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ANALYSIS OF DRILLED-SHAFT FOUNDATIONS FOR OVERHEAD-SIGN STRUCTURES

Gerald Lowe and Lymon C. Reese

RESEARCH REPORT 244-2F

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Development of Procedures for the Design of Drilled Foundations for Support of Overhead Signs

Research Project 3-5-78-244

conducted for

Texas
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in cooperation with the
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by the

CENTER FOR TRANSPORTATION RESEARCH
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May 1982

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

This is the second of two reports for Research Project 3-5-78-244.

Presented in this report are design procedures for drilled shafts to be used for the foundation of Overhead Sign Bridges. Summaries of procedures for the design of single shafts in tension and compression are made as well as suggested procedures for shafts subjected to axial and lateral loads in conjunction with flexural loadings. The design of closely spaced shafts is also summarized and their interaction evaluated. Results of field tests conducted in San Antonio are also presented.

The authors would like to thank several individuals for their assistance, both in the field and in the office. Mssrs. Maltsberger and Hoy of SDHPT as well as Mr. Hank Franklin and Mr. Jim Anagnos contributed greatly to the field test efforts. Lola Williams and Cathy Collins provided support in the office for both field testing and manuscript preparation. Charles Covill, as engineer-representative of SDHPT, also made suggestions and offered many hours of help.

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ABSTRACT

Design procedures are outlined for drilled-shaft foundations subjected to lateral, axial, and flexural stresses. Single-shaft as well as double-shaft systems are investigated. The effects of shaft interaction for systems involving more than one shaft are treated and suggestions for their design are made. Comparison of computer analysis to field tests performed on two sets of uninstrumented shafts are made. Design charts for single-shaft systems formulated by SDHPT are also investigated and compared to results of a computer based analysis.

KEY WORDS: drilled shafts, lateral loads, soil-structure interaction, design procedures, uninstrumented shaft testing, group shafts, design aids

SUMMARY

This study concerns the design of drilled-shaft foundations for use with Overhead Sign Bridges. Design procedures for single- and double-shaft systems were presented with attention given to the effects of soil-structure and structure-structure interaction. Design charts formulated by SDHPT were checked and found to be adequate for design within stated conditions. Alternate methods of design for unusual cases were advanced for both single- and double-shaft systems.

The results of two field tests on uninstrumented shafts were presented and comparisons to predicted results were made. The observed results indicated that the computer-based analysis gave conservative results.

IMPLEMENTATION STATEMENT

This study presents design procedures for foundations of Overhead Sign Bridges. A procedure for design by charts as well as a computer-based procedure are presented, with the appropriate method of design to be selected on the basis of site information that is available.

Where reliable and adequate data are available, the computer-based method should be used. When only a limited amount of information can be obtained, the procedure utilizing the charts should be followed.

It is suggested that, conditions permitting, double-shaft systems be replaced by adequately designed single-shaft systems. Thus, a more efficient system will be attained.

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τ_{BA}	average shear stress on a freebody due to load Q_A
ρ_a	axial deflection of pile
α	correlation factor
ρ_F	deflection of a single pile calculated by elastic methods
$\bar{\rho}_F$	deflection of a single pile due to a unit load
ρ_k	deflection of the k^{th} pile
$\bar{\phi}$	effective angle of internal friction
γ	effective unit weight of soil
I_{ρ_F}	influence factor
$\alpha_{\rho_{F_{kj}}}$	influence factor
α_1	ratio of settlements
ϕ	reduction factor
ρ	reinforcement ratio
β	relational angle between piles in a group
ϵ_{50}	strain at 50 percent of failure
ρ_T	total axial displacement of shaft
γ_T	unit weight of soil
γ_W	unit weight of water
A	cross sectional area of shaft
A_B	area of shaft base
A_S	side area of shaft

c	circumference of shaft
c_Q	undrained shear strength of clay or shale
d	diameter of shaft
D	diameter of shaft bell
d_u	depth at which f_u occurs
EI	flexural stiffness
E_c	modulus of elasticity of concrete
E_s	soil modulus
F_c	breakout factor for clay
F_q	breakout factor for sand
f	side friction
f_c'	concrete compression strength
f_u	ultimate side resistance
f_y	yield stress of reinforcing steel
H	total depth of embedment of shaft
H_T	total lateral load on pile group
H_{avg}	average lateral load on pile in a group
H_j	load on the j^{th} pile
H_k	load on the k^{th} pile
H_m	load on the m^{th} pile
I_1	an influence factor
I_{gr}	gross moment of inertia
I_{tr}	moment of inertia with transformed steel area
K	lateral earth pressure coefficient
K_R	ratio of pile stiffness to soil stiffness

k_f	base movement factor
k	kips (1000 lbs)
k_s	modulus of subgrade reaction
L	shaft embedment length
l	effective shaft length
M_{max}	maximum shaft moment
M_n	nominal moment
M_t	moment at shaft head
M_u	design moment
m	number of piles in the group
N	blow count SPT or SDHPT-THD Penetrometer Test
N_c	bearing capacity factor
n	number of cycles for cyclic loading case
P_t	lateral load at shaft head
P_{t_a}	allowable lateral load on shaft
P_u	ultimate uplift capacity of shaft
P_x	axial load
P_1	correlation factor
p	soil reaction
$\overline{p}$	effective overburden pressure
p_{max}	maximum allowable soil reaction
p_u	ultimate soil reaction
Q_A	load on pile A
Q_{BA}	load induced on pile B due to Q_A
Q_b	capacity of shaft in end bearing
Q_s	capacity of shaft due in skin friction
Q_u	uplift capacity of shaft underream

q_b	base capacity at 5 percent tip movement
q_s	load transferred along shaft sides
R	ratio of deflections calculated by p-y methods and elastic methods
S	center to center spacing of shafts
s	height of bell
s_{max}	maximum allowable rotation at shaft head
s_t	rotation at shaft head
w	effective weight of shaft
y	deflection of shaft at a given point
y_G	groundline deflection of pile group
y_{max}	maximum allowable deflection at shaft head
y_s	deflection calculated by p-y method
y_t	groundline deflection

CHAPTER 1. INTRODUCTION

OVERHEAD-SIGN SYSTEM

USAGE

Since the inception of the Federal Interstate Highway System in the 1950's, the number of miles of divided, multi-lane, limited access roadway in use has continued to increase yearly. One need that arose with this highway system was for a sign system that is easily legible and understandable to the motorist, and the development of the overhead sign has provided an acceptable solution to this problem. Spanning the full width of the roadway, this system quickly provides directional information in an unambiguous form; the proper lane for a given destination can be easily marked overhead. The structural problem of the sign support has been solved by the use of steel trusses with spans of up to 150 feet (45.7 m). The structure must carry the dead load of the signs, lighting, and truss, as well as the live loadings from wind, snow, and ice. The loads are transmitted through vertical support towers to the foundation (Fig 1.1), which typically consists of one or more drilled shafts. This paper presents methods of analysis and design for both single- and double-shaft systems, and an economic comparison is made.

SUPERSTRUCTURE CONFIGURATIONS

There are currently three configurations for overhead-sign systems that are used or proposed for use by the Texas State Department of Highways and Public Transportation (SDHPT). The first and most commonly observed configuration consists of a horizontal truss supported by vertical trusses at either end. The horizontal truss is a box-type structure consisting of planar Pratt trusses fabricated from steel angles. All signs and lighting are bolted to this structure. At either end of this horizontal structure, vertical trusses, consisting of wide flanges for chords and angles for diagonal web members, carry all loads to the foundation. These vertical trusses are

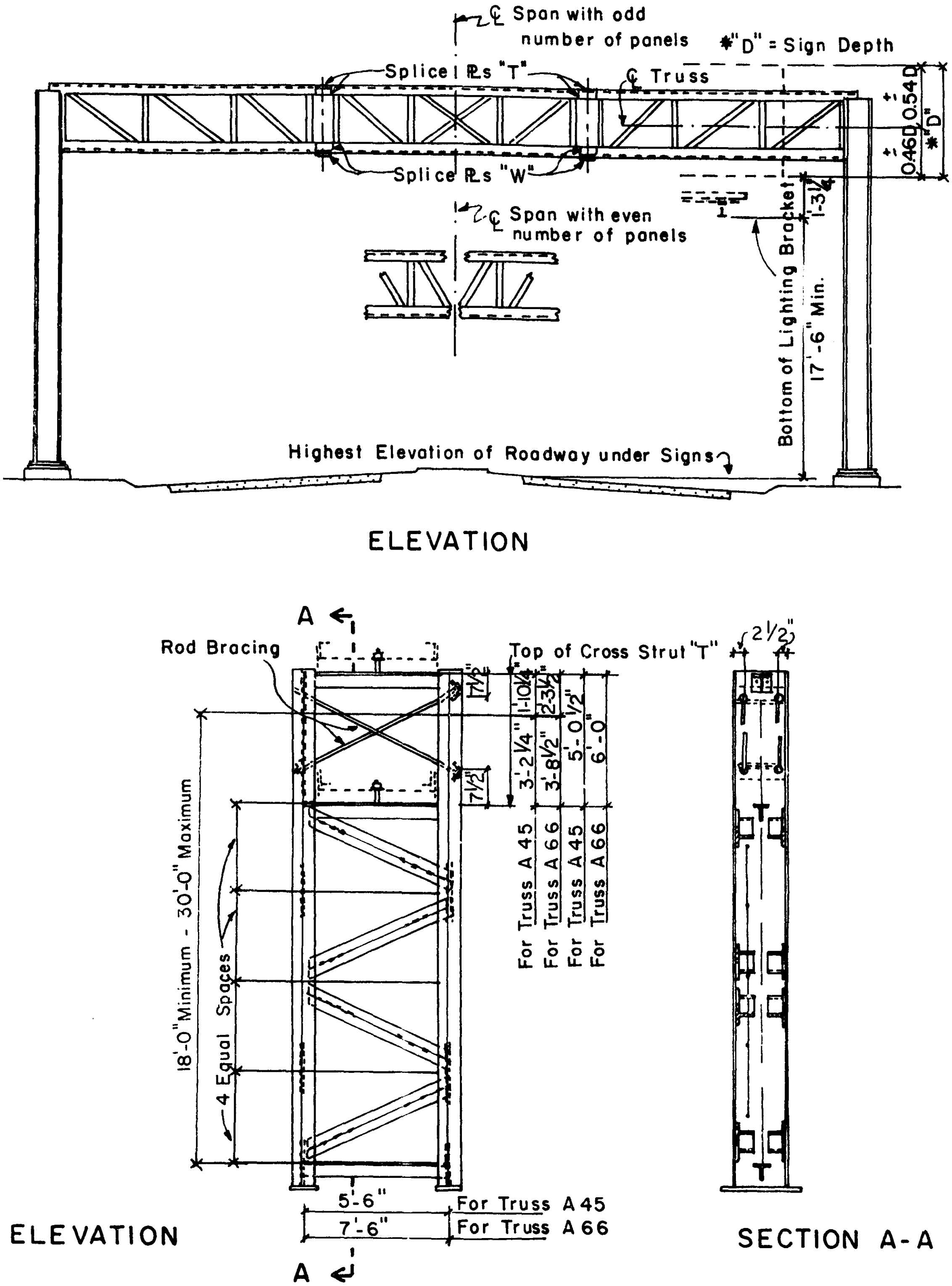


Fig 1.1. Typical configuration of an overhead-sign structure.

(1 ft = .3048 m)

connected to the heads of the foundation shafts by the use of bolted anchor plates. These plates are welded to the wide-flange chords and bolted to anchor bolts which have been cast into the shaft heads. Leveling nuts are then used to attain plumbness of the truss and grout is injected to form a bearing pad between base plate and shaft head. In this case, the connection can be considered to transmit little or no moment. In fact, the moments caused by the horizontal loadings at the main truss are transmitted as either tensile or compressive forces to the bases. Therefore, the foundation system must resist shears and either tensile or compressive forces, but little moment.

The second type of configuration is similar to the first one in several respects and is proposed for use. A horizontal box-type truss is used to attach the signs and lighting; the truss is the same as for the first system, which is described above. However, single columns rather than trusses carry the load to the foundation. This column is concrete, however, not steel, and has a circular cross section that is usually, though not always, the same diameter as the foundation shaft. It can be assumed that the connection between the shaft and concrete column is as strong as the shaft or column.

The third and last configuration consists of a horizontal truss, of either box or planar type, cantilevered out from a single steel-pipe column. This column, in turn, must transmit all loads to the shaft in the form of moments and shears. In all three cases, the loads must be transmitted to a foundation and in turn distributed to the surrounding soil. Typically, this foundation will consist of a cast-in-place, reinforced-concrete drilled shaft on the order of 30 to 48 inches (0.76 to 1.22 m) in diameter, with depths of up to 40 feet (12.2 m). Although drilled shafts can go much deeper, the relatively small loadings that occur rarely call for lengths in excess of 40 feet.

FOUNDATION CONFIGURATION

Although three types of sign configurations exist, the design or analysis of the foundations can be grouped into two main categories, i.e., single shaft and double shaft. The double-shaft system is used in conjunction with the first sign system that was discussed. In this system, each foundation shaft must primarily resist axial forces of a compressive or tensile nature in combination with a horizontal component. Relatively speaking, shaft moments caused by the horizontal shears are small.

For the last two sign systems mentioned, the single-shaft-foundation system is subjected to a slightly different loading condition. For the structure with supports at each end, the vertical loads due to dead load as well as the horizontal shears are practically the same as in the double-shaft system. However, the moments produced by the horizontal loads are no longer transmitted as axial forces; they are transmitted to the shafts as moments and must be resisted by the shafts in bending. The cantilever-type structure is subjected to torsion along with shear and moment. The cantilever design will not be discussed in this report.

AVAILABLE METHODS OF ANALYSIS AND DESIGN

ANALYSIS

The processes of analysis and design of systems using drilled shaft foundations are continually being refined. Newer and more capable methods of computation have allowed the use of systems of analysis and design heretofore unavailable. A problem can now be solved not only by the use of differential equations but also by the use of non-dimensional coefficients or computer-based finite difference methods (Refs 5, 13, 15, and 17). The desired accuracy of the model used for solution of the problem at hand will determine which method of analysis is selected.

DESIGN

The use of computers has encouraged the development of simplified design charts. While these charts are, of practical necessity, restrictive in their application, they can be utilized by the engineer in everyday practice. Under the proper circumstance they can be used for an adequate and quick solution to a given problem. If the situation is too complex, the charts may still be used to give an idea of an appropriate starting point for a computer-based solution. Such computer-based solutions allow a higher degree of freedom in modelling to match the complexities encountered in more difficult problems.

BENEFITS OF IMPROVED METHODS OF ANALYSIS AND DESIGN

Improved methods of analysis and design will in turn lead to the better use of both materials and manpower. A quick and accurate design of the system will allow consideration of construction methods and site-related problems that affect shaft capacities and will allow comparisons to be made with other possible solutions. Situations in which single-shaft foundations may be used in lieu of group shafts or piling, as well as situations in which double-shaft or group systems will perform better than single-shaft systems, will be more easily recognizable. Since not all situations are amenable to the single-shaft solution, the appropriate use of an alternative system will be encouraged by a rigorous investigation.

The ability to establish several different approaches quickly will allow more time to be spent in the evaluation and comparison of economic and construction factors pertinent to each solution. In many instances, the economics will clearly indicate one solution over another, but in some instances the choice may not be as obvious. Under these circumstances, the ability to perform an accurate analysis and design is important and can lead to savings in available funds. In addition, the funamentals that are outlined herein are applicable, without modification, to the problems encountered in the analysis and design of foundations for bent caps, abutments, retaining walls, and similar structures.

The methods of analysis have been treated quite extensively in other papers. The major thrust of this paper is to present design methods and design aids; thus, little time will be spent on analysis other than for a brief review of the existing methods.

CHAPTER 2. ANALYSIS AND DESIGN OF SINGLE-SHAFT SYSTEM

ANALYTICAL METHODS

Before a logical procedure for design can be formulated, a rational method of analysis must be established. Through the use of simplifying assumptions, the problem must be reduced to such a state that a manageable mathematical model can be constructed. Once this is accomplished, the desired design procedure can be established, with the understanding that the solution will never be "exact." Although such a design solution may not be theoretically correct, it may be close enough to real life phenomena to be acceptable. In essence, the solution of the problem that is presented herein reduces to insuring that the soil can provide sufficient reaction to the shaft and that the shaft itself will not fail while keeping the design economically viable.

SYSTEM CONFIGURATION

The loadings on the foundation system can be reduced to lateral load, axial load, and moment, all applied at the pile head. The application of these loads, single or in various combinations, will result in the establishment in the soil system of a reaction which, in turn, produces additional load on the shaft (Fig 2.1). The shaft can be idealized as acting as a beam under concentrated and distributed loads, and the governing differential equations of beam theory can be used for a solution of the problem. If the scheme shown in Fig 2.1 is sufficiently simplified, a closed-form solution can be made. Non-dimensional-coefficient solutions can be used if a more generalized scheme is desired. The greatest degree of freedom, however, is offered by the use of a computer solution using the finite difference method for the approximate solution of the governing differential equations (Ref 13).

The foundation of the cantilever-type structure is also subjected to torsion but, as noted earlier, the cantilever design will not be treated in detail in this report.

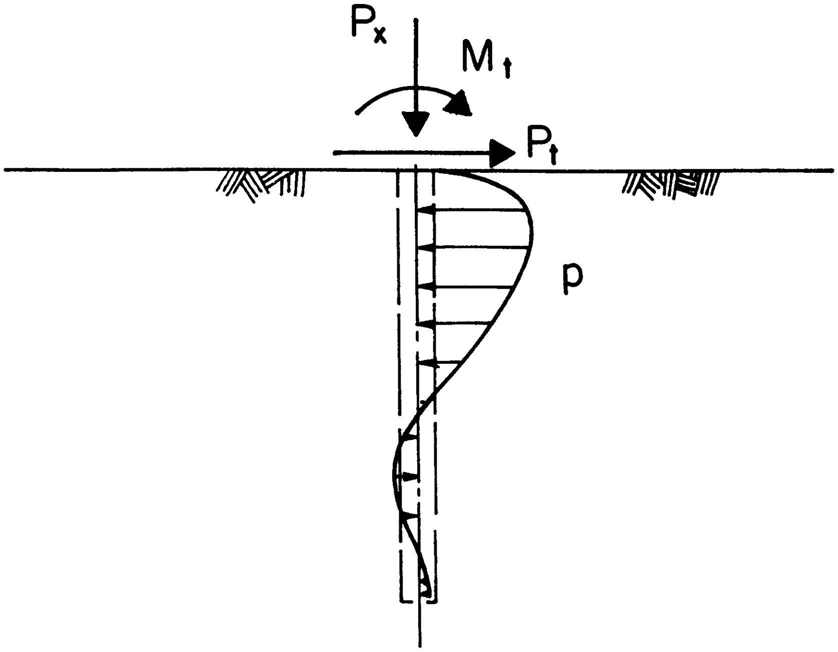


Fig 2.1. Loadings and soil reactions on a shaft.

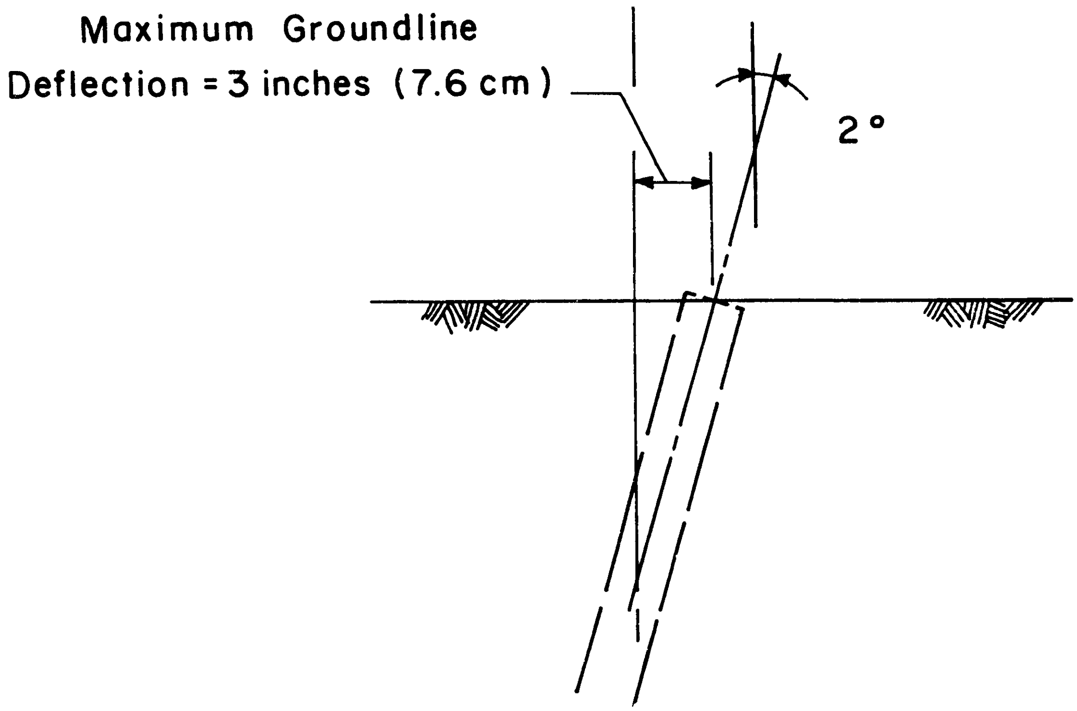


Fig 2.2. Failure limits used in the generation of SDHPT design charts.

AVAILABLE METHODS OF ANALYSIS

The major differences in the three methods are an indication of their ability to adequately model the problem. The closed-form solution restricts the user to using no axial forces. Furthermore, the flexural stiffness (EI) of the pile and the soil modulus (E_s) must be constant with depth even though the soil strength usually increases with depth. It is highly improbable that the model that must be used for the closed-form solution will lead to satisfactory results. The non-dimensional form likewise limits the user to no axial load and a constant EI. However, the soil modulus, E_s, may vary linearly with depth or may have other specified variations with depth. Thus, the non-dimensional solution is a definite improvement over the closed-form solution. The finite-difference method is capable of solutions that allow axial loads to be applied and there may be variations in the EI of the shaft. The soil modulus E_s may also vary in any manner with depth. The one drawback to the finite-difference method is that it requires the use of a computer; in light of the sophistication of the solution and the availability of computer facilities, this may be a relatively minor inconvenience (Refs 5, 13, 14, 15, and 17).

APPLICATION OF ANALYTICAL METHODS TO THE DESIGN PROCESS

The finite-difference method is easily adapted to design and its use has been outlined in the Drilled Shaft Manual, Vol II, and in other publications. Design charts in which a wide variety of design parameters can be considered can be developed by the use of the finite-difference technique (Refs 11 and 13). The non-dimensional method should be used when computers are not available and as a check to a computer solution.

SDHPT DESIGN

SDHPT DESIGN AIDS

Description

Charts were developed at SDHPT for the design of drilled shafts to be used as sign foundations. These charts involve establishing certain loadings, shaft diameters, reinforcement patterns, and shaft embedment lengths and were generated by the use of a finite-difference program called BMCOL 45. The limits imposed upon the solution will be stated, and a design problem will be presented to illustrate the procedures that are used to arrive at a design solution. Finally, an analysis of this solution will be performed using another finite-difference program, COM623, from which relative factors of safety will be determined.

Failure Criteria for SDHPT Design Aids

Three initial limitations were established for the SDHPT design charts. The first limit was on the rotation of the head of the shaft. The limit of the tangent departure at the groundline was set as 2^{o} as the maximum (Fig 2.2). The second limit was that the groundline deflection, y_t , of the shaft was not to exceed 3 inches (7.6 cm) (Fig 2.2). The third limiting factor was that seven-tenths of the ultimate soil resistance was not to be exceeded at any point in the soil system. After establishing these limits to the problem, various combinations of lateral, axial, and moment loadings were run for different soil-pile systems. For instance, an axial load of 26 k (115.6 kN), a lateral load of 50 k (222.4 kN), and a moment of 1500 ft-k (2034 kN-m) were applied to a 30-inch (76.2-cm)-diameter shaft placed in a submerged sandy soil (angle of internal friction of 36^{o}) and the system was then analyzed by the computer for varying lengths. The shortest length was chosen, which insured that none of the three limits was exceeded. The results of these analyses were then used to generate a series of design charts (Figs 2.3 to 2.5). These charts can be used for cases where the heights, spans, wind loading zone, and soil properties are known.

ZONE 4 70 M.P.H. WIND

		SPAN	REACTIONS			COLUMN BENDING MOMENTS (FT. KIPS)																			
		Ft.	D.L.	W.L.	Torque	14'	15'	16'	17'	18'	19'	20'	21'	22'	23'	24'	25'	26'	27'	28'	29'	30'	31'	32'	Height
3/4"ϕ X 1'-3" ANCHOR BOLTS	W8X15 BEARING BEAM	40	2.65	5.03	8.97	81	86	91	96	101	106	112	117	122	127	132	137	142	147	153	158	163	168	173	30"ϕ COLUMN
		45	2.98	5.66	10.09	91	97	102	108	114	120	126	131	137	143	149	154	160	166	172	178	183	189	195	
		50	3.33	6.29	11.21	101	107	114	120	127	133	140	146	152	159	165	172	178	184	191	197	204	210	217	
		55	3.81	6.93	12.34	111	118	125	133	140	147	154	161	168	175	182	189	196	203	210	217	224	231	239	
		60	4.15	7.57	13.46	122	129	137	145	152	160	168	176	183	191	199	206	214	222	230	273	245	253	261	
		65	4.55	8.21	14.58	132	140	149	157	165	174	182	190	199	207	216	224	232	241	249	257	266	274	283	
		70	5.09	8.85	15.71	142	151	160	169	178	187	196	205	214	223	232	241	250	259	268	277	287	296	305	
		75	5.44	9.49	16.83	152	162	172	181	191	201	210	220	230	239	249	259	269	278	288	298	307	317	327	
		80	6.02	9.87	18.06	159	169	179	189	199	209	219	229	240	250	260	270	280	290	300	310	320	330	340	
		85	6.61	10.51	19.19	169	180	191	201	212	223	234	244	255	266	276	287	298	309	319	330	341	352	362	
		90	7.03	11.15	20.32	180	191	202	214	225	236	248	259	271	282	293	305	316	327	339	350	362	373	384	
		95	7.55	11.79	21.45	190	202	214	226	238	250	262	274	286	298	310	322	334	346	358	370	382	394	406	
		100	8.20	12.81	23.65	207	220	233	246	259	272	285	298	311	324	337	350	363	376	390	403	416	429	442	
		105	8.91	13.48	24.83	217	231	245	259	272	286	299	314	327	341	355	369	382	396	410	424	437	451	465	
		110	9.34	14.15	26.02	228	243	257	271	286	300	315	329	344	358	372	387	401	416	430	445	459	473	488	
		115	9.87	14.81	27.20	239	254	269	284	299	314	329	344	360	375	390	405	420	435	450	465	480	495	511	
1"ϕ X 1'-6" A.B.		120	10.45	15.52	29.82	251	26[illegible]	283	299	315	331	346	362	378	394	410	426	441	457	473	489	505	521	536	
		125	11.30	16.20	31.06	262	279	295	312	328	345	362	378	395	411	428	444	461	477	494	510	527	543	560	
		130	12.04	16.87	[illegible]	273	[illegible]	308	325	342	359	376	394	411	428	445	462	480	497	514	53[illegible]	549	566	583	
		135	12.60	17.55	33.56	284	302	320	338	356	374	392	409	427	445	463	481	499	517	535	553	571	588	606	
		140	13.76	18.28	34.81	296	314	333	352	370	389	408	426	445	464	482	501	520	538	557	576	594	613	631	
		145	14.26	19.01	36.06	308	327	346	366	385	404	424	443	463	482	501	521	540	560	579	598	619	637	657	36"ϕ COL
		150	15.72	19.75	37.32	319	339	360	380	400	420	440	460	481	501	521	541	561	581	601	622	642	662	682	
		155	16.25	20.54	38.57	332	353	374	395	416	437	458	479	499	520	541	562	583	604	625	646	667	688	709	

(1 ft = .3048 m, 1 lb-ft = 1.356 N-m, 1 lb = .4536 kg)

Fig 2.3. Column moment selection. Sheet OSBC-SC-Z4, Texas SDHPT Design Charts.

DRILLED SHAFT MOMENTS

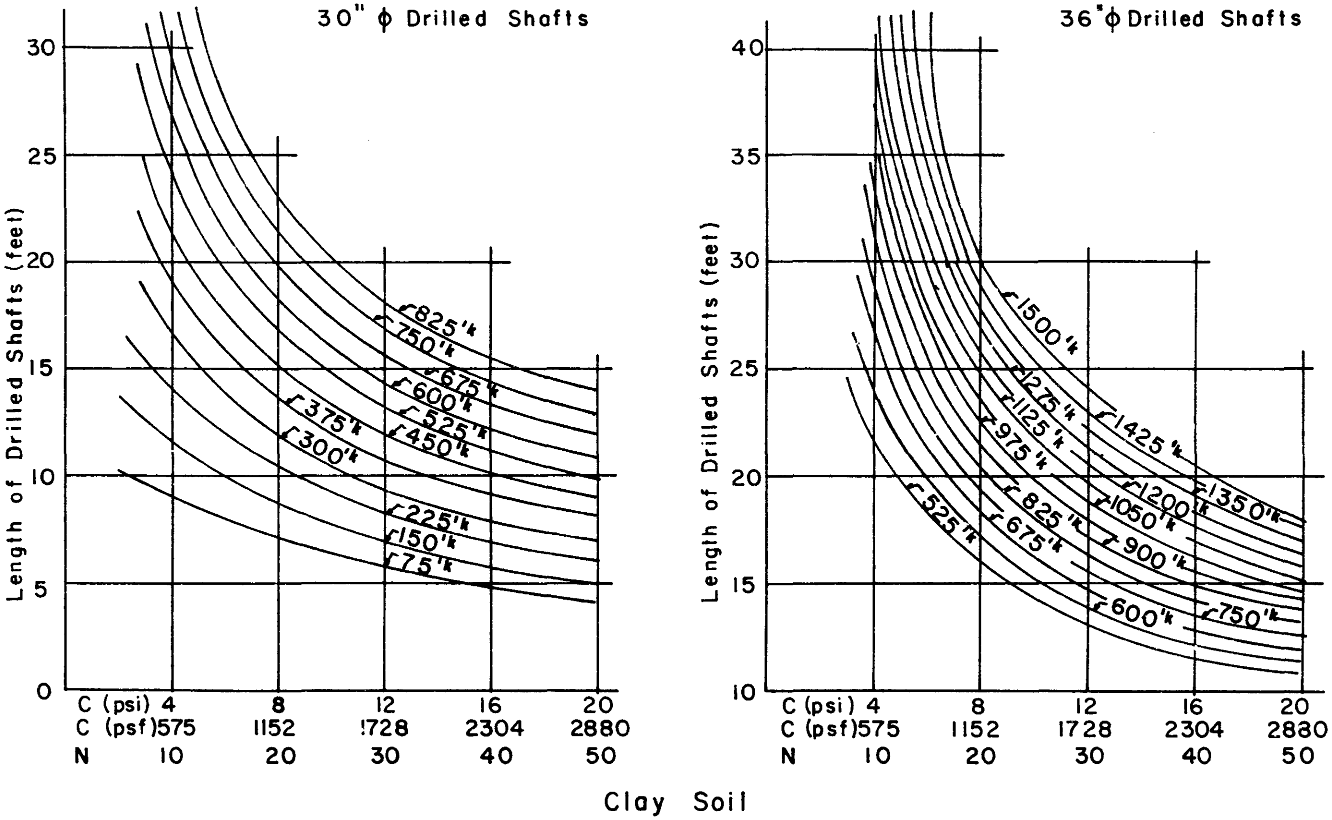
CLAY SOIL												
	30"ϕ DR. SHAFT				36"ϕ DR. SHAFT				42"ϕ DR. SHAFT			
COL. MOMENT \ C	4	8	12	20	4	8	12	20	4	8	12	20
N	10	20	30	50	10	20	30	50	10	20	30	50
75	77	76	75	75								
150	155	154	153	151								
225	234	231	230	228								
300	317	309	307	305								
375	399	388	385	382								
450	484	470	464	461								
525	571	550	544	538	564	548	543	537				
600	657	636	625	615	650	628	623	616				
675	742	717	705	682	739	712	703	694				
750	820	801	787	778	825	795	785	773				
825		888	871	857	921	882	865	853	909	875	863	851
900					1020	968	949	934	998	959	944	931
975					1108	1052	1032	1015	1091	1043	1027	1011
1050					1195	1138	1113	1095	1180	1128	108	1092
1125					1293	1224	1198	1177	1273	215	1193	1173
1200					1383	1317	1282	1258	1373	1301	1277	1264
1275						1404	1369	1339	1460	1393	1359	1336
1350						1490	1448	1420	1565	1478	1443	1416
1425						1577	1539	1504	1655	1568	1532	1498
1500						1664	1627	1584		1657	1614	1579
1575												
1650												
1725												
1800												
1875												
1950												
2025												

1.0 ft = .3048 m , 1 lb = .4536 kg , 1 lb-ft = 1.356 N-m

COLUMN OR DRILLED SHAFT REINFORCING STEEL (GR. 60)				
MOMENT	COLUMN OR SHAFT SIZE			
	30"ϕ	36"ϕ	42"ϕ	48"ϕ
100	8 ~ #9			
150	8 ~ #9			
200	8 ~ #9			
250	8 ~ #9			
300	9 ~ #9			
350	11 ~ #9			
400	13 ~ #9	9 ~ #9		
450	10 ~ #10	11 ~ #9		
500	13 ~ #10	12 ~ #9		
550	12 ~ #11	13 ~ #9		
600	13 ~ #11	15 ~ #9		
650	14 ~ #11	16 ~ #9	10 ~ #10	
700		14 ~ #10	11 ~ #10	
750		15 ~ #10	12 ~ #10	
800		16 ~ #10	13 ~ #10	
850		18 ~ #10	14 ~ #10	
900		19 ~ #10	15 ~ #10	
950		16 ~ #11	13 ~ #11	
1000		17 ~ #11	14 ~ #11	
1050		18 ~ #11	14 ~ #11	
1100		19 ~ #11	15 ~ #11	12 ~ #11
1150			16 ~ #11	13 ~ #11
1200			17 ~ #11	14 ~ #11
1250			17 ~ #11	14 ~ #11
1300			18 ~ #11	15 ~ #11
1350			19 ~ #11	16 ~ #11
1400			20 ~ #11	16 ~ #11
1450			21 ~ #11	17 ~ #11
1500			21 ~ #11	17 ~ #11
1550			22 ~ #11	18 ~ #11
1600			23 ~ #11	19 ~ #11
1650				19 ~ #11
1700				20 ~ #11
1750				21 ~ #11
1800				21 ~ #11
1850				22 ~ #11
1900				23 ~ #11
1950				23 ~ #11
2000				24 ~ #11
2050				
2100				
2150				
2200				

All Column and Shaft Reinforcing to be Grade 60

Fig 2.4. Drilled shaft moment selection and drilled shaft or column reinforcement selection. Sheet OSBS-SC, Texas SDHPT Design Charts.



Minimum embedment of drilled shafts is two diameters; add 3'-0" to required design length of drilled shaft.

1 ft = .3048 m, 1 ft K = 1.356 N-m , 1 K = .4536 Mg

Fig 2.5. Sheet OSB-FD-SC. SDHPT Design Charts.

Design Example Using SDHPT Design Aid

A set of typical parameters was chosen and used to design a shaft from these charts. The relative magnitude of the variables was chosen at random, although the specific values were chosen for convenience for use with the charts so that a minimum amount of interpolation would be needed. It was assumed that the sign system would be 30 feet (9.1 m) in height with a span of 140 feet (42.7 m). This sign was to be founded in a uniform clay with an SDHPT-THD Cone Penetrometer Test value, N , of 30 blows per foot (which corresponded to a chart shear strength, c_Q , of 1,730 lb/ft^2 , or 82.7 kPa). The site for the sign was chosen to be within zone 4, i.e., that area of the state in which 70-mi/hr (113-km/hr) maximum winds (50 year) are expected. Given this information, the design of the shaft foundation and column superstructure is completed in 3 main steps. They are as follows.

Step 1: from sheet OSBC-SC-Z4 (Fig 2.3) obtain the bending moment in the column. For a height of 30 feet (9.1 m) and a span of 140 feet (42.7 m), the moment in the column is found to be 594 ft-k (805 kN-m). The column diameter is 30 inches (76 cm).

Step 2: from sheet OSB-FD-SC (Fig 2.5), obtain the shaft length, using an N value of 30 and the "Clay Soil" graphs. For a 36-inch (91-cm) shaft, L = 13.8 feet (4.2 m); for a 30-inch (76-cm) shaft, L = 14.4 ft (4.4 m). In choosing these lengths, 594 ft-kips was first rounded up to 600 ft-kips (813 kN-m), and the graphs were then employed. From the General Notes, a required 3-foot (0.9-m) length is added to the shaft length, giving L = 16.8 feet $\cong$ 17 feet (5.2 m) for a 36-inch shaft and L = 17.4 $\cong$ 18 feet (5.5 m) for a 30-inch-diameter shaft.

Step 3: from sheet OSBS-SC (Fig 2.4), select the shaft moment and shaft and column reinforcing. Using a column moment of 600 ft-kips, N = 30 and the table for Clay Soils, a shaft moment of 625 ft-kips (847 kN-m) is given for a 30-inch-diameter shaft and 623 ft-kips (845 kN-m) for a 36-inch-diameter shaft. Both shaft moments are rounded up to 650 ft-kips (881 kN-m) and the reinforcement is chosen from the table, Column or Drilled Shaft Reinforcing Steel (GR 60). From this table the values chosen are 14 No. 11 bars for a 30-inch shaft and 16 No. 9 bars for a 36-inch shaft. For the 30-inch column, 13 No. 11 bars are chosen.

Table 2.1 shows a summary of the design. As shown in the table, a cost estimate was made for two combinations of shaft and column sizes.

TABLE 2.1. SUMMARY OF SHAFT DESIGN USING SDHPT DESIGN CHARTS

Shaft Diameter/Column Diameter	36"/30"	30"/30"
Column Moment, ft-kips	594	594
Column Reinforcement	13 # 11's	13 # 11's
Shaft Moment, ft-kips	623	625
Shaft Reinforcement	16 # 9's	14 # 11's
Shaft Length, feet	17	18
Approximate Dollar Cost of Concrete[1]	$954	$700
Steel, lb	925	1338
Approximate Dollar Cost of Steel[1]	$370	$535
Total Approximate Cost[2]	$1324	$1235

[1] Based on lettings in Dallas, August 1979, concrete = $212/c.y., steel = $ 0.40 per lb (U.S. dollars)

[2] Does not include column superstructure, truss, signs, etc.

1 ft = 0.3048 m

1 ft-k = 1.356 kN-m

1 k = 0.4536 Mg

Differences in design length are small; however, because the area of a 36-inch shaft is about 45 percent greater than that of the 30-inch shaft, there is a significant increase in total concrete yardage for the 36-inch shaft. There is more steel required for the 30-inch shaft but this increased cost is offset by the differences in cost of the concrete. For the unit prices that were used, the 30-inch shaft is the most economical choice. Variances in unit prices between steel and concrete could obviously change this conclusion and each case must be investigated to find the most economical design under prevailing market prices. This completes the design using the SDHPT design charts.

ANALYSIS USING COM623

Formulation of Data Set for Computer Solution

The system as design by the SDHPT charts was analyzed with the aid of COM623. The given values, as required for design with the SDHPT charts, left other values to be assumed as necessary for analysis by the computer. These additional parameters were selected on the basis of information given in the literature. The given values, as previously stated, were height = 30 feet, span = 140 feet, and c_Q = 1730 lb/ft^2 for a clay soil. In addition, a value of horizontal load of 18.3 k (87.4 kN) was obtained from sheet OSBC-SC-Z4 (Fig 2.3). The truss weight was 13.8 k (61.2 kN). The column weight was computed as 22.1 k (98.3 kN) using 150 lb/ft^3 (23.6 kN/m^3) for the weight of concrete. The total axial load was therefore 22.1 k + 13.8 k = 35.9 k (160.0 kN). The moment at the shaft head was computed by adding the product of the wind load times the sign height to the value obtained from Fig 2.3. Thus M_t = 34.8 ft-k + 18.3 k (30) feet = 583 ft-k (790 kN-m) .

