods. Also notable was the marked instability of strain rate which they attributed to a modification of the soil structure and the absence of a secondary or constant strain rate phase.

In summary, with respect to creep strength, soils can be classified as those that lose strength with time, those that gain strength, and those whose strength is essentially independent of time. On the basis of strain, after long periods under constant stress conditions, the strain rates may almost cease (terminating strain), they may continue at ever decreasing rates, or they may increase, eventually resulting in failure (non-terminating strain).

It is difficult to visualize how the vertical strain observed in a laboratory compression test can be related to the mode of deformation that occurs in the field. However, a more sophisticated test is not economically warranted for the design of footings for minor service structures. A simplified approach is to obtain the creep strength of the soil and then determine what relationship exists between the 5° overturning load predicted from the creep strength and the actual creep loads observed in the laboratory model tests. Thus, the laboratory tests described below were developed to obtain the creep strength of the soils as well as the creep strain-time relationship.

Test Technique

a) Ottawa Sand. For these tests, 3- by 6-inch triaxial specimens were used. These specimens were constructed in a forming jacket in the manner described by Lambe[7] at a void ratio of 0.51, the average void ratio of the sand in the model bins. The samples were tested in a dry state at confining pressures of 5, 15, and 30 psi, under drained conditions. The vertical stress was applied by means of a platform scale which provided a convenient means of applying and maintaining a constant stress. However, any dead weight loading system would be a satisfactory method of applying the load.

Each sample was then subjected to approximately 60% of its ultimate failure load for each confining pressure. Axial deformation measurements were made until the specimen movement stabilized (or reached terminating strain), at which time a new loading increment was applied. This process was continued until non-terminating creep was obtained. A typical set of results for a single confining pressure is shown in Figure 8. The strains at the conclusion of each loading increment are shown for the same specimen in Figure 9. Finally, Figure 10 shows the combined results of the tests at all confining pressures plotted as creep strain versus the percent of ultimate vertical stress. Within the limits of experimental

