CANADIAN FOUNDATION ENGINEERING MANUAL

4th EDITION

CANADIAN GEOTECHNICAL SOCIETY 2006

Preface

NEPAL CENTRAL LIBRARY

The Canadian Foundation Engineering Manual is a publication of the Canadian Geotechnical Society. It is originally based on a manual prepared under the auspices of the National Research Council of Canada Associate Committee on the National Building Code, Subcommittee on Structural Design for the Building Code. A draft manual for public comment was published in 1975. In 1976, the Canadian Geotechnical Society assumed responsibility for the Manual and placed it under the Technical Committee on Foundations. This committee revised the 1975 draft and published in 1978 the first edition of the Canadian Foundation Engineering Manual, which incorporated suggestions received on the 1975 draft.

The Society solicited comments on the Manual and suggestions for revisions and additions in Seminars across the country. In 1983, the Society requested that the Technical Committee review the comments and suggestions received and prepare a second edition of the Manual published in 1985. A third edition was produced in 1992, including various revisions and additions. Further developments in applied GeoEngineering and Ground Engineering are included in this fourth edition, published in 2006.

The Manual is truly produced by the membership of the Canadian Geotechnical Society. The number of individuals who have contributed to the manual – first, the preparation of the 1975 draft, then the 1978 first edition, the 1985 second edition, the 1992 third edition and this 2006 fourth edition – is very large. Specific individuals who contributed to the fourth edition were:

D.E. Becker and I. D. Moore (Editors)
J. Lafleur (Editor, French Edition)
S.L. Barbour
R.J. Bathurst
S. Boone
R.W.I Brachman
B. Brockbank
M. Diederichs
M.H. El Naggar
J. Fannin
D. Fredlund
J. Howie
D.J. Hutchinson
J.M. Konrad
S. Leroueil
K. Novakowski
J. Shang

...AL CENTRAL LIBRARY

The Manual provides information on geotechnical aspects of foundation engineering, as practiced in Canada, so that the user will more readily be able to interpret the intent and performance requirements of the National Building Code of Canada (the release of this fourth edition coincides with publication of the NBCC, 2005) and the Canadian

Highway Bridge Design Code, 2000. The Manual also provides additional material on matters not covered by these Codes.

Foundation engineering is not a precise science, but is to a large extent based upon experience and judgement. The Manual assumes that the user is experienced in and understands the specialized field of geotechnical and ground engineering. The Manual is not a textbook, nor a substitute for the experience and judgement of a person familiar with the many complexities of foundation engineering practice.

The Manual contains:

1. Acceptable design guidelines for the solution of routine foundation engineering problems, as based on sound engineering principles and practice.

2. An outline of the limitations of certain methods of analysis.

3. Information on properties of soil and rock, including specific conditions encountered in Canada.

4. Comments on construction problems, where these influence the design or the quality of the foundation.

The Manual contains suggested rather than mandatory procedures. It is the intention of the Canadian Geotechnical Society to continue the process of review, and to update the Manual as the need arises. While reasonable efforts have been made to ensure validity and accuracy of information presented in this Manual, the Canadian Geotechnical Society and its membership disclaim any legal responsibility for such validity or inaccuracy.

Layout and design of this Manual were carried out by Barbara Goulet, Calgary, Alberta.

Comments and suggestions on the technical contents of the Manual are welcome. Such comments should be addressed to:

Canadian Geotechnical Society
Vice-President, Technical
Email: cgs@cgs.ca

Table of Contents

1 Introduction

Chapters 2 to 5 of the Canadian Foundation Engineering Manual cover fundamental matters common to all aspects of foundation engineering, such as notations, definitions of terms and symbols, the classification of soil and rock, and discussion of special site conditions. During the preparation of this 4th edition of the Manual by members of the Canadian Geotechnical Society, a companion document has been under development to focus explicitly on site characterization. Since the Manual is being published before that companion document, Chapter 4 continues to include details of site characterization and subsurface investigation in soil and rock. It is likely that a future edition of the Manual will be modified and cross-reference the Characterization Guidelines.

Chapters 6 to 8 contain general discussions of foundation design, dealing with earthquake resistant design in Chapter 6, a more general discussion of foundation design in Chapter 7, and specific treatment of Limit States Design methodologies in Chapter 8. The evolution of geotechnical engineering practice has not yet come to a point where the whole Manual can be converted to a limit states (LSD) or load and resistance factor design (LRFD) framework. Again, this will be left as a major contribution in a subsequent edition of the Manual when the status of foundation engineering practice has moved more comprehensively towards the adoption of LSD or LRFD design concepts.

Chapters 9 to 11 deal with strength and deformation of shallow foundations on rock and soil. Chapters 12, 13, 14 and 15 deal with specific considerations associated with drainage, frost action, machine foundations and foundations on expansive soils, respectively. Chapter 16 contains a discussion of techniques for ground improvement in association with foundation design and construction.

Chapters 17 to 21 deal explicitly with the design of deep foundations. Chapter 22 has a brief discussion associated with control of groundwater. Chapter 23 contains a comprehensive discussion of the design and use of geosynthetics to solve geotechnical engineering problems. Chapters 24 to 27 deal with earth retaining structures, unsupported excavations, and supported excavations and flexible retaining structures, including reinforced soil walls.

2 Definitions, Symbols and Units

2.1 Definitions

The following is a partial list of definitions of some of the terms commonly used in foundation design and construction, which are referred to in this Manual. Other terms are defined or explained where they are introduced in the text. For additional terms, see Bates and Jackson (1980).

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

Basal heave - the upward movement of the soil or rock at the base of an excavation.

Bearing pressure, allowable - in working stress design it is the maximum pressure that may be applied to a soil or rock by the foundation unit considered in design under expected loading and subsurface conditions towards achieving desired performance of the foundation system. In limit stress design, allowable bearing pressure commonly corresponds to serviceability limit states for settlement not exceeding 25 mm towards achieving desired performance of the foundation.

Bearing or contact pressure - the pressure applied to a soil or rock by a foundation unit.

Bearing pressure for settlement means the bearing pressure beyond which the specified serviceability criteria are no longer satisfied.

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

Capacity or bearing capacity or geotechnical capacity - the maximum or ultimate soil resistance mobilized by a loaded foundation unit, e.g., a footing, or a pile. (The structural capacity of a foundation unit is the ultimate resistance of the unit itself as based on the strength of the building materials).

Deep foundation - a foundation unit that provides support for a structure by transferring loads either by toe-bearing to soil or rock at considerable depth below the structure or by shaft resistance in the soil or rock in which it is placed. Piles and caissons are the most common type of deep foundation.

Downdrag - the transfer of load (dragload) to a deep foundation unit by means of negative skin friction, when soil settles in relation to the foundation unit.

Dragload - the load transferred to a deep foundation unit by negative skin friction occurring when the soil settles in relation to the foundation unit.

Dynamic method of analysis - the determination of the capacity, impact force, developed driving energy, etc. of a driven pile, using analysis of measured strain-waves induced by the driving of the pile.

Effective stress analysis - an analysis using effective stress strength parameters and specifically accounting for the effects of pore water pressure.

Excavation - the space created by the removal of soil or rock for the purpose of construction.

Factored geotechnical bearing resistance (of a foundation unit) - the factored resistance of a foundation unit, as determined by geotechnical formula using unfactored (characteristic) soil strength parameters to calculate ultimate capacity (resistance) that is multiplied by an appropriate geotechnical resistance factor, or, the ultimate capacity (as determined in a field-test loading) multiplied by an appropriate geotechnical resistance factor.

Factored geotechnical bearing resistance means the calculated ultimate (nominal) bearing resistance, obtained using characteristic ground parameters, multiplied by the recommended geotechnical resistance factor.

Factored Geotechnical Resistance at ULS - the product of the geotechnical resistance factor and the geotechnical ultimate (nominal) soil or rock resistance.

Factored load - nominal (characteristic) or specified load multiplied by the appropriate load factor.

Factored geotechnical pull out resistance (i.e. against uplift) means the calculated ultimate (nominal) pull out resistance, obtained using characteristic ground parameters, multiplied by the recommended geotechnical resistance factor.

Factored resistance (of a foundation unit) the factored geotechnical or structural resistance of the unit.

Factored geotechnical sliding resistance means the calculated ultimate (nominal) sliding resistance, obtained using characteristic ground parameters, multiplied by the recommended geotechnical resistance factor.

Factor of safety - in working stress design, the ratio of maximum available resistance to the resistance mobilized under the applied load.

Fill - artificial (man-made) deposits consisting of soil, rock, rubble, industrial waste such as slag, organic material, or a combination of these, which are transported and placed on the natural surface of soil or rock. It may or may not be compacted.

Foundation - a system or arrangement of structural members through which the loads from a building are transferred to supporting soil or rock.

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

Frost action - the phenomenon occurring 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, and/or the build-up of expansive forces under confined conditions, and the subsequent thawing that leads to loss of soil strength and increased compressibility.

Frost-susceptible soil - soil in which significant ice- segregation will occur resulting in frost heave, or heaving pressures, when requisite moisture and freezing conditions exist.

Geotechnical Reaction at SLS - the reaction of the soil or rock at the deformation associated with a SLS condition.

Geotechnical Resistance at ULS - the geotechnical ultimate resistance of soil or rock corresponding to a failure mechanism (limit state) predicted from theoretical analysis using unfactored geotechnical parameters obtained from test or estimated from assessed values.

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

Groundwater - free water in the ground.

Groundwater, artesian - a confined body of water under a pressure that gives a level of hydrostatic pore pressure (phreatic elevation) higher than the top surface of the soil unit in which the pore water pressure exists. Flowing artesian corresponds to the condition when the phreatic elevation is higher than the ground surface.

Groundwater level (groundwater table) - the top surface of free water in the ground.

Groundwater, perched - free water in the ground extending to a limited depth.

Hydrostatic pore pressure - a pore water pressure varying as pressure in a non-moving free standing column of water.

Ice-segregation - the growth of ice in lenses, layers, and veins in soil, commonly, but not always, oriented normal to the direction of heat loss.

Lateral pressure (load), design - the maximum pressure (load) that may be applied in the horizontal direction to a soil or rock by a foundation unit.

Load, service - the load actually applied to a foundation unit and which is not greater than the design load.

Load factor - the factor used to modify (usually increase) the actual load acting on and from a structure, as used in ultimate limit states design.

Negative shaft resistance - soil resistance acting downward along the side of a deep foundation unit due to an applied uplift load

Negative skin friction - soil resistance acting downward along the side of a deep foundation unit due to downdrag

Overconsolidation ratio (OCR) - the ratio between the preconsolidation pressure and the current effective overburden stress.

Peat - a highly organic soil consisting chiefly of fragmented remains of vegetable matter that is sequentially deposited.

Pier - a deep foundation unit with a large diameter to length ratio, usually, a large diameter bored pile or caisson

Pile - a slender deep foundation unit, made of materials such as wood, steel, or concrete, or combinations 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).

Pile head - the upper end of a pile.

Pile toe - a premanufactured separate reinforcement attached to the bottom end (pile toe) of a pile to facilitate driving, to protect the pile toe, and/or to improve the toe resistance of the pile.

Pile toe - the bottom end of a pile.

Pore pressure ratio - the ratio between the pore pressure and the total overburden stress.

Rock - a natural aggregate of minerals that cannot readily be broken by hand.

Rock shoe - a special type of pile shoe.

Rock quality designation (RQD) - a measure of the degree of fractures in rock cores, defined as the ratio of the accumulated lengths (minimum 100 mm) of sound rock over the total core length.

Safety factor - a factor modifying -reducing- overall capacity or strength as used in working stress design. The safety factor is defined as a ratio of maximum available resistance to mobilized resistance or to applied load.

Safety margin - the margin (dimensional) between mobilized resistance, applied load, or actual value and maximum available resistance or acceptable value, e.g., the margin between the mobilized shear stress and the shear strength, or the margin between calculated settlement and maximum acceptable settlement.

Shaft resistance - the resistance mobilized on the shaft (side) of a deep foundation. Upward acting is called positive shaft resistance. Downward acting is called negative shaft resistance (See also negative skin friction).

Shallow foundation - a foundation unit that provides support for a building by transferring loads to soil or rock located close to the lowest part of the building.

Site investigation (characterization) - the appraisal of the general subsurface conditions by analysis of information gained by such methods as geological and geophysical surveys, in-situ testing, sampling, visual inspection, laboratory testing of samples of the subsurface materials, and groundwater observations and measurements.

Slaking - crumbling and disintegration of earth material when exposed to air and moisture.

Soak-sensitive soil - soil which, when saturated, or near saturated, and subjected to a shearing force, will lose all or part of its strength. The dominant grain size fraction in this soil is usually medium and coarse silt. Soak-sensitive soil is frost-susceptible soil and, if ice-segregation occurs, when thawing it will become very soft and slough easily.

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

Specifications - project specific requirements indicating applicable codes, standards, and guidelines. Normally, Performance Specifications stipulate the end-results without detailing how to achieve them, whereas Compliance or Prescriptive Specifications detail mandatory methods, materials, etc. to use.

Total stress analysis - an analysis using undrained soil parameters and not separating the influence of pore water pressure.

2.2 Symbols

Wherever possible, the symbols in the Canadian Foundation Engineering Manual are based on the list that has been prepared by the Subcommittee on Symbols, Units, and Definitions of the International Society of Soil Mechanics and Foundation Engineering (ISSMFE, 1977, and Barsvary et al, 1980).

2.2.1 The International System of Units (SI)

In the SI-System, all parameters such as length, volume, mass, force, etc. to be inserted in a formula are assumed to be inserted with the value given in the base unit. It is incorrect to use formulae requiring insertion of parameters in other dimensions than the base units, because this would require the user to memorize not just the parameter, but also its "preferred" dimension, which could vary from reference to reference. For instance, in the well-known Newton's law, F = ma, force is to be inserted in N, mass in kg, and acceleration in m/s^2. Thus, a force given as 57 MN must be inserted as the value 57×10^6. In other words, the multiples are always considered as an abbreviation of numbers. This is a clear improvement over the old system, where every formula had to define whether the parameter was to be input as lb, tons, kips, etc. Therefore, unless specifically indicated to the contrary, all formulae given in the Manual assume the use of parameters given in base SI-units .

The term **mass** in the SI-System is used to specify the quantity of matter contained in material objects and is independent of their location in the universe [Unit =kilogram (kg); the unit Mg to indicate 1000 kg should not be used, as gramme (g) is not a base unit; nor should the unit tonne be used].

The term **weight** is a measure of the gravitational force acting on a material object at a specified location. [unit = newton (N); standard gravity at sea level = 9.81 m/s^2. In practical foundation engineering applications, the gravity constant is often taken as equal to 10 m/s^2].

The term **unit weight** in the SI System is the gravitational force per unit volume [Unit - N/m^3].

The term **density** refers to mass per unit volume [Unit = kg/m^3].

Stress and **pressure** are expressed as the force per unit area (N/m^2=Pa). The unit kilopascal (kPa) is commonly used in Canadian practice.

A **prime** denotes effective stress (e.g., σ')

A **bar** above a symbol denotes an average property (e.g., $\bar{u}$)

A **dot** above a symbol denotes a derivative with respect to time, also referred to as rate (e.g., $\dot{\varepsilon}$).

For symbols indicating force, an upper case letter is used for total force, or force per width or linear length, and a lower case letter is used for force per unit area, i.e., pressure, stress, shear resistance.

Normally, when the abbreviating symbols are not used, the units Newton, metre, kilogram, and second are spelled without plural endings (e.g., 50 kiloNewton, 200 metre, etc.)

Table 2.1 contains a list of terms, symbols, SI units, and recommended multiples for Canadian practice. The numeral 1 in the unit column denotes a dimensionless quantity.

For a complete table, see Barsvary et al. (1980).

TABLE 2.1 *List of Terms, Symbols, S.I. Units, and Recommended Multiples*

I - General		
Term	**Symbol**	**Unit and Recommended Multiples**
Length	L, l	m (km, mm, μm)
Breadth, width	B,b	m (km, mm, μm)
Thickness	H, h	m (km, mm, μm)
Height	H, h	m (km, mm, μm)
Depth	D,z	m (km, mm, μm)
Diameter	D	m (km, mm, μm)
Planar coordinates	x,y	m (km, mm, μm)
Polar coordinates	r	m (km, mm, μm)
	θ	degrees
Area	A	m^2 (km^2, ha, cm^2, mm^2)
Volume	V	m^3 (cm^3, mm^3)
Time	t	s
Velocity	v	m/s
Acceleration	a	m/s^2
Gravity acceleration	g	m/s^2
Mass	m	kg
Density	ρ	kg/m^3
Unit weight	γ	kN/m^3
Pressure, stress	σ	Pa, N/m^2 (MPa, kPa)
Shear stress	τ	Pa, N/m^2 (MPa, kPa)
Force, load	Q	N (MN, kN)
Temperature	T	degree Celsius (°C)
Energy, work	E,W	J, Nm (kJ, kNm)
Moment of Force, torque	M,T	Nm (MNm, kNm)
Safety factor	F	-
3.14	π	-
2.718 (base of natural logarithm.)	e	-
Natural logarithm	ln	-
Logarithm base 10	log	-

II - Physical Properties		
Term	**Symbol**	**Unit and Recommended Multiples**
Density and Unit Weights		
Density	ρ	kg/m^3
Unit weight	γ	kN/m^3
Density of solid particles	ρ_s	kg/m^3
Unit weight of solid particles	γ_s	kN/m^3
Density of Water	ρ_w	kg/m^3
Unit weight of water	γ_w	kN/m^3

II - Physical Properties		
Term	**Symbol**	**Unit and Recommended Multiples**
Dry density	ρ_d	kg/m³
Dry unit weight	γ_d	kN/m³
Saturated density	ρ_{sat}	kg/m³
Saturated unit weight	γ_{sat}	kN/m³
Void ratio	e	-
Porosity	n	-
Water content	w	-
Degree of saturation	S_r	-
Relative density (formerly specific gravity)	D_r	-
Consistency		-
Liquid limit	w_L	
Plastic limit	w_P	-
Shrinkage limit	w_s	-
Plasticity index	I_P	-
Liquidity index	I_L	-
Consistency index	I_C	-
Void ratio in loosest state	e_{max}	-
Void ratio in densest state	e_{min}	-
Density index (formerly relative density)	I_D	-
Grain Size		
Grain diameter	D	
n percent diameter	D_n	m (mm, μm)
Uniformity coefficient	C_u	m (mm, μm)
Curvature coefficient	C_c	-
Hydraulic Properties		-
Hydraulic head or potential	h	m (mm)
Rate of flow	q	m³/s
Flow velocity	v	m/s
Hydraulic gradient	i	-
hydraulic conductivity (permeability)	k	m/s
Seepage force per unit volume	j	N/m³(kN/m³)

III - Mechanical Properties		
Term	**Symbol**	**Unit and Recommended Multiples**
In-Situ Tests		
Cone tip-resistance	q_c	Pa (kPa)
Local side-shear	f_s	Pa (kPa)
Standard penetration test (SPT) index	N	blows/0.3 m
Dynamic cone penetrometer blow count	N_{dc}	blows/0.3 m
Pressuremeter limit pressure	P_ℓ	Pa (kPa)
Pressuremeter modulus	E_M	Pa (kPa)

III - Mechanical Properties		
Term	**Symbol**	**Unit and Recommended Multiples**
Strength		
Effective cohesion intercept	c'	Pa (MPa, kPa)
Apparent cohesion intercept	c	Pa (MPa, kPa)
Effective angle of internal friction	ϕ'	degrees
Undrained shear strength	s_u, c_u	Pa (MPa, kPa)
Residual shear strength	s_R	Pa (MPa, kPa)
Remoulded shear strength	s_r	Pa (kPa)
Sensitivity	S_t	-
Uniaxial compressive strength	σ_c	Pa (MPa, kPa)
Tensile strength	σ_t	Pa (MPa, kPa)
Point load settlement index	I_s	-
Consolidation (One-Dimensional)		
Coefficient of volume change	m_v	Pa^{-1} (kPa^{-1})
Compression index	C_c	
Recompression index	C_{cr}	
Coefficient of secondary consolidation	C_α	
Modulus number	m	
Recompression modulus number	m_r	
Swelling index	C_s	
Permeability change index	C_k	
Coefficient of consolidation (vertical)	c_v	m^2/s (cm^2/s)
Coefficient of consolidation (horizontal)	c_h	m^2/s (cm^2/s)
Time factor; vertical drainage	T_v	-
Time factor; horizontal drainage	T_h	-
Degree of consolidation	U	-
Preconsolidation Pressure	σ'_p	Pa (kPa)

IV - Stress and Strain		
Term	**Symbol**	**Unit and Recommended Multiples**
Pore pressure	u	Pa (kPa)
Pore-water pressure	u_w	Pa (kPa)
Pore-air pressure	u_a	Pa (kPa)
Total normal stress	σ	Pa (MPa, kPa)
Effective normal stress	σ'	Pa (MPa, kPa)
Shear stress	τ	Pa (MPa, kPa)
Principal stresses (major, intermediate and minor)	σ_1, σ_2, σ_3	Pa (MPa, kPa)
Average stress or octahedral normal stress	σ_{oct}	Pa (MPa, kPa)
Octahedral shear stress	τ_{oct}	Pa (MPa, kPa)
Linear strain	ε	
Volumetric strain	ε_v	

IV - Stress and Strain		
Term	**Symbol**	**Unit and Recommended Multiples**
Shear strain	γ	
Principal strains (major, intermediate, and minor)	$\varepsilon_1, \varepsilon_2, \varepsilon_3$	
Poisson's ratio	ν	
Modulus of linear deformation	E	Pa (GPa, MPa, kPa)
Elastic axial deformation	δ	m (mm, μm)
Displacement	Δ	m (mm, μm)
Modulus of shear deformation	G	Pa (GPa, MPa, kPa)
Modulus of compressibility	K	Pa (GPa, MPa, kPa)
Tangent modulus	M_t	Pa (GPa, MPa, kPa)
Secant modulus	M_s	Pa (GPa, MPa, kPa)
Modulus number	m	
Stress exponent	j	
Coefficient of Friction	μ	
Coefficient of viscosity	η	Ns/m^2 (kNs/m^2)

V - Design Parameters		
Term	**Symbol**	**Unit and Recommended Multiples**
Earth Pressure		
Earth pressure thrust, total: active and passive	P_a, P_p	N (kN, MN)
Earth pressure, unit: active and passive	p_a, p_p	Pa (kPa, MPa)
Angle of wall friction	δ	degrees
Coefficient of active and passive earth pressure	K_a, K_p	-
Coefficient of earth pressure at rest	K_o	-
Coefficient of earth pressure acting against a pile shaft	K_s	-
Foundations		
Breadth of foundations	B	m
Length of foundation	L	m
Depth of foundation beneath ground	D	m
Total length of a pile	L	m
Embedment length of a pile	D	m
Diameter of a pile	B, b	m (mm)
Applied load	Q	N (MN, kN)
Applied vertical load	Q_v	N (MN, kN)
Applied horizontal load	Q_h	N (MN, kN)
Applied (axial) pressure	q	Pa (MPa, kPa)
Settlement	s, S	m (mm)
Eccentricity of load	e	m (mm)
Inclination of load	i	degrees
Modulus of subgrade reaction	k_s	N/m^3 (kN/m^3)
Bearing capacity coefficients	N_c, N_q, N_γ, N_t	-

V - Design Parameters		
Term	**Symbol**	**Unit and Recommended Multiples**
Slopes		
Vertical height of slope	H	m
Depth below toe of earth slope to hard stratum	D	m
Angle of slope to horizontal	β	degrees
Dip of planar rock joint	φ	degrees
Depth to water table	z_w	m
Pore-pressure ratio	r_u	-
Compaction		
Maximum dry density	$\rho_{d\,max}$	kg/m^3
Maximum wet density	ρ_{max}	kg/m^3
Optimum dry density	ρ_{opt}	kg/m^3
Water content at optimum dry density	w_{opt}	-

Greek Letter Notations			
Alpha	Α	α	secondary (subscript)
Beta	Β	β	angle of slope to horizontal
Gamma	Γ	γ	shear strain unit weight
Delta	Δ	δ	angle of wall friction
	Displacement		deflection
			elastic axial deformation
			inclination to vertical
Epsilon	Ε	ε	strain
Zeta	Ζ	ζ	
Eta	Η	η	viscosity coefficient
Theta	Θ	θ	
Iota	Ι	ι	
Kappa	Κ	κ	
Lambda	Λ	λ	
Mu	Μ	μ	friction coefficient
Nu	Ν	ν	Poisson's ratio
Xi	Ξ	ξ	
Omicron	Ο	ο	
Pi	Π	π	3.14
Ro	Ρ	ρ	density
Sigma	Σ	σ	pressure, stress
	Summation sign		
Tau	Τ	τ	Shear stress, strength
Ypsilon	Υ	υ	
Phi	Φ	φ	angle of internal friction
Chi	Χ	χ	
Psi	Ψ	ψ	
	Planar joint dip		
Omega	Ω	ω	

3 Identification and Classification of Soil and Rock

3. Identification and Classification of Soil and Rock

3.1 Classification of Soils

3.1.1 Introduction

Soil is that portion of the earth's crust that 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 - containing particles that are large enough to be visible to the naked eye. They include gravels and sands and are often referred to as cohesionless or non-cohesive soils.

Fine-grained soils - containing particles that 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 often referred to as cohesive soils.

Strictly defined, coarse-grained soils are soils having more than 50% of the dry weight larger than particle size 0.075 mm (see Subsection 3.1.3.1), and fine-grained soils are soils having more than 50% of the dry weight smaller than particle size 0.075 mm.

Organic soils - containing a high natural organic content.

3.1.2 Field Identification Procedures

The following procedures and tests may be carried out in the field to identify and describe soils.

3.1.2.1 Coarse-Grained Soils or Fractions

Coarse-grained soils are easily identified in the field because the individual particles are large enough to be visible to the naked eye. The smallest particles that may be distinguished individually are approximately 0.1 mm in diameter (approximately the size of the openings of the No. 200 sieve (0.075 mm) used in the laboratory identification test). Coarse-grained soils and silts are identified on the basis of grain size diameter as follows:

Silt - particles of size	0.002 - 0.060 mm
Sand - particles of size	0.06 - 2.0 mm
Gravel - particles of size	2 - 60 mm
Cobbles - particles of size	60 - 200 mm
Boulders - particles	>200 mm

The silt, sand, and gravel fractions are further divided into fine, medium, and coarse proportions, as follows:

Silt:	Fine	0.002 - 0.006 mm
	Medium	0.006 - 0.020 mm
	Coarse	0.020 - 0.060 mm
Sand:	Fine	0.06 - 0.20 mm
	Medium	0.20 - 0.60 mm
	Coarse	0.60 - 2.00 mm
Gravel:	Fine	2.0 - 6.0 mm
	Medium	6.0 - 20.0 mm
	Coarse	20.0 - 60.0 mm

Other physical properties of soils that may influence engineering characteristics should also be identified. They are:

- Grading describes particle size distribution. A soil that has a predominance of particles of one size is 'poorly graded', whereas soil that has particles of a wide range of sizes with no dominating size is 'well graded'.
- Shape and surface conditions of grains: particles may be platy, elongated, or equidimensional, and they may be angular, sub-angular, sub-rounded, or rounded.
- A qualitative term describing the compactness condition of a cohesionless soil is often interpreted from the results of a Standard Penetration Test (SPT). This test is described in more detail in Subsection 4.5.2. Compactness and penetration values are often related according to Table 3.1, which was proposed by Terzaghi and Peck (1967). Notice that the term "compactness condition" replaces the earlier term "relative density" used in the past.

TABLE 3.1 *Compactness Condition of Sands from Standard Penetration Tests*

Compactness Condition	SPT N-INDEX (blows per 0.3 m)
Very loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very dense	Over 50

Other relationships between the SPT N-index and the compactness condition attempt to take into account the magnitude of the overburden pressure at the sampling depth to be taken into consideration. Three sets of such correlations are now available: the most commonly used set was proposed by Gibbs and Holtz (1957), but it has been modified by Schultze and Melzer (1965).

To be of practical value, the split-spoon sampling method of indirectly determining the compactness of cohesionless soil must satisfy three conditions:

1. the SPT N-index must be independent of the operator and the boring method;

2. the correlation between the SPT N-index and the compactness condition must be accurate to within acceptable limits; and

3. the same correlation between the SPT N-index and the compactness condition must be used by all.

None of these conditions is fully satisfied. It must be recognized, therefore, that the SPT is a very subjective test, and different operators can report substantially different N-values without the differences necessarily corresponding to actual variables in soil condition. A recent improvement in the testing method has been the adoption by some countries of a free-failing trip-hammer.

3.1.2.1 Fine-Grained Soils or Fractions

These procedures are to be performed on the soil fraction passing sieve No. 40, the openings of which are about 0.4 mm in diameter. For field classification purposes screening is not required because the coarse particles that interfere with the tests are simply removed by hand.

3.1.2.2(1) Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about 10 cm. If necessary, add enough water to make the soil soft but not sticky. Then, 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 from the surface, the pat stiffens, and finally 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.

3.1.2.2(2) Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mould 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. 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 specimens. Fine sand feels gritty, whereas typical silt has the smooth feel of flour.

3.1.2.2(3) Toughness (consistency near plastic limit)

After removing particles larger than the No. 40 sieve size, a specimen of soil about 10 cm in volume 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 3 mm in diameter. The thread is then folded and rolled repeatedly. During the manipulation, the moisture content is gradually reduced and the specimen stiffens, until it is no longer malleable and crumbles. This indicates that the plastic limit has been reached. After the thread has crumbled, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more active 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 in the plasticity chart; see Figure 3.1.

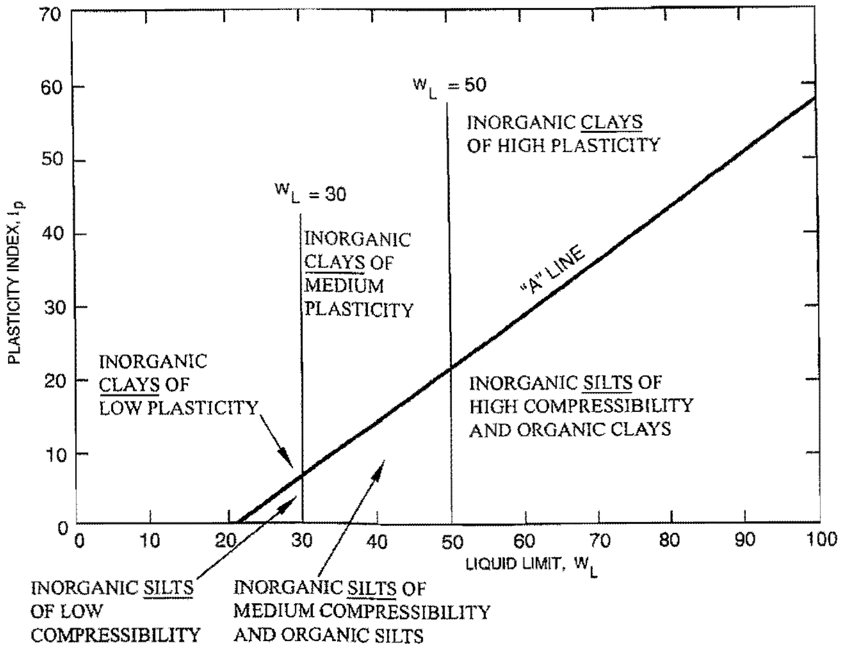


FIGURE 3.1 *The plasticity chart (after Casagrande, 1948)*

Highly organic clays have a weak and spongy feel at the plastic limit. Other physical properties of fine-grained soils, which may influence their engineering characteristics, should also be identified. Typical such properties are as follows:

3.1.2.2(4) Consistency of Cohesive Soil at Natural Water Content

TABLE 3.2 *Approximate Consistency of Cohesive Soils*

Consistency	Field Identification
Very soft	Easily penetrated several centimeters by the fist
Soft	Easily penetrated several centimeters by the thumb
Firm	Can be penetrated several centimeters by the thumb with moderate effort
Stiff	Readily indented by the thumb but penetrated only with great effort
Very stiff	Readily indented by the thumb nail
Hard	Indented with difficulty by the thumbnail

The consistency notations given qualitatively in Table 3.2 are similar to those defined by values of shear strength in Table 3.3, below. However, the field identification methods in Table 3.2 are not suitable for the quantitative determinations of soil strength.

3.1.2.2(5) Discontinuities

Discontinuities of the undisturbed soil should be identified, such as bedding, the presence of joints, cracks, fissures, or slickensides, and evidence of weathering or cementation, and thickness, orientation, and distortion.

3.1.2.2(6) Colour

Colour may be described by the Munsell system (Goddard, 1979).

3.1.2.2(7) Odour

Odour, if any, can provide evidence of the presence of organic material.

3.1.2.2 Organic Soils

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

3.1.3 Laboratory Identification Tests

3.1.3.1 Grain-Size Tests

In the laboratory, grain-size tests are carried out according to a test method, which includes procedures for analysis of coarse-grained soils (i.e., fractions larger than 0.075 mm) by sieving, and the analysis of fine-grained soils by the hydrometer test (ASTM D422).

The results of the grain size test are used to classify the soil beyond the rough separation into fine grained and coarse grained. The classification is based on amounts by weight within the respective grain-size fractions, as follows:

noun	gravel, sand, silt, clay	> 35 % and main fraction
"and"	and gravel, and silt, etc.	> 35 %
adjective	gravelly, sandy, silty, clayey, etc.	20 % - 35 %
"some"	some sand, some silt, etc.	10 % - 20 %
"trace"	trace sand, trace silt, etc.	1 % - 10 %

A soil with 30 % clay, 45 % silt, 18 % sand, and 7 % gravel would thus be named "clayey silt, some sand, trace gravel." However, the clay fraction in such a soil forms the dominant matrix, and a soil of this composition will behave geotechnically much like a clay soil. Some classification systems base the description on the plasticity chart. For example, if the I_p and w_L for the soil were to plot above the A-Line, the description would be silty clay, some sand trace gravel.

3.1.3.2 Atterberg Limits

The range of water content, called plasticity index, $I_p = w_L - w_p$, over which a fine-grained soil is plastic, is an important indicator of its probable engineering behaviour. The Atterberg limits, w_p = plastic limit and w_L = liquid limit, defining these water contents are determined in accordance with the standard ASTM methods (ASTM D423 and ASTM D424, respectively). The liquid limit can also be determined by the Swedish fall-cone test (Garneau and Lebihan, 1977). The preparation of soil samples for these tests should be determined according to Procedure B of the ASTM Standard Method for "Wet Preparation of Soil Samples for Grain-Size Analysis and Determination of Soil Constants" (ASTM D2217).

The liquid limit, w_L, is used to classify clays and silts as to degree of plasticity, as follows:

Low	degree of plasticity	$w_L < 30$
Medium	degree of plasticity	$30 < w_L < 50$
High	degree of plasticity	$50 < w_L$

In the plasticity diagram, Figure 3.1, the liquid limit is combined with the plasticity index. Experience has shown that soils with similar origin and properties plot in specific areas in the diagram, which makes the diagram a very useful tool for identifying and classifying fine-grained soils.

3.1.3.3 Classification by Undrained Strength

Fine-grained soils can be classified in broad terms in relation to undrained strength going from very soft to hard consistency (see Table 3.3). Originally, the table was based on results from unconfined compression tests. Today, however, field and laboratory vane tests, laboratory fall-cone tests, shear-box tests, unconfined compression and triaxial tests, and other test methods may be used. This implies, of course, that the classification is somewhat arbitrary, as different tests do not give the same values of strength. It also implies that when the consistency values are given, the testing method should be identified.

Commonly, the consistency and undrained shear strength of clay soils is correlated to the SPT N-Index values as shown in Table 3.3 (Terzaghi and Peck 1967). It is noted that this correlation needs to be used with caution as the correlation is only very approximate.

TABLE 3.3 *Consistency and Undrained Shear Strength of Cohesive Soils*

Consistency	Undrained Shear Strength (kPa)	Spt N-Index (blows/0.3m)
Very soft	< 12	<2
Soft	12 - 25	2 - 4
Firm	25-50	4 - 8
Stiff	50 - 100	8 - 15
Very stiff	100 - 200	15 - 30
Hard	> 200	>30

3.1.3.4 Classification by Sensitivity

Sensitivity is an important characteristic of fine-grained soils. It is defined as the ratio of intact to remoulded undrained shear strength, and is measured in the laboratory by means of the Swedish fall-cone test or in the field by means of the vane test.

Classes of sensitivity may be defined as follows:

low sensitivity	$S_t < 10$
medium sensitivity	$10 < S_t < 40$
high sensitivity	$40 < S_t$

3.1.3.5 Density Index ID

The density index, ID, of cohesionless soils is defined as

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$$

or

$$I_D = \frac{\rho_{dmax}}{\rho_d} \times \frac{\rho_d - \rho_{dmin}}{\rho_{dmax} - \rho_{dmin}} = \frac{\frac{1}{\rho_d} - \frac{1}{\rho_{dmax}}}{\frac{1}{\rho_{dmin}} - \frac{1}{\rho_{dmax}}}$$

The reference densities ($\rho_{d\,min}$ and $\rho_{d\,max}$) or the void ratios (e_{max} and e_{min}), corresponding to the loosest and the densest condition of the material under consideration, are not defined in the strict sense of the word because they are essentially related to the method used for measuring them. In today's practice, a large number of methods are in use, but the ASTM D4253 and D4254 Standard Method are generally preferred.

The in-situ dry density, ρ_d (or void ratio, e,) of the soil can be measured directly by the "sand cone" or 'rubber balloon" methods at shallow depth. At both shallow and deep depths, the "nuclear method" may be used. In addition, by means of an appropriate sampling method, an undisturbed sample of the cohesionless material might be retrieved for direct measurement of its density.

A sample of the soil is used to determine in the laboratory the minimum and maximum densities by means of an appropriate testing method, preferably the ASTM D2049 Standard. From these values, the density index can be calculated. To be of practical value in design, the measurement of all input densities must be:

- independent of the testing method;
- independent of the operator; and
- of a suitable accuracy.

Recent investigations have shown that these conditions are not fully satisfied. The density index is therefore to be regarded as very approximate, to be used only in conjunction with experience and considerable engineering judgment (Tiedemann, 1973, Tavenas et al., 1973; and Tavenas, 1973). Notice that the term "density index" replaces the earlier term "relative density."

3.1.3.6 References

The classification of soils in Canada follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. This standard is similar in many respects to the Unified Soil Classification System (USCS) used in the United States (Casagrande, 1948). Sometimes used in Canada is the system of the American Association of State Highway and Transportation Officials (AASHTO), which differs in the definitions of soil classes from the USCS. A comprehensive compilation of different classification systems was published by Holtz and Kovacs (1981).

Standards for the testing and laboratory classification of soils in Canada follow closely the standards of the American Society for Testing and Materials (ASTM). Standards of particular importance for the identification and classification of soil and rock are found in the Annual Book of ASTM Standards, Section 4, Construction, Volume 04.08 Soil and Rock; Building Stones.

3.2 Classification of Rocks

3.2.1 Introduction

Rock is a natural aggregate of minerals that cannot be readily broken by hand and that will not disintegrate on a first wetting and drying cycle. A rockmass comprises blocks of intact rock that are separated by discontinuities such as cleavage, bedding planes, joints, shears and faults. These naturally formed surfaces create weakness zones within the rockmass, thereby reducing the material strength. Even the strongest rock may contain potentially unstable blocks formed by sets of discontinuities or possibly even by a single discontinuity (Wyllie, 1992). It is usual, therefore, to investigate the structural geology of a site thoroughly, and to distinguish between the properties of the intact rock and the properties of the much larger rockmass, which includes the effects of the rock discontinuities.

The influence of the discontinuities upon the material strength depends upon the scale of the foundation relative to the position and frequency of the discontinuities.

In the following text, reference is made to the standards for rock testing developed and published by the International

Society for Rock Mechanics (ISRM).

3.2.2 Geological Classification

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

- Igneous rocks, such as granite, diorite and basalt, which are formed by the solidification of molten material, either by intrusion of magma at depth in the earth's crust, or by extrusion of lava at the earth's surface;
- Sedimentary rocks, such as sandstone, limestone and shale, which are formed by lithification of sedimentary soils; and
- Metamorphic rocks, such as quartzite, schist and gneiss, which were originally igneous, metamorphic or sedimentary rocks, and which have been altered physically and sometimes chemically or mineralogically, by the application of intense heat and/or pressure at some time in their geological history.

3.2.3 Structural Features of Rockmasses

Geological structures generally have a significant influence on rockmass properties, increasing the rockmass deformability and reducing the rockmass strength, as compared to the deformability and strength of intact rock. In some cases, discontinuities provide planes of weakness along which slip or excessive deformation can occur, leading to structurally controlled failure of the mass. Some important definitions follow:

Rockmass
An aggregate of blocks of solid rock material containing structural features that constitute mechanical discontinuities. Any in-situ rock with all of its inherent geomechanical discontinuities.

Rock material or intact rock
The consolidated aggregate of mineral particles forming solid material between structural discontinuities. The pieces may range from a few millimeters to several meters in size.

Structural discontinuities
All geological features that separate solid blocks of the rockmass, such as joints, faults, bedding planes, foliation planes, cleavage planes, shear zones and solution cavities. These features are weaker than the intact rock, thereby reducing the strength of the rockmass and increasing its deformability. A list of the different types of rockmass discontinuities and their characteristics is given in Table 3.4.

Major discontinuity or Major structure
A structural discontinuity that is sufficiently well developed and continuous such that shear failure along it will not involve shearing of the intact rock.

3.2.4 Engineering Properties of Rock Masses

The quality of a rockmass for foundation purposes depends mainly on the strength of the intact rock material and on the spacing, persistence, aperture, roughness, filling, weathering and orientation of the discontinuities. The presence of groundwater within the discontinuities may also alter the strength of the rockmass. The influence of discontinuities on rockmass characteristics and strength is further discussed by Hoek, Kaiser and Bawden (1995). Engineering properties of rockmasses can be determined from methods for estimating joint strength, for estimating rockmass strength and deformability or from rockmass classification systems.

3.2.4.1 Strength of Intact Rock

Strength is the maximum stress level that can be carried by a specimen. Rocks can be classified on the basis of their intact strength using values ranging from extremely weak to extremely strong as defined by the approximate field strength criteria set out in Table 3.5. The strength grades are related to Uniaxial Compressive Strength, σ_{ci}, and to the Point Load Strength Index. Uniaxial Compressive Strength is determined from tests on prepared cylindrical samples of intact rock as per the ISRM Standard (1979). Alternatively, strength can be determined from pieces of core or from irregularly shaped, unprepared samples of rock, using the Point Load test as per the ISRM Standard (1985). Additional strength testing on core can be by triaxial tests (ISRM, 1978; ASTM D2664-86) or by tensile strength tests (Brazilian Test, ISRM 1978; Direct Tension Test, ISRM 1978; ASTM D2936-84).

TABLE 3.4 *Rockmass Discontinuity Descriptions (after Hunt, 1986)*

Discontinuity	Definition	Characteristics
Fracture	A separation in the rockmass, a break.	Signifies joints, faults, slickensides, foliations and cleavage.
Joint	A fracture along which there has been no observable relative movement.	Most common defect encountered. Present in most formations in some geometric pattern related to rock type and stress field. Open joints allow free movement of water, increasing decomposition rate of mass. Tight joints resist weathering and the mass decomposes uniformly.
Faults	A fracture along which there has been an observable amount of displacement.	Fault zones usually consist of crushed and sheared rock through which water can move relatively freely, increasing weathering. Faults generally occur as parallel to sub-parallel sets of fractures along which movement has taken place to a greater or lesser degree.
Slickensides	Pre-existing failure surface: from faulting, landslides, expansion.	Shiny, polished surfaces with striations. Often the weakest elements in a mass, since strength is often near residual.
Foliation planes	Continuous foliation surface results from orientation of mineral grains during metamorphism.	Can be present as open joints or merely orientations without openings. Strength and deformation relate to the orientation of applied stress to the foliations.
Cleavage	Stress fractures from folding.	Found primarily in shales and slates; usually very closely spaced.
Bedding planes	Contacts between sedimentary rocks.	Often are zones containing weak materials such as lignite or montmorillonite clays.
Mylonite	Intensely sheared zone.	Strong laminations: original mineral constituents and fabric crushed and pulverized.
Cavities	Openings in soluble rocks resulting from groundwater movement, or in igneous rocks from gas pockets	In limestone, range from caverns to tubes. In rhyolite and other igneous rocks, range from voids of various sizes to tubes.

TABLE 3.5 *Classification of Rock with Respect to Strength (after Marinos and Hoek, 2001)*

Grade *	Term	Uniaxial Comp. Strength (mpa)	Point Load Index (mpa)	Field Estimate of Strength	Examples
R6	Extremely strong	> 250	> 10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 – 250	4 – 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, rhyolite, tuff
R4	Strong	50 – 100	2 – 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25 – 50	1 – 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5 – 25	***	Can be peeled with a pocket knife with difficulty, shallow indentation made by a firm blow with the point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt
R1	Very weak	1 – 5	***	Crumbles under firm blows with point of a geological hammer, can be peeled with a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25 - 1	***	Indented by thumbnail	Stiff fault gouge

* Grade according to ISRM (1981).

** All rock types exhibit a broad range of uniaxial compressive strengths reflecting heterogeneity in composition and anisotropy in structure. Strong rocks are characterized by well-interlocked crystal fabric and few voids.

*** Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.

Some natural materials, which geologically may be referred to as rock, should be treated from an engineering point of view as soils. Some examples of materials that fall into this category include:

- Soft or weakly cemented rocks with unconfined or uniaxial compressive strength < 1 MPa;
- Any material that can be dug by hand with a shovel;
- Cemented sands and gravels, in which the cementing is discontinuous; and
- Rocks such as: marl and volcanic tuff, highly altered or crushed rocks, rocks with closely spaced continuous joints, and residual soils containing rock fragments.

The strength of sedimentary rocks derived from clay and silt sized particles, such as shale or mudstone, generally degrades when exposed to repeated cycles of wetting and drying. The slake durability test can be used to determine whether the rock will degrade, and if so, how rapidly this will occur. Standards for the slake durability test are provided by the ISRM (1979).

3.2.4.2 Characteristics of Discontinuities

The structural integrity of a rockmass will be affected by the presence of discontinuities. Major, discrete, through-going structures such as shears, faults or other major weakness zones will dominate the rockmass behaviour where they are present. Ubiquitous (present everywhere) structure will also affect the behaviour of the rockmass. Systems of extension joints and minor shear structures will have formed under historical stress fields, which were relatively consistent over a local region. As a result, there are usually several distinct groups of similarly oriented structures within a rockmass, termed joint sets or joint families. Ungrouped joints are defined as random. Both discrete and ubiquitous features should be measured, characterized and analysed.

Full characterization of a rockmass requires measurement of a number of characteristics of the discontinuities, including discontinuity orientation (Section 3.2.4.3), discontinuity strength (Section 3.2.4.4) and discontinuity spacing (Section 3.2.4.5). Guidance for description of discontinuities in rockmasses is provided by the ISRM (1978).

3.2.4.3 Discontinuity Orientation

Discontinuities are considered to be adversely oriented if they provide minimal or limited resistance to sliding under the applied load. Joint orientation can be found from logging drill cores, surveying boreholes and/or from mapping surface exposures of the rockmass.

To determine joint orientation from core logging, measurements must be made on oriented core. It is essential that the orientation of the borehole be recorded. It is then necessary to take two measurements of orientation for each joint or discontinuity: alpha, the minimum angle between the maximum dip vector of the discontinuity and the core axis, and beta, the dip direction of the plane, measured clockwise from north or the reference line for the core. The true orientation of the discontinuities with respect to north can then be calculated following procedures defined by Priest (1985).

Joint orientation can also be found by surveying the drill holes. Surveys can be conducted by inspecting the holes with borehole cameras, periscopes or probes. Generally, the orientation of each feature can be determined by examining the angle the feature makes with the hole, and the length of the inscribed circle or oval created by the discontinuity around the perimeter of the borehole. The calculation of the true orientation of the feature depends upon both the orientation and the diameter of the drill hole.

Mapping of joint characteristics can also be carried out on exposures of the rockmass on outcrops, or in other excavations where the rock is exposed, such as shafts, trenches and adits. In these locations, the dip and dip direction of each discontinuity can be measured directly on surface exposures of each structure, using a geological compass. It is important when mapping rockmass exposures that the length of the sampling window, or scanline, is of sufficient length to sample enough features to provide a statistically valid basis for analysis. A minimum of 100 local measurements are normally required to define the structure in a localized zone of rock (Hutchinson and Diederichs, 1996).

Priest and Hudson (1976) suggest that between 150 and 350 measurements should be taken at a number of sample locations, selected to provide data about different lithologies, or about highly variable discontinuity characteristics. When establishing a mapping program it is important to consider the following issues:

- Increased numbers of measurements improve the data precision as well as confidence in the output.
- Increased length of sampling or scan lines leads to increased precision in the data.
- Measurements taken from scan lines of similar orientation will be subject to data bias. Therefore it is advisable to orient successive scanlines in different orientations where possible, and to correct for bias (Terzaghi, 1965).

Where the rockmass quality and nature are variable, it is important to separate the data into sub-sets, on the basis of distinct geological conditions, if possible. For example, where discontinuities have been measured in a rockmass comprising two distinct and substantial lithologies, the structural analysis should be carried out on the full data set, and then on sub-sets divided on the basis of lithology, to determine if the structural patterns are different.

As noted previously, it is important to distinguish between discrete and ubiquitous structures in analysis of the rockmass stability and strength. The ubiquitous structures can generally be grouped into one or more sets with similar orientation. Random joints may also be present in the rockmass. The visual examination and statistical grouping of structural data into sets is best accomplished using a stereonet. The outcome of this work is generally a representative (mean) orientation for each cluster or set of joint data. Further information regarding information plotting and data analysis on stereonets is provided by Hoek et al (1995) and Priest (1993).

3.2.4.4 Discontinuity Strength

Discontinuity strength can be defined using several distinct formulations. These include the strength criterion proposed by Barton and Choubey (1977) and further discussed by Hoek at al (1995), as well as the simplified Mohr-Coulomb analysis, requiring input parameters of friction, φ, and cohesion, c, discussed by Wyllie (1992).

The most accurate measurement of discontinuity strength is made by performing direct shear tests, which can be carried out in the laboratory or in-situ on undisturbed samples. Guidelines for performing these tests are given by Wyllie (1992) and the ISRM (1974).

The strength of a discontinuity depends upon the roughness, persistence, and aperture, as well as upon the presence of any infilling or water. Each of these parameters, defined below, should be measured during any geotechnical mapping program.

Roughness of a discontinuity adds to its resistance to shear, especially when the asperities on one side of the discontinuity interlock with those on the other side.

The importance of surface roughness declines as the aperture, filling thickness and previous displacement along the discontinuity increase. Roughness is generally measured by comparing observations to published surface profiles providing an estimate of the Joint Roughness Coefficient (JRC) (Barton, 1973; Barton and Choubey, 1977; Hoek et al, 1995).

Roughness can be divided into small-scale and larger-scale roughness. The small-scale roughness, measured over a sample distance of up to 10 cm, is defined as rough, smooth or polished (slickensided). Roughness at the metre scale is termed stepped, undulating or planar.

Joint persistence is an estimate of the length of each individual joint. Joints may range from non-persistent or not continuous, through to highly persistent or fully continuous. Joints, which are highly persistent (long), are more likely to combine with other structures to form large free blocks of rock, than are short joints.

Joint aperture is the perpendicular distance separating the adjacent walls of an open discontinuity, which may be water filled. Other fillings of the discontinuity should be described separately, as discussed in the next point. Aperture provides an indication of the secondary permeability of the rockmass as well as some idea of its looseness. Unfortunately, apertures that can be observed directly are usually disturbed by blasting, excavation and weathering. Observations of the less disturbed rockmasses exposed within boreholes, using a borehole camera or periscope, can be very useful.

Where possible, the joint aperture should be measured using feeler gauges, or a measuring tape, and classified as shown in Table 3.6. Impression packer testing can also be used to provide a measurement of the aperture as well.

TABLE 3.6 *Classification of Joint Aperture*

Joint Aperture	Description
< 0.5 mm	Closed
0.5 to 10 mm	Gapped
> 10 mm	Open

Where permeability of the joints is of importance, in-situ permeability testing should be carried out. During the mapping program, observations of any evidence of current or previous water flow along the joints should be recorded. Classification of the joints based on these observations can be made using Table 3.7.

TABLE 3.7 *Classification of Discontinuities depending upon Water Flow*

Class	Description
1	Water flow not possible
2	No evidence of water flow
3	Evidence of water flow (e.g. rust staining)
4	Dampness
5	Seepage
6	Flow (volume per unit of time)

Joint infilling is the material separating the adjacent rock walls of discontinuities. It may be formed by the in-situ weathering or alteration of the rock adjoining the discontinuity, or it may be transported. It may be described by the methods used for the field identification of soils (see Section 3.1). The width of the filled discontinuity, the mineralogy of the infilling, and the roughness of the discontinuity walls will all affect the strength and deformability of the discontinuity and should be examined and described. Water flow can be described in terms of the classes shown in Table 3.8.

TABLE 3.8 *Classification of Filled Discontinuities depending upon Water Flow Proposed by the ISRM (1981)*

Class	Description
1	Filling is dry and has low permeability
2	Filling is damp; no free water is present
3	Filling is wet; drops of free water are present
4	Filling shows outwash; continuous flow of water is present
5	Filling is locally washed out and there is considerable water flow along channels

LIBRARY

3.2.4.5 Discontinuity Spacing

Discontinuity spacing is important because closely spaced joints result in a smaller block size, increasing the potential for internal shifting and rotation as the rockmass deforms, and thereby reducing stability.

Discontinuity spacing is defined by Priest (1993) as the distance between a pair of discontinuities measured along a line of specified location and orientation (or scanline). He defines three main types of discontinuity spacings as follows:

1. Total spacing is the spacing between a pair of immediately adjacent discontinuities measured along a line of any specified orientation.

2. Set spacing is the spacing between a pair of immediately adjacent discontinuities from a particular discontinuity set, measured along a line of any specified orientation.

3. Normal set spacing is the set spacing measured parallel to the mean normal to the set.

The spacing of discontinuities can vary from extremely wide to extremely close, as shown in Table 3.9. In this case, the distance between adjacent discontinuities is measured over a sampling length not shorter than 3 meters. The sampling length should be greater than ten times the estimated discontinuity spacing, if possible (ISRM, 1981).

TABLE 3.9 *Classification of Rock with Respect to Discontinuity Spacing (ISRM, 1981)*

Spacing Classification	Spacing Width (m)
Extremely close	< 0.02
Very close	0.02 to 0.06
Close	0.06 to 0.20
Moderately close	0.20 to 0.6
Wide	0.6 to 2.0
Very wide	2.0 to 6.0
Extremely wide	> 6.0

Rock Quality Designation, or RQD, originally proposed by Deere et al. (1967) is an indirect measure of the number of fractures within a rockmass. The method provides a quick and objective technique for estimating rockmass quality during diamond drill core logging, as shown in Table 3.10.

RQD is calculated as follows:

$$\text{RQD}(\%) = \frac{\Sigma \text{ Length of core pieces} > 10 \text{ cm} \times 100}{\text{Total length of core run}}$$

TABLE 3.10 *Classification of Rock with Respect to RQD Value*

Rqd Classification	Rqd Value (%)
Very poor quality	< 25
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

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 should be fitted together and counted as one piece. Some judgment is necessary in the case of thinly bedded sedimentary rocks and foliated metamorphic rocks, and the method is not as precise in these cases as it is for igneous rocks or for thickly bedded limestones or sandstones. The system has been applied successfully to 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 rockmasses where recovery is poor. This is appropriate because poor core recovery usually reflects poor quality rock. Poor drilling equipment and techniques can cause poor recovery. For this reason, double-tube core barrels of at least NX size (54 mm in diameter) must be used, and proper supervision of drilling is imperative. It is noted that the original definition for RQD Index was based on N size core.

Philosophically, RQD provides a crude estimate of the percentage of the rockmass which can be expected to behave in a fashion similar to a laboratory sample (typically 10 cm long). Rockmass with a low RQD (< 50%) has few intact blocks larger than 10 cm. In such rockmasses, joints and fractures dominate the rock's response to stress. The strength and stiffness of the rock, as determined in the lab, has little relevance here. On the other hand, rockmasses with RQD > 95% possess strength and stiffness much closer to the values obtained in the lab. Joints may still dominate behaviour, especially in the low stress environments of most foundations. A semi-empirical technique for evaluating rockmass strength and deformability is discussed in the following section.

A great deal of work has been done to correlate RQD with joint frequency, rockmass stiffness and other properties. The interested reader is referred to Deere and Miller (1966), Deere and Deere (1988), Cording and Deere (1972), Coon and Merritt (1970) and Bieniawski (1979).

3.2.4.1 Jointed Rockmass Strength and Deformability

The strength of the rockmass will depend on such factors as the shear strength of the surfaces of the blocks defined by discontinuities, their continuous length, and their alignment relative to the load direction (Wyllie, 1992). If the loads are great enough to extend fractures and break intact rock, or if the rockmass can dilate, resulting in loss of interlock between the blocks, then the rockmass strength may be diminished significantly from that of the in-situ rock. Where foundations contain potentially unstable blocks that may slide from the foundation, the shear strength parameters of the discontinuities should be used in design (Section 3.2.4.4), rather than the rockmass strength.

Direct measurements of rockmass deformability are best conducted in-situ for foundations carrying substantial loads, for example major bridge footings. The tests available include borehole jacking tests, plate load tests and radial jacking tests for the rockmass modulus. Direct shear tests are used to determine the shear strength of the fractures. Further details regarding these tests and the use of the data so derived are provided by Wyllie (1992). He also notes that the test results should be checked against values calculated from the performance of other foundations constructed in similar geological conditions.

Strength and deformation properties of jointed rockmasses can be estimated using the Hoek-Brown failure criterion (Hoek and Brown, 1997) from three parameters (Hoek and Marinos, 2000; Marinos and Hoek, 2001):

- The uniaxial compressive strength of the intact rock elements contained within the rockmass (see Section 3.2.4.1).
- A constant, mi, that defines the frictional characteristics of the component minerals within each intact rock element.
- The Geological Strength Index (GSI) which relates the properties of the intact rock elements to those of the overall rockmass (Table 3.12).

The generalized Hoek-Brown failure criterion is defined as: $$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \qquad (3.1)$$

where σ'_1 and σ'_3 are the maximum and minimum effective stresses at failure
σ_{ci} is the uniaxial compressive strength of the intact rock pieces
m_b is the value of the Hoek-Brown constant m for the rockmass, and

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right)$$

m_i is the Hoek-Brown constant for the intact rock (Table 3.11)
s and a are constants which depend upon the rockmass characteristics.

For GSI > 25, $a = 0.5$, and $s = \exp\left(\frac{GSI - 100}{9}\right)$

For GSI < 25, s = 0, and $a = 0.65 - \frac{GSI}{200}$

The deformation modulus for weak rocks (σ_{ci} < 100 MPa), can be estimated from the following equation (Marinos and Hoek, 2001):

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}}10^{\left(GSI-10/40\right)} \tag{3.2}$$

Marinos and Hoek (2001) caution that this criterion is only applicable to 'isotropic' rockmasses, wherein the strength of the whole mass controls its behaviour. In anisotropic rockmasses, such as a strong, blocky sandstone, where the blocks are separated by clay coated and slickensided bedding surfaces, the rockmass behaviour is controlled by the discontinuities.

The Hoek-Brown constant, m_i, can be determined from triaxial testing of core samples, using the procedure discussed by Hoek et al (1995), or from the values given in Table 3.11. Most of the values provided in the table have been derived from triaxial testing on intact core samples. The ranges of values shown reflect the natural variability in the strength of earth materials, and depend upon the accuracy of the lithological description of the rock. For example, Marinos and Hoek (2001) note that the term granite describes a clearly defined rock type that exhibits very similar mechanical characteristics, independent of origin. As a result, m_i for granite is defined as 32 ± 3. On the other hand, volcanic breccia is not very precise in terms of mineral composition, with the result that m_i is given as 19 ± 5, denoting a higher level of uncertainty. The ranges of values depend upon the granularity and interlocking of the crystal structure. The higher values are associated with tightly interlocked and more frictional characteristics.

Values for the Geological Strength Index (GSI), which relates the properties of the intact rock elements to those of the overall rockmass, are provided in Table 3.12. A similar table, developed for heterogeneous rockmasses, is provided by Marinos and Hoek (2001).

3.2.4.2 Rockmass Classification

A number of classification systems have been developed to provide the basis for engineering characterization of rockmasses. An excellent overview of these techniques is provided by Hoek et al. (1995). Most of the classification systems incorporating a number of parameters (Wickham et al., 1972; Bieniawski, 1973, 1979, 1989; Barton et al., 1974), were derived from civil engineering case histories in which all components of the engineering geological character of the rockmass were considered. More recently, the systems have been modified to account for the conditions affecting rockmass stability in underground mining situations.

While no single classification system has been developed for or applied to foundation design, the type of information collected for the two more common civil engineering classification schemes, Q (Barton et al, 1974) and RMR (Bieniawski, 1989) should be considered. These techniques have been applied to empirical design situations, where previous experience plays a large part in the design of the excavation in the rockmass. Empirical techniques are not used in foundation engineering, where a more concentrated expenditure of effort and resources is required and possible, due to the much smaller spatial extent of the work, and the relatively high external loads applied to the rockmass.

TABLE 3.11 *Values of Hoek-Brown Constant m_i for Intact Rock, by Rock Group (after Marinos and Hoek, 2001)*

Sedimentary	Clastic		Conglomerate Breccia *	Sandstone 17±4	Siltstone 7±2 Greywacke (18±3)	Claystone (4±2) Shale (6±2) Marl (7±2)
	Non-clastic	Carbonates	Crystalline Limestone (12±3)	Spartic Limestone (10±2)	Micritic Limestone (9±2)	Dolomite (9±3)
		Evaporites		Gypsum 8±2	Anhydrite 12±2	
		Organic				Chalk 7±2
Metamorphic	Non-foliated		Marble 9±3	Hornfels (19±4) Meta Sandstone (19±3)	Quartzite 20±3	
	Slightly foliated		Migmatite (29±3)	Amphibolite 26±6	Gneiss 28±5	
	Foliated **			Schist 12±3	Phyllite (7±3)	Slate 7±4
Igneous	Plutonic	Light	Granite 32±3 Granodiorite (29±3)	Diorite (25±5)		
		Dark	Gabbro 27±3 Norite 20±5	Dolerite (16±5)		
	Hypabyssal		Porphyry (20±5)		Diabase (15±5)	Peridotite (25±5)
	Volcanic	Lava		Rhyolite (25±5) Andesite 25±5	Dacite (25±3) Basalt (25±5)	
		Pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±5)	

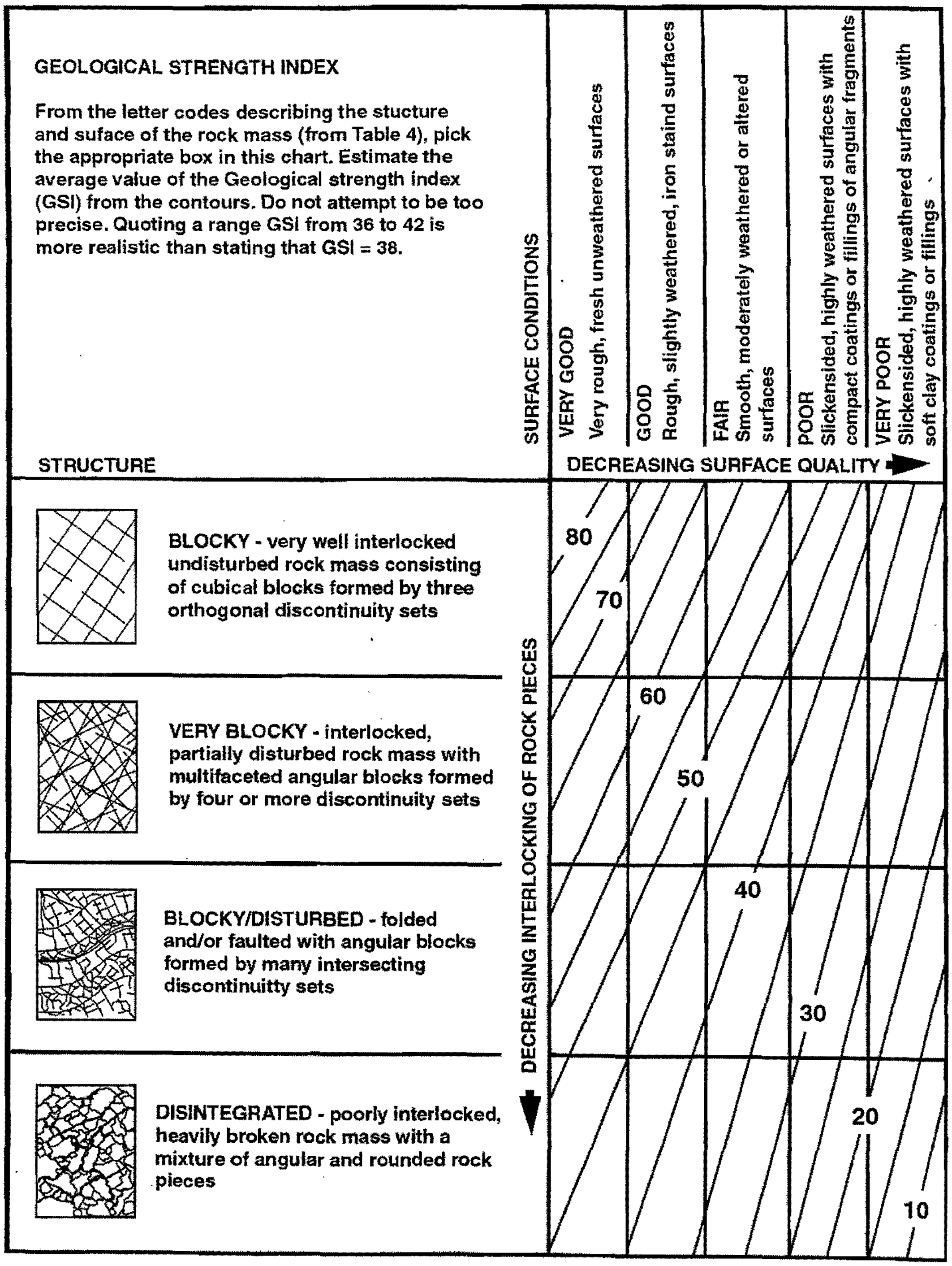
Notes:

Values in parentheses are estimates.

* Conglomerates and breccias may have a wide range of values, depending on the nature of the cementingmaterial and the degree of cementation. Values range between those of sandstone and those of fine-grained sediments.

** These values are for intact rock specimens tested normal to bedding or foliation. Values of m_i will be significantly different if failure occurs along a weakness plane.

TABLE 3.12 *GSI Estimates for Rockmasses, from Hoek and Marinos (2000)*



4 Site Investigations

4. Site Investigations

4.1 Introduction

A site investigation involves the appraisal and characterization of the general subsurface conditions by analysis of information gained by such methods as geological and geophysical surveys, drilling boreholes, and sampling, in-situ testing, laboratory testing of samples of the subsurface materials, groundwater observations, visual inspection, and local experience.

The site investigation is one of the most important steps in any foundation design, and should be carried out under the direction of a person with knowledge and experience in planning and executing such investigations. Drilling crews should be experienced specifically in borings for geotechnical explorations. A valuable guide is provided by ASCE (1976).

4.2 Objectives of Site Investigations

An engineer requires sufficient knowledge of the ground conditions at a site to estimate the response of the soils or rocks to changes induced by the site works. Peck (1962) noted that the three factors of most importance to the successful practice of subsurface engineering were:

- Knowledge of precedents
- A working knowledge of geology
- Knowledge of soil mechanics.

A knowledge of precedents in similar ground conditions helps to ensure that no surprises are encountered in the design and construction of the works; knowledge of geology should enable the engineer to anticipate the range of possible variations in ground conditions between the locations of any borings; and knowledge of soil or rock mechanics should minimize the chances of inadequate performance of the ground during and after construction.

A site characterization should be carried out for all projects. The level of detail of any characterization should be appropriate to the proposed site use and to the consequences of failure to meet the performance requirements. The engineer should be able to prepare a design that will not exceed ultimate and serviceability limit states (see Chapters 7 and 8 for further discussion). This means that there should be no danger of catastrophic collapse and deformations and other environmental changes should be within tolerable limits. Depending on the particular nature of the proposed development, the site characterization may or may not involve field exploration.

Once the scope of work has been established for the proposed engineering works, the site characterization should comprise three components:

- Desk Study and Site Reconnaissance
- Field Exploration

- Reporting.

The first component is the most critical. It consists of a review of existing information about the site including the geology. Attention to detail in this phase in conjunction with a site reconnaissance to review existing surface conditions will minimize the potential for surprises during subsequent field exploration and construction. The extent of this phase of the work will depend on the experience of the engineer in the particular geological environment and with similar foundation systems or soil structures.

Upon completion of this phase, a preliminary sub-surface model of the site should have been established, enabling consideration of foundation design issues and preliminary selection of foundation options. The engineer may proceed to plan an appropriate field exploration.

The primary objectives of field exploration are to determine as accurately as may be required:

- the nature and sequence of the subsurface strata;
- the groundwater conditions at the site;
- the physical properties of the soils and rock underlying the site; and
- other specific information, when needed, such as the chemical composition of the groundwater, and the characteristics of the foundations of adjacent structures.

Site investigations should be organized to obtain all possible information commensurate with project objectives for a thorough understanding of the subsurface conditions and probable foundation behaviour. Additional information on the objectives, planning and execution of site investigations is provided by Becker (2001).

At the very least, the field exploration should confirm the preliminary subsurface model developed during the planning phase and should provide sufficient characterization of material properties to allow estimation of the response of the site to the proposed engineering works. In many cases, the macrostructure of the ground such as jointing and fissuring will control the site and foundation performance during and after construction. An understanding of site geology will allow the engineer to anticipate such cases and field exploration should determine the presence of any layers or zones likely to cause difficulty during construction or operation of the facility. For example, thin weak layers may be critical for stability or thin permeable layers may be critical in excavations. The selection of an appropriate exploration technique should be based on a clear understanding of the critical failure modes and on the types of layers likely to be present.

Upon completion of the stratigraphic logging and material classification, appropriate design parameters can be selected. This can be done on the basis of one or a combination of the following:

- Experience with similar foundations in similar ground conditions,
- Correlation with the known properties of soils or rocks from other sites with similar classification properties,
- Sampling and laboratory testing
- In-situ testing.

4.3 Background Information

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

- the type of structure to be built, its intended use, characteristics of the structure, intended construction method, starting date, and estimated period of construction;
- the behaviour of existing structures adjacent to the site, as well as information available through local experience; and
- the probable soil conditions at the site by analysis of geological and geotechnical reports and maps, aerial photographs, and satellite photographs.

4.4 Extent of Investigation

4.4.1 Introduction

The extent of the ground investigation is determined by the soil type and variability of soil and groundwater, the type of project, and the amount of existing information. It is important that the general character and variability of the ground be established before deciding on the basic principles of the foundation design of the project.

The combination of each project and site is likely to be unique, and the following general comments should therefore be considered as a guide in planning the site investigation and not as a set of rules to be applied rigidly in every case.

The greater the natural variability of the ground, the greater will be the extent of the ground investigation required to obtain an indication of the character of the ground. The depth of exploration is generally determined by the nature of the project, but it may be necessary to explore to greater depths at a limited number of locations to establish the overall geological conditions.

The investigation should provide sufficient data for an adequate and economical design of the project. It should also be sufficient to cover possible methods of construction and, where appropriate, indicate sources of construction materials. The lateral and vertical extent of the investigation should cover all ground that may be significantly affected by the project and construction, such as the zone of stressed ground beneath the bottom of a group of piles, and the stability of an adjacent slope, if present.

The boreholes should be located so that a general geological view of the whole site can be obtained with adequate details of the engineering properties of the soils and rocks and of groundwater conditions. More detailed information should be obtained at the location of important structures and foundations, at locations of special engineering difficulty or importance, and where ground conditions are complicated, such as suspected buried valleys and old landslide areas. Rigid, preconceived patterns of boreholes should be avoided. In some cases, it will not be possible to locate structures until much of the ground investigation data has been obtained. In such cases, the program of investigations should be modified accordingly. In the case of larger projects, the site investigation is often undertaken in stages. A preliminary stage provides general information and this is followed by a second stage and, if required, additional stages as the details of the project and foundation design develop.

Reference is made to boreholes as the means of site investigation. However, in some cases, boreholes can be replaced by, or supplemented by, test pits, test trenches, soundings or probe holes. Regardless of the type of investigation, it is essential that the locations and ground levels for all exploration points be established, if necessary, by survey.

Information and recommendations on the extent of site investigations, both depth and number of boreholes, can be found in various references. The references that have served as the basis for some of the comments presented in this section include ASCE (1976), British Standards Institution, BS 5930 (1981) and Navfac DM 7.01 (1986).

Robertson (1997) suggested the risk-based approach to characterization shown on Figure 4.1. For low risk projects (small to medium sized jobs with few hazards and limited consequences of failure), it is only necessary to classify the soils visually and, perhaps, by index testing to allow selection of design parameters. Design may then be based on presumptive bearing pressures. For medium risk projects, some form of in-situ testing will be necessary. The in-situ testing conventionally consists of penetration testing from which some estimate of the soil properties can be obtained by correlation. Design methods are also available where in-situ test results are used directly to select design values of bearing pressure. Where the consequences of unexpected ground response result in an unacceptable level of risk, a much more elaborate field and laboratory program should be carried out.

Suggestions for the depth of boreholes and spacing of boreholes are considered in the following sections. The suggestions for minimum depth of boreholes can be more definitive since there is a logical analytical basis. The minimum depth is related to the depth at which the increase in soil stress caused by foundation loads is small and will not cause any significant settlement. The suggestions for spacing of boreholes are however, more difficult to make and less definitive since much depends on the soil variability, type of project, performance requirements, and foundation type selected.

4.4.2 Depth of Investigation

The site investigation should be carried to such a depth that the entire zone of soil or rock affected by changes caused by the structure or construction will be adequately explored. The following recommendations are provided as guidelines:

- A commonly used rule of thumb for minimum depth of boreholes is to extend the boreholes to such a depth that the net increase in soil stress under the weight of the structure is less than 10% of the applied load, or less than 5 % of the effective stress in the soil at that depth, whichever is less. A reduction in the depth can be considered if bedrock or dense soil is encountered within the minimum depth. In the case of very compressible normally consolidated clay soils located at depth, it may be necessary to extend boreholes deeper than determined by the 10 % and 5 % rules.
- The net increase in soil stress should appropriately take into account the effect of fill or excavation that may be required for site grading.
- The soil stress increase should take into account adjacent foundations since they may increase the soil stress at depth, and the corresponding minimum depth of boreholes.
- Boreholes should extend below all deposits that may be unsuitable for foundation purposes such as fill ground, and weak compressible soils.
- The minimum borehole depth beneath the lowest part of the foundation generally should not be less than 6 m, unless bedrock or dense soil is encountered at a shallower depth.
- If rock is found the borehole should penetrate at least 3 m in more than one borehole to confirm whether bedrock or a boulder has been found. Three meters may not be adequate for some geological conditions; e.g., where large slabs of rock may occur as rafts in till deposits. No guidance can be given in such cases but where doubt arises, consideration should be given to drilling deeper boreholes.
- In the case of end bearing piles on rock, the boreholes should be deep enough to establish conclusively the presence of bedrock as considered previously. Furthermore, the boreholes or selected number of boreholes should be extended to a sufficient depth to minimize the possibility of weaker strata occurring below the bedrock surface which could affect the performance of piles. In addition, when weathered rock is present, the boreholes should extend to a sufficient depth into the unweathered rock.
- Since the foundation type and design is not always finalized at the beginning of the site investigation, it may be prudent to drill holes deeper than originally estimated to allow some variation during project development.
- Not all boreholes need to be drilled to the same depth since shallower intermediate boreholes may provide adequate information for more lightly loaded foundations. Also, the level of detailed sampling and in-situ testing may vary considerably from borehole to borehole, depending on the design needs.
- Pile-supported rafts on clays are often used solely to reduce settlement. In these cases, the depth of exploration is governed by the need to examine all strata that could contribute significantly to the settlement. A commonly used approximation in settlement calculations for piled rafts is to assume that the entire load is carried on an imaginary raft located at a depth equal to two-thirds of the pile length. The borehole depth should extend to the level at which the soil stress increase from the imaginary raft is small and will not cause significant settlement. In practice, on many occasions, this would lead to an excessive and unnecessary depth of exploration so the engineer directing the investigation should terminate the exploration at the depth where the relatively incompressible strata have been reached.
- Fill ground, and weak compressible soils seldom contribute to the shaft resistance of a pile and may add downdrag to the pile load. The entire pile load, possibly with the addition of downdrag, will have to be borne by the stronger strata lying below the weak materials. This will increase the stress at the bottom of the piles and consequently the corresponding depth of boreholes.
- For driven pile foundations the length of the piles is not known with any accuracy until installation of test piling or construction begins. Selection of the depth of boreholes should make an allowance for this

uncertainty. General guidance can be provided from previous experience in the area.

- If any structure is likely to be affected by subsidence due to mining or any other causes, greater exploration depths than those recommended above may be required.