The soil as presented in the design charts was both homogeneous and of constant strength with depth. The soil was modelled as a stiff clay above the water table, with an effective unit weight, γ , of 115 lb/ft^3 (18.1 kN/m^3) and an undrained shear strength, c_Q , of 1730 lb/ft^2. From the literature, values of strain at 50 percent of failure, ϵ_{50} , of 0.010 and an initial modulus of subgrade reaction, k_s , of 5.0×10^5 lb/ft^3 (7.86×10^7 kN/m^3) were assumed. Since the SDHPT charts were presented with constant soil properties with depth, the parameters used in the computer analysis were also made constant with depth. The diameter of the shaft was selected as 30 inches to agree with the result from the SDHPT procedure. The gross moment of inertia for a 30-inch circular section is about 39,800 in.^{4} (0.01657 m^4). A value of 3.0×10^6 lb/in.^{2} (0.0683×10^7 kN/m^2) was used for the modulus of elasticity of concrete (E_c).

Variation of Parameters Used in Computer Solution

The value of several parameters were varied in turn to establish the general behavior of the foundation. The effect of change in length was obtained by analyzing the shaft using lengths such that the full range of behavior occurred, from the "fence post" (rigid body) action of short piles to the "infinite pile" (flexible member) action of long piles. In addition, the relative position of the water table was varied. This was accomplished by using the total unit weight of the soil, γ_T, for the case where the water table is well below the shaft tip and the buoyant unit weight of the soil ($\gamma_T = \gamma_T - \gamma_W$) for the case where the level of the water table is at the shaft head. Effects of variation in the flexural rigidity of the shaft were also investigated. Analyses were made using both the gross moment of inertia previously mentioned, I_{gr}, and an uncracked, transformed moment of inertia, I_{tr}, in which the steel areas were transformed but it was assumed that the section remained uncracked. Shaft loadings were also varied from values of $0.7\ P_t$ to $2.0\ P_t$ and were applied in both a cyclic and a static manner. This manner of loading is explained subsequently. Center for Transportation Research Report 244-1, "Analysis of Single Piles Under Lateral Loading" (Ref 5), makes use of the same variations plus variations of additional parameters such as undrained shear strength and strain at 50 percent (ϵ_{50}). Although parametric studies in Report 244-1 were conducted on reports from the literature, the basic pattern of behavior will be similar in all cases.

Effects of Parameter Variation on Shaft Behavior

There are basically two patterns of behavior of a shaft or pile under lateral load. The first such pattern is a rigid body rotation of the entire shaft. This "fence post" action is characterized by small curvatures of the shaft itself, accompanied by large deflections of both the shaft head and shaft tip. The second pattern relates to a situation in which more than one point of zero deflection occurs along the shaft length. This results in an increased curvature of the shaft. However, both the head and tip deflection are reduced. The deflected shapes of the shaft, analyzed at lengths of 18 feet and 26 feet, are shown in Fig 2.6. It is obvious that the shorter shaft has a greatly increased groundline deflection, about 3-1/2 times larger than that of the longer pile. In addition, an increase of shaft curvature is also apparent for the long shaft, resulting in a small increase in bending moment. The bending moment is 616 ft-k (835. kN-m) for the long shaft as compared to 608 ft-k (825. kN-m) for the 18-foot shaft. Figure 2.7 presents the variation of groundline deflection as a function of shaft length. For the constant lateral load, the groundline deflections increase as embedment length is decreased. The increase in deflection increases rapidly when the pile length drops below the length necessary to support a long or "infinite pile" action.

To investigate the effect of varying the lateral load on the shaft, solutions were made where the lateral load ranged from $0.7\ P_t$ to $2.0\ P_t$, where P_t was the design value of 18.3 k (81.4 kN). In addition, solutions were made for static and cyclic loading. Static loading mentioned earlier is for the case where the load is applied in a short-term, non-impact manner. Cyclic loading is for the case where the load is applied in a non-impact manner, from zero to the load value desired and back to zero. This application is repeated for n number of cycles (in this case, $n = 20$).

The selection of the method of load application for use with the computer solution must be consistent with the nature of loading in the field. It is implausible at best that an isolated and sudden, yet non-impact, application of 18 k will occur. It is almost equally implausible that exactly 20 cycles of exactly 18 k will load the system from exactly the same direction in each cycle and then dissipate to nothing. The physical description should more nearly approach a system in which the load was applied many times, sometimes even in an impact manner, from different directions and in varying intensities. This would be a much better approximation of the effects of a storm system typical of Gulf Coast hurricanes. However, this would also present serious problems, both in modelling and in the system capacity necessary for such a complex model.

In view of the above arguments, the results shown in Fig 2.7 for static loading may be considered as a lower bound and those for cyclic loading may be considered as an upper bound.

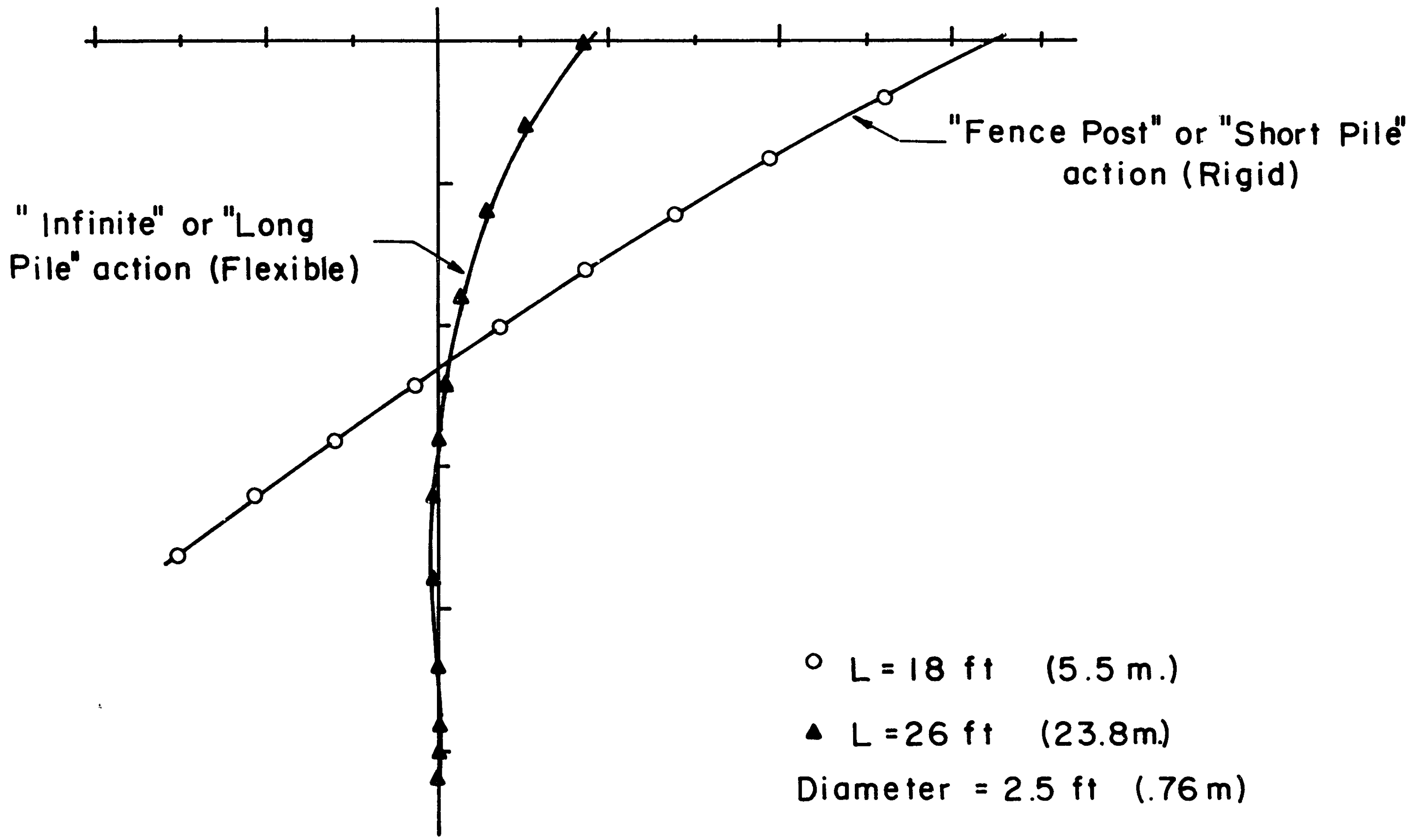
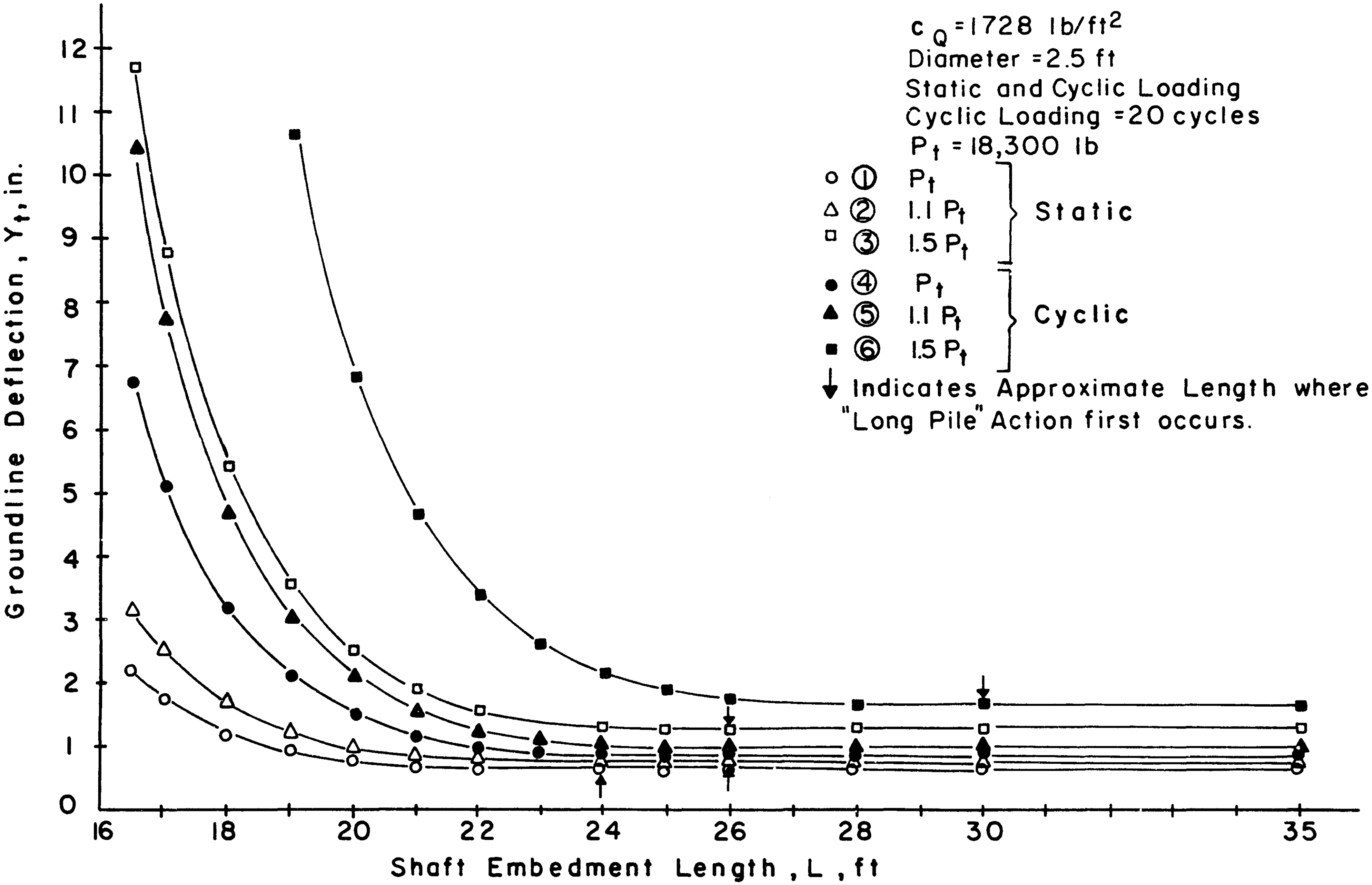


Fig 2.6. Deflected shapes - rigid and flexible behavior.



1 ft = .3048 m, 1000 lb = .4536 Mg, 1000 lb/ft^2 = 47.88 MPa

Fig 2.7. Shaft embedment length versus groundline deflection - variations in loading.

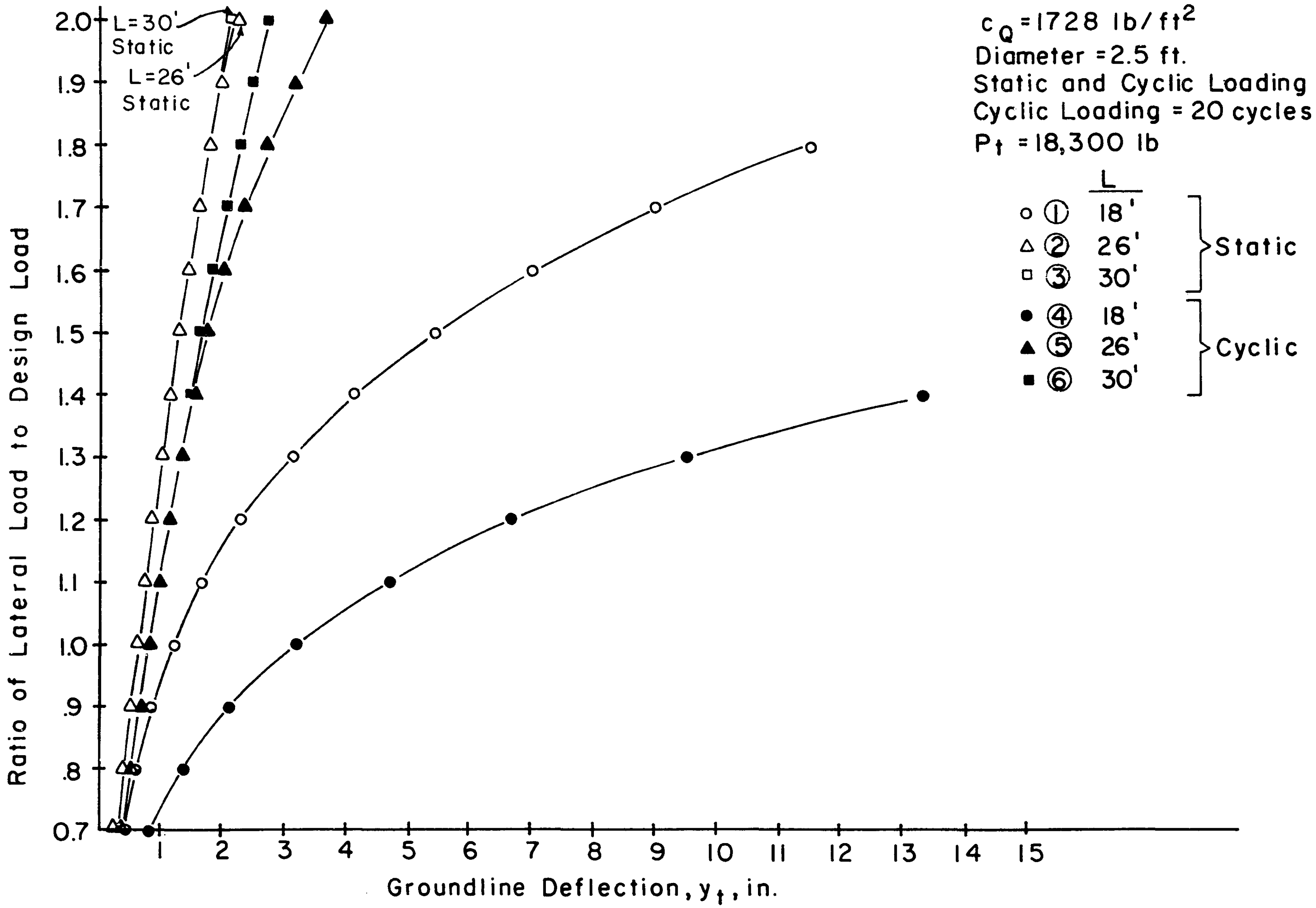
The cyclic-loading method is thought to be applicable to the design of overhead-sign structures because wind velocity is seldom constant. The gusting of the wind will cause repeated loadings to occur on the foundations. It is known that a degradation of the soil resistance occurs with cyclic loadings and the loss in soil resistance can be quite severe (Ref 4). The increased deflections caused by a lessening of soil support can be accompanied by an overstress of structural elements. Therefore, it is important to know not only the behavior of the system subjected to a given load but also the behavior of the system under cyclic loading.

The curves presented in Fig 2.7 represent changes in both load intensity and method of load application. Curves 1 through 3 indicate the behavior to be expected due to an increase in load, as do curves 4 through 6. In general, both sets of curves exhibit the same characteristics. It is of interest to note that the ratio of cyclic to static deflection increases as the shaft length approaches that for rigid-body behavior. For example, at the design load with a shaft length of 26 feet (7.9 m) the ratio is 1.31, while for a length of 18 feet (5.5 m) the ratio is 2.7.

Figure 2.8 is presented to show more clearly the effect of lateral load on groundline deflection. As may be seen, the groundline deflection increases almost linearly with load for shaft lengths of 26 and 30 feet; however, great nonlinearity is shown for the shaft length of 18 feet.

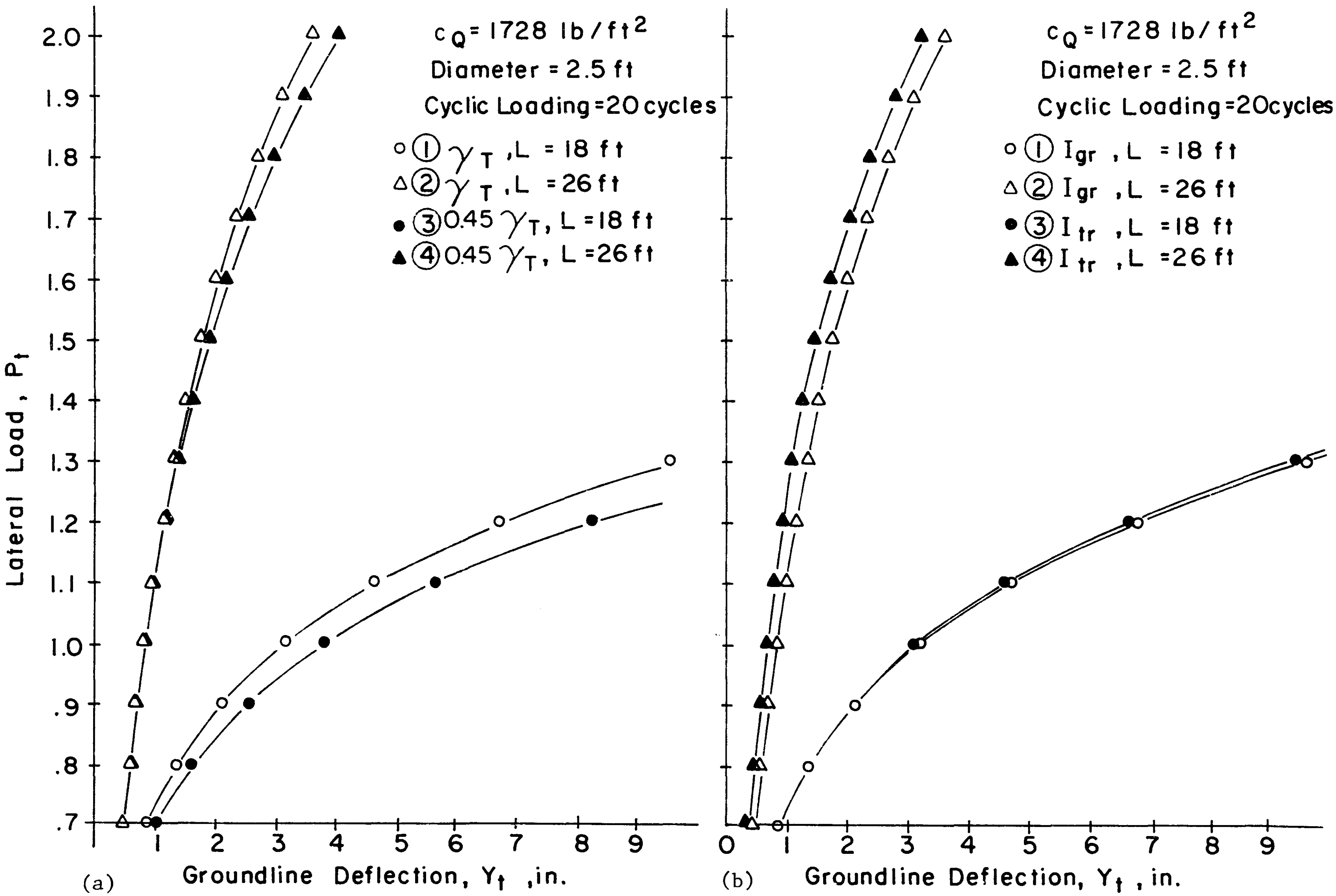
Variations in the water table level were also studied by varying the unit weight of the soil as noted earlier. Curves 1 and 2 in Fig 2.9a present load-deflection curves for the case where the water table was well below the pile tip. Curves 3 and 4 represent the same pile system but with the ground water level at the head of the pile. For the longer shaft, curves 2 and 4, there is no change in deflection until a load of around 1-1/2 times the design load is applied. On the other hand, curves 1 and 3, for the 18-foot shaft at design load, reflect a 20 percent increase in the groundline deflection as the water table level is raised.

Figure 2.9b presents results for the case where shaft rigidity (EI) was varied. The relative stiffness of the pile or shaft was varied from EI_{gr} to EI_{tr} , as mentioned earlier. For the example computations, the relative increase in the moment of inertia between I_{gr} and I_{tr} was roughly 30 percent. Curves 1 and 3 in Fig 2.9b show results for a shaft that



1 ft = .3048 m, 1000 lb = .4536 Mg, 1000 lb/ft^2 = 47.88 MPa

Fig 2.8. Groundline deflection versus lateral load - load and length variations.



1 ft = .3048 m, 1000 lb = .4536 Mg, 1000 lb/ft^2 = 47.88 MPa, 100 lb/ft^3 = 1.602 Mg/m^3

Fig 2.9. Groundline deflection versus lateral load - variations in shaft moment of inertia and unit weight of soil.

was 18 feet long, with the moment of inertia being varied, as shown. The figure indicates that the stiffness of the shaft for a short shaft has little effect on overall deflections. Curves 2 and 4 show results where the moment of inertia of a 26-foot shaft was varied. There is a noticeable change in deflections; there is a relative increase of 25 percent at design load. However, the actual increase in deflection is small, from approximately 0.67 inch (1.7 cm) to 0.85 inch (2.16 cm), or about 0.2 inch (0.51 cm).

The variations in the parameters mentioned lead to several conclusions. These may be summarized as follows.

(1) Deflections are sensitive to both shaft length and nature of loading.

(2) Stiffness of the shaft has a relatively small effect on the deflection pattern.

(3) Changes in soil unit weights will not have a great effect on shaft deflection although relatively short shafts will be affected more than longer shafts.

COMPARISON OF COMPUTER SOLUTION TO DESIGN-AID SOLUTION

Use of the charts resulted in selecting a design shaft length of 18 feet (5.5 m) for a 2.5-foot (76.2-cm)-diameter shaft. Curve 1 in Fig 2.7 shows that the 18-foot shaft will behave as a rigid body under a static loading. The deflection of the shaft head is approximately 1.2 inches (3.1 cm), well within the 3-inch limit established for the charts. The slope at the shaft head is 0.74°, well within the limits set for slope. The computer analysis was used to calculate the soil reaction at various points along the pile according to the formula

$$p = E_s y \quad . \tag{2.1}$$

At the shaft base, $p = 1218$ lb/in., whereas p_u is 2807 lb/in. and $0.7p_u$ is 1965 lb/in. Factors of safety were then computed based on the following:

$$\frac{y_{max}}{y_t} \quad \text{for deflection} \quad , \tag{2.2}$$

$$\frac{s_{max}}{s_t} \quad \text{for slope} \quad , \tag{2.3}$$

and

$$\frac{p_{max}}{p} \quad \text{for soil reaction} \tag{2.4}$$

where y_t and s_t are the deflection and slope at the shaft head, p is the greatest value of soil reaction occurring along the shaft, and p_{max} , y_{max} , and s_{max} are the maximum allowable values established for soil reaction, deflection, and slope. The values of factor of safety thus computed are

$$\text{deflection} \quad \frac{3}{1.2} = 2.5$$

$$\text{slope} \quad \frac{2}{.74} = 2.7$$

$$\text{soil reaction} \quad \frac{1965}{1218} = 1.6$$

These values indicate that the chart gave a shaft length for the example problem such that the limits established for behavior of the drilled shaft were not exceeded.

CHAPTER 3. ANALYSIS AND DESIGN OF DOUBLE-SHAFT SYSTEMS

APPLICATION OF LOAD

The basic configuration of the double-shaft system was mentioned in Chapter 1. In brief review, the loading of the sign structure produces both lateral loads and moments, which are transmitted by vertical trusses to the foundations. The lateral forces are transmitted to the shaft heads as shears while the moment is transmitted as a couple by the truss action. The couple causes a tensile force on one shaft and a compressive force on the other. The design of the shaft or pile must, therefore, account for both axial and lateral forces. Appropriate care must be taken in the design process to insure the adequacy of the shaft for resisting the axial forces in light of the fact that the bending and deflection of the shaft under the lateral load have an influence on its axial behavior. The first step in the process is to formulate a design procedure for the axial loadings. The second step is to check the influence of the shear and moment on the axial solution. There are two recommended design procedures for axial loadings; one for compressive forces and the other for tensile forces. The case for tensile forces is treated first.

ANALYSIS OF A SHAFT SUBJECTED TO TENSILE LOADING

Alexis Sacre proposed a method of design based on experiments conducted on several test shafts (Ref 16). Equations are advanced for determining the capacity of a shaft when loaded by an uplifting (tensile) force. Cohesive soils, clays and clay-shales, and cohesionless soils are considered.

COHESIVE SOILS

Several factors are involved in the computation of the shaft capacity. The most obvious factor is the soil type. For clays, the following equation for the shaft capacity is given:

$$P_u = f(L - 5)\,\pi d + W' \qquad (3.1)$$

where

P_u = ultimate uplift capacity of the shaft,

L = length of the shaft,

d = diameter of the shaft,

W' = effective weight of the shaft (accounting for buoyancy),

f = side friction,

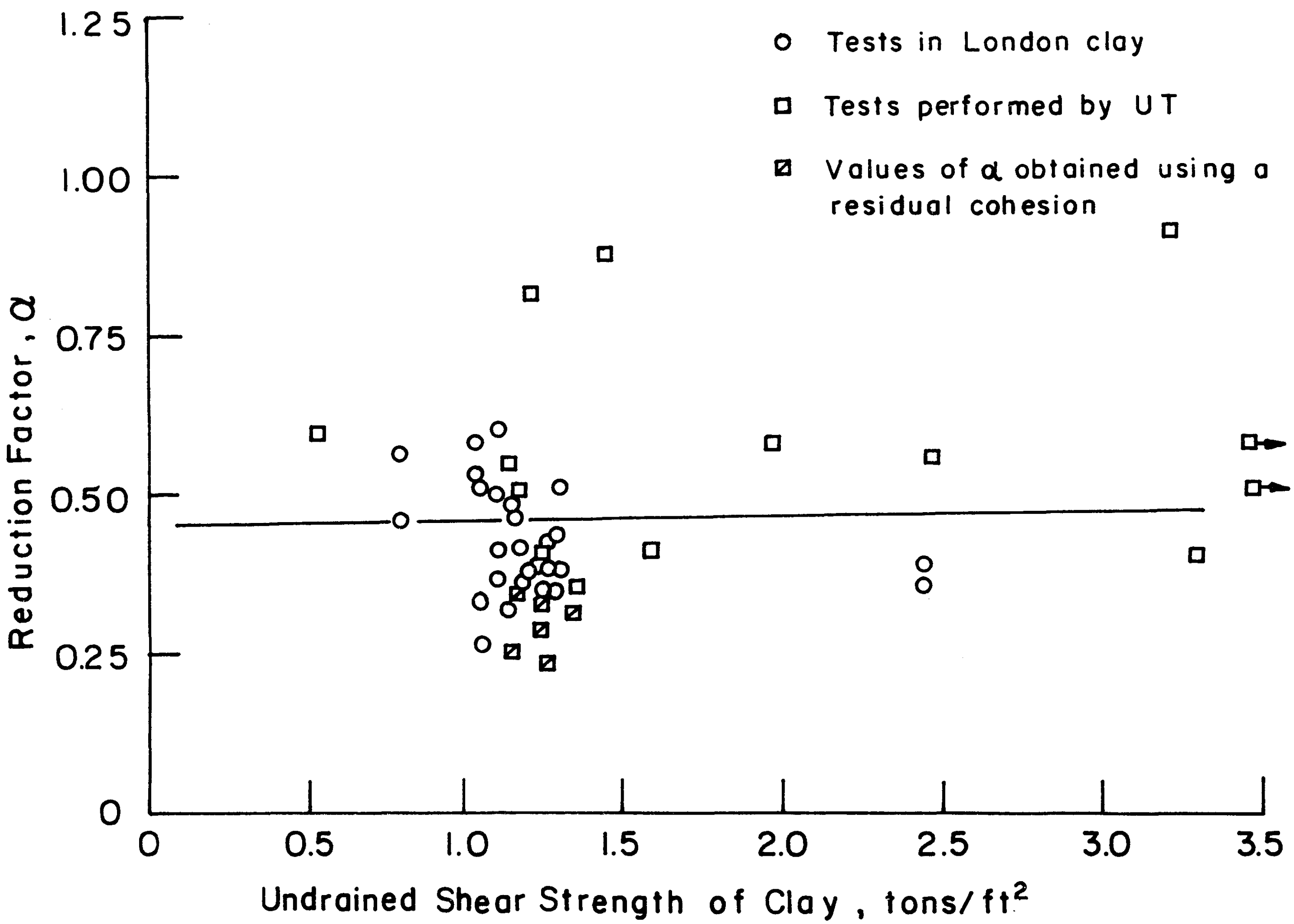
= $\alpha\, c_Q$,

α = correlation factor (see Table 3.1), and

c_Q = undrained shear strength of clay.

P_u is the ultimate capacity of the shaft to resist pullout. The first term in Equation 3.1 represents the capacity of the shaft developed by the interaction of the soil and shaft. In this first term the quantity f is the "side friction" (also termed "skin friction"), with f as a function of the soil shear strength. Various factors such as construction technique, soil properties, and concrete condition will affect the capacity of the soil to develop a given loading. The factor α attempts to account for this variability (Ref 11). Figure 3.1 shows values of α that were computed from a number of load tests. There appears to be a large scatter in the results; however, some of the tests from London were not performed using modern construction techniques. The tests performed by The University of Texas, not using the residual cohesion, are thought to be most representative.

In addition to the reduction of the soil capacity by the α-factor, the top 5 feet of the shaft should not be counted on to contribute to the shaft



1000 lb/ft^2 = 47.88 MPa

Fig 3.1. Correlation factor, α, for design of shafts in tension.

capacity. The first term therefore is seen as the capacity of the soil to resist the load in the shaft. The second term, W', is simply the net weight of the shaft itself, taking into account the buoyant effect of the water.

The value of α to be used in Eq 3.1 may be selected as 0.6 for good construction methods, for example, with the dry method of construction if the excavation is not allowed to remain open for many hours. If there is an inward deformation of the soil due to creep there can be a reduction in shear strength. In such cases and in other instances of questionable construction procedures, the value of α should be reduced (Ref 12).

The limit on side shear for clays is nominally 2 tons/ft^2 but values of load transfer much larger have been measured in experiments in shale, as discussed below. The limit in side shear is established as the maximum value that has been measured in experiments with instrumented drilled shafts.

For drilled shafts in clay-shales that are subjected to tensile loading, the uplift capacity may be computed by use of Eq 3.1. The same values for α can be used for the clay-shales as for the clays; however, load transfer values as high as 7 tons/ft^2 have been measured in a test of an instrumented drilled shaft (Ref 1). Load transfer values of such a magnitude would need to be used with caution, of course, because of the small number of load tests that have been performed on instrumented drilled shafts in clay-shale.

The assumption implicit in Eq 3.1 is that the shaft is straight-sided. If an underream is added, the capacity of the shaft is changed and the capacity of the underream is added to that of the shaft previously computed using Eq 3.1 except that the length of the shaft must be reduced. Underream capacities can be computed by the following formula:

$$Q_u = (c_Q F_c + \bar{\gamma}\, l F_q)(D^2 - d^2)\frac{\pi}{4} \tag{3.2}$$

where

Q_u = uplift capacity of the underream,

c_Q = undrained shear strength of clay or shale,

F_c and F_q are breakout factors for clay and sand, respectively (see Figs 3.2 and 3.3),

ℓ = L - 1.5D - S (see Fig 3.4),

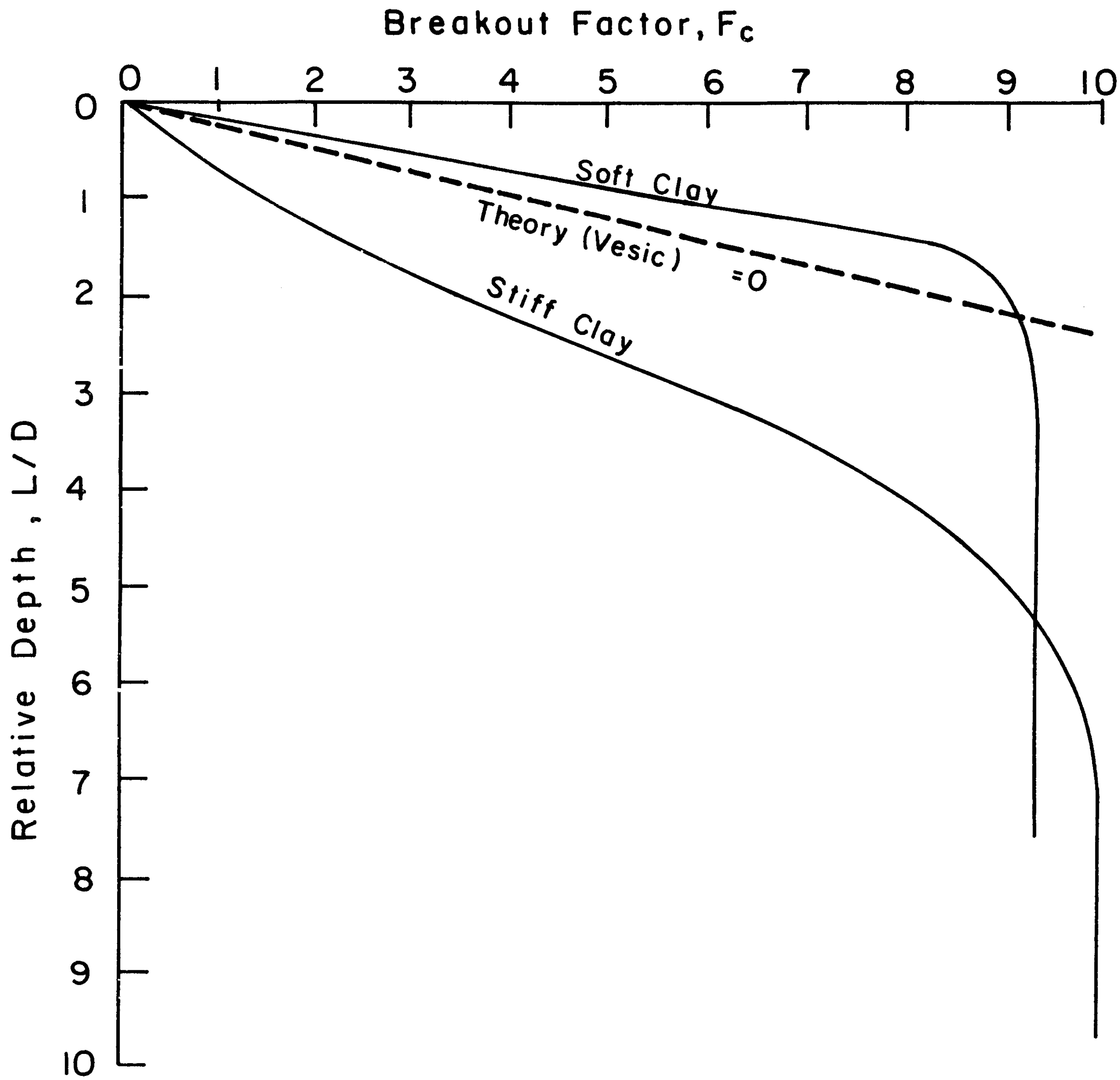


Fig 3.2. Breakout factor, F_c , for clay soils.

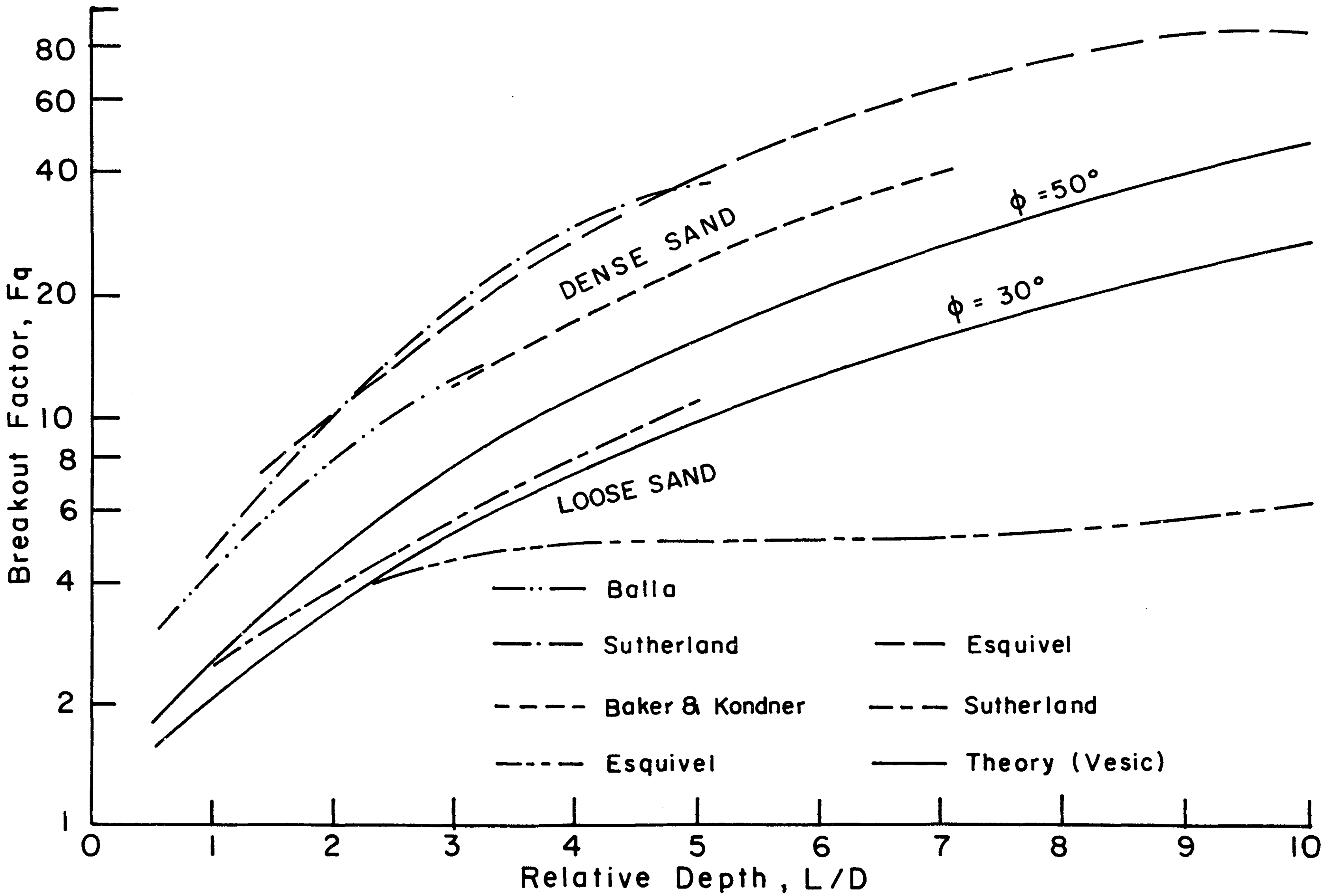


Fig 3.3. Breakout factor, F_q , for sand soils.