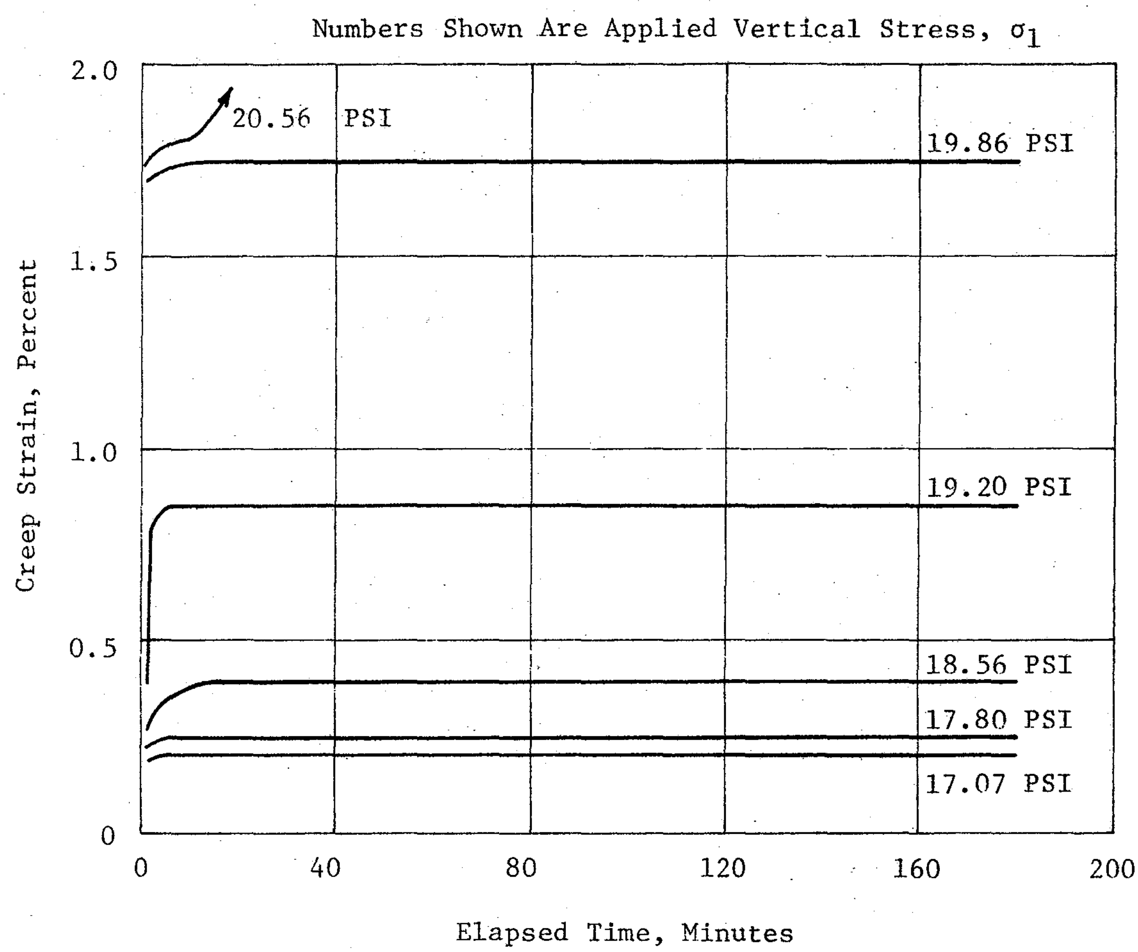


FIGURE 8, CREEP STRAIN VS. TIME CURVE FOR OTTAWA SAND AT CONFINING PRESSURE OF 5 PSI

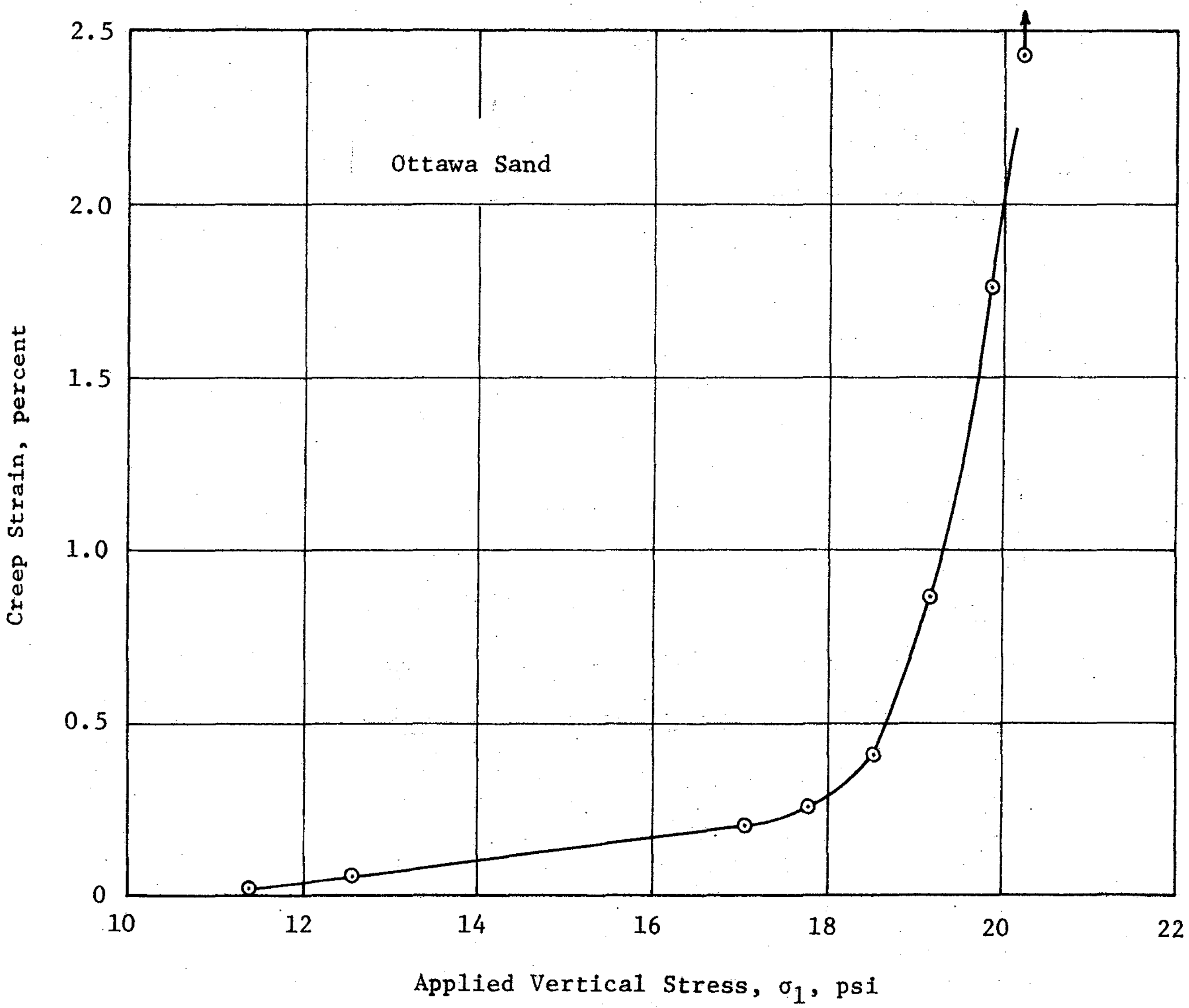


FIGURE 9, RELATIONSHIP BETWEEN CREEP STRAIN AND APPLIED VERTICAL STRESS AT CONFINING PRESSURE OF 5 PSI.

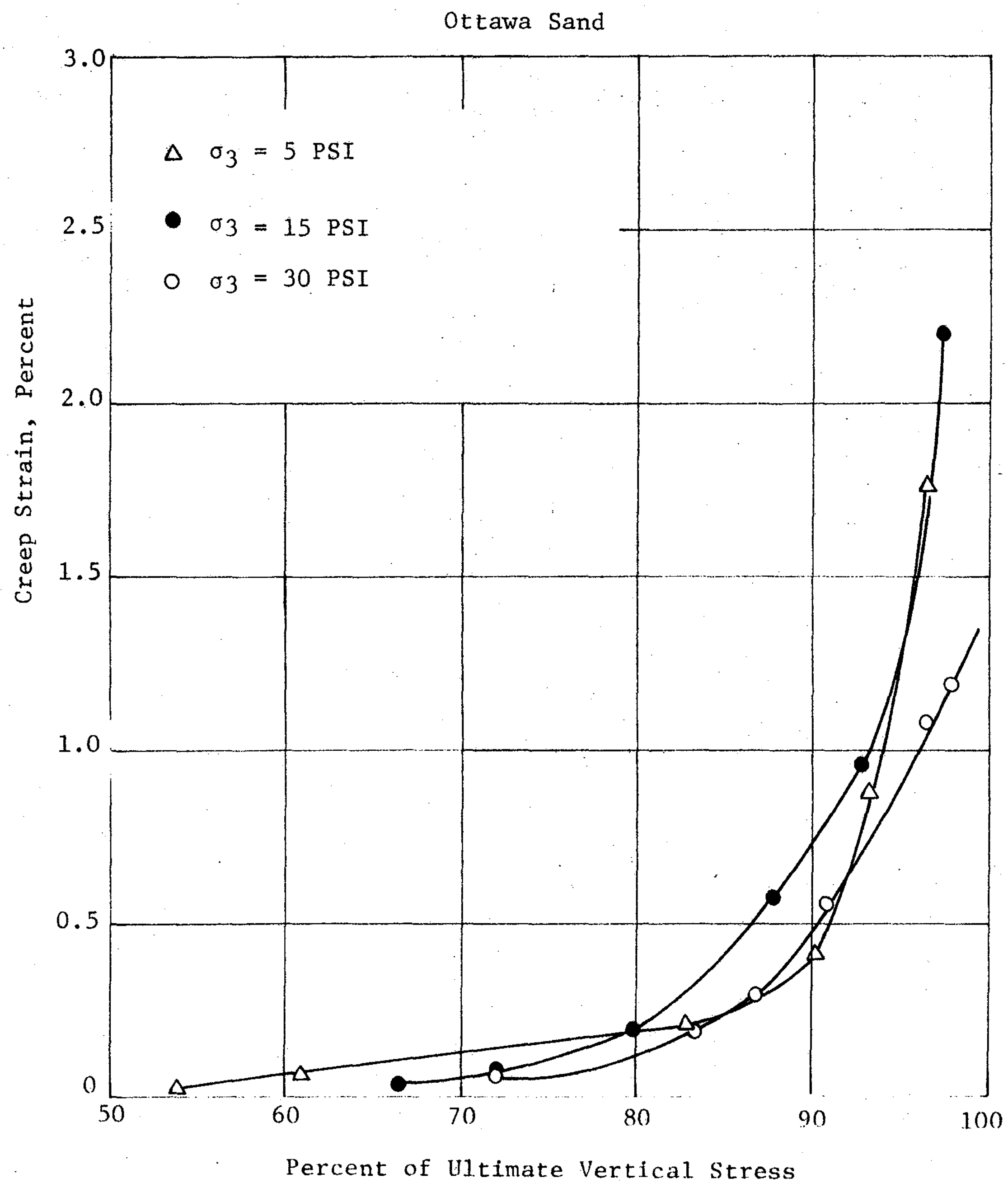


FIGURE 10, RELATIONSHIP BETWEEN CREEP STRAIN AND PERCENT OF ULTIMATE VERTICAL STRESS error, the latter curves coincide, indicating that the creep strain of this soil is somewhat independent of the confining pressure and dependent on the percent of ultimate vertical stress applied to the specimen.

b) Laboratory Clayey Sand. Test specimens of the clayey sand were obtained by cutting an undisturbed block sample from the bin. The block sample was then trimmed to a 1 1/2- by 3-inch specimen for creep testing. After trimming, the specimen was mounted in the triaxial cell and covered with a latex membrane, the confining pressure was applied, and it was subjected to a vertical stress in the same manner as described previously for the Ottawa Sand specimens. Confining pressures of 5, 15, and 30 psi were used.

The initial load was applied on each specimen immediately after the confining pressure was applied. Thus, for this material, which was relatively impermeable, the initial load was applied under undrained conditions. The vertical load was maintained, allowing specimen drainage, until terminating creep was obtained. Thereafter, each additional vertical load increment was applied in the same manner. The test was considered complete when non-terminating strain was reached.