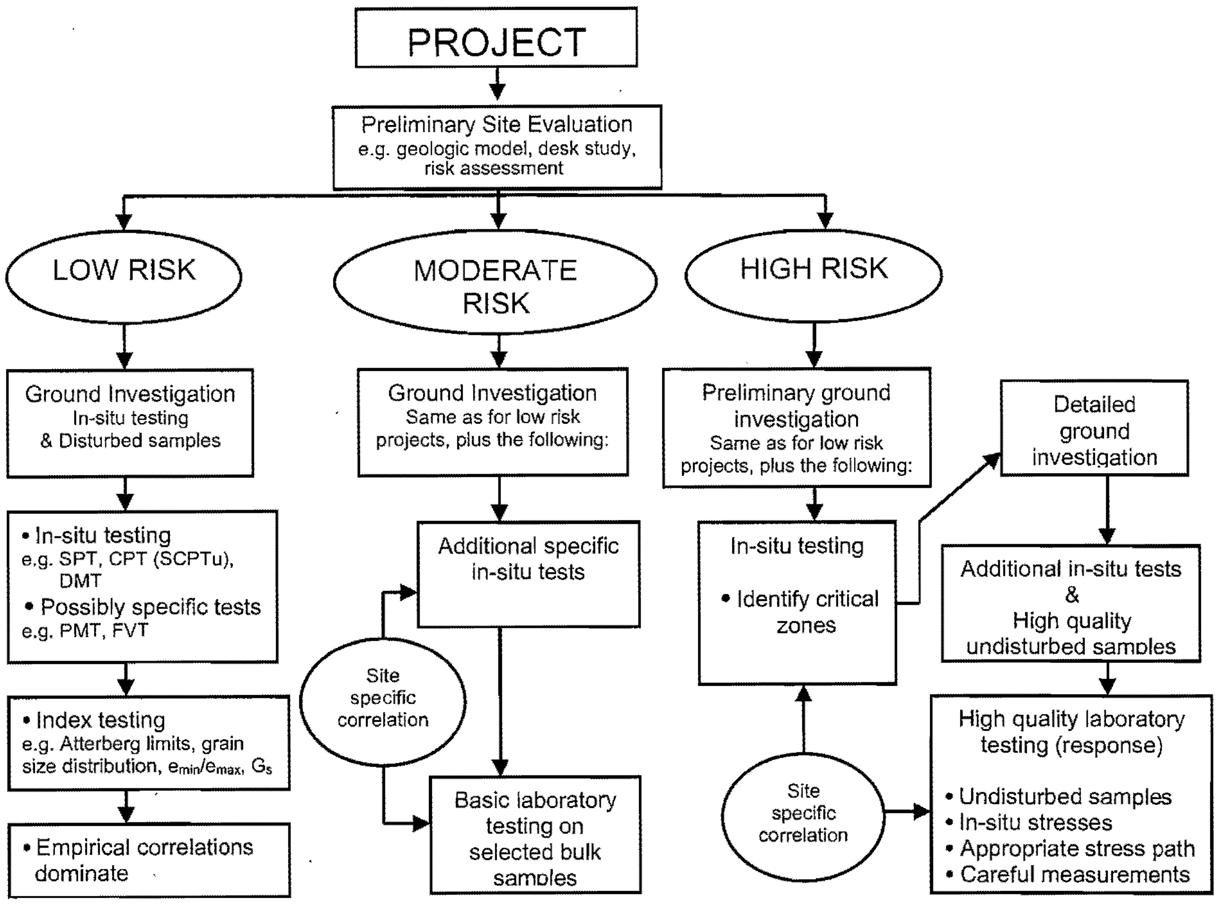


FIGURE 4.1. *Generalized flow chart to illustrate the likely geotechnical site investigation based on risk (after Robertson, 1997)*

4.4.3 Number and Spacing of Boreholes

Determination of the minimum depth of boreholes has a logical basis which is related to the depth at which the increase in soil stress caused by the foundation loads is small and will not cause any significant settlement. The basis for determining the spacing of boreholes is less logical, and spacing is based more on the variability of site conditions, type of project, performance requirements, experience, and judgment. More boreholes and closer spacing is generally recommended for sites which are located in less developed areas where previous experience is sparse or non-existent. The following comments are given for planning purposes. The results of the site investigation may indicate more complex foundation soil conditions which may require additional boreholes.

For buildings smaller than about 1000 m^2 in plan area but larger than about 250 m^2, a minimum of four boreholes where the ground surface is level, and the first two boreholes indicate regular stratification, may be adequate. Five boreholes are generally preferable (at building corners and centre), and especially if the site is not level. For buildings smaller than about 250 m^2, a minimum of three boreholes may be adequate. A single borehole may be sufficient for a concentrated foundation such as an industrial process tower base in a fixed location with the hole made at that location, and where the general stratigraphy is known from nearby boreholes.

The use of a single borehole for even a small project should be discouraged and not considered prudent except for special circumstances as noted above, otherwise three boreholes is the minimum. The results of one borehole can be misleading, for example, drilling into a large boulder and misinterpreting as bedrock. Many experienced geotechnical engineers know from direct experience or have personal knowledge that the consequences of drilling a single borehole can be significant. In practical terms, once a drill rig is mobilized to the site, the cost of an additional one or two boreholes is usually not large.

The preceding comments are intended to provide guidance on the minimum number of boreholes for smaller structures where the performance of the foundations are not particularly critical. Drilling of less than the suggested minimum number of boreholes should have a sound technical basis.

The determination of the number of boreholes and spacing for larger, more complex, and critical projects forms a very important part of the geotechnical design process, and cannot be covered by simple rules which apply across the entire country. Establishing the scope of a geotechnical investigation and subsequent supervision requires the direction of an experienced geotechnical engineer.

4.4.4 Accuracy of Investigation

Subsurface investigations should call for a variety of methods to determine the soil properties critical in design. In particular it is good practice, whenever possible, to use both field and laboratory tests for soil strength and compressibility determinations. The accuracy of the stratigraphy, as 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 In-Situ Testing of Soils

4.5.1 Introduction

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

The measurement of soil properties by in-situ test methods has developed rapidly during the last two decades. Improvements in equipment, instrumentation, techniques, and analytical procedures have been significant.

In-situ test methods can be divided into two groups: logging methods and specific methods.

Commonly, the logging methods are penetration-type tests which are usually fast and economical. When based on empirical correlations, logging methods provide qualitative values of various geotechnical parameters for foundation design. Specific methods are generally more specialized and often slower and more expensive to perform than the logging methods. They are normally carried out to obtain specific soil parameters, such as shear strength or deformation modulus.

The logging and the specific methods are often complementary in their use. The logging methods are best suited for stratigraphic logging with a preliminary and qualitative evaluation of the soil parameters, while the specific methods are best suited for use in critical areas, as defined by the logging methods, where more detailed assessment is required of specific parameters. The investigation may include undisturbed sampling and laboratory testing.

The logging method should be fast, economic, continuous, and most importantly, repeatable. The specific method should be suited to fundamental analyses to provide a required parameter. One of the best examples of a combination of logging and specific test methods is the cone penetrometer and the pressuremeter.
Reviews of in-situ testing techniques and their applicability have been published by several authors, e.g., Mitchell et al. (1978), Campanella and Robertson (1982), and Lunne, et al. (1989). Common in-situ techniques are listed in Table 4.1.

4.5.2 Standard Penetration Test (SPT)

The introduction in the United States in 1902 of driving a 25-mm diameter open-end pipe into the soil during the wash-boring process marked the beginning of dynamic testing and sampling of soils. Between the late 1920s and early 1930s, the test was standardized using a 51-mm O.D. split-barrel sampler, driven into the soil with a 63.5-kg weight having a free fall of 760 mm. The blows required to drive the split-barrel sampler a distance of 300 mm, after an initial penetration of 150 mm, is referred to as the SPT N value. This procedure has been accepted internationally with only slight modifications. The number of blows for each of the three 150-mm penetrations must be recorded. The Standard Penetration Test (SPT) is useful in site exploration and foundation design. Standard Penetration Test 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.

TABLE 4.1 *Summary of Common In-Situ Tests*

Type of test	Best suited to	Not applicable to	Properties that can be determined	Remarks	References*
Standard Penetration Test (SPT)	Sand	Soft to firm clays	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	(See Section 4.5.2)	ASTM D 1586-84 Peck et al. (1974) Tavenas (1971) Kovacs et al. (1981) ESOPT II (1982) ISOPT (1988) Schmertmann (1979) Skempton (1986)
Dynamic Cone Penetration Test (DCPT)	Sand	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	(See Section 4.5.3)	ISSMFE (1977b, 1989) Ireland et al. (1970) ISOPT (1988)
Cone Penetration Test (CPT)	Sand, silt, and clay	Gravels	Continuous evaluation of density and strength of sands. Continuous evaluation of undrained shear strength in clays.	(See Section 4.5.4.) Test is best suited for the design of footings and piles in sand; tests in clay are more reliable when used in conjunction with vane tests	Sanglerat (1972) Schmertmann (1970, 1978) ISOPT (1988) ISSMFE (1977b, 1989) ASTM D3441-79 Robertson and Campanella (1983a, b) Konrad and Law (1987a, b)
Becker Penetration Test (BPI)	Gravelly and cobbly material	Soft soils	Qualitative evaluation of compactness	(See Section 4.5.5)	Anderson (1968) Harder and Seed (1986) Sy and Campanella (1992a, b)
Field Vane Test (FVT)	Clay	Sands and Gravels	Undrained shear strength	See Section 4.5.6) Test should be used with care, particularly in fissured, varved and highly plastic clays.	ASTM D 2573-72 Bjerrum (1972) Schmertmann (1975) Wroth and Hughes (1973) Wroth (1975)

Type of test	Best suited to	Not applicable to	Properties that can be determined	Remarks	References*
Pressure-meter Test (PMT)	Soft rock, dense sand, gravel, and till	Soft sensitive clays loose silts and sands	Bearing capacity and compressibility	(See Section 4.5.7)	Menard (1965) Eisenstein and Morrison (1973) Baguelin et al. (1978) Ladanyi (1972)
Flat Dilatometer Test (DMT)	Sand and clay	Gravel	Empirical correlation for soil type, K_0, overconsolidation ratio, undrained shear strength, and modulus	(See Section 4.5.8)	Marchetti (1980) Campanella and Robertson (1982, 1991) Schmertmann (1986)
Plate Bearing Test and Screw Plate Test	Sand and clay		Deformation modulus. Modulus of subgrade reaction. Bearing capacity.	(See Section 4.5.9) Strictly applicable only if the deposit is uniform; size effects must be considered in other cases.	ASTM D 1194-72
Permeability Test	Sand and gravel		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.	Hvorslev (1949) Sherard et al. (1963) Olson and Daniel (1981) Tavenas et al. (1983a, b)

* See corresponding Sections of this chapter for a more complete list of references.

Details of the split-barrel sampler and procedure for the Standard Penetration Test are described in ISSMFE (1989) and ASTM D1586. The split-barrel sampler commonly used in the United States often differs from such samplers used elsewhere in that the inner liner is not used. As a result, the inner diameter of the sampler is greater than specified, and since the soil friction developed inside the sampler is reduced, the N value may be underestimated by up to 20 %.

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 above a grain size of about 2 mm.

Improper drilling and sampling procedures which can affect the Standard Penetration Test (SPT) N value are listed in Table 4.2.

For the foregoing reasons, it is readily apparent that the repeatability of the Standard Penetration Test is questionable. In addition, relationships developed for SPT N value versus an exact density should be used with caution. The Standard Penetration Test is, however, useful in site exploration and foundation design and provides a qualitative guide to the in-situ properties of the soil and a sample for classification purposes. The evaluation of the test results should be undertaken by an experienced geotechnical engineer. A detailed discussion of the possible errors in SPT results has been presented by Schmertmann (1979) and Skempton (1986).

TABLE 4.2 *Procedures that may affect the SPT N Value*

Inappropriate test procedure	Potential consequence
Inadequate cleaning of the borehole	SPT is not entirely undertaken in original soil; sludge may be trapped in the sampler and compressed as the sampler is driven; increase the blow count; (this may also prevent sample recovery)
Not seating the sampler spoon on undisturbed soil	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 the borehole throughout the entire drilling, sampling, and testing procedure	The water level in the borehole must be at least equal to 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, rising inside the casing.
Overdrive sampling spoon.	Higher N-values usually result from overdriven sampler.
Sampling spoon plugged by gravel.	Higher N-values result when gravel plugs sampler, and resistance of an underlying stratum of loose sand could be highly overestimated.
Plugged casing	High N-values may be recorded for loose sand when sampling below the groundwater table if hydrostatic pressure causes sand to rise and plug casing.
Overwashing ahead of casing.	Low N-values may result for dense sand since sand is loosened by overwashing.
Drilling method.	Drilling techniques such as using a cased hole compared to a mud stabilized hole may result in different N-values for some soils.
Not using the standard hammer drop	Energy delivered per blow is not uniform (European countries have adopted an automatic trip hammer, which currently is not in common use in North America)
Free fall of the drive hammer 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 hammer.
Not using correct weight of drive hammer	Driller frequently supplies drive hammers with weights varying from the standard by as much as 5 kg
Drive hammer does not strike the drive cap concentrically	Impact energy is reduced, increasing the N-values
Not using a guide rod	Incorrect N-values obtained

Inappropriate test procedure	Potential consequence
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 standard	Heavier rods result in incorrect N-values
Extreme length of drill rods	Experience indicates that at depth over about 15 m, N-values are too high, due to energy losses in the drill rods; use of a down-the-hole hammer should be considered
Loose connection between rods, top rod, and drive cap	Insufficient tightening of drill rods results in and drive cap poor energy transmission and increased N-values
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 that seriously disturb the soil will adversely affect the N-values, e.g., drilling with cable-tool equipment. The use of wash boring with a side discharge bit or rotary with a tricone drill bit and mud flush is recommended.
Using drill holes that are too large	Holes greater than 100 mm in diameter are not recommended; use of large diameter-holes may decrease the blow count, especially in sands.
Inadequate supervision	Frequently a sampler will be impeded by gravel or cobbles, causing a sudden increase in blow count; this is often 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

Numerous studies have shown considerable variations in the procedures and equipment used throughout the world for this supposedly standardized test. However, the SPT, with all its problems, is still the most commonly used in-situ test today. As a result considerable research on individual aspects of the standard penetration test equipment and procedures have been carried out in North America and Japan in an effort to better understand the factors affecting the test (Schmertmann, 1979; Kovacs and Salomone, 1982; Yoshimi and Tokimatsu, 1983). Considerable improvements in the understanding of the dynamics of the SPT have occurred in recent years (Schmertmann and Palacios; 1979, Kovacs et al., 1981; Kovacs and Salomone, 1982; Sy and Campanella, 1991a and b). Skempton (1986) and Decourt (1989) present thorough reviews of SPT corrections and correlations with soil properties.

On the basis of the studies referred to above and other investigations, several corrections for adjusting or standardizing the field standard penetration test value, N, are considered in the following paragraphs. While the corrected N values may be required for design purposes, the original field N values should always be given on the borehole logs. These corrections or adjustments to N values can include:

- Correction for the actual energy delivered to the drill rod. Energy levels vary significantly, depending on the equipment and procedures used.

- Correction for the influence of the overburden stress on N values.
- Correction to account for the length of the drill rod.
- Correction to account for absence or presence of a liner inside the split-spoon sampler.
- Correction to account for the influence of the diameter of the borehole.

Energy measurement during recent studies has shown that ER_r, the energy delivered to the rods during an SPT expressed as a ratio of the theoretical free-fall potential energy, can vary from about 30 % to 90 % (Kovacs and Salomone, 1982; Robertson et al. 1983). The energy delivered to the drill rod varies with the hammer release system, hammer type, anvil and operator characteristics. The type of hammer and anvil appear to influence the energy transfer mechanism.

In view of the variation of energy input during the SPT for various situations, there is clearly a need to be able to adjust or normalize the N values to allow comparison on a common basis. Schmertmann and Palacios, (1979), have shown that the SPT blowcount is approximately inversely proportional to the delivered energy. Kovacs et al. (1984), Seed et al. (1984) and Robertson et al. (1983) have suggested that an energy level of 60 % appears to represent a reasonable historical average for most SPT based empirical correlations. Seed et al. (1984) clearly specify that for liquefaction analyses the SPT N values must be corrected to an energy level of 60 %.

N-values measured with a known or estimated rod energy ratio, ER_r, in percent, can be normalized to an energy level of 60 %, that is to N_{60}, by the following conversion:

$$N_{60} = N\left(\frac{ER_r}{60}\right) \tag{4.1}$$

Based on data summarized by Skempton (1986) and Seed et al. (1984), recommended generalized energy ratios, ER_r, in percent, are given in Table 4.3. These values represent broad global correlations and should be used with caution.

TABLE 4.3 *Generalized SPT Energy Ratios (Based on Seed et al., 1984; Skempton, 1986)*

Country	Hammer	Release	Er_r (%)	Er_r/60
North and South America	Donut Safety Automatic	2 turns of rope 2 turns of rope Trip	45 55 55 to 83	0.75 0.92 0.92 to 1.38
Japan	Donut Donut	2 turns of rope Auto-Trigger	65 78	1.08 1.3
China	Donut Automatic	2 turns of rope Trip	50 60	0.83 1.0
U.K.	Safety Automatic	2 turns of rope Trip	50 60	0.83 1.0
Italy	Donut	Trip	65	1.08

TABLE 4.4. *Approximate Corrections to Measured SPT N-Values (after Skempton, 1986)*

Correction Factor	Item	Correction Factor Value
C_r	Rod Length (below anvil): ≥10 m 6-10 m 4-6 m 3-4 m	1.0 0.95 0.85 0.70
C_s	Standard Sampler US Sampler without liners	1.0 1.2
C_d	Borehole diameter: 65 - 115 mm 150 mm 200 mm	1.0 1.05 1.15

The International Reference Test procedure (ISSMFE, 1989) recommends that in situations where comparisons of SPT results are important, calibrations should be made to evaluate the efficiency of the equipment in terms of energy transfer. Table 4.3 provides only a guide to anticipated average energy levels. The recommended method of SPT energy measurement is specified in ASTM D4633-86 and ISOPT (1988). For projects where SPT results are important, such as liquefaction studies, or where major project decisions rely on the SPT, energy measurements should be made.

The SPT N values vary with the confining stress, and consequently, the overburden pressure. An overburden stress correction is required to normalize the field blowcounts to a constant reference vertical effective normal stress as done for liquefaction studies. This correction eliminates the increase in blowcount at constant density due to the increase in confining stress.

A variety of methods of correcting for overburden pressure have been suggested by various investigators and several of these have been summarized by Liao and Whitman (1986). Liao and Whitman (1986) also proposed a correction factor
which is very similar to the other acceptable correction factors and is simple to use. The correction factor used elsewhere in this Manual, however, is that proposed by Peck et al. (1974) and is described in the following paragraphs.

A commonly used overburden reference effective stress level, particularly for liquefaction studies, is 1.0 tsf or 1.0 kg/cm^2, and the corresponding value in SI units, is approximately 96 kPa. If the N-value at depth corresponding to an effective overburden stress of 1.0 tsf (96 kPa) is considered, the correction factor C_N to be applied to the field N values for other effective overburden stresses is given approximately by

$$C_N = 0.77 \log_{10}\left(\frac{1920}{\sigma_v'}\right) \tag{4.2}$$

where C_N = overburden correction factor
σ_v' = effective overburden stress at the level of N-value in kPa

The equation for C_N is not valid for σ_v' less than about 0.25 tsf (24 kPa) since for low overburden pressures the equation for C_N leads to unreasonably large correction factors. To overcome this problem, Peck et al. (1974) have proposed using the chart given as Figure 11.8 (Chapter 11) which is a plot of C_N versus effective overburden stress (pressure). For values of overburden pressure more than 24 kPa, the correction factor C_N on Figure 11.8 corresponds to that obtained from the equation for C_N. To avoid excessively large values of C_N for small effective overburden pressures, the plot on Figure 11.8 has been arbitrarily extended to a C_N value of 2.0 at zero effective overburden pressure. Although the maximum value of C_N of 2.0 has been suggested, it is probably prudent in practice not to use values larger than about 1.5, unless justified by special studies.

The normal practice in liquefaction studies is to normalize the N-values to an energy ratio of 60 %, and also for an effective overburden pressure of 1.0 tsf (96 kPa), (see Seed et al., 1984) This normalized value, known as $(N_1)_{60}$, is given by the following equation:

$$(N_1)_{60} = N\left(\frac{ER_r}{60}\right)(C_N) \tag{4.3}$$

where
- $(N_1)_{60}$ = N value corrected and normalized for energy ratio of 60 % and normalized for effective overburden pressure of 1.0 tsf or 96 kPa (SI units)
- N = field blowcount
- $ER_r/60$ = rod energy ratio normalized to 60 % (Table 4.3)
- C_N = overburden stress correction

Further corrections to N values can also be made, when appropriate for the effects of rod length, sampler type and borehole diameter. Approximate correction factors are given in Table 4.4. Wave equation studies (Schmertmann and Palacios, 1979) show that the theoretical energy ratio decreases with rod length less than about 10 m. The approximate correction factor, Cr, is given in Table 4.4. Note, however, that when applying Seed's simplified liquefaction procedure, the $(N_1)_{60}$ value should be corrected by multiplying with a rod length correction factor of 0.75 for depths less than 3 m as recommended by Seed, et al. (1984).

Studies by Schmertmann (1979) also found that removing the liner from an SPT sampler designed for a liner improved sample recovery but reduced the measured blowcounts by about 20 %. The corresponding correction factor in Table 4.4 is C_S.

Although good modern practice has the SPT undertaken in a borehole with a diameter between 65 mm and 115 mm, many countries allow testing in boreholes up to 200 mm in diameter. The effect of testing within relatively large diameter boreholes can be significant in sands and probably negligible in clays. Approximate correction factors for the borehole diameter, C_d, are given in Table 4.4.

In addition to the foregoing, there are some other factors which may require consideration and possible correction for specialized applications. These factors include grain size, overconsolidation, aging and cementation (Skempton, 1986). Also, special consideration may be required if heavy or long rods (greater than about 20 m) are used. Energy losses and damping may result in N-values that are too high.

While using normalized $(N_1)_{60}$ values together with other corrections as appropriate has merit, many of the standard penetration N-value empirical relationships given in this Manual were developed before it was common practice to correct field N-values. The question then arises as to whether, and in what manner the N-values should be corrected and the following comments are provided for guidance.

A review of the procedures recommended for correcting N-values by authors of foundation engineering text books indicates that there is some difference of opinion. Das (1990) and Fang (1991) both recommend the use of the overburden pressure correction for the Standard Penetration Test. Bowles (1988) perhaps provides one of the more comprehensive evaluations of N-value corrections. He states that since there are several opinions on N corrections, then the following three basic approaches are possible:

1. Do nothing which, with current equipment and conditions, may be nearly correct for some situations.

2. Adjust only for overburden pressure.
3. Use the equation for $(N_1)_{60}$ and when appropriate apply corrections for rod length, C_r, sample liner, C_S, and borehole diameter, C_d. This is probably the best method but requires equipment calibration for ER. This procedure may become mandatory to allow extrapolation of N data across geographic regions where different equipment is used.

In view of the absence of general agreement on the application of N-value corrections, the following guidelines are given for use in this Manual. The N values should be corrected to the $(N_1)_{60}$ values, together with any other corrections as appropriate, when used for liquefaction studies. The N-values should also be corrected as specifically identified in the various chapters of this Manual but such corrections may not include all the possible factors.

In the absence of any specific recommendations in this Manual on corrections to the N-values prior to using empirical relationships, it is difficult to provide specific guidance. Often no corrections are used and this may be reasonably appropriate in Canadian practice for some conditions as suggested by the following comments.

The energy efficiency of much of the Standard Penetration Test equipment currently in use in Canadian practice is very similar to that used when the various N-value empirical relationships were developed, that is (ER_r) was 45 to 60 percent so the energy correction may be small. The rod length correction C_r is applicable for rod lengths less than 10 m. However, most existing empirical correlations with SPT N-values did not incorporate C_r and hence this correction may not be necessary in many cases. In usual Canadian practice, the sampler liner correction, C_S, and the borehole diameter correction, C_d, are both 1.0 so no correction is required. Consequently, for the usual Canadian practice, the most likely correction to field N-values for use in the N-value empirical relationships that may be considered is the overburden correction factor, C_N, which may apply in cases where overburden pressure is a significant factor.

The overburden correction factor, however, is not always used in current practice, and the significance of this omission will depend on the type of problem and empirical relationship for N being considered. Ignoring the correction factor for N-values at shallow depths will be conservative. Ignoring the overburden correction factor at greater depths may be unconservative if the empirical relationship being considered does not extend to the same depth range, or makes no allowance for influence of depth.

4.5.3 Dynamic Cone Penetration Test (DCPT)

The dynamic cone penetration test is a continuous test which utilizes a dropping weight to drive a cone and rod into the ground. The number of blows for each 300-mm penetration (200 mm in European practice) is recorded. A variety of equipment is used in different areas. The Dynamic Probe Working Party of the ISSMFE Technical Committee on Penetration Testing has published suggested international reference test procedures in the Proceedings of the First International Symposium on Penetration Testing/ISOPT-1/ORLANDO/March 1988. This reference contains useful discussions of the test.

Usually in North American practice, the rods consist of the same 44.4 mm diameter rods used for the Standard Penetration Test (SPT), and the drive weight and height of fall is the same as in the SPT. A variety of cones are used. They may be fixed or disposable (to reduce resistance on withdrawal) and usually are pointed. The diameter of the cones used range from 50 mm to 100 mm and may be short or sleeved, depending on the soil strata and the desired information. Some experience has suggested that short cones should be avoided and that a cone with 45° taper from a 30 mm diameter blunt tip to a 60 mm diameter with a minimum 150 mm long sleeve reduces rod friction compared to a short (unsleeved) cone.

In cohesive soils if a dynamic cone is used to delineate the boundary between stiff to firm clay and soft to very soft clay, experience has shown that very large cones, 100 mm or larger, with a sleeve that is 2.5 times the diameter, could provide a better resistance contrast between the strata.

The dynamic penetrometer is subject to all of the disadvantages of the Standard Penetration Test and should not be used for quantitative evaluation of the soil density and other parameters. One major problem with the Dynamic Cone Penetration Test is rod friction which builds up as the probe depth increases. At depths beyond 15 m to 20 m, the effect of rod friction tends to mask the cone tip resistance, making interpretation of test results difficult. Rod friction can be minimized by use of an outer casing which "follows" behind the cone, or by periodic drilling and continuing the Dynamic Cone Penetration Tests from the bottom of the drill hole. In some areas, local experience and calibration with information from sampled drill holes have made the dynamic cone penetration test a useful in-situ technique. The main advantage of the dynamic cone penetration test is that it is fast and economical, and a continuous resistance versus depth profile is obtained that can provide a visual relationship of soil type or density variations.

4.5.4 Cone Penetration Test (CPT)

Many static cone penetrometers were developed and used in Europe before gaining acceptance in North American practice (Table 4.5). The main reasons for the increasing interest in cone penetration tests (CPT) are the simplicity of testing, reproducibility of results, and the greater amenability of test data to rational analysis. A cone point with a 10 cm_2 base area and an apex angle of 60° has been specified in European and American standards (ISSMFE, 1989, and ASTM D3441). A friction sleeve with an area of 150 cm_2 is located immediately above the cone point. Mechanical cone penetrometers (Begemann, 1965) have a telescopic action, which requires a double rod system. With the electrical cone penetrometers, the friction sleeve and cone point advance continuously with a single rod system.

Not withstanding that the mechanical penetrometers offer the advantage of an initial low cost for equipment and simplicity of operation, they have the disadvantage of a slow incremental procedure, ineffectiveness in soft soils, requirement of moving parts, labour-intensive data handling and presentation, and limited accuracy. The electric cone penetrometers have built-in load-cells that record continuously the point-pressure, q_c, and the local side shear, f_s. The load-cells can be made in a variety of capacities from 50 to 150 kN for point resistance and 7.5 to 15 kN for local side shear, depending on the strength of the soils to be penetrated. Typically, an electric cable connects the cone-and-sleeve load-cells with the recording equipment at the ground surface although other data transfer technologies are available.

TABLE 4.5 *Types of Cone Penetration Tests (Adapted from Schmertmann, 1975)*

Type	Point advance		Practiced	Comments
Static or quasistatic	Hydraulic or mechanical jacking	20 mm/s	Worldwide	Usually 10 cm^2 60° cone
Weight- sounding (screw)	Rotation of a weighted helical cone	variable	Sweden Finland Norway	

The electric cone penetrometer offers obvious advantages over the mechanical penetrometer, such as: it is a more rapid procedure, it provides continuous recording, higher accuracy and repeatability, there is the potential for automatic data logging, reduction, and plotting, and additional sensors can also be incorporated in the cone point. Electric cones carry a high initial cost for equipment and require highly skilled operators with knowledge of electronics. They also require adequate back-up in technical facilities for calibration and maintenance.

The most significant advantage that electric cone penetrometers offer is their repeatability and accuracy. An important application of the cone-penetration test is to determine accurately the soil profile. Extensive use is made of the friction ratio, i.e., the ratio between the point-pressure and the side shear, as a means of soil classification (Begemann, 1965, Schmertmann, 1975, Douglas and Olsen, 1981). It has been shown over the past several years that stress normalization of cone point resistance and friction ratio is correct from a fundamental perspective, and its use provides a much better correlation with retrieved samples. It must, however, be kept in mind at all times that the CPT provides an indication of soil type behaviour, which is different from explicit soil type in some instances, but is what the geotechnical engineer ultimately requires for design purposes. Robertson (1990) presents stress normalized soil classification charts.

A significant development in the electric cone-penetration testing has been the addition of a pore-pressure gauge at the base of the cone. Pore-pressure measurement during static cone-penetration testing provides more information on the stratification and adds new dimensions to the interpretation of geotechnical parameters, especially in loose or soft, fine-grained deposits (Konrad and Law, 1987a). The continuous measurement of pore pressures along with the point resistance and side shear makes the electric cone penetrometer the premier tool for stratification logging of soil deposits (Campanella and Robertson, 1982; Tavenas, 1981).

The excess pore pressure measured during penetration is a useful indication of the soil type and provides an excellent means for detecting stratigraphic detail, and appears to be a good indicator of stress history (Konrad and Law, 1987b). In addition, when the steady penetration is stopped momentarily, the dissipation of the excess pore pressure with time can be used as an indicator of the coefficient of consolidation.

Finally, the equilibrium pore-pressure value, i.e., the pore pressure when all excess pore pressure has dissipated, is a measure of the phreatic elevation in the ground.

Cone resistances and pore pressures are governed by a large number of variables, such as soil type, density, stress level, soil fabric, and mineralogy. Many theories exist to promote a better understanding of the process of a penetrating cone, but correlations with soil characteristics remain largely empirical.

Empirical correlations have also been proposed for relating the results of the cone penetration test to the Standard Penetration Test (SPT), as well as to soil parameters, such as shear strength, density index, compressibility, and modulus (Campanella and Robertson, 1981; Robertson and Campanella, 1983 a, b).

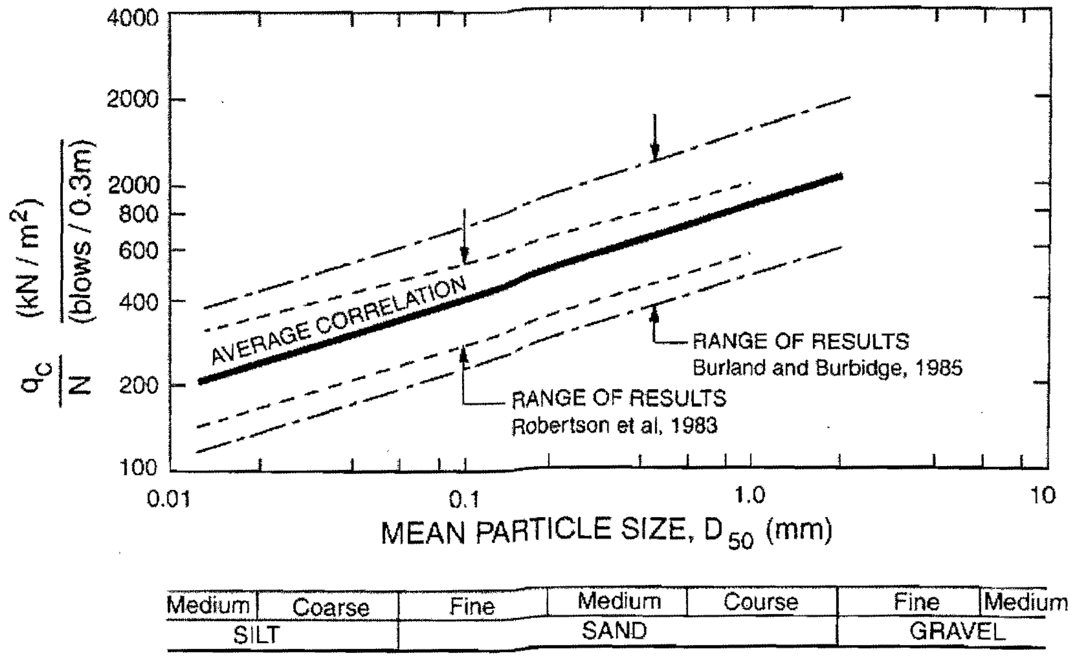


FIGURE 4.2 *Variation of q_c/N - ratio with mean grain size (adapted from Robertson et al., 1983; and Burland and Burbidge, 1985). The dashed lines show the upper and lower limits of observations.*

The use of the CPT to estimate equivalent SPT values is a common application for foundation design. The major advantages of the CPT over the SPT are its continuous profile and the higher accuracy and repeatability it provides; subsequently if a good CPT-SPT correlation exists, very comprehensive equivalent SPT values can be obtained. The relationship between the CPT, represented by the tip resistance, q_c, and the SPT, represented by the blowcount N, has been determined in a number of studies over the past 30 years (Meigh and Nixon, 1961; Thornburn, 1970; Schmertmann, 1970; Burbidge, 1982; Robertson et al., 1983; Burland and Burbidge, 1985). The relationship between CPT and SPT is expressed in terms of the ratio q_c/N (kN/m^2 per blows per 0.3 m); q_c/N data from available literature is summarized on Figure 4.2 against the mean particle size of the soils tested.

4.5.5 Becker Penetration Test (BPT)

The Becker hammer drill was developed in 1958 in Alberta, Canada, initially for seismic oil explorations in difficult gravel sites. The drill is now widely used in North America in mining explorations and in geotechnical investigations for drilling, sampling and penetration testing in sand, gravel and boulder formations. The drill consists of driving a specially designed double-walled casing into the ground with a double-acting diesel pile hammer and using an air injection, reverse-circulation technique to remove the cuttings from the hole. The Becker drill system is more or less 'standardized', being manufactured by only one company, Drill Systems, in Calgary, Alberta. The hammer used in the Becker system is an international Construction Equipment, Inc. (ICE) Model 180 double-acting atomized fuel injection diesel pile hammer; with a manufacturer's rated energy of 11.0 kJ. The casings come in 2.4 m or 3.0 m lengths and are available in three standard sizes: 140 mm O.D. by 83 mm I.D., 170 mm O.D. by 110 mm I.D. and 230 mm O.D. by 150 mm ID. The main advantage of the Becker hammer drill is the ability to sample or penetrate relatively coarse-grained soil deposits at a fast rate. More details of the hammer drill can be found in Anderson (1968).

The Becker casing can be driven open-ended with a hardened drive bit for drilling and sampling, in which case compressed air is forced down the annulus of the casing to flush the cuttings up the centre of the inner pipe to the surface. The continuous cuttings or soil particles are collected at the ground surface via a cyclone which dissipates the energy of the fast-moving air/soil stream. The drilling can be stopped at any depth and the open-ended casing allows access to the bottom of the hole for tube sampling, standard penetration test or other in-situ tests, or for rock coring. Undisturbed sampling or penetration testing conducted through the casing in saturated sand and silt may not be reliable, since stoppage of drilling and air shutoff result in unequal hydrostatic conditions inside and outside the casing, causing disturbance or "quicking" of the soil formation below the casing level. This is manifested in the field by soil filling up the bottom section of the casing when drilling is stopped. On completion of drilling, the casing is withdrawn by a puller system comprising two hydraulic jacks operating in parallel on tapered slips that grip the casing and react against the ground.

The Becker casing can also be driven close-ended, without using compressed air, as a large-scale penetration test to evaluate soil density and pile driveability. In this mode, commonly referred to as the Becker Penetration Test (BPT), the driving resistances or blowcounts are recorded for each 0.3 m of penetration. Because of the larger pipe (or sampler) diameter to particle size ratio, the BPT blowcounts have been considered more reliable than SPT N-values in gravelly soils. As a result, numerous attempts have been carried out in the past to correlate the BPT blowcounts to standard penetration test (SPT) N-values for foundation design and liquefaction assessment. Most of these BPT-SPT correlations, however, have limited or local applications, since they do not take into account two important factors affecting the BPT blowcounts: variable hammer energy output and shaft resistance acting on the Becker casing during driving.

Like all diesel hammers, the Becker hammer gives variable energy output depending on combustion conditions and soil resistances. Harder and Seed (1986) have proposed a practical method using hammer bounce chamber pressure measurements to correct the measured BPT blowcounts to a reference "full combustion rating curve" before correlating with corrected SPT N-values. The method is rig or hammer specific and requires a BPT-SPT correlation be established for each drill rig. A more fundamental method of correcting BPT blowcounts based on transferred energy is proposed by Sy and Campanella (1992a). This energy method, however, requires measuring force and acceleration near the top of the casing during the BPT, similar to dynamic monitoring of pile driving

(ASTM D4945-89).

The Becker Penetration Test also simulates the driving of a displacement pile and can be used for pile driveability evaluations (SDS Drilling Ltd.; Morrison and Watts, 1985; Sy and Campanella, 1992b).

4.5.6 Field Vane Test (FVT)

The field vane test is the most common method of in-situ determination of undrained shear strength of clays. The vane is best suited for soft-to-firm clays; it should not be used in cohesionless soils.

The vane equipment consists of a vane blade, a set of rods, and a torque measuring apparatus. The vane blade should have a height-to-diameter ratio of 2; typical dimensions are 100 by 50 mm. The effect of soil friction on the measured torque should be eliminated or be measurable. The torque-measuring apparatus should permit accurate, reproducible readings, preferably in the form of a torque-angular deformation curve. Specific details of the vane shear test and equipment can be found in ASTM D2573. The vane may be rectangular or tapered.

The vane-test performance and interpretation are subject to some limitations or errors, which should be taken into account when using the test results. The insertion of the vane blade produces a displacement and remolding of the soil. Experience shows that thicker blades tend to produce reduced strengths. For acceptable results, the blade thickness should not exceed 5 % of the vane diameter.

The failure mode around a vane is complex. The test interpretation is based on the simplified assumption of a cylindrical failure surface corresponding to the periphery of the vane blade (Aas, 1965). The undrained shear strength can be calculated from the measured torque, provided that the shear strengths on the horizontal and vertical planes are assumed equal, by the following relation:

$$s_u = \frac{2T}{\pi D^3}\left(H/D + a/2\right) \tag{4.4}$$

where

s_u = undrainedshear strength
T = maximum applied torque
H = vane height
D = vane diameter
a = factor which is a function of the assumed shear distribution along the top and bottom of the failure cylinder
a = 0.66 if uniform shear is assumed (usual assumption)
a = 0.50 if triangular distribution is assumed (i.e., shear strength mobilized is proportional to strain)
a = 0.60 if parabolic distribution is assumed

For the assumption of a = 0.66, which is the usual assumption, and a vane height to vane diameter ratio of 2.0, the above equation becomes:

$$s_u = \frac{T}{3.66D^3} \tag{4.5}$$

The above equations are for a rectangular vane. For a tapered vane refer to the ASTM D2573, and for a vane with a 45 degree taper, H/D = 2.0, a = 0.66, and vane rod diameter d, the undrained shear strength is given by the following relation:

$$s_u = \frac{T}{\pi D^3 + 0.37(2D^3 - d^3)} \tag{4.6}$$

The vane shear test actually measures a weighted average of the shear strength on vertical and horizontal planes. It is possible to determine the horizontal and vertical shear strength for either plane by performing the test in similar soil conditions using vanes of different shapes or height/diameter ratios. It has been found that, in general, the ratio of horizontal/vertical shear strength is less than unity and when this is applicable, the field vane value of s_u, is a conservative estimate of the shear strength along the vertical plane. Becker et al. (1988) provide an interpretation where vane strength is essentially controlled by horizontal stresses on the vertical plane.

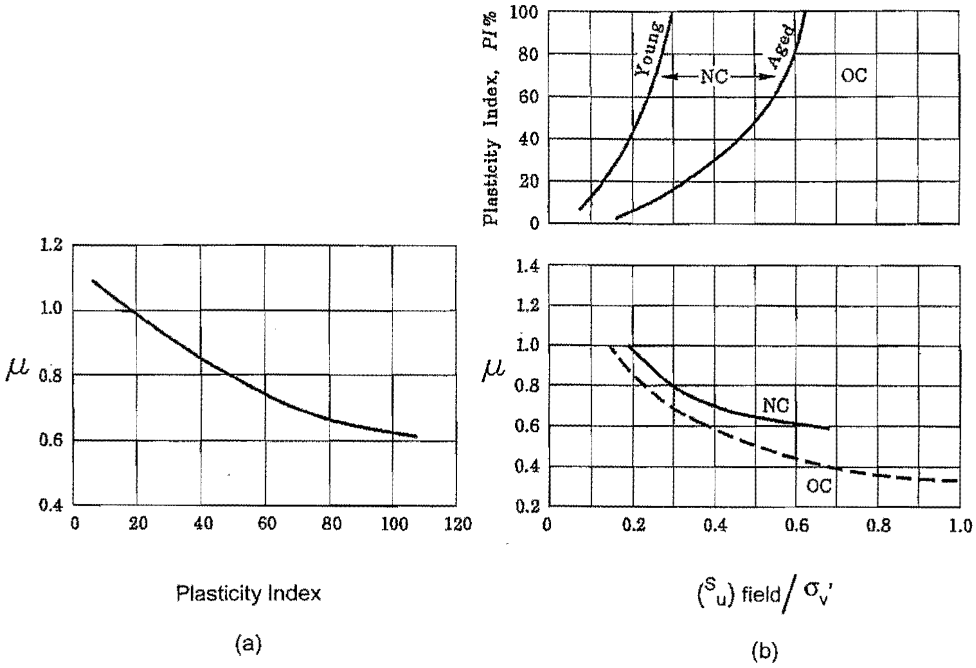


FIGURE 4.3 *Vane correction factor (after (a) Bjerrum, 1972, and (b) Aas, et al., 1986)*

The measured field vane shear strength may require a correction and Bjerrum (1972, 1973) proposed a correction factor, μ, which relates the corrected vane strength, $(s_u)_{corr}$, to the field vane shear strength, $(s_u)_{field}$, as follows:

$$(s_u)_{corr} = \mu\, (s_u)\text{field} \tag{4.7}$$

where μ varies with plasticity index as shown on Figure 4.3(a). Aas et al. (1986) undertook a substantial re-evaluation of the Bjerrum chart to include overconsolidation ratio (OCR) and aging to produce a revised chart, Figure 4.3(b) where μ is given as a function of the ratio (s_u)field,/σ_v', and σ_v' is the effective overburden pressure. Figure 4.3(b) is used by entering the top chart with PI and $(s_u)_{field}$,/ σ_v' to establish whether the clay is within the normally consolidated (NC) range between the limits 'young' and 'aged', or overconsolidated (OC). The bottom chart of Figure 4.3(b) is then used to obtain, μ for the ratio $(s_u)_{field}$,/ σ_v' and the corresponding NC or OC curve. Aas et al. recommend a maximum design value for μ of 1.0 for $(s_u)_{field}$,/ σ_v' less than 0.20 since μ is rather sensitive for low values of $(s_u)_{field}$,/ σ_v'. Refer to Aas et al. (1986) for further details.

Although the correction for plasticity index is used by many practitioners, Leroueil et al. (1990) and Leroueil (2001) provide information that this correction may not be necessary for soft clays. The vane shear value can also be used to estimate the OCR (Mayne and Mitchell, 1988).

The vane shear strength is usually plotted against depth to provide a strength profile. It is a good practice to carry out, in parallel with vane tests, other in-situ tests such as static cone, or piezocone-penetration tests, which yield continuous profiles and to correlate these results with the vane test values. ASTM STP 1014 (1988) contains papers on the testing and interpretation of vane shear tests.

4.5.7 Pressuremeter Tests (PMT)

4.5.7.1 Introduction

Pressuremeters are used to measure the in-situ deformation (compressibility) and strength properties of a wide variety of soil types, weathered rock, and low to moderate strength of intact rock. Two major types of pressuremeters have been developed which are currently in use in Canada; the pre-bored pressuremeter and the self-boring pressuremeter. The Menard-type pressuremeter is a well-known type of pre-bored pressuremeter. Each type of pressuremeter has advantages and limitations largely governed by the type of material to be tested and the method of geotechnical analysis to be carried out. All types of pressuremeter tests are sensitive to the method of probe installation and testing, and highly trained staff who possess a thorough understanding of the equipment and test procedures are required to obtain reliable results. Pressuremeter testing is generally carried out by specialized drilling and/or testing contractors although some engineering consultants and public agencies have their own equipment and trained personnel.

The pressuremeter test was first developed by Louis Menard in 1956. The Menard-type pressuremeter test procedure basically consists of horizontal expansion of a membrane mounted on a relatively long probe placed in a slightly oversized, pre-bored hole. Lateral displacement of the membrane and borehole wall is achieved by injecting either liquid or gas into the probe at selected pressure increments. Displacements are measured either in terms of the volume of liquid injected into the probe or more directly by callipers or displacement transducers for the gas inflated systems. Pressures are measured either with a surface gauge or pressure transducer in the probe. The pressuremeter test allows the determination of the load-deformation characteristics of the tested soil.

The Menard-type tests are sensitive to the degree of soil disturbance caused by drilling the borehole. In order to minimize the soil disturbance, the self-boring pressuremeter was developed independently in France (Baguelin et al., 1972) and in England (Wroth and Hughes, 1973). The principles of the test are similar to the Menard-type test, however, a small rotating cutting head is located in the tip of the probe. The probe is advanced by pushing the probe into the soil.

Displaced soil enters the cutting head where it is removed using water or a bentonite slurry pumped through a double rod assembly. Self-boring pressuremeters can be equipped with a pore-pressure transducer mounted on the exterior of the probe. The membrane is inflated using either liquid or gas in a manner similar to the Menard-type pressuremeter. Similarly, lateral displacements of the borehole wall during the test can be measured either by the volume of injected liquid, or more commonly, with displacement transducers, and the test pressures are measured with a surface gauge or pressure transducers located in the probe.

Relatively small, full displacement pressuremeters have also been combined with static cone penetrometers (Hughes and Robertson, 1985; and Withers et al., 1986) in order to provide a multipurpose tool for site investigations.

4.5.7.2 Menard-Type Pressuremeter Tests

The following discussion will deal with pressuremeters of the Menard design because they are the most common in engineering practice today. This discussion may not be entirely applicable to other pressuremeter designs.

Equipment

The standard Menard pressuremeter consists of a probe connected to a pressure-volume control unit with stiff tubing. Probes are generally available in three diameters consistent with commonly utilized drill hole sizes (A, B and N). The probe consists of a metal cylinder covered with an inflatable membrane and protective sheath comprised of a series of metal strips. The probe is separated into three independent cells; the two end cells are guard cells used to reduce end effects on the middle cell to produce predominantly radial strains in the soil interval tested. Lateral displacements are measured only in the middle cell. All cells are normally filled with water or antifreeze although some systems use gas to inflate the guard cells. Pressure is applied to the fluid in a series of increments by a gas control system acting on a reservoir in the control unit. Volume changes in the reservoir are measured by graduated transparent tubes on the control unit. A more complete description of the Menard system is presented in Baguelin et al. (1978).

Other pressuremeter probes without the two end cells have been introduced. The test results from such probes may need to be corrected before use in common pressuremeter design methods.

Borehole Preparation

It is extremely important to minimize disturbance of the borehole wall during the drilling process. Appropriate drilling procedures are described by Baguelin et al. (1978). Normal drilling and sampling techniques are generally intended to minimize disturbance of the collected samples and may not be suitable for pressuremeter testing. Drilling methods should be selected to prevent collapse of the borehole wall, minimize erosion of the soil, and prevent softening of the soil (Finn et al., 1984). When pressuremeter tests are conducted in a soil type where limited local experience in pressuremeter testing is available, several methods of drilling should be evaluated to determine the optimum method. General guidance regarding the initial selection of drilling methods for various soil types is presented in Table 4.6.

Test Procedure

Typically, Menard-type pressuremeter tests are carried out as stress controlled tests by applying a series of increasing pressure increments. The maximum pressure expected during the test should be divided into a minimum of ten equal pressure increments. Each pressure increment is maintained for a one minute period with volume or radial strain measurements recorded at intervals of 15, 30, and 60 seconds. All pressure increments should be maintained for the same time period. Tests are generally considered to be complete when the volume of the liquid injected during the test is equal to the initial volume of the borehole. In hard soils and rocks it may not be possible to inject this volume and the test is terminated at the maximum pressure for the system. If the sides of the borehole are enlarged excessively either by improper sizing of the drilling equipment or erosion of the borehole wall, the maximum inflation volume of the probe may be reached prior to injection of the required volume.

TABLE 4.6 *Methods of Borehole Preparation for Menard-type Pressuremeter Tests*

Soil Type	Drilling Methods
Firm to Stiff Clay	Pushed tube with internal camfer
Stiff to Hard Clay	Pushed or driven tube with internal camfer Core drilling with mud or possibly foam flush Continuous flight auger
Silt	Pushed or driven tube with internal camfer Core drilling with mud or possibly foam flush (very stiff to hard silts)
Sand	Pushed or driven tube with internal camfer (with mud below the water table) Core drilling with mud flush (dense to very dense sands)
Gravel	Very difficult to avoid disturbance. A driven slotted casing is sometimes used, however disturbance is significant due to lateral displacement of the soil
Glacial Till	Core drilling with mud (very dense finer grained tills with high silt and/or clay content) Driven thick-walled tube with internal camfer (medium dense to dense finer grained tills as above) Driven slotted casing (applicable only to medium dense tills - very high soil disturbance)
Weak or Weathered Rock	Core drilling with mud or possibly foam flush
Sound Rock	Core drilling with water, mud or foam flush

Strain controlled tests are possible for instruments which measure displacements of the borehole wall directly with either callipers or transducers. Computer controlled load application greatly simplifies the test procedure; however, the availability of the equipment is limited. Strain rate selection is important for clays, particularly in the plastic stress range (Anderson, 1979; Windle and Wroth, 1977).

Test Interpretation

The results of a standard Menard-type pressuremeter test corrected for volume and membrane resistance are shown in Figure 4.4 as the Pressuremeter Curve. The pressure must be corrected for the hydrostatic pressure in the measuring circuit above the water table. In the first stage of the test, the volume increases rapidly with small changes in pressure as the probe is inflated against the soil. The volume at the point where the curve becomes approximately linear is termed v_o, which is equal to the difference between the volume of the hole and the initial volume of the probe. The corresponding pressure at this point is called p_o; however, this pressure does not represent the true in-situ pressure in the ground because of stress relief during the formation of the hole. At higher pressures the volume increases slowly with pressure. The creep volume change in this pressure range is small and approximately constant, which indicates pseudo-elastic behaviour of the soil. The slope of the volume - pressure curve in this range is related to the shear modulus of the soil as discussed below. The pressure corresponding to the end of the constant creep volume measurements is called the creep pressure p_f. At higher pressures the volume and the creep volume increase rapidly indicating the development of soil failure around the probe. The pressure - volume curve tends to an asymptotic limit corresponding to the limit pressure p_l.

The theoretical basis for the pressuremeter test is the radial expansion of a cavity in an infinite elastic medium which was developed first by Lamé (1852). Details of the cavity expansion theory are presented in Baguelin et al. (1978) and Mair and Wood (1987). The equation for the radial expansion of a cylindrical cavity in an infinite elastic medium is:

$$G = V\frac{\Delta p}{\Delta V} \tag{4.8}$$

where

G = the shear modulus
V = the volume of the cavity
p = pressure in the cavity

The pressuremeter test produces only shear stresses in the soil; no compressive stresses are involved although the test would appear to be entirely compressive. The modulus value calculated from the pressuremeter test is, therefore, a shear modulus (G_M). While the slope of the pressuremeter curve, $\Delta p/\Delta V$ is constant from v_o to v_f, the volume V is not. Therefore, the shear modulus G is dependent on the volume of the cavity selected, which for the pressuremeter test is, by convention, selected at the midpoint of the pseudo - elastic portion of the pressure - volume curves (Figure 4.4). The corresponding shear modulus is defined as G_M. The shear modulus is calculated using the equation:

$$G_m = (v_c + v_m)(p/v) \tag{4.9}$$

where v_c = the initial volume of the probe prior to inflation
v_m = $(v_o + v_f)/2$ (Figure 4.4)
p/v = $(p_f - p_o)/(v_f - v_o)$

The term p/v is the slope of the pressure - volume line in the pseudo-elastic range

The test results are most often presented in terms of an equivalent Young's modulus (E) assuming an isotropic elastic soil using the equation:

$$E = 2G(1 + v) \tag{4.10}$$

where

v = Poisson's ratio

The standard Menard approach is to assume a Poisson's ratio of 0.33 and the resulting modulus is called the Menard modulus (E_M) where

$$E_M = 2G_M(1 = 0.33) = 2.66G_M \tag{4.11}$$

When the previous equation for (G_M) is substituted, then the Menard modulus (E_M) is given by:

$$E_M = 2.66(v_c + v_m)(p/v) \tag{4.12}$$

Other values of Poisson's ratio may be more appropriate depending on the soil or rock type and the drainage conditions, i.e., fine grained vs. coarse-grained soil and undrained vs. drained loading. General guidance on the selection of appropriate Poisson's ratios is presented in Mair and Wood (1987).

Similar interpretation techniques are used for tests which have been cyclically unloaded and reloaded. A shear modulus can be calculated for either portion of the load cycle. The volume v_m used in the calculation is the average volume over the load cycle. The shear modulus computed from the cyclic portion of pressuremeter tests is generally considered to be representative of the "elastic" stiffness of the soil provided the strains are small (Wroth, 1982). Shear modulus is sensitive to effective stress and strain level and the use of the test results in design should consider these factors.

The Menard limit pressure is defined as that pressure at which the volume is equal to twice the initial volume of the hole, that is $2(v_o + v_c)$. Various methods are available to determine the limit pressure, as described by Baguelin et al, (1978). In cases where the borehole is oversized or the oversized or the soil shear strength is very high, the limit pressure may not be attainable during the test. In these cases the limit pressure may be estimated from the creep pressure (p_f) using the following empirical relationship:

$$0.5 < p_f / p_l < 0.75 \tag{4.13}$$

The ratio of the pressuremeter modulus to the limit pressure tends to be a constant characteristic of any given soil type. Typical values are shown in Table 4.7.

TABLE 4.7 *Typical Menard Pressuremeter Values*

Type Of Soil	Limit Pressure (kPa)	E_M / p_l
Soft clay	50 – 300	10
Firm clay	300 – 800	10
Stiff clay	600 - 2500	15
Loose silty sand	100 – 500	5
Silt	200 - 1500	8
Sand and gravel	1200 – 5000	7
Till	1000 – 5000	8
Old fill	400 – 1000	12
Recent fill	50 - 300	12

LIBRARY

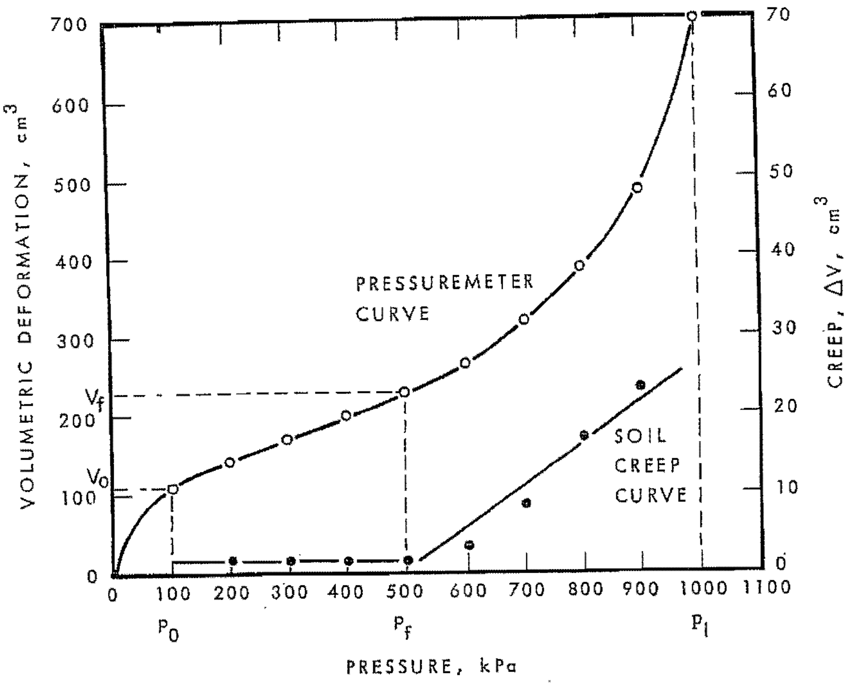


FIGURE 4.4 *Typical pressuremeter and creep curves - Menard type pressuremeter*

Use of Menard-Type Pressuremeter in Foundation Design

In France the Menard-type pressuremeter test results have been empirically correlated to foundation design and performance for many soil types. If these design methodologies are used, the tests must be carried out in accordance with standardized test procedures. Foundation designs must be limited to soil conditions similar to those used to develop the empirical correlations.

The pressuremeter test is a useful tool for investigation of firm to hard clay, silt, sand, glacial till, weathered rock, and low to moderate strength intact rock. The test can also be used for frozen soil and soil containing gas in the pores. The Menard-type pressuremeter is not recommended for general use in clean gravelly soil or soft clay.

4.5.7.3 Self-Boring Pressuremeter Test (SBPMT)

In an effort to minimize soil disturbance in relatively soft soils, the self-boring pressuremeter test (SBPMT) was developed (Baguelin et al., 1972; Wroth and Hughes, 1973).

The sell-boring pressuremeter is similar to a Menard-type pressuremeter as it consists essentially of a thick-wall tube with a flexible membrane attached to the outside. The instrument is pushed into the ground and the soil displaced by a sharp cutting shoe is removed up the centre of the instrument by the action of a rotating cutter or jetting device just inside the shoe of the instrument. The cuttings are flushed to the surface by drill mud, which is pumped down to the cutting head.

Once the instrument is at the desired depth, and following the dissipation of excess pore-water pressure, the membrane surrounding the instrument is expanded against the soil. The expansion at the centre of the instrument is measured by displacement transducers. Pore pressure cells can be incorporated into the membrane to monitor changes in pore-water pressures.

The self-boring pressuremeter can be installed into relatively soft soils and the test results can be interpreted using analytical methods. A summary of the methods of interpretation is presented in Mair and Wood (1987).

The Menard-type pressuremeter test and the self-boring pressuremeter test should be considered as two distinct and separate tests. The Menard-type pressuremeter test is usually interpreted using empirical correlations related to specific design rules. In very stiff soils or rocks, where a pre-bored hole can be made with only elastic unloading of the soil, the Menard type pressuremeter data can be analysed from a more fundamental basis.

4.5.8 Dilatometer Test (DMT)

The flat plate dilatometer test is used in certain regions of North America (Marchetti, 1980) for foundation design. The tool can be classified as a logging tool that is easy to use and provides a range of empirically predicted soil parameters.

Detailed requirements for equipment, test procedure, accuracy of measurements and presentation of test results were recommended by ASTM Subcommittee D.18.02 (Schmertmann, 1986).

A good review of the dilatometer test is provided by Jamiolkowski et al. (1985) and Robertson (1986). An overview of the dilatometer test and interpretation of in-situ test results is given by Lunne et al. (1989). Details of equipment developments are presented by Mitchell (1988).

The flat plate dilatometer is 14 mm thick by 95 mm wide, with a flexible steel membrane 60 mm in diameter on the face of the blade. The pressure for lift-off of the diaphragm, the pressure required to deflect the centre of the diaphragm 1 mm into the soil, and the pressure at which the diaphragm returns to its initial position (closing pressure) are recorded at each depth. Readings are made every 200 mm in depth and the dilatometer, which has a sharpened blade, is advanced at a constant rate of 20 mm/s, with a cone penetrometer rig or similar pushing apparatus. Correlations have been developed between dilatometer readings and soil type, earth pressure at rest, overconsolidation ratio, undrained shear strength, and constrained modulus. However, correlations should be used with caution and verified by local experience before use in any specific case.

4.5.9 The Plate-Load and Screw-Plate Tests

Plate-load tests have been a traditional in-situ method for estimating the bearing capacity of foundations on soil, and for obtaining the soil modulus for the purpose of estimating the settlement of foundations on soil or rock. Plate-load tests involve measuring the applied load and penetration of a plate being pushed into a soil or rock mass. The test is most commonly carried out in shallow pits or trenches but is also undertaken at depth in the bottom of a borehole, pit or adit. In soils, the test is carried out to determine the shear strength and deformation characteristics of the material beneath the loaded plate. The ultimate load is not often attainable in rock where the test is primarily used to determine the deformation characteristics.

The test is usually carried out either as a series of maintained loads of increasing magnitude or at a constant rate of penetration. In the former, the ground is allowed to consolidate under each load before a further increment is applied; this will yield the drained deformation characteristics and also strength characteristics if the test is continued to failure. In the latter, the rate of penetration is generally such that little or no drainage occurs, and the test gives the corresponding undrained deformation and strength characteristics. The degree of drainage is governed by the size of the plate, the rate of testing, and the soil type.

The results of a single plate-load test apply only to the ground which is significantly stressed by the plate and this is typically a depth of about one and a half times the diameter or width of the plate. The depth of ground stressed by a structural foundation will, in general, be much greater than that stressed by the plate-load test and, for this reason, the results of loading tests carried out at a single elevation do not normally give a direct indication of the allowable bearing capacity and settlement characteristics of the full-scale structural foundation. To determine the variation of ground properties with depth, it will generally be necessary to carry out a series of plate tests at different depths. These should be carried out such that each test subjects the ground to the same effective stress level it would receive at working load. Because of the difficulty in undertaking a series of tests at different depths, screw-plate tests which are described later, may be considered.

One of main limitations of the plate-load test lies in the possibility of ground disturbance during the excavation to gain access to the test position. Excavation causes an unavoidable change in the ground stresses, which may result in irreversible changes to the properties which the test is intended to study.

For example, in stiff fissured over- consolidated clay, some swelling and expansion of the clay due to opening of fissures and other discontinuities will inevitably occur during the setting-up process, and can considerably reduce the values of the deformation moduli.

In spite of this effect, the moduli determined from plate-load tests may be more reliable and often many times higher than those obtained from standard laboratory tests. In a project that involves a large deep excavation, the excavation may cause disturbance to the ground beneath, with a consequent effect on the deformation characteristics. In such a case, it will be necessary to allow for this unavoidable disturbance when interpreting the results of loading tests.

Plate-load testing procedures are described in ASTM D-1194-72, (1987) and British Standards Institution Code of Practice, BS 5930 (1981). It is recommended that dial gauges, reading to an accuracy of 0.02 mm, be used for deformation measurements. Interpretation of the test results are given in BS 5930 (1981) and Navfac DM 7.01, (1986).

It may not be possible or practical to perform plate-load tests at depth in the soil. An alternative method developed in Europe is the screw-plate test, which uses a flat-pitch auger device that can be screwed to the desired depth in the soil and loaded in a similar manner to a plate-load test. The horizontally projected area over the single 360° auger flight is taken as the loading-plate area.

A variety of loading procedures for the screw-plate test can be applied depending on the soil type and data required. Constant rate of load or deformation can be applied and load versus deformation plotted to obtain the modulus and strength of the soil. Some success has been reported (Janbu and Senneset, 1973) in obtaining consolidation data from the screw-plate test. It may also be possible to estimate the preconsolidation pressure in a sand deposit from the test (Dahlberg, 1974).

Plate-load tests and screw-plate load tests are only a part of the necessary procedure for soil investigation for foundation design, and should be undertaken in conjunction with other methods. These tests should be carried out under the direction of experts thoroughly conversant with foundation investigations and design.

4.6 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 procedures used to retrieve them.

4.6.1 Boring

Many different methods may be used to advance a borehole in soils. The more common boring methods are summarized in Table 4.8, which has been adapted from Navfac DM 7.01 (1986). The method of advancing a casing and washing the inside with water (washboring) is one of the most commonly used in Canada. It results in a good quality borehole, provided the washing is done properly, i.e., using a limited water pressure and washing to, but never beyond, the bottom of the casing. In loose sands and silts, material may rise up in the casing during washing; bentonite mud should be used instead of water in such cases. Auger boring, including hollow stem auguring, and rotary drilling are also commonly used methods of drilling boreholes in Canada.

4.6.2 Test Pits

Test pits excavated by a backhoe can often provide valuable information on soil characteristics at shallow depth. Care should be exercised in excavating such pits, especially in loose sands, soft clays, or close to the water table. General comments on test pits and test trenches are summarized in Table 4.9, which has been adapted from Navfac DM 7.01 (1986).

4.6.3 Sampling

For the purpose of this Manual, four classes of samples based on degree of disturbance have been defined as listed in Table 4.10. Mechanical properties, which serve as bases for the design of foundations, can be measured only on samples of Class 1. Such samples should usually be retrieved for the design of foundations on clays. Problem soils, as referred to in Chapter 5, may require special sampling procedures as indicated therein.

Common samplers for disturbed and undisturbed soil samples and disturbed rock cores are summarized in Tables 4.11 and 4.12, both have been adapted from Navfac DM 7.01 (1986).

TABLE 4.8 *Types of Borings*

Boring Method	Procedure Utilized	Applicability
Auger Boring	Hand or power operated augering with periodic removal of material. In some cases continuous auger may be used requiring only one withdrawal. Changes indicated by examination of material removed. Casing generally not used.	Ordinarily used for shallow explorations above water table in partly saturated sands and silts, and soft to stiff cohesive soils. Can clean out hole between drive samples. Fast when power-driven. Large diameter bucket auger permits hole examination. Hole collapses in soft and sandy soils below water table.
Hollow-Stem Flight Auger	Power operated. Hollow stem serves as a casing.	Access for sampling (disturbed or undisturbed) or coring through hollow stem. Should not be used with plug in granular soil. Not suitable for undisturbed sampling in sand and silt below groundwater table.
Wash-Type Boring	Chopping, twisting, and jetting action of a light bit as circulating drilling fluid removes cuttings. Changes indicated by rate of progress, action of rods, and examination of cuttings in drill fluid. Casing may be needed to prevent caving.	Used in sands, sand and gravel without boulders, and soft to hard cohesive soils. Usually can be adapted for inaccessible locations, such as on water, in swamps, on slopes, or within buildings. Difficult to obtain undisturbed samples.
Rotary Drilling	Power rotation of drilling bit as circulating fluid removes cuttings from hole. Changes indicated by rate of progress, action of drilling tools, and examination of cuttings in drilling fluid. Casing usually not required except near surface.	Applicable to all soils except those containing large gravel, cobbles, and boulders (where it may be combined with coring). Difficult to determine changes accurately in some soils. Not practical in inaccessible locations for heavy truck-mounted equipment (track-mounted equipment is available). Soil and rock samples usually limited to 150 mm diameter.
Percussion Drilling (Churn drilling)	Power chopping with limited amount of water at bottom of hole. Water becomes a slurry that is periodically removed with bailer or sand pump. Changes indicated by rate of progress, action of drilling tools, and composition of slurry removed. Casing required except in stable rock.	Not preferred for ordinary exploration or where undisturbed samples are required because of difficulty in determining strata changes, disturbance caused below chopping bit, difficulty of access, and usually higher cost. Sometimes used in combination with auger or wash borings for penetration of coarse gravel, boulders, and rock formations. Could be useful to probe cavities and weakness in rock by changes in drill rate.
Rock Core Drilling	Power rotation of a core barrel as circulating water removes ground-up material from hole. Water also acts as coolant for core barrel bit. Generally hole is cased to rock.	Used alone and in combination with other types of boring to drill weathered rocks, bedrock, and boulder formations.

TABLE 4.8 *Types of Borings (continued)*

Boring Method	Procedure Utilized	Applicability
Wire-Line Drilling	Rotary type drilling method, where coring device is an integral part of the drill rod string which also serves as a casing. Core samples obtained by removing inner barrel assembly from the core barrel portion of the drill rod. The inner barrel is released by a retriever lowered by a wire-line through drilling rod.	Efficient for deep hole coring over 30 m on land and offshore coring and sampling.

TABLE 4.9 *Use, Capabilities and Limitations of Test Pits and Trenches*

Exploration Method	General Use	Capabilities	Limitations
Hand-Excavated Test Pits and Shafts	Bulk sampling, in-situ testing, visual inspection.	Provides data in inaccessible areas, less mechanical disturbance of surrounding ground.	Expensive, time-consuming, limited to depths above groundwater level.
Backhoe Excavated Test Pits and Trenches	Bulk sampling, in-situ testing, visual inspection, excavation rates, depth of bedrock and groundwater.	Fast, economical, generally less than 5 m deep, can be up to 10 m deep.	Equipment access, generally limited to depths above groundwater level, limited undisturbed sampling.
Drilled Shafts	Pre-excavation for piles and shafts, landslide investigations, drainage wells.	Fast, more economical than hand excavation. Diameters typically range from 760 mm to 2.0 m.	Equipment access can be difficult. Undisturbed samples and block samples can be obtained with some effort. Slotted casing limits visual inspection.
Dozer Cuts	Bedrock characteristics, depth of bedrock and groundwater level, rippability, increase depth capability of backhoes, level area for other exploration equipment.	Relatively low cost, exposures for geologic mapping.	Exploration limited to depth above groundwater level.
Trenches for Fault Investigations	Evaluation of presence and activity of faulting and sometimes landslide features.	Definitive location of faulting, subsurface observation up to 10 m.	Costly, time-consuming, requires shoring, only useful where dateable materials are present, depth limited to zone above groundwater level.

TABLE 4.10 *Classification of Soil Samples*

Class	Quality	Identification	Properties that can be measured	Footnote no.
1	Undisturbed	A – Block samples B – Stationary piston sampler	A,B,C,D,E,F,G,H,I,J,K A,B,C,D,E,F,G,H,I,J,K	1, 2, and 4 3
2	Slightly disturbed	Open thin-walled tube sampler	A,B,C,D,E,G,H,I	3
3	Substantially disturbed	Open thick-walled tube sampler, such as a 'split spoon'	A,B,C,D,E,G	
4	Disturbed	Random samples collected by auger or in pits	A,C,D,E,G	5

A – Stratigraphy
B – Stratification
C – Organic content
D – Grain size distribution
E – Atterberg limits
F – Density Index
G – Water content
H – Unit Weight
I – Permeability
J – Compressibility
K – Shear strength

Notes

1. Block samples are best when dealing with sensitive, varved, or fissured clays. Wherever possible, block samples should be taken in such soils.
2. Samples of Class 1 are best stored in a vertical position in a room with constant humidity and at a constant temperature. The relative humidity should not be less than 80%.
3. Testing should occur as quickly as possible after sampling. Whenever possible, testing should be performed immediately after extrusion.
4. Because of inevitable stress relief, samples of all classes may be disturbed. The disturbance depends on the consistency of the sampled soil and increases with depth of sampling.
5. Water-content samples should be taken from freshly cut faces of a pit as the pit is advanced. Small diameter spiral augers are suitable for obtaining water-content samples of cohesive soils, if care is taken to remove free water from the sample, as well as all soil scraped from upper layers in the wall of the borehole. Water-content samples should be placed immediately in airtight containers to prevent evaporation.

TABLE 4.11 *Common Samplers for Disturbed Soil Samples and Rock Cores*

Sampler	Dimensions	Best results in soil or rock types	Methods of penetration	Causes of disturbance or low recovery	Remarks
Split Barrel	50 mm OD – 35 mm ID is standard. Penetrometer sizes up to 100 mm OD – 89 mm ID available.	All fine-grained soils in which sampler can be driven. Gravels invalidate drive data.	Hammer driven.	Vibration.	SPT is made using standard penetrometer with 63.56 kg hammer falling 762 mm. Undisturbed samples often taken with liners. Some sample disturbance is likely.

TABLE 4.11 *Common Samplers for Disturbed Soil Samples and Rock Cores (continued)*

Sampler	Dimensions	Best results in soil or rock types	Methods of penetration	Causes of disturbance or low recovery	Remarks
Retractable Plug	25 mm OD tubes 150 mm long. Maximum of six tubes can be filled in single penetration.	For silts, clays, fine and loose sands.	Hammer driven.	Improper soil types for sampler. Vibration.	Light weight, highly portable units can be hand carried to job. Sample disturbance is likely.
Augers:					
Continuous Helical Flight	75 mm to 406 mm diameter. Can penetrate to depths in excess of 15 m.	For most soils above water table. Will not penetrate hard soils or those containing cobbles or boulders.	Rotation.	Hard soils, cobbles, boulders.	Rapid method of determining soil profile. Bag samples can be obtained. Log and sample depths must account for lag between penetration of bit and arrival of sample at surface.
Hollow Stem	Generally 150 mm to 200 mm OD with 75 mm to 100 mm ID hollow stem.	Same as bucket.	Rotation.	Soil too hard to penetrate.	A special type of flight auger with hollow centre through which undisturbed samples or SPT can be taken.
Disc	Up to 1067 mm diameter	Same as flight auger.	Rotation.	Hard soils, cobbles, boulders.	Rapid method of determining soil profile. Bag samples can be obtained.
Bucket	Up to 1220 mm diameter common. Larger available. With extensions, depths greater than 25 m are possible.	For most soils above water table. Can dig harder soil than above types, and can penetrate soils with cobbles and small boulders when equipped with a rock bucket.	Rotation.	Soil too hard to dig.	Several types of buckets available, including those with ripper teeth and chopping buckets. Progress is slow when extensions are used.
Diamond Core Barrels	Standard sizes 38 mm to 75 mm OD, 22 mm to 54 mm core. Barrel lengths 1.5 m to 3.0 m for exploration.	Hard rock. All barrels can be fitted with insert bits for coring soft rock or hard soil.	Rotation.		

TABLE 4.11 *Common Samplers for Disturbed Soil Samples and Rock Cores (continued)*

Sampler	Dimensions	Best results in soil or rock types	Methods of penetration	Causes of disturbance or low recovery	Remarks
Single Tube		Primarily for strong, sound and uniform rock.		Fractured rock. Rock too soft.	Drill fluid must circulate around core – rock must not be subject to erosion. Single tube not often used for exploration.
Double Tube		Non-uniform, fractured friable and soft rock.		Improper rotation or feed rate in fractured or soft rock.	Has inner barrel or swivel which does not rotate with outer tube. For soft, erodible rock. Best with bottom discharge bit.
Triple Tube		Same as Double Tube.		Same as Double Tube.	Differs from Double tube by having an additional inner split tube liner. Intensely fractured rock core best preserved in this barrel.

TABLE 4.12 *Common Samplers for Undisturbed Samples*

Sampler	Dimensions	Best results in soils types	Method of penetration	Causes of disturbance	Remarks
Shelby Tube	75 mm OD – 73 mm ID most common. Available from 50 mm to 127 mm OD. 762 mm sample length is standard.	For cohesive fine-grained or soft soils. Gravelly soils will crimp the tube.	Pressing with fast, smooth stroke. Can be carefully hammered.	Erratic pressure applied during sampling, hammering, gravel particles, crimping tube edge, improper soil types for sampler.	Simplest sampler for undisturbed samples. Boring should be clean before lowering sampler. Little waste area in sampler. Not suitable for hard, dense or gravelly soils.
Stationary Piston	75 mm OD most common. Available from 50 mm to 127 mm OD. 726 mm sample length is standard.	For soft to medium clays and fine silts. Not for sandy soils.	Pressing with continuous, steady stroke.	Erratic pressure during sampling, allowing piston rod to move during press. Improper soil types for sampler.	Piston at end of sampler prevents entry of fluid and contaminating material. Requires heavy drill rig with hydraulic drill head. Generally less disturbed samples than Shelby. Not suitable for hard, dense or gravelly soil.

TABLE 4.12 *Common Samplers for Undisturbed Samples (continued)*

Sampler	Dimensions	Best results in soils types	Method of penetration	Causes of disturbance	Remarks
Hydraulic Piston (Osterberg)	75 mm OD most common – available from 50 mm to 100 mm OD, 914 mm sample length.	For silts-clays and some sandy soils.	Hydraulic or compressed air pressure.	Inadequate clamping of drill rods, erratic pressure.	Needs only standard drill rods. Requires adequate hydraulic or air capacity to activate sampler. Generally less disturbed samples than Shelby. Not suitable for hard, dense or gravelly soil.
Denison	Samplers from 89 mm OD to 197 mm OD. (60.3 mm to 160 mm size samples.) 60 mm sample length is standard.	Can be used for stiff to hard clay, silt and sands with some cementation, soft rock.	Rotation and hydraulic pressure.	Improperly operating sampler. Poor drilling procedures.	Inner tube face projects beyond outer tube which rotates. Amount of projection can be adjusted. Generally takes good samples. Not suitable for loose sands and soft clays.
Pitcher Sampler	Sampler 105 mm OD, uses 75 mm Shelby Tubes. 610 mm sample length.	Same as Denison.	Same as Denison.	Same as Denison.	Differs from Denison in that inner tube projection is spring controlled. Often ineffective in cohesionless soils.
Hand cut block or cylindrical sample.	Sample cut by hand.	Highest quality undisturbed sampling in cohesive soils, residual soil, weathered rock, soft rock.		Change of state of stress by excavation.	Requires accessible excavation. Requires dewatering if sampling below groundwater.

4.6.4 Backfilling

Backfilling of boreholes and test pits should be done carefully. The quality and compaction of the backfill material should be sufficient to prevent hazard to persons or animals, and should prevent water movement or collapse, particularly in drilling for deep excavations or tunnels. In the case of a contaminated site, care is required to minimize possible flow through the boreholes to water supply aquifers.

4.7 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, the more common tests are summarized in Tables 4.13, 4.14, 4.15, 4.16 and 4.17 to provide some guidance on standard (ASTM) or suggested test procedures, the variations that maybe appropriate, and the type and size of samples required. These tables have been adapted from Navfac DM 7.01 (1986). Testing procedures references given in the above tables are summarized for convenience in Table 4.18. The index property tests in Table 4.13 are considered in more detail in Chapter 3. Other comments for general guidance are given in the following paragraphs and these comments are essentially those given in Navfac DM 7.01 (1986).

4.7.1 Sample Selection

Samples to be tested should be representative of each significant stratum, or be an average of the range of materials present, depending on the design and project requirements. A thin stratum can be significant if it has engineering features such as being weak or cemented. If it appears difficult to obtain representative samples because of variations in the stratum, it may be necessary to consider subdivision of the stratum for sampling, testing, and design purposes. In general, tests on samples of mixed or stratified material, such as varved clay, should be avoided. Usually such results are not indicative of material characteristics and better data for analysis can be obtained by testing the different materials separately.

Undisturbed samples for structural properties tests must be treated with care to avoid disturbance. An "undisturbed" sample found to be disturbed before testing normally should not be tested. Fine-grained cohesive samples naturally moist in the ground should not be allowed to dry before testing, as irreversible changes can occur; organic soils are particularly sensitive. Soils with chemical salts in the pore water may change if water is added, diluting the salt concentration, or if water is removed, concentrating or precipitating the salt. Organic soils require long-term low temperature (60°C) drying to avoid severe oxidation (burning) of the organic material.

4.7.2 Index Property Tests

Index properties are used to classify soils, to group soils in major strata, and to correlate the results of structural properties tests on one portion of a stratum with other portions of that stratum or other similar deposits where only index test data are available. Procedures for most index tests are standardized (Table 4.13). Either representative disturbed or undisturbed samples are utilized.

Tests are selected after review of borehole data and visual identification of samples recovered. In general, the test program should be planned so that soil properties and their variation can be defined adequately for the lateral and vertical extent of the project concerned.

4.7.3 Tests for Corrosivity

The likelihood of soil adversely affecting foundation elements or utilities (concrete and metal elements) can be evaluated on a preliminary basis from the results of the tests referenced in Table 4.13. The tests should be run on samples of soil which will be in contact with the foundations and/or utilities in question and typically these will be only near-surface materials. Usually the chemical tests are run only if there is reason to suspect the presence of those ions.

4.7.4 Structural Properties Tests

Tests for structural properties should be planned for particular design problems. Rigid standardization of test programs is inappropriate. Typical tests for determining structural properties are given in Table 4.14. Perform tests only on undisturbed samples or on compacted specimens prepared by standard procedures. In certain cases, completely remolded samples are utilized to estimate the effect of disturbance. Plan tests to determine typical properties of major strata rather than arbitrarily distributing tests in proportion to the number of undisturbed samples obtained. A limited number of high quality tests on carefully selected representative undisturbed samples are preferred.