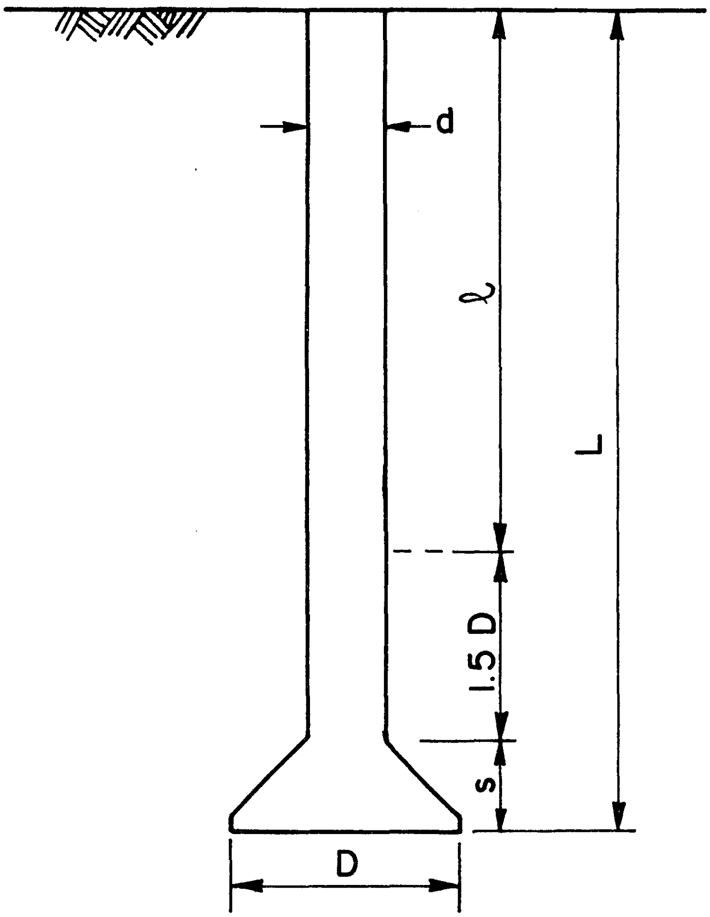


Fig 3.4. Effective shaft length, ℓ, for shafts with underreams.

L = depth to base of bell,

S = height of bell,

D = diameter of bell, and

d = diameter of shaft.

While Eq 3.2 deals with the case where the underream is cut into sand, it is rare when such a construction procedure is possible. An underream cut into sand could very well collapse even though drilling fluid is employed to maintain the shape of the excavation.

If a comprehensive soil study has not been performed in which undisturbed samples have been taken and in which various in-situ techniques have been employed, the value of the undrained shear strength may be obtained from the results of penetration tests. Table 3.1 shows such correlations (Ref 2).

TABLE 3.1. CORRELATION BETWEEN BLOW COUNT FROM PENETRATION TESTS AND UNDRAINED SHEAR STRENGTH (after Ref 1)

Values of c_Q in tons/ft^2

Clay Type	Blow Count, N: SPT	Blow Count, N: SDHPT Pen. Test
Homogeneous - CH	0.10 N	0.07 N
Silty Clay - CL	0.09 N	0.063 N
Sandy Clay - CL	0.076 N	0.053 N
Clay Shale	0.0188N	0.0133N

(1000 lb/ft^2 = 47.88 MPa)

COHESIONLESS SOILS

The tensile capacity of a straight-sided shaft in cohesionless soils is given by Eq 3.3:

$$P_u = \left[d_u \frac{f_u}{2} + (L - d_u) f_u \right] \pi d + W' \tag{3.3}$$

where

P_u = uplift capacity of shaft,

f_u = ultimate side resistance (see Fig 3.5),

d_u = depth at which f_u occurs,

$= \dfrac{f_u}{\bar{\gamma} \; K \tan \bar{\phi}}$,

$\bar{\gamma}$ = effective unit weight of sand,

K = lateral earth pressure coefficient,

$\bar{\phi}$ = angle of internal friction of sand,

L = shaft length,

d = shaft diameter, and

W' = effective weight of shaft.

Side resistance increases from 0 at the groundline to some limiting value, f_u , at depth, d_u . Figure 3.5 presents f_u values as a function of the Standard Penetration Test (SPT) blow count (Ref 13). For SDHPT purposes, correlations between the SDHPT pen test and SPT have been made (Ref 18). As in the equation for clay soils, the first term is the capacity of the soil to resist the loading and the second term is the effective weight of the shaft.

As an example of the use of Eq 3.3, assume that a drilled shaft that is 4 feet in diameter has been installed in a sand with a ϕ of 40 degrees and a submerged unit weight of 60 lb/ft^3. A value of 0.7 is selected for K . Using Fig 3.5, the ultimate side resistance is 1.38 $tons/ft^2$. The depth d_u at which this ultimate side resistance will develop is computed to be 78.3 ft.

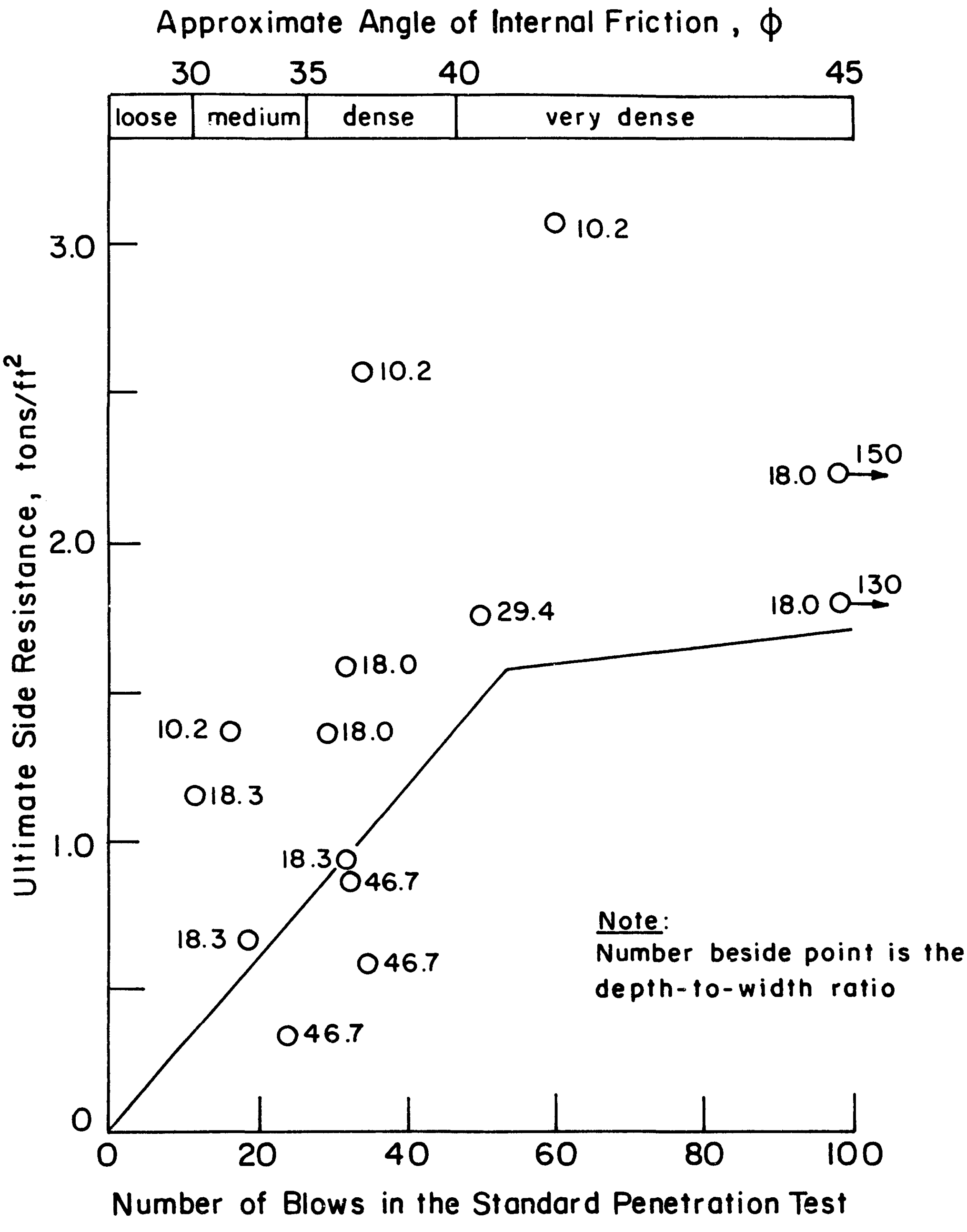


Fig 3.5. Ultimate side resistance, f_u, for design of shafts in tension.

Therefore, the side resistance at 40 feet would be 0.70 tons/ft^2. Thus, the first term in Eq 3.2 yields a value of P_u of 176 tons. Had d_u been computed as less than 40 feet, Eq 3.3 would have been employed without change.

ANALYSIS OF A SHAFT SUBJECTED TO A COMPRESSIVE LOADING

A drilled shaft under compressive load usually distributes its load to the supporting soil both in skin friction and end bearing. The relative magnitude of the load carried in skin friction and end bearing depends principally on the nature of the soil deposit but the shaft geometry will also play an important role. The settlement of a drilled shaft under a given load will also depend on the soil profile and on the geometry of the shaft. The following paragraphs present a review of methods of analysis of drilled shafts in compression.

A basic formula for the computation of the capacity of a drilled shaft in compression is given by Quiros and Reese (Ref 10):

$$Q_{ult} = Q_s + Q_b \tag{3.4}$$

where

Q_s = capacity of shaft in skin friction, and

Q_b = capacity of shaft in end bearing.

The categories of construction techniques are discussed in some detail elsewhere (Refs 10 and 11). As with the section on tensile capacity, the capacity in compression is discussed first for cohesive soils and then for cohesionless soils.

COHESIVE SOILS

For deposits that are predominantly clay, the values of Q_s and Q_b are obtained from Eqs 3.5 and 3.6:

$$Q_s = \alpha c_Q A_s \tag{3.5}$$

and

$$Q_b = N_C c_Q A_B \tag{3.6}$$

where

α = correlation factor (Table 3.2),

c_Q = average undrained shear strength,

N_C = bearing capacity factor (Table 3.2),

A_s = area of shaft surface, and

A_B = area of shaft base.

In addition to giving values of α and N_C, Table 3.2 shows the portions of a drilled shaft in compression that are assumed to be noncontributing (Ref 6).

It should be noted that the construction categories have a significant influence on the design parameters. The importance of the construction method was noted in the discussion of the design of drilled shafts to sustain tensile loadings. The construction categories are recognized in a more formal way in Table 3.2, as follows:

Category A:

Subcategory A.1: Shafts installed dry or by the slurry displacement method.

Subcategory A.2: Shafts installed with drilling mud along some portion of the hole such that entrapment of drilling mud between the sides of the shaft and the natural soil is possible.

Category B: Underreamed drilled shafts in either homogeneous or layered clay with no soil of exceptional stiffness relative to the soil around the stem, below the base.

Subcategory B.1: Shafts installed dry or by the slurry displacement method.

TABLE 3.2. DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY

(Primary Design Procedure)

Parameter		Design Category A.1	A.2	B.1	B.2	C	D
Side resistance* in clay	α_{avg}	0.6	0.3[a]	0.3	0.15[c]	0	0
	Limit on side shear (tsf)	2.0	0.5[b]	0.5	0.3[d]	0	0
Tip resistance** in clay	N_c	9	9	9	9	9	9

[a]May be increased to category A.1 value for segments of shaft drilled dry.

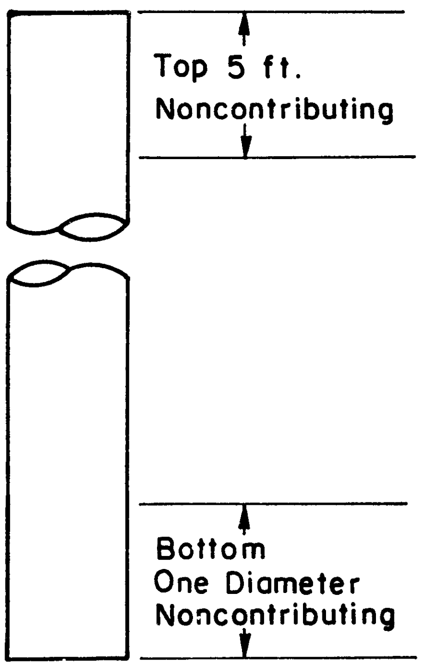
[b]Limiting side shear = 2.0 tsf for segments of shaft drilled dry.

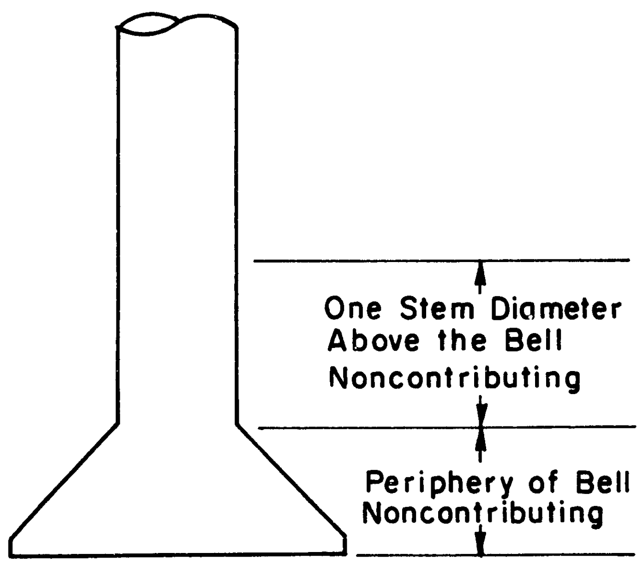
[c]May be increased to category B.1 value for segments of shaft drilled dry.

[d]Limiting side shear = 0.5 tsf for segments of shaft drilled dry.

*Equation for computing side resistance: $(Q_s)_{ult} = \alpha_{avg}\, s_u\, A_s$

**Equation for computing base resistance: $(Q_B)_{ult} = N_c\, c_Q\, A_B$





Subcategory B.2: Shafts installed with drilling mud along some portion of the hole such that entrapment of drilling mud between the sides of the shaft and the natural soil is possible.

Category C: Straight-sided shafts with base resting on soil significantly stiffer than the soil around the stem. The stiffer soil will not allow the shaft side resistance to be developed.

Category D: Underreamed shafts with base resting on soil significantly stiffer than the soil around the stem. The stiffer soil will not allow the shaft side resistance to be developed.

Limiting values on side shear are also given in Table 3.2. If a detailed analysis of the soil deposit is not available, the capacity may be computed using the results of a Standard Penetration Test or an SDHPT Pen Test. The contribution due to skin friction, Q_s, is computed as in Eq 3.5, with values of c_Q being given in Table 3.1 (Ref 2) and α in Table 3.2. However, for the base capacity, Eq 3.8 must be used:

$$Q_B = \frac{N}{p} A_B \qquad (3.8)$$

where

N = blow count SPT or SDHPT Pen Test,

p_1 = correlation factor obtained from Table 3.3, and

A_B = area of the shaft base.

For the design of drilled shafts under compressive loads in clay-shales, the procedures set forth for clay may be used except that the limiting side resistance can be increased to as high as 7 tons per square foot and that the bearing capacity factor N_c should be decreased to 8.

TABLE 3.3. DESIGN PARAMETERS FOR BASE RESISTANCE FOR DRILLED SHAFTS IN CLAY

(After Quiros and Reese, 1977)

Parameter	Design Category					
	A.1	A.2	B.1	B.2	C	D
p_1 (SPT)	1.6	1.6	1.6	1.6	1.6	1.6
p_1 (SDHPT)	2.8	2.8	2.8	2.8	2.8	2.8
Limit on bearing pressure (tsf)	35	35	35	35	35	35

Note: Equation for computing base resistance:

$$Q_B = \frac{\bar{N}}{p} A_B$$

COHESIONLESS SOILS

For shafts used in sand deposits the same basic equation (3.4) is used. The resistance to load offered by the shaft side surface is

$$Q_S = \alpha_{avg}\, c \int_0^H \bar{p} \tan \bar{\phi}\, dz \tag{3.9}$$

where

c = circumference of the shaft,

H = total depth of embedment of the shaft,

$\bar{\phi}$ = effective angle of internal friction,

$\bar{p}$ = effective overburden pressure, and

α_{avg} = correlation factor (Table 3.4).

For the base resistance

$$Q_B = \frac{\pi D^2}{4k} q_b \tag{3.10}$$

where

D = base diameter,

q_b = base capacity at 5 percent tip movement (Table 3.6), and

k_f = base movement factor (Table 3.5).

When the design procedure for drilled shafts in sand under compressive loading is based directly on the results of penetration tests, the following procedure may be used.

The equation for side resistance is

$$Q_S = q_s A_s \tag{3.11}$$

TABLE 3.4. DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY-SHALE

(After Quiros and Reese, 1977)

	Parameter	Design Category A	Design Category B	Design Category C
Side resistance* in clay-shale	α_{avg}	0.75	0.50	0.50
Tip resistance** in clay-shale	N_c	8	8	7

Category A: Shafts installed by the dry method

Category B: Shafts installed by the casing method

Category C: Shafts installed by the slurry displacement method

*Equation for computing side resistance: $Q_s = \alpha_{avg} s_u A_s$

**Equation for computing base resistance: $Q_B = N_c c_Q A_B$

TABLE 3.5. TIP MOVEMENT FACTOR, k_f

Base Diameter, D, ft	k_f
< 1.67	1.0
> 1.67	0.6D

(1 ft = 0.3048 m)

TABLE 3.6. DESIGN PARAMETERS FOR DRILLED SHAFTS IN SAND

(After Quiros and Reese, 1977)

	Parameter	Value	Remarks
Side resistance* in sand	q_s(tsf)	$0.014N_{SDHPT}$	for SDHPT cone penetration test results
		$0.026N_{SPT}$	for SPT results
			side resistance should be limited to 2.0 tsf
Base resistance** in sand	q_B(tsf)	0	loose sand
		16	medium-dense sand
		40	very dense sand

*Equation for computing side resistance: $Q_s = q_s A_s$

**Equation for computing base resistance: $Q_B = \frac{\pi D^2}{4k} q_b$

Tip movement is limited to one inch. The ultimate bearing pressure, q_b , can be interpolated for intermediate densities.

where

q_s = load transferred along shaft sides (Table 3.6).

The equation for end bearing remains the same as before, with N_{SPT} or N_{SDHPT} being used to ascertain whether or not the sand is loose, medium dense, or very dense.

ADDITIONAL FACTORS AFFECTING SHAFT CAPACITY

In the case of clay soils, it has been pointed out that the top 5 feet of shaft are to be ignored in computing the shaft capacity. This is due to several factors. One major factor is the shrinkage of a desiccated soil layer, which will result in poor or even no contact between the shaft surface and the soil medium. It is also recognized that, on many occasions, the upper several feet of soil may be substantially weaker due to factors such as weathering, fissuring due to cycles of expansion and contraction, the addition of organic substance due to plant growth, and the decay of plant growth.

A factor that also affects clays and will affect some cohesionless soils is the degradation of capacity near the surface due to lateral loadings and deflections. Any lateral load will cause the shaft to deflect laterally. If this deflection is severe enough, a pronounced separation of the soil and shaft will occur. For this reason, lateral loadings and deflections must be accounted for within the axial design. This holds for both tensile capacities and compressive loadings. In both cases the lateral deflections should be analyzed and judgments made as to their effect on the skin friction capacities of the shaft.

The factor of safety should be selected after a careful consideration of all the elements affecting the design. In general, the best approach is to compute the collapse load or the load that produces excess deflection and to compute the factor of safety as the ratio of the computed ultimate load to the working load.

AVAILABLE PROGRAMS FOR COMPUTER ASSISTED ANALYSIS OF SHAFTS IN COMPRESSION

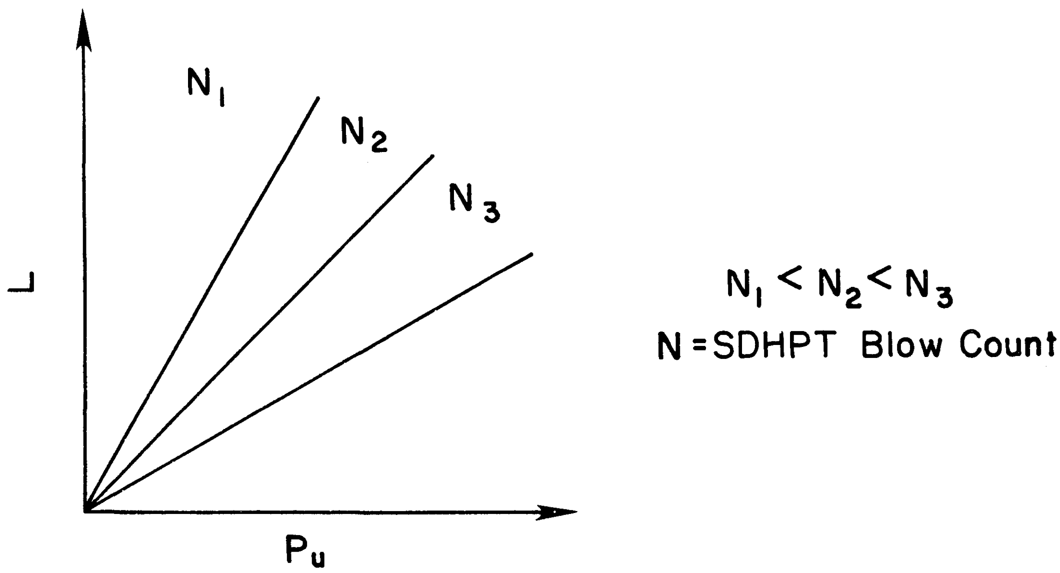
The relative simplicity of the equations involved in computing the axial capacities in compression allowed two computer programs to be developed as aids for design. BSHAFT and SHAFT1 allow computation of shaft capacities and relative shaft efficiencies based upon load per unit volume. SHAFT1 is used when the soil properties are well known, i.e., it utilizes the basic soil properties. BSHAFT makes use of input based upon the results of a dynamic penetration test.

SHAFT1 allows a sophisticated model of the soil-shaft system to be developed. The soil system can be accurately modelled to reflect a layered system if desired. In addition, the changing of all or some of the design parameters is easily accomplished. BSHAFT does not allow the same modelling capabilities as SHAFT1. However, the outputs of both programs are similar and can be useful. The results can be used to generate graphs of shaft length versus shaft capacity and can be further broken down into shaft capacity due to skin friction and shaft capacity due to tip resistance (see Figs 3.6a and 3.6b). Both programs have the capability to make computations for a series of shaft diameters. If several diameters are studied, a design for a "step-taper" type of foundation can be made. Further details for both programs can be found in the Quiros and Reese report (Ref 12).

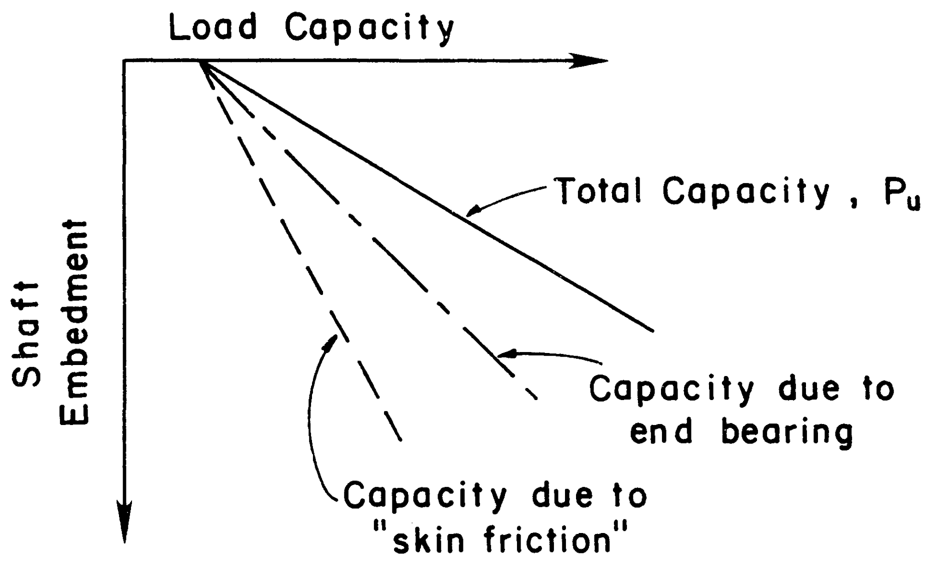
DESIGN OF CLOSELY SPACED SHAFTS

INTERACTION OF CLOSELY SPACED SHAFTS UNDER LATERAL LOADS

The design of any shaft that is subject to a combination of loadings must account for the effect of each load. This has been pointed out in previous paragraphs with reference to the interaction between lateral loads and deflections and computation of axial capacities. For shaft groups, i.e., two or more closely spaced shafts, the lateral deflection of any shaft in the group is increased relative to a single-shaft deflection due to the influence of adjacent shafts under load. The problem is not unique to any one application and occurs not only for double-shaft foundations for overhead signing but also for group piling for abutments, bents, retaining walls, and similar structures. Several solutions to the situation have been advanced. Poulos has presented a solution based upon elastic theory (Refs 9 and 10).



(a) Typical shaft capacity vs. embedment length



(b) Typical breakdown of total shaft capacity vs. shaft length

Fig 3.6. Shaft capacity versus embedment length.

Proposed Methods of Analysis

Since Poulos' solution is based upon the theories of elasticity, solutions made for high levels of stress in the soil will be inaccurate. The nonlinear response of soil to imposed stresses, like that of concrete, dictates that elastic theories be restricted to use with low stress levels where the assumption of linearly elastic behavior is best approximated. In addition, Poulos confines his solution to a constant value of soil modulus (E_s) and pile or shaft modulus (E). While this may be acceptable for the shaft, where material properties will most likely be constant or can be approximated as such, the soil may and probably will have an E_s widely varying with depth or even horizontal location. In addition, the solution is available only for fixed- and free-head piles or shafts and yields only the deflections of the shaft or pile head. With these limitations in mind, the Poulos solution can be utilized and, in fact, is partially utilized by Focht and Koch to account for the deflections of a pile or shaft within a group (Ref 3).

The method of solution proposed by Focht and Koch involves the combination of Poulos' elastic solution with the nonlinear, subgrade-reaction solutions proposed by Reese and Matlock (Ref 3). The former is an attempt to provide a solution that recognizes that the stress levels imposed by the shear load on individual shafts will cause plastic deformations and will be accompanied by a lesser deformation caused by lower stress levels due to the interaction of adjacent shafts. In a group, the total group load must be distributed, though not equally, among the shafts. Each shaft will, therefore, be acted upon by a horizontal load that will develop stresses within the soil mass. Immediately adjacent to the shaft these stresses may become quite large. However, as the load is distributed out into the soil mass, the induced stress level diminishes. Boussinesq developed equations (based upon the theory of elasticity) which attempted to quantify this phenomenon (Ref 7). Therefore, the stresses at a pile will be increased, even if no load is applied to that pile, if an adjacent pile is under load. Since these stresses will normally be small, a solution for deflections due to these small stresses and based upon the theory of elasticity will be satisfactory.

Poulos proposes that, for any pile in a group, the deflection will be given by

$$\rho_k = \bar{\rho}_F \left(\sum_{\substack{j=1 \\ j \neq k}}^{m} H_j \alpha_{\rho_{F_{kj}}} + H_k \right) \tag{3.12}$$

where

ρ_k = the deflection of the kth pile,

$\bar{\rho}_F$ = the deflection due to a unit load acting upon a single pile,

$\sum_{\substack{j=1 \\ j \neq k}}^{m} H_j \alpha_{\rho_{F_{kj}}}$ = the summation of the effective loads on the kth pile due to all the other piles in the group,

H_j = the load on the jth pile,

$\alpha_{\rho_{F_{kj}}}$ = an influence factor, based on the kth pile and its geometry with respect to other piles in the group, and

H_k = the load acting upon the kth pile.

Equation 3.12 simply states that the total deflection of a pile is equal to the deflections caused by adjacent piles influencing that pile plus the deflection due to the pile's own load. Focht and Koch accepted that part of Eq 3.12 which describes the influence of adjacent piles upon the pile in question, but they modified the term involving the deflection of the pile under its own load in an attempt to account for the inelastic effects that are likely to occur. The equation proposed is

$$\rho_k = \rho_F \left(\sum_{\substack{j=1 \\ j \neq k}}^{m} H_j \alpha_{\rho_{F_{kj}}} + RH_k \right) \tag{3.13}$$

This equation is basically the same as 3.12 except for the term R .

Focht and Koch proposed that

$$R = \frac{y_s}{\rho} \tag{3.14}$$

where

R = a relative stiffness factor or ratio,

y_s = the deflection of an isolated pile calculated by p-y methods (i.e., the piles, "plastic" deflection), and

ρ = the deflection of an isolated pile calculated by the Poulos method (i.e., the piles "elastic" deflection).

Application of Analytical Method to Design

The analysis and design of a multi-shaft or multi-pile group is based upon Eq 3.13. Analyses of two-shaft groups will be the simplest to perform although larger groupings will only be more tedious, not more complex. In general, the problem may be approached in the following manner:

(1) Determine the initial parameters for the pile or shaft and for the soil on the site, including loading on the group.

(2) Develop a set of p-y curves for a single pile or shaft.

(3) Compute y_s , the groundline deflection, using COM623 and an average load on the pile

$$H_{avg} = \frac{H_T}{m} \tag{3.15}$$

where

H_T = total load on the pile or shaft group and

m = number of piles in the group.

(4) Select a value of Young's modulus for the soil, E_s , that represents a low stress level in the soil. An initial modulus from the laboratory stress-strain curves may be employed.

(5) Using E_s from (4) and H_{avg} from (3), compute the groundline deflection, ρ_F, for a single pile using Poulos' method for a single pile.

(6) With the results of (5) compute the deflection, $\overline{\rho}_F$, due to a unit load as

$$\overline{\rho}_F = \frac{\rho_F}{H_{avg}} . \tag{3.15}$$

(7) Compute R by dividing the value from (3), y_s, by the value calculated in (5), $\overline{\rho}_F$.

(8) Using the Focht-Koch equation (Eq 3.13), write an equation for each pile in the group. Influence coefficients, $\alpha_{\rho_{F_{kj}}}$, can be obtained using graphs from Poulos.

(9) Write an equation of equilibrium for the shear load on the group, i.e.,

$$H_T = H_1 + H_2 + \ldots + H_m \tag{3.16}$$

where

H_T = total load on group and

H_m = load on mth pile.

(10) Solve the equations generated in steps (8) and (9), knowing that the deflection of each pile in the group must be equal. The group deflection and values of load for each pile will be obtained.

(11) Use a set of y-multipliers (2, 3, 4 and so on) to modify the curves generated in step (2). Then compute groundline deflections using the load from step (10) and the modified p-y curves. This generates a relationship between groundline deflection and the y-multipliers.

(12) Using the relationship established in (11), find the value of the multiplier that will give the same deflection as that computed in step (10).

(13) Modify p-y curves by the factor obtained in step (12) and use the new set of curves to compute the bending moment produced in the pile or shaft that supports the largest load.

(14) Check the adequacy of the pile or shaft design, using the results of (13).

Example Problem

Figure 3.7 depicts a typical double shaft foundation in which it is assumed that the pile cap shown is equivalent in action to the vertical trusses of the overhead signs. Step (1) gives the following data (see Fig 3.7 for definition of symbols):

For the shaft system

$$H = 18{,}300 \text{ lb.}$$

$$P_x = 72{,}000 \text{ lb}$$

$$d = 2.50 \text{ feet}$$

$$I_{gr} = 39{,}761 \text{ in.}^4$$

$$A = 4.91 \text{ ft}^2$$

$$L = 25.0 \text{ feet}$$

$$S = 7.50 \text{ feet}$$

$$E_c = 3.15 \times 10^6 \text{ lb/in.}^2$$

For the soil system

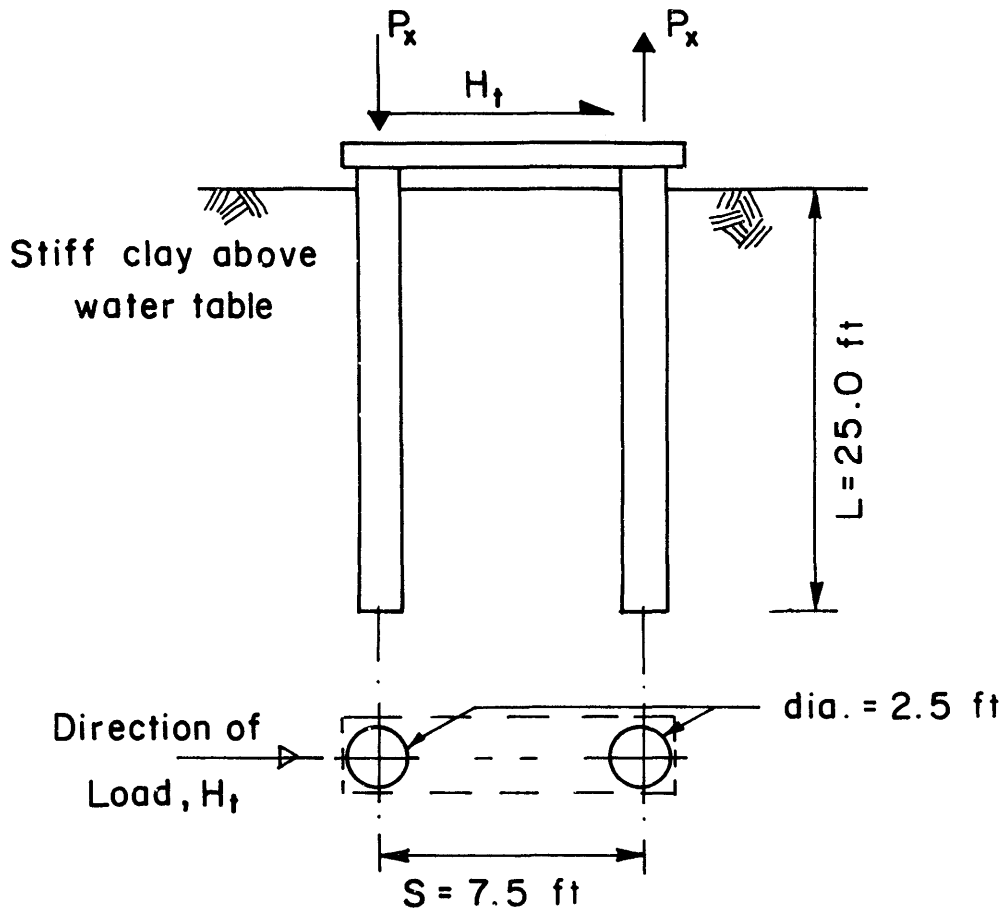
$$c_Q = 1728 \text{ lb/ft}^2$$

$$\bar{\gamma} = 110 \text{ lb/ft}^3$$

$$k = 5 \times 10^5 \text{ lb/ft}^3$$

$$\epsilon_{50} = 0.010$$

The soil is characterized as a stiff clay above the water table. Computer program COM623 was used to obtain p-y curves as well as groundline deflection, y_s (Steps 2 and 3). The value of H_{avg} used in the computation was 9,150 lb and y_s was computed to be 0.00987 in.



1 ft = 0.3048 m

Fig 3.7. Double-shaft group example problems.

Step (4) requires the computation of E_s. For this problem it is assumed that

$$E_s = 250c_Q = 3000 \text{ lb/in.}^2$$

The elastic groundline deflection of a single pile or shaft may now be computed by methods given by Poulos (Step 5).

$$\rho_F = I_{\rho_F}\left[\frac{H}{E_s L}\right]$$

where I_{ρ_F} is from Fig 3.8.

$$L/d = \frac{25}{2.5} = 10$$

$$K_R = \frac{EI}{E_s L^4} = \frac{(3.15 \times 10^6 \text{ lb/in.}^2)(39{,}761 \text{ in.}^4)}{(3000 \text{ lb/in.}^2)(300 \text{ in.})^4}$$

$$= 0.00515$$

$$\text{Use } K_R = 5 \times 10^{-3}$$

From Fig 3.8 generated by Poulos

$$I_{\rho_F} = 3.5$$

$$\rho_F = 3.5\left[\frac{9150 \text{ lb}}{(3000 \text{ lb/in.}^2)(300 \text{ in.})}\right]$$

$$= 0.036 \text{ in.}$$

The unit deflection is then computed (Step 6):

$$\bar{\rho}_F = \frac{\rho_F}{H_{avg}}$$

$$= \frac{0.036 \text{ inch}}{9150 \text{ lb}}$$

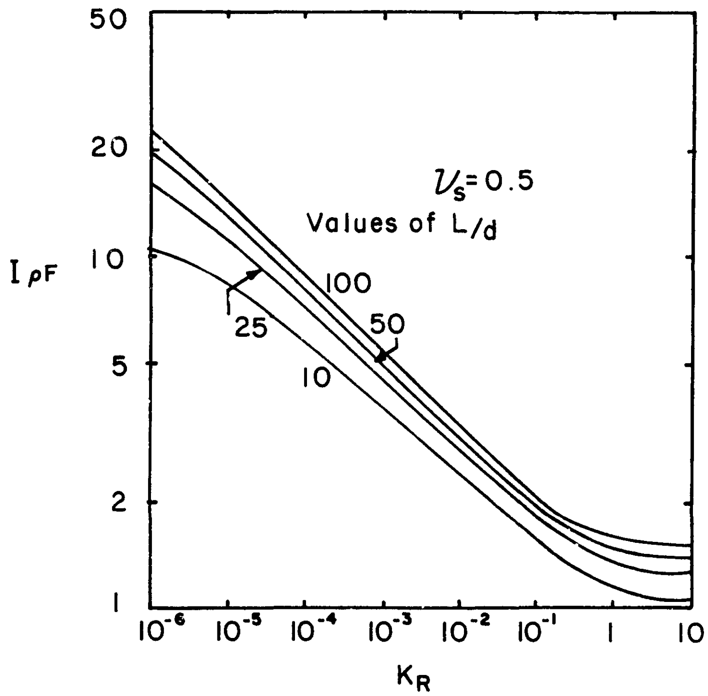


Fig 3.8. Influence factor I_{ρ_F} for fixed-head piles (from Poulos, 1971).

$$= 3.93 \times 10^{-6} \text{ lb/in.}$$

R is now computed (Step 7):

$$R = \frac{y_s}{\rho_F} = \frac{0.00987}{0.036 \text{ inch}}$$

$$= 0.274$$

Use $R = 1.0$. The use of $R = 1.0$ will be discussed subsequently. Step 8 gives

$$L/d = 10 \text{ , } s/d = 3 \text{ , } \beta = 0^{\circ}$$

Use $K_R = 0.1$ from Fig 3.9, which gives

$$\alpha_{\rho_F} \cong 0.51 = \alpha_{12} = \alpha_{21}$$

Substituting into 3.16, two equations involving shaft head displacements are obtained:

$$\rho_1 = \bar{\rho}_F (H_2 \alpha + RH_1) \qquad (1)$$

$$\rho_2 = \bar{\rho}_F (H_1 \alpha + RH_2) \qquad (2)$$

Step 9 gives an equilibrium equation:

$$H_T = H_1 + H_2 \qquad (3)$$

Since $\rho_1 = \rho_2 = \rho_G$ (total deflection of group), Step 10 is the solution of the equations (1) - (3) , yielding

$$H_1 = H_2 = \frac{18{,}300 \text{ lb}}{2} = 9150 \text{ lb}$$

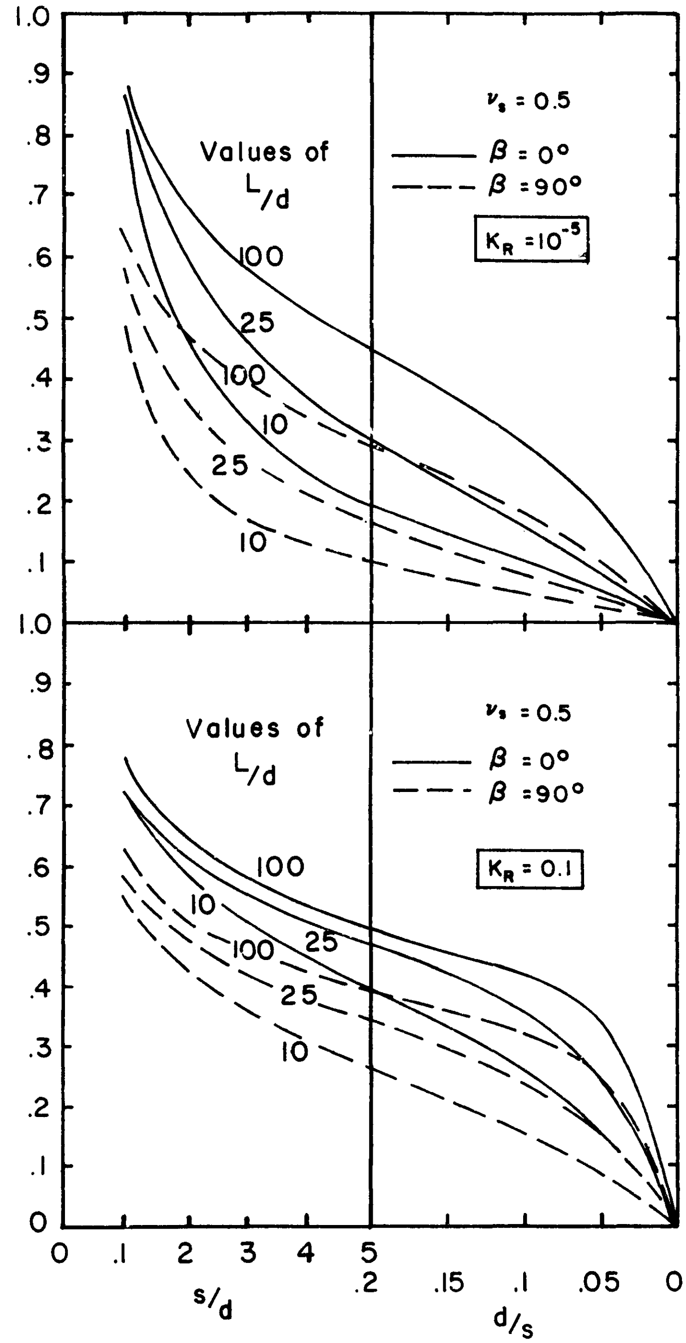


Fig 3.9. Interaction factors $\alpha_{\rho f}$ for fixed-head piles (from Poulos, 1971).

and

$$y_G = 3.93 \times 10^{-6} \left[0.51(9150) + 9150\right]$$
$$= 0.054 \text{ in.}$$

Using modified p-y curves, COM623 is employed to generate a curve showing groundline deflection versus the y-multiplier (Step 11, Fig 3.10). A multiplier is then selected which gives the same deflection as the deflection found in Step 10 (Step 12).