The results of all tests on the clayey sand are shown in Figure 11, which is a plot of creep strains versus percent of ultimate vertical stress. The time required to test each clayey sand specimen was considerably longer than that needed for the Ottawa Sand. Presumably, this was a manifestation of the lower permeability of the clayey sand. In addition, for the same percentage of ultimate vertical stress, the clayey sand underwent significantly higher strains than did the Ottawa Sand.

Both materials show an abrupt increase in the creep strain at approximately 85% of the ultimate vertical stress.

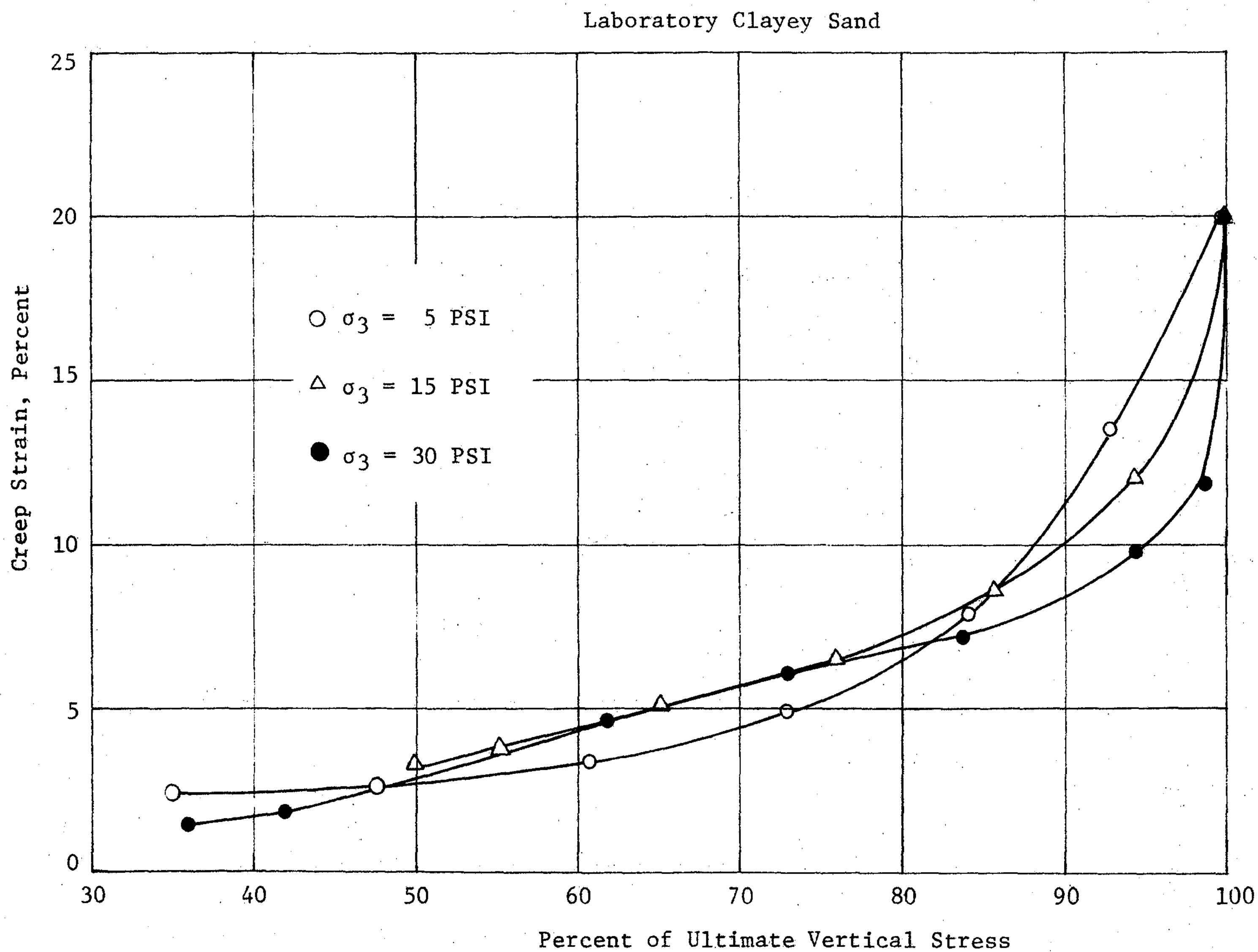


FIGURE 11, RELATIONSHIP BETWEEN CREEP STRAIN AND PERCENT OF ULTIMATE VERTICAL STRESS

APPLICATION OF LABORATORY TESTS AND FIELD OBSERVATIONS

If the vertical and lateral stresses for each laboratory creep specimen at non-terminating strain are plotted in terms of Mohr's circles, a Mohr failure envelope can be developed which defines the creep strength of the soil (see Figure 12). For the Ottawa Sand, this envelope has an angle of 36.5° and a cohesion of 0. As might be expected, there is little difference between this and the 37° obtained from a standard laboratory triaxial test where drainage was allowed. Corresponding values for the clayey sand are 27.8° and 330 psf. By the nature of the creep test, this is probably close to the consolidated, drained shear strength of the soil. This compares with the unconsolidated, undrained values of 5.25° and 673 psf reported earlier in the report.

In an attempt to correlate the creep test results with the laboratory model tests, the 5° overturning load was obtained (using the theory presented in Research Report 105-3[3]) for various percentages of the creep strength. This load is termed the "creep strength 5° overturning load". (It should be emphasized that this load, based on various percentages of the <u>creep strength</u>, is not the same as taking a percentage of the load based on <u>peak strengths</u>, which is the procedure recommended in Research Report 105-3.) These results are shown in Figures 13 and 14 for the Ottawa Sand and the laboratory clayey sand, respectively. Superimposed on these figures are the results of the model tests showing the amount of rotation in degrees undergone by each footing as well as the actual load applied to the footings.