4.7.5 Dynamic Tests

Dynamic testing of soil and rock involves three ranges: low frequency (generally less than 10 hz) cyclic testing, resonant column high frequency testing, and ultrasonic pulse testing (Table 4.15). The dynamic tests are used to evaluate foundation support characteristics under cyclic or transient loadings such as machinery, traffic, or earthquakes. For earthquake loading, a primary concern is often liquefaction. Young's modulus (E_s), shear modulus (G), and damping characteristics are determined by cyclic triaxial, cyclic simple shear, and resonant column tests as shown on Table 4.15. Table 4.16 shows the range of strain levels for which each test is applicable.

From the resonant frequency of the material in longitudinal and torsional modes, Poisson's ratio can be computed from test data. Foundation response to dynamic loading and the effect of wave energy on its surroundings is studied in the light of these test results. The ultrasonic pulse test also evaluates the two moduli and Poisson's ratio, but the test results are more reliable for rocks than for soils. Dynamic tests can be run on undisturbed or compacted samples and the number of tests will depend on project circumstances.

4.7.6 Compaction Tests

In exploring for borrow materials, the number of index tests or compaction tests may be required in proportion to the volume of borrow involved or the number of samples obtained. The requirements for compacted soil sample tests are given in Table 4.17.

Structural properties tests are assigned after borrow materials have been grouped in major categories by index and compaction properties. Select samples for structural tests to represent the main soil groups and probable compacted condition. The number of compaction tests will depend on project requirements and borrow variability.

4.7.7 Typical Test Properties

Various correlations between index and structural properties are available showing the probably range of test values and relation of parameters. In testing for structural properties, correlations can be used to extend results to similar soils for which index values only are available. Correlations are of varying quality, expressed by standard deviation, which is the range above and below the average trend, within which about two-thirds of all values occur. These relationships are useful in preliminary analyses but must not supplant careful tests of structural properties. The relationships should never be applied in final analyses without verification by tests of the particular material concerned.

TABLE 4.13 *Requirements for Index Properties Tests and Testing Standards*

Test	Reference for standard tests (a)	Variations from standard test Procedures, sample requirements	Size or weight sample for Test
Moisture content of soil	ASTM D2216 (1)	None. (Test requires natural moisture content.)	As large as convenient.
Moisture, ash, and organic matter of peat materials	ASTM P2974 (1)	None.	
Dry unit weight	None.	Determine dry a sample of measured total volume. (Requires undisturbed sample.)	As large as convenient.
Specific gravity: (relative density)			
Material smaller than No. (4.75 mm) sieve size	ASTM P854 (1)	Volumetric flask preferable; vacuum preferable for de-airing.	25 g to 50 g for fine-grained soil; 150 g for coarse- grained soils.

TABLE 4.13 *Requirements for Index Properties Tests and Testing Standards (continued)*

Test	Reference for standard tests (a)	Variations from standard test Procedures, sample requirements	Size or weight sample for Test
Material larger than No. 4 (4.75 mm) sieve size	ASTM C127 (1)	None.	500 g
Atterberg Limits:		Use fraction passing No. 40 (0.425 mm) sieve; material should not be dried before testing.	
Liquid Limit	ASTM D423 (1)	None.	100 g to 500 g
Plastic Limit	ASTM D424 (1)	Ground glass plate preferable for rolling	15 g to 20 g.
Shrinkage Limit	(4)	In some cases a trimmed specimen of undisturbed material may be used rather than a remolded sample.	30 g
Gradation:			
Sieve analysis	ASTM D422 (1)	Selection of sieves to be utilized may vary for samples of different gradation.	500 g for soil with grains to 9.5 mm; to 5,000 g for soil with grains to 75 mm.
Hydrometer analysis	ASTM D422 (1)	Fraction of sample for hydrometer analysis may be that passing No. 200 (0.075 mm) sieve. For fine-grained soil entire sample may be used. All material must be smaller than No. 10 (2.0 mm) sieve.	65 g for fine-grained soil; 115 g for sandy soil.
Corrosivity:			
Sulphate content	(5)	Several alternative procedures in reference.	Soil/water solution prepared, see reference.
Chloride content	(5)	Several alternative procedures in reference.	Soil/water solution prepared, see reference.
pH	ASTM D 1293 (1)	Reference is for pH of water. For mostly solid substances, solution made with distilled water and filtrate tested; standard not available.	

TABLE 4.13 *Requirements for Index Properties Tests and Testing Standards (continued)*

Test	Reference for standard tests (a)	Variations from standard test Procedures, sample requirements	Size or weight sample for Test
Resistivity (laboratory)	None.	Written standard not available. Follow guidelines provided by manufacturers of testing apparatus.	
Resistivity (field)	(6)	In-situ test procedure.	

(a) Number in parenthesis indicates reference number in Table 4.18
(b) Samples for tests may either be disturbed or undisturbed; all samples must be representative and non-segregated; exceptions noted.
(c) Weights of samples for tests on air-dried basis.

TABLE 4.14 *Requirements for Structural Properties*

Test	Reference for suggested tests (a)	Variations from suggested test procedures	Size of weight of sample for test (undisturbed, remolded, or compacted)
Permeability:			
Constant Head (moderately permeable soil)	(2), (4)		Sample size depends on maximum grain size, 40 mm diameter by 350 mm height for silt and fine sand.
Variable Head	(2), (4)	Generally applicable to fine-grained soils.	Similar to constant head sample.
Constant Head (coarse-grained soils)	ASTM D2434 (1),(4)	Limited to soils containing less than 10% passing No. 200 (0.075 mm) sieve size. For clean coarse-grained soil the procedure in reference (4) is preferable.	Sample diameter should be at least ten times the size of the largest soil particle.
Capillary Head	(2)	Capillary head for certain fine-grained soils may have to be determined indirectly.	200 g to 250 g dry weight.
Consolidation:			
Consolidation	(2)	To investigate secondary compression, individual loads may be maintained for more than 24 hours.	Diameter preferably 63 mm or larger. Ratio of diameter to thickness of 3 to 4.
Swell	AASHTO T258 (7)		Diameter preferably 63 mm or larger. Ratio of diameter to thickness of 3 to 4.1
Collapse Potential	(8)		Two specimens for each test, with diameter 63 mm or larger. Diameter to height ratio 3 to 4.

TABLE 4.14 *Requirements for Structural Properties (continued)*

Test	Reference for suggested tests (a)	Variations from suggested test procedures	Size of weight of sample for test (undisturbed, remolded, or compacted)
Shear Strength:			
Direct Shear	ASTM D3080 (1), (2)	Limited to tests on cohesionless soils or to consolidated shear tests on fine-grained soils.	Generally 12 mm thick, 75 mm by 75 mm or 100 mm by 100 mm in plan, or equivalent.
Unconfined Compression	ASTM D2166 (1),(2)	Alternative procedure given in Reference 4.	Similar to triaxial test samples.
Triaxial Compression:			
Unconsolidated-undrained	ASTM D2850 (1)		Ratio of height to diameter should be less than 3 and greater than 2. Common sizes are: 71 mm diameter, 165 mm high. Larger sizes are appropriate for gravelly materials to be used in earth embankments.
Consolidated-undrained	(2),(3),(4)	Consolidated-undrained tests may run with or without pore pressure measurements, according to basis for design.	
Consolidated-drained	(2),(3),(4)		
Vane Shear			Block of undisturbed soil at least three times dimensions of vane.

(a) Number in parenthesis indicates reference number in Table 4.18.

TABLE 4.15 *Requirements for Dynamic Tests*

Test	Reference for suggested tests (a)	Variations from suggested test procedures	Size of weight of sample for test (undisturbed, remolded, or compacted)
Cyclic Loading			
Triaxial	(9)		Same as for triaxial test for structural properties
Simple Shear	(9)		
Torsional Shear	(10)	Can use hollow specimen.	
Resonant Column	ASTM D40151(1) (11)	Can use hollow specimen.	Same as for triaxial test for structural properties; lengths sometimes greater.
Ultrasonic Pulse			
Soil	(12)		Same as for triaxial test for structural properties.
Rock	ASTM D2845 (1)		Prism, length less than five times lateral dimension; lateral dimension at least five times length of compression wave.

(a) Number in parenthesis indicates reference number in Table 4.18
(b) Except for the ultrasonic pulse test on rock and resonant column tests, there are no recognized standard procedures for dynamic testing. References are to descriptions of tests and test requirements by recognized authorities in those areas.

TABLE 4.16 *Capabilities of Dynamic Testing Apparatus*

Shearing Strain Amplitude (%) 10^{-4} 10^{-3} 10^{-2} 10^{-1} 1	Shear Modulus G	Youngs Modulus E	Damping	Cyclic Stress Behaviour	Attenu-ation
Resonant column (solid sample)	X	X	X		
Resonant column (hollow sample)	X	X	X		
Ultrasonic pulse	X	X			X
Cyclic Triaxial		X	X	X	
Cyclic Simple Shear	X		X	X	

X - Indicates the properties that can be determined.

Typical Motion Characteristics		
Properly Designed Machine	Strong Ground Shaking Earthquake	Close in Nuclear Explosion

10^{-4} 10^{-3} 10^{-2} 10^{-1} 1

TABLE 4.17 *Requirements for Compacted Soil Sample Tests*

Test	Reference for standard test procedures (a),(b)	Variations from standard test procedures	Size or weight of sample for test (c)
Moisture-density relations:			
Standard Proctor 2.49 kg hammer, 305 mm drop	ASTM D698 (1)	Preferable not to reuse samples for successive compaction determinations.	Each determination (typically 4 or 5 determinations per test): Method A: 3.0 kg Method B: 6.5 kg Method C: 4.5 kg Method D: 10 kg

TABLE 4.17 *Requirements for Compacted Soil Sample Tests (continued)*

Test	Reference for standard test procedures (a),(b)	Variations from standard test procedures	Size or weight of sample for test (c)
Modified Proctor 4.54 kg hammer, 457 mm drop	ASTM D1557 (1)	Preferable not to reuse samples for successive compaction determinations.	Method A: 3.5 kg Method B: 7.5 kg Method C: 5.5 kg Method D: 11.5 kg
Maximum and Minimum Densities of Cohesionless Soils	ASTM D2049 (1)		Varies from 4.5 kg to 60 kg depending on maximum grain size.
California Bearing Ratio	ASTM D1883 (1)	Compaction energy other than that for Modified Proctor may be utilized.	Each determination requires 7 kg to 11.5 kg depending on gradation.
Resistance R-value	ASTM D2844 (1)		4.5 kg to 7 kg depending on gradation.
Expansion Pressure	AASHTO T190 (7)	Alternatively, testing procedures of Table 4.14 may be utilized.	4.5 kg to 7 kg depending on gradation.
Permeability and compression	(13)	Best suited for coarse-grained soils. Alternatively, testing procedures of Table 4.14 may be utilized.	7 kg of material passing No. 4 (4.75 mm) sieve size.

(a) Number in parenthesis indicates reference number in Table 4.18
(b) For other sources of standard test procedures, see Table 4.6.
(c) Weight of samples for tests given on air-dried basis.

TABLE 4.18 *References Cited in Tables 4.13, 4.14, 4.15 and 4.17*

1. American Society for Testing and Materials "Annual Book of ASTM Standards, Part 19 - Natural Building Stone, Soil and Rock, Peat, Mosses, and Humus; Part 14 - Concrete and Mineral Aggregates; Part 4 - Structural Steel"; :ASTM, Philadelphia, Pennsylvania.

2. Lambe, T.W. (1951). "Soil Testing for Engineers"; John Wiley, New York.

3. Bishop, A.W. and D.J. Henkel (1962). "The Measurement of Soil Properties in the Triaxial Test"; Edward Arnold Ltd., London.

4. Office of the Chief of Engineers (1970). "Laboratory Soils testing"; Department of the Army, Engineering Manual EM 1110-01-1906, Washington, D.C.

TABLE 4.18 *References Cited in Tables 4.13, 4.14, 4.15 and 4.17 (continued)*

5. American Society of Agronomy and the American society for Testing and Materials (1965). "Methods of Soil Analysis, Chemical and Microbiological Properties"; part 2, Black, C.A., ed., American Society of Agronomy, Inc., Madison, WI.

6. National Bureau of Standards. "Underground Corrosion"; Circular C450, United States Government printing Office.

7. American Association of State Highway and Transportation Officials (1978) "Standard Specifications for Transportation Materials and Methods of Sampling and testing"; Part II, AASHTO, Washington, D.C.

8. Jennings, J.E. and K. Knight (1975). "A Guide to Construction on or with Materials Exhibiting Additional Settlement Due to Collapse of Grain Structures"; Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering.

9. Silver, Marshal L. (1976). "Laboratory Triaxial Testing Procedures to Determine the Cyclic Strength of Soils"; prepared under contract to US NRC, (contract No. WRC-E(11-1)-2433), Report No. NUREG-31.

10. Wood, R.D. (1978). "Measurement of Dynamic soil Properties"; ASCE Geotechnical Division Special Conference on Earthquake Engineering and Soil Dynamics.

11. Dernevich, V.P., B.O. Hardin and D.J. Shippy (1978). "Modulus and Sampling of Soils by the Resonant Column Method"; ASTM, STP 654.

12. Stephenson, R.W. (1978). "Ultrasonic Testing for Determining Dynamic Soil Modulus"; ASTM, STP 654.

13. Bureau of Reclamation (1974). "Permeability and Settlement of Soils"; Earth Manual, Designation E-13, United States Government Printing Office.

4.8 Investigation of Rock

4.8.1 General

Determination of the character and condition of the rockmass is required for design of foundations which extend to or into the rock surface or for excavations in rock. The site investigation techniques used in rock should reflect the design data required. Pertinent information to be determined should include:

- Geological characteristics of the site to provide an overview of the site and provide the basis for correlation between borings and outcrop mapping. Review of existing published data is useful.

- Rock, if present on the surface, should be mapped and the outline of the rock surface and rock surface elevations recorded. Outcrops should be mapped using conventional mapping techniques. Geophysical techniques such as seismic refraction techniques may be useful for detecting top of rock surfaces covered with overburden. Geophysical techniques should be confirmed by boreholes when position of rock surface is critical.

- Rock at depth should be investigated using boreholes. The recovered rock samples should be classified and described as noted below. Use of down hole geophysics can add valuable data to a single borehole log. The nature of the seams washed away by drilling may be determined and, in some instances, engineering properties can be correlated by geophysical logs or borehole camera logging.

- Extent and character of alteration and weathering, and an assessment of the sensitivity or resistance to weathering or chemical reaction. (Includes slaking, swelling or acid drainage generation).

- Characteristics and orientations (including folds and fold axes) of discontinuities such as bedding planes, faults, joints, foliations or cleavage planes.

- Strength and compressibility of the rock mass.

- Permeability and groundwater levels.

- In permafrost rich areas of the Canadian north, care must be taken to determine ice content within discontinuities. Ice rich lenses could melt and cause settlements. Special drilling procedures with cooled drilling fluids may be warranted.

4.8.2 Core Drilling of Rock

When information is required at depth in rock, boring may be required. Attention to the overall geological setting may indicate if detached bedrock may be present. The borehole or boreholes should be carried well below the first encountered top of rock to confirm the presence of bedrock.

Boreholes for the investigation of rock can be advanced by many different methods, as discussed in detail by Franklin and Dusseault (1989). These may include:

- rotary core drilling with double or triple core barrels, with or without wireline, with air or water flush;

- rotary tricone drilling with air or water flush; and

- percussion drills, down-the-hole-hammers, etc.

Rock drilling provides rock core of various diameters, typically ranging from NQ to HQ sizes for geotechnical investigations. Cores recovered using triple tube wire line core barrels are the least disturbed and are useful for assessing discontinuity characteristics. Oriented core may be used to determine spatial relationships of the discontinuities. Core recovered using wireline double tube systems provide pieces of sequential core but often the discontinuities are disturbed and the true nature may be difficult to determine. Sheared zones may be badly disturbed. Tricone drilling provides cuttings of the rock material which do not allow any assessment of discontinuity characteristics. Infilling material is often lost. Percussion drill and down-the-hole hammer drills are excellent for production drilling.

Drilling of soft bedrock may require the use of a Pitcher sampler or Christenson spring loaded bit. Soft seams of sheared material may still be lost during drilling. Large diameter (1 m) holes augured or churn drilled to depth, then mapped and sampled from a mobile cage, have been successfully used to identify zones of weakness and to recover samples for direct shear testing.

When a drilling program is designed it is often prudent to seek the advice of an experienced drilling contractor, particularly with respect to drill suitability. Where it is important to recover high quality cored rock or if testing down hole is an integral part of the program, an hourly rate, testing rate, or some combined basis for payment should be sought, as opposed to a rate per length drilled.

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

Care should be taken when drilling through overburden to bedrock to ensure that bedrock has in fact been reached and that a floating large slab of rock in a till or colluvium or residual soil has not been misinterpreted. The borehole should be drilled a minimum of 3 meters into bedrock, in more than one borehole, to confirm whether bedrock or a boulder has been found. For some geological conditions, such as when floating rock slabs are possible, the depth of drilling should be increased.

In-situ testing in the borehole is recommended whenever possible. The rock exposed along a borehole will be disturbed by drilling, but the position and orientation of the discontinuities will not be affected. Testing using downhole geophysical techniques and observation using a borehole camera or probe can provide very useful information about the integrity of the rock mass.

4.8.3 Use of Core Samples

4.8.3.1 Identification and Classification

Information about identification and classification of rocks is presented in Chapter 3 of this manual. Core logging procedures should include collection of this data. Particular attention should be paid to the identification of the rock discontinuities, including their nature and origin, geometry and weathering. Colour photographs of the rock core, presented in the correct stratigraphic sequence and with the core depths indicated, are a useful record and can assist office studies.

4.8.3.2 Laboratory Testing of Core Samples

Laboratory tests (described in Chapter 3) are useful for determining the strength and deformability of the intact rock elements. Such results may not be representative of the actual rock mass, since they are performed on samples free of discontinuities. The relative importance of the rock characteristics versus the rockmass characteristics depends upon the size of the foundation and the effect of the discontinuities. The range of possible discontinuity conditions is considered in the Geological Strength Index approach (GSI) discussed in Chapter 3. In this method, a combination of the surface conditions of the discontinuities and the rockmass structural state provide a factor to modify the intact rock strength to more representative rockmass strength. This evaluation relies upon an assessment of the intact rock strength and the rockmass conditions. Where large structures are to be founded on or in rock, insitu tests such as described in the next section should be conducted.

4.8.4 In-situ Testing

In-situ testing of rockmass deformation characteristics should be carried out for design of large structures supported in and on rocks. A variety of tests, as discussed by Lo and Hefny (2001) are summarized here:

- Plate load test. This is the most common in-situ rock mechanics test method. Standards for testing procedures and interpretation are given by ISRM (1979a, b) and ASTM (D4394-84 and D4395-85). In this simple test, a load is applied to a prepared flat surface of the rock mass through a plate and the deformation is measured. The deformation modulus is then calculated from this data. The main disadvantages of this technique include the expense of preparing the site for the testing, only a small volume of rock is tested, and the common presence of a disturbed zone around the excavation usually leads to conservative results.

- Large flat jack test. In this simple test (ISRM, 1986), large hydraulic flat jacks are inserted into a narrow slot cut into an exposed rock surface. Pressure applied to the flat jacks results in measured normal rock deformation. The rock mass deformation modulus can be determined from this data. The advantages of this test include the fact that a large volume of the rock mass is influenced by the test, and that it is performed in a relatively undisturbed zone of the rock mass. The disadvantages include the need for skilled drilling personnel, the weak theoretical background for the interpretation, seating problems when conducting the test, and the fact that most flat jacks are generally non-recoverable.

- Dilatometer test. Dilatometer tests may be carried using either flexible or stiff equipment. In the flexible type, (ISRM, 1987) a uniformly distributed pressure is applied to the borehole wall by hydraulically expanding a flexible membrane. The resulting hole expansion is determined by measuring the volume of fluid injected or directly by displacement transducers contained within the probe. The deformation modulus is determined from the relationship between the applied pressure and deformation.

In the stiff type, (ISRM, 1996; ASTM, D4971-89) unidirectional pressure is applied to the borehole wall by two opposed curved steel platens, each covering a 90-degree sector. The advantage of the easily performed and inexpensive dilatometer test is the ability to perform the test at different depths and locations. As a result, the variation of deformability with depth and across the site can be determined. The main disadvantage of the test is that only a small volume of rock is influence by the test. Therefore the modulus obtained is comparable to the laboratory modulus, but not to the rock mass modulus.

Lo and Hefny (2001) and ASCE (1996) describe other in-situ tests involving tunnelling, dynamic testing using seismic waves and stress relief testing.

4.9 Investigation of Groundwater

4.9.1 General

Groundwater is a critical factor in foundation design and construction. Many foundation problems are directly or indirectly related to groundwater, hence groundwater conditions, both physical and chemical, should be given careful attention during all stages of a soils investigation.

Factors of importance are:

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

A thorough evaluation of groundwater measurement, instrumentation selection, installation and observations is beyond the scope of this Manual. Other references should be consulted such as Dunnicliff (1988). Considerable care is required to ensure that the appropriate groundwater level measuring instrumentation is selected and installed. A good system will provide the required information for design while a poor system can give misleading results.

4.9.2 Investigation in Boreholes

Field records should be made during drilling of all borehole observations related to groundwater and these records should include observations on colour, rate of flow, partial or total loss of water, and the first appearance of artesian conditions. The water level should be measured during drilling and after the completion of the borehole. All information should be recorded on the boring log, along with the depth of the borehole and the depth of the casing at the time of observation.

The groundwater observations made in open boreholes should be treated with caution. 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 the borehole reaches equilibrium. Soil collapsing in the borehole can also lead to erroneous results.

One of the more common methods for measuring groundwater levels is to install an open observation well for the full depth of the borehole. The observation well usually consists of a pipe with a perforated section at the bottom. The pipe extends to the ground surface and is backfilled for the entire hole with sand, with a seal at the ground surface. The major disadvantage is that different soil strata may be under different hydrostatic pressure, and the groundwater level recorded may be inaccurate and misleading. Furthermore the continuous sand backfill may allow cross-connection of water in different strata and this could result in misleading observations. Most of the disadvantages of the open borehole or observation well can be overcome by installing open standpipe piezometers that are sealed into specific strata and these are discussed below.

4.9.3 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 (pore-pressure meters). 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, and proper installation is critical to the performance of piezometers. In particular, when installed in a borehole, piezometers should be isolated from the borehole by, for instance, sealing with bentonite a small distance above and below the piezometer tip (which should be surrounded by clean sand).

The simplest and generally considered to be the most reliable piezometer is the open standpipe piezometer installed in the borehole at the depth required with sand backfill placed around the porous end within the depth of the stratum being observed. This stratum is isolated by placing bentonite seals above and below the sand backfill. The borehole above the upper bentonite seal should be backfilled with a special sealing grout. For further details refer to Dunnicliff (1988).

If the foundation strata in which the piezometers are to be located are of low permeability and the time lag for open standpipe piezometer measurements is excessive, or if piezometers are required in locations inaccessible for reading the vertical open standpipe piezometers, then different types of piezometers will be required. Other types of piezometers can be grouped into those that have a diaphragm between the transducer and the porewater and those that do not. Instruments in the first group are piezometers with pneumatic, vibrating wire, and electrical resistance strain gage transducers. Instruments in the second group are open standpipe and twin-tube hydraulic piezometers. Refer to other sources such as Dunnicliff (1988) for further details.

4.10 Geotechnical Report

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

Text

Terms of reference of the investigation
Scope of the investigation
Procedures and equipment used in the investigation
Proposed-structure or structures
Geological setting
Topography, vegetation, and other surface features
Soil profile and properties
Groundwater observations
Existing adjacent structures
Foundation studies, including alternatives
Recommended field instrumentation and monitoring
Recommended construction procedures, if appropriate
Recommended field services
Conclusions and recommendations

Limitations of the investigation

Graphic presentations

Map showing the site location, including north arrow
Detailed plan of the site showing contours and elevations, and location of proposed structures, boreholes, and adjacent structures and features of importance
Boring logs, including all the necessary pertinent information on soil, rock, and groundwater
Stratigraphical and geotechnical profiles
Groundwater profiles
Laboratory data
Special graphic presentations

4.11 Selection of Design Parameters

4.11.1 Approach to Design

There are four distinct categories of calculation methods in geotechnical design as follows (Hight and Leroueil 2002):

1. Empirical	Direct use of in-situ or laboratory test results, relying on correlation with performance data and experience
2. Semi-empirical	Indirect use of in-situ or laboratory test results, combining field experience and simple theory
3. Analytical	Theoretical models based on elasticity, plasticity, etc.
4. Numerical	Complex soil models based at least in part on real soil behavior

The complexity of soil behaviour has resulted in a need for empiricism and so a substantial number of current design methods in geotechnical engineering practice fall in categories 1 and 2. This has led to the development of a large number of design methods, each applicable to one specific design case. Charts are frequently available to aid in design. Because design methods were developed using properties determined in a particular manner, it is important to follow design approaches in their entirety as the previous success of the approach may rely on compensating errors. One area in which this is particularly important is pile design. Pile installation alters soil properties. The magnitude of the change in soil properties depends on the installation method and on the initial conditions. This effect of changes in ground conditions as a result of foundation construction must be specifically considered during site characterization and selection of design parameters.

Historically, design has involved separate consideration of strength and deformation. Limit equilibrium has been used to design against failure and linear elasticity or the non-linear theory of consolidation has been used to estimate deformation. In the limit equilibrium approach, the mobilized strength at failure will likely vary along the particular failure surface under consideration and will differ from peak strength. Site variability and soil strength anisotropy become important when selecting the design strength.

Advances in numerical modeling have given engineers the capability to model soil response to all stages of site development. Constitutive models have been developed which account for some or all of the above aspects of material behaviour. These models have been implemented in numerical models in commercially available computer programs. The determination of appropriate input parameters requires judgment and a good understanding of soil behaviour. It is critical that any model used in design should be calibrated by comparison to case histories of similar foundation elements or systems in similar soil conditions.

4.11.2 Estimation of Soil Properties for Design

To characterize the engineering behaviour of the soil or rock at a site, the following parameters are critically important:

- In-situ stresses
- Ground water conditions
- Overconsolidation ratio to allow definition of yield stresses
- Initial stiffness and its variation with stress and strain level
- Potential for strain weakening or swelling, that is, the existence of soil structure or expansive clay minerals
- The presence of any joints or other macrostructure that may dominate the engineering behaviour.

Once the materials have been identified, estimates of characteristic behaviour can be based on one or all of the following:

- Previous experience in materials with similar classification properties and of similar geological origin and history
- Site specific in-situ testing
- Site specific laboratory testing
- Prototype testing, e.g. footing or pile load tests.

Comparison to similar materials

For estimates of soil properties based on the known behaviour of similar material, it is necessary to have a means of identifying how closely the materials at the site resemble others for which data have been published. Examples of materials well documented in the technical literature are London Clay, Weald Clay, Leda Clay, Boston Blue clay, San Francisco Bay Mud, Ottawa sand, Fraser River sand, Toyoura sand, Leighton Buzzard sand, Ticino sand, etc. Classification properties such as Atterberg Limits or soil gradations can be used to assist the engineer to make this determination.

In-situ testing

The results of in-situ testing can also be used as an index of soil behaviour. Traditionally, property characterization has been based on blow counts or SPT N-values measured during split-spoon sampling. More reliable techniques such as the piezometer cone penetration testing (CPT) are now available. It is important to note that the loading conditions during an in-situ test are usually very different from the loading conditions under the proposed engineering works.

The following points are critical:

- If the in-situ test parameters are to be correlated to engineering behaviour, the soil being investigated should resemble very closely the soil used to develop the correlation. This requires similarity of drainage conditions, hydraulic conductivity, mineralogy, stress history and stress state, soil structure, compressibility, shear stiffness and strain-rate dependence.
- The in-situ test must be carried out in exactly the same way as it was during the development of the correlation.
- Failure to observe either of these conditions can lead to errors in interpretation. Consequently, the engineer interpreting the data must have a strong understanding of soil behaviour and must exercise extreme diligence in the selection, specification and observation of the in-situ tests. This is particularly important when attempting to apply correlations developed in one geological regime to soils or rocks in another.

Laboratory testing

All sampling causes some soil disturbance. The effect of sample disturbance on the soil behaviour obtained in laboratory tests will vary depending on the care and attention taken during sampling, storage of samples, and during preparation of test specimens. In general, disturbance leads to a reduction in stiffness and peak strength of soils when tested at stresses representative of in-situ conditions. Disturbance may also make it difficult to delineate the yield stress of the soil. In sands, it has been observed that attempts to obtain undisturbed samples by methods other than in-situ freezing and coring, typically result in samples of loose sand that are denser than the in-situ condition and samples of dense sand that are looser than the in-situ condition.

Prototype testing

Material behaviour can also be characterized by load testing a prototype of a particular foundation element. The soil elements affected by the test will experience a range of stresses and strains and it is important to ensure that the zone of soil influenced by the test is representative of the soil to be loaded by the actual foundation. The strain rates imposed during the tests must also be considered in relation to those in effect under application of working loads and the significance of any differences in loading rates should be assessed.

4.11.3 Confirmation of Material Behaviour by Construction Monitoring

Much can be learned from monitoring of soil behaviour during construction and during the service life of structures. This is the ultimate test of the success of the characterization of the material properties.

4.12 Background Information for Site Investigations

Background information for site investigations can be obtained from various government sources (federal, provincial and territorial). A variety of information is also now directly available on the internet, or government and private sources can be located using an internet search. Examples of valuable resources are as follows:

- Topographic Maps and Surveys
- Geological Surveys
- Aerial Photographs
- Satellite and Unusual Imagery
- Hydrology
- Waterwells
- Flood-Plain Maps
- Hydrographic Charts and Surveys
- Soil Surveys
- Land Use and Planning Surveys
- Climate
- Mine Records
- Seismicity
- Catalogues and Standards

5 Special Site Conditions

5. Special Site Conditions

5.1 Introduction

The following sections give brief descriptions of the types of soil, rock, or conditions that require precautionary measures to achieve satisfactory design and performance. Early recognition of these types of soil, rock, or conditions is essential to allow sufficient time for adequate investigations and the development of designs. An excellent overview of the various soils in Canada is provided by Legget (1965 and 1976).

5.2 Soils

5.2.1 Organic Soils, Peat and Muskeg

Soils containing significant amounts of organic materials, either as colloids or in fibrous form, are generally weak and will deform excessively under load. Such soils include peat and 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 that can result from loading the soil.

Many parts of Canada, especially in northern regions, have muskeg deposits that pose many significant and challenging geotechnical design and construction problems. The interested reader is referred to MacFarlane (1969) and Radforth and Brawner (1977) for detailed information and discussion concerning this special site condition.

5.2.2 Normally Consolidated Clays

Organic clays of soft to medium consistency, which have been consolidated only under the weight of existing conditions, are found in many areas. Typical of these are the clays of the Windsor-Lake St. Gaff 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 settlement 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 on or above these clays, in order 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 previously driven piles 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.

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

5.2.3 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 100, are known.

Typically, such clays have field water 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, for example, the Champlain clays of the St. Lawrence and Ottawa River Valleys. Where such clays have been preconsolidated by partial desiccation, or by the weight of materials subsequently eroded, foundations may be placed on the clays, provided that the foundation load produces shearing stresses under the foundations that are well within the shear strength of the clay, or else excessive settlement and possibly catastrophic failure will result. Disastrous flow slides have developed in the Champlain clays in a number of places, 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 sensitive 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 require sophisticated equipment and techniques, and should be attempted only by competent personnel experienced in this type of work.

5.2.4 Swelling and Shrinking Clays

Swelling and shrinking clays are clays that expand or contract markedly upon changes in water content. Such clays occur widely in the provinces of Alberta, Manitoba and Saskatchewan, and are usually associated with lacustrine deposits. Shallow foundations constructed on such clays may be subject to movements brought about by volume changes, because of changes of the water content in the clays Deep foundations supporting structural floors can be damaged if the enclosing clay is confined. Special design provisions should be made, which take into account the possibility of movements or swelling pressures in the clays (see Chapter 15).

5.2.5 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, as a rule, will be only slightly compacted by most building vibration, because of friction developed between the grains from capillary forces. Usually for sands in a compact 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 (discussed in detail in Chapter 14). 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, earthquakes have brought about the liquefaction of very loose sands, such as occurred in Niigata, Japan. In this event, structures supported above such soils may be completely destroyed. Loose sands will settle significantly under static load only. Such settlements may exceed allowable tolerance. 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 or within them.

5.2.6 Metastable Soils

Metastable soils include several types of soil, abnormally loosely deposited, which may collapse on saturation. Such collapses will cause severe or even catastrophic settlement of structures founded in or above these soils. Loess is the most common metastable soil.

Because metastable 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 the 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 to determine accurate in-place densities.

5.2.7 Glacial Till

Till is unsorted and unstratified glacial drift deposited directly by and underneath glaciers. Its soil grains are usually angular and all size fractions are normally present (Legget, 1962 and 1979; Legget and Karrow, 1983). Basal till (consolidated under the full weight of the glacier) is normally very dense, whereas ablation till (deposited from the glacier during ablation) may not be dense. Till is generally a good foundation material, but problems have arisen with the presence of soft layers and large boulders. Till may be difficult to excavate. Fine-grained till is generally susceptible to frost.

5.2.8 Fill

An engineered fill placed under careful control may be an extremely dense material, more uniform, more rigid, and stronger than almost all natural deposits. When not placed under controlled conditions, it may be a heterogeneous mass of rubbish, debris, and loose soil of many types useless as a foundation material. It may, of course, also be some combination intermediate between these extremes.

Unless the conditions and quality control under which a fill was placed are fully known, the fill must be presumed unsatisfactory for use under foundations. Investigations must establish its limits, depths, and characteristics throughout.

5.3 Rocks

5.3.1 Volcanic Rocks

Parts of the Canadian Cordillera and the Western Interior Plains have extensive deposits of geologically young volcanic rocks. Some tuffs within these volcanic sequences have high porosities, low densities, and low shear and compressive strengths. These materials weather rapidly, in some places, to smectites (swelling clay minerals; montmorillonite).

5.3.2 Soluble Rocks

Rocks such as limestone, gypsum, rock salt, and marble are subject to high rates of solution by groundwater, and may contain solution channels, caverns, and sinkholes, which may cave to the earth's surface. These conditions present special foundation problems (Calembert 1973).

5.3.3 Shales

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

In Western Canada, the Bearpaw Formation and other shales of Cretaceous age have been found to swell considerably when stress release or unloading leads to the absorption of water by the clay minerals, in combination with exposure to air. Bearpaw shales also have a low frictional resistance, which may create slope stability problems for both excavations and construction on or near natural slopes in Bearpaw shales. Special advice should be sought if Bearpaw or comparable shales are encountered along deep river valleys.

In Eastern Canada, volumetric expansion of some shale formations, caused by the weathering of iron sulphide minerals (mainly pyrite), accelerated by oxidizing bacteria, has occurred in a few localities. Conditions leading to mineralogical alteration seem to be related to lowering of 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 sulphide minerals. Where these conditions are encountered, special provisions should be considered to reduce heat loss from the building spaces to the supporting shale. Shales often weather rapidly when exposed to air in excavations. Special measures are warranted to avoid prolonged contact with air.

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

5.4 Problem Conditions

5.4.1 Meander Loops and Cutoffs

Meandering streams from time to time develop chute cutoffs across meander bends, leaving disused, crescent-shaped water-filled channels, called oxbow lakes, which later fill with very soft, organic silts and clays. Frequently, these crescent-shaped features can be detected in aerial photographs or from accurate topographic maps. The soils filling these abandoned waterways can be weak and highly compressible. It is necessary, therefore, to determine their limits and to establish the depths of the soft, compressible soils.

5.4.2 Landslides

The possibility of landslides should always be considered. Whereas landslides in an active state are readily identifiable, old landslides or unstable soils in a potential landslide state are more difficult to detect. They may be signalled by hummocky conditions, by bowed trees, by tilted or warped strata, or by other evidence of displacement. The presence of sensitive clays increases significantly the risk of landslides. The stability of such an area may be so marginal that even minor disturbances such as a small excavation near the toe of a slope, or slight changes in groundwater conditions or drainage, may activate a slide. It is simpler to take precautions to avoid triggering a landslide than to stop one in motion, but it is better still to avoid the landslide or potential landslide area altogether.

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. Ongoing sloughing of a slope is often an indication of incipient failure (Eden and Jarrett, 1971).

When a potential landslide area is identified, care should be taken to investigate it thoroughly and to adopt construction procedures and designs that will improve the stability. Both the steepness and height of slopes are important factors influencing the stability. Steepening a natural slope, or excavating near the toe, or placing fill at the top of slopes, either temporarily or permanently, will adversely affect the stability of the slope and may result in slope failure. Proper design analysis is required whenever such construction works are contemplated. In particular, the design must consider the aspects of a seasonally varying groundwater regime, as well as the effect of freezing and thawing of the ground. Arrangement for drainage may be necessary, at both the top and the toe of the slope. High slopes may require additional drainage placed horizontally in the sides of the slopes.

5.4.3 Kettle Holes

During the deposition of glacial outwash by the retreating continental ice sheets, large blocks of ice commonly became stranded or trapped in the outwash deposits. Upon melting, these blocks left depressions in the outwash mantle, many of which were subsequently filled with peat or with soft organic soils. Such depressions, known as kettle holes, range in diameter from a few meters to several hundred meters. Usually, the depths of kettle holes do not exceed 40 % of their minimum lateral dimensions; the depths are limited to the angles of repose of the surrounding materials. Kettle holes are normally easily identified as shallow surface depressions. In some localities, however, all obvious surface expression has been destroyed by farming or levelling operations. In such places aerial photographs will often reveal a difference in vegetation cover.

5.4.4 Mined Areas

Sites above or adjacent to mined areas may be subject to severe ground movements and differential settlements, resulting from subsidence or caving. For coal mines and other types of mines in horizontal strata, the zone of disturbance generally does not extend laterally from the edge of the mined areas for a distance more than half the depth of the mine below the surface. There is little control of the solution process that occurs in potash or salt mines, and subsidence may extend several hundred meters 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 differential settlement conditions are suspected. While good maps for active or recently closed mines may be available, the accuracy and reliability of maps on plans for long abandoned mines are frequently poor. Furthermore, there are many mined-out areas, especially in the older mining regions, for which no records are now available.

5.4.5 Permafrost

Permafrost is the thermal condition of the earth's crust and surficial deposits, occurring when temperature has been below the freezing point 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 great depths, or in the discontinuous zone where permafrost is thinner and there are areas of unfrozen ground (Brown 1970, Johnston 1981, Andersland and Anderson 1978).

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 and pore ice 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 districts 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. Expected changes in global climate are exacerbating these problems.

The accumulated experience from careful, scientifically planned and conducted investigations makes it technically possible to build practically any structure in the permafrost area (Rowley et al., 1975). Design and construction in permafrost should be carried out only by those who possess special expertise.

5.4.6 Noxious or Explosive Gas

Noxious or explosive gases, of which methane is the most common, are occasionally encountered in clay or silt deposits and in landfill sites containing decaying organic matter. They constitute a hazard to workers constructing caissons or deep excavations. Gases may 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, which have discharged into the construction area. The history of the local area of discharge of gas from borings, even if only for short periods of time, should be especially noted and suitable precautions taken.

A particular problem may exist in tunnels or drainage systems where the oxidation of iron sulphides by bacteria can deplete the free oxygen supply in poorly ventilated areas so much that persons entering may be asphyxiated. Such areas should be thoroughly purged with clean air before anyone enters, and adequate ventilation must be assured while people are present.

5.4.7 Effects of Heat or Cold

Soils should be protected against contact with surfaces that will be extremely hot or cold. Desiccation of clay soils beneath furnaces or alongside ducts carrying hot gases will cause differential settlements Therefore, insulation and ventilation is necessary around high-temperature structures.

To prevent the potential collapse of retaining walls in the winter due to ice lens formation, the walls must be back-filled with non frost-sensitive material for a distance equal to maximum frost penetration. The extent of the backfill may be reduced by means of insulation behind the wall. Proper drainage must also be provided.

5.4.8 Soil Distortions

Soils distort both laterally and vertically under surface loadings. Lateral distortion is generally not significant, but severe lateral distortions may develop in highly plastic soils toward the edge of surface loadings, even where the loads are not sufficient to cause rupture or mud waves. These lateral distortions may affect foundations, or structure-supporting piles, or pipe trenches located in or adjacent to areas subject to high-surface loading such as 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 when displacement-type piles are driven. Cohesive soils are especially subject to such displacement. Previously driven piles or existing foundations may be displaced, or the soil movements may result in excessive pressures on retaining walls, on sheeting for excavations, or on buried pipes. Heaved piles may be redriven and used. If there is significant lateral displacements the piles may be kinked or bowed beyond the safe limit of use. These hazards must be evaluated in the investigation program. Provision should be made in design and construction procedures to ensure that 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 risk of disturbing existing structures or previously driven piles.

5.4.9 Sulphate Soils and Groundwater

Sulphates in the soil and groundwater can cause significant deterioration of Portland cement concrete. Because contact of concrete with sulphates invariably is due to sulphate solution in the groundwater, isolation of the concrete by interception or removal of sulphate-laden waters will prevent deterioration of the concrete. An alternative solution is to use sulphate-resistant cement in the concrete.

The presence of sulphates in the groundwater does not automatically justify the use of sulphate-resistant cement. High-quality watertight concrete is less susceptible to deterioration by sulphates than lower quality concrete. Furthermore, the use of sulphate-resistant cement does not necessarily make the concrete sulphate-proof.

6 Earthquake - Resistant Design

6 Earthquake - Resistant Design

6.1 Introduction

Earthquake shaking is an important source of external load that must be considered in the design of civil engineering structures because of its potential for disastrous consequences. The degree of importance of earthquake loading at any given site is related to a number of factors including:

- the composition the probable intensity and likelihood of occurrence of an earthquake;
- the magnitude of the forces transmitted to the structures as a result of the earthquake ground motions (displacement, velocity and acceleration);
- the amplitude, duration and frequency content of strong ground motion; and
- and behaviour of the subsoils.

Hazards associated with earthquakes include ground shaking, structural hazards, liquefaction, landslides, retaining structure failures, and lifeline hazards. The practice of earthquake engineering involves the identification and mitigation of these hazards. With the advancement of our knowledge regarding earthquake phenomena and the development of better earthquake-resistant design procedures for different structures, it is possible to mitigate the effects of strong earthquakes and to reduce loss of life, injuries and damage. However, it is extremely difficult, and in many cases impossible, to produce an earthquake-proof structure. Depending on the type of structure and its use, the foundation conditions, and the costs involved, a structure can, generally, only be designed to be more resistant (not immune) to an earthquake.

Many important developments in the field of earthquake engineering have occurred in the last four decades. Advanced structural seismic analysis methods, comprehensive experimental procedures for the assessment and evaluation of the behaviour of different types of soil, and considerable data on the performance of different structures and soil profiles during earthquakes are available to help designers in producing earthquake-resistant designs. Geotechnical earthquake engineers have to address a number of issues when designing safe structures in a seismic environment. They have to establish design ground motions, assess the seismic capacity and performance of foundations, consider the interaction effects between structures and the supporting ground, and evaluate the effects of the earthquake excitation on the strength parameters of the soil. Each of these issues represents a category of problems that varies according to the type of structure under consideration.

The purpose of this chapter is to present some of the key concepts and procedures used by geotechnical earthquake engineers to design safer structures in a seismic environment. References that give detailed accounts of the procedures will be provided as needed. However, situations that involve a high risk of seismic hazards, and bridges, tall buildings or dams resting on soft foundation soils, generally require detailed dynamic analysis by engineers very knowledgeable in earthquake engineering. Some of the seismological concepts and terminology will be given first to enable the geotechnical engineer to understand the basis of both earthquake characterization and seismic design concepts.

6.2 Earthquake Size

The size of an earthquake can be described based on its effects (Earthquake Intensity); the amplitude of seismic waves (Earthquake Magnitude); or its total released seismic energy (Earthquake Energy).

6.2.1 Earthquake Intensity

Earthquake intensity is the oldest measure and uses a qualitative description of the earthquake effects based on observed damage and human reactions. Different scales of intensity include the Rossi-Forel scale (RF); the Modified Mercalli Intensity scale (MMI) that represents conditions in California; the Japanese Meteorological Agency scale (JMA) used in Japan; and the Medvedev-Sponheuer-Karnik scale (MSK) used in Central and Eastern Europe.

6.2.2 Earthquake Magnitude

Most scales of earthquake magnitude are based on some measured quantity of ground shaking and are generally empirical. Most of these magnitude scales are less sensitive in representing stronger earthquakes (referred to as saturation.)

Richter Local Magnitude (Richter 1958): Defines a magnitude scale for shallow, local (epicentral distance less than 600 km) earthquakes in southern California.

$$M_L = \log A \tag{6.1}$$

where

A = the maximum trace amplitude (in microns) recorded on a Wood-Anderson seismometer located 100 km from the epicentre of the earthquake.

Surface Wave Magnitude: A worldwide magnitude scale based on the amplitude of Rayleigh waves with a period of about 20 s. It is used to describe the size of shallow (focal depth < 70 km), distant (epicentral distance > 1000 km) or moderate to large earthquakes. It is given by

$$M_s = \log A + 1.66 \log \Delta + 2.0 \tag{6.2}$$

where

A = maximum ground displacement (microns) and $\Delta = \dfrac{\text{epicentral distance}}{\text{earth circumference}} \text{ x } 360^0$.

Body Wave Magnitude: A worldwide magnitude scale based on the amplitude of the first few cycles of p-waves. It is used for deep focus earthquakes and is given by

$$M_b = \log A - \log T + 0.01\,\Delta + 5.9 \tag{6.3}$$

where

A = p-wave amplitude in microns, T = p-wave period (about 1 s), and

$\Delta = \dfrac{\text{epicentral distance}}{\text{earth circumference}} \text{ x } 360^0$.

Moment Magnitude M_w: This is the only magnitude scale that is not subject to saturation because it does not depend on ground shaking-levels. It is based on the seismic moment and is given by

$$M_w = \frac{\log M_0}{1.5} - 10.7 \tag{6.4}$$

in which

M_0 = the seismic moment in dyne-cm $= \mu A_r \overline{D}$,

where

μ = the rupture strength of the material along the fault, A_r = the rupture area and $\overline{D}$ = the average amount of slip.

These quantities can be estimated from geologic records for historical earthquakes or from the long-period components of a seismogram (Bullen and Bolt 1985).

6.2.3 Earthquake Energy

The total seismic energy released during an earthquake is estimated by

$$\log E = 11.8 + 1.5\,M_s \tag{6.5}$$

where

E is expressed in ergs. This relationship is also applicable to moment magnitude.

6.3 Earthquake Statistics and Probability of Occurrence

The rate of occurrence of an earthquake with a magnitude equal to or greater than M for a given area and time may be estimated by (Gutenberg and Richter 1944)

$$\log_{10} N(M) = a - bM \tag{6.6}$$

where

$N(M)$ is the number of earthquakes $\geq$ M (commonly per year) and a and b are constants for a given seismic zone and are established by fitting the available earthquake data. Fitting Equation 6.6 to incomplete data may indicate, incorrectly, higher occurrence rates for larger earthquakes. It is also worth noting that Equation 6.6 does not always hold.

The probability of occurrence of at least one earthquake with a magnitude $\geq$ M in a given time can be calculated by

$$p_e = 1 - e^{-Nt} \tag{6.7}$$

where

N is the rate of occurrence per year and t is the time period in years under consideration.

The seismic loads used in the National Building Code of Canada (NBCC 2005) are based on a 2 per cent probability of exceedance over 50 years (a 2475-year earthquake). This means that over a 50-year period there is a 2 per cent chance that the ground motions given in the NBCC (2005) will be exceeded.

6.4 Earthquake Ground Motions

The ground motions produced by earthquakes at a particular site are influenced by many factors and can be quite complicated. They are a function of the distance from the earthquake's causative fault, and the depth, mechanism and duration of the fault rupture causing the earthquake as well as the characteristics of the soil profile at the site.

In practice, three translational components, the vertical and two perpendicular horizontal directions of ground motion are recorded. The significant characteristics of the ground motion (known as ground motion parameters) for engineering purposes are: the amplitude; frequency content; and duration of the motion.

To evaluate the ground motion parameters, measurements of ground motions in actual earthquakes are required. Instruments used to accomplish these measurements are seismographs that produce seismograms (velocity response) and accelerographs that produce accelerograms (acceleration response).

6.4.1 Amplitude Parameters

The ground motion is commonly described with a time history of the acceleration, velocity or displacement. The amplitude is generally characterized by the peak value of acceleration (measured). Peak values of velocity and displacement can be calculated by integrating the acceleration time history. Alternatively, when using the response spectrum approach, the peak values of velocity and displacement can be computed approximately by

$$a(\omega)=\omega v(\omega)=\omega^2 u(\omega) \tag{6.8}$$

where

u, v and a are the transformed displacement, velocity and acceleration obtained by subjecting the measured acceleration time history to a Fourier transform, and ω is the predominant circular frequency of the earthquake.

6.4.1.1 Peak Acceleration

The peak horizontal acceleration (PHA) is obtained as the maximum resultant due to the vector sum of two orthogonal components. It is unlikely that the maximum acceleration in two orthogonal components occur simultaneously, however, and the PHA is taken in practice as the maximum measured horizontal acceleration. Horizontal accelerations are used to describe ground motions and their dynamic forces induced in stiff structures. The peak vertical acceleration (PVA) is less important for engineering purposes and can be taken to be approximately as two thirds of PHA. Ground motions with high peak accelerations and long duration are usually destructive.

6.4.1.2 Peak Velocity

The peak horizontal velocity (PHV) better characterizes the ground motions at intermediate periods, 0.4 s > T > 0.2 s. For flexible structures, the PHV may provide a more accurate indication of the potential for damage during earthquakes in the intermediate period range.

6.4.1.3 Peak Displacement

Peak displacements are associated with the lower frequency components of the ground motion. They are difficult to determine accurately and, as a result, are less commonly used as a measure of ground motion.

6.4.1.4 Seismic Regions of Canada

Ground motion probability values are given in terms of probability of exceedance, that is the likelihood of a given horizontal acceleration or velocity on firm soil sites, being exceeded during a particular time period. The 2005 National Building Code of Canada (NBCC 2005) presents the seismic hazard for Canada in terms of a probabilistic based uniform hazard spectrum, replacing the probabilistic estimates of peak ground velocity (PGV) and peak ground acceleration (PGA) in the earlier codes. Spectral acceleration at 0.2, 0.5, 1.0 and 2.0 second periods and peak acceleration form the basis of the seismic provisions of NBCC (2005).

Eastern and western Canada are treated slightly differently because of the different properties of the crust in these regions. Figure 6.1 shows the earthquakes and the regionalization used and identifies in a general way the low-seismicity central part of Canada defined as "stable Canada." The different physical properties of the crust in eastern and western Canada and the different nature of the earthquake sources in south-western Canada required the use of four separate strong ground motion relations as detailed by Adams and Halchuk (2004). Seismic hazard to the west of the leftmost dashed line on Figure 1 has been calculated using western strong ground motion relations; eastern relations are used for the remaining regions.

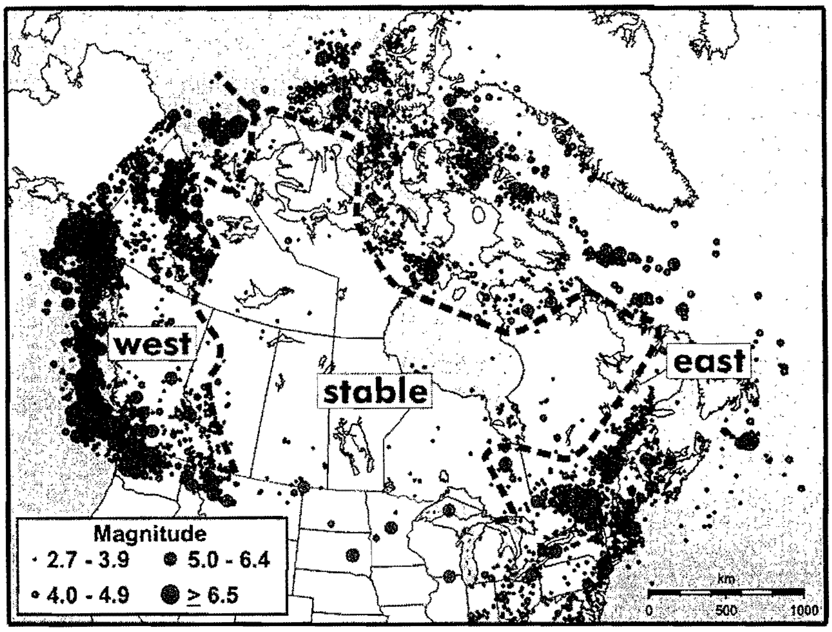


FIGURE 6.1 *Map of Canada (showing the earthquake catalogue used for the 4th Generation model together with dashed lines delimiting the eastern and western seismic regions and the "stable Canada" central region.)*

The spectral acceleration parameters are denoted by Sa(T), where T is the period and are defined later in Tables 6.1B and C (Section 6.5.1.1) for different soil conditions. The PGA values are also presented for use in liquefaction analyses. The NBCC (2005) explicitly considers ground motions from the potential Cascadia subduction earthquake located off the west coast of Vancouver Island. While the amplitudes of such an earthquake are expected to be smaller than from local crustal earthquakes, the duration of shaking will be greater which has implications for liquefaction assessment.

Seismic hazard values were calculated for a grid extending over Canada and used to create national contour maps such as Figure 6.2. Figure 6.3 shows the Uniform Hazard Spectra (UHS) for a few major cities to illustrate the range and period dependence of seismic hazard across Canada.

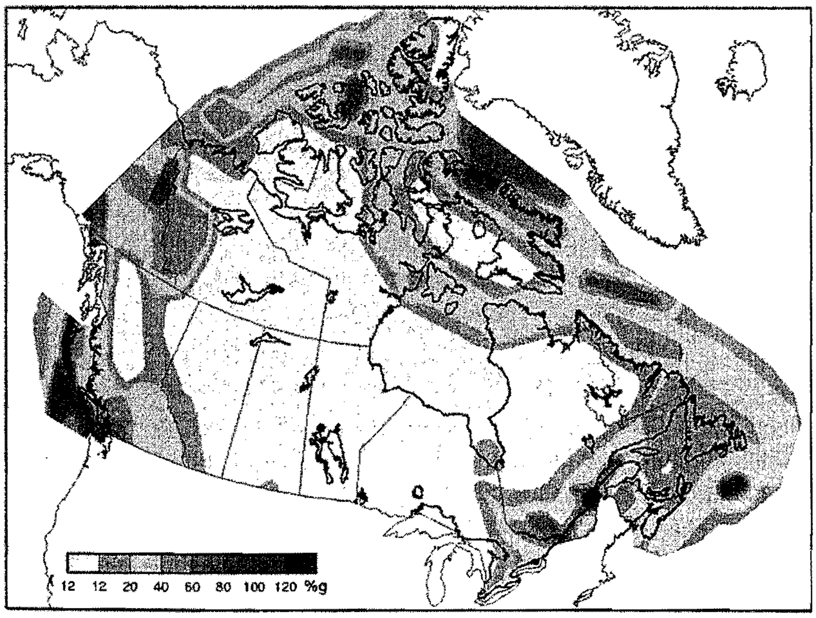


FIGURE 6.2 *Sa(0.2) for Canada (median values of 5 % damped spectral acceleration for Site Class C and a probability of 2 %/50 years)*

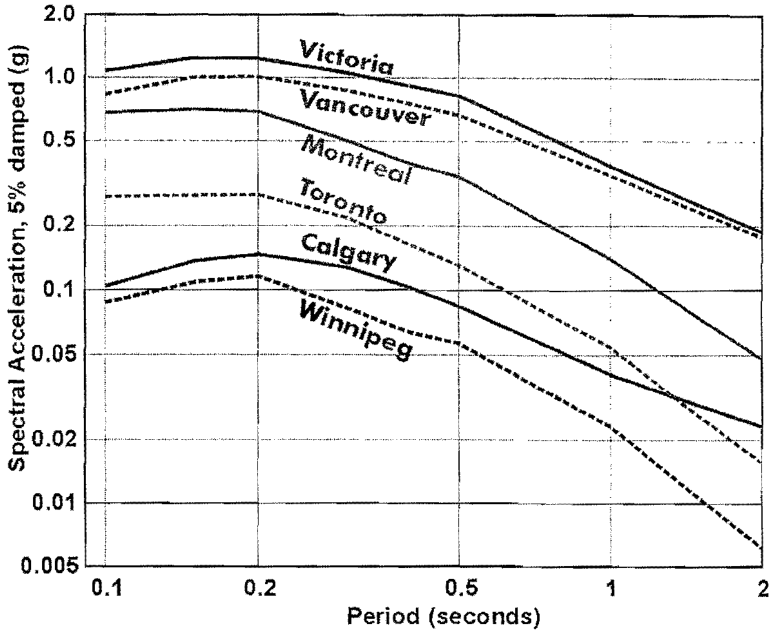


FIGURE 6.3 *Uniform Hazard Spectra for median 2 %/50 year ground motions on Site Class C for key cities*

6.4.2 Frequency Content

The dynamic response of structures is very sensitive to the frequency content of the loading. Earthquake excitations typically contain a broad range of frequencies. The frequency content describes the distribution of ground motion amplitudes with respect to frequency, which can be represented by a Fourier Amplitude Spectrum (i.e., a plot of Fourier amplitude versus frequency) or a Response Spectrum. The predominant circular frequency, ω, in Equation 6.8 is defined as the frequency corresponding to the maximum value of the Fourier amplitude spectrum. The value of ω can be approximated by the number of zero crossings per second in the accelerogram multiplied by 2π.

6.4.3 Duration

The duration of shaking significantly influences the damage caused by an earthquake. The liquefaction of loose saturated sand depends on the number of stress reversals that take place during an earthquake. Earthquakes of longer duration are most likely to cause more damage.

The duration is evaluated from the accelerogram. Different methods are specified to evaluate the duration of strong motion in an accelerogram. The duration can be defined as the time between the first and last exceedances of a threshold acceleration (usually 0.05 g), or as the time interval between the points at which 5 % and 95 % of the total energy has been recorded.

6.5 Building Design

It is almost impossible to design buildings that remain elastic for all levels of earthquakes. Therefore, the intention of building codes and provisions is not to eliminate earthquake damage completely. Rather, structures should be designed to resist:

1. a moderate level earthquake, which has a high probability of occurring at least once during the expected life of the structure, without structural damage, but possibly with some non-structural damage; and

2. a major level earthquake, which has a low probability of occurrence, without collapse, but possibly with some structural damage.

In general, there are two procedures to the earthquake-resistant design of buildings: a static analysis procedure in which the earthquake loading is characterized by equivalent static forces and dynamic analysis procedures. The dynamic analysis procedures include linear analysis using either the Modal Response Spectrum Method where the earthquake loading is characterized by design response spectra or the linear time-history analysis, and nonlinear time-history analysis.

6.5.1 Equivalent Static Force Procedure

The static approach specified in the NBCC (2005) is used for structures satisfying the conditions of sentence 4.1.8.6 of the code (e.g., regular building with a height less than 60 m and natural lateral period less than 2 s). The procedure involves calculating a design seismic base shear proportional to the weight of the structure. The equivalent lateral seismic force procedure of the NBCC (2005) specifies that a structure should be designed to resist a minimum seismic base shear, V, given by

$$V = S(T_a)M_vI_EW/(R_dR_o) \tag{6.9}$$

except that V shall not be less than, $V = S(2.0)M_vI_EW/(R_dR_o)$

where

T_a is fundamental period of the structure, S(T) = the design spectral acceleration, expressed as a ratio to gravitational acceleration, for a period of T, M_v= Factor to account for higher mode effect on base shear, as defined in NBCC Sentence 4.1.8.11.(5), I_E= Earthquake importance factor of the structure, as described in NBCC Sentence 4.1.8.5.(1), W= weight of the structure, R_d= Ductility related force modification factor and R_o= Overstrength related force modification factor.

6.5.1.1 Design Spectral Acceleration, S (T)

The design spectral acceleration values of S(T) is determined as follows (linear interpolation is used for intermediate values of T):

$$\begin{aligned} S(T) &= F_aS_a(0.2) \text{ for } T \leq 0.2 \text{ s} \\ &= F_vS_a(0.5) \text{ or } F_aS_a(0.2) \text{ whichever is smaller for } T = 0.5 \\ &= F_vS_a(1.0) \text{ for } T = 1.0 \text{ s} \\ &= F_vS_a(2.0) \text{ for } T = 2.0 \text{ s} \\ &= F_vS_a(2.0)/2 \text{ for } T \geq 4.0 \text{ s} \end{aligned} \tag{6.10}$$

where

$S_a(T)$ = the 5 % damped spectral response acceleration values for the reference ground conditions (Site Class C in NBCC Table 4.1.8.4.A), and F_a and F_v are acceleration and velocity based site coefficients given in NBCC Tables 4.1.8.4.B and 4.1.8.4.C using linear interpolation for intermediate values of $S_a(0.2)$ and $S_a(1.0)$.

6.5.1.2 Foundation Effect

The soil conditions at a site have been shown to exert a major influence on the type and amount of structural damage that can result from an earthquake. As the motions propagate from bedrock to the surface, the soil layers may amplify the motions in selected frequency ranges around their natural frequencies. In addition, a structure founded on soil, with natural frequencies close to those of the soil layers, may undergo even more intense shaking due to the development of a state of quasi-resonance between the structure and the foundation soil. The natural circular frequency of a soil layer in horizontal direction, ω_u, is given by

$$\omega_u = \frac{\pi V_s}{2h} \tag{6.11}$$

where

V_s is the shear wave velocity of the soil layer and h is its thickness.

Direct calculation of the local site effects is possible using suitable mathematical models such as lumped mass approaches and finite element models with realistic soil properties and assuming vertically propagating shear waves or Rayleigh waves from the bedrock during the earthquake. In these analyses, the source mechanism of the earthquake and the geology of the travel path are incorporated in the bedrock input motion.

The seismic provisions of the NBCC (2005) incorporate site effects by categorizing the wide variety of possible soil conditions into seven types classified according to the average properties of the top 30 m of the soil profile. This classification is based on the average shear wave velocity, V_s, standard penetration resistance, N_{60}, or undrained shear strength, s_u, as shown in Table 6.1A. The factors F_a and F_v given in Tables 6.1B and 6.1C reflect the effect of possible soil amplification (or de-amplification) and soil-structure interaction resonance into the estimation of the seismic design forces for buildings having no unusual characteristics.

While the site coefficients F_a and F_v provide a simple way of introducing surface layer effects for conventional buildings, a fuller evaluation of amplification should be completed for areas of significant seismic activity and/or non-conventional buildings.

Quasi-resonance conditions are of particular importance when the predominant period of the input rock motion (or firm ground) is close to the fundamental period of the less-firm surface layers since this results in amplifications of two to five. In this case, the firm ground or underlying rock accelerations must be modified for potential amplification by less-firm surface layers. The site coefficients are fairly realistic except for this case.

TABLE 6.1A *Site Classification for Seismic Site Response (Table 4.1.8.4.A. in NBCC 2005)*

Site Class	Soil Profile Name	Average Properties in Top 30 m as per Appendix A NBCC 2005		
		Soil Shear Wave Average Velocity, $\overline{V}_s$ (m/s)	Standard Penetration Resistence $\overline{N}_{60}$	Soil Undrained Shear Strength, s_u
A	Hard Rock	$\overline{V}_s > 1500$	Not applicable	Not applicable
B	Rock	$760 < \overline{V}_s \leq 1500$	Not applicable	Not applicable
C	Very Dense Soil and Soft Rock	$360 < \overline{V}_s < 760$	$\overline{N}_{60} > 50$	$s_u > 100$kPa
D	Stiff Soil	$180 < \overline{V}_s < 360$	$15 \leq \overline{N}_{60} \leq 50$	$50 < s_u \leq 100$kPa
E	Soft Soil	$\overline{V}_s < 180$	$\overline{N}_{60} < 15$	$s_u < 50$kPa
E		Any profile with more than 3 m of *soil* with the following characteristics: • Plastic index $I_p > 20$ • Moisture content $w \geq 40\%$, and • Undrained shear strength $s_u < 25$ kPa		
F	(1) Others	Site Specific Evaluation Required		

Note (1) Other soils include:

a) Liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, and other soils susceptible to failure or collapse under seismic loading.
b) Peat and/or highly organic clays greater than 3 m in thickness.
c) Highly plastic clays ($I_p > 75$) with thickness greater than 8 m.
d) Soft to medium stiff clays with thickness greater than 30 m.

TABLE 6.1B *Values of F_a as a Function of Site Class and $S_a(0.2)$ (Table 4.1.8.4.B in NBCC 2005)*

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	(2)	(2)	(2)	(2)	(2)

TABLE 6.1C *Values of F_v as a Function of Site Class and $S_a(1.0)$ (Table 4.1.8.4.C in NBCC 2005)*

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	(2)	(2)	(2)	(2)	(2)

Note (2) F_a and F_v for site Class F are determined by performing site specific geotechnical investigations and dynamic site response analyses.

The seismic design procedures outlined in the NBCC (2005) are based on the assumption that the structures are founded on a rigid base that moves with the ground surface motion. Real foundations possess both flexibility and damping capacity that alter the structural response. The flexibility of the foundation increases the fundamental period of a structure and the damping dissipates energy by wave radiation away from the structure and by hysteretic damping in the foundation, thus increasing the effective damping of the structure. These effects are referred to as soil-structure interaction and are not considered explicitly in the code. For most buildings considered by the code, neglecting soil-structure interaction results in conservative designs. However, neglecting soil-structure interaction effects may not be conservative for tall structures and/or structures with substantial embedded parts and should be considered explicitly in a dynamic analysis.

6.5.1.3 Importance Factor, I_E

Some structures are designed for essential public services. It is desirable that these structures remain operational after an earthquake (defined as post disaster in the code). They include buildings that house electrical generating and distribution systems, fire and police stations, hospitals, radio stations and towers, telephone exchanges, water and sewage pumping stations, fuel supplies and schools. Such structures are assigned an I_E value of 1.5. The importance factor I = 1.3 is associated with special purpose structures where failure could endanger the lives of a large number of people or affect the environment well beyond the confines of the building. These would include facilities for the manufacture or storage of toxic material, nuclear power stations, etc.

6.5.1.4 Force Reduction Factors, R_d and System Overstrength Factors, R_o

The values of R_d and R_o and the corresponding system restrictions shall conform to NBCC Table 4.1.8.9 (Table 6.2). When a particular value of R_d is required, the associated R_o shall be used. For combinations of different types of SFRS acting in the same direction in the same storey, R_dR_o shall be taken as the lowest value of R_dR_o corresponding to these systems.

TABLE 6.2 *SFRS Force Modification Factors (R_d), System Overstrength Factors (R_o) and General Restrictions* [1]

(Table 4.1.8.9. in NBCC)
Forming Part of Sentence 4.1.8.9 (1)

Type of SFRS	R_d	R_o	Restrictions [2]				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where
			<0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	>0.75	$I_EF_vS_a(1.0)$ >0.3
Steel Structures Designed and Detailed According to CSA S16							
▪ Ductile moment resisting frames	5.0	1.5	NL	NL	NL	NL	NL
▪ Moderately ductile moment resisting frames	3.5	1.5	NL	NL	NL	NL	NL
▪ Limited ductility moment resisting frames	2.0	1.3	NL	NL	60	NP	NP
▪ Moderately ductile concentrically braced frames							
• Non-chevron braces	3.0	1.3	NL	NL	40	40	40
• Chevron braces	3.0	1.3	NL	NL	40	40	40
• Tension only braces	3.0	1.3	NL	NL	20	20	20
▪ Limited ductility concentrically braced frames							
• Non-chevron braces	2.0	1.3	NL	NL	60	60	60
• Chevron braces	2.0	1.3	NL	NL	60	60	60
• Tension only braces	2.0	1.3	NL	NL	40	40	40
▪ Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
▪ Ductile frame plate shearwalls	5.0	1.6	NL	NL	NL	NL	NL
▪ Moderately ductile plate shearwalls	2.0	1.5	NL	NL	60	60	60
▪ Conventional construction of moment frames, braced frames or shearwalls	1.5	1.3	NL	NL	15	15	15
▪ Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP
Concrete Structures Designed and Detailed According to CSA A23.3							
▪ Ductile moment resisting frames	4.0	1.7	NL	NL	NL	NL	NL
▪ *Moderately ductile moment resisting frames*	2.5	1.4	NL	NL	60	40	40
▪ Ductile coupled walls	4.0	1.7	NL	NL	NL	NL	NL

	Type of SFRS	R_d	R_o	Restrictions (2)				
				Cases Where $I_E F_a S_a(0.2)$				Cases Where
				<0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	>0.75	$I_E F_v S_a(1.0) > 0.3$
▪	Ductile partially coupled walls	3.5	1.7	NL	NL	NL	NL	NL
▪	Ductile shearwalls	3.5	1.6	NL	NL	NL	NL	NL
▪	Moderately ductile shearwalls	2.0	1.4	NL	NL	NL	60	60
▪	Conventional construction • Moment resisting frames • Shearwalls	 1.5 1.5	 1.3 1.3	 NL NL	 NL NL	 15 40	 NP 30	 NP 30
▪	Other concrete SFRS(s) not listed above	1.0	1.0	15	15	NP	NP	NP
	Timber Structures Designed and Detailed According to CSA 086							
▪	Shearwalls • Nailed shearwalls-wood based panel • Shearwalls - wood based and gypsum panels in combination	 3.0 2.0	 1.7 1.7	 NL NL	 NL NL	 30 20	 20 20	 20 20
▪	Braced or moment resisting frame with ductile connections • Moderately ductile • Limited ductility	 2.0 1.5	 1.5 1.5	 NL NL	 NL NL	 20 15	 20 15	 20 15
▪	Other wood or gypsum based SFRS(s) Not listed above	1.0	1.0	15	15	NP	NP	NP
	Masonry Structures Designed and Detailed According to CSA S304.1							
▪	**Moderately ductile shearwalls**	2.0	1.5	NL	NL	60	40	40
▪	Limited ductility shear walls	1.5	1.5	NL	NL	40	30	30
▪	Conventional Construction • Shearwalls • Moment resisting frames	 1.5 1.5	 1.5 1.5	 NL NL	 60 30	 30 NP	 15 NP	 15 NP
▪	Unreinforced masonry	1.0	1.0	30	15	NP	NP	NP
▪	Other masonry SFRS(s) not listed above	1.0	1.0	15	NP	NP	NP	NP

Notes to Table 6.2:
(1) See NBCC Sentence 4.1.8.10.
(2) Notes on restrictions:
NP in table means not permitted.
Numbers in table are maximum height limits in metres.
NL in table means system is permitted and not limited in height as an SFRS. Height may be limited elsewhere in other Parts.

6.5.1.5 Higher Mode Factor M_v and Base Overturning Reduction Factor J

The seismic lateral force acting on a building during an earthquake is due to the inertial forces acting on the masses of the structures caused by the seismic motion of the base. The motion of the structure is complex, involving the superposition of a number of modes of vibration about several axes. Table 4.1.8.11 of the NBCC (2005) (Table 6.3) assigns M_v and J values to different types of structural systems, which are established based on design and construction experience, and the performance evaluation of structures in major and moderate earthquakes. These values account for the capacity of the structural system to absorb energy by damping and inelastic action through several cycles of load reversal.

TABLE 6.3 *Higher Mode Factor M_v and Base Overturning Reduction Factor J* [1,2]

(Table 4.1.8.11. in NBCC)
Forming Part of Sentence 4.1.8.11.(5)

$S_a(0.2)/S_a(2.0)$	Type of Lateral Resisting Systems	M_v For $T_a \le 1.0$	M_v For $T_a \ge 2.0$	J For $T_a \le 0.5$	J For $T_a \ge 2.0$
< 8.0	Moment resisting frames or "coupled walls" [3]	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
	Walls, wall-frame systems, other systems [4]	1.0	1.2	1.0	0.7
≥ 8.0	Moment resisting frames or "coupled walls" [3]	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall-frame systems, other systems [4]	1.0	2.5	1.0	0.4

Notes:

(1) For values of M_v between periods of 1.0 and 2.0 s, the product $S(T_a) \cdot M_v$ shall be obtained by linear interpolation.
(2) Values of J between periods of 0.5 and 2.0 s shall be obtained by linear interpolation.
(3) Coupled wall is a wall system with coupling beams where at least 66 % of the base overturning moment resisted by the wall system is carried by the axial tension and compression forces resulting from shear in the coupling beams.
(4) For hybrid systems, use values corresponding to walls or carry out a dynamic analysis.

6.5.1.6 Distribution of Base Shear

The base shear is the sum of the inertial forces acting on the masses of the structures caused by the seismic motion of the base. The motion of the structure is complex, involving the superposition of a number of modes of vibration about several axes.

For structures with fundamental periods less than 0.7 s, the addition of the spectral-modal responses results in a lateral inertial force distribution that is approximately triangular in shape, with the apex at the base. For buildings having longer periods, higher forces are induced at the upper portion of the structure due to increasing contributions to top storey amplitudes by all the contributing modes. The redistribution of forces is accounted for by applying part of the base shear as a concentrated force, F_t, to the top of the structure.

The total lateral seismic force, V, shall be distributed such that a portion, F_t, shall be assumed to be concentrated at the top of the building, where F_t is equal to 0.07 $T_a V$ but need not exceed 0.25 V and may be considered as zero where T_a does not exceed 0.7 s; the remainder, $V - F_t$ shall be distributed along the height of the building, including the top level, in accordance with the formula.

$$F_x = (V - F_t)W_x h_x / (\sum_{i=1}^{n} W_i h_i) \tag{6.12}$$

where

F_x is the inertial force induced at any level x which is proportional to the weight W_x at that level.

6.5.1.7 Overturning Moments

The lateral forces that are induced in a structure by earthquakes give rise to moments that are the product of the induced lateral forces times the distance to the storey level under consideration. They have to be resisted by axial forces and moments in the vertical load-carrying members. While the base shear contributions of modes higher than the fundamental mode can be significant, the corresponding modal overturning moments for the higher modes are small. As the equivalent static lateral base shear in the NBCC (2005) also includes the contributions from higher modes for moderately tall and tall structures, a reduction in the overturning moments computed from these lateral forces appears justified. This is achieved by means of the multiplier J as given in NBCC (2005) Table 4.1.8.11 (Table 6.3). If, however, the response of the structure is dominated by its fundamental mode, the overturning moment should be calculated without any J-factor reductions. Alternatively, a dynamic analysis should be used to calculate the maximum overturning moment.

6.5.1.8 Torsional Moments

The inertial forces induced in the structure by earthquake ground motions act through the centre of gravity of the masses. If the centre of mass and the centre of rigidity do not coincide because of asymmetrical arrangement of structural elements or uneven mass distributions, torsional moments will arise. The design should endeavour to make the structural system as symmetrical as possible and should consider the effect of torsion on the behaviour of the structural elements.

6.5.2 Dynamic Analysis

For critical buildings and buildings with significant irregularities, the dynamic analysis approach is recommended to improve the accuracy of calculation of the seismic response including the distribution of forces in the building. The dynamic analysis approach includes response spectrum methods and time domain response methods.

6.5.2.1 Response Spectra

The response spectrum describes the maximum response of a Single Degree Of Freedom System (SDOF) to a particular input motion and is a function of the natural frequency and damping ratio of the SDOF system, and the frequency content and amplitude of the input motion. The response may be expressed in terms of acceleration, velocity or displacement.

The maximum values of acceleration, velocity and displacement are referred to as the spectral acceleration, S_a, spectral velocity, S_v, and spectral displacement, S_d, respectively. They can be related to each other as follows:

$$S_d = |u|_{max} \tag{6.13a}$$

$$S_v = |\dot{u}|_{max} \approx \omega_0 S_d \tag{6.13b}$$

$$S_a = |\ddot{u}|_{max} \approx \omega_0^2 S_d \tag{6.13c}$$

where

ω_0 is the natural circular frequency of the SDOF system. These response spectra provide a meaningful characterization of earthquake ground motion and can be related to structural response quantities, i.e.:

$$E_{\max} = \frac{1}{2} m S_v^2 \tag{6.14a}$$

$$V_{\max} = m S_a \tag{6.14b}$$

where

E_{max} is the maximum earthquake elastic energy stored in the structure, V_{max} is the structure elastic base shear and m is the mass of the structure. The base shear, however, would be less than that calculated by Equation 6.14b for structures that experience inelastic material behaviour (e.g., cracking of concrete and yielding of steel) during an earthquake. However, this reduction is only allowed for structures that have the capacity to deform beyond the yield point without major structural failure (ductile structures).

Most structures are not SDOF systems and higher modes may contribute to the response. This effect can be accounted for approximately using the higher mode factors given in Table 6.3.

6.5.2.2 Design Spectra

Spectral shapes from real records are usually smoothed to produce smooth spectral shapes suitable for use in design. Most of the design spectra commonly used are based on the Newmark-Hall approach. As an example, Figure 6.4 shows the design spectra (normalized to the maximum ground acceleration) developed by Seed and Idriss (1982) and recommended for use in building codes. The National Building Code of Canada (NBCC 2005) includes similar smooth design spectra (which are not based on Newmark-Hall approach, but obtained directly from probabilistic seismic hazard assessments based on spectral amplitudes, i.e., uniform hazard spectrum). Acceleration levels of probable earthquakes can be used to scale the spectral shapes to provide design spectra of particular projects.

$$S_{DS} = F_a S_s$$

$$S_{D1} = F_v S_1$$

$$T_o = 0.2\,T_s$$

$$T_s = \frac{S_{D1}}{S_{DS}}$$

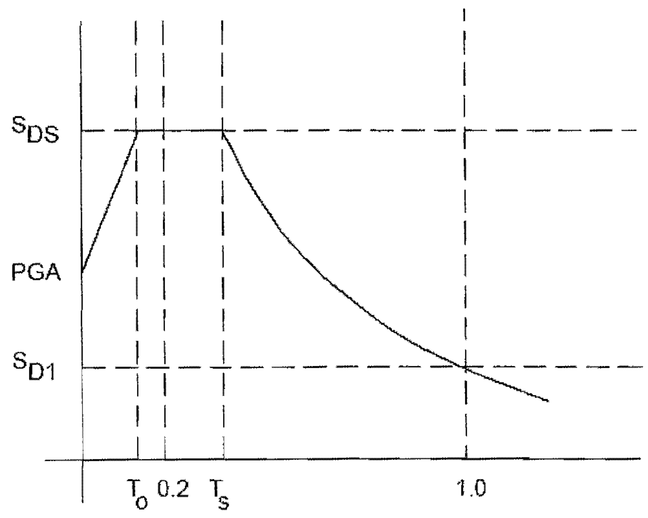


FIGURE 6.4 *Example of a design spectra*

6.5.2.3 Site Specific Response Spectra

Site-specific response spectra are developed with due consideration of the following aspects:

1. Seismotectonic characterization: includes evaluation of seismic source, wave attenuation from the sources to the site and site evaluation.

