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CANADIAN MANUAL ON FOUNDATION ENGINEERING

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CANADIAN MANUAL

ON

FOUNDATION ENGINEERING

1975

PREFACE

The Canadian Manual on Foundation Engineering was prepared under the auspices of the Associate Committee on the National Building Code by the Subcommittee on Foundations of the Standing Committee on Structural Design.

It provides a "state of the art" report on foundation engineering containing recommended procedures for the design, installation and construction of foundations. It is intended to assist the enforcing official and the designer in satisfying the intent of Section 4.2 (Foundations) of the National Building Code of Canada 1975.

There are eight chapters in all. Chapter 1 is of an introductory nature, Chapters 2 and 3 deal with the definitions of terms and the classification systems for soils and rocks, Chapters 4 to 7 contain the various technical aspects of foundation engineering, and Chapter 8 comprises commentaries on some special aspects of foundation engineering.

Although the Manual was originally intended as a supplementary document to the Foundations Section of the 1975 edition of the National Building Code, no decision has yet been made on its final format and source of publication. The Associate Committee has, therefore, agreed to release the material in its preliminary form in advance of this decision in order to obtain wide public review.

The ACNBC is grateful for permission to use a number of illustrations from outside sources the origin of which are noted in the text or figure captions.

Comments and suggestions on the technical content of the Manual and on its value as a background document to the National Building Code of Canada are welcomed. Such comments should be addressed to: The Secretary, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, Ontario K1A 0R6.

CANADIAN MANUAL ON FOUNDATION ENGINEERING

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SCOPE

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CHAPTER 1

SCOPE

1.1 GENERAL

The Canadian Manual of Foundation Engineering was prepared as a supplementary document to Section 4.2 Foundations of the National Building Code 1975. It provides recommended procedures to be followed in the design, installation and construction of foundations with a view to ensuring safety, quality, economy and fitness for purpose.

The Canadian Manual of Foundation Engineering provides

- the designer with methods for complying with the performance requirements of Section 4.2 Foundations, and
- the *authority having jurisdiction* with means of assessing the safety of the designs submitted for its approval, including guidance on inspection of construction practices.

In the preparation of this Manual it was recognized that it was neither appropriate nor possible to present the subject matter in strict specification form in the manner used for Codes invoked by Sections 4.3, 4.4, 4.5 and 4.6 of the National Building Code. This stems from the fundamental difference in the controlability of in-place geological materials and conditions compared with that of manufactured or preselected materials brought to the construction site to fulfill specific design purposes. In addition, primarily because of the infinite variety of materials and conditions that may be encountered, foundation engineering is a less precise science than structural design, and although great strides have been made in testing and analysis, supported by field observations, foundation engineering remains, to an important extent, an art based upon experience and judgement. The material in this Manual is presented therefore in a descriptive form as a series of suggested rather than mandatory procedures which reflect sound and safe techniques.

1.2 FORMAT

The Manual has been arranged in eight chapters, which apart from Chapter 1 present various aspects of foundation engineering.

Chapters 2 & 3 cover the basic matters of defining some of the terms used both in the Manual and Section 4.2 National Building Code, the presentation of symbols used, and classification systems for soils and rocks.

Chapter 4 covers procedures used in subsurface explorations by which samples required for testing and other basic field information needed for design are obtained.

Chapters 5, 6 and 7 cover the subjects of excavations and retaining structures, shallow foundations and deep foundations respectively. Each of these chapters present, in general,

- a basic design method of acceptable quality,
- alternative design methods of increasing sophistication and technical quality,
- discussions on the limits of validity of each method and references in which the methods are discussed in greater detail, and
- comments on specific construction problems where such problems govern the design or the quality of the foundation.

Chapter 8 contains a number of commentaries which cover certain aspects of foundation engineering that warrant separate detailed discussions not appropriate to the treatment of material in the previous chapters. Some of these present assessments of the limitations and errors inherent in techniques that are widely used and accepted such as the standard penetration test and the determination of relative density of cohesionless soil. Some present information on problems not directly related to the static loading of soil by a structure, but which may lead to intolerable differential movements if not accommodated in design, such as the effect of water content change on swelling and shrinking clays, and that of induced freezing conditions on frost-susceptible soils. One deals with the use of pile driving formulas for the determination of pile bearing capacity, a practice not advocated in this Manual; another with lateral loading of piles, a complicated subject increasingly encountered and considered in building construction. The subject of earthquake resistant design of foundations is also treated, and a final commentary presents an assessment of the use of the pressuremeter, a very useful exploratory technique which has found wide acceptance in Europe but which is still relatively little used in North America.

A decimal numbering system similar to that in the National Building Code has been used throughout. It follows the logical subdivision of topics treated in each chapter, and its main purpose is to facilitate referencing. Chapters 5, 6 and 7 bear the same titles as Subsections 4.2.5., 4.2.6., and 4.2.7. in Section 4.2 but correlation of individual articles is not intended.

1.3 LIMITATION

The methods presented in the Canadian Manual of Foundation Engineering are applicable to most design problems. It should be understood, however, that strict use of these methods will not always yield the best technical or most economical solutions. Moreover, the design of unusual structures or the occurrence of unusual subsurface conditions may require the use of novel design approaches or methods of analysis beyond the scope of this Manual.

1.4 EXPERIENCE AND JUDGEMENT

Much of the material in this Manual is simple and obvious, and so it should be, since neglect of the obvious causes more problems than an inability to fathom the obscure. Nevertheless, in the engineering application of the methods shown, neither this Manual nor the textbooks and papers to which it refers should be considered a substitute for the experience and judgement of a person familiar with the complexities of foundation practice.

CHAPTER 2

DEFINITIONS, SYMBOLS AND UNITS

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CHAPTER 2

DEFINITIONS, SYMBOLS AND UNITS

2.1. DEFINITIONS

The following is a list of definitions of some of the terms commonly used in foundation design and construction which are referred to in this Manual and Section 4.2. Foundations of NBC 1975. Other terms are defined or explained where they are introduced in the text. With the exception of the headings of various paragraphs such terms are the only ones that appear in italics.

Adfreezing means the adhesion of *soil* to a *foundation unit* resulting from the freezing of *soil* water. (Also referred to as "frost grip.")

Bearing pressure, allowable means the maximum pressure that may be safely applied to a *soil* or *rock* by the *foundation unit* considered in design under expected loading and subsurface conditions.

Bearing pressure, design means the pressure applied by a *foundation unit* to a *soil* or *rock* and which is not greater than the *allowable bearing pressure*.

Bearing surface means the contact surface between a *foundation unit* and the *soil* or *rock* upon which it bears.

Caisson (See *pile*).

Deep foundation means a *foundation unit* that provides support for a building by transferring loads either by end-bearing to *soil* or *rock* at considerable depth below the building, or by adhesion or friction, or both, in the *soil* or *rock* in which it is placed. *Piles* are the most common type of *deep foundation*.

Excavation means the space created by the removal of *soil*, *rock* or *fill* for the purposes of construction.

Fill means *soil*, *rock*, rubble, industrial waste such as slag, organic material or a combination of these that is transported and placed on the natural surface of *soil* or *rock* or organic terrain. It may or may not be compacted.

Foundation means a system or arrangement of *foundation units* through which the loads from a building are transferred to supporting *soil* or *rock*.

Foundation unit means one of the structural members of the *foundation* of a building such as a footing, raft or *pile*.

Frost action means the phenomenon that occurs when water in *soil* is subjected to freezing which, because of the water ice phase change or ice lens growth, results in a total volume increase or the build-up of expansive forces under confined conditions or both, and the subsequent thawing that leads to loss of *soil* strength and increased compressibility.

Grade means the average level of finished ground adjoining a building at all exterior walls.

Groundwater means a free standing body of water in the ground.

Groundwater, artesian means a confined body of water under pressure in the ground.

Groundwater level (groundwater table) means the top surface of a free standing body of water in the ground.

Groundwater, perched means a free standing body of water in the ground extending to a limited depth.

Load, allowable means the maximum load that may be safely applied to a *foundation unit* considered in design under expected loading and subsurface conditions.

Load, design means the load applied to a *foundation unit* and which is not greater than the *allowable load*.

Peat means a highly organic *soil* consisting chiefly of more or less fragmented remains of vegetable matter sequentially deposited.

Pile means a slender *deep foundation unit*, made of materials such as wood, steel or concrete, or combination thereof, which is either premanufactured and placed by driving, jacking, jetting or screwing, or cast-in-place in a hole formed by driving, excavating or boring. (Cast-in-place bored *piles* are often referred to as *caissons* in Canada.)

Rock means that portion of the earth's crust which is consolidated, coherent and relatively hard and is a naturally formed, solidly bonded, mass of mineral matter which cannot readily be broken by hand.

Shallow foundation means a *foundation unit* which derives its support from *soil* or *rock* located close to the lowest part of the building which it supports.

Soil means that portion of the earth's crust which is fragmentary, or such that some individual particles of a dried sample may be readily separated by agitation in water; it includes boulders, cobbles, gravel, sand, silt, clay and organic matter.

Subsurface investigation means the appraisal of the general subsurface conditions at a building site by analysis of information gained by such methods as geological surveys, *in situ* testing, sampling, visual inspection, laboratory testing of samples of the subsurface materials and *groundwater* observations and measurements.

2.2. SYMBOLS

The following is a list of symbols and abbreviations encountered in this Manual. As far as possible they agree with those widely recognized in foundation engineering and the geotechnical sciences. In some cases, however, where usage is not uniform in the literature and where identical symbols used for different parameters might otherwise lead to confusion new symbols or symbols with different subscripts have been introduced.

A	-	average cross-sectional area of pile
	-	cross-sectional area of cone (Dutch cone penetration test)
	-	anchor load
A_c	-	cross-sectional area of concrete pile
A_p	-	cross-sectional area of pile tip
A_s	-	effective surface area of anchorage
B	-	width of foundation
	-	width of excavation
C	-	constant representing energy losses in pile driving system
	-	critical point
C_c	-	compression index
C_{cr}	-	recompression index
C.P.V.	-	pressure and volume control unit (Pressuremeter test)
c	-	cohesive strength of clay
c_u	-	undrained shear strength of clay, apparent cohesion
c_{ua}	-	unit adhesion related to c_u

c_v - coefficient of consolidation

c_α - coefficient of secondary consolidation

D - diameter of anchorage, pile, rock socket, etc.

D' - depth of penetration of pile

D_e - inside diameter of cutting edge of sampling tube

D_f - depth of foundation, embedment of footing

D_{fe} - equivalent depth of foundation

D_i - inside diameter of sampling tube

D_o - outside diameter of sampling tube

D_r - relative density

d - depth factor

E - modulus of elasticity of pile material

E_b - average pressuremeter modulus for heterogeneous soil deposit (See E_p)

E_n - rated energy of pile driver hammer

E_p - stress-strain modulus from pressuremeter test

E_s - modulus of elasticity of steel
- stress-strain modulus of soil from compression test

E_v - stress-strain modulus of soil from vane test

e - void ratio
- efficiency of pile hammer
- penetration per blow (Dutch cone penetration test)
- eccentricity of load

e_B - eccentricity of load related to width of foundation

e_L - eccentricity of load related to length of foundation

F - friction force at base of retaining wall

F_c - average skin friction from cone tests

F_m - moment coefficient for horizontally loaded pile

F_n - total negative skin friction

F.I. - Freezing Index

FS - factor of safety

FS_b - factor of safety against base heave

FS_s - factor of safety against sliding

FS_{req} - factor of safety required

F_z - depth reduction factor related to swelling and shrinking soil

F_δ - deflection coefficient for horizontally loaded pile

f - settlement coefficient related to soil type and geometry of foundation (Pressuremeter test)

f_c - settlement coefficient (Critical point method)

f'_c - specified strength of concrete

f_f - ultimate skin friction on pile

f_{pe} - effective stress in concrete due to prestress after losses

f_{so} - stress after losses in prestressing steel

H - vertical height of retaining wall
- thickness of soil deposit
- depth of excavation
- fall of hammer

H_c - critical depth related to pile capacity in granular soils

H_s - depth of rock socket

H_w - head of water

h - height of masonry wall

I - moment of inertia of pile cross section

I_w - plasticity Index

i - area ratio of sampling tube, percent
- influence value of stress
- angle of slope of backfill with horizontal

i_c -
i_q - bearing capacity factors related to inclination of load
i_γ -

K - coefficient of earth pressure

K_A - coefficient of active earth pressure

K_b - bearing capacity factor related to depth and diameter of rock socket

K_d - bearing pressure coefficient related to depth of foundation in rock

K_f - anchorage coefficient related to soil type and density

K_g - bearing capacity factor (Pressuremeter test)

K_H - factor for horizontal component of earth pressure

K_o - coefficient of earth pressure at rest

K_p - coefficient of passive earth pressure

K'_p - reduced coefficient of passive earth pressure

K_s - coefficient of horizontal subgrade reaction

K_{sp} - bearing pressure coefficient related to spacing of discontinuities in rock

K_v - factor for vertical component of earth pressure

k - permeability

L - length of foundation

L_p - length of pile

L_w - liquid limit

M - mass of hammer (Dutch cone penetration test)

M_p - moment in pile at depth z

m - coefficient of lateral stress

N - Standard Penetration Index

$\overline{N}$ - average Standard Penetration Index

N_b - stability factor related to geometry of excavation

N_{cone} - dynamic cone penetration resistance factor

N_c -
N_q - bearing capacity factors related to angle of shearing resistance ϕ
N_γ -

N_c^* -
N_q^* - bearing capacity factors for piles

n - empirical factor (Hiley Pile Formula)
- depth ratio

n_h - constant of horizontal subgrade reaction related to soil density

P - mass of pipe (Dutch cone penetration test)
- lateral load on pile head
- governing combination of loads multiplied by appropriate load factors (See Section 4.1 NBC)

P_A - active earth pressure

P_H - horizontal component of load on retaining structure

P_p - passive earth pressure

P_v - vertical component of load on retaining structure

P_{ult} - horizontal load capacity of vertical pile

p_c - preconsolidation pressure

p_f - yield pressure from pressuremeter test

p_i - in place horizontal pressure from pressuremeter test

p_L - limit pressure from pressuremeter test

p_{Le} - equivalent limit pressure from pressuremeter test

p_o - total overburden pressure

p'_o - effective overburden pressure

Q	-	applied pile load, test load
Q_a	-	allowable load
Q_f	-	failure load, ultimate load
Q_{fs}	-	ultimate shaft resistance related to adhesion in clay
Q_L	-	line load (vertical)
Q_p	-	point load (vertical) on foundation
q	-	design pressure, applied pressure
	-	surcharge per unit area
q'	-	distributed stress below pile group
q_a	-	allowable bearing pressure
q_{cone}	-	cone resistance (Static cone penetration test)
q_d	-	design pressure
q_{fp}	-	ultimate point resistance
q_{net}	-	net design bearing pressure
q_o	-	at rest horizontal stress in rock
q_u	-	unconfined compressive strength of cohesive soil
q_{u-core}	-	average unconfined compressive strength of rock cores
R	-	resultant load, force
R_d	-	unit resistance (Dutch cone penetration test)
R_h	-	height of resultant force above base of retaining wall
R_p	-	average point resistance of static cone
R.Q.D.	-	Rock Quality Designation
r	-	radius
	-	distance from point load, Q_p
S	-	settlement
	-	pile set per blow
S_b	-	allowable bond strength between grout and rock for rock anchorages
S_g	-	settlement of pile group
S_p	-	pile spacing
S_c	-	bearing capacity factors related to geometry of foundation
S_q	-	bearing capacity factors related to geometry of foundation
S_γ	-	bearing capacity factors related to geometry of foundation
S_1	-	settlement of 1 ft sq loading plate
S.P.T.	-	Standard Penetration Test

T	-	relative stiffness of pile-soil system
T_a	-	allowable load on anchorage
T_c	-	computed allowable load on anchorage
T_n	-	consolidation time factor
T_t	-	test load capacity of anchorage
t_n	-	time related to degree of consolidation
U%	-	degree of consolidation
V	-	volume
V_m	-	volume of water (Pressuremeter test)
V_o	-	initial volume of measuring cell (Pressuremeter test)
W	-	weight of pile hammer
	-	weight of soil mass
W_p	-	weight of pile
x	-	horizontal distance from line load, Q_L
Y	-	horizontal displacement of retaining wall
z	-	depth below ground surface
α	-	reduction factor for earth anchorages related to undrained shear strength of soil
α_c	-	coefficient of compressibility (Static cone penetration test)
α_g	-	coefficient of settlement for pile group
α_m	-	coefficient of settlement related to structure of rock mass
α_p	-	structure factor related to settlement (Pressuremeter test)
α_ϕ	-	skin friction coefficient related to ϕ
β	-	density coefficient (Static cone penetration test)
β	-	angle of back of retaining wall with horizontal
Δ_p	-	pressure change
δ	-	differential settlement between columns
	-	elastic deformation of pile shaft
δ_a	-	effective angle of friction between soil and retaining structure, active pressure case

δ_f - effective angle of friction between clay and pile shaft

δ_p - effective angle of friction between soil and retaining structure, passive pressure case

δ_z - horizontal deflection of pile at depth z

ε_c - strain in concrete at failure

γ - unit weight of material

γ' - effective unit weight of material

γ_d - dry unit weight of material

γ_{sub} - submerged unit weight of material

γ_w - unit weight of water

λ_2 -
λ_3 - $\Big\}$ form factors related to settlement (Pressuremeter test)

ν - Poisson's ratio

ϕ - angle of shearing resistance
- performance factor (Section 4.1 NBC)

ϕ_1, ϕ_2 - exit gradient factors, groundwater seepage

ϕ' - effective angle of shearing resistance

ϕ_u - angle of shearing resistance for the undrained condition

σ - stress

σ'_h - effective horizontal stress

σ'_v - effective vertical stress

σ_z - vertical stress at depth z

τ_a - allowable bond strength between concrete and rock

τ_f - ultimate skin friction from the pressuremeter test

τ_n - unit negative skin friction

τ_s - effective shaft friction

$\tau_{s\ avg}$ - average effective shaft friction

2.3. UNITS

Although it is recognized that the use of metric units is not only highly desirable but will become official in Canada within a short space of time, only Imperial units appear in this Manual. The reasons are that the various chapters of this Manual were prepared using the Imperial system of units simply because this is the system presently in use in this field of engineering, and that the constrictions of time have not made it possible to convert all of the pertinent material to the metric system. Where it is necessary to convert to or from S.I. or other metric units the user is directed to CSA Standard Z234.1 "Metric Practice Guide" and CSA Standard Z234.2 "The International System of Units (S.I.)".

There is one exception: Commentary 8.8 on The Pressuremeter Test was prepared using metric units because the available literature on the subject is written using that system.

The following is a list of units and abbreviations generally encountered in geotechnical and foundation engineering that are used in this Manual.

blows/ft	-	blows per foot
ft	-	foot (feet)
ft lb	-	foot pound(s)
ft sq	-	foot (feet) square
in.	-	inch(es)
lb	-	pound(s)
lb/sq ft	-	pound(s) per square foot
lb/cu ft	-	pound(s) per cubic foot
lb/sq in.	-	pound(s) per square inch
No	-	number(s)
sq ft	-	square foot (feet)
ton	-	ton(s)
ton/sq ft	-	ton(s) per square foot
ton/cu ft	-	ton(s) per cubic foot

Metric units

* bar	-	bar(s)
mm	-	millimeter
cm	-	centimetre
cm^3	-	cubic centimetre

* 1 bar $\simeq$ 1 ton/sq ft.

CHAPTER 3

IDENTIFICATION AND CLASSIFICATION OF SOILS AND ROCKS

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CHAPTER 3

IDENTIFICATION AND CLASSIFICATION OF SOILS AND ROCKS

3.1 CLASSIFICATION OF SOILS

3.1.1. *GENERAL*

Soil is that portion of the earth's crust which is fragmentary, or such that some individual particles of a dried sample may be readily separated by agitation in water; it includes boulders, cobbles, gravel, sand, silt, clay and organic matter.

There are three major groups of soils:

Coarse-grained Soils - particles of which are large enough to be visible to the naked eye. They include gravels and sands and are generally referred to as *cohesionless* or *non-cohesive* soils.

Fine-grained Soils - particles of which are not visible to the naked eye. They are identified primarily on the basis of their behaviour in a number of simple indicator tests. They include silts and clays. Clays are generally referred to as *cohesive* soils.

Organic Soils - which are those having a high natural organic content.

3.1.2. *CLASSIFICATION SYSTEM*

For purposes of this Manual the soils are identified and classified according to their particle size and distribution (coarse-grained soils) and their plasticity (fine-grained soils) based on the "Unified Soil Classification System." The main aspects and features of this system are presented in Table 3.1.

Note: Particles or rock fragments larger than those included in the Unified Soil Classification System are recognized. They are *cobbles* and *boulders*. (See 3.1.3.1.(1))

3.1.3. *FIELD IDENTIFICATION PROCEDURES*

The following are procedures and tests which may be carried out in the field and by which soils may be identified and described.

3.1.3.1. *Coarse-grained Soils or Fractions*

Coarse-grained soils are most easily identified in the field because the individual particles are large enough to be visible to the naked eye. (In general, the smallest particles that may be distinguished individually are approximately 0.003 in (0.075 mm) in diameter, which corresponds closely with the size of the openings of the N° 200 sieve used in the laboratory identification test.)

(1) *Grain size*

Coarse-grained soils are identified on the basis of grain size as follows:

Sand means particles smaller than ¼ in. and larger than 0.003 in. in diameter

Gravel means particles smaller than 3 in. and larger than ¼ in. in diameter

Cobbles means particles smaller than 8 in. and larger than 3 in. in diameter

Boulders means particles larger than 8 in. in diameter.

(2) *Other physical properties* of coarse-grained or cohesionless soils which may influence their engineering characteristics should also be identified. They are:

a) *Grading*, which is a term describing particle size distribution. A soil that has a predominance of particles of one size is referred to as *poorly-graded*, whereas as soil that has particles of sizes assorted over a wide range with no one size predominating is referred to as *well-graded*.

b) *Shape & surface conditions of grains* Particles may be platy, elongated or equidimensional, and they may be angular, subangular or rounded. *Angular* particles have sharp edges and relatively plane sides with unpolished surfaces; *subangular* particles are similar to angular particles but have rounded edges; *rounded* particles have smoothly curved surfaces and no edges.

c) *Density*, which is a term describing the compactness of the soil and is interpreted from the results of a penetration test carried out in accordance with CSA A119.1-60 "Code for Split-barrel Sampling of Soils". Density and penetration values are related in Table 3.2.

TABLE 3.2

Density of sands from Standard Penetration Tests

Density	Standard Penetration Test N-values. (Blows per foot)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	Over 50

d) *Structural characteristics* of the undisturbed soil such as the presence or absence of a systematic arrangement of the grains or grain size components in layers, and evidence of weathering or cementation. Thickness, orientation and distortion of layers in included.

e) *Colour* of soil or particles.

f) *Odour* if any, which gives evidence of the presence of organic material.

3.1.3.2. *Fine-grained Soils or Fractions*

These procedures are to be performed on the minus N^{o} 40 sieve size particles, approximately 1/64 in. in diameter. For field classification purposes screening is not intended; simply remove by hand the coarse particles that interfere with the tests.

(1) *Dilatancy* (Reaction to shaking)

After removing particles larger than N^{o} 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear

TABLE 3.1

<table>
<tr><th colspan="9">UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION</th></tr>
<tr><th colspan="6">FIELD IDENTIFICATION PROCEDURES
(Excluding particles larger than 3 inches and basing fractions on estimated weights)</th><th>GROUP SYMBOLS</th><th>TYPICAL NAMES</th><th>INFORMATION REQUIRED FOR DESCRIBING SOILS</th></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS
More than half of material is <u>larger</u> than No 200 sieve size
(The No. 200 sieve size is about the smallest particle visible to the naked eye)</td><td rowspan="4">GRAVELS
More than half of coarse fraction is larger than No 4 sieve size</td><td rowspan="2">CLEAN GRAVELS
(Little or no fines)</td><td colspan="3">Wide range in grain size and substantial amounts of all intermediate particle sizes</td><td>GW</td><td>Well graded gravels, gravel-sand mixtures, little or no fines</td><td rowspan="8">Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.

For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.

EXAMPLE:-
<u>Silty sand</u>, gravelly; about 20% hard, angular gravel particles ½-in maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing.</td><td>GP</td><td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES
(Appreciable amount of fines)</td><td colspan="3">Non-plastic fines (for identification procedures see ML below)</td><td>GM</td><td>Silty gravels, poorly graded gravel-sand-silt mixtures.</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>Clayey gravels, poorly graded gravel-sand-clay mixtures.</td></tr>
<tr><td rowspan="4">SANDS
More than half of coarse fraction is smaller than No. 4 sieve size
(For visual classifications, the ¼" size may be used as equivalent to the No. 4 sieve size.)</td><td rowspan="2">CLEAN SANDS
(Little or no fines)</td><td colspan="3">Wide range in grain sizes and substantial amounts of all intermediate particle sizes.</td><td>SW</td><td>Well graded sands, gravelly sands; little or no fines.</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>SP</td><td>Poorly graded sands, gravelly sands; little or no fines.</td></tr>
<tr><td rowspan="2">SANDS WITH FINES
(Appreciable amount of fines)</td><td colspan="3">Non-plastic fines (for identification procedures see ML below)</td><td>SM</td><td>Silty sands, poorly graded sand-silt mixtures</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below).</td><td>SC</td><td>Clayey sands, poorly graded sand-clay mixtures</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS
More than half of material is <u>smaller</u> than No. 200 sieve size.</td><td colspan="5">IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE</td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2"></td></tr>
<tr><td colspan="2" rowspan="4">SILTS AND CLAYS
Liquid limit less than 50</td><td>DRY STRENGTH (CRUSHING CHARACTERISTICS)</td><td>DILATANCY (REACTION TO SHAKING)</td><td>TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)</td></tr>
<tr><td>None to slight</td><td>Quick to slow</td><td>None</td><td>ML</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity</td><td rowspan="7">Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses.

For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.

EXAMPLE:-
<u>Clayey silt</u>, brown; slightly plastic; small percentage of fine sand, numerous vertical root holes; firm and dry in place; loess; (ML)</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Medium</td><td>CL</td><td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td></tr>
<tr><td>Slight to medium</td><td>Slow</td><td>Slight</td><td>OL</td><td>Organic silts and organic silt-clays of low plasticity</td></tr>
<tr><td colspan="2" rowspan="3">SILTS AND CLAYS
Liquid limit greater than 50</td><td>Slight to medium</td><td>Slow to none</td><td>Slight to medium</td><td>MH</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td></tr>
<tr><td>High to very high</td><td>None</td><td>High</td><td>CH</td><td>Inorganic clays of high plasticity, fat clays</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Slight to medium</td><td>OH</td><td>Organic clays of medium to high plasticity.</td></tr>
<tr><td colspan="3">HIGHLY ORGANIC SOILS</td><td colspan="3">Readily identified by color, odor, spongy feel and frequently by fibrous texture.</td><td>Pt</td><td>Peat and other highly organic soils.</td></tr>
</table>

from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil. Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

(2) *Dry strength* (Crushing characteristics)

After removing particles larger than N° 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the clay fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for inorganic clays of high plasticity. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

(3) *Toughness* (Consistency near plastic limit)

After removing particles larger than the N° 40 sieve size, a specimen of soil about one-half inch cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached. After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. The tougher the thread near the plastic limit the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line. Fig. 3.1.

Highly organic clays have a very weak and spongy feel at the plastic limit.

(4) *Other physical properties* of fine-grained soils which may influence their engineering characteristics should also be identified. They are

a) *Consistencies* of cohesive soils at natural water content which may be related to the approximate undrained shear strength as indicated in Table 3.3.

TABLE 3.3

Consistencies of Cohesive Soils

Consistency	Field Identification	Approximate undrained shear strength* lb/sq ft
Very soft	Easily penetrated several inches by the fist	less than 250
Soft	Easily penetrated several inches by the thumb	250-500
Firm	Can be penetrated several inches by the thumb with moderate effort	500-1000
Stiff	Readily indented by the thumb but penetrated only with great effort	1000-2000
Very Stiff	Readily indented by the thumbnail	2000-4000
Hard	Indented with difficulty by the thumbnail	Over 4000

*The undrained shear strength is taken as ½ of the compressive strength

b) *Structural characteristics* of the undisturbed soil such as the presence or absence of a systematic arrangement of grain size components in layers, or cracks, fissures or slickensides and evidence of weathering or cementation. Thickness, orientation and distortion of layers is included.

c) *Colour*

d) *Odour* - if any, - which gives evidence of the presence of organic material.

3.1.3.3. *Organic Soils*

These are readily identified by colour, odour, spongy feel and frequently by fibrous texture.

3.1.4. *LABORATORY IDENTIFICATION TESTS*

3.1.4.1. *Grain-size Tests*

In the laboratory, grain-size tests are carried out according to the Standard Method for "Particle-size Analysis of Soils" A.S.T.M. D422-63(1972). This test method includes procedures for analysis of coarse-grained soils or fractions larger than 0.075 mm by sieving, and the analysis of fine-grained soils or fractions by the hydrometer test. (0.075 mm is approximately 0.003 in.)

3.1.4.2. *Atterberg Limits*

The range of water content over which a fine-grained soil is plastic is an important indicator of its probable engineering behaviour. The Atterberg limits defining these water contents are determined in accordance with the Standard Methods of Test for "Liquid Limit of Soils" ASTM D423-66(1972) and for "Plastic Limit and Plasticity Index of Soils" ASTM D424-59(1971).

Note:- Preparation of soil for these tests in accordance with the Standard Method for "Dry Preparation of Soil Samples for Particle-size Analysis and Determination of Soil Constants" ASTM D421-58 (1972) is not appropriate for testing clays of medium to high plasticity. The liquid limit should be determined on such samples prepared according to Procedure B of the Standard Method for "Wet Preparation of Soil Samples for Grain-Size Analysis and Determination of Soil Constants" ASTM D-2217-66 (1972).

Results of Atterberg Limits tests are referred to the Plasticity Chart shown in Fig 3.1 to aid in classification.

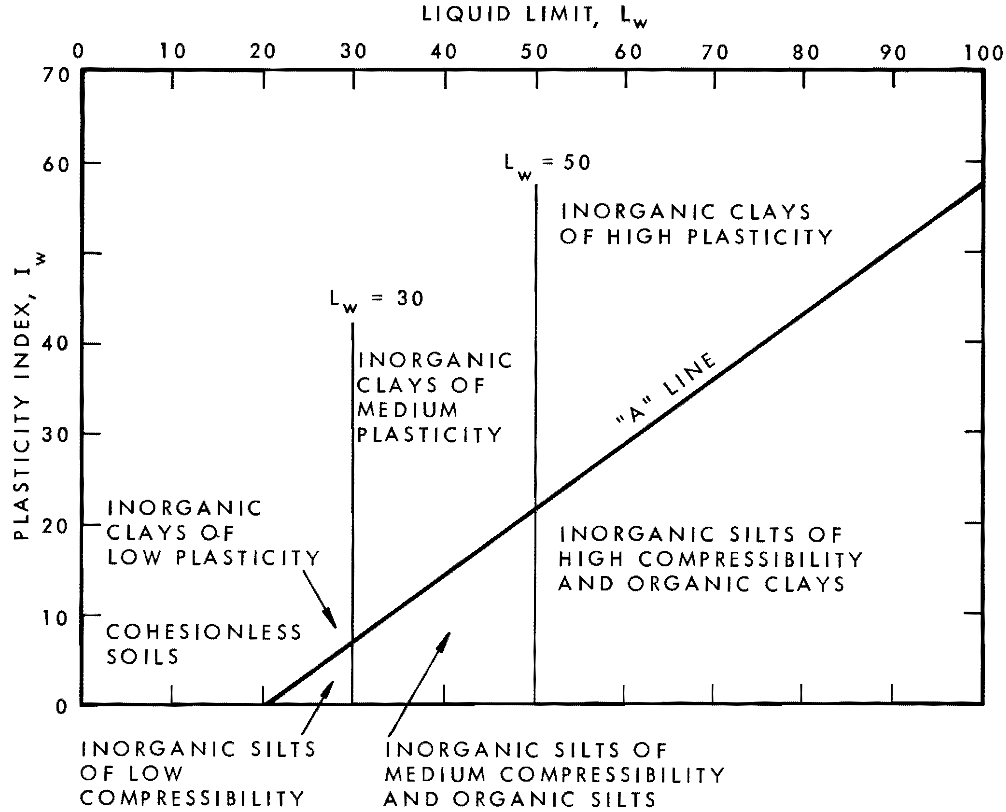


FIG 3.1 (After Casagrande)

PLASTICITY CHART

REFERENCES

Unified soil classification system, *Corps of Engineers, U.S. Army, Waterways Experiment Station, Vicksburg, Miss., Tech. Memo. 3-357, 1953.*

CASAGRANDE, A., 1947. Classification and identification of soils, *Proc. Am. Soc. Civil Engrs., 73, 783-810.*

CSA Standard A119.5 (1966). Recording of borehole and test pit information.

Guide to the field description of soils for engineering purposes, *Associate Committee on Soil and Snow Mechanics, National Research Council, NRC 3813, 1955, Tech. Memo 37.*

Standards

ASTM D421-58 (1972)	Dry preparation of soil samples for particle-size Analysis and determination of soil constants.
ASTM D422-63 (1972)	Particle size analysis of soils
ASTM D423-66 (1972)	Liquid limit of soils
ASTM D424-59 (1971)	Plastic limit and plasticity index of soils
ASTM D2217-66 (1972)	Wet preparation of soil samples for grain-size analysis and determination of soil constants.
CSA A119.1 1960	Code for split-barrel sampling of soils.

3.2 CLASSIFICATION OF ROCKS

3.2.1. *GENERAL*

Rock is that portion of the earth's crust which is consolidated, coherent and relatively hard, and is a naturally formed, solidly bonded mass of mineral matter which can not be readily broken by the hands nor will disintegrate on its first drying and wetting cycle.

3.2.1.1. *Rock Considered As Soil*

Some natural materials which geologically may be referred to correctly as rocks should be treated as soils. These materials are:

soft or weakly cemented rocks with unconfined compressive strength lower than 125 lb/sq in.

any material that can be dug by hand with a shovel or pneumatic spade;

cemented sands and gravels in which the cementing is discontinuous.

Some examples are:- very weak rocks such as chalk, marl and volcanic tuff; highly altered or crushed rocks; rocks with very closely spaced continous joints; and residual soils containing rock fragments.

3.2.2. *GEOLOGICAL CLASSIFICATION*

Rock is classified with respect to its geological origin as follows:

Igneous rocks

Igneous rocks, such as granite, diorite and basalt, are those formed by the solidification of molten material, either by intrusion at depth in the earth's crust or by extrusion at the earth's surface.

Sedimentary rocks

Sedimentary rocks, such as sandstone, limestone and shale, are those rocks formed by deposition,usually under water, of products derived by the disaggregation of pre-existing rocks.

Metamorphic rocks

Metamorphic rocks, such as quartzite, schist and gneiss, may be either igneous or sedimentary rocks which have been altered physically and sometimes chemically by the application of intense heat and pressure at some time in their geological history.

3.2.3. *STRUCTURAL FEATURES OF ROCK MASSES*

Geological structures generally have a significant influence on the rock mass properties. Some of the important features are described as follows:

Rock mass

Rock mass means an aggregate of blocks of solid rock material containing structural features which constitute mechanical discontinuities. Rock mass refers to any *in situ* rock with all inherent geomechanical discontinuities.

Rock material or intact rock

Rock material or intact rock means the consolidated aggregate of mineral particles forming solid material between structural discontinuities. Properties attributed to it refer to rock material free of geomechanical discontinuities.

Geomechanical or structural discontinuities

Geomechanical or structural discontinuities means all geological features which separate solid blocks of the rock mass, such as joints, faults, bedding planes, cleavage planes, shear zones, and solution cavities. These features constitute planes of weakness which reduce the strength of the rock mass appreciably.

Major discontinuities or major structures

Major discontinuities or major structures means those geological features constituting structural discontinuities which are sufficiently well developed and continuous that shear failure along them would involve little or no shearing of intact rock material.

3.2.4. *ENGINEERING PROPERTIES OF ROCK MASSES*

The quality of a rock mass for foundation purposes depends mainly upon the strength of rock material and on the spacing, the nature (width, roughness, waviness, weathering, etc.) and the orientation of discontinuities. Classification of rock according to some of these properties is given in the following paragraphs.

3.2.4.1. *Classification of Rock with Respect to Strength*

The strength of rock material varies from *very high* to *very low* and may be related to the unconfined compressive strength as indicated:

very high strength means rock much stronger than concrete, with a compressive strength greater than 32,000 lb/sq in.;

high strength means rock stronger than concrete, with a compressive strength from 8,000 lb/sq in. to 32,000 lb/sq in.;

medium strength means rock comparable to concrete with a compressive strength from 2,000 lb/sq in. to 8,000 lb/sq in.;

low strength means rock comparable to brick masonry with a compressive strength from 500 lb/sq in. to 2,000 lb/sq in.;

very low strength means rock weaker than brick masonry with a compressive strength from 125 lb/sq in. to 500 lb/sq in.

Note:- Rocks with compressive strengths lower than 125 lb/sq in. should be treated as soils. (See 3.2.1.1.)

3.2.4.2. *Classification of Rock Mass with Respect to the Spacing of Discontinuities*

The spacing in a given system varies from *very wide* to *very close* as indicated:

very wide spacing denotes a system of discontinuities with an average spacing greater than 10 ft;

wide spacing denotes a system of discontinuities with an average spacing from 3 ft to 10 ft;

moderately close spacing denotes a system of discontinuities with an average spacing from 1 ft to 3 ft;

close spacing denotes a system of discontinuities with an average spacing from 2 in to 1 ft;

very close spacing denotes a system of discontinuities with an average spacing smaller than 2 in.

3.2.4.3. *Nature and Orientation of Rock Discontinuities*

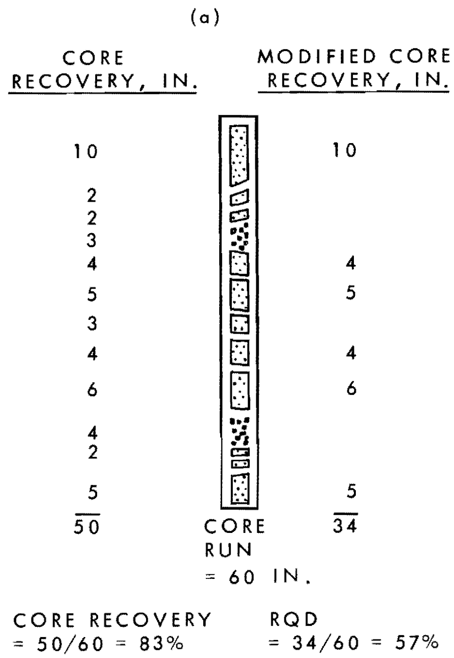
For foundation purposes, the nature of rock discontinuities may be expressed in terms of their width, the degree of weathering of rock contact faces, and the character of infilling materials.

In addition to the strength of rock material, and the spacing and nature of discontinuities, the quality of a rock mass for foundation purposes is affected by the orientation of discontinuities with respect to the applied load. A rock mass is said to contain adversely oriented discontinuities if under the action of the resultant foundation load the minimum resistance to sliding occurs when the sliding surface is considered to be along these discontinuities.

3.2.5. *ROCK QUALITY DESIGNATION*

This is a general method by which the quality of the rock at a site based on the relative amount of fracturing and alteration is obtained.

The Rock Quality Designation (RQD) is based on a modified core recovery procedure which, in turn, is based indirectly on the number of fractures and the amount of softening or alteration in the rock mass as observed in the rock cores from a drillhole. Instead of counting the fractures, an indirect measure is obtained by summing the total length of core recovered by counting only those pieces of hard and sound core which are 4 in. or greater in length.



(b)

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0 - 25	VERY POOR
25 - 50	POOR
50 - 75	FAIR
75 - 90	GOOD
90 - 100	EXCELLENT

FIG 3.2

MODIFIED CORE RECOVERY AS AN INDEX OF ROCK QUALITY

From "Rock mechanics in enginnering practice" by STAGG & ZIENKIEWICZ, 1968.
Used with permission of J. Wiley & Sons, Inc.

An example is given in Fig 3.2 from a core run of 60 in. For this particular case the total core recovery is 50 in, yielding a core recovery of 83%. On the modified basis, only 34 in. are counted and the RQD is 57%.

If the core is broken by handling or during drilling (i.e. the fracture surfaces are fresh irregular breaks rather than natural joint surfaces), the fresh broken pieces are fitted together and counted as one piece. Some judgement is necessary in the case of thinly bedded sedimentary rocks and foliated metamorphic rocks, and the method is not so exact in these cases as it is for igneous rocks, thick-bedded limestones, sandstones, etc. However, the system has been applied successfully even for shales, although it is necessary to log the cores immediately upon removing them from the core barrel before air-slaking and cracking can begin.

The procedure obviously penalizes the rock where recovery is poor. This is appropriate because poor core recovery usually reflects poor quality rock. However, poor drilling equipment and techniques can also cause poor recovery. For this reason, double-tube core barrels of at least NX size (2 1/8 in. diameter) must be used, and proper supervision of drilling is imperative.

As simple as the procedure appears, it has been found that, as an indicator of general quality of rock for engineering purposes, the numerical value of RQD is more sensitive and consistent than gross percentage core recovery. The relationship between RQD and rock quality is given in Fig. 3.2.(b).

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DEERE, D.V., 1968. Geological considerations. In STAGG, K.G. and ZIENKIEWICZ, O.C., (Editors). Rock mechanics in engineering practice. *J. Wiley & Sons. N.Y.*

ASTM D2938-71a. Unconfined compressive strength of rock core specimens.

CHAPTER 4

SUBSURFACE INVESTIGATIONS

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CHAPTER 4

SUBSURFACE INVESTIGATIONS

4.1 GENERAL

Subsurface investigation means the appraisal of the general subsurface conditions at a building site by analysis of information gained by such methods as geological surveys, *in situ* testing, boring and sampling, visual inspection, laboratory testing of samples of the subsurface materials and groundwater observations and measurements.

The subsurface investigation is the first and most important step in any foundation design. Such an investigation should be carried out for all structures, even modest ones, before design is undertaken or a building permit is issued.

It is important that subsurface investigations be carried out under the direction of engineers and personnel with knowledge and experience in planning and executing such investigations. It is desirable that drilling crews be experienced specifically in borings for geotechnical explorations.

4.2 OBJECTIVE OF INVESTIGATION

The primary objective of a subsurface investigation is to determine as accurately as may be required;

- the nature and sequence of strata,
- the groundwater conditions at the site,
- the physical properties of the soils and rock underlying the site,
- the mechanical properties, such as strength and compressibility of the different soil or rock strata, and
- other specific information, when needed, such as chemical composition of the groundwater, and characteristics of foundations of adjacent structures.

Subsurface investigations should be organized in such a way that all possible information be obtained that will provide a thorough understanding of the subsurface conditions and probable foundation behaviour.

4.3 BACKGROUND INFORMATION

Before the actual field investigation is started, information should, whenever possible, be collected on;

- the type of building to be built; its' intended use, characteristics of the structure, starting date, intended construction method, and estimated period of construction.
- the probable soil conditions by analysis of geological and geotechnical maps (Aerial stereophotographs are often of use in the evaluation of general soil conditions and of specific problems such as the stability of natural slopes in the vicinity of the site),and
- the soil conditions beneath, the foundation systems and behaviour of existing structures adjacent to,the site, as well as other related local experience.

4.4. EXTENT OF INVESTIGATION

4.4.1. *GENERAL*

Subsurface conditions at a building site may be relatively uniform or extremely variable. These conditions will largely determine the complexity of the problems to be faced both in the design and construction of the foundations. The subsurface investigation must therefore be of sufficient extent to provide enough information for a thorough understanding of the interaction of proposed foundations and supporting soil or rock on which to base a safe and economical design. To assist in planning a subsurface investigation a list of items to be considered may be found in Appendix B Check List for Foundation Investigations, *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs 98: SM8, 779-785, 1972.*

4.4.2. *DEPTH OF INVESTIGATION*

The subsurface investigation should be carried to such a depth that the entire zone of soil or rock affected by changes caused by the building or the construction will be adequately explored. This depth occurs approximately at a level where the vertical stress induced by the new construction is less than 10% of the existing overburden stress at that level. (HVORSLEV 1949).

Where the depth of investigation cannot be related to background information as described in 4.3. the following guidelines are suggested.

- It is good practice to have one boring carried to bedrock or at least to well below the anticipated level of influence of the building.
- For light structures, insensitive to settlement the borings should be extended to a depth equal to 4 times the probable footing width but to not less than 20 ft below the lowest part of the foundation.
- For more heavily-loaded structures such as multi-storey structures and for framed structures at least 50 percent of the borings should be extended to a depth equal to 1.5 times the width of the building below the lowest part of the foundation.
- Where bedrock is encountered it should be proved by coring to a minimum depth of 10 ft.

4.4.3. *ACCURACY OF INVESTIGATION*

It is advisable to check the agreement of geotechnical tests. Subsurface investigations should call for various methods for measuring the soil properties critical in design; in particular, it is good practice to combine *in situ* tests and laboratory tests for strength and compressibility whenever possible.

The accuracy of stratigraphy determined by geophysical methods such as seismic reflection or refraction, or resistivity measurements should always be checked by borings or other direct observations.

4.5 INVESTIGATION OF SOILS

The physical and mechanical properties of soils are determined either by *in situ* testing, by laboratory testing or a combination of both. Both approaches have advantages, disadvantages and limitations in their applicability.

4.5.1. *IN SITU TESTING*

The common *in situ* testing methods are listed in Table 4.1. The various *in situ* tests must be carried out with utmost care and according to either standardized or generally accepted procedures. Because of their variability, *in situ* tests should be repeated. This is particularly important for the Standard Penetration Test.

TABLE 4.1 IN SITU TESTS

TYPE OF TEST	TYPE OF SOIL Best Suited to	Not Applicable to	Properties that can be determined	REMARKS	REFERENCES
1 - STANDARD PENETRATION TEST (SPT)	Sand	Clay	Qualitative evaluation of compactness Qualitative comparison of subsoil stratification	(See Commentary 8.1)	(1) CSA A119.1-1960 (2) ASTM D1586-67 (3) FLETCHER (1965) (4) PECK ET AL (1963) (5) TAVENAS (1971)
2 - DYNAMIC CONE TEST	Sand & Gravel	Clay	Qualitative evaluation of compactness Qualitative comparison of subsoil stratification		
3 - STATIC CONE TEST	Sand		Continuous evaluation of density and strength of sands and gravel Continuous evaluation of undrained shear strength in clays	Test is best suited for the design of piles in sand. Tests in clay are reliable only when used in conjunction with vane tests	(1) SANGLERAT (1972) (2) SCHMERTMANN (1970) (3) LADANYI & EDEN (1969)
4 - PLATE BEARING TEST	Sand		Modulus of subgrade reaction Ultimate bearing capacity	Strictly applicable only if the deposit is uniform. Size effects must be considered in other cases	(1) ASTM D 1194-72
5 - VANE TEST	Clay	Silt Sand Gravel	Undrained shear strength c_u	Test should be used with care particularly in fissured, varved and highly plastic clays	(1) ASTM D 2573-72 (2) BJERRUM (1972) (3) AAS (1965) (4) LO (1972)
6 - PRESSUREMETER TEST	Soft rock Sand	-	Ultimate bearing capacity and compressibility	(See Commentary 8.8)	(1) MENARD (1965) (2) EISENSTEIN et al (1973) (3) TAVENAS (1971)
7 - PERMEABILITY TEST	Sand & Gravel	Clay	Evaluation of coefficient of permeability	Variable head tests in boreholes have limited accuracy. Results reliable to one order of magnitude are obtained only from long term, large scale pumping tests	(1) HVORSLEV (1949) (2) NAVFAC DM7 (1971) (3) SHERARD ET AL

REFERENCES

Standard Penetration Test

CSA A 119.1-1960. Code for split-barrel sampling of soils.

ASTM D 1586-67. Penetration test and split-barrel sampling of soils.

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PECK, R.B., HANSON, W.E. and THORNBURN, T.H., 1974. Foundation engineering. *J. Wiley & Sons, N.Y.*

TAVENAS, F.A., 1971. Discussion of "The standard penetration test." *Proc. Pan. Am. Conf. Soil Mech. Found. Eng. 4th Puerto Rico, 1971, 3: 64-70.*

Static Penetration Test

SANGLERAT, G., 1972. The penetrometer and soil exploration. *Elsevier Publ. Co. Amsterdam.*

SCHMERTMANN, J.H., 1970. Static cone to compute static settlement over sand. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs., 96: SM3, 1011-1043.*

LADANYI, B. and EDEN, W.J., 1969. Use of the deep penetration test in sensitive clays. *Proc. Inter. Conf. Soil Mech. Found. Eng., 7th, Mexico. 1969 1: 225-230.*

Plate Bearing Test

ASTM D 1194-72. Test for bearing capacity of soil for static load on spread footings.

Vane Test

ASTM D 2573-72. Field vane shear test in cohesive soil.

AAS, G., 1965. A study of the effect of vane shape and rate of strain on the measured values of *in situ* shear strength of clays. *Proc. Internat. Conf. Soil Mech. Found. Eng., 6th, Montreal, 1965, 1: 141-145.*

BJERRUM, L., 1972. Embankments on soft ground. In: *Proc. Am. Soc. Civil Engrs. Conf. Earth and Earth Supported Structures. Purdue U. 1972 2: 1-54.*

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Pressuremeter Test

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TAVENAS, F.A., 1971. Contrôle du roc de fondation de pieux forés à haute capacité. *Can. Geotech. J. 8: 400-416.*

Permeability Test

HVORSLEV, M.J., 1949. Subsurface exploration and sampling of soil for civil engineering purposes. *Soil Mech. Found. Div., Am. Soc. Civil Engrs. Cttee. Sampling and Testing. Vicksburg*

NAVFAC, D.M. 7, 1971. Soil mechanics, foundations and earth structures. *Design Manual 7. Dept. Navy, Naval Facilities Eng. Command. Wash. D.C.*

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4.5.2. *BORING AND SAMPLING*

The properties of soils can be determined from laboratory tests on samples recovered from boreholes. The quality of the samples depends mainly on the boring method, the sampling equipment and the procedure used in retrieving them.

4.5.2.1. *Classes of Samples*

For the purpose of this Manual, four classes of samples have been defined, which are listed in Table 4.2.

Mechanical properties which serve as basis for the design of foundations can be measured only on samples of class I. Such samples should always be retrieved for the design of foundations on clays.

Problem soils, as referred to in paragraph 4.8, may require special sampling procedures as indicated therein.

REFERENCES

CSA A 119.1-1960. Code for split-barrel sampling of soils.

ASTM D 1587-67. Thin-walled tube sampling of soils.

ASTM D 1586-67. Penetration test and split-barrel sampling of soils.

HVORSLEV, M.J., 1949. Subsurface exploration and sampling of soil for civil engineering purposes. *Soil Mech. Found. Div., Am. Soc. Civil Engrs., Cttee. Sampling & Testing. Vicksburg.*

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TERZAGHI, K. and PECK, R.B., 1967. Soil mechanics in engineering practice. *J. Wiley and Sons, N.Y. 289-360.*

TABLE 4.2 Sample Classification

CLASS	QUALITY	IDENTIFICATION	PROPERTIES THAT CAN BE MEASURED											NOTE
			STRATIGRAPHY	STRATIFICATION	ORGANIC CONTENT	GRAIN SIZE DISTRIBUTION	ATTERBERG LIMITS	SPECIFIC GRAVITY	WATER CONTENT	UNIT WEIGHT	PERMEABILITY	COMPRESSIBILITY	SHEAR STRENGTH	
1	UNDISTURBED	a - Block samples	+	+	+	+	+	+	+	+	+	+	+	1-4-6
		b - Stationary piston sampler 3" minimum diameter	+	+	+	+	+	+	+	+	+	+	+	2-3-4-5-6
2	SLIGHTLY DISTURBED	Open thin-walled tube sampler 2" minimum diameter	+	+	+	+	+	+	+	+	+			3
3	SUBSTANTIALLY DISTURBED	Open thick-walled tube sampler such as split-barrel sampler	+	+	+	+	+	+	+					
4	DISTURBED	Random samples collected by auger or in pits	+		+	+	+	+	+					7

NOTES TO TABLE 4.2

1 Block samples are best when dealing with sensitive, varved or fissured clays. Wherever possible block samples should be taken in such soils.

2 3" diameter stationary piston samples may be impossible to obtain in some materials such as very stiff clays. If shear strength and compressibility of such materials are required they may be determined using class 2 samples but due consideration must be given to the lower quality of such samples.

3 Samples of classes 1b and 2 must be taken with tubes conforming to the following geometric requirements:

The area ratio $i = \frac{D_o^2 - D_i^2}{D_i^2} < 12\%$

The inside clearance $0.5\% < \frac{D_i - D_e}{D_i} < 1\%$

The angle of the cutting edge must be not greater than 30^o

where D_o = outside diameter of the tube

D_i = inside diameter of the tube

D_e = inside diameter of the cutting edge

4 Samples of class 1 are best stored in a vertical position in a room with constant humidity of 80% minimum and constant temperature of 50^oF maximum

5 Samples of class 1b are best extruded with the tube in a vertical position. Extrusion and testing should occur as quickly as possible after sampling. Whenever possible testing should be performed immediately after extrusion.

6 Because of inevitable stress relief samples of all classes may be disturbed. The disturbance is dependent upon the consistency of the sampled soil. Disturbance also increases with depth of sampling.

7 Water content samples should be taken from freshly-cut faces of the pit as it is advanced. Small diameter spiral augers are suitable for obtaining water content samples of cohesive soil if care is taken to remove from the sample free water and soil scraped from upper layers in the wall of the bore hole.
Water content samples should be placed immediately in air tight containers to prevent evaporation.

4.5.3. *LABORATORY TESTING OF SOIL SAMPLES*

It is beyond the scope of this Manual to cover in detail all laboratory testing techniques now in use in soil mechanics. However, it is necessary to insist on some basic requirements.

4.5.3.1. *Quality of Test Results*

The quality of test results is determined:

- by the quality of samples as defined in 4.5.2.1.
- by conformance of test equipment and methods to those stipulated in the pertinent standards or implicit in the current state of the art, and
- by the quality of testing, which can only be ensured by adequate initial education, continuous control and improvement in the skill of laboratory personnel.

4.5.3.2. *Identification and Classification*

Identification and classification of soils is presented in Chapter 3 of this Manual.

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ASTM D 2435-70. Test for one-dimensional consolidation properties of soils.

ASTM D 2850-70. Test for unconsolidated, undrained strength of cohesive soils in triaxial compression.

4.6 INVESTIGATION OF ROCK

4.6.1. *GENERAL*

Frequently, determination of the character and condition of rock by means of core boring methods and borehole inspection will be necessary. This will occur where foundations may be extended to the rock surface or into bedrock.

Where investigation of bedrock is necessary, pertinent information to be determined includes;

- geological characteristics of the site,
- elevation of the rock surface and variation over the site,
- rock type and core strength,
- extent and character of weathering and weatherability,
- extent and distribution of solution channels in soluble rocks such as limestone,
- discontinuities such as bedding planes, faults, and joints,
- folds and structural orientation,
- foliation or cleavage planes,
- permeability, and
- strength and compressibility of the rock mass.

4.6.2. *CORE DRILLING OF ROCK*

Boreholes for the investigation of rock should be advanced by the diamond core drilling method.

The minimum quality of equipment should conform to ASTM D 2113-70 "Diamond core drilling for site investigations." Better equipment may be needed for drilling and sampling of soft rocks.

Care must be exercised to ensure maximum possible core recovery. Changes in drilling noise, vibrations, pressure on the drilling bit, colour, pressure and flow of drilling water and all other observations relative to the drilling operations should be carefully recorded.

4.6.3. *USE OF CORE SAMPLES*

4.6.3.1. *Identification and Classification*

Identification and classification of rocks is presented in Chapter 3 of this Manual.

Particular attention should be paid to the identification or rock discontinuities: nature and origin, spacing, geometry, weathering, etc.

4.6.3.2. *Laboratory Tests of Core Samples*

Laboratory tests for measuring the mechanical properties of rock give results of limited value since they are performed on sound samples free of discontinuities. Such results may not be representative of the actual rock mass.

Tests most frequently conducted are unconfined compression tests, triaxial compression tests and sonic velocity tests. These should be performed in accordance with the standards listed below.

REFERENCES

ASTM D 2938-71a. Test for unconfined compressive strength of rock core specimens.

ASTM D 2664-67. Test for triaxial compressive strength of undrained rock core specimens without pore pressure measurements.

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4.7 INVESTIGATION OF GROUNDWATER

4.7.1. *GENERAL*

Groundwater is a critical factor in foundation design and construction. It should therefore be given careful attention during all stages of soil investigation.

Parameters of importance are;

- the existence of groundwater; normal, perched, or artesian,
- the exact level of the groundwater table and of the lower limit of perched groundwater,
- thicknesses of strata and the piezometric level of artesian groundwater,
- the variation of these characteristics over the site and with time, and
- the chemical composition of groundwater

4.7.2. *INVESTIGATION IN BOREHOLES*

In most cases where normal groundwater conditions are encountered they can be investigated during boring. The water level should be measured at regular intervals during the advancement and after completion of each borehole.

During each boring, field records should be made of all observations related to groundwater; such as change in color and rate of flow, partial of total loss of water, first appearance of artesian conditions.

All information related to groundwater should be recorded on the boring log, along with the depth of the borehole and depth of casing at the time of observation.

Groundwater observations made at the time of boring are not representative in clay and other fine-grained soils because of the low permeability of these materials and the longer periods of time required before the water level in such a borehole reaches equilibrium.

4.7.3. *INVESTIGATION BY PIEZOMETERS*

In all cases where groundwater conditions are important in design, or are difficult, or where direct borehole observation is not applicable, the groundwater conditions should be investigated by the installation and observation of piezometers. In designing such installations, attention should be paid to the stratigraphy (for location of the piezometer tips) and the soil type (for selection of the type of piezometer). Time lag is a particularly important parameter in the selection of piezometer type. Equipment and methods of installation are described in detail in the following references.

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4.8 PROBLEM SOILS, ROCKS AND CONDITIONS

There are certain types of soils and rocks which pose particular difficulties or special problems, such as highly sensitive clays and expansive soils and rocks. Those problem soils, rocks and conditions most commonly encountered are described in Appendix 4A.

4.9 REPORT

Data from subsurface investigations usually are referred to continuously and for many different purposes during the construction period and frequently after completion. Appropriate reports should therefore be prepared for each subsurface investigation. They should be clear, complete and accurate. The following outline may be used as a guide in arranging data in such reports:

4.9.1. *TEXT*

Scope of the investigation,

Proposed structure or structures,

Geological setting,

Existing adjacent structures,

Field explorations,

Laboratory investigation (testing),

Analysis of data,

Foundation studies including alternatives,

Recommended construction procedures, if appropriate,

Conclusions and recommendations, and

Limitations of the investigation.

4.9.2. *GRAPHIC PRESENTATIONS*

Map showing the site location,

Detailed plan of the site showing contours, elevations, proposed structures, borehole locations, and adjacent structures.

Boring logs including all the necessary pertinent information,

Stratigraphical, geotechnical profiles,

Laboratory data, and

Special graphic presentations.

REFERENCE

CSA A119.5-1966. Recording of borehole and test pit information.

APPENDIX 4A

PROBLEM SOILS, ROCKS AND CONDITIONS

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APPENDIX 4A

PROBLEM SOILS, ROCKS AND CONDITIONS

GENERAL

Brief descriptions of certain types of soil, rock or conditions which require special care or precautions, if satisfactory designs and performance are to be achieved, are given in the following paragraphs. Early recognition of such soils, rocks or conditions is important in order that more adequate investigations may be undertaken in good time and designs developed to meet the conditions found. Successful investigation and analysis of these conditions require special knowledge and should usually be placed in the hands of competent foundation consultants.

PROBLEM SOILS

ORGANIC SOILS

Soils containing significant amounts of organic materials, either as colloids or in fibrous form, will usually be found weak and subject to excessive deformation under load. Such soils include peat associated with muskeg terrain, organic silts and clays typical of many estuarine, lacustrine or fluvial environments. Such soils are usually not satisfactory as foundations for even very light structures because of excessive settlements resulting from compression and consolidation.

NORMALLY CONSOLIDATED CLAYS

Clays of soft to medium consistency which have been consolidated only under the weight of existing conditions are found in many areas. Typical are the clays of the Windsor - Lake St. Clair region and the varved clays in the northern parts of Manitoba, Ontario and Quebec. Imposition of additional load, such as a building, will result in significant long-term settlement. The magnitude and approximate rate of such settlements can be predicted from analyses based on carefully conducted consolidation tests on undisturbed samples. Such studies should be made before any significant structure is founded above these clays to determine whether settlements will be acceptable, considering the characteristics and purpose of the structure.

Driving piles through normally consolidated plastic clays may cause heave or displacements of piles previously driven or adjacent structures. The bottom of excavations made in such soils may heave and adjoining areas of structures may move or settle, unless the hazards are recognized and proper precautions taken to prevent such movements.

In the case of varved clays special precautions may be necessary in sampling and testing. Any analysis should take into account the important differences in properties between the various layers in the clays.

SENSITIVE CLAYS

Sensitive clays are defined as having a remolded strength of 25% or less of the undisturbed strength. Some clays are much more sensitive than this, and clays having a remolded to undisturbed strength ratio of 1 to 20, or even 1 to 50, are known. Typically, such clays have field moisture contents equal to or greater than their liquid limits, and such relations may indicate their presence. Extensive deposits of sensitive clays occur in some areas as, for example, the Leda clays of the St. Lawrence River Valley. Where such clays have been preconsolidated by partial desiccation or by the weight of materials subsequently eroded, foundations may be placed above such clays, provided that the gross additional load imposed by the structure is appreciably less than the preconsolidation load of the clay, and shearing stresses under the foundations are well within the shear strengths of the clay. Exceeding either of these limits will result in excessive settlements and possibly in catastrophic failure. Disastrous flow slides have developed in these clays in a number of instances and the hazard must always be considered. Deep excavations in sensitive clays are extremely hazardous because of possible severe loss in shear strength resulting from strains within the soil mass beneath and adjacent to the excavation.

Determination of the physical properties necessary for evaluating the significance of such clays to a proposed structure requires taking and testing of both undisturbed and remolded samples of the clays and thorough analysis of the possible hazards involved. Because of the extreme sensitivity of such clays to even minor disturbances, taking and testing undisturbed samples requires extremely sophisticated equipment and techniques. It should be attempted only by competent personnel experienced in this type of work.

SWELLING AND SHRINKING CLAYS

Swelling and shrinking clays are clays which expand or contract markedly upon changes in moisture content. Such clays occur widely in the provinces of Alberta, Manitoba and Saskatchewan and are usually associated with lacustrine deposits. Shallow foundations founded on such clays may be subject to movements brought about by volume changes due to changes of moisture content in the clays. Deep foundations supporting structural floors can be damaged if such a system confines the clay. Special design provisions should be made taking into account the possibility of movements or swelling pressures in the clays.

LOOSE, GRANULAR SOILS

All granular soils are subject to some compaction or densification when subjected to vibration. Normally, this is of significance only below the permanent water table. Sands above the water table usually will be only slightly compacted by most building vibration because of friction developed between the grains from capillary forces. Usually for sands of medium dense to dense state, settlements induced by vibration will be well within normal structural tolerance, except for very heavy vibration as from forging hammers or similar equipment. However, if the sands are in a loose to very loose state, significant settlement may result from even minor vibrations or from nearby pile driving. In some cases, spontaneous liquefaction of very loose sands has resulted from earthquakes, as occurred in Niigata in Japan. In this event structures supported above such soils may be completely destroyed. Loose sands will settle significantly under static loading only. Such settlements may exceed allowable tolerances. Consequently, loose sands should be investigated carefully, and their limits established; densification or compaction of such deposits may be essential before structures can safely be founded above them.

METASTABLE SOILS

Metastable soils include several types of soil which are abnormally loose as deposited and which may collapse on saturation. Such collapses will cause severe or even catastrophic settlement of structures founded in or above these soils. Loess, which is found in some areas such as the Okanogan region is the most common. Because such soils are strong and stable when dry, they can be misleading in investigations, and extreme care should be taken to ensure identification and proper foundation design wherever such soils occur. The open, porous structure which is the usual means of identification may be completely collapsed by set boring techniques. Where such conditions may be anticipated, borings should be done by auger methods and test pits should be dug from which undisturbed samples may be taken for determining accurately in-place densities.

ARTIFICIAL FILL

Artificial fill may be extremely dense granular material placed under careful control which is more uniform, more rigid and stronger than almost all natural deposits; it may be a heterogeneous mass of rubbish, debris and loose soil of many types totally useless as a foundation material or some combination intermediate between these extremes. Unless the conditions and control under which it was placed are fully known, it must be presumed unsatisfactory. The investigations must be adequate to establish its limits, depth, and characteristics throughout.

PROBLEM ROCKS

CHEMICAL WEATHERING

Mechanical properties of both the rock mass and rock cores provide a generally reliable guide to the quality of rock for foundation purposes. However, all rock masses involved in foundation engineering occur within the near surface zone of the earth and are subject to alteration by inorganic and organic chemical processes particularly in the presence of groundwater.

Chemical alteration or weathering of rock may take the form of removal of material in solution or volumetric expansion upon wetting, resulting in both cases, in reduction of the strength properties of the rock mass.

Under Canadian climatic conditions the rate of chemical weathering for igneous and most metamorphic and sedimentary rocks is generally sufficiently slow to be of little importance in foundation engineering. There are, however, some exceptions.

SEDIMENTARY AND METAMORPHIC ROCKS

Sedimentary and metamorphic rocks such as limestone, gypsum, rock salt and metamorphic marble are subject to accelerated rates of chemical attack resulting in solution channels and caverns below bedrock surface or sinkholes at the earth's surface. These conditions may present special foundation problems.

SHALES

Shales are the most abundant of sedimentary rocks and commonly the weakest from the foundation standpoint. Two special problems with certain shale formations have been identified in Canada.

In Western Canada, the Bearpaw and other Cretacious shales have been found to swell considerably when stress release or unloading leads to the absorption of water by the clay minerals. When such shales are encountered along deep river valleys special advice should be sought.

In some shale formations in Eastern Canada volumetric expansion due to a weathering process of sulphide minerals (pyrite) accelerated by oxidizing bacteria, has occurred in isolated instances. Conditions leading to mineralogical alteration seem to be related to lowering the groundwater table and to raising of the temperature in the shale, particularly when the shale is highly fractured. These conditions enhance bacterial growth and oxidation of the sulfide minerals. In these cases, special provisions should be considered to reduce heat loss from the building spaces to the supporting shale.

Note - Since the effect of chemical degradation of foundation rock on the performance of the structure may become obvious only after several years following completion, the problem can only be avoided by recognition of potential difficulties at the time of subsurface exploration and the taking of remedial measures during design and construction phases of the project.

PROBLEM CONDITIONS

MEANDER LOOPS AND CUTOFFS

Slow, meandering streams, from time to time, develop cutoffs across a neck between two loops leaving an abandoned channel which later fills with very soft organic silts and clays. These conditions are very common along the Red and similar rivers. Such meander loops can be identified by their crescent shape. Frequently, these can be detected in aerial photographs or from accurate topographic maps. The soils filling these abandoned waterways are extremely weak and highly compressible. It is necessary that the limits of such areas be accurately located and the depths of the soft, compressible soils filling them established.

LANDSLIDES

In areas of appreciable relief, the possibility of landslides should always be considered. Landslides in an active state are readily identifiable. Old landslides or unstable soils in a potential landslide state may be indicated by hummocky conditions, by bowed trees, by tilted or warped strata, or by other evidences of displacement. Such areas are almost always in a state of marginal stability and even minor disturbances, as by small excavations near the toe, or minor changes in groundwater conditions or drainage, may cause such landslide areas to become active. Stopping a landslide once it is in active motion is always more difficult than taking proper precautionary measures to avoid triggering such a landslide or avoiding the landslide area in the first place. If sensitive clays are present, hazards are increased significantly.

Consequently, care should be taken to locate potential landslide areas, to investigate them thoroughly, and to adopt construction procedures and designs which will be safe. The banks of actively eroding rivers are always in a state of marginal stability. This is particularly true of the outside bends of such rivers, because active cutting is usually in progress, especially during periods of high water.

KETTLE HOLES

In areas of glacial outwash, trapping or stranding of blocks of ice torn loose from the glaciers was a common occurrence. Later, when these blocks melted, they left depressions in the outwash mantle, many of which subsequently filled with peat or with soft, organic soils. These depressions which are referred to as kettle holes, vary in size from a few feet across and a few feet deep to moderate size ponds several hundred feet across. They can usually be detected as shallow surface depressions by careful examination, although occasionally all surface expression has been destroyed by farming or leveling operations. Ordinarily they can be located from aerial photographs because of the difference in vegetation. In areas where they are suspected, it is necessary that their locations and extent be established. Because their depths are limited by the angle of repose of the material surrounding the hole left by the ice, depths of such deposits cannot exceed about 40% of the minimum lateral dimension.

MINED AREAS

Sites located over or adjacent to mined areas may be subject to severe ground movements and differential settlements caused by the collapse of a mine roof. Generally, for coal mines and similar mines in horizontal strata, the zone of disturbance does not extend laterally from the edge of the mined areas a distance much more than half the depth of the mine below the surface. There is little control of the solution process for mining potash or salt, and, in such areas, subsidence may extend from 2,000 ft to 4,000 ft beyond the edges of the mine or well field. Some evidence indicates that the solution may extend farthest up the dip of the strata.

Investigations must be extremely thorough and all possible data on old mines should be obtained wherever such conditions are suspected. While maps may be available for active mines or recently closed mines the accuracy of such maps frequently is poor. Further, there are many mined-out areas, especially in the older mining regions, for which no records are available. Careful surface examination of suspected areas, especially in the slanting light of sunset, may show depressions resulting from ground subsidence and so permit identification of mined areas where records are incomplete.

PERMAFROST

Permafrost is the thermal condition of the earth's crust when its temperature has been below 32°F continuously for a number of years. Half of Canada's land surface lies in the permafrost region - either in the continuous zone where the ground is frozen to a depth of hundreds of feet, or in the discontinuous zone where permafrost is thinner, and there are areas of unfrozen ground.

The existence of permafrost causes problems for the development of the northern regions extending into the Arctic. Engineering structures are, of course, greatly affected by the low temperatures. Ice layers give soil a rock-like structure with high strength. However, heat transmitted by buildings often causes the ice to melt, and the resulting slurry is unable to support the structure. Many settlements in northern Canada have examples of structural damage caused by permafrost. In construction and maintenance of buildings normal techniques must, therefore, be modified at considerable additional cost.

Accumulated experience with careful scientifically planned and conducted investigations make it technically possible to build practically any structure in the permafrost area. Design and construction in permafrost should only be carried out by those who possess this type of very special expertise.

NOXIOUS OR EXPLOSIVE GASES

Noxious or explosive gases, methane being the most common, are occasionally encountered in clay or silt deposits. They constitute a hazard to workmen constructing caissons or in deep excavations. Gases may also be found in shale or other sedimentary rock deposits in various areas of the country. These may be a special hazard in deep excavations or where borings have encountered such gases and are permitted to discharge into the construction area. The history of the area or discharge of gas from borings, even if only for short periods of time, should be especially noted and suitable precautions taken.

A special problem may exist in tunnels or drainage systems where certain iron consuming bacteria are present. These can so severely deplete the oxygen supply in poorly ventilated areas that persons entering may be asphyxiated. Such areas should be thoroughly purged with clean air before entering and adequate ventilation assured while persons are in such areas.

EFFECTS OF HEAT OR COLD

Soils should be protected against contact with surfaces which will be extremely hot or extremely cold. Desiccation of clay soils beneath furnaces or along-side ducts carrying hot gases will cause excessive and severe differential settlements. Spaces or tanks which are permanently below freezing temperature cause frost heave and distress in anything but clean, coarse sands and gravels, unless isolated from the soil. Insulation is not sufficient under these conditions, as it merely slows down the rate of heat transmission to or from the soil mass. A heat source is essential under low temperature structures and ventilation is necessary around high temperature structures.

Collapse of retaining walls may occur in cold climates from ice lens formation unless the walls are back-filled with nonfrost-heave material for a distance equal to maximum frost penetration, and proper drainage provided.

SOIL DISTORTIONS

Soils distort laterally as well as vertically under surface loadings. Usually this is not significant; however, severe lateral distortions may develop in highly plastic soils toward the edge of surface loadings, even though the loads are not sufficient to cause rupture or mud waves. These laterial distortions may affect foundations or piles for structures located in or adjacent to areas subject to high surface loading, such as structures along the edge of fills or a coal pile. Lateral distortions are a special hazard if sensitive clays are present. In such soils, shearing strains accompanying the distortions may lead to significant loss of shear strength or possibly even to flow failures or slides.

Both lateral and vertical displacements may develop in soil when displacement type piles are driven, especially in cohesive soils. Pressures or displacements which develop may cause displacements of previously driven piles or existing foundations, or result in excessive pressures on retaining walls, sheeting for excavations, or buried pipes. Heaved piles may be redriven and used. If there is significant lateral displacement, the piles may be kinked or bowed beyond the safe limit of use. These hazards must be evaluated in the investigational program, and provision made in design and construction procedures to be sure other structures or piles are not damaged or displaced by the driving of adjacent piles. Preboring through the cohesive strata should be required if there is any hazard of disturbing exiting structures or previously driven piles.

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EXCAVATIONS AND RETAINING STRUCTURES

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EXCAVATIONS AND RETAINING STRUCTURES

5.1 UNSUPPORTED EXCAVATIONS

The safety and stability of unsupported excavations depend on the soil and groundwater conditions and on the depth and slope of the cut. In granular materials, slope failures will generally be fairly shallow; in clays, however, deep rotational failures involving not only the sides, but also the base of the excavation, are possible.

Many cuts in clay will stand unsupported to quite large depths for a period of time and then fail. The operational shear strength of clay masses changes with time subsequent to stress release caused by excavation. This can lead to a progressive deterioration in the stability conditions; which can be rapid in stiff highly fissured soils, but is less rapid in softer clays. The important factor affecting stability is the piezometric level or groundwater level in the slope. High piezometric levels reduce the effective stresses along the surface of sliding and create extra driving forces where open tension cracks exist at the back of the overstressed zone.

In sensitive clays such as the Champlain Sea clays (which includes the Leda clays), massive retrogressive flow slides can result once failure is provoked. In these soils considerable caution should be used during excavation operations and deformations should be rigidly controlled and monitored.

Clay soils may fail either under undrained conditions (short term) or under drained conditions (long term). In general, excavations will be more stable in the short term and less stable in the long term. The length of time required before the long term (or drained) condition becomes relevant to stability depends on many factors and it is therefore advisable to check both drained and undrained stability before adopting any given excavation design.

The principles of analysis of the stability of slopes are dealt with in TERZAGHI & PECK, (1967) which details further references covering the techniques of analysis for specific problems.

5.2 SUPPORTED EXCAVATIONS

5.2.1 *WALL PRESSURES*

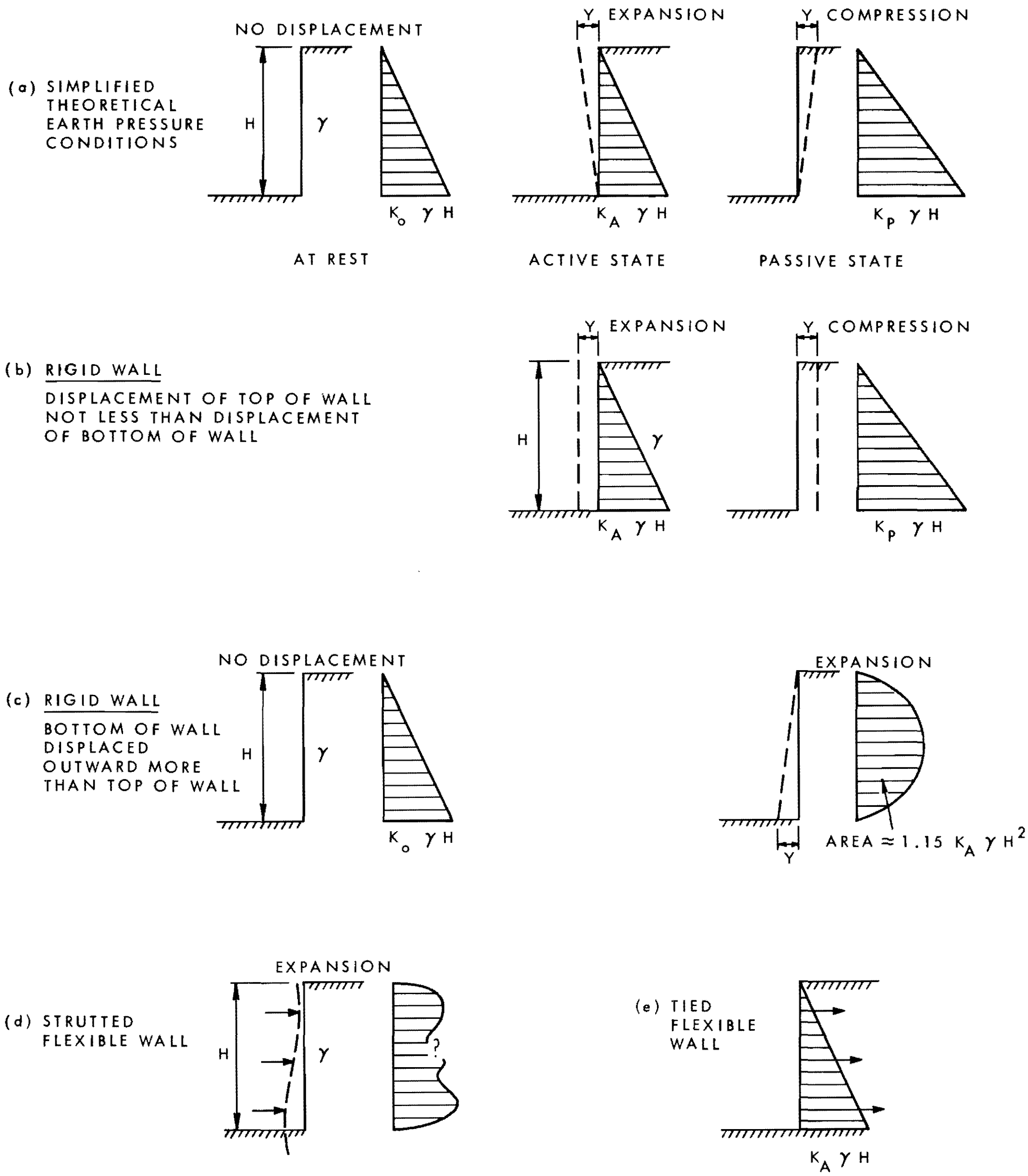
For rigid, inflexible walls such as free standing retaining walls, earth, water and surcharge pressures can be computed adequately from theory for most real situations. The relevant information is contained in Appendix 5A.

For flexible and semi-flexible walls such as those commonly used for the support of vertical faces of excavations, and which may have a variety of support conditions, no satisfactory general theoretical solutions for earth pressures are available. A guide to the probable earth pressures for various situations is given in 5.2.2.

5.2.2 *EARTH PRESSURES AS RELATED TO DEFORMATION*

The earth pressure which acts on an earth supporting structure is strongly dependent on the lateral deformations which have occurred in the soil (Fig 5.1(a). Consequently, unless the deformation conditions can be estimated with reasonable accuracy, no rational attempt at predicting either the total force or the distribution of earth pressure is possible.

For rigid walls, a fairly simple relationship exists between the wall movement and the earth pressure provided that the displacement of the top of the wall is not less than the displacement of the bottom of the wall. As shown in Fig. 5.1(b) the pressure distribution remains close to a triangular form and ranges between the failure limits of the



NOTE: VALUES OF K_A, K_o, K_P ARE GIVEN IN APPENDIX 5A

FIG 5.1

EFFECT OF DEFORMATION ON EARTH PRESSURES *active* case (failure due to lack of support) and the *passive* case (failure due to excessive lateral thrust).

Where the base of a rigid wall is displaced outward further than the top of the wall, a parabolic pressure distribution as shown in Fig. 5.1(c) results. The corresponding force on the wall for this condition is generally about 10 per cent to 15 per cent greater than the force under active failure conditions.

For flexible walls, the deformations and hence the earth-pressures are much more complex. The yield of one part of a flexible wall throws pressure onto the more rigid parts. Hence the pressures in the vicinity of supports are higher than in unsupported areas, and the loads on individual supports vary, depending largely on the stiffness characteristics of the supports themselves.

For strutted walls, it has been shown that the final deformation conditions are approximately as shown in Fig. 5.1(d). This profile results mainly from deformation which occurs below the base of the cut, and before the installation of struts. The final average deflection condition is not greatly different from that shown for rigid walls in Fig. 5.1(c) and the total horizontal force is generally with ± 30 per cent of the theoretical total pressure for this condition. However, the detailed deflection conditions and hence the detailed pressure distribution is almost entirely a function of minute details in the construction technique and procedure. Individual loads in 'identical' struts in any particular set of observations have been found to vary from the average value for those particular struts by up to ± 60 per cent. (LAMBE et al, (1970)).