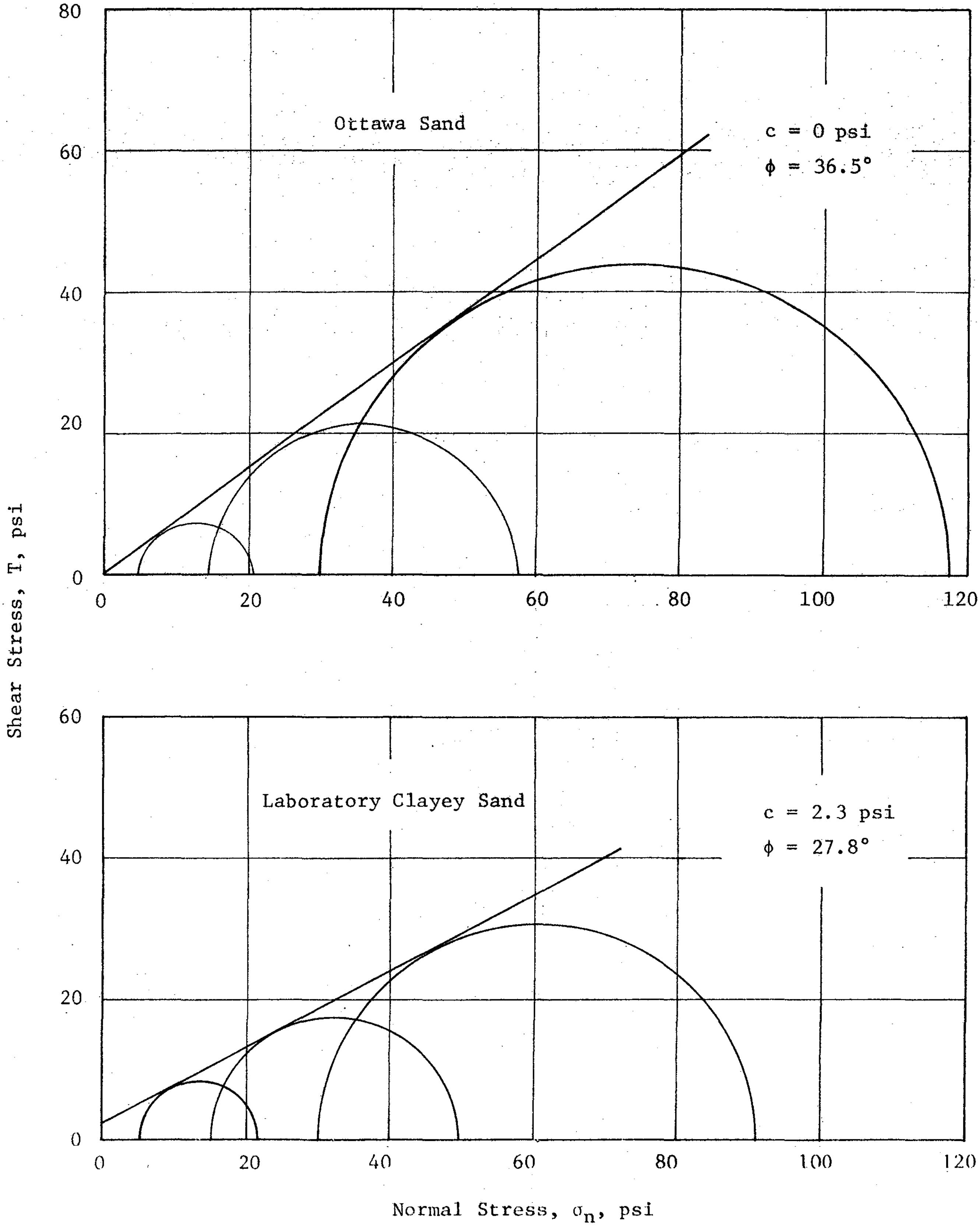


FIGURE 12, MOHR FAILURE ENVELOPE BASED ON CREEP TESTS

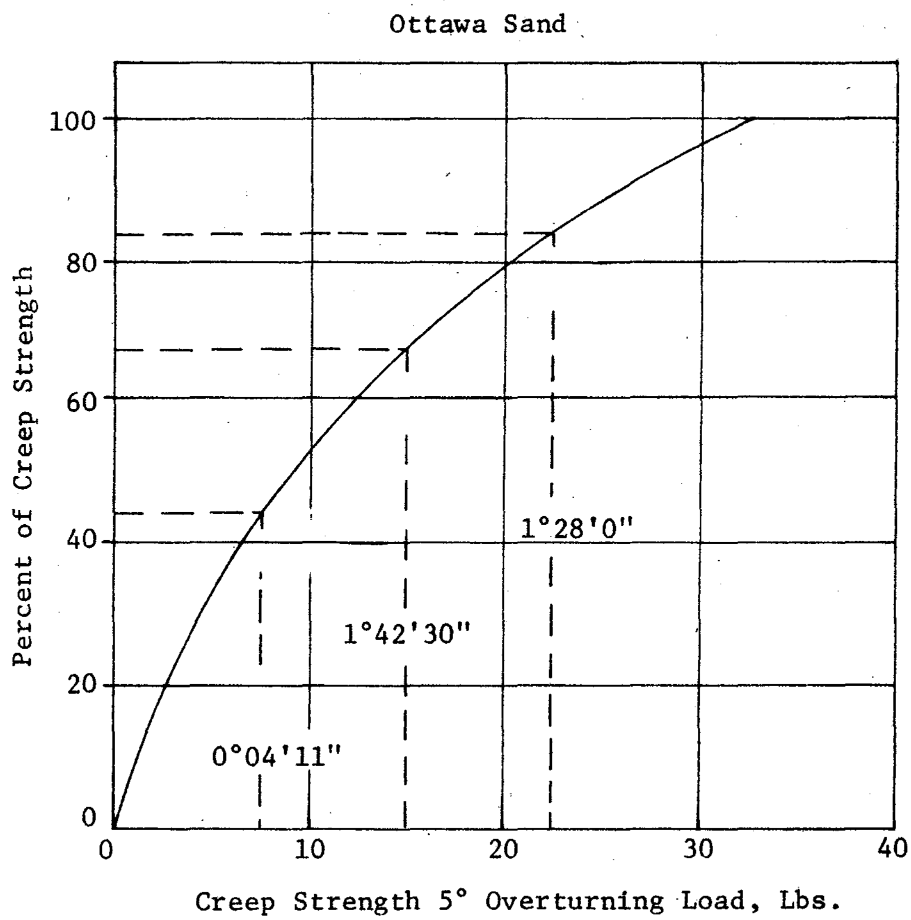


FIGURE 13, CREEP STRENGTH VS. OVERTURNING LOAD

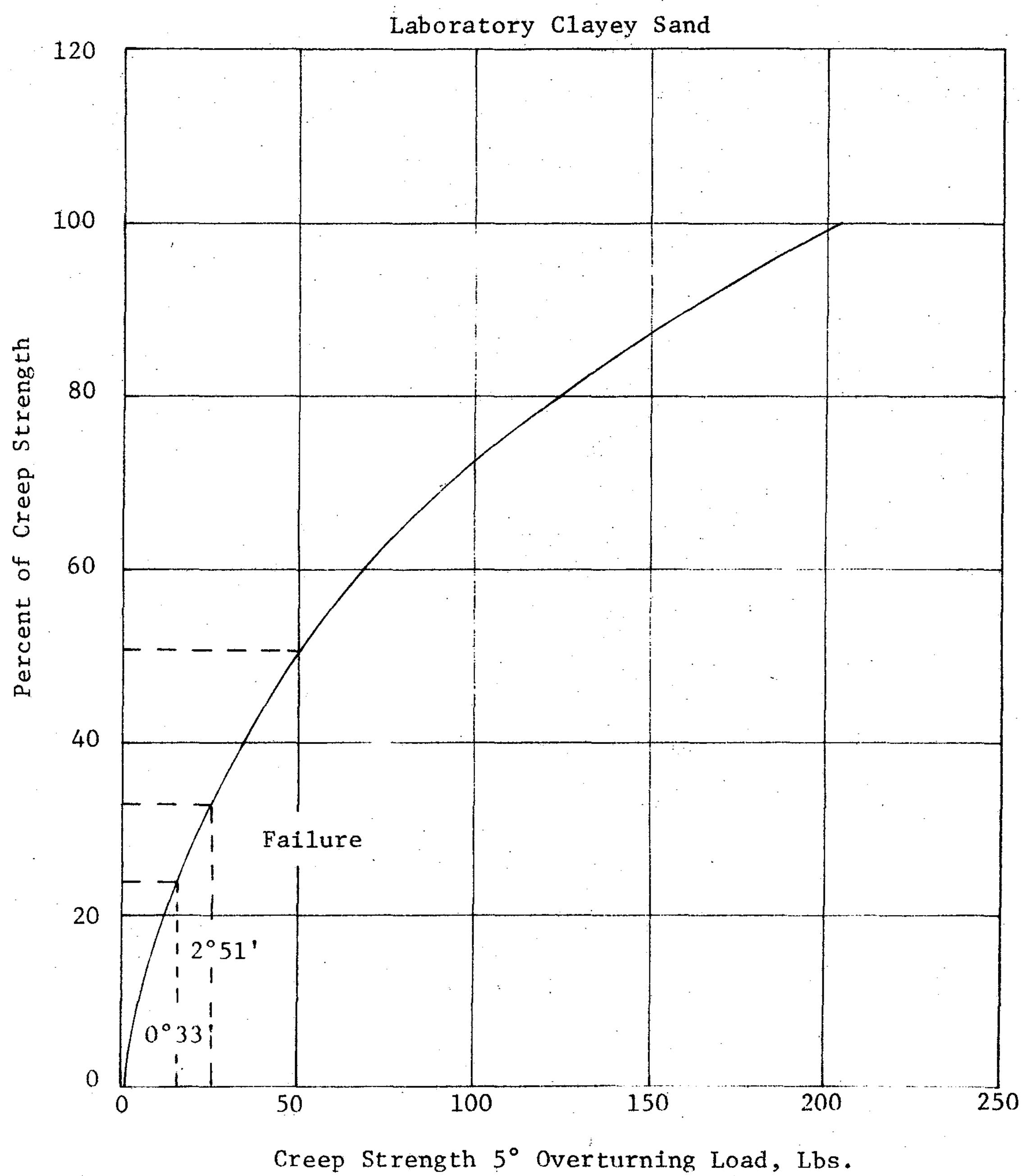


FIGURE 14, CREEP STRENGTH VS. OVERTURNING LOAD

Figure 13 shows for the Ottawa Sand that when nearly 85% of the creep strength was utilized, the model footing rotation was still small (less than 2°). On the other hand, for the clayey sand (Figure 14), failure occurred when approximately 50% of the creep strength was utilized. When 33% of the creep strength was utilized, the rotation was nearly 3°. This behavior is qualitatively indicated by Figures 10 and 11: when 85% of the ultimate stress was applied to the triaxial creep specimens of Ottawa Sand, they strained only 0.3%, whereas the clayey sand strained approximately 3.0% at 50% of the ultimate stress.

Thus, based on the limited test results available, it does not appear that there is a single limiting or "threshold" percentage of the creep strength that can be applied for all soil types beyond which the footing will rotate excessively. It is felt that the principle is sound, but for quantitative purposes, additional test records must be obtained encompassing many different soil types and the creep strain must also be considered. Since creep tests are somewhat time consuming, it may be some time before this information is available.