2. Assessment of seismic exposure: involves probabilistic analysis of data from possible significant earthquake

sources including nearby, mid-field and far-field events to establish the events with the most likely significant contribution to ground motions at the site.

3. Ground motion characterization: encompasses selection and scaling of rock input motion records from earthquakes with magnitude, epicentral distance, types of faulting and site conditions similar to those of the design events for the specific site. The site specific motions are then determined from the rock input motion using ground response analysis (e.g., Schnabel et al., 1972); and

4. Design ground motion specification: includes the specification of smooth response spectra, and the selection of sets of representative ground motion time histories suitable for use in dynamic analysis of the structural response.

For critical structures, spectra are usually developed for two different levels of motions, namely operating level events and major level events. Operating level events are moderate earthquakes with a high probability of occurrence. Structures are designed to survive these events without significant damage and to continue to operate. Major level events are severe earthquakes with a low probability of occurrence, and significant damage, but not collapse, is therefore acceptable. Furthermore, the seismic design of critical structures usually involves dynamic time-history analyses using a number of ground motion records representative of operating level and design level events.

6.5.2.4 Soil-Structure Interaction Effects

The NBCC considers buildings sitting on firm ground (360 m/s < $\overline{V}_s$ < 760 m/s. However, in most cases, buildings are constructed with flexible foundations embedded in soil layers. The soil-structure interaction (SSI) influences the seismic response of structures and should be investigated for cases involving critical or unconventional structures. The soil-structure interaction modifies the dynamic characteristics of the structure:

1. It reduces the natural frequency of the soil-structure system to a value lower than that of the structure under fixed-base conditions (structures found on rock are considered to be fixed-base).

2. It increases the effective damping ratio to a value greater than that of the structure itself. SSI also has some important effects on the ground input motion and the seismic response of the structure. For example, large foundation slabs can reduce the high frequency motions and hence reduce the input motions to the structure, and uplift of foundation slabs can reduce forces transmitted to the structure. Furthermore, SSI reduces the maximum structural distortion and increases the overall displacement by an amount that is inversely proportional to soil stiffness. Thus it tends to reduce the demands on the structure but because of the increased flexibility of the system, the overall displacement increases. These effects can be important for tall, slender structures or for closely spaced structures that may be subject to pounding when relative displacements become large.

In a seismic soil-structure interaction analysis, a structure with finite dimensions interacts dynamically through the structure-soil interface with a soil of infinite dimensions. A detailed analysis for this problem may be desirable and can be accomplished effectively using the finite element (or finite difference) method. Methods for the analysis of soil-structure interaction can be divided into two main categories: direct methods and multistep methods.

Direct Method: The entire soil-foundation-structure system is modelled and analysed in one single step. Free-field input motions are specified along the base and sides of the model and the resulting response of the interacting system is computed. It is preferable that the base of the mesh is placed at the top of the bedrock. The governing equations of motion for this case are

$$[M]\{\ddot{u}\}+[K^*]\{u\}=-[M]\{1\}\ddot{u}_b(t) \tag{6.15}$$

in which

$\{u\}$ are the relative motions between nodal points in the soil or structure and the top of the rock and $\{\ddot{u}_b(t)\}$

are the specified free-field accelerations at the boundary nodal points.

The ground motion, $\ddot{u}_b$, is prescribed for the surface of bedrock or firm ground. When the near surface soils are not firm ground, as is most often the case, the corresponding free-field motion of the model, $\ddot{u}_b$, is applied at the appropriate depth as outcrop motion and the surface motion is predicted accordingly. The surface motion predicted reflects the soil conditions at the site. In this process, nonlinearity of soil behaviour should be accounted for in order to avoid unrealistic amplification of the response.

Equation 6.15 is solved in the frequency domain using FFT, time domain using the Wilson θ or modified Newton-Raphson method, or in terms of modal analysis.

Several software packages are available now that have the capability to analyse the soil-structure interaction problem in the time domain accounting for nonlinearity within the soil and at the structure-soil interface. This type of analysis is recommended for critical structures or when performance-based design is considered.

Multi-step Method: In this method, emphasis is placed on the notations of the kinematic and inertial interaction. This is accomplished by isolating the two primary causes of soil-structure interaction. Because this method relies on superposition, it is limited to the analysis of linear (or equivalent linear) systems. The analysis is described as follows.

Kinematic interaction: In the free-field, an earthquake will cause soil displacements in both the horizontal and vertical directions. If a foundation on the surface of, or embedded in, a soil deposit is so stiff that it cannot follow the free-field deformation pattern, its motion will be influenced by kinematic interaction, even if it has no mass. Kinematic interaction will occur whenever the stiffness of the foundation system impedes development of the free-field motions. Kinematic interaction can also induce different modes of vibration in a structure. For example, if vertically propagating S-waves have a wavelength equal to the depth of the foundation embedment, a net overturning moment can be applied to the foundation, thereby causing the foundation to rock as well as to translate. Horizontally propagating waves can, in a similar manner, induce torsional vibration of the foundation.

The multi-step analysis proceeds as follows:

1. A kinematic interaction analysis, in which the foundation-structure system is assumed to have stiffness but no mass, is performed and the foundation input motion is obtained.

2. The foundation input motion is applied to obtain an inertial load on the structure in inertial interaction analysis in which the mass of the foundation and structure is included.

6.6 Liquefaction

Massive failures occurred during the Alaska (1964) and Niigata (1964) earthquakes showed the importance of damage caused by ground failure and the need for an analysis of the suitability of the site selected for the structure before its design and construction. While in certain cases of ground failure it is possible to design safe structures by properly designing their foundations, in other cases some mitigating measures must be taken such as soil improvement.

Seismic liquefaction refers to a sudden loss in stiffness and strength of soil due to cyclic loading effects of an earthquake. The loss arises from a tendency for soil to contract under cyclic loading, and if such contraction is prevented or curtailed by the presence of water in the pores that cannot escape, it leads to a rise in pore water pressure and a resulting drop in effective stress. If the effective stress drops to zero (100 % pore water pressure rise), the strength and stiffness also drop to zero and the soil behaves as a heavy liquid. However, unless the soil is very loose it will dilate and regain some stiffness and strength, as it strains. The post-liquefaction strength is called the residual strength and may be 1 to 10 times lower than the static strength.

If the residual strength is sufficient, it will prevent a bearing failure for level ground conditions, but may still result in excessive settlement. For sloping ground conditions, if the residual strength is sufficient it will prevent a flow slide, but displacements commonly referred to as lateral spreading, could be excessive. In addition, even for level ground condition where there is no possibility of a flow slide and lateral movements may be tolerable, very significant settlements may occur due to dissipation of excess pore water pressures during and after the period of strong ground shaking.

During an earthquake, significant damage can result due to instability of the soil in the area affected by the seismic waves. The soil response depends on the mechanical characteristics of the soil layers, the depth of the water table and the intensity and duration of the ground shaking. If the soil consists of deposits of loose granular materials it may be compacted by the ground vibrations induced by the earthquake, resulting in large settlement and differential settlements of the ground surface. This compaction of the soil may result in the development of excess hydrostatic pore water pressures of sufficient magnitude to cause liquefaction of the soil, resulting in settlement, tilting and rupture of structures.

Liquefaction does not occur at random, but is restricted to certain geologic and hydrologic environments, primarily recently deposited sands and silts in areas with high ground water levels. Generally, the younger and looser the sediment, and the higher the water table, the more susceptible the soil is to liquefaction. Sediments most susceptible to liquefaction include Holocene delta, river channel, flood plain, and aeolian deposits, and poorly compacted fills. Liquefaction has been most abundant in areas where ground water lies within 10 m of the ground surface; few instances of liquefaction have occurred in areas with ground water deeper than 20 m. Dense soils, including well-compacted fills, have low susceptibility to liquefaction.

6.6.1 Factors Influencing Liquefaction

The following factors influence the liquefaction potential of a given site:

1. Soil type: saturated granular soils, especially fine loose sands and reclaimed soils, with poor drainage conditions are susceptible to liquefaction.

2. Relative density: loose sands are more susceptible to liquefaction, e.g., sand with $D_r > 80\%$ is not likely to liquefy.

3. Confining pressure: the confining pressure, σ_0, increases the resistance to liquefaction.

4. Stress due to earthquake: as the intensity of the ground shaking increases, the shear stress ratio, (τ/σ_0), increases and the liquefaction is more likely to occur.

5. Duration of earthquake: as the duration of the earthquake increases, the number of stress cycles increases leading to an increase in the excess pore water pressure, and consequently liquefaction.

6. Drainage conditions: poor drainage allows pore pressure build-up and consequently liquefaction.

6.6.2 Assessment of Liquefaction

Liquefaction assessment involves addressing the following concerns:

- evaluation of liquefaction potential, i.e., will liquefaction be triggered in significant zones of the soil foundation for the design earthquake, and if so,
- could a bearing failure or flow slide occur and if not,
- are the displacements tolerable?

These effects can be assessed from simplified or detailed analysis procedures.

Simplified analysis of liquefaction triggering involves comparing the Cyclic Stress Ratio, CSR caused by the design earthquake with the Cyclic Resistance Ratio, CRR that the soil possesses due to its density.

6.6.3 Evaluation of Liquefaction Potential

Liquefaction potential can be evaluated if the cyclic shear stress imposed by the earthquake and the liquefaction resistance of the soil are characterized. Methods used to evaluate the liquefaction potential can be categorized into two main groups: methods based on past performance and analytical procedures.

6.6.3.1 Liquefaction Potential Based on Past Performance

Based on the damage survey and field observations after earthquakes, the liquefaction potential can be identified from the performance of similar deposits. An example for this approach is the method developed based on observations from the Niigata Earthquake (1964). In this method, the standard penetration resistance, N, and the confining pressure are used to characterize the liquefaction resistance of soil. Based on this approach, it may be suggested that sands with N > 20 are not susceptible to liquefaction. The earthquake magnitude, M, and the epicentral distance of liquefied sites are used to characterize the cyclic loading from the earthquake. Based on observations from previous earthquakes, it may be suggested that earthquakes with magnitudes less than 6 and/or epicentral distances greater than 500 km may not induce liquefaction.

6.6.3.2 Analytical Procedure

A number of approaches have been developed over the years to evaluate the liquefaction potential. The most common of these, the cyclic stress approach, is briefly presented.

Following the procedure proposed by Seed and Idriss (1971), the initial liquefaction is defined as the point at which the increase in pore pressure, u_{excess}, is equal to the initial effective confining pressure [i.e., when $u_{excess} = \sigma'_{3c}$].

The cyclic stress approach involves two steps and their comparison:

1. Calculation of cyclic shear stresses due to earthquake loading at different depths expressed in terms of cyclic stress ratio, CSR.
2. Characterization of liquefaction resistance of the soil deposits expressed in terms of cyclic resistance ratio, CRR.

These two steps are described as follows.

6.6.3.2(1) Characterization of Earthquake Loading

The cyclic stress approach is based on the assumption that excess pore pressure generation is fundamentally related to the cyclic shear stresses. The earthquake loading is characterized by a level of uniform cyclic shear stress, derived from ground response analysis or from a simplified procedure, applied at an equivalent number of cycles.

Ground response analyses should be used to predict time histories of shear stress at different depths within a soil deposit. An equivalent uniform shear stress is then calculated as 0.65 of the peak shear stress obtained.

Seed's Simplified Equation: For small projects, the simplified procedure proposed by Seed and Idriss (1971) can be used to estimate the cyclic shear stress due to the earthquake for level sites, in terms of the cyclic stress ratio, CSR, i.e.:

$$CSR = \frac{\tau_{cyc}}{\sigma'_{v_0}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_{v_0}} r_d \qquad (6.16)$$

where

a_{max} = the peak ground surface acceleration for the design earthquake, g = gravity acceleration, σ_v = total vertical overburden pressure, σ'_{v_0} = the initial effective overburden pressure and r_d = stress reduction value at the depth of interest that accounts approximately for the flexibility of the soil profile. The stress reduction coefficient, r_d, can be approximated by

$$r_d = 1.0 - 0.00765z \text{ for } z \leq 9.15\text{m} \quad (6.17a)$$

$$r_d = 1.174 - 0.0267z \text{ for } 9.15\text{m} < z \leq 23\text{m} \quad (6.17b)$$

where

z is the depth below the ground surface in metres.

Ground response analysis using equivalent-linear total stress programs: Liquefaction Triggering is traditionally assessed by conducting an equivalent-linear-total-stress ground response analysis using the 1D program SHAKE. The analyses can also be conducted in 2D using the program FLUSH and others.

The induced cyclic stress ratio (CSR) (0.65 of the peak value of τ_{cyc}/σ'_{vo}) from the ground response analysis is equated to the cyclic resistance ratio (CRR) to obtain a factor of safety against liquefaction triggering as indicated in equation 6.18. Input for the ground response analysis would be the firm ground time histories. As indicated in equation 6.18 below, corrections are typically made for magnitude (K_m), confining stress (K_σ) and sometimes static bias (K_α):

$$\text{Factor of Safety against liquefaction} = (\text{CRR x } K_\sigma \text{ x } K_m \text{ x } K_\alpha)/\text{CSR} \quad (6.18)$$

Ground response analysis using non-linear total-stress program with hysteretic damping: In the equivalent linear analyses, the same damping is used for both small strain and large strain cycles throughout the duration of shaking. In reality, small strain cycles will have significantly lower damping than high strain cycles. This shortfall can be addressed by using a constitutive model with hysteretic damping. Such models have been developed to run within FLAC and other programs and can be used to assess liquefaction triggering in both 1D and 2D approximations. The CSR would typically be set equal to 0.65 of the peak value and factor of safety against liquefaction would be calculated using equation 6.18. The method should be calibrated using measured responses from actual earthquakes prior to use. Other advantages of the method are that it can be readily used in 2D analyses and therefore used with sloping ground surface. Structural elements can be included and soil-structure effects modeled if desired.

2D total stress models which track the dynamic shear stress history within each element and trigger liquefaction if a specified threshold is reached are also available.

Ground response analysis using non-linear effective stress programs: These procedures can be used to assess both liquefaction triggering and the consequences of liquefaction.

6.6.3.2(2) Seismic Hazard, Choice of Magnitude and Records

This section deals with the earthquake hazard, the magnitude of the earthquake to be used in liquefaction assessment, and suggestions on earthquake records to be used.

Hazard

Use the spectra given in the NBCC (2005) for firm ground conditions for the 1:2475 hazard (for Vancouver, use the Cascadia subduction hazard). If the 1:475 hazard is needed this can be scaled from the 1:2475 spectrum or found in Geological Survey of Canada web site.

Magnitude for use in Liquefaction Assessment

Deaggregation of the hazard for Vancouver for the 1:2475 probability gives magnitudes of M6.5 to M6.9 depending on whether the mean or median values, and the Sa(.2) or Sa(1) deaggregation is considered. Using the 80th percentile on the deaggregation results gives a range of M7.0 to M7.3. The results for the 1:2475 Sa(0.2) and Sa(1.0) deaggregation is shown in the Table 6.4 below. The maximum recorded crustal earthquake in the Vancouver region has been M7.3, but the hazard calculations assume an upper bound of M7.7 as being possible. It is suggested to use the Sa(1) deaggregation because it gives larger values and the period of 1 second is closer to the first period of many soft sites than is the period of 0.2 seconds.

The 80th percentile deaggregation value should be used because the seismic hazard is substantially influenced by the upper tail of the seismic hazard, as the larger ground motions have a much higher probability of causing damage. Therefore, for Vancouver, a magnitude of M7.25 should be used in assessing liquefaction for the 1:2475 hazard, and M8.2 should be used for the Cascadia subduction earthquake. If the 1:475 hazard is considered, use M6.5.

TABLE 6.4 *Earthquake Magnitude for Vancouver Evaluated from Deaggregation*

Measure	Sa(0.2)	Sa(1.0)
Mode	7.13	6.88
Mean	6.52	6.90
Median	6.51	6.82
80%ile	6.93	7.30

Selection of earthquake records

The Geological Survey of Canada is assembling a suite of records for both the 1:2475 and 1:475 probabilistic hazard and for the Cascadia subduction earthquake. However, it is not easy to find a suite of records that give a good fit to the spectrum and have the appropriate duration and/or number of cycles. Some useful guidelines for choosing records are:

The records should have a spectrum close to the UHRS, and should have duration consistent with the magnitude. The record should be scaled so that the spectrum matches the design spectrum in the period range of interest (related to the fundamental period of the site), or the records should be spectrum matched to the design spectrum. The record should have a number of large cycles, for example the NCEER assessment criteria assume that a M7 earthquake record has 10 significant full cycles greater than 0.65 PGA.

6.6.3.2(3) Characterization of Liquefaction Resistance

The Cyclic Resistance Ratio, CRR, is a measure of the soils ability to resist liquefaction and the development of large strains, and depends mainly on the soil type and density or state. There are two approaches to the characterization of liquefaction resistance, namely methods based on the results of laboratory tests, and methods based on in-situ tests.

Laboratory tests: Different laboratory tests are performed mostly on isotropically consolidated triaxial specimens or on K_0- consolidated simple shear specimens. In these tests, liquefaction failure is defined as the point at which initial liquefaction was reached or at which some limiting cyclic strain amplitude (commonly 5-20 %) was reached. The measured cyclic stress at the onset of liquefaction failure is the liquefaction resistance and is frequently given in terms of the cyclic resistance ratio, CRR = τ_{cyc}/σ'_{v0}.

Comments on testing methods: Undisturbed samples retrieved using specialized sampling techniques (such as ground freezing) should be used in the tests. The simple shear test is the most common test although it is difficult to eliminate its problems. The torsional shear test is sometimes used to ensure uniform distribution of the shear stress but it is very costly and difficult to obtain a hollow sample. Shaking table tests suffer from the lack of suitable confining pressure. Cyclic triaxial tests are also used, however, they impose different loading conditions than the soil experiences during an earthquake and their cyclic stresses need to be corrected.

Cyclic simple shear tests are considered most representative of field conditions during earthquake loading, The results of such a test for loose Fraser River sand are shown in Figure 6.5. The effective stress path shows the normal effective stress reducing with each cycle of shear stress from an initial value of 100 kPa to essentially zero after 6 cycles. Figure 6.5 also shows the shear stress Vs shear strain response, where it may be seen that strains are very small, less than 0.1 %, for the first 5 cycles and become very large, 10 %, on the 6th cycle, when liquefaction is triggered. The applied stress ratio for this sample was 0.1 and caused liquefaction in 6 cycles. The CRR is generally specified as the stress ratio to cause liquefaction in 15 cycles, and from additional tests carried out on this material $(CRR)_{15} = 0.085$.

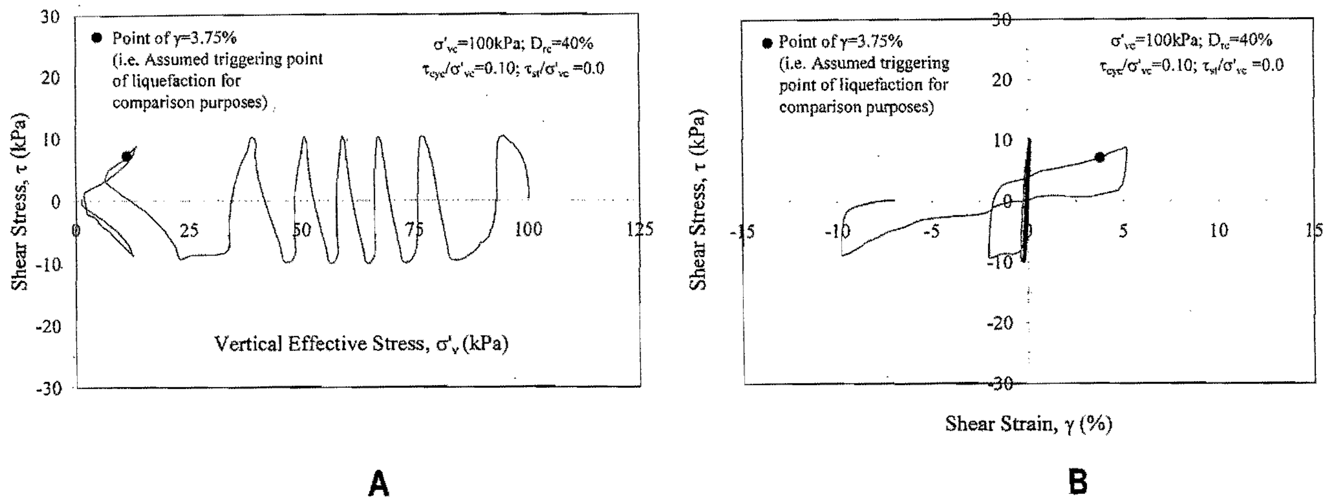


FIGURE 6.5 *Stress path and shear stress-strain response of loose Fraser River sand, cyclic simple shear tests (Wijewickreme et al. 2005)*

The liquefaction response shown in Figure 6.5 is typical for loose sands where the application of an additional cycle of load triggers an abrupt change in behaviour from stiff to soft. The soft post-liquefaction response is controlled by dilation. The drop in shear stiffness upon liquefaction can be in the range of 100 to 1000 times. The strength or strength ratio available after liquefaction, called the residual strength can be significant, and from Figure 6.5, the strength ratio is at least 0.1 for loose Fraser river sand. However, field experience indicates that the strength ratio can be significantly lower than values obtained from undrained tests. The reason for this may be due to upward flow of water associated with generated excess pore water pressures. This may cause some elements to expand and lose their dilation effect, particularly those beneath layers of lower permeability.

For silt and clay material the response to cyclic loading and liquefaction can be quite different than for sand as shown in Figure 6.6. This figure shows effective stress path and shear stress-strain response for loose normally consolidated Fraser River silt under cyclic simple shear loading. The effective stress path shows the normal effective stress reducing with each cycle from its initial value 100 kPa, but not dropping below 10 kPa. After the initial few cycles, loading is associated with an increase in effective stress resulting from dilation. Only the unloading shows strong contraction effects. The shear stress-strain response shows a gradual increase in strain with number of cycles, and there is no abrupt change in shear stiffness from stiff to soft. There is also no indication of a strength reduction below the applied stress ratio of 0.2, thus the post-liquefaction or residual strength ratio is at least 0.2 for the tested silt. The stiffness reduces with each cycle, and after 11 cycles is 10 to 20 times softer than the first cycle.

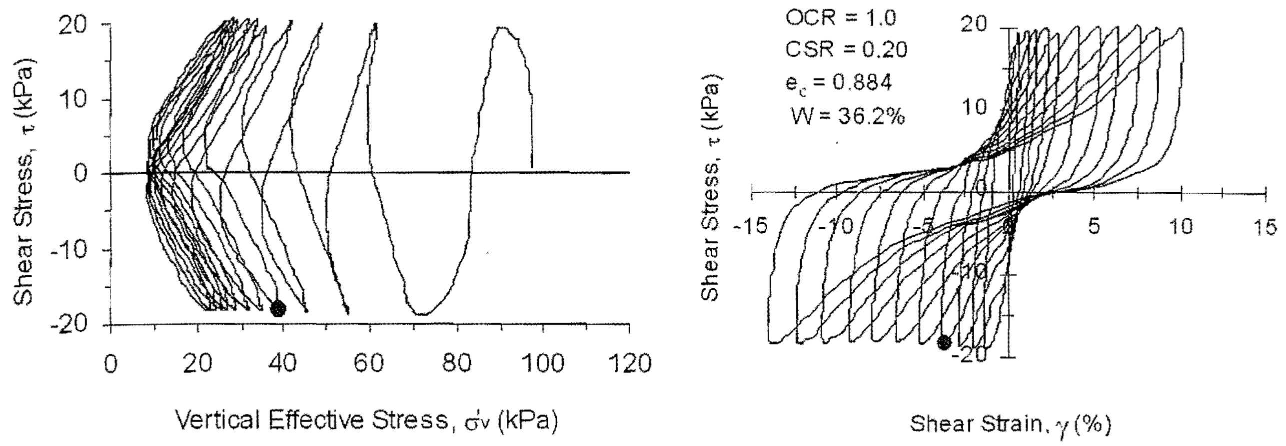


FIGURE 6.6 *Stress path and shear stress-strain response of Fraser River silt, cyclic simple shear tests (Sanin and Wijewickreme, 2006)*

These test results indicate that fine-grained normally consolidated silts and clays of low plasticity can be far more resistant to liquefaction than loose sands.

Test results together with field experience suggest that the liquefaction response of coarse-grained soils, gravels, sands and non-plastic silts should be handled differently than fine-grained silts and clays. While it might seem desirable to recover undisturbed samples (it is possible to do so in fine-grained soils) and obtain a direct measure of liquefaction resistance from cyclic testing, it is very difficult and expensive to obtain undisturbed samples in coarse-grained soils. It is therefore recommended that CRR for coarse-grained soils be based on penetration resistance in accordance with NCEER (2001). For fine-grained soils, it is recommended that CRR be based on Atterberg limits and/or direct testing.

In-situ tests: The soil parameters determined from in-situ tests are used as liquefaction resistance parameters.

Standard penetration resistance: The corrected SPT resistance is plotted vs. cyclic resistance ratio for clean sand (Figure 6.7) sites where liquefaction was or was not observed in earthquakes of M = 7.5 to determine the minimum cyclic stress ratio at which liquefaction could be expected. CRR for other magnitudes may be obtained by multiplying the CRR for M = 7.5 earthquakes by a correction factor, K_M, as recommended by NCEER (2001), i.e.:

$$K_M = \frac{10^{2.24}}{M^{2.56}} \tag{6.19}$$

The data used in Figure 6.7 are for cyclic resistance ratios associated with overburden pressure, $\sigma'_{v0} = 100$ kPa. For higher overburden pressure values, the cyclic resistance ratio must be corrected using a correction factor K_σ given by

$$K_\sigma = \frac{CRR_{\sigma'_{v0}>100kPa}}{CRR_{\sigma'_{v0}=100kPa}} \tag{6.20}$$

Values for K_σ may be taken from the average curve of Seed and Harder (1990) (Figure 6.8).

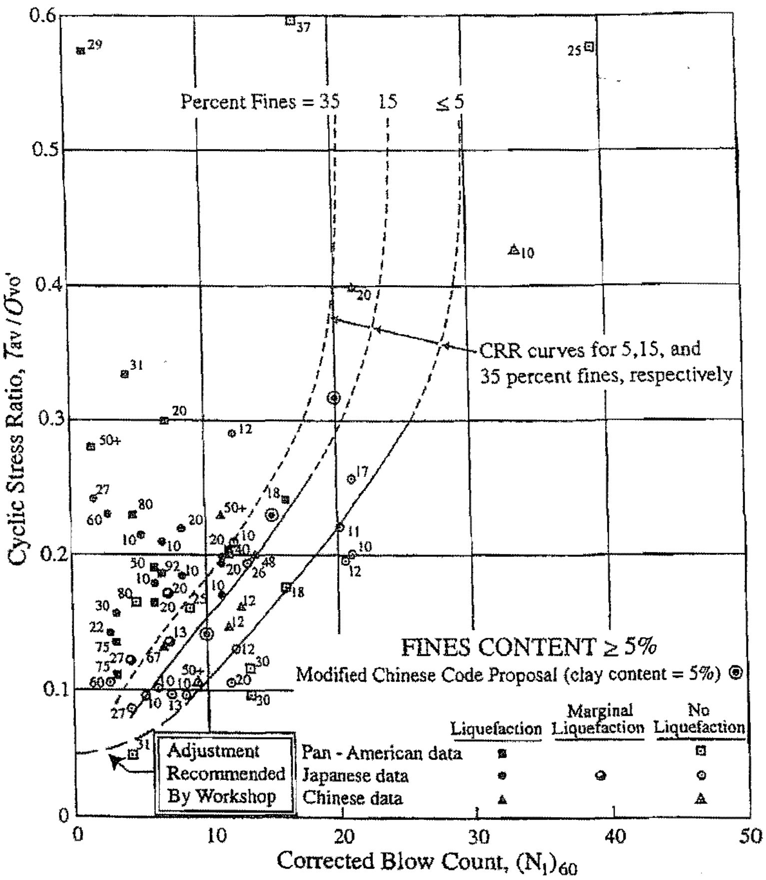


FIGURE 6.7 *CRR_1 vs $(N_1)_{60}$ (Youd et al. 2001)*

Cone penetration test: The tip resistance from the cone penetration test (CPT) is used as a measure of liquefaction resistance. CPT-based liquefaction curves have been developed based on correlation with laboratory test and theoretically derived values of CPT resistance (Figure 6.9). In CPT-based liquefaction evaluations, the tip resistance is normalized to a standard effective overburden pressure of 96 kPa by

$$q_{c1} = q_c \left(\frac{96}{\sigma'_{v0}} \right)^{0.5} \quad \text{or} \quad q_{c1} = \frac{1.8}{0.8 + \sigma'_{v0}/96} \tag{6.21}$$

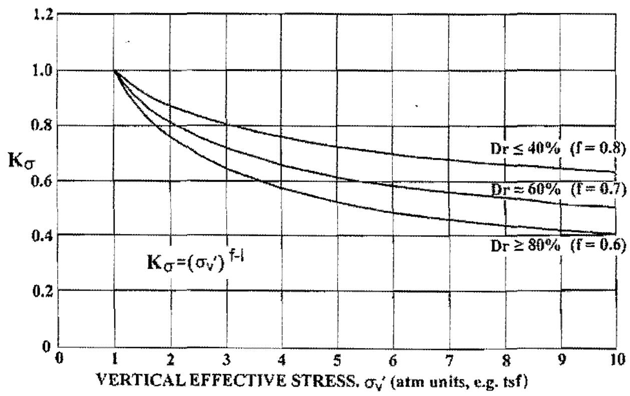


FIGURE 6.8 *Recommended curves for estimating K_σ for engineering practice (Youd et al. 2001)*

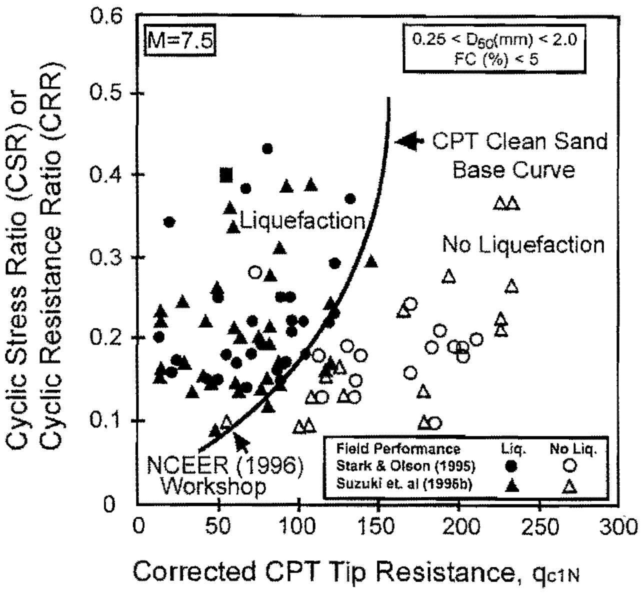


FIGURE 6.9 *Curve recommended for calculation of CRR from CPT data along with empirical liquefaction data from compiled case histories (reproduced from Robertson and Wride 1998) (Youd et al. 2001)*

Shear wave velocity: The measured shear wave velocities can be used to assess the liquefaction resistance, (usually in addition to assessment using SPT or CPT). Measured shear wave velocities are normalized to a standard effective overburden pressure of 96 kPa by

$$V_{s1} = V_s(\sigma'_{v0}/96)^{-1/n} \tag{6.22}$$

where

n = 3 to 4. The normalized shear wave velocity is plotted vs. CRR in Figure 6.10, which can be used to evaluate the liquefaction potential directly, or is used to evaluate the CRR, which is used in turn to evaluate the liquefaction potential.

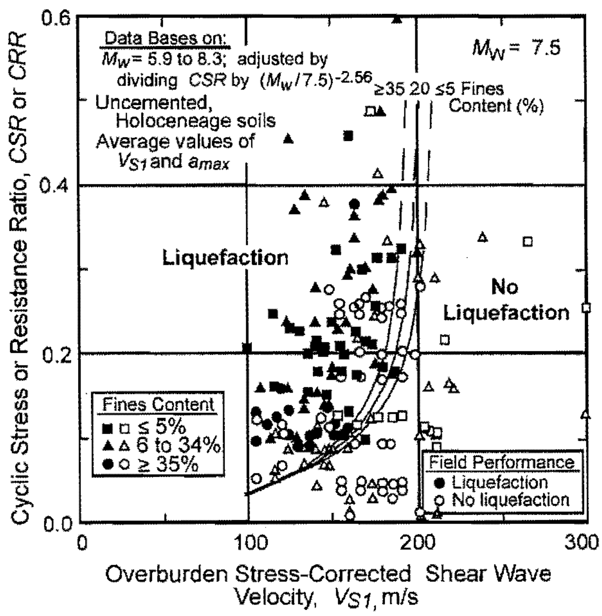


FIGURE 6.10 *Liquefaction relationship recommended for clean, uncemented soils with liquefaction data from compiled case histories (Reproduced from Andrus and Stokoe 2000) (Youd et al. 2001)*

The base CRR obtained from these figures will be given the symbol CRR_1. The CRR for a general condition is given by:

$$CRR = CRR_1 * K_m * K_\sigma * K_\alpha \qquad (6.23)$$

where

K_m is a correction factor for earthquake magnitudes other than M7.5,
K_σ is a correction factor to account for effective overburden stresses other than 100 kPa, and
K_α is a correction factor for ground slope.

The recommended K_m (MSF) curve is shown in Figure 6.11, and for an M7 earthquake $K_m = 1.25$.

The recommended K_σ curves depend on relative density as was shown in Figure 6.8. NCEER does not make a recommendation regarding K_α. The default value is unity, $K_\alpha=1$.

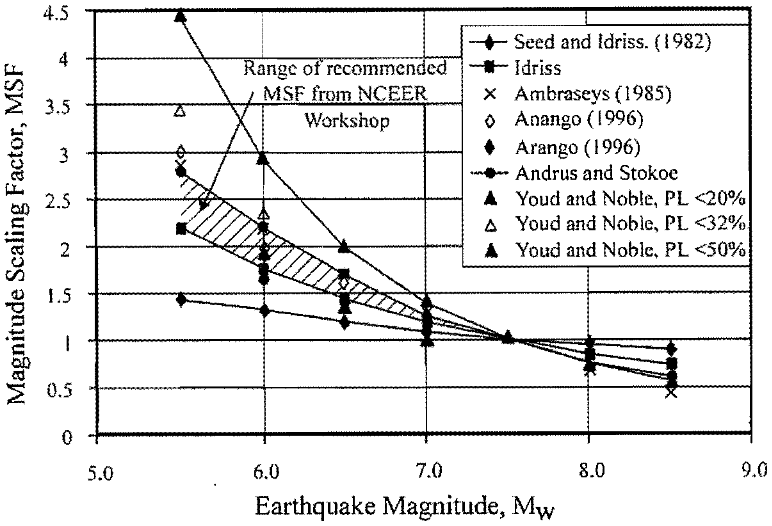


FIGURE 6.11 *Magnitude Scaling Factors derived by various investigators (Reproduced from Youd and Noble 1997) (Youd et al. 2001)*

6.6.3.2(4) Evaluation of Initiation of Liquefaction

The evaluation is easily performed graphically. First, the variation of cyclic stress ratio, CSR, with depth is plotted. The variation of the cyclic resistance ratio, CRR, with depth is then plotted on the same graph. Liquefaction can be expected at depths where the loading exceeds the resistance or when the factor of safety against liquefaction, FS_L, is less than 1, where:

$$FS_L = \frac{CRR}{CSR} \qquad (6.24)$$

6.6.3.2(5) Residual Strength for Gravel, Sands, and Non Plastic Silts

Field experience during past earthquakes indicates that residual strengths can be much lower than values obtained from undrained tests on undisturbed samples. This may be due to upward flow of water associated with generated excess pore water pressures. This may cause some elements to expand to a higher void ratio, and hence a lower critical state strength. Based on back analysis of field case histories, Seed and Harder (1990) proposed upper and lower bounds on residual strength as shown in Figure 6.12. It may be noted that there are no data points associated with large movements or flow slides for SPT blowcounts greater than 16.

Olson and Stark (2002) present residual strength in terms of strength ratio, Figure 6.13. Their values range between about 0.05 and 0.1 for SPT blowcounts in the range 2 to 12. They also developed residual strength ratios in terms of CPT tip resistance. Their relationship is shown in Figure 6.14.

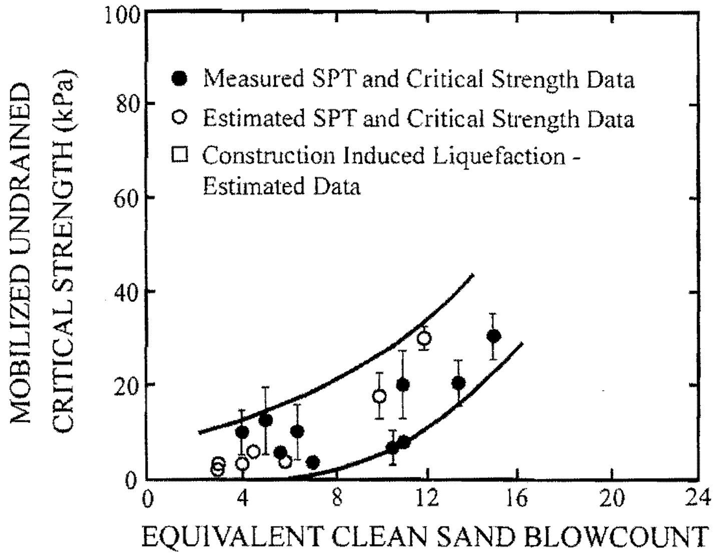


FIGURE 6.12 *Recommended relationship between $s_{u,r}$ and $N_{1,60,CS}$ (Seed and Harder 1990)*

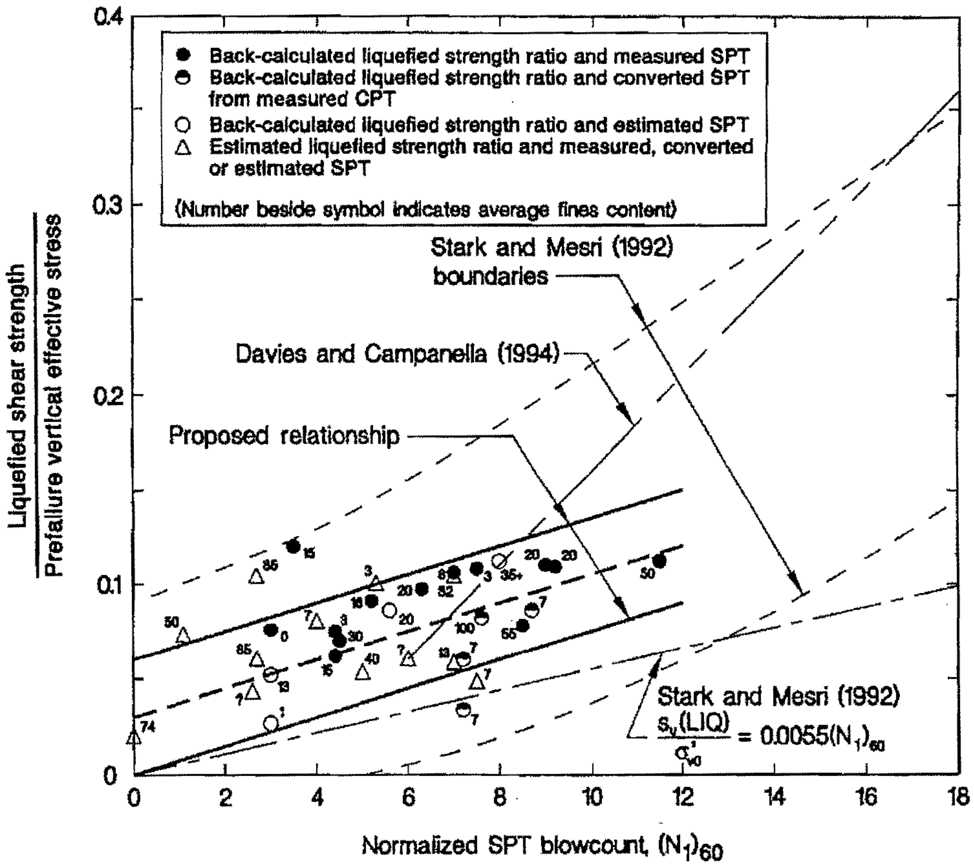


FIGURE 6.13 *A comparison of liquefied strength ratio relationships based on normalized SPT blowcount (Olson and Stark 2002)*

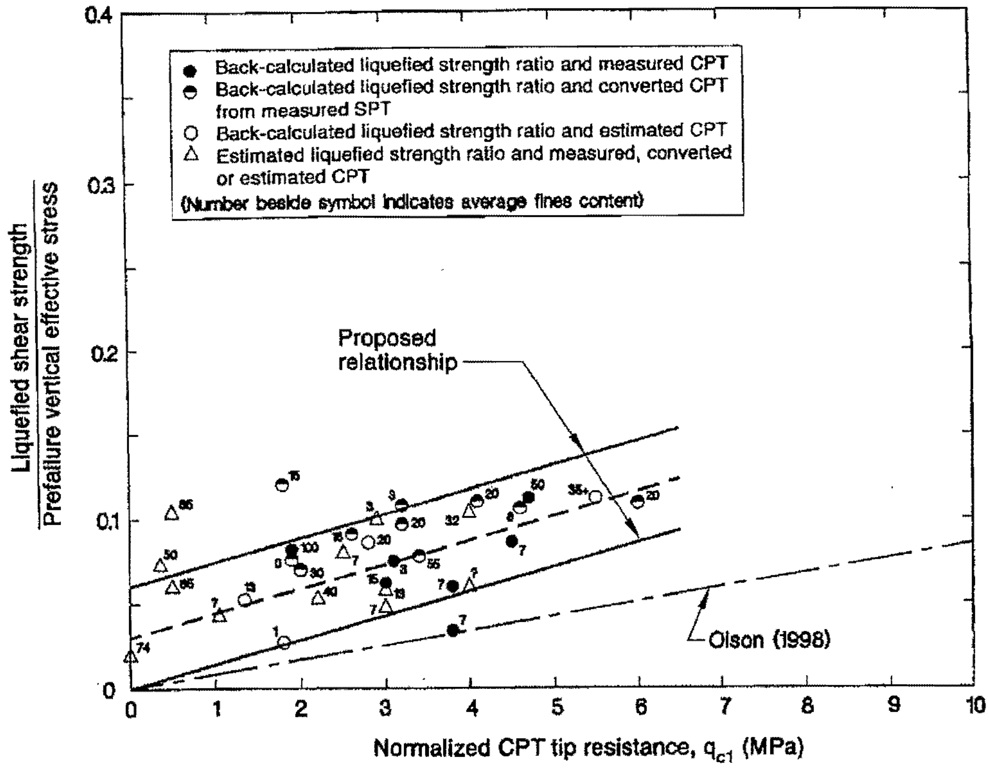


FIGURE 6.14 *A comparison of liquefied strength ratio relationships based on normalized CPT tip resistance (Olson and Stark 2002)*

It is recommended that for zones predicted to liquefy, the residual strength be estimated as follows:

1. For normalized SPT blowcounts less than or equal to 15, use mean values from Seed and Harder (1990) and/or Stark and Olson (2002).
2. For normalized SPT blowcounts greater than or equal to 25, use drained strength values.
3. For normalized SPT blowcount values between 15 and 25, use a linear variation of residual strength.

Although liquefaction can be triggered in dense sands having normalized SPT blowcount values greater than 25, the drained strength values can be used, as dilation upon straining will cause the pore pressures to drop to their pre-earthquake values or lower.

6.6.3.2(6) CRR for Silts and Clays

It has been noted that some fine-grained soils that classify as non-liquefiable according to commonly used empirical "Chinese Criteria" (Wang 1979; Koester 1992; Finn et al. 1994) have in fact experienced liquefaction during earthquakes (Boulanger et al. 1998, Bray et al. 2004). Some data from laboratory cyclic shear testing of silts also confirmed the limitation of Chinese Criteria as a tool to identify potentially liquefiable soils (Sanin and Wijewickreme 2004; Boulanger and Idriss 2004).

As an alternative, Boulanger and Idriss (2004) recommend that fine-grained soils be classified as "sand-like" (susceptible to liquefaction) if $I_p < 7$, and "clay-like" if $I_p \geq 7$. However, some limitations in this approach have been noted from cyclic direct simple shear tests conducted on specimens from a channel fill silt from the Fraser River Delta (Sanin and Wijewickreme 2005).

Based on the field performance of fine-grained soil sites in Adapazari following the 1999 Kocaeli (Turkey) earthquake, combined with data from laboratory cyclic shear testing, Bray et al. (2004) have proposed alternate empirical criteria to delineate liquefaction susceptibility of fine-grained soils. It is recommended that the use of Chinese Criteria be discontinued, and Bray et al. (2004) criteria (Figure 6.15) be used to determine liquefaction susceptibility of fine-grained soils:

a) $w/w_L \geq 0.85$ and $I_p \leq 12$: Susceptible to liquefaction or cyclic mobility*;
b) $w/w_L \geq 0.8$ and $12 < I_p < 20$: Moderately susceptible to liquefaction or cyclic mobility*;
c) $w/w_L < 0.8$ and $I_p \geq 20$: No liquefaction or cyclic mobility, but may undergo significant deformations if cyclic shear stresses > Static undrained shear strength (s_u).

*This classification may be revised on a site-specific basis using data from laboratory cyclic shear testing of good quality field samples [e.g., samples obtained using thin-walled tube samples with sharpened (i.e., <5°) cutting edge and no inside clearance].

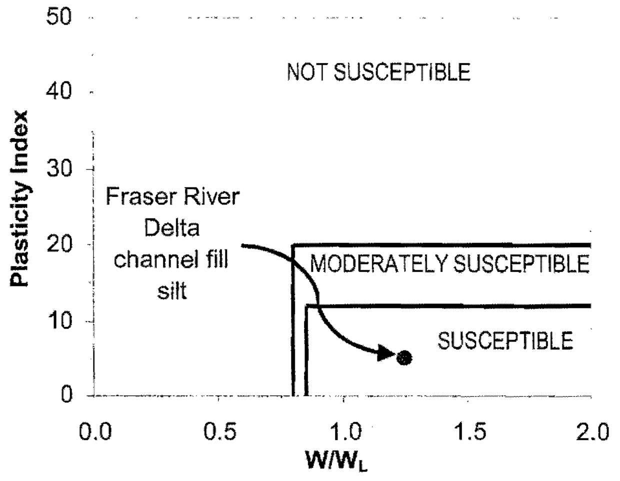


FIGURE 6.15 *Bray et al. (2004) criteria for liquefaction assessment of fine-grained soils*

6.6.3.2(7) Residual Strength for Silts and Clays

It is recommended that the residual strength (S_r) for silt and clay zones be determined as per guidelines given below:

a) $w/w_L \geq 0.85$ and $I_p \leq 12$: S_r = remolded shear strength ($S_{remolded}$), unless appropriate testing of undisturbed samples can show greater strength;
b) $w/w_L \geq 0.8$ and $12 < I_p < 20$: $S_r = 0.85 s_u$, where s_u = static undrained shear strength;
c) $w/w_L < 0.8$ and $I_p \geq 20$: $S_r = s_u$.

This approach essentially employs the liquefaction potential determined using the recommended Bray et al. (2004) criteria as the basis for the determination of S_r. This assumes that the full static undrained strength (s_u), or most part of it, is available as the residual strength after cyclic loading, unless the soil is susceptible to liquefaction.

6.6.4 Liquefaction-Like Soil Behaviour

The liquefaction potential of loose, saturated sands is well recognized as described above. Similar abrupt structural changes, however, could be caused by earthquakes also in some highly sensitive clays such as the Canadian Leda clay or the Norwegian quick clay.

6.6.5 Induced Ground Movements

There are several empirical and approximate procedures for estimating ground movement for situations where liquefaction may be triggered.

The lateral spreading equation of Youd gives ground displacement as a function of simple site properties, soil profile properties, and earthquake magnitude and distance. Post-liquefaction settlement is discussed in the following subsection.

6.6.5.1 Post-Liquefaction Settlements for Coarse-Grained Soil

Post-liquefaction settlements occur during and after earthquake shaking. For level ground conditions, the amount can be computed from the volumetric reconsolidation strains induced as the excess pore pressures dissipate. Based on field experience during past earthquakes, the amount of strain depends on SPT blowcount and the CSR applied by the design earthquake. The curves proposed by Cetin et al. (2004) are shown in Figure 6.16 and indicate that volumetric reconsolidation strains can range between about 10 % for very loose sand to 1 % for very dense sands. These curves are recommended.

The settlement calculated from this chart is induced by consolidation of the liquefied soil only. Footings and other structures founded over or within liquefied soil will also deform due to shear strain within the liquefied soil. This shear strain typically occurs during the period of strong shaking whereas the consolidation settlements often occur following the period of strong shaking. The shear strain deformations are additional to the consolidation settlements and can be of similar or greater magnitude.

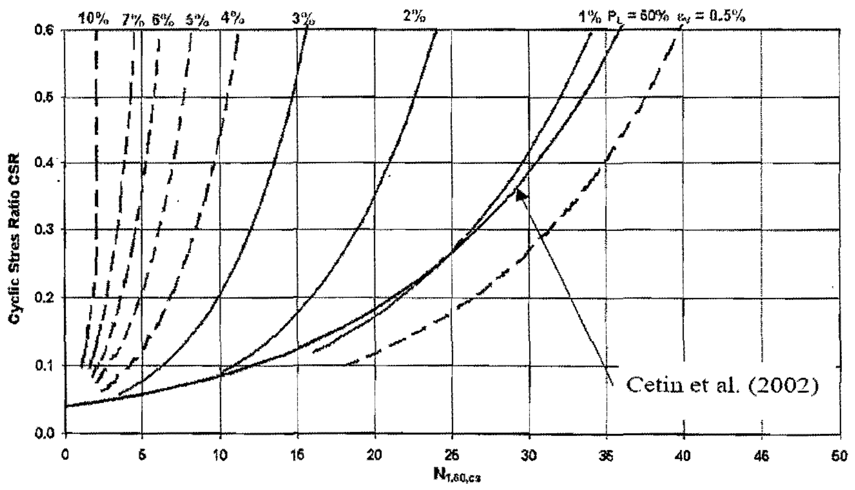


FIGURE 6.16 *Recommended relationships for volumetric reconsolidation strains as a function of equivalent uniform cyclic stress ratio and $N_{1,60,CS}$ for M_w = 7.5 (Wu 2002)*

6.7 Seismic Design of Retaining Walls

The dynamic response of retaining walls is quite complex. Walls can translate and/or rotate, and the relative amounts of translation and rotation depend on the wall design. The magnitude and distribution of dynamic wall pressures during an earthquake are influenced by the mode of wall movement. The maximum soil thrust acting on a wall generally occurs when the wall has moved toward the backfill. The minimum soil thrust occurs when the wall has moved away from the backfill. The shape of the earth pressure distribution on the back of the wall changes as the wall moves. The position of the resultant of the dynamic pressure is highest when the wall has moved toward the soil. Dynamic wall pressures are influenced by the dynamic response of the wall and backfill, and can increase significantly near the natural frequency of the wall-backfill system. Permanent soil displacements also increase at frequencies near the natural frequency of the wall-backfill system. Because of the complexity of the problem, simplified models that make various simplifying assumptions are used for the seismic design of retaining walls.

6.7.1 Seismic Pressures on Retaining Walls

Seismic pressures on retaining walls are usually estimated using simplified methods. Some of these methods are given here.

6.7.1.1 Active Earth Pressure Conditions M-O Method

This method is based on a pseudostatic analysis of seismic earth pressure on retaining structures and has become known as the Mononobe-Okabe (M-O) method. The M-O method is a direct extension of the static Coulomb theory to pseudo-static conditions.

For dry cohesionless backfill, the total active thrust can be expressed in a form similar to that developed for static conditions, i.e.:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v) \tag{6.25}$$

where

the dynamic active earth pressure coefficient, K_{AE}, is

$$K_{AE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi)\left[1 + \sqrt{\dfrac{\sin(\delta + \phi)\sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi)\cos(\beta - \theta)}}\right]^2} \tag{6.26}$$

where

ϕ = soil angle of internal friction, θ = slope of backfill with horizontal, β = slope of the back face of the retaining wall with vertical, δ = angle of friction of wall-backfill interface,

$\psi = \tan^{-1}\left(\dfrac{k_h}{1 - k_v}\right)$, and k_h and k_v are seismic coefficients in the horizontal and vertical directions, respectively, for φ-$\beta \geq \psi$. The seismic coefficient in the horizontal direction, k_h, is defined as a ratio of the peak ground acceleration in the horizontal direction to the gravity acceleration, g, i.e.:

$$k_h = \frac{a_h}{g} \tag{6.27}$$

The seismic coefficient in the vertical direction, k_v, is defined similarly.

The total active thrust, P_{AE}, can be divided into a static component, P_A and a dynamic component, ΔP_{AE}:

$$P_{AE} = P_A + \Delta P_{AE} \tag{6.28}$$

where

$$P_A = \frac{1}{2} K_A \gamma H^2 \tag{6.29}$$

in which,

K_A = the coefficient of static active earth pressure (from Coulomb theory), i.e.:

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2\theta\cos(\delta + \theta)\left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin(\phi - \beta)}{\cos(\delta + \theta)\cos(\beta - \theta)}}\right]^2} \tag{6.30}$$

The total active thrust may then be considered to act at a height, h, from the base of the wall,

$$h = \frac{P_A(H/3) + \Delta P_{AE}(0.6H)}{P_{AE}} \tag{6.31}$$

6.7.1.2 Passive Earth Pressure Conditions M-O Method

The total passive thrust on a wall retaining a dry cohesionless backfill is given by

$$P_{PE} = \frac{1}{2}K_{PE}\gamma H^2(1 - k_v) \tag{6.32}$$

where

the dynamic passive earth pressure coefficient, K_{PE}, is

$$K_{PE} = \frac{\cos^2(\phi + \theta - \psi)}{\cos\psi\cos^2\theta\cos(\delta - \theta + \psi)\left[1 - \sqrt{\frac{\sin(\delta + \phi)\sin(\phi + \beta - \psi)}{\cos(\delta - \theta + \psi)\cos(\beta - \theta)}}\right]^2} \tag{6.33}$$

The total passive thrust can also be divided into static and dynamic components:

$$P_{PE} = P_P + \Delta P_{PE} \tag{6.34}$$

where

P_P is the static passive thrust, given by

$$P_P = \frac{1}{2}K_P\gamma H^2 \tag{6.35}$$

where

$$K_P = \frac{\cos^2(\phi + \theta)}{\cos^2\theta\cos(\delta - \theta)\left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin(\phi + \beta)}{\cos(\delta - \theta)\cos(\beta - \theta)}}\right]^2} \tag{6.36}$$

Note that the dynamic component, ΔP_{PE}, acts in the opposite direction of the static component, P_P, thus reducing the available passive resistance.

Discussion: The M-O analysis provides a useful means of estimating earthquake-induced loads on retaining walls. A positive horizontal seismic coefficient causes the total active thrust to exceed the static active thrust and the total passive resistance to be less than the static passive resistance. Since the stability of a particular wall is generally reduced by an increase in active thrust and/or a decrease in passive resistance, the M-O method produces seismic loads that are more critical than the static loads.

The M-O analysis has some limitations. The determination of the seismic coefficient is difficult; the analysis is not appropriate for soils that experience significant loss of strength during earthquakes, and it over predicts the actual total passive thrust, particularly for $\delta > \phi/2$.

6.7.2 Effects of Water on Wall Pressures

The water exerts loads on waterfront retaining walls both during and after earthquakes. The water outboard of a retaining wall and within the backfill can exert dynamic pressures on the wall. The total water pressures that act on retaining walls in the absence of seepage within the backfill can be divided into two components: hydrostatic pressure that increases linearly with depth and acts on the wall before, during and after earthquake shaking, and hydrodynamic pressure that results from the dynamic response of the water itself.

6.7.2.1 Water Outboard of Wall

The hydrodynamic pressures on a retaining wall are usually estimated from Westergaard's solution for the case of a vertical rigid dam retaining a semi-infinite reservoir of water that is excited by harmonic, horizontal motion of its rigid base. Westergaard computed the amplitude of the hydrodynamic pressure at a depth z_w below water surface as

$$p_w = \frac{7}{8} k_h \gamma_w \sqrt{z_w H} \tag{6.37}$$

where

H = depth of the water. The resultant hydrodynamic thrust is given by

$$P_w = \frac{7}{12} k_h \gamma_w H^2 \tag{6.38}$$

The total actual thrust due to the water is equal to the sum of the hydrostatic and hydrodynamic thrusts.

6.7.2.2 Water in Backfill

The presence of water in the backfill behind a retaining wall can influence the seismic loads on the wall in a number of ways. It alters the inertial forces within the backfill and develops hydrodynamic pressures within the backfill.

For low permeability soils, the inertial forces due to earthquake shaking will be proportional to the total unit weight of the soil. In this case, the M-O method can be modified to account for the presence of porewater within the backfill using

$$\gamma = \gamma_b (1 - r_u) \tag{6.39a}$$

and

$$\psi = \tan^{-1}\left[\frac{\gamma_{sat} k_h}{\gamma_b (1 - r_u)(1 - k_v)}\right] \tag{6.39b}$$

where

γ_b = unit weight of backfill and $r_u = \frac{u_{excess}}{\sigma'_0}$.

An equivalent hydrostatic thrust based on a fluid of unit weight $\gamma_{eq} = \gamma_w + r_u \gamma_b$ must be added to the soil thrust. Soil thrusts from partially submerged backfills may be computed using an average unit weight based on the relative volumes of soil within the active wedge that are above and below the phreatic surface.

For high permeability soils, the inertial forces will be proportional to the submerged unit weight of the soil. In this case, the porewater pressure acting on the wall is given by the Westgaard solution, i.e., Equations 6.37 and 6.38.

6.7.3 Seismic Displacement of Retaining Walls

The serviceability of retaining walls is related to permanent deformations that occur during earthquakes. Therefore, analyses that predict permanent wall deformations provide a more useful indication of retaining wall performance.

6.7.3.1 Deterministic Approach

This method is developed for the seismic design of gravity walls based on allowable wall displacements. In this method, the yield acceleration, defined as the acceleration that is just large enough to cause the wall to slide on its base, is calculated by (Richard and Elms, 1979)

$$a_y = \left[\tan\phi_b - \frac{P_{AE}\cos(\delta+\theta) - P_{AE}\sin(\delta+\theta)}{W}\right]g \qquad (6.40)$$

in which

P_{AE} is calculated using the M-O method with $k_h = \dfrac{a_y}{g}$, and W is the weight of the retaining wall.

The permanent displacement can then be calculated from

$$d_{perm} = 0.087\frac{v_{max}^2 a_{max}^3}{a_y^4} \qquad (6.41)$$

where

v_{max} = the peak ground velocity and a_{max} = the peak ground acceleration.

6.7.3.2 Statistical Approach

Whitman and Liao (1985) used a statistical approach to evaluate the permanent displacement of retaining walls due to earthquake excitation. They studied the results of sliding block analyses of 14 ground motions and found that the permanent displacements were lognormally distributed with mean values

$$\bar{d}_{perm} = \frac{37v_{max}^2}{a_{max}}\exp\left(\frac{-9.4a_y}{a_{max}}\right) \qquad (6.42)$$

6.7.3.3 Finite Element Analysis

The finite element analysis can be used to compute the earthquake-induced deformations of retaining walls. A rigorous analysis should be capable of accounting for nonlinear, inelastic behaviour of the soil and of the interfaces between the soil and the elements of the wall. Some considerations have to be included in the analysis with respect to the boundaries and elements size.

6.7.4 Seismic Design Consideration

The design of retaining walls for seismic conditions is similar to the design for static conditions. Seismic design procedures make use of simplifying assumptions to allow the use of available procedures for static conditions.

6.7.4.1 Gravity Walls

Gravity walls are customarily designed using one of two approaches: a seismic pressure-based approach or a permanent displacement-based approach.

Design Based on Seismic Pressures: The M-O method is commonly used along with an inertial force with the same pseudo-static acceleration applied to the active wedge as is applied to the wall itself. Pseudo-static accelerations are generally considerably smaller than anticipated peak accelerations (values between 0.05g and 0.15g are used). The wall must be designed to avoid sliding, overturning and bearing capacity failure. The pseudo-static forces along with static analysis procedures are used in this approach.

Design Based on Allowable Displacements: This approach allows the designer to consider the consequences of permanent displacement for an individual wall when selecting an allowable displacement for design. Design procedures based on Richard-Elms (1979) and Whitman-Liao (1985) methods for estimation of permanent displacement as discussed in Sections 6.7.3.1 and 6.7.3.2.

The Richard-Elms procedure is summarized as follows:

1. Select an allowable permanent displacement, d_{all}.
2. Calculate the yield acceleration required to produce the allowable permanent displacement as

$$a_y = \left(\frac{0.087 v_{max}^2 a_{max}^3}{d_{all}} \right)^{1/4} \tag{6.43}$$

3. Calculate P_{AE} using the M-O method with the yield acceleration from step 2 as the pseudostatic acceleration.
4. Calculate the wall weight required to limit the permanent displacement to the allowable permanent displacement as

$$W = \frac{P_{AE}\cos(\delta + \theta) - P_{AE}\sin(\delta + \theta)\tan\phi_b}{\tan\phi_b - a_y / g} \tag{6.44}$$

5. Apply a factor of safety to the weight of the wall. A factor of safety, FS = 1.1 to 1.2 is suitable.

Gravity walls can be designed using the Whitman-Liao approach on the basis of allowable displacements that have defined probabilities of exceedence. The yield acceleration in this case is calculated as

$$a_y = \frac{a_{max}}{9.4} \ln\left(\frac{37 \overline{M} v_{max}^2}{a_{max} d_{all}} \right) \tag{6.45}$$

where

$\overline{M}$ = model error =3.5. Then, the same procedure as Richard-Elms is followed.

6.7.4.2 Reinforced Soil Walls

During an earthquake, a reinforced soil wall is subjected to a dynamic soil thrust at the back of the reinforced zone and to inertial forces within the reinforced zone in addition to static forces. The wall must be designed to avoid external instability (sliding, overturning and bearing capacity failure) and internal instability (pullout failure of the reinforcement).

External Stability: A reinforced earth wall can be treated like a gravity wall. The external stability of an earth reinforced wall can be evaluated as follows:

1. Determine the peak horizontal ground surface acceleration, a_{max}.
2. Calculate the peak acceleration at the centroid of the reinforced zone from the equation

$$a_c = \left(1.45 - \frac{a_{max}}{g} \right) a_{max} \tag{6.46}$$

3. Calculate the dynamic soil thrust from

$$\Delta P_{AE} = 0.375 \frac{a_c \gamma_b H^2}{g} \tag{6.47}$$

where

γ_b = unit weight of backfill.

4. Calculate the inertial force acting on the reinforced zone

$$P_{IR} = \frac{a_c \gamma_r H L}{g} \tag{6.48}$$

where

γ_r is the unit weight of reinforced zone.

5. Add P_{AE} and 50 % of P_{IR} and check the external stability. FS (Seismic) ≥ 75 % FS (Static).

Internal Stability: Internal stability is evaluated as follows:

1. Determine the pseudo-static inertial force acting on the potential failure zone,

$$P_{IA} = \frac{a_c W_A}{g} \tag{6.49}$$

where

W_A is the weight of the failure mass (Figure 6.17).

2. Determine the share of each reinforcement layer from P_{IA}, according to its resistance area (this is the earthquake-induced tensile force for each reinforcement layer).
3. Determine the total tensile force for each layer as the sum of the dynamic and static components.
4. Check that the reinforcement allowable tensile strength > 75 % of the total tensile force for each layer.
5. Check the length of the reinforcement so that the FS against pullout failure > 75 % FS (static conditions).

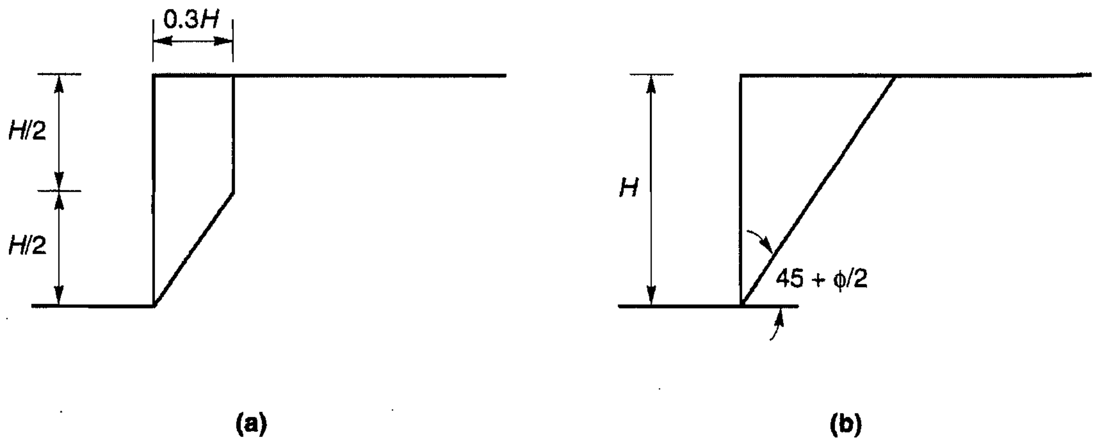


FIGURE 6.17 *Critical potential failure surfaces for evaluation of internal seismic stability of reinforced earth walls: a) inextensible reinforcement; b) extensible reinforcement*

6.8 Seismic Stability of Slopes and Dams

Slopes, embankments and dams may be damaged or may even fail due to earthquake induced shaking of the ground. Landslides often occur in earthquakes and dam failures have also been reported. There is no doubt that earthquakes can pose a serious threat to the stability of slopes and can induce significant damage. The damage manifests itself in the form of slides, slumping, cracks and permanent deformations.

6.8.1 Mechanisms of Seismic Effects

The mechanism leading to slope failures can be attributed to two factors: the earthquake induced forces and stresses; and the radical structural change of the soil that may be brought about by these seismic stresses.

The first effect is present even in soils that do not experience any basic change as a result of the shaking such as stiff clay, gravel or dense, coarse sand. In this case, some movement, could be substantial, of the slope occurs when the total stress exceeds the strength available. On the other hand, fine, loose, saturated sands may undergo a complete change of character when they liquefy. Liquefaction may occur in a sizeable bulk of soil or only in narrow seams and lenses of liquefiable material enclosed in relatively impermeable deposits. The liquefaction potential of loose, saturated sands is well recognized but similar abrupt structural changes could also be caused by earthquakes in some highly sensitive clays such as the Canadian Leda clay.

6.8.2 Evaluation of Seismic Slope Stability

The stability of slopes is influenced by many factors, and a complete slope stability evaluation must consider the effects of each factor. Geological, hydrological, geometrical and material characteristics are needed to reliably perform both static and seismic slope stability analyses.

The seismic stability of a slope is strongly influenced by its static stability because slopes with low factors of safety against failure under static conditions need low additional dynamic stresses to reach yield. Therefore, the factor of safety of any slope under static conditions must be significantly greater than 1.0 to accommodate seismic demands. The acceptable value of the factor of safety depends on the uncertainty in the model used for the analysis, the soil parameters and the magnitude and duration of seismic excitation, in addition to the potential consequences of slope failure.

An analysis of seismic stability of slopes has to consider the effects of dynamic stresses induced by earthquake shaking; and the change in the strength and stress-strain behaviour of the slope materials due to the seismic loading. These effects may lead to yield and plastic deformations due to inertial or weakening effects. The inertial effects occur when the earthquake-induced dynamic stresses reach the shear strength of the soil (that may remain constant), producing slope deformations. The weakening effects occur when the soil is weakened due to the earthquake loading (liquefaction or softening) and cannot remain stable under earthquake-induced stresses. When the available shear strength becomes smaller than the static shear stress required to maintain equilibrium, flow failures occur. Deformation failures occur when the shear strength of a soil is reduced below the earthquake-induced (dynamic) shear stresses.

The potential of a flow slide is commonly evaluated by conventional static slope stability analyses using soil strengths based on end-of-earthquake conditions.

In a typical analysis, the following procedure is used:

1. the liquefaction potential is calculated at all points on a potential failure surface;
2. Residual strengths are assigned to the failure surface portions with factor of safety against liquefaction < 1;
3. If FS against liquefaction > 1, strength values are based on the effective stresses at the end of the earthquake; and
4. Using these strength values, conventional limit equilibrium slope stability analyses are performed to calculate an overall FS against flow sliding. If the overall FS is less than 1, flow sliding is expected.

A number of techniques have been developed for the analysis of seismic inertial effects on slopes. These techniques differ in the way the earthquake motion and the dynamic response of the slope are modelled.

The knowledge of seismic forces makes it possible to examine the stability of the embankment approximately using the so-called pseudo-static approach and to establish the deformations that seismic forces produce. However, experience has shown that pseudo-static analyses can be unreliable for soils that build up large pore pressures or show more than 15 % degradation of strength due to earthquake shaking. Pseudo-static analyses produced factors of safety well above 1 for a number of dams that later failed during earthquakes. These cases illustrate the inability of the pseudo-static methods to evaluate the seismic stability of slopes.

Because of the difficulty in the assignment of appropriate pseudo-static coefficient, the use of this approach has decreased. Methods based on evaluation of permanent slope deformation are being used increasingly for seismic slope stability analysis.

6.8.3 Evaluation of Seismic Deformations of Slopes

In practice, the dynamic response of earth dams and embankments is usually computed using equivalent linear analyses. These analyses are conducted in terms of total stresses and thus the effects of the seismic porewater pressures are not accounted for. Also, these analyses fail to predict the permanent deformation as they assume elastic behaviour. Therefore, these analyses can only predict the distribution of accelerations and shear stresses in the embankment and semi-empirical methods are usually used to estimate the permanent deformations and porewater pressures using the acceleration and stress data (Seed et al. 1975). A detailed review of these methods is given in Finn (1993).

6.8.3.1 Newmark Sliding Block Analysis

The serviceability of a slope after an earthquake is controlled by deformations. Therefore, analyses that predict slope displacements provide a more useful indication of seismic slope stability.

The Newmark method (Newmark 1965) is the most common approach used to predict seismic slope displacement. In this method, the behaviour of a slope under earthquake-induced accelerations is given by the displacement of a block resting on an inclined plane (Figure 6.18a). At a particular instant of time, the horizontal acceleration of the block will induce a horizontal inertial force, k_hW (Figure 6.18b). As k_h increases, the dynamic factor of safety decreases, and there will be some positive value of k_h that will produce a factor of safety of 1.0.

This coefficient, termed the yield coefficient, k_y, corresponds to the yield acceleration, $a_y = k_y g$. The yield coefficient is given by

$$k_y = \tan(\phi - \beta) \tag{6.50}$$

where

ϕ is the angle of friction of the slope material (assuming purely frictional soil) and β is the slope angle . When a slope is subjected to a pulse of acceleration that exceeds its yield acceleration, it will undergo some permanent deformations.

Using the Newmark approach, the total relative displacement, d_{rel}, of the slope can be given by

$$d_{rel} = \frac{1}{2}(A - a_y)\Delta t^2 \frac{A}{a_y} \tag{6.51}$$

where

A is the amplitude of a rectangular pulse acceleration greater than the yield acceleration and Δt is its duration. Equation 6.51 shows clearly that the total relative displacement depends strongly on both the amount by which and the duration of the acceleration that exceeded the yield acceleration.

Using the rectangular pulse solution, Newmark related single-pulse slope displacement to peak base velocity, v_{max}, by

$$d_{rel} = \frac{v_{\max}^2}{2a_y}\left(\frac{1 - a_y}{A}\right) \tag{6.52}$$

Newmark found that a reasonable upper bound to the permanent displacements produced by several earthquake motion normalized to peak accelerations of 0.5g and peak velocities of 0.76 m/s was given by

$$d_{\max} = \frac{v_{\max}^2}{2a_y}\frac{a_{\max}}{a_y} \qquad (a_y / a_{\max} \geq 0.17) \tag{6.53}$$

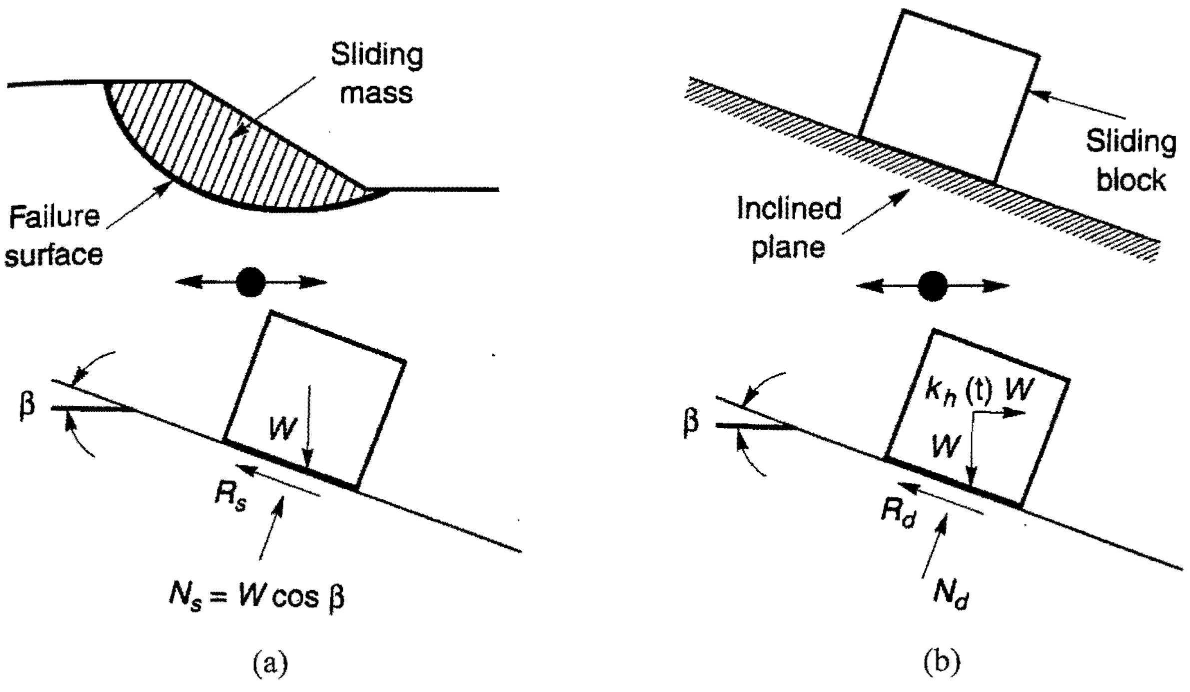


FIGURE 6.18 *a) Analogy between potential landslide and a block resting on inclined plane; b) Forces acting on a block resting on an inclined plane*

6.8.3.2 Nonlinear Analysis

Nonlinear methods of analysis were also developed to calculate the seismic response of slopes accounting for the effects of the intrinsic nonlinear behaviour of the soil. Although some of these procedures include elaborate representation of the basic behaviour of the soil, their reliability and suitability are limited due to the complexity and the need for some soil parameters that are not usually measured in field or laboratory testing. Finn (2000) reviewed the main nonlinear procedures used in current practice and outlined their advantages and limitations.

6.9 Seismic Design of Foundation

The soil-structure interaction effects that take place during the seismic excitation govern the seismic response of foundations. Except for cases where liquefaction occurred, or sensitive clays lost their strength under cyclic loading, foundations failures during earthquakes are rare. The strength and stiffness of the foundation elements in regard to transient dynamic loading are a function of the rate of loading. In general, the stiffness, and for most soils, the strength, increase with the rate of loading.

6.9.1 Bearing Capacity of Shallow Foundations

The effect of the inertia forces within the soil mass is to generate shear stresses that would reduce the capacity. Several studies have shown that the reduction in the bearing capacity due to soil inertia is not more than 15 % to 20 % for $k_h \leq 0.3$ (Shi and Richards, 1995). Therefore, the main seismic consideration in the design of foundations would be the effects of eccentric and/or inclined loading conditions due to the induced horizontal inertial seismic loads from the superstructure.

To account for the effects of horizontal seismic forces on the bearing capacity of a footing, the resultant inclined eccentric load is considered in the calculation of the bearing capacity of the footing. In this case, a reduced effective footing width and load inclination factors are used in the analysis as described in Chapter 10 of this manual. Because of the short duration of the seismic loads, a smaller factor of safety can be adopted for the seismic design of foundations.

6.9.2 Seismic Design of Deep Foundations

The response of deep foundation to earthquake loading is quite complex. The main factors that govern the seismic behaviour of deep foundations are the interactive soil-pile forces and the loss of the soil support to the piles. For piles in a group, the pile-soil-pile interaction effects add to the complexity of the problem.

The proper evaluation of the seismic response characteristics of pile groups requires dynamic analyses that require the use of computer programs. The main features that should be considered in these analyses are the nonlinear behaviour of the soil adjacent to the piles, the slippage and separation that occur at the soil-pile interface and the energy dissipation through different damping mechanisms. These analyses can be used to calculate the response of the foundation system to the seismic loading, and the capacity of the foundation can be evaluated based on some ultimate displacement considerations.

6.9.3 Foundation Provisions

The National Building Code of Canada, NBCC (2005) includes the following provisions to ensure matching the foundation seismic capacity with the capacity of the seismic force resisting system (SFRS).

1. Foundations shall be designed to resist the lateral load capacity of the SFRS, except that when the foundations are allowed to rock, the design forces need not exceed 0.5 R_dR_o times those determined in Sentence 4.1.8.7.(1).
2. The design of the foundations shall be such that they are capable of transferring the earthquake loads and effects between the building and the ground without yielding and without exceeding the capacities of the soil and rock.
3. For cases where $I_EF_aS_a(0.2)$ is equal to or greater than 0.2, the following requirements shall be satisfied:
   a. Piles or pile caps, drilled piers, and caissons shall be interconnected by continuous ties in not less than two directions.
   b. Piles, drilled piers, and caissons shall be embedded a minimum of 100 mm into the pile cap or structure.
   c. Piles, drilled piers, and caissons other than wooden piles shall be connected to the pile cap or structure for a minimum tension force equal to 0.15 times the factored compression capacity of the pile.
4. At sites where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, basement walls shall be designed to resist earthquake lateral pressures from backfill or natural ground.
5. At sites where $I_EF_aS_a(0.2)$ is greater than 0.75, the following requirements shall be satisfied:
   1. A pile, drilled pier, or caisson shall be designed and detailed to accommodate cyclic inelastic behaviour when the design moment in the element due to earthquake effects is greater than 75 % of its moment capacity.
   2. Spread footings founded on soil defined as Site Class E or F shall be interconnected by continuous ties in not less than two directions.
6. Each segment of a tie between elements shall be designed to carry by tension or compression a horizontal force at least equal to the greatest factored pile cap or column vertical load in the elements it connects multiplied by a factor of 0.15 $I_EF_aS_a(0.2)$, unless it can be demonstrated that equivalent restraints can be provided by other means.
7. The potential for liquefaction and the consequences, such as significant ground displacements and loss of soil strength and stiffness, shall be evaluated based on Ground Motion Parameters and shall be taken into account in the design of the structure and its foundations.

