COMMENTARY ON GEOTECHNICAL PRACTICES

DRILLED PIER DESIGN CRITERIA FOR LIGHTLY LOADED STRUCTURES IN THE DENVER METROPOLITAN AREA

Colorado
Association of
Geotechnical
Engineers

PREFACE

The enclosed "Commentary on Geotechnical Practices, Drilled Pier Design Criteria for Lightly Loaded Structures in the Denver Metropolitan Area" was developed by the Colorado Association of Geotechnical Engineers (CAGE), Professional Practice Committee and approved by vote of the members of the organization. The "Commentary on Geotechnical Practices, Drilled Pier Design Criteria for Lightly Loaded Structures in the Denver Metropolitan Area" was formally adopted at the CAGE meeting of December 8, 1999 and represents a consensus of geotechnical practices in the Denver metropolitan area as of that date. These criteria will likely continue to evolve over time as more is learned about soil-pier interaction and the mechanics of swelling soils.

Colorado Association of Geotechnical Engineers

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DRILLED PIER DESIGN CRITERIA FOR LIGHTLY LOADED STRUCTURES IN THE DENVER METROPOLITAN AREA

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INTRODUCTION

The purpose of this document is to present a summary of the historical development and current state of the practice for pier design as it relates to lightly loaded structures in the Denver Metropolitan Area. This paper only addresses issues of vertical load capacity and uplift resistance; it does not address issues of bending or lateral loads.

Drilled piers have been a common foundation type in the Rocky Mountain area since the early 1950's. Drilled piers are a means of transferring structural loads from an upper layer of undesirable material to a lower layer of more desirable material. This lower layer provides support for wall and column loads and also provides anchorage of piers in a zone of more stable moisture below the zone where moisture changes occur that result in expansion. The use of drilled piers has provided a reasonably economical and performance based solution for foundations of structures on expansive soils.

Early pier designs utilized both straight shaft and belled piers. Belled piers were used to develop increased anchorage from uplift and increased vertical capacity with less pier shaft size. By the early 1960 pier design had primarily changed to straight-shaft piers. The use of belled piers decreased in the Denver area. Belled piers are currently used primarily when water and soil conditions limit pier installation depths.

The design of drilled piers in the Denver area has been an evolutionary process consistent with the observational methods of geotechnical engineering. An analytical procedure for evaluating soil/pier interaction was developed in the 1960's. The design assumptions have changed based on performance with little modification of the procedure. The growth in the Denver region over the past 10 years has resulted in construction occurring in differing geologic environments that have experienced higher swelling soils, variable depths of wetting, and in some instances, poor foundation performance. As a consequence, pier lengths have increased. Significant modifications in the basic design procedure have not developed, however, because the numerous mechanisms involved in soil/pier interaction have not been fully characterized. The variability of the geologic and construction conditions has resulted in a simplistic design approach that relies heavily on empiricism, performance, and judgment.

Pier performance problems appear to be most frequent in lightly loaded structures. Performance problems, however, have also been encountered in larger, more heavily loaded structures.

DETERMINATION OF PIER LENGTHS

MECHANICS OF THE DRILLED PIER SYSTEM IN EXPANSIVE SOILS

Drilled pier design includes an evaluation of the vertical load-carrying capacity of the piers and their resistance to uplift. The load carrying capacity of the piers is most often based on an empirical evaluation of the strength of the bearing stratum, commonly evaluated from penetration resistance.

For expansive soils, the evaluation of uplift on a pier is based on an assumption that moisture variation will occur in a zone to some depth below the top of the pier after the pier is installed. This zone is called the zone of moisture variation (Z). Moisture increases within this zone will result in subsequent swelling of the soil and/or bedrock above that depth of moisture variation. This swelling will transmit uplift force from the soil/bedrock to the pier shaft. Because wetting is not considered to occur instantaneously throughout the zone of moisture variation by some engineers, a "zone of influence" can be considered in pier design. The zone of influence (L_w) is defined as the zone through which instantaneous wetting and uplift occurs. Historically, the zone of influence and the zone of moisture variation were considered the same. Some engineers still consider these the same while others consider the zone of influence to be less than the zone of moisture variation. The portion of the pier below the zone of moisture variation resists the uplift. Design includes a calculation of the swelling pressure of the soil, uplift on the pier, the zone of moisture variation, zone of influence, and the shear resistance of the portion of the pier founded below the zone of influence. Figure 1 defines key terms used in pier design.

For capacity, full-scale load tests are the best method of evaluating design criteria. Load tests are not common because of the relative ease and economics of pier installation for the loads they carry and since load tests would be less practical for evaluating expansive soil performance. Empiricism has become the basis of the standard of practice for evaluation and design. Where load tests have been performed, they have generally been performed to evaluate load carrying capacity and have indicated pier capacity exceeds values obtained using the local design approach. Failures due to expansive soil heaving have been much more common than failures due to insufficient load carrying capacity.

LOCAL METHODS OF PIER CAPACITY CALCULATIONS

Allowable End Pressure

A formula for allowable end pressure of a drilled pier in bedrock has been developed and used with relative success in the Denver area. This formula is:

Equation (1)

$$q_a = \frac{N}{2}$$

where q_a = allowable end pressure in kips per square foot, typically limited to 40 ksf or less for lightly loaded structures due to the lack of need for higher capacities

N = ASTM D 1586 Standard Penetration Test blow count or locally modified form of this test utilizing at 2-inch I.D. modified California Sampler

Experience and load testing have indicated this to be conservative when blow counts are taken in bedrock. In situ Menard pressure meter tests have yielded similar results. Typical laboratory testing to verify this analysis, especially when in softer material, has consisted of undrained strength tests and high capacity consolidation tests.

Skin Friction for Resistance of Vertical, Downward Loads

Design skin friction of the soil or bedrock materials below the zone of moisture variation is typically assigned as 10 percent of the allowable end pressure. This value of skin friction is used for determining the required embedment of the pier. On sites where piers will be anchored in bedrock below soil or fill, skin friction is typically not used for any length of pier in the overburden soils or fill. Where piers are founded entirely in bedrock, typically some depth of the bedrock below the top of pier is not used in the skin friction capacity evaluation. On sites where "friction" piers in soil are utilized, skin friction is typically ignored for some portion of the soil below the top of the pier.

Potential Pier Uplift

Pier uplift caused by expansion of soil/bedrock in the zone of moisture variation is calculated as follows:

Equation (2)

$$U = \Pi D L_w \propto S_p$$

Where U = pier uplift (kips)

D = pier diameter (feet)

L_w = zone of influence (feet)

$\propto$ = swell pressure coefficient. This is an empirical factor relating swell pressure measured in the laboratory swell-consolidation test to pressure acting along the pier. The swell pressure coefficient that has been frequently used is 0.15 but may vary. This coefficient has been determined from laboratory testing by Chen (1988)[1] and is based on limited data and varies with the engineer's judgement.

S_p = swell pressure determined from load-back during swell-consolidation test (ksf)

Uplift Resistance

Resistance to uplift forces is by the shear resistance of the portion of the pier below the zone of moisture variation and by the dead load applied to the pier. This resistance is calculated as follows:

Equation (3)

$$R = \Pi D L_e S_u + W_d$$

Where R = uplift resistance (kips)

D = pier diameter (feet)

L_e = length of pier below zone of moisture variation (feet)

S_u = shear resistance for uplift (kips per square foot)

W_d = dead load (kips)

The shear resistance, S_u, used to resist expansive soil uplift forces, is typically taken as some percentage of the skin friction resistance used to resist downward structural loads. This value varies between designers and has varied over time. Currently values of S_u range from 50 to 100 percent of the skin friction for downward vertical load resistance.

[1] Chen, F.H. Foundations on Expansive Soils, Developments in Geotechnical Engineering 54, Elsevier, 1988.

Pier Length Determination

Minimum Embedment Length

Pier lengths are determined by equating the potential pier uplift force to the uplift resisting force. In general, this equation takes the following form:

Uplift (Equation 2) = Shear Resistance of Portion of Pier below zone of moisture variation + Structural Dead Load (Equation 3)

In equation form:

Equation (4)

$$\Pi D L_w \alpha S_p = \Pi D L_e S_u + W_d$$

Engineers have used considerable judgement on where to apply factors of safety to this equation and to what dead loads should be specified. Some firms specify relatively high dead loads which is typically presented as a "dead load pressure" or the ratio of dead load to the pier cross sectional area. Where dead loads cannot be met, soils engineers have recommended increased pier length and embedment. Some firms believe that provision of dead load is more effective than increasing pier length and, therefore, encourage structural designs, which maximize dead loading

Minimum Pier Length

Minimum pier lengths are typically determined by summing the required minimum length of embedment and the length of the zone of moisture variation. In equation form:

Equation (5)

$$L_{min} = L_e + Z$$

Where L_{min} = minimum pier length (feet)

L_e = minimum length of embedment below zone of moisture variation

Z = zone of moisture variation

Note that Z, the zone of moisture variation, is the depth over which moisture variation is expected to occur. Historically, this zone of moisture variation was considered equal to the zone of influence. Now some engineers, based on judgment, assume an instantaneous wetting for uplift calculation that is less than the total anticipated zone of moisture variation. This assumption takes the form of a constant pressure distribution along the portion of the pier in the zone of influence above the embedment zone. In these instances, the engineer may use the entire zone of moisture variation to determine minimum pier length rather than just the zone of influence.

Pier Reinforcing for Tension

The net uplift on the upper portion of the pier results in tensile stresses in the pier. Reinforcement is required to resist these stresses. Reinforcement recommendations have been given in the following forms:

1. Uplift forces (U in Equation (2)) are presented or an uplift pressure acting over a certain pier length is given and the reinforcement design is left to the structural engineer.
2. Minimum reinforcement in terms of a specified number of reinforcing bars and occasionally the grade of the bars.
3. Reinforcement as a function of pier circumference or diameter.

EMPIRICAL METHODS FOR PIER LENGTH DETERMINATION

Empirical methods for pier lengths have been developed by observation of pier performance. Depending upon the engineer, this observational process may have been initiated by an analytical type evaluation or by experience on other projects. As performance data have been gathered, drilling equipment changed, and new areas developed, such as the dipping bedrock areas and locations of highly swelling materials, pier lengths have increased.

CAGE members reviewed reports prepared over the past 30 years to determine how recommended minimum pier lengths have changed. The results of this review are presented in Figure 2. The longest lengths are typically associated with high to very high swelling materials and the shorter lengths are typically associated with low swelling soils or conditions where piers were used to support loads below non-suitable bearing material rather than the pressure of expansive subsurface materials.

RESULTS OF CAGE SURVEY

A survey was performed by the Professional Practices Committee of CAGE to gauge the current practice of pier design. This survey included three examples that were prepared in November 1998 and collated in March 1999. Examples 1 and 2 were for single residential structures. Example 3 was for a small subdivision. A copy of the three-question survey is in Appendix A.

Example 1 had a profile of a clay consisting of weathered claystone or a sand consisting of weathered sandstone overlying claystone and sandstone bedrock. No groundwater existed. Swell-consolidation testing indicated high to very high swell.