The alternate approach is to use the standard soil test results and design on the basis of a percentage of the 5° overturning load. In connection with this approach, the results of all long-term tests, both model and field, are plotted in Figure 15, which shows the footing rotation versus the percent of calculated 5° overturning load. The soils are divided into three basic groups:

> Soft clays, which include the laboratory clayey sand and the Galveston tests,
>
> Stiff, non-fissured clays, which are the Bryan tests, and
>
> Sands, which include the Navasota and the Ottawa Sand tests.

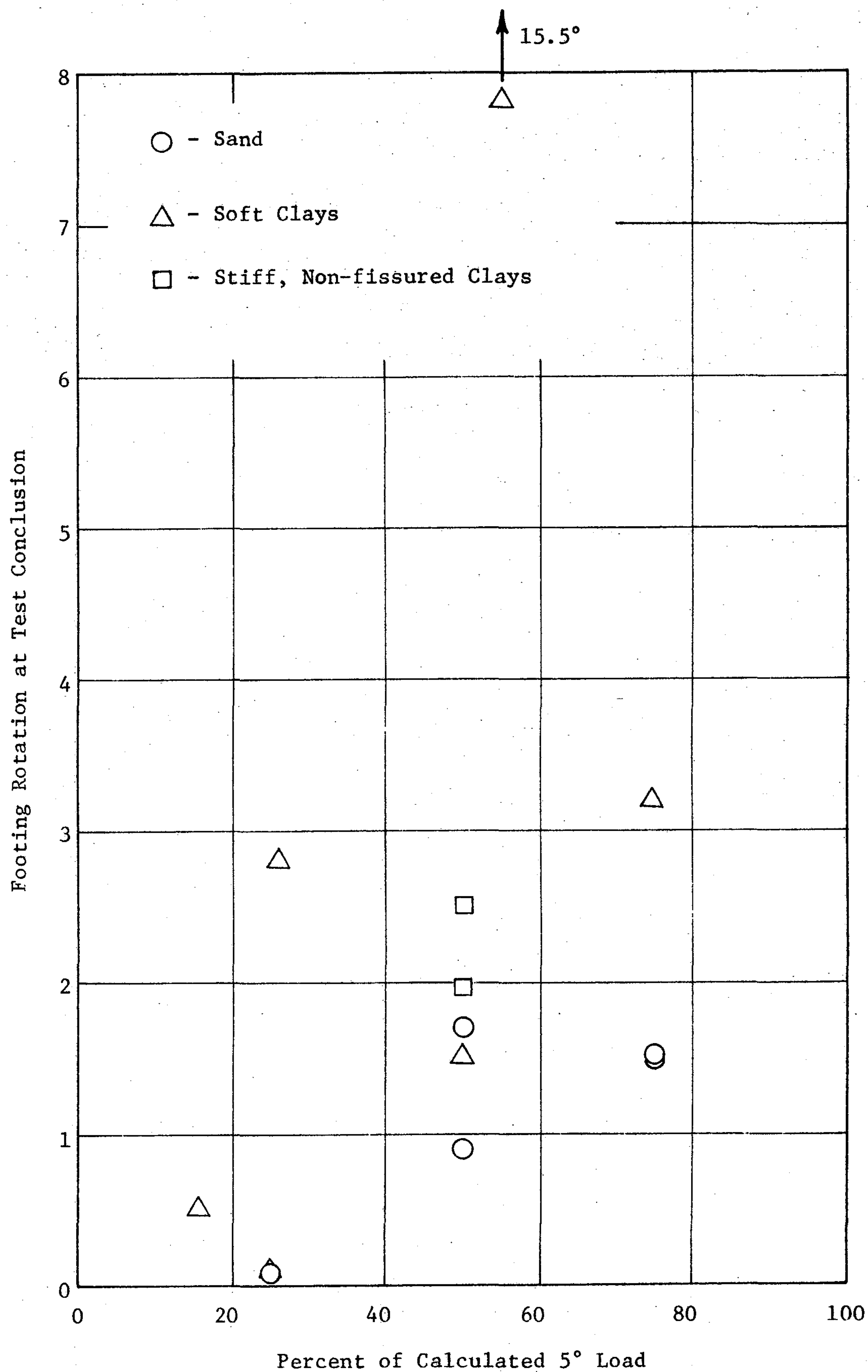


FIGURE 15, FOOTING ROTATION BY SOIL TYPE

On this basis the following conservative conclusions can be made:

> Soft clays should not be subjected to long-term loads greater than 1/3 of their standard 5° overturning load,
>
> Stiff, non-fissured clays may be safely subjected to long-term loads of 1/2 of their standard 5° overturning load, and
>
> Sands may be safely subjected to long-term loads of 1/2 of their standard 5° overturning load and indications are that 3/4 of their 5° load may be satisfactory.