Using this modifier, a solution is made with COM623. The results of this solution (Step 13) are given in Fig 3.11. The final step is to check the structural adequacy of the shaft.

The solution that was just presented indicated that the deflection computed by the p-y method was less than that computed by the Poulos elastic method. Such a result is an anomoly because the p-y approach should represent the "true" behavior of the pile. The conclusion would then be that the elastic modulus that was selected for the soil for use in the Poulos method was too low. However, for this particular solution the Poulos solution was assumed to be correct for purposes of completing the solution.

As shown in Fig 3.11, the deflection is certainly tolerable. The maximum bending moment of 63 k-ft results in a bending stress of 285 lb/in.^2. Therefore, the design that is presented is conservative from the standpoint of bending moment.

The Focht-Koch-Poulos procedure is rational and is being employed widely at the present time. However, it is unproven by having been compared with a sufficient number of results from prototype tests in the field to allow a judgement to be made about the validity of the method. The few comparisons that have been made (unpublished) show that the Focht-Koch-Poulos method gives reasonable agreement with experimental results.

The next step in checking the design shown in Fig 3.7 is to check the adequacy of the design under the axial loads that are shown.

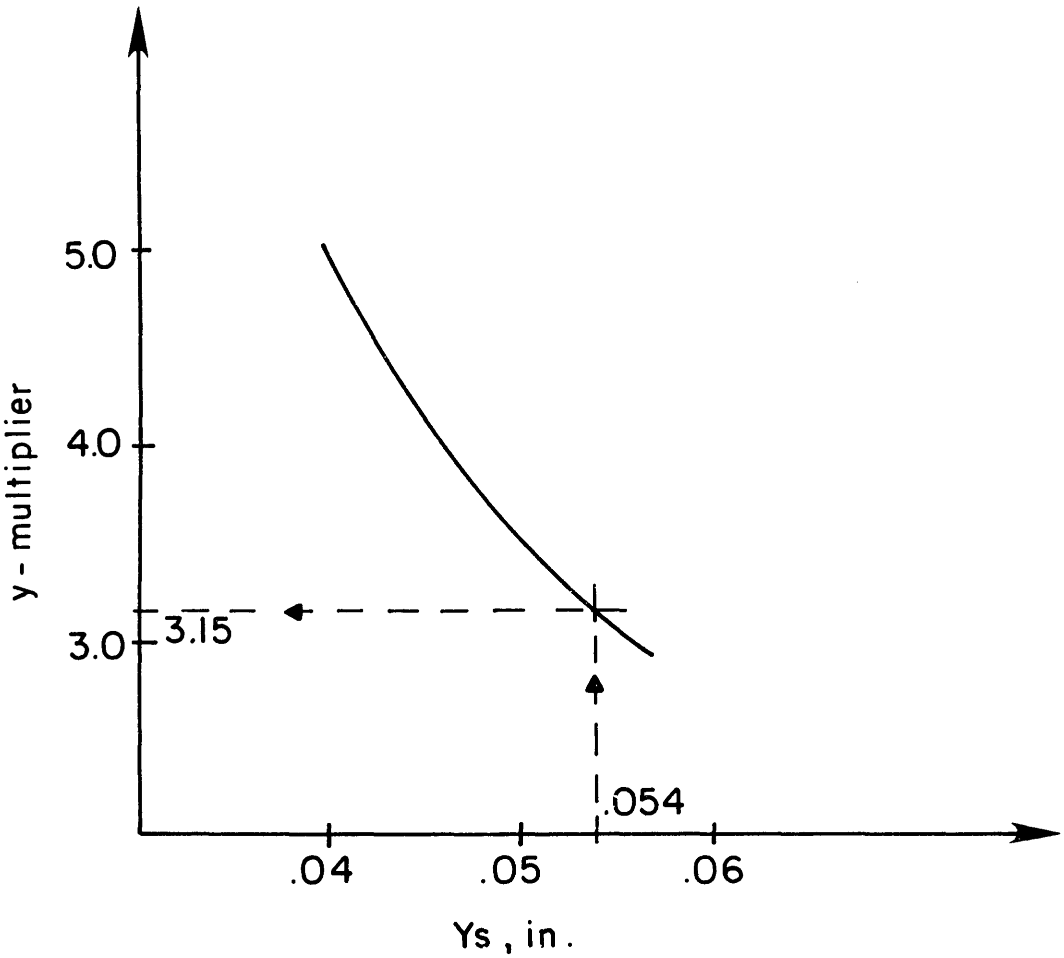
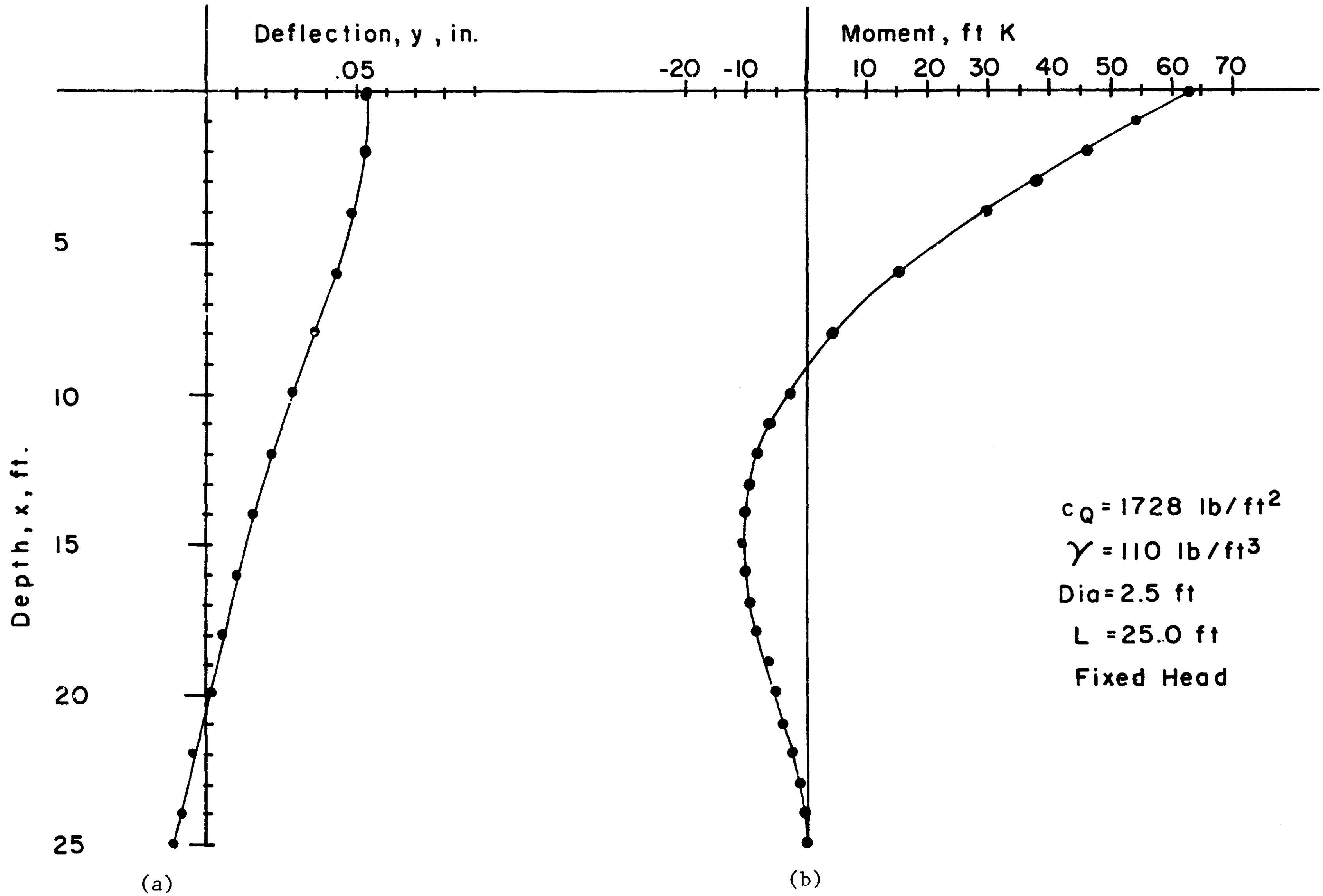


Fig 3.10. Selection of p-y curve modifier.



1 ft = 0.3048 m; 1000 lb/ft^2 = 47.88 MPa; 1 ft-k = 1.356 kNm

Fig 3.11. Equivalent shaft deflections and moments for example problem for double-shaft group.

INTERACTION OF CLOSELY SPACED SHAFTS UNDER AXIAL LOAD

In the preceding pages, methods of analysis and design have been outlined for axial loading of single shafts and for the lateral loading of shaft groups. The procedure fcr lateral loading accounted for the interaction between shafts in the group. The following discussion will treat the problem of such interaction due to axial loading.

Poulos presents methods of analysis for a two-pile grouping (Refs 8 and 11). His methods, based upon theories of elasticity, treat the problems of axial capacity and settlement. His methods must be recognized as approximate, but give results that improve the ability of the designer to make reasonable decisions.

Axial Deflection

For a two-shaft grouping, as has been mentioned previously, one shaft will be subjected to tensile loads while the other will be subjected to compressive loads. The movements involved under these two types of loading will be opposite in direction. Poulos gives the following expression for the shaft-head displacement of a shaft under axial loading:

$$\rho_a = \frac{P_x I_1}{E_s d} \tag{3.19}$$

where

P_x = applied axial load,

I_1 = an influence factor,

E_s = Young's modulus of the soil, and

d = shaft diameter.

I_1 is a factor that is a function of pile or shaft diameter, base diameter, and length. Figure 3.12 (from Poulos) gives values of I_1. As shown in Fig 3.13, Poulos also gives values of an α factor for the computation of shaft interaction effects on axial movement where

$$\alpha_1 = \frac{\text{additional settlement due to adjacent shaft}}{\text{settlement of shaft under its own load}} \quad . \tag{3.17}$$

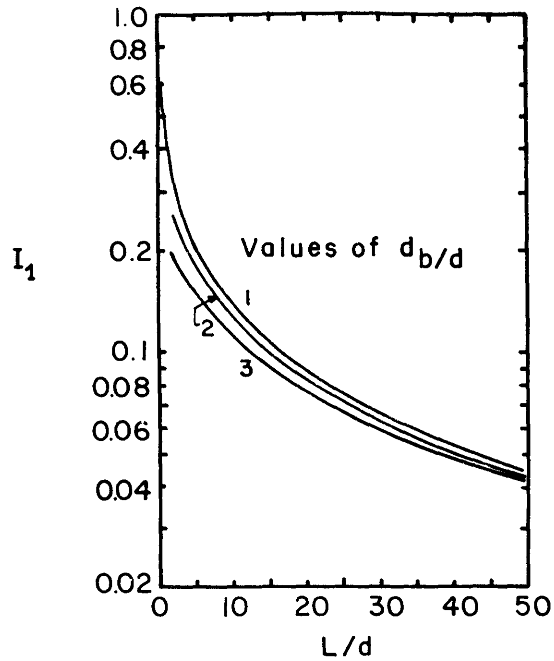


Fig 3.12. Influence factor I_1 for axial displacement (after Poulos, 1971).

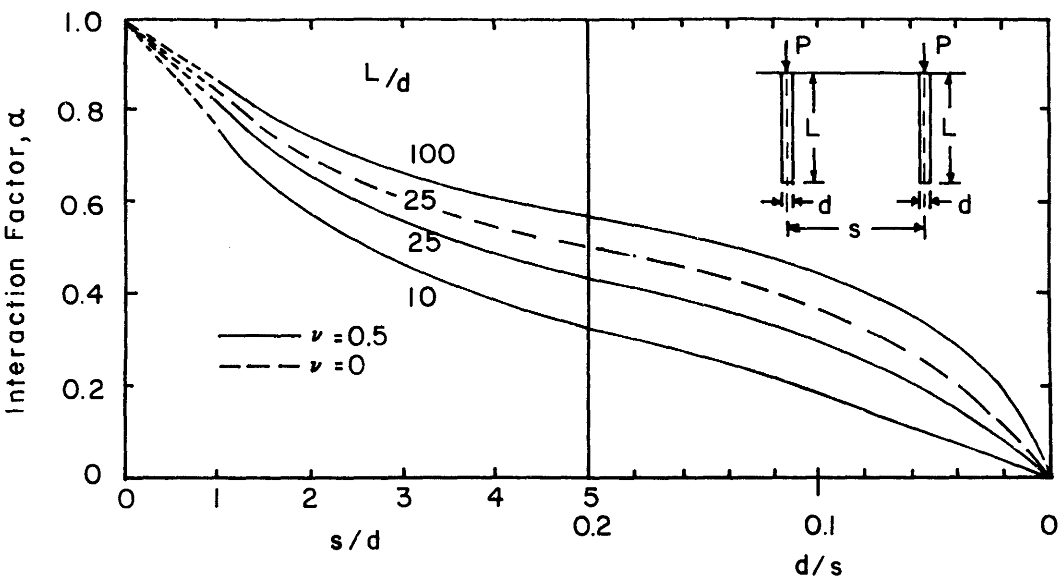


Fig 3.13. Interaction factor α_1 for axial displacement (after Poulos, 1971).

The curves of Fig 3.12 for I_1 and Fig 3.13 for α_1 allow an estimate to be made of the interaction between drilled shafts supporting an overhead sign. Figure 3.7 shows the typical loading for the two-shaft group. One load tends to cause an upward movement as opposed to the downward movement of the other shaft. This means that the "additional settlement" quantity should actually be negative in value since the effect of the adjacent pile or shaft is to cause movement opposite to the affected shaft movement.

As an illustrative example of the Poulos method, consider the two-shaft system of Fig 3.7, with

$$L/d = 10 \quad , \quad \frac{d_b}{d} = 1.0 \quad \text{where} \quad d_b = \text{diameter of shaft base.}$$

From Fig 3.12

$$I_1 = 0.15$$

$$\therefore \bar{\rho}_x = \frac{(72{,}000 \text{ lb})(0.15)}{(3000 \text{ lb/in.}^2)(30 \text{ in.})}$$

$$= 0.12 \text{ inch}$$

From Fig 3.13, with $L/d = 10$ and $s/d = 3$, $\alpha_1 = 0.45$. However, as pointed out, this value should be negative in value.

With this value of α_1 , the movement of either shaft may be computed and is

$$\rho_T = \rho_x + \alpha_1 \rho_x$$

$$= 0.12 + (-0.45)(0.12)$$

$$= 0.066 \text{ inch}$$

This result indicates that the axial movement of a shaft of the size used for overhead signs is relatively small and in most cases may be ignored. The second point of interest is that the total movement of either shaft in the double-shaft system will be less than that of the single shaft.

Shaft Capacities

Interaction between two shafts will affect the axial capacities of the shafts as well as the axial deflections. A qualitative examination of a two-shaft group will be presented. The problem should not be viewed as an exact solution but will indicate the trend of the behavior to be expected.

Figure 3.14 presents a simplified view of a two-shaft system. A free body of shaft A is taken to include the soil mass enclosed within a cylindrical shape of radius S and length L . The average shear stress on the outer face of this soil cylinder may be approximated by

$$\tau_{BA} = \frac{Q_A}{2\pi SL} \tag{3.19}$$

If this value is used as the average shear stress between the surface of shaft B and the soil mass, then the upward load due to the stresses induced by the load at A is

$$Q_{BA} = \tau_{BA} \pi dL$$

$$Q_{BA} = \frac{Q_A}{2\pi SL} (\pi dL)$$

$$= \frac{Q_A d}{2S} \tag{3.20}$$

For the previously presented problem then, the effect of A on B could be approximated as

$$Q_{BA} = \frac{(72{,}000 \text{ lb})(2.5 \text{ ft})}{2(7.5 \text{ ft})}$$

$$= 12{,}000 \text{ lb} \quad .$$

Thus, the compressive load can be considered to be reduced by 12,000 lb as is the tensile load.

This figure should not be viewed as exact. For example, the load due to a compressive load at one shaft calculated using Eq 3.20 ignores the fact that only a portion of the load will be felt as an induced shear stress since part of the load will be carried by the shaft base. However, the ideas expressed

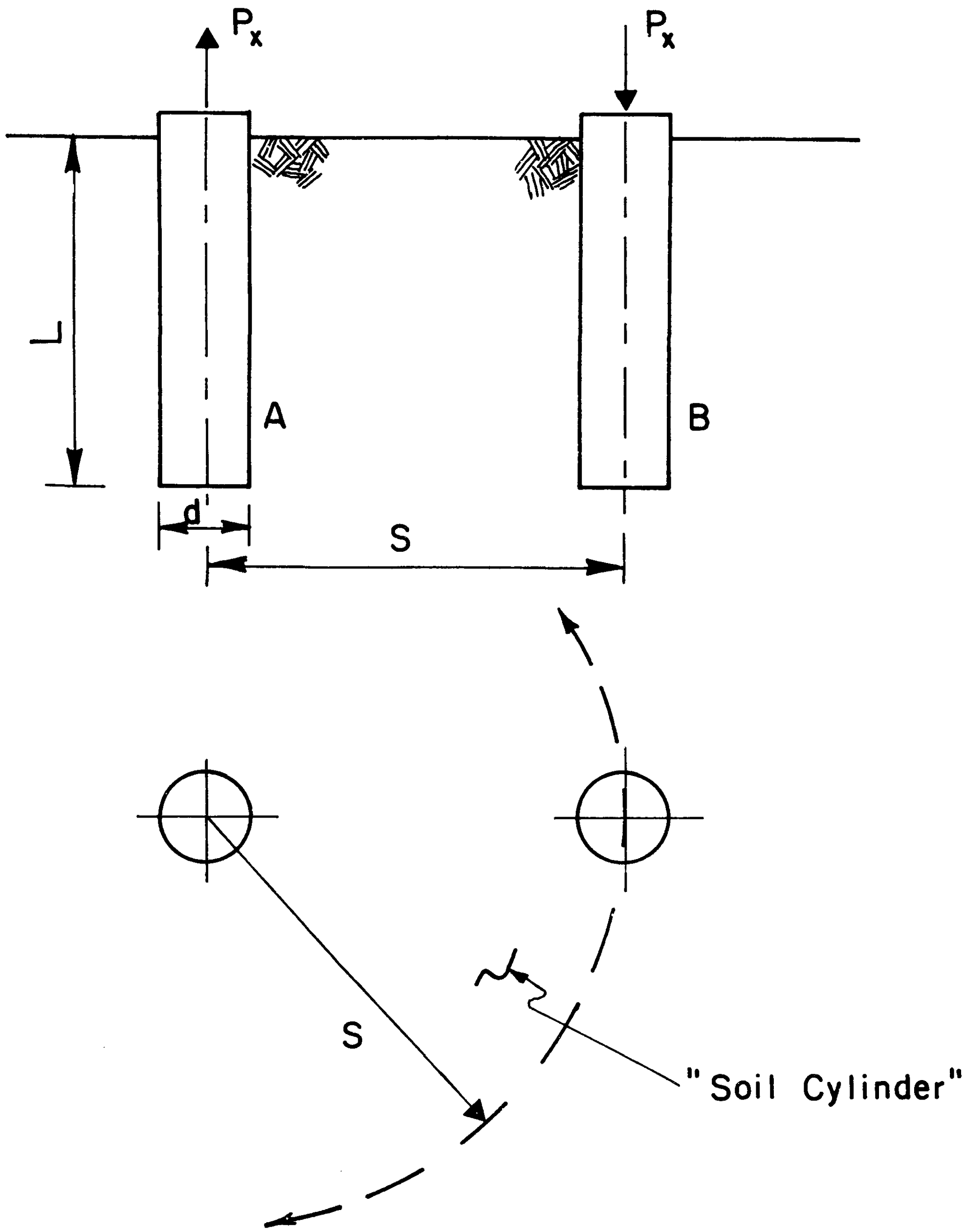


Fig 3.14. Free body of two-shaft system.

are intuitively sound and may be thought of as increasing the overall factor of safety of the system.

The methods presented in the preceding pages for the analysis and design of two-shaft systems are felt to be both easy to use and as accurate as is presently possible. As in all areas of design, care should be used to understand the theories, the assumptions, and, in particular, the limitations involved in the use of the presented procedures. A major portion of design must be the exercise of experience and good judgement.

CHAPTER 4. DESIGN PROCEDURE USING COMPUTER-BASED METHODS OF ANALYSIS

PRESENT CAPABILITIES

With the advent of high-speed digital computers many engineering problems which were formerly too complex mathematically or simply too tedious to solve have become manageable. The problem of the laterally loaded pile or shaft was one such problem. Computer programs, making use of the finite difference approach, currently make the solution of this problem relatively straightforward. The programs COM622 and COM623 have been specifically developed to generate the necessary solutions.

Input and output examples for COM622 have been reported in the literature (Ref 15). In addition, a detailed report on the capabilities and input/output formats for COM623 has been supplied to SDHPT (Ref 17). Some important aspects of the latter program will be summarized at this point.

COMPUTER-BASED ANALYSIS

COM623 allows great flexibility in problem formulation on the part of the design engineer. The input format makes variation possible in both the soil and structural parameters. For instance, the moment of inertia can vary along the length of the shaft, independently of all other variables. Likewise, soil parameters, such as soil unit weight, undrained shear strength of clay, and internal angle of friction, can be varied, thus allowing the designer to make the model of the system as simple or as complex as desired or as the available data allow.

In addition, the program is structured in such a manner as to make multiple solutions both easy and practical. If a load-deflection curve is desired, the simple addition of extra load-value cards at the end of the data deck will cause the necessary output to be generated. Output can also be manipulated to some degree to avoid generation of excess output information.

Instead of station by station values of moment and outer fiber stresses, any number of stations can be omitted or the entire table can be deleted so that only the maximum values are output in a summary table. As has been stated, a most complete and informative guide has already been developed and is available if further questions arise.

APPLICATION OF ANALYTICAL METHODS TO DESIGN

Within the context of design, the several methods of analysis previously mentioned can be manipulated to achieve a design, although some are much more easily manipulated than others. If only the crudest of approximations to the best solution is desired, a closed-form solution of the differential equation would suffice. However, this would of necessity be overly restrictive, complex, and time consuming.

In direct contrast to a closed-form solution would be a design procedure based on a computer analysis of the system. A design procedure has been formulated which attempts to capitalize on the computer's availability and speed. This procedure will be outlined, using as an example problem the shaft and basic parameters used earlier in the section on charts developed at SDHPT for design of single shafts. A basic summary of the steps will be made and then each step will be discussed using the example problem for illustration.

SUGGESTED DESIGN PROCEDURE

Single-Shaft System - COM623 Design Procedure

(1) Determine soil properties; select shaft properties and cross section geometries.

(2) Compute shaft design moment: $M_u = \phi M_n$

(3) Estimate EI as $E_c I_{gr}$

(4) Estimate shaft length, L , such that "long" pile action occurs (i.e., two points of zero deflection occurring along the shaft)

(5) Using values from Steps 1, 3, and 4, and COM623, generate, for a range of lateral loads, P_t , curves of

   (a) lateral load versus maximum shaft moment, M_{max} , and

   (b) lateral load versus groundline deflection, y_t .

(6) With a value of M_u from Step 2, enter 5a and find the allowable load, P_{t_a}

(7) With P_{t_a} enter 5b to

(a) insure that y_t is not in the "critical" area of the curve, and

(b) check y_t for esthetic criteria, if any.

(8) With P_{t_a} and COM623, generate a y_t versus L curve by decreasing L until y_t begins to increase significantly. Choose a design length, L .

(9) Using PMEI (Appendix) or a similar program, generate a moment versus EI curve. Refine the estimate of EI , if deemed necessary, and rerun COM623 with new EI values.

(10) Check results of Step 9 for unacceptable changes in y_t or M_{max} .

EXAMPLE PROBLEM

The first stage of the procedure involves the collection and evaluation of all pertinent design data. Design solutions, in the end, are limited by design input and, therefore, every effort should be made to identify and select appropriate values. For purposes of illustration, certain parameters used here have been simplified.

The soil deposit is input as a homogeneous, single-layer, clay deposit. Shear strength, unit weight, and ϵ_{50} are assumed constant with depth and equal to 1728 lb/ft^2, 115 lb/ft^3 and 0.010 respectively (Fig 4.1). The water table is below the shaft base and the initial modulus of subgrade reaction, k_s , is equal to 5.0×10^5 lb/ft^3. Loading is chosen to be cyclic in nature. It should be mentioned that it is usual for most of these parameters to be unavailable to the design engineer. However, values that will provide a sufficient order of accuracy can be obtained from many different sources in

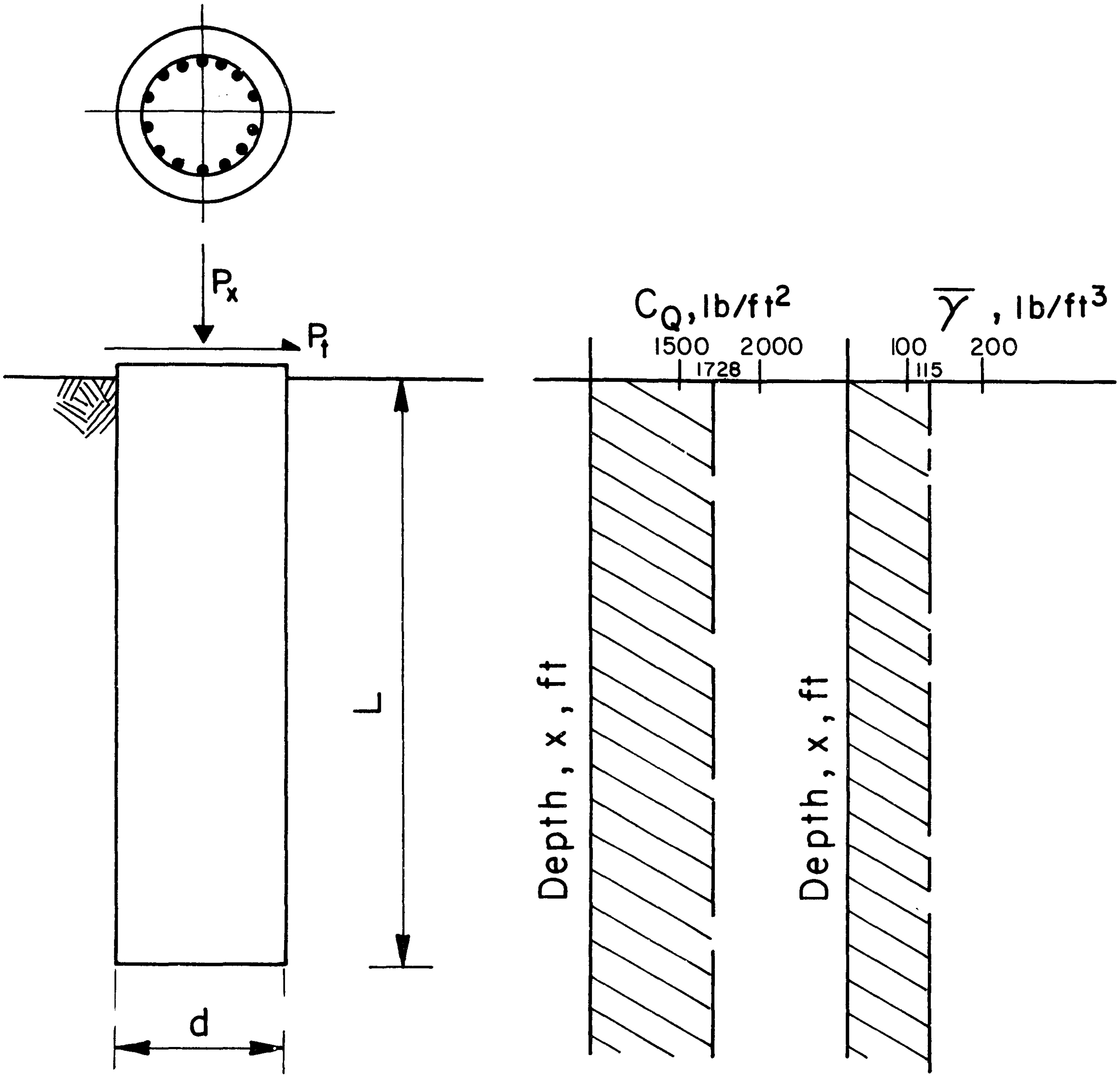


Fig 4.1. Parameters for design of single shaft.

the literature when necessary. The engineer's judgment and experience, as in all other design processes, may also be called upon.

If there are no limits set upon shaft geometries, a trial section must be formulated. In many instances, certain limits, as to diameters and reinforcement arrangements, are in effect and must be observed. At SDHPT, certain sizes are more frequently used than others and a natural preference seems to have evolved for a 30-inch-diameter shaft. For this problem, a 30-inch-diameter shaft has been chosen with a circular pattern of 14 # 11 bars (ρ = 3.1 percent) enclosed with a #3 spiral. Concrete is designated as Class A with a 28-day strength of 3000 lb/in.^{2} and the reinforcing steel is Grade 60 (ultimate strength is 60,000 lb/in.^{2}).

With the basic soil and shaft parameters established, Step 2 is performed. Several programs exist, or are in the process of being written, for computing the design moment of a circular section. Simpler versions for use with hand-held programmable calculators are also being developed, although such an approach of necessity requires some manual bookkeeping. ACI design handbooks can also be used to obtain the desired value.

Step 3 involves the estimation and computation of the shaft's stiffness. The modulus of elasticity can be computed using the ACI 318-77 formula for normal weight concrete:

$$E_c = 57{,}000 \ f_c'^{\ 1/2} \tag{4.1}$$

The moment of inertia of the cross section can be computed in one of several ways. The small reinforcing can be transformed to an equivalent concrete area and consideration can be given to the effects of cracking. However, as has been mentioned, the product EI has a relatively small effect on the behavior of the system. For this reason, the moment of inertia used in this problem is the gross moment of inertia. It is felt that most design problems will be of such a nature that this gross value will be adequate and no further refinement is needed.

The next part of the procedure involves the estimation of an initial shaft length that will insure that "long pile" action occurs. This could be any value desired and is used only as a guide to the rest of the design process. For this example a length of 40 feet was selected. This length can be checked by making an initial solution with a load 20 to 30 percent greater than the expected design load. If the results of groundline deflection show at least 2 points of zero deflection along the shaft length, the estimated L is sufficient; if there is only one point of zero deflection, the length should be increased. Familiarity with shaft design will enhance the designer's ability to choose these initial lengths with a minimum amount of guesswork. Once a length is selected, the next stage of the procedure can be performed.

With the initial shaft length, the computer is used to simulate behavior for a range of loads. The range to use will depend upon the designer. For the purpose of illustration, loads of from $0.7\ P_t$ to $2.0\ P_t$ have been used. Two curves are generated, one being a load (P_t) versus shaft moment (M) curve, the other a load (P_t) versus groundline deflection (y_t) (Figs 4.2 and 4.3). The value of design moment, M_u, is now entered into the load versus moment curve. From this curve, the maximum design load P_t is selected. This load represents the maximum allowable load that can be put on a shaft of length L without exceeding the shaft moment capacity or a predetermined deflection limit. If the load selected in this manner is less than the expected design load, a larger shaft diameter is indicated.

If this load is larger than expected, a larger factor of safety will be accepted or a smaller shaft diameter may be investigated. The shaft length may now be selected by applying the allowable load P_{t_a} to shafts of decreasing length. A plot of groundline deflection versus shaft length is then generated (see Fig 4.4). As long as two or more points of zero deflection occur along the shaft length, the groundline deflection will remain essentially constant. However, as soon as the deflected shape is characterized by only one point of zero deflection, increases in deflection will occur. This condition then represents a limit on shaft length for the given system. Shaft length may be chosen as that length which occurs at the point of tangency of this curve to the horizontal, or in some instances the designer may choose a lesser length. The curve of groundline deflection (y_t) versus load (P_t) should be examined to insure that there is a sufficient reserve in penetration, taking into account all of the factors for the particular design.

For the problem at hand, Fig 4.2 is entered with a moment capacity of 8.06×10^6 in.-lb, indicating a capacity of 19,760 lb. Since this load is

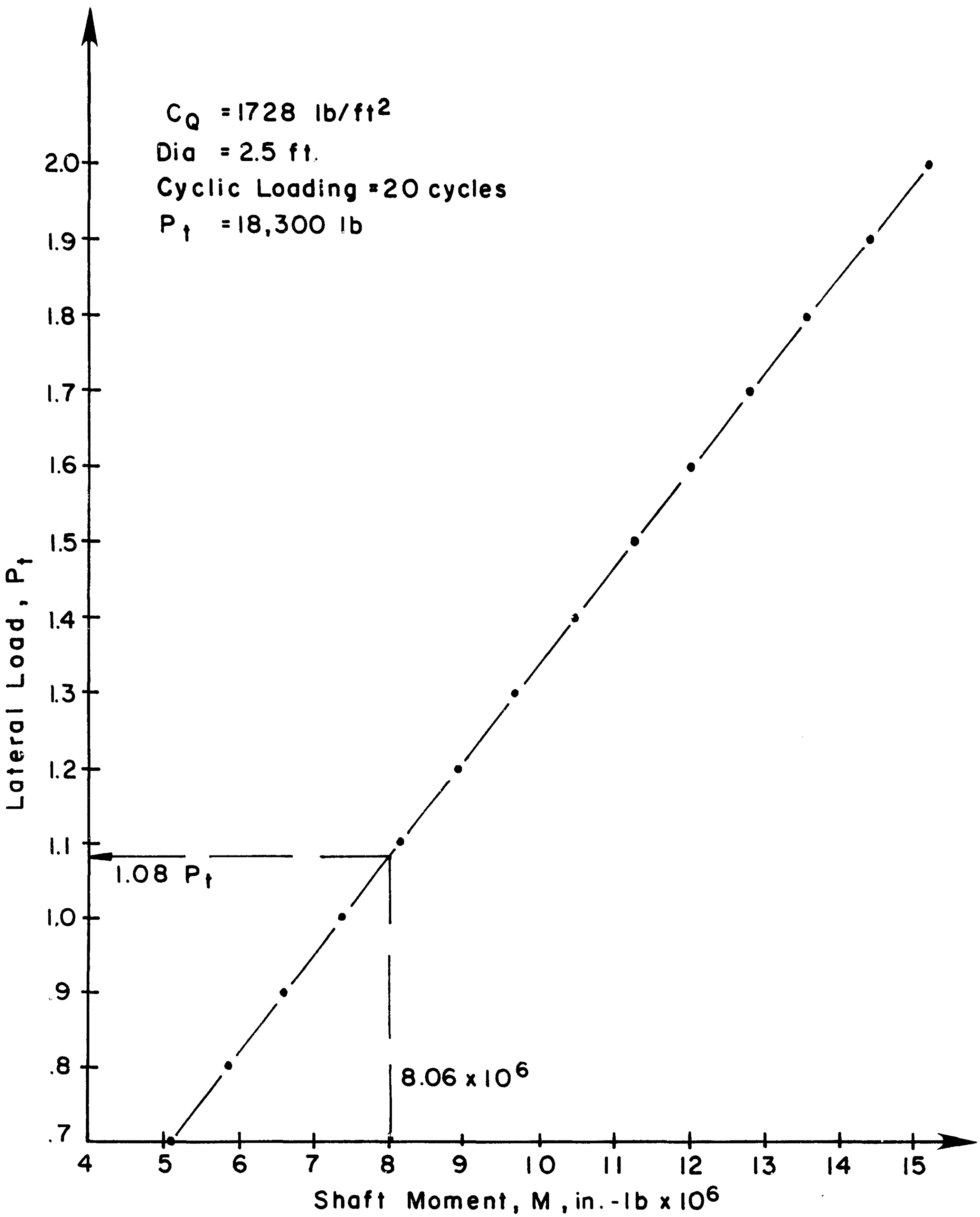


Fig 4.2. Shaft moment versus lateral load.