For anchored walls, the deflection characteristics and hence the pressure distribution differ from strutted walls. Once installed and stressed, struts can be considered basically to be fixed deflection supports; anchors, on the other hand, generally approximate fixed load supports. Anchored walls will therefore come much nearer than strutted walls to having triangular pressure distributions. In addition, stressing of anchors on the basis of higher lateral pressures tends to reduce wall movements subsequent to anchor placement (Article 5.7.1).

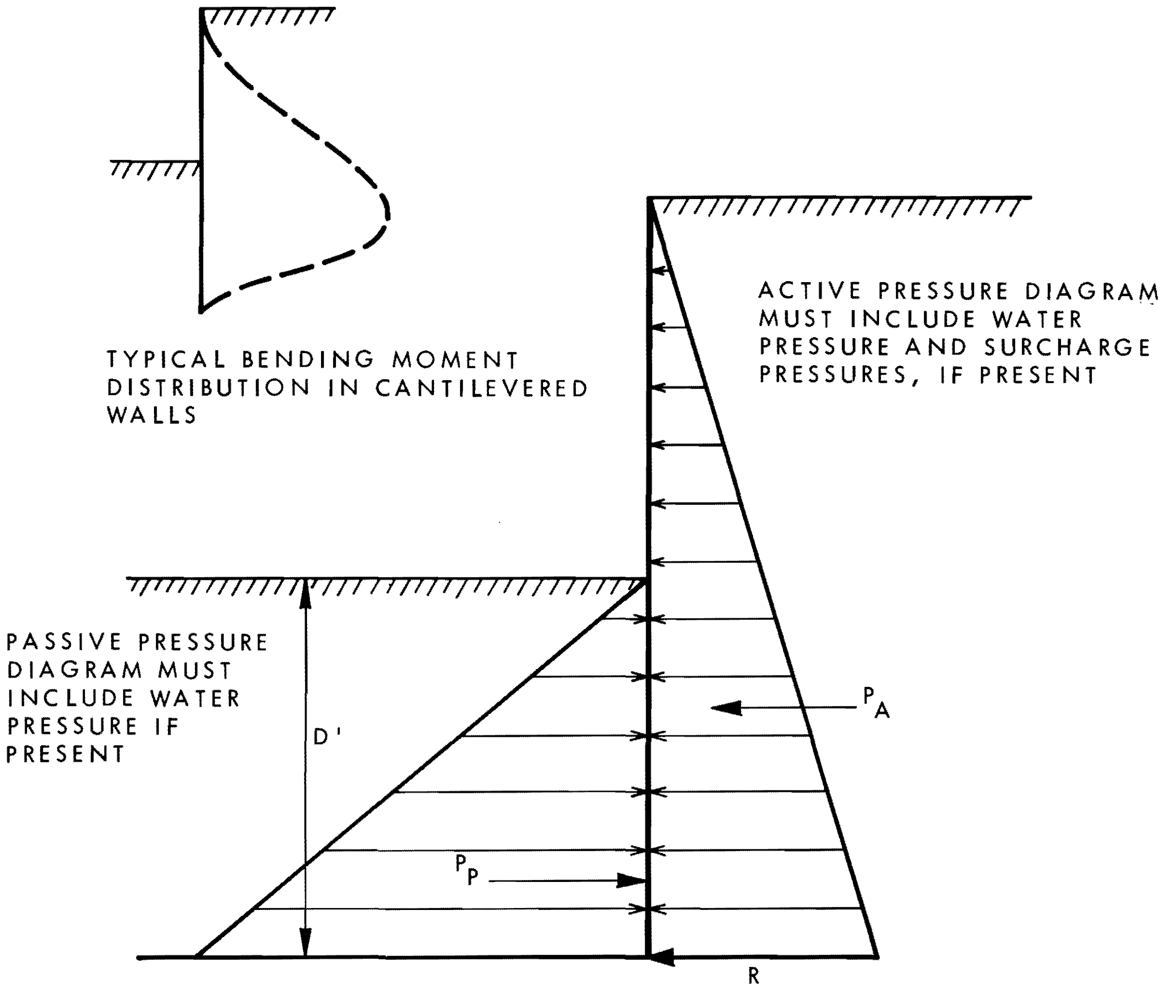
The pressure distribution on flexible walls with large unsupported spans such as in flexible bulkheads differs from the above cases and is discussed in BJERRUM et al, (1972), and TERZAGHI, (1953).

5.2.3 *CANTILEVERED (UNBRACED) WALLS*

Cantilevered walls (Fig. 5.2) are frequently used to support soil faces up to about 15 ft in height. They are generally considered to act as rigid structures and to rotate about some point beneath the base of the excavation. The earth pressures acting on the walls are therefore considered to approximate to the *active* and *passive* failure conditions (Appendix 5A).

Cantilevered walls are not suitable for permanent support in clay soils except those having low compressibility. Where used for permanent support in these soils, they should be analysed on the basis of effective stresses, using ϕ' the effective angle of shearing resistance, and neglecting cohesion. For temporary support in clay soils, design is on the basis of the undrained shear strength c_u, and computed earth pressures may be negative; a minimum earth pressure of 0.25 γz, at any depth z, should be used on the *active* side of the wall.

The method of analysis is shown in Fig. 5.2. Note that where water occurs behind the wall, the relevant water pressures must be added to earth pressures in all effective stress analyses; in total stress analyses, water pressure must be added where computed *active* pressures are negative.



$\Sigma M_R = O$ GIVES D' AND HENCE P_P , P_A

$\Sigma H = O$ GIVES R AND HENCE MOMENT DIAGRAM

(ACTIVE AND PASSIVE PRESSURE DIAGRAMS SHOULD BE OBTAINED FROM APPENDIX 5A)

FIG 5.2

FORCES ON CANTILEVERED UNBRACED WALL

5.2.4 *ANCHORED WALLS*

5.2.4.1 *Earth Pressures*

The actual earth pressures which finally act on an anchored wall will depend on;

- the wall stiffness relative to the soil,
- the anchor spacing,
- the anchor yield, and
- the prestress locked into the anchors at installation.

Two possible design methods are outlined below.

(1) *Analytical method*

The pressure diagrams are assumed to be triangular in form (See 5.2.2). For all soils, it is preferable that pressures be computed on the basis of effective stresses using ϕ', the effective angle of shearing resistance, neglecting cohesion.* (See Appendix 5A for details of earth pressure diagrams).

(a) *'Active' pressures*

i) If moderate wall movements can be permitted (See 5.2.7.), *active* pressure may be computed using the coefficient of *active* earth pressure K_A.

ii) If foundations of buildings or services exist at shallow depth at a distance less than H (height of the wall) behind the top of the wall and not closer than 0.5H, the pressure should be computed using a coefficient $K = 0.5\ (K_A + K_o)$.

iii) If foundations of buildings or services exist at shallow depth at a distance less than 0.5H behind the top of the wall, pressure should be computed using the coefficient of earth pressure at rest K_o.

iv) Where foundations of adjacent buildings extend to below the base of the wall, *active* pressure may be computed as in i) above.

(b) *'Passive' pressures*

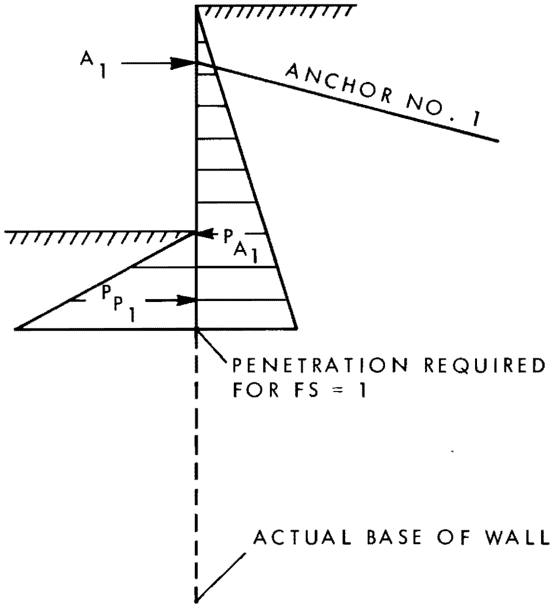
Passive pressures, relating to that portion of the wall below the base of the excavation, should be computed using a reduced coefficient of *passive* pressure $K_p' = K_p\ (\frac{1}{FS})$, where the factor of safety FS is not less than 1.5.

(2) *Empirical method*

If installation and deformation conditions are considered to approximate those obtained in strutted excavations, the pressure diagrams recommended in paragraphs 5.2.5.1 to 5.2.5.3 may be used to estimate the pressures on the wall.

---

* Where the excavation is in stiff cohesive soil and is open for only a limited period, pressures may be computed on the basis of the 'short term' or 'undrained' condition using the undrained shear strength c_u, with $\phi_u = 0$. Where computed *active* pressures are zero, a minimum earth pressure of $0.25\ \gamma z$ at any depth z, should be used in computations. Below the water table, water pressures are included where computed *active* pressures are negative.



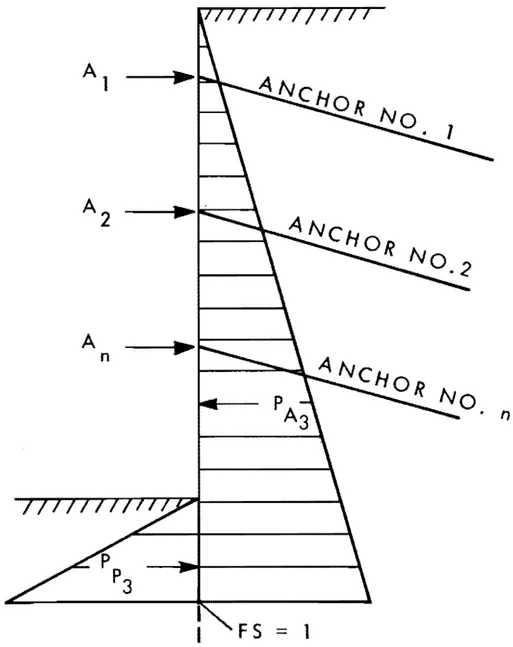
P_A AND P_P SHOULD BE DETERMINED AS IN 5.2.4.1(1) AND SHOULD INCLUDE WATER PRESSURE AND SURCHARGE EFFECTS

A IS THE HORIZONTAL COMPONENT OF THE ANCHOR FORCE

TO DETERMINE A_1, SOLVE FOR

$$\Sigma H = 0$$
$$\Sigma M = 0$$

(a) ANALYSIS FOR FIRST ANCHOR



FOR A_1 - A_{n-1}, USE THE PREVIOUSLY CALCULATED VALUES

P_A, P_P DETERMINED AS IN FIG. 5.3a)

TO DETERMINE A_n, SOLVE FOR

$$\Sigma H = 0$$
$$\Sigma M = 0$$

(b) ANALYSIS FOR INTERMEDIATE ANCHORS

FIG 5.3
CALCULATION OF ANCHOR FORCES

5.2.4.2 *Computations of Loads on Anchors*

(1) *Analytical method*

Where lateral pressures are computed on the basis of paragraph 5.2.4.1.(1), the following steps in computing anchor loads are recommended:

— Add relevant water pressures and the effect of any surcharge loads (Appendix A).

— Assume that the highest load on the n^{th} level anchor occurs just before placing the $(n+1)$ anchor and draw the excavation cross-section for that condition (Fig. 5.3).

— For all anchors other than the lowest, determine the depth of penetration of the wall required to establish a factor of safety of 1.0 against rotation using the pressure diagrams previously established, and taking into account the design forces in previously installed anchors.

— Determine the required force in the n^{th} anchor for stability of the wall, based on equilibrium of all horizontal forces.

— For the next to lowest anchor, check that the required depth of penetration as indicated by the analysis is in fact available.

— For the lowest anchor, take the depth of penetration at the proposed design value and calculate the anchor force from horizontal force equilibrium.

— Check the bending moments that will develop in the wall at each stage of construction. Critical conditions will occur immediately before each anchor is installed.

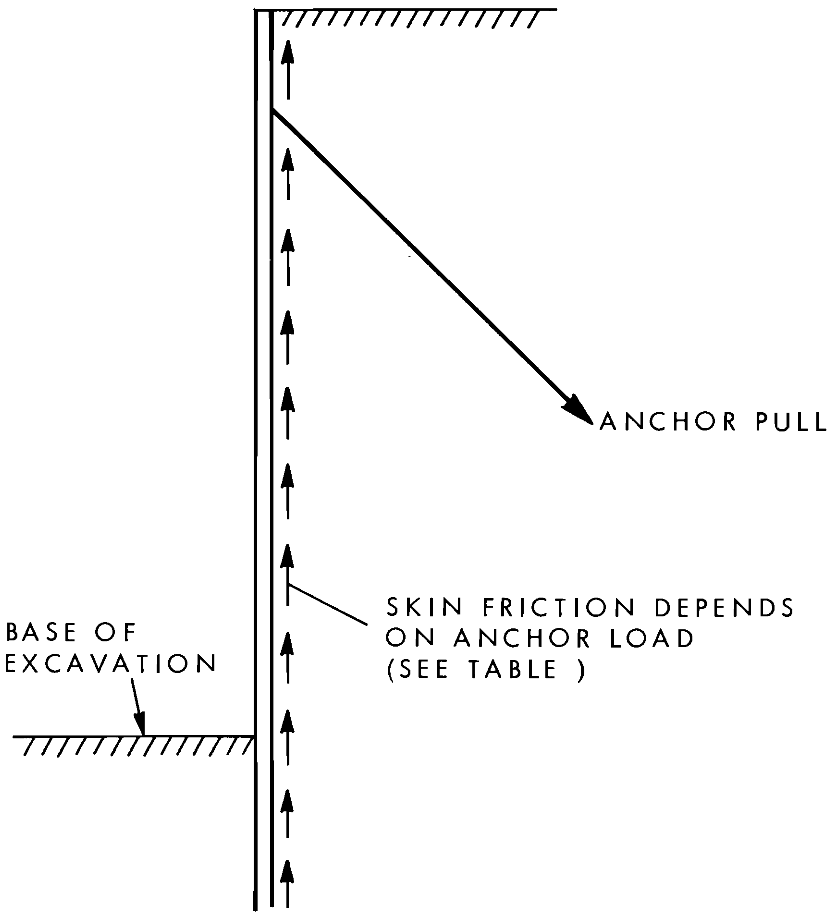
— In general, where the lowest anchor is more than a few feet from the bottom of the wall, the wall should penetrate below the base of the cut at least to the depth at which the computed resultant earth pressure is zero. (Where this is not so, substantial bending moments may exist in the bottom section of the wall and the load on the lowest anchor may increase as a result of stress redistribution.)

5.2.4.3. *Effects of Anchor Inclination*

Anchors are usually inclined downwards, transmitting the vertical component of the anchor force into the anchored vertical member. This force should be considered in design, together with the weight of the vertical member itself.

Forces which resist downward movement due to the inclined anchor load are skin friction and the reaction at the base of the vertical member. The range of possible skin friction mobilized to resist downward movement for diaphragm walls is shown in Fig. 5.4. The reaction of the base of the vertical member should be computed in accordance with Chapters 6 and 7 of this Manual.

When soldier piles are used, vertical forces are concentrated in the piles. Only minimal friction, if any, can be mobilized. Such vertical forces must therefore be supported in end-bearing at the base of the pile. The base capacity of the pile must be checked, otherwise unacceptable vertical and horizontal deformations may take place. It is sound practice for the base of a steel WF or H section soldier pile to be placed in a clean pre-bored hole filled with concrete. This markedly increases available base capacity.



BASIS OF ANCHOR DESIGN	δ_f	c_a
K_A	0	0
$\frac{1}{2}(K_A + K_o)$	1/3 ϕ	c_u
K_o	2/3 ϕ	c_u

FIG 5.4

POSSIBLE SKIN FRICTION ON DIAPHRAGM WALL

Settlement of vertical members produces some reduction in anchor loads with a consequent tendency for outward displacement of the supported face. It is therefore essential to monitor vertical and horizontal movements at the top and bottom of the excavation at regular intervals throughout the course of the work.

5.2.4.4. *Design of Soil and Rock Anchors*

(1) *General*

The anchors discussed in this article are considered to be temporary. Each consists of a stressing tendon (rod or cable) connecting a fixed anchorage (within the soil or rock mass) to a surface anchorage or head. In cohesionless soils and rock the fixed anchorages are almost invariably formed by pressure grouting techniques while in stiff cohesive soils tremie methods may also be used except where the inclination of the hole to the horizontal is not very great. Typical anchor details are shown in Fig. 5.5.

The performance of soil and rock anchors is dependent, not only on minor variations in soil and groundwater conditions, but also on construction techniques and details. Consequently, the prediction of anchor capacity is difficult. Anchorage capacities calculated using the procedures outlined here are considered to represent reasonable design limits, but must be proved by test or proof loading during construction.

(2) *Allowable anchor load in soils*

The load capacity of an anchor in soils should, wherever possible, be established by a pull-out test (5.2.4.4.(3)). The allowable anchor load T_a, is determined by dividing the test load capacity T_t, of the anchor by a factor of safety FS.

$$T_a = \frac{1}{FS} \ (T_t)$$

Required minimum values of FS vary between 1.5 and 2.0 depending upon inclination and are shown in Fig 5.6. Values between those given may be obtained by linear interpolation.

Where no pull-out tests are carried out, the allowable anchor load T_a, is obtained by dividing the computed load capacity T_c, of the anchor 5.2.4.4.(4)) by a minimum factor of safety FS = 3. In this case:

$$T_a = \frac{T_c}{3}$$

(3) *Anchor load capacity established from pull-out tests*

Where the load capacity of anchors are to be determined by pull-out tests, it is recommended that at least one anchor in ten of those actually used in the project, with a minimum of three in each soil or rock type, be tested.

The pull-out capacity of the anchor T_t , is defined as that load at which withdrawal of the anchor begins. If the load is not clearly apparent from the test data, the pull-out capacity is taken as the maximum load at which withdrawal is still tolerable for the structure. If an ultimate capacity is not reached, or no withdrawal is observed in the test loading, the greatest applied test load should be assumed as the pull-out capacity for calculation of the allowable anchor load T_a.

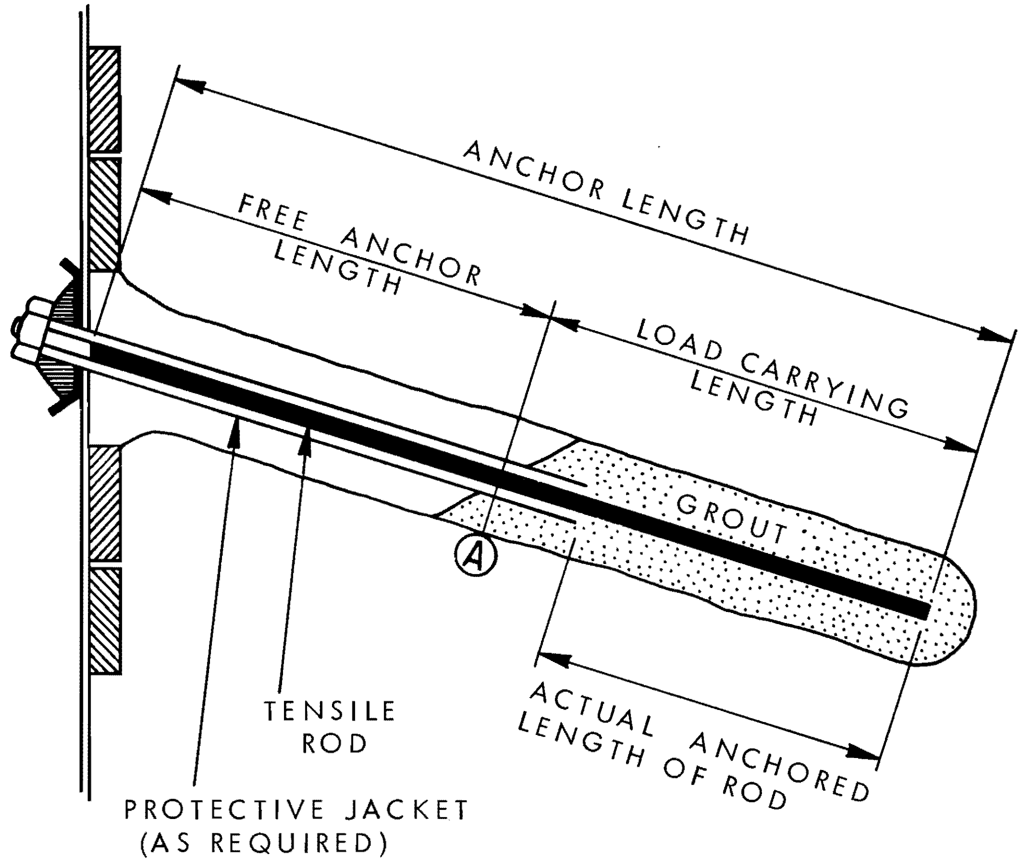


FIG 5.5

TYPICAL ANCHOR DETAILS

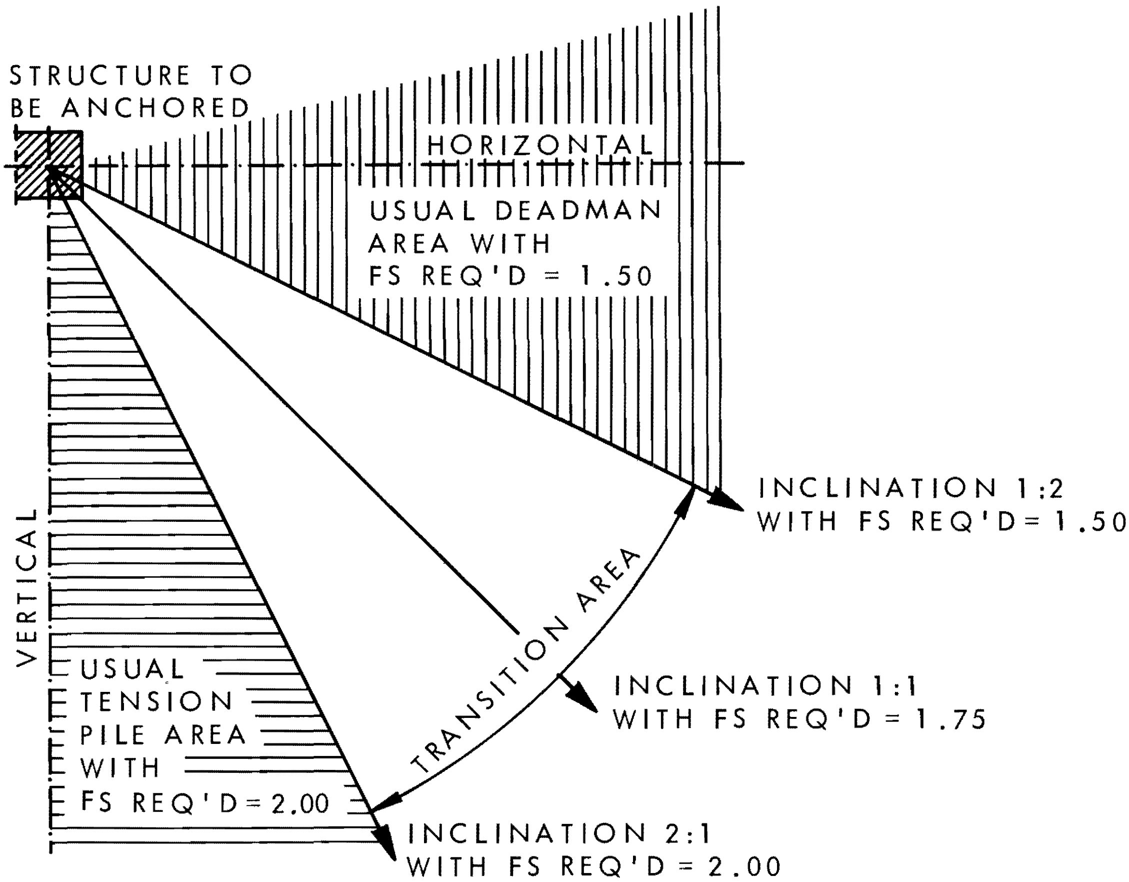


FIG 5.6
REQUIRED SAFETY FACTORS FOR LOAD TESTED ANCHOR

(4) *Computations of anchor load capacity in soils*

(a) *cohesionless soils*

Computation of the anchor load capacity T_c, for grouted anchors in cohesionless soils can be estimated from the equation.

$$T_c = \sigma'_z A_s K_f$$

where σ'_z = effective vertical stress at the midpoint of the load carrying length (Fig. 5.5)

A_s = effective surface area of the anchorage

and K_f = anchorage coefficient dependent on the soil type and density as given in Table 5.1

TABLE 5.1

Variations in K_f

Soil Type	Density		
	Loose	Compact	Dense
Silt	1	4	10
Fine Sand	1.5	6	15
Medium Sand	5	12	20
Coarse Sand, Gravel	10	20	30

(b) *cohesive soils*

Computation of the anchor load capacity T_c, in stiff to very hard cohesive soils can be estimated from the equation.

$$T_c = A_s c_u \alpha$$

where A_s = effective surface area of the anchorage

c_u = average undrained shear strength of the soil over the anchorage length

and α = reduction factor related to the undrained shear strength (Fig. 5.7).

Anchors should not be formed in *soft* or *firm* clays (c_u = 250 to 1,000 lb/sq ft) or in sensitive clays because of the large deformations which can occur, both at and subsequent to loading.

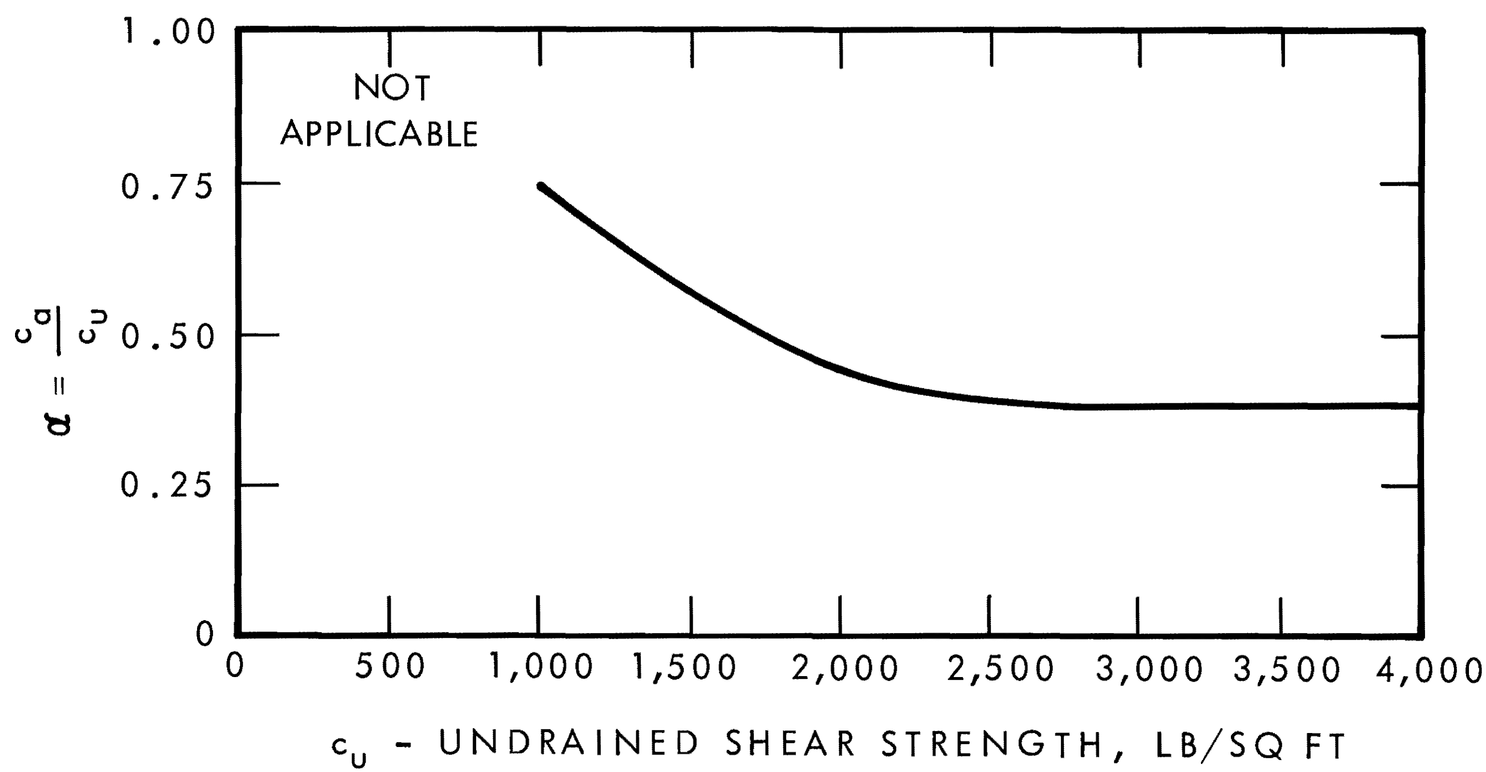


FIG 5.7
RELATIONSHIP BETWEEN α AND UNDRAINED SHEAR STRENGTH FOR ANCHORAGE DESIGN

(5) *Allowable anchor load in rock*

Anchorage design in rock is based on an allowable grout to rock bond stress S_b, acting over the fixed anchorage length. S_b should not exceed the minimum value given by the following criteria;

$S_b \ngtr$ 1/30 (unconfined compressive strength of the rock)

$\ngtr$ 1/30 (unconfined compressive strength of the grout)

$\ngtr$ 200 lb/sq in.

Using these criteria, the allowable anchor load T_a, is given by the equation:

$$T_a = A_s S_b$$

where A_s = effective area of the anchorage, with a minimum anchorage length of 10 ft.

(6) *Location of anchorages*

The depth of overburden above any anchorage should not be less than 15 ft in soil (Fig. 5.8) and not less than 5 ft in *sound* rock where *sound* rock is defined in Chapter 4 of this Manual. Unsound or weathered rock should be treated as soil.

Where multiple anchors are used, the minimum spacing between anchorages in a line should be equal to 4D, where D = anchorage diameter (Fig 5.8).

(7) *Installation of anchorages*

The advancement of the hole for a soil or rock anchor must be carried out in a manner that precludes the possibility of loss of ground or flow of wet soil into the hole. Where penetrating water-bearing zones or *wet soil* are encountered, holes must be temporarily cased. Such casing should only be withdrawn after that section of the hole in water-bearing zones is backfilled with concrete or grout to the level of hydrostatic pressure within the water-bearing zone.

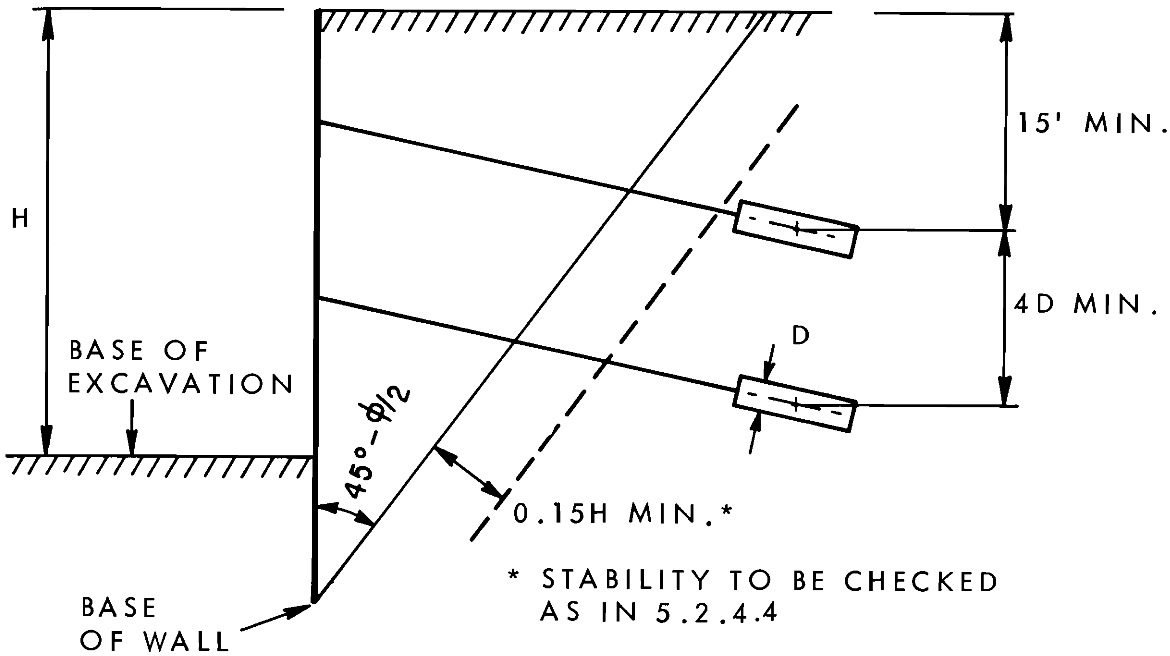
In common practice, anchorages in soil are effected by advancing a hole using a hollow stem auger to the full anchorage depth. Where the hole is 8 in. diameter or less, grout is injected through the hollow stem at pressures often considerably in excess of 100 lb/sq in. to achieve a grouted anchorage length. Care must be taken to ensure that high grout pressures will not cause damage to adjacent structures or services. Where the hole is up to 12 in. diameter, concrete rather than grout is pumped through the hollow stem as the auger is withdrawn. Since the hole is of large diameter, it is not necessary to use high pressures for the concrete.

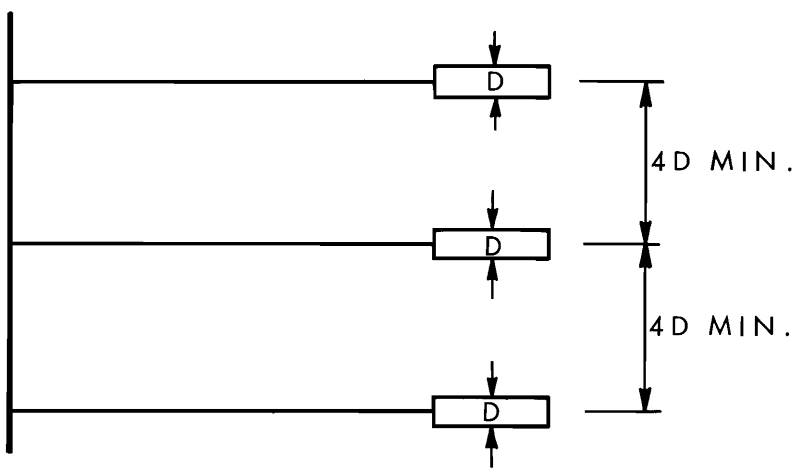
(8) *Stressing and proof loading of anchors*

Each installed anchor should be stressed and proof loaded to 1.33 times the allowable or design working load for the anchor. The following procedure is recommended:

i) Test load the anchor to 80 per cent of the ultimate tensile strength of the tendon, hold for five minutes and then reduce the load to zero.

ii) Restress the anchor to the required working load plus 10 per cent and record tendon movement at the ram as the load is incrementally applied. During this second loading cycle, the load-extension graph obtained should compare closely with the estimated extension of the free length





PLAN VIEW

FIG 5.8

MINIMUM SPACING AND DEPTH FOR SOIL ANCHORS of tendon. Lock off the anchor at working load, plus an allowance (usually 10 per cent) for relaxation and pull-in of wedges. Working load should not exceed 60 per cent of the ultimate tensile strength of the tendon.

iii) Check the anchor after 15 minutes. If a loss of prestress in excess of 5 per cent is recorded, restore to working load plus 10 per cent by shimming.

iv) Repeat step (iii).

v) If a further loss of prestress is recorded, reduce the anchor load until creep ceases. A safe working load for the anchor is then equal to 60 per cent of the load showing no creep after 15 minutes.

5.2.4.5. *Overall Stability of Anchorage System*

The overall stability of the anchorage system is checked by analysing the stability of the block of soil lying between the wall and the anchorages.

Note: It is assumed that overall stability of the excavation has initially been checked by the methods given under 5.2.6 Basal Instability and 5.4.3 Overall Stability.

(1) *Single-level anchor systems*

The *anchoring body* (Figs. 5.9 and 5.10) is analysed for stability with respect to movement along a lower failure plane DF. This plane extends from the base of the retaining wall to the mid-point of the anchorage. For the case where the anchorage lies below the base of the retaining wall, stability of the *anchoring body* is assumed.

(2) *Multiple-level anchor systems*

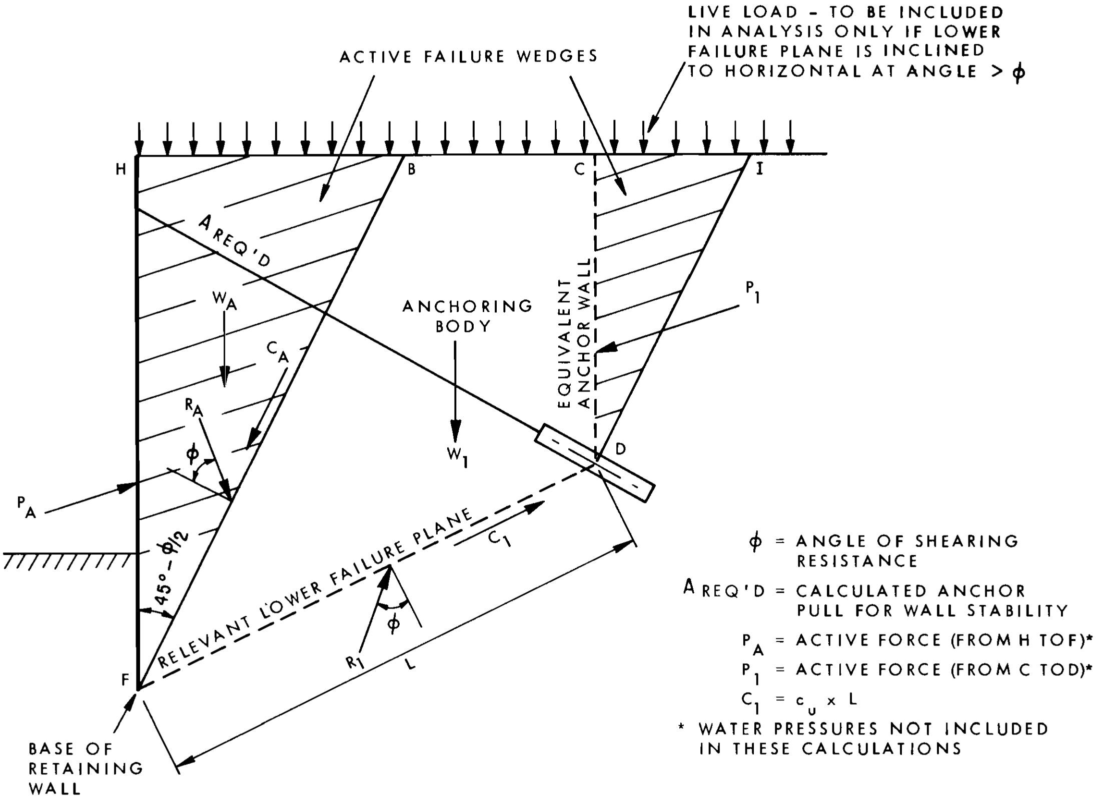
The stability of each level of the anchoring system should be checked, commencing at the top anchor. At each level, the required anchor force is the sum of all anchor forces above the relevant lower failure plane.

Three possible cases according to the location of the anchorages with respect to the base of the retaining wall are shown in Fig. 5.11. The failure planes requiring stability analyses are indicated in each case. The method of analysis for each *anchoring body* is the same as that indicated for the single anchorage system.

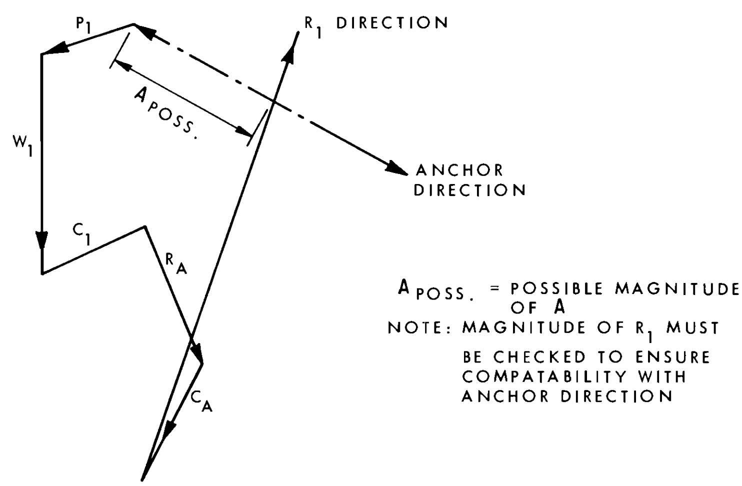
5.2.5 *STRUTTED WALLS*

5.2.5.1 *Design Loads - Earth Pressures*

The distribution of stress against the walls of strutted excavations cannot be adequately predicted from theory. Field measurements show that the actual stress distribution varies from section to section depending on many construction variables. Since for a safe excavation no single strut may be overloaded, design is based on an envelope of probable distributions, determined from field experience.



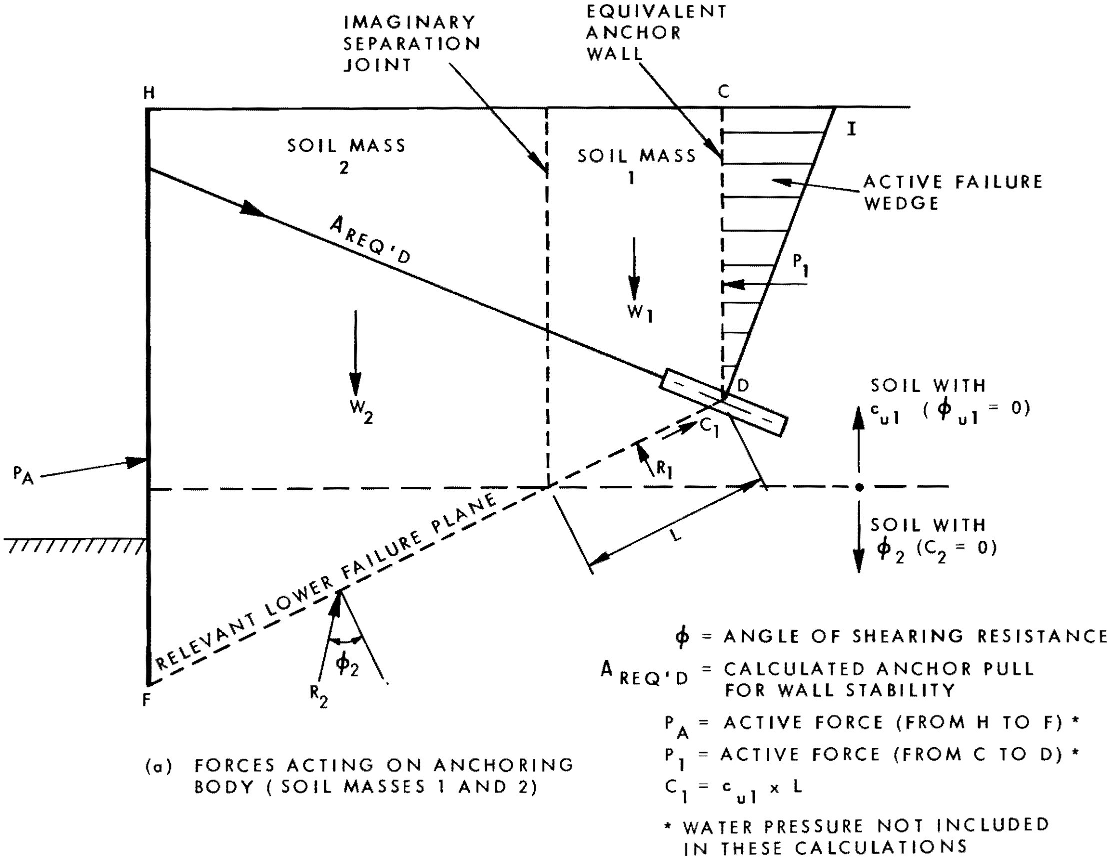
(a) FORCES ACTING ON ANCHORING BODY



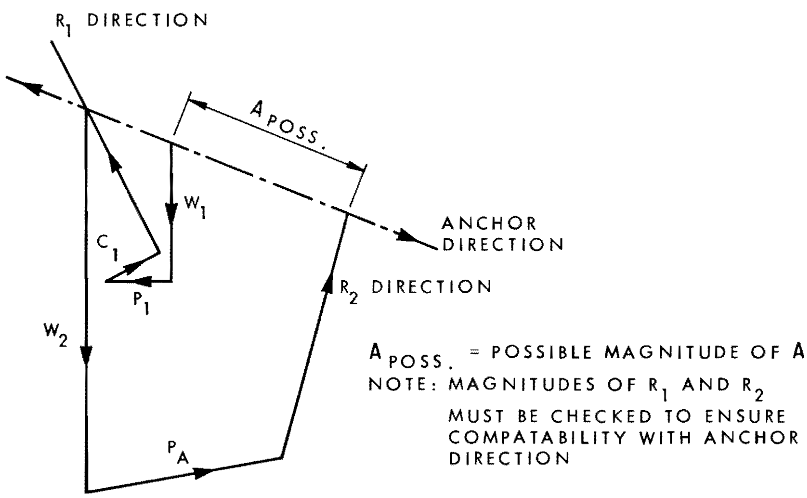
(b) VECTOR DIAGRAM

FIG 5.9

GRAPHICAL ANALYSIS OF ANCHORED WALLS



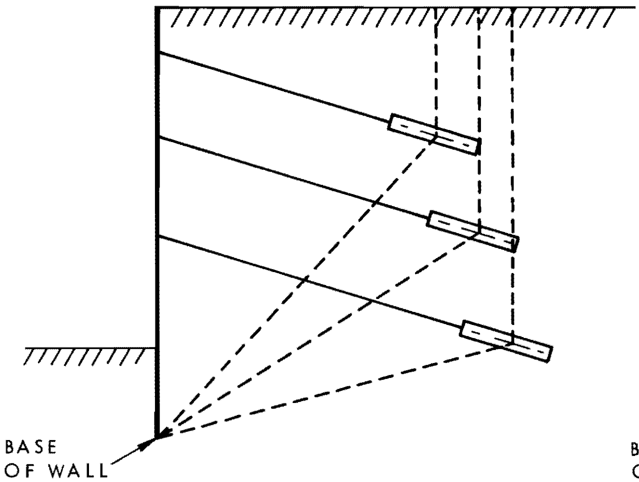
(a) FORCES ACTING ON ANCHORING BODY (SOIL MASSES 1 AND 2)



(b) VECTOR DIAGRAM

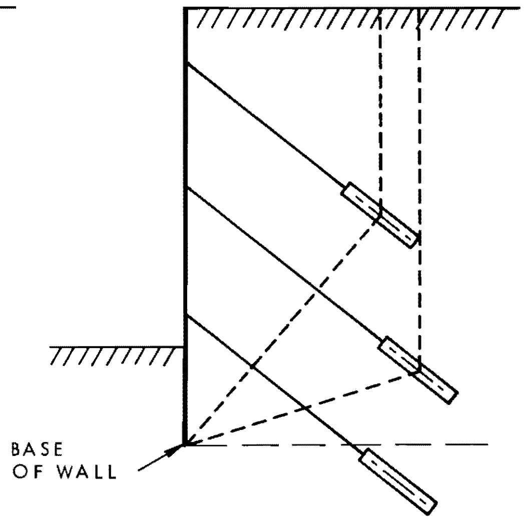
FIG 5.10

GRAPHICAL ANALYSIS OF ANCHORED WALLS (TWO LAYERS)



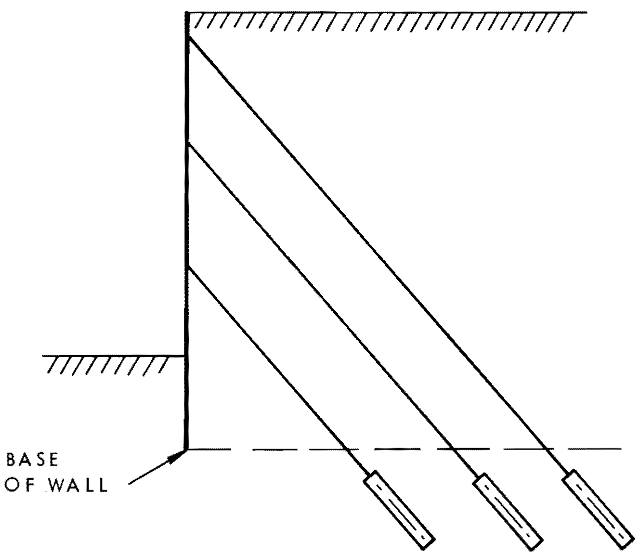
CASE A

ALL ANCHORS ABOVE LEVEL
OF BASE OF RETAINING WALL



CASE B

ANCHORS ABOVE AND
BELOW LEVEL OF BASE
OF RETAINING WALL



CASE C

ALL ANCHORS BELOW LEVEL OF BASE
OF RETAINING WALL. NO STABILITY
ANALYSIS REQUIRED

FIG 5.11

TYPICAL MULTIPLE LEVEL ANCHOR SYSTEMS SHOWING POTENTIAL
FAILURE PLANES REQUIRING STABILITY ANALYSIS

(1) *Cohesionless soils*

For cohesionless soils, the pressure distribution to be used in design is shown in Fig 5.12(a). The area of this rectangular pressure diagram produces a lateral thrust about 30 per cent greater than the Rankine *active* value.

(2) *Soft to firm clays (c_u = 250 to 1,000 lb/sq ft)*

For soft to firm clays, the pressure distributions to be used are given in Fig 5.12 (b), where the parameters referred to in the text and figures are:

γ = unit weight of material, lb/cu ft

H = depth of excavation, ft

c_u = undrained shear strength of clay beside and immediately beneath the cut, lb/sq ft

FS_b = factor of safety against base heave. (See 5.2.6.1.)

Where a great depth of soft clay exists below the excavation, use the pressure diagram in Fig 5.12(b) and a value for $m = 0.4\ FS_b$.

Where a much more resistant layer is encountered at or near the base of the excavation, use Fig 5.12(b) and a value for m = 1.0.

In no case should the maximum pressure ordinate be less than 0.3 γH.

(3) *Stiff to very hard clays (c_u > 1,000 lb/sq ft)*

For stiff clays, the pressure diagram shown in Fig 5.12(c) is recommended. The variation in the value of maximum stress level, ranging from 0.2 γH to 0.4 γH, is dependent on the character of the clay, the degree of jointing or fissuring, and the reduction in strength of the clay with time. The choice within this range can only be made on the basis of experience and detailed knowledge of the clay deposit.

5.2.5.2. *Surcharge Loading*

The design of all members must include the effects of loads of street traffic, construction equipment, supported utilities, adjacent structures which are not underpinned, and any other loads that must be carried by the walls of the excavation during the construction period. (TERZAGHI & PECK, 1967)

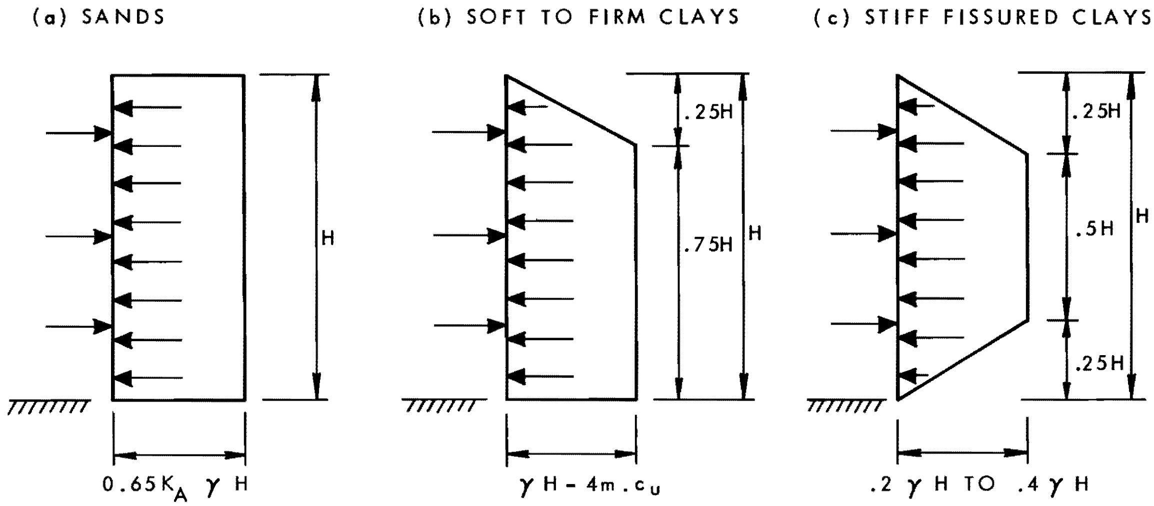
5.2.5.3. *Effect of Seepage and Drainage*

Groundwater pressures estimated in design should be consistent with the required or permissible drawdown levels. Where soldier beams with wood lagging are to be utilized, groundwater is generally assumed to be at, or below, the base of the interior of the excavation. When the wall is intended to prevent all leakage of groundwater, maximum exterior groundwater pressures should be used.

5.2.5.4. *Design and Installation of Members*

(1) *Structural design*

Members such as walls, struts, soldier piles, and sheeting should be sized for the loads defined in 5.2.5.1 to 5.2.5.3 in accordance with the structural requirements of Part 4 of the National Building Code 1975. The effects of combined axial and flexural loading, unsupported span lengths and lateral stability of the members must be considered in the design.



NOTES:

1. CHECK SYSTEM FOR PARTIAL EXCAVATION CONDITION
2. IF THE FREE WATER LEVEL IS ABOVE THE BASE OF THE EXCAVATION THE HYDROSTATIC PRESSURE MUST BE ADDED TO THE ABOVE PRESSURE DISTRIBUTION IN SANDS
3. IF SURCHARGE LOADINGS ARE PRESENT AT OR NEAR THE GROUND SURFACE THESE MUST BE INCLUDED IN THE LATERAL PRESSURE CALCULATION.
4. VALUES OF m ARE GIVEN IN 5.2.5.1(2)

FIG 5.12 (After Peck)

PRESSURE DISTRIBUTION COMPLETE EXCAVATION

Details on contractor's shop drawings should show appropriate means for posting of struts and walers, lacing of struts in both vertical and horizontal planes to provide lateral stability, web and connection stiffeners, brackets, and provisions for wedging and jacking of struts to prevent horizontal movement. Details are a vital element in the adequacy and safety of temporary earth retaining structures and should be shown completely on the contractor's shop drawings in conjunction with the methods and sequence of installation of all elements of the structure. Particular attention should be given to procedures for pre-stressing, wedging, or jacking to maintain tight contact for all bracing members and to provide for uniformity of distribution of load to struts and walers.

(2) *Struts*

Struts should be designed for the loads calculated from 5.2.5.1. to 5.2.5.3. on the assumption that the members subjected to bending stresses are hinged at each strut position.

Long struts may be subjected to large temperature-induced stresses when exposed to the sun and it may be necessary to make an allowance in design for this effect.

(3) *Rakers and raker footings*

Rakers and their connections may be designed in the same way as horizontal struts.

Raker footings should be designed in accordance with the design principles for shallow foundations subject to inclined loading, as outlined in Chapter 6 of this Manual. Footings and the foundation material should be protected from freezing or deterioration.

All raker footings should be located outside the zone of influence of the buried portion of soldier piles and at a distance of not less than 1.5D from the piles, where D = depth of penetration of the piles below the base of the excavation. No excavation should be made within two footing widths of the raker footings on the side opposite the rakers.

(4) *Soldier piles*

The design loads defined in 5.2.5.1 to 5.2.5.3 should be used for the design of soldier piles or soldier beams. Soldier piles should be designed as continuous members supported at strut or tie back points,and stresses should be checked for various stages of construction when only partial support may exist. For preliminary sizing, the members may be selected assuming walers and piles to be hinged at the support points (i.e. the whole system is simply supported) and the calculated bending moments reduced by 25 per cent.

Interim construction conditions must be analysed to check flexural stresses in the soldier piles. When sloping berm excavation procedures are employed, the depth to the *equivalent support point* which allows the effective span of the pile to be determined, may be estimated using the method illustrated in Fig 5.13.

Unless large soil movements adjacent to the excavation can be tolerated, the soldier piles should be in place before excavation commences and should remain in contact with the soil at all times. Consequently, no excavation behind soldier piles should be allowed.

If soldier piles are installed in pre-augered holes, sloughing or caving of the holes must be prevented. Immediately after installation of the piles, the hole should be backfilled with lean concrete. If, because of possible caving or sloughing, pre-augering is not possible, the soldier piles should be installed by driving.

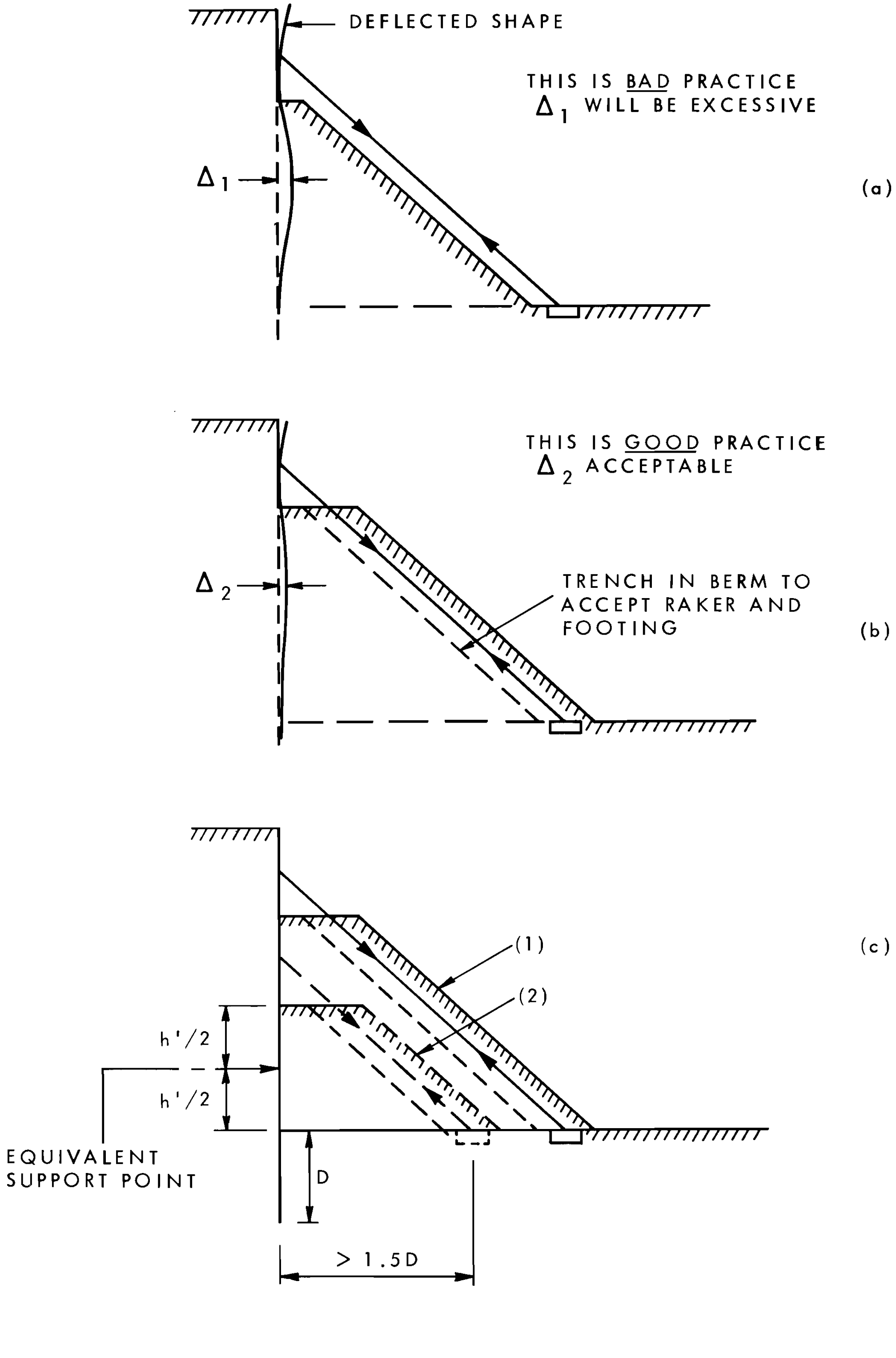


FIG 5.13
PLACEMENT OF RAKER STRUTS

(5) *Lagging*

The design of timber planks or lagging should conform with good practice and the lagging should be of good quality hardwood. Lagging is installed by hand after a depth of several feet is excavated. The maximum depth made each time before a section of lagging is placed depends on the soil characteristics. Soft clay and cohesionless soils must be planked in short depths to reduce the amount of soil moving into the excavation. Immediately after placement of lagging, wedges should be driven to force it tightly against the soil. Voids behind the lagging should be packed by hand to reduce the amount of loss of ground. The depth of excavation below any lagging boards that have not been backfilled should not exceed four feet.

To minimize the possibility of erratic loss of ground in local areas when excavating sands and silts below original groundwater, it is essential that straw packing, burlap, or in extreme conditions, grouting be used behind the lagging as it is installed.

The design of timber lagging, in common practice, is empirical. In general, the following practice has been found satisfactory for excavation depths 25 ft or less.

TABLE 5.2

Thickness of lagging related to spacing of soldier piles

Spacing of Soldier Piles	Thickness of Lagging
Up to 6.5 ft	2 in
6.5 ft to 8.5 ft	3 in
8.5 ft to 10 ft	4 in

For excavation depths greater than 25 ft but less than 75 ft, the lagging thickness should be increased by 1 in.

(6) *Diaphragm walls, sheetpiling*

Generally diaphragm walls and sheetpiled walls used for excavation support are designed as continuous walls between supports. (TERZAGHI, 1954).

The installation and construction *in situ* of diaphragm walls is critically dependent on construction techniques and should only be carried out by contractors of recognized competence in this field of work.

(7) *Penetration of vertical members*

If the bracing system is designed such that there are no struts near the bottom of the excavation, the depth of penetration provided should be 1.5 times the depth required for moment equilibrium about the lowest strut.

The resistance provided to the portion of wall penetrating below the base of the excavation is computed using the *passive* pressure and ignoring wall friction.

For driven soldier piles, the maximum horizontal force on the flange of the soldier pile below the bottom of the excavation may be taken as 1.5 times the values computed for the width of the flange, providing that the pile spacing is not less than 5 times the flange width.

For piles placed in a concreted base, the diameter of the concrete-filled hole may be used in place of the flange width as discussed in the preceding paragraph.

5.2.5.5 *Interim Construction Conditions*

The design of all members including struts, walers, sheetpiling, walls and soldier piles should be checked for several stages of partial excavation when the wall is assumed to be continuous over the strut immediately above the excavation level and supported some distance below the excavation level by the available *passive* resistance. (See Fig 5.13 for the case where only a berm remains to support the wall.) This condition could produce the maximum loading in struts and walers.

Where excessive stresses or loads would result from interim construction conditions using regular construction procedures, trenching techniques can be employed to advantage.

The design of members should also be checked for the condition when portions of the building within the excavated area are completed and lower struts are removed. Consideration must be given to the possible increase in loading on the upper struts remaining in place; also the span between that portion of the building that has been completed and the lowest strut then in place must be considered in relation to flexural stresses.

Because of the possibility of delays in construction, it is essential that the safety of the excavation is satisfactory for *long term* as opposed to *short term* conditions. The pressure distribution diagrams given in Fig 5.12 are for *short term* conditions only and in certain cases the pressure distribution can vary considerably with time. It is therefore essential that monitoring of deformation (and hence implied stresses) be carried out systematically during construction and additional struts added if required.

5.2.6. *BASAL INSTABILITY*

5.2.6.1 *Soft to Firm Clays* *(c_u = 250 to 1,000 lb/sq ft)*

Deep excavations in these soils are subject to base heave failures which result from overstressing the soil in shear. (Fig 5.14). The factor of safety with respect to base heave is:

$$FS_b = \frac{N_b \ c_u}{\gamma H + q}$$

where c_u is the undrained shear strength of the soil below base level*, N_b is stability factor dependent upon the geometry of the excavation, and the remaining parameters are those defined in 5.2.5.1.(2).

As the potential for bottom instability increases, the heave in the base of the excavation increases and the loss of ground adjacent to the excavation increases. It should be noted that, in the case of soft clays underlying the base of the excavation where FS_b is less than 2, substantial deformations may result with consequent loss of ground. If soft clay extends to a considerable depth below the excavation, the beneficial effects of even relatively stiff sheeting in reducing deformation have been found to be minimal. However, if the lower portion of the sheeting is driven into a hard stratum, the effectiveness of the sheeting in reducing deformation is increased appreciably. No satisfactory theoretical procedures exist to determine sheeting or wall pressures at depth below the base of the excavation.

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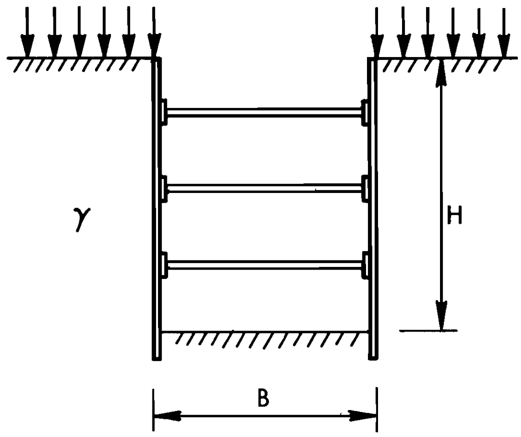
* For clay soils of moderate to high plasticity, definition of c_u by conventional means can lead to an overestimate of shear strength (BJERRUM, 1972).



$$FS_b = \frac{N_b c_u}{\gamma H + q}$$

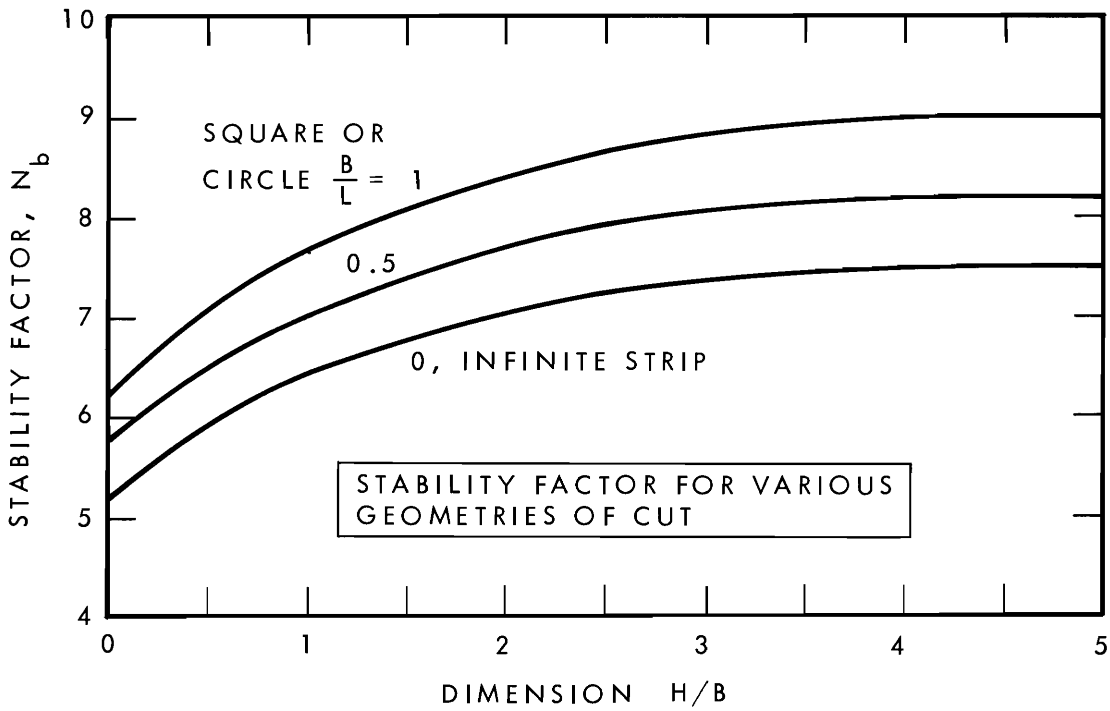


FIG 5.14 (After Janbu)

FACTOR OF SAFETY WITH RESPECT TO BASE HEAVE

5.2.6.2. *Cohesionless Soils*

In cohesionless soils, basal instability takes the form of piping or heave and is associated with groundwater flow. Groundwater control can be achieved by drainage, by using sheetpiling to support the face of the excavation and providing adequate penetration of the piling for cut-off purposes, or by a combination of the two methods. This is discussed in detail in 5.3 - CONTROL OF GROUNDWATER IN EXCAVATIONS .

5.2.7. *MOVEMENTS ASSOCIATED WITH EXCAVATION*

5.2.7.1. *General*

Movements associated with excavations are primarily related to construction technique and commonly consist of lateral yield of the soil and support system towards the excavations with corresponding vertical movement adjacent to the excavation walls. Both lateral and vertical movements due to yield are generally of the same order of magnitude; however, if very flexible soldier piles are used, lateral movements can be grossly increased. Where construction technique is poor, erratic movements can also occur due to loss of ground or erosion behind the wall.

5.2.7.2. *Strutted Excavations*

Movements due to yield in strutted excavations are, to a large extent, unavoidable since they are controlled not by design assumptions but by construction details and procedures. Such movements develop in each excavation phase before the next level of struts is installed.

(1) *Magnitude of movements*

For well-constructed support systems, designed in accordance with the requirements of 5.2.5, STRUTTED WALLS it has been found that deformations are dependent on the wall height and related to the soil type.

a) *cohesionless soils*

If the struts are installed as soon as the support level is reached and prestressed to 100 per cent of the design load, the lateral movements in the system can be expected to be of the order of 0.2 per cent of the depth of the excavations.

b) *soft to firm clays* (c_u *= 250 to 1,000 lb/sq ft)*

Substantial movements often occur when vertical cuts are made in soft clays. These movements occur in spite of well-constructed and installed support system. Measurements have shown that 60 to 80 per cent of the total lateral yield occurs below the excavation level. Struts should be installed and pre-stressed as soon as the excavation reaches the support level. The applied prestress should be 100 per cent of the design load. However, lateral movements below the bottom support will increase significantly if the excavation reaches a depth where the factor of safety against base heave becomes less than about 2.0. Even if the system is properly installed, the maximum lateral movement of the support system is likely to be 1 to 2 per cent of the excavation depth.

c) *stiff clay* (c_u *> 1,000 lb/sq ft)*

The lateral movements of temporary support systems decrease sharply as the shear strength of the soil increases. Limited available data indicates that maximum lateral movements of excavations in stiff clays with c_u > 1,5000 lb/sq ft will be less than 0.2 per cent of the excavation depth and often less than 0.1 per cent provided struts are installed as soon as the support level is reached and prestressed to 100 per cent of the design load.

(2) *Means of reducing movements*

To reduce the magnitude of movements it is necessary to reduce the shear stresses induced in the ground by excavation. Two possible methods can be utilized to effect this:

a) The unsupported depth of wall between supports can be shortened by using more levels of struts. Generally, a vertical spacing of 8 ft between strut levels is considered a minimum from a construction viewpoint, with 12 ft to 16 ft being preferred. The maximum spacing for small lateral deformation is generally close to 12 ft, but where underpinning of small or light adjacent structures is omitted, and tightly braced excavation walls are intended to prevent movement of such adjacent structures, the vertical spacing should be kept to the minimum value of 8 ft.

b) The unsupported depth of wall can be shortened by use of the trenching method as illustrated in Fig 5.13.

5.2.7.3. *Anchored Walls*

The yield movements of anchored walls are controlled more by design methods than with strutted walls. The number of anchors and the vertical spacing of such anchors, plays a significant part in controlling the degree of lateral deformation. In normal practice, movements due to yield of anchored diaphragms, sheeted or soldier pile walls are usually less than for strutted walls for the same depth of excavation.

(1) *Magnitude of movements*

If the wall and anchor system is designed on the basis of an earth pressure coefficient $K = K_A$, assuming good construction technique, lateral wall movements and adjacent vertical settlements are generally about 0.2 per cent of the excavation depth.

If an earth pressure coefficient $K = K_o$ is used in design, associated movements are generally about 0.1 per cent of the excavation depth.

There is no definitive evidence to date to relate associated movements with stiffness of the wall, but limited data available suggests that diaphragm walls tend to induce lesser movements than sheet piled or soldier pile walls.

In sensitive soils experience has shown that heavy prestressing of ground anchors with the intention of reducing lateral movements can in fact lead to overstress of the soil and result in increased vertical movement adjacent to the wall.

5.2.7.4. *Loss of Ground Behind Excavations*

(1) *Cohesionless soils*

Because lateral yield of strutted or anchored excavations in cohesionless soils is usually small, the loss of ground behind such systems is also usually small. However, placement of lagging and backfill behind the lagging must be emphasized (see 5.2.5.4.(5)). With good workmanship and attention to detail, settlements can often be kept to less than 0.05 per cent of the depth of the excavation.

Two exceptions to the above general rule are sometimes encountered. These are loss of ground due to flow of water into the cut with concomitant soil erosion, and loss of ground due to densification of loose cohesionless deposits. It is difficult to estimate settlements associated with flow or migration of sands into a cut because of dependence on construction techniques, groundwater levels and local soil situations. Settlements due to densification of loose cohesionless deposits can be of the order of 1.5 per cent of the depth of the cut.

(2) *Soft to firm clays* (c_u = *250 to 1,000 lb/sq ft)*

Because significant lateral yield occurs in cuts in soft clays, the surface settlements associated with such cuts are also substantial. The magnitude and extent of these surface settlements may be estimated using the relationships shown in Fig 5.15.

5.2.8. *UNDERPINNING*

Structures adjacent to excavations will frequently need to be supported. The support required will depend on the soil type, and the magnitude of the foundation loads and their locations with respect to the excavation. The structural loads may be carried by direct underpinning of the foundations, or by the provision of additional lateral support to the face of the excavation. The following recommendations assume the foundation material to be soil. Rather less underpinning and more face support might well be considered for rock foundations.

5.2.8.1. *General Support Requirements*

The geometry of zones within which support for adjacent structures is usually considered necessary is shown on Fig 5.16. In general, foundations of adjacent heavy structures which lie within the *active* earth Zone A surrounding the excavation will need to be underpinned. For vertical cuts, this is defined as a zone inside of the line rising at a slope of 2 vertical on 1 horizontal from a point 2 ft below the edge of the base of the excavation. The limiting slope angle within which underpinning may be required, Zone B, ranges from 2 vertical on 1 horizontal, to 1 vertical on 1 horizontal, depending on the character of the soils. Where building foundations lying immediately between these limits are so heavy that they would expand the *active* zone, underpinning should be provided.

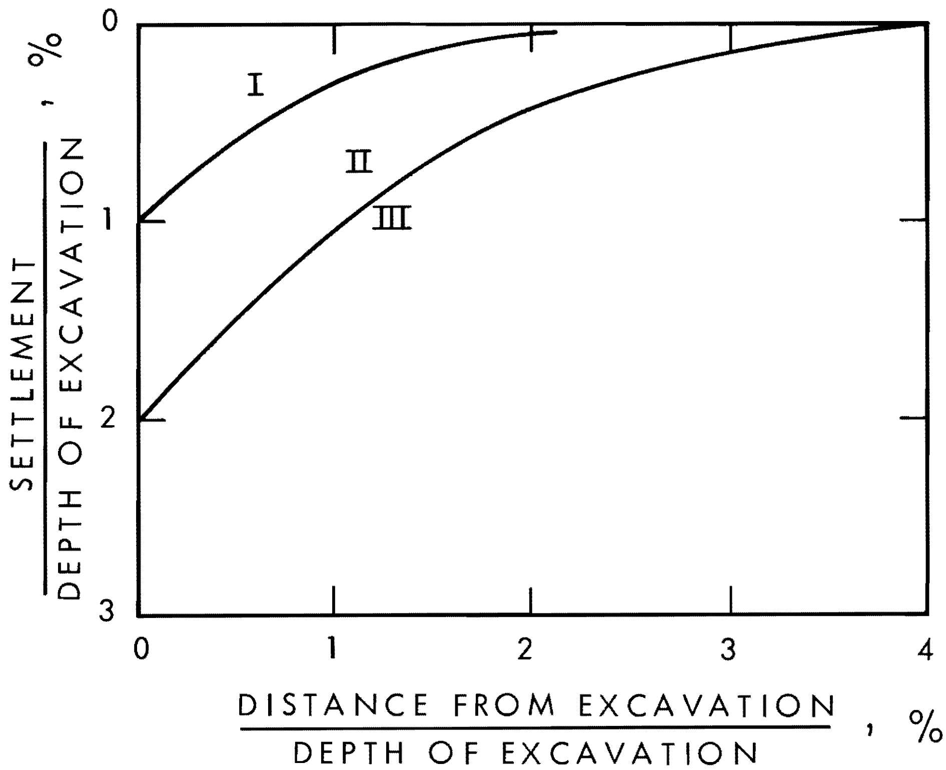
Where foundations of smaller structures lying in the *active* Zone A adjacent to the excavation apply an equivalent line load on the front wall or on side walls perpendicular to the street totalling less than 2,000 lb/lin ft, it might be possible to eliminate underpinning and control movement by careful excavation within tightly braced excavation walls.

In all cases of excavation in soil where foundations of adjacent structures supported in Zones A and B are not underpinned, the temporary retaining structure and the permanent subsurface structure must be designed to resist the horizontal and vertical pressures applied by these foundations, computed in the manner described in Appendix 5A.

5.2.8.2. *Requirements for Underpinning Supports*

For excavation in soil, all portions of the bearing area or tip of the underpinning members should extend into Zone C of Fig 5.16, below a line rising at a slope of 1 vertical on 1 horizontal from a point 2 ft below the edge of the base of the excavation. The support provided to the underpinning member below this line should accommodate the total applied load with adequate safety factor. In this case no pressures from the underpinned structure need be considered in the design of the excavation support system.

Underpinning walls, piers, or piles which form a portion of the excavation support system should be extended to a depth not less than 2 ft below the lowest nearby subgrade of the excavation. The bearing support for such underpinning members should provide an adequate safety factor during excavation and construction as well as after the completion of construction. Where underpinning members



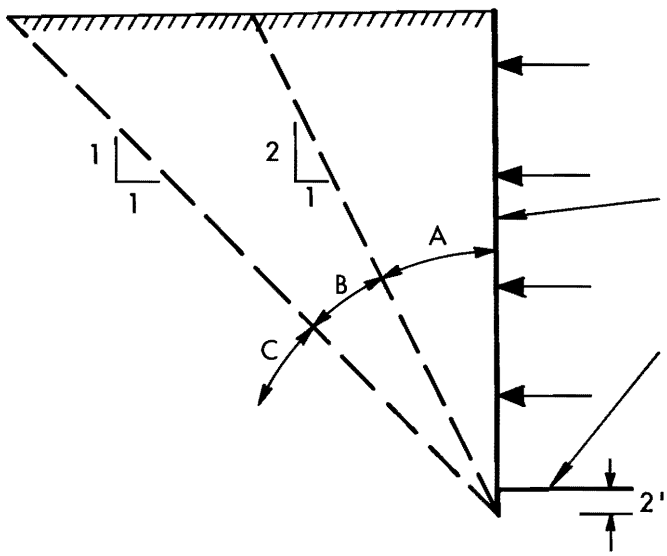
ZONE I - SAND AND SOFT TO HARD CLAY, (c_u > 500 LB/SQ FT)

ZONE II - VERY SOFT TO SOFT CLAY (c_u < 500 LB/SQ FT)
1) - LIMITED DEPTH OF CLAY BELOW BASE OF EXCAVATION
2) - SIGNIFICANT DEPTH OF CLAY BELOW BASE OF EXCAVATION WHERE FS_b < 1.3

ZONE III - VERY SOFT TO SOFT CLAY, (c_u < 500 LB/SQ FT)
1) - SIGNIFICANT DEPTH OF CLAY BELOW BASE OF EXCAVATION AND WHERE FS_b > 1.3

FIG 5.15 (After Peck)

SETTLEMENTS ADJACENT TO OPEN CUTS



ZONE A:

FOUNDATIONS WITHIN THIS ZONE GENERALLY REQUIRE UNDERPINNING. HORIZONTAL AND VERTICAL PRESSURES ON EXCAVATION WALL OF NON-UNDERPINNED FOUNDATIONS MUST BE CONSIDERED

ZONE B:

FOUNDATIONS WITHIN THIS ZONE GENERALLY DO NOT REQUIRE UNDERPINNING. HORIZONTAL AND VERTICAL PRESSURES ON EXCAVATION WALL OF NON-UNDERPINNED FOUNDATIONS MUST BE CONSIDERED

ZONE C:

UNDERPINNING TO STRUCTURES MUST BE FOUNDED IN THIS ZONE. PRESSURES FROM UNDERPINNING GENERALLY NEED NOT BE CONSIDERED

FIG 5.16

REQUIREMENTS FOR UNDERPINNING

will be exposed at the sides of the excavation, they must be capable of resisting any horizontal loads applied to them by non-underpinned foundations in Zones A and B. These loads may be calculated on the basis of the information given in 5.A.6. of Appendix 5A.

Design bearing pressures for foundations of underpinning members should be limited to the allowable values described elsewhere in the Manual. Note however, that lower values than usual might well be desired for underpinning members in order to restrict possible settlements.