One important soils group not tested was the stiff, fissured clays, which are very prevalent along the Texas gulf coast area. Under certain conditions of loading, such as the active pressure element around the footing, these materials tend to open along pre-existing joints and lose strength with time. Standard laboratory tests usually do not reveal this strength loss. Until positive information is available, it is suggested that 1/3 of the calculated 5° pullover load be used in these materials.

SUMMARY

The purpose of the field and laboratory long-term loading tests was to determine what values of long-term overturning loads could be safely applied to drilled shaft footings without undue rotation and also to develop a laboratory creep test which would aid in these predictions.

Based on the limited number of laboratory creep tests which were performed, it does not appear that these tests can be used until additional information is obtained on several soil types. Until this information becomes available, it is suggested that the following percentages of the calculated 5° overturning load, based on the soil tests recommended in Research Report 105-3, be used for admissible long-term creep loads:

Soft clays	33%
Stiff, non-fissured clays	50%
Sands	50 to 75%
Stiff, fissured clays	33%

Although the use of these percentages for allowable long-term loads should result in a terminating rotation, this rotation will probably be significant (on the order of one degree). The judgement of the engineer is necessary to decide what rotation is acceptable for a particular footing. If the acceptable rotation is severely limited by functional or aesthetic considerations, the use of significantly smaller percentages of the 5° load may be necessary.

Even though the data developed are limited and the correlation between soil creep tests and footing creep tests are not fully reconciled, it is the authors' opinion that this study has produced some very usable

information which will allow the design engineer to consider a loading condition, the effects of which were almost totally undefined prior to this work.