7 Foundation Design

7 Foundation Design

7.1 Introduction and Design Objectives

The basic purpose of foundations (shallow and deep) is to safely and adequately transfer load effects, from and acting on any given structure, to the ground. The term ground is general; it includes both soil and rock. Foundation design essentially involves two basic considerations

- The foundation unit(s) must not collapse (i.e., not induce overall shear failure of the supporting ground); and
- Post-construction settlement of the foundation unit(s) must be within tolerable limits.

As discussed in Chapter 8, the first consideration involves Ultimate Limit States (ULS), and the second consideration involves Serviceability Limit States (SLS).

The primary objectives of engineering design are safety, serviceability, and economy. Safety and serviceability can be improved by increasing the design margins or levels of safety to reduce the probability of failure. However, this generally increases costs. Considerations of overall economy in design involve balancing the increased cost associated with increased safety (and improved performance) against the potential losses (costs, lives and other factors) that could result from unsatisfactory performance or failure. The basic design criterion is that the resistance of the system must be greater than the imposed load effects, while achieving an acceptable or required level of safety and performance.

7.2 Tolerable Risk and Safety Considerations

Design must assure an acceptable risk or a required level of safety; but how does one rationalize what is an acceptable or tolerable level of risk?

The probability of failure that is associated with a given design needs to be compatible with the level of risk that people (i.e., society) are willing to accept in specific situations or from natural and constructed works. This is referred to as tolerable risk. Tolerable risk refers to a willingness to live with a risk so as to secure certain benefits, and in the confidence that risk is being properly controlled or managed.

The specified desired level of safety for design is defined by relevant jurisdictional codes of practice (e.g., the National Building Code of Canada (NBCC), the Canadian Highway Bridge Design Code (CHBDC) and others). Codes generally describe recommended good engineering design practice by defining a set of requirements, or provisions, that are aimed at achieving a minimum level of technical quality, and the desired or specified level of safety. Codes can be viewed as documents for the quality assurance of the design of engineering structures and facilities. Codes are legal documents and, as such, compliance with the code is required by law. A code represents a legal means to facilitate sound, rational design decisions to be made by engineers. It assists the engineer in making the "right" decisions that lead to sufficiently safe structures. A good code does not necessarily lead to no failures, but leads to design situations where the number of failures are acceptable or the risk level is tolerable.

7.3 Uncertainties in Foundation Design

Significant and varying degrees of uncertainty are inherently involved in the foundation design process. Allowances during design must be made to account for these uncertainties. The sources of uncertainty can be grouped into four main categories:

1. Uncertainties in estimating the load effects
2. Uncertainties associated with inherent variability of the ground
3. Uncertainties in evaluation of geotechnical material properties
4. Uncertainties associated with the degree to which the analysis represents the actual behaviour/response of the foundation, structure, and the ground that supports the structure.

The above uncertainties involve both structural and geotechnical aspects and other considerations that contribute to the overall risk. Standard design philosophies and procedures generally take uncertainty into account through the application of specified safety factors to manage risk satisfactorily. In working (allowable) stress design this is handled by the overall global factor of safety; whereas in limit states design, the use of separate partial factors on loads and resistances are used (refer to Chapter 8). Natural ground variability and evaluation of geotechnical properties usually constitute the greatest uncertainty, commensurate with the complex geological processes involved with the deposition and formation of soil and rock.

In contrast, gross errors including human errors or omissions that occur in practice are not quantified or taken into account through safety factors in design. These errors are usually handled by, or mitigated through, quality control and quality assurance programs, and independent third party reviews on larger projects. It is noted that gross or human errors are probably responsible for most of the failures that occur.

7.4 Geotechnical Design Process

The geotechnical design process, as it relates to foundation design, is schematically summarized in Figure 7.1. The design process starts off with the project description (e.g., a building with specific capacity and serviceability requirements based on the client's needs). A basic design issue, from the perspective of geotechnical engineers, is related to determining the most appropriate type and size of foundation units.

To assist in the design process and to ensure a specified level of safety and compliance with a minimum specified level of technical quality, engineers refer to a jurisdictional code of practice. The purpose of codes is to assist engineers in making appropriate design decisions. Codes may also provide general guidance for site investigation requirements. From an interpretation of the results from the investigation, geotechnical engineers formulate a geotechnical model of the site in terms of stratigraphy, soil and groundwater conditions, and engineering properties. Codes and their reference documents also usually provide guidance for the choice of appropriate geotechnical parameters, present a discussion on appropriate theory and calculation models or equations for geotechnical resistances, and specify load combinations and load effects for design.

The geotechnical parameters are dependent on many factors and are subject to significant inherent variability and uncertainty. There is no unique answer to questions associated with the shear strength and deformation parameters that are the most appropriate for design purposes. Depending on past experience and judgment, different engineers could arrive at and use different values of shear strength or compressibility, even for the same site.

The selection of characteristic design values of soil and rock properties needs to account for the following issues:

- Geological and other background information including data from previous projects
- Inherent variability of the ground and its properties

- Extent or zone of influence in the ground that contributes to overall behaviour and performance of the ground under load effects for possible limit states or failure modes
- Effect of construction activities on in-situ ground properties and characteristics
- Influence of workmanship on constructed or improved ground
- Scale effects and possible differences between the results of discrete small sized laboratory and field tests relative to the overall ground mass due to factors such as:
  - presence of fissures, joints and other planes/zones of weakness
  - testing rate effects
  - stress path effects
  - brittleness or ductility (stress-strain response)
- Other factors considered to be relevant for the site and project.

In summary, the selection of the characteristic value for design should appropriately take into account all factors that influence the property or parameter under consideration. The selection of suitable characteristic values, therefore, requires engineering judgment and experience. Additional discussion on characteristic values for design is presented in Chapter 8.

The selection of the procedure used to determine ultimate geotechnical resistances will be influenced by the scope of the site investigation and the complexity of subsurface conditions at the site. The calculation procedure or design equation for geotechnical resistance is usually based on theories of elasticity, plasticity and other relevant theoretical frameworks. In addition, ultimate bearing capacity and many geotechnical design parameters are frequently selected on the basis of empirical correlations to in-situ tests such as the Standard Penetration Test (SPT), piezocone penetration test (CPT), pressuremeter test and other in-situ tests. These correlations involve inherent uncertainty and may be site specific. Such empirical correlations need to be applied judiciously and with caution. Some people suggest that the geotechnical community should reduce, if not avoid, reliance on these types of correlation models. Nevertheless, traditional, empirical correlations are expected to remain in use and will continue to be an integral part of design practice for some time. This is because the geotechnical professional heritage is embodied in empirical correlations.

A sound, basic design approach requires a thorough understanding of the key design issues, of the geological setting and geotechnical conditions, and of the interaction between them. In most cases, a good understanding of these factors is as important, if not more so, as the analytical/numerical methods used for analysis and calculation. It is important to initially capture the essence of the problem, and then proceed with appropriate, simple analysis followed by an increasing level of sophistication and complexity, as required or as the project demands.

For the calculation model and load effects, codes specify safety factors aimed at producing a design with an acceptable risk or level of safety. The safety factors specified help to account for and to mitigate uncertainties in the design process, such as those related to loads, material properties, design equations, and inherent variability in the ground conditions at the site. For large, complex and special projects that involve a high degree of risk (e.g., long-span bridge) a comprehensive site investigation may be able to provide sufficient data for the geotechnical parameters for strata at various depths to be described in terms of a mean and standard deviation. If sufficient data are available to describe adequately both loads and resistances, a complete or fully probabilistic method, involving reliability theory, may be used for design and for risk management.

As shown in Figure 7.1, the geotechnical model of the site, calculation model, and load effects are considered in the geotechnical analysis of load carrying capacity and settlement of the foundation. The results from the analysis, when appropriately tempered or modified by engineering judgment and experience, are then used in the decision making process as to what constitutes the most appropriate type and size of foundation unit for the building.

7.5 Foundation Design Methodology

A detailed flow chart for the design of foundations is shown on Figure 7.2. In many cases, the flow chart can be simplified depending on the project requirements. However, the figure illustrates the key factors and interaction that affect the design and selection of the most appropriate choice of foundation for a given site and project.

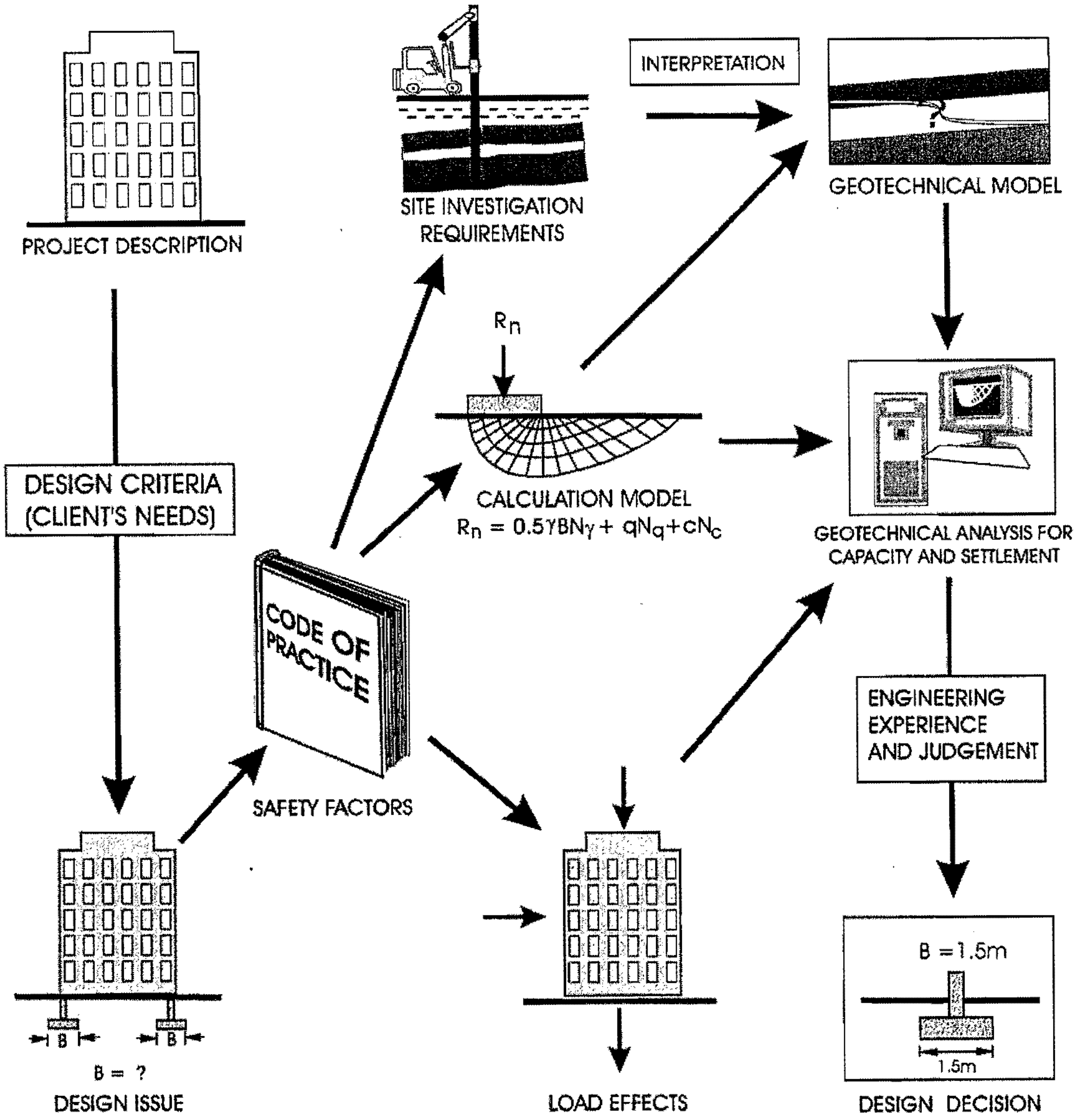


FIGURE 7.1 *Components of foundation design and role of codes of practice (after Ovesen 1981, 1993 and Becker 1996a).*

An important aspect of the flow chart (that is inherent to limit states design methodology) is the distinct and explicit separate treatment of ultimate and serviceability limit states. Although the traditional working (allowable) stress design approach also considers both ultimate capacity and settlement, the separation or distinction between them was not clear or evident. For example, the traditional global factor of safety of three in working (allowable) stress design often is used to limit settlements to acceptable values, while at the same time to account for uncertainties associated with applied loads and ultimate geotechnical bearing capacity. The separate and distinct treatment of ultimate capacity and settlement (serviceability) are key aspects and form the kernel concept of limit states design that is all too frequently missed, or not well understood or appreciated by foundation engineers. Additional information and discussion on limit states design is provided in Chapter 8.

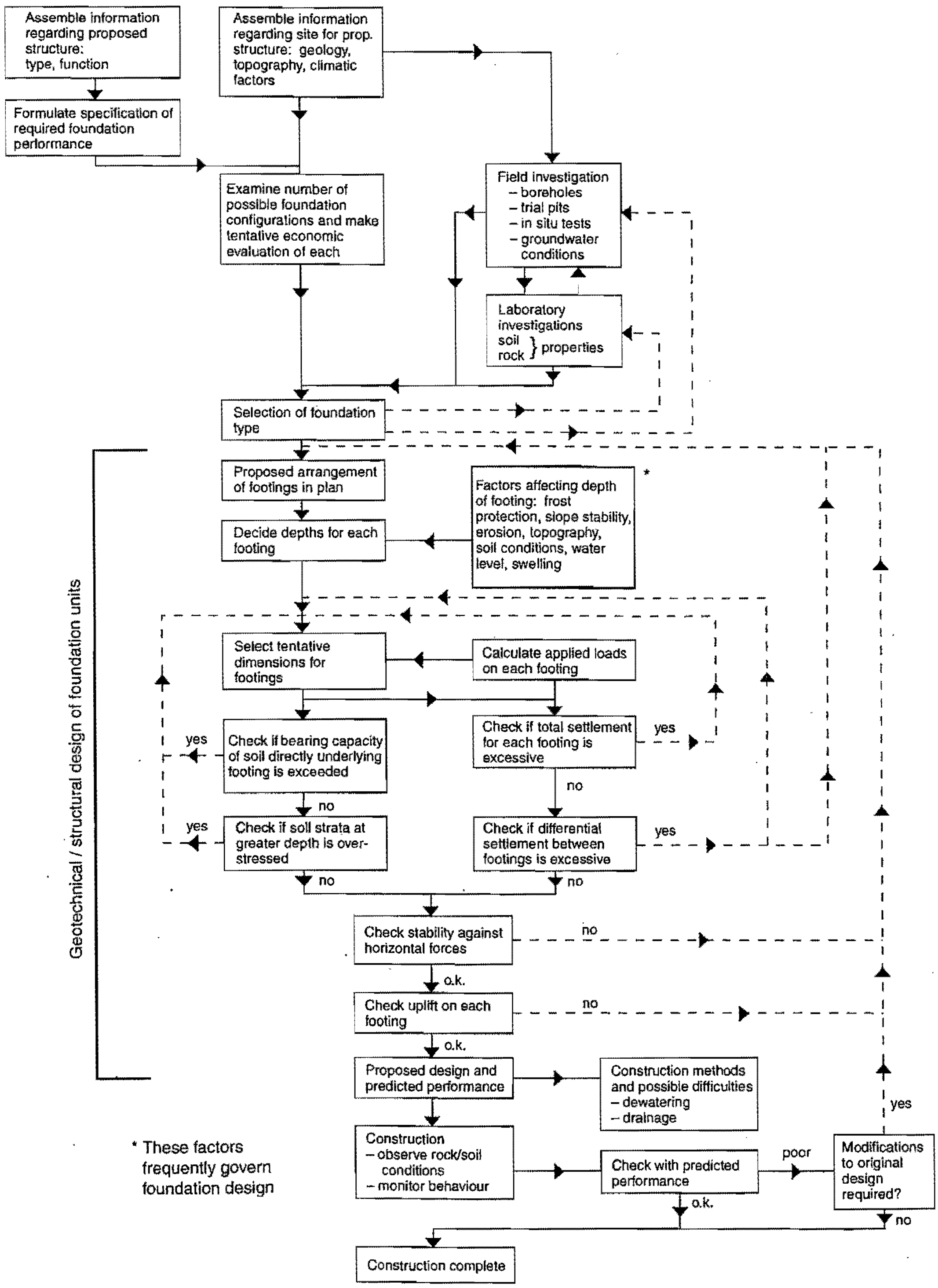


FIGURE 7.2 *Flow diagram for design of foundations (from NBCC (2005) - User's Guide)*

The key aspects of the design flow chart are:

- Assimilation of all relevant geotechnical and structural information and data
- Appropriate field and laboratory investigation to define the geotechnical model and characteristic design values.
- Identification of all possible foundation limit states or "failure" mechanisms that would result in unsatisfactory

performance. The key geotechnical ultimate limit states (ULS) are bearing capacity, sliding, overturning, uplift and excessive foundation deformation that would cause a ULS condition to occur in the structure. For serviceability limit states (SLS), the main consideration is deformation (in terms of settlement and horizontal displacement, vibration effects and others).
- Checking (through appropriate analysis) of each identified limit state to ensure that they either would not exist, or are within acceptable levels of risk (probability of occurrence).

7.6 Role of Engineering Judgment and Experience

Engineering judgment and experience are, and always will be, an essential part of geotechnical engineering; they are vital for managing safety (risk) of geotechnical structures. There will always be a need for judgment, tempered by experience, to be applied to new technologies and tools. Many aspects of geotechnical design are heavily reliant on engineering judgment and experience.

The spirit of the limit states design concept, as it was originally conceived, is particularly important in geotechnical engineering. The proper identification of potential modes of failure or limit states of a foundation, which is the first step in design, is not always a trivial task. This step generally requires a thorough understanding and appreciation of the interaction between the geological environment, loading characteristics, and foundation behaviour.

Reasonable analyses can be made using relatively simple models if the essence of geotechnical behavior and soil-structure interaction is captured in such models. There must also be a sufficient data and experience base to calibrate these models. Empirically based models are only applicable within the range of specific conditions reflected or included in the calibration process. Extrapolation beyond these conditions can potentially result in erroneous predictions of performance.

In summary, engineering judgment and experience play an integral role in geotechnical engineering analysis and design. Uncertainties in loads, material strengths (resistance), models, identification of potential failure modes or limit states, and geotechnical predictions all need to be considered collectively in controlling or ensuring an adequate level of safety in the design. The role of the geotechnical engineer through his or her judgment and experience, and that of others, in appreciating the complexities of geotechnical behavior and recognizing the inherent limitations in geotechnical models and theories is of considerable importance. The management of safety (risk) in geotechnical engineering design is distributed amongst the many aspects of the overall design process, including experience and judgment.

7.7 Interaction Between Structural and Geotechnical Engineers

Geotechnical resistance and reaction are a coupled function of applicable geotechnical parameters and of applied loading effects. Consequently, close and effective communication and design interaction between structural and geotechnical engineers need to take place to assure compatibility with the various design criteria, and achievement of desired performance and economy. Although this interaction and effective communication should occur for all classes of problems, it is especially important, if not essential, for more complex soil-structure interaction considerations where the design procedure involves, or is based on a modulus of subgrade reaction (vertical or horizontal). Examples include horizontal deformation and capacity of piles, retaining walls and raft/floor slab foundations. Additional discussion is presented in Section 7.7.1.

Some codes, such as the Canadian Highway Bridge Design Code (CHBDC), formally require that appropriate design interaction and communication occur between geotechnical and structural engineers. This legal requirement of such design interaction is an important precedent and step towards safe, economical design of foundations.

7.7.1 Raft Design and Modulus of Subgrade Reaction

In the design of a raft foundation, structural engineers usually ask for the value of the coefficient (modulus) of subgrade reaction of the supporting soil. Because of local variations in soil type under the raft, disturbances that take place during excavation and placement of steel reinforcing, and limitations of the theory, only approximate indications of the magnitude of the coefficient of subgrade reaction can be given. In addition, because the stresses from the raft affect the soil to considerable depth below bearing level, longer-term consolidation settlements may develop; these settlements also may vary, depending on the differences in soil compressibility existing at different points under the raft. Such considerations need to be taken into account by the geotechnical engineer when assessing appropriate values for subgrade reaction.

Unlike strength and compressibility, the modulus of subgrade reaction is not a fundamental soil property. Rather, it is a common design approach used by structural engineers to model the interface between the foundation soil and concrete footing (i.e., soil-structure interaction). The modulus of subgrade reaction is a number required by structural engineers to model the deformation and stiffness response of a footing (raft) on soil. The modulus of subgrade reaction is defined as:

$$k = q/\delta \tag{7.1}$$

where

k = modulus of subgrade reaction
q = applied bearing or contact pressure on footing
δ = settlement of footing under applied pressure q

The modulus of subgrade reaction, though simple in its definition, is a very difficult parameter to evaluate properly because it is not a unique fundamental property that is readily measured. Its value depends on many factors, including size and shape of footing (raft), type of soil, relative stiffness of footing and soil, duration of loading relative to the hydraulic conductivity of the loaded soil, and others. The value of modulus of subgrade reaction can also vary from one point to another beneath a footing or raft (e.g., centre, edge or corner) and can change with time, in particular for soils with low hydraulic conductivity such as clays.

Field plate load tests are commonly used to determine numerical values for the modulus of subgrade reaction. A database of numerical values and types of soil has been developed. Because the modulus value can change with size of footing, a one foot (300 mm) square footing has been adopted as the standard basis for comparison purposes, and frequently serves as the starting point for design. The technical literature cites typical values for the modulus of subgrade reaction, k_{v1}, (for a one-foot square plate) for a variety of soil types. Typical ranges in k_{v1} are summarized in Table 7.1. Appropriate design values for modulus of subgrade reaction generally decrease if the size of the loaded plate (or footing) is larger than one foot (300 mm) by one foot (300 mm). The manner in which the value of modulus of subgrade reaction decreases with increasing footing size varies with the type of foundation soil. Additional information is provided below, as well as by Terzaghi (1955), NavFac (1982) and Winterkorn and Fang (1975).

TABLE 7.1 *Typical Ranges In Vertical Modulus Of Subgrade Reaction*

Soil Type	k_{v1} (MPa/m) [1]
Granular Soils (Moist or Dry) [2]	
Loose	5 - 20
Compact Sand	20 – 60
Dense	60 – 160
Very Dense	160 – 300 [3]
Cohesive Soils	
Soft	< 5
Firm	5 – 10
Stiff	10 – 30
Very Stiff	30 - 80
Hard	80 – 200 [3]

Notes:

1. For a 1 ft. (300 mm) x 1 ft. (300 mm) plate

   For granular soils, $k_{vb} = k_{v1}\left(\frac{b+1}{2b}\right)^2$

   For strip footing on cohesive soil, $k_{vb} = k_{v1}/b$

   If the loaded area on cohesive soil is of width b and length mb, $k_{vb} = \frac{k_{v1}}{b}\left[\frac{m+0.5}{1.5\,m}\right]$

where

k_{vb} = modulus for actual footing dimension b
b = foundation width

When using the above expressions, care must be taken to ensure that the units are consistent. These equations were initially derived for b in units of feet. Therefore, when using b in meters, the expression (b+1) needs to become (b+0.3) and (m+0.5) becomes (m+0.15).

2. If below groundwater table, these values should be multiplied by 0.6.
3. Higher values to be used only if assessed on basis of adequate test results and settlement calculation.

Values for modulus of subgrade reaction can be derived from the results of plate load tests using elastic displacement theory as represented by:

$$\delta = Iqb(1-v^2)/E \qquad (7.2)$$

where

I = an influence factor that is dependent on geometry of footing and thickness of compressible soil relative to footing width
b = width of footing
v = Poisson's ratio (v = 0.5 for undrained condition and typically about 0.3 for drained conditions for most soils)
E = modulus of deformation (E_u if examining undrained condition and E' for drained condition)

Rearrangement of Equation (7.2) gives

$$q/\delta = E/Ib(1-v^2) = k_{vb} \qquad (7.3)$$

Therefore, if values of E are known for the soil within the zone of influence, beneath a footing of width b, reasonable estimates can be made for the modulus of subgrade reaction, k_{vb}, using Equation 7.3. Typical values for E are provided in references such as Bowles (1988), NavFac (1982) and many others.

It is generally considered that the use of settlement calculation is a more rational method of assessing modulus of subgrade reaction than is the use of adjusting typical values of k_{v1} for a one-foot square plate. The value of modulus of subgrade reaction for the footing or raft under consideration is the applied pressure at a given location divided by the settlement calculated at that location for the applied pressure (i.e., k=q/δ).

It is emphasized that values of k_{vb} as determined from extrapolation of plate bearing tests or from k_{v1} should be used with judgment and care. The deformation response of a smaller sized plate may not be representative of the response of the larger sized actual foundation because the zone of influence extends much deeper for the actual foundation. This aspect is especially important in ground with variable stratigraphy and engineering properties with depth, in particular for the case of softer soil at depth to which the zone of influence for a small plate would not extend. The results from the test plate would not reflect the response of the soft layer at depth. Further, the results from plate load tests on clays and clayey silts may be unreliable because the time associated with the testing may not permit complete consolidation (drainage of excess porewater pressure) of these fine-grained soils. An assessment of whether an undrained or drained condition prevailed during the test must always be made. For design, the test results obtained would need to be adjusted (corrected) as appropriate.

8 Limit States and Limit States Design

8 Limit States and Limit States Design

8.1 Introduction

The geotechnical engineering profession in Canada and elsewhere throughout the world is in the process of evaluating and incorporating limit states design (LSD) into codes of practice for geotechnical aspects of foundation engineering. A benefit of LSD for geotechnical aspects of foundation design is that it provides a consistent design approach/ philosophy between structural and geotechnical engineers. Information on the background and development of LSD for structures and for foundations is provided by Allen (1975), MacGregor (1976), Meyerhof (1982, 1984 and 1995), Duncan et al. (1989), Green (1989, 1991 and 1993), Ovesen and Orr (1991), Becker (1996a and b), Green and Becker (2000) and Becker (2003). In addition, the proceedings of international workshops and symposia, including DGS (1993), LSD 2000 and IWS Kamakura 2002 provide substantial information and discussion.

To date (i.e., early 2000's) geotechnical engineering practice largely continues to use traditional working (allowable) stress (WSD) design for foundations. However, most structural design is carried out using LSD concepts. Therefore, a significant degree of inconsistency exists in the design interaction between structural and geotechnical engineers, which could lead to different levels of safety and to errors. There is no basic reason why limit states design principles cannot be applied to the design of foundations. Ground (soil or rock) can be treated as an engineering material, albeit one that may exhibit considerable variability and deformability. Models can be developed to show how ground resists forces and deformations, and how ground can induce load on structures. The principles of engineering mechanics and of deformable bodies can be applied in conjunction with analytical procedures to analyse foundation units for serviceability and ultimate limit sates.

Both structural and geotechnical engineers have the common mandate of achieving a specified level of safety and minimizing repair and loss of function during the life of a structure. The design should also be efficient from an economic viewpoint. Economic advantage can be realized if all members-components of the structure are designed to a consistent appropriate level of safety or reliability. This objective is enhanced if both geotechnical and structural aspects of foundation design are based on the same design approach and concepts. Therefore, a strong motivation for the use of LSD in foundation engineering is the need, benefit and importance of a compatible design process between structural and geotechnical engineers. However, there are important technical benefits associated with the use of LSD for geotechnical aspects of foundation design. LSD has significant merit and advantages over the traditional WSD approach for foundation design (Becker 1996a). LSD can be viewed as a logical extension to WSD. It is considered that LSD will eventually become the general state of practice by geotechnical engineers for foundation design.

To date, some existing Canadian Codes such as the Canadian Highway Bridge Design Code (CHBDC 2000), the Ontario Highway Bridge Design Code (OHBDC 1983 and 1992), the National Building Code of Canada (NBCC 2005), the Canadian Standard Association (CSA) S472 Standard for Foundations in the Offshore Code (CSA 1992) have introduced or require LSD for foundations. Green and Becker (2000) and Becker (2003) provide a status of LSD in Canada for geotechnical engineering design practice.

8.2 What Are Limit States?

Limit states are defined as conditions under which a structure or its component members no longer perform their intended function. Whenever a structure or part of a structure fails to satisfy one of its intended performance criteria, it is said to have reached a limit state. A limit state is associated with unsatisfactory performance.

Limit states are classified into the two main groups of ultimate limit states and serviceability limit states.

Ultimate limit states (ULS) are primarily concerned with collapse mechanisms of the structure and, hence, safety. For foundation design, ULS consist of:

- exceeding the load carrying ability of the ground that supports the foundation (i.e., ultimate bearing capacity),
- sliding,
- uplift,
- overturning,
- large deformation of the foundation subgrade that leads to an uls being induced in the structure, and
- loss of overall stability.

Because of their relationship to safety, ULS conditions are designed for a low probability of occurrence that is consistent with the desired or specified level of safety and reliability.

Serviceability limit states (SLS) represent conditions or mechanisms that restrict or constrain the intended use, function or occupancy of the structure under expected service or working loads. SLS are usually associated with movements or deformations that interrupt or hinder the function (i.e., serviceability) of the structure. For foundation design, SLS generally consist of:

- excessive movements (e.g., settlement, differential settlement, heave, lateral movement, cracking, tilt),
- unacceptable vibrations, and
- local damage and deterioration.

SLS have a much higher likelihood or probability of occurrence than ULS. SLS may be viewed as those things that make life difficult, but are not necessarily dangerous.

The distinction between ULS and SLS may be better appreciated by the following example. A building that does not collapse under specified loading has performed satisfactorily against an ULS condition. However, if deformation has occurred to the extent that the owner cannot open doors to the building or if the floor and walls are severely cracked, then SLS have not been satisfied. Because the building did not collapse, safety has been assured; however, damage to the building no longer allows it to perform its intended use or occupancy (serviceability).

Allowable movements of foundations and structures depend on soil-structure interaction, desired serviceability, harmful cracking, vibration, and distortion restricting the use of a given structure. Empirical damage criteria are generally related to relative rotation (i.e., angular distortion, deflection ratio, or tilt of the structure). For superstructures, these criteria differ for frame buildings (bare or cladded), load-bearing walls (sagging or hogging), and other structures, depending on the differential settlement after the end of the construction. Additional information is provided by Burland et al. (1977) and Feld (1965).

The allowable (tolerable) movements and deformations of structures should be determined for each particular case. For common types of buildings and for some other types of engineering structures, tentative safe limits have been suggested as a guide (Bjerrum 1963 and Meyerhof 1982). Appropriate guides are also given in other parts of this Manual. However, these guidelines should not stand in the way of direct communication and interaction between geotechnical and structural engineers.

The loads that are applied to a foundation consist of permanent (dead) and transient (e.g., live, snow, wind) loads. The full values of live (transient) load effects do not necessarily need to be used in a calculation or analysis of magnitude of foundation settlement. Full or complete values of permanent load effects always need to be included; however, whether the total magnitude of live (transient) load effects needs to be used depends on the consolidation characteristics of the soils that exist within the zone of influent below the foundation.

For cohesionless soils, settlement estimates should be based on the maximum (dead and live) loads with an allowance for any dynamic effects. For fine-grained soils that have relatively low rates of consolidation settlement, the duration of the transient load effects is usually not sufficient for a substantial portion of consolidation settlement to take place under transient loading. In these cases, ignoring the transient load effects or using only a proportion of the total transient load in a settlement analysis may be appropriate. The appropriate proportion of total transient load effects for a given circumstances depends primarily on the duration of the applied transient loading relative to the coefficient of consolidation of the foundation soils. Although relevant Codes of Practice may specify or provide guidance as to suitable proportions for use in settlement analyses, this task is usually left to the discretion of geotechnical engineers. Settlement estimates for cohesive soils therefore, could be based on dead loads, plus a reduced load for live and other transient loads.

The effects of elastic displacements, shear distortions and permanent hysteresis effects that may be induced by transient loading effects should be considered and included in settlement analysis, as appropriate.

8.3 Limit States Design (LSD)

In essence, limit states design (LSD) involves the identification of all possible limit states or "failure" mechanisms, and the subsequent checking that the probability or likelihood of occurrence of each limit state identified will be within an acceptable or specified level of safety or reliability. The term "failure" is used here in the general sense of unsatisfactory performance. It does not necessarily mean rupture or collapse. The applicable, acceptable level of safety or reliability is usually defined by the target reliability index that is specified by governing codes.

Each potential limit state identified is considered separately, and through the design process its occurrence is demonstrated to be sufficiently improbable (or eliminated) or to be acceptable.

ULS conditions are checked using separate, partial factors on loads and on nominal (ultimate) geotechnical resistance. The values of these partial factors are specified by applicable codes and manuals (guidelines) for state-of-practice. The magnitudes of the partial factors are usually based on calibration to WSD (including engineering judgment) or on reliability theory, or a combination of both (Becker 1996a and b, Green and Becker 2000, Kulhawy and Phoon 2002, and Phoon et al. 2003). The magnitude of the specified partial factors serve as a means of risk management towards achieving the desired or target level of safety/reliability.

The SLS conditions are checked using working or service loads and unfactored geotechnical properties. In essence, a partial factor of one is used on all specified loads and on the characteristic values for deformation and compressibility properties of the ground. Geotechnical characteristic values are generally based on conservative (cautious estimate) mean values obtained from in-situ and laboratory tests. In this sense, the methodology of SLS calculation is virtually identical between LSD and WSD approaches.

The explicit distinction between safety (ultimate) and deformation (serviceability) analyses/calculations, and the classification of performance that flows from this distinction, reflect the kernel concept of limit states design. This distinct and explicit separate treatment of ULS and SLS is the essence and most important fundamental aspect of limit states design.

Although the traditional working (allowable) stress design approach considers ultimate capacity and settlement, the separation or distinction between them was not clear or evident. For example, the traditional global factor of safety of three in working (allowable) stress design often is used to limit settlements to acceptable values, while at the same time to account for uncertainties associated with applied loads and ultimate geotechnical resistance (capacity).

The separate, distinct treatment of ultimate capacity and settlement (serviceability) is the key aspect of limit states design that is all too frequently missed, or not well understood or appreciated by geotechnical engineers.

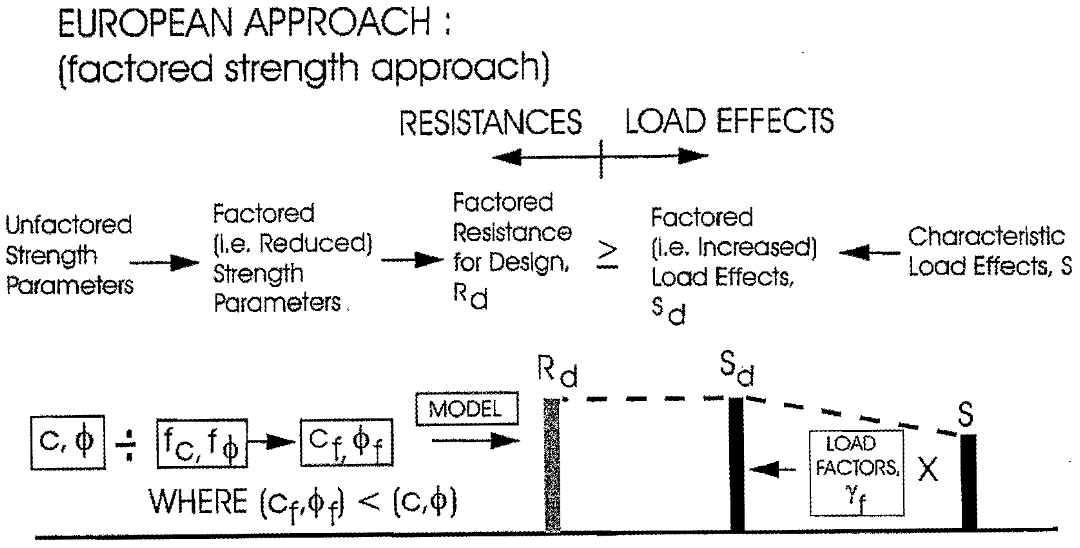
The historical development of geotechnical LSD has been described and summarized by Ovesen and Orr (1991), Meyerhof (1995), Becker (1996a) and others. The approaches to LSD have developed differently in North America and in Europe, mainly in the manner for calculating factored geotechnical resistances at ULS.

In the factored strength (European) approach, specified partial factors are applied directly to the geotechnical strength parameters of cohesion and angle of internal friction. The resulting factored strength parameters are then used in traditional equations/formulae for the direct calculation of factored geotechnical resistance at ULS for design. This is the approach advocated and required by Eurocode 7 (ENV 1991, 1994, 1997, Eurocode 7 (1987 and 1990)).

In North America, a factored resistance methodology, such as load and resistance factor design (LRFD), has become the standard approach. In this method, an overall specified resistance factor is applied to the calculated or assessed ultimate geotechnical resistance for each applicable limit state. The ultimate resistance is firstly calculated from "real" or unfactored (characteristic) strength parameters using traditional equations or formulae; the calculated ultimate resistance is then multiplied by a single, specified geotechnical resistance factor to obtain the factored geotechnical resistance at ULS for design.

Figure 8.1 summarizes the comparison of these two LSD approaches. The advantages and disadvantages of the two approaches are a subject of debate by geotechnical engineers throughout the world. The interested reader is referred to Becker (1996a) for a detailed discussion. For the purposes of this manual, the factored resistance approach is used because, as stated in Chapter 7, it forms the basis of many existing codes of practice currently in use in Canada and the United States.

It is noted that this LSD approach does not alter the methods for calculating ultimate geotechnical resistance (capacity). The calculations are performed according to the same traditional and classical methods that are familiar to all geotechnical engineers using working (allowable) stress (WSD) design. The key difference is the manner in which the design value is obtained and used. In WSD, a single global factor of safety is used; whereas in LRFD, several partial load and resistance factors are employed. The only difference in the execution of calculation for LSD design values is that the ultimate geotechnical resistance (capacity) is multiplied by a different (both in terms of rationale and magnitude) factor of safety.



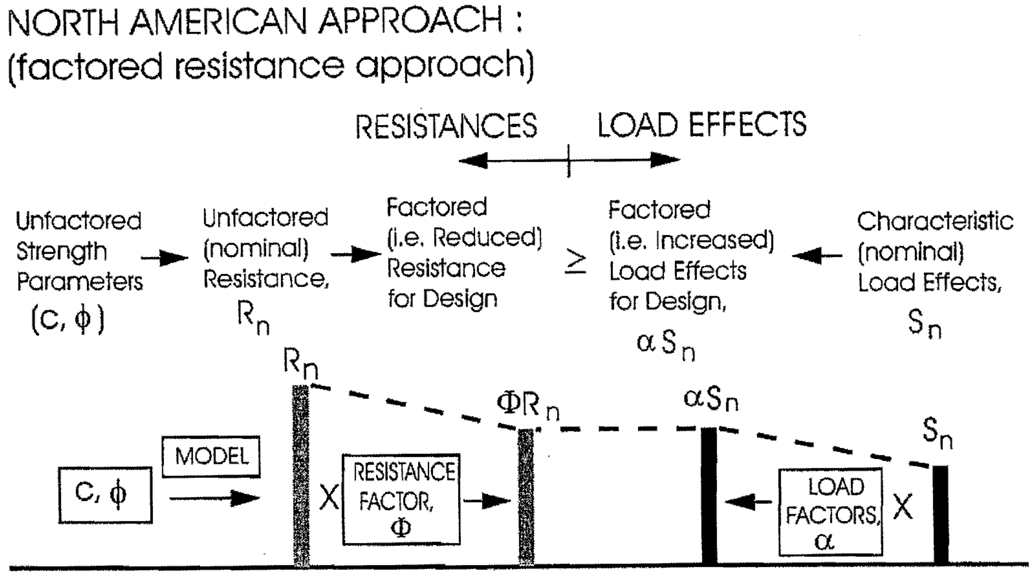


FIGURE 8.1 *Comparison of limit states design approaches for ultimate limit states (after Ovensen and Orr, 1991; Becker, 1996a)*

8.4 LSD Based on Load and Resistance Factor Design (LRFD)

Significant and varying degrees of uncertainty are inherently involved in foundation and other geotechnical design. Therefore in recent years, there as been a trend towards the use of reliability-based design and probabilistic methods in geotechnical engineering design. However, complete probabilistic design is difficult to apply reliably and appropriately, in particular in most practical geotechnical design situations, generally because of lack of statistically viable information. Complete probabilistic methods are also time-consuming and expensive, which makes them practical or suitable for large, special projects only. Because of these difficulties, simpler, yet probabilistically based design procedures have been developed. LRFD is an example where the partial factors have been based on or calibrated using probability and reliability concepts. For the consideration of ultimate limit states, the separate consideration of loads, materials and performance provides the opportunity for the design to be more responsive to the differences between types of loads, material types, fundamental behaviour of the structure and of the ground, and consequence of different modes of unsatisfactorily performance (i.e., limit states).

The basic design equation is:

$$\Phi R_n \geq \Sigma \alpha_i S_{ni} \tag{8.1}$$

where

ΦR_n	is the factored geotechnical resistance
Φ	is the geotechnical resistance factor
R_n	is the nominal (ultimate) geotechnical resistance determined through engineering analyses (e.g., bearing capacity) using characteristic (unfactored) values for geotechnical parameters or performance data (e.g., pile load test); it represents the geotechnical engineer's best estimate of resistance, that has appropriately taken into account all factors influencing resistance
$\Sigma \alpha_i S_{ni}$	is the summation of the factored overall load effects for a given load combination condition
α_i	is the load factor corresponding to a particular load; it accounts for uncertainties in loads

S_{ni} is a specified load component of the overall load effects (e.g., dead load due to weight of structure or live load due to wind); and

i represents various types of loads such as dead load, live load, wind load, etc.

The values for load factors (α), geotechnical resistance factors (Φ) and load combinations are specified by applicable codes (e.g., NBCC, CHBDC, AASHTO, etc.).

The load factors, α, are usually greater than one; they account for uncertainties in loads and their probability of occurrence. The resistance factors (or performance factors as they are sometimes called), Φ, are less than one and account for variabilities in geotechnical parameters and analysis uncertainties when calculating geotechnical ultimate (nominal) resistances.

The design equation can be visualized by inspecting the interaction of the probability distribution curves for resistance and load effects, as shown schematically on Figure 8.2. It should be noted that the resistance and load effects are assumed to be independent variables, which is approximately true for the case of static loading. The characteristic or nominal values for load effects (S_n) and resistance (R_n) do not necessarily need to be taken as the mean values of S and R, respectively. The nominal or characteristic values for design are related to the mean values as follows:

$$R_n = \frac{\bar{R}}{k_R} \quad \text{and} \quad S_n = \frac{\bar{S}}{k_S} \tag{8.2}$$

where

k_R is the ratio of mean value to nominal (characteristic) value for resistance; and
k_S is the ratio of mean value to specified (characteristic) value for load effects.

The factors of k_R and k_S are used to define characteristic values of design based on the mean values of the resistance and load distribution curves, respectively. Typically, k_R values are equal to or greater than one (i.e., $R_n \le \bar{R}$) and k_S values are less one (i.e., $S_n \ge \bar{S}$). The terms k_R and k_S are also referred to as bias factors by some researchers. The bias factor is one if the mean value is used as the characteristic value.

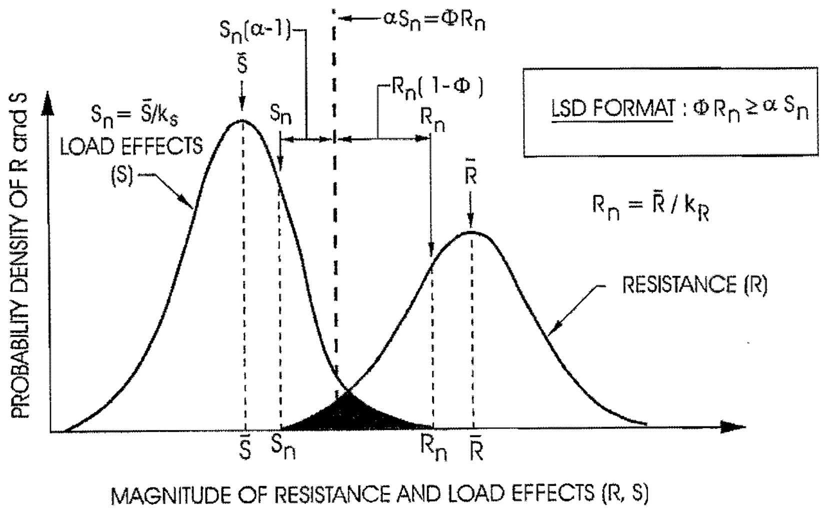


FIGURE 8.2 *Load and resistance factor design (LRFD)*

In practice, values for α and Φ are specified in codes. They are based on target values of reliability or acceptable probabilities of failure selected to be consistent with the current state-of-practice. In general, different values of α and Φ are provided for different limit states. Although values of α may differ between codes in various countries, load factors are typically in the range of 0.85 to 1.3 for dead loads and in the range of 1.5 to 2.0 for live and environmental loads. A load factor of less than 1.0 for dead loads is used when the dead load component contributes to the resistance against overturning, uplift, and sliding. Typical values of Φ range from about 0.3 to 0.9, depending on ground type, method of calculating resistance, and class of structure such as foundation type or retaining structure.

8.5 Characteristic Value

It is important to note that the load and resistance factors are interrelated to each other. That is, the value of α is dependent on the value of Φ, and vice versa. The values of α and Φ depend on the target reliability index for design, the variability of the parameters that affect loads and resistances, and the definition of their characteristic values. Load and resistance factors have been derived (calibrated) based on characteristic values that have been defined in a specific manner. Therefore, consistent sets of these factors must be used in design as per their intended purpose and specific evaluation. It is inappropriate to use a set of resistance factors (that have been derived for specific values of load factors) and directly use them with other load factors that have been taken from an unrelated source, or vice versa. For consistent and rational design in practice, the selection of a given characteristic value for geotechnical resistance needs to be made in the same manner as that used to derive the specified resistance factor. That is, if the mean value was used in the derivation of the resistance factor, the mean value of a given geotechnical property should be used in the calculation of geotechnical resistance. The use of the mean value or a value slightly different from the mean is frequently used in reliability analysis for the determination (calibration) of load and geotechnical resistance factors.

The key statistical parameters (i.e., the ratio of the mean value to characteristic value and the coefficient of variation) for geotechnical resistance depend on many factors, including site investigation method, quality and quantity of testing (laboratory and in the field), construction quality control, and method of analysis.

The selection of nominal or characteristic strength for design varies with local state-of-practice and with the training, intuition, background, and experience of the individual geotechnical engineer. Frequently, the mean value or a value slightly less than the mean is selected by geotechnical engineers as the characteristic value for design purposes. Eurocode 7 proposes a "cautious estimate" of the mean value for the characteristic value.

The geotechnical engineer selects representative (characteristic) geotechnical parameters based on the results of appropriate investigations (field and laboratory). Representative in this sense refers to the geotechnical engineer's best estimate of the likely values of parameters required for design. As discussed in Chapter 7 (Section 7.4), the selection of the characteristic value, for a given design situation, should appropriately take into account all factors that have influence on the parameter or property for the volume of ground (zone of influence) under consideration. The selection of appropriate characteristic values is assisted by engineering judgment and experience. In addition and as mentioned above, the geotechnical engineer should be cognizant of the interrelationship between resistance and load factors and characteristic value when selecting characteristic geotechnical parameters for design purposes. A cautious estimate of the mean value for the affected volume of ground (zone of influence) is generally considered to be a logical value to use for the characteristic value.

8.6 Recommended Values for Geotechnical Resistance Factors

The recommended resistance factors are specified in applicable codes and manuals of practice. Although the values recommended by various codes tend to be similar, there are some specific differences. For example, the values in the NBCC (2005) and CHBDC (2000) are shown in Table 8.1 and Table 8.2, respectively. The reliability index associated with these resistance factors ranges from 2.8 to 3.5, a range that is generally consistent with values commonly specified for the design of structures.

TABLE 8.1 *Geotechnical Resistance Factors for Shallow and Deep Foundations NBCC (2005)*

	Description	Resistance Factor, Φ
1.	**Shallow foundation**	
(a)	Vertical bearing resistance from semi-empirical analysis using laboratory and in-situ test data	0.5
(b)	Sliding	
	(i) based on friction ($c = 0$)	0.8
	(ii) based on cohesion/adhesion ($\tan\phi = 0$)	0.6
2.	**Deep foundation**	
(a)	Resistance to axial load	
	(i) semi-empirical analysis using laboratory and in-situ test data	0.4
	(ii) analysis using static loading test results	0.6
	(iii) analysis using dynamic monitoring results	0.5
	(iv) uplift resistance by semi-empirical analysis	0.3
	(v) uplift resistance using loading test results	0.4
(b)	Horizontal load resistance	0.5

TABLE 8.2 *Geotechnical Resistance Factors – CHBDC (2000)*

Shallow Foundations		Φ
Bearing Resistance		0.5
Passive Resistance		0.5
Horizontal Resistance (Sliding)		0.8
Ground Anchors (Soil or Rock)		
Static Analysis Tension		0.4
Static Test Tension		0.6
Deep Foundations – Piles		
Static Analysis	Compression	0.4
	Tension	0.3
Static Test	Compression	0.6
	Tension	0.4
Dynamic Analysis Compression		0.4
Dynamic Test Compression (field measurement and analysis)		0.5
Horizontal Passive Resistance		0.5

The AASHTO Code (1997 and 1998) specifies many more resistance factors than is provided by CHBDC and NBCC. For each class of foundation, AASHTO specifies resistance values that are to be used for different methods of calculation and geotechnical data. For example, a different value is given if the geotechnical data is based on Standard Penetration Testing (SPT), Pieco-cone Penetration Testing (CPT), or laboratory testing. As a result, the number of specified resistance factors in the AASHTO Code exceeds that of CHBDC by more than an order of magnitude.

Although there is a merit in what the AASHTO Code has done, the approach for both the CHBDC and NBCC was to keep the process simple, at least during the initial stages of transition between working stress design and limit states design. For the NBCC and CHBDC, it was felt that it is more important that the fundamental principles of limit states design for foundations be conveyed to and understood by geotechnical practitioners, as well as structural engineers designing the foundations. The initial transition should be as gradual and smooth as possible. Providing a myriad of partial factors that cover a large range of methods used in practice may not be conducive to better understanding and acceptance of the design method by geotechnical engineers, who are accustomed to using only a few values of global factor of safety. Refinements and level of sophistication and details can come later when more experience with limit states design for foundations has been gained. In addition, the existing geotechnical database in terms of bias factor, coefficient of variation and other statistical parameters need to be further developed and better understood before levels of refinement such as those included in AASHTO can be reliably developed for Canadian codes.

8.7 Terminology and Calculation Examples

The various codes tend to use slightly different terminology for LSD design values. When designing based on a given code, the geotechnical engineer needs to be cognizant of the specific terms and definitions that are specified by that code. For example, the NBCC Commentary L Foundations (2005) uses the following terms for expressing recommended geotechnical criteria for the design of the building structure, including its foundations.

Bearing pressure for settlement means the bearing pressure beyond which the specified serviceability criteria are no longer satisfied.

Factored bearing resistance means the calculated ultimate (nominal) bearing resistance, obtained using characteristic ground parameters, multiplied by the recommended geotechnical resistance factor.

Factored sliding resistance means the calculated ultimate (nominal) sliding resistance, obtained using characteristic ground parameters, multiplied by the recommended geotechnical resistance factor.

Factored pull out resistance (i.e., against uplift) means the calculated ultimate (nominal) pull out resistance, obtained using characteristic ground parameters, multiplied by the recommended geotechnical resistance factor.

CHBDC (2000) uses the following definitions.

Factored Geotechnical Resistance at ULS – the product of the geotechnical resistance factor and the geotechnical ultimate (nominal) soil or rock resistance.

Geotechnical Reaction at SLS – the reaction of the soil or rock at the deformation associated with a SLS condition.

Geotechnical Resistance at ULS – the geotechnical ultimate resistance of soil or rock corresponding to a failure mechanism (limit state) predicted from theoretical analysis using unfactored geotechnical parameters obtained from test or estimated from assessed values.

8.7.1 Calculation Examples

The following two examples demonstrate the simple calculation of design values for factored geotechnical resistance at ULS.

The basic equation for factored geotechnical resistance is ΦR_n where Φ is the geotechnical resistance factor and R_n is the ultimate (nominal) geotechnical resistance.

Shallow Foundation
An ultimate bearing capacity of 800 kPa has been calculated using the classical bearing capacity equation. For LRFD, the factored bearing resistance at ULS is 400 kPa (i.e., 0.5 x 800, where $\Phi = 0.5$ from Table 8.1).

Deep Foundation
A static pile load test has shown an ultimate axial capacity of 2,500 kN. The factored axial geotechnical resistance at ULS is 1,500 kN (i.e., 0.6 x 2,500, where $\Phi = 0.6$ from Table 8.1).

8.8 Working Stress Design and Global Factors of Safety

Working stress design (WSD) was one of the first rational design methods used in civil engineering. It has been the traditional design basis since it was first introduced in the early 1800's. WSD is also referred to as allowable stress or permissible stress design. The basis of the design is to ensure that throughout the structure, when it is subjected to the working or service applied load, the induced stresses are less than the allowable stresses. A single, global factor of safety is used, which collectively considers or lumps all uncertainty associated with the design process into a single value, with no distinction made as to whether it is applied to material strength and resistances or to load effects.

The assessment of the level of safety of the structure is made on the basis of global factors of safety, that were developed from previous experience with similar structures in similar environments or under similar conditions. The values of the global factor of safety selected for design reflect past experience and the consequence of failure. The more serious the consequence of failure or the higher the uncertainty, the higher the global factor of safety. Similar values of global factor of safety became customary for geotechnical design throughout the world. The ranges of customary global factors of safety are shown in Table 8.3.

TABLE 8.3 *Ranges of Global Factor of Safety Commonly Used for Foundation Design*

Failure Type	Item	Factor of Safety, FS
Shearing	Earthworks	1.3 to 1.5
	Earth retaining structures, excavations	1.5 to 2
	Foundations	2 to 3
Seepage	Uplift heave	1.5 to 2
	Exit gradient, piping	2 to 3
Ultimate pile	Load tests	1.5 to 2.0
Loads	Dynamic formulae	3

Note: Data after Terzaghi and Peck (1948, 1967).

A global factor of safety represents a relationship between allowable and applied quantities. Although this concept is simple and useful, it is also accompanied by difficulties and ambiguity. Problems arise when factors of safety are used without first defining them and understanding why they were introduced. A single global factor of safety would have unambiguous meaning if carefully prescribed standard procedures for selecting capacity, for defining loads, and for carrying out the analysis or calculations were always used in design. However, these steps are usually not well defined, or followed uniformly or consistently by all geotechnical engineers. In practice, different engineers will use different approaches and select different values of strength for design, even for the same site. For example, some engineers may use a mean value for strength, while others will use a much more conservative value such as minimum or lower bound values in measured strength. Therefore, for the same numerical value of global safety factor, the actual margin of safety can be very different. Further, the value of the global factor of safety tells us very little quantitatively as to the possibility or probability of failure.

The global factor of safety (FS) can be defined in many ways. The traditional FS is defined as the ratio of ultimate resistance (R_u) to the applied load (S_a):

$$FS = \frac{R_u}{S_a} \quad or \quad S_a = \frac{R_u}{FS} \tag{8.3}$$

For FS = 1, a limiting condition theoretically exists where the resistance equals the load effects (i.e., a state of failure).

The limitations of WSD and the use of a single global factor of safety have been discussed by Green (1989) and Becker (1996a). Despite all its apparent limitations, the global FS and WSD approach is a simple approach that has worked well in geotechnical engineering design. WSD has been the traditional design method for over 100 years. Consequently, an extensive database and experience have been assimilated over the years towards the development of good engineering practice. Improvements and refinements have been incorporated as the need arises. It would be foolish and inconceivable to ignore this substantial database and experience gained in WSD. It is noted that despite the shortcomings of WSD, the development of limit states design (in some codes using partial factors) has utilized the WSD experience for calibration purposes to produce designs with comparable levels of safety as those existing in previous design codes based on WSD.

Figure 8.3 shows the relationship between global safety factor, resistance factor and reliability index based on statistical assumptions for variability in bearing resistance (coefficient of variation equal to 0.3 and a ratio of mean to nominal value of 1.1) typical for shallow and deep foundations. An advantage of Figure 8.3 is that the reliability index may be more readily appreciated by geotechnical engineers who have considerable experience in using the traditional values of global safety factor. This may assist in bridging the gap, during the transitional stage, between the use of working stress design and limit states design.

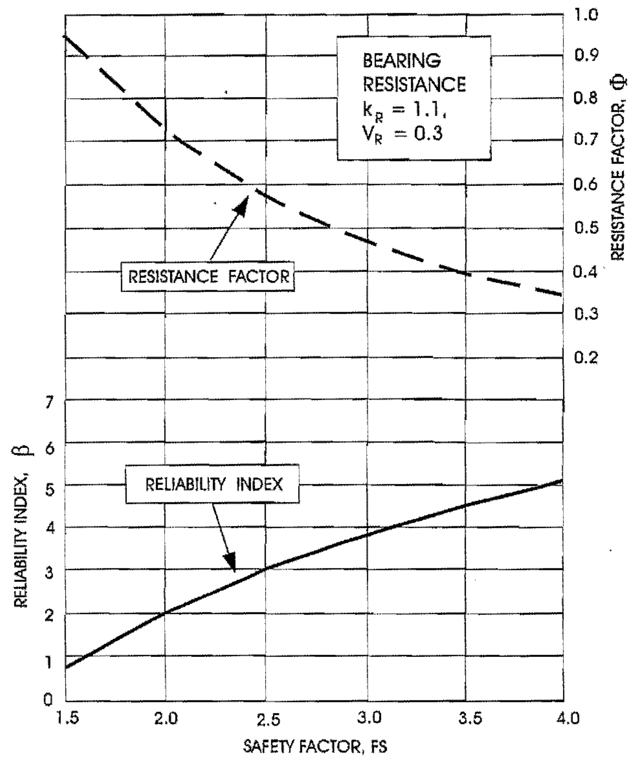


FIGURE 8.3 *Relationship between FS, Φ, and β for bearing capacity k_R = 1.1, V_R = 0.3 (from Becker, 1996b).*

9 Bearing Pressure on Rock

9 Bearing Pressure on Rock

9.1 Introduction

Rock is usually recognized as the best foundation material. Generally, bearing capacity failure and factored bearing resistance at ultimate limit states are rarely an issue for sound, intact rockmasses. 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. Such failure may be due to either deformation or failure of intact, weak rock or due to sliding failure along unfavourably oriented structural planes of weakness. A foundation on rock should be designed with the same care as a foundation on soil.

Failure of rock foundations may occur as the result of one of several mechanisms as shown in Figure 9.1 (from Franklin and Dusseault, 1989). The failure modes are described as:

- Bearing capacity failures, which occur when soil foundations are overloaded (Figures 9.1a and b), are uncommon in rock. However, such failures may occur beneath heavily loaded footings on weak clay shales.

- Consolidation failures are quite common in weathered rocks were the footing is placed within the weathered profile (Figures 9.1c and e). In this case, unweathered rock corestones are pushed downward under the footing load, because of a combination of low shear strength along clay-coated lateral joints and voids or compressible fillings in the horizontal joints.

- A punching failure (Figure 9.1d) may occur where the foundation comprises a porous rock type, such as shale, tuff and porous limestones (chalk). The mechanism comprises a combination of elastic distortion of the solid framework between the voids and the crushing of the rock where it is locally highly stressed (Sowers and Sowers, 1970). Following such a failure, the grains are in much closer contact. Continued leaching and weathering will weaken these rock types, resulting in further consolidation with time.

- Slope failure may be induced by foundation loading of the ground surface adjacent to a depression or slope (Figure 9.1f). In this case, the stress induced by the foundation is sufficient to overcome the strength of the slope material.

- Subsidence of the ground surface may result from collapse of strata undercut by sub-surface voids. Such voids may be natural or mining induced. Natural voids can be formed by solution weathering of gypsum or rocksalt and are commonly encountered in limestone terrain (Figure 9.1g). When weathering is focused along intersecting vertical joints, a chimney-like opening called a pipe will form, which may extend from the base of the soil overburden to a depth of many tens of meters. When pipes are covered by granular soils, the finer silt and sand components can wash downward into the pipes, leaving a metastable arch of coarse sand and gravel which may subsequently collapse (Figure 9.1h).

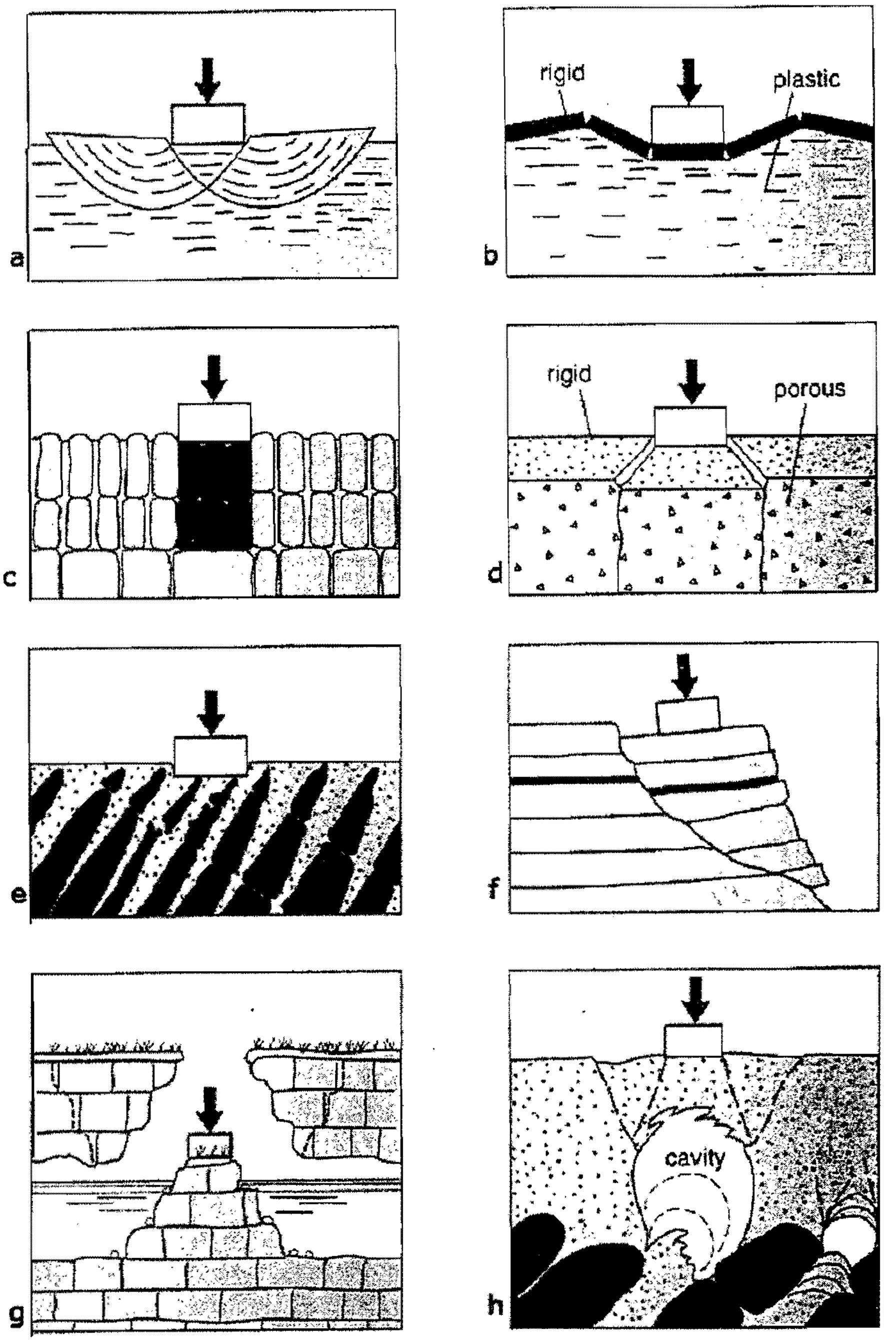


FIGURE 9.1 *Mechanisms of foundation failure (from Franklin and Dusseault, 1989; adapted from Sowers, 1976): a) Prandtl-type shearing in weak rock; b) shearing with superimposed brittle crust; c) compression of weathered joints; d) compression and punching of porous rock underlying a rigid crust; e) breaking of pinnacles from a weathered rock surface; f) slope failure caused by superimposed loading; g) collapse of a shallow cave; and h) sinkhole caused by soil erosion into solution cavities*

The methods proposed in this Manual for the determination of design bearing pressure on rock are suitable for various ranges of rock quality. The design bearing pressure is generally for serviceability limit states for settlements not exceeding 25 mm. The bearing pressure assessment is for relatively sound rock not subject to the special conditions shown in Figures 9.1b through h. Guidance on the applicability of the proposed methods is provided in Table 9.1.

TABLE 9.1 *Applicability of Methods for the Determination of Design Bearing Pressure on Rock depending upon Rockmass Quality*

Rockmass Quality	Basis of Design Method
Sound rock Rockmass with *wide* or *very wide* discontinuity *spacing*	Core strength (see Section 9.2)
Rockmass with closed discontinuities at *moderately close, wide* and *very wide spacing*	Core strength (see Section 9.2)
Low to very low strength rock Rockmass with *close* or *very closely spaced* discontinuities	Pressuremeter
Very low strength rock Rockmass with *very closely spaced* discontinuities	Soil mechanics approach

Note: Italicised terms are defined in Section 3.2. Preliminary estimates are provided in Table 9.3 and Section 9.3.

In all cases, field tests may also be used to assess the capacity and load-deformation characteristics of the rockmass, as discussed in Chapter 4.

9.2 Foundations on Sound Rock

For the purpose of this section, a rock mass is considered sound when the spacing of discontinuities is in excess of 0.3 m.

When the rock is sound, the strength of the rock foundation is commonly in excess of the design requirements, provided the discontinuities are closed and are favorably oriented with respect to the applied forces. Geotechnical investigations should, therefore, concentrate on the following foundation aspects:

- identification and mapping of all discontinuities in the rock mass within the zone of influence of the foundation, including the determination of the aperture of discontinuities;
- evaluation of the mechanical properties of these discontinuities, such as frictional resistance, compressibility, and strength of filling material; and
- identification and evaluation of the strength of the intact rock material.

Such investigations should be carried out by a person competent in this work, and following the guidelines set out in Chapters 3 and 4.

The final determination of the design bearing pressure on rock may be governed by the results of 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 (e.g., the rock surface is perpendicular to the foundation, the load has no tangential component, the rock mass has no open discontinuities), the design bearing pressure may be estimated from the following approximate relation:

$$q_a = K_{sp} \times q_{u\text{-core}} \tag{9.1}$$

where

q_a = design bearing pressure
$q_{u\text{-}core}$ = average unconfined compressive strength of rock (as determined from ASTM D2938).
K_{sp} = an empirical coefficient, which includes a factor of safety of 3 (in terms of working stress design) and ranges from 0.1 to 0.4 (see Table 9.2 and Figure 9.2).

TABLE 9.2 *Coefficients of Discontinuity Spacing, K_{sp}*

Discontinuity Spacing		K_{sp}
Description	**Distance (m)**	
Moderately close	0.3 to 1	0.1
Wide	1 to 3	0.25
Very wide	> 3	0.4

The factors influencing the magnitude of the coefficient are shown graphically in Figure 9.2. The relationship given in the figure is valid for a rock mass with spacing of discontinuities greater than 300 mm, aperture of discontinuities less than 5 mm (or less than 25 mm, if filled with soil or rock debris), and for a foundation width greater than 300 mm. For sedimentary rocks, the strata must be horizontal or nearly so.

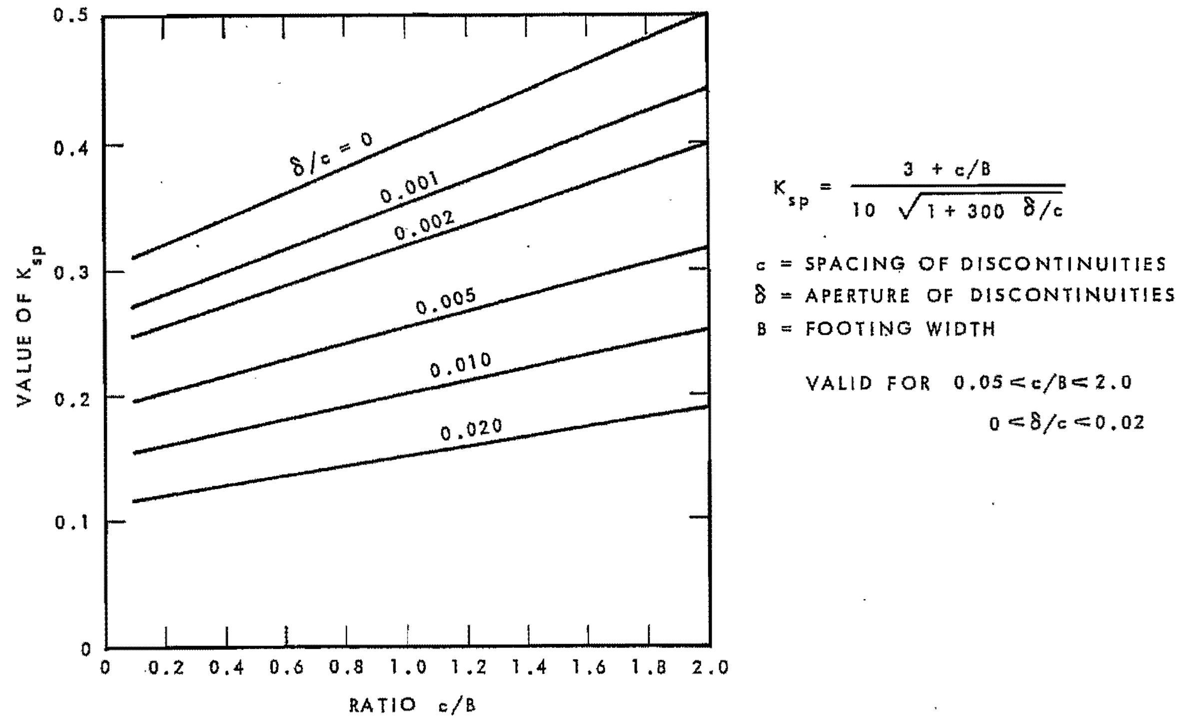


FIGURE 9.2: *Bearing pressure coefficient K_{sp}*

The bearing-pressure coefficient, K_{sp}, as given in Figure 9.2, takes into account the size effect and the presence of discontinuities and includes a nominal safety factor of 3 against the lower-bound bearing capacity of the rock foundation. The factor of safety against general bearing failure (ultimate limit states) may be up to ten times higher. For a more detailed explanation, see Ladanyi et al. (1974). Franklin and Gruspier (1983) discuss a special case of foundations on shale.

9.3 Estimates of Bearing Pressure

Universally applicable values of design bearing pressure cannot be given. The design bearing pressure is generally for serviceability limit states for settlement not exceeding 25 mm, or by the settlement criteria, as described in Chapter 11. Nevertheless, it is often useful to estimate a bearing pressure for preliminary design on the basis of the material description. Such values must be verified or treated with caution for final design. Table 9.3 gives presumed preliminary design bearing pressure for different types of soils and rocks.

TABLE 9.3 *Presumed Preliminary Design Bearing Pressure*

Group	Types and Conditions of Rocks and Soils	Strength of Rock Material	Preliminary Design Bearing Pressure (kPa)	Remarks
Rocks	Massive igneous and metamorphic rocks (granite, diorite, basalt, gneiss) in sound condition (2)	High-very high	10 000	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-high	3000	
	Sedimentary rocks: cemented shale, siltstone, sandstone, limestone without cavities, thoroughly cemented in conglomerates, all in sound condition (1) (2)	Medium-high	1000-4000	
	Compaction shale and other argillaceous rocks in sound condition (2) (4)	Low-medium	500-1000	
	Broken rocks of any kind with moderately close spacing of discontinuities (0.3 m or greater), except argillaceous rocks (shale)		1000	
	Limestone, sandstone, shale with closely spaced bedding		(See note 3)	
	Heavily shattered or weathered rocks		(See note 3)	
Coarse-grained soil	Dense gravel or dense sand and gravel		>600	Width of foundation (B) not smaller than 1 m. Groundwater level is assumed to be at a depth equal to B or more than B below the base of the foundation.
	Compact gravel or compact sand and gravel		200-600	
	Loose gravel or loose sand and gravel		<200	
	Dense sand		>300	
	Compact sand		100-300	
	Loose sand		<100	

TABLE 9.3 *Presumed Preliminary Design Bearing Pressure (continued)*

Group	Types And Conditions Of Rocks And Soils	Strength Of Rock Material	Preliminary Design Bearing Pressure (kPa)	Remarks
Fine-grained soil	Very stiff to hard clays or heterogeneous mixtures such as till		300-600	Fine-grained soils are susceptible to long-term consolidation settlement due to imposed loads and are often susceptible to severe swelling or shrinking due to changed moisture conditions. If the Plasticity Index (I_p) exceeds 30 and the clay content exceeds 25 %, the long-term performance of the foundation may be significantly affected by swelling or shrinking of the subsoils, and a complete assessment of these possibilities is necessary as discussed in Chapter 15
	Stiff clays		150-300	
	Firm clays		75-150	
	Soft clays and silts		<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 the 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 should be assessed by a person knowledgeable in this field of work.
2. Sound rock conditions allow minor cracks at spacing not closer than 1 m.
3. To be assessed by examination in-situ, including test loading if necessary.
4. These rocks are apt to swell on release of stress, and on exposure to water they are apt to soften and swell.
5. The above values are preliminary estimates only and may need to be adjusted upwards or downwards in a specific case. No consideration 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.

9.4 Foundations on Weak Rock

Conditions are frequently encountered where the rock material is very weak, has very closely spaced discontinuities, or is heavily weathered or fragmented. It is common practice in such cases to consider the rock as a soil 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. For more details on the estimation of strength and deformation parameters of rock masses, see the discussion in Chapter 3. Additional detail may also be found in Barton et al. (1974), Bieniawski (1976), and Hoek and Brown (1980). Table 9.3 provides suggestions for preliminary estimates.

9.5 Special Cases

9.5.1 Bearing Capacity of Jointed and Layered Rockmasses

The design for a foundation on rock that is jointed is dependent on the joint spacing and aperture, the area to be loaded, and the location of the load with respect to the joints. These characteristics dictate whether the rock will undergo compression, unconfined compression, or splitting. Where a weak compressible layer is present in the foundation rockmass, the hard rock layer can fail in flexure or punching. If the ratio of the thickness of the hard rock layer to the width of the foundation is small, then the rock will likely fail by punching. If the ratio is large, and the flexural strength of the rock is small, the rock will fail by flexure. This analysis can also be used for designs with hard rock layers over voids.

Bearing capacity calculations for this range of conditions are proposed by Lo and Hefny (2001) and by the ASCE (1996). The design bearing pressure for serviceability considerations can be taken as the ultimate bearing capacity divided by the factor of safety. Generally, the minimum factor of safety is 3 for a structure load comprising the full dead load and full live load. For factored geotechnical bearing resistance at ultimate limit states, the ultimate bearing capacity is multiplied by the geotechnical resistance factor of 0.5 as per Tables 8.1 and 8.2 (Chapter 8).

9.5.2 Foundations on Karstic Formations

Sinkholes are often the cause of foundation failure on karstic formations. These cavities, caused by the chemical reaction between limestone and acidic water, are irregular and difficult to predict. Sinkholes may develop at any time; therefore, investigations are necessary throughout the life of the structure.

Sinkholes can be detected using a number of geophysical techniques, including ground-penetrating radar (GPR), electromagnetic conductivity measurements (EM), and by drilling core samples. Sinkhole remediation can be performed by: concrete plugging; underground bridging; load eccentricity; and replacement of collapsed material with concrete. For more information on the detection of sinkholes and remedial measures, see Wyllie (1992).

9.6 Differential Settlement

Differential settlement occurs when adjacent footings are subjected to unequal settlements. Settlement (δ) for footings on elastic medium can be calculated by the following equation from Lo and Cooke (1989):

$$\delta = C_s q B \frac{\left(1-\nu^2\right)}{E} \tag{9.2}$$

where

C_s = influence factor for the shape of the foundation
q = applied pressure
B = width of the footing
ν = Poisson's ratio
E = elastic modulus

Maximum differential settlement should be calculated and tested for during design stages to avoid redesign of the footings. Settlement in rock with seams or faults can be estimated by plate load testing as discussed in Chapter 4.

10 Bearing Capacity of Shallow Foundations on Soil

10 Bearing Capacity of Shallow Foundations on Soil

10.1 Introduction

One possible ultimate limit state of a shallow foundation involves the case where the applied loads exceed the resistance of the ground beneath the foundation. The geotechnical resistance at this ultimate limit state is termed the ultimate bearing capacity of the ground that supports the foundation. The ultimate bearing capacity depends on the strength of the ground, ground conditions (e.g., thickness and presence of weak layers, depth to bedrock), and the nature of applied loading (e.g., vertical, horizontal and inclined forces; moments). Methods to estimate the ultimate bearing capacity of shallow foundations on fine- and coarse-grained soils are presented in this Chapter. Other possible ultimate limit states for shallow foundations may include sliding, overturning and general slope stability and their influence on foundation design need to be assessed for each individual project. The serviceability limit state of the foundation is considered separately from the ultimate limit state, as presented in Chapter 11. Shallow foundations are those constructed on or embedded near the ground surface such that the distance from the ground surface to the underside of the foundation is not greater than the width (or least plan dimension) of the foundation.

10.2 Conventional Bearing Capacity

10.2.1 Bearing Capacity Equation

The ultimate bearing capacity (i.e. the geotechnical bearing resistance at the ultimate limit state) of a shallow foundation on uniform soil as shown in Figure 10.1 with shear strength parameters c and ϕ may be calculated from:

$$q_u = c\,N_c\,S_c + q_s\,N_q\,S_q + \frac{1}{2}\gamma\,B\,N_\gamma\,S_\gamma \tag{10.1}$$

where:

q_u	=	ultimate bearing capacity (denoted as R_n in limit states design–see Section 8.4),
N_c, N_q, N_γ	=	dimensionless bearing capacity factors (see 10.2.3),
S_c, S_q, S_γ	=	dimensionless modification factors for foundation shape, inclination, depth and tilt and ground slope (see 10.2.4),
q_s	=	vertical stress acting at the elevation of the base of foundation (see 10.2.2),
B	=	width of foundation or least plan dimension of the foundation,
c	=	soil cohesion (see 10.2.2),
γ	=	soil unit weight (see 10.2.6).

Unless otherwise noted, any consistent set of units may be used for the parameters in Equation 10.1.

Equation 10.1 expresses the ultimate bearing capacity of a foundation experiencing general shear failure as the sum of: the shear resistance of a weightless material with cohesive strength parameter c (N_c term), the shear resistance of a frictional but weightless material with angle of friction ϕ' on addition of a surcharge q_s at the foundation level (N_q term), and the shear resistance of a frictional material with angle of friction ϕ' and weight γ but no surcharge (N_γ term).

Shear strength parameters c and ϕ' are normally selected within depth B beneath the base of the foundation.