5.3. CONTROL OF GROUNDWATER IN EXCAVATIONS

5.3.1. *METHODS FOR THE CONTROL AND REMOVAL OF GROUNDWATER*

Water may be removed from excavations by gravity drainage or by pumping from sumps, well points or bored wells. The method adopted will depend upon;

- soil conditions, such as the permeability of pervious layers, the sequence of the soil strata and local variations of permeability within the soil profile,
- the depth of excavation below groundwater level or relative to piezometric levels in underlying strata,
- the method of supporting the sides of the excavation, i.e. open or sheeted excavations, and
- the necessity or otherwise of safeguarding existing adjacent structures.

Good practice requires that the following conditions be fulfilled when dewatering excavations,

A dewatering method be chosen that will assure the stability of sides and bottom of excavations as well as the integrity and safety of adjacent structures.

The lowered water table be kept constantly under full control thus avoiding fluctuations liable to cause instability of the excavation.

Effective filters be provided where necessary to prevent loss of ground.

Adequate pumping and standby pumping capacity be provided.

Pumped water be discharged in a manner that will not interfere with the excavation.

Pumping methods be adopted for groundwater lowering that will not lead to damage of adjacent structures, such as by settlement.

For most soils, the groundwater table during construction must be maintained at least 2 ft to 5 ft below the bottom of the excavation in order to ensure *dry* working conditions. It needs to be maintained at a somewhat lower level for silts than for sands to keep traffic from pumping water to the surface and making the bottom of the excavation wet or spongy.

5.3.2. *GRAVITY DRAINAGE*

Where site conditions permit, water can be drained by gravity from an excavation.

5.3.3. *PUMPING FROM INSIDE THE EXCAVATION*

Frequently groundwater levels are controlled by pumping from sumps inside the excavation. This method often creates hazards in construction and in many instances is used when pumping should be carried out from outside the excavation.

The location of drainage channels leading to the sumps should be a matter for careful consideration in order to ensure that the whole of the excavated area is drained at all stages. The efficient design and maintenance of drainage ditches are particularly important where water seeps down a sheeted or sloping face and is intercepted by the ditches. The slope of the ditches should be sufficient to avoid silting up due to soil carried into them, but they should not be so steep that erosion occurs. It is often convenient to pipe the drainage ditches using slotted or perforated pipe surrounded by graded gravel filter material.

Loss of ground from around the sump must be prevented. The best method is to install the filter medium between the ground and the sump. This can be accomplished by placing a cage of perforated metal inside the sump excavation and filling the space between the cage and the ground with graded gravel filter material, the sheeting for the sump excavation being withdrawn as the filter material is placed.

5.3.3.1. *Pumping from Unsupported Excavations*

Generally, where excavations are in rock, groundwater will seep down the face of the excavations, where it can be collected by drainage ditches and led to a sump without causing instability of the face. However, where faces of excavations are in permeable soil, the velocity of the water seeping into the excavation may be sufficient to cause movement of soil particles which leads to collapse of the sides. To avoid this trouble, the face of the excavation should be cut back to a stable slope. The water level is lowered by pumping and the water, as it emerges, at or near the toe of the slope, can drain into a graded gravel or stone filter.

(1) *Heave due to artesian pressure at depth*

Where an excavation is underlain by an impermeable layer, such as a stratum of silt or clay that is, in turn, underlain by a pervious stratum of sand under artesian pressure, upward seepage from the deeper stratum may keep the bottom of a large excavation wet, even though drainage sumps may be in use. If this situation exists, it may be necessary to lower the head in the deep sand stratum below the bottom of the excavation by means of relief wells. If the intervening clay stratum, as shown in Fig 5.17 is impervious, the hydrostatic head in the deep sand can be somewhat higher than the bottom of the excavation, but in no event should the net head above the bottom of the excavation exceed 80 per cent of the submerged weight of the soil above the top of the artesian aquifer. Otherwise heave may occur in the bottom of the excavation.

(2) *Use of relief wells*

If relief wells are installed within the excavation, it should be noted that the allowable upward seepage gradient depends upon the uniformity and permeability of the fine-grained soils overlying the pervious stratum. In clays, gradients as high as 0.5 may be safe, whereas in silty soils it is necessary to lower the artesian head below the bottom of the excavation in order to control upward seepage and achieve a dry, stable bottom. Stratification of the soil will also affect the allowable uplift pressure.

5.3.3.2. *Pumping from Sheeted Excavations*

If a sheeted excavation is made using closed sheet piling or an *in situ* impermeable diaphragm taken down into a thick impermeable stratum, the flow of water in the overlying pervious ground will be completely cut off and the dewatering of the area enclosed by the cofferdam is simple.

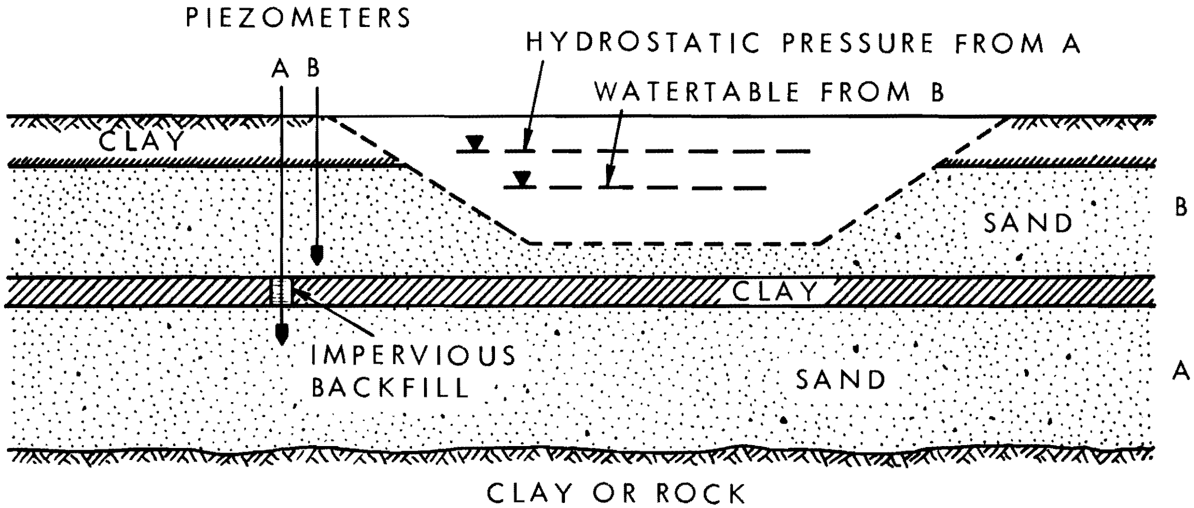


FIG 5.17

ARTESIAN GROUNDWATER CONDITIONS BELOW EXCAVATION

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(1) *Basal instability of sheeted excavations due to seepage*

If the sheeting or diaphragm does not penetrate into an impermeable layer, flow will occur under the sheeting or diaphragm and up into the excavation. Unless groundwater control is adequate, this flow will cause instability of the base, generally *piping* in dense sands or *heave* in loose sands. *Piping* occurs if the seepage exit gradient at the base of the excavations equals about one. *Heave* occurs if the uplift force at the sheeting toe exceeds the submerged weight of the overlying soil column. To prevent *piping* or *heave*, sheeting must penetrate a sufficient depth below the base of the excavation. Fig 5.18 indicates the seepage exit gradients related to sheeting penetration in isotropic sands.

For clean sand, exit gradients between 0.5 and 0.75 will cause unstable conditions for men and equipment operating on the subgrade. To avoid this, the sheeting penetration should be sufficient to provide a safety factor of 1.5 to 2 against *piping* or *heave*.

The sheeting penetration required in layered subsoils is given in Fig 5.19.

(2) *Heave due to artesian pressure at depth*

See paragraph 5.3.3.1.(1)

(3) *Use of relief wells*

See paragraph 5.3.3.1.(2)

5.3.4. *PUMPING FROM OUTSIDE THE EXCAVATION*

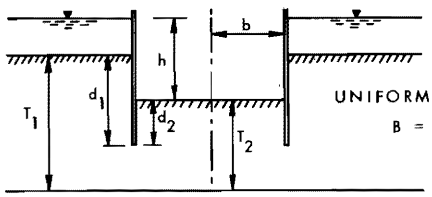
The object of an external groundwater lowering system is to lower the water table below the level at which work is to be carried out or to reduce the pressures in underlying pervious layers so that the stability of the excavation is ensured at all times.

The methods used for lowering the groundwater level outside an excavation can be listed as follows:

- i) Excavated wells or sumps with independent pumps.
- ii) A number of small diameter well points *(the well point system)*.
- iii) Multiple bored filter wells with independent submersible pumps in each well *(the deep well system)*, or where the quantities of water to be pumped are small, well point in jet eductors *(the eductor system)*.
- iv) Multi-stage installations of (ii) and (iii) above.
- v) Vacuum well methods.

In all methods, loss or disturbance of the ground should be prevented by the use of filters.

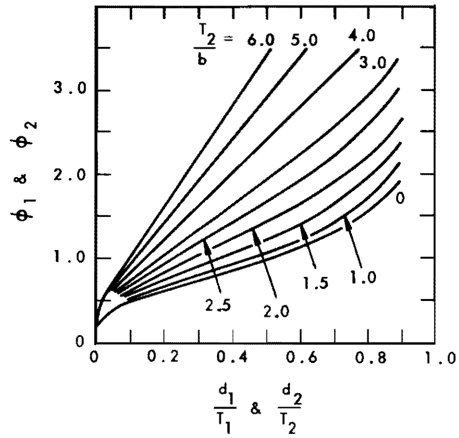
When the water is pumped from a well, the quantity pumped depends on the level to which the water immediately outside the well screens is lowered, on the radius of the well and on the permeability of the ground. Pumping causes the water table around the well to take the form of an inverted cone known as the cone of depression. When water is pumped simultaneously from a number of wells, the cones of depression intersect. The lowering in level of the enclosed water table (Fig 5.20) depends upon the spacing and size of the wells as well as upon the reduction in the water table immediately adjacent to the wells. The fact that the cones of depression of the wells intersect means that the yield of water pumped from any one of the wells is considerably less than that of a single isolated well for the same lowering in water level.



(a)

NOTE:

Where groundwater level is below ground surface T_1 and d_1 are taken from the groundwater level



(b)

1) FOR TWO PARALLEL WALLS

$$i_{exit} = \frac{h}{d_2} \times \frac{\phi_2}{\phi_1 + \phi_2}$$

$$Q = \frac{kh}{\phi_1 + \phi_2} \quad \text{PER UNIT LENGTH}$$

ϕ_1 AND ϕ_2 FROM FIG. (b)

2) FOR A CIRCULAR EXCAVATION

$$i_{exit} = 1.3\,\frac{h}{d_2} \times \frac{\phi_2}{\phi_1 + \phi_2}$$

$$Q = 0.8\,\frac{kh}{\phi_1 + \phi_2}\,2\pi R$$

3) FOR A SQUARE EXCAVATION

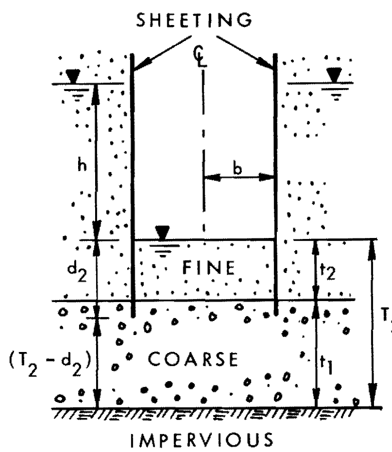
$$i_{exit} = 1.3\,\frac{h}{d_2} \times \frac{\phi_2}{\phi_1 + \phi_2} \quad \text{(MIDDLE SECTION OF THE SIDES)}$$

$$i_{exit} = 1.7\,\frac{h}{d_2} \times \frac{\phi_2}{\phi_1 + \phi_2} \quad \text{(IN THE CORNERS)}$$

$$Q = 0.7\,\frac{kh}{\phi_1 + \phi_2}\,4B$$

FIG 5.18

PENETRATION OF SHEETING AND EXIT GRADIENTS FOR ISOTROPIC SANDS

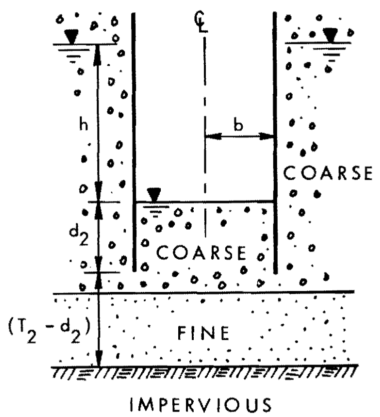


COARSE SAND UNDERLYING FINE SAND

PRESENCE OF COARSE LAYER MAKES FLOW IN FINE MATERIAL MORE NEARLY VERTICAL AND GENERALLY INCREASES SEEPAGE GRADIENTS IN THE FINE LAYER COMPARED TO THE HOMOGENEOUS CROSS-SECTION OF FIG. 5.18

IF TOP OF COARSE LAYER IS AT A DEPTH BELOW SHEETING TIPS GREATER THAN WIDTH OF EXCAVATION, EXIT GRADIENTS OF FIG. 5.18 FOR INFINITE DEPTH APPLY

IF TOP OF COARSE LAYER IS AT A DEPTH BELOW SHEETING TIPS LESS THAN WIDTH OF EXCAVATION, THE UPLIFT PRESSURES ARE GREATER THAN FOR THE HOMOGENEOUS CROSS-SECTION. IF PERMEABILITY OF COARSE LAYER IS MORE THAN TEN TIMES THAT OF FINE LAYER, FAILURE HEAD (h) = THICKNESS OF FINE LAYER (t_2)

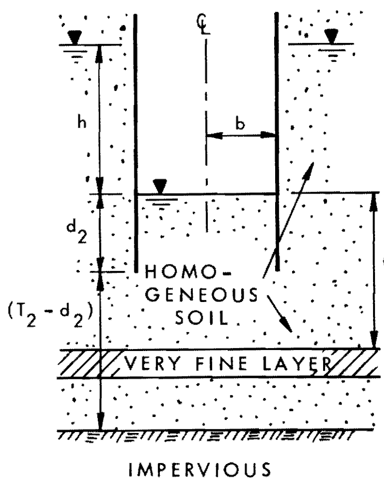


FINE SAND UNDERLYING COARSE SAND

PRESENCE OF FINE LAYER CONSTRICTS FLOW BENEATH SHEETING AND GENERALLY DECREASES SEEPAGE GRADIENTS IN THE COARSE LAYER

IF TOP OF FINE LAYER LIES BELOW SHEETING TIPS, EXIT GRADIENTS ARE INTERMEDIATE BETWEEN THOSE CALCULATED FOR AN IMPERMEABLE BOUNDARY AT TOP AND BOTTOM OF THE FINE LAYER IN FIG. 5.18

IF TOP OF THE FINE LAYER LIES ABOVE SHEETING TIPS THE EXIT GRADIENTS OF FIG. 5.18 ARE SOMEWHAT CONSERVATIVE FOR PENETRATION REQUIRED



FINE LAYER IN HOMOGENEOUS SAND STRATUM

IF THE TOP OF FINE LAYER IS AT A DEPTH GREATER THAN WIDTH OF EXCAVATION BELOW SHEETING TIPS, EXIT GRADIENTS OF FIG. 5.18 APPLY, ASSUMING IMPERVIOUS BASE AT TOP OF FINE LAYER.

IF TOP OF FINE LAYER IS AT A DEPTH LESS THAN WIDTH OF EXCAVATION BELOW SHEETING TIPS, PRESSURE RELIEF IS REQUIRED SO THAT UNBALANCED HEAD BELOW FINE LAYER DOES NOT EXCEED HEIGHT OF SOIL ABOVE BASE OF LAYER

IF FINE LAYER LIES ABOVE SUBGRADE OF EXCAVATION, FINAL CONDITION IS SAFER THAN HOMOGENEOUS CASE, BUT DANGEROUS CONDITION MAY ARISE DURING EXCAVATION ABOVE THE FINE LAYER AND PRESSURE RELIEF IS REQUIRED AS IN THE PRECEDING CASE

FIG 5.19

PENETRATION OF SHEETING REQUIRED TO PREVENT PIPING IN STRATIFIED SANDS

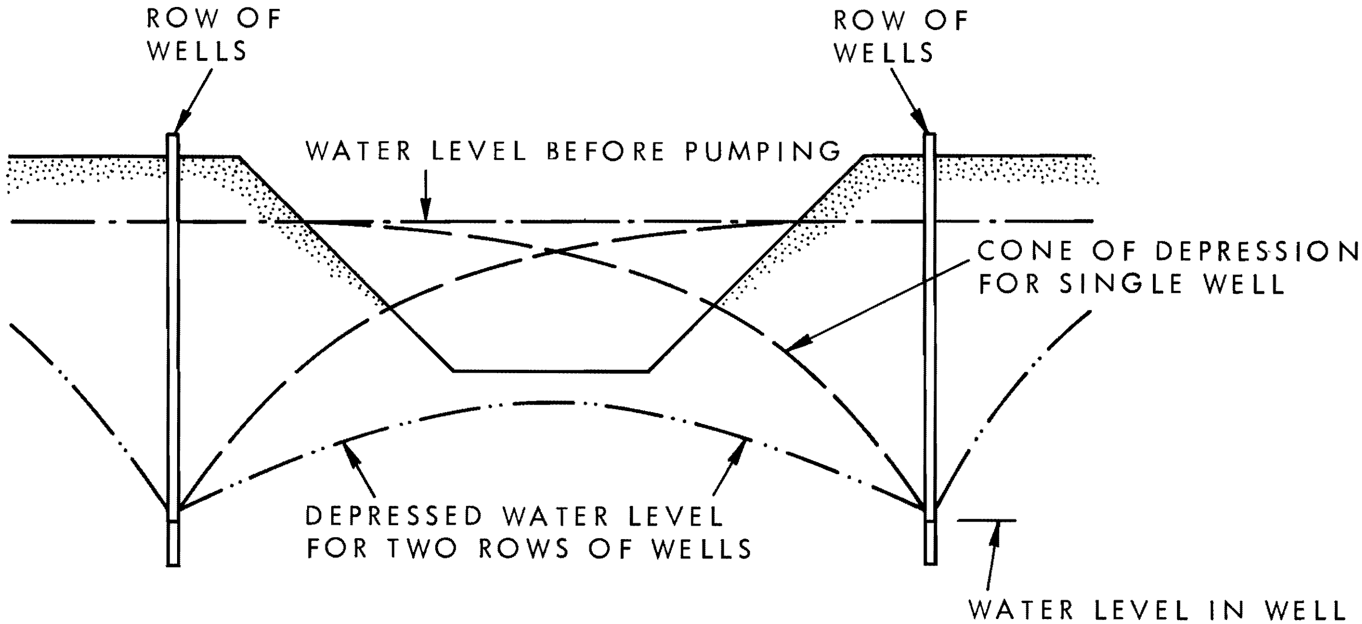


FIG 5.20
REDUCTION OF WATER LEVELS BELOW AN EXCAVATION BY WELL GROUNDWATER LOWERING SYSTEM

The details of these methods and their design is given in various texts. (The reader is directed to TERZAGHI & PECK, 1967 for a comprehensive description.)

5.4. FOUNDATION AND RETAINING WALLS

5.4.1. *DESIGN LOADS*

Theoretical wall pressures are discussed in Appendix 5A. The following modifying factors should be considered.

5.4.1.1. *Effect of Wall Movement and Wall Restraint*

For a discussion of the effect of wall movement on the coefficient of lateral earth pressure against rigid walls, see 5.A.2. Appendix 5A.

(1) *Yielding rigid walls*

For cohesionless backfill, reduction of pressures to active values requires wall rotation. Y/H, of only a few tenths of a percent, (H = height of the wall and Y = lateral displacement of the top of the wall). For cohesive backfills, movements necessary to produce *active* earth pressures can be much greater.

(2) *Restrained rigid walls*

Where a rigid wall is prevented from moving, lateral earth pressures depend greatly on the compaction procedure (See 5.4.1.2.).

5.4.1.2. *Effect of Compaction*

Compaction of backfill in a confined wedge behind the wall tends to increase the horizontal pressures.

(1) *Cohesionless soils*

Cohesionless soils compacted behind rigid, unyielding walls can produce horizontal pressures up to nearly twice the *at rest* values depending on the amount of compaction. Typical values of K_o range from 0.4 to 0.8.

For moderately compacted fill behind rigid yielding walls, design can be based on *active* values. Typical values of K_A range from 0.2 to 0.4.

(2) *Cohesive soils*

In cohesive soils, residual compaction pressures can vary substantially. Where compaction is light to moderate, *at rest* pressures ($K_o = 1 - \sin \phi'$) may be assumed. Where compaction is heavier (to a density greater than 95% of Standard Proctor), the following points should be noted:

a) *yielding walls*

The walls should be designed for *at rest* pressures; however, some movement is likely as a result of compaction.

b) *unyielding walls*

Lateral earth pressures corresponding to K = 1.0 or higher are likely to occur.

5.4.1.3. *Effect of Backfill Type*

(1) *Cohesionless soils*

Soils classified as GW, GP, SW, or SP, provide excellent backfill material and theroetical earth pressures may be considered valid for design purposes.

(2) *Sandy clays and clayey sands*

Soils classified as SC, SM, GC or GM, provide suitable wall backfill if kept dry, but are subject to frost action when wet. Where drainage is adequate, theoretical earth pressures may be considered valid for design purposes.

(3) *Silts and clayey silts*

Soils classified as CL, MH, ML, OL are often subject to excessive frost action and swelling when used as wall backfill. Under these conditions, design for *active* pressures is inadequate, even for yielding walls, as resulting wall movement is likely to be excessive and continuous; design using an earth pressure coefficient $K = 1.0$ is recommended.

5.4.1.4. *Low Walls*

For walls less than about 20 ft in height and where the total cost is not great, detailed testing to determine soil properties, and elaborate pressure computations, are usually not justified. Wall pressures can be adequately estimated on the basis of equivalent fluid pressures providing drainage requirements are satisfied.

(1) *Equivalent fluid pressures*

Equivalent fluid pressures for straight slope backfill may be obtained using Fig 5.21, and for broken slope backfill using Fig 5.22. A dead load surcharge should be included as an equivalent weight of backfill, and a line load surcharge included as a resultant force on the back of the wall obtained using Fig 5.23.

(2) *Drainage*

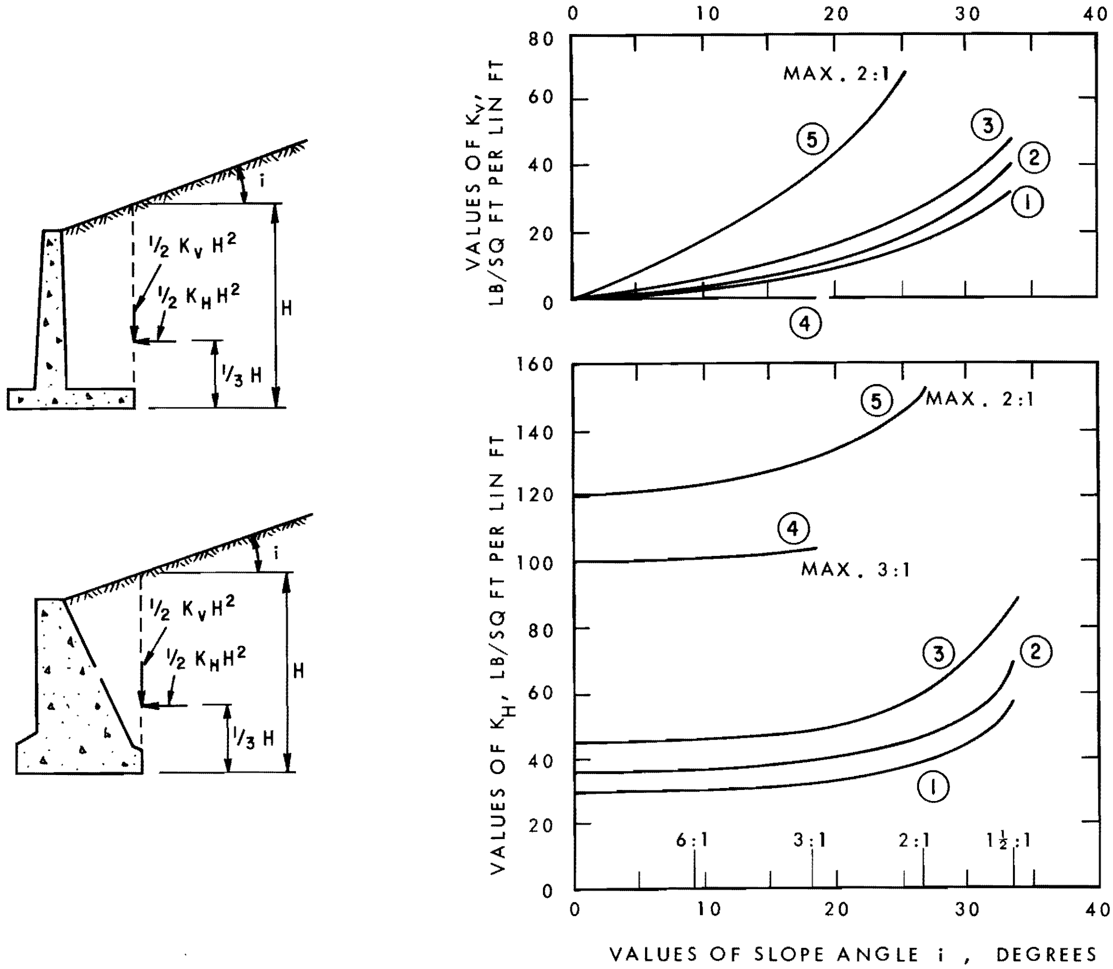
The equivalent fluid pressures include effects of seepage and time conditioned changes in the backfill. However, provisions should be made to prevent accumulation of water behind the wall. As a minimum measure weep holes should be provided for drainage. Backfill of soil types in 5.4.1.3.(2) & (3) should be covered with a surface layer of impervious soil.

5.4.2. *RETAINING WALL DESIGN*

Design criteria for overturning and sliding are given in Fig 5.24.

5.4.2.1. *Stability Against Sliding*

The base should be placed at least 3 ft below ground surface in front of the wall and below the depth of frost action, the zone of seasonal volume change and the depth of scour. Sliding stability must be adequate without including the *passive* pressure at the toe. If insufficient sliding resistance is available, a pile foundation should be provided or the base of the wall should be lowered; in which case the *passive* resistance below frost depth should be considered in the analysis.



CIRCLED NUMBERS INDICATE THE FOLLOWING SOIL TYPES:

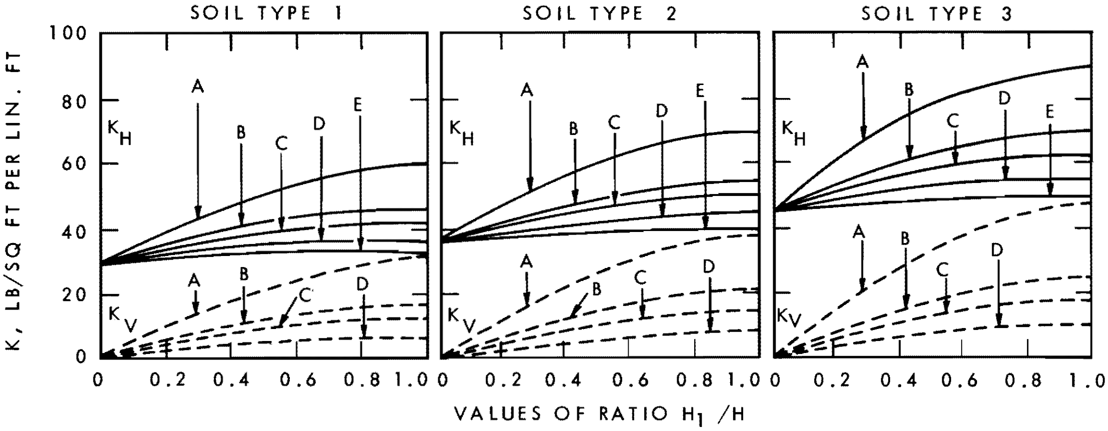
1. CLEAN SAND AND GRAVEL: GW, GP, SW, SP.
2. DIRTY SAND AND GRAVEL OF RESTRICTED PERMEABILITY: GM, GM-GP, SM, SM-SP
3. STIFF RESIDUAL SILTS AND CLAYS, SILTY FINE SANDS, CLAYEY SANDS AND GRAVELS: CL, ML, CH, MH, SM, SC, GC
4. VERY SOFT TO SOFT CLAY, SILTY CLAY, ORGANIC SILT AND CLAY: CL, ML, OL, CH, MH, OH
5. MEDIUM TO STIFF CLAY DEPOSITED IN CHUNKS AND PROTECTED FROM INFILTRATION: CL, CH

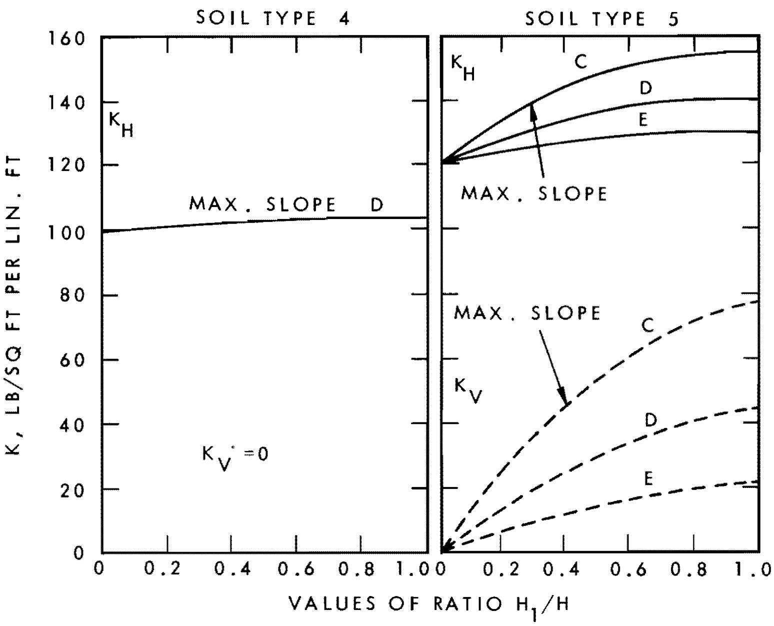
FOR TYPE 5 MATERIAL H IS REDUCED BY 4 FT. RESULTANT ACTS AT A HEIGHT OF (H-4)/3 ABOVE BASE

FIG 5.21

DESIGN LOADS FOR LOW RETAINING WALLS (STRAIGHT SLOPE BACKFILL)

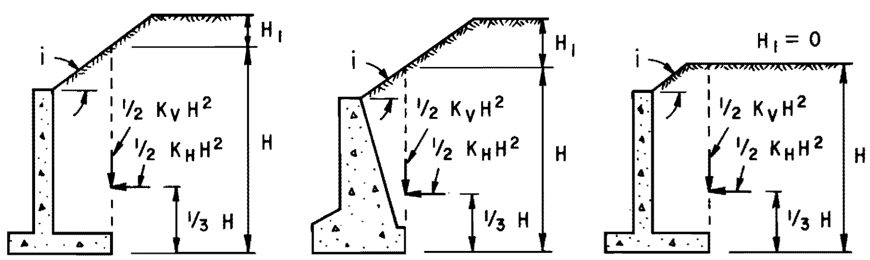
(From 'Soil Mechanics in Engineering Practice' by TERZAGHI & PECK, 1948. Used with permission of J. Wiley & Sons Inc.).





SLOPE

A	$1\frac{1}{2}:1$
B	$1\frac{3}{4}:1$
C	2 :1
D	3 :1
E	6 :1

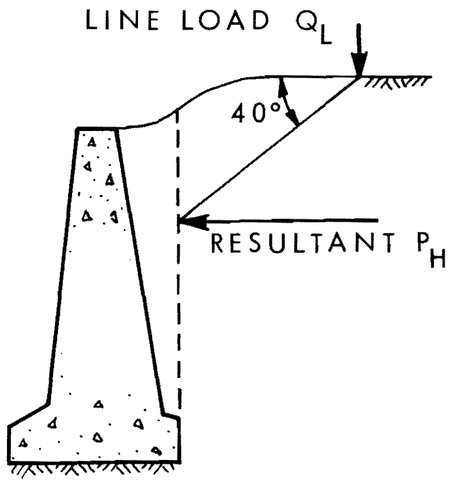


FOR TYPE 5 MATERIAL H IS REDUCED BY 4FT, RESULTANT ACTS AT A HEIGHT OF (H - 4)/3 ABOVE BASE.
FOR DESCRIPTION OF SOIL TYPE SEE FIG. 5.21

FIG 5.22

DESIGN LOADS FOR LOW RETAINING WALLS (BROKEN SLOPE BACKFILL)

(From 'Soil Mechanics in Engineering Practice' by TERZAGHI & PECK, 1948. Used with permission of J. Wiley and Sons Inc.).



RESULTANT $P_H = KQ_L$

SOIL TYPE FIG. NO. 5.21	VALUE OF K
1	0.27
2	0.30
3	0.39
4	1.00
5	1.00

FIG 5.23
RESULTANT FORCE FROM LINE LOAD (APPROXIMATE METHOD FOR LOW RETAINING WALLS)

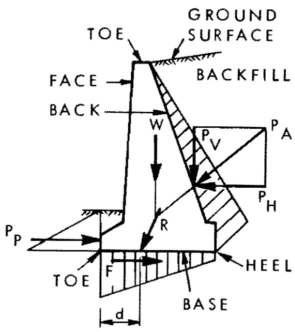
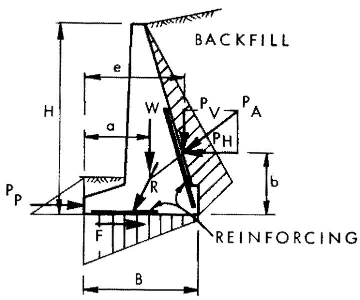
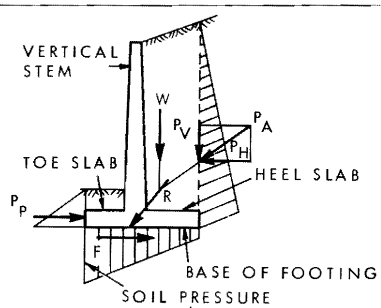
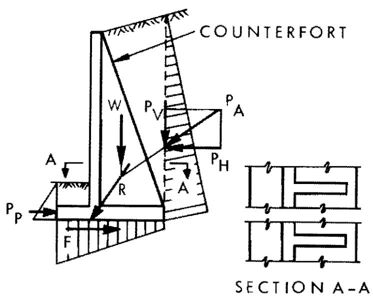
TYPE OF WALL	LOAD DIAGRAM	DESIGN FACTORS
GRAVITY		LOCATION OF RESULTANT Moments about toe: $d = \dfrac{Wa + P_V e - P_H b}{W + P_V}$ Assuming $P_P = 0$
SEMI-GRAVITY		RESISTANCE AGAINST SLIDING $FS_s = \dfrac{(W + P_V)\tan\delta}{P_H} \geqq 1.5$ $FS_s = \dfrac{(W + P_V)\tan\delta + P_P}{P_H} \geqq 2.0$ $F = (W + P_V)\tan\delta$
CANTILEVER		$\tan\delta$ = Friction factor between soil and base W = Includes weight of wall and soil in front for gravity and semi-gravity walls. Includes weight of wall and soil above footing, for cantilever and counterfort walls
COUNTERFORT		OVERALL STABILITY, CONTACT PRESSURE(S) For analysis of overall stability and contact pressures, see Chapter 5

FIG 5.24

DESIGN CRITERIA FOR RETAINING WALLS

5.4.2.2. *Stability Against Bearing Failure and Overturning*

Allowable bearing pressure at the base of the wall should be checked by the method defined in Chapter 6 of this Manual. If this method is used, no separate check on overturning is required.

5.4.2.3. *Settlement*

If the foundation soil is compressible, the settlement should be computed and the tilt of the rigid wall due to the settlement estimated. If the consequent tilt would exceed several degrees, the wall must be proportioned to keep the resultant force at the midpoint of the base.

5.4.3. *OVERALL STABILITY*

Where retaining walls are founded on deep layers or strata of cohesive soils, there is a possibility of failure occurring along a surface passing at some depth below the wall. The stability of the soil mass containing the retaining wall should be checked with respect to the most critical surface of sliding. A minimum factor of safety of 2.0 is desirable.

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APPENDIX 5A

THEORETICAL WALL PRESSURES

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APPENDIX 5A

THEORETICAL WALL PRESSURES

5.A.1. *COEFFICIENT OF LATERAL EARTH PRESSURE K*

The coefficient of lateral earth pressure K, at any point, is defined as the ratio of the horizontal effective stress σ_h', to the vertical effective stress, σ_v', at that point.

$$\text{i.e.} \quad K = \frac{\sigma_h'}{\sigma_v'} \quad \text{or} \quad \sigma_h' = K\sigma_v' \qquad \text{A.1}$$

5.A.2. *EARTH PRESSURE AT REST AND EFFECT OF LATERAL STRAIN*

The horizontal effective stress which exists in a natural soil in its undisturbed state is defined as the earth pressure *at rest*. For normally consolidated soils, the coefficient of earth pressure *at rest* K_o, is given approximately by the equation:

$$K_o = 1 - \sin \phi' \qquad \text{A.2}$$

For heavily overconsolidated soils, K_o can be of the order of 3 or higher.

Any lateral strain in the soil will alter its horizontal stress condition. Depending on the strain involved, the final horizontal stress can lie anywhere between two limiting (failure) conditions. The limiting stresses occur at the *active* and *passive* failure states.

Fig 5A.1 illustrates the dominant role of soil strain in determining the horizontal stress acting on the supporting structure. Much larger strains are necessary to achieve failure condition. Orders of magnitude of wall rotation Y/H, required to achieve failure conditions in various soil types are indicated in the following table;

Soil Type	Rotation Y/H*	
	Active	Passive
Dense Cohesionless	.0005	.002
Loose Cohesionless	.002	.006
Stiff Cohesive	.01	.02
Soft Cohesive	.02	.04

* Y = horizontal displacement
H = height of the wall

5.A.3. *ACTIVE EARTH PRESSURE*

The *active* earth pressure is the minimum value of lateral earth pressure which a soil mass can exert against a yielding retaining structure. It represents the failure condition at which the shear strength in the soil is fully mobilized in resisting gravity forces. The lateral strain (expansion) required to mobilize the soil strength is relatively small, but is nevertheless only possible in structures which are not rigidly restrained.

The ratio of lateral to vertical effective stress under *active* failure conditions K_A, can be obtained from the table in Fig 5A.2 for vertical retaining structures. Where the soil structure interface is not vertical, Fig 5A.3 can be used.

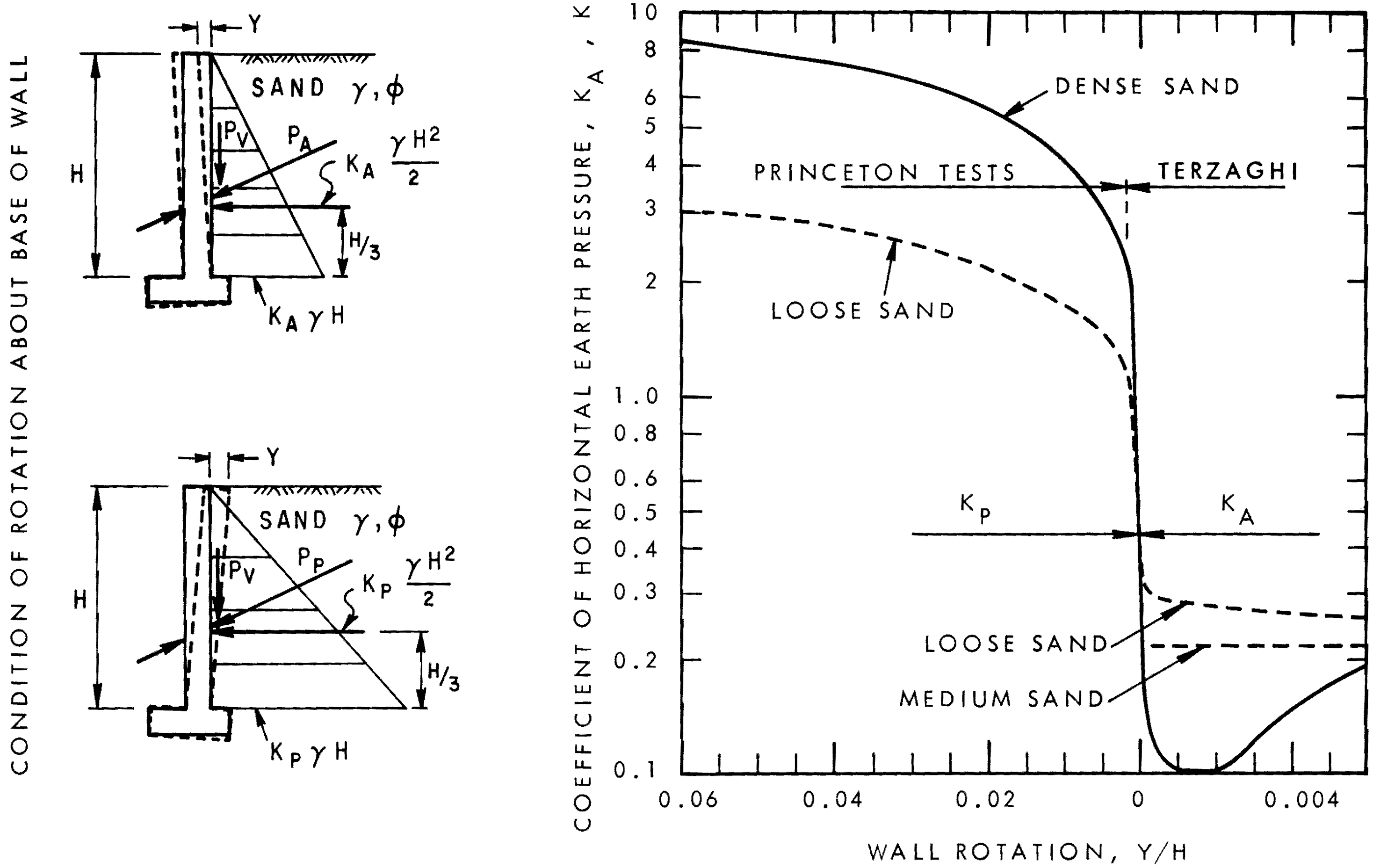
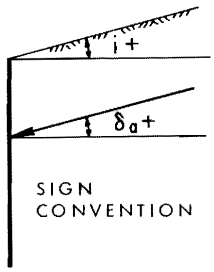


FIG 5A.1

EFFECT OF WALL MOVEMENT ON WALL PRESSURES



K_A, $\delta_a = 0$

i/ϕ \ ϕ	10°	15°	20°	25°	30°	35°	40°	45°
-1.0	0.639	0.513	0.413	0.333	0.267	0.213	0.168	0.131
-0.8	0.644	0.520	0.421	0.340	0.273	0.219	0.174	0.136
-0.6	0.656	0.534	0.434	0.353	0.287	0.230	0.183	0.144
-0.4	0.669	0.549	0.450	0.368	0.300	0.242	0.192	0.152
-0.2	0.685	0.567	0.468	0.385	0.315	0.255	0.205	0.161
0	0.704	0.589	0.490	0.406	0.333	0.271	0.217	0.172
+0.2	0.727	0.615	0.517	0.431	0.356	0.290	0.233	0.184
+0.4	0.756	0.648	0.551	0.464	0.386	0.316	0.254	0.201
+0.6	0.793	0.693	0.599	0.510	0.428	0.353	0.286	0.226
+0.8	0.850	0.763	0.674	0.586	0.500	0.419	0.342	0.272
+1.0	1.044	1.021	0.979	0.922	0.850	0.767	0.676	0.580

FIG 5A.2
ACTIVE EARTH PRESSURE COEFFICIENTS

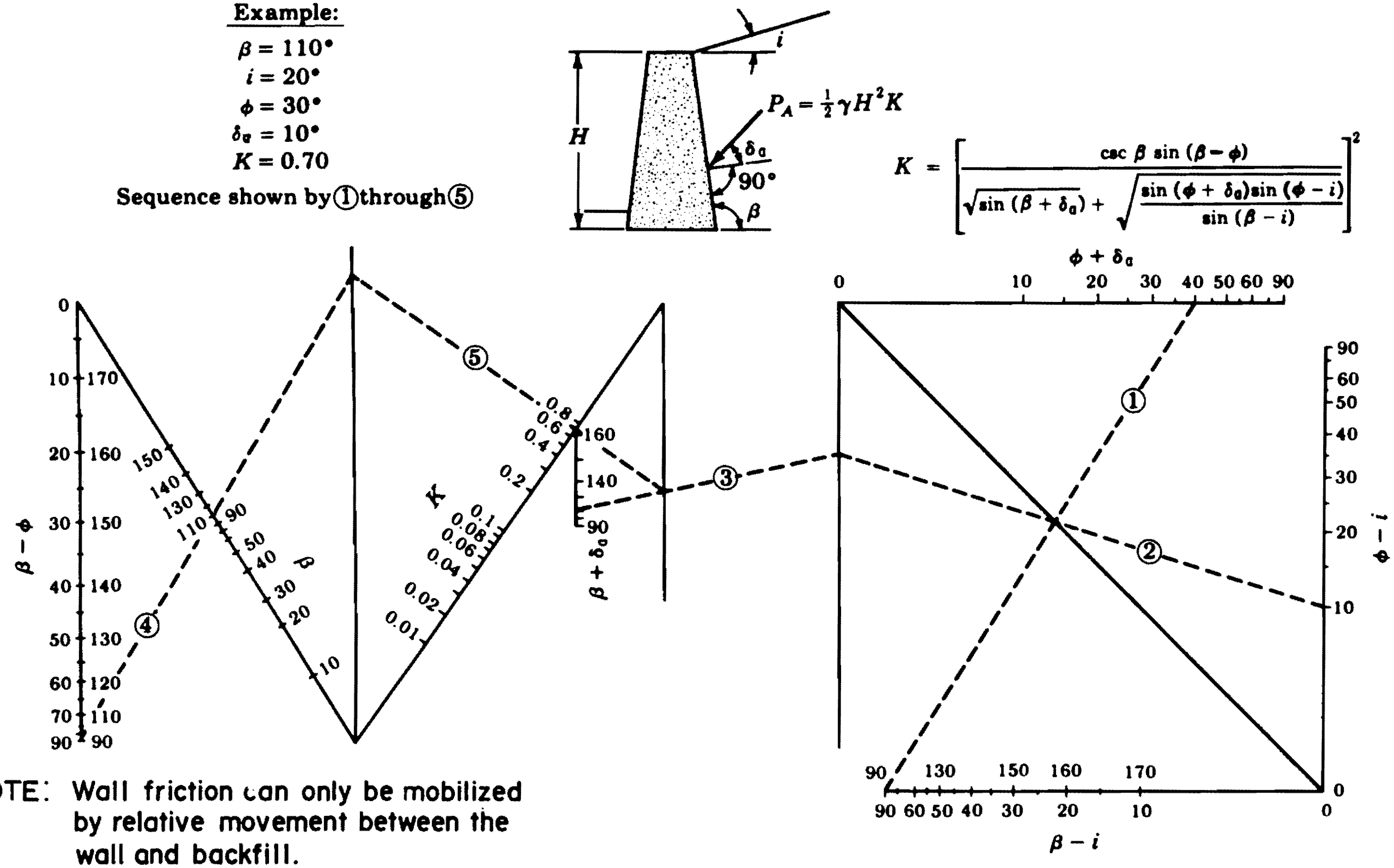
Example:

$\beta = 110°$
$i = 20°$
$\phi = 30°$
$\delta_a = 10°$
$K = 0.70$

Sequence shown by ① through ⑤

$$P_A = \tfrac{1}{2}\gamma H^2 K$$

$$K = \left[\frac{\csc\beta \sin(\beta-\phi)}{\sqrt{\sin(\beta+\delta_a)} + \sqrt{\dfrac{\sin(\phi+\delta_a)\sin(\phi-i)}{\sin(\beta-i)}}}\right]^2$$



NOTE: Wall friction can only be mobilized by relative movement between the wall and backfill.

FIG 5A.3

NOMOGRAM FOR ACTIVE EARTH PRESSURE COEFFICIENT K_A

Static Groundwater

For stratified soils with a horizontal, or no, groundwater table, K_A can be determined for each soil type from the figures using the effective angle of shearing resistance ϕ', and neglecting cohesion. In general, the lateral earth pressure inclined at δ to the normal, at any depth = $K_A\ \sigma'_z$, where σ'_z = effective stress at depth z. Net water pressure must be added to obtain the total pressure on wall. Formulae are given in Fig 5A.4.

Sloping Groundwater

For more complex situations such as sloping groundwater table or uneven backfill, refer to 'Soil mechanics, foundations, and earth structures.' *NAVFAC Design Manual. 7, U.S. Dept. Navy, Naval Facilities Eng. Comd. Wash. D.C. 1971.*

5.A.4. *PASSIVE EARTH PRESSURE*

The *passive* earth pressure is the maximum value of lateral earth pressure which can be mobilized by a structure moving against a soil mass. It represents the failure conditions at which the shear strength in the soil is fully mobilized in resisting the lateral forces. The lateral strain (compression) required to mobilize the soil strength can be quite large and the ability of the wall to move the required distance should be checked (see 5.A.2). If movement is restricted, lower pressure can be expected.

The ratio of lateral to vertical effective stress under *passive* failure conditions, K_P, can be obtained from the table in Fig 5A.5 for the case of a vertical soil-structure interface. For other conditions, refer to 'Soil mechanics, foundations, and earth structures." *NAVFAC Design Manual. 7 U.S. Dept. Navy, Naval Facilities Eng. Comd. Wash. D.C. 1971.*

Static Groundwater

For stratified soils with a horizontal, or no groundwater table, K_P can be determined for each soil type from the figures using the effective angle of shearing resistance, ϕ', and neglecting cohesion. In general, the lateral earth pressure, inclined at δ to the normal, at any depth = $K_P\sigma'_z$, where σ'_z = effective stress at depth z. Net water pressure must be added to obtain the total pressure on wall. Formulae are given in Fig 5A.6.

5.A.5. *WALL FRICTION*

Unless a wall is settling, friction on its back acts upward on the *active* wedge (angle δ is positive), reducing *active* pressures. Wall friction acts downward against the *passive* wedge (angle δ is negative), resisting its upward movement and increasing *passive* pressures.

> In general, the effect of wall friction on *active* pressures is small and ordinarily is disregarded.
>
> The effect of wall friction on *passive* pressure is large, but definite movement is necessary for mobilization of wall friction.

In the absence of specific test data, the angle of wall friction δ, where applicable, should be estimated to be in the range of 1/2 to 2/3 of ϕ'.

5.A.6. *EFFECT OF SEEPAGE AND DRAINAGE*

Horizontal Groundwater

The effect of the greatest unbalanced water head that will act across the wall must be included in the pressure computations. For the effect of flow on K_A, K_P, see Fig 5A.7(a).

For the effect on wall pressures, see Fig 5A.7(b). For static differential head with insignificant seepage, water pressures on walls should be computed using the formula in Fig 5A.7(b).

In cohesionless soils, the *active* force on the wall with static water level at the top of the backfill is frequently more than double that for dry backfill.

5.A.7. *SURCHARGE LOADING*

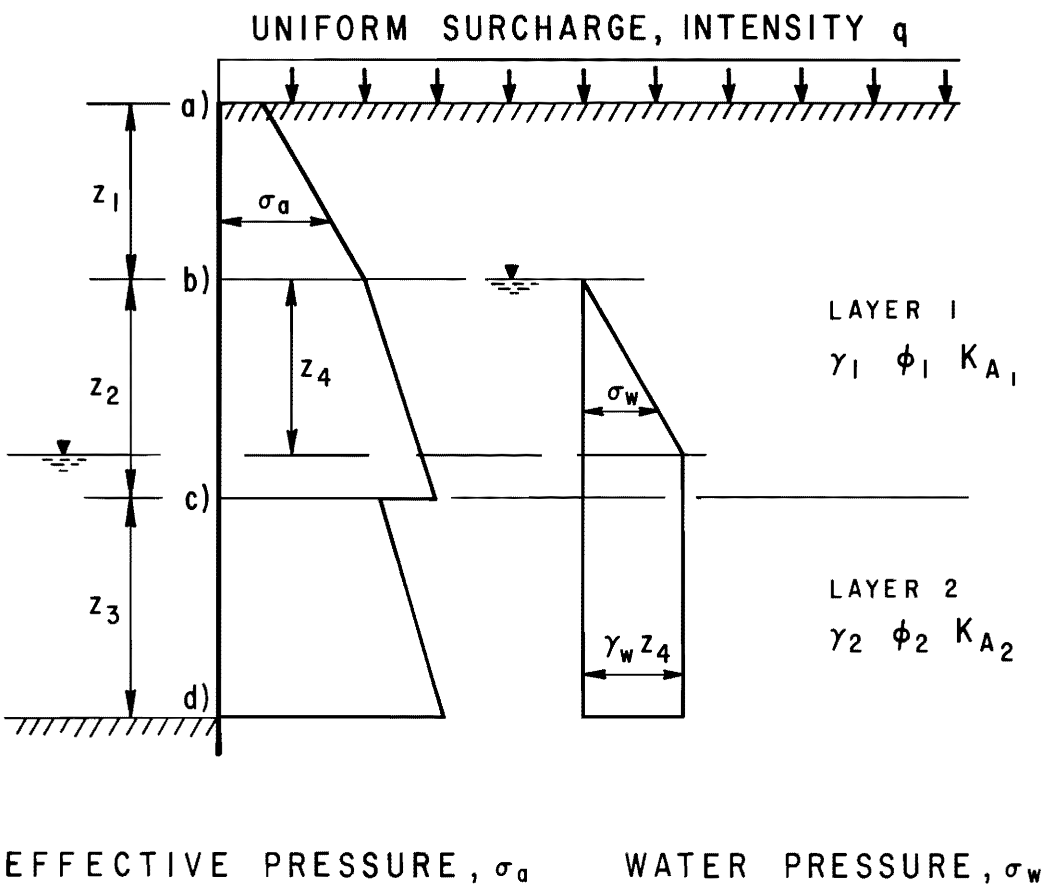
Area Loads

Where the surcharge behind a wall consists of a uniform area load, the weight of the surcharge as illustrated in Figs 5A.4 and 5A.6 must be included in the design analysis.

Point or Line Loads

Where the surcharge behind a wall consists of a point or line load whose intensity is small compared to total backfill forces, (total force on wall from surcharge is less than 30% of the active force), the additional wall pressures may calculated using the formulae in Fig 5A.8.

For heavy surcharges a wedge analysis should be used. (See "Soil mechanics, foundations, and earth structures." *NAVFAC Design Manual. 7 U.S. Dept. Navy, Naval Facilities Eng. Comd. Wash. D.C. 1971.)*



IN GENERAL

$$\sigma_a = K_A \sigma'_z$$

$$\sigma = \sigma_a + \sigma_w$$

$$\sigma_w = \gamma_w \times \text{NET WATER HEAD}$$

AT a) $\sigma_a = q K_{A_1}$

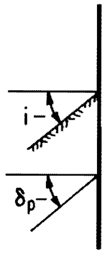
b) $\sigma_a = (q + z_1 \gamma_1) K_{A_1}$

c) $\sigma_a = (q + z_1 \gamma_1 + z_2 \gamma'_1) K_{A_1}$ IN LAYER 1

$\sigma_a = (q + z_1 \gamma_1 + z_2 \gamma'_1) K_{A_2}$ IN LAYER 2

d) $\sigma_a = (q + z_1 \gamma_1 + z_2 \gamma'_1 + z_3 \gamma'_2) K_{A_2}$

FIG 5A.4
CALCULATION OF ACTIVE PRESSURES



SIGN
CONVENTION

K_p: $\delta_p = 0$

i/ϕ \ ϕ	10°	15°	20°	25°	30°	35°	40°	45°
0	1.42	1.70	2.04	2.46	3.00	3.69	4.58	5.83
- 0.1	1.38	1.63	1.92	2.27	2.67	3.10	3.77	4.54
- 0.2	1.36	1.56	1.81	2.08	2.38	2.71	3.09	3.50
- 0.3	1.32	1.49	1.69	1.89	2.10	2.30	2.50	2.68
- 0.4	1.28	1.42	1.57	1.70	1.82	1.92	1.99	2.02
- 0.5	1.23	1.34	1.44	1.51	1.56	1.58	1.56	1.48
- 0.6	1.18	1.25	1.30	1.32	1.32	1.28	1.19	1.06
- 0.7	1.12	1.15	1.17	1.14	1.09	1.00	0.880	0.731
- 0.8	1.07	1.06	1.03	0.97	0.882	0.764	0.626	0.480
- 0.9	1.00	0.961	0.895	0.803	0.686	0.557	0.422	0.292
- 1.0	0.848	0.749	0.637	0.520	0.404	0.296	0.200	0.123

K_p: $\delta_p = -\frac{1}{2}\phi$

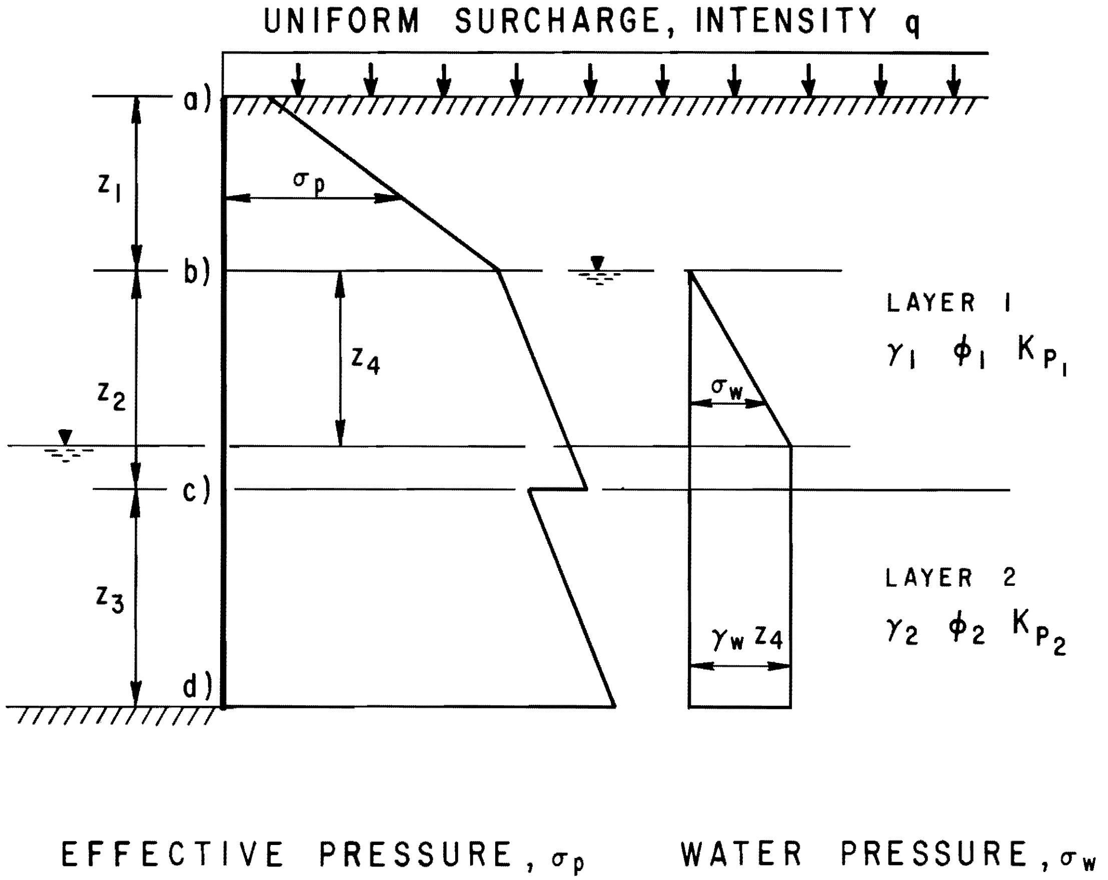
i/ϕ \ ϕ	10°	15°	20°	25°	30°	35°	40°	45°
0	1.54	1.97	2.55	3.38	4.62	6.55	9.73	15.48
- 0.1	1.51	1.90	2.40	3.12	4.12	5.63	8.00	12.06
- 0.2	1.48	1.81	2.26	2.06	3.66	4.81	6.56	9.52
- 0.3	1.44	1.73	2.11	2.59	3.23	4.09	5.30	7.11
- 0.4	1.39	1.64	1.96	2.33	2.80	3.41	4.23	5.36
- 0.5	1.35	1.55	1.80	2.08	2.41	2.81	3.31	3.94
- 0.6	1.29	1.45	1.63	1.82	2.03	2.27	2.52	2.82
- 0.7	1.22	1.34	1.46	1.57	1.67	1.78	1.87	1.94
- 0.8	1.17	1.23	1.29	1.33	1.36	1.35	1.32	1.27
- 0.9	1.09	1.11	1.12	1.10	1.06	0.988	0.895	0.776
- 1.0	0.925	0.868	0.797	0.714	0.623	0.525	0.425	0.327
$\cos\delta_p$	0.996	0.991	0.984	0.976	0.965	0.953	0.939	0.923

K_p: $\delta_p = -\frac{3}{4}\phi$

i/ϕ \ ϕ	10°	15°	20°	25°	30°	35°	40°	45°
0	1.59	2.08	2.76	3.78	5.31	7.97	12.63	22.11
- 0.1	1.55	2.00	2.61	3.48	4.79	6.86	10.39	17.22
- 0.2	1.52	1.91	2.45	3.19	4.26	5.86	8.51	13.26
- 0.3	1.49	1.83	2.29	2.90	3.75	4.98	6.89	10.16
- 0.4	1.44	1.73	2.12	2.61	3.26	4.16	5.49	7.65
- 0.5	1.39	1.64	1.95	2.33	2.80	3.42	4.30	5.62
- 0.6	1.33	1.52	1.76	2.03	2.36	2.76	3.27	4.02
- 0.7	1.26	1.41	1.58	1.75	1.95	2.16	2.42	2.77
- 0.8	1.20	1.30	1.39	1.49	1.58	1.65	1.72	1.82
- 0.9	1.12	1.17	1.21	1.23	1.23	1.20	1.16	1.10
- 1.0	0.956	0.917	0.864	0.799	0.724	0.64	0.553	0.466
$\cos\delta_p$	0.991	0.980	0.965	0.947	0.923	0.896	0.866	0.843

FIG 5A.5

PASSIVE EARTH PRESSURE COEFFICIENTS



IN GENERAL

$$\sigma_p = K_p \sigma'_z \qquad \sigma = \sigma_p + \sigma_w$$

$\sigma_w = \gamma_w$ X NET WATER HEAD

AT a) $\sigma_p = q K_{P_1}$

b) $\sigma_p = (q + z_1 \gamma_1) K_{P_1}$

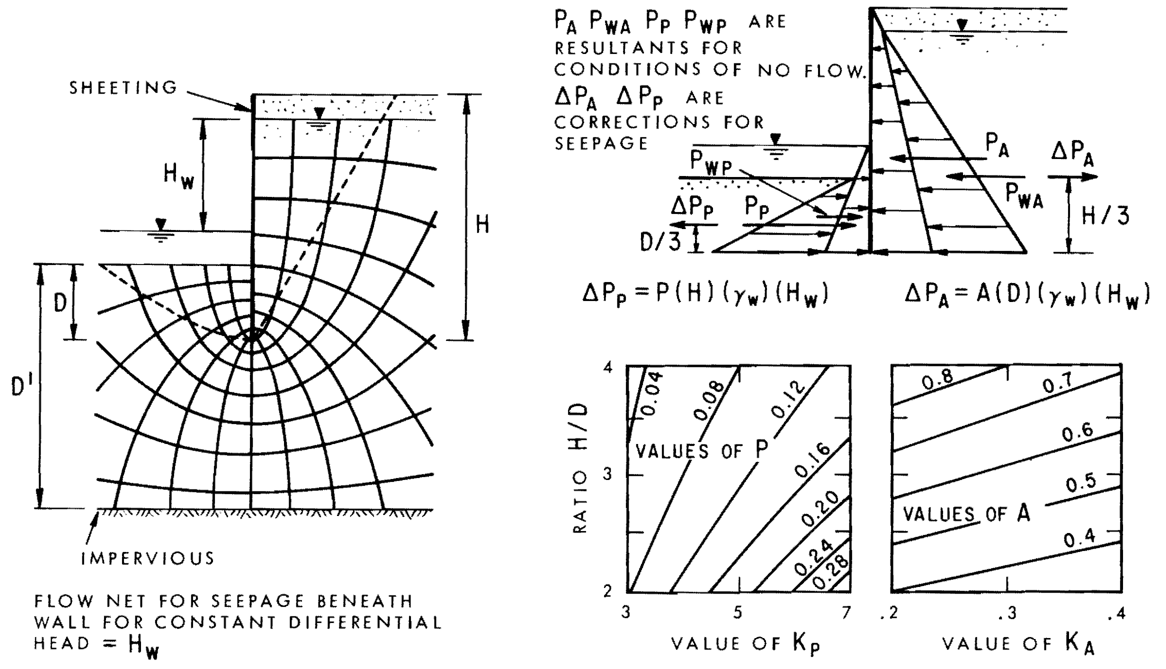
c) $\sigma_p = (q + z_1 \gamma_1 + z_2 \gamma'_1) K_{P_1}$ IN LAYER 1

$\sigma_p = (q + z_1 \gamma_1 + z_2 \gamma'_1) K_{P_2}$ IN LAYER 2

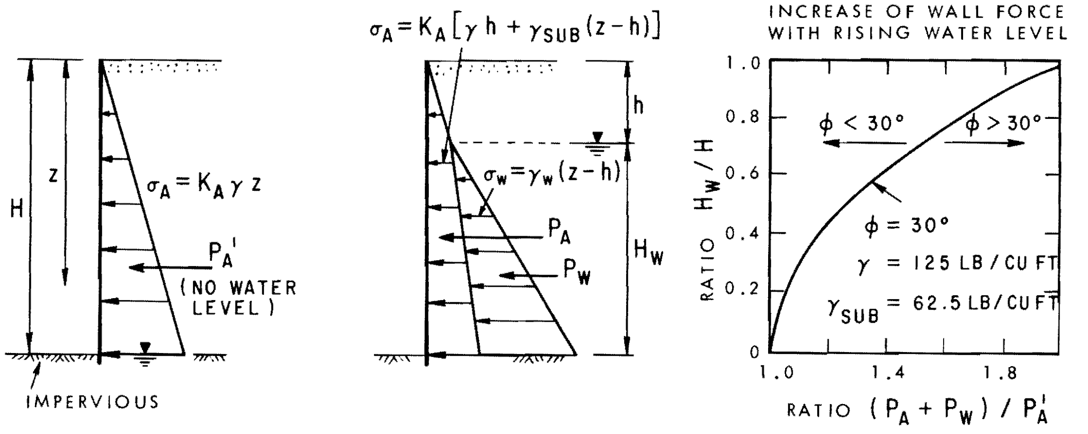
d) $\sigma_p = (q + z_1 \gamma_1 + z_2 \gamma'_1 + z_3 \gamma'_2) K_{P_2}$

FIG 5A.6

CALCULATION OF PASSIVE PRESSURES



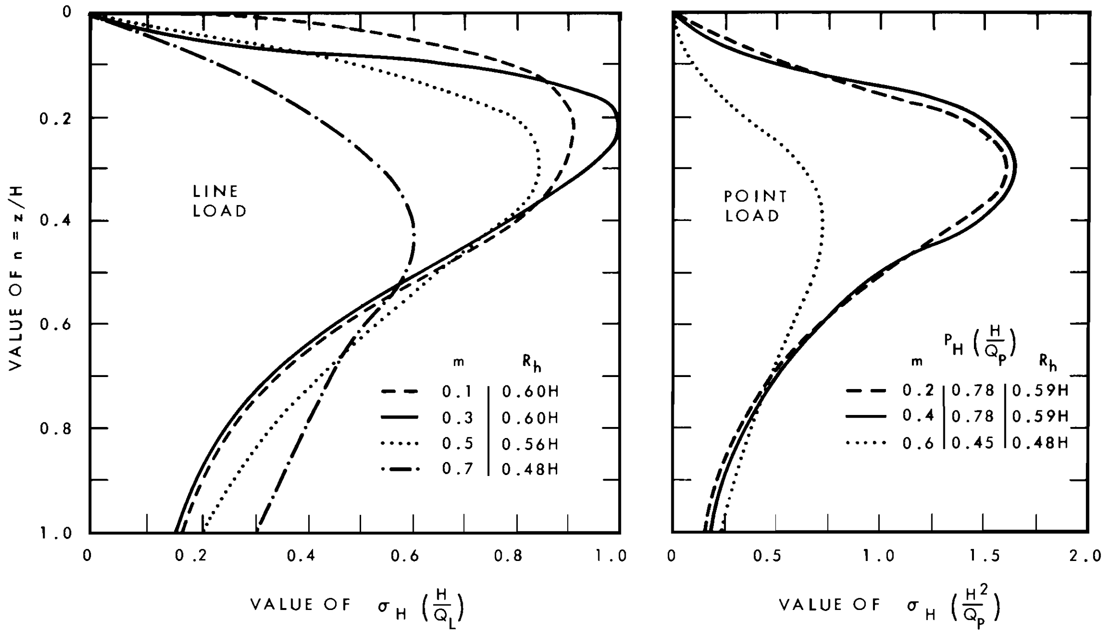
(a) EFFECT OF SEEPAGE BENEATH WALL

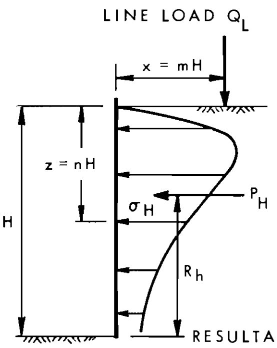


(b) EFFECT OF STATIC WATER LEVEL

FIG 5A.7

EFFECTS OF SEEPAGE AND STATIC GROUNDWATER LEVEL





FOR m ≦ 0.4:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

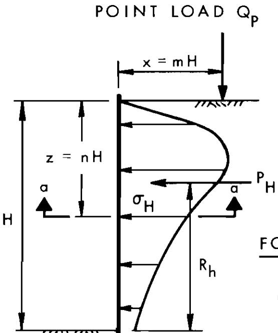
$$P_H = 0.55\ Q_L$$

FOR m > 0.4:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{1.28\ m^2 n}{(m^2 + n^2)^2}$$

$$\text{RESULTANT } P_H = \frac{0.64\ Q_L}{(m^2 + 1)}$$

PRESSURES FROM LINE LOAD Q_L
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



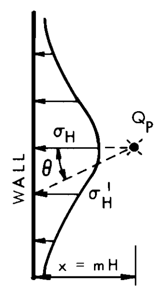
FOR m ≦ 0.4:

$$\sigma_H\left(\frac{H^2}{Q_P}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4:

$$\sigma_H\left(\frac{H^2}{Q_P}\right) = \frac{1.77\ m^2 n^2}{(m^2 + n^2)^3}$$

$$\sigma'_H = \sigma_H \cos^2(1.1\ \theta)$$



SECTION a - a
PRESSURES FROM POINT LOAD Q_P
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

FIG 5A.8
HORIZONTAL PRESSURES ON WALL DUE TO SURCHARGE

CHAPTER 6

SHALLOW FOUNDATIONS

TABLE OF CONTENTS

CHAPTER 6

SHALLOW FOUNDATIONS

6.1 GENERAL

A shallow foundation generally derives its support from the soil or rock close to the lowest part of the building which it supports. The depth of the bearing area below the adjacent ground is usually about equal to or less than the width of the bearing area, and vertical loads on the sides of the foundation due to adhesion or friction may normally be neglected.

Shallow foundations include such common footing types as slabs, rafts, spread footings, strip footings, pads, mats and sills.

6.1.1. *VALIDITY OF THE METHOD OF DESIGN FOR SHALLOW FOUNDATIONS*

6.1.1.1. *Bearing Capacity and Settlement*

The design of a foundation unit normally requires that both bearing capacity and settlement be checked. While either bearing capacity or settlement criteria may provide the limiting condition, it is normal for settlement to govern. Structural distress from settlement as evidenced by such occurrences as cracking, and distortion of doors and window frames, is common experience. The drastic effects of a bearing capacity failure are rare except perhaps during construction where shallow temporary footings are frequently used with falsework.

(1) *Bearing capacity*

The bearing capacity of both *cohesive* and *non-cohesive* soils can be determined with reliability by relatively simple calculations assuming that the strength parameters for the bearing soil are accurately known within the depth of influence of the footing.

(2) *Settlement*

(a) *Cohesive soil*

The settlement of a structure on *cohesive* soil can be calculated with less accuracy than the bearing capacity. Such a calculation is affected by a number of complicating factors usually requiring judgement to assess. The most important of these is an estimate of the preconsolidation pressure; that is, the maximum past pressure on the *in situ* soil. Because of the various uncertainties, errors of a factor of two should be expected in the calculation of settlement.

(b) *Non-cohesive soil*

The settlement of a structure on *non-cohesive* soil normally can only be estimated by empirical methods. Such an estimate usually is taken to mean the settlement directly related to the load, but this settlement occurs rapidly and frequently during the construction period. Post-construction settlement in such a case will be negligible and may be considerably less than the predicted settlement.

Post-construction settlement may occur at a considerable period after construction and after a period of successful performance of the structure as the result of vibrations or changes in the groundwater conditions, whether natural or man-made; for example, earthquake or blasting, flooding or groundwater lowering. Settlement of this nature is not usually included in an empirical estimation, but may be assessed and allowed for.

6.1.1.2. *Stress Distribution*

The stress distribution beneath a structure can be assessed using conventionally acceptable procedures based on the Boussinesq or similar equations. Various charts, graphs and tables of influence values are available to aid in such calculations. It is easily possible for the stress analysis to be carried out in detail not warranted by the potential accuracy of a succeeding settlement analysis.

Using such a stress analysis, it can be seen that the loaded area beneath a large footing is greater than beneath a small one and it follows that the settlement under the larger footing will also be greater for the same intensity of loading. The concept is illustrated in Fig 6.1.

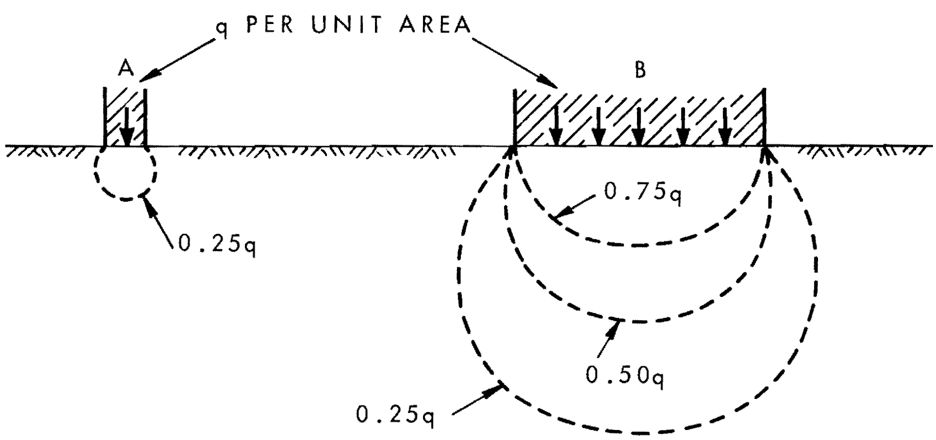


FIG 6.1

EFFECT OF SIZE OF FOOTING ON STRESS DISTRIBUTION

(From 'Soil Mechanics in Engineering Practice' by TERZAGHI & PECK, 1948. Used with permission of J. Wiley & Sons, Inc.).

6.1.1.3. *Foundation Flexibility*

Shallow foundations may be flexible or rigid. Design methods of calculating stresses and resulting settlement normally are based on an assumption of complete flexibility of the foundation. Such a case, however, seldom occurs. Normally, foundations are not flexible and the stress distribution will be different from the assumptions. It follows that the settlement will also be different from that calculated. Corrections can be made for this effect and calculations of total settlement adjusted. Such a procedure is, however, not justified for routine problems.*

6.1.1.4. *Construction*

The calculation of bearing capacity, the distribution of stresses, and the prediction of settlement may be labour in vain and the choice of allowable load may be grossly in error if the construction techniques are not considered or if they do not conform to good practice. It is necessary to consider such factors as the following which may alter the conditions assumed in design beyond recognition:

---

*This problem is avoided in the *Critical Point Method* of design; See 6.5.6.

(1) *Occurrence during excavation*

- bottom heave,
- wetting, swelling and softening of an expansive clay or rock,
- piping in sands and silts, and
- remoulding of silts and sensitive clays.

(2) *Adjacent construction activities*

- groundwater lowering,
- excavation, and
- blasting,

(3) *Other effects during or following construction*

- scour and erosion,
- frost action, and
- flooding.

6.1.2. *ESTIMATES OF ALLOWABLE BEARING PRESSURE*

Universally applicable values of allowable bearing pressure cannot be given. Many factors affect bearing capacity, as discussed in 6.2 BEARING PRESSURES ON ROCK and the allowable load will frequently be controlled by settlement criteria, as described in 6.5 SETTLEMENT. Nevertheless, it is often useful to estimate the allowable bearing pressure for preliminary design on the basis of the material description, although such values must be checked or treated with great caution for final design.

6.2 BEARING PRESSURE ON ROCK

6.2.1. *GENERAL*

Rock is usually recognized as the best foundation material. However, design engineers should be aware of the dangers associated with unfavourable rock conditions since overstressing a rock foundation may result in large settlement or sudden failure. A foundation on rock should be designed with at least the same care as a foundation on any type of soil.

The methods proposed in this Manual for the determination of the allowable bearing pressure on rock apply for various ranges of rock quality. Guidance on the applicability of the proposed methods is outlined in Table 6.1.

TABLE 6.1.

Applicability of Methods for the Determination of Allowable Bearing Pressure on Rock

Basis of Design Method	Rock Quality
Rock Description (See Table 6.2 for preliminary estimate)	Sound rock and broken rock with *wide* or *very wide spacing* of discontinuities.
Core Strength (See 6.2.2.)	Rock mass with closed discontinuities at *moderately close, wide* and *very wide spacing*
Pressuremeter (See 6.2.3.1. and Commentary 8.8)	Rock of *low* to *very low strength*: rock mass with discontinuities at *close* or *very close spacing*; fragmented or weathered rock.
Soil Mechanics Approach	Rock of *very low strength*: rock mass with discontinuities at *very close spacing*; fragmented or weathered rock.

(Terms in italics are defined in 3.2.4.)

6.2.2. *FOUNDATION ON SOUND ROCK*

For the purpose of this section, a rock is considered as *sound* where the unconfined compression strength is in excess of 125 lb/sq in and the spacing of discontinuities is in excess of 3 ft. This includes rock of *very low strength.*

Where the rock is *sound*, the strength of the rock foundation is generally much in excess of the design requirements, provided the discontinuities are closed and are favourably oriented with respect to the applied forces. The investigation should, therefore, be concentrated on:

- The identification and mapping of all discontinuities in the rock mass within the zone of influence of the foundation including the determination of the thickness of discontinuities.
- An evaluation of the mechanical properties of these discontinuities: frictional resistance, compressibility and strength of infilling material; and
- The identification and evaluation of the strength of the rock material.

Such investigations should be carried out by a person competent in this field of work.

The final determination of the bearing pressure on rock results from the analysis of the influence of the discontinuities on the behaviour of the foundation. As a guideline, in the case of a rock mass with favourable characteristics (i.e., the rock surface is perpendicular to the foundation, the load has no tangential component, the rock mass has no open discontinuities), the allowable bearing pressure may be estimated from:

$$q_a = K_{sp}\, q_{u\text{-}core}$$

where q_a = allowable bearing pressure,

$q_{u\text{-}core}$ = average unconfined compressive strength of rock cores, as determined from ASTM D2938-71, and

K_{sp} = empirical coefficient depending on the spacing of discontinuities and including a factor of safety of 3 as follows:

Spacing of Discontinuities	K_{sp}
Very wide	0.4
Wide	0.25
Moderately close	0.1

(Terms in italics are defined in 3.2.4.)

Note: The factors influencing the magnitude of coefficient K_{sp} are shown graphically in Fig 6.2 to provide additional understanding of the effects of discontinuities. The relationship given in Fig 6.2 is valid for a rock mass with spacing of discontinuities greater than one foot, thickness of discontinuities less than ¼ inch (or less than one inch if filled with soil or rock debris) and for a foundation width greater than one foot. For sedimentary or foliated rocks, the strata must be level or nearly so.

6.2.3. *FOUNDATION ON POOR ROCK*

Conditions are frequently encountered where the rock is of *very low strength*, has discontinuities at a *very close spacing*, or is weathered or fragmented. It is common practice in such cases to consider the rock as a granular mass and to design the foundation on the basis of conventional soil mechanics. However, the strength parameters necessary for such a design are difficult to evaluate.