Example 2 had a soil profile consisting of a very stiff to hard clay for the full depth explored. No groundwater existed. Swell-consolidation testing indicated moderate to high swell.

Example 3 indicated a subsurface profile of a stiff to very stiff, native clay which was overlain in some areas by a stiff to very stiff clay with claystone fill. The native clay was underlain by claystone bedrock. Groundwater was encountered in three of the seven borings. Swell-consolidation testing indicated low to very high swell.

Boring logs and test data were provided for each example problem. CAGE member firms were asked to provide pier design recommendations for each example just as they would for a typical report. Additionally, the member firms were asked to state and show all assumptions and calculations for their analyses.

The results of this survey were summarized and are presented in Tables 1, 2, and 3 for Examples 1, 2, and 3, respectively. Out of 25 firms solicited, ten responded.

In all but one entry for Example 1, there was a general trend of allowable end pressure being based on "N/2" or less. Similarly, most respondents used allowable skin friction for compression equal to 10 percent of allowable end pressure. Allowable skin friction for tension ranged from 67 to 100 percent of the value used for compression for all examples with a majority of firms using 100 percent.

The zone of influence for the three examples ranged from 6 to 10 feet except for one responding firm, which used 12 feet. The depth of moisture variation ranged from 8 to 16 feet with all but one respondent using 10 feet or greater.

Uplift coefficients of 0.15 appeared to be the norm. Only one respondent differed and used 0.05 and 0.07.

Design swell pressures for Examples 1 and 2 appeared to be near the maximum swell pressure value. For Example 3, design swell pressures appeared to vary from an average to an estimation of the highest potential swell pressure. Estimation was required because one swell test was not able to achieve the initial volume when it was loaded to 25,000 psf. No additional load was added.

Minimum pier lengths varied from 18 to 28 feet for Example 1, 20 to 34 feet for Example 2, and 18 to 31 feet for Example 3. Of all responses for the three examples, only one response for Example 1 and one for Example 3 had pier lengths of less than 20 feet.

Steel reinforcement in general was specified by all firms except one for all three examples and two for Example 3. For the one firm that did not specify steel for all three examples, the firm gave forces for the structural engineer to design for. For the other non-entry, no guidance was given.

TABLE 1
SUMMARY OF EXAMPLE 1

Respondent	q_a Allowable End Bearing Pressure (psf)	S Allowable Skin Friction Compression (psf)	S_u Allowable Skin Friction Tension (psf)	L_w Zone of Influence (feet)	Z Depth of Moisture Variation Zone (feet)	Uplift Coefficient	S_p Design Swelling Pressure (psf)	W_d Specified Minimum Dead Load Pressure (psf)	L_e Minimum Embedment Depth (feet)	Minimum Length of Shaft Below Basement (B) Non-Basement (NB) B (feet)	NB (feet)	Foundation Void Space (inches)	Minimum Steel Reinforcement Grade 60
1	25,000	2,500	1,900	10	10	0.15	13,000	20,000	9	20	20	8	3 - #5
2	20,000	2,000	2,000	-	-	-	-	25,000	10	28	28	6	3- #6 (See Note 1)
3	30,000	2,000	1,333	6	10	0.15	13,000	13,000	9	19	19	-	See Note 2
4	20,000	2,000	2,000	7	10	0.15	16,000	0	10	19	19	6	See Note 3
5	20,000	2,000	2,000	10	10	0.15	15,000	15,000	6	26	26	6	3 - #5
6	15,000	1,500	1,500	10	16	0.15	14,000	20,000	11	21	25	8	2 - #6
7	15,000	1,500	1,500	8	-	0.15	12,500	0	12	18	18	4	2 - #5
8	15,000	1,500	1,130	10	-	0.05	13,000	20,000	10	20	20	-	See Note 4
9	40,000	4,000	4,000	-	-	-	-	15,000	6	18	18	-	-
10	20,000	2,000	1,000	7	-	0.15	15,000	20,000	11	21	21	-	See Note 6

Notes:

1) Piers that cannot be loaded to meet the minimum dead-load requriements should have the tension steel increased by 50%.

2) Minimum reinforcing steel not provided in report. Structural engineer must size reinforcing steel based upon an uplift of 1850 pounds per square foot (psf) for the top six feet of each pier.

3) Minimum reinforcing steel not provided in report. Structural engineer must size reinforcing steel based upon the difference between the uplift force and the dead load on the piers. Uplift force, in pounds, should be computed by multiplying the pier diameter in feet by 53, 000.

4) One #5 or #6 per 16 inches of circumfrence. If high swelling soils exist may consider additional steel or give the structural engineer some parameters.

5) "-" symbol indicates no value given.

6) 1 #5 per 18 inches of circumference or 2 #5 per 10 inches of diameter

TABLE 2
SUMMARY OF EXAMPLE 2

Respondent	q_a Allowable End Bearing Pressure (psf)	S Allowable Skin Friction Compression (psf)	S_u Allowable Skin Friction Tension (psf)	L_w Zone of Influence (feet)	Z Depth of Moisture Variation Zone (feet)	Uplift Coefficient	S_p Design Swelling Pressure (psf)	W_d Specified Minimum Dead Load Pressure (psf)	L_e Minimum Embedment Depth (feet)	Minimum Length of Shaft Below Basement (B) Non-Basement (NB) B (feet)	NB (feet)	Foundation Void Space (inches)	Minimum Steel Reinforcement Grade 60
1	8,000	800	600	10	10	0.15	12,000	20,000	NA	34	34	8	3 - #5
2	15,000	1,500	1,500	-	-	-	-	15,000	NA	30	30	6	3- #6 (See Note 1)
3	13,000	1,300	866	10	10	0.15	12,000	10,000	NA	25	25	6	See Note 2
4	8,000	800	800	7	13	0.15	16,000	0	NA	33	33	6	See Note 3
5	8,000	800	800	10	10	0.15	15,000	15,000	NA	28	28	6	3 - #5
6	10,000	1,000	1,000	10	16	0.15	12,000	20,000	NA	24	28	8	2 - #6
7	6,000	1,500*	1,500*	8	8	0.15	12,000	0	NA	20	20	4	3 - #5
8	8,000	650	650	10	-	0.15	15,000	20,000	NA	25	25	-	See Note 4
9	10,000	1,000	1,000	8	-	0.15	15,000	15,000	NA	26	26	-	-
10	0	See Note 6	450	7	-	0.15	15,000	25,000	NA	31	31	-	See Note 7

*Shear Rings Specified

Notes:

1) Piers that cannot be loaded to meet the minimum dead-load requriements should have the tension steel increased by 50%.

2) Minimum reinforcing steel not provided in report. Structural engineer must size reinforcing steel based upon an uplift of 1800 pounds per square foot (psf) for the top six feet of each pier.

3) Minimum reinforcing steel not provided in report. Structural engineer must size reinforcing steel based upon the difference between the uplift force and the dead load on the piers. Uplift force, in pounds, should be computed by multiplying the pier diameter in feet by 53,000.

4) One #5 or #6 per 16 inches of circumfrence. If high swelling soils exist may consider additional steel or give the structural engineer some parameters.

5) "-" symbol indicates no value given.

6) 1300 / 3 Dead Load or 1300 / (2 Dead Load + Live Load)

7) 1 #5 per 18 inches of circumference

TABLE 3
SUMMARY OF EXAMPLE 3

Respondent	q_a Allowable End Bearing Pressure (psf)	S Allowable Skin Friction Compression (psf)	S_u Allowable Skin Friction Tension (psf)	L_w Zone of Influence (feet)	Z Depth of Moisture Variation Zone (feet)	Uplift Coefficient	S_p Design Swelling Pressure (psf)	W_d Specified Minimum Dead Load Pressure (psf)	L_e Minimum Embedment Depth (feet)	Minimum Length of Shaft Below Basement (B) Non-Basement (NB) B (feet)	NB (feet)	Foundation Void Space (inches)	Minimum Steel Reinforcement Grade 60
1	20,000	2,000	1,500	10	10	0.15	21,411	25,000	18	28	28	12	2 - #8
2	-	-	-	-	-	-	-	-	-	-	-	-	-
3	25,000	2,500	1,700	6	10	0.15	28,000	10,000	15	25	25+	6	See Note 2
4	15,000	1,200	1,200	7	13	0.15	20,000	0	21	21	31	6	See Note 3
5	20,000	2,000	2,000	10	10	0.15	20,000	20,000	6	24	24	6	3 - #5
6	20,000	2,000	2,000	10	16	0.15	20,000	20,000	13	23	27	8	2 - #7
7	20,000	2,000 for <2'	2,000 for <	8	8	0.15	24,000	0	18	26	26	6	3 - #5
8	20,000	2,000	1,500	12	-	0.07	30,000	30,000	12	25	25	-	See Note 4
9	20,000	2,000	2,000	-	-	0.15	-	25,000	8	18	18	-	-
10	20,000	2,000	1,000	7	-	0.15	17,000	30,000	20	30	30	-	See Note 6

Notes:

1) Piers that cannot be loaded to meet the minimum dead-load requriements should have the tension steel increased by 50%.

2) Minimum reinforcing steel not provided in report. Structural engineer must size reinforcing steel based upon an uplift of 3500 pounds per square foot (psf) for the top six feet of each pier.

3) Minimum reinforcing steel not provided in report. Structural engineer must size reinforcing steel based upon the difference between the uplift force and the dead load on the piers. Uplift force, in pounds, should be computed by multiplying the pier diameter in feet by 66,000.

4) One #5 or #6 per 16 inches of circumfrence. If high swelling soils exist may consider additional steel or give the structural engineer some parameters.

5) "-" symbol indicates no value given.

6) 1 #5 per16 inches of circumference

MOISTURE VARIATION ASSUMPTIONS IN PIER DESIGN

The evolution of the local pier design method described in this paper has primarily involved changes in the assumed zone of influence for pier uplift calculations. In the 1970's and early 1980's it was common to use a depth of moisture variation and zone of influence of about 4 to 5 feet in pier design. This generally resulted in piers on the order of 12 to 16 feet in length. Forensic data developed by some firms in the mid to latter 1980's indicated that depth of moisture variation on the order of 14 to 16 feet below the ground surface had occurred. Some firms increased the design zone of influence to 6 to 8 feet.