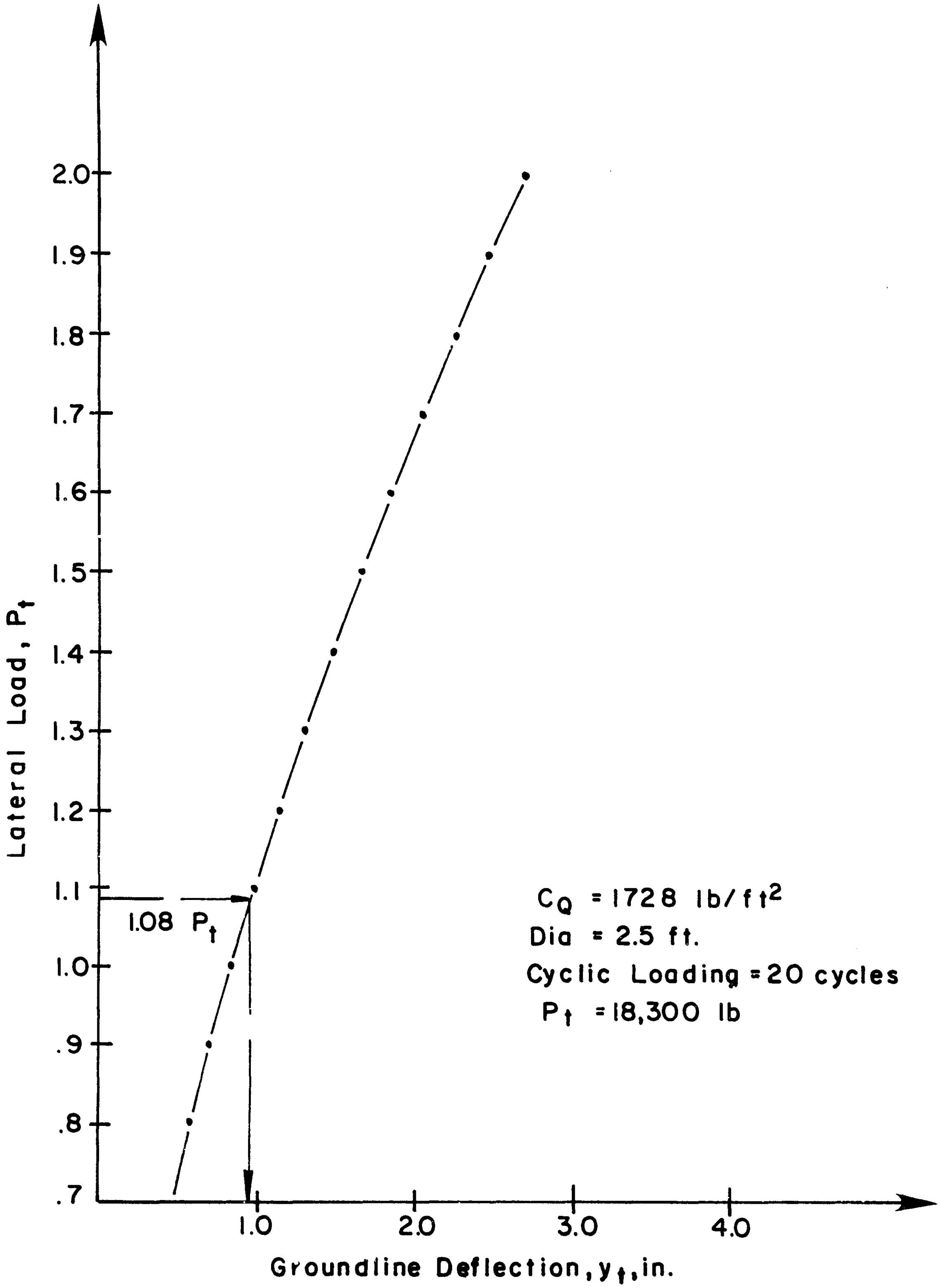


Fig 4.3. Groundline deflection versus lateral load.

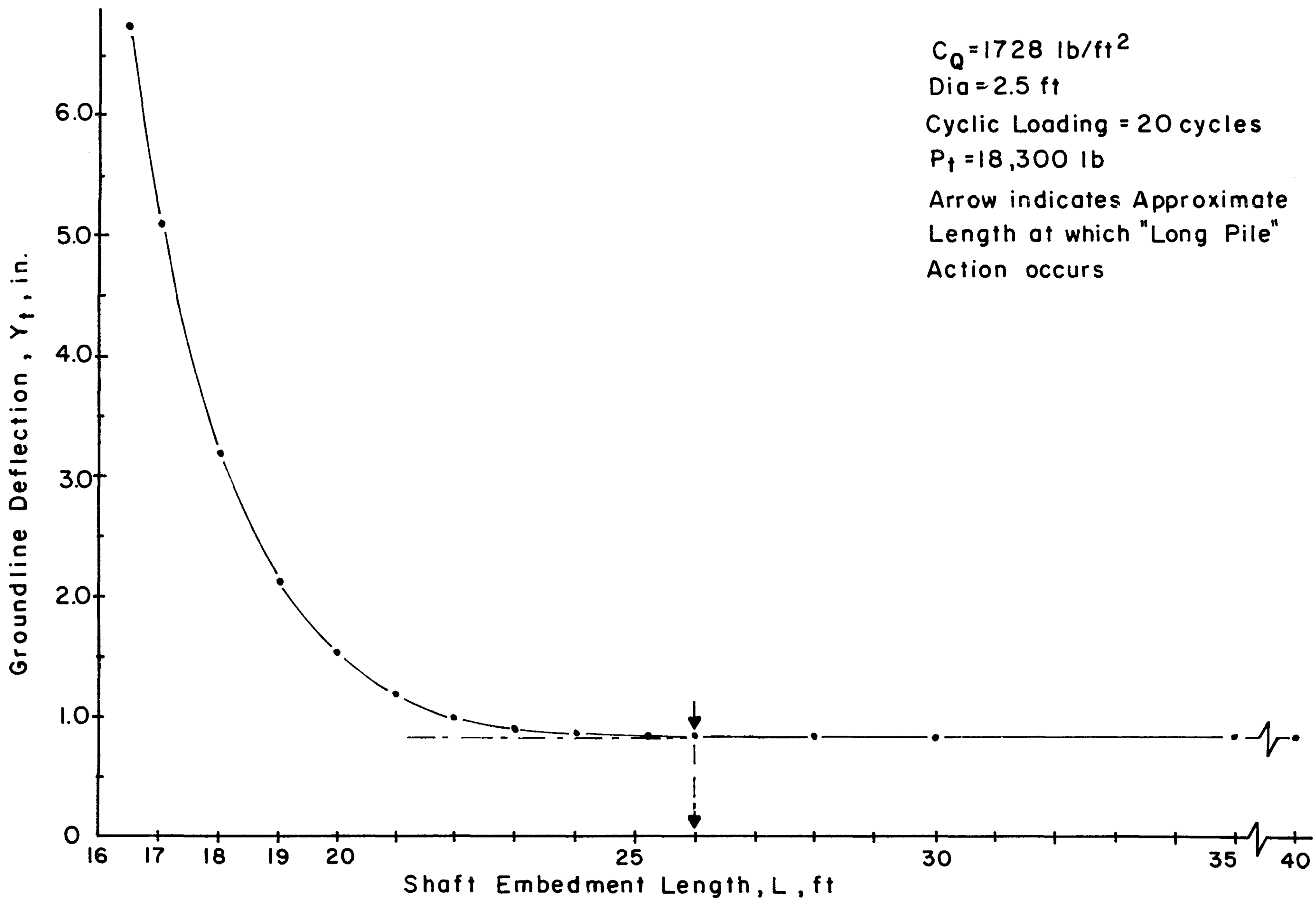


Fig 4.4. Groundline deflection versus embedment length of shaft.

larger than the design load, Fig 4.3 can be utilized to check the groundline deflection. A deflection of approximately 0.95 inch is indicated. This is determined to be acceptable and Fig 4.4 is now used to establish the required shaft length at 26 feet. If desired, another pass could be made using a smaller diameter shaft. However, this would require an increase in reinforcing since the shaft capacity in bending is almost fully utilized already.

At this point an additional factor of the program output may be utilized. Because the bending moment decreases with depth, a plot of shaft moment versus depth will indicate the areas where shaft reinforcing may be reduced. Moment versus depth for the 26-foot shaft is presented in Fig 4.5. Some economy may be achieved through the tailoring of the reinforcement cage.

COMPARISON OF COMPUTER-BASED DESIGN AND CHART DESIGN

In this section, the design problem was accomplished by the use of a computer program. In Chapter 2, the same problem was investigated using design charts that were generated through the use of a computer. The advantages and disadvantages of each approach bear discussion.

The quantifiable results of a particular method of design are easily checked and two different systems can be used equally well if both methods result in solutions that are approximately equal and correct. If this is the case, then the method chosen will frequently be selected on the basis of ease of use, familiarity with design concepts, time and money considerations, and desired accuracy.

The use of design tables and charts offers the engineer a quick and relatively straightforward design procedure. Charts often reflect the fact that a standardized system has either been established or encouraged. Within such a system a design procedure can be established which will relieve the designer of certain decisions with regard to the quality of input data. The rapid solution that will result could be at the expense of a more economical design. However, the refinement of the design process could result in little economy. For example, with shaft diameters restricted to four basic diameters (24, 30, 36, and 42 inches), the process of continual design refinement will be limited to choosing the smallest adequate diameter rather than obtaining the optimum diameter. The variation of diameters of drilled shafts in increments of

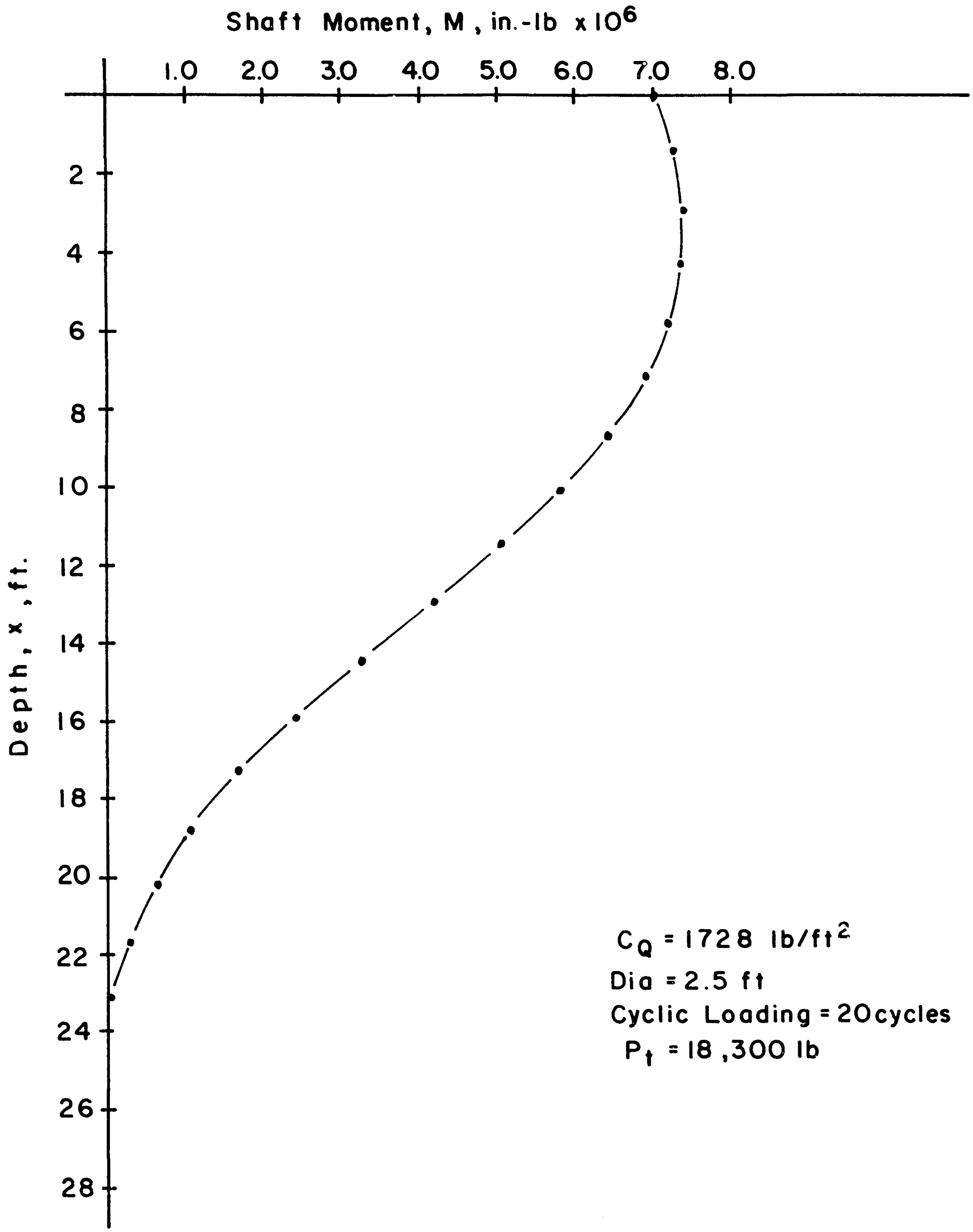


Fig 4.5. Bending moment in shaft versus depth.

6 inches is stipulated by contractors organizations; therefore, the selection of the diameter by use of charts can usually be done with confidence. Refinements in the design will principally involve the selection of the amount and placement of the reinforcing steel and the selection of the required penetration.

Care must be used when designing by chart. The greatest possibility for error occurs when the charts are applied to a situation for which they were not formulated. In order for the charts to be correctly used, the designer should be acquainted with the basic theories and assumptions underlying both the problem and the technique for generation of the charts. Two examples of where it might be difficult to use the charts as presented are if the soil profile consists of a weak clay over a very hard clay or if the soil profile consists of interbedded layers of clay and sand. When only the grossest aspects of the field conditions are known, general information concerning field conditions is available. In some instances, where design data are sketchy at best, and there is to be no foreseeable expenditure for soils testing, the charts may prove to be almost as effective as a more advanced technique.

A computer-based design procedure, similar to the one previously outlined, will also have its good and bad points. In comparison to the charts, there is definitely an increase in design time, although practice in data-set building and the use of remote terminals can lead to a large reduction in the amount of design time. Of course, any increase in design time translates into increased cost. In addition, there will be the added cost of computation time on the computer system. This program (COM623) is relatively inexpensive in comparison to others in use at SDHPT. As with the use of charts, the computer-based design can be made simple and straightforward, although it is critical that the designer have an understanding of the details of the method and its limitations.

The computer-based design is an extremely versatile arrangement. Whereas charts must face restrictions in certain areas, the computer design can usually vary the same parameter in an almost limitless manner. Human judgment can reduce these choices to a practical number and yet still allow a large degree of freedom. The chart's inability to treat a layered system becomes merely a problem of where to describe the layers in the computer design. The number of layers, type and strength of materials, and pattern of layering can all be easily input and a whole series of structural variations can be examined in a very short period of time. Another aspect of the computer-based design is the production of not only a final size but also a complete description of the behavior of the soil-structure system.

CHAPTER 5. GENERATION OF DESIGN AIDS

Design aids such as the ones developed by the Texas State Department of Highways and Public Transportation are popular for a variety of reasons. In many instances they provide design solutions that are safe and economical, and the aids are relatively easy to use. Situations are often such that highly exact, theoretical solutions are no better than less rigorous, approximate solutions, simply because the designer is unable to obtain accurate and reliable data for input. Where this is the case, design aids based upon simplifying assumptions and generalization of certain parameters will provide adequate solutions. However, the aids must be formulated using sound theory, and the limitations and generalizations used must be fully understood by both the author of the aids and the user.

For example, tables and curves generated using soil properties such as shear strength or unit weight that are constant with depth should not be expected to give more than approximate design in cases where extensive explorations indicate that a complex, layered system exists. In such an instance design aids would best be used as a point of departure in a more detailed process. Conversely, many soil profiles show somewhat constant characteristics, and designs in such instances can be adequately made using the appropriate tables and curves.

GENERATION PROCEDURE

The generation of design aids can be accomplished with the aid of COM623. The process can be approached in several different ways, with no single approach being essential. Certain generalizations peculiar to one approach may or may not be made in another. One such method of generation is discussed below.

ESTABLISHMENT OF RANGE OF TABLES AND CURVES

The process is initiated by selecting a limited series of shaft diameters with a convenient increment in size. For instance, diameters of 24, 30, 36, and 42 inches could be used. With the shaft diameters set, the capacities of the shaft must then be determined for tabulation. Values of reinforcement ratio, ρ , cover to reinforcement, grade of steel, f_y , and strength of concrete, f_c' , are set. Two or three reinforcement patterns and several values of axial load are selected. For each axial load selected, values of M_u are computed for each bar size. Tables of M_u can then be arranged as shown in Fig 5.1.

GENERATION OF CURVES

The next phase involves the generation of a series of curves which will describe the soil-structure behavior. For a chosen diameter, a shaft length, L , is selected such that "long pile" action occurs when the shaft is loaded with approximately 150 percent of the expected design loading. This length will provide the basis for the production of the first two curves. Values of ϵ_{50} and k_s are selected as the constants that are most compatible with the expected range of soil properties. The moment of inertia has been established by the pile geometry and is best based upon the gross section. As an alternative, program PMEI (Appendix A) may be used to generate values of EI in which I is based upon a "cracked" section with appropriately modified areas of steel reinforcement.

With the appropriate values established, a given value of shear strength, c_Q , is chosen and curves are generated for a range of loads. For each value of c_Q , curves of lateral load, P_t , versus shaft moment, M , and lateral load, P_t , versus groundline deflection, y_t , are established (see Fig 5.2). After these curves are generated, the shaft length previously selected is used as a starting point for a series of solutions in which the shaft length is decreased for each new analysis. A curve is generated showing the behavior of the shaft with respect to embedment. Figure 5.3 presents a set of such curves, each generated as outlined for different shear strengths. The necessary curves are now complete. A typical design problem could be approached as shown in the next section.

P_x = 0 k				
# of Bars/M_u ; Values of Moment in ft-k	Shaft Diameter, in.			Notes
	30	36	42	Based on P_x = 0 k
9	8/375*	10/591	14/1006	f_c' = 3.6 ksi
10	6/349	8/593	12/1078	f_y = 60.0 ksi; $\rho \simeq$ 1.0 percent
11	-/ -	6/541	10/1116	2.25-in. cover (clear) to # 3 spiral (6-in. pitch)

P_x = 500 k				
# of Bars/M_u ; Values of Moment in ft-k	Shaft Diameter, in.			Notes
	30	36	42	Based on P_x = 500 k
9	8/536*	10/858	14/1228	f_c' = 3.6 ksi
10	6/520	8/854	12/1267	f_y = 60.0 ksi; $\rho \simeq$ 1.0 percent
11	-/ -	6/840	10/1286	2.25-in. cover (clear) to # 3 spiral (6-in. pitch)

P_x = 1000 k				
# of Bars/M_u ; Values of Moment in ft-k	Shaft Diameter, in.			Notes
	30	36	42	Based on P_x = 1000 k
9	8/483*	10/904	14/1448	f_c' = 3.6 ksi
10	6/470	8/901	12/1489	f_y = 60.0 ksi; $\rho \simeq$ 1.0 percent
11	-/ -	6/874	10/1500	2.25-in. cover (clear) to # 3 spiral (6-in. pitch)

1 ft = 0.3048 m; 1 ft-k = 1.356 kN-m; 1000 lb/ft^2 = 47.88 MPa;
1000 lb = 0.4536 Mg

Fig 5.1. Tables of ultimate moment.

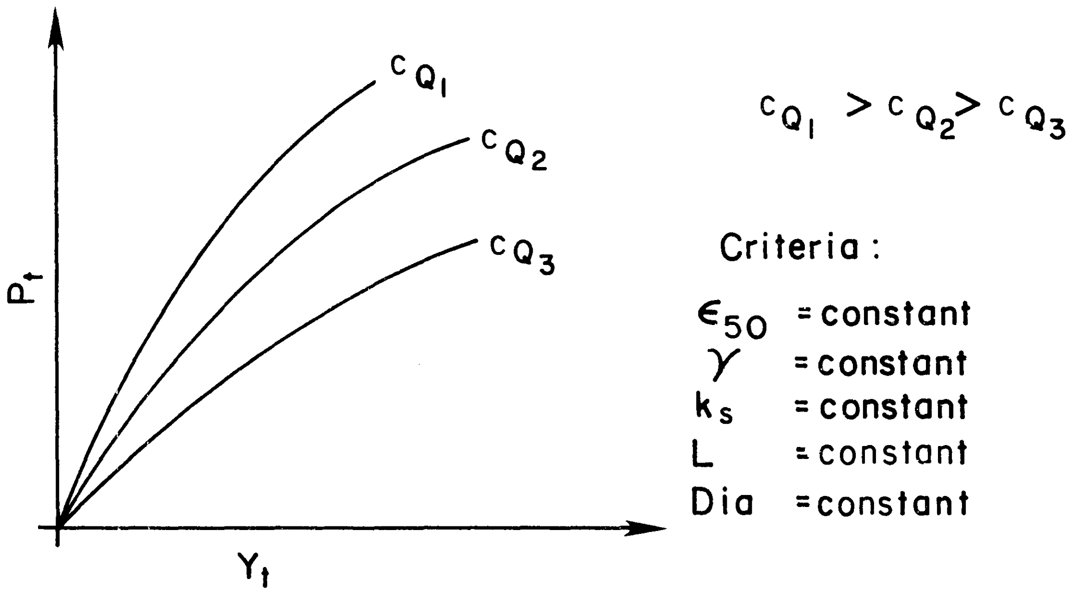


Fig 5.2a. Groundline deflection versus lateral load.

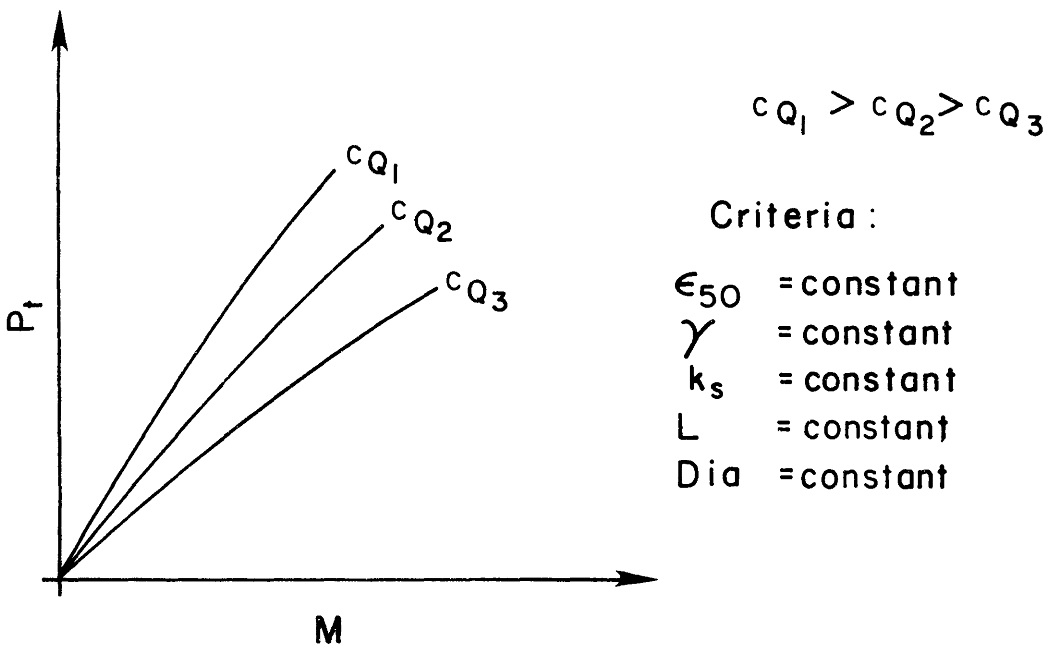


Fig 5.2b. Groundline deflection versus shaft moment.

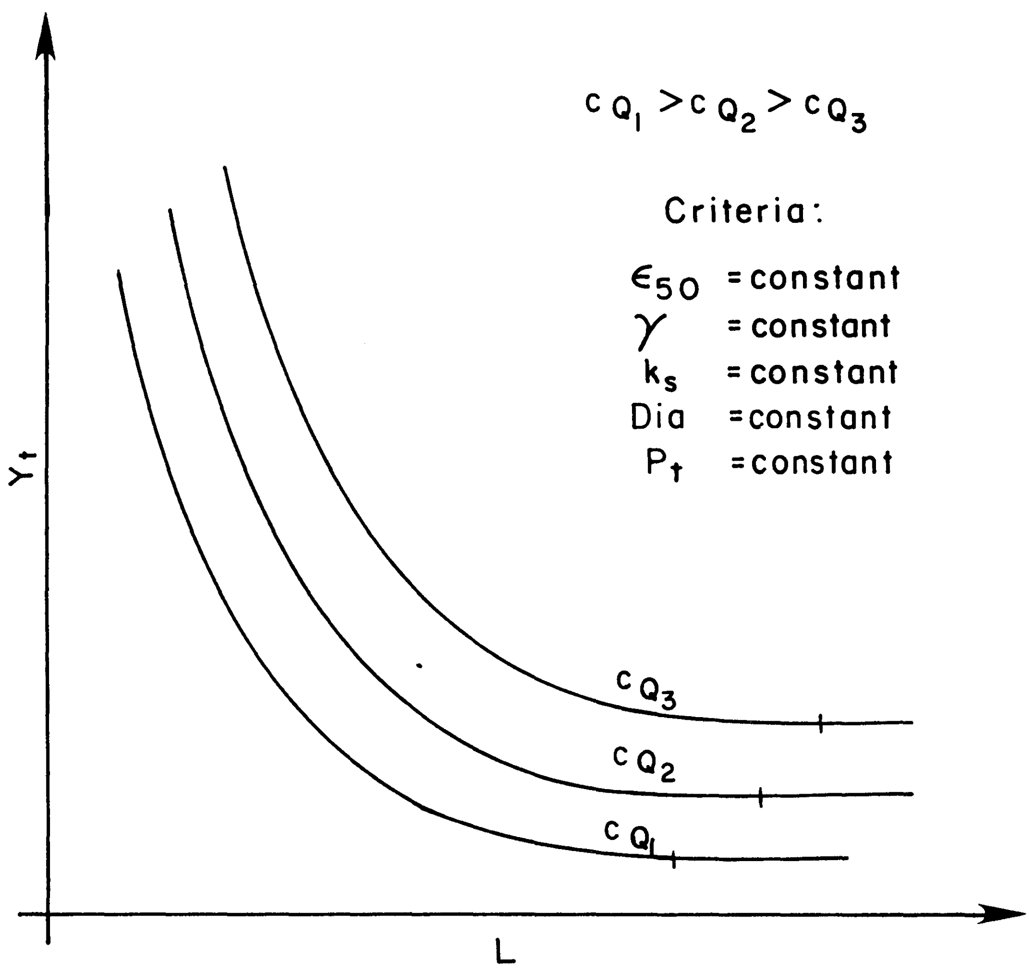


Fig 5.3. Groundline deflection versus shaft length.

USE OF TABLES AND CURVES

All available data are examined and values of shear strength, unit weight, and soil type are selected. A trial diameter and reinforcement pattern is selected for the given design loads. With a value of P_t , enter the P_t versus M chart and find the shaft moment, M . Check the M_u table to ensure that the capacity of the cross section is not exceeded. If the shaft moment is less than the cross section capacity, the next step is to enter the P_t versus y_t chart. If the shaft moment is greater than the section capacity, a large diameter should be chosen. Once the chosen section checks as adequate the deflections may be checked.

With the design load, P_t , and the P_t versus y_t charts, the deflections may be determined. If these appear to be less than the allowable, the procedure continues; if not, a larger diameter must be chosen. The shaft length can be set once the deflection criteria have been satisfied. The y_t versus L chart will give the design length by any method desired. When generating these curves, care should be taken to mark each curve with the length at which pile behavior changes from that of having two points of zero deflection to that of having only one point. This can be used as a reference point in selecting a design length. Another method, giving slightly shorter lengths, would be to lay a straight edge along the straight (horizontal) portion of the appropriate curve and determine the point of tangency to the curved segment. This could then be the chosen design length. As a third option, a value of deflection could be predetermined. The curves could then be entered with this value and a corresponding length chosen. This procedure would, however, ignore the beneficial behavior of the longer pile in resisting load and overload.

The design aids are limited by several factors and earlier sections discuss the restrictions inherent in such design aids. With this in mind, the method can be used to find a preliminary design and a final design when the particular design warrants such an approach.

The brief presentation in this section provides no guidance for integrating the tables and curves into a series of charts, tables, curves, or nomographs. Such integration is, of course, possible. A possible approach would be to develop the required number of tables and families of curves as illustrated and to present them in a manual with appropriate indexing.

CHAPTER 6. SAN ANTONIO TEST AND RESULTS

In December 1978 and January 1979, two sets of drilled-shaft foundations were made available by SDHPT for testing. The shafts were located on the western section of IH 410 in San Antonio, Texas. They had been in use as foundations for an overhead-sign structure that spanned the southbound lane. The existing signing and supports were to be moved to another position and the shaft foundations removed to allow the construction of a new access roadway. The major portions of the shafts were to remain in the ground with only the top several feet being removed and the holes backfilled. Because the shafts were of no further use, testing to failure was permissible. The vertical-support trusses were available for use in the testing. However, they were to be reused and could not be damaged during testing. Furthermore, analysis seemed to indicate that the anchor bolts in the heads of the shafts could not withstand the loads that were expected to be applied in order to cause shaft failure. For these reasons, it was decided not to use the vertical trusses or the anchor bolts for purposes of load application. This decision ruled out the possibility of loading the shaft at some point above the shaft head (a loading which would produce both shear and moment at the top of the shaft). It was decided instead to test the shaft by applying the load at the top of the shaft.

TEST SITE AND CONDITIONS

The aims of the testing program in San Antonio were as follows: to obtain data by which the analytical procedures could be evaluated, to obtain a direct indication of the strength of a drilled shaft in a typical installation, and to obtain physical evidence concerning the interaction of a drilled shaft with the supporting soil.

The two test sites were near the intersection of U.S. 90 and IH 410 in west San Antonio. The sign structure had spanned the southbound lane, with one set of foundations located between the northbound and southbound main lanes and the other set located between the southbound main lane and a southbound feeder road (see Fig 6.1). The latter set was designated Site 1 for further discussion, with the shafts between the north and southbound main lanes as Site 2.

Site 1 lay within a drainage ditch that sloped up to the south (see Fig 6.2a). It can be assumed that some cut and fill took place in the area, for construction of the roadways, with the entire area resodded after construction. Site 2 was somewhat different in that, instead of resodding, a base coat of crushed rock about 6 inches thick and covered by an inch of asphaltic material was placed after the road and drainage ditch had been graded. It can be assumed that the soil in the first few feet had undergone considerable compaction relative to the Site 1 material. Figures 6.2a and 6.2b give detailed layouts of both sites.

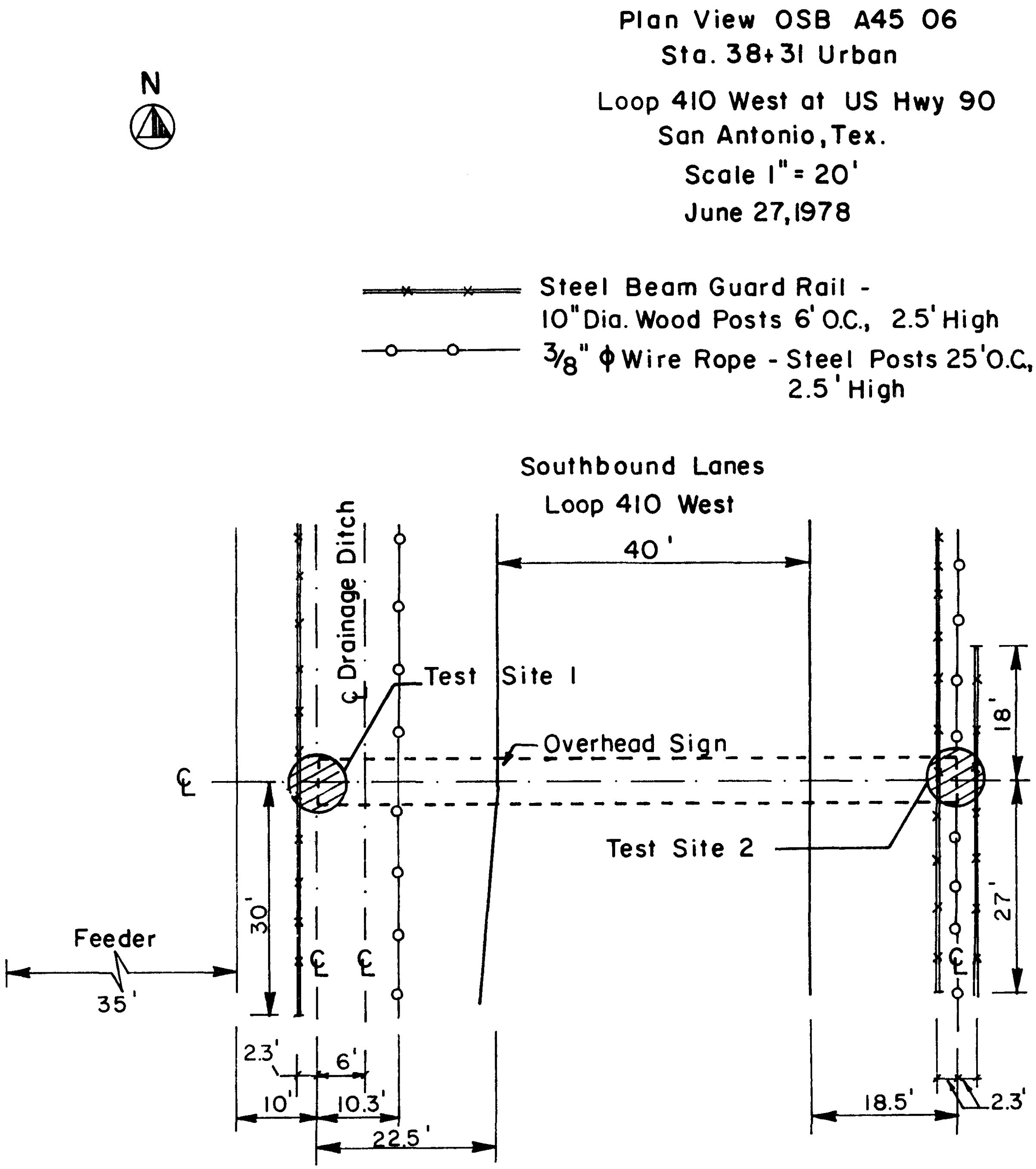
The test at Site 1 was performed on December 15, 1978, and the test at Site 2 on January 31, 1979. In both cases, the testing was carried out immediately following the passing of a weather front from the north. The weather was therefore dry and cold with some gusting northerly winds and clear skies. However, due to the passing of the storm, the soil at Site 1 was fairly well saturated and soft enough for a small utility van to become stuck near the site.

Soils testing was performed at Site 1 in July 1978. This was done immediately after a period of severe rains and the ground was extremely soft. An SDHPT drilling rig was used to obtain 3-inch "undisturbed" samples as well as to perform the SDHPT pen test. The samples were tested at the site with a pocket penetrometer and pocket torvane device. Q-type triaxial tests were performed in the laboratory at The University of Texas at Austin. The results of these tests are given in Fig 6.3. It is to be noted that the soil was badly fissured and that the fissures opened very quickly after being extruded from the sampling tubes, making trimming and testing difficult.

TEST DETAILS

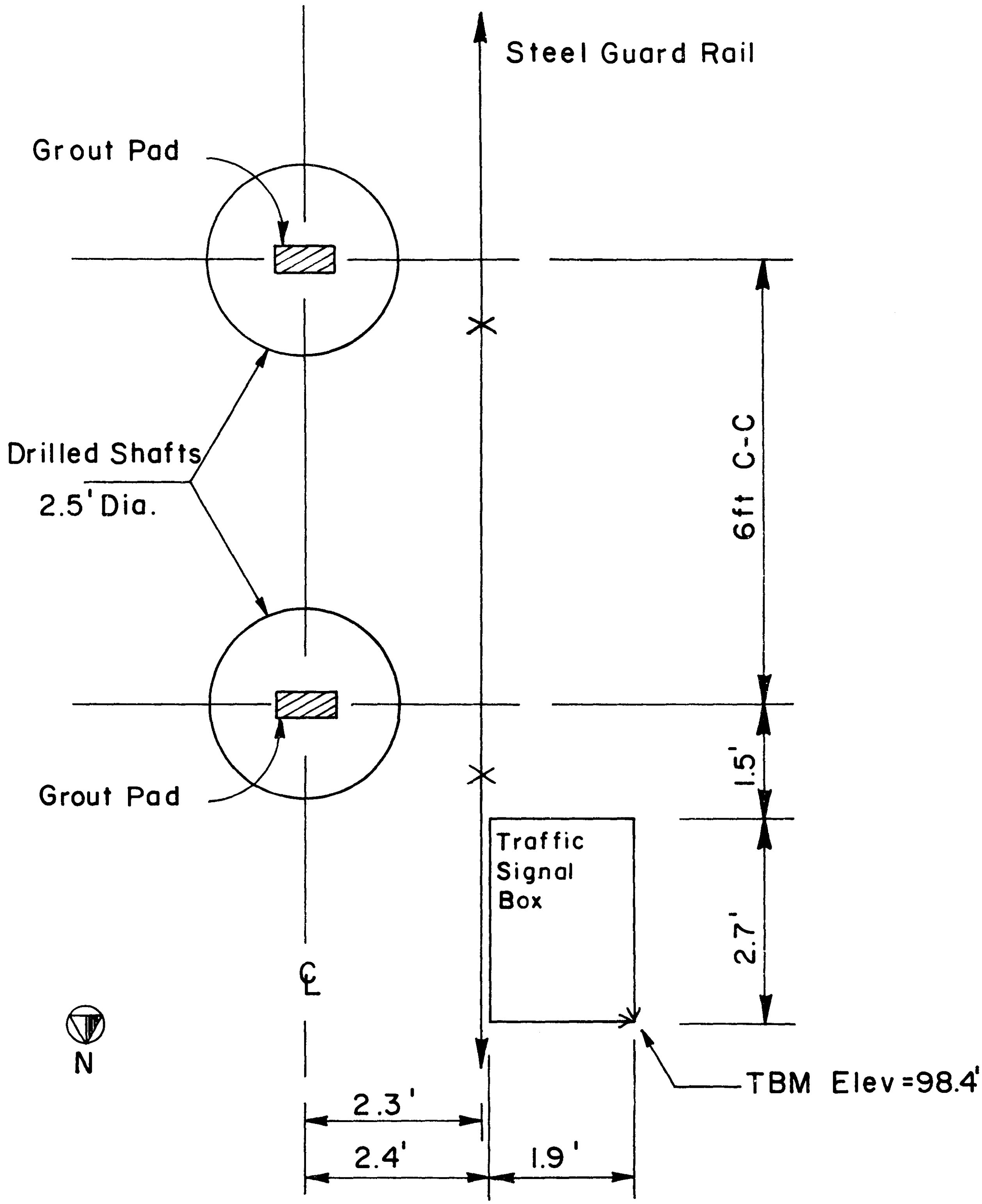
SHAFT CONFIGURATION

As mentioned previously, Site 1 was located within a drainage ditch which had been resodded after construction. The point of load application to the shafts was to be at the shaft head. Because of the shaft positions it was



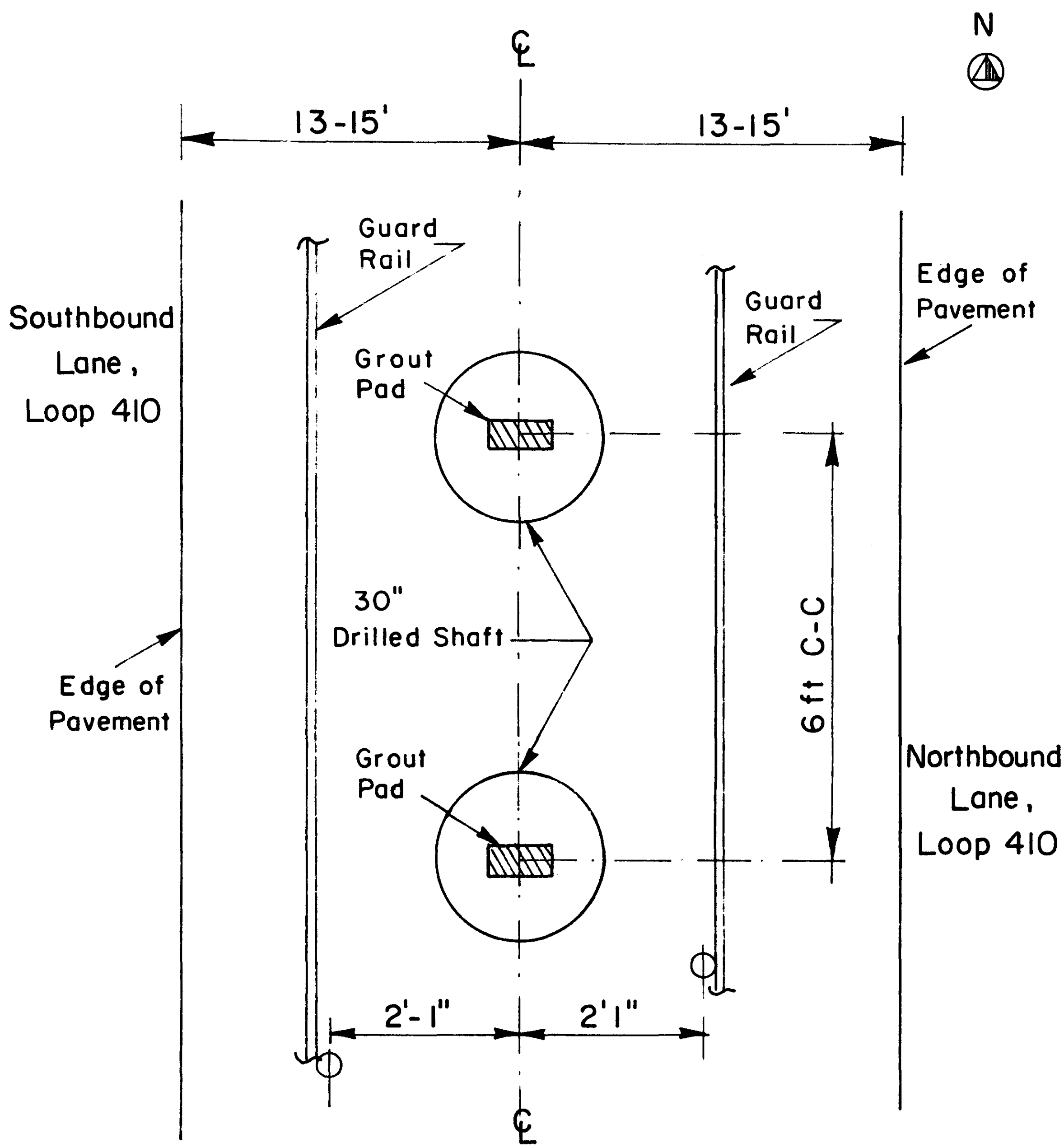
1 ft = 0.3048 m

Fig 6.1. San Antonio test site.