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APPENDIX

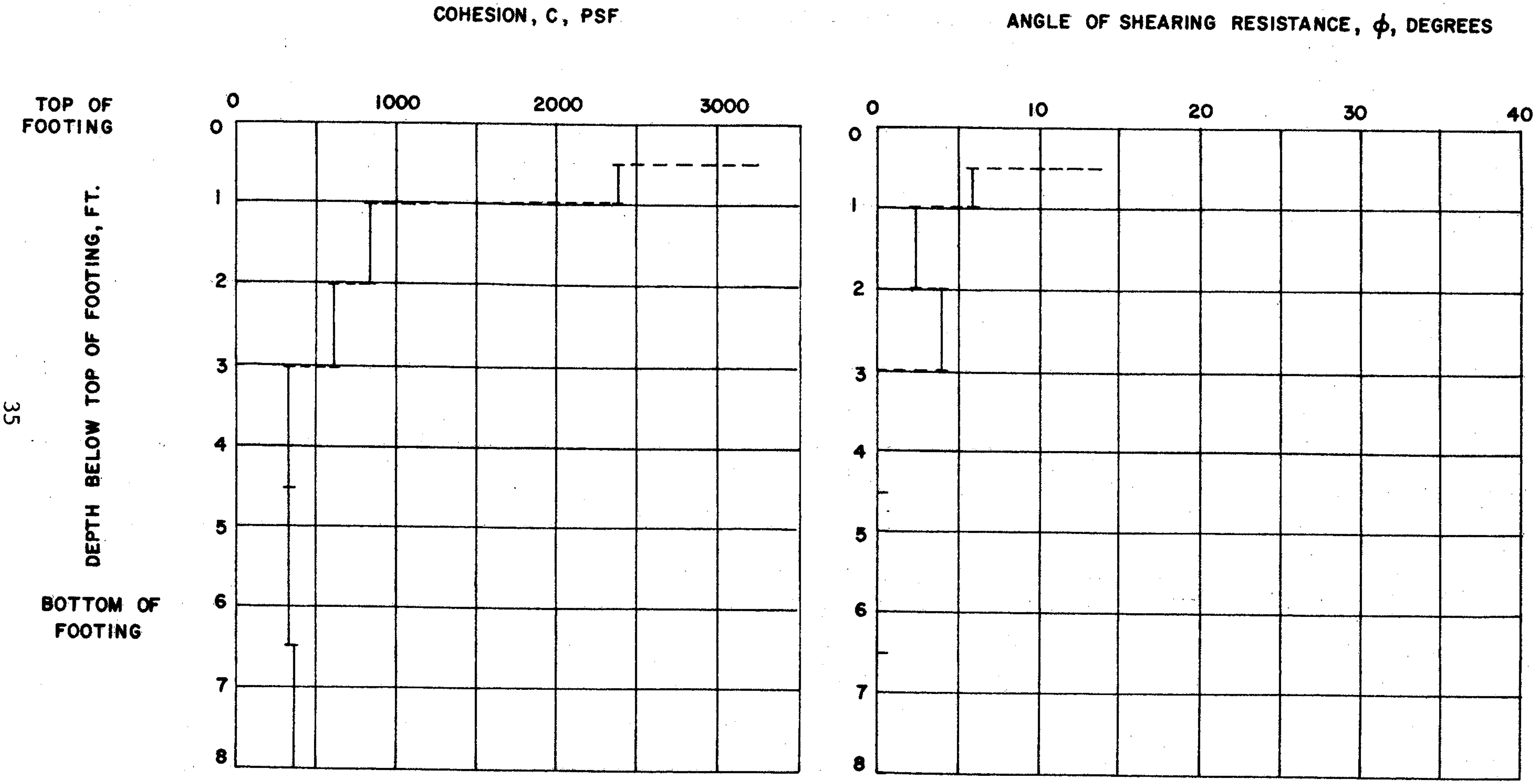

FIGURE 16, SOIL COEFFICIENTS OF GALVESTON CLAY

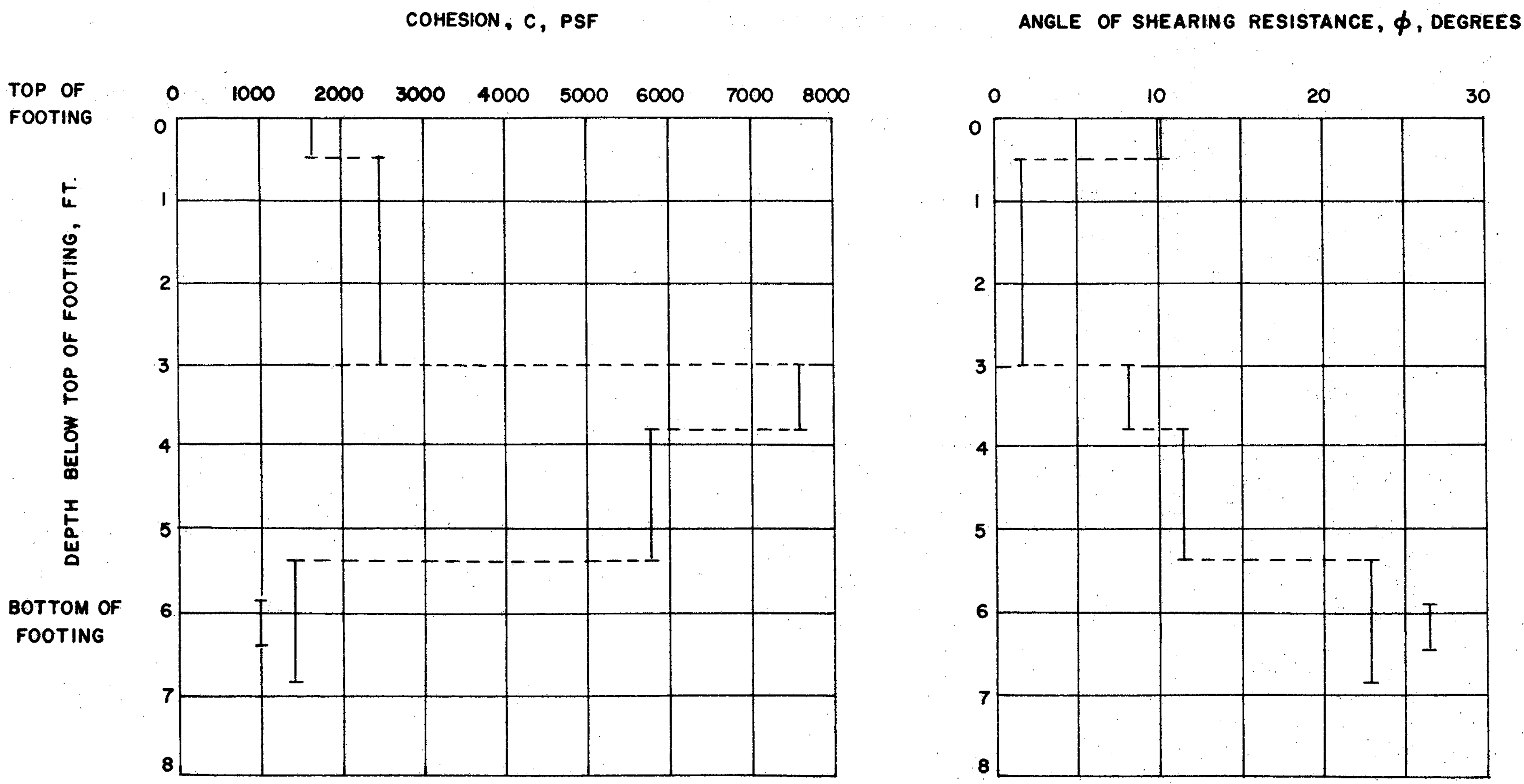

FIGURE 17, SOIL COEFFICIENTS OF BRYAN SANDY CLAY

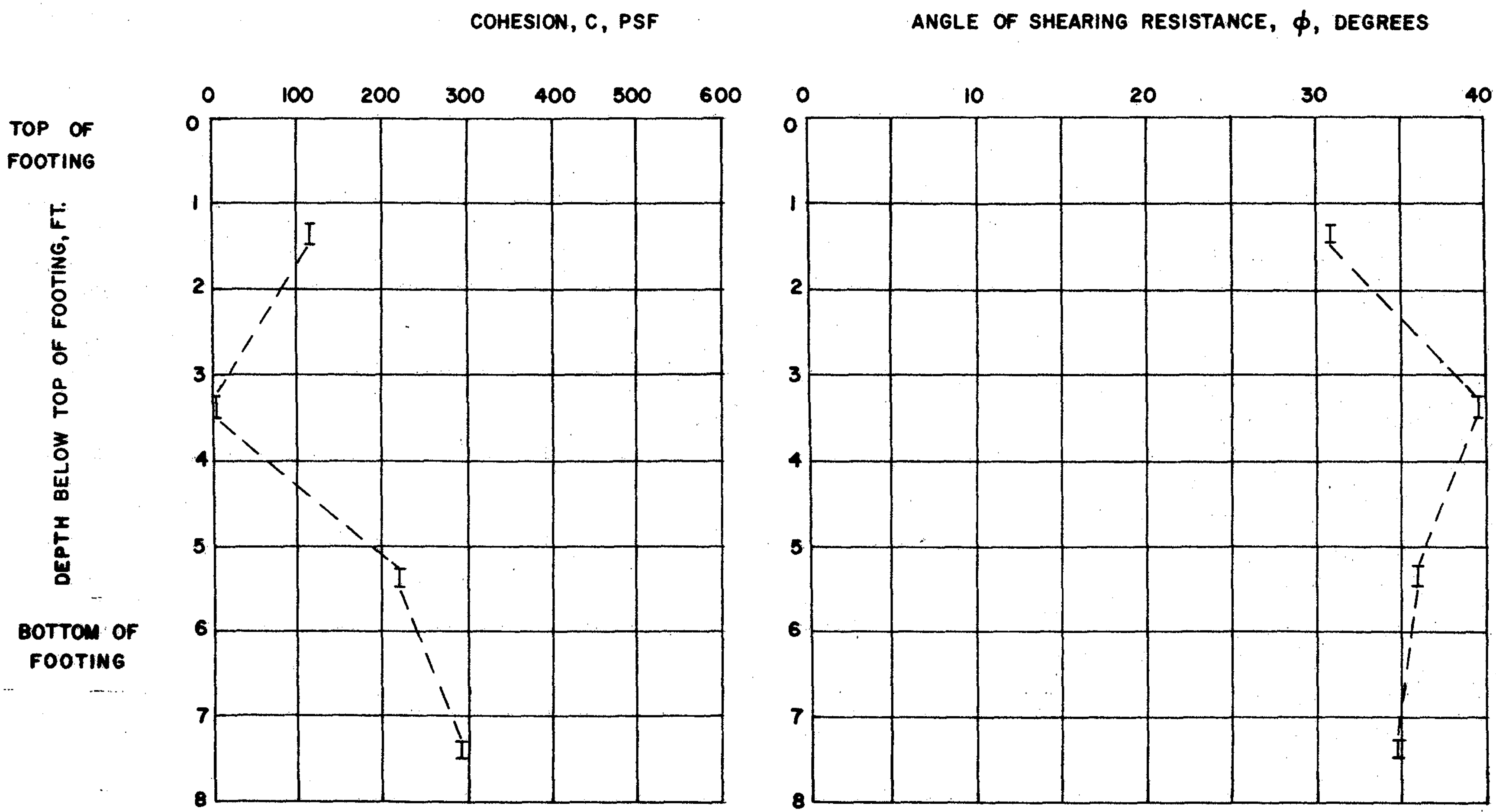

FIGURE 18, SOIL COEFFICIENTS OF NAVASOTA SAND

TEST NUMBER S1

Footing Description

H = 24" D = 12"
d = 4"

Soil Description

Ottawa 20-30 Sand

c = 0 PSF ∅ = 37°

Load = 22.5 lbs = 75 % Ultimate

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
1			0	49	50
8			0	50	0
15			0	50	0
60			0	50	0
150			0	50	20
210			0	50	20
	1		0	50	40
	1.7		0	51	50
	14		1	24	20
	28		1	26	20
	50		1	26	40
	63		1	26	40
	133		1	27	10
	203		1	28	0

TEST NUMBER S1'

Footing Description

H = 24" D = 12"
d = 4"

Soil Description

Ottawa 20-30 Sand

c = 0 PSF ϕ = 37°

Load = 22.5 lbs = 75 % Ultimate

Elapsed Time		Footing Rotation		
Minutes	Days	Degrees	Minutes	Seconds
11		1	8	30
13		1	9	20
259		1	9	30
	1	1	9	40
	4	1	10	10
	42	1	26	0