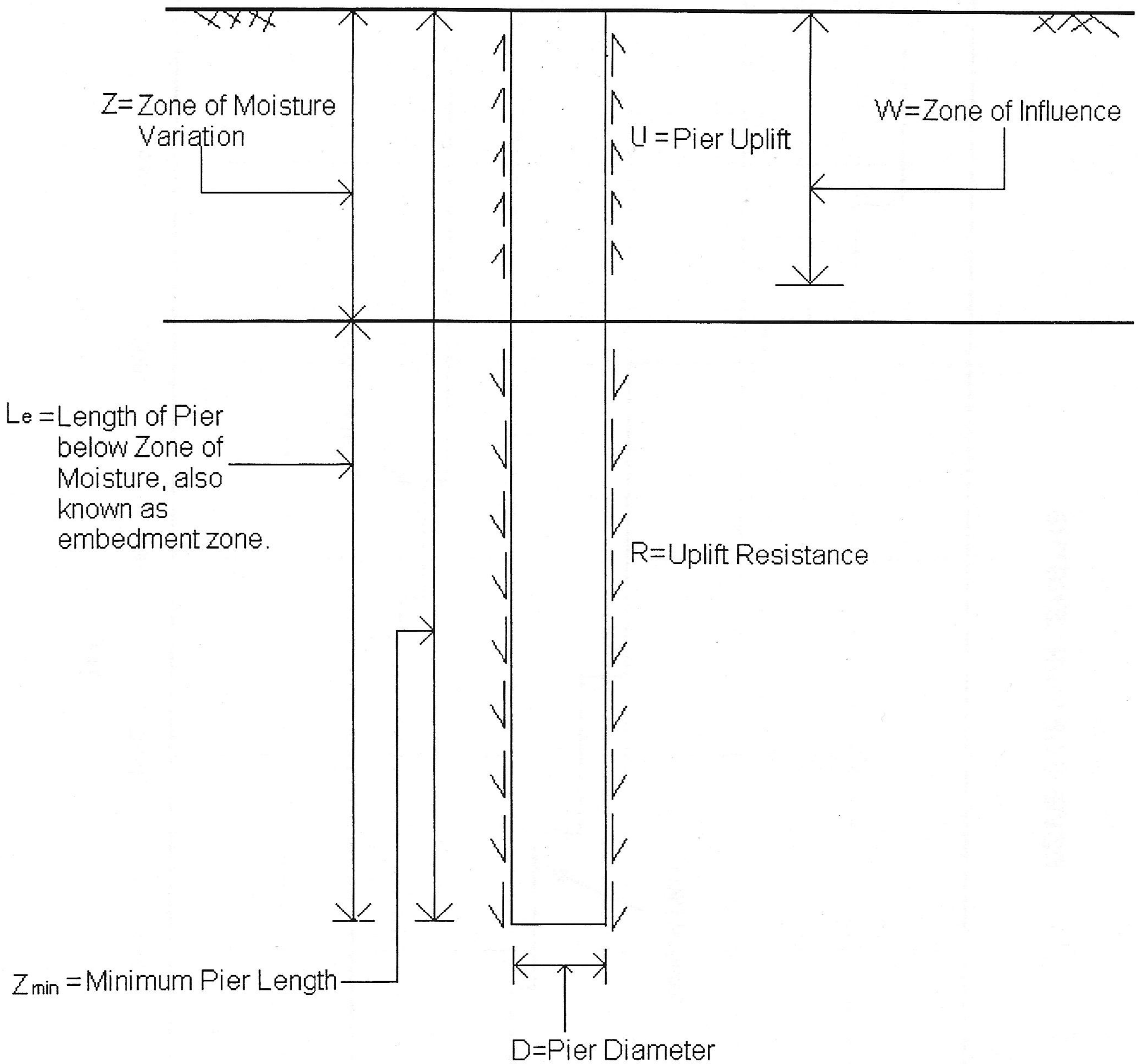
Some lightly loaded structures, such as residences, are built with basements. Although drilled piers in these instances are drilled from differing excavation levels, the state of practice of drilled pier design was, and to a large degree is, to assume that the zone of influence for piers drilled from the basement level is the same as the zone of influence for piers drilled from the non-basement level. This results in the same minimum pier length for basement level and non-basement level piers. Some firms now recommend increasing the pier length in non-basement areas so that they bottom very near the bottom of basement-level piers.

Observations of foundation performance have indicated that the majority of residences constructed using piers designed with a zone of influence assumption of 4 to 5 feet performed satisfactorily. As construction moved into areas of higher swelling soils and bedrock in parts of metropolitan Denver in the mid to late 1980's some houses designed with this assumption performed poorly. The increase in pier lengths resulting from an increase in the likely zone of influence to 6 to 10 feet resulted in a lower frequency of problems into the mid 1990's.

Within the last two years, data have become available indicating depth of moisture variation on the order of 20 feet can occur. This has led some firms to increase the zone of influence assumption in pier design to 10 to 12 feet, or more. Geologic factors can significantly influence moisture penetration and resulting performance of drilled piers. For example, in areas of steeply dipping bedrock (30 degrees or more), depth of moisture variation of 25 to 30 feet or more has been measured by some firms. The presence of permeable lenses of sandstone and fractures in claystone bedrock also influence depth of moisture variation and zone of influence. Considerable judgement is required when evaluating the potential influence of moisture changes and resulting soil and bedrock heave.

CLOSING

The practice of geotechnical engineering remains a dynamic process subject to significant uncertainty. No survey can either establish the standard of care for past practice or predict that standard for the future. While we believe studies such as this present valuable empirical data; each project must continue to be considered on its own merits. Thus, this paper should not be considered either a design guide or an attempt to define a standard of practice in a profession where the state of the art continues to evolve.



Z= Zone of Moisture Variation
U = Pier Uplift
W=Zone of Influence
Le =Length of Pier below Zone of Moisture, also known as embedment zone.
R=Uplift Resistance
Zmin =Minimum Pier Length
D=Pier Diameter

FIGURE 1

REVIEW OF PIER LENGTHS

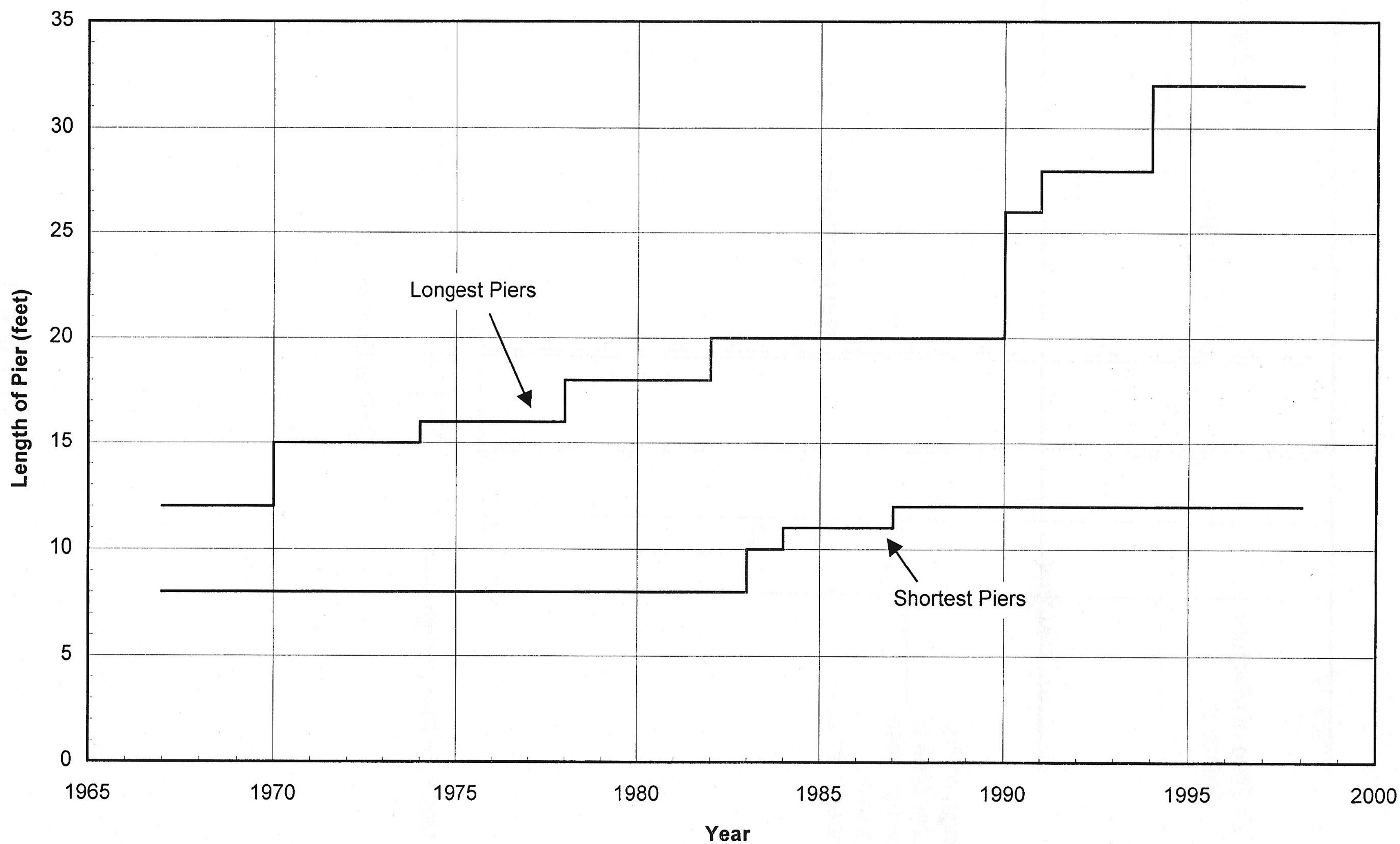


Figure 2

APPENDIX A

November 1998
CAGE Questionnaire

Colorado
Association of
Geotechnical
Engineers

November 2, 1998

Dear CAGE member firm:

As you are aware, the Professional Practices Committee is presently in the process of drafting a document which will present the current state of the practice for drilled pier design for residential structures. The document will be similar in scope to the Guideline for Slab Performance Risk Evaluation previously published by CAGE.

In order to help with our compilation and evaluation of the current practice, we need your honest input on drilled pier design. Attached you will find three basic examples containing subsurface logs and laboratory test data. Examples 1 and 2 are for a single structure and Example 3 is for a small subdivision. We would like each member firm to provide pier design recommendations for the examples as you would in a typical report. In addition, we would like you to submit your design assumptions and calculations performed during your analysis. The backup information is important to the process of developing what is the current practice in drilled pier design. If there is some additional information that you typically require please state the nature of the information. Where possible, please make an assumption on the missing information and complete the design.