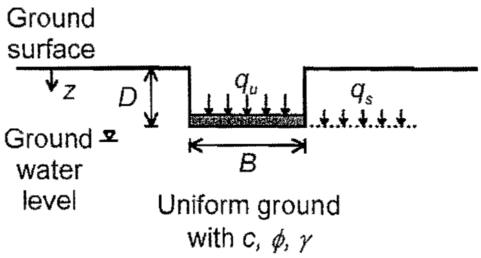


FIGURE 10.1 *Definition of geometry and parameters for ultimate bearing capacity of a shallow foundation*

10.2.2. Undrained and Drained Conditions

The values of c and ϕ' for use in the general bearing capacity equation (Equation 10.1) depend on the type of soil and whether short-term (undrained) or long-term (drained) conditions are being examined. The short-term stability of a foundation involving fine-grained soils can be calculated by taking c equal to the undrained shear strength, s_u, and $\phi = 0$. The long-term stability of a foundation can be obtained with c equal to the effective cohesion intercept, c', and ϕ' equal to the effective angle of internal friction of the soil, ϕ'. In most cases, short-term stability controls design, especially for soft to very stiff clays.

The surcharge q_s for use in the general bearing capacity also depends on whether undrained or drained conditions are being considered. For undrained conditions q_s is the total vertical stress acting adjacent to the base of the foundation; whereas, for drained conditions it is equal to the vertical effective stress and consequently will be influenced by the position of the groundwater level (see Section 10.2.6).

10.2.3 Bearing Capacity Factors

Bearing capacity factors have been derived based on modified plasticity solutions for uniform ground conditions. Bearing capacity factors N_c and N_q have been reported by Meyerhof (1963), Hansen (1970) and Vesic (1975) to be equal to:

$$N_c = (N_q - 1)\cot\phi \tag{10.2}$$

$$N_q = e^{\pi \tan\phi} \tan^2\left(45^\circ + \frac{\phi}{2}\right) \tag{10.3}$$

Several formulations of the bearing capacity factor N_γ are available (Terzaghi, 1943; Meyerhof, 1963; Hansen, 1970; Vesic, 1975) but tend to overestimate N_γ when compared with the more rigorous plasticity solution of Davis and Booker (1971). An approximate value of N_γ suitable for $\phi' > 10^\circ$ obtained from Davis and Booker (1971) is:

$$N_\gamma \cong 0.0663e^{0.1623\phi} \tag{10.4}$$

for a smooth interface between the foundation and the ground, while for a rough interface it equals:

$$N_\gamma \cong 0.0663e^{0.1623\phi} \tag{10.5}$$

where ϕ is in degrees.

For the case of undrained stability ($c = s_u$, $\phi' = 0$) the bearing capacity factors become:

$$N_c = (2 + \pi) \tag{10.6}$$
$$N_q = 1 \text{ and} \tag{10.7}$$
$$N_\gamma = 0 \tag{10.8}$$

Bearing capacity factors N_c, N_q, and N_γ for uniform ground conditions are presented in Table 10.1 and plotted in Figure 10.2.

TABLE 10.1 *Bearing capacity factors N_c and N_q from Meyerhof (1963) and N_γ from Davis and Booker (1971)*

ϕ °	N_c	N_q	N_γ rough	N_γ smooth
0	5.1	1	0	0
10	8.3	2.5	0.6	0.3
15	11	3.9	1.3	0.8
20	15	6.4	3.0	1.7
21	16	7.1	3.6	2.0
22	17	7.8	4.2	2.4
23	18	8.7	5.0	2.8
24	19	9.6	5.9	3.3
25	21	11	7.0	3.8
26	22	12	8.2	4.5
27	24	13	9.7	5.3
28	26	15	11	6.2
29	28	16	14	7.3
30	30	18	16	8.6
31	33	21	19	10
32	35	23	22	12
33	39	26	27	14
34	42	29	31	17
35	46	33	37	19
36	51	38	44	23
37	56	43	52	27
38	61	49	61	32
39	68	56	73	37
40	75	64	86	44

Small (2001) and Poulos et al. (2001) present useful summaries of bearing capacity factors for soils with an increase in strength with depth, finite depth, fissured clays, layered soils, and foundations near slopes.

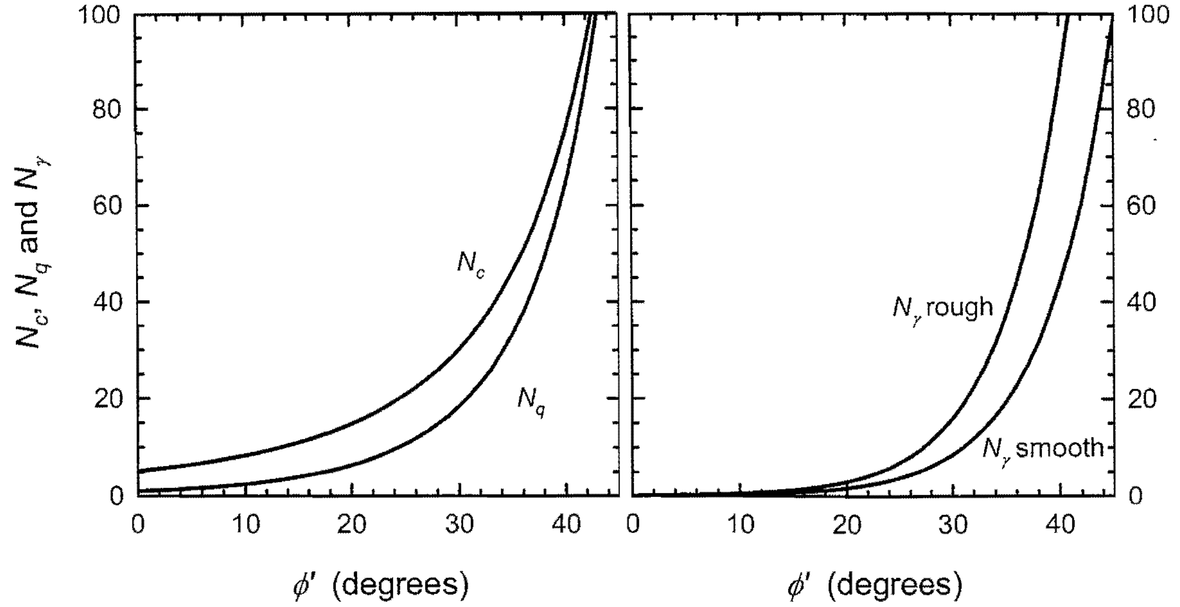


FIGURE 10.2 *Bearing capacity factors N_c and N_q from Meyerhof (1963) and N_γ from Davis and Booker (1971)*

10.2.4 Modification Factors

The bearing capacity factors were derived for the case of strip footing on a level base subjected to loading perpendicular to the foundation. Deviations from these conditions can be accounted for, where appropriate, by factors to modify the bearing capacity factors for the effects of foundation shape (S_{cs}, S_{qs} and $S_{\gamma s}$), load inclination (S_{ci}, S_{qi} and $S_{\gamma i}$), foundation depth (S_{cd}, S_{qd} and $S_{\gamma d}$), surface slope ($S_{c\beta}$, $S_{q\beta}$ and $S_{\gamma\beta}$)and foundation tilt ($S_{c\delta}$, $S_{q\delta}$ and $S_{\gamma\delta}$) via:

$$S_c = S_{cs}\, S_{ci}\, S_{cd} S_{c\beta}\, S_{c\delta} \tag{10.9}$$
$$S_q = S_{qs}\, S_{qi}\, S_{qd}\, S_{q\beta}\, S_{q\delta} \tag{10.10}$$
$$S_\gamma = S_{\gamma s}\, S_{\gamma i}\, S_{\gamma d}\, S_{\gamma\beta}\, S_{\gamma\delta} \tag{10.11}$$

where expressions for the various modification factors are given in Table 10.2 based on Vesic (1975).

TABLE 10.2 *Modification Factors for General Bearing Capacity Equation (based on Vesic, 1975)*

Factor	S_c	S_q	S_γ
Foundation shape, s	$S_{cs} = 1 + \dfrac{B'}{L'}\dfrac{N_q}{N_c}$	$S_{qs} = 1 + \dfrac{B'}{L'}\tan\phi$	$S_{\gamma s} = 1 - 0.4\dfrac{B'}{L'}$
Inclined loading, i [1]	$\phi = 0$, $S_{ci} = 1 - \dfrac{mH}{B'L'cN_c}$ $\phi > 0$, $S_{ci} = S_{qi} - \dfrac{1 - S_{qi}}{N_c \tan\phi}$	$S_{qi} = \left(1 - \dfrac{H}{V + B'L'c\cot\phi}\right)^m$	$S_{\gamma i} = \left(1 - \dfrac{H}{V + B'L'c\cot\phi}\right)^{m+1}$
Foundation depth, d [2]	$\phi = 0$, $S_{cd} = 1 + 0.4k$ $\phi > 0$, $S_{cd} = S_{qd} - \dfrac{1 - S_{qd}}{N_c \tan\phi}$	$S_{qd} = 1 + 2\tan\phi\,(1 - \sin\phi)^2 k$	$S_{\gamma d} = 1$

TABLE 10.2 *Modification Factors for General Bearing Capacity Equation (based on Vesic, 1975) (continued)*

Surface slope, β[3]	$\phi = 0$, $S_{c\beta} = 1 - \dfrac{2\beta}{\pi + 2}$ $\phi > 0$, $S_{c\beta} = S_{q\beta} - \dfrac{1 - S_{q\beta}}{N_c \tan\phi}$	$S_{q\beta} = (1 - \tan\beta)^2$	$S_{\gamma\beta} = (1 - \tan\beta)^2$ [4]
Base inclination, δ[5]	$\phi = 0$, $S_{c\delta} = 1 - \dfrac{2\delta}{\pi + 2}$ $\phi > 0$, $S_{c\delta} = S_{q\delta} - \dfrac{1 - S_{q\delta}}{N_c \tan\phi}$	$S_{q\delta} = (1 - \delta\tan\phi)^2$	$S_{\gamma\delta} = (1 - \delta\tan\phi)^2$

[1] V = vertical force; H = horizontal force; m depends on direction of inclined loading θ relative to long side of the foundation: If force inclined in B direction (θ=90°) $m = m_B = (2+B/L)/(1+B/L)$, if inclined in L direction (θ=0°) $m = m_L = (2+L/B)/(1+L/B)$, and if inclined at angle θ to L direction $m = m_\theta = m_L\cos^2\theta + m_B\sin^2\theta$.
[2] $k = D/B$ if $D/B \leq 1$; $k = \tan^{-1}(D/B)$ if $D/B > 1$.
[3] β = inclination below horizontal of the ground surface away from the edge of the foundation (see Figure 10.4); for $\beta < \pi/4$; β in radians.
[4] For sloping ground case where $\phi = 0$ $N_\gamma = -2\sin\beta$ must be used in bearing capacity equation.
[5] δ = inclination from the horizontal of the underside of the foundation (see Figure 10.4); for $\delta < \pi/4$; δ in radians.

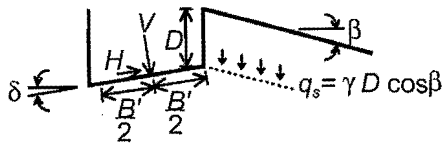


FIGURE 10.4 *Definition of parameters for shallow foundation with ground slope* β *and base tilt* δ

10.2.5 Eccentric Forces and Moments

If the foundation is subjected to vertical forces that act eccentric to the centroid of the foundation, the size of the foundation used in the bearing capacity equation should be reduced:

$$B' = B - 2e_B \tag{10.12}$$
$$L' = B - 2e_L \tag{10.13}$$

where

B, L	=	actual foundation dimensions,
B′, L′	=	reduced dimensions for use in bearing capacity equation, and
e_B, e_L	=	eccentricities of force in directions B and L from the centroid.

This is an approximate but reasonable approach to account for eccentricities provided that the resultant loading acts within the middle third of the foundation (i.e. e < B/6). Values of B′ and L′ are to be used in all bearing capacity calculations. The term k for depth modification factors S_{cd} and S_{qd}, and the term m for load inclination factors S_{ci}, S_{qi} and $S_{\gamma i}$ as shown in Table 10.2 remain in terms of L and B.

Foundations that are subject to moments M_B and M_L in the B and L directions and vertical load V acting through the centroid can be treated as an equivalent loading system with vertical load V acting at eccentricities e_B and e_L as shown in Figure 10.3.

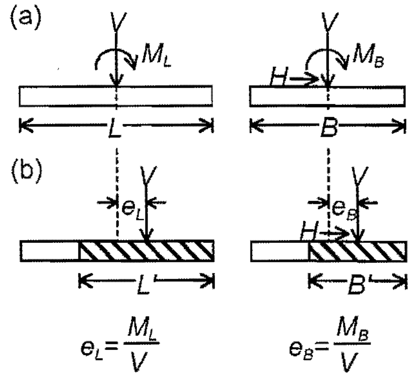


FIGURE 10.3 *Shallow foundation subjected to moments and vertical force*

10.2.6 Influence of Groundwater

The position of the groundwater level will influence the selection of γ and q_s for use in the general bearing capacity equation when considering drained conditions as summarized in Table 10.3.

TABLE 10.3 *Unit Weight and Surcharge for Drained Conditions in the General Bearing Capacity Equation depending on Depth from Surface to the Groundwater Level z (as defined in Figure 10.1). The foundation is located at depth D beneath the ground surface*

Depth from surface of groundwater level	Unit weight γ for N_γ term	Surcharge term q_s
$z = 0$	γ_{sub}	$\gamma_{sub} D$
$z = D$	γ_{sub}	$\gamma_{bulk} D$
$D < z < D+B$	$\gamma_{sub} + \frac{z-D}{B}(\gamma_{bulk} - \gamma_{sub})$	$\gamma_{bulk} D$
$z > D+B$	γ_{bulk}	$\gamma_{bulk} D$

The bulk unit weight γ_{bulk} should be selected based on the minimum water content of the soil above the water table. Effective stresses can be introduced into the N_γ term by using the submerged unit weight γ_{sub}, which is equal to:

$$\gamma_{sub} = \gamma_{sat} - \gamma_w \tag{10.14}$$

where
γ_{sat} is the saturated unit weight and γ_w is the unit weight of water.

In all cases in Table 10.3, q_s is the vertical effective stress adjacent to the foundation at its base.

10.3 Bearing Capacity Directly from In-Situ Testing

10.3.1 Standard Penetration Test (SPT)

There is no direct relationship between standard penetration test (SPT) resistance N and the ultimate bearing capacity. Shear strength parameters for use in the general bearing capacity equation can be estimated from empirical correlations with SPT-N (e.g., Hatanaka and Uchida, 1996; Terzaghi et al., 1996). Empirical design charts relating the design bearing pressure for foundations on sand to SPT-N are available; however, since these are also based on limiting settlement of the foundation they are presented in Section 11.8.1. Such empirical correlations need to be treated with caution and adjusted as appropriate by experience.

10.3.2 Cone Penetration Test (CPT)

Shear strength parameters for use in the general bearing capacity equation can be estimated from empirical correlations with cone penetration test (CPT) results (e.g., Lunne et al., 1997). Empirical methods are also available to estimate the ultimate bearing capacity directly from CPT tip resistance q_c.

For coarse-grained soils:

$$q_u = K_\phi \, \overline{q}_c \tag{10.15}$$

where

K_ϕ = empirical factor relating ultimate bearing capacity and average CPT tip resistance for coarse-grained soils, and

$\overline{q}_c$ = average tip resistance over a depth B beneath the foundation.

Values of K_ϕ depend on soil density and foundation shape and range between 0.16 to 0.3 (Lunne et al., 1997). A value of $K_\phi = 0.16$ can be used for most cases, recognizing that limiting settlement will generally control foundation design.

For fine grained soils and undrained conditions:

$$q_u = K_{su} \, \overline{q}_c + \gamma D \tag{10.16}$$

where

K_{su} = empirical factor relating ultimate bearing capacity and average CPT tip resistance for fine-grained soils, and

all other parameters are as previously defined. Factor K_{su} ranges from 0.3 to 0.6 depending on foundation shape and embedment, and soil stress history and sensitivity. A value of $K_{su} = 0.3$ can be conservatively used for most cases.

These empirical correlations need to be treated with caution and adjusted where appropriate based on experience.

10.3.3 Pressuremeter and Dilatometer Tests

In-situ tests such as the pressuremeter test (PMT) and flat dilatometer test (DMT) can be used to obtain shear strength parameters for use in the general bearing capacity equation (e.g., Lunne et al., 1989; Marchetti et al., 2001).

10.3.4 Plate Load Test

A plate load test, if loaded to failure, can be used to assess the ultimate bearing capacity. In this test a reduced-scale foundation is subjected to load and the deflection is recorded. The plate load test involves the actual ground material beneath the foundation and can be useful to obtain soil parameters and to verify the method of analysis. The general bearing capacity equation can be used to interpret results if ground conditions are homogeneous with depth. Scale effects are important as the results will depend on the size of the reduced-scale foundation relative to the underlying sequence of soil strata. Appropriate engineering judgment must be exercised prior to any extrapolation to larger foundations. An additional disadvantage is the costs required to conduct the tests. As a result, plate load tests may only be appropriate for medium to higher risk projects. The plate load test is also useful in the evaluation of ground stiffness (e.g., see Sections 7.7.1 and 11.7)

10.4 Factored Geotechnical Bearing Resistance at Ultimate Limit States

Geotechnical resistance at the ultimate limit state is reduced (multiplied by the appropriate geotechnical resistance factor (see Tables 8.1 and 8.2 in Chapter 8) to provide the factored geotechnical bearing resistance for foundation design.

10.4.1 Net Ultimate Bearing Pressure

The ultimate bearing capacity q_u is the total stress that can be applied at foundation level. If an excavation is made for the foundation, stresses in excess of the original overburden stress at the foundation level contribute to bearing failure. The net bearing capacity is defined as:

$$q_{netu} = q_u - q_{ob} \tag{10.17}$$

where

q_{netu} = net bearing capacity,
q_u = ultimate bearing capacity, and
q_{ob} = total overburden stress removed at foundation level.

There is no possibility of bearing failure if the applied load at the foundation level is equal to that of the excavated soil. This is the basis for the design of what is termed full-compensated (or floating) foundations.

10.4.2 Allowable Bearing Capacity

In a working stress design (WSD) approach (see Chapter 8) all uncertainty is accounted for in one parameter called the global factor of safety against ultimate bearing capacity FS. The allowable bearing capacity, q_{all}, that can be applied at the foundation level is:

$$q_{all} = \frac{q_{netu}}{FS} + q_{ob} \tag{10.18}$$

The value of FS against ultimate bearing capacity of a shallow foundation is normally taken equal to 3 (see Section 8.8 in Chapter 8).

For shallow foundations on the ground surface or neglecting the effect of the excavated ground, the allowable bearing pressure becomes:

$$q_{all} = \frac{q_u}{FS} \tag{10.19}$$

10.4.3 Factored Geotechnical Bearing Resistance

Using the load and resistance factor design (LRFD) approach (see Chapter 8), uncertainty in loads acting on the foundation and the resistance of the foundation are treated separately. Loads acting on the foundation are increased using appropriate factors for live and dead loads, while the geotechnical resistance is decreased using a geotechnical resistance factor Φ.

For the bearing resistance of shallow foundations the geotechnical resistance factor Φ may be taken to be 0.5 (see Tables 8.1 and 8.2 in Chapter 8).

11 Settlement of Shallow Foundations

11 Settlement of Shallow Foundations

11.1 Introduction

The settlement of a foundation must be within tolerable or acceptable limits to satisfy the specified serviceability limit states criteria (see Chapter 8) for a given project. Methods to estimate the possible magnitude of ground settlement, the rate of settlement and the maximum allowable settlement are presented in this Chapter.

The settlement of shallow foundations depends on the magnitude of the applied forces, geometry of the foundation, type of ground conditions, ground stiffness and in some cases ground strength.

The rate of settlement depends on the rate of loading relative to the rate of excess pore pressure dissipation. For saturated soils, if the rate of loading exceeds the rate of dissipation, pore pressures in excess of steady-state values will be generated. Settlement of the foundation will then increase with time until the excess pore pressures are dissipated. Thereafter, creep settlement can continue with time. Soil type, permeability, drainage conditions and magnitude of loads influence how quickly excess pore pressures can dissipate.

Maximum allowable settlements (i.e., the serviceability limit states criteria) largely depend on type and end use of the structure, nature of the ground conditions and risk of the project.

11.2 Components of Deflection

Vertical deflections of a shallow foundation may arise from:

- undrained shear distortions that occur with no change in void ratio (or volume);,
- drained settlements arising from change in void ratio (or volume) and shear distortions that occur from an increase in effective stresses; and
- creep settlements arising from change in void ratio (or volume) that occur at constant effective stresses.

Undrained distortions occur from shear strains when the rate of loading is fast relative to the time required for excess pore pressures to dissipate (i.e., under conditions of undrained loading). Since undrained distortions arise from shear strains they occur for situations other than one-dimensional loading and become more prominent as the size of the loaded area decreases relative to the thickness of the compressible layer. Drained settlements are time-dependent displacements associated with primary consolidation (i.e., decrease in void ratio) of the foundation soils as the effective stresses increase. Drained settlements may occur rapidly for coarse-grained soils (e.g., sand, gravel), or very slowly for fine-grained soils (e.g., silt, clay). Creep settlements are time-dependent settlements associated with secondary consolidation. Reference to the total final settlement of a foundation in the subsequent sections neglects creep settlement. For most practical cases creep settlements may be added to the total final settlement as discussed in Section 11.10.

11.2.1 Settlement of Fine-Grained Soils

For most foundation applications, fine-grained soils typically experience both undrained distortions and drained settlements, and possibly secondary compression. Undrained distortions can be a significant proportion of the total settlement for overconsolidated clays, but often can be small relative to the drained settlements for normally consolidated clays.

The total final settlement, S_{TF}, and the settlement at time t, S_{Tt}, are equal to (Davis and Poulos, 1968):

$$S_{TF} = S_i + S_{CF} = S_i + (S_{TF} - S_i) \qquad (11.1)$$

$$S_{Tt} = S_i + U_s\, S_{CF} = S_i + U\,(S_{TF} - S_i) \qquad (11.2)$$

where

S_i = undrained distortion
S_{CF} = final consolidation settlement, and
U = degree of consolidation settlement.

Methods to estimate the total final settlement and undrained distortions for fine-grained soils are presented in Sections 11.3.3, 11.3.4, and 11.4.2. The selection of appropriate parameters discussed for use in these methods is discussed in Section 11.7. Methods to estimate the degree of consolidation settlement are presented in Section 11.11.

11.2.2 Settlement of Coarse-Grained Soils

For most foundation applications, coarse-grained soils do not experience undrained distortions since they are sufficiently permeable to dissipate excess pore pressures rapidly, relative to the rate of applied loading. Coarse-grained soils experience drained settlements from compression of the soil skeleton (i.e., decreases in void ratio) for increases in effective stress. Since excess pore pressures dissipate rapidly, the settlement at time t is essentially equal to the total final settlement S_{TF}. Coarse-grained soils may also experience creep or additional deflections from cyclic loading.

The total final settlement of foundations on coarse-grained soils can be calculated using the elastic displacement method described in Section 11.3 (with the selection of appropriate parameters discussed in Section 11.7) or by direct methods related to *in-situ* testing as described in Section 11.8.

11.3 Three-Dimensional Elastic Displacement Method

11.3.1 Approximating Soil Response as an Ideal Elastic Material

When subject to increases in stress by loading from foundations, soil materials exhibit nonlinear and inelastic stress-strain response, such that increments in stress are not linearly proportional to increments in strain, and permanent strains remain upon unloading. Additionally, the stress-strain response may be dependent on the stress path. Estimates of settlement (serviceability limit state) are made at service loads (i.e., working stresses) that are usually well below the ultimate limit state.

For such conditions, the issue of soil nonlinearity may be resolved by selecting secant (or average) stiffness parameters for the appropriate stress (and/or strain) increment of the ground loaded by the foundation. Thus, despite the fact that soils are not usually elastic materials, elastic displacement theory can be used to obtain estimates of foundation settlement for most practical cases.

In elastic displacement theory the soil is treated as a saturated two phase material that is normally assumed to have a homogeneous and isotropic elastic soil skeleton with Young's modulus E' and Poisson's ratio ν' and incompressible pore water (solutions exist that explicitly consider soil anisotropy e.g., Rowe and Booker, 1981a,b). It is the responsibility of the geotechnical engineer to evaluate these elastic parameters in the context of the true modulus of deformation of the ground loaded by the foundation. Guidance on the selection of Young's modulus E and Poisson's ratio ν are provided in Sections 11.3.2 and 11.7.

11.3.2 Drained and Undrained Moduli

The total final settlement can be estimated from elastic displacement theory by using the change in effective stress (once all excess pore pressures have dissipated) and drained modulus E' and ν'. Undrained distortions can be calculated using the change in total stresses and undrained modulus E_u and Poisson's ratio of $\nu_u = 0.5$ to satisfy the conditions of zero volume change.

11.3.3 Three-Dimensional Elastic Strain Integration

The total final settlement and undrained distortion can be calculated by summing the vertical strains $\Delta\varepsilon_z$ arising from loading on the foundation. This approach may be useful for some problems where different layers are encountered or ground properties vary beneath the foundation. These calculations can be easily conducted using spreadsheet computer programs.

The increase in vertical strain is related to the increase in stress using three-dimensional elasticity in x, y and z coordinates (where z is in the vertical direction). Similar expressions can be written for polar coordinates for use with circular foundations (e.g., see Poulos and Davis, 1974). The total final settlement can be calculated using:

$$S_{TF} = \sum_{i=1}^{n} \left[\Delta\varepsilon_z \, \delta h\right]_i = \sum_{i=1}^{n} \left[\frac{1}{E'}\left(\Delta\sigma'_z - \nu'(\Delta\sigma'_x + \Delta\sigma'_y)\right)\delta h\right]_i \tag{11.3}$$

where

$\Delta\varepsilon_z$	=	increment in vertical strain from the increase in effective stresses of sublayer i,
$\Delta\sigma'_x$, $\Delta\sigma'_y$, $\Delta\sigma'_z$	=	increment in effective stresses in x, y and z directions of sublayer i,
E', ν'	=	secant drained Young's modulus and Poisson's ratio for the appropriate stress increment and layer i,
n	=	number of sublayers, and
δh	=	thickness of sublayer i.

The increment in effective stress can be found using available solutions for stress distribution with depth for the appropriate loaded region (see Section 11.6). The number of sublayers should be selected to provide a sufficient integration of vertical strain with depth and also to capture different ground conditions beneath the foundation.

The undrained distortion can be calculated in a similar manner using:

$$S_i = \sum_{i=1}^{n} \left[\Delta\varepsilon_z \, \delta h\right]_i = \sum_{i=1}^{n} \left[\frac{1}{E_u}\left(\Delta\sigma_z - \nu_u(\Delta\sigma_x + \Delta\sigma_y)\right)\delta h\right]_i \tag{11.4}$$

where

$\Delta\varepsilon_z$	=	increment in vertical strain from the increase in total stresses of sublayer i,
$\Delta\sigma_x$, $\Delta\sigma_y$, $\Delta\sigma_z$	=	increment in total stresses in x, y and z directions of sublayer i,
E_u, ν_u	=	secant undrained Young's modulus and Poisson's ratio for the appropriate stress increment and layer i, and all other parameters as previously defined.

11.3.4 Elastic Displacement Solutions

Elastic displacement solutions for various foundation shapes, soil homogeneity, finite layer depth, mutlilayered soils, foundation roughness, foundation stiffness, and drainage conditions have been provided by Poulos and Davis

(1974). Results from these solutions are presented in a graphical manner. This can be useful to illustrate the influence of key parameters on foundation settlement (e.g., size of loaded area relative to the thickness of compressible deposit). They can also provide a useful check on the results from more elaborate analyses.

Elastic displacement solutions are presented in the subsequent sections in terms of Young's modulus E and Poisson's ratio ν and can be used to find the total final settlement using $E=E'$ and $\nu=\nu'$, and the undrained distortion using $E=E_u$ and $\nu=\nu_u=0.5$.

a) Flexible Strip Foundation

The settlement beneath the centre of a flexible strip foundation on the surface of a uniform layer of isotropic elastic material of thickness h and subject to uniform vertical pressure q is equal to:

$$S = \frac{q\,B}{E} I_S \tag{11.5}$$

where

- q = average pressure applied to the ground by the foundation,
- B = width of strip foundation,
- E = drained or undrained modulus of ground,
- I_S = influence factor for a strip foundation given in Figure 11.1a, and
- h = distance from ground surface to an incompressible base.

b) Circular Foundation

The settlement beneath the centre of a circular foundation on the surface of a homogeneous and isotropic elastic material of thickness h and subject to uniform vertical pressure q is equal to:

$$S = \frac{q\,B}{E} I_C \tag{11.6}$$

where

- q = average pressure applied to the ground by the foundation,
- B = diameter of the circular foundation,
- E = drained or undrained modulus of ground, and
- I_C = influence factor for a circular foundation given in Figure 11.1b.

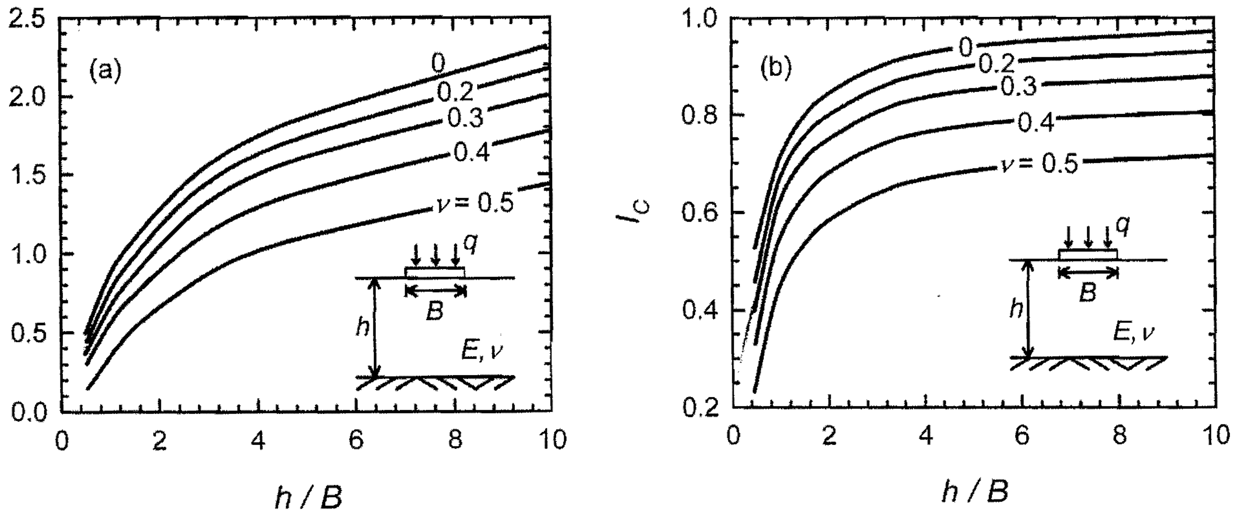


FIGURE 11.1 *Influence factors for the settlement beneath the centre of flexible: (a) strip foundation of width B and (b) circular foundation of diameter B, on a uniform isotropic compressible material of thickness h. Modified from Rowe and Booker (1981a,b)*

For the more general cases involving non-uniform ground stiffness, foundation rigidity and burial beneath the surface, as defined in Figure 11.2, the settlement beneath the centre of a shallow circular foundation resting on an isotropic elastic material of finite thickness whose stiffness increases linearly with depth and is subject to uniform vertical pressure q can be estimated using (Mayne and Poulos, 1999):

$$S = \frac{q\, B\, I_G\, I_F\, I_E\,(1-\nu^2)}{E_o} \tag{11.7}$$

where

- q = average pressure applied to the ground by the foundation,
- B = diameter of the circular foundation,
- I_G = influence factor for nonuniform ground stiffness given in Figure 11.2a,
- I_F = influence factor for foundation stiffness given in Figure 11.2b,
- I_E = influence factor for foundation embedment given in Figure 11.2c,
- ν = Poisson's ratio, and
- E_o = drained or undrained modulus at the ground surface.

The influence factor for nonuniform ground stiffness is plotted against the dimensionless term:

$$\beta = \frac{E_o}{k\, B} \tag{11.8}$$

where

k is the increase in modulus with depth.

The influence factor for foundation stiffness is defined in terms of the dimensionless foundation flexibility ratio K_F which is equal to:

$$K_F = \frac{E_F}{\bar{E}}\left(\frac{2t}{B}\right)^3 \tag{11.9}$$

where

E_F is the Young's modulus of the foundation material (e.g., concrete, steel), t is the thickness of the foundation, and $\bar{E}$ is the average modulus of the ground within depth B beneath the foundation.

Although developed for circular footings, this method can be used for square and rectangular footing (provided the length is less than three times the breadth) with an equivalent diameter used for B such that the total force applied to the foundation is the same.

11.4 One-Dimensional Consolidation Method

11.4.1 Oedometer Test

The stiffness parameters for many practical settlement calculations involving fine-grained soils can be obtained from one-dimensional consolidation laboratory tests, referred to herein as the oedometer test. In principle, the settlement of coarse-grained soils could also be assessed using the oedometer test; however, this is not normally practical given the difficulties in obtaining undisturbed samples of coarse-grained soils.

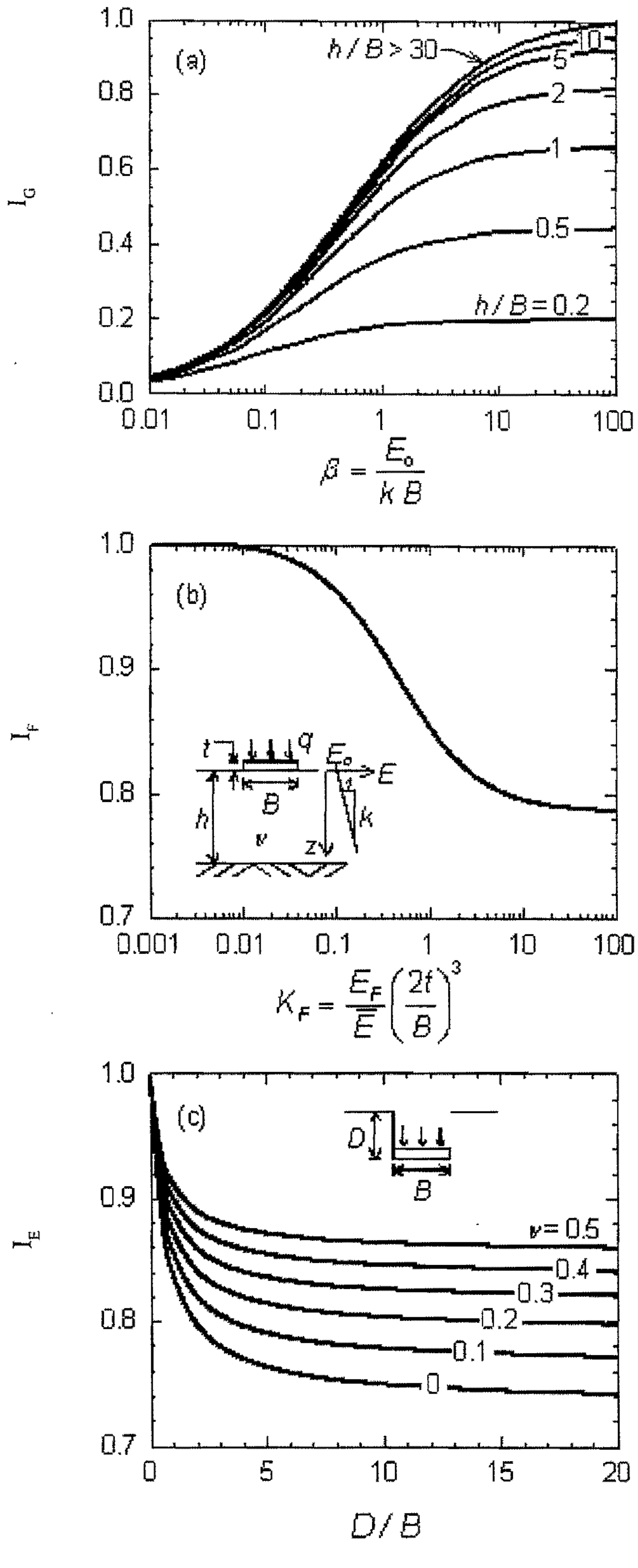


FIGURE 11.2 *Influence factors for the settlement beneath the centre of a uniformly loaded circular foundation of diameter* **B** *on a finite compressible layer of thickness* **h** *whose stiffness increases linearly with depth. Influence factors for: (a) nonuniform ground stiffness, (b) foundation rigidity and (c) foundation embedment. Modified from Mayne and Poulos (1999)*

Specific details on the procedures to conduct and interpret results from the oedometer test can be found elsewhere (e.g., ASTM D2435; Holtz and Kovacs, 1981). In this test, soil samples retrieved from the field are subjected to increments in total vertical stress under conditions of zero lateral strain. Excess pore pressures that generate in the sample from an increment in total stress are allowed to dissipate (normally for a 24 hour period – see ASTM D2435 for possible deviations) prior to placement of an additional total stress increment. It is often assumed that the increase in effective stress is equal to the increase in total stress at the end of each increment. The change in void ratio (obtained from the change in height of the sample) is recorded for each stress increment. The void ratio at the end of each increment is plotted versus the logarithm of the effective stress on the sample as illustrated in Figure 11.3.

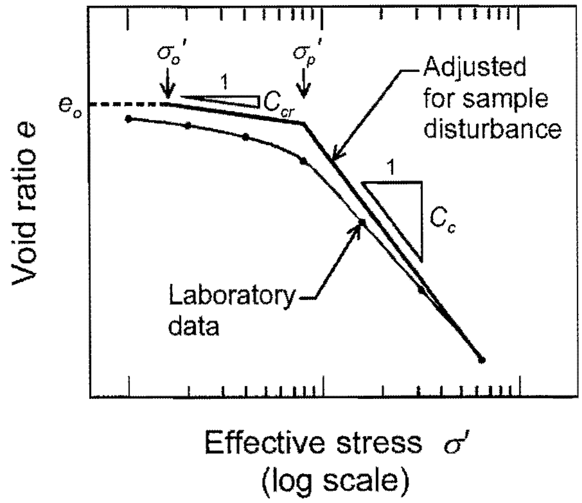


FIGURE 11.3 *Oedometer test results showing definition of parameters to calculate one-dimensional settlement*

The following parameters can be defined in reference to Figure 11.3:

e_o = initial void ratio of the sample corresponding to the initial (or in-situ) vertical effective stress σ'_o,
σ'_p = preconsolidation pressure which corresponds to the previous maximum vertical effective stress experienced by the sample,
C_c = compression index, and
C_{cr} = recompression index (this portion of the plot is present only if $\sigma'_p > \sigma'_o$).

The preconsolidation pressure is related to the stress history of the deposit where normally consolidated soils have σ'_p approximately equal to σ'_o and overconsolidated soils have σ'_p greater than σ'_o. The magnitude of the preconsolidation pressure may also be presented in terms the overconsolidation ratio, OCR, where:

$$OCR = \frac{\sigma_p'}{\sigma_o'} \tag{11.10}$$

The preconsolidation pressure can be estimated using the empirical and graphical Casagrande procedure (for specific details see Holtz and Kovacs, 1981). This approach is normally sufficient for estimation of foundation settlement provided there is a defined change in slope of the e–logσ' plot. An alternate approach may be necessary for soils with a more gradual change in slope of the e–logσ' plot (e.g., Becker et al., 1987). Geologic information about the site can also be used to assist with the estimation of the preconsolidation pressures.

Sampling disturbance decreases the preconsolidation pressure obtained from the oedometer test and also increases the calculated settlements (Leroueil, 1996). Empirical methods exist to modify the measured laboratory curve to account for changes in sample compressibility arising from sampling disturbance (e.g., see Holtz and Kovacs, 1981).

Slopes C_c and C_{cr} are dimensionless parameters. Although they represent the compressibility of a particular soil sample with a single value over a certain stress range, this does not imply that its stiffness is constant over that stress range. Rather the value of C_c combined with the logarithmic scale captures the strain-hardening behaviour of soils (i.e., they become stiffer as the effective stresses increase).

11.4.2 One-Dimensional Settlement: e–logσ′ Method

For cases where the loaded area of the foundation is large relative to the thickness of the compressible deposit, lateral strains may be sufficiently small such that the foundation settlement can be approximated with one-dimensional strain models. Since one-dimensional strain conditions are imposed during a conventional oedometer test, one-dimensional settlement is denoted herein as oedometer settlement S_{oed}.

One-dimensional settlement from an increase in initial vertical effective stress σ'_o to final vertical effective stress σ'_f is obtained by summing the increase in vertical strains with depth. The increment in vertical strain is obtained from the change in void ratio Δe for an increase in effective stress from laboratory oedometer data viz:

$$S_{oed} = \sum_{i=1}^{n} \left[\Delta\varepsilon_z \,\delta h\right]_i = \sum_{i=1}^{n} \left[\frac{-\Delta e}{1+e_o}\,\delta h\right]_i \tag{11.11}$$

where

$\Delta\varepsilon_z$ = increment in vertical strain from the increase in vertical effective stresses of sublayer i,
Δe = change in void ratio from the increase in vertical effective stresses(i.e., $\sigma'_f - \sigma'_o$) of sublayer i,
e_o = initial void ratio of the sample corresponding to the initial (or *in-situ*) vertical effective stress σ'_o of sublayer i,
n = number of sublayers, and
δh = thickness of sublayer i.

The negative sign in front of the Δe term is to account for the decrease in void ratio for an increase in effective stress. The change in void ratio depends on the stress history of the soil and magnitude of the final vertical effective stresses relative to the preconsolidation pressure. Final vertical effective stresses can be obtained using elastic solutions (Section 11.6) and incorporating changes in water levels beneath the foundation. If the soil is normally consolidated, then the change in void ratio is equal to:

$$\Delta e = -C_c \log_{10}\left(\frac{\sigma'_f}{\sigma'_o}\right) \tag{11.12}$$

If the soil is overconsolidated and $\sigma'_f < \sigma'_p$, then the change in void ratio is equal to:

$$\Delta e = -C_{cr} \log_{10}\left(\frac{\sigma'_f}{\sigma'_o}\right) \tag{11.13}$$

while if overconsolidated and $\sigma'_f > \sigma'_p$, the change in void ratio is equal to:

$$\Delta e = -C_{cr} \log_{10}\left(\frac{\sigma'_p}{\sigma'_o}\right) - C_c \log_{10}\left(\frac{\sigma'_f}{\sigma'_p}\right) \tag{11.14}$$

Alternatively, one-dimensional settlement can be expressed in terms of the coefficient of volume decrease, m_v:

$$S_{oed} = \sum_{i=1}^{n} \left[\Delta\varepsilon_z \,\delta h\right]_i = \sum_{i=1}^{n} \left[m_v \,\Delta\sigma'_z \,\delta h\right]_i \tag{11.15}$$

The coefficient of volume decrease is the slope obtained from a plot of effective stress (plotted on a linear scale) versus vertical strain obtained from an oedometer test. An appropriate secant value of m_v should be selected for the effective stress increment expected beneath the foundation since m_v is dependent on stress level and stress history. Calculation of one-dimensional settlement is a special case of the more general three-dimensional elastic settlement presented in Section 11.3 where lateral strains are neglected (i.e., $v = 0$) and a one-dimensional constrained modulus ($1/m_v$) is used for the elastic modulus.

11.4.3 Modifications to One-Dimensional Settlement

For foundations with one-dimensional conditions there are no undrained distortions $S_i=0$ and the total final settlement will be equal to the one-dimensional settlement $S_{TF}= S_{oed}$. This would be applicable for foundations where the loaded area is large relative to the thickness of the compressible deposit.

Modification to S_{oed} may be required for foundations with other than one-dimensional conditions (e.g., foundations where lateral strains will occur). For normally consolidated clays, S_{oed} provides a good approximation for the final consolidation settlement S_{CF}, whereas for stiff overconsolidated clays S_{oed} is a good approximation to the total final settlement S_{TF} (Burland et al., 1977; Poulos, 2000). Thus the following modifications are required to one-dimensional settlement theory for applications to two- and three-dimensional condition.

For normally consolidated clays:

$$S_{TF} = S_i + S_{oed} \tag{11.16}$$

For stiff overconsolidated clays.

$$S_{TF} = S_{oed} \tag{11.17}$$

$$S_{CF} = S_{oed} - S_i \tag{11.18}$$

11.5 Local Yield

The undrained distortion of heavily loaded foundations on weak soils may be larger than those calculated using elastic displacement theory because of local ground yield (shear failure) beneath the foundation. The consolidation settlement and the rate of settlement are not greatly affected by local yield (Small et al., 1976; Carter et al. 1979). Based on the results provided by D'Appolonia et al. (1971), local yield may have an influence on undrained distortions for foundations with a global factor of safety against bearing capacity of three or greater ($FS \geq 3$) if:

$$(1 - K_o)\, \sigma'_o < s_u \tag{11.19}$$

where

K_o = coefficient of lateral earth pressure,
σ'_o = initial vertical effective stress beneath the base of the foundation, and
s_u = undrained shear strength of the soil within depth B beneath the base of the foundation (B is the least plan dimension of the foundation).

For cases that satisfy Equation 11.19, the effects of local yield on undrained distortions can be quantified using modification factors for strip foundations reported by D'Appolonia et al. (1971) or by using numerical methods (see Section 11.9). Local yield can be neglected for cases that do not satisfy Equation 11.19.

11.6 Estimating Stress Increments

Increments in total stress beneath a loaded region can be estimated using elastic theory. The following solutions for the stress increments in a homogeneous, isotropic, semi-infinite elastic medium when subject to different loaded areas were obtained from Poulos and Davis (1974). This reference also provides a useful compilation of elastic stress distribution solutions for other loading conditions and nonuniform ground conditions.

11.6.1 Point Load

The stress increments at a point with coordinates r and z beneath a point load of magnitude P on the ground surface (Figure 11.4) are:

$$\sigma_z = \frac{3Pz^3}{2\pi R^5} \tag{11.20a}$$

$$\sigma_r = -\frac{P}{2\pi R^2}\left[\frac{-3r^2 z}{R^3} + \frac{(1-2\nu)R}{R+z}\right] \tag{11.20b}$$

$$\sigma_\theta = -\frac{(1-2\nu)P}{2\pi R^2}\left[\frac{z}{R} - \frac{R}{R+z}\right] \tag{11.20c}$$

$$\tau_{rz} = \frac{3Prz^2}{2\pi R^5} \tag{11.20d}$$

where

σ_z, σ_r, σ_θ and τ_{rz} are the vertical, radial, tangential and shear stresses induced by the point load.

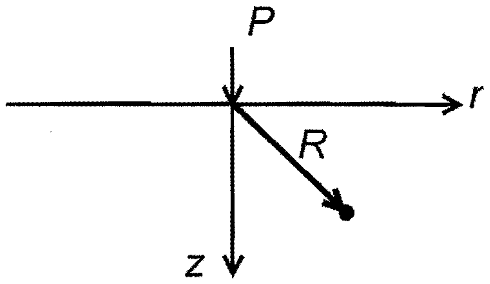


FIGURE 11.4 *Vertical force P acting on the ground surface*

11.6.2 Uniformly Loaded Strip

The stress increments beneath an infinitely long strip of width $2b$ subject to uniform vertical pressure q on the ground surface are:

$$\sigma_z = \frac{q}{\pi}[\alpha + \sin\alpha \cos(\alpha + 2\delta)] \tag{11.21a}$$

$$\sigma_x = \frac{q}{\pi}[\alpha - \sin\alpha \cos(\alpha + 2\delta)] \tag{11.21b}$$

$$\sigma_y = \frac{2q}{\pi}\nu\alpha \tag{11.21c}$$

$$\tau_{xz} = \frac{q}{\pi}\sin\alpha \sin(\alpha + 2\delta) \tag{11.21d}$$

where

σ_z, σ_x, σ_y and τ_{xz} are the vertical, horizontal, axial and shear stresses, and α and δ are angles in radians as shown in Figure 11.5a. Positive angles are counter clockwise from the vertical. Contours of vertical stresses from Equation 11.21a are plotted in Figure 11.5b.

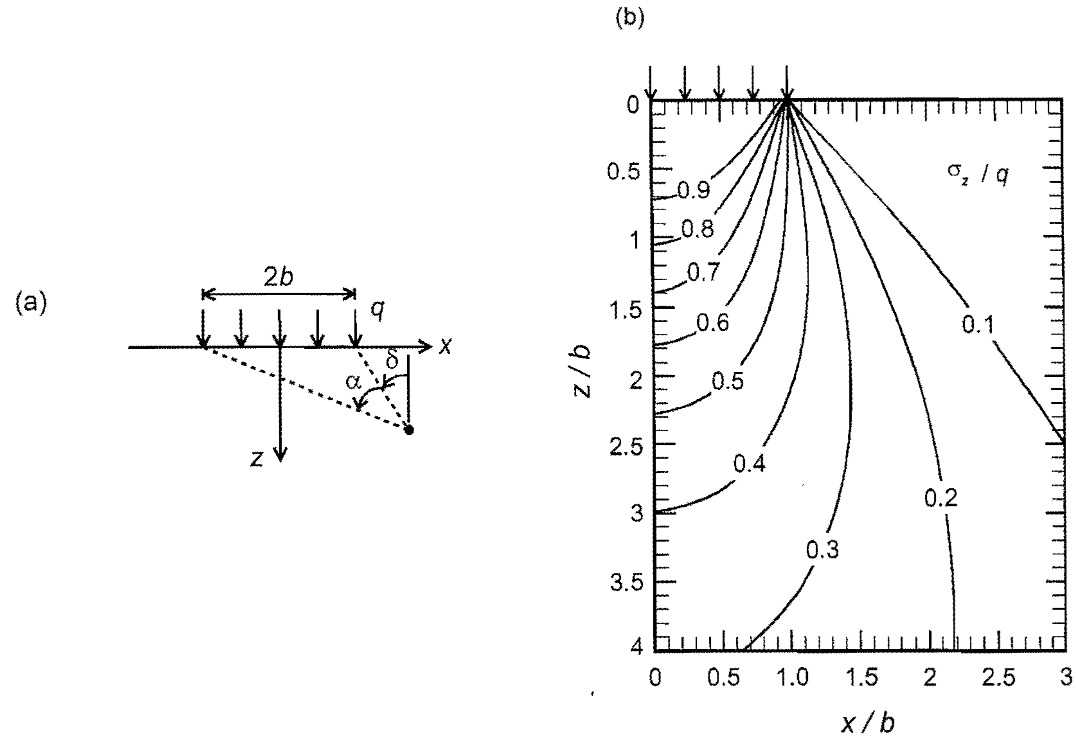


FIGURE 11.5 *Vertical stress σ_z beneath a uniformly loaded strip of width 2b on the ground surface subject to vertical pressure q*

11.6.3 Uniformly Loaded Circle

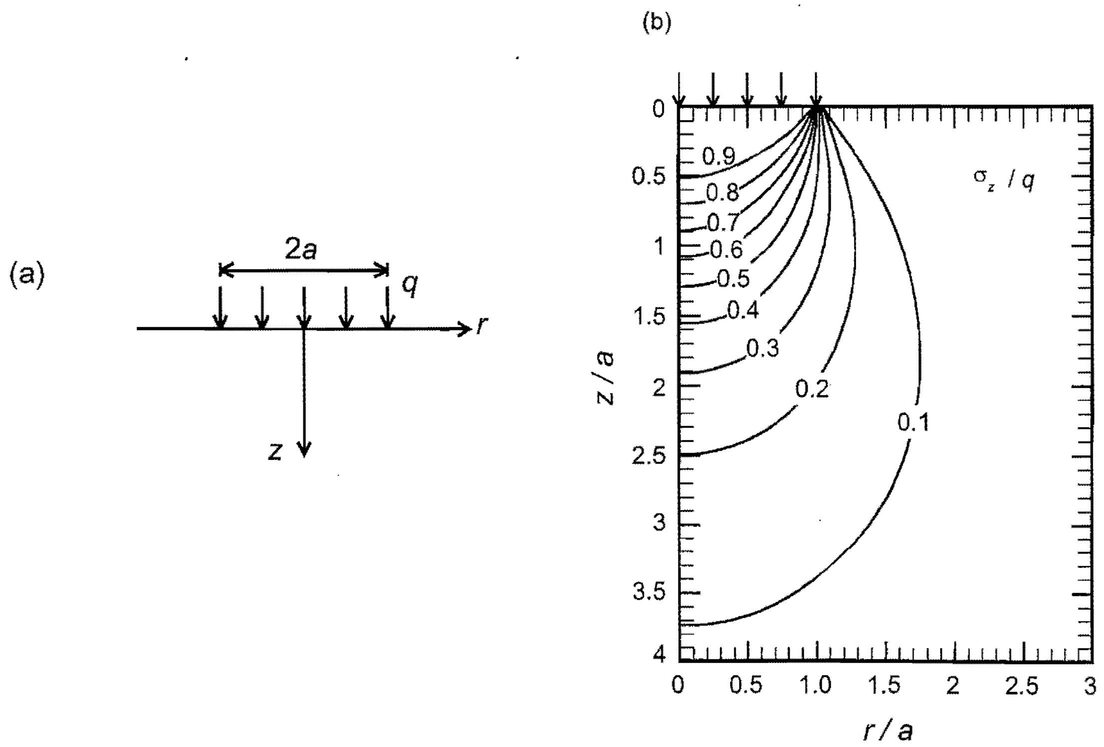


FIGURE 11.6 *Vertical stress σ_z beneath a uniformly loaded circle of diameter 2a on the ground surface subject to vertical pressure q*

The stress increments beneath the centre of a circular area (i.e., along $r = 0$) with radius a subject to uniform vertical pressure q on the ground surface (Figure 11.6a) are:

$$\sigma_z = q\left[1-\left(\frac{1}{1+\left(\frac{a}{z}\right)^2}\right)^{\frac{3}{2}}\right] \tag{11.22a}$$

$$\sigma_r = \sigma_\theta = \frac{q}{2}\left[(1+2\nu)-\frac{2(1+\nu)z}{\left(a^2+z^2\right)^{\frac{1}{2}}}+\frac{z^3}{\left(a^2+z^2\right)^{\frac{3}{2}}}\right] \tag{11.22b}$$

where

σ_z, σ_r, and σ_θ are the vertical, radial and tangential stresses.

The vertical stresses beneath a uniformly loaded circle are plotted in Figure 11.6b.

11.6.4 Uniformly Loaded Rectangle

The stress increments beneath the corner of a rectangle of length L and width b subject to a uniform vertical pressure q on the ground surface (Figure 11.7a) are:

$$\sigma_z = \frac{q}{2\pi}\left[\tan^{-1}\left(\frac{Lb}{zR_3}\right)+\frac{Lbz}{R_3}\left(\frac{1}{R_1^{\,2}}+\frac{1}{R_2^{\,2}}\right)\right] \tag{11.23a}$$

$$\sigma_x = \frac{q}{2\pi}\left[\tan^{-1}\left(\frac{Lb}{zR_3}\right)-\frac{Lbz}{R_1^{\,2}R_3}\right] \tag{11.23b}$$

$$\sigma_y = \frac{q}{2\pi}\left[\tan^{-1}\left(\frac{Lb}{zR_3}\right)-\frac{Lbz}{R_2^{\,2}R_3}\right] \tag{11.23c}$$

$$\tau_{xz} = \frac{q}{2\pi}\left[\frac{b}{R_2}-\frac{z^2b}{R_1^{\,2}R_3}\right] \tag{11.23d}$$

$$\tau_{yz} = \frac{q}{2\pi}\left[\frac{L}{R_1}-\frac{z^2L}{R_2^{\,2}R_3}\right] \tag{11.23e}$$

$$\tau_{xy} = \frac{q}{2\pi}\left[1+\frac{z}{R_3}-z\left(\frac{1}{R_1}-\frac{1}{R_2}\right)\right] \tag{11.23f}$$

where

$$R_1 = \left(L^2+z^2\right)^{\frac{1}{2}} \tag{11.23g}$$

$$R_2 = \left(b^2+z^2\right)^{\frac{1}{2}} \tag{11.23h}$$

$$R_3 = \left(L^2+b^2+z^2\right)^{\frac{1}{2}} \tag{11.23i}$$

and σ_z, σ_x, and σ_y are the vertical, horizontal and axial stresses and τ_{xz}, τ_{yz} and τ_{xy} are the shear stresses.

Alternatively, the vertical stress beneath the corner of a uniformly loaded rectangle is given by:

$$\sigma_z = qI_R$$

where

I_R is an influence coefficient plotted in Figure 11.7b. The stress at points other than beneath the corner of the rectangle can be obtained from linear superposition (i.e., addition and/or subtraction of influence coefficients). For example, the stress beneath the centre of a rectangle with dimensions $2L$ by $2b$ is equal to four times the stress beneath the corner of a rectangle with length L and width b.

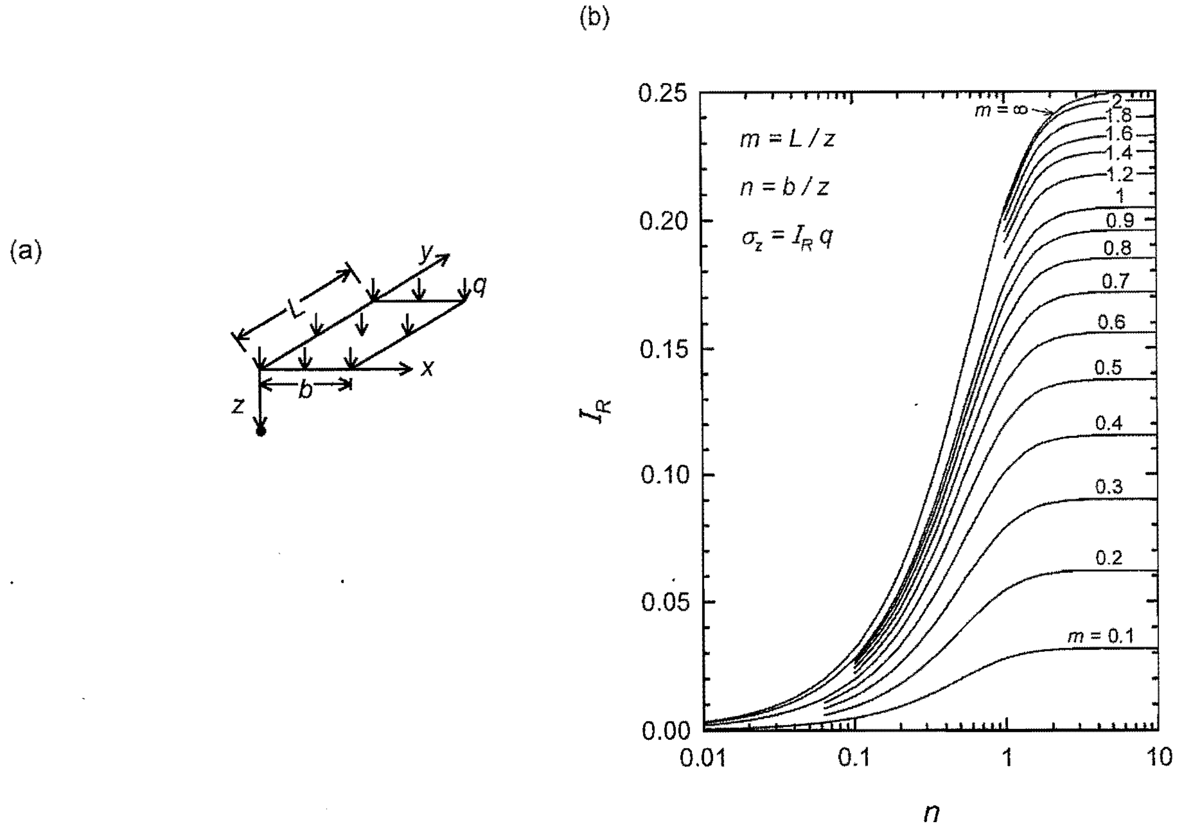


FIGURE 11.7 *Vertical stress σ_z beneath the corner of a uniformly loaded rectangle of width b and length L on the ground surface subject to vertical pressure q*

11.7 Obtaining Settlement Parameters

Selection of ground stiffness or compressibility parameters is an important step to estimate the settlement of shallow foundations. For example, this may involve obtaining estimates of drained and undrained moduli (E', ν', and E_u) for use in the elastic displacement methods presented in Section 11.3. Compressibility parameters for use in the one-dimensional e–logσ' method (C_c, and C_{cr}) are normally obtained from the oedometer test (Section 11.4.1).

The soil parameters for input into any settlement calculations should not be viewed as constants but rather dependent on many factors including: ground conditions, geologic setting, type of foundation (i.e., shallow or deep) and nature of loading. Engineering judgement is required in the selection of stiffness parameters, and consequently, they should always be selected by a qualified and experienced engineer.

Settlement parameters may be estimated using several different methods ranging from empirical correlations with penetration tests (e.g., SPT, CPT), to laboratory tests on high-quality samples from the field (oedometer, triaxial testing), to field testing directed at obtaining parameters for shallow foundations (e.g., plate load tests, measurements of shear wave velocity). Becker (2001) provides a summary of available field (*in-situ*) testing methods. Often settlement parameters are assessed using different methods to provide a bound on the parameters and to check for consistency between values.

The extent of the testing involved in the selection of settlement parameters can be based on the risk of the foundation project. For foundations projects of low-risk (e.g., those involving few hazards with a low probability of occurrence and limited consequences), parameter selection may be largely based on assessed values based on past local experience or on empirical correlations with standard penetration test (SPT) blow count and/or cone penetration test (CPT) tip resistance. Whereas for medium and higher risk projects, in addition to the use of empirical correlations, laboratory testing on high-quality samples and/or specialized field (*in-situ*) testing are normally warranted.

Values of drained Poisson's ratio are not normally measured for most foundation projects, but rather estimated from published values for similar soil and strain levels. Mayne and Poulos (1999) suggest that a Poisson's ratio between $0.1 < \nu' < 0.2$ can be used for both fine and coarse-grained soil for the strain levels expected beneath shallow foundations.

Field penetration results from the Standard Penetration Test (SPT) blow count and/or Cone Penetration Test (CPT) tip resistance are often available from the site investigation. Both of these penetration tests do not provide direct measurements of soil stiffness since they do not simulate the stress path or strain level of shallow foundations. Empirical correlations exist between SPT results and modulus for coarse-grained soils (e.g., Berardi and Lancellotta, 1991) and may be used as an initial guide. CPT data is generally more reliable and reproducible compared to the SPT. Available correlations between CPT tip resistance and modulus for coarse-grained soils can be found elsewhere (e.g., see Baldi et al., 1989). Modulus can also be inferred from field results from the flat dilatometer test (DMT) or the pressuremeter test (PMT), e.g., see Lunne et al. (1989), Marchetti et al. (2001). Regardless of the testing procedure, such correlations should not be extrapolated to ground conditions different from those that they were derived for (e.g., soil type, fines content, stress history, etc.). All empirical correlations need to be treated with caution and adjusted as appropriate by experience.

Both drained and undrained moduli can be obtained from laboratory triaxial testing on high quality samples with values selected over the appropriate stress range. The challenges of obtaining undisturbed samples of coarse-grained soils often preclude laboratory testing on these materials for most foundation projects. Drained modulus for use in three-dimensional calculations can also be estimated from the constrained modulus ($D'=1/m_v$) from oedometer test results. For an isotropic elastic material the drained Young's modulus for three-dimensional conditions E' is related to the one-dimensional constrained modulus D' by:

$$E' = \frac{(1+\nu')(1-2\nu')D'}{(1-\nu')} \tag{11.24}$$

For fine-grained soils, correlations have been developed relating undrained modulus to undrained shear strength and have been summarized by Lade (2001).

For both fine- and coarse-grained soils, values of drained and undrained moduli can also be related to the small strain shear modulus G_{max}. The small strain shear modulus is the same for static and dynamic loading, characterizes both drained and undrained deformations, and is relatively insensitive to OCR of both sands and natural clays (e.g., see Poulos et al., 2001; Burland, 1989). The small strain shear modulus can be obtained from the shear wave velocity V_s and total mass density of the soil ρ_T via:

$$G_{\max} = \rho_T V_s^2 \tag{11.25}$$

Shear wave velocity can be measured in the field from seismic cone penetration test (sCPT, e.g., see Lunne et al., 1997), or from cross-hole wave tests (ASTM D4428). Shear modulus G decreases from G_{max} as shear strains increase. Consequently, adjustments to modulus depending on level of stress or strain of the foundation can then be made (e.g., see Fahey and Carter, 1993; Lehane and Fahey, 2002). Poulos et al. (2001) provide a summary of findings on shear modulus dependence on strain level and propose a simple framework to incorporate these into practical estimations of foundation settlement.

For fine-grained soils, undrained modulus E_u can be obtained from shear modulus using:

$$E_u = 3G \tag{11.26}$$

For stiff overconsolidated clays, the drained modulus E' can be found from G using the relationship for an ideal elastic material:

$$E' = 2(1 + \nu')\,G \tag{11.27}$$

Equations 11.26 and 11.27 can be used to relate the drained and undrained moduli for an overconsolidated clay. For soft compressible clays, the ratio of drained to undrained moduli may be much smaller than that derived from elastic theory, with the ratio becoming smaller as the soil becomes more compressible.

11.8 Settlement of Coarse-grained Soils Directly from In-Situ Testing

11.8.1 Standard Penetration Test (SPT)

a) Method of Peck et al. (1974)

Peck et al. (1974) provided an empirical chart that relates the design bearing pressure for a foundation on sand with the results from Standard Penetration Test (SPT) N resistance, foundation width and foundation embedment as given in Figure 11.8. The design bearing pressures from Figure 11.8 are expected to produce settlements smaller than 25 mm. This figure can be used to estimate preliminarily geotechnical bearing resistance at serviceability limits not exceeding 25 mm of total settlement.

SPT-N values need to be adjusted for depth (overburden pressure effects) using the relationship in Figure 11.8d before using Figures 11.8 a-c.

A representative value of SPT-N should be used to a depth of B beneath the foundation. This approach was developed from field data gathered prior to the 1970s, thus N probably is for an energy ratio of 50-55 %. This approach was also developed for conditions where the groundwater level is located deep beneath the foundation elevation. If the groundwater level rises to the ground surface, no more than half the pressure values indicated in Figure 11.8 should be used. For intermediate positions of the groundwater level (i.e., $0 < z \leq D+B$) the design bearing pressure from Figure 11.8 can be multiplied by the factor C_w, given by:

$$C_w = 0.5 + 0.5\frac{z}{D+B} \tag{11.28}$$

where

z is the depth to the groundwater level and D is the depth to the underside of the foundation, both relative to the ground surface.

Estimates of design bearing pressure from Figure 11.8 are generally viewed as being conservative. Tan and Duncan (1991) found that the results using the method of Peck et al. (1974) were not very accurate as they overestimated settlements by an average factor of 2.7 when compared with 76 cases involving shallow foundations on sand (with $B < 10$ m). Although inaccurate, Tan and Duncan (1991) also found this approach to be reliable, as settlements were underestimated in only 20 % of the 76 cases. Consequently with appropriate engineering judgement, the approach of Peck et al. (1974) may be suitable for foundation design of low risk projects and assessing geotechnical bearing resistance at serviceability limit states not exceeding 25 mm of total settlement.

b) Method of Burland and Burbidge (1985)

Burland and Burbidge (1985) provided an empirical expression to obtain the settlement for foundations on normally consolidated coarse-grained soils from SPT data that can be expressed as:

$$S = B^{0.75}\frac{1.6}{\overline{N}_{60}^{\,1.4}}q \tag{11.29}$$

where

- S = drained settlement (mm),
- $\overline{N}_{60}$ = average standard penetration resistance within depth $B^{0.75}$ beneath the foundation adjusted to energy ratio of 60 %,
- B = width of foundation (m), and
- q = average pressure applied to the ground by the foundation (kPa).

In this approach, the SPT-N value is adjusted to an energy ratio of 60 % (e.g., see Terzaghi et al., 1996) but it is not necessary to modify the value of N for overburden effects. When selecting the average penetration resistance, N_{60} should be adjusted for very dense fine or silty sands using:

$$N'_{60} = 15 + \frac{1}{2}(N_{60} - 15) \tag{11.30}$$

If the thickness of the compressible coarse-grained layer is less than $B^{0.75}$ the actual thickness can be substituted for $B^{0.75}$ in Equation 11.29. For overconsolidated sands Burland and Burbidge (1985) found that the settlement was approximately one-third of that for normally consolidated sands.

Tan and Duncan (1991) found that the method of Burland and Burbidge (1985) was more accurate (overestimated settlements by a factor of 1.5) but less reliable (underestimated settlements 50 % of the time) than that of Peck et al. (1974) when compared with 76 cases involving shallow foundations on sand.

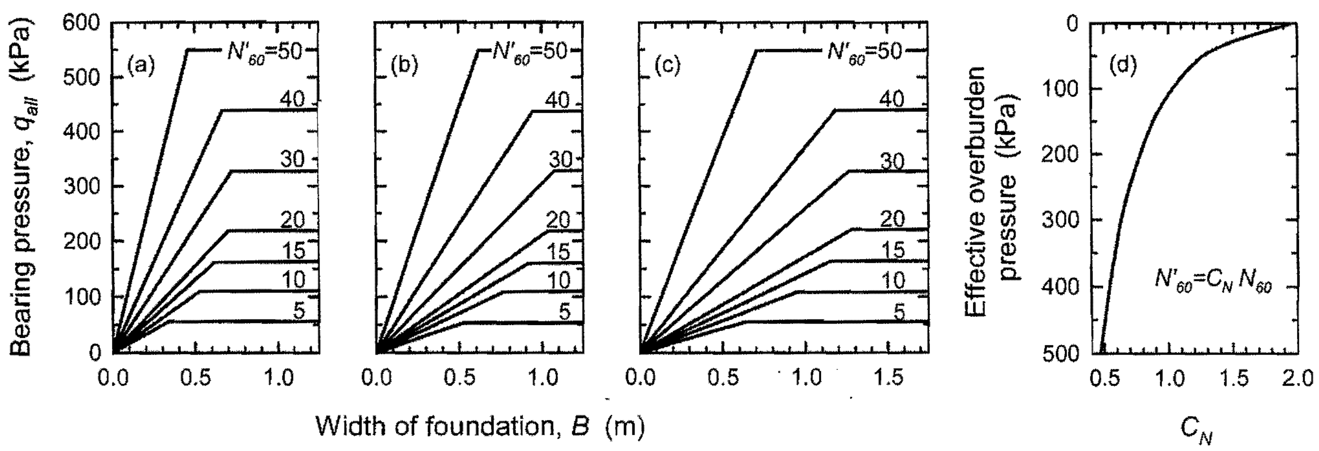


FIGURE 11.8 *Design bearing pressure for foundations on sand for settlement not exceeding 25 mm based on SPT-N results for: (a) D/B=1, (b) D/B=0.5, and (c) D/B=0.25. SPT-N value from field to be modified by factor C_N given in (d) for use in (a)-(c). Modified from Peck et al. (1974)*

11.8.2 Cone Penetration Test (CPT)

Cone penetration test (CPT) tip resistance q_c can be used to estimate foundation settlement in coarse-grained soils using the approach of Schmertmann et al. (1978). This approach uses a simple approximation of elastic strain distribution with the drained modulus obtained from the correlations with the CPT tip resistance. The sand is divided into a number of layers (n) of thickness Δz down to a depth below the base of the foundation equal to $2B$ for a square footing and $4B$ for a strip footing (length of footing, $L > 10B$). A representative value of q_c is assigned to each layer. The settlement is then given by:

$$S = C_1\, C_2\, C_3\, \Delta q \sum_{i=1}^{n} \frac{\Delta z_i}{E'_i} I_z \qquad (11.31)$$

where

C_1 = factor to allow for strain relief from embedment,

$$= 1 - 0.5\frac{q'_s}{\Delta q} \qquad (11.32)$$

C_2 = factor to account for creep and cyclic loading,

$$= 1 + 0.2\log_{10}\left(\frac{t}{0.1}\right) \qquad (11.33)$$

C_3 = factor to account for foundation shape,

$$= 1.03 - 0.03\left(\frac{L}{B}\right) \geq 0.73\text{, for strip foundations} \qquad (11.34)$$

= 1.0, for circular and square foundations,

Δq = net foundation pressure = q - q'_s,

q = average pressure applied to the ground by the foundation,

q'_s = initial vertical effective stress at foundation depth D,

t = time since load application in years,

I_z = strain influence factor (see Figure 11.9),

Δz_i = thickness of layer i,

E' = modulus of the sand for layer i ,

= $3.5q_c$ for strip footings ($L/B > 10$), or (11.35a)

= $2.5q_c$ for square or circular footings ($L/B = 1$), and (11.35b)

q_c = average CPT tip resistance for each layer.

The triangular distributions used to approximate the vertical strains with depth are given in Figure 11.9.

The peak value of the strain influence factor (I_{zp}) occurs at a depth (z_{Dp}) of $B/2$ beneath square or circular foundations and a depth of B beneath strip foundations, and has a value given by:

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta q}{q'_p}} \qquad (11.36)$$

where

q'_p is the initial vertical effective stress at the depth corresponding to the peak value of strain.

A useful refinement to this method would be to use the actual strain distribution beneath the foundation given by elastic theory in Section 11.3 instead of the triangular approximation, which could be readily programmed in a spreadsheet for easy calculation (e.g., see Mayne and Poulos, 1999).

The modulus values obtained with the correlation with q_c are reasonable for recent normally consolidated sands. Estimates of sand modulus from q_c can also be obtained from Baldi et al. (1989) as a function of the degree of loading, soil density, stress history, cementation, age, grain shape and mineralogy. These correlations suggest ratios of E'/q_c from 2 to 4 for recent, normally consolidated sands; 4 to 6 for aged (>1000 years), normally consolidated sands; and 6 to 20 for overconsolidated sands.

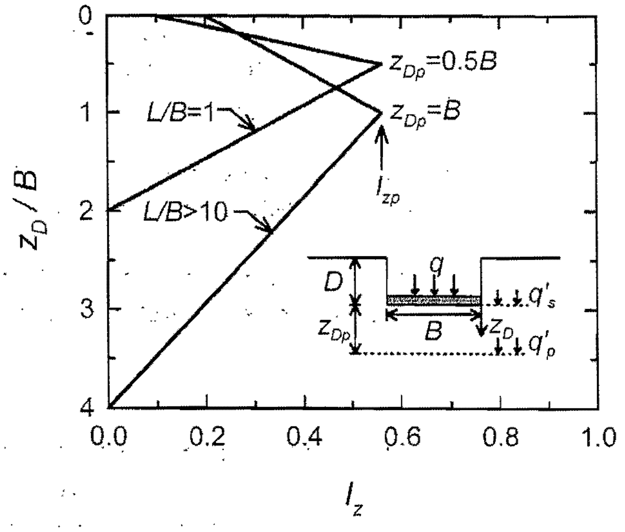


FIGURE 11.9 *Influence factor I_z for estimating settlement of foundation on sand using Schmertmann's method (modified from Schmertmann et al. 1978)*

11.9 Numerical Methods

Finite element or finite difference numerical methods may also be used to estimate foundation settlement. Numerical methods provide the opportunity to model complex ground conditions (if known). It is also possible to model both the structure and the ground (and the associated interactions between the two) which may provide additional information on the influence of intermediate foundation rigidity on ground response and/or the structural response of the foundation (e.g., bending moments, shear forces, deflections) for use in structural design. More elaborate constitutive relations for the ground may be employed in a numerical method to possibly better capture the influence of soil nonlinearity or yielding; however, this requires knowledge of the constitutive relationship and the ability to measure the required parameters.

Numerical methods may be more appropriate for medium and high-risk foundations where there is sufficient data available to justify more elaborate analysis. Additionally the finite element mesh or finite difference grid should have sufficient refinement to correctly approximate stresses and displacements.

Mesh refinement can readily be verified by conducting analyses with progressively increased mesh refinement until there is negligible change in the numerical solution. Consideration must also be given to the selection of boundary conditions such that they adequately model the foundation. The elastic solutions presented in Section 11.3.4 provide simple solutions that may be used to verify the results from numerical analyses.

11.10 Creep

For fine-grained soils, laboratory and field data suggest that creep (i.e., secondary compression) displacements occur simultaneously with primary consolidation (Leroueil, 1996). For most practical cases involving low compressible clays with $C_c/(1+e_o) < 0.25$, creep during primary consolidation does not need to be explicitly calculated. Consequently, creep settlements are added to the total final settlement to account for displacements of the foundation when the effective stresses are constant (i.e., at the end of primary consolidation).

Foundation displacements from secondary compression at time t can be estimated from:

$$S_{sc} = \frac{C_a}{1+e_o} H_o \log_{10}\left(\frac{t}{t_p}\right) \tag{11.37}$$

where

C_α = secondary compression index in terms of void ratio,
t_p = duration of primary consolidation, and
H_o = thickness of compressible layer.

Values of C_α may be obtained from the oedometer test. Often a reasonable estimate for normally consolidated inorganic clays and silts is equal to $0.04C_c$, with values for other ground types reported in Terzaghi et al. (1996).

For highly compressible clays with $C_c/(1+e_o) > 0.25$, viscous effects may contribute to foundation displacements during the time frame of primary consolidation and may be estimated as discussed by Leroueil (1996).

11.11 Rate of Settlement

The rate of settlement may be of importance for foundations on fine-grained soils and depends on how quickly excess pore pressure can dissipate. Generally the rate of settlement depends on the type of soil, hydraulic conductivity of the soil, and drainage boundary conditions. The rate of settlement is quantified by the average degree of consolidation U for use in Equation 11.2 and may be obtained using one- or three- dimensional consolidation theories depending on the foundation conditions.

11.11.1 One-Dimensional Consolidation

One-dimensional consolidation theory of Terzaghi (for details see Terzaghi et al., 1996) assumes that pore pressures can dissipate only in a vertical direction (i.e., there is no lateral flow). It may be used to estimate the rate of settlement for foundations where the assumption of one-dimensional drainage may be reasonable (e.g., foundations where the surface load is large relative to the layer thickness).

The average degree of consolidation U obtained from Terzaghi's one-dimensional theory is plotted in Figure 11.10 versus dimensionless time factor, T_v, where:

$$T_v = \frac{c_v\, t}{H^2} \qquad (11.38)$$

and

c_v = one-dimensional coefficient of consolidation,
t = time, and
H = drainage path of the consolidating layer.

The one-dimensional coefficient of consolidation is normally obtained from oedometer results for load increments taken over the appropriate stress range (for the graphical procedures see Holtz and Kovacs, 1981) and may also be estimated from *in-situ* cone penetration tests (CPT) with pore pressure measurements (e.g., see Lunne et al., 1997).

The drainage path H relates to the boundary conditions above and below the consolidating layer. Conditions of two-way drainage exist if excess pore pressures can dissipate at the top and bottom of the consolidating layer and the drainage path would be equal to one-half of the thickness of the consolidating layer. One-way drainage conditions exist if the excess pore pressures can only dissipate to one of the layer boundaries and H is equal to the thickness of the consolidating layer. Figure 11.10 may be used for conditions involving two-way drainage with initial linearly distributed excess pore pressures and for one-way drainage where the initial excess pore pressures are uniform throughout the consolidating layer.