TEST DISTURBED

TEST NUMBER S2

Footing Description

H = 24" D = 12"
d = 4"

Soil Description

Ottawa 20-30 Sand

c = 0 PSF Ø = 37°

Load = 15 lbs = 50 % Ultimate

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
3			0	05	40
7			0	21	0
9			0	21	0
27			0	21	20
105			0	21	30
	0.9		0	34	0
	2.3		0	35	50
	3.1		0	36	30
	4.4		0	36	10
	6.0		0	36	50
	6.6		0	34	20
	9.0		0	35	10
	11.0		0	34	10
	35		0	47	30
	108		1	01	0
	124		1	01	10
	175		1	23	40
	189		1	24	20
	259		1	34	30
	319		1	42	30

. TEST NUMBER S3

Footing Description

H = 24"   D = 12"
d = 4"

Soil Description

Ottawa 20-30 Sand
c = 0 PSF ϕ = 37°

Load = 7.5 lbs = 25 % Ultimate

Elapsed Time			Footing Rotation			
Minutes	Days		Degrees	Minutes	Seconds	
2			0	0	30	
6			0	03	20	
18			0	03	30	
	0.8		0	04	10	
	2.8		0	04	10	No Movement
	11		0	04	10	
	15		0	04	10	
	23		0	04	10	
	35		0	04	10	
	60		0	04	10	

TEST NUMBER C1

Footing Description

H = 24" D = 12"
d = 4"

Soil Description

Clayey Sand

c = 673 PSF ϕ = 5.25°

Load = 50 lbs = 55 % Ultimate

Elapsed Time			Footing Rotation		
Minutes	**Days**		**Degrees**	**Minutes**	**Seconds**
11			4	50	
14			5	06	
17			5	11	
27			5	43	
43			5	58	
152			6	37	
207			6	46	
405			7	19	
	1.4		9	05	
	1.8		11	16	
	5.2		12	45	
	5.8		13	07	
	8.2		15	33	

TEST NUMBER C2

Footing Description

H = 24" D = 12"
d = 4"

Soil Description

Clayey Sand

c = 673 PSF Ø = 5.25°

Load = 25 lbs = 27 % Ultimate

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
8			0	55	
15			1	06	
28			1	14	
105			1	31	
160			1	41	
220			1	47	
358			2	09	
	1.3		2	30	
	1.9		2	32	
	4.0		2	35	
	5.8		2	37	
	9.2		2	43	
	17.0		2	43	
	41		2	44	
	102		2	45	
	120		2	45	
	170		2	47	
	184		2	48	
	254		2	49	
	314		2	51	

TEST NUMBER C3

Footing Description

H = 24"   D = 12"
d = 4"

Soil Description

Clayey Sand

c = 673 PSF Ø = 5.25°

Load = 15 lbs = 16 % Ultimate

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
2			0	9	40
5			0	11	30
10			0	12	30
65			0	16	10
100			0	17	20
400			0	18	20
	2		0	21	30
	15		0	23	50
	66		0	27	10
	80		0	28	20
	150		0	28	40
	210		0	33	0

TEST NUMBER NAV-1

Footing Description

H = 12'-0" D = 6'-0"
d = 2'-2"

Load = 4600 lbs = 50 % Ultimate

Soil Description

Sand

c_t = 81 PSF ϕ_t = 36.9°

c_b = 140 PSF ϕ_b = 36.9°

Elapsed Time			Footing Rotation			
Minutes	Days		Degrees	Minutes	Seconds	
35			0	37	30	
42			0	39	40	
	4		0	47	30	
	6		0	47	30	
	7		0	47	30	
	10		0	47	30	
	12		0	47	30	
	14		0	47	50	
	17		0	47	50	
	19		0	47	50	
	21		0	50	0	
	24		0	51	40	
	26		0	47	50	
	33		0	50	0	
	35		0	50	0	
	40		0	52	0	
	54		0	52	0	
	70		0	53	50	Stable
	83		0	53	50	
	117		0	53	50	
	146		0	53	50	
	258		0	53	50	
	320		0	53	50	

TEST NUMBER BRY-1

Footing Description

H = 12'-0" D = 6'-0"
d = 2'-2"

Load = 6,200 lbs = 50 % Ultimate

Soil Description

Sandy Clay

c_t = 2350 PSF ϕ_t = 3.0°

c_b = 2590 PSF ϕ_b = 20.6°

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
18			0	21	0
45			0	25	30
52			0	27	40
	.9		0	40	20
	2.0		0	40	20
	8		0	47	10
	10		0	51	20
	15		0	49	40
	20		0	53	40
	29		0	59	50
	45		1	19	30
	65		1	25	10
	92		1	29	10
	101		2	0	20
	121		2	2	20
	170		2	4	30
	233		2	16	50
	290		2	29	0

TEST NUMBER BRY-2

Footing Description

H = 12'-0"   D = 6'-0"
d = 2'-2"

Load = 6,200 lbs = 50 % Ultimate

Soil Description

Sandy Clay

c_t = 2350 PSF ϕ_t = 30°

c_b = 2590 PSF ϕ_b = 20.6°

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
18			0	12	40
30			0	37	50
37			0	46	20
97			0	47	40
172			0	48	40
249			0	50	40
329			0	52	50
	10		0	55	0
	58		1	12	0
	122		1	33	20
	180		1	59	40

TEST NUMBER GAL-1

Footing Description

H = 12'-0"   D = 6'-0"
d = 2'-2"

Load = 4120 lbs = 75 % Ultimate

Soil Description

Soft Clay

c_t = 1580 PSF ϕ_t = 4.2°

c_b = 350 PSF ϕ_b = 0 °

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
160			0	8	20
173			0	4	20
178			0	5	20
190			0	26	40
202			1	20	50
275			1	33	30
280			1	49	30
290			1	50	30
325			1	50	40
335			1	50	40
	21		2	33	0
	50		2	52	0
	146		3	4	30
	180		3	13	20

TEST NUMBER GAL-2

Footing Description

H = 12'-0"   D = 6'-0"
d = 2'-2"

Load = 2750 lbs = 50 % Ultimate

Soil Description

Soft Clay

c_t = 1580 PSF ϕ_t = 4.2°

c_b = 350 PSF ϕ_b = 0 °

Elapsed Time		Footing Rotation		
Minutes	Days	Degrees	Minutes	Seconds
5		0	44	0
15		0	45	30
32		0	46	30
103		0	48	30
	.8	0	53	0
	9.0	1	1	10
	31	1	1	20
	52	1	5	20
	94	1	9	10
	114	1	19	50
	143	1	29	10
	239	1	31	20
	273	1	31	40

TEST NUMBER GAL-3

Footing Description

H = 12'-0" D = 6'-0"
d = 2'-2"

Load = 1375 lbs = 25 % Ultimate

Soil Description

Soft Clay

c_t = 1580 PSF ϕ_t = 4.2°

c_b = 350 PSF ϕ_b = 0°

Elapsed Time			Footing Rotation		
Minutes	Days		Degrees	Minutes	Seconds
3			0	9	30
30			0	8	40
58			0	8	30
	1		0	6	30
	8		0	8	20
	14		0	10	30
	30		0	10	30
	51		0	10	40
	93		0	10	40