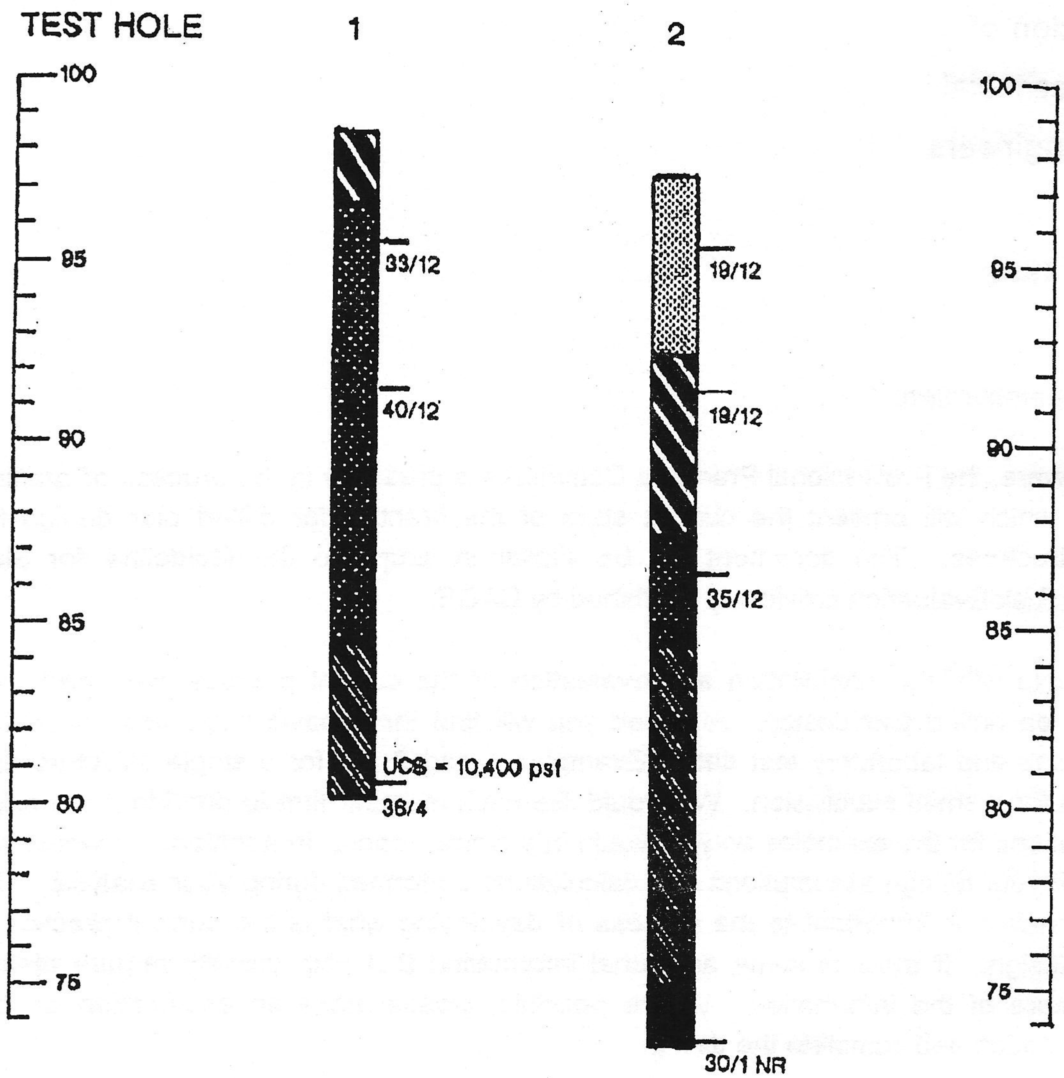
As a senior member of your firm we ask for your cooperation in completing this effort as it involves our livelihood. If possible, please bring this work to the next CAGE meeting or send it to:

CAGE Professional Practices Committee
c/o Terracon
10625 West I-70 Frontage Road North, Suite 3
Wheat Ridge, Colorado 80033
Attn: Jere Strickland

or fax to 303-423-3353

Sincerely,

CAGE Professional Practices Committee



LEGEND

CLAY, Weathered CLAYSTONE, medium to high plasticity, very stiff, medium moist to moist, dark gray and brown (CL-CH)

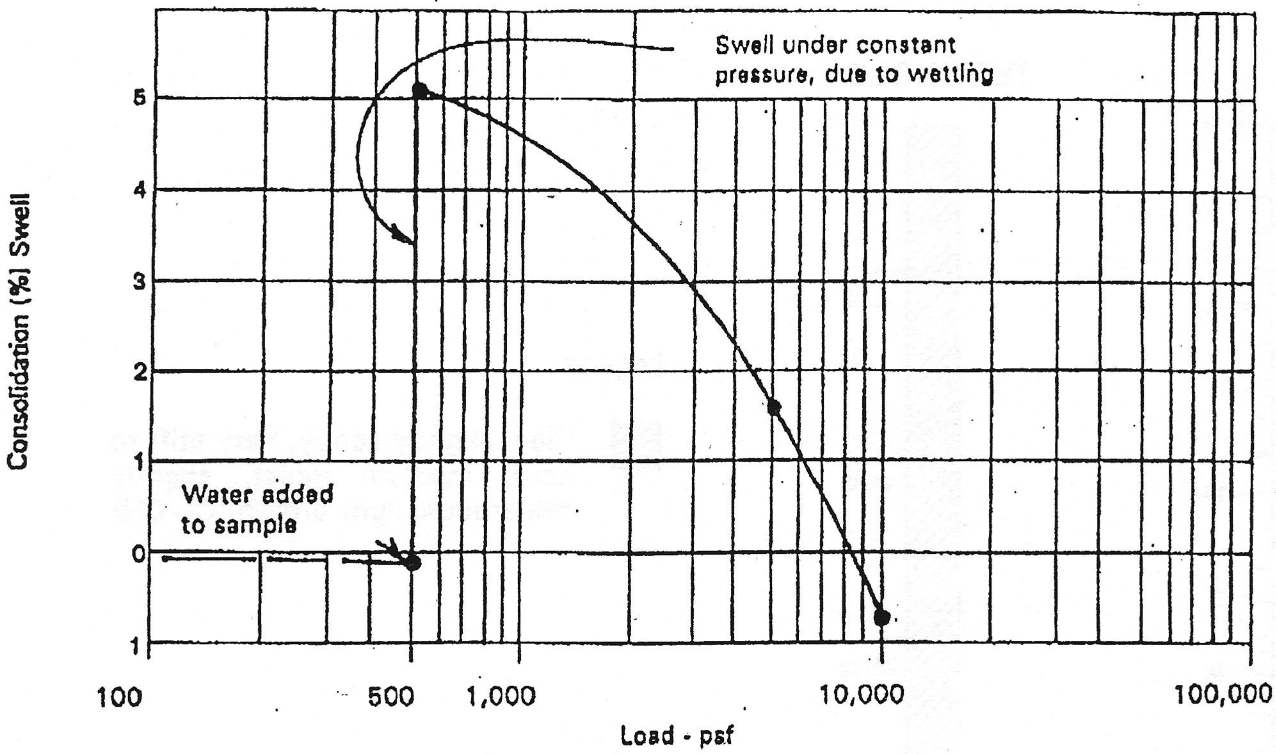
SAND, Weathered SANDSTONE, fine to medium grained, weak cementation, silty, medium dense, slightly moist, light brown (SM)

CLAYSTONE BEDROCK, medium to high plasticity, hard, medium moist, iron staining noted, gray (CL-CH)

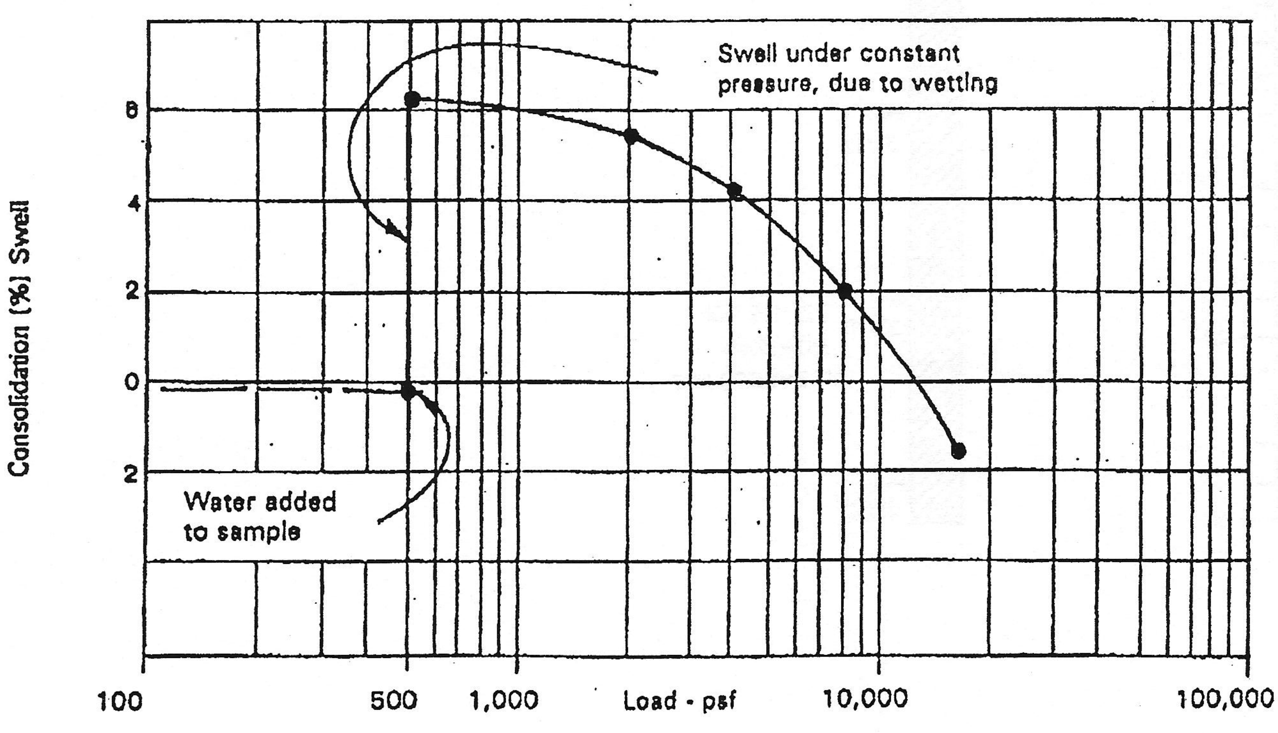
CLAYSTONE BEDROCK, low to high plasticity, very hard, medium moist to moist, iron staining and soluble sulfates noted, yellow brown and gray (CL-CH)

SANDSTONE BEDROCK, fine grained, well cemented, silty, very hard, slightly moist, light brown (SM)