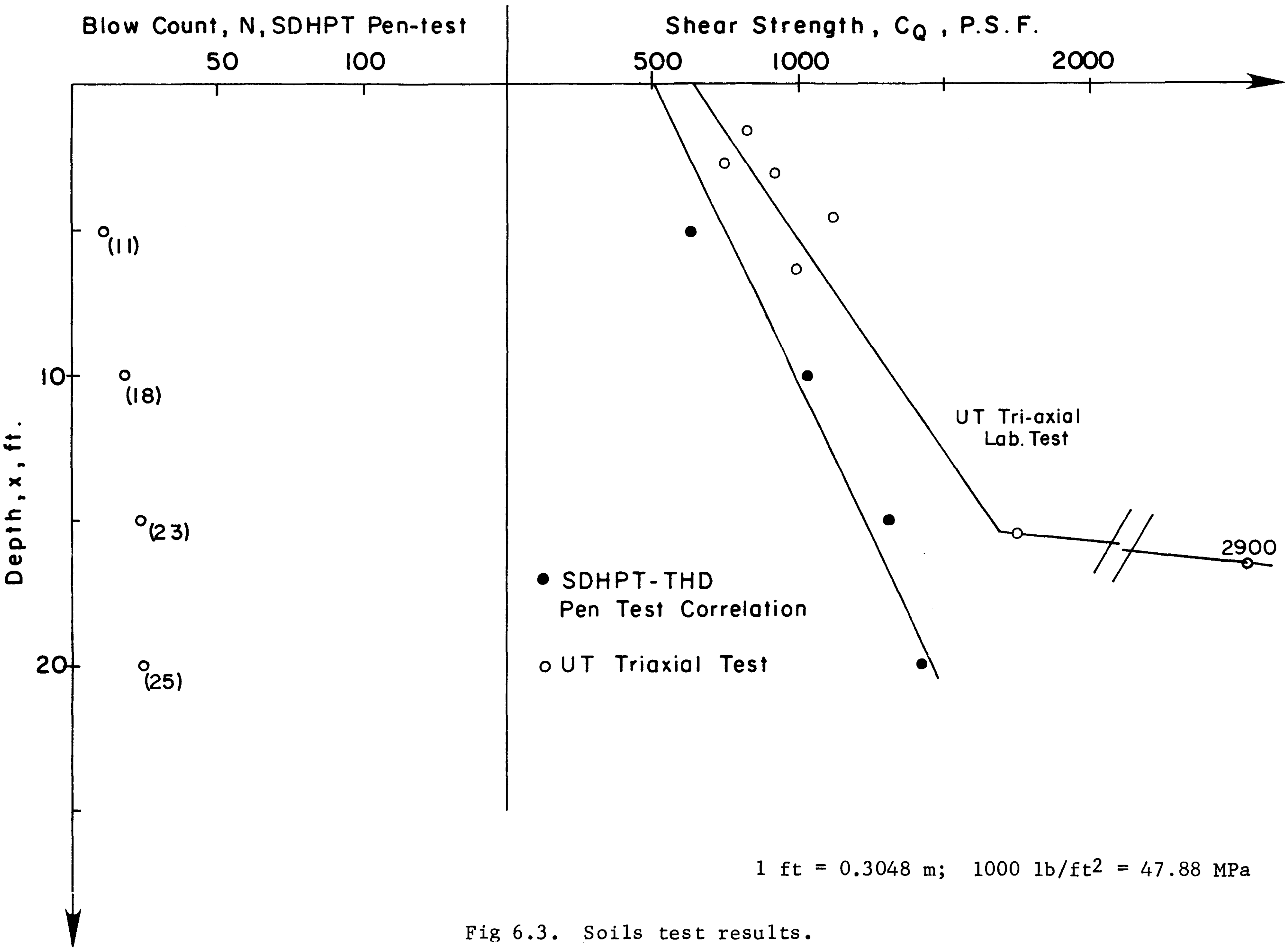
1 ft = 0.3048 m

Fig 6.2a. San Antonio test site 1.



1 ft = 0.3048 m

Fig 6.2b. San Antonio test site 2.



1 ft = 0.3048 m; 1000 lb/ft^2 = 47.88 MPa

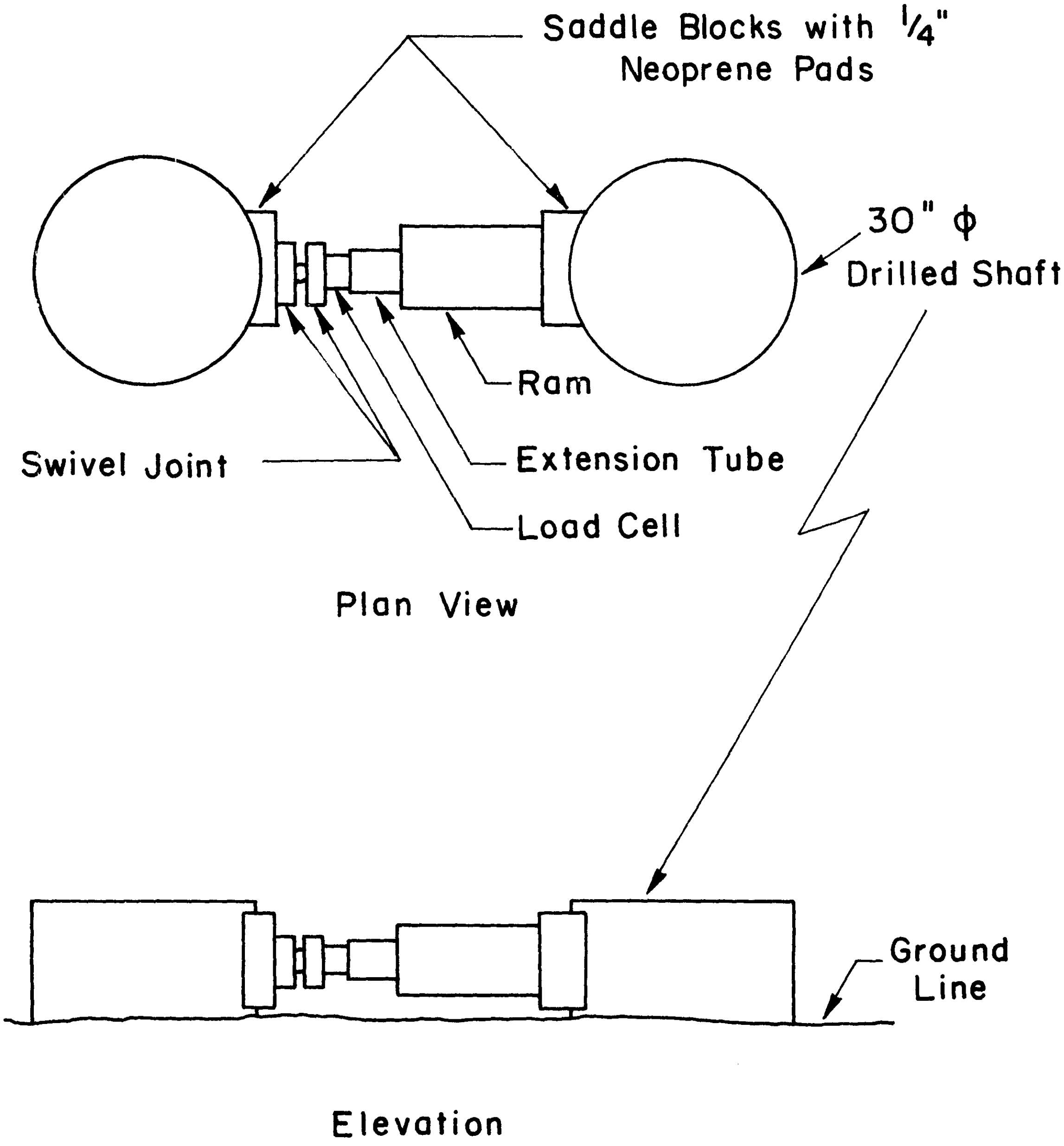
Fig 6.3. Soils test results.

felt that the application of load between the two shafts would be acceptable. The size of the jacking equipment dictated that an inch or so of soil be removed in areas near the southernmost shaft. Site 2 presented a similar problem. The tops of the shafts were located 2 to 3 inches above the asphalt, necessitating the removal of 6 to 8 inches of material. This material was also removed for a distance of 3 to 4 inches all around the shafts at Site 2 in an attempt to keep the increased stiffness of the base and asphaltic materials from influencing the test results.

The shafts for both sites were straight-sided 30-inch-diameter shafts spaced 6.0 feet center to center. The lengths were specified on the original plans to be 17 feet at Sites 1 and 2. However, there was no way to verify the embedment. The ground surface around the shaft heads varied from being almost level with the shaft head to being as much as 6 to 8 inches below the shaft head. At both sites, minor excavation was performed between shafts to allow room for the jacking system. At Site 1 a guard rail was located approximately 2-1/2 feet to the west of the shafts. Site 2 had guard rails located on both sides of the shafts.

LOAD SYSTEM

The load system consisted of a hydraulic ram, "saddle" blocks, spacers, a load cell, and a swivel joint (Fig 6.4). The saddle blocks were constructed of steel plate and were cushioned on the shaft faces by neoprene pads. The hydraulic ram was a double-acting ram of 60 kips capacity (Site 1) and 100 kips capacity (Site 2), each with 10-inch strokes. The spacers were two round steel pipes 2-1/2 inches in diameter with 1/2-inch-thick walls and approximately 4 and 6 inches in length. The swivel consisted of 2 cylindrical steel shapes, each having one face dished out. A hardened steel ball was placed between the two pieces, allowing the loading system to pivot about a point as the two shaft faces rotated from the vertical. The ram was powered by a hydraulic pump. Pressures were regulated by a system of valves to attain predetermined strain readings from the load cell.



1 ft = 0.3048 m

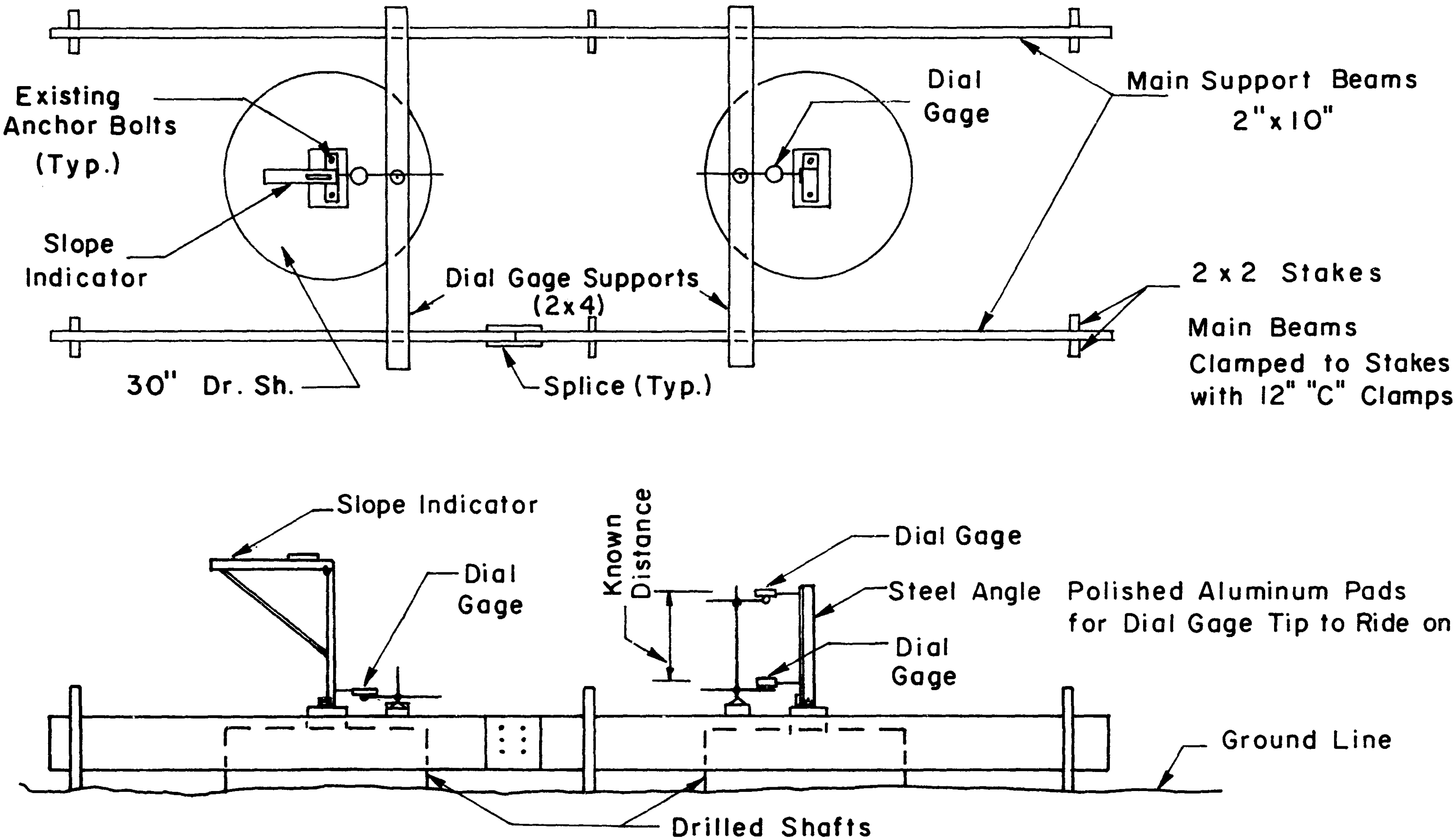
Fig 6.4. Load system configuration.

SYSTEM FOR MEASURING DEFLECTION

Deflections and rotations of the shaft heads were measured using a system of dial gauges mounted on a wooden frame (Fig 6.5). The frame consisted of 2-in.-by-4-in. stakes driven into the ground approximately 6 feet from the shafts. Beams were then clamped to the stakes and cross members were fastened to the beams. Dial gauge stands and support rods were attached and dial gauges were mounted to the rods. Vertical steel angles were bolted to the shaft heads using the existing anchor bolts. Small aluminum plates were in turn clamped to the angles to provide a relatively smooth surface for the dial-gauge stems to ride on. Two dial gauges were used on each shaft, the lower gauge located approximately 3 inches above the top of the shaft and the other about 18 inches above the lower gauge. In addition, one shaft had a device mounted on the vertical steel angle which measured the shaft-head rotation. For each increment of load and shaft rotation, an arm on which a level vial was mounted was re-leveled using a barrel micrometer. With the geometries known, the amount of rotation could be computed. Readings of gauges and slope indicator were made after each load application.

LOADING AND RESULTS OF FIELD TEST

The loading sequence consisted basically of loading the shaft to a predetermined level, unloading, and then reloading to the same level. After a series of load cycles, the load was increased to a higher level and a new series of load cycles performed at this new load. Measurements of deflection and slope were made at given increments of load during the loading process. The rate of unloading was not controllable and no effort was made to measure deflections during unloading. After all load was released, shaft movements continued for a short period of time. After shaft movements were essentially finished upon unloading, measurements were made as soon as the movements appeared to stabilize. At very high load levels, near the end of the test, movement would continue. If the pressure in the system was not constantly increased, the load on the shafts would decrease due to the movement of the shaft, and there would be a consequent decrease in pressure and load. In order to maintain a constant load, the pressure was regulated by hand as measurements were made.



1 ft = 0.3048 m

Fig 6.5. Deflection and slope measurement system configuration.

The results of the testing are presented in Figs 6.6, 6.7, and 6.8. The capacities of the shafts were approximately the same for both sites. Loads of from 60 k to 70 k caused behavior that could be interpreted as failure. Within this load range the rate of deformation occurring did not seem to decrease with time. When an attempt was made to increase the load, the deflection rate would increase. Loading was continued until the system was extended enough that an unstable configuration occurred at the swivel joint. At Site 1 this represented a deflection of around 4 inches at each shaft; at Site 2, around 2-1/2 inches for each shaft.

Figure 6.7 illustrates the effect of cyclic loading on deflections. For each additional cycle of load the deflection increases with the increase being larger at higher loads. Dashed lines indicate limits that could be expected for static or cyclic loading. A static loading would follow the upper dashed line. A cyclic loading would, for a given load value, produce a point on the lower dashed line. In this manner the two lines may be thought of as upper and lower limits on the shafts' behavior.

It was noted that at the conclusion of any given loading a certain amount of permanent deformation existed. Figure 6.8 presents the residual or permanent deflections noted during the cycling of the 60-k load. Figure 6.9 illustrates the patterns of soil disturbance noted at the end of testing. Semi-annular openings, on the order of 3/4 inch to 1-1/4 inch in width and extending around the shaft for about 120^{o}, were noted. At both sites these openings occurred not at the shaft surface but at a distance of 2 to 3 inches from the surface.

This indicates that the soil-structure interface remained intact and that failure actually occurred within the soil mass. At Site 2 a meter stick was inserted into the opening to a depth of 35 inches. At both sites radial cracks in the soil surface were noted on the side opposite to the load application. A slight mounding of the soil was observed in this area. None of the shafts indicated any distress in the concrete.

COMPARISON OF FIELD TEST RESULTS TO COMPUTER ANALYSES

An analysis was made using COM623 and all available data for the site. The results are presented in Fig 6.10. If a shaft length corresponding to that indicated on the plans is used, the results are quite conservative, especially for higher load levels. Several different parameters can be varied

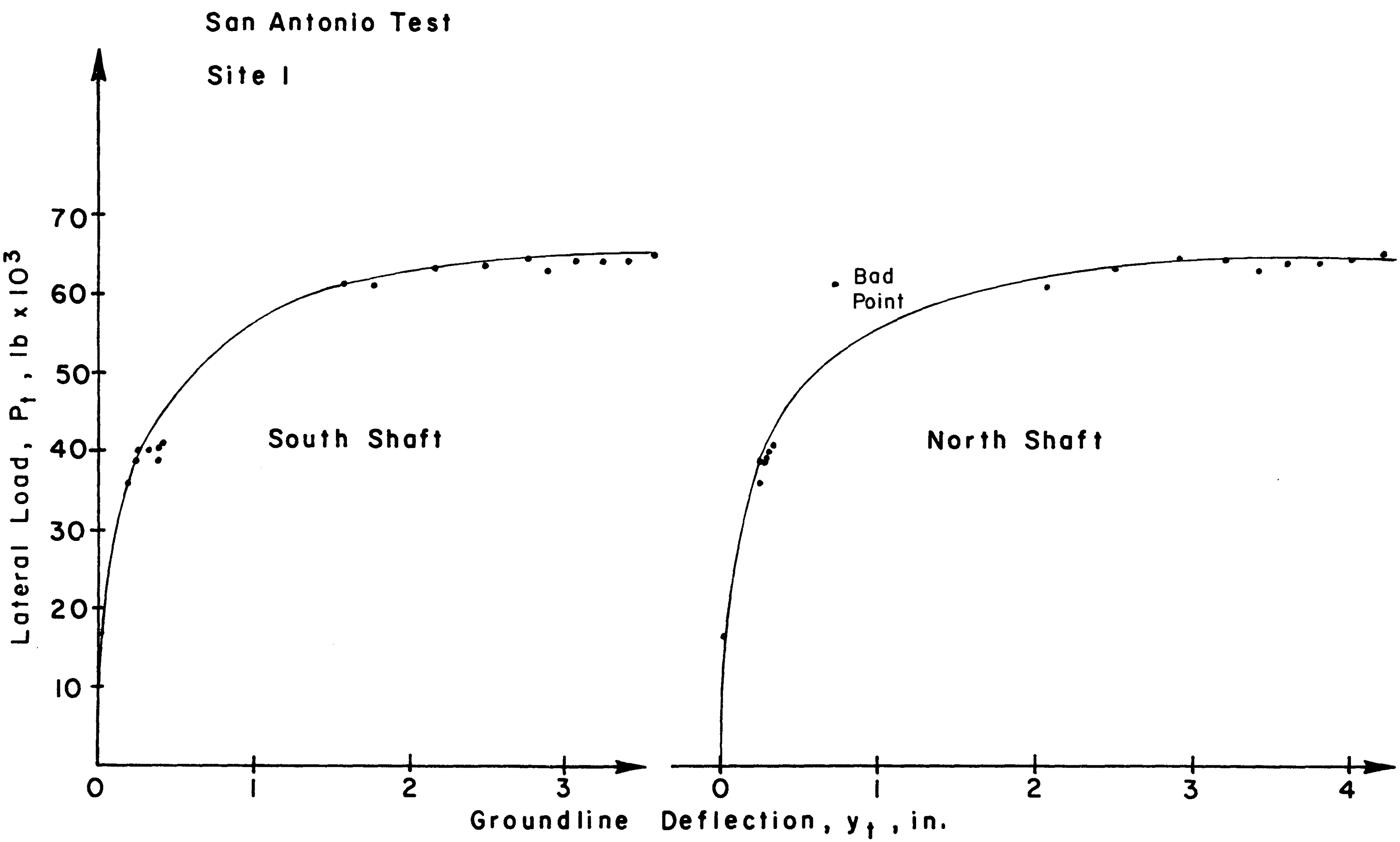


Fig 6.6. Lateral load versus groundline deflection.

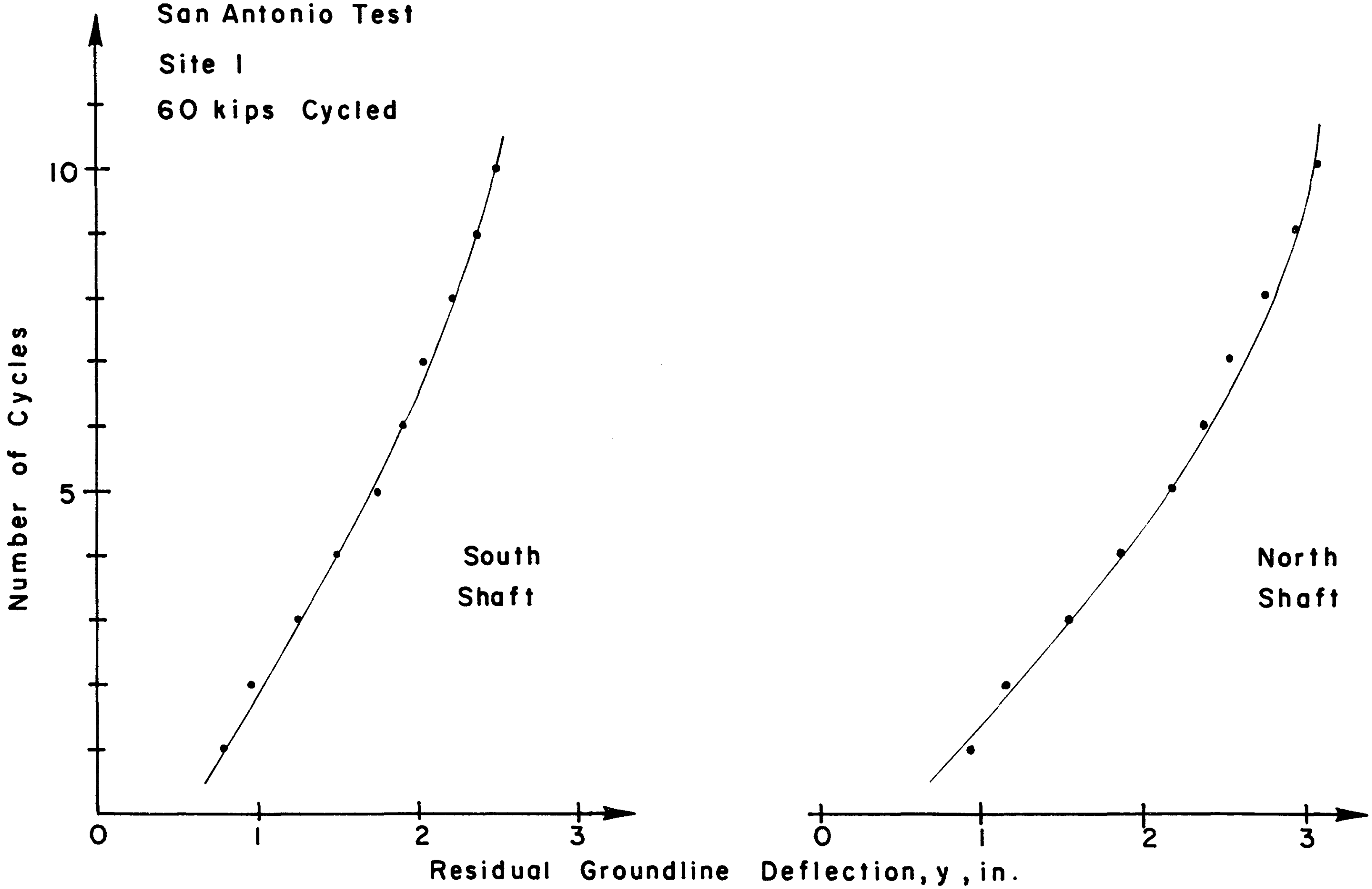


Fig 6.7. Lateral load versus groundline deflection.

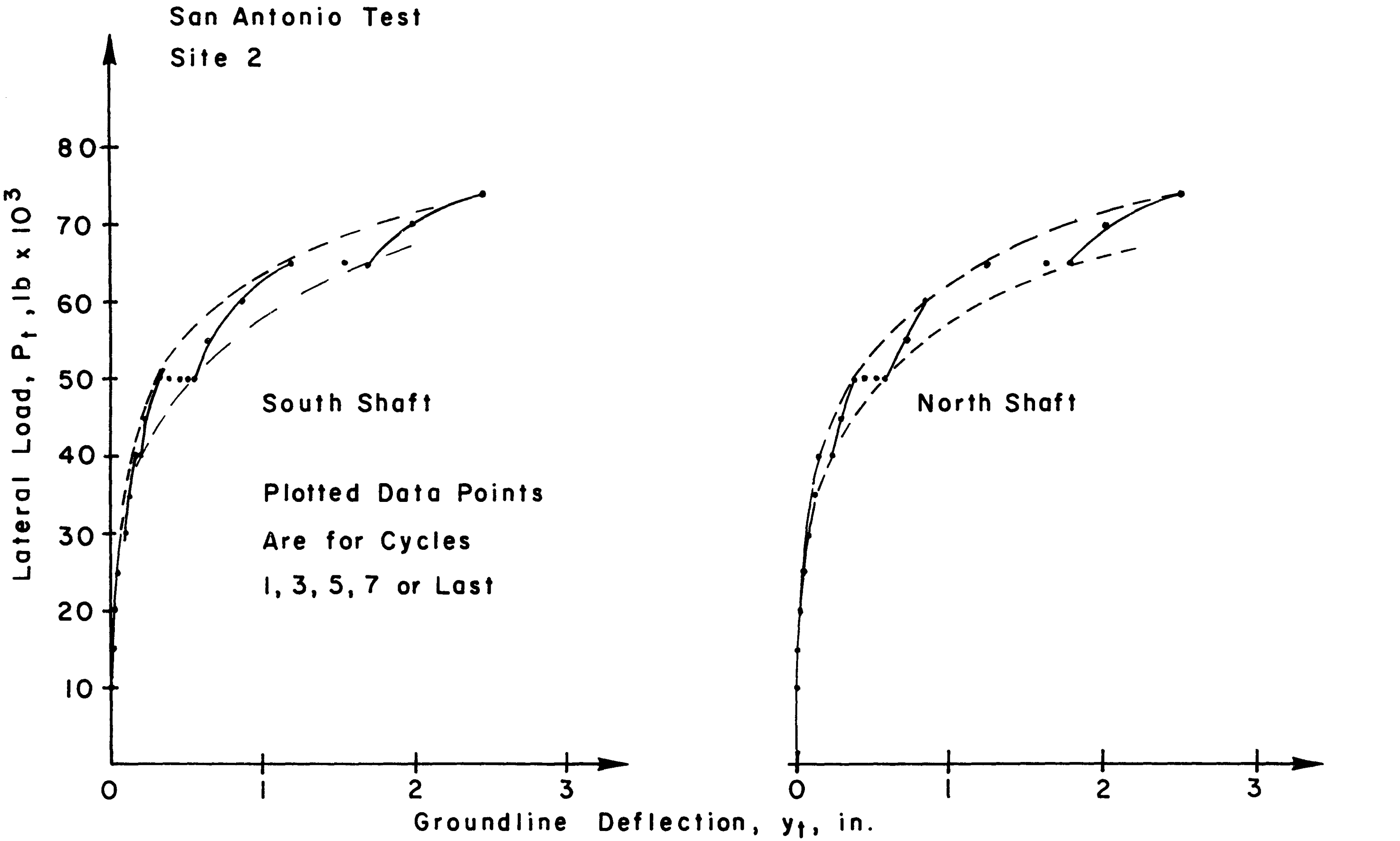


Fig 6.8. Lateral load versus groundline deflection.

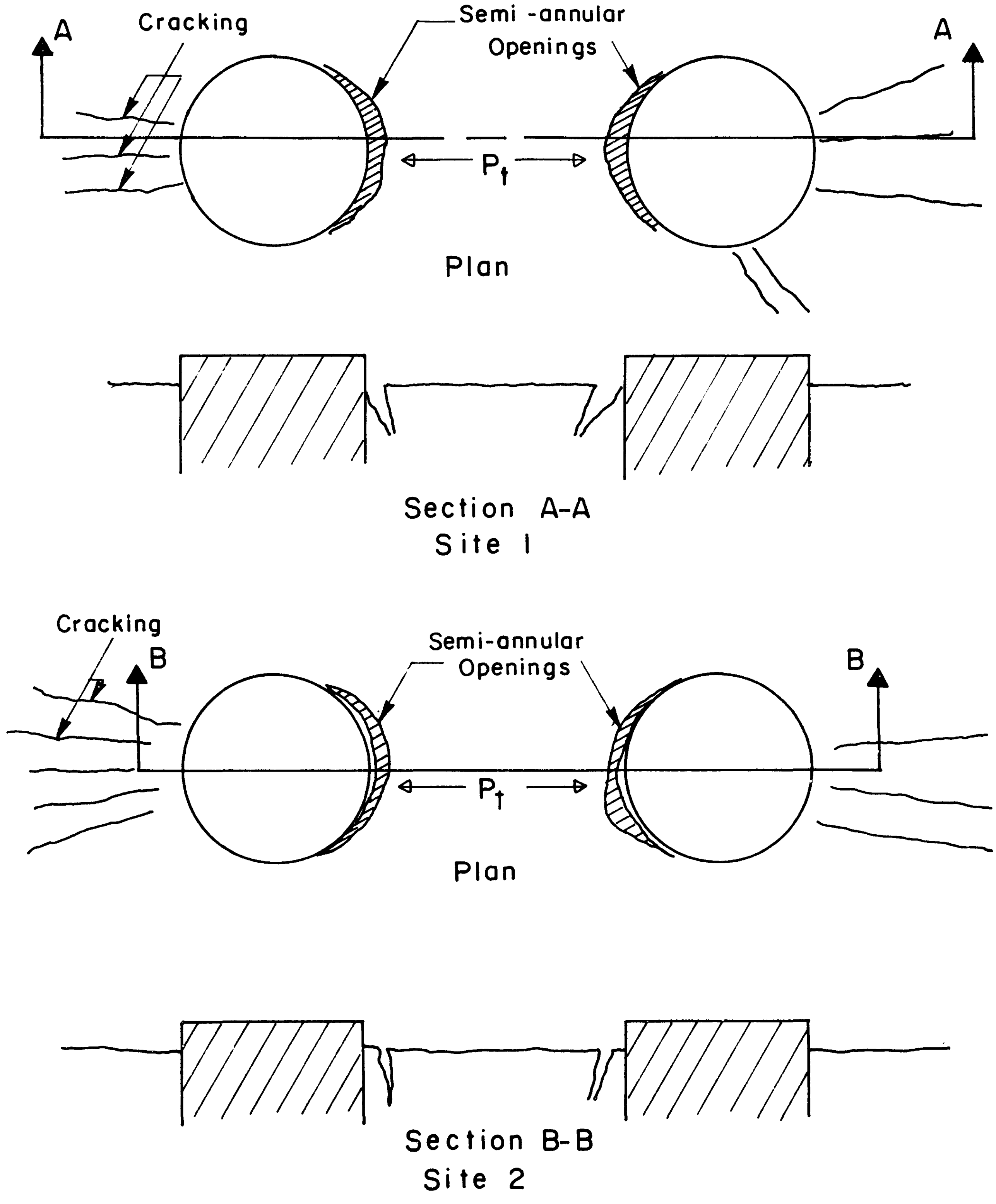
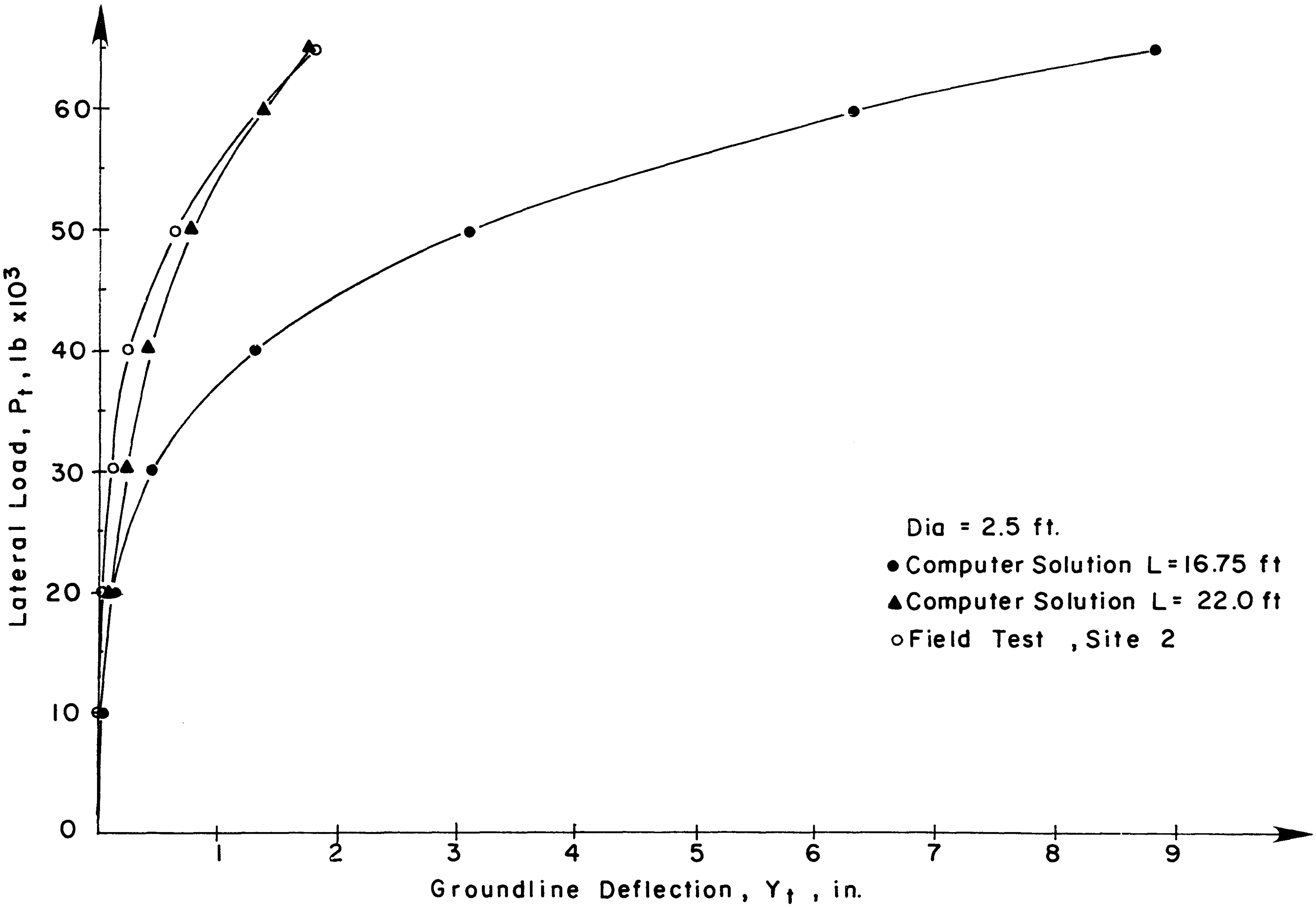


Fig 6.9. San Antonio test, observed failure patterns.



1 ft = 0.3048 m; 1000 lb = 0.4536 mg

Fig 6.10. San Antonio test, computer analyses and field test results.

in an attempt to provide a better match to the field curve. For instance, a length of 22 feet was used and produced a curve that is close to the field curve. However, this is an approach that is quite difficult to use when no field curves exist for comparison.

CONCLUSIONS

Test results indicate that theories in use are correct. The pattern of reduction in soil capacity noted is the same as that observed in earlier testing programs. The capacities predicted by the computer analysis and the capacities observed indicate that the method of analysis used will give conservative results. Observed shaft capacity would also indicate that a reserve strength is available to resist possible overload.

CHAPTER 7. COMPARISON OF SINGLE AND DOUBLE SHAFT SYSTEMS

For any given situation the foundation system that is selected will depend on several different variables. The parameters that are involved will range from those with well known values to those that are of a general or indeterminate nature. For instance, the loading to which a system will be subjected may be well known whereas the variability of the construction process may lead to the necessity of using an estimated value for the shaft stiffness. The objective-subjective attitude of the designer will influence the final design. Soil profiles represented by visual inspections and blow-count reports are much more susceptible to subjective interpretation than the reported results of tests of concrete cylinders. Personal preference for large factors of safety for certain types of loading, construction methods and site conditions may exert an undue influence on the type of foundation that is finally selected. Although many factors will enter into the design process and many decisions will be subjective in nature, an attempt will always be made to judge the final design by one universal criterion, cost. For each of the preceding design methods a solution was proposed. The final step of the process would be the economic comparison of the solutions that are obtained.