6.2.3.1. *Pressuremeter*

The pressuremeter allows for a direct determination of the strength of a rock mass, including the effect of discontinuities and weathering for the design of foundations on poor rock. The allowable bearing pressure may be calculated with a factor of safety of 3 against failure using the following relationship:

$$q_a = \frac{1}{3}\left\{\gamma D_f + K_d\ (p_L - \gamma D_f)\right\}$$

where q_a = allowable bearing pressure, ton/sq ft,

p_L = limit pressure determined by pressuremeter, ton/sq ft,

γ = unit weight of soil or rock, ton/cu ft,

D_f = depth of footing, ft, and

K_d = empirical coefficient as follows:

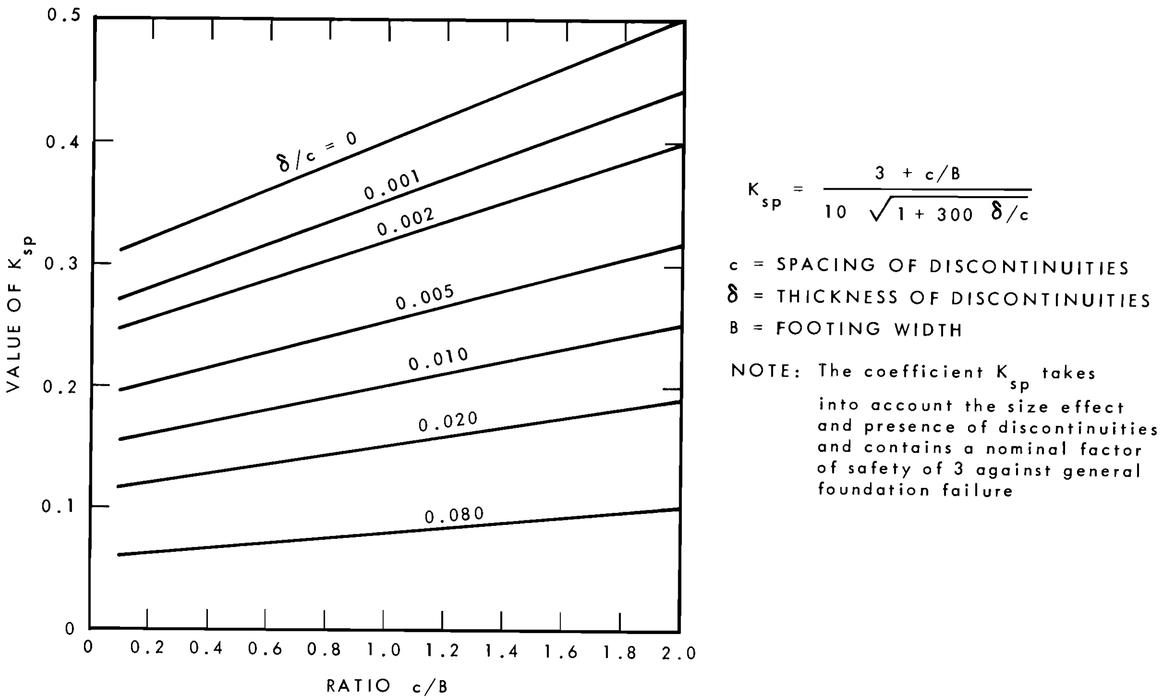


FIG 6.2

BEARING PRESSURE COEFFICIENT K_{sp}

Depth of Footing	K_d
Load at rock surface	0.8
Radius of foundation unit	2.0
4 x radius of foundation unit	3.6
10 x radius of foundation unit	5

6.2.3.2. *Limitations*

The pressuremeter test is an *in situ* test carried out with specialized equipment. Its use and the interpretation of the results should be restricted to geotechnical specialists.

REFERENCES

MENARD, L., 1965. Règles pour le calcul de la force portante et du tassement des fondations en fonction des résultats pressiométriques. *Proc. Inter. Conf. Soil Mech. Found. Eng., 6th, Montreal, 2: 295-299.*

MENARD, L., 1972. Rules for the calculation of bearing capacity and foundation settlement based on pressuremeter results. *Draft Translation 159, U.S. Army Corps of Engrs. Cold Regions Research and Engineering Laboratory.*

6.3. BEARING PRESSURE ON SOIL

6.3.1. *GENERAL*

The allowable bearing pressure may be estimated from Table 6.2 on the basis of a description of the material type. In the following paragraphs, a variety of methods are presented for arriving at the allowable bearing pressure based on some form of field or laboratory test procedure. Generally, settlement must be considered separately from the allowable bearing pressure.

6.3.2. *STANDARD PENETRATION TEST IN NON-COHESIVE SOILS*

The allowable bearing pressure can be roughly estimated in sands from the results of the Standard Penetration Test (SPT) by using the relationships between N, the number of blows per foot, and the bearing pressure as shown on Fig 6.3.*

By entering Fig 6.3(a) with the width of footing B and the value of N, the allowable soil pressure for a footing surrounded by no surcharge can be obtained. If a surcharge exists, Fig 6.3(b) indicates the additional allowable soil pressure due to the surcharge.

The diagrams are applicable without modification if the groundwater level is at a depth of B or more below the base of the footing. If the groundwater is or is likely to be at the base of the footing, the safe soil pressure obtained from Fig 6.3(a) should be divided by two. If the groundwater is at the top of the surcharge surrounding the footings, the increment of allowable soil pressure due to the surcharge, as given in Fig 6.3(b), should also be divided by two.

* The charts of PECK, HANSON & THORNBURN have been altered in the 1974 Edition which includes a useful discussion of the bearing capacity of *non-cohesive* soil.

TABLE 6.2

ESTIMATES OF ALLOWABLE BEARING PRESSURE

These presumed values of the allowable bearing pressure are estimates and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation. Reference should be made to other parts of the Manual when using this table.

Group	Types and conditions of rocks and soils	Strength of Rock Material	Presumed Allowable Bearing Pressure Ton /sq ft	Remarks
Rocks	Massive igneous and metamorphic rocks (granite, diorite, basalt, gneiss) in sound condition (2)	High to very high	100	These values are based on the assumption that the foundations are carried down to unweathered rock.
	Foliated metamorphic rocks (slate, schist) in sound condition (1) (2)	Medium to high	30	
	Sedimentary rocks: cemented shale, siltstone, sandstone, limestone without cavities, thoroughly cemented conglomerates, all in sound condition (1) (2)	Medium to high	10-40	
	Compaction shale and other argillaceous rocks in sound condition (2) (4)	Low to medium	5	
	Broken rocks of any kind with moderately close spacing of discontinuities (1 ft or greater), except argillaceous rocks (shale)		10	
	Thinly bedded limestone, sandstones, shale		See note (3)	
	Heavily shattered or weathered rocks		See note (3)	
Non-cohesive soils	Dense gravel or dense sand and gravel		>6	Width of foundation (B) not less than 3 ft. Groundwater level assumed to be at a depth not less than B below the base of the foundation.
	Compact gravel or compact sand and gravel		2-6	
	Loose gravel or lôose sand and gravel		<2	
	Dense sand		>3	
	Compact sand		1-3	
	Loose sand		<1	
Cohesive soils	Very stiff to hard clays or heterogeneous mixtures such as till		3-6	Cohesive soils are susceptible to long-term consolidation settlement
	Stiff clays		1.5-3	
	Firm clays		0.75-1.5	
	Soft clays and silts		<0.75	
	Very soft clays and silts		not applicable	
Organic soils	Peat and organic soils		not applicable	
Fill	Fill		not applicable	

NOTES:

(1) The above values for sedimentary or foliated rocks apply where the strata or foliation are level or nearly so, and, then only if the area has ample lateral support. Tilted strata and their relation to nearby slopes or excavations shall be assessed by a person knowledgeable in this field of work.

(2) Sound rock conditions allow minor cracks at spacing not less than 3 feet.

(3) To be assessed by examination *in situ*, including loading tests if necessary, by a person knowledgeable in this field of work.

(4) These rocks are apt to swell on release of stress, and on exposure to water they are apt to soften and swell appreciably.

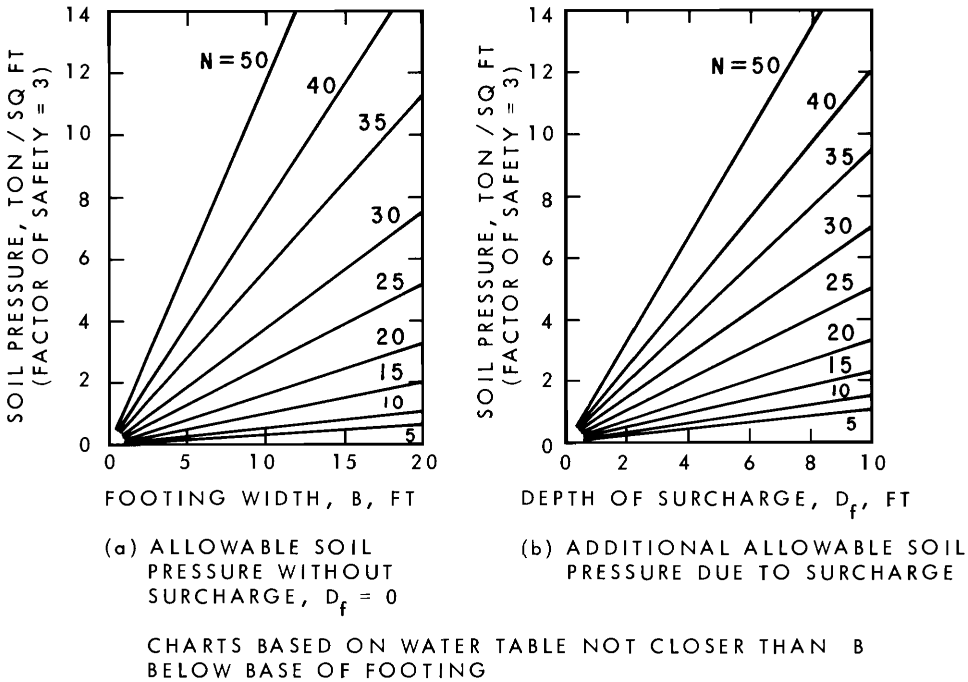


FIG 6.3

ALLOWABLE SOIL PRESSURES BENEATH FOOTINGS NON-COHESIVE SOILS AS DETERMINED BY BEARING CAPACITY

(From 'Foundation Engineering' by PECK, HANSON and THORNBURN, 1953. Used with permission of J. Wiley & Sons, Inc.).

6.3.2.1. *Limitations*

The Standard Penetration Test is subject to many errors in practice, including the skill of the operator. The material at the bottom of a borehole is frequently disturbed by the drilling process or by the upward flow of water, leading to an underestimation of N. The drop distance of the 140 lb hammer is frequently too small and the friction of the rope on the shieves and winch drum is frequently too high, leading to an overestimation of N. (See Commentary 8.1 on the Standard Penetration Test, Chapter 8.) The relationship between N and the angle of shearing resistance ϕ, is remote and the calculation of bearing capacity from N values is therefore highly suspect. It remains a fact that bearing capacity is frequently calculated from the Standard Penetration Test, particularly in granular soils and in many cases this is the only approach readily available. Such estimates should be treated with caution.

REFERENCES

CSA A119.1-1960, Code for split-barrel sampling of soils.

MEYERHOF, G.G., 1956. Penetration tests and bearing capacity of cohesionless soils. *Proc. Am. Soc. Civil Engrs., 82: SM1, Paper 866, 19 p.*

PECK, R.B., HANSON, W.E. and THORNBURN, T.H., 1973. Foundation Engineering. *J. Wiley & Sons, N.Y.*

6.3.3. *DYNAMIC CONE TEST IN NON-COHESIVE SOILS*

Where the soil classification is known, the allowable bearing capacity may be estimated from the results of a dynamic cone test. This test is frequently used to provide subsurface data between conventional boreholes where standard penetration test data is available.

A standard dynamic cone test uses a 2-1/4 in. diam., 60 degree cone driven into the ground by blows of a 140 lb hammer with a 30 in. drop. The blow count for every foot of penetration is recorded.

6.3.3.1. *Application*

The dynamic cone penetration resistance, N_{cone}, may be related to the Standard Penetration Test.

$$N_{cone} = 1.5\ N$$

The allowable bearing pressure is then estimated as described in 6.3.2.

Alternatively, the allowable bearing pressure for shallow foundations (D/B < 4) can be estimated using the Dutch formula. The allowable soil pressure is estimated by dividing the dynamic cone resistance by a factor of 20.

$$q_a = \frac{R_d}{20} \quad \text{where} \quad R_d = \frac{M^2\ H}{A\ e\ (M + P)}$$

where R_d = unit resistance, lb/sq ft

e = penetration per blow, ft

M = mass of hammer, lb

H = height of fall of hammer, ft

P = mass of pipe, lb, and

A = cross-sectional area of cone, sq ft

6.3.3.2. *Limitations*

The dynamic cone penetration test is subject to most of the limitations of the Standard Penetration Test, although it avoids the errors imposed by the process of making a borehole. Without a borehole, friction on the drilling rods must be accounted for and use of a sleeved cone is recommended.

REFERENCES

GADSBY, J.W., 1971. Discussion of "The correlation of cone size in the dynamic cone penetration test with the standard penetration test", *Geotech. 21, 2: 188-189.*

MOHAN, D., AGGARWAL, U.S. and TOLIA, D.S., 1970. The correlation of cone size in the dynamic cone penetration test with the standard penetration test; *Geotech. 20: 315-319.*

SANGLERAT, G., 1972. The penetrometer and soil exploration; *Elsevier Publ. Co., Amsterdam.*

6.3.4. *STATIC CONE PENETRATION TEST IN NON-COHESIVE SOIL*

Where the soil classification is known, the allowable bearing pressure may be estimated from the results of a static cone penetration test. A standard cone is considered to be 10 cm^2 in cross-section or 1.4 in. in diameter, with an apex angle of 60 degrees.

For shallow footings of commonly used dimensions with an embedment of about 3 ft, the allowable bearing pressure may be estimated from the approximate relationship:

$$q_a = \frac{q_{cone}}{10}$$

where q_a = allowable bearing pressure, and

q_{cone} = cone resistance

This formula should be used with caution for simple cases only. For other cases, the relationship developed by MEYERHOF (1956) may be used (See Fig 6.4).

6.3.4.1. *Limitations*

The static cone penetration test is free of many of the objections to the Standard Penetration Test described in 6.3.2. However, the static cone was developed for use in deep deposits of *loose*, uniform, fine-grained soil. The equipment normally used is effective in such soils, but may give trouble in *dense* or mixed grain deposits.

REFERENCES

MEYERHOF, G.G., 1956. Penetration tests and bearing capacity of cohesionless soils. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs. 82: SM1, Paper 866, 19 p.*

SANGLERAT, G., 1972. The penetrometer and soil exploration. *Elsevier Publ. Co., Amsterdam.*

6.3.5. *PRESSUREMETER TEST*

The allowable bearing pressure may be derived from the results of *in situ* pressuremeter tests by the relationship:

$$q_a = \frac{1}{3}\left[\gamma D_f + K_g\ (p_L - \gamma D_f)\right]$$

where q_a = allowable bearing pressure, ton/sq ft

p_L = limit pressure as obtained from pressuremeter tests within a depth of 2B below the foundation level, ton/sq ft

γ = unit weight of the soil, ton/cu ft

D_f = depth of the footing, ft, and

K_g = empirical coefficient, which is a function of the nature of the soil and the geometry of the footing, as follows:

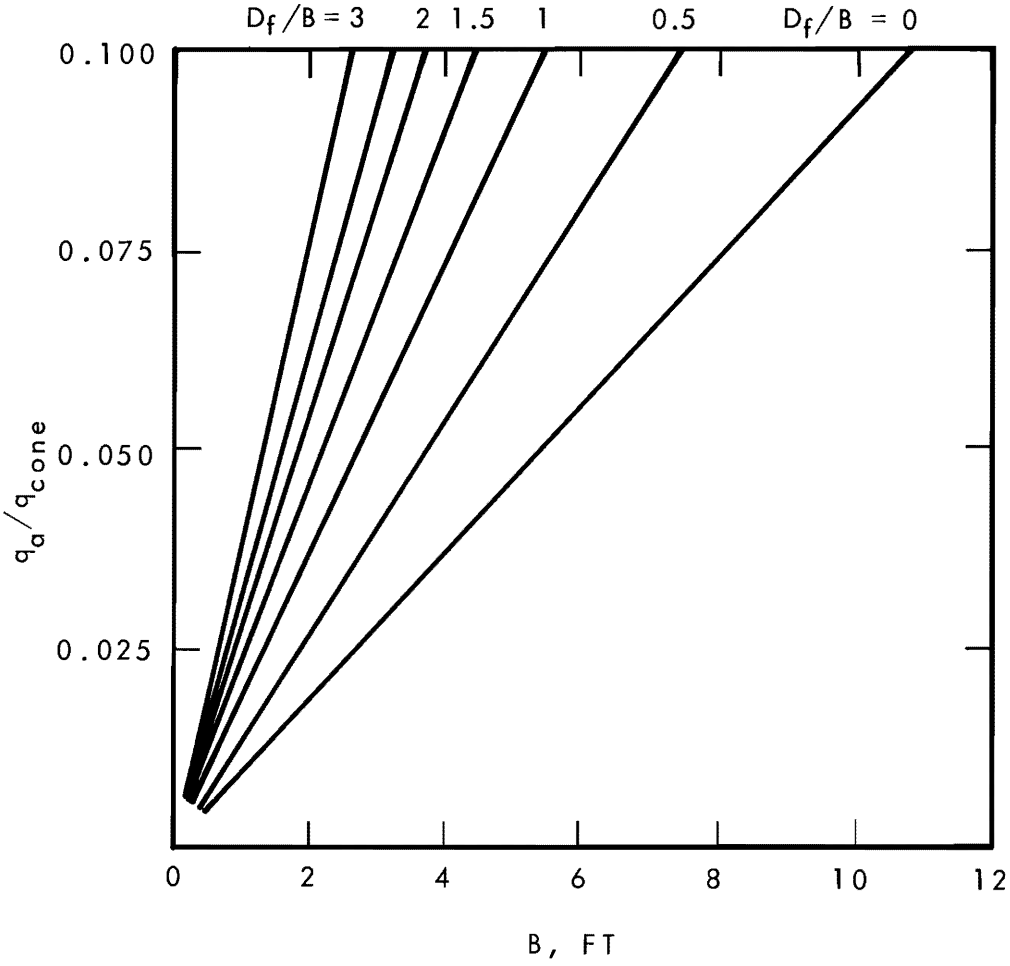


FIG 6.4

ALLOWABLE BEARING PRESSURE FROM STATIC CONE PENETRATION TESTS

Type of Soil	K_g
Cohesive	1 + 0.2 B/L
Non-cohesive, *loose*	1.1 + 0.22 B/L
Non-cohesive, *dense*	1.2 + 0.4 B/L

where B = width of the footing, and
L = length of the footing

6.3.5.1. *Limitations*

The pressuremeter test is an *in situ* test carried out with specialized equipment. The results of which are highly dependent on the quality of the borehole. Use of the pressuremeter and the interpretation of the results of the test should be restricted to geotechnical specialists. The relationship between P_L and q_a is empirical and should be used only for soils in which it has already been proved applicable; i.e., in all non-cohesive soils as well as in stiff, non-sensitive clays. The relationship should not be used for sensitive clays for which no experience is available.

The pressuremeter method is particularly well suited for soils generally difficult to investigate, such as sand, gravel and tills.

REFERENCES

MENARD, L., 1965. Règle pour le calcul de la force portante et du tassement des fondations en fonction des résultats pressiométriques; *Proc. Internat. Conf. Soil Mech. Found. Eng. 6th Montreal, 2: 295-299.*

MENARD, L., 1972. Rules for the calculation of bearing capacity and foundation settlement based on pressuremeter tests. *Draft Translation 159, U.S. Army Corps of Engrs., Cold Regions Research and Eng,g. Lab.*

6.3.6. *ULTIMATE BEARING CAPACITY CALCULATED FROM THE SOIL SHEARING STRENGTH*

The ultimate bearing capacity may be calculated using values of the shearing strength of the soil. The ultimate bearing capacity describes the load at which general shear failure of the soil beneath a footing takes place. The allowable bearing pressure is the ultimate bearing capacity divided by an appropriate factor of safety; it can be determined for a continuous strip footing where the strength of the subsoil is known by use of the following equation:

$$q_a = \frac{1}{FS}\left(c\,N_c + \gamma\,D_f N_q + \tfrac{1}{2}\gamma\,B\,N_\gamma\right)$$

where q_a = allowable bearing pressure, lb/sq ft

B = width of strip footing, ft

c = cohesive strength, lb/sq ft

γ = unit weight, lb/cu ft

D_f = depth of foundation, ft, and

N_c, N_q, N_γ = bearing capacity factors, depending on the angle of internal friction or angle of shearing resistance (ϕ or ϕ'). (See Fig. 6.5)

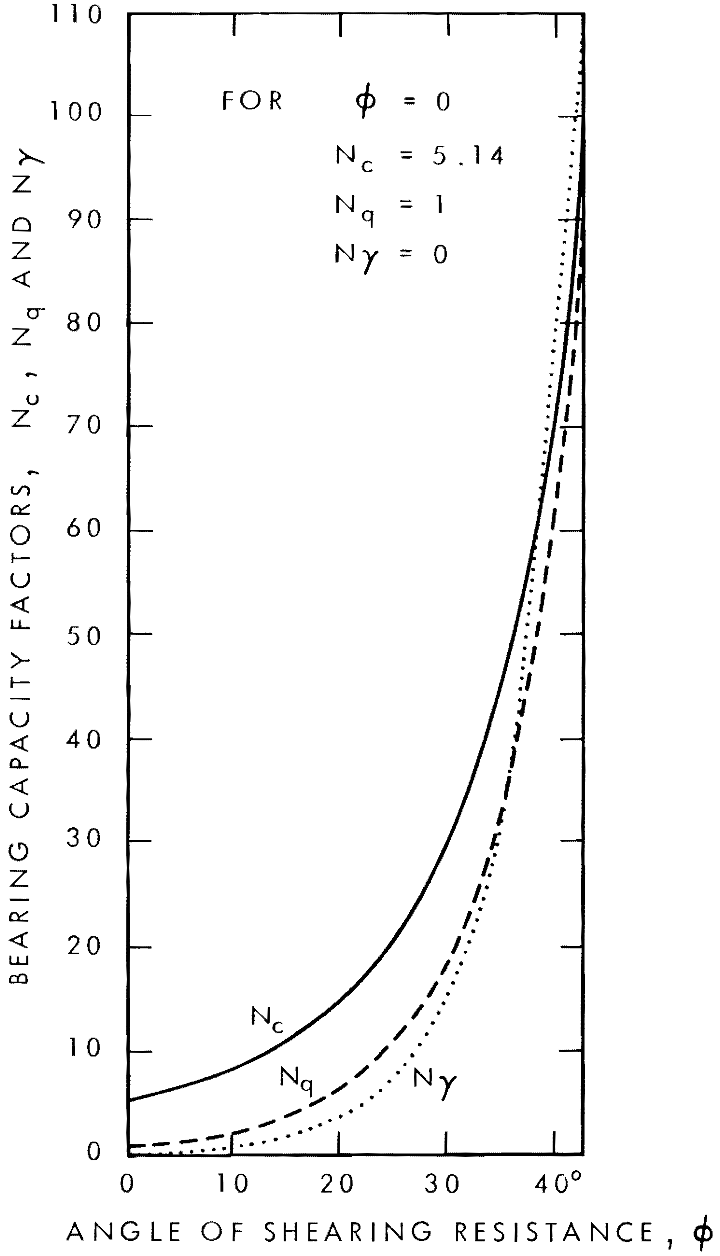


FIG 6.5 (After Hansen)

BEARING CAPACITY FACTORS

Note: The determination of ϕ, the angle of shearing resistance, is difficult for soils sensitive to sampling and testing techniques. Since small changes in ϕ greatly affect the bearing capacity factor, judgement must be used in its selection. Frequently cohesive soils are overconsolidated and the angle of shearing resistance ϕ (ϕ' in terms of effective stresses) will appear to be variable, depending upon the rate of loading in either the laboratory or the field case. The choice of ϕ and of the bearing capacity factors for use in calculation must then be made with the loading rate in mind, using values appropriate to the field problem.

6.3.6.1. *Limitations*

The use of this expression for calculation of the limiting equilibrium assumes that the ground surface is level and that the soil properties are known correctly and remain constant within the zone affected by the footing; i.e., to a depth below the bearing area greater than the width B, of the bearing area. In addition, N_c, N_q and N_γ are variable, depending on the theories used to compute them. The values selected here are those of Hansen as taken from D1N 4017.

6.3.6.2. *Effect of Groundwater Table*

The depth to the groundwater table should be considered, particularly when dealing with footings on granular soils and can be accounted for by using the submerged unit weight γ', where the groundwater table is above the bearing level, or intermediate values where intermediate groundwater levels apply.

6.3.6.3. *Shape Factor*

The shape of the footing can be considered using the following shape factors in the modified equation;

$$q_a = \frac{1}{FS}\left(c\,N_c\,S_c \;+\; \gamma\,D\,N_q\,S_q \;+\; \tfrac{1}{2}\,\gamma\,B\,N_\gamma\,S_\gamma\right)$$

SHAPE FACTORS

Shape of Footing	Shape factors	
	S_c, S_q	S_γ
Strip	1.0	1.0
Rectangular	1 + 0.3 B/L	1 - 0.4 B/L
Square or round	1.3	0.6

(from D1N 4017)

where B = width of the footing, and
L = length of the footing

6.3.6.4. *Eccentricity and Inclination Factor*

The eccentricity of loading and the effect of an inclined load may be considered by reducing the effective bearing surface and by using inclination factors in a modified formula using dimensions and symbols as shown in Fig 6.6 (See MEYERHOF 1963).

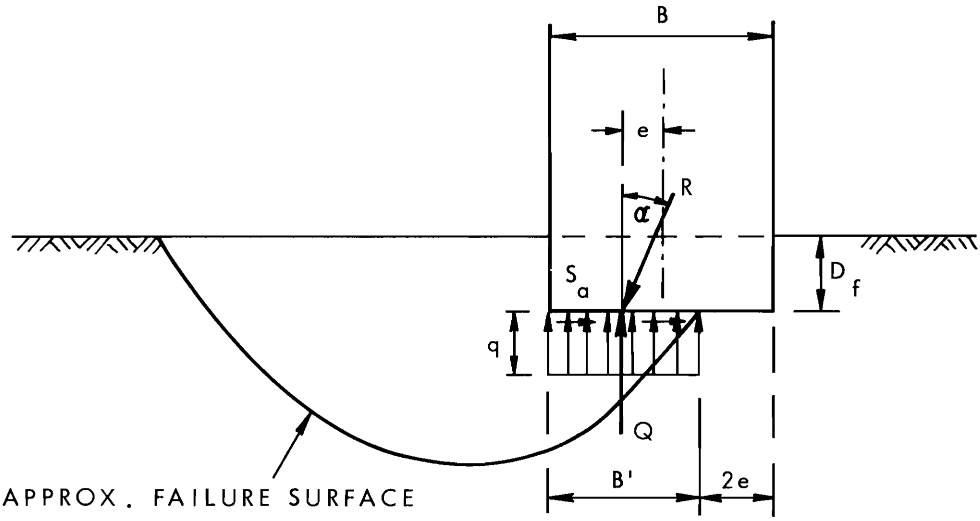


FIG 6.6 (After Meyerhof)

BASE UNDER ECCENTRIC INCLINED LOAD AT FAILURE

In considering eccentricity e, of the resultant load R, on the base of a foundation of width B, the effective width should be considered as the actual width, less twice the eccentricity:

$$B' = B - 2e$$

For eccentricity in two directions, the corrections may be made in both dimensions:

$$B' = B - 2e_B \quad \text{and}$$

$$L' = L - 2e_L$$

For loads inclined at an angle to the vertical, the effect can be considered by using inclination factors in the modified equation:

$$q_a = \frac{1}{FS} \left(c\, N_c i_c + \gamma\, D\, N_q i_q + \tfrac{1}{2}\, \gamma\, B\, N_\gamma i_\gamma \right)$$

where $\quad i_c = i_{qa} = (1 - \alpha/90°)^2$

and $\quad i_\gamma = (1 - \alpha/\phi)^2$

The effect of inclination may also be considered using slightly more complicated relationships by reference to DIN 4017.

6.3.6.5. *Slope Factor*

Strip footings on sloping ground or on level ground at the top of a slope may also be considered by using modified equations, (MEYERHOF, 1957). The bearing capacity is decreased by increasing the steepness of the slope. The decrease may be small for clays, but can be considerable for sands and gravels.

REFERENCES

MEYERHOF, G.G., 1963. Some recent research on bearing capacity of foundations, *Can. Geotech. J., 1: 16-26.*

MEYERHOF, G.G., 1957. "The ultimate bearing capacity of foundations on slopes", *Proc. Internat. Conf. Soil Mech. Found. Eng. 4th, London. 1: 384.*

DEUTCHER NORMENAUSSCHUSS, DIN 4017 (1973)

Bl 1 Vornorm Baugrund; Grundbruchberechnungen von lotrecht mittig belasteten Flachgründen. Richtlinien (8) (1965)

Bl 2 Vornorm Baugrund; Grundbruchberechnungen von aussermittig und schräg belasteten Flachgründungen. Empfehlungen (4) (1970)

6.3.7. *FACTOR OF SAFETY*

A factor of safety of three is commonly applied to the calculated ultimate bearing capacity to arrive at the allowable bearing pressure. Occasionally, under particular loading conditions, lower factors of safety may be justified; however, where allowable settlements govern, higher factors of safety may be required.

6.3.7.1. *Depth Term*

There is little or no ambiguity associated with the depth term γD, in the bearing capacity equations; i.e., the soil beneath the footing has already supported the weight removed in excavation. It is common to handle this term, unmodified by the bearing capacity factor N_q, independently with a factor of safety of unity.

6.4. STRESS DISTRIBUTION

6.4.1. *GENERAL*

To calculate settlement under an imposed loading, it is necessary first to calculate the increase of stresses within the ground resulting from this loading. This is conventionally done for a number of conveniently chosen increments of depth using influence values developed from the Boussinesq equations.

The Boussinesq equations are based on theories of elasticity assuming a perfectly flexible loaded area. Calculations will remain tolerably accurate within a loading range in which stresses are related to strains by constant ratios. This will normally be true at loadings where conventional factors of safety are used. It will normally not be true where failure is imminent or where distinctly non-linear stress-strain curves are typical of the particular soil. More rigorous solutions considering anisotropy of the soil mass are available, but it is likely that the errors in calculating settlement will outweigh the possible advantage of such detailed stress calculations.

6.4.2. *CALCULATION OF STRESSES IMPOSED BY A LOADED AREA*

The stresses in the ground resulting from a rectangular, uniformly loaded area may be calculated beneath a corner using the relationships illustrated in Fig 6.7(a)

The stress at any location under a loaded area may be calculated by dividing the surface in question into rectangles. The corner stress for each of four rectangles may be calculated and the stress at the point in question is the sum of these, as shown in Fig 6.7(b). Similarly, the stress outside the projected area of the footing may be calculated by constructing rectangles as shown in Fig 6.7(c).

The stress at any location under a line or point load may be calculated using the relationships illustrated in Fig 6.8.

The stress at any location under various configurations of surface loads may be calculated using the tables prepared by JURGENSEN, 1934.

The stress at any location under a loaded area of irregular shape may be calculated using the charts developed by NEWMARK, 1942. This is described by TERZAGHI and PECK, 1948, 1967; TAYLOR, 1948; and will be found in many other textbooks.

6.4.3. *SIMPLIFIED METHOD*

The stress imposed by a loaded area may also be calculated by assuming a uniform spread of the load. It is common practice to assume a spread of one horizontal to two vertical. The load is assumed to be distributed uniformly over the area of any horizontal plane within the frustum of a pyramid extending downward from the perimeter of the foundation unit.

REFERENCES

JURGENSEN, L., 1934. The application of elasticity and plasticity to foundation problems. *J. Boston Soc. Civil Engrs, 21: 206-241.*

NEWMARK, N.M., 1942. Influence charts for computation of stresses in elastic foundations. *Univ. Illinois Eng. Exp. Sta. Bull. 338, 28 p.*

STEINBRENNER, W., 1934. Tafeln zur Setzungsberechnung, *Die Strasse, 1: 121-124.*

STEINBRENNER, W., 1936. A rational method for determination of vertical normal stresses under foundations. *Proc. Internat. Conf. Soil Mech. Found. Eng. 1st Cambridge, Mass., 2: 142-143.*

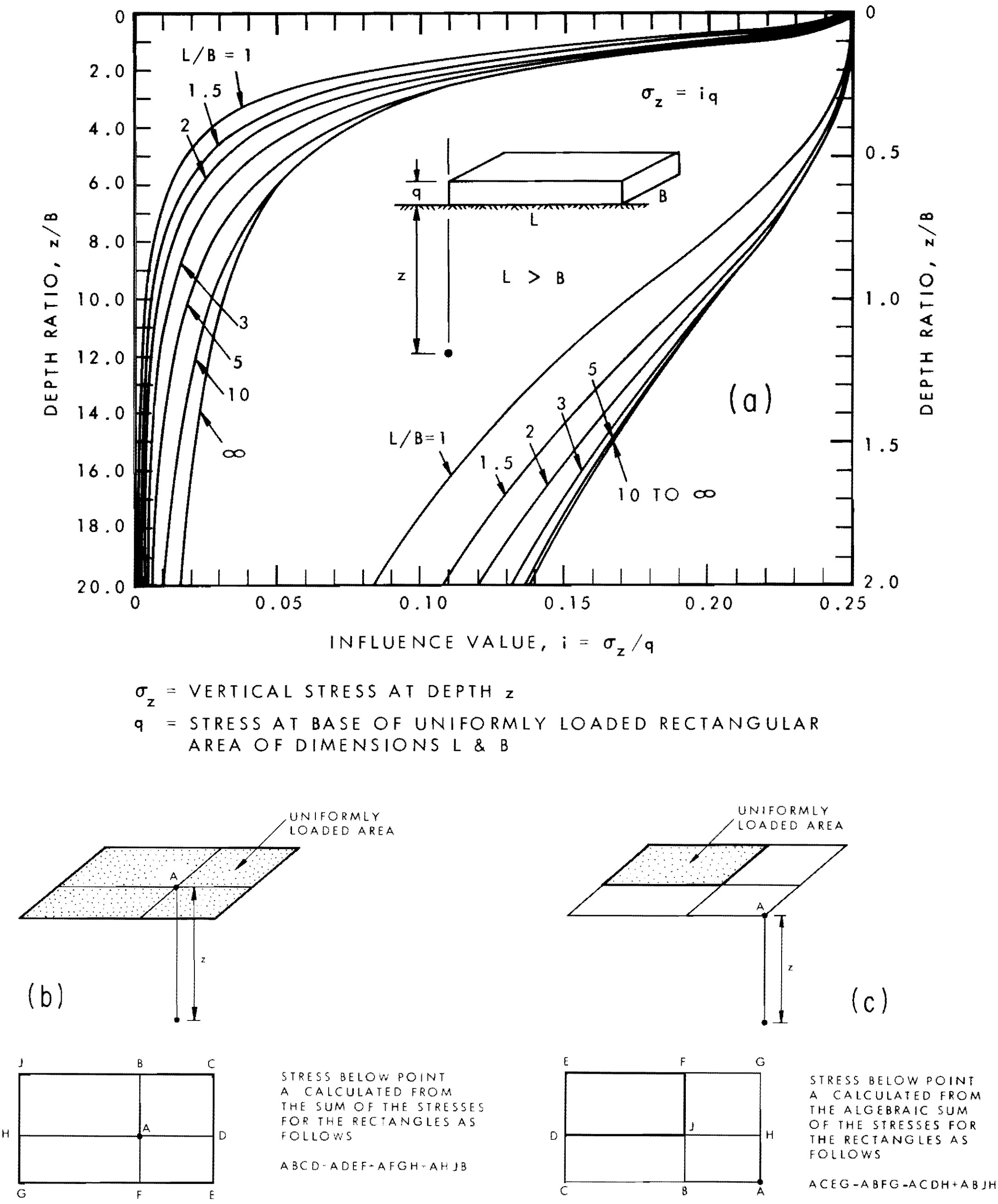
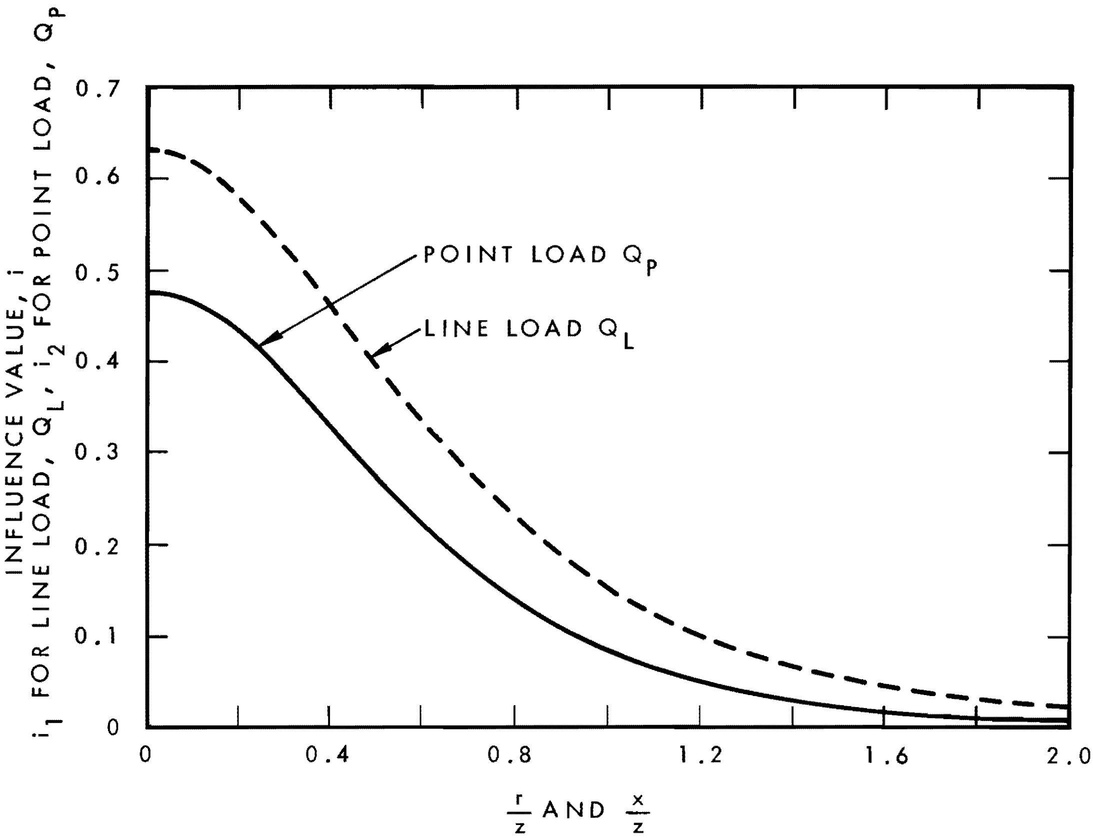
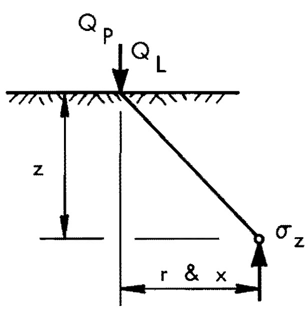


FIG 6.7 (After Steinbrenner)

DETERMINATION OF STRESS BELOW CORNER OF UNIFORMLY LOADED RECTANGULAR AREA





$$i_1 = \sigma_z \frac{z}{Q_L} = \frac{2}{\pi} \frac{1}{\left[\left(\frac{x}{z}\right)^2 + 1\right]^2}$$

$$\sigma_z = i_1 \frac{Q_L}{z}$$

$$i_2 = \sigma_z \frac{z^2}{Q_P} = \frac{3}{2\pi} \frac{1}{\left[\left(\frac{r}{z}\right)^2 + 1\right]^{5/2}}$$

$$\sigma_z = i_2 \frac{Q_P}{z^2}$$

FIG 6.8

STRESS AT DEPTH z FOR A LINE LOAD OR POINT LOAD AT DISTANCE x OR r RESPECTIVELY

6.5. SETTLEMENT

6.5.1. *GENERAL*

Settlement of a structure is the result of the deformation of the supporting subsoil. It may be evidence of:

- elastic deformation,
- volume changes due to a reduction of the water content (consolidation), or of the air content (compaction),
- general shear movement, or
- other factors such as subsoil collapse as in sink-hole formation or mining subsidence.

Of these, elastic deformation is usually too small to be significant and may normally be neglected. General shear movement, is considered under 6.2. BEARING PRESSURES ON ROCK and 6.3. BEARING PRESSURES ON SOIL and is of little concern where factors of safety, as considered in 6.3.7, are used. Subsoil collapse, is a local occurrence usually considered on the basis of regional experience.

Consolidation settlement involves a reduction in the water content of the subsoil and can be estimated and measured. It occurs in all soils.

6.5.1.1. *Cohesive (Fine-Grained) Soils*

The permeability of clay and silt is low, settlement is slow, and the prediction of its magnitude and rate is generally of importance.

6.5.1.2. *Non-Cohesive (Coarse-Grained) Soils*

The permeability of sands and gravels is sufficiently great that consolidation normally takes place during the construction period. Settlement of sands and gravels is largely the result of rearrangement of the particles and may be significant, particularly in loose deposits. Settlement, even when very low soil pressures are used in design, is likely to follow submergence, soaking, or vibration from blasting, machine operations, or earthquake.

6.5.2. *STANDARD PENETRATION TEST IN NON-COHESIVE SOIL*

The settlement of shallow footings may be roughly related to the N value obtained from the Standard Penetration Test. However, the accuracy of settlement predicted this way is questionable. As a simplified approach, TERZAGHI & PECK have suggested the relationship shown in Fig 6.9. The allowable bearing pressure obtained from this relationship is such that the resulting settlement will be about 1 in.

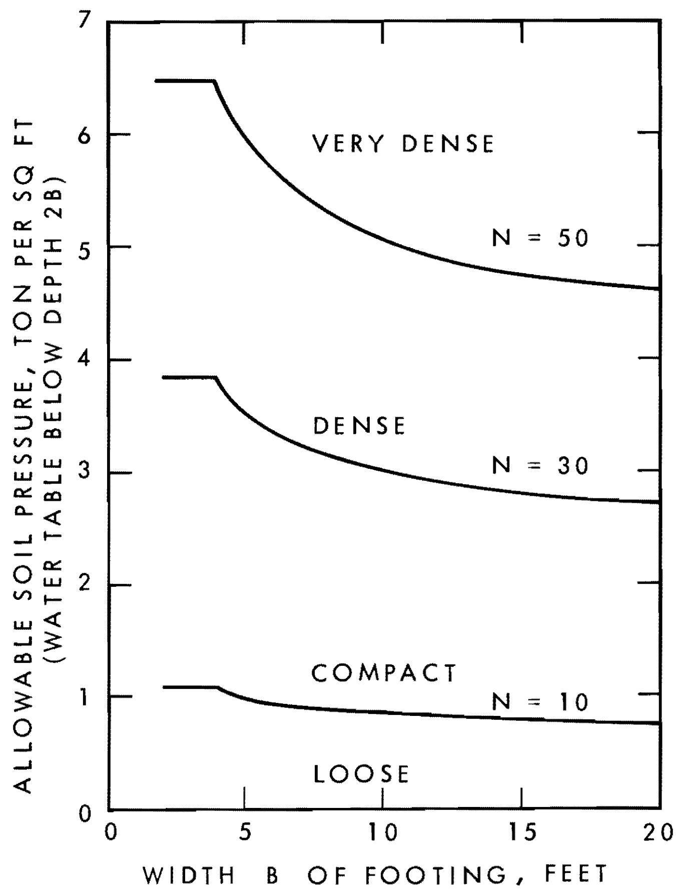


FIG 6.9

ALLOWABLE SOIL PRESSURE FOR FOOTINGS ON NON-COHESIVE SOILS ON THE BASIS OF 1 IN. SETTLEMENT

(From 'Soil Mechanics in Engineering Practice' by TERZAGHI & PECK, 1948. Used with permission of J. Wiley & Sons, Inc.).

6.5.2.1. *Submergence*

According to theory, the submergence of the sand located beneath the base of a footing should approximately double the settlement, provided the base is located at or near the surface of the sand. The values obtained from Fig 6.9. should be reduced by 50 per cent.

This procedure leads to conservative and probably over-conservative results (MEYERHOF, 1965). Submergence may reduce the penetration resistance, in which case the use of the N values determined in the field inherently includes a correction for submergence. In practice, therefore, it is common to neglect the effect of submergence and this may quite properly be done where local experience supports the procedure or where the possibility of greater settlement is not of controlling importance in the design.

6.5.2.2. *Limitations*

Settlement calculated using this procedure is generally greater than that actually observed.

The method is of limited value for soils containing gravel, cobbles, or boulders where single fragments may affect the blow count, and is not valid for cohesive or cemented soil.

REFERENCES

D'APPOLONIA, D.J., D'APPOLONIA, E. and BRISSETTE, K.F., 1968. Settlement of spread footings on sand. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs., 94: SM3, 735-760.*

FLETCHER, G.F.A., 1965. Standard Penetration Test: Its uses and abuses. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs., 91: SM4, 67-75.*

MEYERHOF, G.G., 1965. Shallow foundations. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs., 91: SM2, 21-31.*

PECK, R.B., HANSON, W.E. and THORNBURN, T.H., 1954. Foundation Engineering. *J. Wiley & Sons, N.Y.*

6.5.3. *STATIC CONE PENETRATION TEST IN NON-COHESIVE SOIL*

Settlement may be estimated from the results of static cone penetration tests by means of the relationship between the coefficient of compressibility, α_c and the cone point resistance, q_{cone}.

$$\alpha_c = \frac{\beta\, q_{cone}}{p_o}$$

where α_c = coefficient of compressibility
q_{cone} = cone resistance
p_o = effective overburden pressure, and
β = is a coefficient depending on soil density as follows"

Soil density	β
Dense sand	<1
Compact sand	1
Loose sand	1.5

Settlement under a shallow foundation can then be estimated by substituting the value of α_c into the settlement equation.

$$s = 2.3\ \frac{H}{\alpha_c} \log \frac{p_o + \Delta p}{p_o}$$

where S = settlement
H = thickness of deposit, and
Δp = pressure change applied to the soil layer

6.5.3.1. *Limitations*

The static cone test was developed for use in loose, uniform, fine-grained soils and field verification has been restricted largely to deposits of these materials. The equipment normally used is effective in such soils, but may give trouble in dense or mixed-grain deposits.

Experience indicates that the α_c calculated by this method is usually low, giving an upper limit to estimated settlements. The static cone penetration test should be supplemented with subsurface data from conventional boreholes.

REFERENCES

SANGLERAT, G., 1972. The penetrometer and soil exploration, *Elsevier Publ. Co., Amsterdam.*

SCHMERTMANN, J.H., 1970. Static cone to compute static settlement over sand, *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs., 96: SM3, 1011-1043.*

6.5.4. *PLATE BEARING TEST ON NON-COHESIVE SOIL*

A plate bearing test may be carried out on non-cohesive soils in which the settlement of a 1 ft sq test plate is measured and related to the expected settlement of a footing. The relationship suggested by TERZAGHI & PECK (1948, 1967), who also describe the conditions required for a *Standard Load Test* is:

$$S = S_1 \left(\frac{2B}{B + 1} \right)^2$$

where S = settlement of footing with width B, ft, and

S_1 = settlement of a 1 ft sq loading plate under the pressure expected to be applied by the footing.

6.5.4.1. *Limitations*

The method is only considered suitable for use in non-cohesive soils where time-dependent settlement relationships are negligible. It tests only a shallow depth of soil which must be representative of the stratum affected by the footing. Extrapolation to large footings should be carried out with caution.

From an inspection of Fig 6.1, it will be obvious that the model footing of a plate loading test smaller than the prototype will stress an entirely different depth of material. It follows that the test will be misleading if the material properties change within the depth affected by the larger footing.

The test is cumbersome to perform and potentially misleading. It requires supplementary information from boreholes and, generally, these will yield sufficient information to allow satisfactory estimates to be made without the use of detailed load tests.

REFERENCES

D'APPOLONIA, D.J., D'APPOLONIA, E. and BRISSETTE, R.F., 1968. Settlement of spread footings on sand. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs., 94: SM3, 735-760.*

6.5.5. *PRESSUREMETER TEST*

Settlement may be estimated from the results of an *in situ* pressuremeter test, by means of the following simplified formula:

$$S = \frac{q_{net}}{E_p} f$$

where S = settlement, ft,

q_{net} = net design bearing pressure, ton/sq ft,

E_p = pressuremeter modulus as obtained from pressuremeter test, within a depth of 2B below the foundation level, ton/sq ft, and

f = empirical coefficient, ft which is a function of the nature of the soil a the geometry of the footing, as given in Fig 6.10.

6.5.5.1. *Limitations*

The pressuremeter test is an *in situ* test carried out with specialized equipment. Its use and the interpretation of the results should be restricted to geotechnical specialists.

The determination of the pressuremeter modulus E_p, is highly sensitive to the method of boring and testing. Reliable results can be expected in stiff or dense soils, provided bentonite mud is used as the drilling fluid. The method is not applicable in loose sand and silt deposits or in soft clays.

REFERENCES

MENARD, L., 1965. Règle pour le calcul de la force portante et du tassement des fondations en fonction des résultats pressiométriques. *Proc. Internat. Conf. Soil Mech. Found. Eng. 6th, Montreal, 2: 295-299.*

MENARD, L., 1972. Rules for the calculation of bearing capacity and foundation settlement based on pressuremeter tests. *Draft Translation 159, U.S. Army Corps of Engrs. Cold Regions Research and Engineering Laboratory.*

6.5.6. *CRITICAL POINT METHOD*

6.5.6.1. *Definition*

The *critical point* is that point of a foundation under which the settlement is independent of the footing's rigidity. Therefore, the settlement under that point, as computed from the Boussinesq solution for flexible footings, will be equal to the settlement of a rigid footing of the same area supporting the same load.

A determination of the settlement can be made simply for the *critical point*, assuming a constant value of the elastic modulus of the soil (KANY, 1959.)

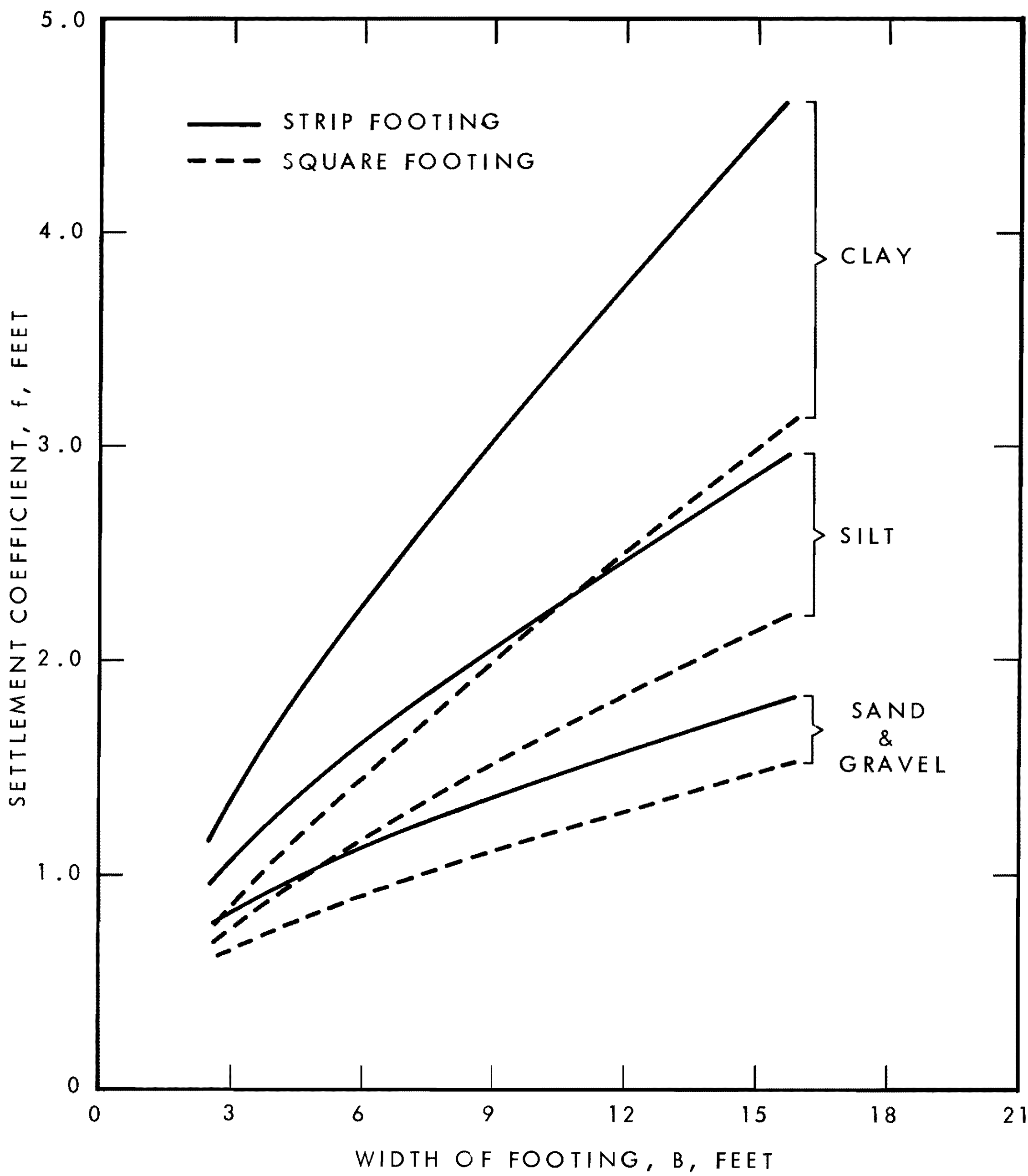


FIG 6.10
SETTLEMENT COEFFICIENT, f (PRESSUREMETER TEST)

6.5.6.2. *Formula*

The settlement may be estimated using the relationship:

$$S = \frac{B\ q_{net}}{E_s}\ f_c$$

where
S = settlement, ft
B = footing width, ft
q_{net} = net design bearing pressure, ton/sq ft
E_s = modulus of elasticity of the soil, ton/sq ft, and
f_c = settlement coefficient, as given in Fig 6.11.

6.5.6.3. *Application*

To apply the method, a representative value of E_s must be selected.

1) *In non-cohesive soils*

The settlement is independent of time. E_s may be determined from:

(a) SPT results as shown in Fig 6.12;

(b) Static cone penetration test results by means of the relationship;

$$E_s = 1.5\ R_p$$

where R_p is the average point resistance, ton/sq ft, or

(c) The density of the soil, as follows:

Modulus of elasticity of cohesionless soils E_s, ton/sq ft

Soil Type	Density		
	Loose	*Medium*	*Dense*
Gravel	300 - 800	800 - 1000	1000 - 2000
Sand	100 - 300	300 - 500	500 - 800
Fine Sand	80 - 120	120 - 200	200 - 300

(after KEZDI)

2) *In cohesive soils*

The settlement is time-dependent; two cases must be considered:

(a) For the immediate settlement, E_s is taken equal to E_u, as determined from the vane strength, c_u, by means of the empirical relationship:

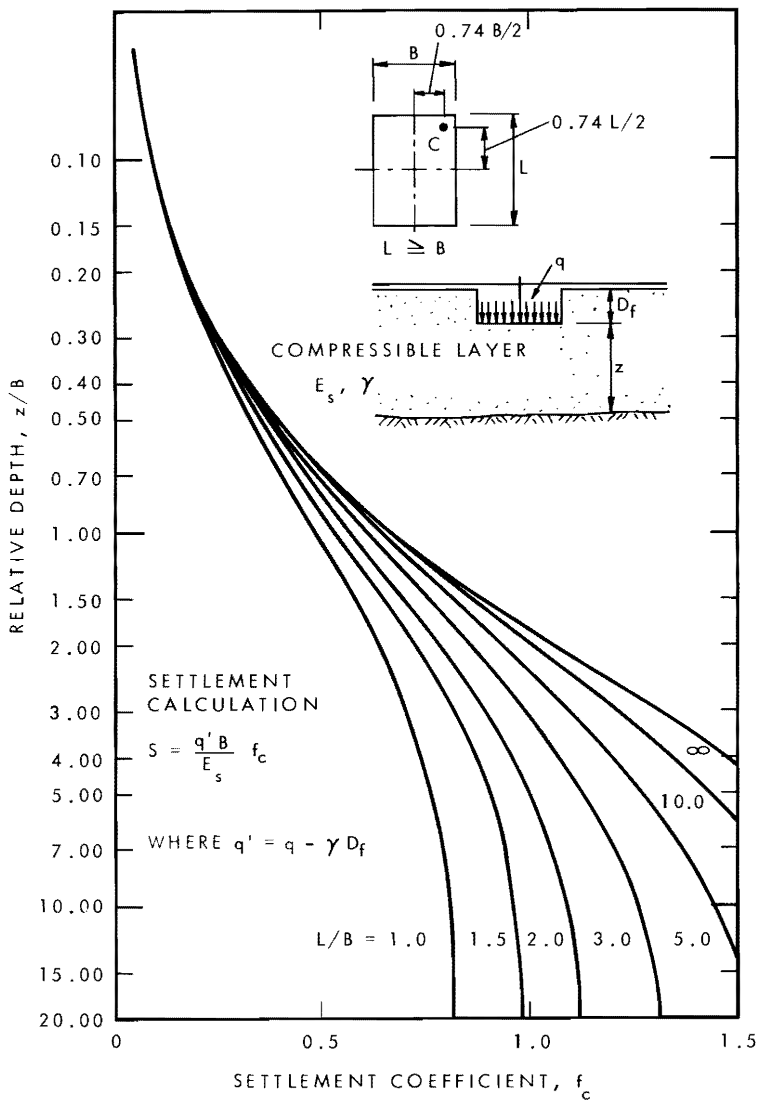


FIG 6.11 (After Kany)

CURVES FOR CALCULATION OF SETTLEMENT AT THE CRITICAL POINT, C

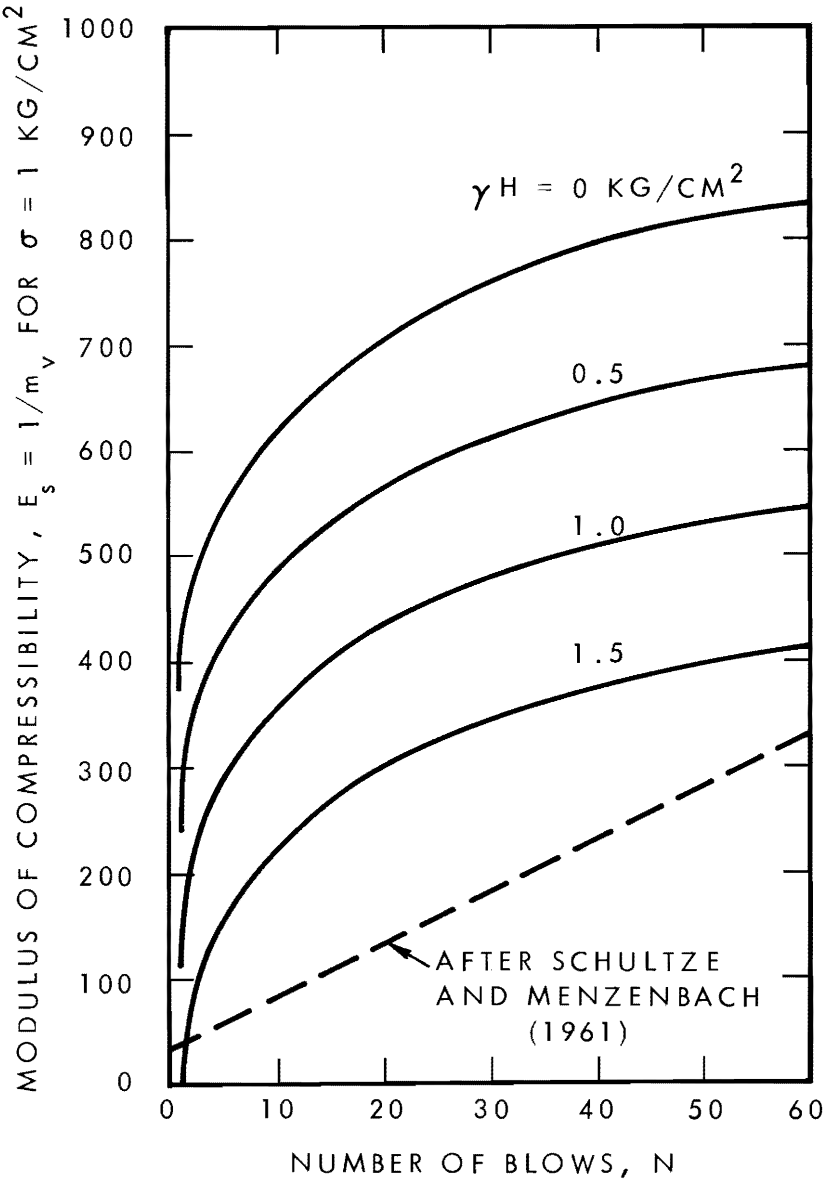


FIG 6.12 (After Schultze and Melzer)

RELATIONSHIP BETWEEN THE MODULUS OF COMPRESSIBILITY, $E_s = 1/m_v$ AND NUMBER OF BLOWS, N

$E_u = 500\ c_u$ for *soft* sensitive clays
$E_u = 1000\ c_u$ for *firm* to *stiff* clays
$E_u = 1500\ c_u$ for *very stiff* clays

(b) For long-term settlement, E_s is taken equal to $1/m_v$, where m_v is determined from consolidation tests. (Note that E_s is the slope of the consolidation curve when plotted on a linear Δh versus load plot).

3) *In stratified deposits*

In cases where layers with different moduli occur within a depth of 3B below the foundation level, this fact may be accounted for by using the modified formula:

$$S = B\ q \left(\frac{f_1}{E_{s1}} + \frac{f_2 - f_1}{E_{s2}} + \frac{f_3 - f_2}{E_{s3}} + \ldots \frac{f_n - f_{n-1}}{E_{sn}} \right)$$

where f_1, f_2, f_3, ... f_n are the settlement coefficients obtained assuming the depth of the compressible layer z to be equal to z_1, z_2, z_3, ... z_n, depth of the bottom of layers 1, 2, 3, ... n and where E_{s1}, E_{s2}, E_{s3}, ... E_{sn} are the moduli of layers 1, 2, 3, ... n. (See Fig 6.11).

6.5.6.4. *Limitations*

This method requires an approximation of the value of the modulus of elasticity E_s. It is well known that E_s is a function of the stress level and therefore variable with depth and applied load. The settlements estimated by this method will therefore be only as good as the estimate of the representative value of E_s selected by the designer.

The method is useful for preliminary design purposes, but should be used with discretion for final design.

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KANY, M., 1959. Beitrag zur Berechnung von Flächengründungen, *Wilh. Ernst & Sohn, Berlin.*

KEZDI, A., 1964. Bodenmechanik. *VEB Verlag für Bauwesen, Berlin, Band 1.*

SCHULTZE, E. and MELZER, K.-J., 1965. The determination of the density and the modulus of compressibility of non-cohesive soils by soundings, *Proc. Internat. Conf. Soil Mech. Found. Eng., 6th, Montreal, 1: 354-358.*

SANGLERAT, G., 1972. The penetrometer and soil exploration, *Elsevier Publ. Co., Amsterdam.*

6.5.7. *CONSOLIDATION SETTLEMENT OF COHESIVE SOIL*

The consolidation settlement of cohesive soil is normally computed on the basis of laboratory tests. Consolidation tests are carried out in the laboratory on undisturbed samples of clay or silt contained in a rigid ring between porous stones and loaded axially. The results of the test and, by analogy, settlement of a confined soil stratum in the field can be expressed as follows:

$$S = H \frac{C_c}{1 + e_o} \log \frac{p_o + \Delta p}{p_o}$$

where S = total settlement,

H = original thickness of stratum,

C_c = compression index,

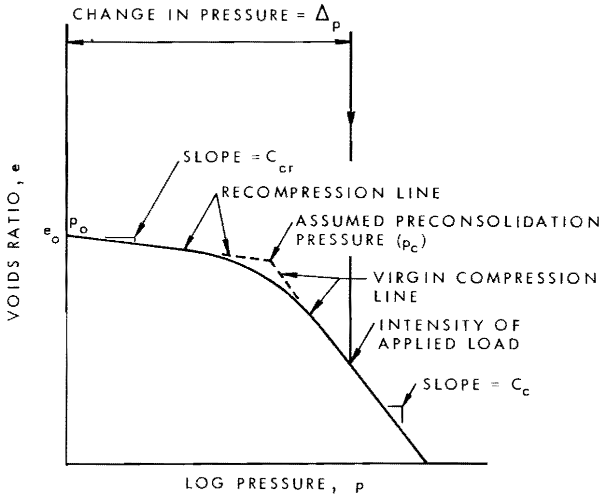
e_o = initial void ratio,

p_o = average initial effective pressure,

Δp = the average change in pressure in the compressible stratum considered.

The values of both e_o and C_c must normally be determined by test. The value of e_o for saturated soil is directly related to water content and can be found simply. The value of C_c is obtained from a consolidation test which requires detailed laboratory procedures.

The value of C_c is grossly affected by the consolidation history of the clay. The value of C_c obtained on the first loading of the soil will be many times greater than the value obtained upon recompression. Computation of settlement, then, must take into account the preconsolidation pressure (over-consolidation) of the clay; that is, the greatest pressure in excess of the existing overburden pressure with which the soil has been in equilibrium. The relationships of the compression indices to void ratio e, and pressure are illustrated in Fig 6.13.



NOTE:
Settlement is dependent upon both C_{cr} and C_c

FIG 6.13

FIELD e - $\log_p$ RELATIONSHIP

(From "The Design of Foundations for Buildings' by JOHNSON & KAVANAGH, Copyright 1968. Used with permission of McGraw-Hill Book Company).

For loadings less than the preconsolidation pressure, p_c settlement will be computed using a value of the compression index representing recompression, C_{cr}. For loadings greater than the preconsolidation pressure, settlement will be computed using the compression index, C_c. Where the increase in pressure represents both recompression and loading in excess of the precompression load, the settlement equation may be written:

$$S = H \left\{ \frac{C_{cr}}{1 + e_o} \log \frac{p_c}{p_o} + \frac{C_c}{1 + e_o} \log \left(\frac{p_o + \Delta p - p_c}{p_c} \right) \right\}$$

The estimation of the preconsolidation pressure is technically complicated and will usually require geological confirmation. Results are frequently ambiguous.

6.5.7.1. *Settlement-Time Relationships*

Consolidation is a time-dependent process and, typically, under a particular load will plot as shown in Fig 6.14.

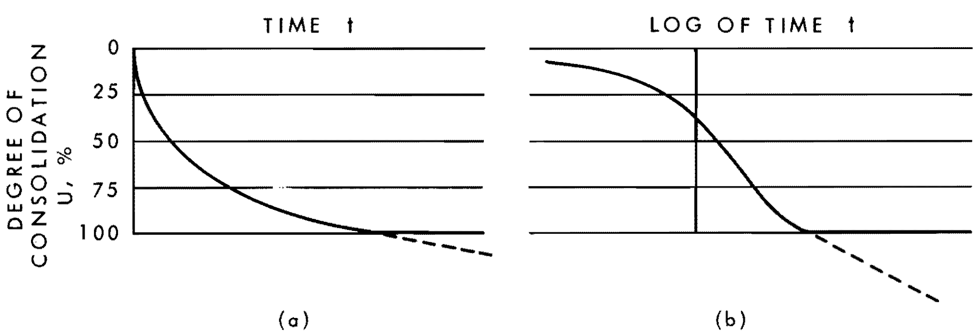


FIG 6.14
TYPICAL TIME - SETTLEMENT RELATIONSHIPS

(From 'Soil Mechanics in Engineering Practice' by TERZAGHI & PECK, 1948. Used with permission of J. Wiley & Sons, Inc.).

Three significant portions of the measured settlement may be considered:

(i) An initial compression which occurs immediately owing at least in part to the compression of gas in the pore space (not shown in Fig 6.14);

(ii) The compression indicated by the solid lines of Fig 6.14 known as primary consolidation, which is accompanied by a corresponding drop in pore water pressure;

The time at which consolidation will take place can be calculated from the equation:

$$t_n = T_n \frac{H^2}{c_v}$$

where t_n = time for the degree of consolidation U, in per cent, expressed by n, in per cent to occur, min.,

H = length of drainage path, ft, (for the usual case of double drainage, 2H equals the thickness of the consolidating stratum),

c_v = coefficient of consolidation* (sq ft/min) for the appropriate range of pressures, and

T_n = time factor, as follows **

U%	10	20	30	40	50	60	70	80	90
T_n	.003	.031	.071	.126	.197	.287	.403	.557	.848

(iii) The compression indicated by the difference between the solid and dashed lines of Fig 6.14, known as *secondary compression,* or the consolidation resulting from the *secondary time effect.* This takes place at constant effective stress with no change in pore water and is related to the portion of the curve in which excess pore water pressures are negligible.

Secondary compression is significant in comparison with *primary consolidation* when considering some highly compressible clays, peat, highly organic soil, and some micaceous soils. For a discussion of the settlement of peat, refer to the Muskeg Engineering Handbook, MacFARLANE (1969).

6.5.7.2. *Limitations*

The settlement calculation must be based on an estimate of the field consolidation curve. The principal difficulty in making such an estimate is that of determining the preconsolidation pressure p_c. Where the preconsolidation pressure is not clearly defined or not carefully determined the indicated values of the compression index C_c, may be in error by as much as an order of magnitude.

Settlement-time predictions are also subject to considerable error. Laboratory values of the coefficient of consolidation c_v, are not easily derived and field drainage conditions may be difficult to determine. Where time is important, such predictions are usually checked by field measurements.

Organic and other materials subject to significant *secondary compression* are difficult to sample and test and estimates of the amount of settlement and time are usually based on field experience.

---

* An approximate value of c_v can be obtained from the relationship:

$$c_v = \frac{kE_s}{\gamma_w}$$

where k = permeability, ft/min,

E_s = modulus of elasticity, as discussed in 6.5.6.3. and

γ_w = unit weight of water, ton/cu ft

** These values relate to a constant initial state of hydrostatic excess pressure and this is the relationship most common in practice. Other relationships are referred to in the literature.

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BJERRUM, L., 1967. Engineering geology of Norwegian normally-consolidated marine clays as related to settlements of buildings. Seventh Rankine Lecture, *Geotechnique, 17: 83-117.*

CRAWFORD, C.B., 1964. Interpretation of the consolidation test. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs. 90: SM5, 87-102.*

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SCHMERTMANN, J.H., 1953. Estimating the true consolidation behaviour of clay from laboratory test results, *Proc. Am. Soc. Civil Engrs., 79: Separate 311.*

SKEMPTON, A.W. and BJERRUM, L., 1957. A contribution to the settlement analysis of foundations on clay, *Geotechnique, 7: 168-178.*

6.5.8. *ALLOWABLE SETTLEMENT*

For any given structure there is a certain amount of settlement, either differential or total, that can be tolerated without:

- overstressing the structure;
- creating an unacceptable maintenance or aesthetic problem.

The foundation must be designed so that anticipated settlements do not exceed the lesser of these amounts.

6.5.8.1. *Differential Settlement*

Allowable displacement criteria in common use are as follows:

(i) *Maximum deflection between supports where L is the span length*

Members supporting walls or partitions	
masonry, glass or other frangible material	L/360
metal cladding or similar nonfrangible finishes	L/240
Steel or concrete frames	L/150 to L/180
Timber frames	L/100
Steel or concrete shear walls	by design

(ii) *Limitation on slope*

Type of construction	Maximum slope*
High continuous brick walls	.005 to .001
Brick dwellings	.003
Brick cladding between columns	.001
Reinforced concrete building frame	.0025 to .004
Reinforced concrete curtain wall	.003
Continuous steel frame	.002
Simply supported steel frame	.005

Similarly values are given by BJERRUM (1963), in Fig 6.15.

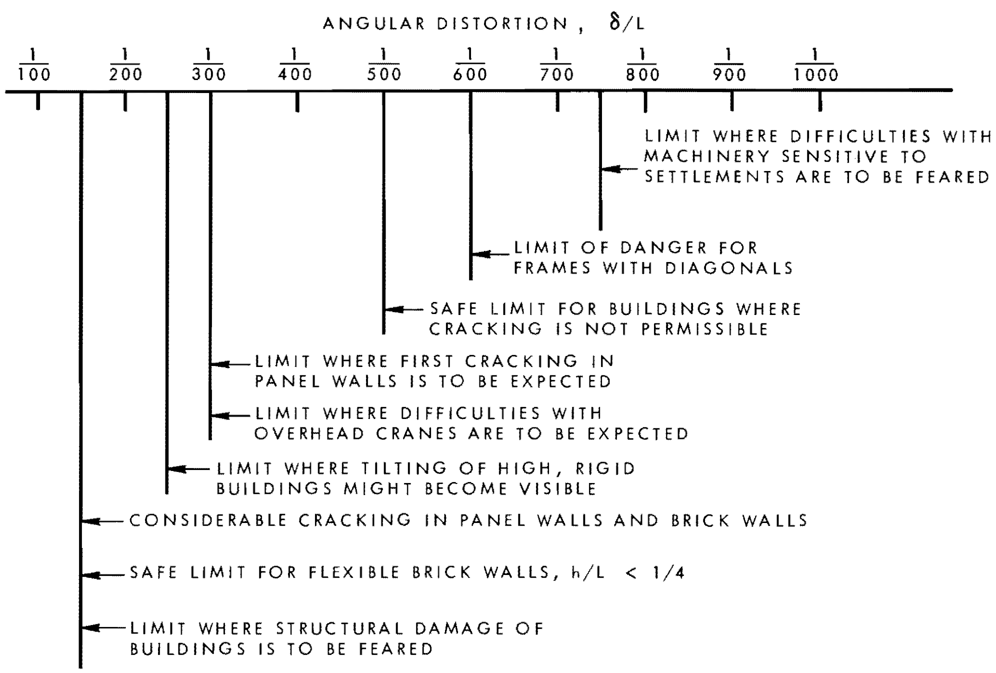

FIG. 6.15 (After Bjerrum)

DIFFERENTIAL SETTLEMENT AND ANGULAR DISTORTION RELATED TO BUILDING PERFORMANCE

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* Maximum slope of deflected configuration from line of reference as-built configuration.

6.5.8.2. *Total Settlement*

Differential settlement will occur in all cases because of the natural variability of soils even where total settlements are calculated to be uniform. The magnitude of these differential settlements may be related to the magnitude of the total settlements, for example, see D'APPOLONIA et al 1968. Consequently, limiting the total settlement of a structure is frequently used as an indirect means of controlling the amount of differential settlement.

The following values are suggested:

	Maximum total settlement
Structures on clay	3 in.
Structures on sand	$1\frac{1}{2}$ in.

6.5.8.3. *Limitations*

Design limits on differential settlement are frequently set in totally unrealistic terms. In fact, each structure should be considered individually with the tabulated values providing only a guide.

Design limits on total settlement are simple criteria to apply and are commonly used. Many successful structures may be seen, however, with total settlement greatly in excess of the values quoted.

REFERENCES

D'APPOLONIA, D.J., D'APPOLONIA, E. and BRISSETTE, R.F., 1968. Settlements of spread footings on sand. *J. Soil Mech. Found. Div., Proc. Am. Soc. Civil Engrs. 94: SM3, 735-760.*

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SKEMPTON, A.W. and MacDONALD, D.H., 1956. The allowable settlement of buildings. *Proc. Inst. Civil Engrs., Part III, Volume 5, 727-768.*

SOWERS, G.F., 1962. Shallow Foundations. In: G.A. LEONARDS. Foundation Engineering. *McGraw-Hill, N.Y.*

6.6. DESIGN BEARING PRESSURE

The design bearing pressure is limited by two considerations:

- the foundation must be safe against shear failure of the supporting soil, and
- post-construction settlement must not be excessive.

The design bearing pressure is the lesser of the values dictated by these two requirements.

A detailed flow diagram for the design of shallow foundations is shown in Fig 6.16. In many cases this can be simplified.

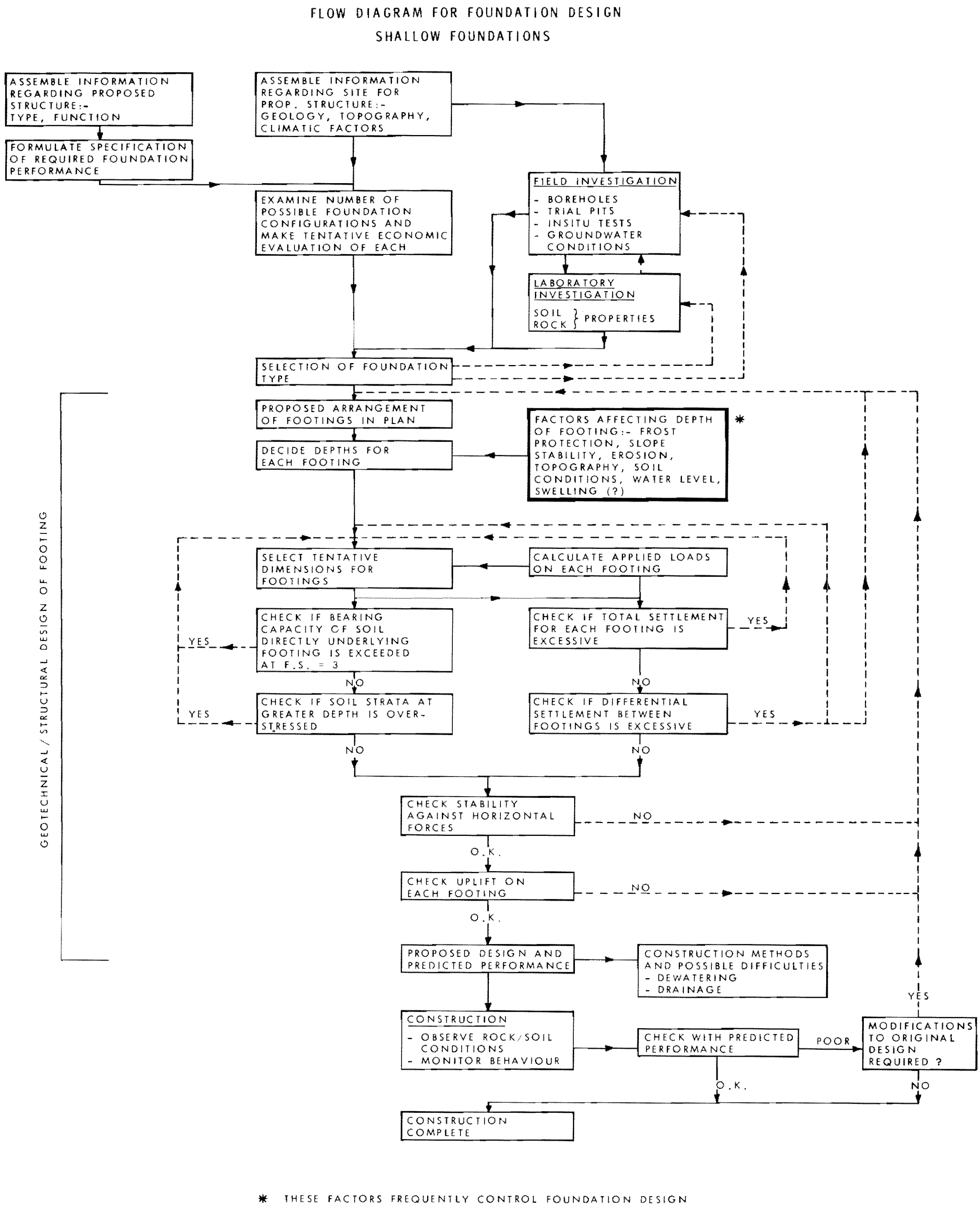

FIG 6.16

FLOW DIAGRAM FOR DESIGN OF SHALLOW FOUNDATIONS

CHAPTER 7

DEEP FOUNDATIONS

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CHAPTER 7

DEEP FOUNDATIONS

7.1. GENERAL

7.1.1. *DEFINITIONS*

A deep foundation is a foundation unit that provides support for a building by transferring loads either by end-bearing to a soil or rock at considerable depth below the building, or by adhesion or friction, or both, in the soil or rock in which it is placed. Piles are the most common type of deep foundation.

Piles can be pre-manufactured or cast-in-place; they can be driven, jacked, jetted, screwed, bored or excavated. They can be of wood, concrete or steel, or combination thereof. Bored piles of large diameter are frequently referred to as caissons in Canada; in this Manual they are considered under bored piles.

7.1.2. *RELATIONSHIP OF NBC SUBSECTION 4.2.7. AND THIS CHAPTER*

The quality of a deep foundation is highly dependent on construction technique, on equipment and on workmanship. Such parameters cannot be quantified nor taken into account in normal design procedures. Consequently, as implied in NBC Subsection 4.2.7. it is highly desirable to design deep foundations on the basis of *in situ* load tests on actual foundation units.

Few projects however, are large enough to warrant full scale load tests during the design phase, and in most cases load tests are performed only during or even after construction of the foundation. Therefore, it is necessary to provide the engineer with appropriate design methods. This chapter presents a series of alternative methods applicable to the various types of deep foundations encountered in practice.

7.1.3. *LIMITATIONS*

Due to the determining influence of construction procedures on the behaviour of deep foundations, the methods presented in this chapter may lead to successful designs of deep foundations only if appropriate inspection of the construction is carried out as required in Article 4.2.2.3. of the NBC. Inspection should be considered as an integral part of the design of any foundation.

Load tests should always be performed to check the validity of the design, since the best design method is still not so reliable as a load test.