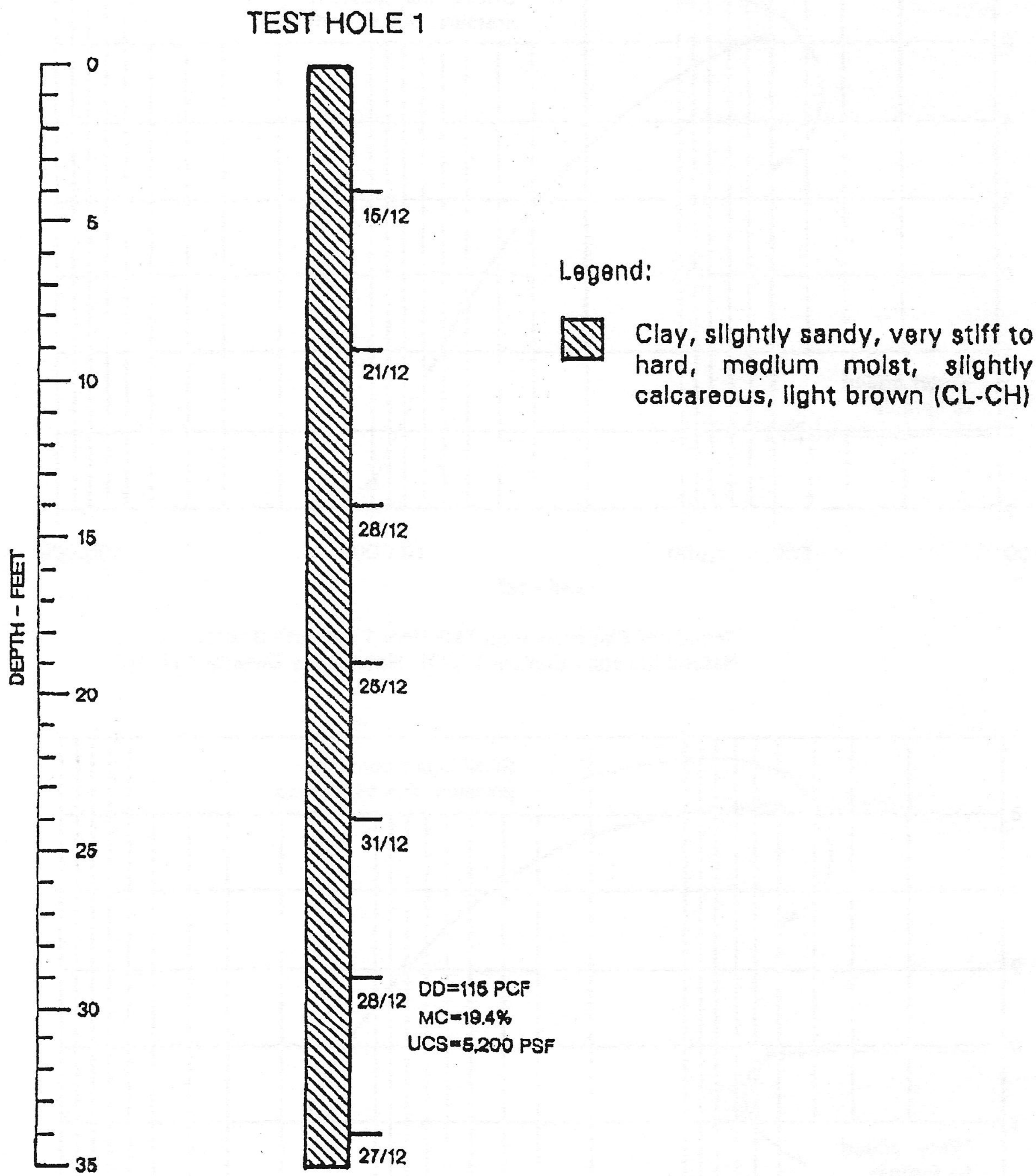
Example 1



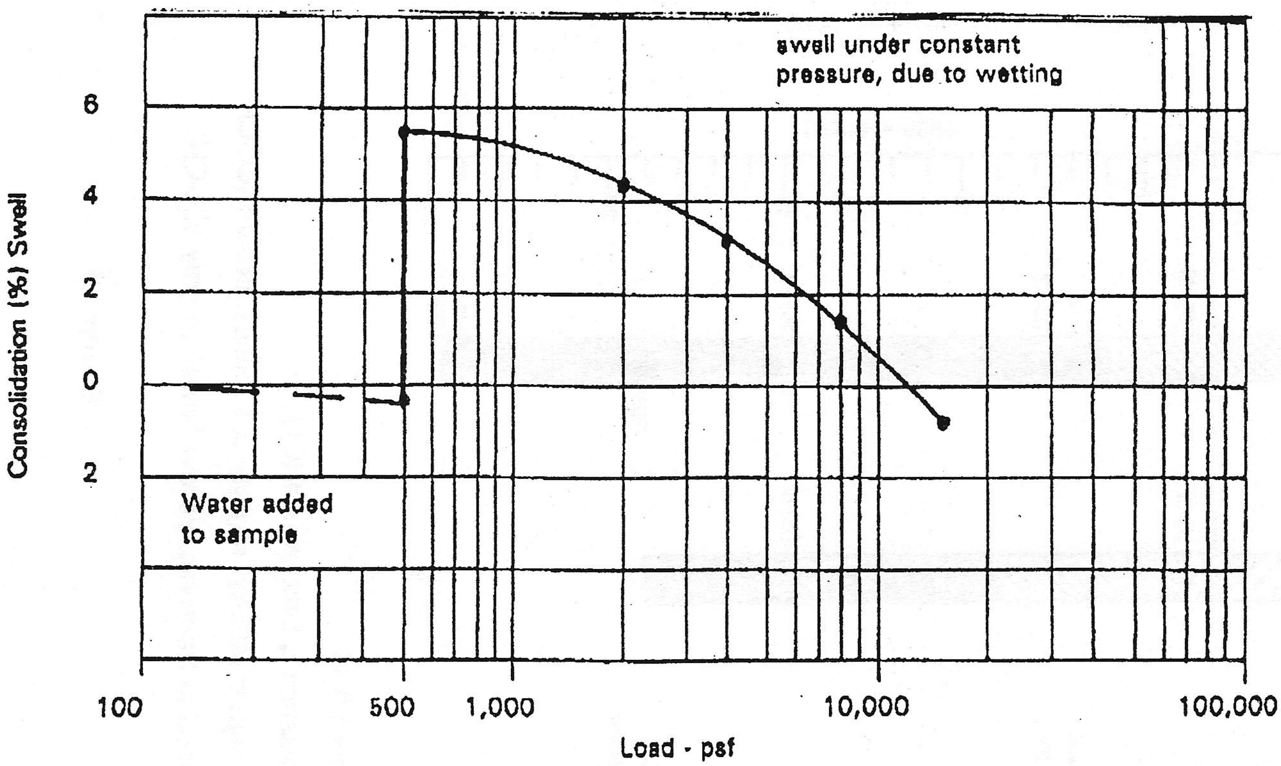
Sample of Claystone from Test Hole 1 at depth 3 feet.
Natural Moisture Content 14.7% Natural Dry Density 119 pcf



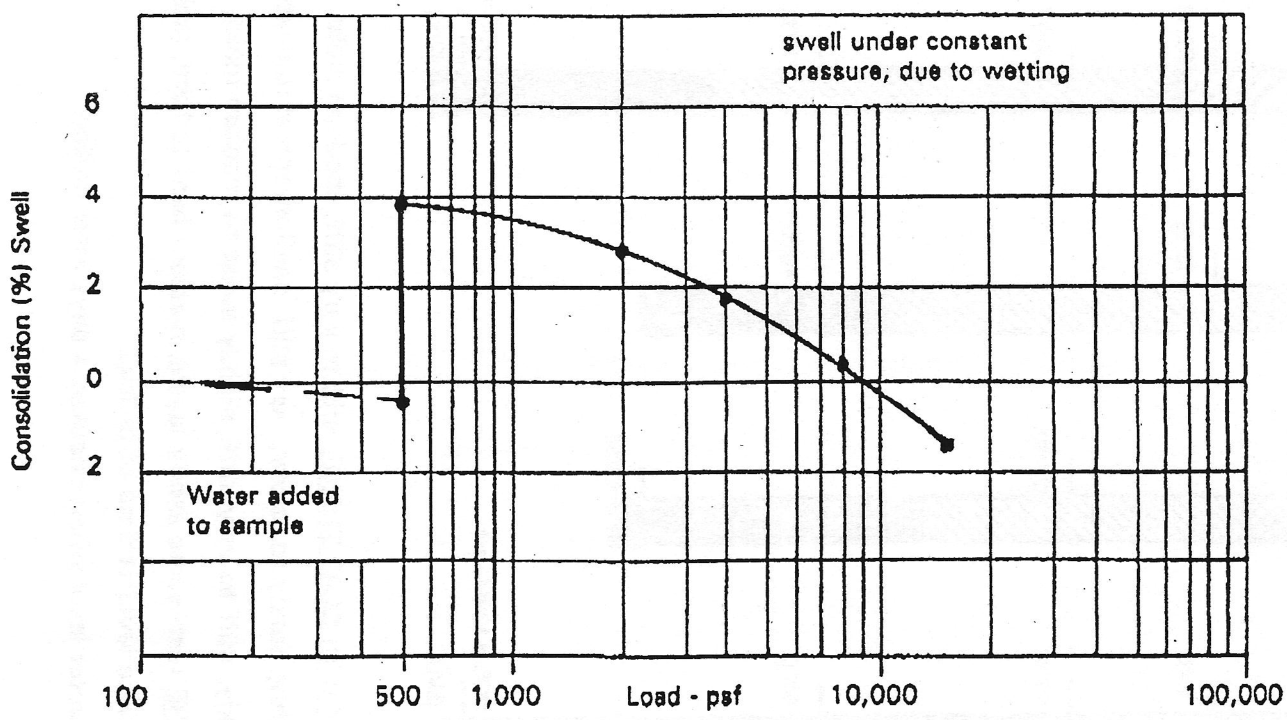
Sample of Clay, weathered Claystone from Test Hole 2 at depth 6 feet.
Natural Moisture Content 14.0% Natural Dry Density 119 pcf


TEST HOLE 1
DEPTH - FEET
0
5
10
15
20
25
30
35
15/12
21/12
28/12
25/12
31/12
28/12
DD=115 PCF
MC=19.4%
UCS=5,200 PSF
27/12
Legend:
Clay, slightly sandy, very stiff to hard, medium moist, slightly calcareous, light brown (CL-CH)