ECONOMIC COMPARISON OF PROPOSED DESIGNS

Initially it is to be assumed that each system that is designed is comparable to all the others with respect to factors such as site suitability, ease of construction, and time of construction. Subjective judgments must be made regarding these factors, particularly with respect to the latter two. The three factors noted above will be assumed equal for each design considered in order to make the conomic comparison simpler. The physical requirements and price for each solution are summarized in Table 7.1. The price given is based upon SDHPT average low bids compiled for the twelve-month period ending in February 1980. The price quoted was $74.84 per linear foot for a 30-inch-diameter drilled shaft and was based upon a total bid quantity of 60,774 linear feet.

TABLE 7.1. COMPARISONS OF SINGLE- AND DOUBLE-SHAFT SYSTEMS

System	Method of Design	Shaft Size and Length	Total Cost, U.S. Dollars
Single Shaft	THD charts	2 @ 30" Ø × 18'	$ 2,694.24
	COM623	2 @ 30" Ø × 25'	3,742.00
Double Shaft	THD charts	4 @ 30" Ø × 19'	5,687.84
	Sacre & Quiros	4 @ 30" Ø × 16'	4,789.76

1 ft = 0.3048 m

Table 7.1 indicates that the cost of the single-shaft system is around one-half the cost of the double-shaft system. If the desired design solution is to be based upon such a simplified cost comparison the single-shaft system would be chosen.

ADDITIONAL FACTORS AFFECTING SELECTION OF FOUNDATION TYPE

In many cases, the decision on foundation type to be used can be based largely on economic consideration. However, the possibility exists that other conditions may influence or even dominate this selection. In every instance the economic analysis must, therefore, be viewed in light of additional variables that are not directly convertible to dollar quantities.

CHAPTER 8. CONCLUSIONS

DESIGN CONCEPTS

TWO-SHAFT SYSTEM

The traditional form of foundation for the overhead sign structure has been a double-shaft system. The loadings, shear, moment, and axial thrust have been resisted largely by the axial resistances of the soil-shaft system in either compression or tension. This system tends to be inefficient in comparison to the single-shaft system for most uses and its application should be restricted to special cases.

SINGLE-SHAFT SYSTEM

The use of single shafts to resist shear, moment, and axial thrust has been suggested. The proper design of such a system leads to the most efficient use of the system materials; the large axial thrusts of the double-shaft system are greatly reduced and at the same time the shaft's capacity in bending is much more fully utilized.

DESIGN PROCEDURES

USE OF CHARTS IN DESIGN

Aids have been developed at SDHPT for use in the design of both single and double-shaft systems. These charts were based upon several simplifying assumptions and failure criteria. The designer should be aware of these assumptions and limits in order to avoid an incorrect application of the chart.

USE OF COMPUTER PROGRAM IN DESIGN (COM623)

The use of a computer program for purposes of design has been presented. This approach allows a maximum amount of variability in the modelling of the system and provides an output of the most probable final configuration of the designed system. Input of data is a simple process and computation times are relatively short.

FACTORS OF SAFETY AND COST COMPARISONS

FACTORS OF SAFETY

Factors of safety have been formulated and used in the generation of the SDHPT charts. However, as mentioned previously, the use of a chart for purposes of design can lead to uncertainty in the actual factor of safety of the final design. The computer program presented has no factor of safety. It can be used for either working stress or load-factor design by the appropriate manipulation of input or output quantities.

COST COMPARISON

A simplified cost comparison has been made and presented for a typical design problem. The single-shaft system was comparatively cheaper than the double-shaft system. Variables exist that can complicate and influence the design in such a manner that cost figures alone cannot be the sole criteria for system selection.

SAN ANTONIO FIELD TEST

TEST RESULTS

Results of a test run in San Antonio indicate that current theories of soil-shaft behavior under loading are correct. The predicted behaviors were conservative in comparison to the observed behaviors, indicating that design procedures based upon the theories involved will yield a safe solution.

FURTHER RESEARCH

Testing of uninstrumented shafts can be relatively inexpensive, and an attempt should be made to perform such tests when an opportunity arises. Testing of instrumented shafts will be relatively expensive but the construction and testing of instrumented shafts will allow further improvements to be made in the design methods.

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APPENDIX

PROGRAM DOCUMENTATION

COMPUTATION OF EI VALUES FOR CONCRETE COLUMNS FOR VARYING BENDING MOMENTS

A.1. PROGRAM IDENTIFICATION

A.1.1. PROGRAM TITLE: Computation of EI values for concrete columns for varying bending moments.

A.1.2. PROGRAM CODE NAME: PMEIX

A.1.3. WRITER: Gangadharan Menon

A.1.4. ORGANIZATION: Department of Civil Engineering, The University of Texas at Austin, Austin, Texas 78712.

A.1.5. DATE: March 1977.

A.1.6. SOURCE LANGUAGE: FORTRAN IV.

A.1.7. AVAILABILITY: A program listing is given following the documentation.

A.1.8. ABSTRACT: The program calculates values of EI (effective product of modulus of elasticity and moment of inertia) of the cross section of a concrete column for a set of values of bending moments under various axial loads.

A.2. ENGINEERING DOCUMENTATION

A.2.1. NARRATIVE DESCRIPTION

A.2.1.1. Statement of the Problem

The flexural behavior of a structural element such as a beam, column, or a pile subjected to bending is dependent upon its flexural rigidity which is expressed as the product, EI, of the modulus of elasticity of the material of which it is made and the moment of inertia of the cross section about the axis of bending. When the values of E and I remain constant for all ranges of stresses to which the member is subjected, the flexural rigidity EI also remains constant. But there are situations where both E and I vary as the stress conditions change. This variation is most pronounced in reinforced concrete members. Because of nonlinearity in stress-strain relationships, the value of E varies; and because the concrete in the tensile zone below the neutral axis becomes ineffective due to cracking, the value of I is reduced.

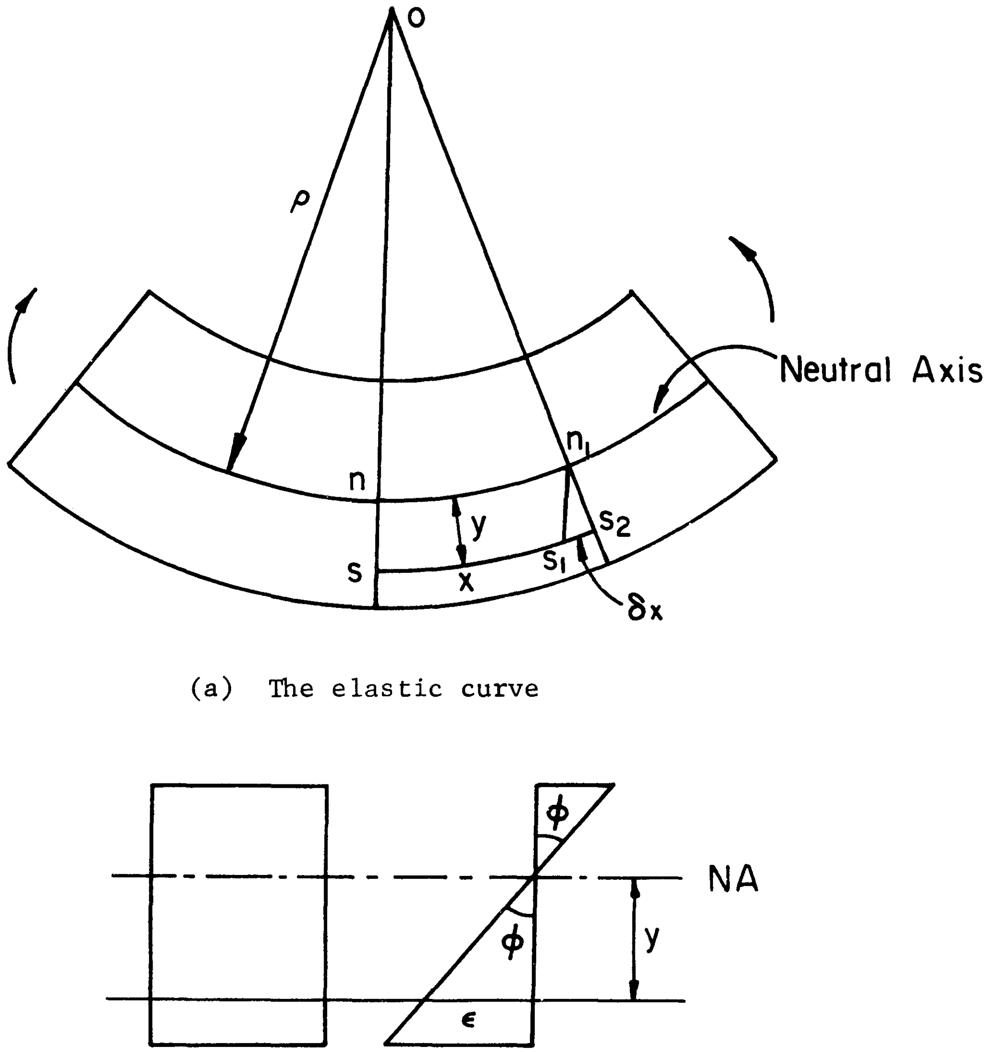
Apart from this, when a member is made up of a composite cross section there is no way to directly calculate the value of E for the member as a whole. Reinforced concrete itself is a composite material, being a combination of concrete and steel reinforcement having different values of E. Other examples are concrete encased in a steel tube or a steel section encased in concrete.

A.2.1.2. Outline of the Solution

The value of EI can, however, be calculated from the moment-curvature relationship of the elastic curve of a beam subjected to bending.

Figure A.1a is a portion of the beam subjected to bending with a radius of curvature ρ . Triangles onn_1 and $n_1s_1s_2$ being similar,

$$\frac{y}{\rho} = \frac{s_1s_2}{nn_1}$$



(a) The elastic curve

(b) Cross section (c) Strain diagram

Fig A.1. Portion of a beam subjected to bending.

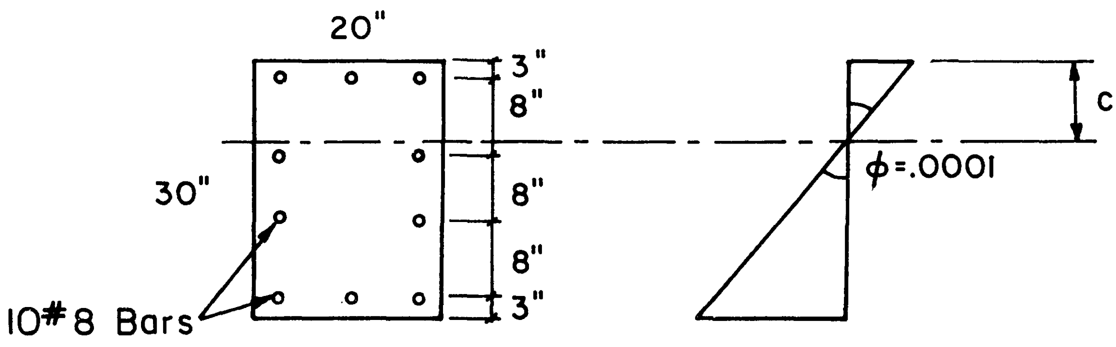


Fig A.2. Beam cross section for example problem.

$$= \frac{s_1 s_2}{s s_1}$$

$$= \epsilon \text{ , the strain at the section considered.} \qquad (1)$$

$$\frac{y}{\rho} = \frac{\sigma}{E}$$

where

$$\sigma = \frac{My}{I}$$

since y is the distance of the strained fiber from the neutral axis. Therefore,

$$\frac{y}{\rho} = \frac{1}{E} \times \frac{My}{I}$$

$$\frac{M}{EI} = \frac{1}{\rho}$$

$$= \frac{\epsilon}{y} \quad \text{from Eq (1)}$$

$$= \tan \phi \text{ , as is obvious from Fig A.1b and c}$$

$$= \phi \text{ , since } \phi \text{ is very small.}$$

Therefore,

$$EI = \frac{M}{\phi} \; . \qquad (2)$$

A.2.1.3. Procedure

The procedure consists of calculating the value of M for an assumed value of ϕ and then computing EI from Eq (2). Then a range of values of ϕ, M, and EI can be obtained.

A.2.1.4. Example

Figure A.2 shows the cross section of a beam subjected to bending moment. The axial load is 200 kips (890 kN), ϕ = .0001 in.$^{-1}$, E_c = 4000 kip/in.^{2} (2800 kN/cm^2), and E_s = 30,000 kip/in.^{2} (20,700 kN/cm^2). Find the values of M and EI.

As the first step, the position of the neutral axis should be determined by trial, such that the net force on the cross section equals the applied load of 200 kips (890 kN). Concrete below the neutral axis will be neglected. A linear stress strain relationship will be assumed here for simplicity.

Trial 1

$$c = 9 \text{ in. } (22.9 \text{ cm})$$

Strains:

At top fiber of concrete:	$.0001 \times 9$	$= .0009$
1st row of bars:	$.0001 \times 6$	$= .0006$
2nd row of bars:	$.0001 \times 2$	$= .0002$
3rd row of bars:	$.0001 \times 10$	$= .001$
4th row of bars:	$.0001 \times 18$	$= .0018$

Forces (stress $\times$ area):

Concrete:	$\dfrac{(.0009 \times 4000)}{2} \times 20 \times 9$	= 324 k comp (1442 kN)
1st row of bars:	$(.0006 \times 30{,}000) \times 3 \times .79$	= 43 k comp (191 kN)
2nd row of bars:	$(.0002 \times 30{,}000) \times 2 \times .79$	= 9 k tension (40 kN)
3rd row of bars:	$(.001 \times 30{,}000) \times 2 \times .79$	= 47 k tension (209 kN)
4th row of bars:	$(.0018 \times 30{,}000) \times 3 \times .79$	= 128 k tension (570 kN)
Net force		= 183 k comp (8144 kN) N.G.

Trial 2

$c = 9.2$ in. (23.4 cm)

Strains:

At top fiber of concrete:	$.0001 \times 9.2$	=	.00092
1st row of bars:	$.0001 \times 6.2$	=	.00062
2nd row of bars:	$.0001 \times 1.8$	=	.00018
3rd row of bars:	$.0001 \times 9.8$	=	.00098
4th row of bars:	$.0001 \times 17.8$	=	.00178

Forces:

Concrete:	$.00092 \times \frac{4000}{2} \times 20 \times 9.2$	= 328 k comp (1459 kN)
1st row of bars:		44 k comp (196 kN)
2nd row of bars:		8 k tension (36 kN)
3rd row of bars:		46 k tension (205 kN)
4th row of bars:		127 k tension (565 kN)
Net force		= 201 k (894 kN) OK

Step 2

Calculate bending moment due to all these forces about the centroidal axis of the cross section. Clockwise moments are taken as positive.

Moment due to compression in concrete $= 338 \left(15 - \frac{9.2 \times 1}{3}\right)$

$= + 4033$ in-kips (455.7 m-kN)

Moment due to compression in row 1 bars $= 44 \times 12 = + 528$ in-kips (455.7 m-kN)

Moment due to tension in row 2 bars $= 8 \times 4 = - 32$ in-kips (- 3.6 m-kN)

Moment due to tension in row 3 bars $= 46 \times 4 = + 1524$ in-kips (172 m-kN)

Moment due to tension in row 4 bars $= 127 \times 12 = + 1524$ in-kips (172 m-kN)

Net Moment M $= + 6237$ in-kips (705 m-kN)

$$EI = \frac{M}{\phi} = \frac{6237}{.0001} = 62{,}370{,}000 \text{ k-in.}^2 \ (179{,}000{,}000 \text{ N-m}^2)$$

The above method, though simple in cases like rectangular cross sections, becomes tedious when cross sections with varying widths are considered. Further, since the actual stress-strain relationship of concrete is a non-linear function, for a circular cross section the computation of forces will involve double integration, one for area and one for the stress. This is not possible by hand calculations.

A.2.2. APPLICATION TO LOAD-DEFLECTION ANALYSIS OF DRILLED SHAFTS OR PILES

In the analyses of drilled shafts or piles subjected to bending moments, the flexural rigidity EI is one of the parameters occurring in the differential equation for the solution of deflections. Typically there will be large variations of bending moment along the length of the column. Consequently there will be variations in EI depending on the moment and axial load if any, and this changed value of EI should be employed in calculations.

A.2.3. PROGRAM CAPABILITIES

Calculation of forces and moments are done in the program by dividing the cross section into a number of horizontal strips and summing. Figures A.3 and A.4 show the stress-strain curves for concrete and steel, respectively, used in the program.

The program gives as output a set of curves for M versus EI values for different axial loads ranging from zero to the axial load capacity for the column. The number of load cases in one run is limited to 10.

Program options allow treatment of the following types of cross sections:

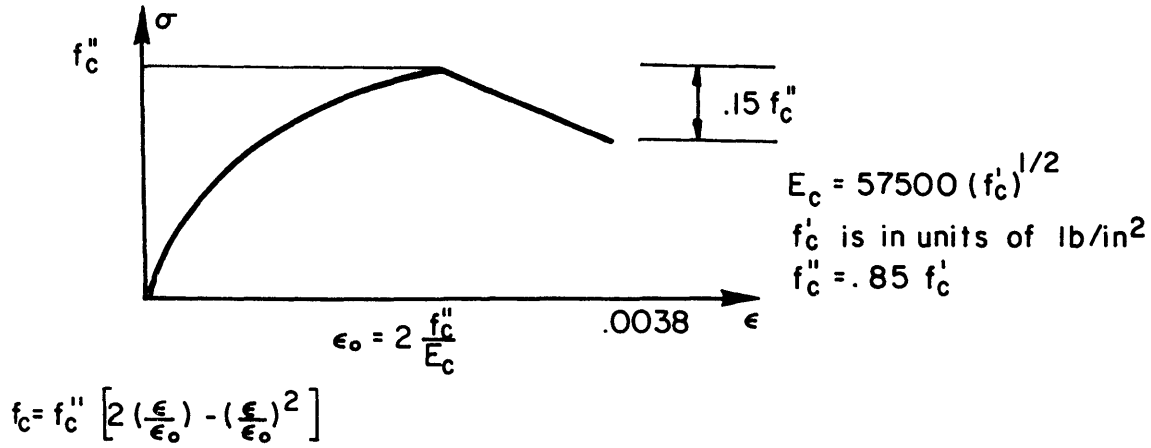


Fig A.3. Stress-strain curve for concrete used by Program PMEI.

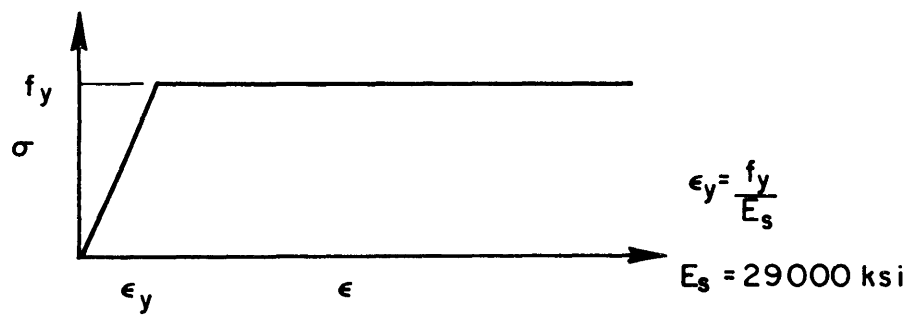


Fig A.4. Stress-strain curve for steel used by Program PMEI.

(1) Square or Rectangular, Reinforced Concrete,

(2) Circular, Reinforced Concrete,

(3) Circular, Reinforced Concrete, with steel tubular shell around the concrete,

(4) Circular, Reinforced Concrete, with steel tubular shell and tubular core, and

(5) Circular, Reinforced Concrete, without shell but with tubular core.

A.2.4. DATA INPUT

The data input form along with the names of variables is shown in Fig A.5. The variables are defined in Table A.1.

A.2.5. PRINTED OUTPUT

The printed output gives a statement of input values, values of E_c and axial load capacity for no moment, and a table each of values of moment, EI, ϕ, maximum strain in the concrete, and depth of neutral axis for each axial load case. The initial value of moment corresponds to a curvature of .00001 in.$^{-1}$. The final value is a step higher than the ultimate failure conditions of concrete. Values at failure can be read off at a concrete strain of 0.0030.

A.2.6. OTHER OUTPUT

If the axial load more than the squash load is applied, the program stops and an error message is printed out.

A.2.7. SAMPLE RUNS

Four example problems were solved. Figures A.6a to A.6d give the cross sections of each example. For each problem three load cases, 0.0 (0.0 kN), 10.0 (44.48 kN), and 1000 k (4448 kN), were used.

(All units in inches and kips)

Card									
	1	10	20	30	40	50	60	70	80
A		ANAME							
	1	5	10						
B	ISHAPE	NP							
	1	10							
C	P	(Repeat NP times)							
	1	10	20	30	40				
D	FC	BARFY	TUBEFY	ES					
	1	10	20	30	40	50			
E	WIDTH	OD	DT	T	TT				
	1	5	10	20					
F	NBARS	NROWS	COVER						
	1	10							
G	AS	(Repeat NROWS times)							
	1	10							
H	XS	(Repeat NROWS times)	Card H required only if ASHAPE is 1, i.e., for rectangular or square cross sections.						

Fig A.5. Data input form for computer program PMEI. A description of variables is given in Table A.1.

TABLE A.1. DETAILED INPUT GUIDE WITH DEFINITIONS OF VARIABLES
(All units in inches and kips)

Card A	(8A10)	ANAME	Alphanumeric description to be printed as title
Card B	(2I5)	ISHAPE	Identification number of the shape of cross section of column/pile 1: Rectangular or square 10: Circular (without shell or core) 20: Circular (with shell but without core) 30: Circular (with shell and core or without shell and with core)
		NP	Number of load cases (axial)
Card C	(F10.2)	P	Axial load. The total number of axial loads per run is limited to 10.
Card D	(4F10.2)	FC	Cylinder strength of concrete
		BARFY	Yield strength of reinforcement
		TUBEFY	Yield strength of shell or core
		ES	Modulus of elasticity of steel
Card E	(5F10.2)	WIDTH	Width of section if rectangular (0.0 if circular)
		OD	Outer diameter, if circular, or depth of section if rectangular
		DT	Outer diameter of core (0.0 if ISHAPE is 1 or 10)
		T	Thickness of shell
		TT	Thickness of core
Card F	(2I5,F10.2)	NBARS	Number of reinforcing bars
		NROWS	Number of rows of reinforcing bars (a number not exceeding 50)
		COVER	Cover of rebar, from center of rebar to outer edge of concrete
Card G	(F10.2)	AS	Area of reinforcement in a row AS(1) is for the top row AS(2) is for the 2nd row from the top, etc. The total number of values should not exceed 50

Note: - In the case of an odd number of bars in a circular cross section the centroidal axis is taken as the diameter passing through one bar. In this case the number of rows will be the same as the number of bars.

Card H	(F10.2)	XS	Distance of row from centroidal axis, starting from top row downwards. Positive for rows above the axis and negative for rows below the axis. The total number of values should not exceed 50.

Note: - Card H is required only in the case of rectangular or square sections.

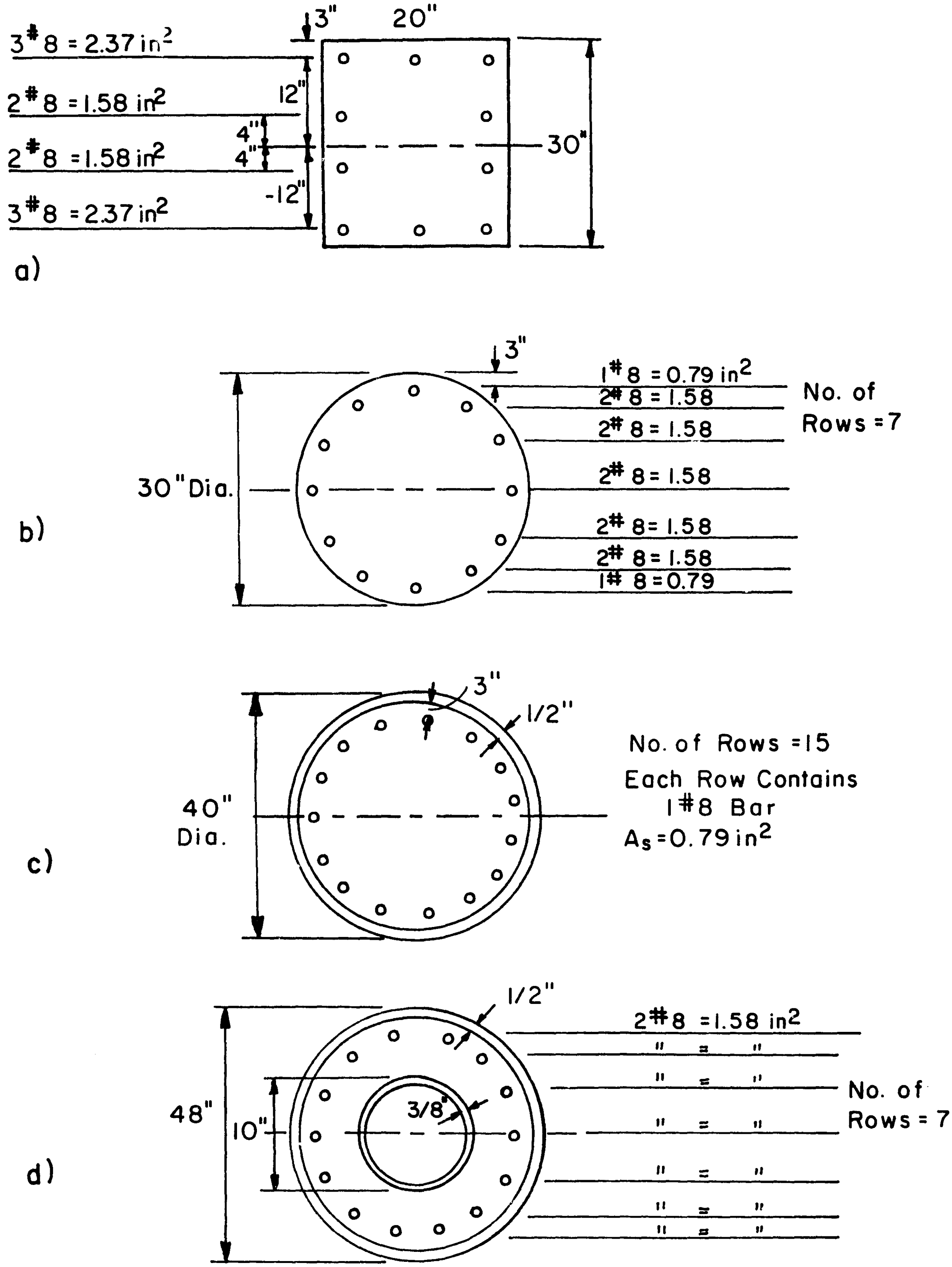


Fig A.6. Concrete column cross sections for example problems.

1 PMEIX RECTANGULAR SECTION P=0.0K

SHAPE : RECTANGULAR

WIDTH	20.00	DEPTH	30.00

NO. OF REBARS	10
ROWS OF REBARS	4
COVER (BAR CENTER TO CONCR EDGE)	3.0

LAYER	AREA	ORDINATE
1	2.37	12.00
2	1.58	4.00
3	1.58	-4.00
4	2.37	-12.00

CONCRETE CYLINDER STRENGTH	4.00KSI
REBARS YIELD STRENGTH	60.00KSI
SHELL/TUBE YIELD STRENGTH	-0.00KSI
MODULUS OF ELAST. OF STEEL	29000.00KSI
MODULUS OF ELAST. OF CONCR	3636.62KSI
SQUASH LOAD CAPACITY	2487.14KPS

1 AXIAL LOAD = 0.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
44.2	44213042.6	.000001	.00001	7.07
220.8	44163492.6	.000005	.00004	7.08
397.0	44114035.8	.000009	.00006	7.10
572.8	44064632.0	.000013	.00009	7.11
748.2	44013949.0	.000017	.00012	7.12
923.2	43963260.8	.000021	.00015	7.14
1097.8	43912526.2	.000025	.00018	7.15
1272.0	43860446.8	.000029	.00021	7.16
1445.7	43808261.0	.000033	.00024	7.18
1619.0	43755926.2	.000037	.00027	7.19
1791.8	43702178.9	.000041	.00030	7.20
1964.2	43648220.7	.000045	.00032	7.22
2136.1	43594008.2	.000049	.00035	7.23
2307.6	43539497.1	.000053	.00038	7.25
3577.8	43106347.8	.000083	.00061	7.36
4616.8	40856722.3	.000113	.00083	7.33
4998.3	34953274.9	.000143	.00100	6.99
5336.8	30848708.4	.000173	.00117	6.74
5436.9	26782949.4	.000203	.00130	6.41
5528.1	23725862.1	.000233	.00143	6.15
5618.7	21363962.7	.000263	.00157	5.97
5701.4	19458644.7	.000293	.00170	5.81
5780.6	17896636.9	.000323	.00184	5.69
5855.8	16588508.2	.000353	.00198	5.60
5918.9	15453981.2	.000383	.00212	5.53
5929.0	14356013.7	.000413	.00224	5.42
5931.6	13389688.2	.000443	.00235	5.29
5934.1	12545585.5	.000473	.00245	5.18
5935.9	11801052.7	.000503	.00256	5.09
5937.5	11139795.2	.000533	.00267	5.01
5943.8	10557323.6	.000563	.00280	4.98

5943.7	10023060.5	.000593	.00291	4.90
5942.9	9539158.7	.000623	.00301	4.84
5942.4	9100220.4	.000653	.00312	4.77
5941.8	8699592.6	.000683	.00322	4.72
5941.1	8332485.2	.000713	.00333	4.67
5940.3	7995069.6	.000743	.00344	4.62
5939.7	7683990.6	.000773	.00354	4.58
5938.9	7395918.6	.000803	.00365	4.55
5938.0	7128420.5	.000833	.00376	4.51
5884.0	6818084.6	.000863	.00388	4.50

1 PHEIX RECTANGULAR SECTION P=10.0 K.

SHAPE : RECTANGULAR

WIDTH	20.00	DEPTH	30.00

NO. OF REBARS	10
ROWS OF REBARS	4
COVER (BAR CENTER TO CONCR EDGE)	3.0

LAYER	AREA	ORDINATE
1	2.37	12.00
2	1.58	4.00
3	1.58	-4.00
4	2.37	-12.00

CONCRETE CYLINDER STRENGTH	4.00KSI
REBARS YIELD STRENGTH	60.00KSI
SHELL/TUBE YIELD STRENGTH	-0.00KSI
MODULUS OF ELAST. OF STEEL	29000.00KSI
MODULUS OF ELAST. OF CONCR	3636.62KSI
SQUASH LOAD CAPACITY	2487.14KPS

1 AXIAL LOAD = 10.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
113.8	113784102.6	.000001	.00002	16.35
298.9	59776892.1	.000005	.00005	9.51
475.4	52821774.5	.000009	.00008	8.51
651.1	50084129.4	.000013	.00011	8.11
826.3	48608699.4	.000017	.00013	7.91
1001.1	47671221.5	.000021	.00016	7.77
1175.4	47015260.3	.000025	.00019	7.69
1349.3	46527200.1	.000029	.00022	7.63
1522.7	46142849.8	.000033	.00025	7.59
1695.7	45829859.2	.000037	.00028	7.56
1868.3	45568473.8	.000041	.00031	7.54
2040.4	45342145.3	.000045	.00034	7.52
2212.0	45143664.2	.000049	.00037	7.51
2383.2	44966063.9	.000053	.00040	7.51
3651.2	43990301.9	.000083	.00062	7.53
4710.9	41689750.8	.000113	.00085	7.49
5089.7	35592525.4	.000143	.00102	7.11
5442.9	31462022.3	.000173	.00119	6.88
5540.1	27291242.7	.000203	.00132	6.52
5634.1	24180856.7	.000233	.00146	6.26
5721.2	21753789.5	.000263	.00159	6.06
5806.5	19817569.3	.000293	.00173	5.92
5884.7	18219037.9	.000323	.00187	5.79
5958.3	16878902.8	.000353	.00201	5.70
6027.6	15737890.9	.000383	.00216	5.63
6037.6	14618931.4	.000413	.00227	5.50
6046.2	13648327.9	.000443	.00240	5.41
6048.0	12786536.6	.000473	.00250	5.30
6049.7	12027200.3	.000503	.00261	5.20
6050.9	11352483.9	.000533	.00272	5.11
6052.0	10749537.2	.000563	.00283	5.03

6090.9	10271304.4	.000593	.00297	5.00
6058.0	9723866.9	.000623	.00308	4.95
6057.2	9275997.3	.000653	.00319	4.88
6056.3	8867236.5	.000683	.00330	4.82
6055.6	8493130.2	.000713	.00340	4.77
6054.8	8149116.1	.000743	.00351	4.72
6054.0	7831871.0	.000773	.00362	4.68
6052.7	7537671.1	.000803	.00372	4.64
6051.9	7265152.0	.000833	.00383	4.60

PMEIX RECTANGULAR SECTION P=1000.0 K.

SHAPE : RECTANGULAR

WIDTH	20.00	DEPTH	30.00

NO. OF REBARS	10
ROWS OF REBARS	4
COVER (BAR CENTER TO CONCR EDGE)	3.0

LAYER	AREA	ORDINATE
1	2.37	12.00
2	1.58	4.00
3	1.58	-4.00
4	2.37	-12.00

CONCRETE CYLINDER STRENGTH	4.00KSI
REBARS YIELD STRENGTH	60.00KSI
SHELL/TUBE YIELD STRENGTH	-0.00KSI
MODULUS OF ELAST. OF STEEL	29000.00KSI
MODULUS OF ELAST. OF CONCR	3636.62KSI
SQUASH LOAD CAPACITY	2487.14KPS

AXIAL LOAD = 1000.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
143.9	143941313.9	.000001	.00048	482.71
719.5	143892074.2	.000005	.00054	108.65
1294.0	143777061.0	.000009	.00060	67.18
1866.8	143596208.0	.000013	.00067	51.28
2436.9	143348949.4	.000017	.00073	42.91
3003.7	143035585.6	.000021	.00079	37.76
3566.4	142654754.9	.000025	.00086	34.30
4124.0	142206637.5	.000029	.00092	31.81
4679.0	141789189.9	.000033	.00099	29.96
5184.2	140114309.6	.000037	.00105	28.47
5617.5	137013403.4	.000041	.00112	27.22
6000.8	133351794.8	.000045	.00118	26.16
6344.0	129468419.0	.000049	.00124	25.24
6656.0	125583988.6	.000053	.00130	24.44
8340.4	100486444.4	.000083	.00171	20.64
9414.2	83311839.9	.000113	.00212	18.80
10129.1	70832542.8	.000143	.00256	17.88
10431.4	60296825.3	.000173	.00304	17.57
10625.7	52343145.1	.000203	.00355	17.50
10636.5	45650078.2	.000233	.00406	17.44

P= 0.0 K. CIRCULAR SECTION. PMETX.