7.2. GEOTECHNICAL DESIGN OF DEEP FOUNDATIONS

7.2.1. *DEEP FOUNDATIONS ON ROCK*

7.2.1.1. *General*

Deep foundations sitting on or socketed into rock normally carry heavy loads. They may be bored or excavated and cast-in-place. In this case the area of contact with rock is known and the load capacity can be evaluated by means of the design methods given in 7.2.1.2.

Deep foundations may also be driven to rock. In this case, which includes steel H piles, pipe piles driven with a closed end or precast concrete piles, the exact area of contact with rock, the depth of penetration into rock as well as the quality of rock at the foundation level are largely unknown. Consequently, the determination of the load capacity of such deep foundations cannot be made by means of the methods given below, and should be made on the basis of driving observations, local experience and load tests.

7.2.1.2. *Load Capacity*

(1) *Design assumptions*

In most cases where cast-in-place deep foundations are socketed into rock the depth of the socket is typically 1 to 3 times the diameter of the foundation. Present Canadian practice for the design of such deep foundations varies from region to region. Three different design assumptions are in use:

(a) The load capacity is assumed to be derived from point resistance only. This assumption can be considered as safe, since the bearing capacity of the rock is available, irrespective of the construction procedure. However, if the bottom of the excavation is not properly cleaned, the bearing capacity may not be mobilized before large settlements occur due to the compression of mud remaining in the bottom of the socket.

Design methods based on this assumption are given in 7.2.1.2. (2) and (3).

(b) The load capacity is assumed to be derived from the bond between concrete and rock along the surface perimeter of the socket. This assumption is not necessarily safe. Theoretical considerations indicate that a uniform mobilization of the bond is possible only if the modulus of elasticity of both concrete and surrounding rock are of the same order of magnitude (COATES 1967). Furthermore the available bond strength is highly dependent on the quality of the rock surface on the walls of the socket.

The design method based on this assumption is discussed in 7.2.1.2. (4).

(c) The load capacity is assumed to be derived from both point resistance and lateral bond. This assumption leads to unreasonably high load capacities. It should not be used unless it can be proved applicable by means of full scale load tests or well-supported local experience.

(2) *Allowable bearing pressure from properties of rock cores*

The method described in Chapter 6 of this Manual is applicable to deep foundations. In this case the effect of depth is included and the formula becomes:

$$q_a = q_{u\text{-core}} K_{sp} d$$

where q_a = allowable bearing pressure,

$q_{u\text{-core}}$ = average unconfined compressive strength of rock core, from ASTM D2938-71,

K_{sp} = empirical factor as given in 6.2.2. including a factor of safety of 3, and

d = depth factor:

$$= 0.8 + 0.2 \frac{H_s}{D} \leqslant 2$$

where H_s = depth of the socket in rock having a strength $q_{u\text{-core}}$

D = diameter of the socket

This method is generally not applicable to soft stratified rocks such as shales or limestones. The values of the basic parameter $q_{u\text{-core}}$, are generally not representative of the actual mechanical properties of the rock mass because of the effect of sampling disturbance and the absence of discontinuities in the test specimens.

The allowable bearing pressure as obtained from this method should be checked against the range of values shown in Table 6.2.

(3) *Allowable bearing pressure from pressuremeter test results*

In situ pressuremeter tests allow for a direct determination of the strength of the rock mass, including the effect of joints and weathering. Where performed properly (see Commentary 8.8) the pressuremeter test gives a strength index of the rock mass called the limit pressure, p_L. The test and the corresponding design methods are particularly suited for weathered or closely jointed rocks and for soft rocks in general.

The allowable bearing pressure is given by:

$$q_a = \frac{1}{3}\left[p_o + K_b\,(p_L - q_o)\right]$$

where q_a = allowable bearing pressure, ton/sq ft (a factor of safety of 3 is included),

p_o = overburden pressure at the elevation of the pile tip, ton/sq ft,

q_o = *at rest* horizontal stress in the rock at the elevation of the pile tip, ton/sq ft (for practical purposes, it can be assumed that: $q_o = p_o$),

p_L = limit pressure as determined from pressuremeter tests in the zone extending 2 pile diameters above and below the pile tip, ton/sq ft, and

K_b = an empirical bearing capacity coefficient as follows:

$\frac{\text{Depth of socket}}{\text{Pile diameter}}$	0	1	2	3	5	7
K_b	0.8	2.8	3.6	4.2	4.9	5.2

(4) *Load capacity from bond between concrete and rock*

It is common practice in some regions to assume that the entire load from the pile is transferred to the rock by adhesion between the concrete of the socket and the surrounding rock. The allowable load capacity is given by:

$$Q_a = \frac{\pi\, D\, H_s\, \tau_a}{2000}$$

where Q_a = allowable load on pile, ton,

D = pile diameter, in.,

H_s = depth of socket in sound rock, in.,

and τ_a = allowable bond strength between concrete and rock, lb/sq in.

The available bond strength τ_a is a function of the strength of concrete and rock as well as of the quality of the contact area resulting from the excavation process. τ_a is generally higher than the bond strength normally considered in concrete design due to the Poisson's effect in the confined concrete socket.

Design values of 100 to 300 lb/sq in. are used but much lower values have been observed on actual sockets where the construction process had produced a poor contact area.

The application of this design method is based on the assumption that the walls of the socket are of sound rock, unshattered by the excavation process and are clean from any drilling mud or smear. Experience shows that this is difficult to achieve particularly in sedimentary rocks. The design method should therefore be used with great caution and a careful visual inspection of the rock socket before concreting is mandatory. Furthermore, and to ensure the safety of the design it is recommended that the load capacity Q_a determined by this method be limited to the maximum value resulting from method (2) or (3).

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MENARD, L., 1965. Règles pour le calcul de la force portante et du tassement des fondations en fonction des résultats pressiométriques. *Proc. Internat. Conf. Soil Mech. Found. Eng., 6th, Montreal, 2: 285-299.*

TAVENAS, F.A., 1971. Contrôle du roc de fondation de pieux forés à haute capacité. *Can. Geotech. J., 8: 400-416.*

7.2.1.3. *Settlements*

Settlement analysis of piles sitting on or socketed in rock is very difficult and frequently unreliable because of the discontinuous nature of rock masses.

In general, in sound rock, settlements are minute and can be neglected. Important rock settlements are generally associated with the presence of open joints in the rock mass and, in sedimentary rocks, with the occurrence of seams of compressible material. Where such conditions are expected to exist special investigations and analysis are required and should be carried out by a person competent in this field of work.

Settlements may also result from the presence of debris between the bottom of the concrete shaft and the rock surface. Careful inspection of the bottom of each excavation is necessary to eliminate this problem especially in the case where the deep foundation has been designed according to 7.2.1.2. (2) or (3).

In some cases, such as for deep foundations of large dimensions or those carrying high loads, a settlement analysis may be desirable. Three methods are available.

(1) *Settlements from tests on rock cores*

Elastic moduli measured on rock core samples have little relation to the actual settlement behavior of rock masses, since the influence of joints and other rock discontinuities is neglected. A settlement analysis based on such moduli must include arbitrary assumptions on the influence of joints, and is therefore of limited practical value.

(2) *Settlements from pressuremeter tests*

Settlements can be estimated on the basis of *in situ* pressuremeter tests. To do so, a large number of tests must be performed to allow for an assessment of the variability of elastic moduli of the rock mass, including some measure of the influence of joints and other discontinuities. In first approximation the settlement is given by:

$$S = \frac{q_d D}{9 \alpha_m E_p}$$

where S = settlement, in.

q_d = design pressure, ton/sq ft

D = tip diameter of the pile, in.

E_p = average pressuremeter modulus in the zone extending 3 diameters below the pile tip, ton/sq ft

α_m = a coefficient which is a function of the structure of the rock mass as follows:

Spacing of Discontinuities	>10 ft	3 to 10 ft	1 to 3 ft	3 in to 1 ft
α_m	1	0.75	0.5	0.25

This method is applicable to homogeneous as well as to stratified rock masses. In the latter case the modulus to be used in the formula is taken as a weighted average of the moduli measured in the different strata, provided the moduli do not differ by more than a factor of 10.

The effect of thin horizontal joints or compressible seams cannot be taken into account in this method and the results may be misleading if such joints or seams occur.

(3) *Settlements from plate load tests*

In situ plate load tests may be used to assess the settlement behaviour of a rock mass under a deep foundation.

The importance of size effects on the results of such tests should be recognized. Ideally the plate should be of the same diameter as the deep foundation. For practical reasons, however, this is seldom possible and smaller plates are generally used. The results obtained from loading smaller plates may generally be considered representative of the actual foundation behaviour provided the diameter of the plate is not less than half the diameter of the foundation, and is always in excess of 1 ft.

Plate load tests are difficult to carry out properly and results are frequently variable. To obtain a reliable evaluation of the foundation behaviour, series of tests have to be carried out. The cost of such tests and of the resulting design is high, and is, in general, only justified for projects of a very large size or when the structure to be supported is very sensitive to settlements.

The performance and interpretation of such plate load tests should be carried out by a person competent in this field of work.

7.2.2. *PILES IN GRANULAR SOILS*

7.2.2.1. *General*

The following paragraphs cover the design of all kinds of piles embedded in granular soils, i.e. gravels, sands, and non-cohesive silts. The design methods described are applicable only to unstratified deposits where granular soils extend to a significant depth beneath the lowest part of the deep foundation or to layered deposits where granular soils are underlain by more competent materials such as tills or rock.

In cases of layered deposits where granular soils are underlain by compressible materials the design methods described in 7.2.4. should be used.

Piles in granular soils derive their load carrying capacity from both point resistance and shaft friction. The relative contributions of point resistance and shaft friction to the total capacity of the pile depend essentially on the density and shear strength of the soil and on the characteristics of the pile.

It is usual to distinguish between a displacement pile, which is driven into the soil and displaces a volume of soil equal to its overall volume and a non-displacement pile, where a volume of soil equal to that of the pile is removed by excavation before the pile is placed. It is generally considered that a displacement pile has an intrinsically higher bearing capacity but none of the available design methods takes this consideration into account.

7.2.2.2. *Allowable Load On A Single Pile*

(1) *Method based on the standard penetration test*

(a) *Ultimate bearing capacity*

The ultimate bearing capacity of a single pile in granular soils may be determined from the results of the Standard Penetration Test as suggested by MEYERHOF, 1956.

$$Q_f = 4\,N\,A_p + \frac{\overline{N}\,A_s}{50}$$

where
Q_f = ultimate pile load, ton
N = average standard penetration index at the pile tip elevation, blows/ft
A_p = cross-sectional area of the pile tip, sq ft
$\overline{N}$ = average standard penetration index along the pile shaft, blows/ft, and
A_s = surface area of the pile shaft, sq ft

(b) *Factor of safety*

The Standard Penetration Test is subject to many errors (See Commentary 8.1) and much care must be exercised when using the test results. For this reason a minimum factor of safety of 4 should be applied to Q_f. The allowable load capacity of a pile is therefore:

$$Q_a \leqslant \frac{Q_f}{4}$$

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MEYERHOF, G.G., 1956. Penetration tests and bearing capacity of cohesionless soils. *J. Soil Mech. Found. Div., Am. Soc. Civil Engrs., 82: SM1, Paper No. 866.*

FLETCHER, G.F.A., 1965. Standard Penetration Test: its uses and abuses. *J. Soil Mech. Found. Div., Am. Soc. Civil Engrs., 91: SM4, 67-75.*

(2) *Method based on the theory of plasticity*

The allowable load on a single pile in a granular soil may be determined from the friction angle of the soil by use of the theory of plasticity (or bearing capacity theory).

(a) *Critical depth*

The bearing capacity of a pile in granular soil is not a continuous linear function of the overburden pressure. It has been demonstrated by

VESIC (1970) that both the point resistance and the skin friction become constant below a critical depth H_c, which, for all practical purposes, is equal to:

$$H_c = 20D$$

where D is the diameter of the pile.

(b) *Ultimate point resistance*

For piles with a length in granular soil less than H_c the ultimate point resistance is given by:

$$q_{fp} = \gamma L_p N_q^*$$

where q_{fp} = ultimate point resistance, lb/sq ft

γ = effective unit weight of the soil, lb/cu ft

L_p = length of the pile in soil, ft

N_q^* = a bearing capacity coefficient for piles as derived from BEREZANTSEV (1961). N_q^* is given as a function of the angle of shearing resistance ϕ of the soil as follows:

ϕ^o	25^o	30^o	35^o	40^o
Soil density	*loose*	*compact*	*dense*	
N_q^*	15	30	75	150

Considering the exponential increase of N_q^* with ϕ the selection of a design value of ϕ should be made with caution.

For lengths of piles in excess of H_c, the ultimate point resistance is constant and equal to:

$$q_{fp} = \gamma H_c N_q^*$$

(c) *Skin friction*

The ultimate skin friction acting on a pile of length L_p is related to the ultimate point resistance by the formula:

$$f_f = \frac{q_{fp}}{\alpha_\phi}$$

where α_ϕ = a coefficient defined by VESIC (1970). α_ϕ is given as a function of the angle of shearing resistance ϕ of the soil as follows:

ϕ	25^o	30^o	35^o	40^o
Soil density	*loose*	*compact*	*dense*	
α_ϕ	35	50	75	110

(d) *Factor of safety*

The factor of safety to apply to q_{fp} and f_f should be at least equal to 3.

The resulting allowable load on a single pile with a diameter D and a length L_p, is computed as follows:

— *for* $L_p < H_c$

$$Q_a = \frac{1}{3}\left[q_{fp} \frac{\pi D^2}{4} + \frac{f_f}{2} \pi D L_p\right]$$

where q_{fp} and f_f are computed at depth = L_p

— *for* $L_p > H_c$

$$Q_a = \frac{1}{3}\left[(q_{fp} \frac{\pi D^2}{4}) + (\frac{f_f}{2} \pi D H_c) + f_f \pi D (L_p - H_c)\right]$$

where q_{fp} and f_f are computed at depth $H_c = 20D$.

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BEREZANTSEV, V.G., KRISTOFOROV, V.S. and GOLUBKOV, V.N., 1961. Load bearing capacity and deformation of piles foundations. *Proc. Internat. Conf. Soil Mech. Found. Eng., 5th, Paris, 2; 11-15.*

VESIC, A.S., 1970. Tests on instrumented piles, Ogeechee River site. *J. Soil Mech. Found. Div., Am. Soc. Civil Engrs., 96: SM2, 561-584.*

(3) *Method based on static penetration tests*

The allowable load on a pile in granular soil can best be computed from the results of static cone penetration tests (Dutch cone). The test is best suited for silts and sands that are *loose* to *dense*. It is difficult to carry out in coarse gravels and in sands that are *very dense*.

(a) *Ultimate load capacity*

The ultimate load capacity of a single pile in granular soil may be determined from:

$$Q_f = R_p A_p + 2 F_c A_s$$

where Q_f = ultimate pile load, ton

R_p = point resistance from cone tests, ton/sq ft. (It is recommended that for piles with D > 18 in. a design value of R_p less than the measured average R_p and equal to the minimum measured R_p be used).

A_p = cross-sectional area of the pile tip, sq ft,

F_c = average skin friction measured by cone tests, ton/sq ft. (The use of a cone equipped with a friction sleeve is recommended).

A_s = surface area of the pile shaft, sq ft.

(b) *Factor of safety*

The results of static cone penetration tests are more reproducible than those of the Standard Penetration Test and a greater confidence can be put in the design method based upon them.

The factor of safety to apply to Q_f should be between 2.5 and 3 depending on the number of cone tests performed and on the observed variability of the test results; the minimum factor of safety corresponding to a large number of results with a variability of less than ± 10% of the average.

REFERENCES

VAN DER VEEN, C. and BOERSMA, L., 1957. The bearing capacity of a pile predetermined by a cone penetration test. *Proc. Internat. Conf. Soil Mech. Found. Eng., 4th, London, 2: 72-75.*

De BEER, E.E., 1963. The scale effect in the transposition of the results of deep sounding tests on the ultimate bearing capacity of piles and caisson foundations. *Geotechnique, 13: 39-75.*

(4) *Method based on load tests*

The design of piles on the basis of theoretical or empirical methods, as described above, are subject to some uncertainties:

- soil properties cannot be measured with great accuracy and are always variable within a building site;
- the correlations between the soil parameters and the bearing capacity of a pile include a margin of error;
- the actual driving or placing conditions vary from pile to pile and cannot be properly taken into account.

Therefore, the best method of assessing the bearing capacity of piles is to load test typical units.

General considerations on the use of load tests, the recommended methods of testing and interpreting the test results are given in 7.4 LOAD TESTS ON DEEP FOUNDATIONS.

For piles in granular soils, it is recommended that test Method A as described in 7.4 be used and that a factor of safety of 2.0 to 2.5 be applied to the ultimate pile capacity. Selection of the appropriate factor of safety will depend on the observed settlement behaviour of the tested pile and on the toleration to settlements of the structure to be supported.

(5) *Compacted concrete piles*

Compacted concrete piles in granular soils derive their bearing capacity from the densification of the soil around the base. The bearing capacity of such piles is therefore entirely dependent on the construction method and can only be assessed from load tests and from well documented local experience.

(6) *Relaxation and freeze*

In some granular soils the ultimate capacity of driven piles is subject to changes with time following driving. In fine-grained soils such as non-cohesive silts and fine sands the ultimate pile capacity may decrease after driving. This effect is known as *relaxation*. It should be taken into account in design and load tests should be carried out only after a sufficient delay following driving.

In sands and coarse grained materials the ultimate load capacity of piles may increase after driving. This effect is known as *freeze*. It may be recognized by means of re-driving tests but it can be taken quantitatively into account in design only when it has been investigated by load tests. The effect of *freeze* should be treated with great caution in large pile groups.

REFERENCE

YANG, N.C., 1970. Relaxation of piles in sand and inorganic silt. *J. Soil Mech. Found. Div., Am. Soc. Civil Engrs., 96: SM2, 395-410.*

7.2.2.3. *Allowable Load On A Pile Group*

It is common practice to define the allowable load on a pile group as the sum of the allowable loads of the individual piles in the group. However, it is known that piles in groups in granular soils develop a larger load capacity than isolated piles: their *group efficiency*, defined as the ratio of the ultimate load capacity of a pile in a group to that of the same pile when isolated, is greater than 100%. Where it would be necessary to take this effect into account in design, the influence of pile spacing and pile cap should be considered.

(1) *Influence of spacing and pile cap*

(a) *Spacing*

Piles in groups:

- act as individual piles at spacing greater than seven times the average pile diameter,
- act as a group at spacing varying from 2.5 to 7 times the average pile diameter,
- should not be installed at spacing less than 2.5 times the average pile diameter.

(b) *Pile cap*

The pile cap on top of a pile group may be in contact with the soil or above the soil surface. Experience has shown that a pile cap in contact with the soil develops a bearing capacity which increases the apparent group efficiency.

(c) *References*

VESIC, A.S., 1969. Experiments with instrumented pile groups in sand. *Am. Soc. Test. Matls. Spec. Tech. Publ. 444. Performance of deep foundations, 177-222.*

KISHIDA, H. and MEYERHOF, G.G., 1965. Bearing capacity of pile groups under eccentric loads in sand. *Proc. Internat. Conf. Soil Mech. Found. Eng., 6th, Montreal: 2, 270-274.*

7.2.2.4. *Settlement Of A Single Pile*

Many factors that cannot be included in theoretical analysis influence the actual settlement of piles, with the result that estimates based only upon considerations of the elastic properties of the soil and pile material are generally so inaccurate as to be of no practical value. Instead, estimates of settlements of piles are based upon empirical relationships.

(1) *Empirical methods*

Experience has shown that the settlement of a single pile in granular soils is a function of the ratio of applied load to ultimate load capacity and of the diameter of the pile.

(a) *Displacement piles*

For normal load levels, the settlement of a pile may be estimated from the empirical formula (VESIC 1970):

$$S = \frac{D}{100} + \delta$$

where
S = settlement of pile head, in.
D = pile diameter, in.
δ = elastic deformation of pile shaft, in. For the purpose of this analysis it is common practice to assume:

$$\delta = \frac{Q\ L_p}{A\ \ E}$$

where
Q = applied pile load, lb.
A = average cross-sectional area of the pile, sq in.
L_p = length of the pile, in.
E = modulus of elasticity of the pile material in lb/sq in.

(b) *Non-displacement piles*

Limited experience has shown that the settlement of non-displacement piles may be four times larger than that of displacement piles under similar conditions.

REFERENCES

SKEMPTON, A.W., YASSIN, A.A. and GIBSON, R.E., 1953. Théorie de la force portante des pieux dans le sable. *Ann. Inst. Tech. Bâti. Travaux Pubs, 63-64, 285-561.*

VESIC, A.S., 1970. Tests on instrumented piles, Ogeechee river site. *J. Soil Mech. Found. Div., Am. Soc. Civil Eng., 96, SM2, 561.*

(2) *Settlement from load tests*

Since time effects are usually negligible in granular soils, the settlements observed during load tests conducted according to method A described in 7.4, can be considered as representative of the long term behaviour of the pile.

7.2.2.5. *Settlement Of A Pile Group*

The settlement of a pile group is evaluated on empirical bases and the methods are less reliable than those used for single piles because of the limited data that are available. It is recommended that the settlement of a pile group be evaluated on the basis proposed by SKEMPTON et al (1953).

(1) *Skempton's method*

The settlement of a pile group S_{group} is always larger than that of the individual piles forming the group.

$$S_{group} = \alpha_g S$$

where S = settlement of a single pile under its allowable load.

α_g = group settlement ratio; a function of the dimension of the group and of the pile spacing, or of the ratio B/D of the width of the pile group to the diameter of the piles as follows:

B/D	1	5	10	20	40	60
α_g	1	3.5	5	7.5	10	12

REFERENCE

SKEMPTON, A.W., YASSIN, A.A., GIBSON, R.E., 1953. Théorie de la force portante des pieux dans le sable. *Ann. Inst. Tech. Bâti. Travaux Pubs., 63-64, 285-290.*

7.2.3. *PILES IN COHESIVE SOILS*

7.2.3.1. *General*

(1) *Limitations of design methods*

Design methods for piles in cohesive soils are in some cases of doubtful reliability. This is particularly so for the bearing capacity of friction piles in clays of medium to high shear strength. Therefore, the design methods described in this section should be used with caution and essentially only for;

— the preliminary design of large foundations. In this case *in situ* full scale load tests should be performed as part of the final design or at the beginning of construction.

— the design of small foundations, provided adequate safety factors are used.

Settlements of groups of friction piles in clay are estimated by means of the methods normally used for shallow foundations with an additional empirical assumption concerning the transfer of load from the piles to the soil (see Chapter 6). Consequently, settlement estimates will be reliable only in terms of an order of magnitude. Differential settlements are difficult to predict.

(2) *Disturbance caused by driving*

Piles driven into cohesive soils induce some disturbance which is a function of;

— the soil properties, particularly sensitivity,

— the geometry of the pile foundation (diameter of piles, number and spacing of piles in the groups), and

— the driving method and sequence.

This disturbance results in loss of strength of the soils and consequently in reduction of support provided to the piles.

In some cases, such as in soft sensitive clays, complete remoulding of the clay may occur with the result that further construction becomes impossible.

The effect of disturbance diminishes with time following driving as the soil adjacent to the pile consolidates. This results in an increase in the bearing capacity of the pile. This phenomenon is substantially influenced by the pile material: consolidation and gain in strength are limited in amount and develop at a slow rate for steel piles; they are maximum and develop within a few weeks for timber piles.

Load testing of a pile in clay should not be carried out without an awareness of these processes. It is advisable not to load test within two weeks of driving.

REFERENCES

CUMMINGS, A.E., KERKHOFF, G.O. and PECK, R.B., 1950. Effect of driving piles into soft clays. *Trans. Am. Soc. Civil Engrs., 115: 275-285.*

EIDE, P., HUTCHINSON, J.N. and LANDVA, A., 1961. Short and long term loading of a friction pile in clay. *Proc. Internat. Conf. Soil Mech. Found. Eng., 5th, Paris, 2: 45-53.*

FLAATE, K., 1972. Effects of pile driving in clay. *Can. Geotech. J. 9: 81-88.*

CLARK, J.I. and MEYERHOF, G.G., 1972. The behaviour of piles driven in clay. I. An investigation of soil stress and pore water pressure as related to soil properties. *Can. Geotech. J., 9: 351-373.*

(3) *Pore water pressures induced by driving*

Driving piles in clay generates high pore water pressures, the effect of which is to:

- temporarily reduce the bearing capacity of the piles,
- affect the process of reconsolidation of the clay around the pile thereby making it necessary to delay the application of the load.
- alter the natural stability conditions in sloping ground. (Examples exist of landslides triggered by pile driving operations).

As demonstrated by LO and STERMAC (1965), pore water pressures at the end of driving can, in first approximation, be assumed equal to the effective initial overburden pressure along the full length of the pile within a ring equal in width to the pile diameter. As reconsolidation of clay around the pile occurs the high pore water pressures are diminished by gradual redistribution of stresses to the less disturbed soil further from the pile.

REFERENCE

LO, K.Y. and STERMAC, A.G., 1965. Induced pore pressures during pile driving operations. *Proc. Internat. Conf. Soil Mech. Found. Eng., 6th, Montreal, 2: 285-289.*

7.2.3.2. *Allowable Load On A Single Pile*

Piles in cohesive soils generally derive their load capacity from shaft adhesion or friction. However, in very stiff clays or in cohesive tills, a substantial point resistance may be mobilized which, for large diameter bored piles, may represent the total bearing capacity of the pile.

(1) *Total stress vs effective stress approach*

Until recent times, it was the general practice to evaluate the bearing capacity of piles in clay from a total stress approach, i.e. on the basis of the undrained shear strength c_u of the clay. Empirical correlations between c_u and the point resistance and skin friction on a pile have been developed, but these have not proved entirely reliable, particularly for c_u in excess of 500 lb/sq ft and analysis in terms of effective stresses appear more rational.

(2) *Driven piles in clays where c_u < 2000 lb/sq ft*

A pile driven in clay with an undrained shear strength of less than 2000 lb/sq ft derives its load capacity almost entirely from shaft adhesion or friction.

(a) *Ultimate capacity in terms of total stresses*

It is common practice to determine the ultimate load capacity of a single pile from the formula:

$$Q_f = c_{ua} A_s$$

where Q_f = ultimate load capacity, lb
A_s = surface area of pile shaft, sq ft
c_{ua} = unit adhesion strength, derived from c_u as in Fig 7.1.

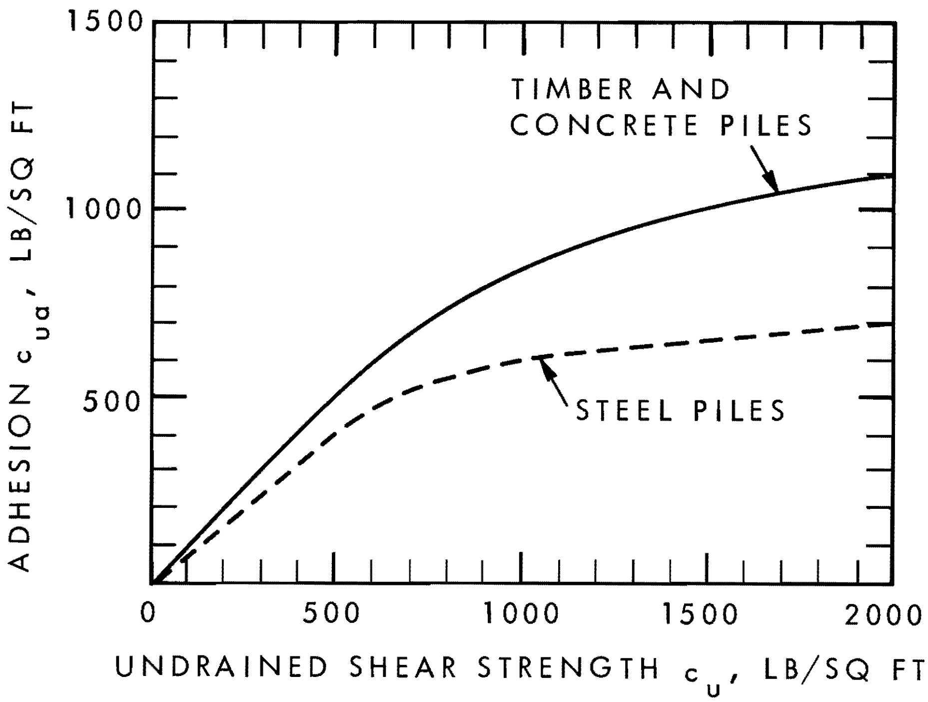


FIG 7.1 (After Tomlinson)
ADHESION ON PILES

The values of c_{ua} are empirical and actual adhesion may differ significantly from these values depending on the geometry of the foundation, the driving method and sequence, the properties of the clay and time effects. The ultimate capacity of piles resulting from the above formula should be confirmed by load tests.

(b) *Ultimate capacity in terms of effective stresses*

Recent investigations suggest that the ultimate load capacity of a single pile in clay may be derived from:

$$Q_f = A_s \tau_{s\ avg}$$

where Q_f = ultimate load capacity, lb

A_s = surface area of pile shaft, sq ft

$\tau_{s\ avg}$ = average effective shaft friction, lb/sq ft

$\tau_{s\ avg}$ is computed from the shaft friction τ_s at various depths along pile shaft.

$$\tau_s = p_o' K_o \tan \delta_f$$

where p_o' = effective overburden pressure at the considered depth, lb/sq ft

K_o = *at rest* earth pressure coefficient

δ_f = effective angle of friction between the clay and the pile shaft.

This method requires that K_o and δ_f be known. Both parameters are difficult to measure. However, available test results indicate that, for clays with c_u less than 2000 lb/sq ft, which are not heavily over-consolidated, the factor ($K_o \tan \delta_f$) varies only from 0.25 to 0.40. For design purposes a typical value of 0.3 may be used, so that:

$$\tau_s = 0.3\ p_o'$$

It is recommended that the calculated ultimate pile capacity be confirmed by load tests.

(c) *Factor of safety*

To obtain the allowable load capacity of the pile, from the ultimate capacities as given in (a) or (b) above, it is recommended that a factor of safety of at least 2.5 be applied provided load tests are carried out during construction of the foundation. In cases where no load tests are performed, a factor of safety of at least 3.0 should be applied.

REFERENCES

TOMLINSON, M.J., 1957. The adhesion of piles driven in clay soils. *Proc. Internat. Soc. Soil Mech. Found. Eng., 4th, London, 2: 66-71.*

CLARK, J.I. and MEYERHOF, G.G., 1973. The behaviour of piles driven in clays. II. Investigation of the bearing capacity using total and effective strength parameters. *Can. Geotech. J., 10: 86-102.*

EIDE, O., HUTCHINSON, J.N. and LANDVA, A., 1961. Short and long term test loading of a friction pile in clay. *Proc. Internat. Soc. Soil Mech. Found. Eng., 5th, Paris, 2: 45-53.*

BURLAND, J.B., 1973. Shaft friction of piles in clay - a simple fundamental approach. *Ground Eng., 6: 3, 30-42.*

(3) *Driven piles in clays where c_u > 2000 lb/sq ft*

A pile driven in clay with an undrained shear strength in excess of 2000 lb/sq ft derives its bearing capacity from both shaft adhesion or friction and point resistance.

The shaft friction of such a pile however, cannot be predicted with any degree of reliability because little is known of the effect of driving on the adhesion and on the final effective contact area between clay and pile.

In this case it is suggested that:

— tapered piles be used to ensure a maximum contact area between soil and pile,

— the ultimate bearing capacity be determined by pile loading tests during design.

(4) *Bored piles in clays where c_u > 2000 lb/sq ft*

Large diameter bored piles with or without enlarged or *belled* bases are successfully used in clays or cohesive tills where $c_u > 2000$ lb/sq ft. They derive their load carrying capacity from both shaft adhesion or friction and point resistance. Present design methods have been derived from extensive studies on bored piles in London clays. Considering the unusual properties of these soils, the generalization of empirical design parameters to other types of cohesive soils should be made with caution.

(a) *Shaft adhesion in terms of total stresses*

The ultimate load, based upon adhesion between the clay and the pile shaft, may be obtained from:

$$Q_{fs} = c_{ua} A_s$$

where Q_{fs} = ultimate shaft resistance, lb

A_s = surface area of pile shaft, sq ft

c_{ua} = ultimate adhesion, lb/sq ft. Experience shows that:

$$c_{ua} = 0.3 \text{ to } 0.4 \quad c_u$$

The actual value of c_{ua} is greatly affected by the excavation process which may cause remoulding or softening of the clay, and by the structure of the clay such as its degree of fissuring. It is recommended that c_{ua} be determined from the minimum undrained shear strength c_u, and that it be limited to a maximum of 2000 lb/sq ft.

(b) *Shaft adhesion in terms of effective stresses*

The same approach and formula as given in 7.2.3.2. (2)(b) may be applied here. However, the earth pressure coefficient K_o is highly dependent upon the geological history of a particular clay. It is therefore impossible to give typical values of (K_o tan δ), and the method may be applied only where K_o has been determined by appropriate methods or evaluated from load tests.

(c) *Point resistance*

The ultimate load that may be carried by point resistance may be estimated from:

$$Q_{fp} = N_c^* c_u A_p$$

where Q_{fp} = ultimate point load, lb

A_p = cross-sectional area of pile point, sq ft

c_u = minimum undrained shear strength of the clay at pile point level, lb/sq ft

N_c^* = a bearing capacity coefficient which is a function of the pile point diameter as follows:

Point Diameter,	N_c^*
Less than 18 in.	9
18 to 36 in.	7
Greater than 36 in.	6

In very stiff clays and tills where samples are difficult to retrieve and c_u is not easily measured, the pressuremeter method, as described in 7.2.1. may be used.

(d) *Allowable loads on bored piles*

The allowable loads on bored piles are determined from a combination of shaft adhesion and point resistance, after the application of appropriate factors of safety. The relative contribution of the shaft adhesion and the point resistance is a function of the rigidity of the pile and the compressibility of the clay around the shaft and below the base of the pile.

If the soil below the base has the same or greater compressibility than the soil around the shaft, the allowable load on the pile may be taken as;

$$Q_a = \frac{1}{2.5}\left(Q_{fs} + Q_{fp}\right)$$

If the soil below the base is less compressible than the soil around the shaft, the movements of the shaft relative to the soil will generally be too small to mobilize the full adhesion. In this case it is recommended that the allowable load on the pile be taken as;

$$Q_a = \frac{1}{2} Q_{fp}$$

While the above formulas may be considered as limiting cases, the decision to consider shaft adhesion in addition to base resistance must be made with care and only after properly designed and interpreted load tests are carried out. Such tests should indicate whether or not the resistance available is commensurate with strain both around the shaft and at the base, and any possibility of reduction in shaft resistance with time. The selection of the allowable load should be based upon permissible pile movement, as determined from these tests.

REFERENCES

WHITAKER, T. and COOKE, R.W., 1966. An investigation on the shaft and base resistances of large bored piles in London clay. *Proc. Symposium on Large Bored Piles, Inst. Civil Engrs., London, 7-49.*

SKEMPTON, A.W., 1959. Cast-in-place bored piles in London clay. *Geotechnique, 9: 153-173.*

(5) *Pile capacity from load tests*

The ultimate load capacity of piles in clays should be determined or confirmed by means of full scale load tests.

(a) *Method*

Load tests cannot be performed slowly enough for an evaluation of the time-settlement behaviour of piles in clays; only the ultimate load capacity may be determined. Under such conditions it is recommended that Method B described in 7.4. LOAD TESTS ON DEEP FOUNDATIONS be used. This method, known as the *constant rate of penetration method*, is best suited for a rapid and accurate evaluation of the ultimate pile capacity.

(b) *Factor of safety*

To obtain the allowable pile capacity a factor of safety of 2.5 should be applied to the ultimate pile capacity determined from 7.4.

7.2.3.3. *Allowable Load On A Pile Group*

(1) *Piles in clays where c_u < 2000 lb/sq ft*

When friction piles are driven in groups in clays with an undrained shear strength of less than 2000 lb/sq ft the ultimate load capacity of the group is usually less than the sum of the ultimate load capacities of the individual piles in the group. For spacings of 2.5 to 4 times the average pile diameter, the group efficiency can be taken to be equal to 70%.

Reference

WHITAKER, T., 1970. The design of piles foundations. *Pergamon Press, London.*

(2) *Piles in clays where c_u > 2000 lb/sq ft*

It is common practice to neglect group effects in the determination of the load capacity of pile groups in clays with c_u in excess of 2000 lb/sq ft.

7.2.3.4. *Settlement Of A Single Pile*

(1) *Piles in clays where c_u < 2000 lb/sq ft*

Piles in clays where c_u is less than 2000 lb/sq ft are seldom used singly but they act as single piles in groups where the spacing is in excess of 7 times the pile diameter and where the pile cap is not in contact with the soil. In this case limited field observations indicate that the settlement is due to local shear deformations along the pile shaft rather than to consolidation settlements, and is therefore very limited. If such cases occur it is recommended that special analyses, based on load tests be performed.

(2) *Piles in clays where c_u > 2000 lb/sq ft*

Because of their high load capacity, bored piles in stiff clays are often used as single piles.

The analysis of settlement of single piles in stiff clays is difficult at the present time because little data is available on the actual behaviour of such piles. Discussions on the validity of available methods of analysis are found in the references hereunder.

Where it is important to evaluate settlements the use of load tests, designed, carried out and interpreted by a person competent in this field is recommended.

REFERENCES

WHITAKER, T. and COOKE, R.W., 1966. An investigation on the shaft and base resistances of large bored piles in London clay. *Proc. Symposium on Large Bored Piles, Inst. Civil Engrs., London, 7-49.*

BURLAND, J.D., BUTLER, F.G. and DUNICAN, P., 1966. The behavior and design of large diameter bored piles in stiff clay. *Proc. Symposium on Large Bored Piles, Inst. Civil Engrs., London, 51-71.*

TROW, W. and BRADSTOCK, J., 1972. Instrumented foundations for two 43-storey buildings on till, Metropolitan Toronto. *Can. Geotech. J., 9: 290-303.*

7.2.3.5. *Settlement Of A Pile Group*

(1) *General*

As mentioned in 7.2.3.1. (1), settlements of groups of piles in clay are estimated by means of methods normally used for shallow foundations, after application of an additional empirical assumption concerning the transfer of load from the pile group to the soil. Total and differential settlement predictions will therefore be less reliable for pile groups than for footings.

(2) *Suggested method*

The following method, proposed by TERZAGHI and PECK (1948), and confirmed by limited field observations, is suggested for the evaluation of of the settlement of pile groups in clay. The load carried by the pile group is assumed to be transferred to the soil through a *theoretical footing* located at 1/3 the pile length up from the pile point (Fig. 7.2.). The load is assumed to spread within the frustrum of a pyramid of side slopes at 30° and to cause uniform additional vertical pressure at lower levels, the pressure at any level being equal to the load carried by the group divided by the cross-sectional area of the pyramid at that level. The settlement calculation then follows the method described in 6.5.7.

REFERENCES

TERZAGHI, K. and PECK, R.B., (1948)(1967). Soil mechanics in engineering practice. *J. Wiley and Sons, N.Y.*

BRZEZINSKI, L.S., 1969. Behaviour of an overpass carried on footings and friction piles. *Can. Geotech. J., 6: 369-382.*

7.2.3.6. *Negative Skin Friction*

(1) *General*

When a clay deposit, in which or through which piles have been installed, is subject to consolidation, the resulting downward movement of the clay around the piles induces downdrag forces on the piles. This force which tends to reduce the useable pile capacity is called *negative skin friction*.

Negative skin friction develops in cases where piles are placed in soil which is consolidating under an applied load, or where a fill is placed around an existing pile foundation. It develops in clay deposits subject to general subsidence resulting from lowering of the ground water table or other causes. It may also be generated by reconsolidation of the remolded clay layer around any driven pile. The magnitude and significance of *negative skin friction* in the design of piles in clays differs widely from case to case.

Negative skin friction is a pile capacity problem only in the case of a true end bearing pile on rock, where the pile capacity is generally controlled by its structural strength and where settlements of the pile are negligible. In all other cases of piles bearing in compressible soils, where the pile capacity is controlled by point resistance and shaft adhesion or friction, the problem of *negative skin friction* may be regarded as a settlement problem. See FELLENIUS (1972).

(2) *Magnitude of negative skin friction*

(a) *Present practice*

The most common method of computing *negative skin friction* τ_n is to assume:

$$\tau_n = c_{ua}$$

where c_{ua} is the adhesion as given in Fig 7.1.

(i) *Isolated piles*

For an isolated pile the total force F_n due to *negative skin friction* is therefore:

$$F_n = c_{ua} A_s$$

where A_s is the area of pile in contact with the settling clay layer.

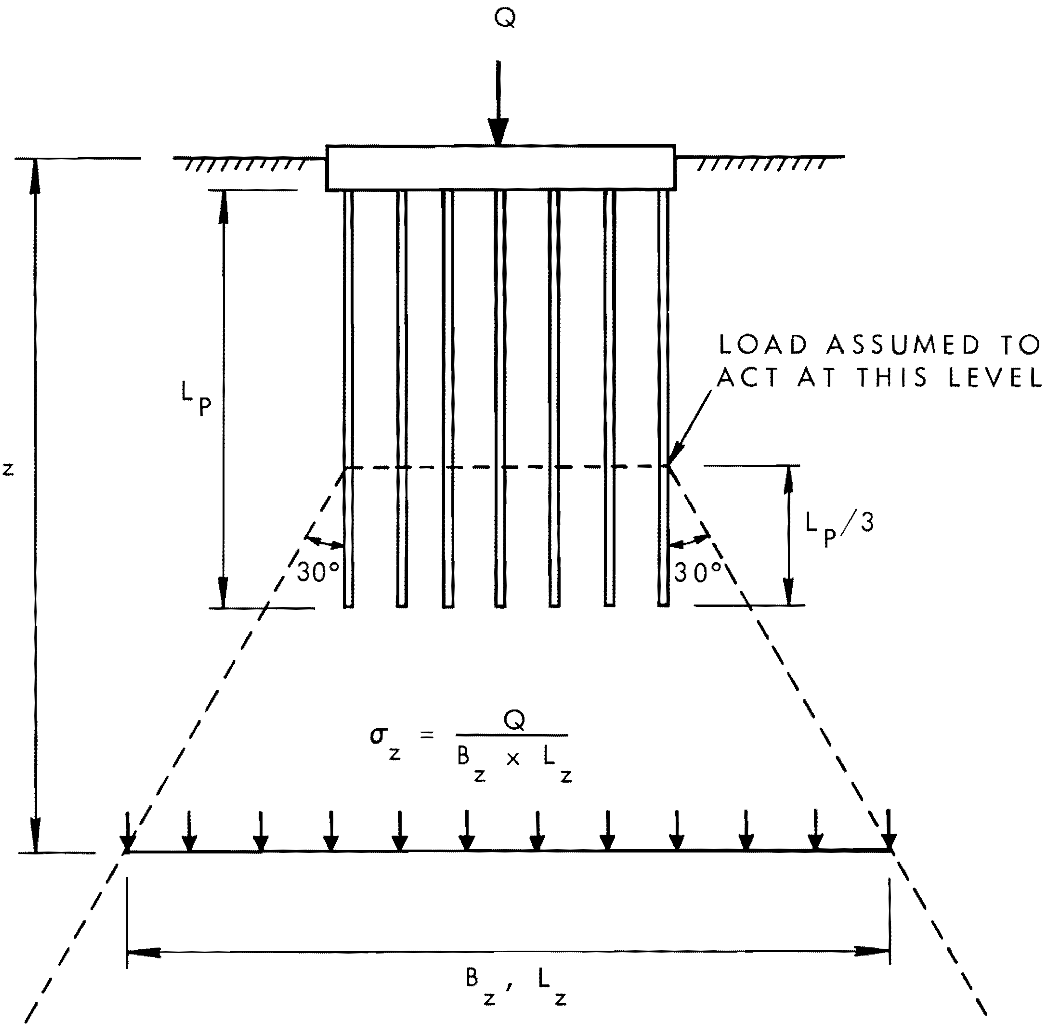


FIG 7.2

STRESS DISTRIBUTION BENEATH PILE GROUP IN CLAY USING THEORETICAL FOOTING CONCEPT

(ii) *Pile groups*

For pile groups the maximum force F_n on a pile is limited by the weight of clay between the piles so that:

$$F_n = c_{ua} A_s \leqslant S_p^2 H \gamma$$

where S_p = pile spacing, ft
H = thickness of the clay layer, ft
γ = unit weight of clay, lb/cu ft.

(b) *Recommended practice*

Field observations on instrumented piles have shown that the magnitude of negative skin friction is a function of the effective stress acting on the pile and may be expressed as:

$$\tau_n = p_o' K \tan \delta_f$$

where p_o' = effective overburden pressure
K = coefficient of earth pressure equal to or greater than K_o
δ_f = effective angle of friction between the clay and the pile material.

For all practical purposes it can be assumed that:

$$\tau_n = 0.25 \ p_o'$$

(3) *Means for reducing the negative skin friction*

For piles driven to rock the occurrence of *negative skin friction* means that a considerable increase of structural strength and bearing capacity above those needed to carry the building load will be required. *Negative skin friction* acting on driven piles may be reduced by the application of bituminous or other viscous coatings to the pile surfaces or in the case of steel piles by using the electro-osmosis technique. For cast-in-place piles, floating sleeves have been used successfully. The choice of appropriate method and evaluation of its effectiveness in any particular case should be left with a person competent in this field of work.

REFERENCES

FELLENIUS, B.H., 1972. Down drag on piles in clay due to negative skin friction. *Can. Geotech. J., 9: 323-337.*

BOZOZUK, M., 1972. Downdrag measurements on a 160 ft floating pipe test pile in marine clay. *Can. Geotech. J., 9: 127-136.*

BJERRUM, L., JOHANNESSEN, I.J. and EIDE, O., 1968. Reduction of negative skin friction on steel piles to rock. *Proc. Internat. Soc. Soil Mech. Found. Eng., 7th, Mexico, 2: 27-34.*

7.2.3.7. *Special Problems*

(1) *Piles driven near slopes*

As discussed in 7.2.3.1.(3), driving piles in clay generates pore water pressures in the clay. After driving, these pore water pressures are distributed in the clay mass over a considerable distance from the piles. If piles are driven in the vicinity of a slope, the increase in pore pressure produced by driving may cause failure of the slope. This phenomenon must be taken into account in design, particularly in sensitive clays by;

- analysis of the stability of the slope before and after driving, and
- instrumentation of the clay layer for pore water pressure measurements during driving.

If necessary, pore water pressures can be reduced by;

- the use of proper driving techniques and sequences. (Pre-boring is an efficient way of reducing pore water pressures), and
- the use of drain strips attached to the surface of the piles.

(2) *Heave due to pile driving*

When piles are driven in clays, the volume of soil displaced by the pile generally causes a heave of the soil surface. The heave of adjacent piles may also occur, with a resulting loss of capacity of these piles. This problem is of particular significance when large pile groups are driven.

Experience has shown that the heaved volume at the ground surface is generally of the order of 40% to 60% of the pile volume. If such heave is unacceptable, pre-boring is the method usually applied to reduce it.

(3) *Piles in swelling clays*

Piles driven in swelling clays may be subjected to uplift forces in the upper active layer as the result of the swelling process. The effect of these forces on the structural integrity of the piles or on the deformations of the foundation must be taken into account in design by:

- neglecting the contribution to the bearing capacity of that part of the pile embedded in the active layer of swelling clay.
- ensuring that the uplift resistance of that portion of the pile located below the active layer of swelling clay is sufficient to withstand uplift forces generated in the swelling clay layer, and
- ensuring that the structural resistance of the pile is sufficient to withstand the uplift forces.

If necessary, uplift forces may be eliminated by isolating the piles from the swelling clay in the active layer. This can be achieved by the use of floating sleeves or of bituminous or other viscous coatings applied to the pile surface.

REFERENCES

BJERRUM, L. and JOHANNESSEN, I., 1961. Pore pressures resulting from driving piles in soft clay. *Proc. Conf. Pore Pressures and Suction in Soils, London.*

ORRJE, P. and BROMS, B., 1967. Effects of pile driving on soil properties. *Am. Soc. Civil Engrs., J. Soil Mech. Found. Div., 93: SM5, 59-74.*

7.2.4. *PILES IN LAYERED DEPOSITS*

7.2.4.1. *General*

Piles are commonly driven through a layer of soft soil to a competent stratum or through alternating layers of competent and non-competent soils. In such cases the pile foundation is generally designed in accordance with the methods described in 7.2.1. to 7.2.3. but with modifications contingent upon the prevailing subsoil conditions. In designing such piles particular attention should be paid to:

- the relative stiffnesses and strengths of the different layers penetrated by the piles. (This will lead to an evaluation of the probable relative contribution of these layers to the pile capacity), and

- the stratigraphy immediately below the pile tip which influences the stability and the settlement of pile groups.

7.2.4.2. *Allowable Pile Capacity*

The relative contribution of the various strata penetrated by a pile to the capacity of that pile is primarily a function of the relative stiffnesses of these layers and of the type of pile.

(1) *End bearing piles*

Piles extending through layers of weaker soils to a very competent stratum such as bedrock or very dense till or gravel should be assumed to derive their bearing capacities only from the resistance mobilized in this supporting stratum. Because of the comparatively high stiffnesses of the supporting stratum and the pile, the relative displacements of pile and soil in the upper layers are generally insufficient to mobilize any significant shaft friction.

Similarly, for compacted concrete piles it should not be assumed that any other resistance will be mobilized than that obtained at the compacted base.

(2) *Piles in a two-layer deposit*

It is generally assumed for piles extending through a layer of soft soil to some depth into a deep deposit of competent soil such as sand that their bearing capacities are derived from point resistance and skin friction only in the lower layer. The upper layer is considered to contribute to the pile capacity only by increasing the overburden pressure used in the computation.

In cases where the bearing stratum is granular soil the critical depth mentioned in 7.2.2.2. (2) is taken from the upper surface of that stratum.

(3) *Piles in a multi-layer deposit*

Piles driven through a multi-layer deposit may derive their load capacities from both skin friction and point resistance. However, the evaluation of the relative importance of skin friction and point resistance are difficult and may need to be confirmed by load tests.

Whenever possible, piles in multi-layer deposits should be driven to a layer of sufficient strength and thickness that it may be assumed that they derive their load capacity entirely from that layer. In such a case, the load capacity may be determined according to the methods given in 7.2.1. to 7.2.3. It is essential to check that the bearing layer extends below the proposed pile tip elevation to a depth sufficient to ensure safety against a punching failure of the bearing layer into a lower weaker material. Safety against a punching failure may be evaluated by the following empirical method.

The total load Q on the pile group is assumed to be transferred to the soil through a *theoretical footing* located at the base of the pile group. The load is assumed to be spread within the frustrum of a pyramid with side slopes at 30^{o}. The resulting stress q' at the upper limit of the lower weaker layer may then be calculated as shown in Fig 7.3. In the general case where this layer is of cohesive soil with an undrained shear strength c_u the margin of safety against a punching failure will be sufficient if:

$$q' \leqslant 3\ c_u$$

7.2.4.3. *Settlement Of Pile Groups*

The methods of evaluating settlements of pile groups given in 7.2.2. and 7.2.3. are applicable to groups in layered deposits provided the layer in which the pile tips are located extends to a depth at least equal to 3 times the width of the pile group below the base of the group.

Where alternating layers of compressible and non-compressible soils are present below the pile tips, the settlement is assumed to originate in the compressible layers only. The total load Q on the pile group is assumed to be transferred to and distributed in the soil as indicated in Fig 7.3. The stresses acting on the compressible layers below the pile tips are computed and the corresponding settlements are determined according to the method given in 6.5. This analysis usually leads to an over-estimate of the settlements.

7.2.5. *PILES SUBJECTED TO HORIZONTAL LOADS*

7.2.5.1. *General*

Horizontal loads or moments on a vertical pile are taken by the mobilization of resistance in the surrounding soils as the pile deflects. The lateral load capacity of the pile depends essentially on the relative stiffnesses of the pile and of the surrounding soil.

For cases of vertical piles subjected to small and transient horizontal loads it is common practice to assume that such piles can sustain horizontal loads of up to 10% of the allowable vertical load without special analysis or design features.

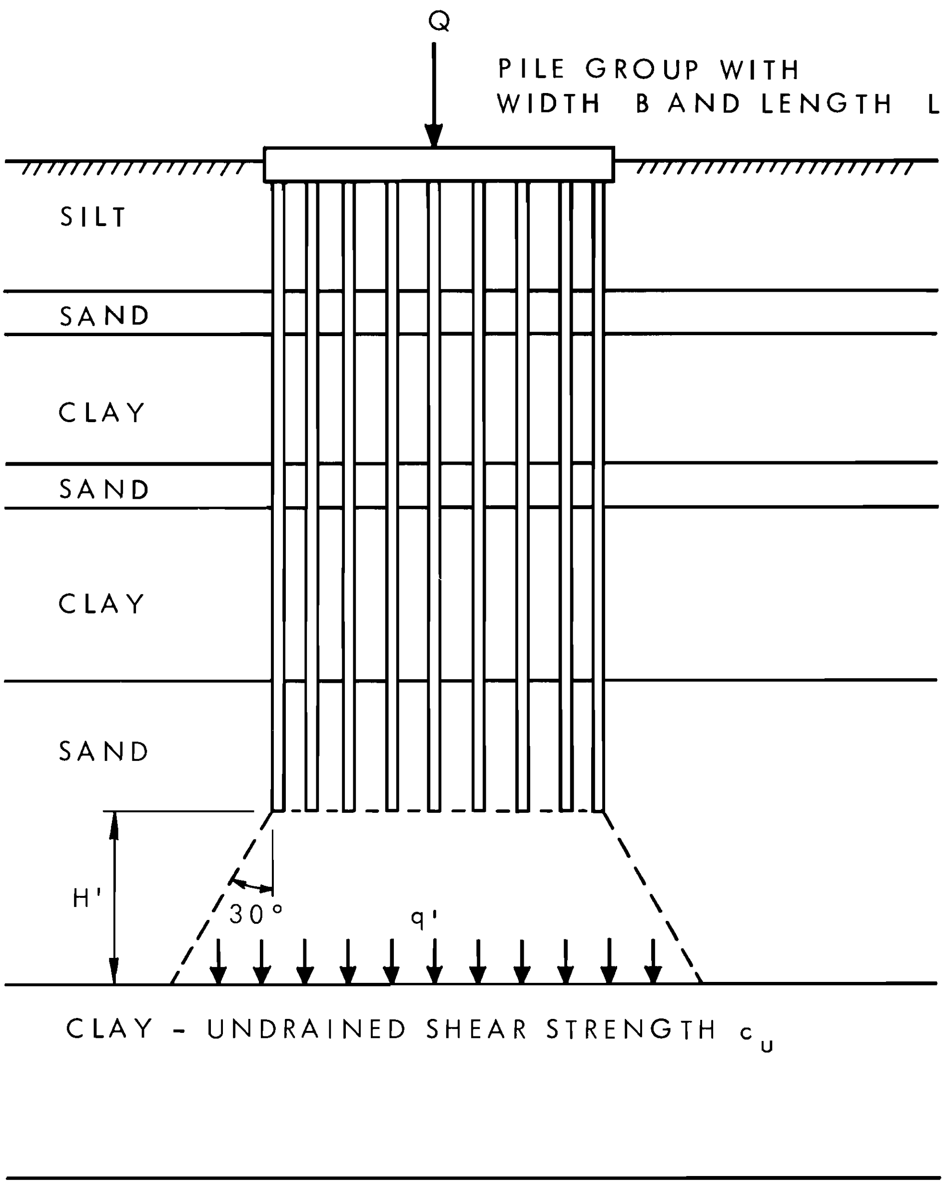
For cases where large transient or permanent horizontal loads must be resisted or where very soft soils occur it is common practice to install inclined piles to take horizontal loads. In some cases, however, large horizontal loads may be safely applied to vertical piles but the design of such piles is difficult. (See Commentary 8.6; THE DESIGN OF PILES SUBJECTED TO HORIZONTAL LOADS).

7.2.5.2. *Pile Groups With Inclined Piles*

In cases where the horizontal loads to be resisted exceed the horizontal load capacity of a group of vertical piles, or for piles installed in soils where this capacity is negligible, it is common practice to make use of inclined piles. For simple cases it is assumed that the horizontal loads are resisted by the horizontal components of the total load capacity of the inclined piles. However, for large loads a detailed analysis as described by CHELLIS (1961) is recommended.

It is important to note that:

- the slope of inclined piles is usually limited to 3 vertical to 1 horizontal because of installation problems.



$$q' = \frac{Q}{(B + 1.15H')(L + 1.15H')}$$

PILE GROUP IS SAFE AGAINST PUNCHING IF

$$q' \leq 3\,c_u$$

FIG 7.3

SAFETY OF PILE GROUPS AGAINST PUNCHING FAILURE

— when inclined piles are used, the horizontal load capacity of the vertical piles in the group cannot be considered to contribute to the horizontal resistance of the pile group because of the restraint of lateral movements provided by the inclined piles.

REFERENCE

CHELLIS, R.D., 1961. Pile foundations. *2nd Ed. McGraw Hill, New York.*

7.2.5.3. *Horizontal Load Capacity Of Vertical Piles*

The design of vertical piles subjected to large horizontal loads is difficult and should be carried out only by a person competent in this field of work.

(1) *Design based on theory*

Three different problems must be considered;

— safety against failure of the soil support.

— magnitude of the movements of the pile head and their influence on the behaviour of the superstructure, and

— magnitude of the bending moments in the pile and their influence on the structural behaviour of the pile.

Methods to analyse these problems have been developed which are summarized in Commentary 8.6; THE DESIGN OF PILES SUBJECTED TO HORIZONTAL LOADS.

The principal difficulty encountered in the application of these methods is proper evaluation of the necessary soil parameters. In particular the basic concept of subgrade reaction and the related coefficient of subgrade reaction K_s should be treated with great caution.

(2) *Design based on load tests*

The most reliable method of designing piles subjected to lateral loads is by means of load tests. However, such load tests are much more difficult to perform properly than vertical load tests. Consequently, they should be designed, carried out and interpreted by a person competent in this field of work.

The following points must be considered:

(a) The method of applying horizontal loads, by inserting horizontal jacks between the heads of two adjacent piles in a group or a row, is not acceptable unless the spacing between the piles is in excess of 10 pile diameters. At closer spacing there will be an interaction between the two piles and the load test results will be on the unsafe side (K_s and P_{ult} will be overestimated).

(b) In most cases it is not sufficient to measure the horizontal displacement of the pile head vs applied horizontal load. To allow for an appropriate evaluation of the elastic behaviour of the pile-soil system, and in particular of K_s, it is also necessary to instrument the pile for the measurement of bending stresses or deformations.

(c) Since horizontal loads applied by the structures are generally of a transient nature (wind loads earthquake, etc...) it is necessary to provide similar cyclic loading conditions in the tests.

7.2.6. *PILES SUBJECTED TO UPLIFT FORCES*

7.2.6.1. *General*

Pile foundations must sometimes resist uplift forces and should be checked both for their resistance to pullout and their structural ability to carry tensile stresses.

7.2.6.2. *Uplift Resistance Of A Single Pile*

Two cases must be considered.

(1) *Piles with straight shaft*

Piles usually have shafts of constant section. The ultimate uplift resistance of the pile is equal to the skin friction which can be mobilized along the surface area of the shaft. The skin friction is commonly assumed equal to that contributing to the bearing capacity of the pile as described in 7.2.2.2. and 7.2.3.2. The same factors of safety apply.

(2) *Piles with variable diameter*

When piles are built primarily to resist uplift forces it is common practice to increase the pullout resistance by providing one or more sections of a diameter larger than the average pile diameter: enlarged base piles, underreamed and multiunderreamed piles, and screw piles are typical.

For such piles, the ultimate pullout resistance is generated by skin friction along the shaft as well as by resistance mobilized above the sections of large diameter. This resistance may be taken equal to the point resistance as described in 7.2.2.2. and 7.2.3.2.

The same factors of safety apply.

(3) *Pile uplift capacity from load tests*

Where the uplift capacity of piles is important in the design of a building, it is recommended that this capacity be determined by means of full scale pullout tests, in which the effects of time can be taken into account. Such tests should be designed, carried out and interpreted by a person competent in this field of work.

The allowable uplift capacity should be determined from the ultimate pullout resistance by applying a factor of safety of 2.0.

7.2.6.3. *Uplift Resistance Of Pile Groups*

The uplift resistance of a pile group is the lesser of the two following values:

- the sum of the uplift resistances of the piles in the group,
- the sum of the shear resistance mobilized on the surface perimeter of the group plus the total weight of soil and piles enclosed in this perimeter.

7.3. STRUCTURAL DESIGN AND INSTALLATION OF DEEP FOUNDATIONS

7.3.1. *GENERAL*

The following paragraphs give information on the use of different types of deep foundations, including special features of structural design and important matters to be considered in the installation of such foundations.

These paragraphs have not been written as specifications although some parts may be suitable for such purposes.

7.3.1.1. *Structural Capacity Of Deep Foundations*

The structural capacity of a deep foundation unit, as resulting from Sentence 4.2.7.4. of the NBC and from considerations given here, represents the maximum load which could be carried by that deep foundation unit.

The allowable load however, will generally be less than the maximum structural capacity. This reduction is necessary for the following reasons:

— The actual placing of deep foundations frequently deviates from the position and alignment assumed in design; the actual stresses on any section of the deep foundation unit may therefore differ from the design stresses; and local over-stressing of the material may occur.

— Once in place, deep foundation units can neither be inspected nor repaired. This lack of serviceability should be reflected in the structural design by a reduction in capacity, particularly for cast-in-place deep foundations.

— The placement of concrete in cast-in-place deep foundations cannot be done with the same control as in structural columns: concrete is placed by tremie, sometimes to great depth, high slump concrete is used, and vibration of concrete cannot be applied.

— Finally, in most cases, the allowable load on a deep foundation unit is governed by geotechnical considerations: the geometry of the unit (length, cross-section) is determined to produce the necessary geotechnical capacity; the structural capacity corresponding to that geometry is generally in excess of the geotechnical capacity.

7.3.1.2. *Wave Equation Analysis*

In this method, the propagation of the stress wave generated by the impact of a given hammer in a pile is analysed taking into account the characteristics of:

— the hammer (weight, drop height or rated energy, impact velocity).

— the driving cap (weight, stiffnesses of the capblock and the cushion, coefficients of restitution of capblock and cushion).

— the pile (weight, stiffness, presence of joints or cracks).

— the soil (deformation characteristics represented by ground quake and damping factors for side friction and point resistance).

Representative values for these parameters can either be measured or taken from published data. (FOREHAND and REESE 1964). The method requires the use of a simple computer program which is readily available (EDWARDS 1967; BOWLES 1974). It can be used to advantage at three different stages of the design and installation of driven deep foundation units:

(1) *Driving stresses in piles*

It can easily be demonstrated that, for driven piles, the maximum stresses in the pile material are developed during driving. Therefore, the structural strength of the pile should be determined for the driving condition. The *wave equation analysis* is the only method available for evaluating the stresses generated in the pile material at different stages of driving. Its use is highly recommended, particularly for the structural design of precast concrete piles.

(2) *Selection of driving equipment*

The *wave equation analysis* is the only rational method for selecting the most appropriate hammer-capblock-cushion combination and the number of blows necessary to drive a given pile to a given load capacity in a given soil. Its use should be considered for large pile foundations or when large diameter piles have to be driven.

(3) *Bearing capacity of piles*

The *wave equation analysis* was developed and can most effectively be used to evaluate the bearing capacity of driven piles. The method yields a correlation between the number of blows per inch and the ultimate bearing capacity of the pile for any selected set of design assumptions concerning the hammer, the driving cap, the pile and the soil. From this correlation it is possible to pre-determine a *refusal-criterion* (minimum blows per inch necessary to ensure a given allowable load) and the probable depth at refusal, as well as to control the construction operations.

In a sense the use of the *wave equation analysis* is similar to that of pile driving formulas. However it is free of the serious fundamental errors involved in these formulas and is therefore much more reliable. (See Commentary 8.5.; THE USE OF PILE DRIVING FORMULAS).

The use of the *wave equation analysis* is therefore highly recommended for the prediction of load capacity of driven piles. Pile driving formula should not be used for that purpose, because of inherent fundamental errors.

REFERENCES

SMITH, A.E.L., 1960. Pile driving analysis by the wave equation. *J. Soil Mech. Found. Div., J. Soil Mech. Found. Div., Am. Soc. Civil Engrs. 86: SM4, 35-61.*

EDWARDS, T.C., 1967. Pile analysis wave equation computer program utilisation manual. *Texas Transportation Institute. Research Report 33-11, Texas A & M University.*

FOREHAND, P.W. and REESE, J.L., 1964. Prediction of pile capacity by the wave equation. *J. Soil Mech. Found. Div., J. Soil Mech. Found. Div., Am. Soc. Civil Engrs. 80: SM2, 1-25.*

BOWLES, J.E., 1974. Analytical and computer methods in foundations. *McGraw Hill, New York.*

7.3.2. *TIMBER PILES*

7.3.2.1. *Use Of Timber Piles*

Timber piles are:

- best suited for use as friction piles in sands, silts and clays because of their naturally tapered shape.
- not recommended for piles to be driven through dense gravel or till, or for end bearing piles to rock, since they are vulnerable to damage at the head and the tip in hard driving.
- commonly used for depths of 20 to 50 ft, for diameters of 8 to 16 in., corresponding to the natural dimensions of available tree trunks, and for design loads of 10 to 50 ton. Note that timber piles are difficult to splice.

7.3.2.2. *Materials*

Timber piles must conform with the requirements of Subsection 4.2.3. of the NBC.

They may be used untreated where they are entirely located below the permanent water table, and in this condition they are extremely resistant to decay, irrespective of the quality of groundwater.

Where untreated timber piles are exposed to soil or air above the permanent water table and in particular when they are subjected to intermittent submergence, they are very vulnerable to decay.

7.3.2.3. *Structural Design*

The structural design of timber piles must conform with the requirements of Subsection 4.2.7. of the NBC. No special consideration need to be given to handling or driving stresses, but special precautions must be taken to protect the pile tip and head from damage.

7.3.2.4. *Installation Of Timber Piles*

The only potential problem associated with the installation of timber piles is the splitting and *brooming* of the pile tip and head during driving.

To avoid this the following steps are recommended:

— the driving energy per blow (ft-lb) should be limited to:

$$1500\ D$$

where D is the diameter of the pile tip, in., and

— the pile head should be provided with protection in the form of a chamfer if easy driving is expected as in soft clays, or of a steel ring if hard driving is expected.

— the pile tip should be provided with protection in the form of a cone-shaped tip for easy driving, a steel ring for medium driving or a special steel point protection or *boot* for hard driving.

Even when these precautions are taken, timber piles cannot withstand very hard driving; overdriving will generally lead to the destruction of timber piles. To avoid this, it is recommended that:

— the maximum driving energy per inch (ft-lb) be limited to:

$$6000\ D$$

where D is the pile tip diameter, in., and

— that driving be stopped immediately when abrupt high resistance to penetration is encountered.

7.3.3. *PRECAST AND PRESTRESSED CONCRETE PILES*

7.3.3.1. *Use Of Precast And Prestressed Concrete Piles*

Because of the structural strength and wide choice of possible dimensions, precast and prestressed concrete piles can have a wide range of load carrying capacity. They are:

— best suited for high capacity piles in sand and gravel and for end bearing piles to rock.

— not recommended for piles subject to uplift forces unless special precautions are taken, nor for driving in soils containing large boulders.

— commonly used for depths of 30 to 45 ft for precast concrete piles without splicing device, 40 to 60 ft for prestressed concrete piles without splicing device, and unlimited depths for piles with splicing device.

Typical cross-sections are square with a width of 12 to 24 in., hexagonal with 10 to 24 in. across the flats, or cylindrical with diameters up to 54 in. (The larger diameter cylinders are usually hollow and prestressed).

Design loads vary over a wide range depending on the geometry of the pile, the strength of concrete and the amount of reinforcing steel or of prestressing.

7.3.3.2. *Materials And Fabrication*

Concrete piles must conform to the requirements of Subsection 4.2.3. of the NBC. In addition, the following special requirements should be considered:

(1) *Concrete*

Concrete used in precast and prestressed concrete piles should have a strength in excess of 5000 lb/sq in. Higher strength is desirable and concrete with strengths in excess of 7500 lb/sq in. is in common use. Such high strengths are necessary to reduce the risk of spalling or cracking during driving.

(2) *Steel*

Reinforcing steel should have a yield stress of at least 60,000 lb/sq in. for normal driving condition, and of 85,000 lb/sq in. when hard driving is expected. Longitudinal reinforcement should be made up of a minimum of 4 bars in square piles and 6 bars in hexagonal or cylindrical piles, spaced symmetrically. Spirals or ties are spaced 4 to 8 in. on centers in the middle of the pile length, but should be spaced no more than 3 in. on centers at each end of the pile for a length at least equal to three times the pile diameter. In order to reduce the risk of spalling the thickness of concrete cover protecting the reinforcing or prestressing steel is reduced to $1\frac{1}{2}$ in. for concrete with a strength at 28 day under 7000 lb/sq in. and to 1 in. for concrete with a strength at 28 day in excess of 7000 lb/sq in.

(3) *Forms*

Forms for precast and prestressed concrete piles must be extremely accurate to ensure perfect straightness of the piles, constant cross-sections and smooth surfaces. The form ends must be exactly perpendicular to the longitudinal axis to avoid destruction of the pile ends during driving.

7.3.3.3. *Pile Splices*

Since the length of precast concrete piles is limited by handling conditions, special pile splices have been developed to allow the construction of very long precast concrete piles. Quality requirements for concrete pile splices are stringent because of the determining influence of splices on the drivability of concrete piles. Pile splices are now produced by specialized manufacturers, and have been the object of extensive design review and testing. General requirements for splices are as follows:

— the strength of the splice must be at least equal to that of the pile in compression, tension or bending.

— the splice must be designed and positioned so as to ensure and maintain perfect alignment of the joined sections of piles.

— the splice must be designed so that the slack between two joined sections of a pile is less than 0.02 inch in either compression or tension. A slack in excess of this amount would produce significant loss of driving energy and impair the drivability of the pile.

7.3.3.4. *Structural Design*

The structural design of precast and prestressed concrete piles must conform with the requirements of Subsection 4.2.7. of the NBC.

(1) *Handling conditions*

The structural capacity of precast and prestressed concrete piles must be checked for the handling condition, where the pile is subjected to bending under its own weight. To allow for impact, it is common practice to compute handling stresses with a design weight equal to 150% of the actual pile weight.

To ensure proper handling it is common practice to provide the pile with handling hooks located according to the design assumptions.

Handling conditions govern the maximum length of precast and prestressed concrete piles.

(2) *Driving conditions*

Driving conditions generally govern the structural design of precast and prestressed concrete piles. Until recent years no design tools were available to check for these conditions and common practice was limited to a careful monitoring of the driving operations to ensure that no damage would occur to the visible portion of the pile. With the development of the wave equation analysis (See 7.3.1.2.) it was possible to evaluate the compressive and tensile stresses generated in concrete piles during driving and to design them to withstand such stresses.

For guidance, in cases where such analysis has not been performed, it has been established that maximum driving stresses in precast and prestressed concrete piles are about 150% of the static stresses corresponding to the achieved load capacity. In other words, to take driving stresses into account, it is recommended that the structural capacity determined from Sentence 4.2.7.4. of the NBC be multiplied by a reduction factor equal to 0.6.

(3) *Working conditions*

(a) *Precast concrete piles*

The design method and details given in CSA Standard A23.3 1973 'Code for the Design of Concrete Structures for Buildings' are those applicable to laterally supported compression members. However, for piles subjected to moments or horizontal loads in addition to vertical loads, the effects of such loads, as determined in 7.2.5., of this Manual, must be taken into account in the structural design of the piles.

(b) *Prestressed concrete piles*

Although design is governed by CSA Standard A23.3 it is recommended that the following formula be used to take into account the reduction of prestress due to the application of compression working loads:

$$P = A_c\phi\left[f'_c - \left(1.1 - \frac{\varepsilon_c E_s}{f_{so}}\right) f_{pe}\right]$$

where P = governing combination of loads multiplied by appropriate load factors as specified in CSA-A23.3, lb.

A_c = gross concrete section of the pile, sq in.

ϕ = capacity reduction factor as defined in CSA-A23.3

ε_c = strain in concrete at failure, assumed = 0.003

E_s = modulus of elasticity of prestressing steel, lb/sq in.

f_{so} = stress after losses in prestressing steel, lb/sq in.

f_{pe} = effective stress in concrete due to prestress after losses, lb/sq in.

f'_c = specified strength of concrete, lb/sq in.

For most practical cases the formula reduces to:

$$P = A_c\phi\left[f'_c - 0.6\, f_{pe}\right]$$

7.3.3.5. *Installation*

Driving of precast or prestressed concrete piles is difficult to perform properly requiring special driving equipment and extreme care. Two problems commonly arise:

- regular horizontal tension cracks may form in the early stages of driving when the resistance to penetration is low; and
- the pile tip or head may be crushed in compression under hard driving. To avoid such problems, the following information is given as a guideline.

(1) *Required quality of pile*

(a) *Structural integrity*

Piles designed and fabricated according to the recommendations in 7.3.3.2. to 7.3.3.4. will have the necessary qualities for successful driving. However, all piles should be carefully inspected before driving and damaged piles should be rejected.

Piles which have become fissured or spalled as the result of mishandling will generally be impossible to drive properly.

(b) *Pile head*

It is essential that the pile head be exactly perpendicular to the pile axis in order to avoid uneven distribution of impact forces. It is good practice to protect the pile head by means of a steel plate which should be at least ½ in. thick. The plate should be anchored into the reinforcing steel of the pile. The pile head should be encased with a steel collar connected to the head plate and extending to a depth equal to half the pile diameter. The plate and collar should be cast with the pile.

When easy driving conditions are expected, the pile head need only be chamfered at the edges and corners. In this case, it is important to ensure that no reinforcing steel or prestressing strands protrude from the head.

(c) *Pile tip*

In most cases the pile tip needs only be chamfered at the edges and corners.

When hard driving conditions are expected and in particular where piles are driven to end bearing on rock it is recommended that a special steel point be attached to the pile tip. The *Oslo Point* is a common type of tip protection; its characteristics are such that it can be chiselled into any type of rock to ensure proper seating.

(d) *Joints*

When joints are used the straightness of the pile across each joint should be checked as driving proceeds. With piles cast in horizontal moulds the face of the pile which was in contact with air during casting and curing has a different modulus of elasticity. This results in uneven dynamic deformations during driving, and, for long piles, in bending. To avoid this it is recommended that this face of the pile element be rotated 180° at each joint.

REFERENCE

REHNMAN, S.E. and BROMS, B.B., 1971. Bearing capacity of piles driven into rock. *Can. Geotech. J., 8: 2, 151-162.*

(2) *Driving hammers*

(a) *Types of hammers*

Drop hammers and diesel hammers are the most common types used for driving precast or prestressed concrete piles. Vibratory hammers are not recommended for precast or prestressed concrete piles because of the high tension stresses they generate.

(b) *Weight of hammer*

The selection of the appropriate weight of hammer is extremely important. In the absence of a wave equation analysis for such selection, it is recommended that a heavy hammer (weight at least equal to that of pile) be used. For the same weight a long hammer is more efficient than a short one.

(c) *Energy*

In the absence of a *wave equation analysis*, it is recommended that the hammer energy be limited to a maximum equal to 200 x $\sqrt{D}$ ft lb/sq in. of pile cross-section, where D is the pile diameter, ft.

Furthermore, it is recommended that the drop height of free fall hammers be limited to a maximum of 30 in. Higher drop heights result in higher impact velocities and unacceptable driving stresses.

To avoid the formation of tension cracks it is recommended that the ram velocity or drop height be reduced during early driving when little soil resistance is encountered, and in general when driving through soft soils. With reduced ram velocity the tensile stresses reflected from the pile tip can be kept within acceptable limits.

(3) *Driving cap*

(a) *Cap dimensions*

To avoid the transmission of torsion or bending forces, the driving cap or helmet should fit loosely but not so loosely as to prevent the proper alignment of hammer and pile.

(b) *Capblock*

A capblock must be placed on top of the driving cap to eliminate the damage caused by direct impact. The capblock must be of a material that attenuates the peak impact force and transmits the impact energy without excessive losses.

The most common material for a capblock is a hardwood block with grain parallel to the pile axis enclosed in a tightly fitting steel sleeve. A typical thickness is 6 in. However, the hardwood changes its properties during driving and rapidly looses its effectiveness. It should not be used therefore once it is crushed or burned, since damage to the pile may result. The use of micarta as a capblock is desirable and recommended because of the greater energy transmission characteristics of this material and because it retains its elastic properties much longer than hardwood.

(c) *Cushion*

To avoid damage to the head of concrete piles as the result of direct impact from the steel driving cap, it is essential that a cushion be provided between the driving cap and the pile head. A typical cushion is made of compressible material such as masonite with a thickness of 3/4 to 1 in. It is recommended that the cushion not be used for more than 5000 blows.

7.3.4. *STEEL H PILES*

7.3.4.1. *Use Of Steel H Piles*

Steel H piles identified as B.P. are available in various standard sections. WF sections are not recommended for use as piles because they have relatively thin web sections.

Steel H piles are:

— best suited for end-bearing piles to rock particularly where they are driven through soft clay deposits. In this case, steel H piles displace a minimum volume of clay and reduce the potential problem of heaving (see 7.2.3.7.(2)).

- not recommended for driving through deposits containing large obstructions. H piles may be destroyed by separation of flanges and web when hitting major obstructions. Similar problems may be encountered in very dense gravels.

- commonly used for any depth since splices are easy, (optimum lengths are 40 to 100 ft), and for loads of 40 to 140 ton.

7.3.4.2. *Materials*

Steel H piles must conform to the requirements of Subsection 4.2.3. of the NBC but as discussed in 7.3.4.4., it is generally not advantageous to use steel with a yield stress in excess of 36000 lb/sq in.

Where conditions are known to be corrosive to steel, an increased steel cross-section, encasement by cast-in-place concrete, precast concrete jackets, or cathodic protection may be used to ensure a full design cross-section.

7.3.4.3. *Splices*

Splices can be made either by riveting, bolting or welding; the latter being the most common. The splice should have at least the same strength as the pile in compression, tension and bending.

Sufficient time should be allowed for welded splices to cool and strengthen before driving is resumed.

7.3.4.4. *Structural Design*

The structural design of steel H piles must conform to the requirements of Subsection 4.2.7. of the NBC. Due to the high strength of steel, handling conditions are usually not considered in the design of steel H piles.

(1) *Driving conditions*

Driving conditions generally govern the structural design of steel H piles. Driving conditions can be investigated in detail by means of the *wave equation analysis* as referred to in 7.3.1.2.

In the absence of such an analysis, the following may be considered in evaluating driving conditions:

(a) The driving process and the generation of the geotechnical capacity of steel H piles is governed, not by the strength of steel used, but by the axial stiffness EA/L of the pile. Therefore the geotechnical capacity of the pile is also independent of the strength of steel and cannot be improved by using, say, grade 60 steel instead of grade 36 steel, since E is the same for both.

(b) For most practical cases, geotechnically allowable pile loads obtained by applying a factor of safety of 2.0 to the ultimate capacity resulting from the driving process will correspond to compressive service stresses in the pile of the order of 12000 to 14000 lb/sq in. Corresponding maximum driving stresses will be of the order of 36000 lb/sq in.

(2) *Working conditions*

The pertinent design method and details given in CSA Standard S16 'Steel Structures for Buildings' are those applicable to laterally supported compression members. However, for piles subjected to moments or horizontal loads in addition to vertical loads, the effects of such loads, as described in 7.2.5. of this Manual, must be taken into account in the structural design of the piles.

Since in most cases it will not be possible to drive steel H piles to allowable load capacities corresponding to service stresses in excess of 14000 lb/sq in., it is recommended:

(a) that the design yield strength of steel be limited to 36000 lb/sq in.

(b) that the computed structural capacity be multiplied by a reduction factor equal to 0.6. (The resulting service stress will be of the order of 13000 lb/sq in.)

(c) that design yield strengths and service stresses in excess of these values be considered only

- for steel H piles driven to true end bearing on rock when the load capacity is not related to driving,
- for piles subject to *freeze* as described in 7.2.2.2. of this Manual, and
- when horizontal loads act on the pile.

7.3.4.5. *Installation*

Driving of steel H piles is generally easy. Problems arise only when driving H piles through very dense gravel or tills containing boulders. If left unprotected under these conditions the pile tip may deform to an unacceptable extent and separation of the flanges and web may occur. To avoid such problems the following are recommended:

— *Protection of the pile*

When hard driving conditions are expected it is recommended that the tips of H piles be protected. *Oslo points* as described by BJERRUM (1957) may be used for driving into hard rock. (The heads of H piles are generally left unprotected; damaged sections are cut from the pile head after driving.)

— *Driving equipment*

All kinds of driving hammers may be used to drive steel H piles. However, the energy of the hammer should be limited to 2000 ft lb/sq in. of cross-sectional area. (The recommendations for driving cap and capblock are as in 7.3.3.5.(3). Cushions are not used when driving steel H piles.)