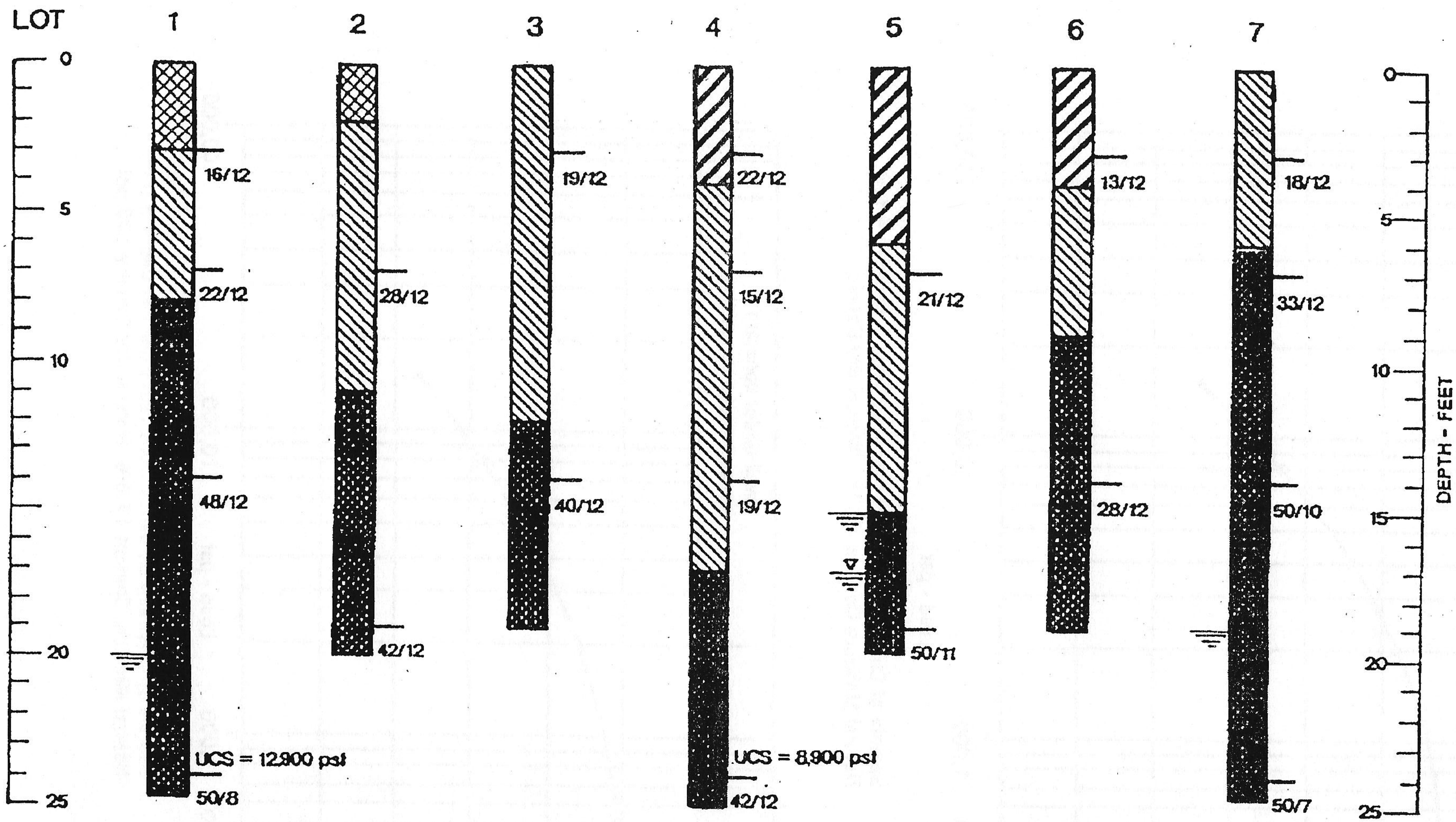
Example 2



Sample of Clay, slightly sandy from Test Hole 1 at depth 4 feet.
Natural Moisture Content 15.2% Natural Dry Density 112 pcf



Sample of Clay, slightly sandy from Test Hole 1 at depth 9 feet.
Natural Moisture Content 17.8% Natural Dry Density 108 pcf



<u>FILL</u>, <u>CLAY with CLAYSTONE</u>, stiff to very stiff, medium moist, mottled brown (CL)

<u>CLAY</u>, slightly sandy to sandy, very stiff, medium moist to moist, medium plasticity, light brown (CL)

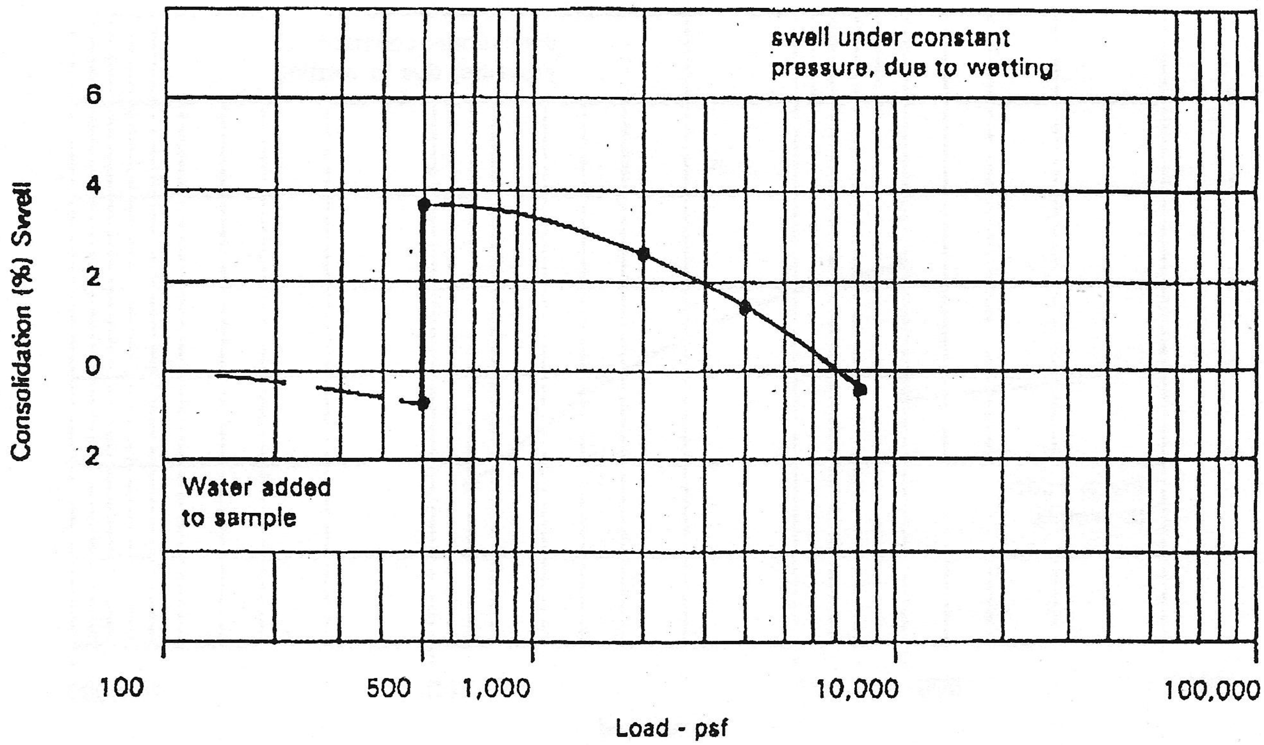
<u>CLAY</u>, sandy, stiff to very stiff, slightly moist to medium moist, medium to high plasticity, slightly calcareous, brown (CL-CH)

<u>CLAYSTONE</u> with some sandy layers, medium hard to hard, moist, iron stained in fractures, yellow brown to gray (CL-CH)

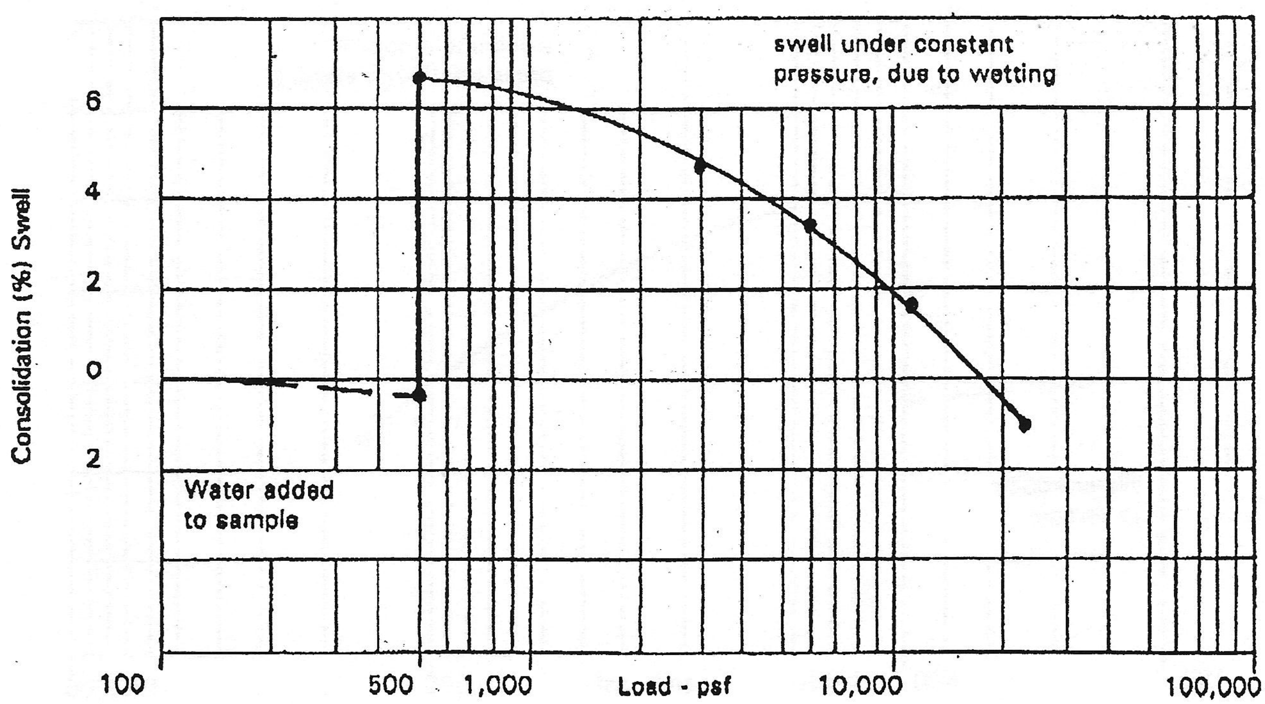
Indicates were level at time of drilling.

Indicates water level when checked 4 days after drilling.