The average degree of consolidation in Figure 11.10 was obtained assuming that the foundation load was rapidly applied and then held constant. An estimate of the influence of gradual loading on the rate of consolidation is given by Terzaghi et al. (1996).

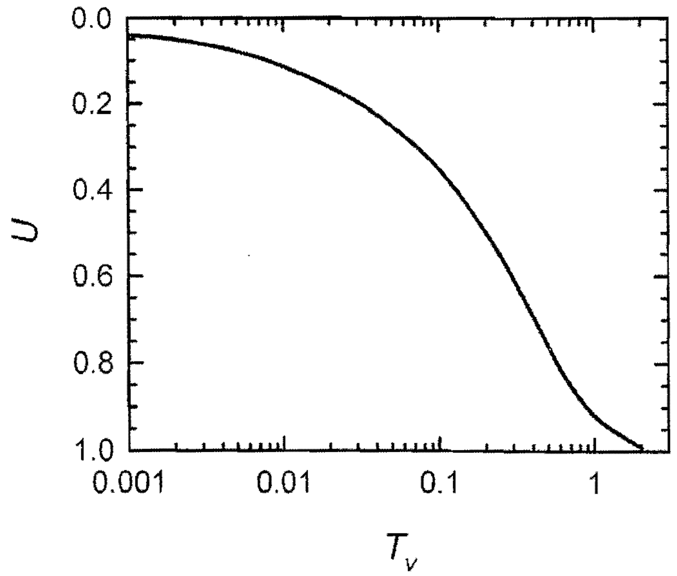


FIGURE 11.10 *Average degree of consolidation for one-dimensional conditions. Modified from Tergazhi et al. (1996)*

11.11.2 Three-Dimensional Consolidation

For many practical foundations, lateral flow of water will occur and consequently Terzaghi's one-dimensional consolidation solution will underestimate the rate of settlement with time. Other factors being equal, smaller foundations will settle faster given the ability of excess pore pressures to dissipate laterally and vertically.

The approximate solutions of Davis and Poulos (1972) may be used to estimate the degree of settlement for two- and three-dimensional drainage. Alternatively, using the solutions of Davis and Poulos (1972), the coefficient of consolidation for use in one-dimensional consolidation theory can be modified to approximately account for three dimensional effects (Poulos, 2000):

$$c_{ve} = R_f \, c_v \tag{11.39}$$

where

c_v = one-dimensional coefficient of consolidation (e.g., obtained from odeometer results over the appropriate stress range),
c_{ve} = modified coefficient of consolidation (for use in Figure 11.10), and
R_f = modification factor to account for three-dimensional effects.

Factor R_f(i.e., c_{ve} / c_v)is plotted in Figure 11.11 and is presented for both strip and circular foundations and for three combinations of drainage boundary conditions that may be encountered in practice that are denoted as:

PT = permeable top surface,
PB = permeable bottom surface,
IF = impermeable foundation, and
IB = impermeable base.

Square foundations can be approximated as a circle. An approximation for rectangular foundations is given by Davis and Poulos (1972). Modifications to account for anisotropic permeability of the consolidating soil are also given by Davis and Poulos (1972).

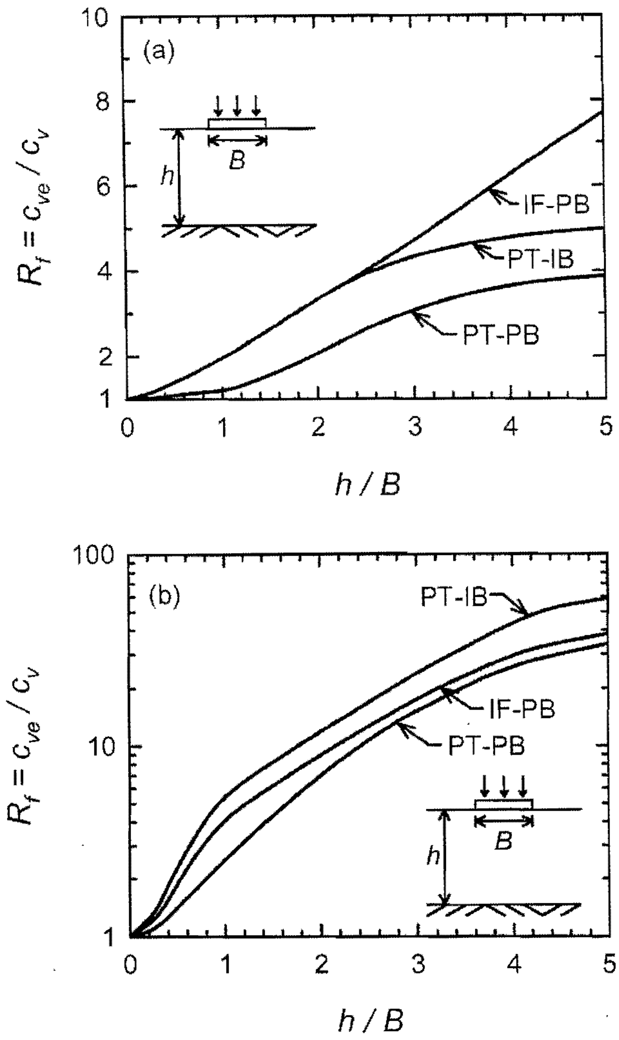


FIGURE 11.11 *Equivalent coefficient of consolidation c_{ve} for use in one-dimensional rate of settlement analysis to account for three-dimensional effects for: (a) strip foundation of width B and (b) circular foundation of diameter B on a uniform layer of consolidating soil of thickness h. Modified from Poulos (2000)*

11.11.3 Numerical Methods

The rate of settlement can also be obtained by employing numerical methods. This approach may only be appropriate for medium and higher risk projects where there is sufficient data to warrant more elaborate analysis. Numerical methods can solve the equations of Biot (1941) to calculate both changes in stresses and pore pressures in response to applied loads. More realistic constitutive models can be used for the soil to characterize effects such as the decrease in hydraulic conductivity with decreases in void ratio during consolidation, as well as possible viscous effects of the soil. Either finite element or finite difference numerical approximations may be employed. It is important to have sufficient refinement of finite element mesh or finite difference grid and sufficiently small time increments to avoid numerical errors.

11.12 Allowable (Tolerable) Settlement

Foundation deflections need to be limited to allowable levels to ensure adequate serviceability of the structure. Figure 11.12 illustrates the types of limiting deflections that need to be considered to avoid damage to the structure. An overlying structure experiences no additional structural loads from a uniform vertical deflection of the foundation (Figure 11.12a). However, limits on the total settlement of the structure are required to prevent damage to services connected to the building (e.g., gas lines, water and sewer pipes). Differential settlements refer to the case where one portion of the foundation settles more than that at other locations. Differential settlements will occur from differences in loads applied to the foundation and/or from the natural variability of the ground beneath the foundation (e.g., from variations in thickness, presence and stiffness of a compressible layer, depth to bedrock). For framed structures (Figure 11.12b), limiting differential settlements are defined in terms of an allowable angular distortion, which is equal to the differential settlement divided by the distance over which the differential settlement occurs. For unreinforced load bearing walls and panels (Figure 11.12c and d), allowable settlement to limit cracking of the wall is expressed as a deflection ratio, which is equal to the relative sag or hog divided by the length of the wall. Overall and local tilt of the structure may also need to be limited (Figure 11.12e).

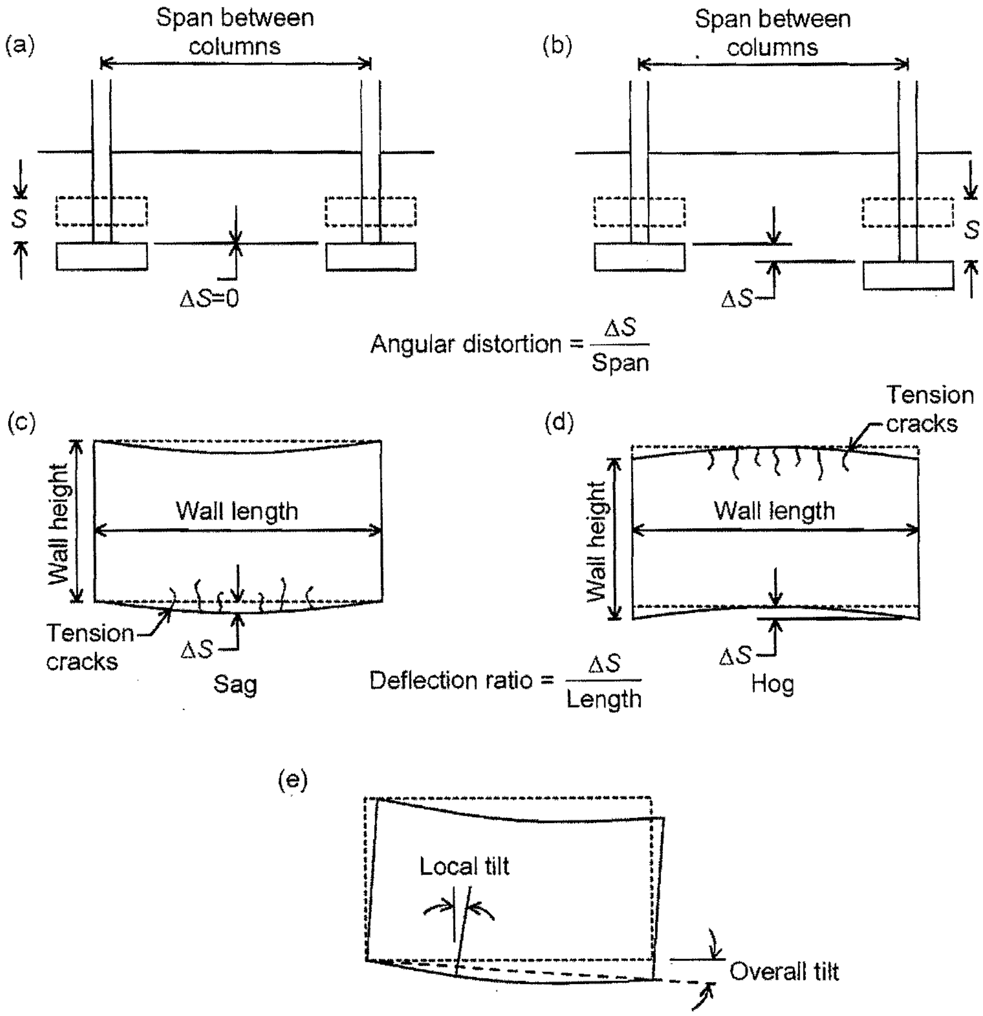


FIGURE 11.12 *Illustration of types of tolerable settlements for shallow foundations. Dashed lines indicate undeflected position of structure. Modified from Burland and Wroth (1974)*

Tolerable limits on foundation deflections listed in Table 11.1 may be used for low risk projects and as an initial guide for higher risk projects. For higher risk projects, consideration should be given to (Boone, 1996): the configuration, flexural and shear stiffness of the building sections; nature of the ground deflection profile; location of the structure relative to the deflection profile; and possible slip between the foundation and the ground. The values cited in Table 11.1 and in Boone (1996) provide realistic estimates of tolerable settlement. They should not, however, preclude specific structural assessment of tolerable settlement of a given building or structure. Communication between the structural and geotechnical engineer is encouraged to address adequately appropriate serviceability limit states criteria.

TABLE 11.1 *Guidelines for Limiting Settlement of Framed Buildings and Load Bearing Walls (adapted from Poulos et al., 2001)*

Type Of Damage	Criterion	Limiting Value
Structural damage	Angular distortion	1/150 – 1/250
Cracking in walls and partitions	Angular distortion	1/500 1/1000 – 1/1400: end bays
Visual appearance	Tilt	1/300
Connection to services	Total settlement	50 – 75 mm: sands 50 – 135 mm: clays
Cracking by relative sag*	Deflection ratio	1/2500: wall length/height=1 1/1250: wall length/height=5
Cracking by relative hog*	Deflection ratio	1/5000: wall length/height=1 1/2500: wall length/height=5

* For unreinforced load bearing walls.

12 Drainage and Filter Design

12 Drainage and Filter Design

12.1 Introduction

Drainage is essential to the performance of earthworks, including slopes, walls and shallow foundations. The drains must provide, over the service life of the structure, a means for the collection and discharge of water that would otherwise impair its performance. The detrimental effects of water on subsurface facilities are manifested in ways that include:

- the ingress and presence of water in locations that were intended to be dry;
- the impact of dissolved salt, which is corrosive to Portland cement concrete; and,
- a reduction of shear strength in the soil as the effective stress diminishes in response to increasing pore water pressure.

Drainage pipes are used to collect and remove subsurface water. The pipes must have structural, hydraulic and durability characteristics that ensure they support the loads to which they are subject during and after construction, while adequately conveying the inflow. Perforated or slotted drainage pipes, into which water seeps, must be protected by filter provisions.

12.2 Filter Provisions

Filter materials, for example one or more specified gradations of coarse-grained soil, or alternatively a geotextile, are used to retain the base soil against which it is placed without adversely impeding subsurface flow from that soil. Accordingly, the filtration process itself is predicated on the development, over time, of a stable interface between base soil and filter material. Geotextile filters are addressed separately in Chapter 23.

A graded granular filter should satisfy the following performance requirements:

1. The voids of the filter should be small enough to restrict particles of the base soil from penetrating or washing through it, fulfilling a criterion of "soil retention."
2. The filter material should be more pervious than the base soil, fulfilling a "permeability criterion."
3. The filter should be sufficiently thick to ensure a representative gradation throughout.
4. The filter should not segregate during processing, handling, placing, spreading or compaction.
5. The filter material should be physically durable, and chemically inert.
6. The filter should not be susceptible to internal instability, whereby seepage flow acts to induce migration of the finer fraction of the gradation.
7. The filter gradation should be compatible with the size, location and distribution of openings in the drainage pipe.

12.3 Filter Design Criteria

Performance requirements are addressed by a series of design criteria. The criteria are empirical, having been established from interpretation of experimental observations, with occasional consideration of theoretical analysis and practical constraints. They are founded on observations of steady unidirectional flow and, accordingly, are appropriate to such conditions in the field. In describing the base soil, its grain size distribution should be determined by wet sieving and without the use of a dispersing agent: the fines fraction so obtained is believed representative of that encountered by the filter (GEO, 1993). Reddi (2003) provides a concise summary of filter requirements in drainage applications, and many of the related design criteria, including a series of worked examples.

12.3.1 Retention Criterion

The pore size distribution of the filter is strongly influenced by its grain size distribution. A pore size that is sufficiently small will restrict the passage of finer grains through the filter. Retention of the base soil is therefore achieved through specifying a maximum value for the ratio of a characteristic grain size of filter (D_{15}) to grain size of base soil (d_{85}). Laboratory testing of Bertram (1940), Karpoff (1955) and Sherard et al. (1984a) confirm the general suitability of a criterion first advocated by Terzaghi in the design of drains for embankment dams, where:

$$D_{15}/d_{85} < 4$$

In a minor variation to the criterion, these studies have led to the recommendation (GEO, 1993) in current practice that filters comprising sands and gravels (D_{15} larger than about 1.0 mm) satisfy:

$$D_{15}/d_{85} < 5$$

Either criterion provides a suitable margin of safety against inadequate retention, the onset of which has been noted to occur at a ratio of D_{15}/d_{85} in excess of ten.

For base soils comprising clays, Sherard et al. (1984b) recommended a sand filter with a D_{15} of 0.5 mm. For sandy clays and silts, the filter criterion $D_{15}/d_{85} < 5$ is reasonable and conservative.

12.3.2 Permeability Criterion

A pore size that is sufficiently large will promote unimpeded flow of water from the base soil, through the filter. Adequate permeability of the filter is therefore achieved through specification of a minimum value for the ratio of a characteristic grain size of filter (D_{15}) to grain size of base soil (d_{15}). Terzaghi first advocated a ratio for base soils,

where

$$D_{15}/d_{15} > 4$$

Recognizing that permeability is, to some extent, a function of the square of the D_{15}/d_{15} ratio, a relative permeability of about 25 is implied by the recommendation for current practice (GEO, 1993) that:

$$D_{15}/d_{15} > 5$$

12.3.3 Other Design Considerations

The following suggestions are made, based on experience reported in the literature, to address additional considerations arising from the requirements of a filter:

- The filter should be sufficiently thick to ensure a representative gradation in the region of inflow. Accordingly, the minimum thickness is strongly influenced by the size of the larger grains. While no specific criterion exists, it is suggested the filter be at least 300 mm thick, to ensure a reasonably consistent distribution of grains.
- The filter should not segregate adversely during processing, handling, placing, spreading or compaction. Experience shows that susceptibility to segregation increases with the range in grain size, and the maximum particle size. The phenomenon is therefore limited by imposing an upper limit on the coefficient of uniformity (C_U), and it is suggested that $C_U < 20$ and $D_{100} < 50$ mm.
- The filter material should be physically durable, and chemically inert. Accordingly, consideration should be given to the mineralogy of the filter material, and its compatibility with the pH of the subsurface water.
- The filter should not be susceptible to internal instability, whereby seepage induces a migration of the finer fraction of the gradation. Experience shows that internal instability is most likely in soils that have a gently inclined gradation in the finer fraction of the grain size distribution, and in soils exhibiting a gap-gradation. Kenney and Lau (1985, 1986) postulate a boundary to internal instability based on the shape of the gradation curve over its finer fraction: the increment of mass fraction (H), over a designated range of grain size (D to 4D) beyond a point on the grading curve (F), defines a ratio H/F that is deemed indicative of potential instability when H/F > 1. It complements an earlier approach (Kezdi, 1979) based on a split gradation analysis, and the principle of soil retention of the finer fraction by the coarser fraction.
- The filter gradation should be compatible with the size, location and distribution of perforations in the drainage pipe. For steady unidirectional flow, experience suggests D_{85} should not exceed the diameter of circular openings, and D_{70} should not exceed the width of slot openings.

12.4 Drainage Pipes and Traps

Drainage pipe must be installed at a slope that is sufficient to induce a flow velocity capable of transporting any fine grains that wash in through the openings of the pipe. The minimum slope is 1%. It is important that traps be installed, which cause the flow to change and result in deposition of suspended solids at locations that can be accessed for purposes of inspection and cleaning. The use of valves may be necessary to ensure flow occurs in the desired direction, and to prevent the possibility of a back-flow in the drainage system.

12.4.1 Construction of Subsurface Drains

Key elements in the configuration of a perimeter drainage system for a shallow foundation are illustrated schematically, for three scenarios, in Figure 12.1. It is important to slope of the base of the trench away from the footing, to slope the wall of the trench such that minor sloughing is avoided during placement of the drain, and to direct surface water away from the trench itself. Intended use of the structure determines the need for damp-proofing the outside face of the wall. A geotextile may be used to separate the foundation soil from that of the filter and backfill: experience does not support wrapping geotextile around the drainage pipe, due to the concentration of flow. It is important to locate the invert of the drainage pipe below the top surface of the basement floor slab. Where concern exists for integrity of the footing, and the efficiency of its bearing action, the invert of the drainage pipe should not be located below the elevation of the footing.

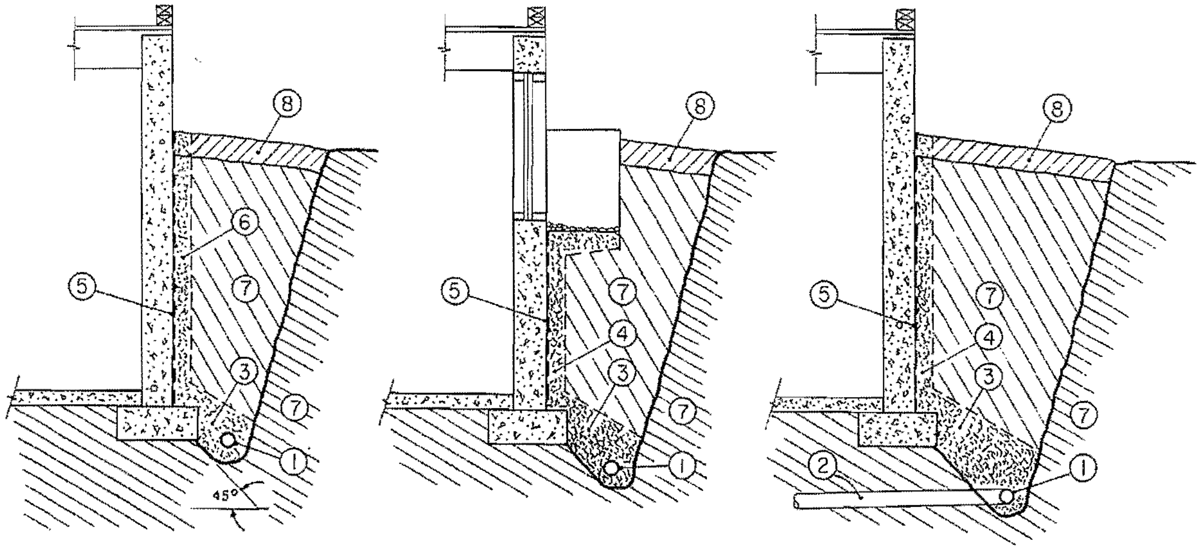


FIGURE 12.1 *Typical Sections Showing Arrangement of Subsurface Perimeter Drains around Shallow Foundations*

(1) perforated or slotted pipe placed about 300 mm below the upper level of the basement floor slab;
(2) unperforated drain pipe connected to appropriate trap and backwater valve before connecting to a sewer. The trap shall have provisions for inspection and cleaning;
(3) filter material that is compatible with the grain size characteristics of the fine-grained foundation and backfill soils, as well as with the perforations of the pipe;
(4) filter material continuously or intermittently placed next to the foundation wall to intercept water from window wells and from low areas near the building (see also 6);
(5) damp-proofing on wall - optional depending on the quality of the concrete wall;
(6) optional use of sheet drain, or synthetic filter blanket, next to the foundation wall to replace the soil filter according to (4);
(7) foundation and backfill soils, which may contain fine-grained and erodible materials; and
(8) "topping-off" material sloping outward to lead off the surface water. It is usually desirable to use low permeability soil to reduce the risk of overloading the pipe.

13 Frost Action

13 Frost Action

13.1 Introduction

The Canadian climate results in freezing of the near-surface ground for several months each winter almost everywhere in Canada. The depth of seasonal frost penetration ranges from minimal to several meters, depending upon local climate, soil conditions and snow cover. Ground freezing frequently results in volumetric expansion of the soil which causes heaving of structures located above or adjacent to the freezing soil. Thaw during the following spring will release the excess water, usually causing loss of strength or complete collapse of the soil structure. This natural seasonal process can be very damaging to infrastructure, such as roads and buried pipelines, and may also cause serious problems for buildings (Crawford, 1968; Penner and Crawford, 1983).

This chapter provides a description of the phenomenon of frost heave, its causes and a brief summary of current predictive capabilities. Guidance is provided for simplified prediction of frost penetration and selection of mitigative design measures. The comments are not intended to deal with structures on a permafrost foundation. A thorough understanding of the nature and distribution of frozen soil is required to predict soil behaviour in permafrost regions. The reader is referred to a comprehensive treatment of this more complex topic such as found in Brown (1970), Andersland and Anderson (1978), Johnston (1981) and Andersland and Ladanyi (2004).

13.2 Ice Segregation in Freezing Soil

Water in soil pores begins to freeze as the temperature is lowered through 0°C. Figure 13.1 illustrates the progressive reduction of unfrozen water content as the relative proportions of water and ice change at sub-zero temperatures for sand, silt and clay. Continued formation of ice in the soil pores at progressively decreasing temperatures confines the remaining water to progressively smaller pore spaces. A pressure differential between the ice and water phases draws water from the unfrozen soil into the freezing soil. Fine-grained soils, which freeze over a broader range of temperature, are particularly susceptible to moisture migration along a pressure gradient, resulting in growth of ice lenses. The resulting heave rate and magnitude depend upon soil type, overburden pressure, groundwater conditions, freezing rate, and other factors. The extent of ice lensing that can occur in a clay soil is illustrated in Figure 13.2.

Where restraint in the form of a building is present, heaving pressures develop that may or may not be able to overcome the restraint. Heaving pressures may be very high, depending upon the restraint offered by the surrounding structure and soil; values equivalent to 1800 kPa were measured on a 300 mm diameter plate (Penner and Gold, 1971).

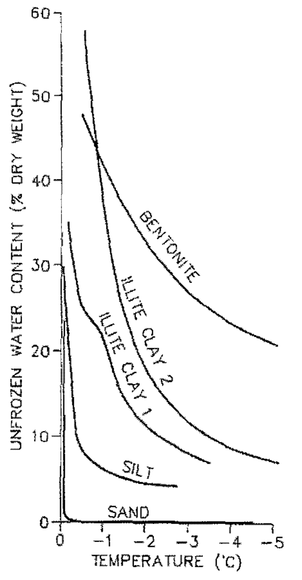


FIGURE 13.1 *Unfrozen water content for a range of frozen soils (after Williams and Smith, 1989)*

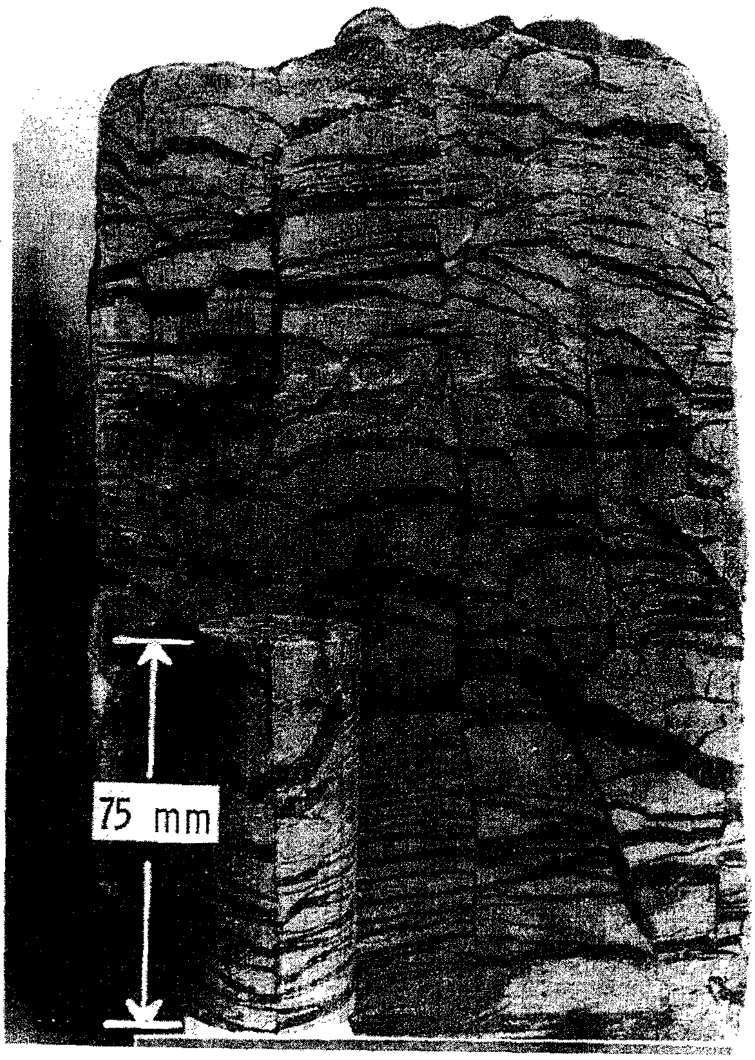


FIGURE 13.2 *Sample of frozen clay showing ice segregation*

The rate of heaving in a frost susceptible soil is limited by the rate of heat extraction from the freezing fringe where water is migrating to feed growing ice lenses. This complex heat and moisture flow phenomenon is normally uncoupled to simplify engineering predictions. Penetration of the freezing isotherm with time and temperature is predicted first by ground thermal analyses without consideration to the impact of moisture redistribution and ice lensing. The predicted extent of frost penetration and knowledge of the thermal gradients that exist within the frozen soil are then used as inputs for prediction of heave magnitudes due to ice segregation.

Engineering methods for predicting ground thermal conditions and frost heave have evolved significantly in the past decade such that practical solution techniques are now available. The remainder of this chapter summarizes current practice in this evolving field together with some practical considerations for mitigating frost heave damage.

13.3 Prediction of Frost Heave Rate

13.3.1 Ice Segregation Models

Several hydrodynamic models have been developed to express the coupled heat and moisture flow that cause frost heave. These models have been reviewed by Nixon (1987, 1991) to evaluate their applicability for practical engineering predictions.

Ice lenses grow within the frozen fringe where the temperature is less than 0°C (Miller, 1978). The temperature of the growing ice lens is related to the overburden pressure (Konrad & Morgenstern, 1982). Ice also forms in the larger pores between the active ice lens and the 0°C isotherm, requiring water to flow through the fringe of partially frozen soil to feed the growing lens. The rate of lens growth is dependent upon the finite hydraulic conductivity of the partially frozen fringe and the rate of heat extraction at the ice lens. All hydrodynamic models therefore relate the velocity of water through the freezing fringe to the temperature gradient, and to the permeability of the partially frozen soil. The heave rate can be computed from the rate of change of the velocity of water in the frozen soil.

A practical method for predicting frost heave magnitude for geotechnical engineering applications was developed by Konrad and Morgenstern (1980). Their semi-empirical formulation does not rely on measurement of the permeability of frozen soils or other physical parameters that characterize the movement of water through the freezing fringe. They relate the water velocity directly to the thermal gradient in the frozen soil. The constant of proportionality is termed the segregation potential (SP). The SP parameter is dependent upon overburden pressure but is considered to be independent of the rate of cooling in the freezing fringe at low cooling rates. The SP parameter must be determined from a series of step temperature freezing tests carried out at various overburden pressures. The tests must reasonably simulate the freezing rates or thermal gradients expected in the field.

The heave rate *(dh/dt)* under field conditions can be predicted from:

$$dh/dt = SP\ G_f + 0.09\ n\ dX/dt \tag{13.1}$$

where

SP	is the segregation potential determined from freezing tests
G_f	is the thermal gradient in the frozen soil at the freezing fringe, determined from geothermal simulations
dX/dt	is the rate of advance of the frost front determined from geothermal simulations,
n	is the soil porosity reduced to account for the percentage of in-situ porewater that remains frozen within the anticipated range of ground temperatures.

A summary of published data relating the SP parameter to overburden pressure for various soils was presented by Nixon (1987), and is shown in Figure 13.3.

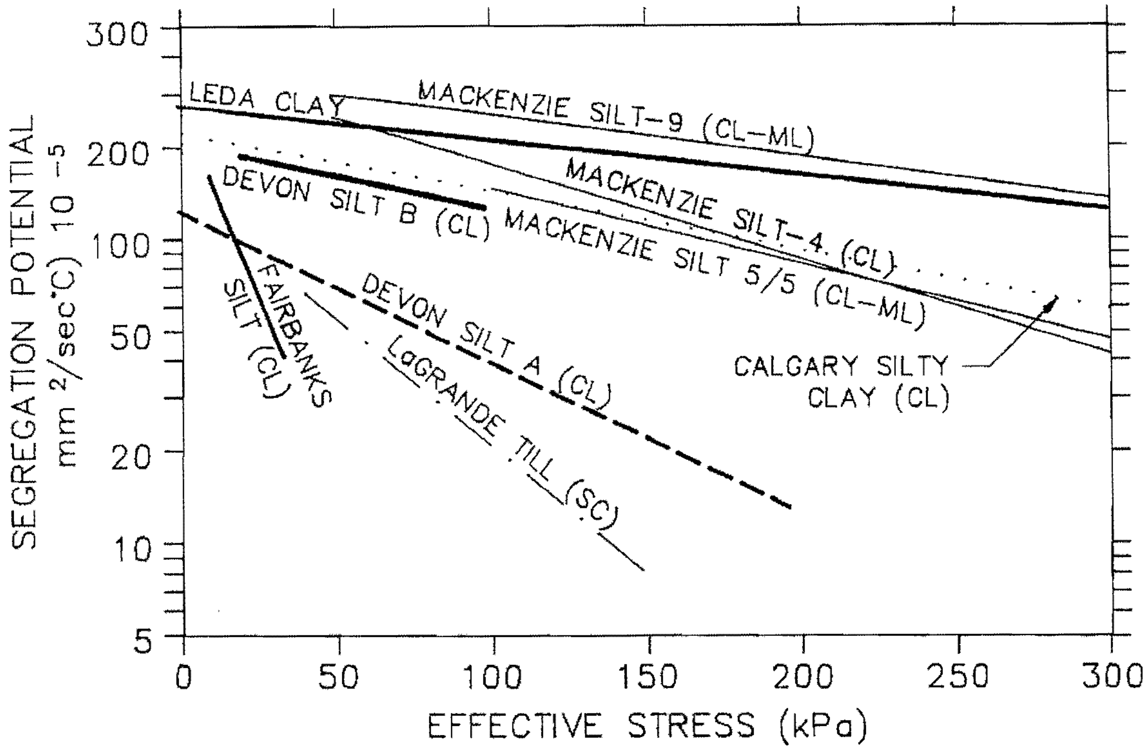


FIGURE 13.3 *Published segregation potential (SP) parameter data (after Nixon, 1987)*

13.3.2 Frost Susceptibility

Frost susceptibility of soils refers to the propensity of the soil to grow ice lenses and heave during freezing. At present, there are no precise criteria for classifying soils according to their frost susceptibility. A common guideline, developed by Casagrande (1932) based on observation and experience, relates frost susceptibility of soils to the percentage of fine fraction less than 0.02 mm.

The Casagrande guide has been extended by the U.S. Corps of Engineers to a widely used classification system, shown in Table 13.1. Soils are listed in four categories, F1 to F4, in approximate increasing order of frost susceptibility and loss of strength during thaw.

Where frost susceptibility and heave are critical parameters in foundation design, laboratory frost heave testing should be carried out. There are no current standards for heave tests; thus, it is important to develop a test program that meets the requirements of the project. This may range from simple confirmation of frost susceptibility and heave rate to determination of specific parameters such as segregation potential (SP) that can be used in a frost heave prediction model.

Frost heave tests are carried out in an insulated freezing cell where precise control can be maintained over temperatures. A sub-zero temperature is applied to the upper or lower sample cap. The other end of the sample may be uncontrolled, insulated or maintained at some positive temperature. The end temperatures might be controlled either as a step temperature change or a time-dependant "ramped" temperature change. The ramped temperature change is chosen if a near-constant freezing rate is desired. The volume of free water drawn into the sample at the unfrozen end cap is measured with time and related to the volumetric increase or sample heave rate. An interpretation of frost heave test data in terms of segregation potential is described by Konrad and Morgenstern (1981).

TABLE 13.1 *U.S. Corps of Engineers Frost Design Soil Classification*

Frost Group	Soil Type	Percentage finer than 0.02 mm, by weight	Typical soil types Under Unified Soil Classification System
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	a) Gravelly soils	>20	GM, GC
	b) Sands, except very fine silty sands	>15	SM, SC
	c) Clays, PI >12	--	CL, CH
F4	a) All silts	--	ML, MH
	b) Very fine silty sands	>15	SM
	c) Clays, PI <12	--	CL, CL-ML
	d) Varved clays and other fined-grained, banded sediments	--	CL and ML; CL, ML, and SM; Cl, CH, and ML; CL, CH, ML, and SM

13.3.3 SP from Soil Index Properties

A comprehensive study conducted by Konrad (1999) established that the segregation potential parameter (SP) of saturated fine-grained soils can be adequately related to a few basic soil index properties. For a soil freezing under zero applied overburden pressure, a reference value of the segregation potential, SP_0, is best empirically related to the mean grain size of the fines fraction (<0.075 mm), $d_{50}(FF)$, the specific surface area of the fines fraction, S_s, and the ratio of water content to the liquid limit, w/w_L as illustrated by Figure 13.4. For a ratio w/w_L close to 0.7, the empirical relationship for clayey silts is:

$$SP_0.S_s = [116 - 75 \log d_{50}(FF)].10^3 \text{ mm}^4/(°C.s.g) \tag{13.2}$$

where $d_{50}(FF)$ is expressed in μm.

In well-graded soils or gap-graded soils, SP is directly proportional to the relative fines content, i.e. the ratio of actual fines and the amount of fines needed to fill all the pore space between the coarser-grained particles. Details on a complete frost-susceptibility assessment methodology is given in Konrad (1999).

Frost susceptibility assessment was recently extended to non-clay soils such as tills and crushed rock by Konrad (2005).

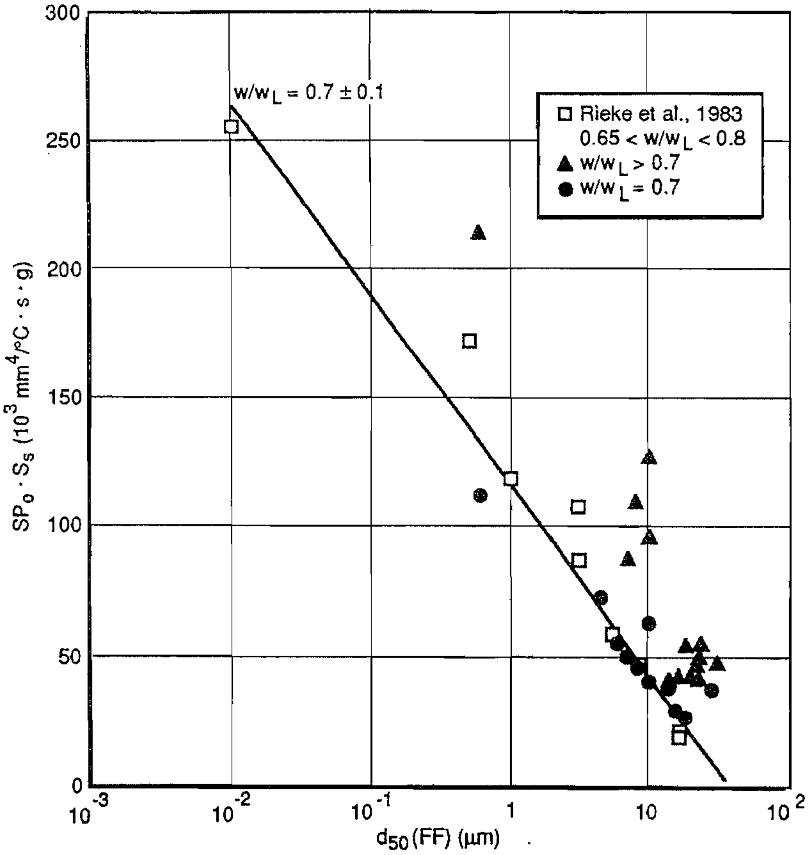


FIGURE 13.4 *Frost susceptibility assessment was recently extended to non-clay soils such as tills and crushed rock by Konrad (2005)*

13.4 Frost Penetration Prediction

13.4.1 Ground Thermal Analyses

The dominant mechanism of heat transfer in soils is thermal conduction. Heat flow in the ground follows Fourier's Law of conduction with a term to account for the release or absorption of latent heat of water during phase change. Heat transfer by mechanisms other than conduction may only be a factor in porous soils where groundwater flow is occurring. Water velocities generally must exceed 10^{-4} cm/s before convective heat flow starts to become significant.

Analytical methods, or closed-form mathematical solutions of the well-known Laplace equation, can provide an approximation of seasonal frost penetration for simple conditions. Prediction of transient ground temperature changes for problems with complex stratigraphy and variable boundary conditions requires solution by numerical methods. Numerical models in common use are either finite difference or finite element solutions. A comprehensive review of numerical methods for ground thermal regime calculations has been provided by Goodrich (1982). Two numerical models in common use in Canada are described by Nixon (1983) and by Hwang (1976).

Numerical methods are required for geotechnical design calculations other than simple prediction of the maximum depth of frost penetration. The usual range of problems involves layered systems, temperature-dependent thermal properties, and time-dependent boundary conditions such as ground surface heat exchange. A realistic simulation of the temperature-dependent liberation or absorption of latent heat during freezing or thawing, associated with the changes in unfrozen water content shown in Figure 13.1, is also an essential feature in any numerical simulation.

Numerical methods are very flexible and can reasonably simulate geotechnical complexities in either one or two dimensions. However, they require familiarity with an appropriate computer program and experience deriving input parameters. The results are normally expressed as temperature isotherms on a two-dimensional plot for various times of interest to the designer. The results can also be expressed as a propagation of the freezing isotherm with time or as a transient thermal gradient which may be input to a subsequent prediction of frost heave in an uncoupled analysis of heat and moisture flow.

13.4.2 Simplified Solutions for Maximum Frost Penetration Neglecting Frost Heave

Frost penetration is proportional to the square root of time for a step change in ground surface temperature. The most useful form of the relationship is the modified Berggren equation as described by Aldrich (1956), Sanger (1963) and Johnston (1981), and shown as Equation 13.3:

$$\mathrm{X} = \lambda\sqrt{\frac{2k_f I_s}{L_s}} \tag{13.3}$$

where

X = depth of frost penetration
I_s = surface freezing index which can be estimated from the air freezine index times a ground surface interface factor "n"
k_f = Thermal conductivity of the frozen soil
L_s = Volumetric latent heat of the soil
λ = A dimensionless coefficient (Figure 13.8)

The surface freezing index expresses the average negative surface temperature and the time over which it applies. The empirical n-factor can be used to determine surface freezing index from the air-freezing index. Published n-factors for various types of surfaces are shown in Table 13.2. The air-freezing index is a summation of the daily mean degree-days for the freezing period. A long-term mean (30 year) air freezing index can be estimated from monthly mean air temperature data published by Environment Canada. Typical variation in air freezing index within Canada is shown in Figure 13.5.

TABLE 13.2 *Values of n-Factors for Different Surfaces (from Johnston, 1981)*

Surface type	Freezing-n
Spruce trees, brush, moss over peat - soil surface	0.29 (under snow)
As above with trees cleared – soil surface	0.25 (under snow)
Turf	0.5 (under snow)
Snow	1.0
Gravel (most probable range)	0.6 – 1.0 (0.9 – 0.95)
Asphalt pavement	0.29 – 1.0 or greater
(most probable range)	(0.9 – 0.95)
Concrete pavement	0.25 – 0.95
(most probable range)	(0.7 – 0.9)

Winter air temperatures vary substantially from year to year everywhere in Canada. Therefore, it is seldom appropriate to use the long-term mean air-freezing index for design purposes.

Common practice is to choose some return period or recurrence interval and to estimate the most severe winter likely to occur within that period. The US Corps of Engineers method, as described by Linell et al. (1963), is to use either the most severe winter of the previous ten years or the average of the three most severe winters in the previous 30 years.

A simple relationship between design freezing index, taken as the coldest winter over the last 10-year period, and mean freezing index was developed by Horn (1987) by curve fitting data for 20 cities across Canada. The relationship is given as:

$$I_d = 100 + 1.29\, I_m \tag{13.4}$$

where

I_d = Design Freezing Index (°C-days)
I_m = Mean Freezing Index (°C-days)

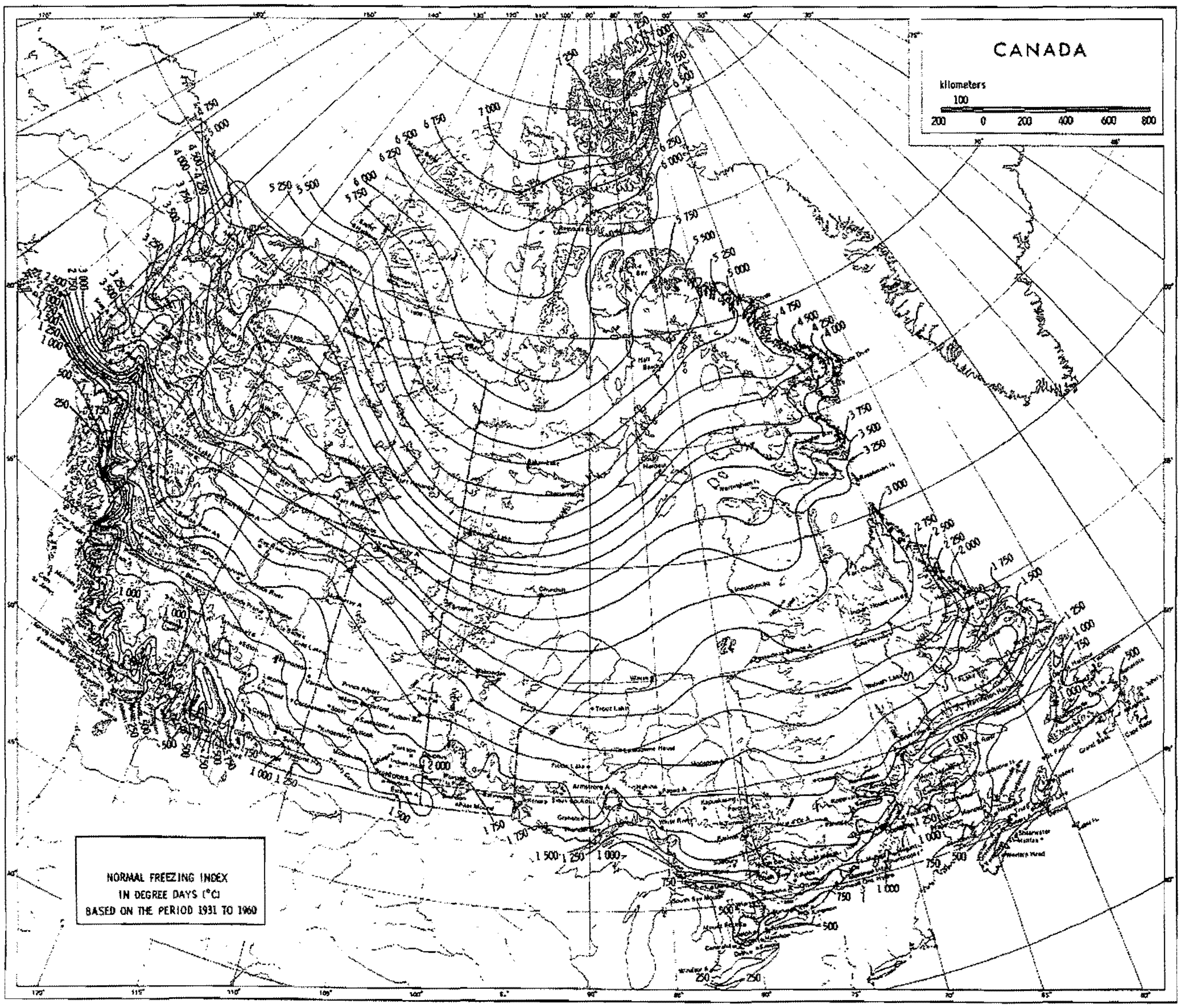


FIGURE 13.6 *Thermal conductivity of frozen coarse-grained soil (after Kersten, 1949)*

This relationship is recommended for the design air freezing index in the absence of an in-depth evaluation of historical climate data. The surface freezing index for the modified Berggren equation then becomes:

$$I_s = n\, I_d \tag{13.5}$$

The thermal conductivity of soil can be estimated from relationships to soil index properties. The relationships developed by Kersten (1949) for frozen coarse and fine-grained soils are shown in Figures 13.6 and 13.7, respectively. Frost penetration depths based on Kersten's relationships for coarse-grained soils may under predict frost depth significantly for unsaturated soils.

The thermal conductivity of coarse-grained soils is also dependent on soil mineralogy. The thermal conductivity of quartz is about four times that of other common soil minerals. The Kersten correlation is only appropriate for sands that have neither a very low nor a very high fraction of quartz particles. A more thorough treatment of soil thermal properties and their variability with index properties and soil constituents has been provided by Farouki (1986). A generalized thermal conductivity model for soils and construction materials is also provided by Côté and Konrad (2005).

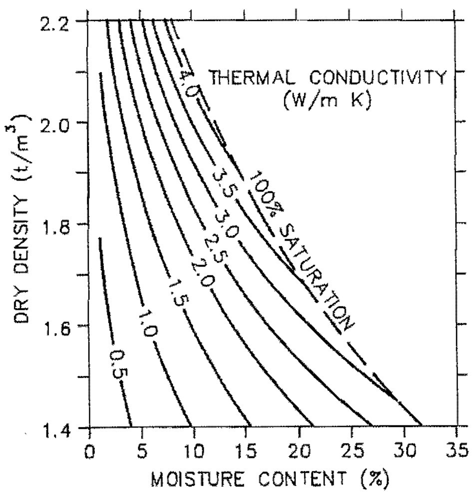


FIGURE 13.6 *Thermal conductivity of frozen coarse-grained soil (after Kersten, 1949)*

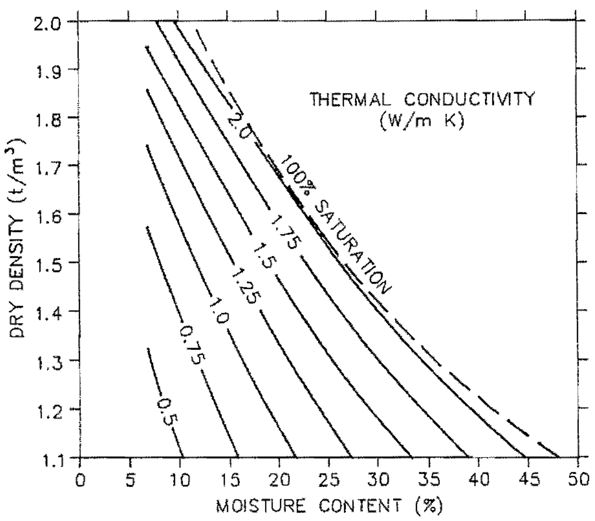


FIGURE 13.7 *Thermal conductivity of frozen fine grained soil (after Kersten, 1949)*

The volumetric latent heat term of the soil (L_s) can be estimated from the relationship:

$$L_s = \gamma_d \, w \, L \tag{13.6}$$

where

γ_d	Is the dry unit weight of the soil
w	Is the gravimetric water content of the soil expressed as a fraction
L	Is the latent heat of fusion of water to ice which can be taken as 334 kJ/kg.

The above relationship for latent heat of the soil, when used in the modified Berggren equation, assumes that all of the water in the soil freezes at 0°C. This will result in under prediction of the freezing depth in fine-grained soils which freeze over a range of temperature, as described in Section 13.2. Alternatively, the volumetric latent heat term can be corrected to account for unfrozen water using the relationships of Figure 13.1 if an average frozen soil temperature can be estimated.

Lambda (λ) is a dimensionless coefficient that is a function of the temperature gradient, the volumetric latent heat of the soil and the volumetric heat capacity of the soil. The coefficient can be determined from a relationship developed by Sanger (1963) shown in Figure 13.8. The dimensionless parameters thermal ratio (β) and fusion parameter (μ) can be determined from:

$$\beta = \frac{MAAT \quad t}{I_S} \quad \text{and} \quad \mu = \frac{CI_S}{Lt}$$

where

$MAAT$	Is the mean annual air temperature (°C) for the site determined from Canadian Climate Normals
t	Is the duration of the freezing period (days)
I_s	Is the ground surface freezing index (°C-days)
C	Is the volumetric heat capacity of the frozen soil

$$C = \gamma_d \, (C_s + C_i w) \tag{13.7}$$

where

C_s	Is the specific heat of dry soil which can be taken as 0.71 kJ/kg °C
C_i	Is the specific heat of ice which can be taken as 2.1 kJ/kg °C
w	Is the gravimetric water content of the soil

For many practical field freezing situations, λ is close to unity. Omitting it from the freezing equation results in a slight over prediction of frost depth.

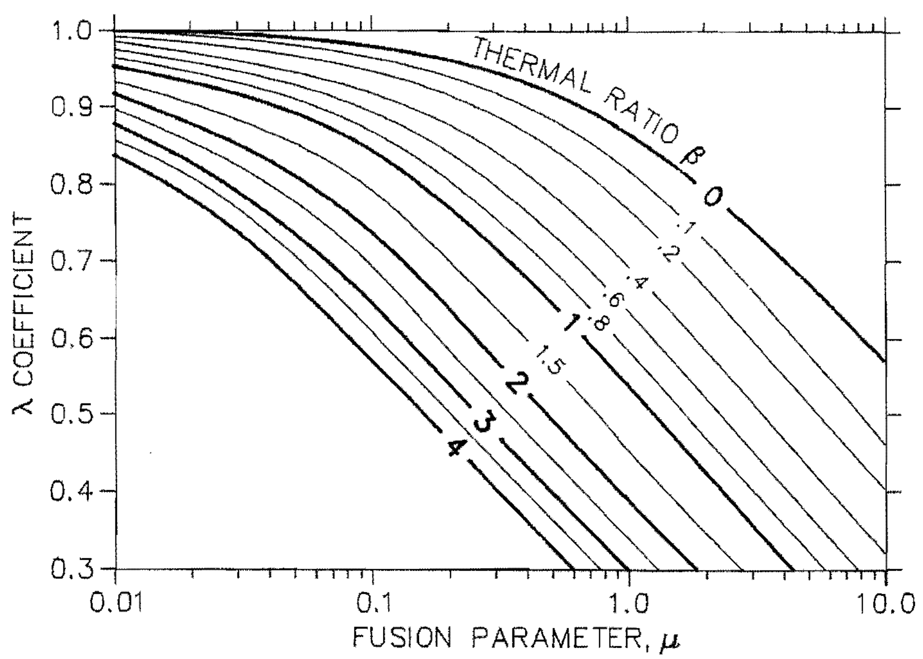


FIGURE 13.8 *Lambda (λ) coefficient for modified Berggren equation (after Sanger, 1963)*

13.4.3 Frost Susceptible Soils

While frost depth in non-frost susceptible soils is readily estimated with the modified Berggren equation, the calculation of frost depth in frost susceptible soils must account for the release of latent heat associated with the formation of ice lenses.

An extension to Stefan's approach yields enough accuracy for practical considerations. Using the segregation potential to quantify the rate of ice formation with Stefan's assumptions gives the modified Stefan equation (Konrad, 2000):

$$X = \sqrt{\frac{2(k_f - SP.L)I_s}{L_s}} \tag{13.8}$$

where

- SP Is the value of the segregation potential in $m^2/s.°C$
- L Is the volumetric latent heat of water, i.e. 334 MJ/m^3
- L_s Is the latent heat of soil (Equation 13.6)
- k_f Is the thermal conductivity of the frozen soil from Kersten's relationship given in Figures 13.6 and 13.7
- I_s Is the ground surface freezing index (°C - days)

13.5 Frost Action and Foundations

The conventional approach for protection of building foundations against frost action is to locate shallow foundations at a depth greater than the design depth of frost penetration. The modified Berggren equation, described in Section 13.4.2, may be used to determine the design depth of frost penetration. This procedure can be used to establish the minimum depth of soil cover over an exterior footing. The depth of perimeter foundation walls for heated structures may be reduced somewhat to account for heat loss from the building. Alternatively, foundation depth for protection against frost action may be specified in local building codes or is frequently determined by local experience. However, caution should be exercised where a significant depth of the footing cover is comprised of dry, coarse-grained soil as frost depths could exceed local experience.

13.5.1 Adfreezing

Soil in contact with shallow foundations can freeze to the foundation, developing a substantial adfreeze bond. Backfill soil that is frost susceptible can heave and transmit uplift forces to the foundation. Spread footings normally have sufficient uplift resistance from their expanded base to resist heave, but the structural design of the wall-footing connection must be sufficient to transmit any load applied through adfreeze. Average adfreeze bond stresses, determined from field experiments, typically range from 65 kPa for fine-grained soils frozen to wood or concrete to 100 kPa for fine-grained soils frozen to steel (Penner, 1974). Design adfreeze bonds for saturated gravel frozen to steel piles can be estimated at 150 kPa (Penner and Goodrich, 1983). The most severe uplift conditions can occur where frost penetrates through frost stable gravel fill into highly frost susceptible soils surrounding a foundation. These conditions result in a heaving situation with maximum adfreeze bond stress and have been known to jack H-piles driven to depths in the order of 13 m (Hayley, 1988).

It is good practice to backfill against foundations with non-frost susceptible soil. Provision should be made for drainage around the foundation perimeter, below the maximum depth of frost penetration. The granular backfill should be capped with less permeable soil and a surface grade provided to shed runoff before it enters the backfill.

13.5.2 Thermal Insulation

It may not always be feasible to place foundation-bearing surfaces below the design depth of frost penetration. Conditions such as high groundwater level or particularly deep predicted frost penetration may make excavation impractical. For these and other cases, thin soil cover may be supplemented with insulating materials. Rigid board insulation, fabricated from extruded polystyrene, is the most common material for subsurface use. This closed cell insulation is manufactured with high compressive strength and a smooth exterior skin to resist deterioration by absorption of moisture. Polystyrene insulation deteriorates rapidly in the presence of hydrocarbons; therefore, alternative materials should be used where the possibility of oil spills exist. A design methodology for insulated foundations has been presented by Robinsky and Bespflug (1973). Summaries of their design charts for heated and unheated structures are shown in Figures 13.10 and 13.11, respectively. These charts can be used as a guide for estimating the required thickness of insulation. However, actual design conditions should be checked using a geothermal analysis of the type described in Section 13.4.1.

Insulation sheets should be placed with minimum soil covers of 300 mm and extend at least 1.2 m out from the building. Deeper placement is warranted in high traffic areas. A sheet of vertical insulation should be fastened to the exterior wall above the horizontal insulation up to the insulated exterior wall. Common practice is to place the required thickness of insulation in two layers with staggered joints and to increase the thickness by 50 to 100 percent at the corners. The surface of the insulation should be sloped such that groundwater contacting the impervious sheets is directed away from the building.

13.5.3 Other Design Considerations

Unheated or partially heated appurtenances to a primary structure are frequently the source of frost heave displacements. Decks, porches and unheated garages often require insulation. Where these structures may be at greater risk of frost heave, they should be separated from the primary structure.

Buildings without basements are often supported on cast-in-place concrete piles with perimeter grade beams. Perimeter concrete grade beams formed and cast on the ground are particularly susceptible to damage by frost action. Uplift forces that may develop under grade beams can be transmitted back to piles resulting in tension failure if reinforcement is not provided. It is common practice to provide cardboard void formers below grade beams where there is a risk of frost action. A minimum thickness of 100 mm is necessary, with greater thicknesses suggested where conditions are anticipated to be severe. Synthetic insulation should not be used as a void former because of its high compressive strength. It is also common practice to make reinforcing in grade beams symmetrical on the top and bottom such that some uplift load can be tolerated without risk of cracking. Tension reinforcement must then be provided in cast-in-place concrete piles with adequate tie-in reinforcement at the connections.

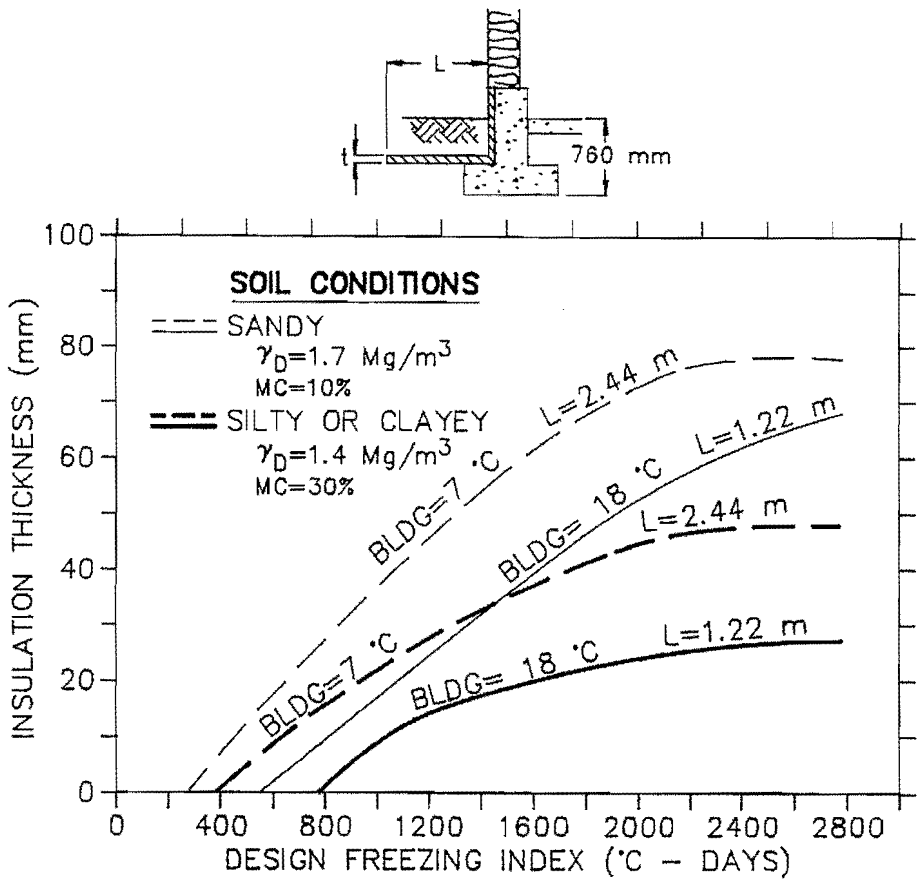


FIGURE 13.10 *Design curves for minimum insulation requirements for heated structures (adapted from Robinsky and Bespflug, 1973)*

13.6 Frost Action during Construction in Winter

Construction in winter is routine in Canada. Special care must be taken to prevent frost action affecting foundations after construction and before heat is applied. Frost heaving and damage frequently occur on construction sites in early winter before temporary heating begins.

13.6.1 Shallow Footings, Pile Caps and Crawl Spaces

Interior footings, which are often placed just below basement floors, are particularly vulnerable to frost action beneath the footings even when straw is used as temporary insulation over the floor surface (Crocker, 1965). Under these circumstances, basement floors may heave causing distortion of partitions or permanent structural damage. Concrete pile caps cast on the ground surface are also vulnerable to frost heave during winter construction. Freezing of supporting soils can lift caps relative to the piles resulting in undesirable deflections during construction as the building load resets the cap. 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 foundation 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 Figure 13.2 was obtained beneath the concrete raft of a seven storey building with crawl space, which was heaved more than 50 mm during construction.

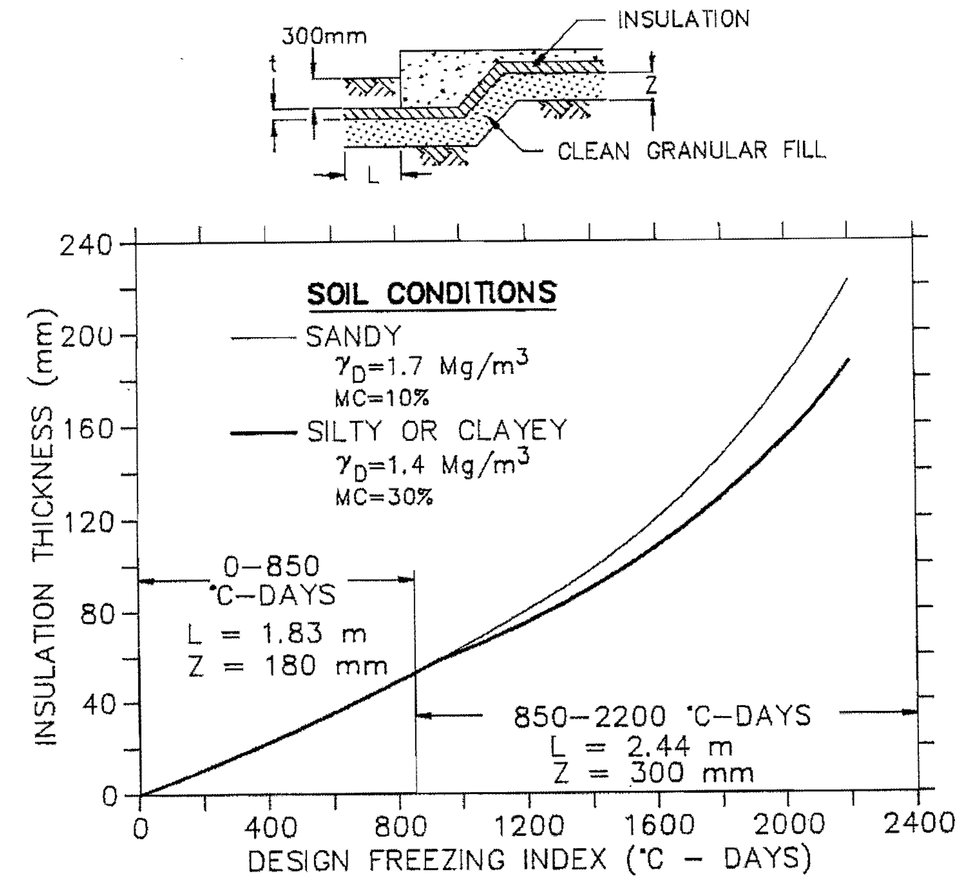


FIGURE 13.11 *Design curves for minimum insulation requirements for unheated structures (adapted from Robinsky and Bespflug, 1973)*

13.6.2 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, being denser than warmer air, flows into below ground openings and accelerates heat extraction from the soil behind the retaining structures.

The direction of heat flow under these conditions is primarily horizontal, producing a preferred ice lens growth direction that is parallel to the walls. This can result in large outward pressures against the wall increasing the loads transferred to the supporting members, which may lead to overstressing (Morgenstern and Sego, 1981). The horizontal components of loads on anchors and rakers may increase considerably. Horizontal struts spanning from wall to wall may be subjected to stress increases with contributions from both walls. Additional loads may develop when struts expand from the heat of the sun.

The development of potentially dangerous conditions must be recognized and mitigative measures taken. Deflection of walls and supporting systems should be monitored for early detection of potential stress increases associated with frost action. This monitoring should be performed 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 25 mm to 50 mm may be tolerable, stresses on the individual tiebacks 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 frozen ground. 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.

14 Machine Foundations

14 Machine Foundations

14.1 Introduction

Geotechnical engineers encounter problems related to machine foundations when designing foundations for machinery and vibrating equipment or designing foundations for vibration-sensitive equipment subjected to vibrations from external sources. In both cases, the foundation design is usually governed by serviceability limit states performance considerations, not strength requirements.

14.2 Design Objectives

The main objective of the foundation design for vibration-sensitive equipment is to limit the response amplitudes to the specified tolerance in all vibration modes. The tolerance is usually set by the machine manufacturer to ensure satisfactory performance of the machine and minimum disturbance for people working in its immediate vicinity. The response of foundations subjected to dynamic loads depends on the type and geometry of the foundation, the flexibility of the supporting ground and the type of dynamic loading.

The dynamic response analysis essentially involves the calculation of the vibration characteristics of the machine-foundation-soil system (i.e. the natural frequencies and the vibration amplitudes due to all sources of vibration). The required complexity of the response analysis depends on the type of the foundation system. For flexible foundation systems (e.g tabletop or mat foundations), dynamic finite element analysis may be necessary. For rigid foundations resting directly on the soil or supported by pile groups, simplified analytical and/or numerical methods are commonly used and are given here.

14.3 Types of Dynamic Loads

14.3.1 Dynamic Loads Due to Machine Operation

A machine causes distinct dynamic forces depending on its manufacturing purpose and the type of motion the machine parts describe, whether it is of a rotating, oscillating or an impacting nature. The machine dynamic forces can be periodic, transient or random.

14.3.1.1 Periodic Loading

Rotating and reciprocating machines produce centrifugal periodic (harmonic) forces due to unbalanced rotors. An unbalanced mass m_e rotating with an eccentricity e and circular velocity ω produces a centrifugal force $P = m_e\, e\, \omega^2$. Examples of machines with predominantly rotating parts are fans, centrifugal separators, vibrators, lathes, centrifugal pumps, electrical motors, turbines and generators. Arya et al. (1979) provide tables for typical values of eccentricity for rotating machines and unbalanced forces and couples for different crank arrangements.

Oscillating parts of machines produce bi-harmonic inertia forces and centrifugal forces associated with the motion of the piston, the flywheel and the crank mechanism. Examples of machines with predominately oscillating parts are piston engines, reciprocating compressors and pumps, presses, crushing and screening machines. The machine manufacturer usually specifies the characteristics of the dynamic force from reciprocating machines.

14.3.1.2 Transient Loading

Impacting parts of machines develop intermittent dynamic forces that are transient in nature. Transient loading is characterized by a non-periodic time history of limited duration. The load time history could be smooth as the one produced by hammer blows or more irregular similar to that generated by crushers and shredders. This type of loading is represented either by an analytical expression or by a set of digital data.

14.3.1.3 Random Loading

Some machines such as mills, pumps and crushers produce fluctuating forces that are random in nature. A random force and its effect is most meaningfully treated in statistical terms and its energy distribution with regard to frequency is described by a power spectral density (power spectrum). Detailed information on dynamic loading is given in Barkan (1962), Richart (1975) and Arya et al. (1979).

14.3.2 Ground Transmitted Loading

In the case of vibration-sensitive equipment, the vibration problems may stem from external sources such as ground-transmitted vibrations from traffic, trains and blasting activities. Vibration criteria supplied by the manufacturer of vibration-sensitive equipment are typically specified in terms of "floor vibrations." Before the facility is built, though, floor vibration cannot be measured directly but, rather, must be predicted by analytical means. Seismic excitation at the site due to ground-transmitted vibration could be, in many cases, an important factor for designing the facility, or even in deciding whether or not it will be built. To assess the level of seismic excitation at the site due to traffic, trains or blasting activities the ground vibration has to be monitored. Ground vibration is usually evaluated in terms of ground acceleration measurements.

14.3.2.1 Vibration Monitoring Equipment

Components of the ground vibration monitoring equipment include sensors, mountings for the sensors, and data acquisition systems. The monitoring system should be designed to provide the required sensitivity, minimize data sampling errors, and achieve the robust performance necessary for the anticipated environmental conditions.

Sensors: ground accelerations can be measured using seismic accelerometers with appropriate sensitivity and suitable operational temperature range. The mounted natural frequency of the sensor should be higher than the maximum excitation frequency of interest to minimize measurement bias in the frequency range of interest.

Mounting arrangement: The sensors are usually mounted on especially designed posts. The posts should be rigid and light. The length of the post should be smaller than the minimum wavelength of soil vibrations for the maximum frequency of interest. It should also enable the simultaneous attachment of accelerometers in three mutually orthogonal directions, with two oriented horizontally and the third vertically. The sensors must be protected from interference from other factors such as wind, rain, snow and electromagnetic fields.

Data acquisition system: The digital data acquisition system should be compatible with the sensors used in measuring the vibration. Proper analog filtering should be used to ensure that no frequency interference occurs. The sampling frequency has to be higher than the highest frequency component of interest.

14.3.2.2 Representation of Ground-Transmitted Excitation

The ground-transmitted excitation can be represented as acceleration time history or in terms of acceleration Fourier transform. The time history will show the maximum acceleration experienced at the location of the foundation, while the Fourier transform will show the frequency content and the distribution of excitation energy with frequency.

14.4 Types of Foundations

Machine foundations are designed as block foundations, wall foundations, mat foundations, or frame foundations. Block foundations are solid blocks of concrete with sizable thickness, wall foundations are block foundations with cavities, and mat foundations are foundations with a limited thickness compared to their surface dimensions. Block foundations, the most common type, and wall foundations behave as rigid bodies. Mat foundations of small depth may behave as elastic slabs. Sometimes the foundation features a joint slab supporting a few rigid blocks for individual machines. The foundations can rest directly on soil (shallow foundations) or on piles (deep foundations). The foundation type results in considerable differences in response.

14.5 Foundation Impedance Functions

The response of soils and foundations to dynamic excitation is frequency dependent and, thus, is a function of the stiffness and damping parameters of the foundation. Therefore, the evaluation of the appropriate stiffness and damping parameters (impedance functions) for the foundation soil or pile/soil system is a key step in the analysis.

The foundation block can be represented in the dynamic analysis as a lumped mass with a spring and dashpot. The block has a mass, m, and is free to move in six directions, i.e., it has six degrees of freedom, three translational and three rotational. These are the displacements along the Cartesian axes x, y and z and rotation about the same axes. The response of the mass depends on the spring and the dashpot that represents the supporting soil medium or pile/ soil system. The spring represents the elasticity of the soil and the dashpot represents damping caused by energy dissipation. This section presents a general introduction to this subject and a summary of approaches and formulae that can be used to evaluate the stiffness and damping of shallow and deep foundations.

14.5.1 Impedance Functions of Shallow Foundations

Shallow foundations are often idealized by a massless circular disc. For circular bases the complex stiffness K_i (also called the dynamic impedance function) associated with direction i is obtained by the determination of the relationship between a harmonic force acting on a massless disc resting on the surface of the halfspace and the resulting displacement of the disc. This complex stiffness can be expressed in terms of the true stiffness constant, k_i, and damping constant, c_i, as

$$K_i = \bar{k}_i [k_i'(a_0) + i a_0 c_i'(a_0)] \tag{14.1}$$

in which is $\bar{k}_i$ static stiffness, $a_0 = \dfrac{\omega R}{V_s}$ = dimensionless frequency, R is the disc radius, $V_s = \sqrt{G/\rho}$ = shear wave velocity of the soil, G and ρ are the soil shear modulus and mass density, respectively, and k'_i and c'_i are stiffness and damping constants normalized as follows: $k_i' = \dfrac{k_i}{\bar{k}_i}$, $c_i' = \dfrac{V_s}{\bar{k}_i R} c_i$. In the case of an isotropic homogeneous halfspace, the approximate static stiffness constants for the vertical translation, v, horizontal translation (sliding), u, rocking, ψ, and torsion, η, are shown in Table 14.1, in which ν is Poisson's ratio and G is the soil shear modulus. The constants k_i' and c_i' are frequency-dependent and may be approximated using the treatment outlined by Wolf (1995):

$$k_i'(a_0) = 1 - \frac{\mu}{\pi} \frac{z_0}{R} \frac{V_s^2}{V} a_0^2 \quad \text{and} \quad c_i'(a_0) = \frac{z_0}{R} \frac{V_s}{V} \tag{14.2}$$

where V is the pertinent wave velocity as given in Table 14.2, in which V_p is the dilational wave velocity $= \sqrt{2 \dfrac{G}{\rho} \dfrac{1-\nu}{1-2\nu}}$. The other parameters are given in Table 14.2.

TABLE 14.1 *Static Stiffnesses of a Disc Resting on the Surface of a Homogeneous Halfspace*

Vertical	Horizontal	Rocking	Torsion
$\bar{k}_v = \dfrac{4GR}{1-\nu}$	$\bar{k}_u = \dfrac{8GR}{2-\nu}$	$\bar{k}_\psi = \dfrac{8GR^3}{3(1-\nu)}$	$\bar{k}_\eta = \dfrac{16GR^3}{3}$

TABLE 14.2 *Parameters of Approximate Solution for Footings Resting on Surface of Soil Halfspace*

	Vertical	Horizontal	Rocking	Torsion
R	$\sqrt{\dfrac{A_0}{\pi}}$	$\sqrt{\dfrac{A_0}{\pi}}$	$\sqrt[4]{\dfrac{4I_0}{\pi}}$	$\sqrt[4]{\dfrac{2J_0}{\pi}}$
V	V_P $\nu \le 1/3$ $2V_s$ $\nu \ge 1/3$	V_s	V_P $\nu \le 1/3$ $2V_s$ $\nu \ge 1/3$	V_s
Z_0/R	$\dfrac{\pi}{4}(1-\nu)\left(\dfrac{V}{V_s}\right)^2$	$\dfrac{\pi}{8}(2-\nu)$	$\dfrac{9\pi}{32}(1-\nu)\left(\dfrac{V}{V_s}\right)^2$	$\dfrac{9\pi}{32}$
μ	0 $\nu \le 1/3$ $2.4\left(\nu-\dfrac{1}{3}\right)\rho A_0 R$ $\nu \ge 1/3$	0	0 $\nu \le 1/3$ $1.2\left(\nu-\dfrac{1}{3}\right)\rho I_0 R$ $\nu \ge 1/3$	0

To account for the material damping, the stiffness and damping constants including the soil hysteretic damping, β, are given by

$$k'_{i\beta}(a_0) = 1 - \frac{\mu}{\pi}\frac{z_0}{R}\frac{V_s^2}{V}a_0^2 - \beta\frac{z_0}{R}\frac{V_s}{V}a_0 \quad \text{and} \quad c'_{i\beta}(a_0) = \frac{z_0}{R}\frac{V_s}{V} + \frac{2\beta}{a_0}k'_i \tag{14.3}$$

For shapes which differ from circular, the real noncircular base is replaced by an equivalent circular base with a suitable radius. The radius of the equivalent circular foundation is usually determined by equating the areas of the actual base (A_0) and equivalent base for vertical and horizontal translations, the moments of inertia (I_0) for rotation in the vertical plane (rocking) and the polar moments of inertia (J_0) for torsion about the vertical axis. For rectangular bases having dimensions a and b, the equivalent radii are given in Table 14.3.

TABLE 14.3 *Equivalent Radii for a Rectangular Footing having Dimensions a and b*

Vertical	Horizontal	Rocking	Torsion
$R = \sqrt{\dfrac{ab}{\pi}}$	$R = \sqrt{\dfrac{ab}{\pi}}$	$R = \sqrt[4]{\dfrac{a^3 b}{3\pi}}$	$R = \sqrt[4]{\dfrac{ab(a^2+b^2)}{6\pi}}$

14.5.2 Embedment Effects

Embedment is known to increase both stiffness and damping but the increase in damping is more significant. The response of embedded footings can be approximated by assuming that soil reactions acting on the base are equal to those of a surface footing and the reactions acting on the footing sides are equal to those of an independent layer overlying the halfspace (Figure 14.1). Novak and Beredugo (1972) and Beredugo and Novak (1972) used plane strain solutions for side reactions and a halfspace solution for base reactions, and the notations in Figure14.1 to derive the stiffness and damping constants given in Table 14.4. The parameters C defining the base stiffness and damping and S defining the side stiffness and damping in Table 14.4 are frequency dependent. However, it is often sufficient to select suitable constant values to represent the parameters over a limited frequency range.