REFERENCE

BJERRUM, L., 1957. Norwegian experiences with steel piles to rock. *Geotechnique 7: 2, 73-96.*

7.3.5. *STEEL PIPE PILES*

7.3.5.1. *Use Of Steel Pipe Piles*

Steel pipe piles may be driven with an open or closed end; they may be left open or filled with concrete. They are;

- best suited for end bearing piles to rock or for piles subjected to horizontal loads or moments. Pipe piles driven open-ended are best for driving through soils containing obstructions such as till, since the obstructions can be broken and removed from under the pile tip.
- not recommended for friction piles in clay, because of the impermeability and smoothness of the steel surface.
- commonly used
  - for variable lengths since splices are easily made,
  - for total lengths of 40 to 120 ft,
  - with diameters of 12 in. to 20 in. (8 in. to 36 in. diam. are used),
  - for loads of 80 to 200 ton, depending upon diameter.

7.3.5.2. *Materials*

(1) *Steel*

The materials to be used for steel pipe piles are specified in Sentence 4.2.3.9. of the NBC. Where conditions are shown to be corrosive to steel, Sentence 4.2.3.11. of the NBC applies; most common protection consists in an increased steel cross-section. Under extreme conditions encasement by cast-in-place concrete or precast concrete jackets or cathodic protection may be used.

For detailed information on corrosion of steel piles see the following references.

REFERENCES

SCHWERDTFEGER, W.G. and ROMANOFF, M., 1972. NBS papers on underground corrosion of steel piling. *NBS Monograph 127, U.S. Dept Commerce, Nat. Bur. Stand.*

BJERRUM, L., 1967. Norwegian experiences with steel piles to rock. *Geotechnique, 7: 2, 73-96.*

(2) *Concrete*

Steel pipe piles may or may not be filled with concrete. When concrete is used it must conform to the requirements of Section 4.5 of the NBC. However, in most cases, the requirements of CSA A23.1 concerning maximum slump (4") cannot be met for concrete placed by tremie. Slumps of about 7" are normally used; the mix must be designed accordingly by a person competent in this field of work.

7.3.5.3. *Structural Design*

The structural design of steel pipe piles must conform to the requirements of Subsection 4.2.7. of the NBC. Due to the properties of steel, handling conditions need not be considered in design.

(1) *Driving conditions*

Two cases are distinguished here:

(a) *Pipe piles driven with an open end*

When a pipe pile is driven with an open end, and when it is shown by inspection that no soil plug forms at the pile tip, driving stresses are not related to the final load capacity of the pile. In this case, driving stresses are generally within acceptable limits. However, in cases of large pile foundations or of piles with large diameters, it is recommended that driving stresses be evaluated by means of a *wave equation analysis*, and that the grade of steel be selected accordingly.

(b) *Pipe piles driven with a closed end*

When a pipe pile is driven with a closed end, the final load capacity of the pile is directly related to the driving stresses which in turn are related to the stiffness of the pipe, and not to the strength of the steel. (See 7.3.4.4.)

In this case it is recommended that driving stresses be determined by a *wave equation analysis*.

(2) *Working conditions*

The structural capacity of steel pipe piles is determined according to the requirements of Subsection 4.2.7. of the NBC, i.e. according to CSA Standard S16;"Steel Structures for Buildings."

(a) *Pipe piles driven with an open end*

Pipe piles driven with an open end, cleaned out and filled with concrete, are to be designed as laterally supported, concrete filled structural sections used as columns. Grade of steel and strength of concrete may be selected to fit the design conditions. For piles subjected to moments or horizontal loads the effects of such loads, as described in 7.2.5. of this Manual, must be taken into account in the structural design of the piles.

(b) *Pipe piles driven with a closed end*

The geotechnical load capacity is governed by driving conditions and is dependent on the stiffness of the pile. Consequently, and as discussed in 7.3.4.4. (2), it is recommended:

- that the design yield strength of steel be limited to 36000 lb/sq in.

- that the contribution of any concrete filling be neglected

- that the structural capacity, as determined from Section 4.6. be multiplied by a reduction factor equal to 0.6.

- that the design yield strength and service stresses in excess of the above mentioned values, as well as the contribution of concrete filling, be considered only,

  - for piles driven to true end bearing where the load capacity is not related to driving,
  - for piles subject to *freeze* as described in 7.2.2.2. of this Manual,
  - when horizontal loads act on the pile, and
  - when results of a *wave equation analysis* show this to be acceptable.

7.3.5.4. *Installation*

Installation of steel pipe piles is generally easy. Problems arise only when driving closed end pipe piles through materials containing obstructions or when driving open end pipe piles through very dense materials. In the first case piles may deflect and deviate from their design alignment to an unacceptable extent. In the second case the tip of the pipe may be deformed.

(1) *Protection of the pile*

(a) *Piles driven with closed ends*

No special protection is necessary for soft or medium driving. When hard driving is expected it is desirable to provide a special pile point of conical shape, made of special steel or alloy. When obstructions such as boulders are expected pipe piles should be driven with open ends with provision for the breaking and removal of such obstructions.

(b) *Piles driven with open ends*

No special protection is necessary for soft or medium driving. When hard driving is expected such as in dense gravel it is recommended that a special driving shoe made of special steel or alloy be provided. When pipe piles are driven with open ends constant control of the driving energy is necessary to identify obstructions and provide for their removal. Regular checks on the level of soil within the tube are necessary to recognize the formation of a soil plug at the pile tip.

(2) *Driving equipment*

All kinds of driving hammers may be used to drive steel pipe piles. However, it is recommended that the energy of the hammer blow be limited to 2000 ft lb/sq in. of cross-sectional area. The recommendations for driving cap and capblock are as described in 7.3.3.5. Cushions are not used when driving steel pipe piles.

7.3.6. *COMPACTED EXPANDED BASE CONCRETE PILES*

7.3.6.1. *Use Of Compacted Concrete Piles*

Compacted concrete piles were originally developed as a patented technique and require the use of special equipment for their installation. Compacted concrete piles develop their bearing capacity primarily from the densification of soil around the expanded base. They are:

- best suited for piles in granular soils, in particular in loose sands where high capacities can be developed at shallow depths, and for piles subjected to uplift forces provided they are structurally designed for this condition.
- not recommended in cohesive soils where compaction of the base is impossible.
- commonly used with shaft diameters of 12 to 24 in., for loads of 60 to 150 ton and for lengths of 10 to 60 ft.

7.3.6.2. *Materials*

Materials used for compacted concrete piles must conform with the requirements of Subsection 4.2.3. of the NBC. However, because of the installation technique, *dry concrete* must be used in the compacted base in all cases and in the compacted shaft when this is used instead of an encased shaft. (*Dry concrete* means a concrete with 0 in. of slump containing about 3.5 gallons of water per cement bag.) The strength of *dry concrete* should be checked on special compacted samples, although there is currently no standard method for such sampling and tests.

Compacted concrete piles are commonly built with an encased shaft. The casing is usually made of light gauge steel tubing and is intended only to provide the necessary protection against intrusion of water or soil during concreting operations.

7.3.6.3. *Structural Design*

The structural design of compacted concrete piles must conform with the requirements of Subsection 4.2.7. of the NBC, i.e. to CSA Standard A23.3. Since compacted concrete piles are cast-in-place, only working conditions need be considered in design.

(1) *Working conditions*

Two cases are distinguished.

(a) *Piles with compacted shaft*

In this case the pile shaft is made of *dry concrete* compacted against the soil and may be reinforced. The structural capacity of the shaft is determined according to the requirements of CSA A23.3. The design 28 days strength of *dry concrete* is taken equal to 3000 lb/sq in. The area of concrete effective in load carrying is taken equal to the nominal area of pile shaft corresponding to the inner diameter of the driving tube. The resulting structural capacity is multiplied by a reduction factor of about 0.7 to take the unusual construction conditions into account.

(b) *Piles with encased shaft*

Piles with encased shafts may be reinforced. The structural capacity of the shaft is determined according to the requirements of CSA A23.3. Concrete with any desired strength may be used. The resulting structural capacity is multiplied by a reduction factor of about 0.7 to take the unusual construction conditions into account.

Where compacted concrete piles have to resist uplift forces, the structural strength of the shafts must be determined accordingly. Consideration must be given to a proper continuity of reinforcing at the junction of the shaft with the base.

7.3.6.4. *Installation*

Installation of compacted concrete piles requires the use of special equipment and is generally carried out in three steps.

(1) *Driving*

A plug of *dry concrete* is placed inside a heavy steel tube properly aligned at ground surface. A heavy ram (5000 to 10000 lb) is then dropped from 10 to 20 ft on the plug. As a result of repeated impacts of the ram the concrete plug is forced into the ground, to the desired depth dragging the tube with it.

(2) *Forming the base*

When the base of the tube has been driven to the design depth, the tube is clamped to the driving rig at ground surface to maintain it at a fixed elevation. By applying blows of the heavy ram the concrete plug is expelled into the ground. *Dry concrete* is added and expelled in a continuing process. It is essential that a minimum amount of *dry concrete* be maintained in the tube at all times and that neither soil nor ground water be allowed to enter the tube. Both the volume of concrete and the total energy are recorded. The relationship shown on Fig. 7.4. may be used with caution as a guideline for estimating the capacity of such a pile.

(3) *Forming the shaft*

After completion of the base, additional small batches of *dry concrete* are placed at the bottom of the tube. With the ram resting on top of each batch, the tube is withdrawn slightly and several blows of the ram are applied to compact it. The cycle is repeated until the top of the pile reaches the design elevation. If the design calls for reinforcing, the steel cage is placed inside the tube before the last batch of *dry concrete* is compacted in the base to ensure an appropriate connection. Care must be exercised to ensure that the cage is not lifted when the ram is raised.

(4) *Encased shaft*

If the design calls for an encased shaft, the steel casing is dropped inside the driving tube after the base has been compacted. A plug of *dry concrete* is then placed and compacted by several blows of the ram to ensure intimate contact with the base. The driving tube is then withdrawn and the steel casing is filled with concrete in the normal manner.

(5) *Common installation problems*

Three main problems may be encountered when using compacted concrete piles. They can be avoided by careful construction and inspection.

(a) *Placing of concrete*

For piles with compacted shafts, extreme care must be exercised in order to maintain a sufficient height of *dry concrete* within the driving tube at all times. If the tube is withdrawn too rapidly or if too much concrete is expelled a void may be created between the top of the compacted concrete column and the bottom of the tube. Water and soil may fill this void and produce a reduction (necking) or even a complete interruption of the concrete shaft. A constant control on the quantities of concrete placed, the elevation of the base of the tube and the elevation of the top of the compacted concrete, is necessary to avoid this problem.

(b) *Heave of adjacent piles*

Under particular soil conditions such as when piles have to be driven through a clay layer into a lower sand deposit, existing piles may heave as the result of driving new piles adjacent to them. A typical case is discussed by BRZEZINSKI et al (1973).

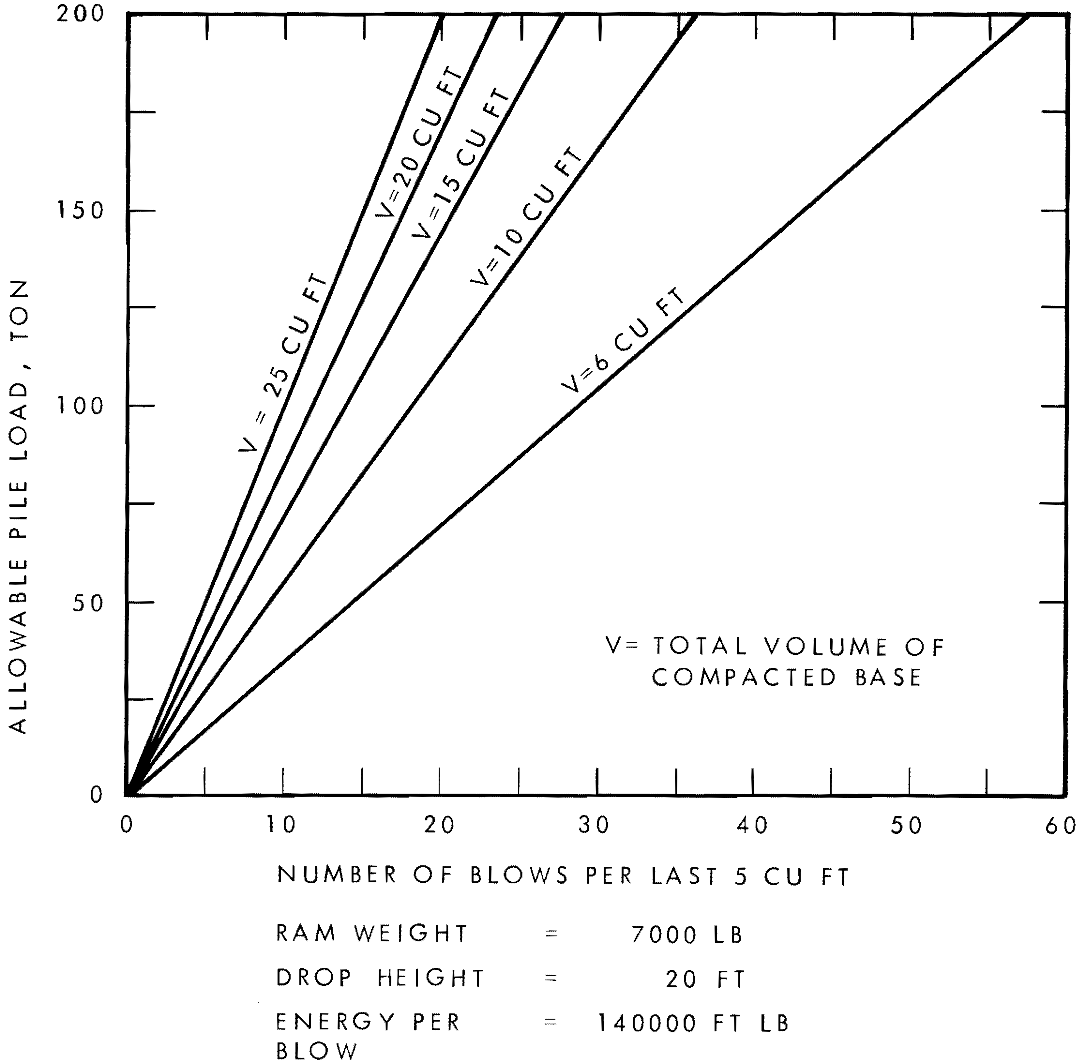


FIG 7.4 (After Nordlund)

ALLOWABLE PILE LOAD FOR COMPACTED EXPANDED BASE CONCRETE PILES

(c) *Insufficient load capacity*

The load capacity of compacted concrete piles is related empirically to the volume of concrete and energy imparted to the compacted base. Problems with insufficient load capacities may occur where such piles are used in areas or soil conditions where little or no experience is available. It should be considered mandatory to check the capacity of compacted concrete piles by load tests and, where this capacity is insufficient, to make the necessary adjustments in the compaction of the base.

REFERENCES

NORDLUND, R.L., 1970. Pressure injected footings. *Proc. Conf. Design and Installation of Pile Foundations and Cellular Structures. Lehigh Univ., 297-308.*

BRZEZINSKI, L.S., SHECTOR, L., MacPHIE, H.L. and Vander NOOT, H.J., 1973. An experience with heave of cast in situ expanded base piles. *Can. Geotech. J. 10: 246-260.*

7.3.7. *BORED PILES*

7.3.7.1. *Use Of Bored Piles*

Bored piles can be made in different shapes and dimensions. Cylindrical piles are the most frequent type; however in recent years elements of diaphragm walls have been used in various combinations (I, H, X) as deep foundation units. Bored piles are increasingly used because of their very high load capacities. Bored piles are:

— best suited for end bearing high capacity piles to rock or dense till. Bored piles are also successfully used in stiff clays.

— not recommended in cases where deposits of loose cohesionless materials have to be penetrated or when artesian groundwater conditions prevail; in such cases it may be impossible to excavate successfully even with the use of bentonite slurry.

— commonly used for variable lengths (bored piles excavated with bentonite slurry have been installed at depths in excess of 300 ft), for diameters in excess of 36 in. and up to 8 ft, for loads up to 2000 ton.

7.3.7.2. *Materials*

The materials to be used for bored piles must conform with the requirements of Subsection 4.2.3. of the NBC. However, where concrete is placed by tremie the requirements of CSA A23.1 concerning maximum slump cannot be met. Slumps of about 7 in. are normally used; the concrete mix should be designed by a person competent in this field of work.

When bored piles are provided with structural steel casings, the appropriate considerations discussed in 7.3.5. of this chapter also apply.

When bored piles are excavated with bentonite slurry the quality of the slurry (density, viscosity, etc...) should be determined by a person competent in this field of work and it should be kept under constant control to ensure that it performs satisfactorily.

7.3.7.3. *Structural Design*

Bored piles may be uncased or cased.

(1) *Uncased piles*

The structural capacity of uncased bored piles must conform with the requirements of Subsection 4.2.7. of the NBC, i.e. according to the requirements of CSA A23.3. As discussed in 7.3.1.1. the full structural capacity as resulting from CSA A23.3, should not be considered in design because of adverse installation conditions. More specifically it is recommended:

- to consider that an outer surface of concrete of 1 inch thickness does not contribute to the structural strength of the pile cross-section. (Experience shows that the surface concrete is often contaminated with soil drilling and has a reduced strength.)
- that the design strength of concrete be limited to f'_c = 4000 lb/sq in.
- that the structural capacity resulting from CSA A23.3 be multiplied by a reduction factor, the magnitude of which should be selected by the design engineer to take into consideration the difficulties related to the construction of the unit. Suggested values of this reduction factor are given as follows:

Type of Deep Foundation Unit	REDUCTION FACTORS FOR UNCASED BORED PILES		
	Nominal size or diameter of unit		
	18 in. and less	18 in. to 36 in.	36 in. and more
Unreinforced	0.8	0.85	0.9
Reinforced	0.6	0.7	0.8

(2) *Cased piles*

The structural capacity of cased bored piles must conform to the requirements of Subsection 4.2.7. of the NBC, i.e. according to the requirements of CSA S16, on concrete-filled hollow structural sections used as columns.

As discussed in 7.3.1.1., the full structural capacity as resulting from CSA S16 should not be considered in design because of adverse installation conditions. More specifically it is recommended:

- that the design strength of concrete be limited to f'_c = 4000 lb/sq in.
- that the structural capacity resulting from CSA S16 be multiplied by a reduction factor, the magnitude of which should be selected by the design engineer to take into consideration the difficulties related to the construction of the unit. Suggested values of this reduction factor are given as follows:

Type of Deep Foundation Unit	REDUCTION FACTORS FOR CASED BORED PILES		
	Nominal size or diameter of unit		
	18 in. or less	18 in. to 36 in.	36 in. and more
Unreinforced	0.8	0.9	0.9
Reinforced	0.7	0.8	0.9

7.3.7.4. *Installation*

(1) *Excavation*

The excavation for a bored pile may be made:

- by using a large diameter auger or bucket drill to remove the soil above the founding level,

- by driving, vibrating or pushing down a heavy casing to the proposed founding level and by removing the soil from the casing either continuously as driving proceeds or in one sequence after the casing has reached the founding level.

- by using a clamshell mounted on a Kelly bar to remove the soil and by keeping the excavation open by use of a bentonite slurry.

- by drilling, coring or chopping when penetration into rock is specified. (Blasting is not permitted since it affects the properties of surrounding soil and rock.)

Selection of the excavation procedure depends on the soil and prevailing groundwater conditions. In stiff cohesive soils, free of water-bearing layers, simple augering is possible. In this case a loose fitting safety liner is required to protect personnel during clean out and inspection. Where weak or water-bearing soil overlies the founding level, a temporary casing or liner is required to support the hole and to hold back the groundwater until the base is cleaned out, inspected and concrete is placed. Where the soil above founding level is very weak and wet, the steel casing may be left in place; in this case the casing may be considered as contributing to the structural strength of the pile, provided its inside surfaces are cleaned of smeared soil.

Whenever possible the steel casing should be pushed tightly into the founding layer to control the flow of groundwater into the excavated hole. Where such flow is too great to be controlled, it may be necessary to clean out the hole and to place the concrete by tremie without removing the water; direct inspection of the hole is then impossible. In this case, excavation without casing but with the use of bentonite slurry may prove more effective. However this is only feasible where the founding medium is bedrock and where inspection of the bottom of the excavation by such means as coring is provided.

Belling of the base, where specified, may be done by machine or by hand. Where the nature of soil requires it, or when groundwater is present, bells should be sheeted and braced to maintain their shape and permit proper placing of concrete.

Regardless of the procedure used for excavation it is essential that the base be cleaned to the sound founding material, and that groundwater be controlled so that excess uplift pressures do not act below the founding level and water and soil do not flow over the prepared base. It is also essential that the walls of a socket in rock be cleaned of loose rock or smear when loads are designed to be transferred to the founding rock by adhesion of the concrete to the walls of the socket. (See 7.2.1.).

(2) *Placing concrete*

After the excavation has been completed, inspected and accepted, concrete may be placed in one continuous operation.

(a) *Placing reinforcement*

Steel reinforcement, steel studs or core sections should be accurately placed and adequately supported. Should the method of pile construction specify removal of the casing, care should be exercised to ensure that the reinforcement is not disturbed or exposed to surrounding soil during the removal process. Spacers, capable of sliding on the casing, should be attached to the reinforcement.

(b) *Placing concrete in a dry excavation*

Where the excavation is dry, concrete may be placed by buckets, chutes or elephant trunks so as not to result in segregation. It is permissible to allow free fall of concrete, but it must be poured through a centering chute which causes it to fall down the center of the hole, well clear of the walls of the shaft. Where free fall of concrete is used, it results in adequate compaction below the top 5 ft. Vibration of the concrete is then required for the upper 5 ft to produce a concrete of uniform strength.

(c) *Withdrawing temporary casing*

If ground conditions are such that the casing may be removed during the concreting of the pile, the procedure used should ensure that the concrete will not be disturbed, pulled apart or pinched off by earth movement. The level of concrete must be maintained at a minimum of 5 ft above the bottom of the casing, a higher head being necessary in cases of high groundwater level in the surrounding soil.

(3) *Common installation problems*

Some common problems associated with the installation of bored piles are:

- Inadequate precautions to control groundwater flow during excavation resulting in loss of ground and potential long term undermining of floor areas.
- The tremie pipe is pulled out of the concrete during placing so that some of the concrete flows through water. The result is a layer or pocket of sand and gravel and a concentration of cement or laitance at cut-off level.
- The temporary liner is withdrawn too fast causing soil to intrude in the theoretical concrete section (necking).
- The temporary liner becomes stuck and is withdrawn after partial set of concrete has taken place, causing cracking of the shaft.
- The concrete is too old when placed. Where delays are expected a retarder should be specified.
- Low slump concrete is used without vibration, causing voids to be formed.

REFERENCES

WOODWARD, R.J., GARDNER, W.S. and GREER, D.M., 1972. Drilled pier foundations. *McGraw Hill, New York.*

ACI Committee 336, 1972. Suggested design and construction procedures for pier foundations. *J. Am. Conc. Inst., August 1972, 461-480.*

7.4. LOAD TESTS ON DEEP FOUNDATIONS

7.4.1. *USE OF LOAD TESTS*

As indicated in 7.1. of this chapter, load testing of piles is the most positive method of determining load capacity. Depending upon the type and size of the foundation, such load tests may be performed at different stages during design and construction.

7.4.1.1. *Load Tests During Design*

The best method of designing a pile foundation consists in performing pile driving and loading tests. The number of tests, type of piles tested, method of driving or of installation and test loading should be selected by the engineer responsible for design. The following points should be considered:

- The test program should be carried out by a person competent in this field of work.
- A detailed soil investigation should be carried out at the test location.
- The piles, equipment, used for driving or other method of installation and procedure should be those intended to be used in the construction of the foundation.

— The piles should be instrumented for shaft deformations to allow for a measurement of settlement at the head of the pile. (Where possible, deformation measurements should also be made at the tip of the pile and at intermediate points to allow for a separate evaluation of point resistance and skin friction).

— The driving process should be observed in detail to allow for an analysis by means of the *wave equation*. (See 7.5.4.).

— The piles should be loaded to at least twice the proposed working load and preferably beyond failure.

REFERENCE

TAVENAS, F.A., 1971. Load test results on friction piles in sand. *Can. Geotech. 7: 7-22.*

7.4.1.2. *Load Tests During Construction*

It is recommended practice to perform load tests on representative deep foundation units at early stages of construction. The purpose of such tests is to ascertain that the allowable loads obtained by design are appropriate, and that the installation procedure is satisfactory.

The selection of the test piles should be made by the engineer responsible for design on the basis of observed driving behaviour or installation features.

7.4.1.3. *Routine Load Tests For Control*

Where full advantage of Sentences 4.2.4.1.(1)(c) and 4.2.7.2.(2) of the NBC is to be taken, a sufficient number of load tests must be carried out on representative units to ascertain the uniformity of the allowable loads and of the behaviour of the constructed foundation. Load tests for control should be performed on one out of each group of 250 units, or portion thereof, of the same type and capacity. Load tests should also be performed on one out of each group of units where driving records or other observations indicate that the soil conditions differ significantly from those normally prevailing at the site. Selection of the deep foundation units to be load tested is the responsibility of the design engineer.

7.4.2. *RECOMMENDED TEST METHODS*

Sentence 4.2.7.2.(2) of the NBC requires that load tests on piles be carried out in accordance with ASTM D1143-69, "Load Settlement Relationship for Individual Vertical Piles Under Static Axial Loads", or other acceptable methods. The ASTM Standard actually presents three alternative methods. In addition, another test method, acceptable under certain circumstances is included here.

7.4.2.1. *ASTM D1143-69 Method (Method A)*

This standard applies to load tests carried out for control of deep foundations as discussed in 7.4.1.2. and 7.4.1.3. However, it is recommended that only the method described in Sentence 5.2.1. of ASTM D1143-69 be used. The following considerations should be taken into account when using this test method

— The loading device described in Sentence 3.1.3. may prove unusable because the accuracy of measurement of both applied load and settlement may be insufficient for clear interpretation of results.

— Incremental strain measurements, as discussed in Sentence 4.1.5. are recommended for all design load tests.

— The elapsed times between driving and testing of piles, mentioned in Sentence 5.1.1. are minimum values. As discussed in 7.2.2. and 7.2.3. variations in the bearing capacity of piles can develop over longer periods of time. In most cases however, the pile capacity increases with time so that early testing will result in an underestimate of the actual pile capacity.

- Depending on the soil condition, the type of pile and the observations during previous stages of the load test, the 200 percent design load may not need to remain on the pile for 24 hours, as required in Sentence 5.2.1.1. A short duration test usually will not indicate the long term behaviour of the pile but testing the pile to failure as required in Sentence 5.2.1. is of more significance in the assessment of the pile behaviour.

7.4.2.2. *Constant Rate Of Penetration Method (Method B)*

Piles in clay cannot be load tested at sufficiently slow rates of loading to give any indication of the settlement of such piles, and only the ultimate capacity may be obtained from load tests. The constant rate of penetration (CRP) method developed by WHITAKER (1970) may be used to determine the ultimate capacity of tested piles. This method is recommended for testing piles in clay and for all tests where the ultimate capacity only is to be measured.

(1) *Test equipment*

Equipment as specified in Sections 3 and 4 of ASTM D1143-69 may be used. To produce the necessary constant rate of penetration, the hydraulic jack must be connected to a pump, electrically operated and equipped with a regulator capable of providing an adjustable constant flow of oil to the jack. Typical test equipment is described by GARNEAU and SAMSON (1974).

(2) *Test procedure*

- The minimum elapsed time between driving and testing is specified in ASTM D1143-69 and commented on in 7.4.2.1.
- The pile head should be forced down at a rate of settlement of 0.02 in./min.
- Readings of the pressure in the jack and of the settlement of the pile head should be taken at regular time intervals not greater than three minutes.
- Loading should continue until it reaches 250% of the design load of the pile, but at least until the observed settlement of the pile head is equal to the elastic deformation of the pile plus 1 inch; the elastic deformation being obtained either by direct measurement or by assuming that the test load acts on the full length of the pile.

REFERENCES

WHITAKER, T., 1970. The design of piled foundations. *Pergamon Press, London.*

Swedish Commission on Pile Research, 1970. Recommendation for pile driving test and routine load testing of piles. *Prelim. Rep. 11, Royal Swedish Academy of Engineering Sciences, Stockholm.*

GARNEAU, R. and SAMSON, L., 1974. A device for the constant rate of penetration test on piles. *Can. Geotech. J., 11: 298-302.*

7.4.2.3. *Other Tests*

It may be necessary to test piles under loading conditions other than the usual axial compressive load, eg. pullout tests and horizontal load tests may be specified. There is no standard method for such tests and they should be carried out under the direction of a person competent in this field of work.

Where the ultimate pullout resistance is to be obtained a reverse CRP method is recommended. The system used for reaction should be arranged to that no compressive load is applied to the soil surface within a distance of 10 ft from the pile.

Horizontal load tests are discussed in 7.2.5.3.

7.4.2.4. *Presentation Of Load Test Results*

The results of load tests performed according to any of the methods described above should be presented in a report conforming to the requirements of ASTM D1143-69, Sentence 6. Graphic presentation of the results should include the following.

(1) *Load-settlement curve*

The loads are computed from the observed jack pressures and the calibration constant of that jack as required in Sentence 3.1.1.1. of ASTM D1143-69. The settlements are the average of the readings on at least two dial gauges, expressed in 1/1000 in.

To facilitate the interpretation of the test results, as discussed in 7.4.3., it is recommended that the scales for the loads and the settlements be selected so that the line representing the elastic deformation δ of the pile be inclined at an angle of about 20° to the load axis. The elastic deformation δ is computed from:

$$\delta = \frac{Q\ L_p}{A\ E}$$

where δ = elastic deformation, in.

Q = test load, lb

L_p = pile length, in.

A = cross-sectional area of the pile, sq in.

E = Young's modulus of the pile material, lbs/sq in.

(2) *Time-settlement curves*

The time-settlement readings taken for each load increment in Method A should be presented in graphical form, with the time in minutes on a linear scale on the abcissa, and the observed settlements in 1/1000 in. on a linear scale on the ordinate.

7.4.3. *INTERPRETATION OF LOAD TEST RESULTS*

Only the results of standard tests, as described in 7.4.2.1. and 7.4.2.2. are considered in the following. The interpretation of pullout or horizontal load tests should be made by the person responsible for the design of such tests.

There is a wide variety of methods for interpreting standard load tests, which can be divided into two groups:

- Those methods giving an acceptability criterion for the tested pile. Typical of these is the method specified in the 1970 edition of the NBC. In these methods no consideration is given to the failure load of the pile. In most cases a pile is deemed acceptable if the observed settlements of pile head are within specified limits, which are selected independently of the type and length of pile.

- Those methods giving the failure load of the tested pile, from which the allowable load may be computed by applying an adequate factor of safety. Such methods are recommended because they provide a better understanding of pile quality.

7.4.3.1. *Method Based On A Failure Criterion*

(1) *Failure criterion*

Different failure criteria have been proposed in the literature. The following criterion is considered applicable to all types of load test and is recommended for use.

The ultimate or failure load Q_f of a pile is that load which produces a settlement of the pile head equal to:

$$S_f = \delta + \frac{D}{30}$$

where S_f = settlement at failure, in.
D = pile diameter, in.
δ = elastic deformation of pile shaft, in.

δ is defined as:

$$\delta = \frac{Q\ L_p}{A\ E}$$

where Q = test load, lb
L_p = pile length, in.
A = cross-sectional area of pile, sq in.
E = Young's modulus for pile material, lb/sq in.

(2) *Determination of the failure load*

The failure criterion defined above is represented by a straight line on the load-settlement curve (Fig. 7.5.). The observed load-settlement curve intersects the failure criterion at point F, the abcissa of which, by definition, is the failure load Q_f of the pile. Where the observed load-settlement curve does not intersect the failure criterion, the maximum test load should be taken as the failure load.

(3) *Factor of safety*

To obtain the allowable pile load, the failure load Q_f should be divided by a factor of safety of at least 2.0. Larger factors of safety may be required:

— for friction piles in clay, in particular when Q_f has been obtained from a CRP test. (A value of 2.5. is recommended).

— where a limited number of load tests is specified and where soils conditions are variable,

— for piles in loose sand and silts where Q_f may decrease with time,

— to ensure satisfactory settlement behaviour.

7.4.3.2. *Other Methods Of Interpretation*

Other methods of interpretation commonly used on results of ASTM D1143-69 tests are listed in the following references.

All methods based on maximum allowable gross settlements, which do not take into account the elastic deformation of the pile shaft are not recommended because the use of such methods results in overestimates of allowable capacities of short piles and in underestimates of allowable capacities of long piles.

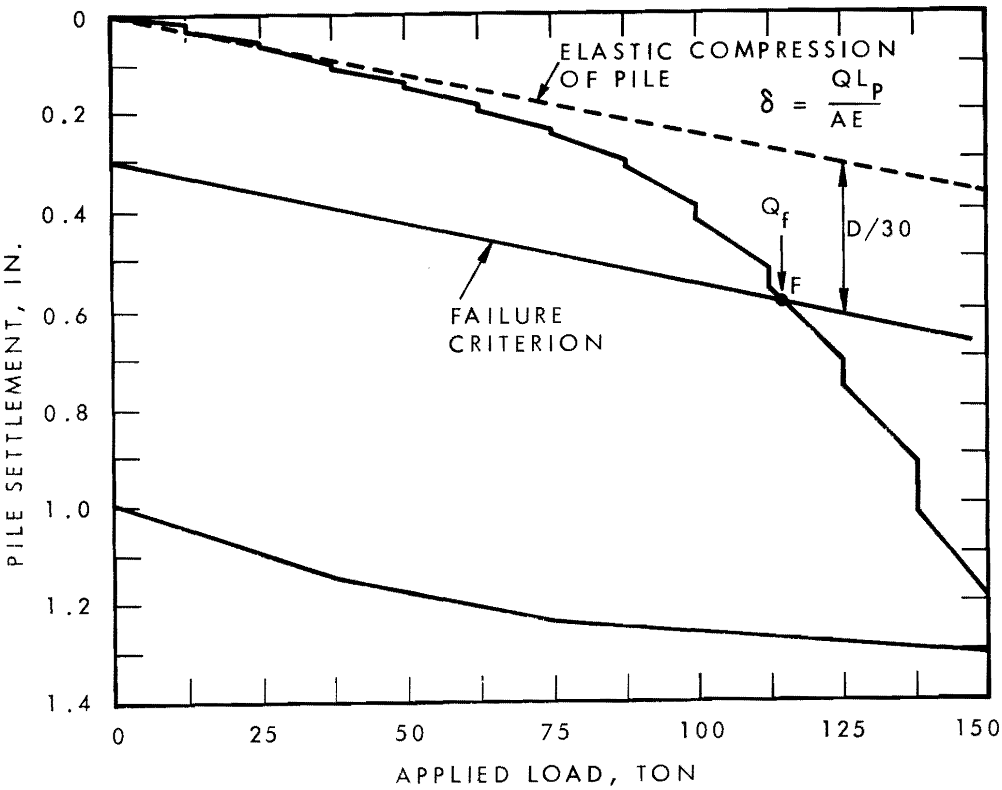


FIG 7.5
EXAMPLE OF RECOMMENDED FAILURE CRITERION

Detailed analysis of pile test results by means of the method described by TROW (1967) is useful particularly where the failure load of the pile cannot be developed.

REFERENCES

CHELLIS, R.D., 1951. Pile foundations. *McGraw Hill, New York.*

FULLER, F.M. and HOY, H.E., 1970. Pile load tests including quick load test method, conventional methods and interpretations. *Highway Research Record, No. 333 HRB US Nat. Research Council, 74-86.*

TROW, W.A., 1967. Analysis of pile load test results. *Proceedings 1967 Convention, Can. Good Roads Assoc., Vancouver, 414-434.*

7.5. INSPECTION OF DEEP FOUNDATIONS

7.5.1. *GENERAL*

The quality of deep foundations is governed by their installation. A proper choice of installation procedure and equipment, good workmanship and tight control of all installation work is essential to the construction of a good deep foundation. Consequently, inspection is of utmost importance. Sentence 4.2.2.3.(1) and (2) of the NBC requires that *Inspection shall be carried out by the designer, or by another suitably qualified person responsible to the designer to ensure that the sub-surface conditions are consistent with the design and that construction is carried out in accordance with the design and good engineering practice. Inspection shall be carried out, on a continuous basis during the construction of all deep foundation units.*

It is essential that inspection personnel be well experienced in this field, so as to be able:

- to recognize faulty construction procedures,
- to properly interpret pile driving data, particularly when piles are driven to rock,
- to properly evaluate actual soil conditions in bored piles.

7.5.2. *DOCUMENTS*

Good inspection begins prior to actual construction, with the examination of all design documents. The following should be available to the inspector on the site:

- Soil investigation report,
- Drawings of the foundation,
- Specification,
- Contract,
- Any other documents on special design features or assumptions.

On the drawings of the foundation, the exact location of each deep foundation unit should be indicated, and each unit identified by a unique designation: pile number, column number or structure designation followed by pile number. This designation should be used for reference throughout the construction and inspection.

If any of the documents contain unclear or contradictory matter, this should be reported by the inspector and clarified immediately.

7.5.3. *LOCATION AND ALIGNMENT*

7.5.3.1. *Location*

Exact location of each deep foundation unit should be staked in advance and checked immediately prior to installation of each unit. After completion of the installation the location of each unit should be checked against design location and permissible deviation as indicated on the design documents.

As required in Sentence 4.2.7.5. of the NBC, permissible deviations from the design location shall be determined by design analysis. For guidance, deviations to the following maximum values are usually considered acceptable:

- 3 in. on units placed in groups of 4 units or more, arranged in more than 1 row.
- 1/20 of the diameter of the unit for single acting units or units arranged in 1 row.

Wrongly located units will result in:

- modified load distributions on the different units in a group and a necessary reduction of the allowable loads on each unit in the group.
- modified stress distribution in the cross-section, of single acting units and a necessary reduction of the structural capacity of the unit.

As required in Sentence 4.2.7.6. of the NBC where a deep foundation unit is wrongly located, the condition of the foundation shall be assessed by the person responsible for design and the necessary changes made.

7.5.3.2. *Alignment*

During and after installation of any deep foundation unit, its alignment should be checked against the design alignment and the permissible deviation as indicated on the design documents.

(1) *Driven piles*

The alignment of driven piles should be checked at regular intervals during driving. In general, this can be done only by checking the alignment of the driving leads and of the visible portion of the pile by means of a mason's level placed against the face of the pile and leads, or against the vertical face of a template with appropriate shape for battered piles. Where the pile is provided with a central hole, the alignment of the pile can be checked at the end of driving. In this case the method used should be such that the deformed shape of the pile may be measured. A typical example is given by FELLENIUS (1972). Methods which only determine whether the upper part of the pile is straight or not are of little value since they do not allow for an analysis of the effect of bending on the structural capacity of the pile.

REFERENCE

FELLENIUS, B.H., 1972. Bending of piles determined by inclinometer measurements. *Can. Geotech. J., 9: 25-32.*

(2) *Cast-in-place piles*

The alignment of cast-in-place piles should be checked during the process of boring or driving the casing and after completion of the excavation. Checking the alignment during boring or driving can be done as discussed in 7.5.3.2.(1). Checking the alignment after completion of the excavation should be made by a method such that the exact shape of the excavation may be measured.

(3) *Permissible deviations*

As required in Sentence 4.2.7.5. of the NBC the permissible deviations from design alignment of deep foundation units shall be determined by design analysis.

Current practice is to limit the total deviation from design alignment to a certain percentage of the final length of the deep foundation unit: 2% is a value in common use. However, such practice does not ensure proper structural behaviour of the unit since it does not take into account the length over which this deviation is distributed. It should be recognized:

— that the total deviation from alignment of a deep foundation unit has little influence on its geotechnical capacity unless it exceeds extreme values such as 10% of the length of the unit.

— that practically all piles, particularly when driven, are more or less out of design alignment. A straight pile is a theoretical concept, seldom achieved in practice.

— that only the radius of curvature of a deep foundation unit is of importance for its structural and geotechnical behaviour. The maximum allowable radius of curvature should be determined by design whenever it is specified that such radius be measured during inspection. A discussion of allowable bending of piles is given by FELLENIUS (1972), (See 7.5.3.2.(1)).

7.5.4. *INSPECTION OF PILE DRIVING OPERATIONS*

7.5.4.1. *General*

Item of importance in driving of different types of piles have been discussed in 7.3. of this Chapter. The following check lists are given for guidance of inspection personnel.

7.5.4.2. *Driving Equipment*

Items to be checked include the following:

(1) Type of hammer as specified

(2) For drop hammers:

— weight of the hammer,

— type of crane and trip mechanism,

— drop height, and

— sliding condition in the leads.

(3) For steam hammers:

— type (single or double acting), make, serial number,

— weight of the hammer and ram,

— positions of the valves, trips, and resulting stroke,

— steam pressure,

— energy rating,

— blows per minute, and

— general condition of the hammer.

(4) For diesel hammers:

— type, make, serial number,

— weight of the hammer and ram,

- stroke,
- energy rating, and
- blows per minute.

(5) For driving cap:

- weight of the cap,
- dimensions as related to pile, hammer and lead dimensions,
- type of capblock,
- thickness of the capblock,
- condition of the capblock (This should be checked regularly and burned, crushed or broomed capblocks should be replaced immediately),
- type of cushions used,
- thickness of cushion, and
- condition of cushion (A new cushion should be used for each pile).

(6) Type and characteristics of other equipment such as drive heads, followers, etc...

7.5.4.3. *Piles*

Items to be checked include the following:

(1) Type of pile is as specified

(2) For steel piles

- that there is a mill certificate indicating that the product meets the specifications (Each shipment),
- that the condition of the piles is satisfactory, not damaged or bent,
- that tip and head protections, if any, are as specified,
- that proper handling and storing procedures are followed,
- that the head of the pile is perpendicular to the longitudinal axis, and
- that splices conform to specifications.

(3) For precast concrete piles:

(a) At the plant;

- that the geometry and other characteristics of the forms are as required,
- that dimensions, form and quality of reinforcing, are as specified,
- that proper curing conditions are provided,
- that proper handling and storage procedures are followed,
- that the quality of the concrete: mix, slump, strength, etc... are as required by CSA A23.1.

And for prestressed piles

- that there is a certificate indicating that the prestressing cables meet specifications, and

- that the prestressing procedure and forces used are as specified.

(b) On site;

- that the age of delivered piles and corresponding strength of concrete (based on test cylinders or Schmidt hammer tests), are as specified,
- that the geometry of piles: heads perpendicular to longitudinal axis, length, straightness, conform to specifications,
- that proper handling and storage procedures are followed,
- that the condition of the piles is satisfactory (not fissured, spalled, etc...), and
- that joins, if any, conform to specifications.

(4) For timber piles:

- that there is a certificate indicating the species and grade of timber,
- that there is a certificate on protective treatment, where specified,
- the length and dimensions at tip, mid-height and head of pile,
- that the piles are straight within the specified tolerances,
- that proper handling and storage procedures are followed,
- that points or boots, if any, conform to specifications and are properly placed, and
- that protective treatment is intact over the full surface of pile where specified.

7.5.4.4. *Driving Operations*

Items to be checked or noted include:

- general information: date; weather conditions; pile identification,
- the exact location of the pile,
- the stability and alignment of the driving rig and leads,
- the number of blows,
- deformations of the pile under blows at various depths,
- the position and quality of splices,
- the location, time, duration of any interruption in driving,
- elastic deformations, permanent set and blows per inch for final blows,
- the elevations of ground surface, pile tip and cut off,
- any erratic or unusual pile behaviour with record of time and corresponding tip elevation,
- possible heave of adjacent piles, and
- other pertinent information.

7.5.5. *INSPECTION OF COMPACTED CONCRETE PILES*

7.5.5.1. *General*

The construction of compacted concrete piles requires the use of special equipment and a particular technique. It should be undertaken only by contractors well experienced in construction of this particular type of deep foundation.

7.5.5.2. *Equipment*

The equipment should be checked for conformity to the specifications or to good practice. Of particular importance are:

- the size and weight of the hammer,
- the dimensions of the driving tube, and
- the adequacy of the clamping equipment to hold the driving tube when the base.

7.5.5.3. *Installation*

Items to be checked or noted include:

- general information: date; weather conditions; pile identification; time driving was started and completed, and time concreting was started and completed,
- the location of the pile,
- the alignment of the driving tube,
- the resistance to driving of the tube: drop height; weight of the hammer; number of blows/ft,
- the elevation of the bottom of the driving tube before forming the base,
- the concrete for the base: the mix used; strength determined from the compacted samples,
- the formation of the base: number of 5 cu ft buckets and number of blows per bucket; hammer weight, drop height and resulting energy per blow; final volume of the base and final driving energy for the last bucket against the specifications or good practice.
- elevation of the bottom of the hammer when forming the base; (Minimum should be 3 in. above the bottom of the driving tube.)
- placement of reinforcing, if any,
- seating into the base of the permanent liner, if any,
- quality of concrete for the shaft: mix, slump, freshness, that there are test cylinders of each day of pour, of each 40 cubic yards, and of any suspect batch,
- the relative position of the bottom of the driving tube and top of the concrete during compaction of the shaft,
- the volume of the concrete in the compacted shaft compared to the length of the shaft,
- the cut-off elevation,
- the elevation of the top of the liner, if any, immediately after installation,
- the elevation of each liner after all adjacent units are driven (to check for possible heave), and

- the backfilling of the annular space around the permanent liner.

7.5.6. *INSPECTION OF BORED DEEP FOUNDATIONS*

7.5.6.1. *Preliminary Information*

In addition to the usual information on soil stratigraphy, type and strength, information on the following should be available:

- presence of water bearing strata of gravel, sand or silt; location and thickness of such strata; piezometric levels in such strata,
- piezometric level in the bedrock if the piles are founded in bedrock,
- rate of flow from water bearing strata or bedrock into the borehole,
- presence of large obstructions above the founding level,
- presence of natural gas in the soil or bedrock, and
- chemical analysis of the groundwater.

7.5.6.2. *Excavation*

Items to be checked or noted include:

- general information: date, weather conditions, unit identification, time excavation was started and completed,
- location of the unit,
- conformity of the excavation technique to the specifications or to good practice,
- alignment and dimensions of the excavation at regular intervals,
- adequacy of the technique and equipment used to penetrate water bearing strata, if any,
- adequacy of the technique and equipment used to penetrate large obstructions, if any,
- log of stratigraphy penetrated during excavation,
- depth of the socket in sound rock, if any, (Elevation of the bottom,),
- elevation and shape of the bell, if any,
- quality of the founding stratum, (This should be done by visual inspection whenever possible. For high capacity units, coring and *in-situ* testing of the material to a depth of 1 to 2 diameters below the base of the unit is recommended,),
- cleanness of bottom and sides of the excavation and permanent liner, if any,
- rate of seepage into the excavation,
- quality of the bentonite slurry, if any, and
- losses of bentonite slurry, if any, (Time, elevation and quantity.)

7.5.6.3. *Concreting*

After the excavation has been inspected and accepted, placing of reinforcing and concrete may proceed. Items to be checked or noted include:

- general information: date, weather conditions, unit identification, time concreting was started and completed,

- quality of the concrete; mix; slump; freshness; that there are test cylinders for each truck load, for any suspect batch and at least three for each foundation unit,

- adequacy of the placing method, proper position of the pouring chute or tube, (Whether or not the bottom of the tremie pipe was always kept below the surface of concrete being placed,),

- that reinforcing and position of the reinforcing cage conform to the drawings and specifications,

- that weight of the concrete is adequate to balance existing groundwater pressure,

- quantity of concrete compared to the height of shaft,

- concrete level in the casing during casing withdrawal,

- vibration of the top 5 to 10 ft of concrete if the concrete has a slump less than 4 in.,

- elevations of cutoffs and exact lengths of units,

- spot checking of completed units by NX corebarrel, inspection of core and borehole by methods such as borehole camera, caliper logging, ultrasonic logging, if specified, and

- correct location of the completed unit.

REFERENCES

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CHAPTER 8

COMMENTARIES

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COMMENTARY 8.1

THE STANDARD PENETRATION TEST

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COMMENTARY 8.1

THE STANDARD PENETRATION TEST

HISTORY

The Standard Penetration Test can be used for sampling most soils. Its main use, however, is in evaluating the *in situ* engineering properties of fine grained granular soils. The introduction in the United States in 1902 of driving a 1-in. diam. open-end pipe into the soil during the wash-boring process marked the beginning of dynamic sampling of soils. Between the late 1920's and early 1930's the test was standardized using a 2-in. O.D. split sampler spoon, driven into the soil with a 140 lb weight having a free fall of 30 in. The blows required to drive the split spoon sampler a distance of 12 in. is referred to as the N value or Penetration Index. This procedure has been widely accepted internationally with only slight modifications.

PROCEDURE

Details of the split-barrel sampler and procedure for the Standard Penetration Test are described in CSA A119.1-1960.

The Standard Penetration Test is extremely useful in site exploration and foundation design. SPT results in exploratory borings give a qualitative guide to the *in situ* engineering properties and provide a sample of the soil for classification purposes. This information is helpful in determining the extent and type of undisturbed samples that may be required.

COHESIONLESS SOILS

TERZAGHI and PECK (1948 and 1968) have suggested that the SPT index N can be related to the *relative density** of sands, as shown below. They emphasize that this relationship should be used with caution and only with carefully controlled tests.

No of blows, N	Density
0 - 4	*Very loose*
4 - 10	*Loose*
10 - 30	*Medium*
30 - 50	*Dense*
Over 50	*Very dense*

This is an empirical relationship. Since its introduction in 1948, it has been and is being misused to establish data on granular soils far beyond the scope of its original intent.

The above relationships were developed for sand deposits above the water table. The influence of submergence on SPT results has not been fully investigated. In some cases submergence reduces the penetration resistance. Reduction of the N value for submerged sands, as proposed by PECK, HANSON and THORNBURN (1953 and 1974) may not be warranted in all cases.

COHESIVE SOILS

TERZAGHI & PECK (1948 and 1968) have also suggested the following crude relationship between the penetration index N, consistency and unconfined compressive strength q_u of clay soils.

* For a discussion of *relative density*, see Commentary 8.2 of this Manual.

No of blows, N	Consistency	q_u ton/sq ft
Below 2	*Very soft*	Below 0.25
2 - 4	*Soft*	0.25 - 0.50
4 - 8	*Medium*	0.50 - 1.00
8 - 15	*Stiff*	1.00 - 2.00
15 - 30	*Very stiff*	2.00 - 4.00
Over 30	*Hard*	Over 4.00

It is emphasized that the results obtained from this test be supported by compression strength tests.

FACTORS AFFECTING THE STANDARD PENETRATION TEST

For all of its wide use and simple procedure, the results of the SPT are greatly affected by the sampling and drilling operations. In addition, it is generally recognized that in granular soils of the same density blow counts increase with increasing grain size.

Improper drilling and sampling procedures which can affect the N values are listed in Table 8.1-A.

TABLE 8.1-A

Inadequate cleaning of the borehole	SPT is not or only partially made in original soil. Sludge may be trapped in the sampler and compressed as the sampler is driven, increasing the blow count. (This may also prevent sample recovery.)
Not seating the sampler spoon on undisturbed material	Incorrect N values obtained
Driving of the sampler spoon above the bottom of the casing	N values are increased in sands and reduced in cohesive soils
Failure to maintain sufficient hydrostatic head in boring	The water table in the borehole must be at least equal the piezometric level in the sand, otherwise the sand at the bottom of the borehole may become quick and be transformed into a loose state.
Not using the standard 30 in. hammer drop	Energy delivered per blow is not uniform. British industry has developed an automatic trip hammer not currently in use in North America.
Free fall of the drive weight is not attained.	Using more than 1½ turns of rope around the drum and/or using wire cable will restrict the fall of the drive weight.
Not using a 140 lb weight	Driller frequently supplies drive hammers with weights varying from the standard by as much as 10 lb.
Weight does not strike the drive cap concentrically	Impact energy is reduced increasing N values

Not using a guide rod	Incorrect N values obtained.
Not using a good tip on the sampling spoon	If the tip is damaged and reduces the opening or increases the end area the N value can be increased.
Use of drill rods heavier than 1 in. diam. extra heavy pipe or A rod	With heavier rods more energy is absorbed by the rods causing an increase in the blow count.
Extreme length of drill rods	Experience indicates that at depths over about 50 ft N values are too high due to energy absorbed by the drill rods
Not recording blow counts and penetration accurately	Incorrect N values obtained.
Incorrect drilling procedures	The SPT was originally developed from wash boring techniques. Drilling procedures which seriously disturb the soil will affect the N value, e.g. drilling with cable tool equipment.
Using drill holes that are too large	Holes greater than 4 in. in diam are not recommended. Use of larger diameters may result in decreases in the blow count.
Inadequate supervision	Frequently a sampler will be impeded by gravel or cobbles causing a sudden increase in blow count; this is not recognized by an inexperienced observer. (Accurate recording of drilling, sampling and depth is always required.)
Improper logging of soils	Not describing the sample correctly.
Using too large a pump	Too high a pump capacity will loosen the soil at the base of the hole causing a decrease in blow count.

CONCLUSIONS

For the foregoing reasons, it is readily apparent that the accuracy of the Standard Penetration Test is questionable. In addition, unique relationships developed for N value versus an exact density (referred to as *relative density*) should be used with caution. It is, however, an extremely useful and simple test. The extrapolation of SPT results beyond the original purpose of providing a guide to the *in situ* density of soil, should be entrusted to experienced geotechnical personnel.

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COMMENTARY 8.2

THE RELATIVE DENSITY OF COHESIONLESS SOILS

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COMMENTARY 8.2

THE RELATIVE DENSITY OF COHESIONLESS SOILS

In the 1975 edition of the National Building Code of Canada, Section 4.2 Foundations and in this Manual, reference to the term *relative density* of cohesionless soils has been avoided. This has been done with full knowledge of the fact that the term *relative density* is of widespread use. The present commentary explains briefly the reasons for such a departure from common practice.

PRESENT METHODS OF MEASUREMENT OF THE RELATIVE DENSITY

DEFINITION

The *relative density* of cohesionless soils is defined as:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

or

$$D_r = \frac{\gamma_{d\ max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\ min}}{\gamma_{d\ max} - \gamma_{d\ min}}$$

The reference unit weights or void ratios corresponding to the loosest and the densest state of the material under consideration are not defined in the strict sense of the word, since they are essentially related to the method used for measuring them. Therefore, there are as many minimum and maximum densities of a given cohesionless material, as there are methods of producing and measuring these densities. A brief investigation of today's practice shows that more than 100 methods are in use, including the ASTM D 2049 Standard method.

MEASUREMENT

Different methods of measuring D_r are available.

Direct Measurement

By means of an appropriate sampling method an undisturbed sample of the cohesionless material is retrieved. The *in situ* density can be measured directly. The sample is then used to determine in the laboratory the minimum and maximum densities by means of an appropriate testing method, preferably the ASTM D 2049 Standard. From these three values, the relative density can be calculated.

The same methods apply to the measurement of D_r at shallow depth where the *in situ* density can be measured directly by the sand-cone, rubber balloon or nuclear method. To be of practical value in design the measurement of all three input densities must be:

- independent of the testing method
- independent of the operator
- of a suitable accuracy.

Recent investigations have shown that none of these conditions are fully satisfied.

Indirect Measurement

It has been suggested by TERZAGHI and PECK (1948) that the Standard Penetration Index N is related to the *relative density* of cohesionless soils, but the proposed relationship was only qualitative in terms of *relative density*:

Standard Penetration Index N (blows/ft)	Relative Density
0 - 4	*very loose*
4 - 10	*loose*
10 - 30	*medium*
30 - 50	*dense*
over 50	*very dense*

Subsequent investigators have proposed "more precise" correlations which supposedly allow the value of D_r to be determined from the Standard Penetration Index. Three sets of such correlations are now available: the most common was proposed by GIBBS and HOLTZ (1957); it has been modified by SCHULTZE and MELZER (1965) and by BAZARAA (1967) but these more recent correlations have not found as wide an acceptance as that proposed by GIBBS and HOLTZ. To be of practical value this method of indirect measurement of D_r must satisfy three conditions:

— The Standard Penetration Index N must be independent of the operator or boring method.
— The correlation of N versus D_r must be sufficiently accurate that the error in D_r is within acceptable limits.
— The correlation of N versus D_r must be the same for all laboratories or engineers using it and for all soils.

Investigations have shown that none of these conditions are fully satisfied.

ACCURACY OF RELATIVE DENSITY MEASUREMENTS

Because of its formulation as the ratio of two small differences between large numbers, the *relative density* is highly sensitive to the errors on each of the three input densities. However, it was not until recent years that the problem of the testing accuracy of the minimum and the maximum densities was considered.

Two investigations have recently been completed; both of which were organized in the form of comparative test programs, where samples of reference materials were sent to different laboratories for testing. The investigation by TIEDEMANN (1971) was limited to 15 U.S. Bureau of Reclamation Laboratories; the more general investigation by TAVENAS ET AL (1973) involved 42 leading laboratories in Canada and the United States.

The results of these investigations allow evaluation of the testing errors for the following cases:

A) Variations between tests within a series performed by a given operator using a given method. This represents the minimum error of $\gamma_{d\ min}$ and $\gamma_{d\ max}$.

B) Variations between tests performed at different laboratories, using a given method. This represents the error involved when using the *relative density* in standard design methods or when comparing *relative densities* as obtained by different laboratories.

ERRORS ON THE MINIMUM DENSITY

The minimum density can be measured most accurately. The ASTM D 2049 Standard method is well accepted and easy to use. Results shown here were obtained using this method, but they are representative of results obtained using any method of measuring $\gamma_{d\ min}$. The errors, expressed in terms of the 95% intervals, i.e. ±2 standard deviations, are given in Table 8.2.1. for two extreme materials and the two cases defined above. Errors applicable to other materials should fall within the values given in Table 8.2.1. For the most common practical case B the probable error on any measurement of $\gamma_{d\ min}$ is about ±4 lb/cu ft.

ERRORS ON THE MAXIMUM DENSITY

The maximum density is difficult to measure accurately. This difficulty applies as much to the ASTM D 2049 standard method of vibratory compaction as to any other method of dynamic compaction. The limiting errors for cases A and B are also given in Table 8.2.1. For the most common practical case B the probable error on any measurement of $\gamma_{d\ max}$ is of the order of ± 7 lb/cu ft.

ERRORS IN THE IN SITU DENSITY

Numerous investigations have shown that the error in any *in situ* density measurement is of the order of ± 2 lb/cu ft which is practically independent of the method of measurement, i.e. sand cone, rubber balloon or nuclear method.

RESULTING ERROR IN THE RELATIVE DENSITY

Any value of *relative density* calculated from measured minimum, maximum and *in situ* densities will be affected by the errors in the input parameters.

TABLE 8.2.1. ERRORS IN MINIMUM AND MAXIMUM DENSITY MEASUREMENTS

DENSITY	SOIL	PROBABLE ERROR IN LB/CU FT	
		FOR CASE A	FOR CASE B
$\gamma_{d\ min}$	Uniform fine sand	±1	±3
	Sandy gravel	±2	±5
$\gamma_{d\ max}$	Uniform fine sand	±1.5	±5
	Sandy gravel	±3	±9

NOTE 1 Case A - Variations within a test series

Case B - Variations between laboratories

NOTE 2 Errors given in the Table are equal to ±2 standard deviations. This interval normally includes 95% of all test results for a given test series.

Table 8.2.2. summarizes the errors on any *relative density* determination. The main conclusions are as follows:

- In no case will the error on a measured value of D_r be less than ± 15%.
- In all practical cases, where D_r is used in conjunction with standard design methods or with empirical correlations to other soil properties the error in D_r will be in excess of ± 30%.
- D_r values obtained using different testing methods cannot be compared.

CONSEQUENCES OF THE USE OF THE RELATIVE DENSITY

The consequences of the demonstrated inaccuracy of *relative density* measurement on the use of this soil parameter have been investigated in detail by TAVENAS (1973). They may be summarized as follows:

CORRELATIONS OF S.P.T. WITH RELATIVE DENSITY

The numerical correlations between N and D_r as proposed by GIBBS and HOLTZ (1957) cannot be used directly since:

- the published correlation is affected by an error equivalent to case A.
- the user will automatically introduce an error equivalent to case B when he uses the value D_r to reproduce samples in his own laboratory.

The qualitative relationship proposed by TERZAGHI and PECK (1948) is therefore the ultimate refinement that can be accepted.

TABLE 8.2.2. ERRORS IN RELATIVE DENSITY MEASUREMENTS

SOIL	PROBABLE ERROR ΔD_r%	
	FOR CASE A	FOR CASE B
Uniform fine sand	±16%	±30%
Sandy gravel	±23%	±45%

NOTE 1 Case A - Variations within a test series

Case B - Variations between laboratories

NOTE 2 Errors given in the Table are in percent *relative density*. They are equal to ±2 standard deviations.

RELATIVE DENSITY AS A COMPACTION CRITERION

A *relative density* of 85% is a well accepted compaction criterion. Considering the errors on D_r it is obvious that:

- the quality of different fills, supposedly compacted to 85% *relative density* under the control of different laboratories, will vary widely.
- it is impossible to demonstrate beyond any doubt that a fill is or is not well-compacted.
- therefore, the *relative density* cannot be accepted as a reliable compaction criterion.

RELATIVE DENSITY AS A DESIGN CRITERION

The *relative density* is an accepted design criterion for foundations on soils sensitive to liquefaction during earthquakes. Present design methods such as those developed by SEED and IDRISS (1971) necessitate a very accurate evaluation of the *relative density* of the foundation soil. This has been demonstrated impossible with the present testing techniques, so that the application of such design methods will have only very limited reliability.

CONCLUSIONS

Because of its formulation and unavoidable testing errors in the three parameters which serve as a basis for the determination of *relative density* of cohesionless soils, the error in any measured value of D_r will be at least in excess of ± 15% and most probably in excess of ± 30%. With such a degree of inaccuracy the *relative density* cannot be used as a quantitative soil parameter in the evaluation of the properties of a natural deposit, as a compaction criterion and even less as a design criterion.

For these reasons, any reference to D_r has been deleted from the 1975 National Building Code of Canada and from the present Canadian Manual on Foundation Engineering.

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COMMENTARY 8.3

THE DESIGN OF FOUNDATIONS ON SWELLING AND SHRINKING CLAYS

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COMMENTARY 8.3

THE DESIGN OF FOUNDATIONS ON SWELLING AND SHRINKING CLAYS

GENERAL

Many natural and man-made deposits of soils which contain substantial proportions of clay mineral particles have potentials for swelling or shrinking with change in water content. The degree to which these potentials are developed and the rate with which volume changes take place are governed by the environmental changes to which these soils are subjected. The magnitude and direction of volume change will depend on many factors, including the mineralogy of the clay minerals present, the relative proportion of active clay-size particles to non-clay particles, the initial moisture and stress conditions of the soil, the new environmental conditions imposed on the soil, and the time available for response by the clay.

Traditional foundations for light structures on these clays usually have very large safety factors with respect to bearing capacity or settlement but often give poor service because they transmit large distortions to the superstructure. These distortions arise from sizeable volume changes in soils below or around the structure caused by external forces of climate and vegetation or reactions by the soils to changed effective stresses and temperatures due to the influence of the structure. In soils of medium to high potential volume change, the foundation design will likely be governed by the need to limit distortions caused by these types of reactions rather than by classical shearing strength-bearing capacity or consolidation-settlement limitations.

FOUNDATIONS ON ACTIVE SUBSOILS

THE ACTIVE ZONE CONCEPT

The active layer is a useful term in permafrost studies to denote the maximum depth of subsurface material which freezes and thaws annually. In the definition of this term it is further recognized that the depth of the active layer is not a fixed dimension at any location but can vary yearly or after any disturbance of the area resulting from development or occupancy.

The term *active zone* is proposed as the key term in a new concept to describe the dynamic environment around structures on or in potentially active subsoils. The *active zone* is considered to encompass all of the subsoil mass around and below a structure which is or will be appreciably affected by the presence of the structure. Included in these effects are cyclic or long term changes in soil moisture contents, soil volume changes, ground water levels, effective stresses, shear strength, soil temperatures, soil chemistry and frost action.

Although the concept of considering shallow foundations with respect to the properties and extent of a potentially *active zone* is similar for subsoils susceptible to these other effects, this commentary will be confined to the subject of swelling and shrinking subsoils within the *active zone* near shallow foundations.

TWO FOUNDATION DESIGN APPROACHES

There are basically two approaches to providing foundations for swelling and shrinking subsoil conditions. For the majority of small buildings, it has been traditional practice to found these on relatively shallow spread footings. Through evolutionary development, in areas of active subsoils, combinations of structural strength in the foundation, adjustable columns and maximum flexibility in the frame, partitions and cladding of superstructures have somewhat improved the performance of light structures. These measures to resist and accommodate vertical deformations usually include reinforcement of perimeter walls to form deep beams, the provision of adjustable length interior columns carrying the main beams and partitions, and the widespread adoption of wood frame construction with careful attention to fastening of plaster board interiors.

This first approach has been reasonably successful over short periods of service in reducing damage to small buildings on subsoils of from low to moderate swelling or shrinking potential (Fig 8.3.1.). Over longer service periods it has not successfully coped with basement floor heaving, differential movement or general tilt of perimeter footings, nor has it provided satisfactory long-term performance of buildings on subsoils of high to very high volume change potentials (Fig 8.3.2.).

A rational engineering design approach is now fairly commonly taken for foundations of somewhat larger commercial, institutional and residential buildings. In these it is common practice to utilize deeper foundation units which induce little or no differential movements in the superstructure. Usually these foundations are designed to develop their bearing capacity in stable ground conditions below the *active zone* (Fig 8.3.3.). Trouble-free performance from these foundations still requires strict attention to many design and construction details including: sufficient tensile strength in bearing walls and piles to resist uplift forces and in foundation beams and walls to resist horizontal and vertical forces; void spaces maintained between the soil and all grade beams, pile caps, footings and structural floor systems; and, special attention to connections and transitions between the main structure and all ground supported appurtenances, such as door steps, sidewalks, driveways, tunnels, planters, water, sewer, gas, power and communication conduits. The large differential movements of the latter are usually sharply contrasted against the stationary structure unless adequate transitions or flexible junctions are provided.

ENGINEERING PROPERTIES OF SWELLING AND SHRINKING CLAYS

EXPANDING AND NON-EXPANDING CLAY MINERALS IN CANADA

The nature, origin, occurence and properties of clay minerals in natural soils is a very complex subject but, fortunately, the immense glacial processes which have reshaped most of Canada's surface have somewhat simplified the problem of classification of "problem" soil deposits because of their relative uniformity and massive proportions. Most of the clay rich deposits of concern in the construction and performance of structures are of relatively recent origin in geologic terms having been laid down by glacial and post-glacial processes in the last few thousand years. Most of the inland, clay-rich soils are found either in lacustrine lakes or in glacial drift and their chemistry has been altered little from that of the preglacial sources from which they were derived. Low natural temperatures and little or no leaching have left unaltered most of the subsoil mineralogy.

Illite and chloritic mica are reported as the predominant minerals found in many lacustrine and glacial drift deposits derived from older sediments of marine origin. Soils consisting of these and other non-clay particles are generally considered to be non-swelling but may be subject to large shrinkage upon drying or spectacular reduction in shearing strength if their high void ratios and flocculated microstructures are changed by drying or remolding. The infamous Leda clay of the Ottawa Valley and St. Lawrence lowlands is one of a number of such clays which were laid down in marine or brackish waters.

Bentonite and the montmorillonitic shales of the Cretaceous formations of the interior great plains of North America have provided the very active clay minerals which give rise to large, reversible swelling and shrinking properties of the lacustrine clays found in what were once some of the worlds largest glacial lakes, including Agassiz, Regina and Edmonton, and many others throughout the western prairies.

Unfortunately, the natural and man-made climatic and vegetative conditions of the regions in which these deposits are found tend to accentuate their potentials for adverse reactions. In the more humid areas, the clays sensitive to shrinkage have not previously been subjected to drying to the extent now occurring due to construction and the introduction of new vegetation. In the more arid regions, the expansive clay types are now often subjected to new wetting conditions which have not been equalled or exceeded since their emergence as land from lake bottom.

The potential volume change of clay-rich soils can be satisfactorily classified from results of Atterberg limits and grain-size tests (Fig 8.3.4.).

VOLUME AND DIMENSION CHANGES

Clay-rich soils undergo first episode shrinkage which is directly proportional to reduction in moisture content from the depositional moisture content to almost the shrinkage limit. The resulting volume change can be in excess of 50% in soils

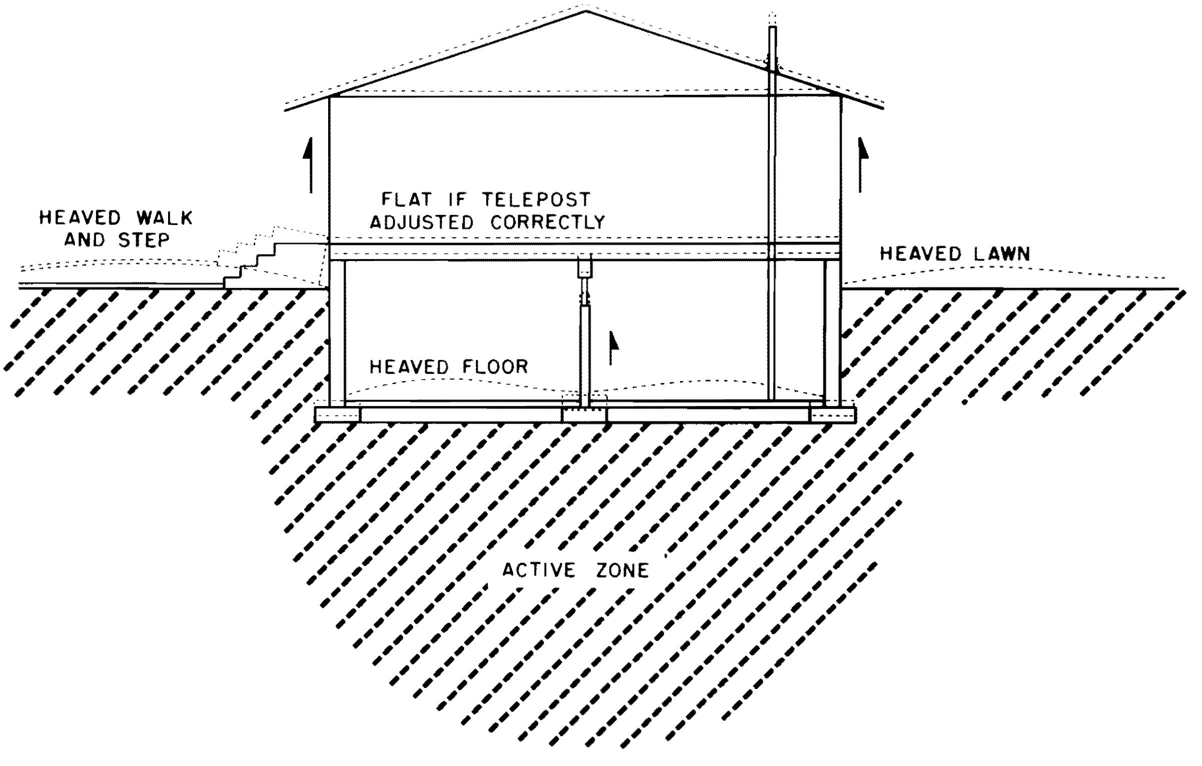


FIGURE 8.3.1

TYPICAL SHORT TERM SHALLOW FOUNDATION PERFORMANCE ON DEEP DEPOSIT OF ACTIVE CLAY SUBSOILS

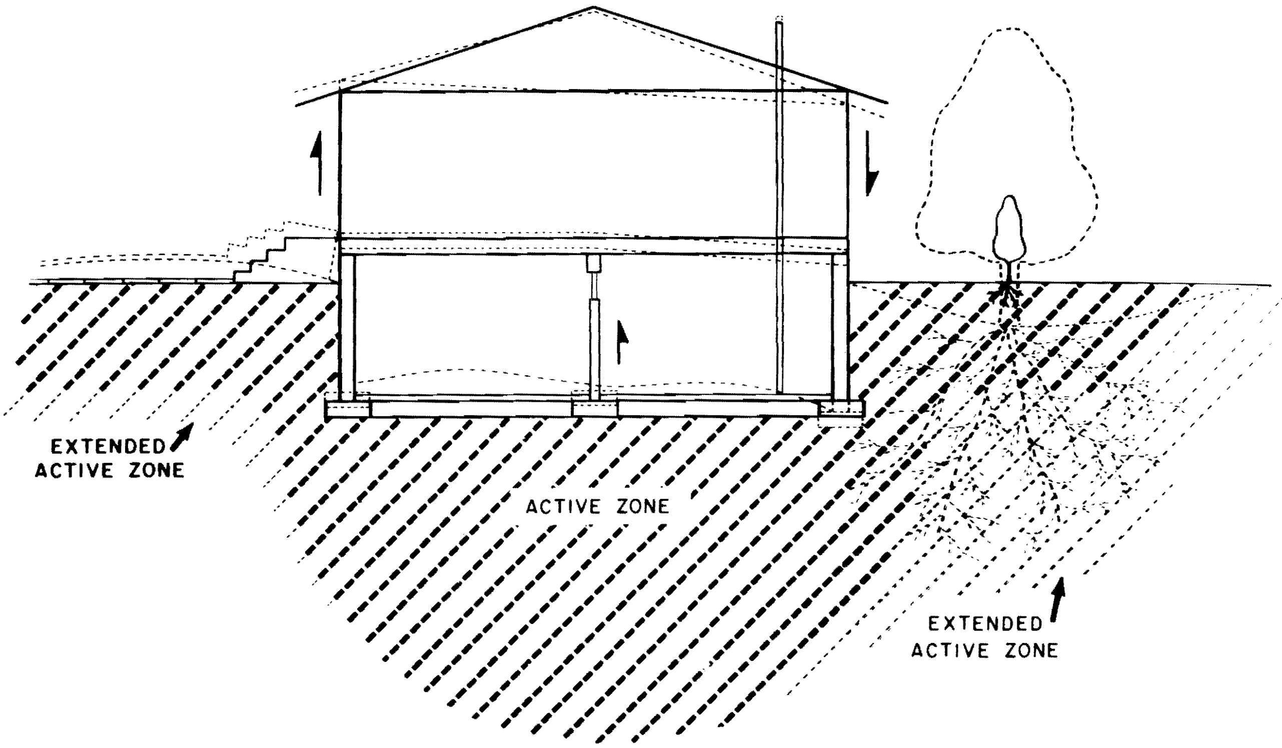


FIGURE 8.3.2

TYPICAL LONG TERM SHALLOW FOUNDATION PERFORMANCE INCLUDING INFLUENCE OF A DEEP ROOTED TREE

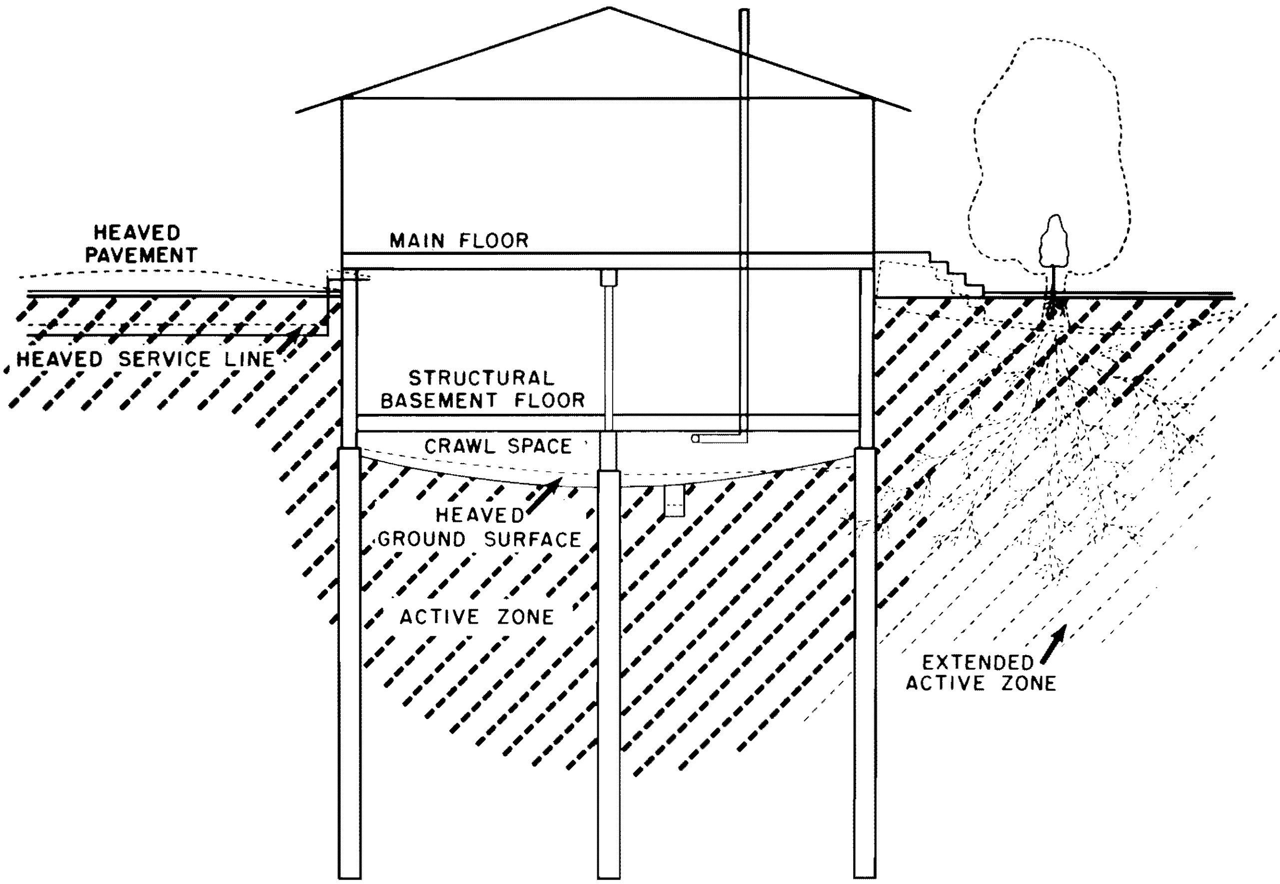


FIGURE 8.3.3

TYPICAL LONG TERM DEEP FOUNDATION PERFORMANCE

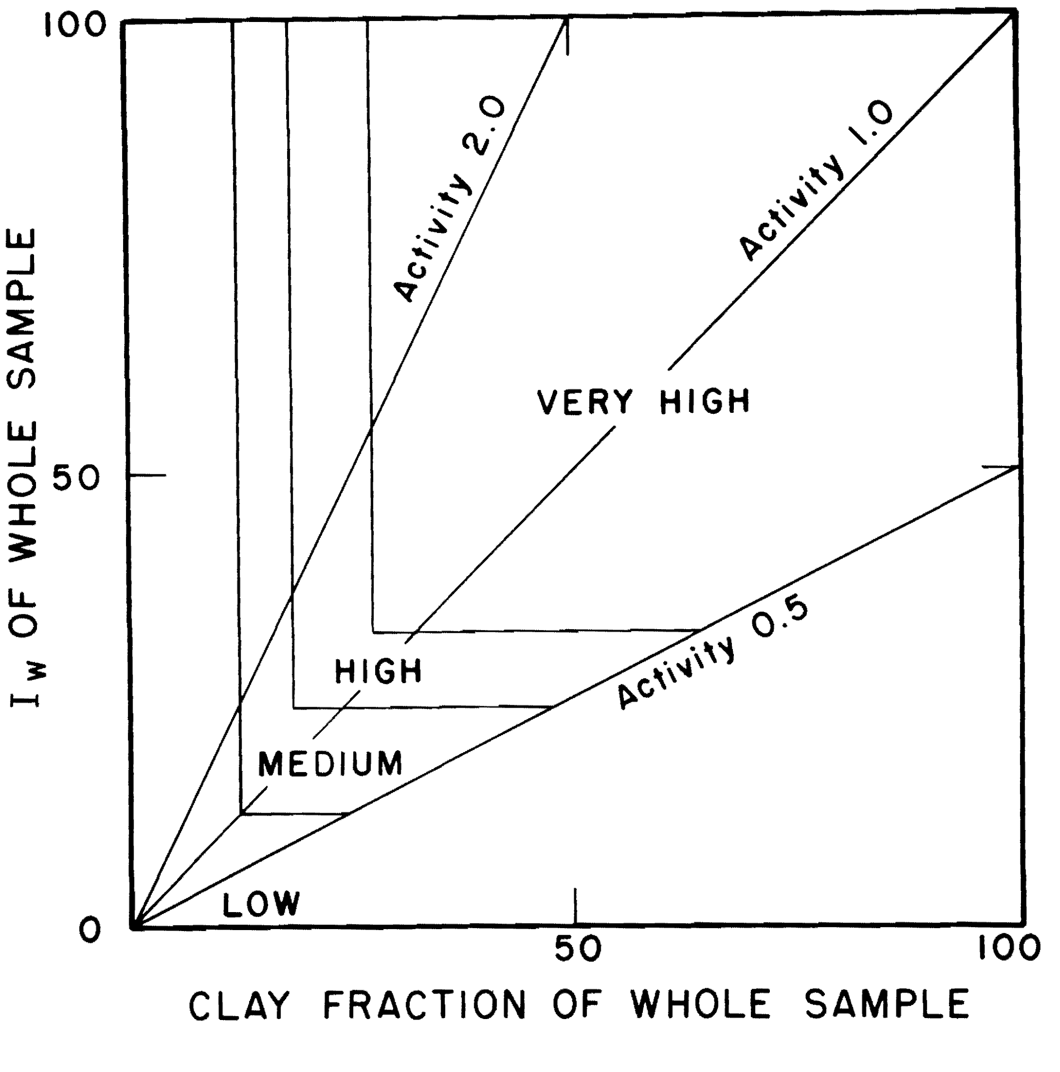


FIG 8.3.4 (After Williams)

VOLUME CHANGE POTENTIAL CLASSIFICATION FOR CLAY SOILS of high initial void ratio. Large permanent horizontal and vertical dimension changes take place during the initial shrinkage of flocculated clays. During rewetting of non-expansive type clays, the rebound expansion is very much less than the original shrinkage (Fig 8.3.5.). The permanent set has been attributed to re-orientation of particles. Reversible shrinking and swelling behavior occurs only after severe drying and large reduction from original volume.