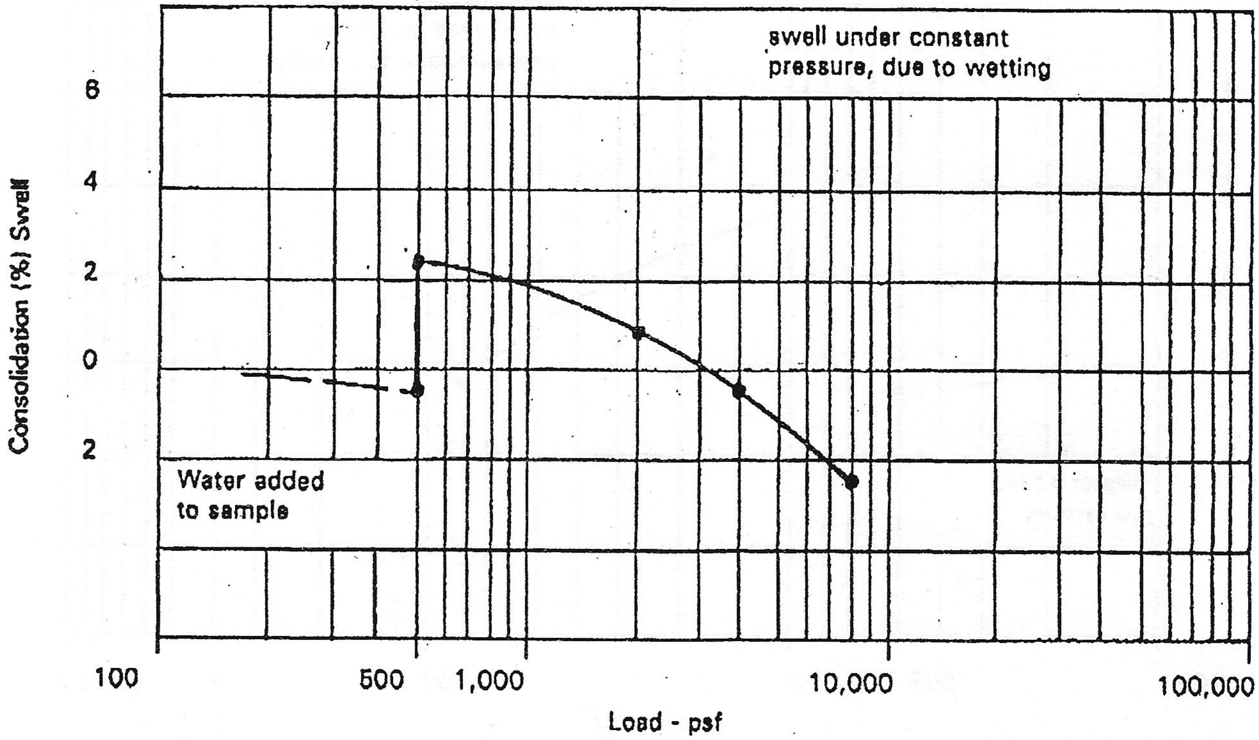
Example 3



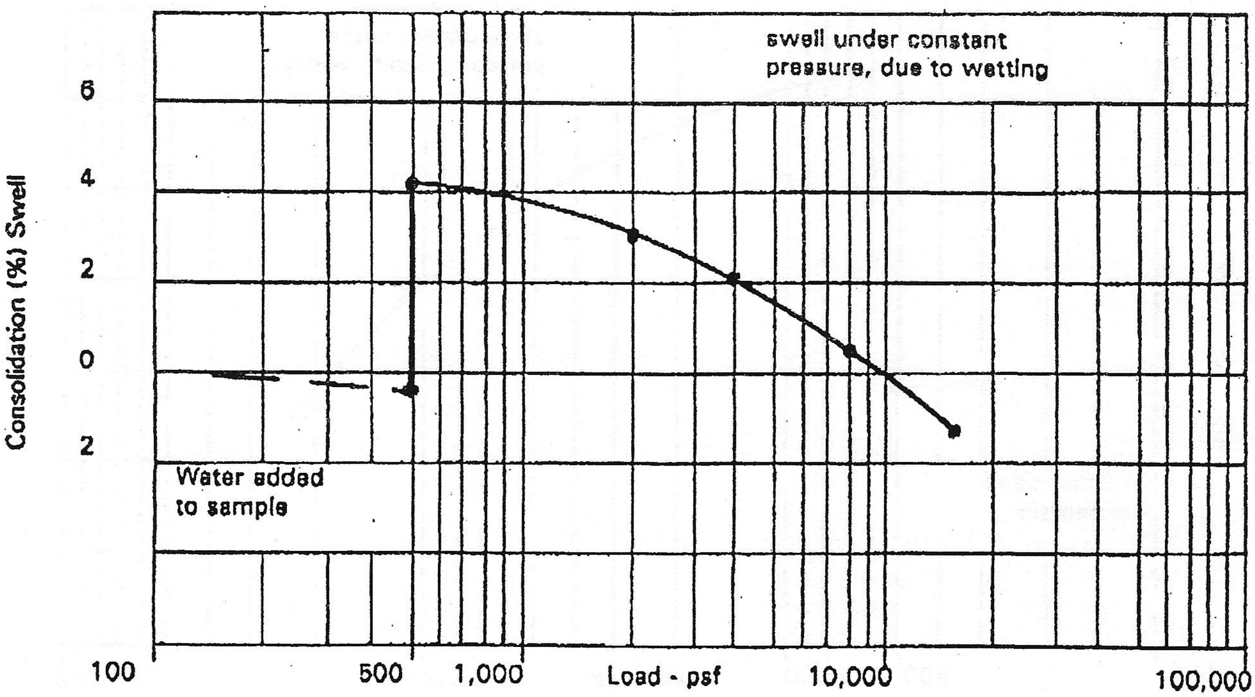
Sample of Clay, slightly sandy from Lot 1 at depth 3 feet.
Natural Moisture Content 15.0% Natural Dry Density 112 pcf



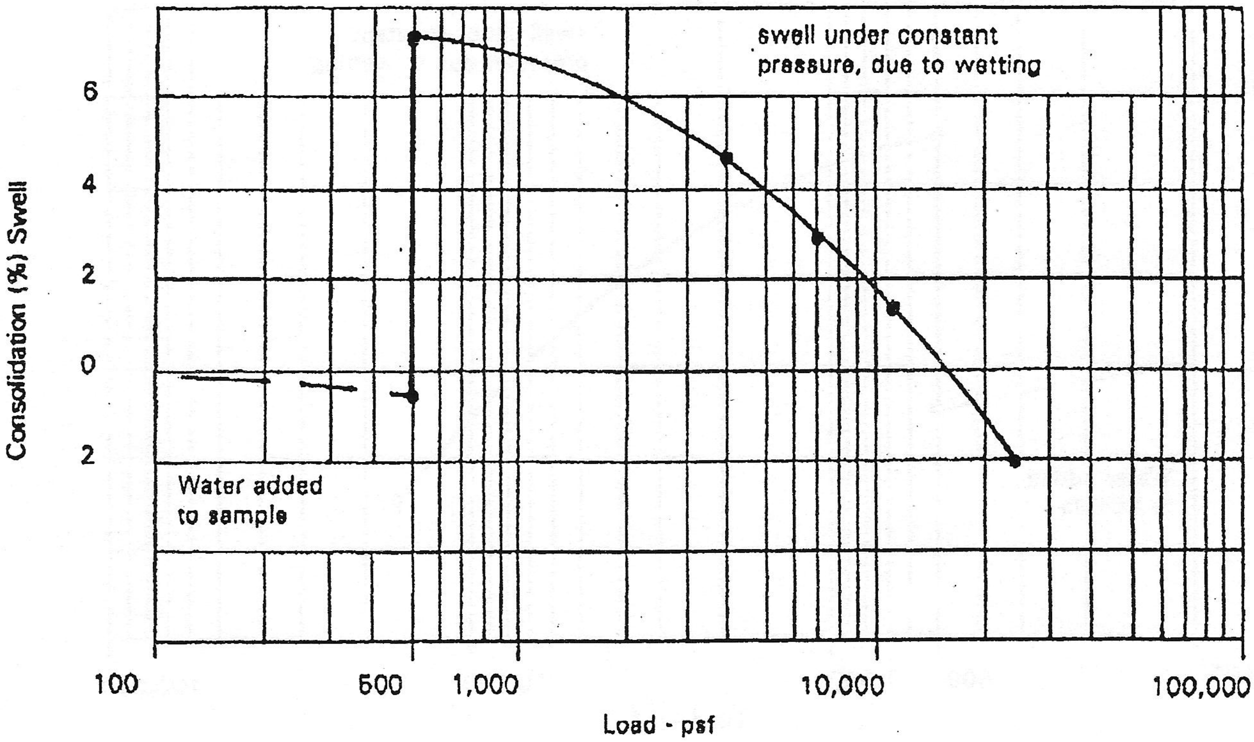
Sample of Claystone from Lot 1 at depth 14 feet.
Natural Moisture Content 22.4% Natural Dry Density 103 pcf



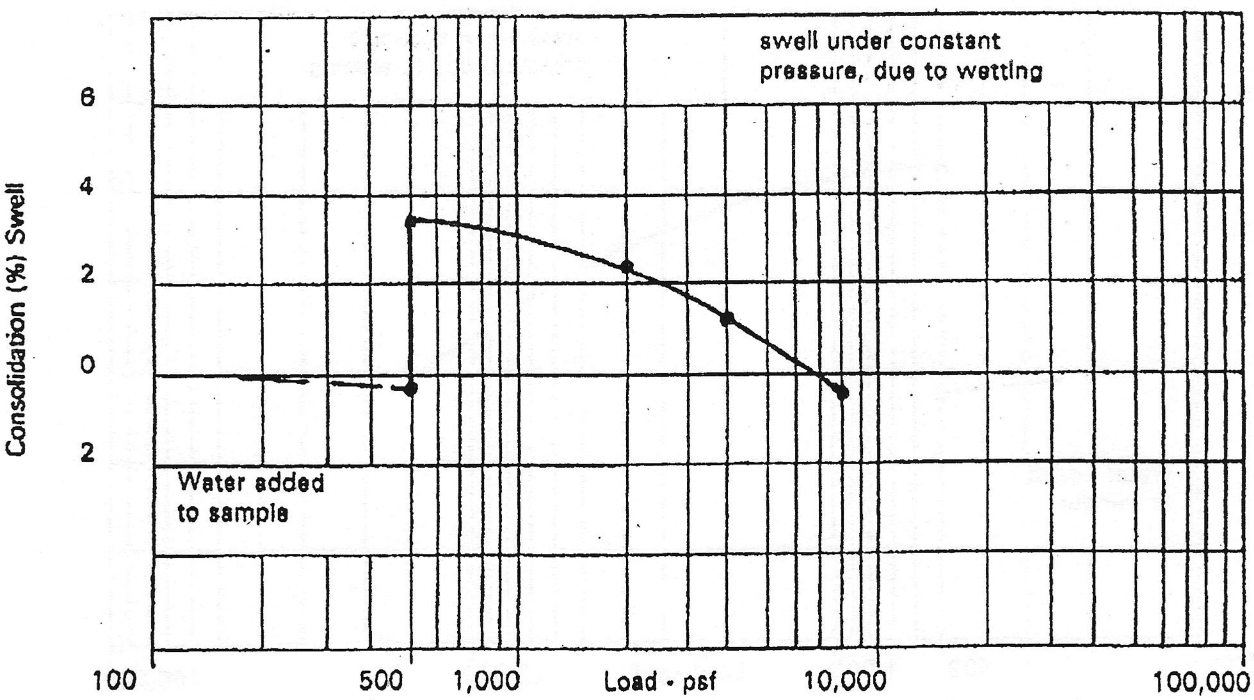
Sample of Clay, sandy from Lot 2 at depth 7 feet.
Natural Moisture Content 19.3% Natural Dry Density 109 pcf