SHAPE : CIRCULAR
DIAMETER 30.00
SHELL THICKNESS -0.00
CORE TUBE O.D. -0.00
CORE TUBE THICKNESS -0.00

NO. OF REBARS 12
ROWS OF REBARS 7
COVER
(BAR CENTER TO CONCR EDGE) 3.0

LAYER	AREA	ORDINATE
1	.79	12.00
2	1.58	10.39
3	1.58	6.00
4	1.58	0.00
5	1.58	-6.00
6	1.58	-10.39
7	.79	-12.00

CONCRETE CYLINDER STRENGTH	4.00KSI
REBARS YIELD STRENGTH	60.00KSI
SHELL/TUBE YIELD STRENGTH	-0.00KSI
MODULUS OF ELAST. OF STEEL	29000.00KSI
MODULUS OF ELAST. OF CONCR	3636.62KSI
SQUASH LOAD CAPACITY	2939.89KPS

AXIAL LOAD = 0.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
41.7	41678266.8	.000001	.00001	8.13
208.1	41625747.5	.000005	.00004	8.15
374.1	41572128.4	.000009	.00007	8.16
539.7	41518668.1	.000013	.00011	8.17
704.9	41465328.8	.000017	.00014	8.18
869.6	41410830.6	.000021	.00017	8.19
1033.9	41355168.2	.000025	.00021	8.21
1197.7	41299555.5	.000029	.00024	8.22
1361.1	41243953.3	.000033	.00027	8.23
1523.9	41187125.6	.000037	.00030	8.24
1686.3	41130251.8	.000041	.00034	8.26
1848.2	41072117.8	.000045	.00037	8.27
2009.7	41013880.1	.000049	.00041	8.28
2170.6	40955497.7	.000053	.00044	8.29
3361.2	40496175.0	.000083	.00070	8.40
4516.9	39972547.2	.000113	.00096	8.50
5120.1	35804951.8	.000143	.00119	8.32
5476.1	31653873.8	.000173	.00140	8.07
5652.4	27844203.6	.000203	.00159	7.83
5817.0	24965774.5	.000233	.00178	7.63
5973.5	22712739.7	.000263	.00197	7.50
6057.1	20672851.7	.000293	.00216	7.36
6089.0	18851498.3	.000323	.00232	7.19
6118.5	17332958.2	.000353	.00249	7.06
6149.7	16056659.5	.000383	.00268	6.99
6173.1	14947048.4	.000413	.00285	6.89
6195.6	13985512.5	.000443	.00302	6.81
6216.9	13143593.1	.000473	.00319	6.74
6237.1	12399840.1	.000503	.00336	6.69
6256.7	11738730.1	.000533	.00354	6.64
6276.0	11147349.1	.000563	.00372	6.60
6290.7	10608309.6	.000593	.00390	6.57

1 P= 10.0 K. CIRCULAR SECTION. PMFIX.

SHAPE : CIRCULAR
DIAMETER 30.00
SHELL THICKNESS -0.00
CORE TUBE O.D. -0.00
CORE TUBE THICKNESS -0.00

NO. OF REBARS 12
ROWS OF REBARS 7
COVER
(BAR CENTER TO CONCR EDGE) 3.0

LAYER	AREA	ORDINATE
1	.79	12.00
2	1.58	10.39
3	1.58	6.00
4	1.58	0.00
5	1.58	-6.00
6	1.58	-10.39
7	.79	-12.00

CONCRETE CYLINDER STRENGTH	4.00KSI
REBARS YIELD STRENGTH	60.00KSI
SHELL/TUBE YIELD STRENGTH	-0.00KSI
MODULUS OF ELAST. OF STEEL	29000.00KSI
MODULUS OF ELAST. OF CONCR	3636.62KSI
SQUASH LOAD CAPACITY	2939.89KPS

1 AXIAL LOAD = 10.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
101.3	101251123.4	.000001	.00002	16.13
275.7	55140961.0	.000005	.00005	10.28
442.1	49117147.3	.000009	.00008	9.41
607.5	46728842.0	.000013	.00012	9.05
772.5	45441105.6	.000017	.00015	8.87
937.0	44618252.9	.000021	.00018	8.76
1101.0	44039503.2	.000025	.00022	8.68
1264.6	43606053.2	.000029	.00025	8.63
1427.7	43263211.9	.000033	.00028	8.59
1590.3	42980867.7	.000037	.00032	8.57
1752.4	42742439.5	.000041	.00035	8.55
1914.2	42536859.9	.000045	.00038	8.54
2075.3	42353429.9	.000049	.00042	8.53
2236.0	42189121.4	.000053	.00045	8.52
3424.6	41259872.9	.000083	.00071	8.54
4577.8	40511181.2	.000113	.00097	8.61
5197.2	36344020.2	.000143	.00121	8.43
5563.3	32157895.4	.000173	.00142	8.18
5739.7	28274147.8	.000203	.00161	7.93
5903.0	25334694.8	.000233	.00180	7.73
6057.8	23033342.8	.000263	.00200	7.59
6152.1	20997023.9	.000293	.00219	7.46

6183.0	19142501.2	.000323	.00235	7.29
6211.7	17596915.9	.000353	.00252	7.15
6238.0	16287313.1	.000383	.00270	7.04
6266.4	15172855.0	.000413	.00288	6.98
6288.3	14194776.4	.000443	.00306	6.90
6308.9	13338028.1	.000473	.00323	6.83
6328.7	12581992.2	.000503	.00340	6.77
6347.9	11909816.2	.000533	.00358	6.72
6366.5	11308166.3	.000563	.00376	6.68
6377.6	10754787.3	.000593	.00395	6.66

1 P= 1000.0 K. CIRCULAR SECTION. PMFIX.

SHAPE : CIRCULAR
DIAMETER 30.00
SHELL THICKNESS -0.00
CORE TUBE O.D. -0.00
CORE TUBE THICKNESS -0.00

NO. OF REBARS 12
ROWS OF REBARS 7
COVER
(BAR CENTER TO CONCR EDGE) 3.0

LAYER	AREA	ORDINATE
1	.79	12.00
2	1.58	10.39
3	1.58	6.00
4	1.58	0.00
5	1.58	-6.00
6	1.58	-10.39
7	.79	-12.00

CONCRETE CYLINDER STRENGTH	4.00KSI
REBARS YIELD STRENGTH	60.00KSI
SHELL/TUBE YIELD STRENGTH	-0.00KSI
MODULUS OF ELAST. OF STEEL	29000.00KSI
MODULUS OF ELAST. OF CONCR	3636.62KSI
SQUASH LOAD CAPACITY	2939.89KPS

1 AXIAL LOAD = 1000.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
134.7	134722916.1	.000001	.00040	402.51
673.5	134691807.2	.000005	.00046	92.58
1211.6	134618931.1	.000009	.00052	58.21
1748.6	134504526.2	.000013	.00059	45.03
2283.9	134348419.5	.000017	.00065	38.08
2817.2	134150025.4	.000021	.00071	33.80
3347.7	133909787.6	.000025	.00077	30.92
3869.7	133436769.4	.000029	.00084	28.84
4342.3	131583987.8	.000033	.00090	27.24
4763.8	128750270.3	.000037	.00096	25.95
5136.9	125290544.8	.000041	.00102	24.87
5475.1	121669952.4	.000045	.00108	23.96
5778.1	117919778.5	.000049	.00114	23.17
6063.5	114405710.4	.000053	.00119	22.49
7668.3	92388681.5	.000083	.00160	19.23
8796.7	77846912.6	.000113	.00199	17.59
9655.0	67517761.3	.000143	.00238	16.67
10311.3	59603136.1	.000173	.00279	16.16
10697.5	52697251.3	.000203	.00323	15.92
10841.2	46528567.5	.000233	.00366	15.72
10878.5	41363199.9	.000263	.00409	15.56

P= 0.0 K. CIRC. SECT. W/STEEL SHELL. PMEIX.

```
SHAPE : CIRCULAR
DIAMETER                 40.00
SHELL THICKNESS            .50
CORE TUBE O.D.           -0.00
CORE TUBE THICKNESS      -0.00

NO. OF REBARS                  15
ROWS OF REBARS                 15
COVER
(BAR CENTER TO CONCR EDGE)    3.0
```

LAYER	AREA	ORDINATE
1	.79	16.41
2	.79	15.69
3	.79	14.29
4	.79	12.26
5	.79	9.70
6	.79	6.71
7	.79	3.43
8	.79	0.00
9	.79	-3.43
10	.79	-6.71
11	.79	-9.70
12	.79	-12.26
13	.79	-14.29
14	.79	-15.69
15	.79	-16.41

```
CONCRETE CYLINDER STRENGTH          4.00KSI
REBARS YIELD STRENGTH              60.00KSI
SHELL/TUBE YIELD STRENGTH          36.00KSI
MODULUS OF ELAST. OF STEEL      29000.00KSI
MODULUS OF ELAST. OF CONCR       3636.62KSI
SQUASH LOAD CAPACITY             6966.01KPS
```

AXIAL LOAD = 0.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
542.8	542753813.6	.000001	.00002	15.57
2709.1	541821847.0	.000005	.00008	15.59
4867.9	540876047.5	.000009	.00014	15.62
7019.0	539925802.3	.000013	.00020	15.64
9162.3	538961248.5	.000017	.00026	15.67
11297.8	537991425.2	.000021	.00032	15.69
13425.3	537011273.5	.000025	.00038	15.72
15544.5	536016078.4	.000029	.00044	15.75
17655.5	535014347.8	.000033	.00050	15.77
19758.0	534001272.0	.000037	.00057	15.80
21852.0	532976504.7	.000041	.00063	15.83
23937.3	531939695.8	.000045	.00069	15.86
26013.6	530890491.8	.000049	.00075	15.89
28021.2	528702326.8	.000053	.00082	15.90
36086.0	434771517.6	.000083	.00119	14.88
39826.8	352449362.9	.000113	.00154	14.17
41419.4	289645982.1	.000143	.00188	13.64
42323.1	244642174.5	.000173	.00221	13.27
42897.5	211317665.4	.000203	.00253	12.96
43245.2	185601645.9	.000233	.00285	12.72
43482.6	165333141.7	.000263	.00318	12.60
43619.5	148872102.9	.000293	.00351	12.48
43689.3	135260999.6	.000323	.00384	12.39

P= 10.0 K. CIRC. SECT. W/STEEL SHELL. PHEIX.

```
SHAPE : CIRCULAR
DIAMETER              40.00
SHELL THICKNESS         .50
CORE TUBE O.D.        -0.00
CORE TUBE THICKNESS   -0.00

NO. OF REBARS                 15
ROWS OF REBARS                15
COVER
(BAR CENTER TO CONCR EDGE)   3.0
```

LAYER	AREA	ORDINATE
1	.79	16.41
2	.79	15.69
3	.79	14.29
4	.79	12.26
5	.79	9.70
6	.79	6.71
7	.79	3.43
8	.79	0.00
9	.79	-3.43
10	.79	-6.71
11	.79	-9.70
12	.79	-12.26
13	.79	-14.29
14	.79	-15.69
15	.79	-16.41

```
CONCRETE CYLINDER STRENGTH         4.00KSI
REBARS YIELD STRENGTH             60.00KSI
SHELL/TUBE YIELD STRENGTH         36.00KSI
MODULUS OF ELAST. OF STEEL     29000.00KSI
MODULUS OF ELAST. OF CONCR      3636.62KSI
SQUASH LOAD CAPACITY            6966.01KPS
```

AXIAL LOAD = 10.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
586.6	586627706.1	.000001	.00002	18.13
2752.9	550572733.0	.000005	.00008	16.12
4911.2	545683934.1	.000009	.00014	15.92
7061.8	543212748.4	.000013	.00020	15.85
9204.6	541446540.8	.000017	.00026	15.83
11339.5	539975334.2	.000021	.00032	15.83
13466.3	538653717.0	.000025	.00038	15.83
15585.1	537416232.0	.000029	.00045	15.85
17695.5	536227618.7	.000033	.00051	15.86
19797.3	535063434.5	.000037	.00057	15.88
21890.8	533921510.0	.000041	.00063	15.90
23975.4	532787606.3	.000045	.00069	15.92
26051.4	531660317.9	.000049	.00076	15.95
28068.4	529591893.1	.000053	.00082	15.96
36166.4	435740255.0	.000083	.00120	14.93
39906.7	353156840.4	.000113	.00155	14.22
41505.3	290247128.9	.000143	.00189	13.69
42411.1	245150664.9	.000173	.00222	13.31
42986.7	211757379.5	.000203	.00254	13.01
43333.1	185979026.0	.000233	.00286	12.76
43566.9	165653566.3	.000263	.00320	12.65
43706.7	149169736.0	.000293	.00352	12.53
43773.3	135520945.2	.000323	.00386	12.44

P= 1000.0 K. CIRC. SECT. W/STEEL SHELL. PMEIX.

SHAPE : CIRCULAR
DIAMETER 40.00
SHELL THICKNESS .50
CORE TUBE O.D. -0.00
CORE TUBE THICKNESS -0.00

NO. OF REBARS 15
ROWS OF REBARS 15
COVER
(BAR CENTER TO CONCR EDGE) 3.0

LAYER	AREA	ORDINATE
1	.79	16.41
2	.79	15.69
3	.79	14.29
4	.79	12.26
5	.79	9.70
6	.79	6.71
7	.79	3.43
8	.79	0.00
9	.79	-3.43
10	.79	-6.71
11	.79	-9.70
12	.79	-12.26
13	.79	-14.29
14	.79	-15.69
15	.79	-16.41

CONCRETE CYLINDER STRENGTH 4.00KSI
REBARS YIELD STRENGTH 60.00KSI
SHELL/TUBE YIELD STRENGTH 36.00KSI
MODULUS OF ELAST. OF STEEL 29000.00KSI
MODULUS OF ELAST. OF CONCR 3636.62KSI
SQUASH LOAD CAPACITY 6966.01KPS

AXIAL LOAD = 1000.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
791.9	791941901.7	.000001	.00018	177.62
3959.2	791847187.2	.000005	.00026	51.61
7116.8	790760776.0	.000009	.00033	37.67
9922.2	763243144.4	.000013	.00041	32.07
12392.1	728947871.8	.000017	.00048	28.88
14695.8	699802035.8	.000021	.00055	26.79
16916.3	676653687.6	.000025	.00062	25.32
19069.3	657561237.3	.000029	.00069	24.22
21189.4	642102110.1	.000033	.00075	23.37
23282.7	629261964.4	.000037	.00082	22.69
25354.7	618407347.6	.000041	.00089	22.14
27409.7	609104006.0	.000045	.00095	21.69
29450.9	601039337.6	.000049	.00102	21.32
31468.0	593735947.3	.000053	.00109	20.99
41739.5	502885245.6	.000083	.00158	19.51
45843.8	405697534.5	.000113	.00205	18.64
47912.0	335048701.9	.000143	.00251	18.04
48848.6	282361605.6	.000173	.00297	17.66
49261.2	242666252.0	.000203	.00343	17.41
49468.9	212312707.3	.000233	.00391	17.29

P= 0.0 K. CIRC. SECT. W/STEEL SHELL A INNER TUBE. PME

```
SHAPE : CIRCULAR
DIAMETER                   48.00
SHELL THICKNESS              .50
CORE TUBE O.D.             10.00
CORE TUBE THICKNESS          .38

NO. OF REBARS                      14
ROWS OF REBARS                      7
COVER
(BAR CENTER TO CONCR EDGE)        3.0

LAYER            AREA        ORDINATE
    1            1.58          19.99

    2            1.58          16.03

    3            1.58           8.89

    4            1.58           0.00

    5            1.58          -8.89

    6            1.58         -16.03

    7            1.58         -19.99

CONCRETE CYLINDER STRENGTH           4.00KSI
REBARS YIELD STRENGTH               60.00KSI
SHELL/TUBE YIELD STRENGTH           36.00KSI
MODULUS OF ELAST. OF STEEL       29000.00KSI
MODULUS OF ELAST. OF CONCR        3636.62KSI
SQUASH LOAD CAPACITY              9357.29KPS
```

AXIAL LOAD = 0.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
1002.3	1002256689.5	.000001	.00002	18.42
5008.1	1000028450.9	.000005	.00009	18.45
8980.0	997779617.6	.000009	.00016	18.49
12941.5	995498869.7	.000013	.00023	18.53
16884.3	993194930.6	.000017	.00031	18.56
20808.0	990856871.5	.000021	.00038	18.60
24712.3	988492960.0	.000025	.00045	18.64
28596.9	986101623.5	.000029	.00053	18.68
32461.2	983672527.3	.000033	.00060	18.72
36304.6	981200719.2	.000037	.00068	18.76
40127.3	978713826.1	.000041	.00075	18.80
43567.0	968155129.2	.000045	.00082	18.78
46106.2	948942860.9	.000049	.00089	18.63
48257.4	910516761.7	.000053	.00095	18.45
58940.5	710126324.8	.000083	.00139	17.26
63313.5	560296199.7	.000113	.00183	16.67
65398.5	457332477.9	.000143	.00224	16.20
66539.4	384620894.3	.000173	.00266	15.90
67158.3	330829066.5	.000203	.00309	15.71
67566.5	289985105.7	.000233	.00352	15.60
67786.4	257743101.5	.000263	.00395	15.50

P= 10.0 K. CIRC. SECT. W/STEEL SHELL & INNER TUBE. PME

```
SHAPE : CIRCULAR
DIAMETER                    48.00
SHELL THICKNESS               .50
CORE TUBE O.D.              10.00
CORE TUBE THICKNESS           .38

NO. OF REBARS                       14
ROWS OF REBARS                       7
COVER
(BAR CENTER TO CONCR EDGE)         3.0

LAYER            AREA         ORDINATE
    1            1.58           19.99

    2            1.58           16.03

    3            1.58            8.89

    4            1.58            0.00

    5            1.58           -8.89

    6            1.58          -16.03

    7            1.58          -19.99

CONCRETE CYLINDER STRENGTH          4.00KSI
REBARS YIELD STRENGTH              60.00KSI
SHELL/TUBE YIELD STRENGTH          36.00KSI
MODULUS OF ELAST. OF STEEL      29000.00KSI
MODULUS OF ELAST. OF CONCR       3636.62KSI
SQUASH LOAD CAPACITY             9357.29KPS
```

AXIAL LOAD = 10.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
1057.8	1057797260.4	.000001	.00002	20.31
5055.1	1011028234.2	.000005	.00009	18.85
9034.3	1003813963.2	.000009	.00016	18.71
12995.1	999620498.4	.000013	.00024	18.68
16937.0	996296112.1	.000017	.00031	18.68
20860.0	993335083.2	.000021	.00038	18.70
24763.5	990538168.5	.000025	.00046	18.72
28647.1	987829676.6	.000029	.00053	18.75
32510.5	985166052.8	.000033	.00060	18.78
36353.4	982524166.7	.000037	.00068	18.82
40174.8	979874297.2	.000041	.00075	18.85
43631.9	969597236.4	.000045	.00083	18.83
46181.2	942472679.3	.000049	.00089	18.68
48339.0	912057069.0	.000053	.00095	18.50
59029.0	711192202.0	.000083	.00139	17.30
63406.4	561118767.6	.000113	.00183	16.71
65496.9	458020100.7	.000143	.00225	16.24
66636.0	385179123.2	.000173	.00267	15.94
67250.7	331284061.9	.000203	.00310	15.76
67656.9	290372797.1	.000233	.00353	15.64
67889.4	258100454.0	.000263	.00396	15.55

P= 1000.0 K. CIRC. SECT. W/STEEL SHELL & INNER TUBE. PMI

SHAPE : CIRCULAR
DIAMETER 48.00
SHELL THICKNESS .50
CORE TUBE O.D. 10.00
CORE TUBE THICKNESS .38

NO. OF REBARS 14
ROWS OF REBARS 7
COVER
(BAR CENTER TO CONCR EDGE) 3.0

LAYER	AREA	ORDINATE
1	1.58	19.99
2	1.58	16.03
3	1.58	8.89
4	1.58	0.00
5	1.58	-8.89
6	1.58	-16.03
7	1.58	-19.99

CONCRETE CYLINDER STRENGTH 4.00KSI
REBARS YIELD STRENGTH 60.00KSI
SHELL/TUBE YIELD STRENGTH 36.00KSI
MODULUS OF ELAST. OF STEEL 29000.00KSI
MODULUS OF ELAST. OF CONCR 3636.62KSI
SQUASH LOAD CAPACITY 9357.29KPS

1 AXIAL LOAD = 1000.00 KIPS

MOMENT IN KIPS	EI KIP-IN2	PHI	MAX STR IN/IN	N AXIS IN
1555.1	1555122010.6	.000001	.00013	134.53
7775.0	1555004922.2	.000005	.00023	46.23
13037.4	1448600364.7	.000009	.00032	35.86
17430.6	1340818167.5	.000013	.00040	31.35
21540.7	1267101609.6	.000017	.00048	28.80
25526.4	1215543540.2	.000021	.00056	27.15
29426.8	1177070360.5	.000025	.00064	25.99
33280.3	1147596211.4	.000029	.00071	25.14
37097.6	1124168217.1	.000033	.00079	24.50
40883.6	1104961604.5	.000037	.00087	24.00
44631.1	1088562897.3	.000041	.00095	23.59
48352.8	1074506363.8	.000045	.00102	23.26
52054.5	1062337494.2	.000049	.00110	23.00
55201.0	1041528943.2	.000053	.00118	22.70
66205.7	797659427.8	.000083	.00171	21.07
70885.5	627305466.5	.000113	.00223	20.26
73113.6	511283953.2	.000143	.00277	19.84
74126.8	428478511.1	.000173	.00330	19.57
74396.0	366482735.8	.000203	.00384	19.39

```
      PROGRAM PMEIX (INPUT,OUTPUT)
C
C     THIS PROGRAM GIVES A SET OF VALUES OF -ET- FOR VARIOUS VALUES
C     OF MOMENTS COMBINED WITH AXIAL LOADS RANGING FROM ZERO TO ANY
C     LOAD LESS THAN THE SQUASH LOAD.
C
C     THE PROGRAM CAN TREAT SQUARE, RECTANGULAR OR CIRCULAR SHAPES
C     OF CONCRETE WITH REINFORCEMENT OF ANY GRADE.. CIRCULAR SHAPES
C     CAN BE SPECIFIED AS WITH OR WITHOUT A STEEL SHELL, WITH OR
C     WITHOUT A TUBULAR STEEL CORE (NO CONCRETE IN CORE).
C
C     ---- ALL UNITS SHALL BE INPUT IN INCHES AND KIPS ----
C
C     INPUT FORMATS ARE AS FOLLOWS:
C
C     1. FORMAT(8A10). THIS LINE IS FOR IDENTIFICATION OF THE
C     PROBLEM.  WILL READ ANY CHARACTER IN THE FIRST 80 COLUMNS.
C
C     2. FORMAT(2I5). THIS LINE IS FOR SHAPE IDENTIFICATION AND
C     NUMBER OF LOAD CASES.
C            SHAPE IDENTIFICATION:
C                1=RECTANGULAR
C               10=CIRCULAR WITHOUT SHELL OR CORE
C               20=CIRCULAR WITH SHELL, WITHOUT CORE
C               30=CIRCULAR WITH SHELL AND CORE
C     THE NUMBER OF LOAD CASES SHALL NOT EXCEED TEN (10).
C
C     3. FORMAT(F10.2). THIS LINE IS FOR THE APPLIED AXIAL LOADS.
C     ONE (1) LINE FOR EACH LOAD CASE.
C
C     4. FORMAT(4F10.2). THIS LINE IS FOR THE COMPRESSIVE STRENGTH
C     OF THE CONCRETE, YIELD STRENGTH OF THE REINFORCEMENT, YIELD
C     STRENGTH OF THE SHELL OR CORE STEEL, AND THE MODULUS OF
C     ELASTICITY OF THE STEEL.
C
C     5. FORMAT(5F10.2). THIS LINE IS FOR THE WIDTH OF THE SECTION
C     (SPECIFY AS 0.0 IF CIRCULAR), DEPTH OF THE SECTION (EXTERNAL
C     DIAMETER IF CIRCULAR), EXTERNAL DIAMETER OF THE INNER TUBE,
C     THICKNESS OF THE OUTER SHELL, AND THE THICKNESS OF THE INNER
C     TUBE
C
C     6. FORMAT(2I5,F10.2). THIS LINE IS FOR THE NUMBER OF REBARS,
C     THE NUMBER OF ROWS OF REBARS, AND THE CONCRETE COVER FROM THE
C     CENTER OF THE REBAR TO THE EDGE OF THE CONCRETE.
C
C     7. FORMAT(F10.2)  THIS LINE IS FOR THE AREA OF REINFORCEMENT
C     IN EACH ROW, STARTING FROM THE TOP ROW OF THE SECTION.
C     ONE LINE FOR EACH ROW OF REBAR.  THE NUMBER OF ROWS SHALL
C     NOT EXCEED FIFTY (50).
C
C     8. FORMAT(F10.2). THIS LINE IS REQUIRED ONLY IF THE SECTION
C     IS RECTANGULAR OR SQUARE.  IT IS FOR THE DISTANCE FROM THE
C     CENTROIDAL AXIS TO EACH ROW OF REBAR, STARTING FROM THE TOP
C     ROW.  VALUES ARE POSITIVE (+) IF THE ROW IS ABOVE THE AXIS,
C     AND NEGATIVE (-) IF THE ROW IS BELOW THE AXIS.
C     ONE LINE FOR EACH ROW.
C
C
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
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```
      COMMON/FOUR/KA,II,KK,PCAP,RCS,ROS,RT,RTS,DIF,ASH(100),IJ,PCT,NBARS
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/SIX/X(100),CSN(100),CSS(100),ACC(100),ATOT(100),SSH(100)
      COMMON/SEVEN/RSSH(100),XX(100),RXX(100),AM(50),XXS(100),RXXS(100)
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
      COMMON/NINE/RASH(100),RATOT(100),RTOT(100),RTACC(100),TOT(100)
      DIMENSION ANAME(8)
      DIMENSION DDNA(2)
      REAL NDNA
      READ 20,(ANAME(I),I=1,8)
      READ 1,ISHAPE,NP
      READ 2,(P(J),J=1,NP)
      READ 3,FC,BARFY,TUBEFY,ES
      READ 4,WIDTH,OD,DT,T,TT
      PRINT 22,(ANAME(I),I=1,8)
      IF (ISHAPE.EQ.1) GOTO 18
      PRINT 6,OD,T,DT,TT
      GOTO 19
   18 PRINT 7,WIDTH,OD
   19 CALL SETUP
      PRINT 5,NBARS
      PRINT 8,NROWS
      PRINT 11,COVER
      PRINT 9
      PRINT 10,(JJ,AS(JJ),XS(JJ),JJ=1,NROWS)
      PRINT 12,FC
      PRINT 13,BARFY
      PRINT 14,TUBEFY
      PRINT 15,ES
      PRINT 16,EC
      PRINT 17,PCAP
      DO 500 J=1,NP
      IF (PCAP-P(J)) 430,26,26
   26 PRINT 30,P(J)
      PRINT 21
      PRINT 23
      PHI=.000001
   35 NSWTCH=0
      DDNA(1)=0.0
      DDNA(2)=OD
      DNA=OD
   40 CALL CSTRESS
      PCON=0.0
      IF (DNA-OD) 33,33,55
   33 K=INT(DNA/DEL)
      GO TO 481
   55 K=60
  481 CONTINUE
      DO 90 I=1,K
      XX(I)=X(I)+RO-DNA
      IF (ISHAPE.EQ.1) GOTO 85
      XXS(I)=XX(I)*XX(I)
      IF (RINT.GE.ABS(XX(I))) GOTO 47
      ACC(I)=0.0
      GOTO 44
   47 ACC(I)=2.*SQRT(RINTS-XXS(I))*DEL
   44 PCON=PCON+CSS(I)*ACC(I)
      GOTO 90
   85 PCON=PCON+AC*CSS(I)
   90 CONTINUE
      CALL STEELP
      PTOT=PCON+PSTEEL
```

```
      IF (ISHAPE.EQ.1) GOTO 86
      IF (ISHAPE.EQ.10) GOTO 86
      CALL SHELLP
      PTOT=PTOT+PSHELL
      IF (ISHAPE.EQ.20) GOTO 86
      CALL COREP
      PTOT=PTOT+PCORE
   86 IF(PTOT .GT. P(J)) GOTO 89
      DDNA(1)=DNA
      IF(NSWTCH .EQ. 1)GOTO 92
      DDNA(2)=5.0*DNA
      GOTO 92
   89 DDNA(2)=DNA
      NSWTCH=1
   92 NDNA=(DDNA(1)+DDNA(2))/2.0
      IF(ABS(DNA-NDNA) .LE. .0001) GOTO 50
      DNA=NDNA
      GOTO 40
   50 AMCON=0.0
      DO 100 I=1,K
      IF(ISHAPE.EQ.1) GOTO 65
      AMCON=AMCON+CSS(I)*ACC(I)*(XX(I))
      GOTO 100
   65 AMCON=AMCON+AC*CSS(I)*(XX(I))
  100 CONTINUE
      AMSTEEL=0.0
      DO 200 I=1,NROWS
  200 AMSTEEL=AMSTEEL+SSS(I)*AS(I)*XS(I)
      AMTOT=AMCON+AMSTEEL
      IF (ISHAPE.EQ.1) GOTO 110
      IF (ISHAPE.EQ.10) GOTO 110
      CALL SHELLM
      AMTOT=AMTOT+AMSHELL
      IF (ISHAPE.EQ.20) GOTO 110
      CALL COREM
      AMTOT=AMTOT+AMCORE
  110 EI=AMTOT/PHI
      CSNMAX=PHI*(DNA-T)
      PRINT 400,AMTOT,EI,PHI,CSNMAX,DNA
      IF (CSNMAX.GT..0038) GOTO 500
      GOTO 450
  430 PRINT 440
      GOTO 500
  450 IF (PHI.GT..00005) GOTO 460
      PHI=PHI+.000004
      GOTO 35
  460 PHI=PHI+.00003
      GOTO 35
  500 CONTINUE
    1 FORMAT (2I5)
    2 FORMAT (F10.2)
    3 FORMAT (4F10.2)
    4 FORMAT (5F10.2)
    5 FORMAT(5X,*NO. OF REBARS                    *,I5)
    6 FORMAT (5X,*SHAPE : CIRCULAR*/,5X,*DIAMETER  *,10X,F5.2,/,5X,
     **SHELL THICKNESS      *,F5.2,/,5X,*CORE TUBE O.D.        *,
     *F5.2/,5X,*CORE TUBE THICKNESS *,F5.2/)
    7 FORMAT (5X,*SHAPE : RECTANGULAR*,/,5X,*WIDTH                          *,
     *F5.2,5X,*DEPTH                    *,F5.2/)
    8 FORMAT (5X,*ROWS OF REBARS              *,I5)
    9 FORMAT(5X,*LAYER*,11X,*AREA*,7X,*ORDINATE*)
   10 FORMAT(5X,I5,5X,F10.2,5X,F10.2/)
```

```
 11   FORMAT (5X,*COVER*,/,5X,*(BAR CENTER TO CONCR EDGE)*,
     *3X,F3.1/)
 12      FORMAT (5X,*CONCRETE CYLINDER STRENGTH    *,F10.2,*KSI*)
 13   FORMAT (5X,*REBARS YIELD STRENGTH           *,F10.2,*KSI*)
 14   FORMAT (5X,*SHELL/TUBE YIELD STRENGTH*,5X,F10.2,*KSI*)
 15   FORMAT (5X,*MODULUS OF ELAST. OF STEEL    *,F10.2,*KSI*)
 16   FORMAT (5X,*MODULUS OF ELAST. OF CONCR    *,F10.2,*KSI*)
 17   FORMAT (5X,*SQUASH LOAD CAPACITY          *,F10.2,*KPS*//)
 20   FORMAT(8A10)
 22   FORMAT(*1*,32X,8A10)
 30   FORMAT(*1*,4X,*AXIAL LOAD = *,F10.2,2X,*KIPS*)
 21   FORMAT(/13X,*MOMENT        EI          PHI    MAX STR   N AXIS*)
 23   FORMAT(13X,*IN KIPS     KIP-IN2              IN/IN       IN*)
 400  FORMAT(10X,F9.1,2X,F12.1,2X,F7.6,2X,F7.5,2X,F6.2)
 440  FORMAT(5X,*APPLIED LOAD MORE THAN SQUASH LOAD*)
      STOP
      END
      SUBROUTINE SETUP
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,PINTS,DNA,C,F,G,TUBEFY
      COMMON/FOUR/KA,II,KK,PCAP,RCS,ROS,RT,RTS,DIF,ASH(100),IJ,PCT,NBARS
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/SIX/X(100),CSN(100),CSS(100),ACC(100),ATOT(100),SSH(100)

      COMMON/SEVEN/RSSH(100),XX(100),RXX(100),AM(50),XXS(100),RXXS(100)
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
      COMMON/NINE/RASH(100),RATOT(100),RTOT(100),RTACC(100),TOT(100)
      FACT=.7854
      EC=57.5*SQRT(FC*1000.)
      STR=1.7*FC/EC
      FFC=FC
      DEL=OD/60.
      RO=.5*OD
      READ 1,NBARS,NROWS,COVER
      READ 2,(AS(JJ),JJ=1,NROWS)
      IF(ISHAPE.EQ.1) GOTO 300
      ANBARS=NBARS
      LA=INT(ANBARS/2.)
      LB=2*LA
      NA=INT(ANBARS/4.)
      NB=4*NA
      ANGLE=6.2832/ANBARS
      RS=RO-COVER-T
      DO 200 JJ=1,NROWS
      AJJ=JJ
      IF ( ANBARS-LB ) 7,6,7
 7    AM(JJ)=(ANBARS+1.)/4.-AJJ/2.
      GOTO 200
   6 IF(NBARS.EQ.NB) GOTO 100
      AM(JJ)=ANBARS/4.-AJJ+.5
      GOTO 200
 100  AM(JJ)=ANBARS/4.-AJJ+1.
 200  XS(JJ)=RS*SIN(ANGLE*AM(JJ))
      GOTO 4
 300  READ 3,(XS(JJ),JJ=1,NROWS)
 4    ARS=0.0
      DO 5 I=1,NROWS
 5    ARS=ARS+AS(I)
      IF(ISHAPE.EQ.1)GOTO 101
      D=OD-T-T
      ROS=RO*RO
```

```fortran
      RINT=.5*D
      RINTS=RINT*RINT
      ARC=FACT*D*D-ARS
      A=FACT*((OD**2)-(D**2))
      IF(ISHAPE.EQ.10) GOTO 103
      IF(ISHAPE .EQ. 20) GOTO 103
      RT=.5*DT
      DC=DT-TT-TT
      RC=.5*DC
      RTS=RT*RT
      RCS=RC*RC
      Z=RO+RT
      ATUBE=FACT*((DT**2)-(DC**2))
      AIN=FACT*DT*DT
      PCAP=FFC*(ARC-AIN)+BARFY*ARS+TUBEFY*(A+ATUBE)
      DIF=RO-RT
      GOTO 104
  103 PCAP=FFC*ARC+BARFY*ARS+TUBEFY*A
      GOTO 104
  101 AC=WIDTH*DEL
      ARC=WIDTH*OD-ARS
      PCAP=FFC*ARC+BARFY*ARS
    1 FORMAT(2I5,F10.2)
    2 FORMAT(F10.2)
    3 FORMAT(F10.2)
  104 RETURN
      END
      SUBROUTINE CSTRESS
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/SIX/X(100),CSN(100),CSS(100),ACC(100),ATOT(100),SSH(100)
      N=INT(DNA/DEL)
      C=DNA-(FLOAT(N))*DEL
      F=DEL-C
      K=60
      IF(DNA-OD) 10,10,20
   10 X(1)=C+.5*DEL
      GOTO 60
   20 X(1)=(DNA-OD)+.5*DEL
   60 DO 90 I=1,K
      IF (I.GT.1) X(I)=X(1)+(FLOAT(I-1))*DEL
      CSN(I)=X(I)*PHI
      IF (CSN(I)-STR) 70,70,80
   70 CSS(I)=FFC*(2.*CSN(I)/STR-(CSN(I)/STR)**2)
      GOTO 90
   80 CSS(I)=FFC*(.85+.15*(.0038-CSN(I))/(.0038-STR))
   90 CONTINUE
      RETURN
      END
      SUBROUTINE STEELP
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
      PSTEEL=0.0
      DO 100 JJ=1,NROWS
      AXS(JJ)=XS(JJ)+DNA-RO
      SSN(JJ)=PHI*AXS(JJ)
      SSS(JJ)=SSN(JJ)*ES
```

```
      IF (SSS(JJ).GT.BARFY) SSS(JJ)=BARFY
      IF (SSS(JJ).LT.-BARFY) SSS(JJ)=-BARFY
      PSTEEL=PSTEEL+AS(JJ)*SSS(JJ)
  100 CONTINUE
      RETURN
      END
      SUBROUTINE SHELLP
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
      COMMON/FOUR/KA,II,KK,PCAP,RCS,ROS,RT,RTS,DIF,ASH(100),IJ,PCT,NBARS
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/SIX/X(100),CSN(100),CSS(100),ACC(100),ATOT(100),SSH(100)
      COMMON/SEVEN/RSSH(100),XX(100),RXX(100),AM(50),XXS(100),RXXS(100)
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
      COMMON/NINE/RASH(100),RATOT(100),RTOT(100),RTACC(100),TOT(100)
      DIMENSION SM(100)
      PSHT=0.0
      PSHELL=0.0
      DO 103 I=1,K
      IF (RO.GT.ABS(XX(I))) GOTO 110
      ASH(I)=0.0
      GOTO 106
  110 ATOT(I)=2.*SQRT(ROS-XXS(I))*DEL
      ASH(I)=ATOT(I)-ACC(I)
  106 SSH(I)=ES*CSN(I)
      IF (SSH(I).GT.TUBEFY) SSH(I)=TUBEFY
      IF (SSH(I).LT.-TUBEFY) SSH(I)=-TUBEFY
  103 PSHELL=PSHELL+ASH(I)*SSH(I)
      IF (DNA-OD) 107,105,105
  107 KA=59-K
      DO 104 I=1,KA
      RX(I)=X(I)-C+F
      RXX(I)=RO-DNA-RX(I)
      RXXS(I)=RXX(I)*RXX(I)
      IF (RINT.GT.ABS(RXX(I))) GOTO 47
      RACC(I)=0.0
      GOTO 44
   47 RACC(I)=2.*SQRT(RINTS-RXXS(I))*DEL
   44 IF (RO.GT.ABS(RXX(I))) GOTO 48
      RASH(I)=0.0
      GOTO 104
   48 RATOT(I)=2.*SQRT(ROS-RXXS(I))*DEL
      RASH(I)=RATOT(I)-RACC(I)
      SM(I)=RX(I)*PHI
      RSSH(I)=ES*SM(I)
      IF (RSSH(I).GT.TUBEFY) RSSH(I)=TUBEFY
      IF (RSSH(I).LT.-TUBEFY) RSSH(I)=-TUBEFY
  104 PSHT=PSHT+RASH(I)*RSSH(I)
      G=F*PHI*ES*T*DEL
      PSHELL=PSHELL-PSHT-G
  105 RETURN
      END
      SUBROUTINE SHELLM
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
      COMMON/FOUR/KA,II,KK,PCAP,RCS,ROS,RT,RTS,DIF,ASH(100),IJ,PCT,NBARS
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/SIX/X(100),CSN(100),CSS(100),ACC(100),ATOT(100),SSH(100)
      COMMON/SEVEN/RSSH(100),XX(100),RXX(100),AM(50),XXS(100),RXXS(100)
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
```

```
      COMMON/NINE/RASH(100),RATOT(100),RTOT(100),RTACC(100),TOT(100)
      AMSHELL=0.0
      AMSHT=0.0
      DO 300 I=1,K
  300 AMSHELL=AMSHELL+SSH(I)*ASH(I)*XX(I)
      IF (DNA-OD) 400,502,600
  400 DO 501 I=1,KA
  501 AMSHT=AMSHT+RSSH(I)*RASH(I)*(RX(I)+DNA-RO)
  502 AMSHELL=AMSHELL+AMSHT+G*(F/2.+DNA-RO)
  600 RETURN
      END
      SUBROUTINE COREP
      COMMON/ONE/PCON,AMCON,PSTEEL,FFC,PSHELL,AMSHELL,AC,PSHT,LL(50)
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
      COMMON/FOUR/KA,II,KK,PCAP,RCS,ROS,RT,RTS,DIF,ASH(100),IJ,PCT,NBARS
      COMMON/FIVE/P(10),AS(50),XS(50),SSN(50),SSS(50),AXS(50),BARFY,STR
      COMMON/SIX/X(100),CSN(100),CSS(100),ACC(100),ATOT(100),SSH(100)
      COMMON/SEVEN/RSSH(100),XX(100),RXX(100),AM(50),XXS(100),RXXS(100)
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
      COMMON/NINE/RASH(100),RATOT(100),RTOT(100),RTACC(100),TOT(100)
      COMMON/TEN/NN,S(100),CST(100),CB(100),H(100),RCX(100)
      COMMON/ELEVEN/CX(100),SN(100),CS(100),B(100),BS(100)
      CDEL=DT/20.
      NN=INT(DNA/CDEL)
      CC=DNA-FLOAT(NN)*CDEL
      CF=CDEL-CC
      IF(DNA-Z)10,10,20
   10 CX(1)=CC+.5*CDEL
      GOTO 600
   20 CX(1)=(DNA-Z)+.5*CDEL
  600 DO 90 I=1,100
      IF(I.GT.1) CX(I)=CX(1)+(I-1)*CDEL
      SN(I)=CX(I)*PHI
      CST(I)=ES*SN(I)
      IF(CST(I).GT.TUBEFY) CST(I)=TUBEFY
      IF(CST(I).LT.-TUBEFY) CST(I)=-TUBEFY
   90 CONTINUE
      PCT=0.0
      PCORE=0.0
      IF(DNA-Z) 1,1,5
    1 IF(DNA-DIF) 80,80,4
    4 IK=INT((DNA-DIF)/CDEL)
      GOTO 6
    5 IK=20
    6 DO 70 I=1,IK
      CB(I)=CX(I)+RO-DNA
      CS(I)=CB(I)*CB(I)
      IF(RT-ABS(CB(I))) 15,15,25
   15 TOT(I)=0.0
      GOTO 30
   25 TOT(I)=2.*SQRT(RTS-CS(I))*CDEL
   30 IF (RC-ABS(CB(I))) 40,40,50
   40 TACC(I)=0.0
      GOTO 60
   50 TACC(I)=2.*SQRT(RCS-CS(I))*CDEL
   60 TA(I)=TOT(I)-TACC(I)
   70 PCORE=PCORE+TA(I)*CST(I)
      IF(DNA-Z) 80,200,200
   80 KK=INT((Z-DNA)/CDEL)
      IF(DNA-DIF) 95,100,100
   95 RCX(1)=DIF-DNA+.5*CDEL
```

```
      KK=20
      GOTO 96
  100 RCX(1)=CF+.5*CDEL
      KK=INT((Z-DNA)/CDEL)
   96 DO 180 I=1,KK
      IF(I.GT.1) RCX(I)=RCX(1)+(I-1)*CDEL
      B(I)=RO-DNA-RCX(I)
      BS(I)=B(I)*B(I)
      IF(RT-ABS(B(I))) 120,120,130
  120 RTOT(I)=0.0
      GOTO 140
  130 RTOT(I)=2.*SQRT(RTS-BS(I))*CDEL
  140 IF(RC-ABS(B(I))) 150,150,160
  150 RTACC(I)=0.0
      GOTO 170
  160 RTACC(I)=2.*SQRT(RCS-BS(I))*CDEL
  170 RTA(I)=RTOT(I)-RTACC(I)
      H(I)=CST(I)*RCX(I)/CX(I)
      IF(H(I).GT.TUBEFY) H(I)=TUBEFY
      IF(H(I).LT.-TUBEFY) H(I)=-TUBEFY
  180 PCT=PCT+RTA(I)*H(I)
      PCORE=PCORE-PCT
  200 RETURN
      END
      SUBROUTINE COREM
      COMMON/TWO/PCORE,AMCORE,ISHAPE,WIDTH,OD,RO,RC,DC,DT,T,TT,Z,COVER
      COMMON/THREE/NROWS,NP,FC,K,DEL,PHI,RINT,RINTS,DNA,C,F,G,TUBEFY
      COMMON/FOUR/KA,II,KK,PCAP,RCS,ROS,RT,RTS,DIF,ASH(100),IJ,PCT,NBARS
      COMMON/EIGHT/TA(100),RTA(100),TACC(100),RACC(100),RX(100),ES,EC,IK
      COMMON/NINE/RASH(100),RATOT(100),RTOT(100),RTACC(100),TOT(100)
      COMMON/TEN/NN,S(100),CST(100),CB(100),H(100),RCX(100)
      COMMON/ELEVEN/CX(100),SN(100),CS(100),B(100),BS(100)
      AMCORE=0.0
      AMCT=0.0
      IF(DNA-Z) 1,1,4
    1 IF (DNA-DIF) 6,6,4
    4 DO 5 I=1,IK
    5 AMCORE=AMCORE+CST(I)*TA(I)*CB(I)
      IF(DNA-Z) 6,9,9
    6 DO 7 I=1,KK
    7 AMCT=AMCT+H(I)*RTA(I)*(RCX(I)+DNA-RO)
      AMCORE=AMCORE+AMCT
    9 RETURN
      END
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(Continued from inside front cover)

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