The volume-moisture content relationship for expansive clay soils is reversible over a wide range of moisture content or stress changes (Figs 8.3.6. and 8.3.7.). For laminated or varved clays, the vertical dimension changes may exceed those in the horizontal direction by a factor of three, or more. The moisture content-volume and stress-volume relationships are hysteritic and, hence, it is found, for example, that confining pressure is quite effective in reducing swelling (See Fig 8.3.8. for test results on Regina clay soils). Combined field and laboratory experience with specific natural clay soils provides the best estimates of end-point equilibrium moisture contents, volumes and pressures for reswelling against various overburden pressures. This experience can be expressed in depth-reduction factors for unit heaving, as shown in Fig 8.3.9., and equilibrium moisture content-depth plots for various surface exposure conditions, as shown in Fig 8.3.10. These then form the bases for reasonably accurate predictions of maximum vertical heaving with depth for specific soil deposits.

The horizontal component of shrinkage is manifested in fissures and cracks of great diversity in spacing and dimensions. The variability in shrinkage cracking seems to be related to previous shrinkage patterns, the rate and nature of the drying forces, and great complexities introduced by seasonal frost action. In-filling of cracks with debris from above or evaporites from within introduces further complications which cannot be discussed in this short overview of the subject.

In addition to normal thermal volume changes, freezing and thawing of clay-rich soils can cause large volume and dimension changes. Freezing shrinkage has been found to be of significant magnitude in both natural and compacted unsaturated clay soils.

SWELLING PRESSURES

The swelling pressures which can be generated in the vertical direction due to rewetting are usually equal or greater than those generated in the horizontal direction in intact natural soils. Exceptions to this include very heavily over-consolidated deposits in which horizontal strain relief has not been possible and in fissured soils where crack filling has been extensive. For the more usual cases of nearly normally consolidated clays, the vertical swelling pressure is of the same order of magnitude as the matrix suction before wetting. For instance, clay soils dried by plant roots stressed to the wilting point, or by air drying, would be expected to exhibit a swelling pressure of several tons per square foot. The effective stress concept provides a basis for understanding the nature of the problem but, unfortunately, laboratory methods of measuring swelling pressures and/or strains are usually considered to be too complex and costly for most small foundation designs. In many sub-humid and arid regions, the potential vertical swelling pressure is often one order of magnitude larger than the net bearing pressures of small to intermediate sized buildings.

LATERAL EARTH PRESSURES

As discussed briefly above, lateral earth pressures in natural field conditions can vary from zero to greater than the overburden pressure with cyclic or long term changes in soil moisture. When a structure is placed in direct vertical contact with undisturbed soil, it may or may not experience large lateral forces depending on the conditions preceding and at the time of construction. On rare occasions, normally adequate basement walls have been jacked-in several inches and severely cracked over a few seasons by progressive infilling of shrinkage cracks during dry periods followed by expansion during wetter periods.

At the present time there is insufficient field data but adequate theoretical basis for predicting design lateral earth pressures against non-yielding earth retaining structures, such as basement walls. As discussed in 5.4.1.4., equivalent fluid pressures ranging from 30 to more than 120 lb/cu ft are appropriate for backfill soils ranging from freely drained coarse grained soils to medium or stiff clay deposited in chunks.

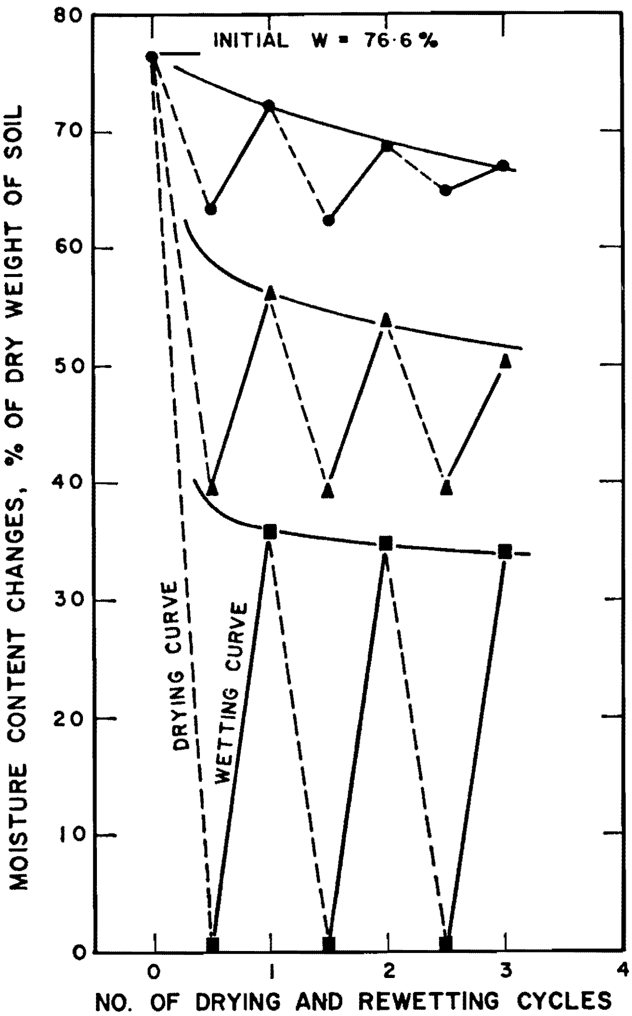


FIG 8.3.5 (After Warkentin and Bozozuk)

EFFECT OF DRYING AND REWETTING ON ULTIMATE MOISTURE CONTENT OF OTTAWA CLAY

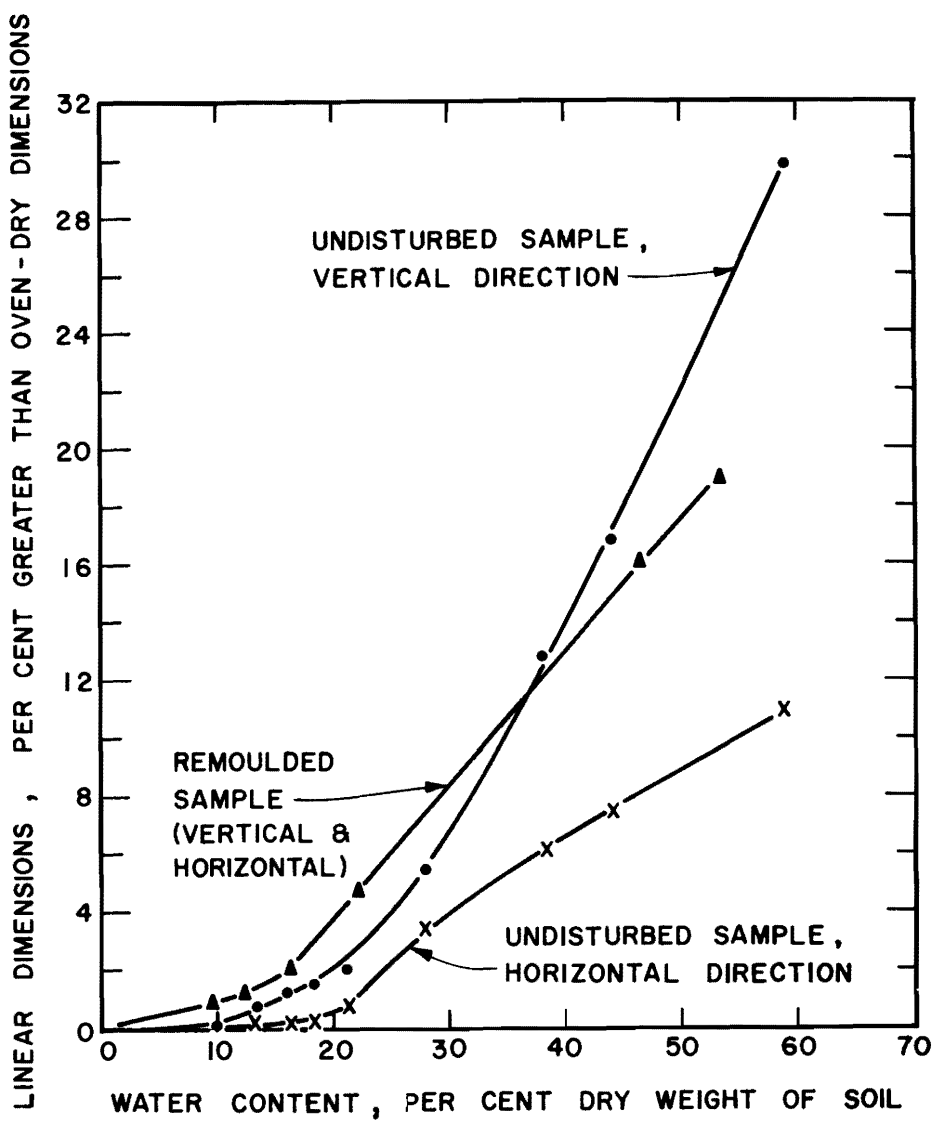


FIG 8.3.6 (After Warkentin and Bozozuk)

DIMENSIONAL SHRINKAGE CURVES FOR SEVEN SISTERS CLAY

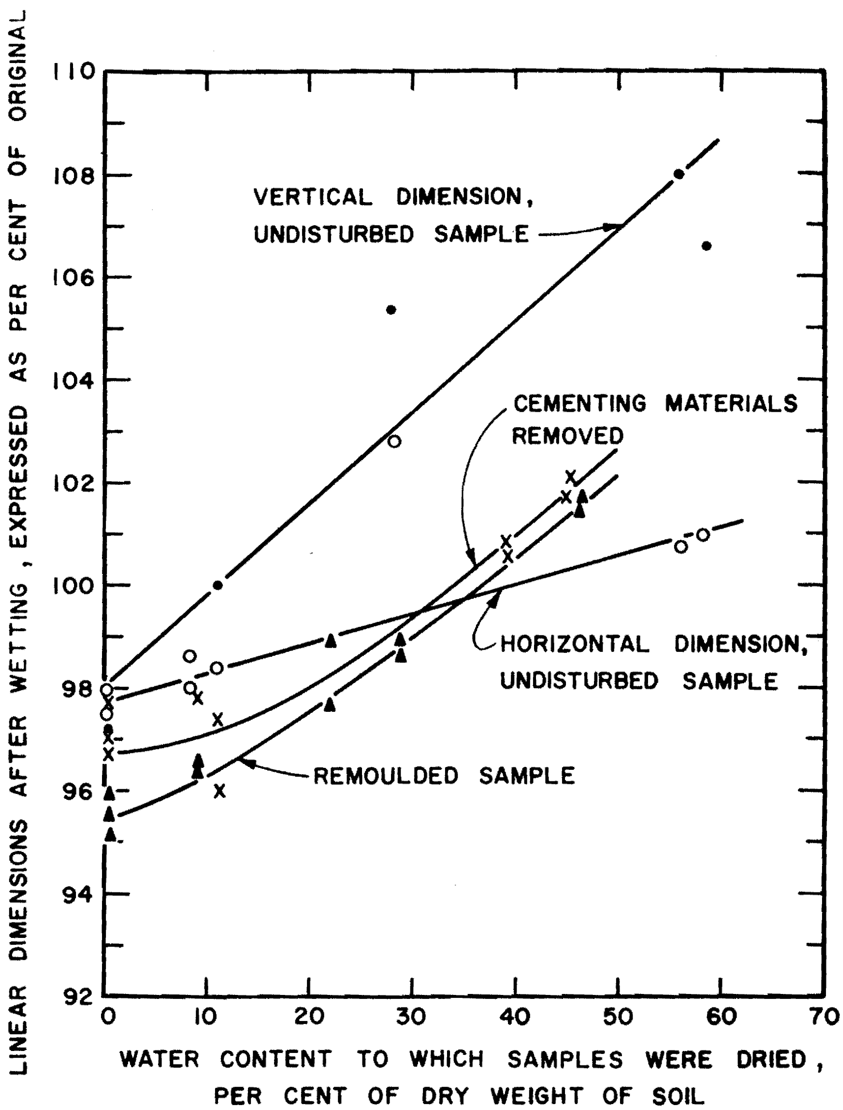


FIG 8.3.7 (After Warkentin and Bozozuk)

DIMENSIONAL REGAIN OF SEVEN SISTERS CLAY SAMPLES ON WETTING AFTER DRYING

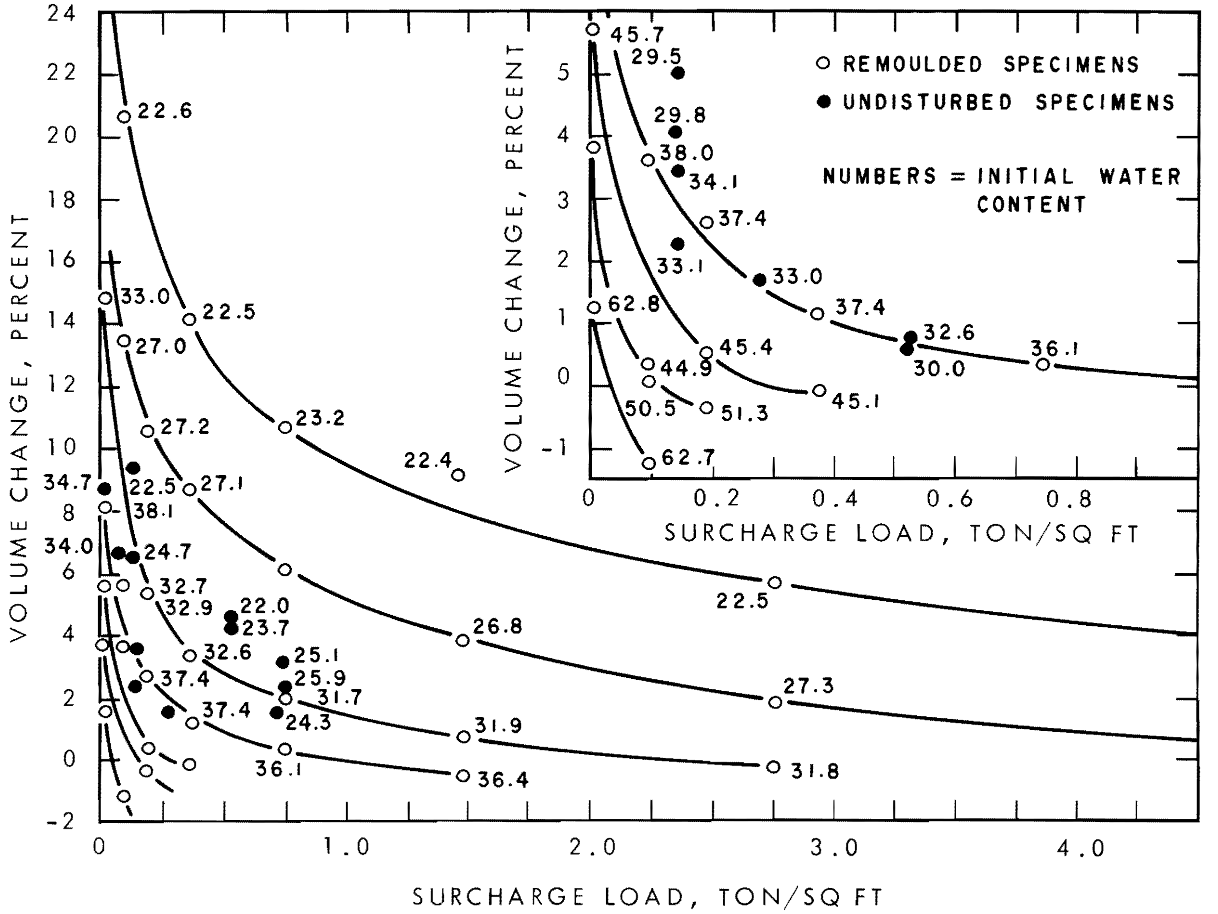
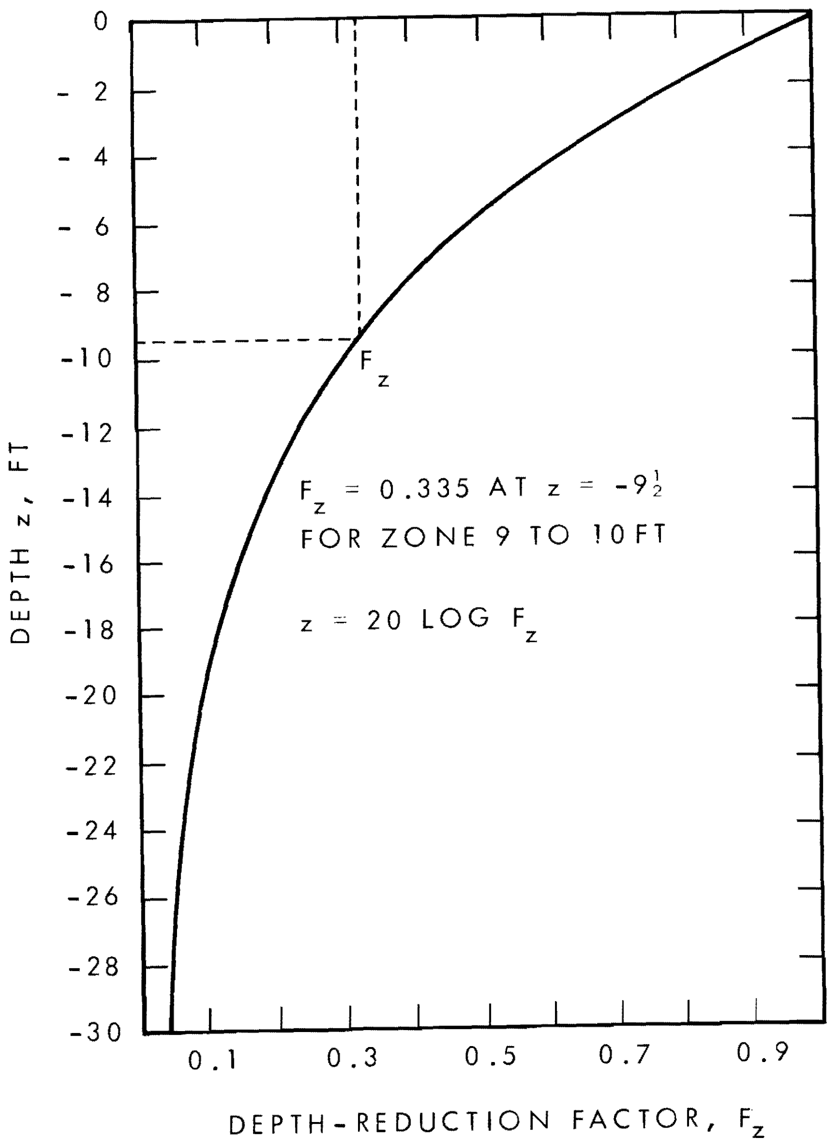


FIGURE 8.3.8 (After Noble)

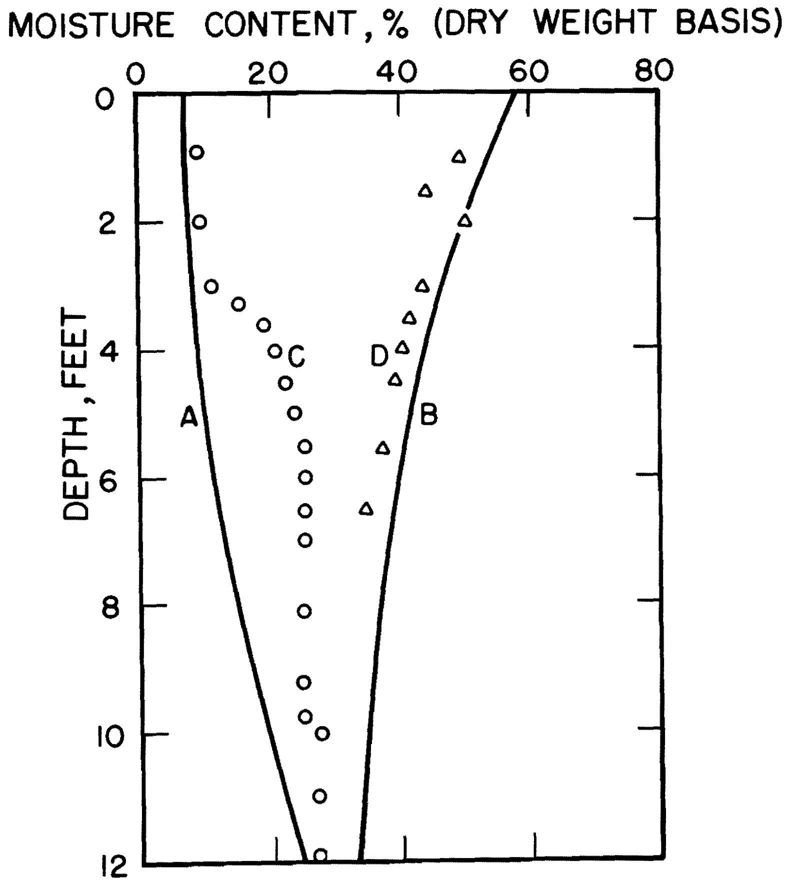
EFFECTS OF SURCHARGE LOAD ON VOLUME CHANGE OF REGINA CLAY



(After Van der Merwe)

FIG 8.3.9

CURVE SHOWING RELATIVE CHANGE IN POTENTIAL HEAVE WITH DEPTH



NOTE:

A EXTREMELY DRY CONDITIONS (HYPOTHETICAL)

B EXTREMELY MOIST CONDITIONS (HYPOTHETICAL)

C SOIL MOISTURE CONDITIONS AFTER 10 YEARS OF DRYING IN AN UNCOVERED CRAWL SPACE (ACTUAL)

D SOIL MOISTURE CONDITIONS AFTER 5 YEARS OF HEAVY LAWN WATERING (ACTUAL)

FIG 8.3.10 (After Hamilton 1969)

PROBABLE EXTREME RANGES OF WATER CONTENT FOR REGINA CLAY

Soils containing substantial proportions of swelling clays should not be placed as compacted backfill against light retaining walls. Unfortunately, the practice of placing clay soils in large chunks and in an uncompacted state against house basements and other shallow retaining walls is widespread because of a lack of more desirable backfill soils in many glacial lake areas. When placed in this manner, the total forces against the backfill soils often exceed the structural strength of lightly reinforced and immature concrete basement walls. In addition, the probability is great of blocked or overcharged drainage systems around these walls and, more for this reason than from a knowledge of actual lateral earth pressures exerted, it is common practice to design for equivalent fluid pressures of from 60 to 65 lb/cu ft. The breakdown and settlement of this chunky backfill is a cause of maintenance problems for many years after construction. The consolidation and later swelling due to moisture change with time of this material, and the addition of fill to restore original grade, can ultimately increase pressures against these walls beyond the 120 lb/cu ft equivalent fluid pressure shown in Figs 5.21 and 5.22.

ASSESSMENTS AND PREDICTIONS OF SWELLING AND SHRINKING POTENTIALS

As a first step, it is useful to classify the potential volume change of clay and silty clay deposits through the use of a chart based on clay-size and plasticity index (Fig 8.3.4.). This separates soils into low, medium, high and very high categories of potential expansiveness. This classification based on simple well established soil mechanics laboratory tests is adequate for preliminary assessments of clayey subsoil conditions. It must be recognized that this classification does not take into account either the conditions of stress or moisture content of the soil at the time of sampling, nor does it indicate changes which may take place in engineering properties of these soils in the new environment around a proposed structure. In some localities, where considerable experience and judgement have resulted in good long-term performance for certain foundation designs, this simple classification of the subsoils may be all that is necessary to organize experience and to call up satisfactory foundations for many small buildings.

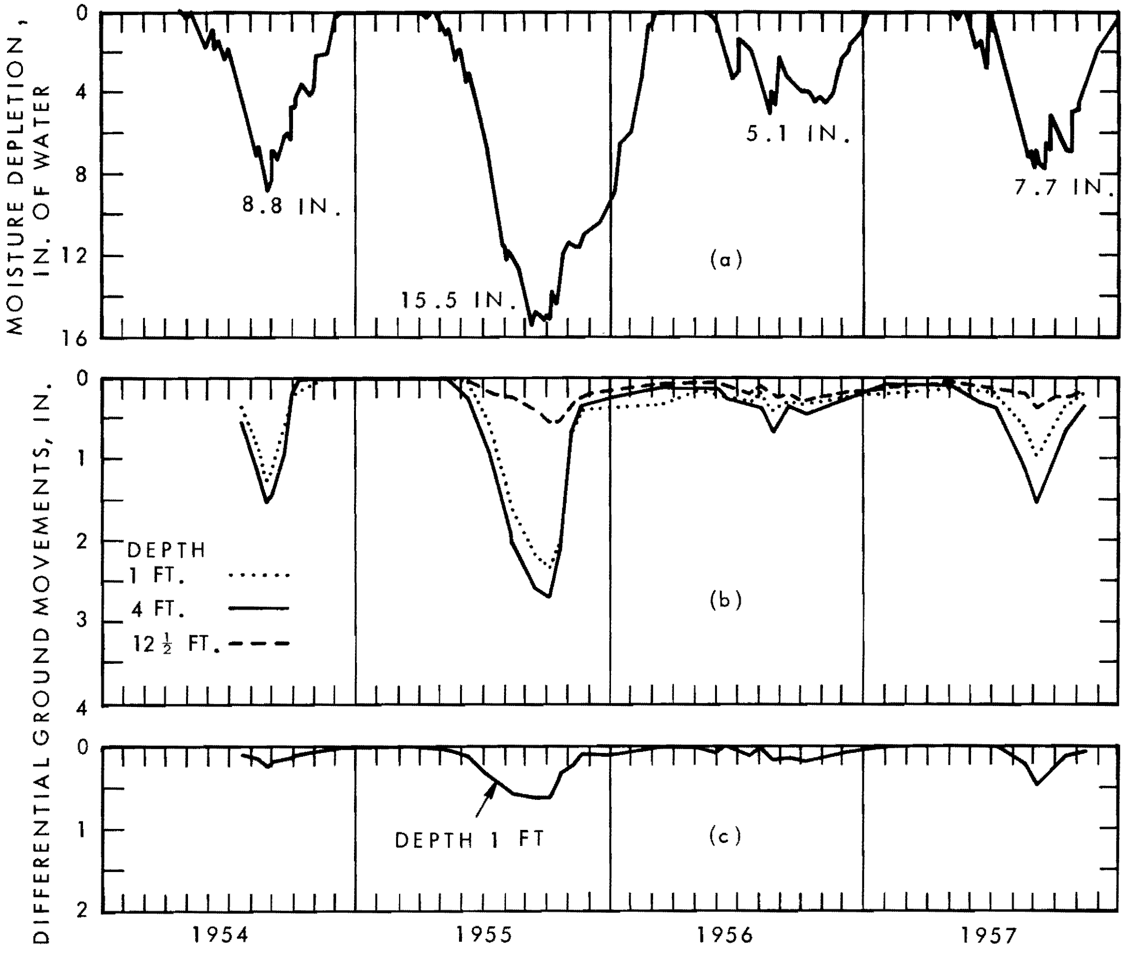
Where more refined assessments and predictions are warranted, more detailed geotechnical investigations are appropriate. Literature on the properties of expansive soils is extensive and many testing and analytical procedures are available for various soil and design conditions. Usually the success of these methods is limited more because of incomplete appreciation of, or ability to predict, the changes in environmental conditions than by any lack in laboratory methods to model specific conditions.

Although the present state-of-the-art in predicting maximum probable heave or settlement is satisfactory for most engineering purposes, methods of predicting the rates at which these volume changes may take place are at a relatively less advanced stage. Laboratory tests can produce heaving rates which are well related to the permeability of intact soils. Field heaving rates are greatly affected by macrostructure of the soil which is difficult, if not impossible, to model in the laboratory, and by the often unpredictable availability of water from surface and subsurface sources.

Field shrinkage rates are affected by the efficiency with which moisture can be removed from subsoils. Evapotranspiration proceeds in a predictable manner when soil moisture contents are very high, (Fig 8.3.11.), but in a much less predictable manner at lower moisture contents because of plant root extensions, plant wilting, soil cracking, etc. First drying or wetting episodes for a soil are much more predictable as to rate and magnitude of volume change than are later cycles because of large hysteresis effects in the volume-moisture content relationships.

BEARING CAPACITY AND COMPRESSIBILITY CONSIDERATIONS

If any generalizations are valid on these properties of the expansive clay soils of the great plains regions, as contrasted with the shrinkable marine clays of coastal lowlands, they might be stated as follows. Except in ground water discharge areas, most of the expansive clays have been subjected to overconsolidation by soil moisture suction and depletion to varying degrees ranging from slight drying to severe desiccation. Preconsolidation pressures range from one half ton to many tons per square foot, and the net loading effects of small to intermediate sized structures seldom, if ever, cause significant consolidation settlements unless serious wetting and softening of the subsoil has taken place during the construction operation. At their normal moisture contents, these clays are stiff to hard, and their shear strengths are usually much above the level of concern for bearing capacity, except in extremely heavily loaded structures. Many of the sensitive marine clays exhibit a drying crust or



(a) SOIL MOISTURE CONDITIONS COMPUTED FROM WEATHER RECORDS
(b) GROUND MOVEMENTS (5 FT FROM 55 FT HIGH ELM TREES)
(c) GROUND MOVEMENTS (40 FT FROM 55 FT HIGH ELM TREES)

FIG 8.3.11 (After Bozozuk and Burn)
SEASONAL VARIATIONS IN MOISTURE CONDITIONS AND VERTICAL GROUND MOVEMENTS IN LEDA CLAY

pseudo-preconsolidation, sometimes attributed to chemical bonding which gives rise to preconsolidation pressures and undisturbed shearing strengths above the normal ranges of concern for classical bearing capacity and consolidation settlement considerations for most small to intermediate sized structures. Some of these clay deposits are found in normally consolidated and weak condition for which their bearing capacity and compressibility must be carefully considered as outlined in Chapter 6.

GROUND AND STRUCTURE MOVEMENTS

OPEN FIELD GROUND MOVEMENTS

Deep deposits of expansive clay soils usually undergo sizeable cyclic ground movements which are often undetected before construction and misinterpreted unless referenced to reliable deep bench marks. The amplitude and periodicity of these movements of the surface and at various depths in the subsoils are manifestations of the net effects of vegetation and climate on subsoil moisture and temperature conditions. In sub-humid to semi-arid regions of western Canada, the annual amplitude of these movements is typically of the order of two to three inches for grass covered, undisturbed profiles (Fig 8.3.11.).

THE EFFECTS OF CONSTRUCTION AND LANDSCAPING ACTIVITIES

Construction and landscaping activities can have very great impact on the magnitude and depth of influence of ground movements. The introduction of deep rooted vegetation in areas where it has not grown previously, or the removal of mature vegetation which has depleted subsoil moisture, has resulted in surface settlement or heaving of the order of one foot in magnitude and extending for great depths and horizontal distances (Fig 8.3.12). Heavy irrigation or changed ground surface covers have had similarly great impact in more arid areas (Fig 8.3.13). Relatively small reductions in total stresses due to lowered grades or excavation have also induced large rebound swelling. Rapid heat flow to or from uninsulated structures has also caused spectacular changes in soil moisture and volume.

TOTAL AND DIFFERENTIAL MOVEMENTS OF FOUNDATIONS

The total movement of a structure is directly related to the effective stress changes in the bearing substrata which occurs during or after its construction, and to the properties of the subsoils within the zone of influence activated by the structure (Fig 8.3.14). This zone of influence will be at least twice as deep as the width of the structure (Chapter 6). If very active soil types are found throughout this depth, then very large total movements can be predicted provided the presence of the structure greatly changes the preceding environmental conditions. For example, if a deep excavation is required for a basement and the weight and flexibility of the structure are such that there is a significant stress reduction (unloading) over the whole area, then the predicted ultimate heaving should be calculated by integrating the heaving for all strata within the zone of influence. In such situations, where all foundation units are placed at the same elevation, it is common for central footings to heave approximately twice as much as perimeter footings because of the unload influence.

Spread footings immediately below deep basements on very active subsoils are usually subject to large total, differential and tilt movements. Slab-on-grade constructions also undergo serious differential movements, often of contrasting appearance to basement movements, with the edges moving more than central areas (Fig 8.3.15).

Deep foundations may reduce or completely eliminate total, differential and tilt movements within structures, but large differential movements of appendages to the building, such as door steps, sidewalks, driveways, fences and service pipes, relative to the main structure should be expected. These may be as large as the predicted heave for the ground surface, and are usually sharply contrasted against the stationary structure. Both shallow and deep foundations on highly reactive subsoils may experience severe differential movements of adjoining constructions. Considerable thought must be given to the difficult problems of transition or increased flexibility at junctions between structures founded at different depths or subject to significantly different environmental conditions.

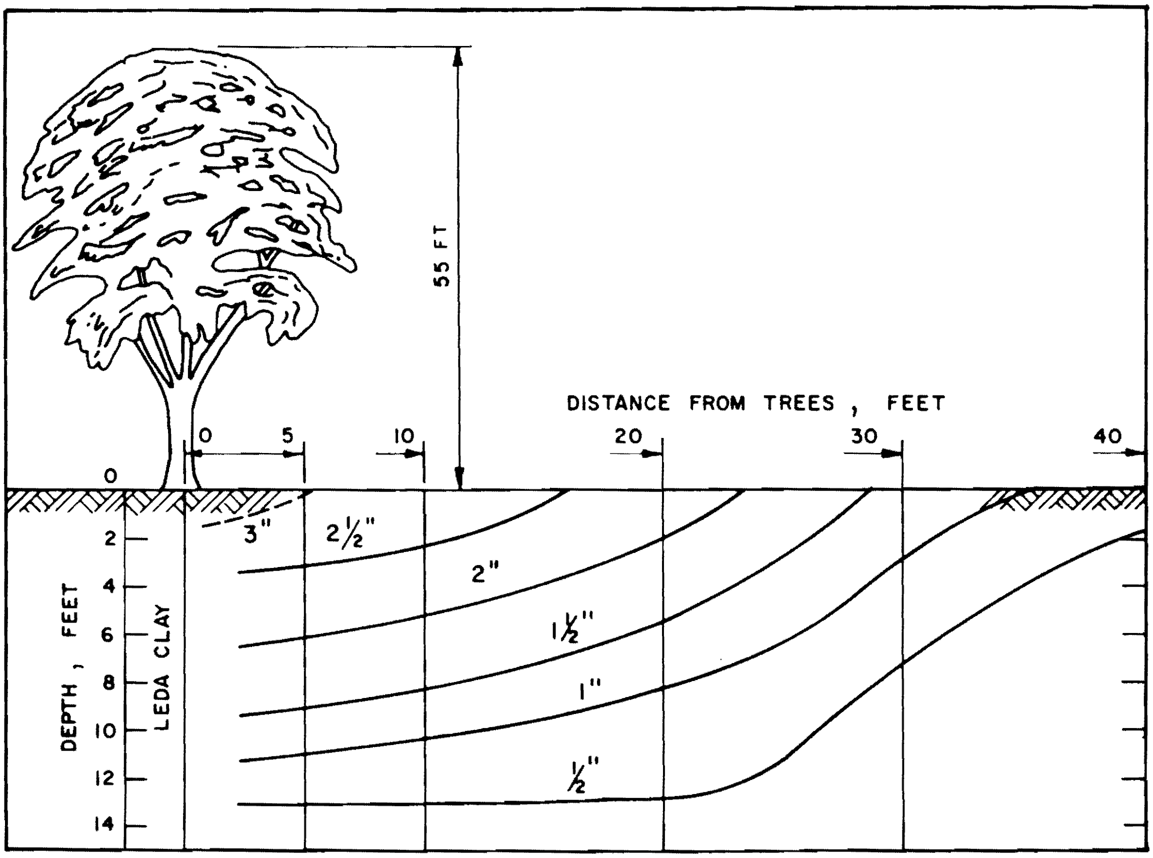
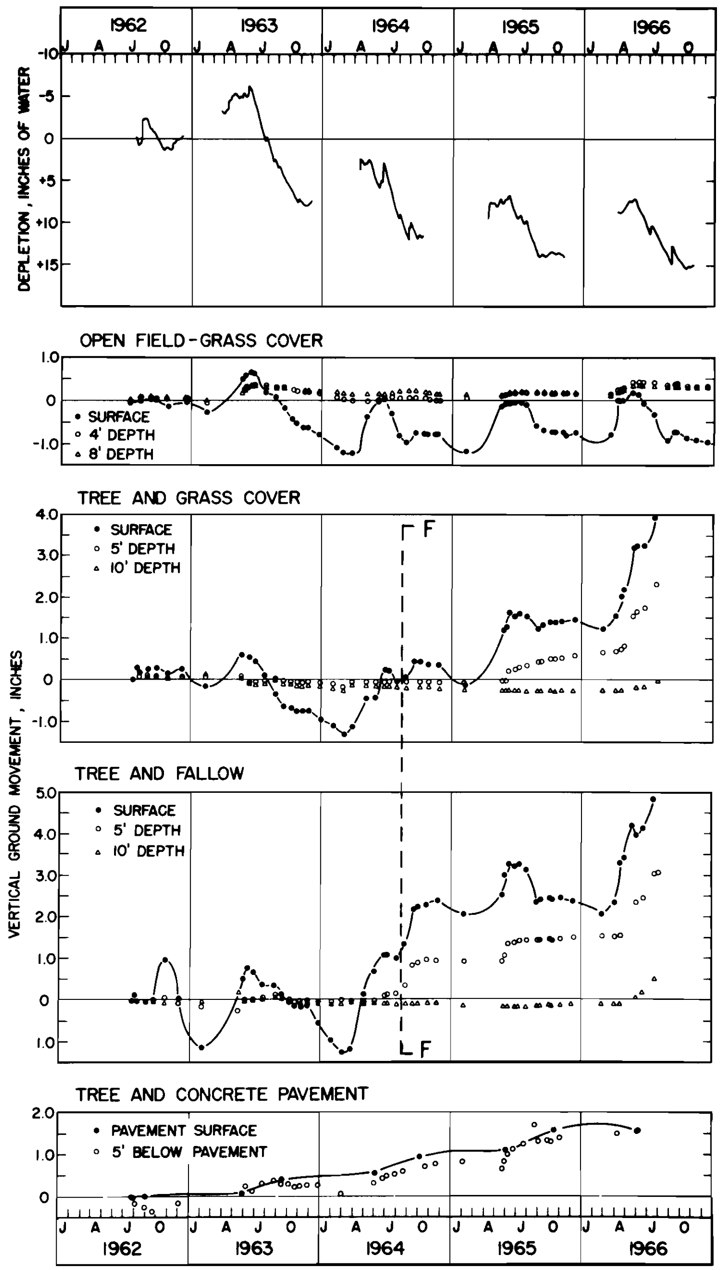


FIG 8.3.12 (After Bozozuk)

VARIATION IN MAXIMUM VERTICAL GROUND MOVEMENTS IN LEDA CLAY NEAR ELM TREES IN 1955



NOTE: F-F INDICATES INJECTION OF FERTILIZER IN VICINITY OF TREE TRUNK

FIG 8.3.13 (After Hamilton 1969)

CALCULATED SOIL MOISTURE DEPLETION AND VERTICAL GROUND MOVEMENTS MEASURED AT THREE DEPTHS UNDER VARIOUS GROUND COVERS AND AT VARIOUS DISTANCES FROM AN ELM TREE IN WINNIPEG

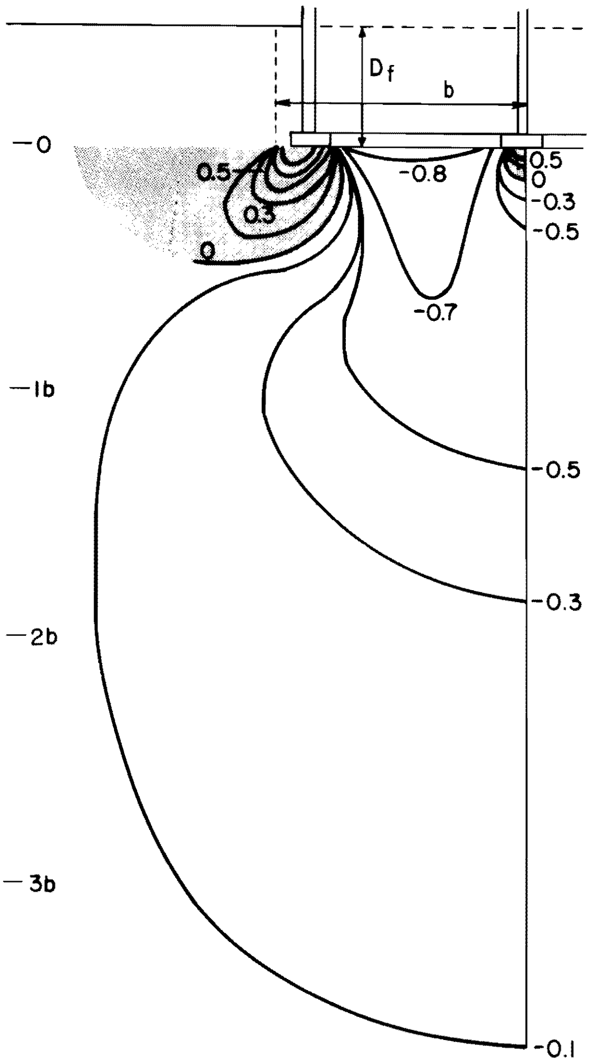
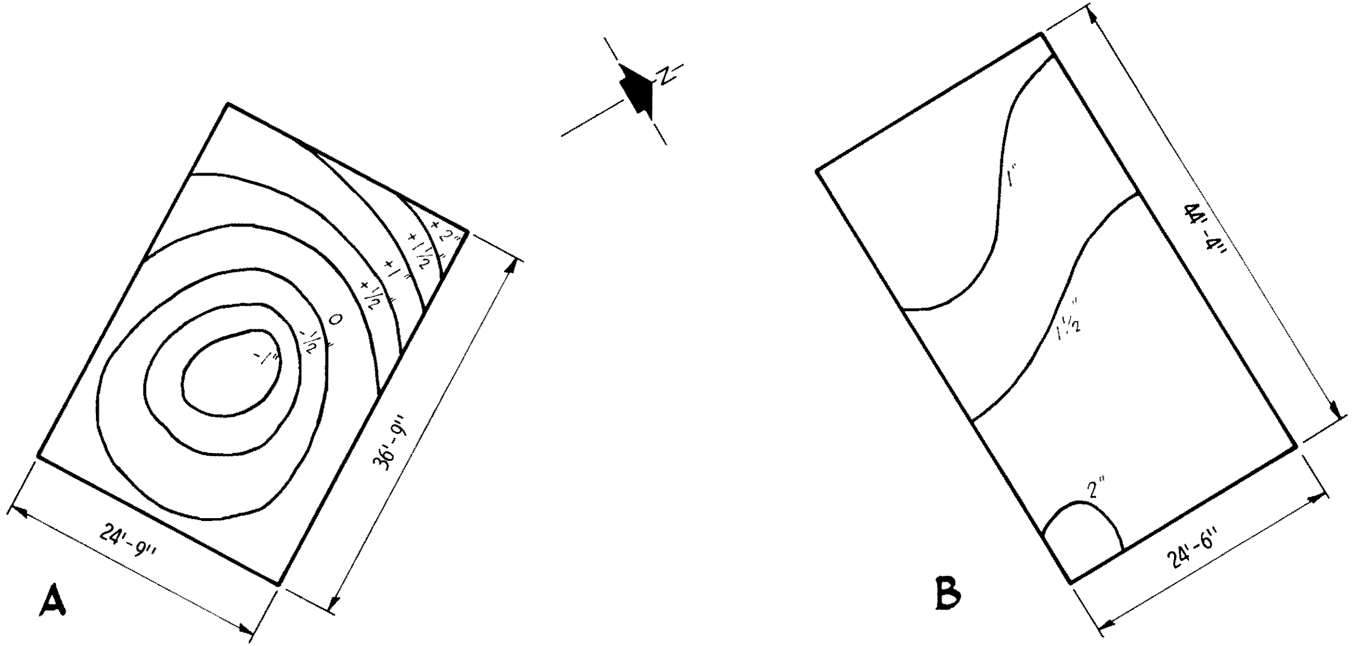


FIG 8.3.14 (After Hamilton 1969)

TOTAL STRESS REDISTRIBUTION DUE TO COMBINED EFFECTS OF EXCAVATION AND HOUSE FOUNDATION LOADS



A: Floor slab contours on July 19, 1960 relative to assumed level condition at time of construction: Oct 28, 1955

B: Floor slab contours on July 9, 1964 relative to level condition at time of construction: July 10, 1960

FIG 8.3.15 (After Hamilton 1965)

CONTOURS OF SLAB-ON-GROUND MOVEMENTS, REGINA, SASKATCHEWAN

KEYS TO GOOD DESIGN, CONSTRUCTION AND PERFORMANCE

The following discussion of this many faceted subject will be divided into two major sub-headings:

- Foundations designed by geotechnical experts, and
- Non-engineered foundations.

FOUNDATIONS DESIGNED BY GEOTECHNICAL EXPERTS

During the past two decades there have been great strides forward in understanding clays and their effects on structures. In most major cities confronted with these problems, there are now at least a few specialists in private practice and governmental agencies who are well equipped with theory and experience to provide designs which will insure satisfactory performance. Many other engineers and architects not specifically in this field of expertise now appreciate the nature of the problems and can refer to the local experts for professional advice. Municipal and provincial building regulations generally permit innovative or non-traditional design approaches when prepared by recognized specialists.

In areas of potentially active subsoils, it is folly on the part of owners or chief designers to proceed beyond very early stages of project planning without the participation of a geotechnical expert. All too often, such expertise is called in long after many important planning and architectural details have been set. The geotechnical expert is often commissioned only to prepare the "soils report" for a specific structure without being given adequate information on the details of the structure, on plans for the surrounding property, or without being given sufficient scope to make the best overall contribution to the success of the project. His report is often obtained and used primarily to satisfy the minimum requirements of the local building regulations, with little or no assurance that his recommendations will be interpreted correctly or heeded in the final design and construction stages. In many building projects, the geotechnical consultant should be the first specialist retained by the owner or his prime consultant. He should be consulted during the assessment of alternate sites and throughout the detailed design, including:

- the selection of depths of excavations,
- elevations of main and basement floors and final grading around the structure,
- location and details for connecting services and structures below grade,
- excavation and shoring procedures,
- details and specifications for waterproofing,
- drainage systems,
- subgrades and
- backfills.

His judgement should also be sought by the contractor on design details and scheduling for construction phases, including excavation, shoring, foundation construction, ground water control, backfilling and protection during adverse weather conditions. In addition to including his report, the detailed drawings and specifications submitted for approval to building authorities should bear the professional seal and signature of the geotechnical engineer indicating that he has been consulted throughout the final design stages and that he concurs with the foundation selection. Similarly, authorities having jurisdiction over planning and approving land development and municipal services should have the benefits of expert geotechnical advice at very early stages of planning in order to make the best possible decisions on optimum land use, surface grading, drainage systems, depths and location of service pipes, and specifications for backfills, subgrades and pavements.

Whenever previous experience or preliminary investigation reveals subsoils of moderate to high volume change potential, the geotechnical engineer should be commissioned to carry out the following additional work to supplement as necessary for design purposes the standard subsurface investigation as outlined in Chapter 4:

- Assess and report on probable changes in volume, strength or stresses in subsoils within the *active zone* around the foundation. This will require investigation of preconstruction, construction, and prediction of post construction, environmental conditions and evaluation of the engineering properties of the subsoils throughout the range of environmental conditions and the projected service life of the structure.

- Provide a selection of foundation design alternatives, together with estimates of their probable life service costs.

- Provide expert guidance during the development of design details and specifications for the excavation, shoring, foundation units, earth retaining structures, subsurface drainage systems, subgrade fills, backfills, surface grades, landscaping, service connections, and bridges or transitions between the main structures and other structures.

- Provide inspection for critical aspects of any of the above construction to insure proper execution and performance, and

- Provide the design, supervise the installation, direct measurements, interpret and report results of any necessary foundation performance monitoring systems.

NON-ENGINEERED FOUNDATIONS

The vast majority of structures built on subsoils of varying swelling or shrinking potentials fall into this category. Many thousands of residential, light commercial and industrial buildings are built each year without the benefit of direct involvement of a geotechnical expert. Many are built in conformance with minimum standards laid down by building bylaws and others are built outside the direct influence of such standards but usually to some traditional standards or rules-of-thumb brought from some other area. Few, if any, structures under $100,000 have the benefit of a comprehensive subsurface investigation and subsequent foundation design by a specialist.

Apart from those structures built on deposits subject to mass movements, i.e., landslides or earthquakes, few low buildings become unsafe for human occupancy in their thirty to fifty year life spans because of inadequate foundations. However, many of these same structures, placed on shallow foundations in moderately to highly reactive subsoils, yield disappointing performance, excessive maintenance costs and short service life. Unlike most of the superstructure, the foundations are nearly impossible for small building owners and operators to maintain or repair themselves. When carried out by qualified contractors, major repairs or replacement of foundations usually cost several times the original cost of the foundation and often ten to twenty times the extra cost of a greatly improved original foundation if designed and built adequately in the first place.

Much of what now appears in the 1970 edition of the Canadian Code for Residential Construction, Sections 12, 15, 16 and 18, should be applied only for stable soils of slight to no potential volume change. For more severe soil conditions, such as described in this commentary, and when more specific guidelines have not been provided, such as described later as part of the subdivision planning, the following suggestions are provided for the guidance of non-specialist designers, builders and building officials, as aids to selecting and detailing satisfactory foundations for moderate to severe swelling and shrinking subsoil conditions. It would appear that the vast majority of foundation selections will continue to be made by non-specialists until such time as technical experts are brought more actively into the planning and decision making system for residential, light commercial and industrial buildings. Considered individually, these "non-engineered" buildings are of small total value but, because of their large number, their aggregate value is probably much in excess of half the building construction expenditure in Canada.

Of prime importance in selecting a satisfactory foundation for potentially reactive subsoils is an understanding of the soil profile and the environmental regime of the site. The depths and potential reactivity of the subsoils, the natural ranges in soil moisture and temperature conditions, and the long-term impact of the building and area landscaping must all be recognized at least qualitatively and the possible consequences provided for in the design.

In humid climates, such as found in coastal areas and the most populous area of Ontario and Quebec, or in ground water discharge areas in all climates, water tables are relatively shallow, soil moisture contents are normally high and natural soils which have not been previously desiccated tend more to shrinkage than to swelling reactions because of their mineralogy, environmental history, and because construction and landscaping usually tend to reduce rather than increase subsoil moisture contents. Rapidly growing, deep rooted trees can cause very severe settlement problems for nearby structures.

In semi-arid to arid areas, when ground water tables are deep, such as in the west central prairies and the interior of British Columbia, acute soil moisture deficiencies and highly reactive soil types usually present very severe swelling problems as a consequence of construction or irrigation of sites. Even within these climatic regions, localized initially wet conditions, such as in ground water discharge areas or in the beds of current lakes or drainage channels, can give rise to future severe shrinkage problems if drying is allowed because of new environmental conditions imposed by land use, structures and vegetation.

Because of the complexities of the soil, climate and vegetation interrelationships, it is difficult if not dangerous to generalize or offer rules-of-thumb to be applied by the non-specialist across this vast country. There is no substitute for local experience and judgement based on the best available scientific principles. It is to be hoped that Canada's expertise in geotechnical engineering will increasingly be applied to solving problems such as those of shallow foundations on swelling and shrinking clays. Until such time as this is general practice and with the understanding that exceptions to rules-of-thumb must always be considered, the following are offered as points to be considered during the selection of foundation types and details for swelling and shrinking soils.

Shallow Spread Footings, Piers and Flexible Slabs on or Near Grade for Heated Buildings

These may be economical and give adequate service for certain structures on subsoils of low to moderate volume change potential in humid to sub-humid regions if reinforced to minimize effects of seasonal edge movements and non-uniform bearing over service trenches, etc., and if free from deep-seated or long term effects of major changes in soil moisture and vegetation conditions. Such shallow foundations will not perform well in more severe environmental conditions. Good practice includes:

- providing positive surface drainage away from the structure by carefully selecting slab surface and outside grade elevations, placing the slab on a thick granular, free draining fill is usually desirable,
- to the extent that is possible, insuring stable uniform soil moisture conditions under and around the foundation,
- exclusion of deep roots and protection against undetected leakage from underground piping or backup through poorly backfilled trenches,
- provision of adequate perimeter insulation to eliminate steep thermal gradients through reactive subsoils under the foundations.

Other precautions worth consideration in superstructure design include:

- utilization of flexible framing, cladding and partitioning construction,
- provision of adjustable length interior columns and slip joints in non-load bearing partitions to accommodate relatively large differential movements; and
- wherever possible, free-spanning of floors and roofs between load-bearing exterior walls and frames.

Crawl Spaces Near or Slightly Below Grade on Shallow Foundations

In addition to the recommendations given above, crawl space designs require special attention to the following:

- provision of adequate slopes for drainage to sumps or drainage tile beds within the crawl space,
- provision of adequate ground cover to control evaporation of moisture from the subsoil,
- provision of adequate heat supply and insulation to prevent frost penetration below footings and to control thermal gradients in reactive soils below and around foundation

units, and to prevent excessive accumulation or drying of moisture in reactive subsoils, and

- provision of adequate ventilation in all seasons to prevent condensation on or within structural materials within the crawl space.

The magnitude of total, differential and tilt movements of shallow foundations will depend on the many factors described earlier relating to the active zone and the reactivity of the subsoils on the site. Even in soils of low volume change potential, some differential movement of perimeter shallow foundation units relative to central units should be expected and provision made for convenient length adjustment of columns supporting central beams and floors. Central load bearing partitions carried directly on strip footings are not recommended unless an effective means can be incorporated for adjusting the elevation of the superstructure below the main floor level.

Deep Basements or Crawl Spaces on Shallow Foundations

The magnitude of total and differential movements experienced by structures on shallow foundations is greatly influenced by net unloading of subsoils as is typical with full basement excavations and light weight one and two storey buildings. Although central footings may be designed to carry equal structural loads and to have similar dimensions to insure similar stress increases in the subsoils, the net area unloading effects of the excavation have much deeper influence and, consequently, deep-seated heaving tends to effect central footings much more than perimeter footings. The precautions suggested earlier, about providing adjustable central columns, partitions and pipes, are, if anything, more important here. Serious attention must also be given by designers to stacks, chimneys, heating ducts, furnaces, and other equipment placed on or through ground supported basement floors. On moderate to very high volume change subsoils, differential heaving of basement floors will likely become excessive for many purposes, and objectionable to many occupants in a period of a few years after construction. This problem can best be attacked at the design stage by providing structural basement floor systems spanning between foundation supports (Fig 8.3.16.), or fully adjustable flooring which can be easily maintained by the occupant or owner. All shallow foundations may be subject to tilt deformations or localized settlement due to non-uniform subsoil reaction to moisture changes or localized influences, such as deep tree roots, leaks, or other localized sources of water.

Grade beams and basement walls which also serve as retaining walls for clay backfills of moderate to high swelling potential should be designed to resist horizontal earth pressures in accordance with the equivalent fluid pressure method, (See the discussion on LATERAL EARTH PRESSURES in this Commentary). Design loads for medium to stiff clays and silty clays deposited in chunks, as given in Figs 5.21 and 5.22 should not be considered too conservative in the light of the limited number of earth pressure measurements available to date.

Deep Foundations in Swelling and Shrinking Soils

This approach is often selected for assured good long-term performance in situations where moderate to severe soil volume changes are anticipated. The details and variations of the methods and materials are many and complex. The approach will likely therefore remain the proper subject for local specialists. As briefly discussed earlier in TWO FOUNDATION DESIGN APPROACHES AND KEYS TO GOOD DESIGN, CONSTRUCTION AND PERFORMANCES in this Commentary, and further in Chapter 7, there are many design and construction details which must be carefully executed to insure good performance. Where experience is limited or lacking with these techniques, specialist professional judgement must be applied and followed up with detailed performance monitoring in order to prove predictions and to advance the state-of-the-art.

GEOTECHNICAL ASSESSMENTS FOR SUBDIVISION PLANNING AND APPROVALS

Where expert knowledge is available, it should be engaged in developing the most economic solutions to problems such as providing suitable foundations for problem subsoil conditions. It would be unrealistic and probably wasteful in areas of relatively uniform stratigraphy, such as the major lacustrine deposits of Western Canada, to require detailed design by a specialist for each and every house foundation. On the other hand, it would be advantageous at the early

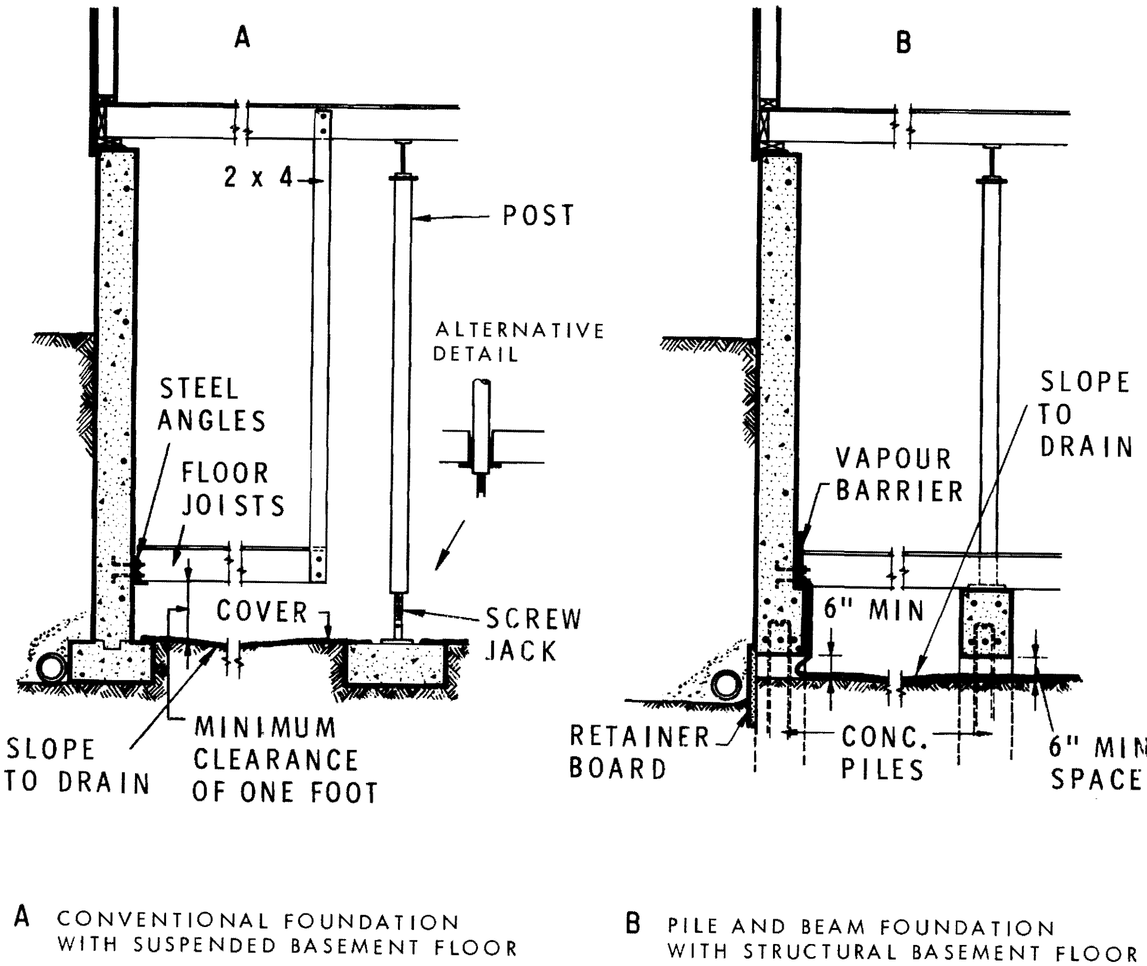


FIG 8 .3.16 (After Hamilton and Handegord)

TYPICAL FOUNDATION AND BASEMENT FLOOR SYSTEMS WHICH OFFER IMPROVED LONG TERM PERFORMANCE development stage of any subdivision or project of more than a few lots, for a geotechnical expert to evaluate the various hazards to foundations and other construction which is involved in urban development. As part of his submission for subdivision approval or development proposal, a developer should be required by the planning authority to provide a comprehensive geotechnical report which describes to the satisfaction of the authority-having-jurisdiction:

- the preconstruction subsurface materials and conditions in sufficient detail for reasonable interpolations at the proposed subdivision scale,
- a comprehensive assessment of potential hazards to good performance of materials, structures and systems which may arise from the geotechnical features of the site and the natural and man-made environmental conditions now in effect and predicted for the future at the site, and
- workable design concepts for various types of foundations, providing examples when necessary to illustrate the recommended foundation practices for this specific development, and guidelines on deciding when foundations for certain structures or ground conditions require individual specific designs by specialists.

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COMMENTARY 8.4

FROST ACTION AND FOUNDATIONS

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COMMENTARY 8.4

FROST ACTION AND FOUNDATIONS

GENERAL

Everywhere in Canada except in south western British Columbia daily mean air temperatures fall below freezing for several weeks or months each winter and except where there is sufficient insulating snow cover the ground freezes to a few inches or a few feet. Ground freezing frequently leads to volumetric expansion of the soil and to heaving of structures located above or adjacent to it. Upon thawing the release of excess water into the soil leads to collapse of the soil structure with great loss of strength. The forces involved in such movements can be very destructive to lightly-loaded structures, but may also cause serious problems in major buildings (CRAWFORD 1968).

Brief descriptions of the phenomenon of *frost action*, its causes, some of the construction problems it presents and steps that may be taken to prevent it are given in this commentary. Some of the comments may be pertinent to the active zone in permafrost regions but in general the solution of construction problems in the north calls for the application of different techniques (JOHNSTON 1975).

A short glossary of terms frequently encountered in the literature on *frost action* is given at the end of this commentary.

EXPANSION OF WATER UPON FREEZING

The change of phase of water to ice results in an increase in volume of about 9%. If water occupies all of the pore spaces in a cohesionless *non frost-susceptible* soil the overall volume increase upon freezing depends upon the relative volumes of water and soil particles but it will be considerably less than 9%. *Frost heave* that occurs under these circumstances may result in minor damage to supported structures but in general the expansion of water upon freezing is of little importance when considering the overall problems of *frost action*.

ICE SEGREGATION AND LENSING

This phenomenon is the basic cause of all problems arising from the freezing of *fine grained soils* and other materials. When the right conditions exist water is drawn to the *frost front* from the unfrozen soil to form distinct layers, lenses or veins of ice which may add significantly to the original water content of the soil. Fig 8.4.1. Formation of ice under these conditions causes large increases in volume which is generally manifested in heaving at the surfaces exposed to cooling. Without physical restraint there is no theoretical limit to the amount of heaving that may occur. Movements in excess of 4 in. on basement floors developing in only three weeks have been recorded. Where restraint in the form of a building load is present heaving pressures develop which may or may not be able to overcome the restraint. Heaving pressures however may be very high; values of 30,000 lbs were measured on a 1 ft diameter plate equivalent to 19 ton/sq ft (PENNER and GOLD 1971) and a seven-storey reinforced concrete frame building on a raft foundation was heaved more than 2 in. when frost was inadvertently allowed to penetrate the soil beneath the foundation.

CONTROLLING FACTORS

For *frost action* to occur it is generally accepted that three basic conditions must exist which are

- the presence of *frost-susceptible* soil
- the availability of water, and
- cooling conditions that cause the soil and water to freeze.

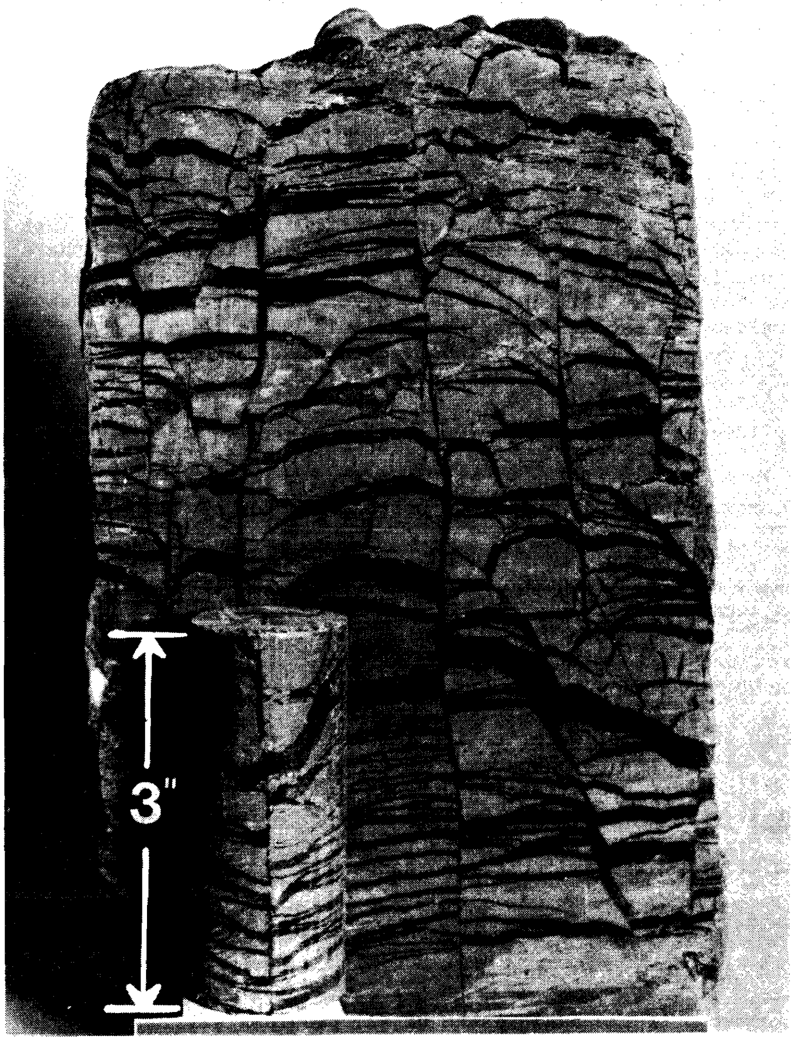

FIG 8.4.1

SAMPLE OF FROZEN CLAY SHOWING ICE SEGREGATION

FROST-SUSCEPTIBLE SOIL

Frost-susceptible soils are those in which there are sufficiently fine pores to support the mechanism of *ice segregation* and the formation of *ice lenses*. Fine pore spaces are related to particle size and to density (PENNER 1968). Several criteria have been devised based upon particle size distribution alone by which it is possible to estimate the frost susceptibility of a soil (TOWNSEND and CSATHY 1963). One of the most widely known of these is that proposed by CASAGRANDE 1932 as follows:

"Under natural conditions and with sufficient water supply one should expect considerable *ice segregation* in uniform soils containing more than 3 per cent of grains smaller than 0.02 mm and in very uniform soils containing more than 10 per cent smaller than 0.02 mm. No *ice segregation* was observed in soils containing less than 1 per cent of grains smaller than 0.02 mm even if the groundwater level was as high as the frost line."

The borderline between soils that are *frost-susceptible* and those that are not is not distinct and those which appear to fall just clear of these criteria should be treated with caution.

AVAILABILITY OF WATER

In order for ice lenses to grow water must be available in the unfrozen soil for movement to the *frost front* (PENNER 1959). Water may be transported in the liquid state, by capillary action and by suction developed by super-cooling at the *frost front* or in the vapour state. In general, if the groundwater table is high with respect to the surface from which heat is extracted, conditions will be suitable for *ice lensing* to occur in *frost-susceptible* soils.

FREEZING CONDITIONS

Temperatures near the ground surface are determined by the balance of heat between that originating in the earth's centre (geothermal heat) and that gained or lost at the earth's surface. During the winter the net effect is one of extraction of heat which in most of Canada results in freezing conditions in the subsoil. The quantity of heat extracted depends upon such climatic factors as air temperatures, solar radiation, snow cover and exposure to wind. Of these, the most effective and significant is air temperature.

To gauge the severity of winter conditions the combined effects of both the duration and intensity of freezing conditions can be estimated directly from air temperature measurements. The cumulative total of the difference between daily mean air temperature and the freezing point is known as the Freezing Index expressed in "°F days" (1 °F day = 1 day for which the mean temperature was 31°F).

Figure 8.4.2. is a map of Canada showing normal values of total Freezing Index for the winter based upon records of 10 to 30 years between 1931 to 1960 from almost 900 weather stations across the country (BOYD 1973). Values vary from less than 100°F days in south western British Columbia, and about 500°F days in Southern Ontario and the south east of Nova Scotia to 7500 F°days in Northern Manitoba and Northern Quebec. Maximum values occur as would be expected in the Arctic archipelago and reach 14,000°F days.

DEPTH OF FROST PENETRATION

The depth to which freezing occurs is related to the rate that heat is extracted which besides being dependent upon climatic conditions is influenced by the thermal properties of the soil which in turn are related to such factors as mineralogical composition, grain size, density and water content.

Elaborate and complex numerical solutions requiring the use of computers are available for determining the depth of frost penetration, but because they generally require making several critical assumptions even when soil thermal properties are known they are of limited practical use.

For most purposes, it has been found that depth of frost penetration can be estimated fairly closely by using one of the correlations between air Freezing Index and field observations which for the most part have been made beneath highways and airport runways where the ground surface

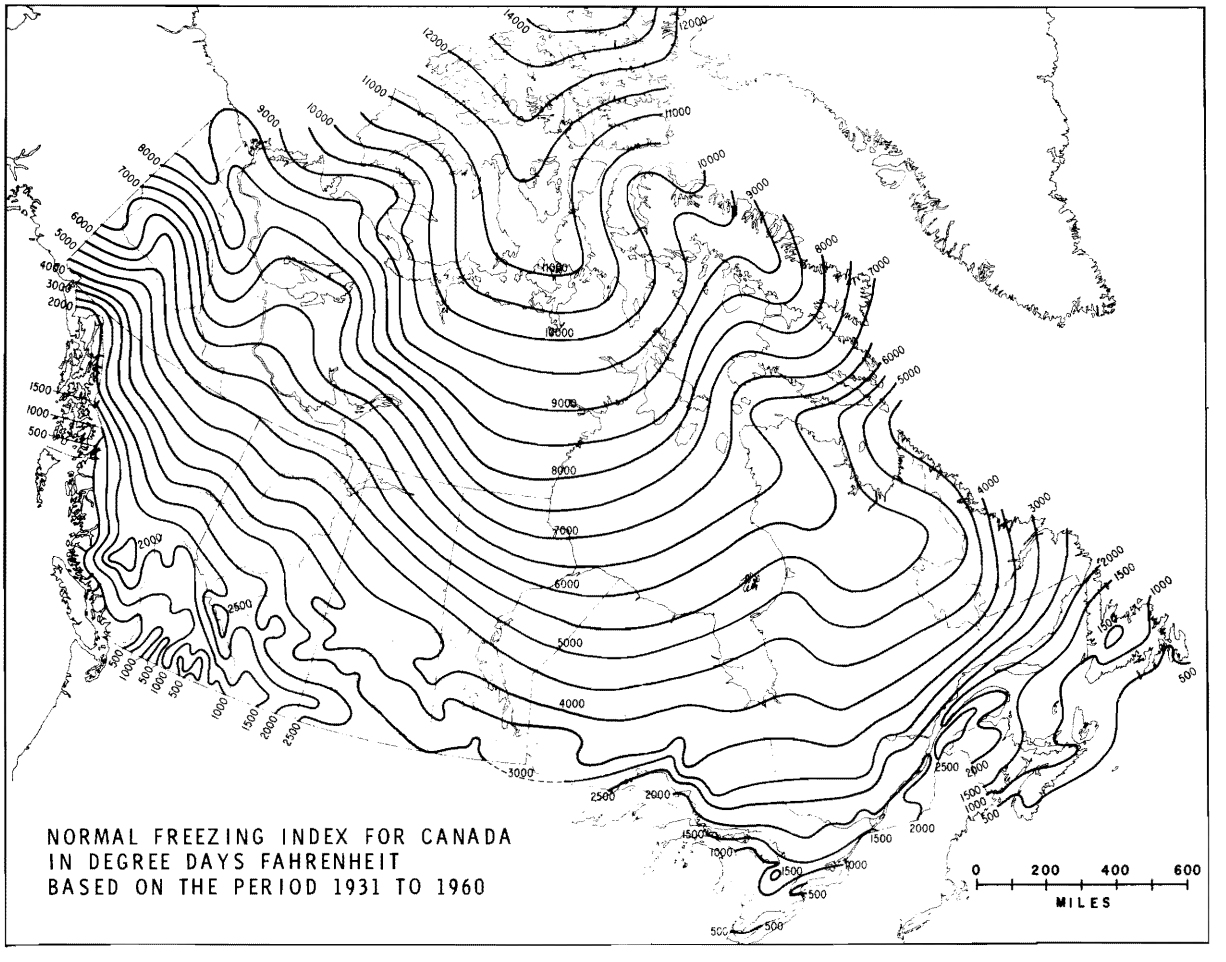

FIG 8.4.2 (After Boyd)

FREEZING INDEX FOR CANADA

was kept clear of snow (U.S. ARMY, CORPS OF ENGRS., 1949, ARGUE 1968). It is known that for the same conditions frost penetration in well-drained cohesionless materials is greater than in *fine grained soils* of higher water content, but these correlations are based upon all available data and do not make any distinction between soil types or drainage conditions.

The correlation shown in Fig 8.4.3. is that by BROWN 1964 based upon both field measurements and theoretical considerations.

FROST ACTION AND FOUNDATIONS

The design of foundations against *frost action* rarely implies incorporating additional structural strength to withstand the stresses that can be generated in *frost-susceptible* soils, but rather the use of techniques to avoid the problem which can be accomplished by eliminating one or more of the factors that together result in *ice segregation* and *frost heaving*.

The conventional approach is that of placing the foundation beyond the depth of expected maximum frost penetration so that the soil beneath the bearing surface will not freeze. This measure alone, however, does not ensure that frost damage will not occur; backfilling the excavation with spoil that is *frost susceptible* may lead to damage resulting from *adfreezing*. Depths at which foundations should be placed are normally determined by local experience as incorporated in building by-laws, but in the absence of such information the data from BOYD (1973) may be used in the correlation by BROWN (1964) to give a safe depth for the foundation.

ADFREEZING

Adfreezing occurs when soil in contact with a foundation wall adheres to the wall surface as it freezes. The soil water changes to ice and a strong bond is formed at the interface. If the soil is *frost-susceptible,* heaving pressures developed at the *frost front* are transmitted through the *adfreezing* bond to the foundation wall resulting in uplift forces that are capable of producing appreciable vertical displacements. Unless the walls are anchored to the footings they may lift from foundation level, or if constructed of concrete block may fail under tension and part near the depth of frost penetration. Relatively little is known of the magnitude of the forces that may be generated, but limited field experiments have shown the bond strength of *adfreezing* average about 15 lb/sq in. for steel surfaces, and about 10 lb/sq in. for wood and concrete. Peak bond strengths reached the 20 to 35 lb/sq in. range (PENNER and GOLD 1971, PENNER 1974).

DRAINAGE

By their very nature *frost-susceptible* soils do not drain well and even though inflow of groundwater may be prevented the quantity of water held in the unfrozen soil is often sufficient to produce significant heaving when drawn upward to the *frost front* by the mechanism of *ice lensing*. Where possible it is good practice to remove *frost-susceptible* soil and replace it with coarse granular material that is easy to drain, and to provide drainage tile around the perimeter of the building which must be connected to some other system for disposal. Such procedures also include the use of less permeable soil near the soil surface and sloping the grade to shed rain. Together the replacement of *frost-susceptible* soil with granular material and proper drainage prevent *adfreezing* from occurring (PENNER and BURN 1970 and 1973).

FREEZING TEMPERATURES AND THERMAL INSULATION

In recent years, with the advent of lightweight plastic insulation, it has become possible to greatly reduce the loss of geothermal heat that normally leads to frost penetration. With the selection of the right thicknesses and its application to the appropriate surfaces of the foundation and soil, temperatures can be kept above the freezing point. The design of such measures around foundations has advanced rapidly in the last few years, but the use of insulation for this purpose should only be undertaken after careful examination of the pertinent conditions and a thorough understanding of the effect it will have on heat flow at the soil-foundation interface (ROBINSKY and BESPFLUG 1973). Insulation is of particular advantage in the design of unheated buildings such as warehouses and garages, and in special facilities for food storage and the maintaining of ice surfaces for winter sports where it is necessary that temperatures inside the building be kept several degrees below freezing.

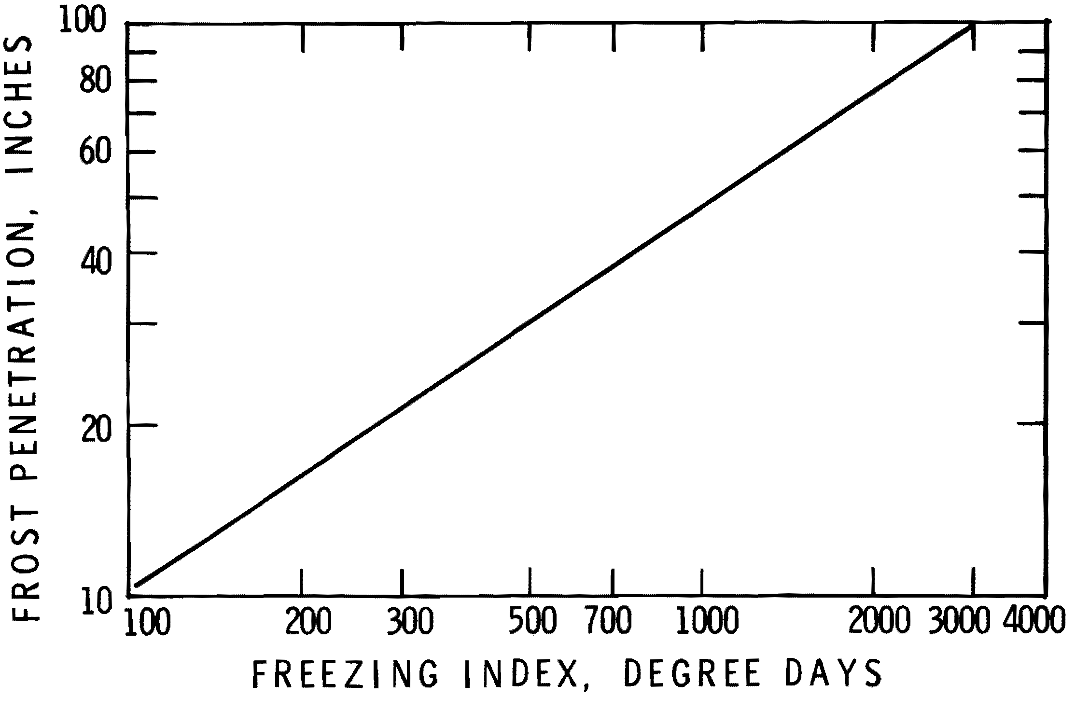

FIG 8.4.3 (After Brown)

RELATIONSHIP OF FREEZING INDEX AND DEPTH OF FROST PENETRATION

Insulation can be manufactured that has relatively high compressive strengths so that it is possible to place slabs of these materials directly below the bearing surfaces of foundations. Substantial economic advantages accrue where such designs are used because it is possible to place foundations closer to the ground surface thus reducing the costs of excavation and transportation of granular fill to replace *frost-susceptible* soil (ROBINSKY and BESPFLUG 1973).

HEATED BUILDINGS

Loss of heat from basement spaces through the supporting soil reduces the depth to which frost penetrates in the immediate vicinity of foundations and foundation walls. Relatively simple analytical methods are available for calculation of such heat losses (LATTA and BOILEAU 1968) but the problem of determining the precise effects these have on frost penetration is more complex. The expense of such calculations is seldom warranted and the usual practice is to ignore the effects of heat losses. With the increased use of insulation to conserve energy the conditions begin to approach those prevailing in unheated buildings. Safe depths for footings on the perimeter of a building therefore are determined from the maximum depth of frost penetration. Interior footings are generally placed at shallower depths.

Basement Garages

Garage spaces are frequently provided in the basement of residences and other buildings. In heated buildings interior footings placed at shallow depths may be included within the garage space or beneath partitions separating the garage from the rest of the basement. Because corrosion of vehicle bodies is accelerated at higher temperatures such spaces are often maintained just above the freezing point. Frost heaving occurs when inadequate heat is supplied during cold snaps or the garage doors are left open. Concrete floors may be lifted and the shallow foundations heaved causing damage to the structure and interior finish of the building (PENNER and BURN 1970). Where such conditions are anticipated it is recommended that foundations beneath all the walls of basement garages be placed at depths beyond maximum frost penetration, properly backfilled and drained or that they be protected from freezing by the use of insulation.