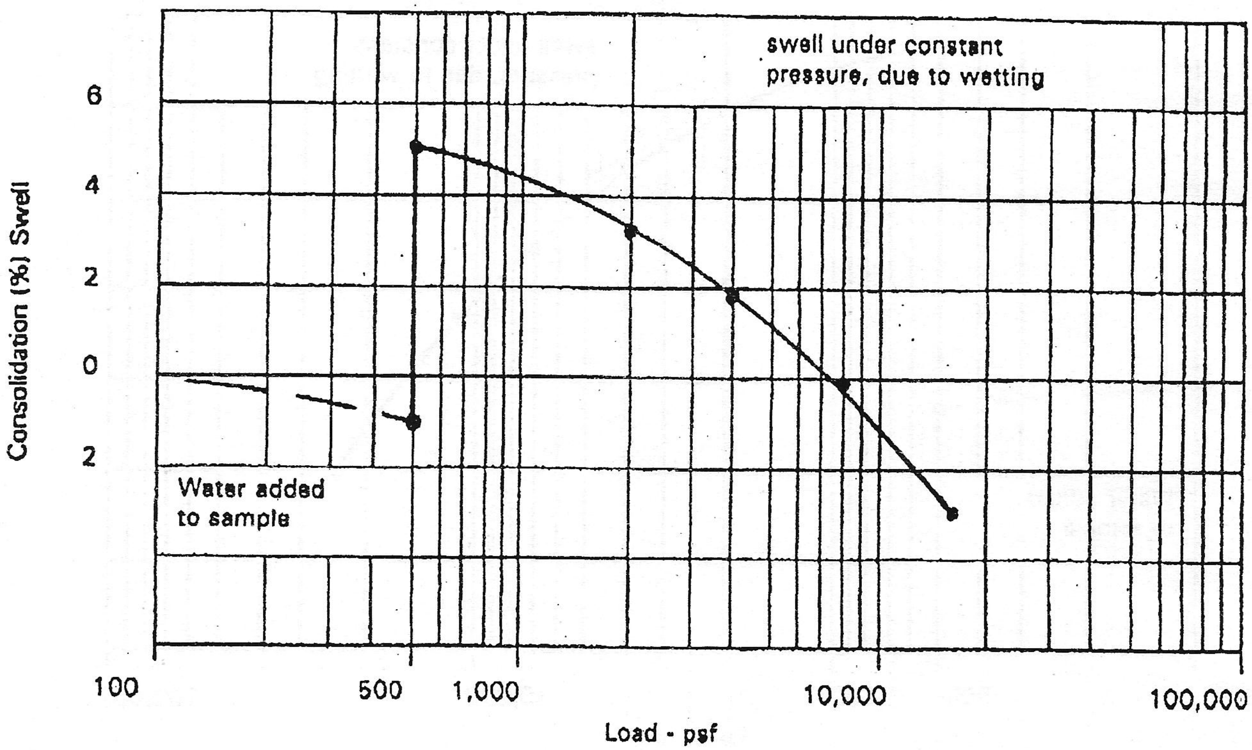
Sample of Clay, sandy from Lot 3 at depth 3 feet.
Natural Moisture Content 18.3% Natural Dry Density 112 pcf



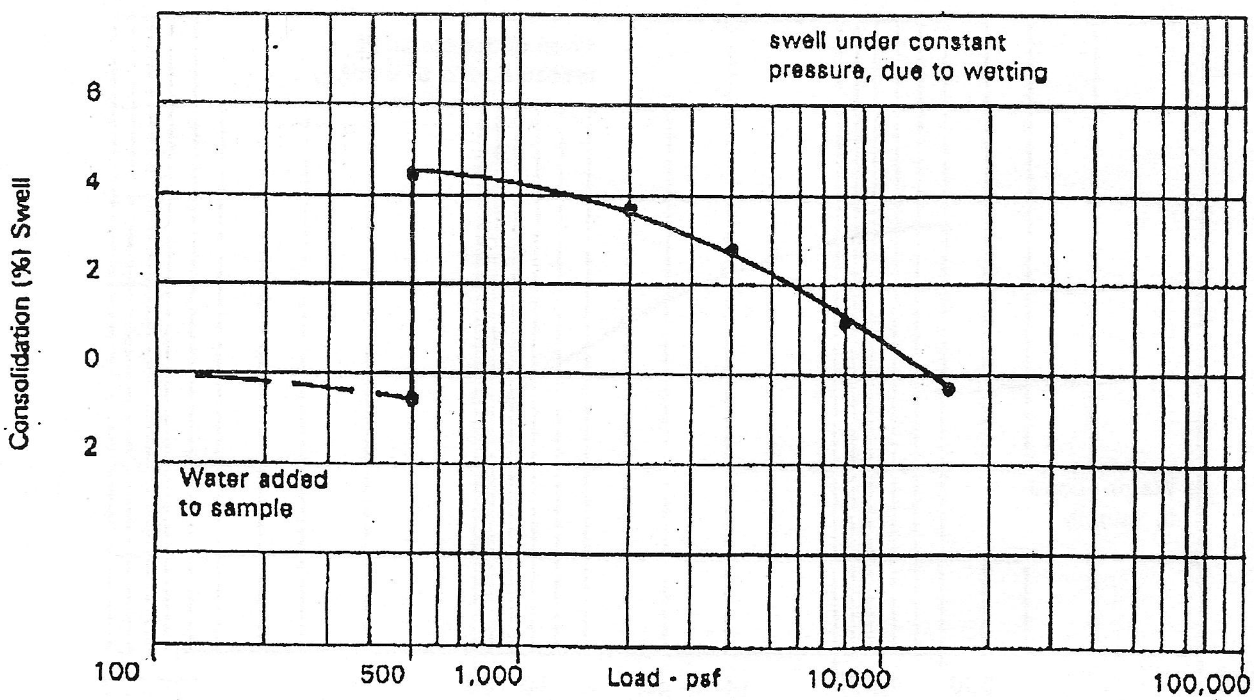
Sample of Clay, sandy from Lot 4 at depth 3 feet.
Natural Moisture Content 14.2% Natural Dry Density 112 pcf



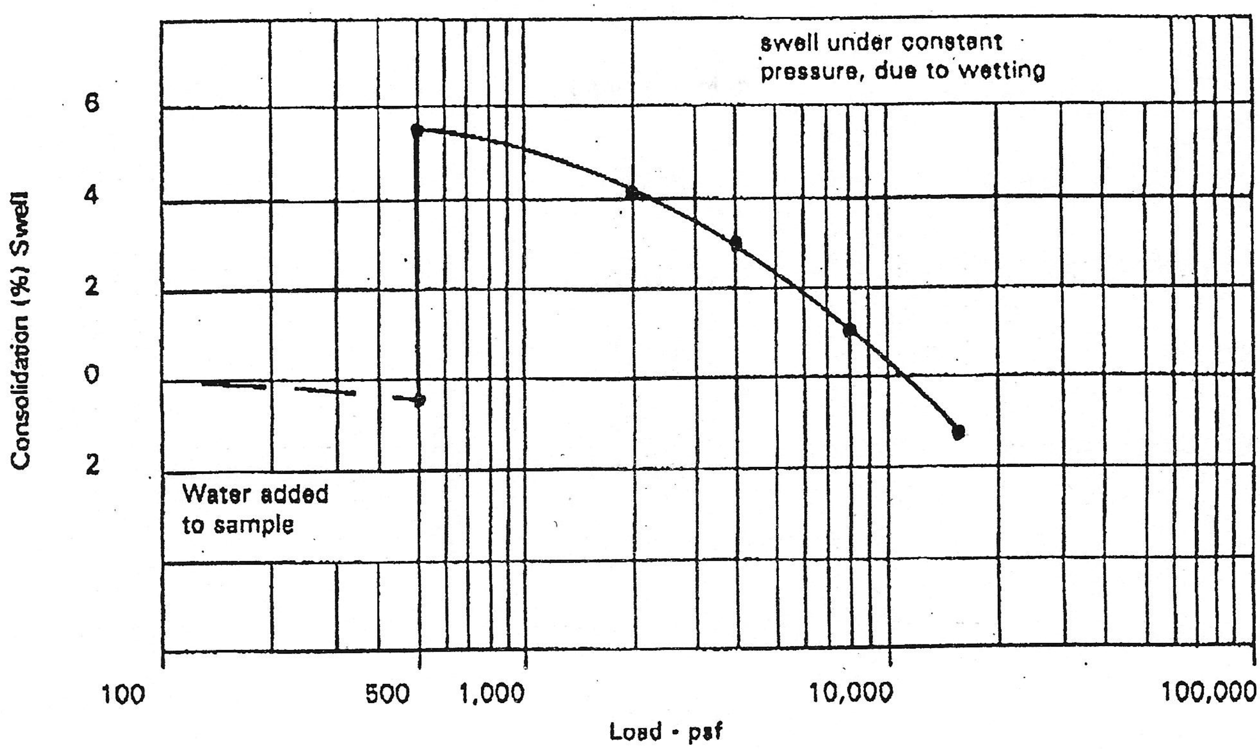
Sample of Clay, sandy from Lot 4 at depth 7 feet.
Natural Moisture Content 15.3% Natural Dry Density 114 pcf



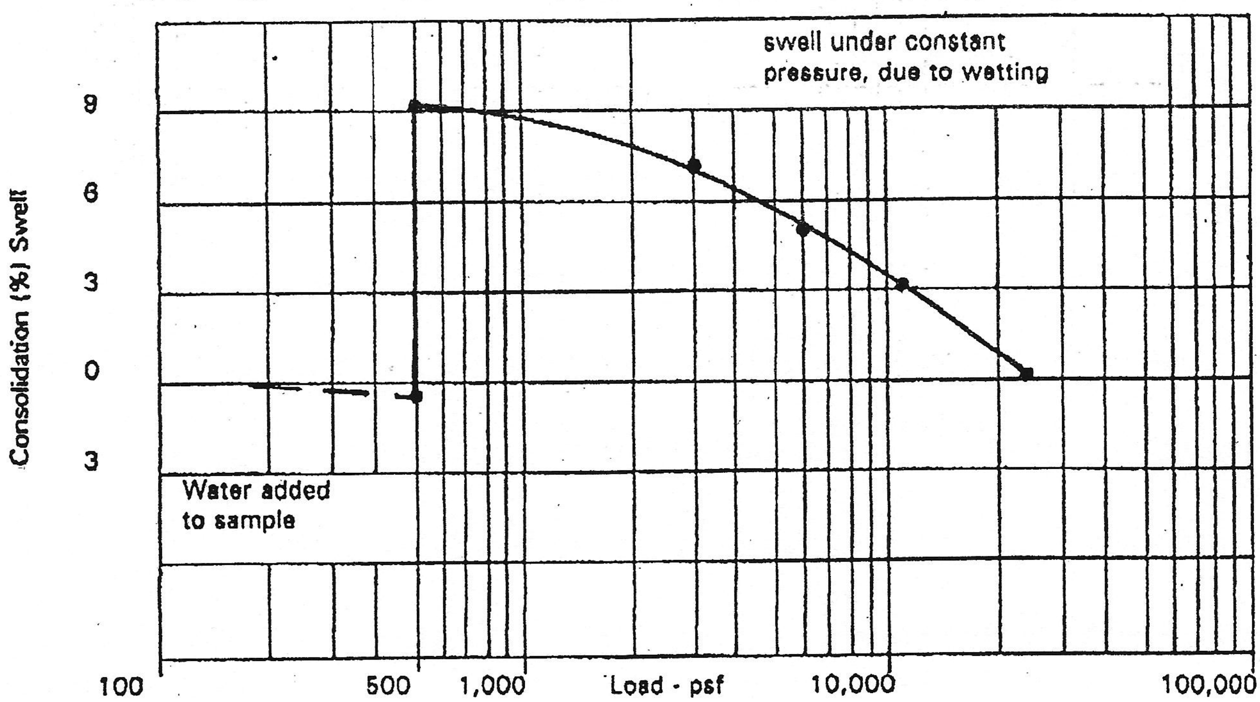
Sample of Clay, slightly sandy from Lot 5 at depth 7 feet.
Natural Moisture Content 17.2% Natural Dry Density 118 pcf



Sample of Clay, sandy from Lot 6 at depth 3 feet.
Natural Moisture Content 12.2% Natural Dry Density 116 pcf



Sample of Clay, slightly sandy from Lot 7 at depth 3 feet.
Natural Moisture Content 17.3% Natural Dry Density 114 pcf



Sample of Claystone from Lot 7 at depth 7 feet.
Natural Moisture Content 21.2% Natural Dry Density 108 pcf