Unheated Ancillary Structures

Small unheated structures such as garages and storage facilities which may be expected to heave when erected on *frost-susceptible* soil should not be attached to other structures which are designed not to heave. The resulting differential movements will rack or destroy connecting walls and roofs and present continual maintenance problems.

FROST ACTION DURING CONSTRUCTION IN WINTER

Construction in winter is now considered routine in Canada (CROCKER 1971) and the handling of building materials in below freezing temperatures is generally well understood by contractors, but special care must be taken to prevent *frost action* affecting foundations before the permanent heating facilities are installed. *Frost heaving* and damage frequently occur on construction sites in early winter before temporary heating begins.

SHALLOW FOOTINGS AND CRAWL SPACES

Interior footings, which are often placed only a few inches below basement floors are particularly vulnerable to *frost action*. The partially completed structure acts like a series of cooling fins accelerating the extraction of heat from immediately beneath the footings even when straw is used as temporary insulation over the floor surface (CROCKER 1965). Under the same circumstances basement floors of concrete may heave causing either crushing of lightweight partitions between floor and frame or further lifting of the frame and distortions which may lead to permanent structural damage. It is important therefore that foundations at shallow depths in buildings designed to be heated be adequately protected during the construction period either by temporary heating or adequate insulation.

Buildings in which crawl spaces are provided between the foundations and the first floor level are also vulnerable to *frost action*. Temporary heating is often only installed above the first floor for the sake of progress of the work and the crawl space is forgotten. Temperatures drop to those prevailing outside and *frost heaving* occurs. The sample of frozen soil shown in Fig 8.4.1. was obtained from beneath the concrete raft of a seven-storey building with crawl space which was heaved more than 2 in. during construction.

EXCAVATION WALLS AND SUPPORTS

Dangerous conditions may develop in the walls of excavations supported by sheet piling or soldier pile and lagging systems if they remain open without heating during winter construction. Cold air is more dense than warmer air and 'flows' into the spaces below ground level thus accelerating the extraction of heat from the soil behind the retaining structures. The direction of heat flow under these conditions is primarily horizontal and *ice lensing* occurs parallel to the walls. This results in large outward pressures against the wall increasing the loads on the supporting members which may lead to overstressing and to inward movement of the walls. The horizontal components of loads on anchors and rakers may increase considerably, but horizontal struts spanning from wall to wall will be subjected to stress increases with contributions from both walls. Additional loads may develop when struts are hit by sunlight and heat absorbed by radiation causes the struts to expand.

The development of potentially dangerous conditions must be avoided and it is therefore necessary to monitor the walls and supporting systems to detect movements and stress increases associated with *frost action*. (This should be done even where increased factors of safety have been used in the design to accommodate the expected stress increases). Where observations indicate that excessive *heaving pressures* are developing against the walls appropriate steps must be taken to prevent overstressing of the support systems. For anchored flexible walls, where inward movements of one or two inches may be tolerable, stresses on the individual tie-backs may be reduced by 'slacking off' on the locking system. Other support systems, such as rakers and horizontal struts, are more difficult to adjust and avoidance of excessive stresses may require a supply of heat to the walls to thaw the frost. Where subsurface conditions are such that excessive *frost action* may be expected and where significant wall movements cannot be tolerated heating systems should be installed to prevent *frost action* from occurring.

Raker Footings

Soil beneath raker footings must not be permitted to freeze. Besides producing increased stresses on the supporting member an unstable condition can develop if the soil is rapidly thawed with subsequent loss of shear strength upon which stability depends. The result may be complete failure of the footing and loss of support of the wall.

GLOSSARY OF TERMS

The following are terms frequently encountered in the literature on *frost action*.

Adfreezing The adhesion of *frost-susceptible* soil in contact with foundation walls due to freezing resulting in sufficient bond to transfer heaving pressures from the soil to the structure. It is also sometimes referred to as *frost grip*.

Frost action A general term for the damage caused by freezing and thawing of moisture in materials and on structures of which they are a part or with which they are in contact.

Frost front The position in the ground at which freezing is taking place at any particular time. It is usually a line roughly parallel to the ground surface or any other surface from which heat is being extracted.

Frost heave The raising of a surface due to the formation of ice in the underlying soil.

Frost-susceptible soil Soil in which significant *ice segregation* will occur resulting in *frost heave* or heaving pressures when the requisite moisture and freezing conditions exist.

Heaving pressures The stresses acting against a structure that result from ice formation in *frost-susceptible* soil.

Ice lenses Ice formations in soil occurring essentially parallel to each other, generally normal to the direction of heat loss and commonly in repeated layers. (See Fig 8.4.1.).

Ice segregation The growth of *ice lenses*, layers, veins and masses in soils, commonly, but not always, oriented normal to the direction of heat loss. (See Fig 8.4.1.).

Non frost-susceptible materials Cohesionless materials such as crushed rock, gravel, sand, slag and cinders or soil in which significant detrimental *ice segregation* does not occur under normal freezing conditions.

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COMMENTARY 8.5

THE USE OF PILE DRIVING FORMULAS

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COMMENTARY 8.5

THE USE OF PILE DRIVING FORMULAS

In the 1975 edition of the National Building Code of Canada, section 4.2, Foundations, and in this Manual, any reference to the use of pile driving formulas for assessing the bearing capacity of driven piles has been avoided. This commentary explains the reasons for this departure from common practice.

HISTORICAL REVIEW OF PILE DRIVING FORMULAS

In the early stages of development of piling practice, only driven piles were used. Since the science of geotechnique was non-existent, the obvious method of "designing" pile foundations was to observe the resistance of the soil to the penetration of the pile during the driving process. This "design method" was officially introduced by WELLINGTON (1893) who proposed the well known Engineering News Formula which is still widely used.

$$Q_f = \frac{W\,H}{S + C}$$

where

Q_f = ultimate bearing capacity of the pile, ton

W = weight of the driving hammer, ton

H = drop height of the hammer, in.

S = the pile set per blow, in.

C = a constant representing energy losses in the driving system at impact, in.

Since then, more than 100 additional formulas have been suggested. They all have the same form but include a variable quantity of empirical constants intended to produce a better simulation of the driving system and to yield more reliable results in terms of bearing capacity.

The Hiley formula widely used in Canada, and typical of these modifications has the following form:

$$Q_f = \frac{e \times E_n}{S + \frac{1}{2}(C_1 + C_2 + C_3)} + \frac{W + n^2 W_p}{W + W_p}$$

where

E_n = rated energy of the driving hammer, in.-ton

e = the efficiency of the hammer

n = an empirical factor normally equal to 0.25

W_p = the weight of the pile, ton

W = the weight of the hammer, ton

$\left.\begin{array}{l} C_1 \\ C_2 \\ C_3 \end{array}\right\}$ = constants representing energy losses at impact due respectively to elastic compression of the pile cap, of the pile and of the soil, in.

S = the pile set per blow, in.

The lack of confidence that soil mechanics engineers have in such design formulas is demonstrated by the fact that the safety factors applied to Q_f to determine the allowable loads are always very large; a value of F.S. = 6 is typical.

DEFICIENCIES OF PILE DRIVING FORMULAS

VALIDITY OF THE BASIC ASSUMPTIONS

Pile driving formulas are based on the assumption that the bearing capacity of a driven pile is a direct function of the energy delivered to it during the last blows of the driving process, and that the energy transmission from the hammer to the pile and the soil is instantaneous on impact.

These two assumptions have been proved wrong by many investigators.

It has been clearly demonstrated that the bearing capacity of a pile is related, not so much to the total energy per blow of the driving system, but more importantly to the distribution of this energy with time at and after impact and by the magnitude and duration of the peak impact force. From the many investigations of pile driving by means of the wave propagation theory, it has been made clear that time effects as related to the propagation of impact forces in the pile have a governing influence on the behaviour of piles during driving.

Under such circumstances, all existing pile driving formulas patterned on the Engineering News Formula must be considered as being inherently incorrect.

QUALITY OF INPUT DATA

All existing pile driving formulas are based on two fundamental parameters: the energy delivered by the driving hammer at each blow, and the set of the pile under each blow. While the set can be measured fairly accurately during the driving process, the energy has to be assumed equal to W x H for a free-fall hammer or to the so-called rated energy as specified by the manufacturers for steam or diesel hammers. This assumption implies that all blows of a given hammer deliver the same energy, ... and is the origin of the poor reliability of pile driving formulas.

The lack of reliability of pile driving formulas was recognized a long time ago, for example by PECK (1942) who stated:

> "*It can be demonstrated by a purely statistical approach that the chances of guessing the bearing capacity of a pile are better than that of computing it by pile-driving formula ... To determine the ultimate bearing capacity of a pile, the following procedure then would be justified: take 100 poker chips and label them with numbers so as to form a geometrically normal array having a mean value of 91 tons and a standard deviation of 1.55. Mix the poker chips and select one. The value written on the chip will be the bearing capacity of the pile. The value from the chip will be nearer to the true bearing capacity more frequently than a value obtained by use of any of the pile driving formulas*".

However, the reasons for this situation were established only more recently by HOUSEL (1965) for driving by steam and diesel hammers and by TAVENAS and AUDY (1972) for driving by free-fall hammers. These investigations showed that the energy per blow delivered to a pile by the same driving equipment varies by as much as ± 70% of the average energy, and, that for steam or diesel hammers, the average energy is generally 30 to 60% lower than the rated energy. Therefore, since it is impossible to assume that the energy delivered is constant from blow to blow and is equal to the rated energy of the driving equipment, it is also impossible to assign a reliable value to the energy delivered in any pile driving analysis.

CONSEQUENCES OF THE USE OF PILE DRIVING FORMULAS

PILE DRIVING FORMULAS FOR EVALUATING Q_f

Since it has been demonstrated that pile driving formulas are inherently incorrect in their assumptions and that the energy delivered to a pile by a given hammer is highly variable and generally entirely different from its rated or assumed energy, it is obvious that pile driving formulas which refer to this rated or assumed energy cannot lead to an acceptable evaluation of the ultimate bearing capacity of the pile. The use of pile driving formulas for designing pile capacities therefore are not recommended.

PILE DRIVING RECORDS FOR COMPARING PILES

It has been demonstrated that the energy actually delivered to piles by a given hammer is highly variable from blow to blow and from pile to pile. These variations are due to energy losses and varying hammer operation, pile cap condition, cushion properties, etc ... and are not necessarily related to variation in the resistance of the soil.

For these reasons the observed driving energy cannot be used as a basis for comparing the bearing capacity of adjacent piles, and more specifically for ensuring that non-load-tested piles are of quality identical to that of load-tested piles.

PILE DRIVING RECORDS FOR EVALUATING INDIVIDUAL PILES

The only acceptable application of driving energy records is in the evaluation of the quality of each pile taken individually. More particularly the pile driving record will normally indicate if a pile has been broken during the driving process. It will also show the changes in soil strata and will therefore permit assessment of the length of pile in the different strata constituting the soil deposit. It will also make it possible to establish if an end bearing pile to rock has actually been driven to refusal.

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COMMENTARY 8.6

THE DESIGN OF PILES SUBJECTED TO HORIZONTAL LOADS

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THE DESIGN OF PILES SUBJECTED TO HORIZONTAL LOADS

Design methods or features applicable to simple cases of piled foundations subjected to limited horizontal loads have been described in Chapter 7 of this Manual. With the development of tall, slender structures such as high-rise buildings, towers etc., the designer is increasingly confronted with cases where greater horizontal loads transmitted by these structures must be resisted by vertical piles. The purpose of this commentary is to present to the designer information on more detailed methods of analysis than those given in Chapter 7 so that he may have some guidance in providing foundation systems for such structures.

GENERAL

Vertical piles resist horizontal loads or moments by deflecting until the necessary reaction in the surrounding soil is mobilized. The behaviour of the foundation under such loading conditions depends essentially on the relative stiffnesses of the pile and the soil.

The horizontal load capacity of vertical piles may be limited in three different ways; the ultimate capacity of the soil may be exceeded resulting in very large horizontal movements of the piles and failure of the foundation, the bending moments may generate excessive bending stresses in the pile material resulting in structural failure of the piles, or the deflections of the pile heads may be too large to be compatible with the superstructure. All three modes of failure must be considered in design.

The methods presently available for design of piled foundations subjected to horizontal loads must be regarded as highly empirical. The input soil data are associated with a high degree of uncertainty. Therefore these methods must be used with great caution and with due consideration of their limitations. They have been summarized in this commentary to help the designer in using references which in many cases contain inaccuracies or contradictory statements.

There is much room for improvement of these design methods and, at present, the best method is still that based on a well-planned and well-executed *in situ* load test, as presented in Chapter 7 of this Manual.

HORIZONTAL LOAD CAPACITY OF A VERTICAL PILE

The maximum horizontal load that can be applied to a pile is limited by the maximum horizontal reaction that can be mobilized in the soil in front of the pile. This limitation generally governs in the case of short rigid piles. The following methods may be used:

SEMI-THEORETICAL METHOD

According to BROMS (1964a&b) the horizontal load capacity of a pile varies with the length of the pile and conditions of restraint at the pile head.

1) *In cohesionless soil*

$$P_{ult} = 1.5\,\gamma'\,L_p^2\,D\,K_p$$

2) *In cohesive soil*

$$P_{ult} = 9\,c_u\,D\,(L_p - 1.5\,D)$$

where P_{ult} = ultimate horizontal load, lb

γ' = effective unit weight of soil, lb/cu ft

L_p = length of pile, ft

D = diameter of pile, ft

K_p = passive earth pressure coefficient of soil (Appendix 5A. Chapter 5)

and c_u = undrained shear strength of clay, lb/sq ft

For other cases, such as long restrained piles, see BROMS (1964 a&b)

EMPIRICAL METHOD

Considering the very close analogy between the behaviour of soils around a horizontally loaded pile and around a pressuremeter probe, an empirical method for determining P_{ult} from pressuremeter test results has been proposed by MENARD (1962). According to this method, which has been checked by means of full scale tests, the horizontal load capacity of a short restrained pile may be expressed by

$$P_{ult} = p_L D (L_p - D)$$

where p_L = limit pressure from pressuremeter tests, lb/sq ft

REFERENCES

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MENARD, L., 1962. Comportement d'une fondation profonde soumise à des efforts de renversement. *Sols Soils, 3:4, 9-23.*

DEFLECTIONS AND MOMENTS IN A PILE

In most cases other than short rigid piles, the maximum horizontal loads that may be safely applied to a vertical pile is limited, not by the load capacity of the surrounding soil, but by the magnitude of the deflection of the pile and of the resulting bending moments in the pile.

The analysis of the behaviour of horizontally loaded piles is based on the concept of elastic reaction. In this concept it is assumed that the soil around a pile can be simulated by a series of horizontal springs, each spring representing the behaviour of a layer of soil of unit height. When the pile is forced against the soil under the action of horizontal loads, the soil deforms and generates an elastic reaction assumed to be identical to the force that would be generated by the simulating spring subjected to the same deformation. With the further assumption that the soil is homogeneous, or that all simulating springs are identical, the soil's behaviour can be determined if the equivalent spring constant is known. This spring constant is called the coefficient of subgrade reaction K_s.

COEFFICIENT OF SUBGRADE REACTION

Though simple in its definition, the coefficient of subgrade reaction has proved to be a very difficult parameter to evaluate. This is due to the fact that it cannot be measured in laboratory tests, but must be backcalculated from full scale field tests. Investigations have shown it to be variable not only with soil type and mechanical properties, but also with stress level and the geometry of the pile.

In the absence of better information, the coefficient of subgrade reaction may be estimated by the following method.

Typical Values

TERZAGHI (1955) has proposed the following formulas and reference constants to assess the value of K_s

1) In cohesionless soil

$$K_s = n_h \frac{z}{D}$$

where K_s = coefficient of horizontal subgrade reaction, ton/cu ft at depth z
z = depth, ft
D = pile diameter, ft
and n_h = constant related to soil density as given in table 8.6.A

TABLE 8.6.A

Values of n_h for cohesionless soils

Soil Density	n_h in ton/cu ft	
	above groundwater	below groundwater
loose	7	4
medium	21	14
dense	56	34

2) In cohesive soil

$$K_s = 67 \frac{c_u}{D}$$

where K_s = coefficient of horizontal subgrade reaction, ton/cu ft
c_u = undrained shear strength of the soil, ton/sq ft
and D = pile diameter, ft

Because of the influence of stress level and geometry of the pile on the value of K_s and the empirical nature of these expressions, the coefficients of subgrade reaction determined in this way include a high degree of uncertainty and must be used with caution.

For a discussion of factors influencing the coefficient of subgrade reaction, see ROWE (1956b).

Pressuremeter Method

According to recent investigations a better method of evaluating the actual field values of K_s is by means of *in situ* pressuremeter tests. As shown by MENARD (1962) and later confirmed by tests on instrumented piles by BAGUELIN & JEZEQUEL (1972), the coefficient of horizontal subgrade reaction may be directly related to the pressuremeter modulus E_p or indirectly to the limit pressure p_L.

1) In cohesionless soil

$$K_s = 3.3\,\frac{E_p}{D} \simeq 25\,\frac{p_L}{D}$$

2) In cohesive soil

$$K_s = 1.6\,\frac{E_p}{D} \simeq 16\,\frac{p_L}{D}$$

where K_s = coefficient of horizontal subgrade reaction, ton/cu ft
E_p = pressuremeter modulus, ton/sq ft
p_L = limit pressure, ton/sq ft
and D = pile diameter, ft

REFERENCES

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DETERMINATION OF MOMENTS AND DEFLECTIONS

Only the common case of piles with a rigid cap at ground surface will be considered here. For other cases refer to MATLOCK and REESE (1960).

The distributions and magnitudes of moments and deflections in a pile subjected to horizontal forces are essentially a function of the relative stiffness T of the pile-soil system. T is given by:

$$T = \left(\frac{E\ I}{K_s}\right)^{1/5}$$

where E = elastic modulus of pile material, ton/sq ft
I = moment of inertia of pile cross section, ft^4
K_s = coefficient of subgrade reaction, ton/cu ft
and T = relative stiffness, ft

From the values of T the moments M_p and the deflections δ_p may be computed at any depth using the following formulas:

$$M_p = F_m \quad (PT)$$

$$\delta_p = F_\delta \left(\frac{P\ T^3}{E\ I}\right)$$

where M_p = moment at depth z, ton ft

δ_p = deflection at depth z, ft

F_m = moment coefficient at depth z, as given in

F_δ = deflection coefficient at depth z, as given in

P = horizontal load, ton

T = relative stiffness, ft

E = modulus of elasticity of pile material, ton/sq ft

and I = moment of inertia of pile cross section, ft^4

— Values of M_p and δ_p are shown graphically in Fig 8.6.1.

REFERENCE

MATLOCK, H., REESE, L.C., 1960. Generalized solutions for laterally loaded piles. *J. Soil Mech. Found. Div., Am. Soc. Civil Engrs., 86: SM5, 63-91.*

GROUP EFFECTS

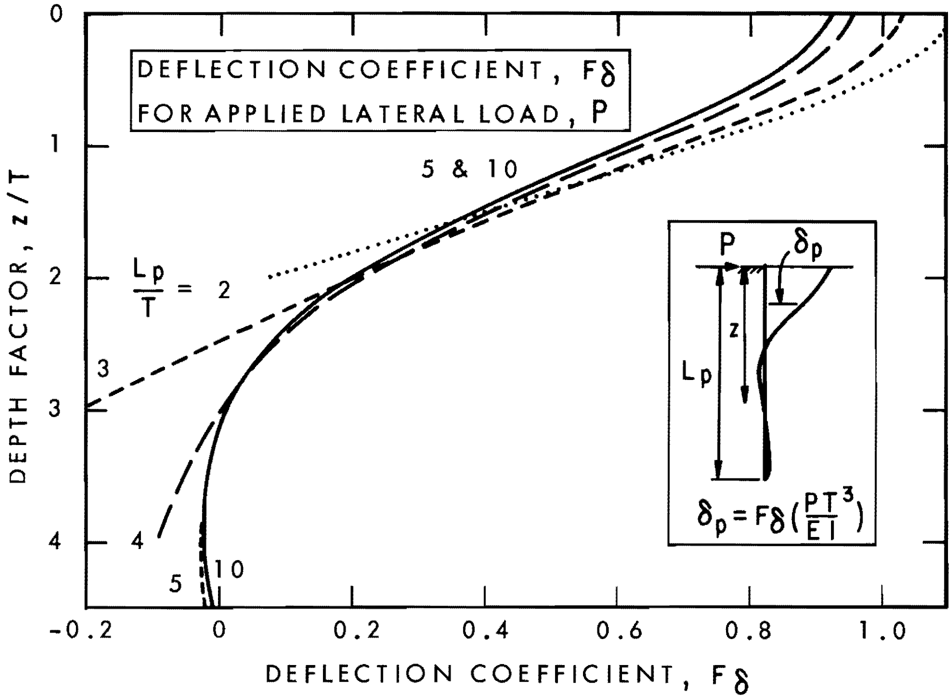
The above considerations apply to individual piles. Little information is available on the behaviour of pile groups but it is recognized that group action produces a reduction of the coefficient of subgrade reaction. The reduction of K_s is a function of pile spacing in the direction of loading, as indicated in Table 8.6.B.

TABLE 8.6.B

Subgrade reaction of pile groups related to pile spacing.

Spacing	K_{group}
8 D	1.00 K_s
6 D	0.70 K_s
4 D	0.40 K_s
3 D	0.25 K_s

Pile spacing normal to the direction of loading has no influence provided it is greater than 2.5 D.



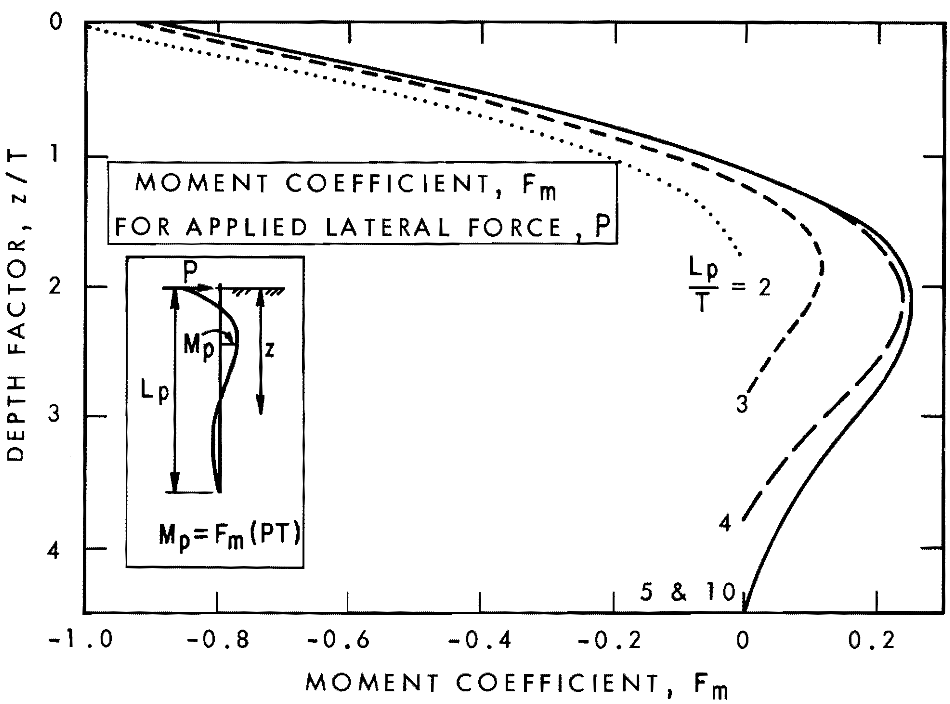


FIG 8.6.1

DEFLECTION AND MOMENT COEFFICIENTS FOR LATERALLY LOADED PILES

COMMENTARY 8.7

EARTHQUAKE RESISTANT DESIGN OF FOUNDATIONS

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COMMENTARY 8.7

EARTHQUAKE RESISTANT DESIGN OF FOUNDATIONS

GENERAL

The earthquake resistant design of a building involves the consideration of a number of factors including:

- the magnitude of the forces transmitted to the structure as a result of the earthquake accelerations;
- the ground velocity and displacement;
- the duration of strong ground motion;
- the behaviour of the subsoils.

The earthquake motion at any particular site is a function of:

- the distance of that site from the earthquake's causative fault, or the earthquake's epicentre;
- the earthquake's magnitude, duration, mechanism, and depth;
- the depth and engineering characteristics of the soils overlying bedrock.

In the past, a main consideration relating to the structural design has been the magnitude of the forces transmitted to the structure by the earthquake. For the comprehensive earthquake resistant design of a structure it is necessary, however, to consider the ground velocity, the ground displacement and the behaviour of the subsoils.

Studies of the damage caused by severe earthquakes in Alaska (1964), Niigata (1964), Chile and San Fernando (1971) show that although in many cases the actual structure was left intact, the building failed due to inadequate connection between the structure and foundation, and/or failure of the subsoil. (Fig 8.7.1.).

Whilst analytical procedures are available, it is extremely difficult to produce quantitative solutions for the complete earthquake resistant design of a structure. Considerable judgement is required to evaluate the behaviour of a building and its subsoils during an earthquake. The purpose of this Commentary is to indicate the range of problems associated with earthquake resistant design. It must be noted by the designer that it is virtually impossible to make a structure entirely earthquake resistant. The degree of a seismic design will depend upon the type of structure, its use, the foundation conditions and costs of making the structure and foundation better resistant to an earthquake.

For additional discussions on a seismic design of buildings see Commentary K, "Dynamic analysis for the seismic response of buildings", Supplement No 4, N.B.C. 1975.

ZONES OF SEISMIC ACTIVITY

Major earthquakes are believed to originate when movement occurs within the earth along major tectonic faults or fracture planes. The fault motion associated with earthquakes may be primarily horizontal (strike-slip faulting) or vertical (normal or thrust faulting).

For the purposes of conventional building structures, Canada is divided into four zones of seismic activity. (Fig 8.7.2.). These seismic zones have been established from an analysis of data obtained from seismograph stations. Where available, historical information (prior to 1900) confirms the solutions obtained from current data.

The available seismic data for the past seventy years has been statistically analyzed by MILNE & DAVENPORT (1969) to produce geographical contours of peak firm ground acceleration for a one hundred year return period. This predicted acceleration is subject to confidence limits in the order of ± 100%.

(Courtesy of H.B. SEED)

FIG 8.7.1
TILTING OF BUILDINGS

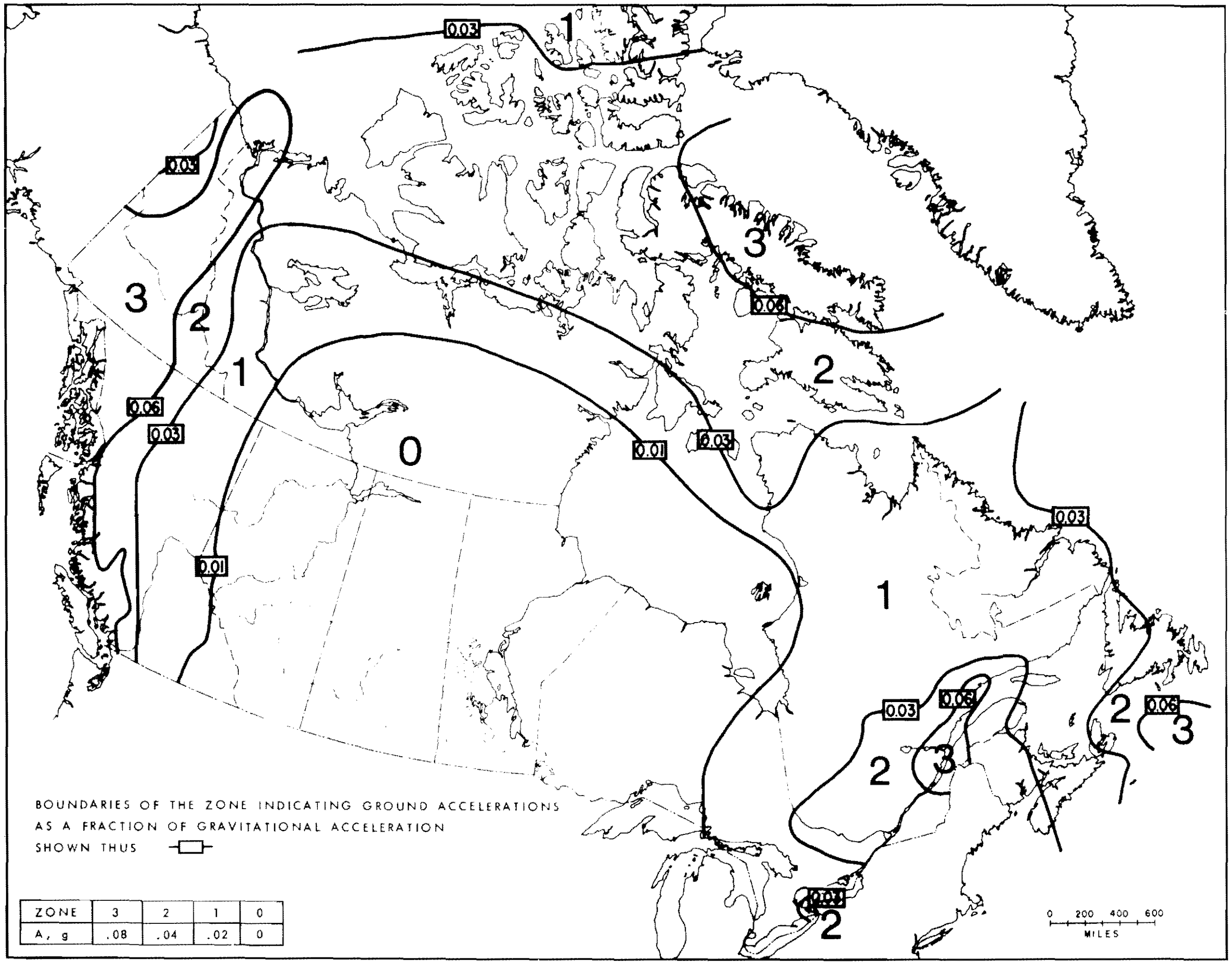

FIG 8.7.2

SEISMIC ZONES IN CANADA

EARTHQUAKE GROUND MOTIONS

The basic input data for seismic analysis are obtained from recordings of ground acceleration during earthquakes. Strong motion accelographs are able to record three orthogonal components of acceleration generated by an earthquake. Although available data of recorded ground accelerations are limited, it is possible to indicate the general trends of this parameter in relation to the earthquake magnitude, distance from causative fault or epicentre, duration of shaking, type of soil deposit, and depth of focus.

For more detailed discussion on earthquake ground motions see Commentary J "Effects of earthquakes", Supplement No 4, N.B.C. 1975.

Factors which affect surface ground motions are (HOUSNER 1970):

(a) nature of the source mechanism

- dimensions and orientation of the causative fault
- depth of focus
- stress drop
- amplitude, direction, time and history of the fault movement.

(b) travel path of the seismic waves

- physical properties of the rock
- geological structure of the region

(c) local geology

- physical properties of soil
- size of soil mass
- orientation of bedding planes.

Depending upon the above factors, there can be an amplification or an attenuation of the bedrock motions at the ground surface. Analytical procedures are available to estimate the ground surface motions. The procedure consists of selecting design bedrock motions and determining the dynamic response of the overlying soil. These analytical procedures are costly to perform for an average building structure. WIEGEL (1970).

SOIL BEHAVIOUR

COHESIONLESS SOILS †

Settlement of Cohesionless Soils

Vibration is recognized as an effective means of densifying *cohesionless* soils. Vibrations caused by earthquakes can lead to densification of *loose cohesionless* soil deposits and associated settlements of the ground surface. Settlements of the ground due to densification can lead to differential settlements in a structure. If one part of a building is seated upon firm materials or a pile foundation and another part founded on a backfill or looser materials, differential settlements caused by earthquake vibrations may seriously affect the continuity of the structure.

Soil Liquefaction

If saturated cohesionless soils are subjected to earthquake ground vibrations, the tendency to densify the sand is accompanied by an increase in the pore water pressure. This build up in pore water pressure reduces the shear strength of the soil to a minimum value. This phenomenon is known as liquefaction. In general, liquefaction occurs primarily in saturated uniform sand deposits of *loose* to *medium* density.

Level ground

In the case of level ground, the build up in pore water pressure causes water to flow upward to the ground surface, emerging as mud spouts or sand boils. The sand may be turned into a *quick* or liquefied condition. The resulting reduction in shear strength of the sand can cause bearing capacity failures and settling of structures into the quicksand. Submerged structures, being of a lower density than the liquefied soil, may float to the surface.

Sloping ground

If liquefaction occurs in or under a slope, the slope will slide towards the unsupported side. This is called a flow slide and occurs in *loose* saturated *cohesionless* materials. Flow slides were observed in the earthquakes occuring in Chile (1960), Alaska (1964) and Niigata (1964).

Backfill liquefaction

Waterfront bulkhead structures are often backfilled with sand. It being difficult to compact the backfill below the water level, the sand is frequently in a *loose* condition. If the backfill is liquefied during an earthquake, the resulting pressure against the bulkhead can be considerably higher than the design pressure resulting in damage to the bulkhead.

Liquefaction of thin layers

Thin layers or lenses of sand often occur within clay deposits. Liquefaction of this sand layer could cause the overlying non-liquefied sloping soil to slide along the liquefied layer. In addition, a zone of soil can collapse or sink into the back end of the sliding mass. This depressed zone is referred to as a graben (Fig 8.7.3.). Buildings located in an area in which a graben might form would be subjected to large differential settlements and pulled apart. In addition, buildings located near the toe of a slide can be heaved upwards or pushed over by the lateral thrust. Many slides of this type were observed in the 1964 Alaskan earthquake.

COHESIVE SOILS †

Slides can occur during earthquakes in clay deposits. As clay deposits often include sand layers, liquefaction of these layers may contribute significantly to such slides. Many sensitive clay deposits are particularly vulnerable to sliding due to earthquake vibrations, but evidence to date indicates that stiff sensitive clays found in Canada are not vulnerable to sliding due to earthquakes.

SLOPING COMPACTED FILLS †

Firm Foundations

Where earthquake vibrations do not set up large pore water pressures in well-compacted fills on firm foundations, the result is generally a slumping of the fill. The slumping can vary from a fraction of an inch to several feet, depending upon the height of the fill.

Weak Foundations

The behaviour of fills upon weak foundations during an earthquake is almost entirely dependent upon the behaviour of the foundation material. Failures seem to occur as a lateral spreading of the base and extensive longitudinal cracking.

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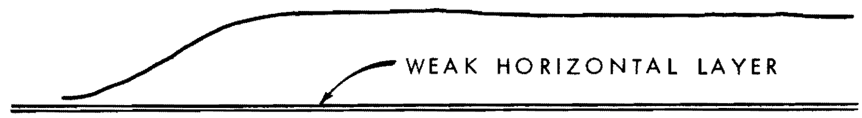
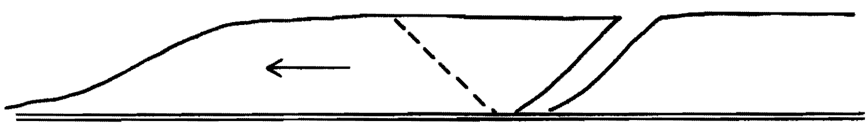
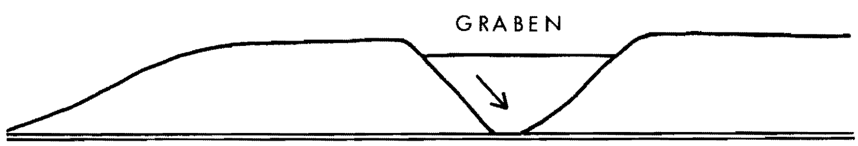
† Photographs illustrating soil behaviour related to earthquake activity are given in SEED (1970).



(a) BEFORE THE EARTHQUAKE



(b) FORWARD MOTION OF MASS OF EARTH



(c) SLIPPING OF BLOCK OF EARTH TO FORM GRABEN

FIGURE 8.7.3
FORMATION OF A GRABEN

FOUNDATION STRUCTURES

RETAINING WALLS

The natural tendency for an earthfill is to slide downhill during an earthquake. This results in an increased pressure on retaining walls which can cause displacements and/or cracking of the wall. This phenomenon was frequently observed in the 1964 Alaskan earthquake.

PILES AND DEEP FOUNDATIONS

The main factors to be considered in the earthquake resistant design of pile foundations are:

- connection of the pile to the structure;
- soil-pile foundation interaction;
- loss of soil support to the pile.

The pile should be tied to the building by adequate structural connections both vertically and horizontally.

Determination of soil-pile interaction can be estimated for deep pile foundations with digital computer programs (PENZIEN 1970). The procedure requires a detailed knowledge of the engineering characteristics of the subsoils, which include creep, damping, and dynamic stress-strain properties. Deformations of the soil-pile mass can be estimated; stresses developed in the piles are controlled by the pile curvature. The analytical procedure is complex and not widely used.

Loss of soil support around piles can be caused by sand liquefaction. In the case of friction piles, this results in a transfer of the load to the lower portion of the pile, which may cause settlement. The unsupported length should also be investigated for buckling. For end bearing piles, the main consideration is the buckling of the piles. Piles embedded in soft loose fill tend to follow the movements of the fill during an earthquake and buildings tend to come off the piles.

GENERAL COMMENTS ON THE EARTHQUAKE RESISTANT DESIGN OF BUILDINGS

Examination of the behaviour of buildings during earthquakes indicates two main problems;

- ground motions caused by the earthquake which will affect the structure and connection of the structure to its foundation;
- behaviour of the soil which can cause loss of support to the foundation.

The majority of soil behaviour problems are associated with *loose* deposits of granular soils and the liquefaction of those soils. Considerable research has been carried out in the last few years and is continuing on the liquefaction of granular soils. At present the phenomenon of liquefaction is not fully understood. It is generally agreed, however, that the susceptibility of a granular soil to liquefy is a function of its

- density
- shape of soil particles
- grading characteristics (Uniformity of particle size)
- amount and intensity of shaking.

In situ densities, and that at which a soil is susceptible to liquefaction, are extremely difficult to determine with any meaningful degree of accuracy, and attempts to simulate field conditions in the laboratory have met with very limited success. Therefore, where granular soils are believed subject to liquefaction the engineering solution usually consists of densifying the soil by the use of compaction piles, for example, or by removing the questionable soil and replacing it with a better graded and more easily compacted soil.

A review of data on recent earthquakes shows that liquefaction generally occurs in deltaic fine grained granular deposits and man made sand fills with little or no compaction. These deposits have a low density and uniformity coefficients generally less than 5.

Site investigations and geotechnical studies for earthquake areas should only be undertaken by specialists in this field.

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COMMENTARY 8.8

THE PRESSUREMETER TEST

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COMMENTARY 8.8

THE PRESSUREMETER TEST

GENERAL

The pressuremeter test developed by MENARD (1956) is an *in situ* loading test carried out in a borehole by means of a cylindrical probe. This test allows the determination of the complete load-deformation characteristics of the tested soil in plane strain conditions. In particular the following parameters are determined:

- the pressuremeter deformation modulus, representative of the elasticity of the soil, which permits the evaluation of settlements,

- the limit pressure, related to the shear strength of the soil, from which the bearing capacity of foundations can be computed.

The pressuremeter test has been very widely used in Europe in the last 15 years and it was introduced in Canada around 1965. The use of the pressuremeter in foundation design is based on a number of empirical correlations which were established from a large number of tests and observations on actual structures. Consequently, the quality of foundation designs based on pressuremeter tests is very good, provided the tests are carried out according to the standard method and in soils similar to those which have been studied in the development of the empirical methods. This means that this test can be used in all soils with the exception of the soft sensitive clays of eastern Canada for which experience is still limited.

The purpose of this commentary is to describe the equipment and the standard testing technique, and to review the methods available for the design of foundations based upon the test results.

NOTE: All pressures and stresses associated with the pressuremeter test are expressed in bars. The following equivalents should be kept in mind:

$$1 \text{ bar} = 10 \text{ KN/m}^2 \simeq 1 \text{ Kg/cm}^2 \simeq 1 \text{ ton/sq ft}$$

In addition, the symbols used in the literature dealing with the pressuremeter are not consistent with those adopted for use in this Manual. Therefore, those symbols which might otherwise lead to confusion have been altered to conform with the Manual.

THE PRESSUREMETER EQUIPMENT

The various apparatus, which are presently in use, all function on the same principle and consist of three components as shown in Fig 8.8.1.; a probe, a pressure and volume control unit referred to as the C.P.V., and connecting tubes. The differences between the various apparatus occur in the details of the probe design.

THE PROBE

The probe consists of a metal cylinder covered with an inflatable rubber membrane under which three independent cells are located. The three cell system has been adopted so as to ensure uniformity of stress and deformation conditions around the central cell which is used for the test measurements.

The results from tests using monocellular probes are strongly influenced by uncontrolled deformations at the ends making it difficult to obtain meaningful and consistent design parameters. Use of such a probe is not recommended, nor should attempts be made to apply the test results using such a probe to the design methods outlined here.

The probe is dilated by injecting a gas or a liquid into the three cells, which are separated by tight inflatable membranes. In the apparatus most commonly used the central cell is inflated with water or a liquid antifreeze, while the guard cells may be inflated with either a gas or a liquid. The volume changes of the central cell during the test are measured by reading the volume of water expelled from the C.P.V. In some experimental probes, the deformations are measured by means of a displacement transducer but such a system may lead to erroneous results in heterogeneous materials.

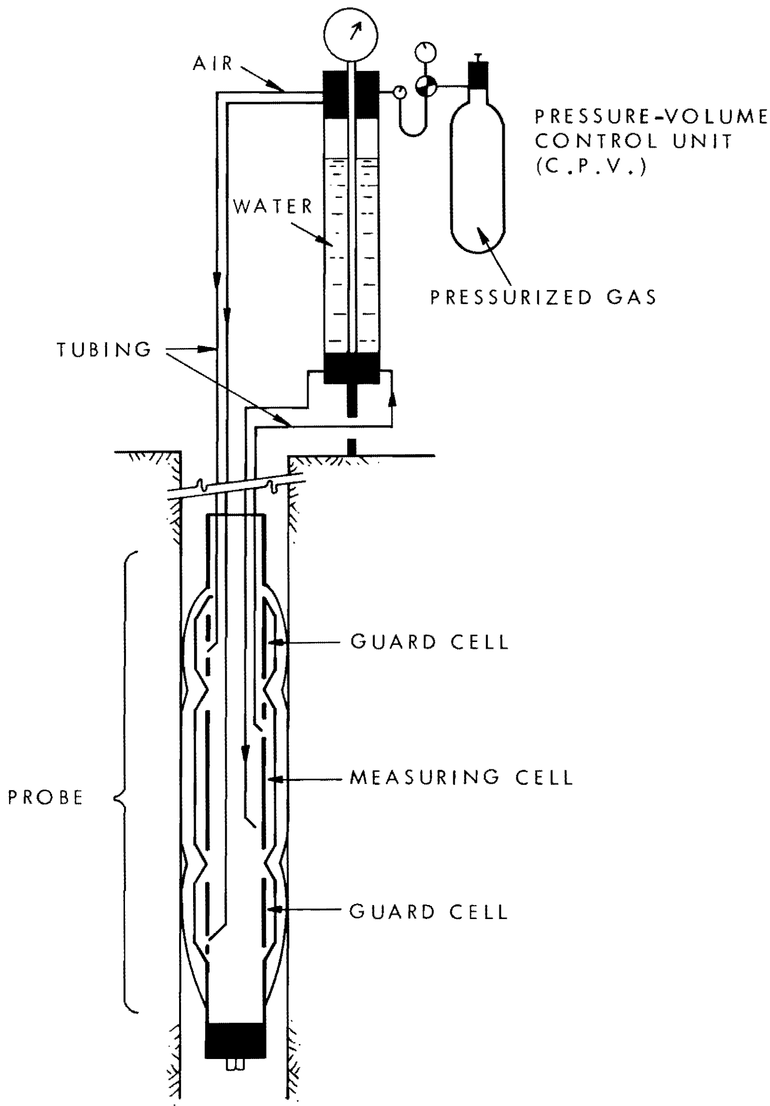


FIG 8.8.1

SKETCH OF PRESSUREMETER SHOWING C.P.V. AND PROBE

THE PRESSURE AND VOLUME CONTROL UNIT (C.P.V.)

The C.P.V. consists essentially of a cylindrical reservoir with a graduated transparent tube which is connected to the central measuring cell of the probe. This reservoir can be pressurized using a pressure-reducing valve connected to a compressed gas tank. The pressure applied to the reservoir can be controlled as required by the test procedure and is read on Bourdon pressure gauges of suitable sensitivity.

Two types of C.P.V. are used which are related to the design of the probe.

C.P.V. for Probes with Gas Inflated Guard Cells

In this case the central cell is isolated by a special membrane. To overcome the stiffness of this membrane, it is necessary to apply in the guard cells a gas pressure which is lower than the liquid pressure acting in the central cell. This is accomplished by having a separate circuit for the guard cells, which is connected to the main pressure circuit through a reducer valve. This reducer valve can be adjusted, according to the depth at which the test is carried out, to maintain a given pressure differential between the central and guard cells.

C.P.V. for Probes with Liquid Inflated Guard Cells

In this case the same pressure is applied to the three cells. The C.P.V. includes a second reservoir for the filling of the guard cells but only a single pressure control is needed.

For both types of C.P.V. a gas circuit is provided which is used at the end of the test to apply pressure on the outside of the central cell so as to force the cell fluid back into the reservoir and to facilitate the retrieval of the probe from the borehole.

TUBING

Two or three flexible tubes are connected to the probe and the C.P.V.

In order to reduce the errors in volume readings which would result from the dilatation of the tubing connected to the central measuring cell, this tubing is run co-axially through the tubing connected to the guard cells. Even so, it is necessary to measure the compliance of the system when tests are carried out in very stiff materials, to ensure that representative moduli are measured.

RECOMMENDED TEST PROCEDURE

Investigations by Ménard and the Laboratoire Central des Ponts et Chaussées, Paris, have led to the development of a standard procedure for pressuremeter tests. A complete description of this procedure is given in LCPC (1971) and a summary is presented here.

PREPARATION OF THE PROBE

Prior to the installation of the probe at the test location, the following operations must be carried out:

- the probe and the connecting tubes must be saturated, with water by flushing out all air bubbles,
- the probe must be predilated to ensure a constant stiffness of the membranes,
- stiffness of the membranes must be calibrated. (This is done by conducting a standard test with the probe in the air and the guard cells open to atmospheric pressure. The measured volume versus pressure relationship represents the stiffness of the probe) and,
- the volume of liquid in the C.P.V. must be adjusted to its theoretical initial value, V_o.

INSTALLATION OF THE PROBE IN THE GROUND

There are three different ways of installing the pressuremeter probe in the ground at the test elevation; by lowering in a borehole, by direct jacking into the ground or by self-boring procedures. The latter requires special equipment which is not presently in common use.

Installation in the Borehole

This is the most common method. In order to obtain satisfactory test results, a technique should be used which creates a minimum of disturbance in the walls of the borehole. The boring method must be selected according to the type of soil and the following are recommended;

In rock	—	Standard core drilling with good quality equipment.
In soft, weathered rock	—	Core drilling with bentonite mud as drilling fluid.
In till	—	Drilling with a bull-nose, tricone or fishtail with bentonite mud as drilling fluid.
In sand	—	Drilling with a fishtail and bentonite mud.
In stiff clay	—	Drilling with a fishtail and bentonite mud; however the hole left after taking a sample with a shelby tube or a piston sampler is acceptable.
In soft clays	—	Drilling with a fishtail or preferably with a special auger and bentonite mud. (Sample holes are not acceptable because of remolding of the clay resulting from the suction developed during the retrieval of the sampling tube.)

Use of bentonite mud is recommended in all cases; in soft or loose soils it is recommended that a hollow pressuremeter probe be used to avoid a piston effect and related soil disturbance while lowering the probe in the hole.

Installation by Direct Jacking into the Ground

In coarse granular soils such as gravels and gravely sands it is impossible to obtain suitable conditions in a borehole and better results are obtained by driving the probe into the ground. In this case the probe is normally protected with a slotted steel tube attached at the bottom of the casing which is first driven to the desired depth. The test is carried out inside the slotted tube, the stiffness of which is measured prior to its installation.

Small diameter pressuremeter probes have also been developed which can be driven directly into the ground. They are used for the control of compaction to depths of less than 25 ft.

Investigations by the Laboratoire Centrale des Ponts et Chaussées have shown that soil disturbance caused by the driving of the tubing, or that caused during boring, has little influence on the measured limit pressure, so that bearing capacity evaluations are possible in all cases. However, the pressuremeter moduli can be reduced significantly by soil disturbance so that settlement predictions are possible only with tests carried out in good boreholes.

TESTING

With all valves closed to isolate the measuring cell, the probe is lowered into position in the ground and fixed at the test elevation. The circuit is then opened and the initial volume read on the C.P.V.

The test itself is carried out by applying pressure in increasing steps of equal magnitude and duration. The pressure increase for each step should be selected so that the limit pressure is achieved after about 10 steps (Tests with 7 to 14 steps are considered acceptable.) The pressure is maintained constant for the duration of each step, i.e. for 1 min.

The variations of the volume are read on the C.P.V. at 15 sec, 30 sec, and 1 min after the application of the pressure. The test is considered completed when the total volume injected into the measuring cell is 700 cm^3, or when the pressure capacity of the apparatus is reached.

Once the test is completed, the pressure is released and the cell is deflated. If necessary a gas pressure is applied in the guard cells to force the water out of the measuring cell and back into the C.P.V.

INTERPRETATION OF THE TEST RESULTS

PRESSUREMETER CURVE

The rough results of a pressuremeter test are presented in the form of a volume versus pressure diagram as shown in Fig 8.8.2. The pressure is indicated in bars on the abscissa, and the volume in cm^3 on the ordinate. The volume read at the end of each step i.e. after each min is used. The pressuremeter curve obtained during the calibration of the probe is generally shown on the same diagram so that the necessary corrections are easy to make.

CREEP CURVE

For each pressure step, the volume change observed between the volume readings at 30 sec and 1 min is calculated. Variations of this volume change with pressure are presented on a graph with the pressure in the abscissa and the volume change in the ordinate. This can be done on the same diagram as the pressuremeter curve. The shape of the creep curve gives an indication of the quality of the test; i.e. the central portion of this curve should be nearly horizontal indicating little volume change, and nearly elastic behaviour of the soil.

CHARACTERISTIC PRESSURES

From the shape of the pressuremeter and creep curves shown in Fig 8.8.2., three characteristic pressures can be defined:

- In the first stage of the test, the volume increases rapidly with pressure and the creep volume decreases, until a pressure, p_i is reached which should normally correspond to the *in situ* total horizontal stress in the ground.
- At pressures higher than p_i, the volume increases slowly and linearly with pressure, and the creep volume remains constant and small, indicating elastic behaviour of the soil around the probe. This elastic stage ends when the pressure equals the *yield pressure*, p_f.
- Beyond the *yield pressure*, p_f, the volume increases rapidly and the creep volume increases with the applied pressure, indicating the development of soil failure around the probe. With increasing pressures, the volume versus pressure curve tends to an asymptotic limit for a pressure p_L called the *limit pressure*.

Correction of the Test Results

Since all test results must be expressed in terms of the total pressure applied on the walls of the borehole around the pressuremeter probe, the pressures read on the Bourdon gauge must be corrected as follows:

- The hydrostatic pressure equal to the difference in elevation between the middle of the probe and the manometer multiplied by the unit weight of the fluid in the apparatus must be added; and the pressure corresponding to the stiffness of the cell at the volume at which the pressure is to be determined must be subtracted. For tests in stiff soils, it may be necessary also to adjust the observed volume to account for the compliance of the C.P.V. and the tubing.

  These adjustments can be made on the full test curve to obtain a corrected test curve. However it is more practical to correct only the relevant pressures forming the results of the test; p_i, p_f, and p_L.

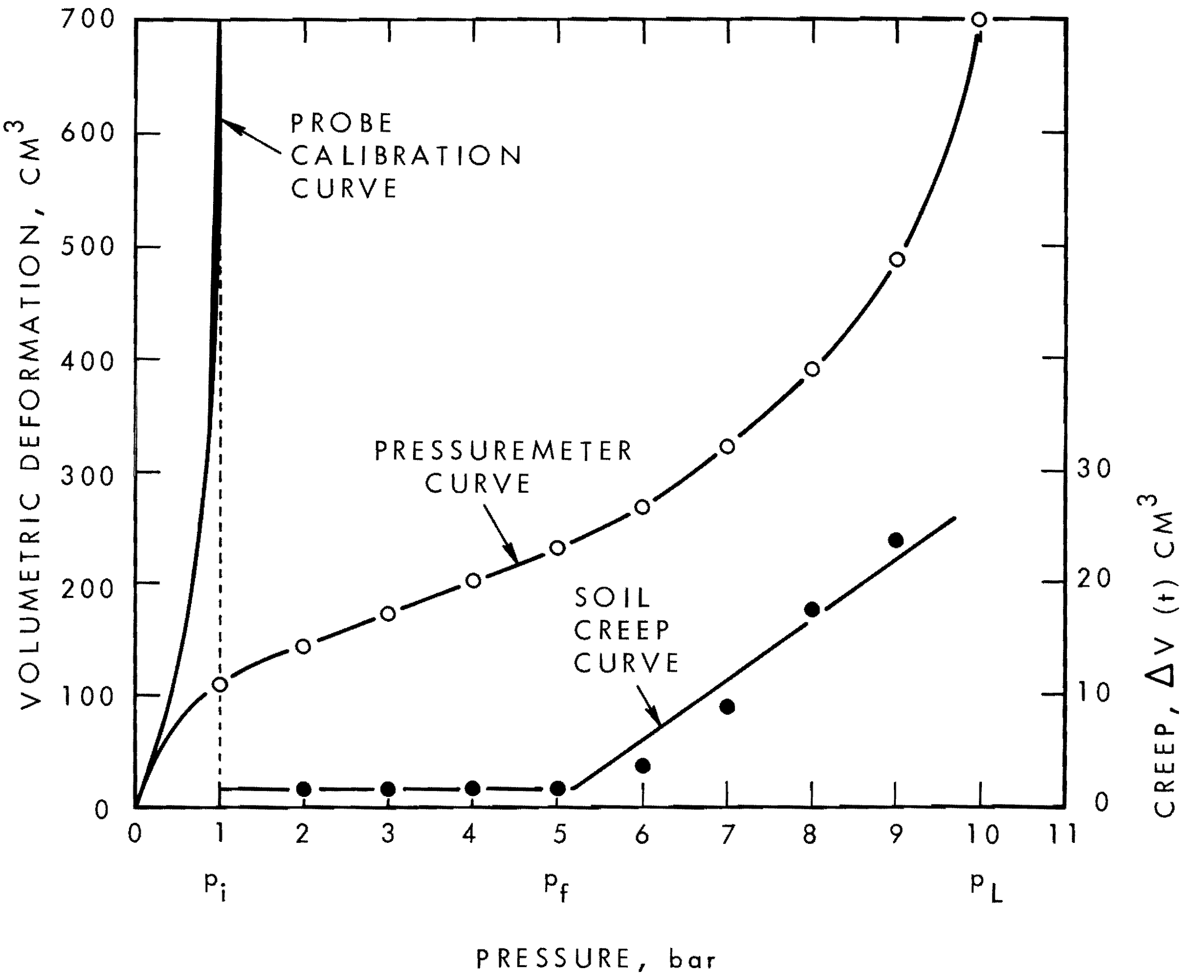


FIG 8.8.2

TYPICAL PRESSUREMETER AND CREEP CURVES

Limit Pressure p_L

The *limit pressure* p_L is generally determined simply as that pressure to which the volume-pressure curve becomes asymtotic or that pressure at which the total producing volume change reaches 700 cm^3.

In some cases, if the strength of the soil is high, the limit pressure cannot be obtained directly because the maximum pressure that may be used in the equipment is limited. In this case, the limit pressure p_L can be deduced from the yield pressure p_f. The yield pressure is determined at the end of the horizontal section of the creep curve or, if this pressure cannot be measured which may happen only for tests in rock, the maximum test pressure is taken as p_f. The limit pressure p_L is then estimated from p_f by applying the following empirical correlation:

$$0.5 < p_f/p_L < 0.75$$

(For a given soil this ratio is a constant)

Pressuremeter Modulus E_p

The pressuremeter modulus is determined from the pseudo-elastic part of the test corresponding to the linear section of the pressuremeter curve.

The pressuremeter modulus is expressed as

$$E_p = 2\,(1 + \nu)\,(V_o + V_m)\,\frac{\Delta p}{\Delta V}$$

where V_o = initial volume of the central measuring cell of the probe,
V_m = volume of water injected under the pressure $p = \frac{p_i + p_f}{2}$,
ν = Poisson's ratio of the soil, generally taken equal to 0.33,
$\frac{\Delta p}{\Delta V}$ = slope of the pressuremeter curve between p_i and p_f.

The pressuremeter modulus is a shear modulus corresponding to a deviatoric stress field. It should not be compared to the oedometer modulus.

Presentation of the Results

The results of pressuremeter tests are presented in the form of diagrams showing the variations with depth of the pressuremeter modulus E_p, the yield pressure p_f, and the limit pressure p_L.

To permit proper evaluation of a deposit, it is recommended that a series of tests at a vertical spacing of 5 ft be carried out.

TYPICAL VALUES OF E_p AND p_L IN DIFFERENT SOILS

From the very wide experience accumulated in France as well as in Canada the following typical values of E_p and p_L may be used for guidance:

Type of Soil	E_p, bar	p_L, bar
Peat and *very soft clays*	2 to 15	0.2 to 1.5
Soft clays	5 to 30	0.5 to 3
Firm clays	30 to 80	3 to 8
Stiff clays	80 to 400	6 to 25
Loose silty sands	5 to 20	1 to 5
Silts	20 to 100	2 to 15
Sands and *gravels*	80 to 400	12 to 50
Till	75 to 400	10 to 50
Recent *fill*	5 to 50	0.5 to 3
Ancient *fill*	40 to 150	4 to 10

DESIGN OF SHALLOW FOUNDATIONS

As mentioned in Chapter 6 the pressuremeter can be used to determine the bearing capacity and the settlements of shallow foundations on soils or rocks. The design methods have been established on the basis of full scale tests, MENARD (1965).

BEARING CAPACITY

The ultimate bearing capacity of a shallow foundation is proportional to the limit pressure p_L and it is given by:

$$q_f - p_o = K_g \, (p_L - p_i)$$

where
- q_f = ultimate bearing pressure, bar,
- p_o = overburden pressure, bar,
- p_L = limit pressure, bar, (within a zone extending the width of the foundation below the foundation level),
- p_i = horizontal pressure measured at the foundation level,
- K_g = bearing capacity factor which is a function of the geometry of the foundation and the type of soil.

It is common practice to apply a factor of safety of 3 to the term $K_g \, (p_L - p_i)$, to obtain the allowable bearing pressure.

Equivalent Limit Pressure

If a foundation sits on a deposit of varying strength, an equivalent limit pressure p_{Le} is used in the bearing capacity formula. p_{Le} is defined as

$$p_{Le} = \sqrt[3]{p_{L1} \times p_{L2} \times p_{L3}}$$

where p_{L1}, p_{L2}, p_{L3} are the limit pressures measured one foundation width above the foundation level, at the foundation level, and one foundation width below the foundation level respectively.

Depth of the Foundation

The depth of the foundation is generally taken directly from the geometry of the foundation. However, if the strength of the soil is variable and an equivalent limit pressure is used, an equivalent depth of foundation D_{fe} defined as

$$D_{fe} = \frac{1}{p_{Le}} \int_{o}^{D_f} p_L\,(z)\,dz$$

should also be used.

Bearing Capacity Factor

The bearing capacity factor K_g is given as a function of the geometry of the foundation (width B, length L, depth D_f) and of the type of soil.

Four soil classes are defined as shown in the following table.

Class	Type of soil	Limit pressure (p_L bar)
1	*soft to firm clays*	0 - 12
	silts	0 - 7
2	*stiff clays*	18 - 40
	dense silts	12 - 30
	loose sands	4 - 8
	very low strength rock	10 - 30
3	*sands and gravels*	10 - 20
	low strength rock	30 - 60
3a	*very dense sands & gravels*	30 - 60
	rocks of medium to high strength	60 - 100 +

Selection of the appropriate soil class should be made by an experienced soils engineer on the basis of information obtained, not only from the pressuremeter tests, but from all methods of investigation used on the given project.

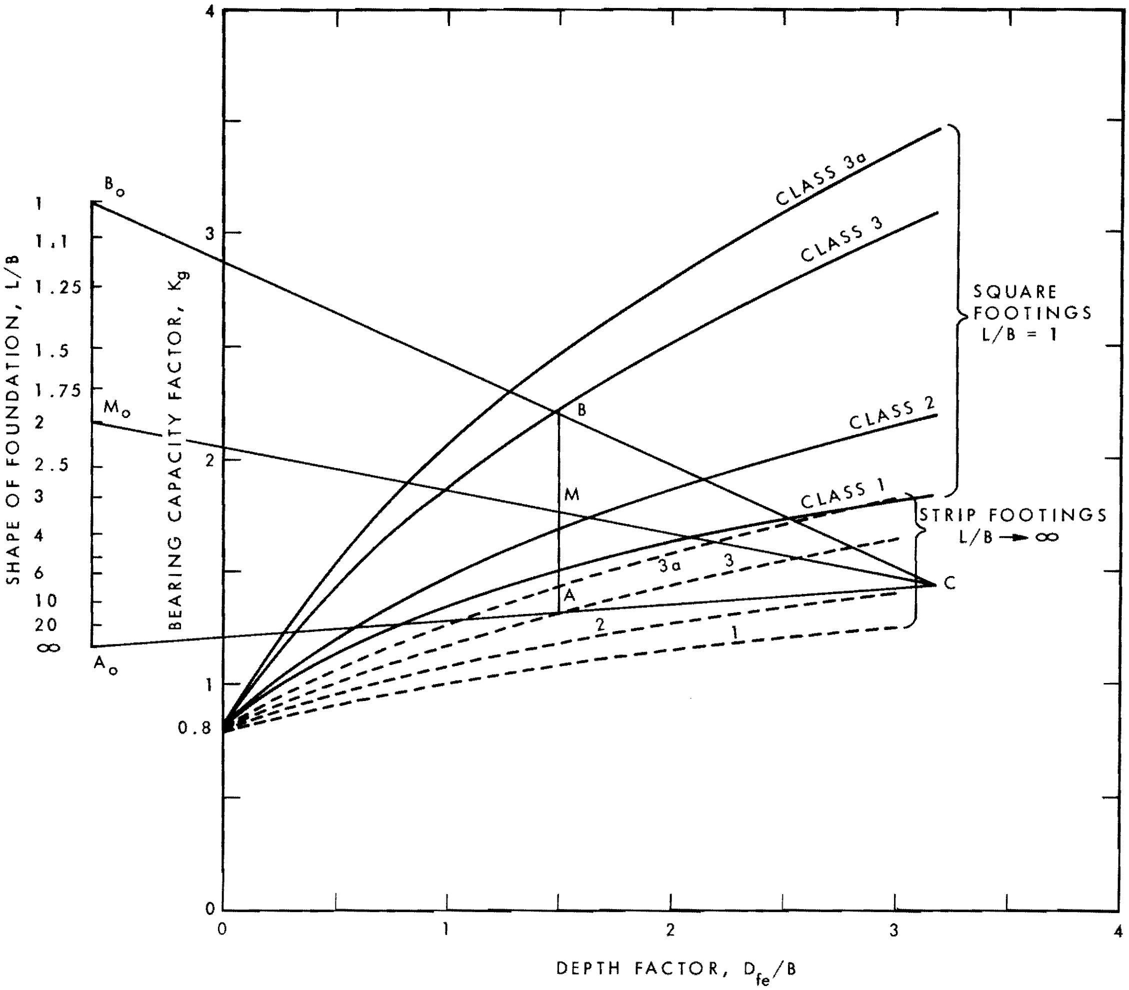
The values of K_g are given in Fig 8.8.3. for the four categories and two limiting values of the L/B ratio. To determine the value of K_g applicable to a rectangular footing, the log scale on the left side of the figure may be used as explained.

SETTLEMENTS

The pressuremeter test gives a shear modulus in the horizontal plane. From classical soil mechanics principles one would assume that this modulus has little relevance to the problem of vertical settlements of footings. However, theoretical as well as full scale experimental studies have shown that this test permits a much better evaluation of foundation settlements. Settlement predictions based on pressuremeter test results are presently the most reliable particularly for granular materials.

The settlement of a footing is given by:

$$s = \frac{1.33}{3\,E_p}\, q_a\, R_o \left(\lambda_2 \frac{B}{2\,R_o} \right) + \frac{\alpha_p}{4.5\,E_p}\, q_a\, \lambda_3 \frac{B}{2}$$



NOTE:

IN ORDER TO DETERMINE VALUES OF K_g FOR RECTANGULAR FOOTINGS THE SCALE ON THE LEFT MAY BE USED TO AID IN INTERPOLATING BETWEEN THE CURVES GIVEN FOR SQUARE AND STRIP FOOTINGS. THE EXAMPLE ILLUSTRATED IS FOR A FOOTING WHERE L/B = 2, A DEPTH FACTOR $2D_{fe}/B = 1.5$ AND A SOIL OF CLASS 3.

THE CONSTRUCTION IS AS FOLLOWS: POINT B_o (AT WHICH L/B = 1) IS CONNECTED TO B ON THE CLASS 3 CURVE FOR SQUARE FOOTINGS WHERE $2D_{fe}/B = 1.5$. POINT A_o (AT WHICH $L/B = \infty$) IS CONNECTED TO A ON THE CLASS 3 CURVE FOR STRIP FOOTINGS WHERE $2D_{fe}/B = 1.5$. LINES B_oB AND A_oA ARE EXTENDED UNTIL THEY INTERSECT AT SOME POINT C. A LINE IS THEN DRAWN FROM C TO THE REQUIRED RATIO OF L/B ON THE SCALE ON THE LEFT, WHICH IN THIS EXAMPLE IS 2, AT M_o. THE REQUIRED VALUE OF K_g IS READ OFF AT M WHERE THE LINE M_oC INTERSECTS THE LINE AB REPRESENTING THE DEPTH FACTOR $2D_{fe}/B = 1.5$. THE VALUE OF K_g OBTAINED IN THIS WAY IS 1.75.

FIG 8.8.3

BEARING CAPACITY FACTOR K_g FOR SHALLOW FOUNDATIONS

where s = settlement, cm
E_p = pressuremeter modulus, bar
q_a = allowable bearing pressure, bar
R_o = reference half-width equal to 30 cm
B = width of the foundation
λ_2, λ_3 = shape factors as given in Fig 8.8.4. and
α_p = structure factor depending on the type of soil as given in the following table.

Type	Peat	Clay		Silt		Sand		Sand and Gravel		Type	Rock
	α_p	E/p_L	α_p	E/p_L	α_p	E/p_L	α_p	E/p_L	α_p		α_p
Overconsolidated or *very dense*	-	>16	1	>14	2/3	>12	1/2	>10	1/3	*Wide spacing* of discontinuities	2/3
Normally consolidated or *dense*	1	9.16	2/3	8.14	1/2	7.12	1/3	6.10	1/4	*Moderately close spacing* of discontinuities	1/2
Under consolidated or *loose*	-	7.9	1/2	5.8	1/2	5.7	1/3		-	*Close spacing* of discontinuities	1/3
										Very close spacing of discontinuities, *very low strength.*	2/3

The first term of the equation represents the settlement caused by shear stresses, the second term, the settlement caused by the increase in confining pressures.

Pressuremeter Modulus in Heterogeneous Deposits

If the measured pressuremeter moduli under a foundation vary by more than 30% it is recommended that an average moduli be used that is determined as follows:

— The modulus used in the first term of the settlement equation should be taken equal to E_b where E_b is defined as

$$E_b = \frac{4}{1/E_{p1} + 1/0.85\,E_{p2} + 1/E_{p3,4,5} + 1/2.5\,E_{p6,7,8} + 1/2.5\,E_{p9\text{ to }16}}$$

where $E_{p1}, E_{p2} \ldots E_{p16}$ are the pressuremeter moduli measured at depths of 1, 2, ... 16 foundation widths below the foundation level.

The modulus used in the second term of the settlement equation should be taken as the arithmetic mean of the moduli between the foundation level and a depth of B/2 below this level.

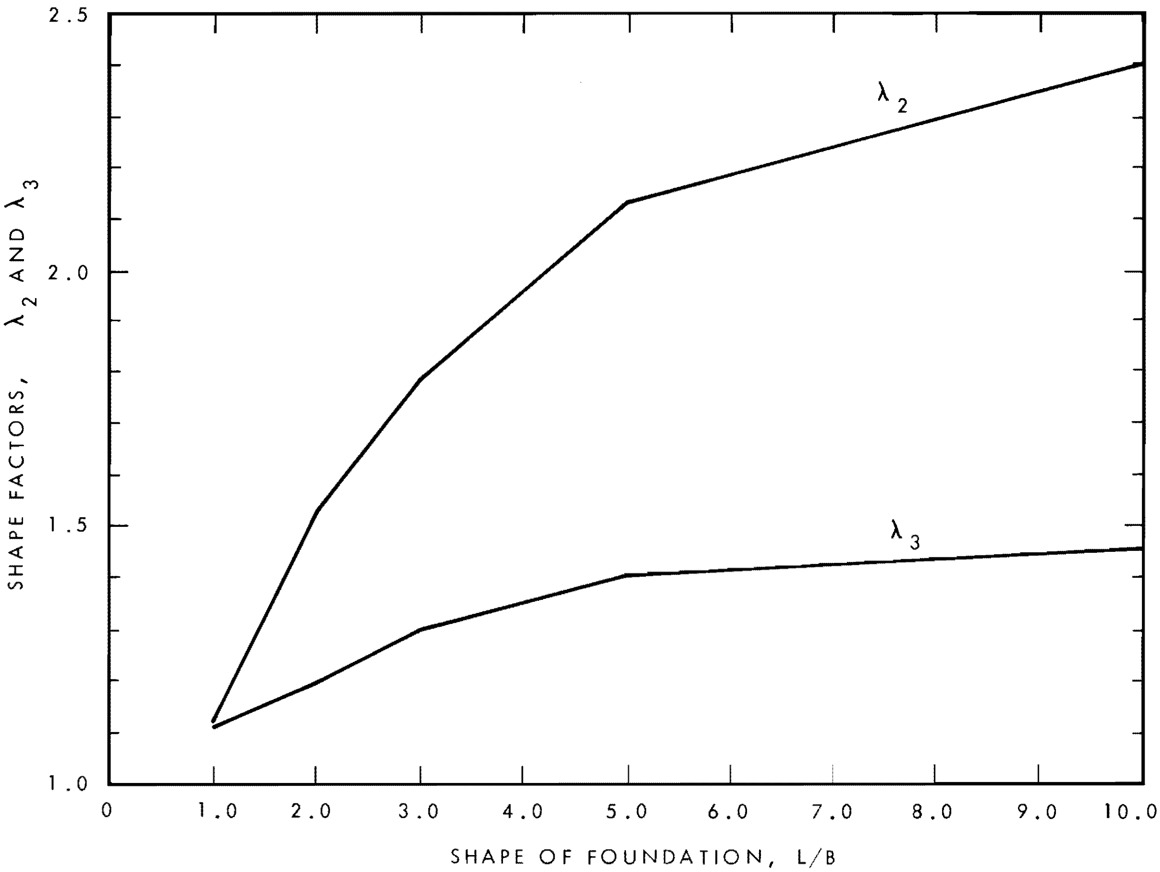


FIG 8.8.4
SHAPE FACTORS FOR SETTLEMENT CALCULATIONS

Note

The above method of predicting settlements is applicable to all non-sensitive soils supporting foundations with a width limited as compared to the depth of the soil deposit. For soft sensitive clays and in general for soils with a pressuremeter modulus of less than 30 bar, as well as for rafts, it is recommended that the predicted settlements be checked by the classical method based on oedometer test results.

The method described in Chapter 6 of this Manual is a further simplification of the method described above. It should be used only in the preliminary design of shallow foundations.

DESIGN OF DEEP FOUNDATIONS

DEEP FOUNDATIONS ON ROCK

The pressuremeter is an ideal tool for the design of deep foundations on rock, in particular for large diameter bored piles. The applicable design methods have been given in Chapter 7 and do not need to be repeated here. It should be noted however that the concepts of equivalent limit pressure p_{Le}, equivalent depth of embedment D_{fe}, and average moduli E_b, should be applied to the design of deep foundations in layered rocks.

DEEP FOUNDATIONS IN SOILS

Pressuremeter test results can be used to design deep foundations in soils, particularly in granular soils.

Point Bearing Capacity

The point bearing capacity of a pile can be estimated from the limit pressure by means of the formula

$$q_f - p_o = K_q (p_L - p_i)$$

The value of K_q applicable here is given in Fig 8.8.5. The soil classes are the same as defined earlier. A factor of safety of 3 should be applied to the term $K_q (p_L - p_i)$.

Skin Friction

The skin friction acting at any depth on the surface of a deep foundation unit can be estimated from the limit pressure p_L at that depth.

The ultimate skin friction τ_f is given in Fig 8.8.6. as a function of p_L.

- For deep foundations in cohesive soils, curve A should be used directly for concrete and timber piles; a 25% reduction should be applied for steel piles.
- For deep foundations in granular materials, curve A should be used for non-displacement, concrete piles and for displacement steel piles; a 50% reduction should be applied for non-displacement steel piles. Curve B should be used for displacement concrete piles. In no case should the skin friction be in excess of 1.2 bar.

It is recommended practice to apply a factor of safety of 2 to the skin friction determined in this way.

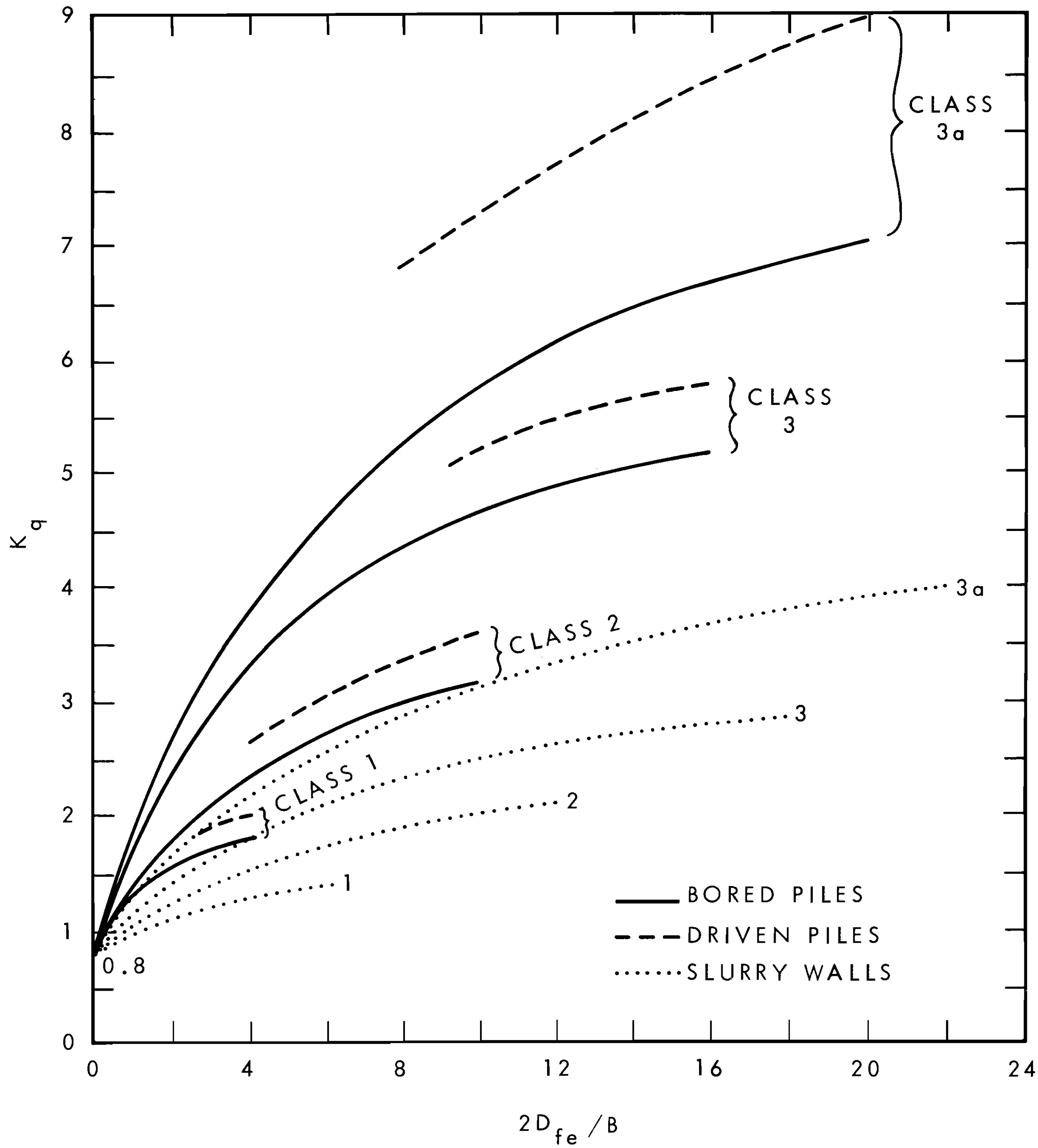


FIG 8.8.5

BEARING CAPACITY FACTOR K_q FOR DEEP FOUNDATIONS

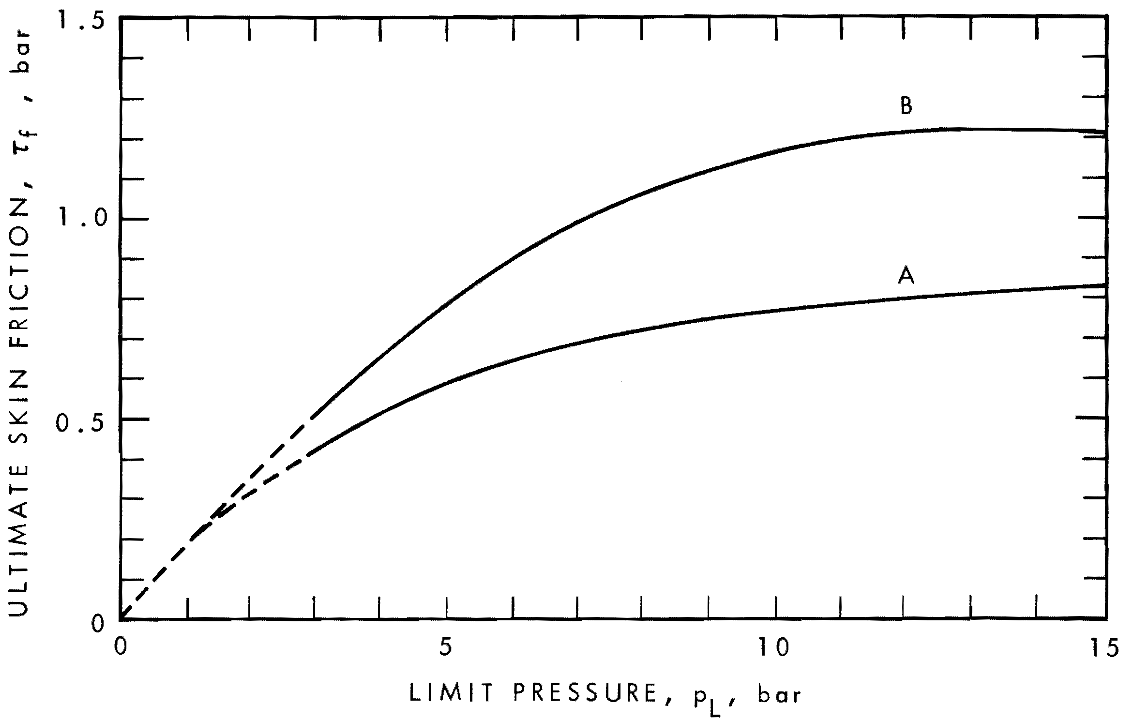


FIG 8.8.6

ULTIMATE SKIN FRICTION τ_f ON PILES

Settlements

While the settlements of deep foundations in soils can normally be predicted on the basis of pressuremeter test results, it is recommended that the methods described in Chapter 7 of this Manual be applied.

DEEP FOUNDATIONS SUBJECTED TO HORIZONTAL LOADS

The results of pressuremeter tests represent the best possible information for the design of piles subjected to horizontal loads since the stress and deformation conditions around the pressuremeter probe and the pile are nearly identical. The methods for designing deep foundations subjected to horizontal loads are given in Commentary 8.6 of this Manual.

REMARKS

The pressuremeter test is an extremely powerful tool for the investigation and design of foundations. Its use however requires a sound understanding of the standard techniques and equipment and conformity to the empirical methods already described. In particular the soil characteristics such as the undrained shear strength of clays and the modulus of deformation E_p will generally differ significantly from the values obtained by conventional tests and, if used in classical design methods would produce erroneous results.

The pressuremeter test is particularly valuable for the design of foundations on soils which are difficult to investigate by means of the conventional geotechnical methods such as dense granular soils, tills, soft rocks and frozen soils. It is particularly well suited for the design of deep foundations in such soils.

The pressuremeter appears very difficult to use in soft sensitive clays where disturbance of the soil during the opening of the borehole results in erroneous evaluation of the clay properties. As a consequence, sufficient reliable data is not yet available upon which to base the specific factors for sensitive clays that are required in these empirical design methods.

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