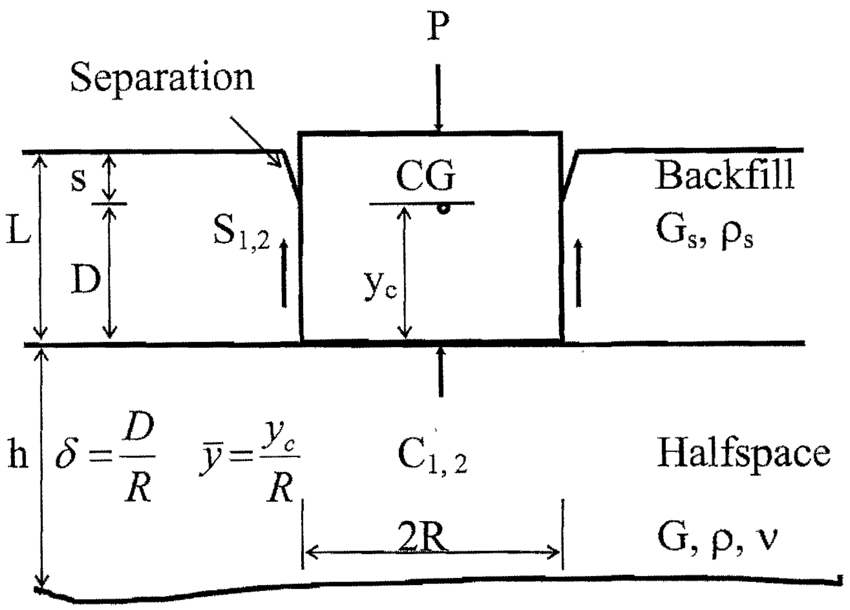


FIGURE 14.1 *Notations for embedded foundation*

TABLE 14.4 *Stiffness and Damping Constants for Embedded Footings*

Motion	Stiffness Constant k	Damping constant c
Vertical	$GR(C_{v1} + \frac{G_s}{G}\delta S_{v1})$	$R^2\sqrt{\rho G}(C_{v2} + S_{v2}\delta\sqrt{\frac{\rho_s}{\rho}\frac{G_s}{G}})$
Torsional	$GR^3(C_{\eta 1} + \frac{G_s}{G}\delta S_{\eta 1})$	$R^4\sqrt{\rho G}(C_{\eta 2} + S_{\eta 2}\delta\sqrt{\frac{\rho_s}{\rho}\frac{G_s}{G}})$
Horizontal	$GR(C_{u1} + \frac{G_s}{G}\delta S_{u1})$	$R^2\sqrt{\rho G}(C_{u2} + \delta\sqrt{\frac{\rho_s}{\rho}\frac{G_s}{G}} S_{u2})$
Rocking	$GR^3[\bar{y}^2 C_{u1} +$ $\frac{G_s}{G}\delta\left(\frac{\delta^2}{3} + \bar{y}^2 - \delta\bar{y}\right)S_{u1}]$ $+ GR_\varphi^3[C_{\varphi 1} + \frac{G_s}{G}\delta_\varphi S_{\varphi 1}]$	$R^4\sqrt{\rho G}[\bar{y}^2 C_{u2} +$ $\delta\sqrt{\frac{\rho_s}{\rho}\frac{G_s}{G}}\left(\frac{\delta^2}{3} + \bar{y}^2 - \delta\bar{y}\right)S_{u2}]$ $+ R_\varphi^4\sqrt{\rho G}[C_{\varphi 2} + \delta_\varphi\sqrt{\frac{\rho_s}{\rho}\frac{G_s}{G}} S_{\varphi 2}]$
Coupling	$-GR[y_C C_{u1} + \frac{G_s}{G}\delta\,(y_C - \frac{D}{2})S_{u1}]$	$-R^2\sqrt{\rho G}[y_C C_{u2} +$ $\delta\sqrt{\frac{\rho_s}{\rho}\frac{G_s}{G}}\left(y_C - \frac{D}{2}\right)S_{u2}]$

Such constant values are suggested in Table 14.5. The lack of confining pressure at the surface often leads to separation of the soil from the foundation, which reduces the effectiveness of embedment. To account for the lack of confining pressure at the surface which leads to separation of the soil from the foundation, an effective embedment depth, D, smaller than the true one, may be used. An extensive set of tables and charts for stiffness and damping constants of embedded footings of arbitrary shapes is given by Gazetas (1991).

TABLE 14.5 *Stiffness and Damping Parameters (β = 0)*

Motion	Soil	Side Layer		Halfspace	
Vertical	Cohesive	S_{v1} = 2.7	S_{v2} = 6.7	C_{v1} = 7.5	C_{v2} = 6.8
	Granular	S_{v1} = 2.7	S_{v2} = 6.7	C_{v1} = 5.2	C_{v2} = 5.0
Horizontal	Cohesive	S_{u1} = 4.1	S_{u2} = 10.6	C_{u1} = 5.1	C_{u2} = 3.2
	Granular	S_{u1} = 4.0	S_{u2} = 9.1	C_{u1} = 4.7	C_{u2} = 2.8
Rocking	Cohesive	$S_{\psi 1}$ = 2.5	$S_{\psi 2}$ = 1.8	$C_{\psi 1}$ = 4.3	$C_{\psi 2}$ = 0.7
	Granular	$S_{\psi 1}$ = 2.5	$S_{\psi 2}$ = 1.8	$C_{\psi 1}$ = 3.3	$C_{\psi 2}$ = 0.5
Torsion	Coh.&Gran.	$S_{\eta 1}$ = 10.2	$S_{\eta 2}$ = 5.4	$C_{\eta 1}$ = 4.3	$C_{\eta 2}$ = 0.7

14.5.3 Impedance Functions of a Layer of Limited Thickness

The stiffness of a layer of limited thickness is higher than that of a halfspace but its geometric damping decreases or even vanishes if the excitation frequency is lower than the first natural frequency of the soil layer. For a homogeneous soil layer, the first vertical and horizontal natural frequencies, ω_v and ω_u, respectively, are:

$$\omega_v = \frac{\pi V_s}{2H}\sqrt{\frac{2(2-\nu)}{1-2\nu}} \quad \text{and} \quad \omega_u = \frac{\pi V_s}{2H} \tag{14.4}$$

The damping parameters at frequencies lower than ω_v and ω_u may be calculated by:

$$S_{u2} = 2\beta\frac{S_{u1}}{a_0} \quad \text{and} \quad S_{v2} = 2\beta\frac{S_{v1}}{a_0} \tag{14.5}$$

The stiffness and damping of a footing embedded in a layer of limited thickness, H, can be defined in a manner similar to Eq. 14.1. However, the static stiffnesses, $\bar{k}_v$, of circular foundations may be given by (Elsabee and Morray, 1977; Kausel and Ushijima, 1979):

$$\bar{k}_v = \frac{4GR}{1-\nu}(1+1.28\frac{R}{H})(1+0.47\delta)[1+(0.85-0.28\delta)\frac{D/H}{1-D/H}] \tag{14.6a}$$

$$\bar{k}_u = \frac{8GR}{2-\nu}(1+\frac{1}{2}\frac{R}{H})(1+\frac{2}{3}\delta)(1+\frac{5}{4}\frac{D}{H}) \tag{14.6b}$$

$$\bar{k}_\varphi = \frac{8GR^3}{3(1-\nu)}(1+\frac{1}{6}\frac{R}{H})(1+2\delta)(1+0.7\frac{D}{H}) \tag{14.6c}$$

$$\bar{k}_{u\varphi} = (0.4\delta - 0.03) R \bar{k}_u \tag{14.6d}$$

$$\bar{k}_{\eta} = \frac{16GR^3}{3}(1 + 2.67\delta) \tag{14.6e}$$

These empirical expressions for the stiffness are referred to the centre of the base and are valid for δ = D/R $\leq$ 1.5, D/H $\leq$ 0.75 and R/H $\leq$ 0.5. The dynamic stiffness and damping can be calculated taking k' and c' equal to the halfspace functions (Equations 14.1-14.3). For frequencies below the first layer natural frequencies, it would be safe to ignore geometric damping completely (first term and damping formula in Equation 14.3). Similar formulae for foundations on shallow layer can be found in Gazetas (1991).

14.5.4 Trial Sizing of Shallow Foundations

The design of a shallow foundation for a centrifugal or reciprocating machine starts with trial dimensions of the foundation block. The trial sizing is based on guidelines derived from past experience. The following guidelines may be used for the trial sizing of the foundation block:

1. Generally, the base of the foundation should be above the groundwater table. It should be resting on competent native soil (no backfill or vibration-sensitive soil).
2. The mass of the block should be 2 to 3 times the mass of the supported centrifugal machine, and 3 to 5 times the supported reciprocating machine.
3. The top of the block should be 0.3 m above the elevation of the finished floor.
4. The thickness of the block should be the greatest of 0.6 m, the anchorage length of the anchor bolts and 1/5 the least dimension of the footing.
5. The width should be 1 to 1.5 times the vertical distance from the base to the machine centerline to increase damping in rocking mode.
6. The length is estimated from the mass requirement and estimated thickness and width of the foundation. The length should then be increased by 0.3 m for maintenance purposes.
7. The length and width of the foundation are adjusted so that the centre of gravity of the machine plus equipment lies within 5 % of the foundation dimension in each direction, from the foundation centre of gravity.
8. It is desirable to increase the embedded depth of the foundation to increase the damping and provide lateral restraint as well.
9. If resonance is predicted from the dynamic analysis, increase or decrease the mass of the foundation to change its natural frequency (try to undertune for rotating machines and overtune for reciprocating machines).

14.6 Deep Foundations

The dynamic stiffness and damping of a pile group are affected by both the interaction between the piles and surrounding soil, and the interaction between individual piles. Therefore, the calculation of the stiffness for a group of identical piles may be performed in two steps. First, the stiffness of the single pile is calculated. Second, the group effect is accounted for using "interaction factors."

14.6.1 Impedance Functions of Piles

The pile length, bending and axial stiffness, tip and head conditions, mass, batter and the surrounding soil properties and their variation with depth and layering, affect the dynamic stiffness of a pile. The impedance functions of piles can be described as

$$K_i = k_i(a_0) + i\omega c_i(a_0) \tag{14.7}$$

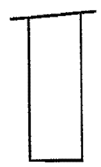
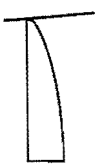
The stiffness constants, k_i, and the constants of equivalent viscous damping, c_i, for individual motions of the pile head suggested by Novak (1974) are shown in Table 14.6.

TABLE 14.6 *Stiffness and Damping Constants for Single Piles*

Vertical	Horizontal	Rocking	Coupling	Torsion
$k_v = \dfrac{E_p A}{R} f_{v1}$	$k_u = \dfrac{E_p I}{R^3} f_{u1}$	$k_\varphi = \dfrac{E_p I}{R} f_{\varphi 1}$	$k_c = \dfrac{E_p I}{R^2} f_{c1}$	$c_\eta = \dfrac{G_p J}{V_s} f_{\eta 2}$
$c_v = \dfrac{E_p A}{V_s} f_{v2}$	$c_u = \dfrac{E_p I}{R^2 V_s} f_{u1}$	$c_\varphi = \dfrac{E_p I}{V_s} f_{\varphi 2}$	$c_c = \dfrac{E_p I}{R V_s} f_{c2}$	$c_\eta = \dfrac{G_p J}{V_s} f_{\eta 2}$

These constants are a function of the pile's elastic modulus, E_p cross-sectional area, A, and its moment of inertia and torsional stiffness I and $G_p J$, respectively. R is the radius of circular piles and equivalent radius for non-circular piles. The symbol $f_{1,2}$ in Table 14.6 represents dimensionless stiffness and damping functions whose subscript 1 indicates stiffness and 2 indicates damping. These functions depend on the following parameters: the relative stiffness of the pile and soil, E_p/G; dimensionless frequency, a_0; the slenderness ratio, L/R, in which L = pile length; material damping of both the soil and pile; the variation of soil and pile properties with depth; and the tip and head conditions. However, E_p/G, the soil profile and, for the vertical direction, the tip condition have the strongest effect on the stiffness. The stiffness and damping parameters, $f_{1,2}$, are given for a few basic cases in Table 14.7, for horizontal response, for a dimensionless frequency, $a_0 = 0.3$. For other cases, see Novak and El Sharnouby (1983).

TABLE 14.7 *Stiffness and Damping Parameters of Horizontal Response (L/R > 25 for homogeneous soil and L/R > 30 for parabolic soil profile) (Reproduced from Novak and El Sharnouby © 1983 with permission of ASCE)*

Soil Profile	ν	E_p/G	$f_{\psi 1}$	f_{c1}	f_{u1}	f_{u1}^P	$f_{\psi 2}$	F_{c2}	f_{u2}	f_{u2}^P
	0.25	10000	0.2135	-0.0217	0.0042	0.0021	0.1577	-0.0333	0.0107	0.0054
		2500	0.2998	-0.0429	0.0119	0.0061	0.2152	-0.0646	0.0297	0.0154
		1000	0.3741	-0.0668	0.0236	0.0123	0.2598	-0.0985	0.0579	0.0306
		500	0.4411	-0.0929	0.0395	0.0210	0.2953	-0.1337	0.0953	0.0514
		250	0.5186	-0.1281	0.0659	0.0358	0.3299	-0.1786	0.1556	0.0864
G_{soil} Constant with Depth	0.4	10000	0.2207	-0.0232	0.0047	0.0024	0.1634	-0.0358	0.0119	0.0060
		2500	0.3097	-0.0459	0.0132	0.0068	0.2224	-0.0692	0.0329	0.0171
		1000	0.3860	-0.0714	0.0261	0.0136	0.2677	-0.1052	0.0641	0.0339
		500	0.4547	-0.0991	0.0436	0.0231	0.3034	-0.1425	0.1054	0.0570
		250	0.5336	-0.1365	0.0726	0.0394	0.3377	-0.1896	0.1717	0.0957
	0.25	10000	0.1800	-0.0144	0.0019	0.0008	0.1450	-0.0252	0.0060	0.0028
		2500	0.2452	-0.0267	0.0047	0.0020	0.2025	-0.0484	0.0159	0.0076
		1000	0.3000	-0.0400	0.0086	0.0037	0.2499	-0.0737	0.0303	0.0147
		500	0.3489	-0.0543	0.0136	0.0059	0.2910	-0.1008	0.0491	0.0241
		250	0.4049	-0.0734	0.0215	0.0094	0.3361	-0.1370	0.0793	0.0398
G_{soil} Varying with Depth (parabola) to G_{soil}	0.4	10000	0.1857	-0.0153	0.0020	0.0009	0.1508	-0.0271	0.0067	0.0031
		2500	0.2529	-0.0284	0.0051	0.0022	0.2101	-0.0519	0.0177	0.0084
		1000	0.3094	-0.0426	0.0094	0.0041	0.2589	-0.0790	0.0336	0.0163
		500	0.3596	-0.0577	0.0149	0.0065	0.3009	-0.1079	0.0544	0.0269
		250	0.4170	-0.0780	0.0236	0.0103	0.3468	-0.1461	0.0880	0.0443

14.6.2 Pile-Soil-Pile Interaction

When piles in a group are closely spaced, they interact with each other because the displacement of one pile to contribute to the displacement of others. To obtain an accurate analysis of dynamic behaviour of pile groups it is necessary to use a suitable computer program. However, a simplified approximate analysis, can be formulated on the basis of interaction factors, α, introduced by Poulos (1971) for static analysis and extended to the dynamic case by Kaynia and Kausel (1982) who presented charts for dynamic interaction. For a homogeneous halfspace, the interaction factors between two piles may be given by (Dobry and Gazetas, 1988 and Gazetas and Makris, 1991)

$$\alpha_v \approx \frac{1}{\sqrt{2}}\left(\frac{S}{d}\right)^{-0.5} e^{-\beta\omega\frac{S}{V_s}} e^{-i\omega\frac{S}{V_s}} \quad \text{and} \quad \alpha_u(\theta^0) \approx \alpha_u(0^0)\cos^2\theta + \alpha_u(90^0)\sin^2\theta \qquad (14.8a)$$

where

$$\alpha_u(0^0) \approx \frac{1}{2}\left(\frac{S}{d}\right)^{-0.5} e^{-\beta\omega\frac{S}{V_{La}}} e^{-i\omega\frac{S}{V_{La}}} \quad \text{and} \quad \alpha_u(90^0) \approx 0.75\alpha_v \qquad (14.8b)$$

where α_v and α_u are vertical and horizontal interaction factors, respectively, S/d = pile spacing to diameter ratio, θ is the angle between the direction of load action and the plane in which piles lie, and V_{La} = the so-called Lysmer's analog velocity = $\frac{3.4V_s}{\pi(1-\nu)}$.

To calculate the dynamic stiffness of a pile group using the interaction factors approach, the impedance functions of single piles and the interaction factors are calculated first, then the group impedance functions are computed. The stiffness and damping constants of individual piles are calculated using expressions given in Table 14.6 or formulae due to Gazetas (1991). The interaction factors are calculated using Equation 14.8 or charts due to Kaynia and Kausel (1982). The impedance functions of a pile group of n piles are then given by

$$K_v^G = \bar{k}_v \sum_{i=1}^{n} \sum_{j=1}^{n} \varepsilon_{ij}^{v} \qquad (14.9a)$$

$$K_h^G = \bar{k}_h \sum_{i=1}^{n} \sum_{j=1}^{n} \varepsilon_{2i-1,2j-1}^{h} \qquad (14.9b)$$

$$K_r^G = \bar{k}_h \sum_{i=1}^{n} \sum_{j=1}^{n} \varepsilon_{2i,2j}^{h} \qquad (14.9c)$$

$$K_c^G = \bar{k}_h \sum_{i=1}^{n} \sum_{j=1}^{n} \varepsilon_{2i-1,2j}^{h} \qquad (14.9d)$$

where K_v^G, K_h^G, K_r^G and K_c^G are the vertical, horizontal, rocking and coupling group stiffness, respectively. In Eq. 14.9 $\bar{k}_v$ is the static vertical stiffness of the single pile, $[\varepsilon^v] = [\alpha]_v^{-1}$ where α_{ij}^v = complex interaction factors between piles i and j, $\alpha_{ii}^v = \bar{k}_v / K_v$, and K_v is the complex vertical impedance function of the single pile. Similarly, $\bar{k}_h$ is the static horizontal stiffness of the pile $[\varepsilon^h] = [\alpha]_h^{-1}$ where α_{ij}^h = complex interaction coefficients for the horizontal translations and rotations. The formulation of the $[\alpha]_h$ can be found in El Naggar and Novak (1995).

14.6.3 Trial Sizing of Piled Foundations

The design of a deep foundation for a centrifugal or reciprocating machine starts with trial dimensions of the pile cap, and size and configuration of the pile group (Step No. 3 in the design procedure). The trial sizing is based on guidelines derived from past experience. The following guidelines may be used for trial sizing the pile cap:

1. The pile cap (block) mass should be 1.5 to 2.5 times the mass of the centrifugal machine and 2.5 to 4 times the mass of the reciprocating machine.

2. The top of the cap should be 0.3 m above the elevation of the finished floor.
3. The thickness of the block should be the greatest of 0.6 m, the anchorage length of the anchor bolts and 1/5 the least dimension of the block.
4. The width should be 1 to 1.5 times the vertical distance from the base to the machine centerline to increase damping in rocking mode.
5. The length is estimated from the mass requirement and estimated thickness and width of the block. The length should then be increased by 0.3 m for maintenance purposes.
6. The length and width of the block are adjusted so that the centre of gravity of the machine plus equipment lies within 5 % of the block dimension in each direction, from the block centre of gravity.
7. It is desirable to increase the embedded depth of the foundation to increase the damping and provide lateral restraint as well.

The following guidelines may be used for the trial configuration of the pile group:

1. The number and size of piles are selected such that the average static load per pile ≤ ½ the pile design load.
2. The piles are arranged so that the centroid of the pile group coincides with the centre of gravity of the combined structure and machine.
3. If battered piles are used to provide lateral resistance (they are better than vertical piles in this aspect), the batter should be away from the pile cap and should be symmetrical.
4. If piers are used, enlarged bases are recommended.
5. Piles and piers must be properly anchored to the pile cap for adequate rigidity (as commonly assumed in the analysis).

14.7 Evaluation of Soil Parameters

The soil parameters required for the dynamic analysis include the shear modulus, G, the material damping ratio, D, Poisson's ratio, ν, and mass density, ρ. Some of the procedures that can be used to evaluate these parameters are given here.

14.7.1 Shear Modulus

The shear strains developed in the supporting soil medium due to the dynamic loading from machine foundations are usually of a much smaller magnitude than the strains produces by static loading. The value of the soil shear modulus at smaller strains is much higher than its value at larger strains. Therefore, the soil shear modulus used for the computation of the foundation impedance functions should be evaluated for smaller strain laboratory field tests (see Richart et al. 1975 for details on experimental procedures). In the absence of measured values, the correlations in Table 14.8 can be used to evaluate the shear modulus.

14.7.2 Material Damping Ratio

Soil material damping is a measure of energy lost due to friction between soil particles during the dynamic loading. Material damping ratio can be obtained from resonant column testing and the Spectral Analysis of Seismic Wave procedure (SASW). The material damping is typically 0.03 to 0.05 for sand and saturated clay.

14.7.3 Poisson's Ratio and Soil Density

The dynamic behaviour of foundations is less sensitive to the values of ν and ρ. Typical values for ν are given in Table 14.9. The soil mass density values should always be calculated from the total unit weight rather than the buoyant unit weight. Total weights are used in dynamic problems because both the solid and liquid phases vibrate.

TABLE 14.8 *Some Correlations for Soil Shear Modulus*

Soil Type	Correlation	Reference
Sand (round-grained)	$G_{max} = \frac{6908(2.17-e)^2}{1+e}\bar{\sigma}_0^{1/2}$ * (kPa)	Hardin and Black (1968)
Sand (angular-grained)	$G_{max} = \frac{3230(2.97-e)^2}{1+e}\bar{\sigma}_0^{1/2}$ * (kPa)	Hardin and Black (1968)
Sand	$G_{max} \approx 35000 N_{60}^{0.34}(\bar{\sigma}_0)^{0.4}$ * (psf)	Seed et al. (1986)
Clay (moderate sensitivity)	$G_{max} = \frac{3230(2.97-e)^2}{1+e}\bar{\sigma}_0^{1/2}(OCR)^K$ ** (kPa)	Hardin and Drnevich (1972)

* $\bar{\sigma}_0 = \frac{1}{3}(\bar{\sigma}_1 + \bar{\sigma}_2 + \bar{\sigma}_3)$ = effective octahedral stress

** OCR is over consolidation ratio and K = function of the soil plasticity index, PI, and is given by

PI (%)	0	20	40	60	80	≥ 100
K	0	0.18	0.3	0.41	0.48	0.5

TABLE 14.9 *Typical Values of Soil Poisson's Ratio*

Soil Type	v
Saturated clay	0.45-0.50
Unsaturated clay	0.35-0.45
Silt, Medium dense sand - Gravel	0.30-0.40
Dense sand - Gravel	0.4-0.5

14.8 Response to Harmonic Loading

The machine foundation can vibrate in any or all six possible modes due to the excitation loading from the vibrating machine it supports. For ease of analysis, some of these modes can be considered separately (e.g. vertical or torsional) and design is carried out by considering the displacement due to these modes separately.

14.8.1 Response of Rigid Foundations in One Degree of Freedom

The response of the foundation in one degree of freedom (1 DOF) to a harmonic load with an amplitude, P, and frequency ω, can be given by

$$v(t) = \frac{P}{\sqrt{(k - m\omega^2)^2 + \omega^2 c^2}} \cos(\omega t + \phi) \tag{14.10}$$

where m, k and c are the mass, stiffness and damping of the foundation, and $\phi = \tan^{-1}\frac{-\omega c}{k - m\omega^2}$ The stiffness and damping constants k and c are established as described in Sections 14.5 and 14.6 and are frequency dependent hence the response has to be calculated using Equation 14.10. However, if they can be considered frequency independent in the frequency range of interest, Equation 14.10 can be rearranged and the real amplitude can be written as

$$v = \frac{P}{k_v} \frac{1}{\sqrt{\left[1-(\frac{\omega}{\omega_0})^2\right]^2 + 4D^2\left(\frac{\omega}{\omega_0}\right)^2}} = v_{st}\varepsilon \tag{14.11}$$

in which the natural frequency, $\omega_0 = \frac{k_v}{m}$, the damping ratio, $D = \frac{c_v}{2\sqrt{k_v m}}$, $v_{st} = \frac{P}{k_v}$, = the static displacement and ε = dynamic amplification factor given by

$$\varepsilon = \frac{1}{\sqrt{\left[1-(\frac{\omega}{\omega_0})^2\right]^2 + 4D^2\left(\frac{\omega}{\omega_0}\right)^2}} \tag{14.12}$$

For a harmonic excitation, the maximum displacement is given by

$$v_{\max} = \frac{P}{k} \frac{1}{2D\sqrt{1-D^2}} \tag{14.13}$$

14.8.2 Coupled Response of Rigid Foundations

The coupled motion in the vertical plane represents an important case because it results from excitation by moments and horizontal forces acting in the vertical plane. The horizontal sliding, $u(t)$, and rocking, $\psi(t)$, describe the coupled motion. For a simple rectangular footing with dimensions a and b, the mass moment of inertia of the system is

$$I = \frac{m_1}{12}(a^2 + b^2) + m_1(y_c - \frac{b}{2})^2 + m_2 y_m^2 \tag{14.14}$$

where m_l is the mass of the footing, m_2 is the mass of the machine, y_c and y_m are distance from C.G. to foundation base and machine centre of gravity, respectively. The stiffness constants, k_{uu}, $k_{u\psi}$, and $k_{\psi\psi}$ are described as the stiffness constants for translation and rotation at the base of the footing, transformed to the Centre of Gravity of the system, CG. If the stiffness constants referred to the centre of the base are k_u and k_ψ (calculated as described in sections 14.5 or 14.6), the stiffness constants referred to CG are

$$k_{uu} = k_u\,,\ k_{\psi\psi} = k_\psi + k_u y_c^2 \text{ and } k_{u\psi} = k_{\psi u} = -k_u y_c \tag{14.15}$$

The response of the foundation system in the coupled motion to an excitation loading given by a horizontal force, P(t), and a moment, M(t) can be evaluated using the modal analysis. First, the natural frequencies and modes of free vibration are calculated, i.e.

$$\omega_{1,2}^2 = \frac{1}{2}(\frac{k_{uu}}{m} + \frac{k_{\psi\psi}}{I}) \mp \sqrt{\frac{1}{4}(\frac{k_{uu}}{m} - \frac{k_{\psi\psi}}{I})^2 + \frac{k_{u\psi}^2}{mI}} \tag{14.16}$$

With these two natural frequencies, ω_j ($j = 1,2$), the two vibration modes are

$$a_j = \frac{u_j}{\psi_j} = \frac{-k_{u\psi}}{k_{uu} - m\omega_j^2} = \frac{k_{\psi\psi} - I\omega_j^2}{-k_{u\psi}} \tag{14.17}$$

Then, the footing translation and rocking are

$$u(t) = \sum_{j=1}^{2} q_j u_j \sin(\omega t + \phi_j) \text{ and } \psi(t) = \sum_{j=1}^{2} q_j \psi_j \sin(\omega t + \phi_j) \tag{14.18}$$

in which q_j and ϕ_j are

$$q_j = \frac{P_j}{M_j\sqrt{(\omega_j^2-\omega^2)^2+4(D_j\omega_j\omega)^2}} \quad \text{and} \quad \phi_j = -\tan^{-1}\left(\frac{2D_j\omega_j\omega}{\omega_j^2-\omega^2}\right) \tag{14.19}$$

where, $P_j = Pu_j + M\psi_j$, $M_j = mu_j^2 + I\psi_j^2$ and $D_j = \frac{1}{2\omega_j M_j}(c_{uu}u_j^2 + c_{\psi\psi}\psi_j^2 + 2c_{u\psi}u_j\psi_j)$

If the damping in the system is small, the results from modal analysis are very close to the results obtained from the direct approach.

14.8.3 Response of Rigid Foundations in Six Degrees of Freedom

When the rigid foundation is of general shape, the response is in six degrees-of-freedom, all of them, possibly, coupled. The stiffness constants are described at the base of the footing, then transformed to the reference point, Centre of Gravity CG. The stiffness and damping are described in terms of impedance functions K_{ij}. Considering the dynamic equilibrium of forces and moments for the system will result in six linear algebraic equations that can be solved for the vibration amplitudes.

14.9 Response to Impact Loading

Shock producing machines generate dynamic effects that differ from those of rotating machines and the design of their foundations, therefore, requires special consideration. Different foundation arrangements are used to support impact-producing machines. The foundation block is most often cast directly on soil. When the transmission of vibration in the vicinity and adjoining facilities is of concern, the block may be supported on vibration isolating elements.

14.9.1 Design Criteria

The design of a hammer foundation must ensure satisfactory performance of the hammer and minimum disturbance to the environment. These objectives are met by limiting the vibration amplitudes, settlement, physiological effects and stresses to the given tolerances.

14.9.1.1 Performance Criteria

The manufacturer should specify the limits on the vibration amplitudes. The physiological effects are related to vibration velocity and acceleration rather than displacement. The vibration velocity can be calculated approximately as $\dot{v}_m = v_m\omega_0$ where v_m = the maximum displacement and ω_0 = the natural frequency of the foundation. For data on human perceptibility collected see Richart et al. (1970). Stresses in all parts of the foundation have to remain within allowable limits. Dynamic stress is repetitive and fatigue effects have to be accounted for by using a factor of safety greater than 3 in the design.

The adequacy of the mass for a hammer foundation is best proven by detailed analysis of stresses and amplitudes. Some guidelines have been suggested for the preliminary choice of the weight of the foundation block. Assuming the anvil weight = 20 G_0, where G_0 is the weight of the head, the weight of the block, G_b, can be estimated by

$$G_b = 75G_0\left(\frac{C_0}{C_r}\right)^2 \tag{14.20}$$

where C_0 = the maximum velocity of the head and C_r = 5.6 m/s, (Rausch, 1950).

14.9.1.2 Vibration Effects on the Environment

Vibration propagates from the footing into the surroundings in the form of ground motion. The vertical amplitude of the ground motion, v_r, at a distance r from the foundation vertical axis can be evaluated approximately as

$$v_r = v_0\sqrt{\frac{r_0}{r}}e^{-\alpha(r-r_0)} \tag{14.21}$$

where v_0 = footing amplitude, r_0 = the distance of the footing edge from its vertical axis and α = empirical coefficient ranges from 0 to 0.05 m^{-1}. The horizontal amplitude may be considered equal to the vertical one. The response of a structure located near the hammer foundation can be predicted using the methods of structural dynamics.

14.9.2 Response of One Mass Foundation

When the anvil is rigidly mounted on the foundation block and the hammer blow does not act eccentrically, the foundation response can be analysed using a one degree of freedom model. The response corresponding to initial velocity of the system, , can be written as

$$v(t) = \frac{\hat{c}}{\omega_0'}e^{-D\omega_0 t}\sin\omega_0' t \tag{14.22}$$

where $\omega'_0 = \omega_0\sqrt{1-D^2}$, $D = \frac{c}{2\sqrt{km}}$. The initial velocity of the system, $\hat{c}$, can be obtained from the consideration of the collision between the head and the foundation. The peak force transmitted into the ground is $F = \hat{v}\sqrt{k^2 + (c\omega_0)^2}$ and the peak stress is $\sigma = F/A_b$, where $\hat{v}$ = peak displacement and A_b = the base area.

14.9.3 Response of Two Mass Foundation

When the anvil rests on an elastic pad, a hammer foundation should be considered as a two mass system. In this model, m_1 is the mass of the anvil and m_2 is the mass of the footing; k_1 and c_1 are the stiffness and damping constants of the pad and k_2 and c_2 are the stiffness and damping of the soil or piles supporting the footing. Stiffness and damping of foundations can be evaluated using the approaches described in sections 14.5 and 14.6. The stiffness and damping constants of a pad can be given by

$$k_p = \frac{E_p A_p}{d} \text{ and } c_p = 2\beta_p \frac{k_p}{\omega_0} \tag{14.23}$$

where E_p, A_p, d, and β_p are Young's modulus, area, thickness and material damping of the pad, respectively, and ω_0 = natural frequency of the block calculated with k_p. With these values, the natural frequencies can be calculated as

$$\omega_{1,2}^2 = \frac{1}{2}\left(\frac{k_1}{m_1} + \frac{k_1+k_2}{m_2}\right) \mp \sqrt{\frac{1}{4}\left(\frac{k_1}{m_1} - \frac{k_1+k_2}{m_2}\right)^2 + \frac{k_1^2}{m_1 m_2}} \tag{14.24}$$

The damped response can be evaluated using the approach developed by Novak and El-Hifnawy (1983).

14.10 Response to Ground-Transmitted Excitation

The basic response to harmonic loading in 1 DOF is given by Eq. 14.10. For ground-transmitted excitation, the forcing function, P(t), is given by {- $m\ddot{u}(t)$ }where $\ddot{u}(t)$ is the absolute ground acceleration time history measured at the location of the future foundation. In this case, there are two approaches to solve for the response of the foundation. In the first approach, the Duhamel integral of $\ddot{u}(t)$ is used to calculate the relative displacement of the foundation, i.e.

$$\delta(t) = \frac{-1}{\omega_0\sqrt{1-D^2}}\int_0^t \ddot{u}(\tau)e^{-\omega D(t-\tau)}\sin[\omega_d(t-\tau)d\tau \tag{14.25}$$

where $\omega_0 = \sqrt{k/m}$, $D = c/2\sqrt{km}$ and $\omega_d = \sqrt{1-D^2}$

The response of the machine-foundation system is influenced by both its natural frequency and the frequency content of loading. The traffic loading is transmitted to the foundation as a combination of seismic waves propagating in the ground at different frequencies. Therefore, alternatively, a Fourier analysis can be used to calculate the response of the foundation to the transient load in the frequency domain. In this type of analysis, the load is represented by the sum of a series of harmonic components obtained by subjecting the load time history to a Fast Fourier Transform (FFT). In the FFT, the forcing function is given at an even number, N, of equidistant points in the time domain, and N/2 frequency components are obtained. Thus, increased accuracy can only be obtained by increasing the number of data points.

The response of the system can be related to the loading by

$$\delta_n(t) = \frac{x_k}{k} \left| H(\omega_n) \right| \cos(\omega_n t + \phi) \tag{14.26}$$

where x_k and ω_n are the amplitude and frequency of that harmonic component and $H(\omega_n)$ is the modulus of the complex transfer function, $H(\omega_n)$ given by

$$H(\omega_n) = \frac{1}{1 - \left(\frac{\omega_n}{\omega_0}\right)^2 + i2D\frac{\omega_n}{\omega_0}} = \left| H(\omega_n) \right| e^{i\phi} \tag{14.27}$$

The principle of superposition gives the total response as $\delta(t) = \sum \delta_n(t)$.

15 Foundations on Expansive Soils

15 Foundations on Expansive Soils

15.1 Introduction

Expansive soils are defined as any soil that has the potential to undergo significant volume change as a result of changes in water content. The magnitude of volume change considered to be significant is defined in terms of the serviceability limit states performance of affected surface structures such as shallow foundations, utilities, or roadways.

Light structures such as the house shown in Figure 15.1, are generally constructed with limited knowledge of the soil conditions. However, the buildings often suffer subsequent distress because of volume changes (deformation) in the soils below the structure.

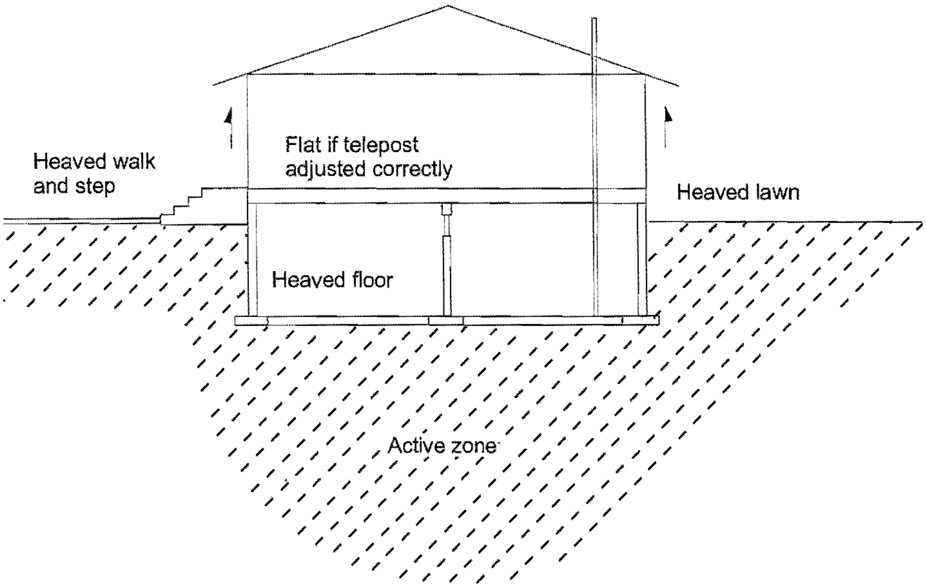


FIGURE 15.1 *Ground movements associated with the construction of shallow footings on an expansive soil (Hamilton, 1977)*

Vertical ground movements generally occur as a consequence of unloading associated with the excavation for the basement of the house, or a change in the normal evaporation and evapotranspiration regime at the ground surface. An example of structure distress can often be seen in floor slabs that are meant to function as "floating" slabs but seldom 'float' (Figure 15.2). This is just one of many ways in which light structures suffer distress due to volume changes in expansive soils.

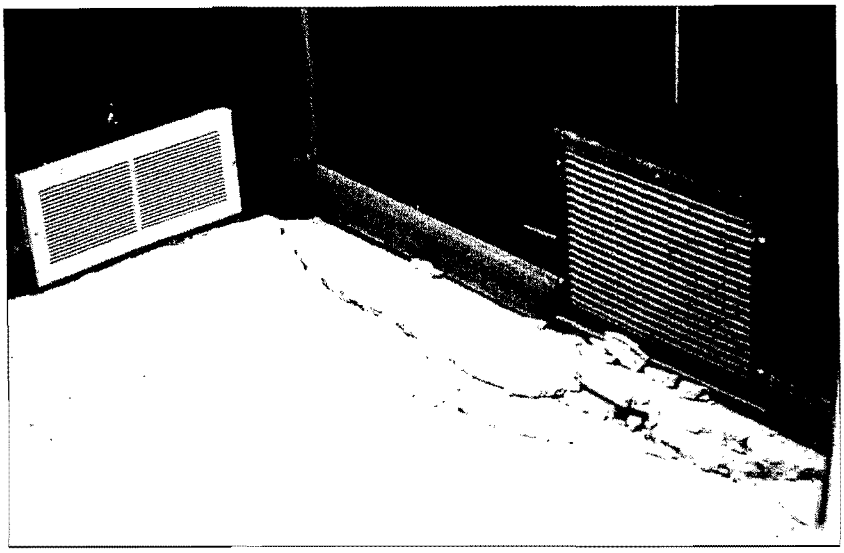


FIGURE 15.2 *Typical cracking pattern around a basement slab that was meant to perform as a "floating slab"*

Light structures most commonly experience distress associated with expansive soils; however, swelling pressures of expansive soils can be high, causing movement to multi-story structures. Relatively short piles below a light structure, along with a structural floor slab, provide a common solution to many expansive soils problems (Figure 15.3).

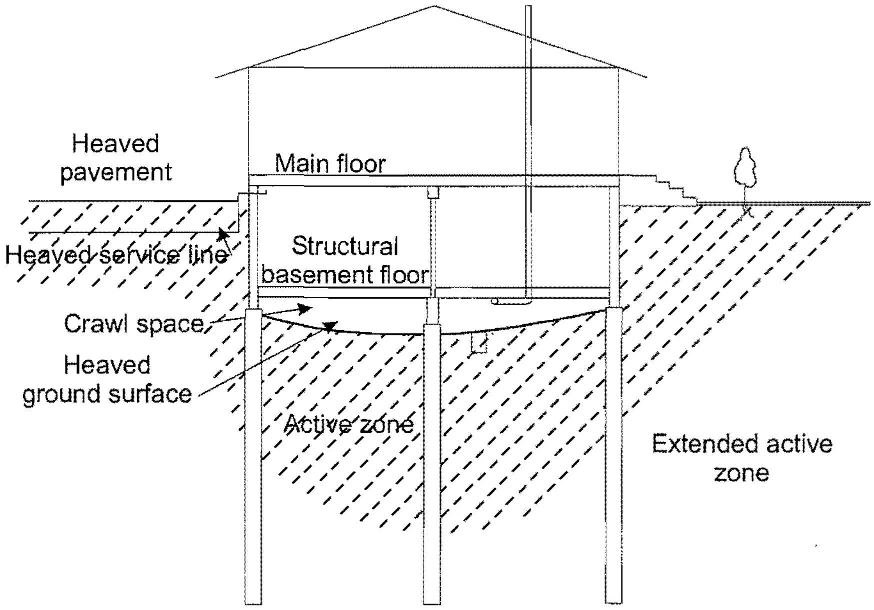


FIGURE 15.3 *Illustration of a short pile and structural floor slab below a light structure such as a house*

The potential for a soil to be expansive is largely controlled by the mineralogy and percentage of the clay-size fraction, while changes in water content are also dependent upon changes in environmental conditions at the ground surface. Environmental conditions result in the wetting and/or drying of the soil in response to moisture transfers across the soil-atmosphere boundary.

Changes in the water content in the soil may be the result of natural causes such as climatic fluctuations or the result of human activity such as surface irrigation, runoff from paved areas or leakage from buried utilities.

Expansive soils problems are encountered in almost every country of the world and have been found to be extremely costly to accommodate fully in original design or remedial design. Expansive soils have been referred to as the 'hidden disaster' in the United States and cause more damage to structures, (particularly light buildings and pavements), than all other natural hazards including earthquakes and floods (Jones and Holtz, 1973). It has been estimated that the average annual losses due to structural distress associated with expansive soils in the United States is in the order of $7 billion (Krohn and Slosson, 1980). While the amount of damage in Canada may be considerably less, it is still substantial (Fredlund, 1979).

Problems related to structures on expansive soils are accentuated since structures incurring the most damage have generally had the least engineering design prior to construction. Engineers are often reluctant to become involved in the study of expansive soils problems because the consulting fees are generally small relative to the potential risk of litigation. There is need to establish accepted standard of practice or "protocol" for geotechnical engineering practice as it relates to expansive soils.

This chapter in the Canadian Foundation Manual does not provide a complete description and analysis of problems related to the behavior of expansive soils. Rather, the goal of this chapter is simply to provide information on factors controlling heave in expansive soils and to present an outline of a simple method based on one-dimensional oedometer test results, to estimate the magnitude of potential heave.

There are several important questions that need to be addressed in order to evaluate the impact that an expansive soil may have on foundation performance:

- How can a potentially expansive soil be identified? (i.e., soil characterization).
- What environmental conditions can cause changes in water content in an expansive soil? (i.e., environment characterization).
- What methods can be used to predict the magnitude of volume change or heave that might be experienced subsequent to completion of construction? (i.e., predictive model).
- What design and remedial measures can be taken to minimize damage to light engineered structures? (i.e., design methods).

The first two questions focus primarily on the identification and characterization of expansive soils. These methods are described in Section 15.2. The next question shows that there is need to have a predictive method based on an appropriate theoretical framework to relate changes in void ratio to changes in stress state. This predictive method is outlined in Section 15.3. The final section of this chapter provides a general discussion of issues related to foundation design and remediation measures that can be taken when dealing with expansive soils.

15.2 Identification and Characterization of Expansive Soils

The geographic regions in Canada where expansive soils problems may occur can be delineated by first identifying those areas containing soils with the prerequisite mineralogy and lithology (Quigley, 1980). Secondly, the climatic conditions must lend themselves to the potential for large changes in water content. Expansive soils are comprised of clay soils that contain a significant fraction of active clay minerals. Glacial and post-glacial processes laid down most of the clay-rich deposits of concern in the construction and performance of surface structures in Canada. These clay-rich soils are found either in glacial lacustrine deposits or in glacial tills (Figure 15.4).

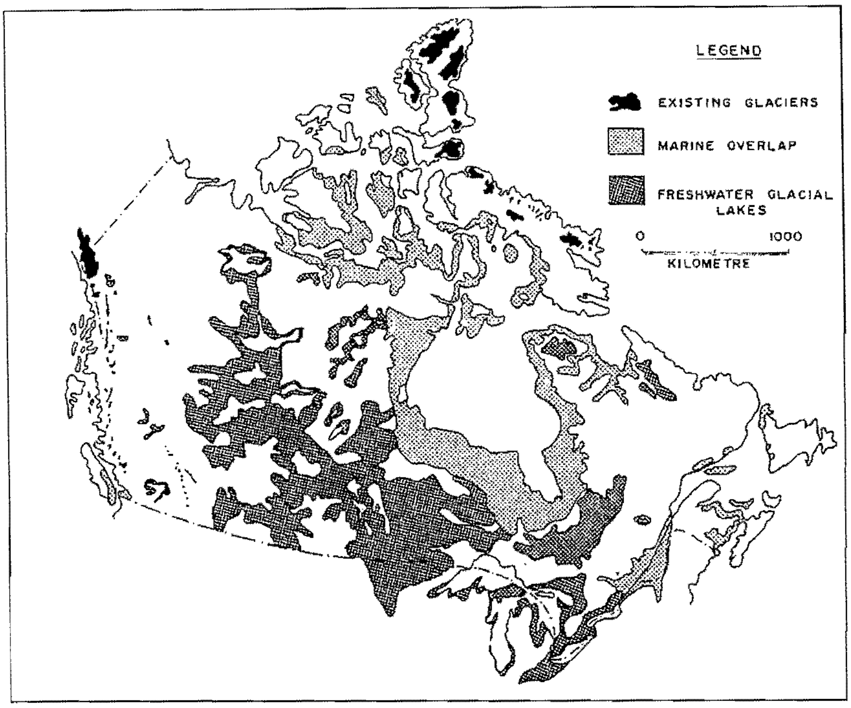


FIGURE 15.4 *Distribution of marine and freshwater glacial and postglacial lakes of Canada (Quigley, 1980)*

Many of the lacustrine deposits in Eastern Canada have illite or chlorite mica as the dominant minerals. Soils consisting of these minerals are generally considered to be non-swelling, although there may be large shrinkage upon drying if the initial void ratios are high. The Champlain Sea (or Leda) clays of the Ottawa Valley and St. Lawrence Lowlands are one of a number of such clays. In the Western provinces, the montmorillonite shales from Cretaceous formations in the Interior Plains provide the active clay minerals that give rise to expansive soils. Most of these deposits are found in lacustrine clay deposits that were once large glacial lakes. The clay deposits surrounding Lake Agazziz near Winnipeg and Lake Regina near Regina are examples of these deposits.

The natural environment, as well as anthropogenic changes in the environment, can produce significant changes in the water content of the surficial soils. In the more humid parts of Canada, clays sensitive to shrinkage have not previously been subjected to drying to the extent now occurring as a result of construction and the introduction of non-native vegetation. The surficial clays in Western Canada have historically been subjected to arid or semi-arid climatic conditions. The development of surface structures, such as light residential housing, inevitably leads to a change in moisture fluxes across the ground surface as the result of irrigation, leakage from underground utilities, or vegetation.

A general description of the soil and environmental conditions that can lead to significant volume changes in the near-ground-surface soils are described in the next section. This information can be used for the preliminary identification of potentially expansive soils areas. This is followed by a description of how to measure appropriate soil properties for use in a heave analyses, as described in Section 15.3.1.

15.2.1 Identification of Expansive Soils: Clay Fraction, Mineralogy, Atterberg Limits, Cation Exchange Capacity

A potentially expansive soil contains a relatively high percentage of highly active clay minerals. The expansion of the diffuse double layers within the clay fraction results in changes in water content. Methods of identifying the key features of potentially expansive soils are described in this section.

Standard hydrometer analyses can be used to identify the 'clay-sized' fraction that is less than two microns in diameter (ASTM D-422). However, not all particles of this size fraction are clay minerals. It is recommended that the mineralogy of the 'clay fraction' be measured. The most common method of identifying and quantifying the clay mineralogy is from an X-ray diffraction analyses. All of the clay mineral types are in close proximity on the X-ray trace and consequently it is important to use the correct opening for the X-rays, (i.e., a narrow slit), along with a qualified technician when interpreting the test results. Of primary importance is the quantification of the amount of montmorillonite (or Smectite) clay mineral in the clay fraction of the soil sample.

The Atterberg Limits, (i.e., Plastic Limit, Liquid Limit, and Shrinkage Limit), can be measured as part of a geotechnical investigation. The difference between the Plastic Limit and Liquid Limit is referred to as the Plasticity Index. The Plasticity Index is related to the percentage of clay-sized particles and the mineralogy of the clay-sized particles.

Van der Merwe (1964) provided a correlation between the Plasticity Index, the percent of clay-sized particles, and the potential for swelling as shown in Figure 15.5. Swelling potential ranged from low to very high. The highest potential for swelling occurred when the soil had a high percentage of clay-sized particles and a high Plasticity Index. A soil can be described as having a high potential for swelling but the expansiveness of the soil will only be revealed when the initial water content of the soil is low.

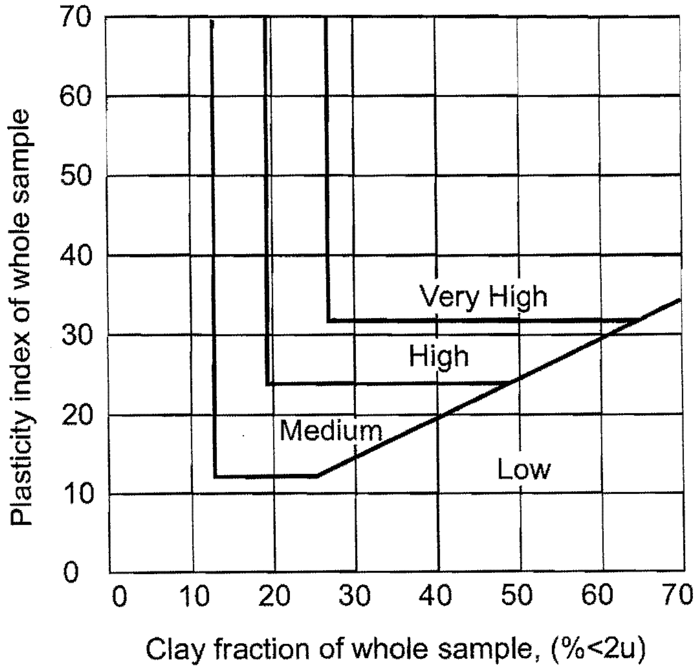


FIGURE 15.5 *Classification of potential severity of an expansive soil based on the plasticity and percent clay-sized particles (van de Merwe, 1964)*

A useful index that can be computed from the Plasticity Index and the percentage of clay fraction (%clay) is the Soil Activity (Ac):

$$Ac = \text{Plasticity Index} / (\%\text{clay}) \tag{15.1}$$

Skempton (1953) classified clays as 'inactive' when Ac was less than 0.75; 'normal' when Ac was between 0.75 and 1.25 and 'active' when Ac was greater than 1.25. It is clays in the 'active' range that cause the greatest difficulty with respect to swelling (and shrinking). Nelson and Miller (1992) listed typical values for the Activity of various clay minerals: kaolinite, 0.33 to 0.46; illite, 0.9; Ca-montmorillonite, 1.5; Na-montmorillonite, 7.2. Figure 15.6 uses the Activity of the soil and the percent clay-sized particles to classify the potential for swelling of compacted clays (Seed et al, 1962). The amount of swelling that can be anticipated with clayey soils can range from less than 1.5 % to more than 25 % depending upon the activity of the soil and the amount of clay-sized particles.

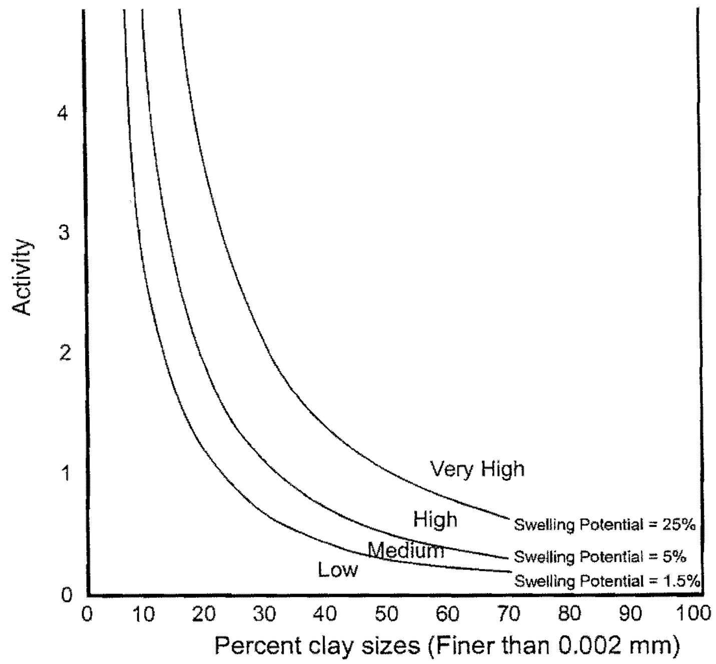


FIGURE 15.6 *Classification of potential swell for compacted clays based on the Activity of the soil (Seed et al, 1962)*

Table 15.1 was first proposed by Holtz and Gibbs (1956) and relates colloidal content (where colloids are defined to be particles less than 0.001 mm in diameter), Plasticity Index and Shrinkage Limit to the potential for volume change. The table separates soils into low, medium, high and very high categories of 'potential for expansion'. This table is not meant to be used as a basis for predicting heave, but rather to provide a preliminary assessment of the potential for volume change. It is useful to augment the table with observations from local experience.

TABLE 15.1 *Potential for Expansion as Estimated from Classification Test Data**

Potential for Expansion	Probable Expansion (% of total volume change)**	Colloidal Content (%)***	Plasticity Index (%)	Shrinkage Limit (%)
Very High	> 30	> 28	> 35	> 11
High	20 - 30	20 – 31	25 - 41	7-12
Medium	10 - 20	13 – 23	15 - 28	10 – 16
Low	< 10	< 15	< 18	> 15

* After Holtz and Gibbs (1956).
** Dry to saturated conditions - under a surcharge of 6.9 kPa (1 psi).
*** Particles less than 0.001 mm in diameter

Figure 15.7 illustrates the general pattern of percent swell for a compacted, highly plastic soil (Holtz and Gibbs, 1956). While the amount of swelling may vary from one soil to another, the pattern of total swell upon wetting from various density and water content conditions should show the same trend. The results illustrate that compaction of a soil at a high density increases the amount of swelling upon wetting. Also, compaction at water contents above optimum water content, results in reduced amounts of swelling upon wetting.

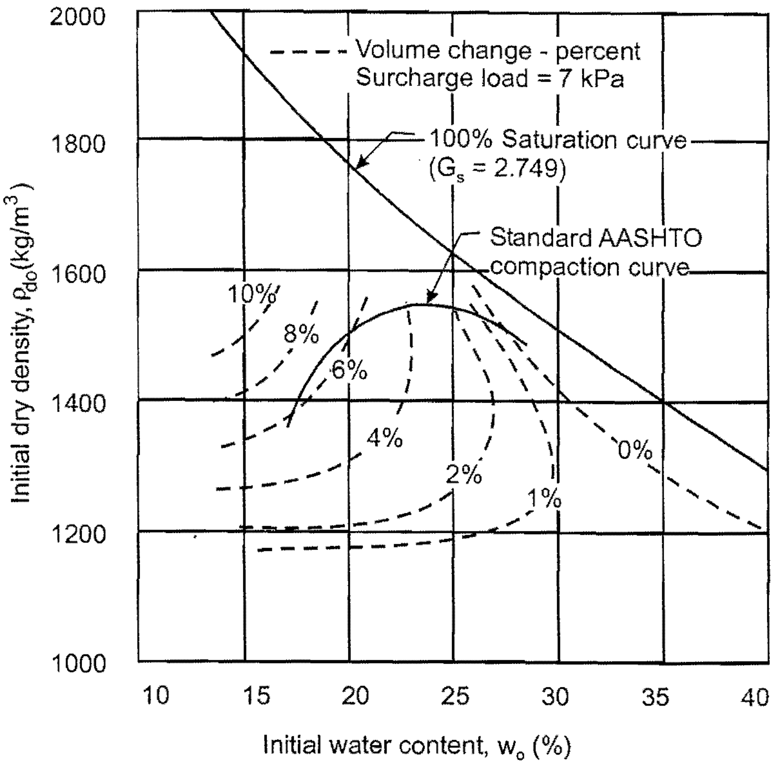


FIGURE 15.7 *Pattern of percent swell for a soil compacted at various water contents and densities (Holtz and Gibbs, 1956)*

Cation Exchange Capacity, CEC, is a measurement of the quantity of positively charged dissolved ions required to satisfy the negative charge imbalance on the surface of clay particles, and is commonly quoted in terms of millieqivalents per 100 grams of dry soil (Mitchell, 1993).

CEC is related to clay mineralogy and the amount of clay-sized particles present in the colloidal fraction. High values of CEC mean that there is a high surface activity in the clay fraction and consequently a greater potential for volume change. CEC measurements are routinely available in most agriculture soil testing laboratories. Typical values in meq/100g of soil, for the three basic clay minerals are; kaolinite = 3 to 15; illite = 10 to 40; and montmorillonite (smectite) = 80 to 150 (Mitchell, 1993).

For a soil with a given CEC, the potential expansion during wetting can also be affected by the valance of the cation adsorbed on the exchangeable sites as well as the chemistry of the pore-fluid. Most agriculture soils laboratories can measure the chemistry of the salts present in the clay using a 'saturation extraction' technique (Klute, 1986). This involves adding water to a dry soil until free water is observed to form on the clays. The sample is then centrifuged and the chemistry of the 'extract' is measured. The greatest potential expansion will occur when the adsorbed cations are monovalent (e.g., $Na+$) and when the pore-fluid is dilute. The presence of divalent cations and concentrated solutions can cause volume change due to swelling to be suppressed (Mitchell, 1993).

A soil property called the coefficient of linear extensibility, or COLE, has been routinely measured by the U.S. Soil Conservation Service, National Soil Survey Laboratory in the United States. The test measures the lineal strain of an undisturbed, unconfined specimen when it is dried from one-third of an atmosphere of suction (i.e., 33 kPa), to oven-dried conditions. The specimens are brought to equilibrium at one-third of an atmosphere and coated with a flexible plastic resin. The COLE value of many soils has been related to the swelling properties of soils and has been quite extensively used in the United States (McKeen and Neilsen, 1978; McKeen and Hamberg, 1981; Nelson and Miller, 1992).

15.2.2 Environmental Conditions

Expansive soils are generally clay-rich sediments deposited in glacio-lacustrine lakes that have undergone extensive drying since deposition. The drying is the result of evaporation from the soil surface and transpiration by vegetation. The soils must be located in an environmental condition in which potential evapotranspiration exceeds precipitation.

A useful index to quantify soil moisture deficiency was developed by Thornthwaite (1948) and is called the Thornthwaite Moisture Index (TMI). The TMI categorizes climate primarily on the average precipitation conditions and potential evaporation conditions. Negative values for the TMI indicate that the climate is arid, and consequently, expansive soil may undergo significant seasonal swelling upon wetting (O'Neill and Poormoayed, 1980). The climate categories and the associated dimensionless Thornthwaite Moisture Indices are shown in Table 15.2.

TABLE 15.2 *Climate Classification According to the Thornthwaite Moisture Index (1948)*

Climate Classification	Thornthwaite Moisture Index
Extremely Humid	> +40
Humid	+20 to +40
Sub-humid	0 to +20
Semi-arid	-20 to -40
Arid	< -40

Computational methods that more accurately compute the actual evapotranspiration from the ground surface have been developed (Wilson et al, 1991). The analysis involves the solution of a coupled heat and moisture mass transport model. The model has been applied to specific sites (e.g., for soil-cover designs) as opposed to being used to develop climatic maps.

15.2.3 Laboratory Test Methods

The one-dimensional oedometer (i.e., consolidation apparatus) has been used in many countries of the world to test and obtain physical soil properties for expansive soils. The objective of the laboratory test is to assess the in situ stress conditions and measure soil properties that can be used for the prediction of vertical heave (Fredlund and Rahardjo, 1993). Although the consolidation test was originally developed as a laboratory simulation of compressible soft clays, it can be also be used to provide valuable information on expansive soils. There are numerous test procedures that have been proposed in the literature but the two most common tests are the Constant Volume swell test (CV test), and the Free Swell test, (FS test). The test procedures for both of these tests can be found in ASTM designation D- 4546-90. Both tests are conducted in a manner similar to a consolidation test with the primary difference related to the procedure for the setup and the commencement of the test.

15.2.3.1 Constant Volume Swell Test Procedure

The Constant Volume swell test is conducted on an undisturbed soil specimen that is trimmed into a consolidation ring. The specimen is placed in the oedometer and seated under a nominal load. The specimen is then inundated with water and as it attempts to swell, the load on the specimen is increased to prevent any volume increase or swelling. When the specimen no longer exhibits a tendency to swell, the applied load is further increased in a series of increments in a manner similar to that of a conventional consolidation test. Once the recompression branch or the 'virgin' branch of the consolidation curve has been established, the specimen is unloaded in a series of decrements in order to establish the swelling index. The loading decrements are usually twice as large as the loading increments.

The Constant Volume swell test provides two important measurements that are required for predicting heave; namely, an estimate of the swelling pressure, P_s, (or more correctly, the corrected swelling pressure, P_s') and the swelling index, C_s. Although the swelling pressure of a soil is sometimes construed to be a soil property, the swelling pressure is more correctly a measure of the in-situ stress state of the expansive soil. The undisturbed soil sample was taken from its in-situ condition where it was subjected to the overburden stress (total stress). As well, the soil was subjected to the effect of negative pore-water pressures (or matric suction). The total stress and matric suction combine on the total stress plane to provide an indication of the initial state of stress in a soil. If the change in stress state is known along with the swelling index, the volume change associated with stress state changes can be computed.

Consider the stress path followed in the laboratory when a soil specimen is tested using the Constant Volume test procedure subsequent to sampling (Figure 15.8). Once the soil specimen is submerged in water, the specimen attempts to swell while the matric suction is dissipated. However, the total stress on the specimen is increased to keep the specimen from increasing in volume. Gradually, the matric suction within the soil specimen is reduced to zero and the volume of the specimen has been maintained constant by the increase in the total stress. Figure 15.8 shows that the swelling pressure represents the sum of the in-situ overburden stress and the matric suction of the soil translated onto the total stress plane. As such, the swelling pressure is dependent upon the in-situ matric suction.

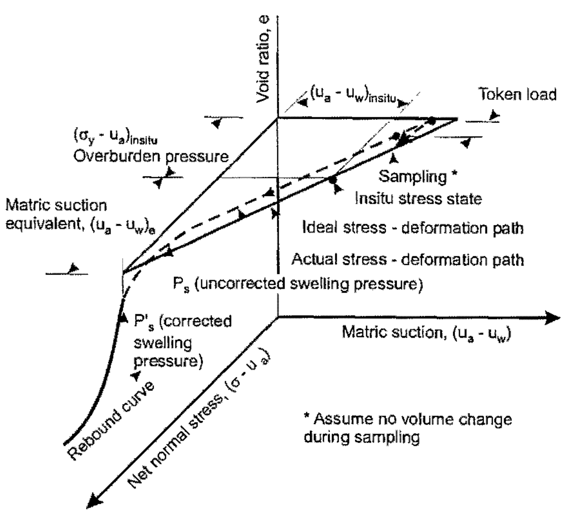


FIGURE 15.8 *Ideal and actual stress state versus void ratio path followed when performing a one-dimensional oedometer test*

The measured swelling pressure will, however, be under-estimated unless the effect of "sampling disturbance" and "apparatus compressibility" are taken into account (Fredlund, 1969). The interpretation of the Constant Volume swell test must include a correction for the compressibility of the consolidation apparatus, the compressibility of filter paper (if filter paper was used during the test), and the seating of the porous stones and the soil specimen. Desiccated swelling soils have a low compressibility and the compressibility of the apparatus can substantially affect the measurement of the swelling pressure as well as the slope of the rebound curve (i.e., swelling index). The compressibility correction can be measured by substituting a steel plug for the soil specimen and measuring deflections accruing to the apparatus under each load increment. This correction is relatively consistent for a particular consolidometer and its accessories. It is recommended that filter paper **not** be placed above and below the soil specimen because of the magnitude of its compressibility. Figure 15.9 illustrates data from a Constant Volume swell test, with and without a correction applied for compressibility.

Sampling disturbance will result in a measured swelling pressure that is lower than the in-situ value. (This phenomenon is similar to the observed effect of "sampling disturbance" on the measurement of preconsolidation pressure in a consolidation test on soft clays.) In the oedometer test, it is **not possible** for the soil specimen to return to its precise in-situ stress state after sampling without displaying some curvature on the void ratio versus effective stress plot (i.e., when going from the swelling pressure to the recompression curve or onto the virgin compression curve). The procedure for determining the 'corrected swelling pressure' begins by correcting the laboratory data to account for the compressibility of the apparatus. The correction for 'sampling disturbance' is then applied in order to establish the "corrected swelling pressure."

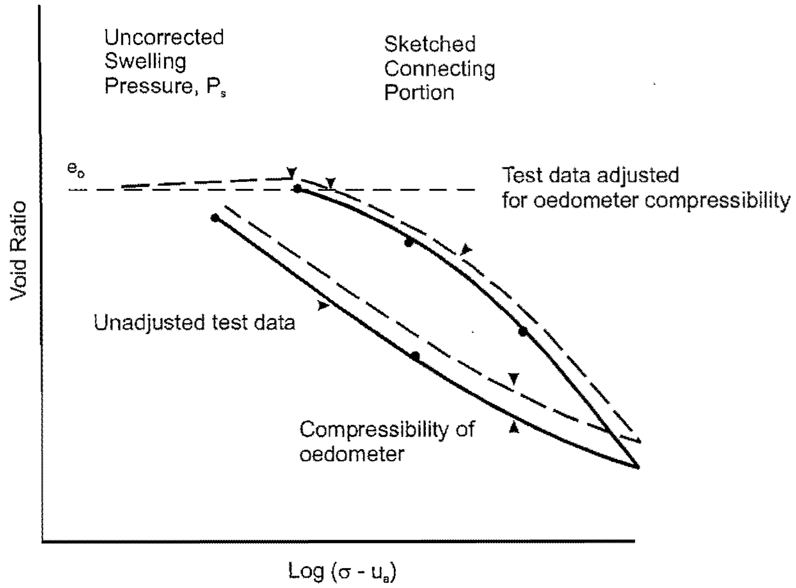


FIGURE 15.9 *Adjustment of one-dimensional oedometer laboratory test data to account for the compressibility of the apparatus (Fredlund, 1983)*

In 1936, Casagrande proposed an empirical construction that could be applied to saturated compressible soils in order to determine more accurately the preconsolidation pressure. The empirical construction was, in essence, a means to compensate for the effects of 'sampling disturbance.' A similar procedure to account for the effect of "sampling disturbance" on the swelling pressure was proposed by Fredlund (1987) and is illustrated in Figure 15.10. The slope of the rebound curve is used as part of the empirical construction procedure (rather than the slope of the virgin compression curve). The final plot of void ratio versus logarithm of total stress gives the plot shown in Figure 15.10.

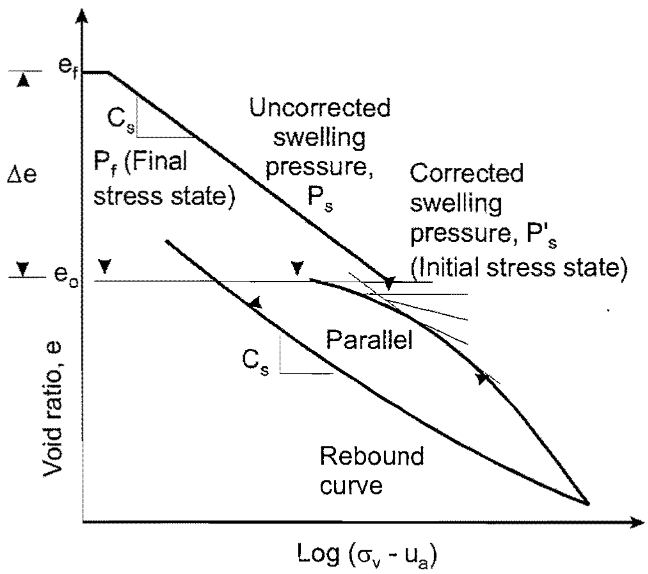


FIGURE 15.10 *Constant Volume swell test results showing the empirical procedure to correct the "swelling pressure" for the effect of sampling disturbance (Fredlund, 1987)*

The "corrected swelling pressure," P'_s, is estimated as shown and the swelling index, C_s, is obtained from the slope of the rebound curve. The 'corrected swelling pressure' and the swelling index are used as input data to the heave analyses.

15.2.3.2 Free Swell Test Procedure

The preparation of the soil specimen for the Free Swell test is similar to that described for the Constant Volume test. Once the soil specimen has been prepared, a token load is applied to the specimen. Water is then added to the oedometer pot and the specimen is allowed to swell freely until an equilibrium condition is attained. The soil specimen is then loaded by doubling the load on the specimen and allowing equilibrium to be attained under each applied load. Using this test procedure, the swelling pressure is defined as the load required for the void ratio to return to its original value as shown in Figure 15.11. It is not necessary to apply a 'correction' for sampling disturbance when using this test procedure. The effects of sampling disturbance are taken into account through the test procedure. The swelling pressure measured from the Free Swell test and the 'corrected swelling pressure' obtained from the Constant Volume test are generally quite similar (Fredlund, 1983).

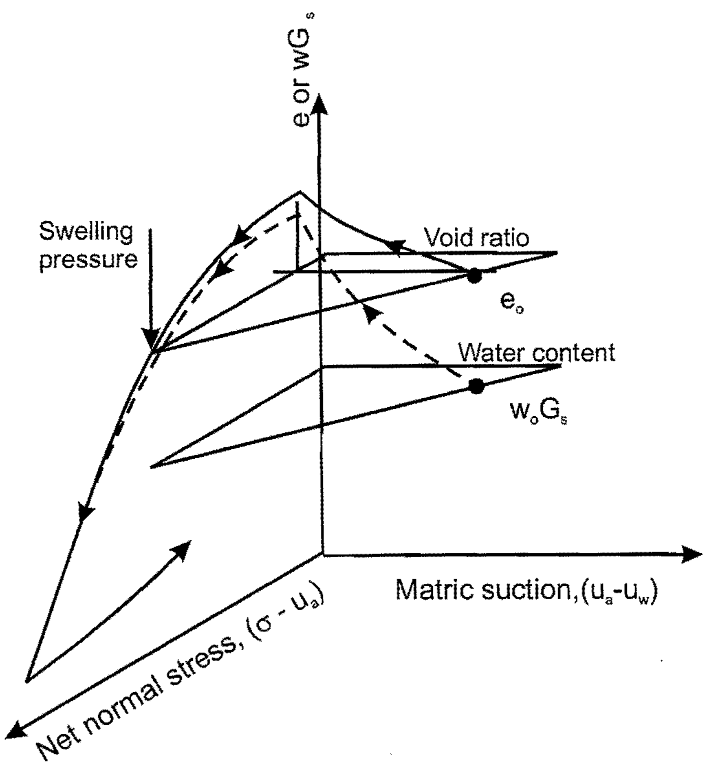


FIGURE 15.11 *Typical plot of data from a Free Swell oedometer test on an expansive soil*

15.3 Unsaturated Soil Theory and Heave Analyses

The volume change experienced in an expansive soil should be understood in terms of the changes occurring in the stress state of the soil. In other words, it is better to describe the expansion (or shrinkage) of a soil in terms of changes in the stress state rather than in terms of water content.

When a soil becomes unsaturated it is necessary to use two independent stress state tensors to define the complete stress state of the soil (Fredlund and Morgenstern, 1977). These two stress tensors are referred to as the "net normal" stress tensor and the 'matric suction' tensor, and are defined as follows:

$$\begin{bmatrix} (\sigma_x - u_a) & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & (\sigma_y - u_a) & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & (\sigma_z - u_a) \end{bmatrix}$$

and

$$\begin{bmatrix} (u_a - u_w) & & \\ & (u_a - u_w) & \\ & & (u_a - u_w) \end{bmatrix} \tag{15.2}$$

where:

σ_x , σ_y , σ = total normal stresses in the x-, y-, and z- directions, respectively,
τ_{xy} , τ_{yz} , τ_{zx} = shear stresses in the x-, y-, and z- planes, respectively,

u_w = pore-water pressure, and
u_a = pore-air pressure.

Matric suction is defined as the difference between the pore-air pressure and the pore-water pressure, (i.e., (u_a – u_w)). Changes in the environment (e.g., rainfall on the ground surface or evaporation of moisture from the ground surface), produce a change in the matric suction in the soil, with time. In other words, the matric suction tensor is changed. Likewise, changes brought about by construction (e.g., excavation of soil or the placement of fill), cause changes in the net normal stress tensor. Independent soil properties are associated with each of the two stress tensors and consequently the stress tensors must be handled in an independent manner.

The osmotic component of soil suction does not need to be taken into consideration unless the salt content of the soil is specifically changed in the problem under consideration. In general, this is not necessary because changes in the salt content in the laboratory and in-situ are similar.

In an expansive soil, the volume of the soil increases as a result of a decrease in matric suction. Similarly, the volume of the soil decreases as a result of an increase in matric suction. The volume of the soil can also decrease in an independent manner as a result of changes in the external loading. Analytical procedures related to the prediction of heave should be visualized and understood in terms of changes in the stress state of the soil. It is particularly important to visualize the expansive soils problem in terms of two independent stress state variables because changes in the pore-water pressure are always three-dimensional in character while external loading imposed by man's design are more commonly one-dimensional or two-dimensional in character. For example, a vertical total load will produce a tendency for an outward movement in the lateral direction while an increase in matric suction will have a tendency for inward movement in the lateral direction.

Numerous testing procedures and analytical procedures have been proposed in the research literature for predicting the amount of heave that can be anticipated in an expansive soil under various soil and design configurations. Generally, the success of each of the methods is somewhat limited by incomplete appreciation of, or inability to predict, the changes in environmental conditions. The present state-of-the-art in predicting maximum probable heave is satisfactory for most engineering purposes; however, the prediction of the rate at which the volume changes may take place is considerably more difficult because it depends upon the availability of water to the soil.

The rate of heave is also related to the coefficient of permeability of the soil. Field rates of heave are strongly influenced by the macrostructure of the soil, which is difficult (if not impossible) to assess from a laboratory test. The unpredictable availability of water from surface and subsurface sources is also difficult to predict.

Field shrinkage rates are affected by the efficiency with which moisture can be removed from the subsoil. Evapotranspiration proceeds in a fairly predictable manner when the water content of the soil is high, but is less predictable at lower water contents because of plant-root extensions, plant wilting, soil cracking and other factors.

15.3.1 Prediction of One-Dimensional Heave

The prediction of heave (or swelling) can be carried out in a manner similar to that used when calculating consolidation or settlement of a soft clay layer (Fredlund et al, 1980). The prediction of heave requires an understanding of the initial and final stress states and the deformation modulus of the soil. The Constant Volume swell test provides the necessary information to assess the initial stress state (i.e., the corrected swelling pressure), while the swelling index, C_s, is taken as the deformation modulus.

The swelling index, C_s, generally ranges from 10 to 20 percent of the compressive index, C_s, for a particular soil. Figure 15.12 shows approximate values for the swelling index values that have been correlated with the liquid limit and the rebound void ratio of a soil (NAVFAC DM-7, 1971). The estimated values of the swelling index are useful for obtaining an estimate of the swelling.

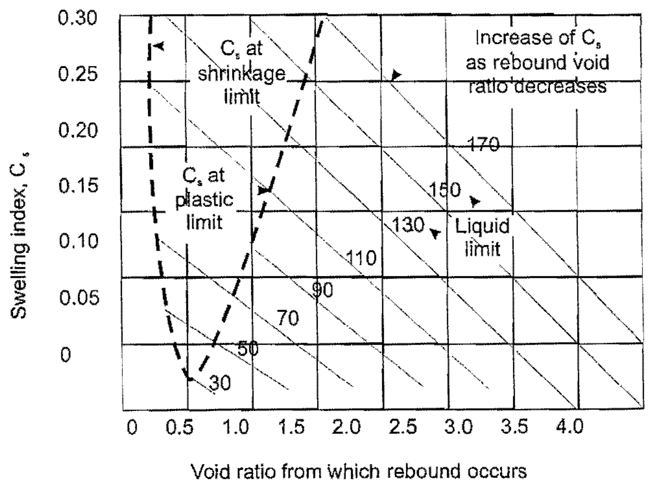


FIGURE 15.12 *Correlation of swelling index, Cs, with the Atterberg Limits and in situ void ratio for an expansive soil (NAVFAC DM-7, 1971)*

The equation of a straight line on a semi-logarithm plot can be used as the basic equation for the prediction of heave. The equation corresponds to the in situ stress paths projected onto the net normal stress plane (Figure 15.13).

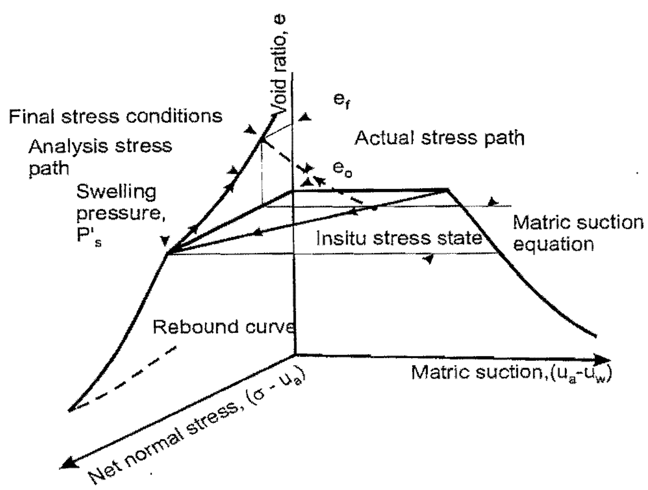


FIGURE 15.13 *Actual stress path in situ and the stress path used in the analyses for total heave*

The stress path followed during the swelling of the soil corresponds to the rebound curve (i.e., C_s) from the initial stress state to the final stress state. The equation for the rebound portion of the swelling curve can be written as follows:

$$\Delta e = C_s \log\left(\frac{P_f}{P_o}\right) \tag{15.3}$$

where:

Δe = change in void ratio (i.e., $e_f - e_o$)
e_o = void ratio
e_f = final void ratio
C_s = swelling index
P_f = final stress state
P_o = initial stress state or the "corrected" swelling pressure, P'_s

The initial stress state, P_o, can be visualized in terms of the overburden pressure plus the matric suction equivalent (see Figure 15.14):

$$P_o = (\sigma_y - u_a) + (u_a - u_w) \tag{15.4}$$

where:

σ_y = total overburden pressure
$(\sigma_y - u_a)$ = net overburden pressure
$(u_a - u_w)$ = matric suction.

The pore-air pressure in the field can be assumed to remain at atmospheric conditions. The initial stress state, P_o, can always be taken as the 'corrected swelling pressure,' P'_s. The final stress state, P_f, must take into account total stress changes and the final pore-water pressure conditions.

$$P_f = \sigma_y \pm \Delta\sigma_y - u_{wf} \tag{15.5}$$

where:

$\Delta\sigma_y$ = change in total stress due to excavation or the placement of fill
u_{wf} = final pore-water pressure.

An estimate of the final pore-water pressures must be made as part of the assessment of the final stress state (Hamilton 1969). Several possibilities can be considered as reasonable long-term pore-water pressure states. First, it could be assumed that hydrostatic conditions above and below an estimated water table would be reasonable. Assuming that this water table rises to ground surface is the most conservative assumption and will produce the greatest estimate of heave. Second, it could also be assumed that soil suctions throughout the soil profile will dissipate to zero but that no positive pore-water pressures will develop. Third, it could be assumed that under long-term equilibrium conditions, the pore-water pressures will remain at a slightly negative value. This assumption produces the smallest prediction of heave. It has been observed that all of these assumptions related to final pore-water pressure conditions generally produce similar estimates of heave since most of the heave occurs in the uppermost soil layer where the matric suction change is largest.

The selection of the final pore-water pressure boundary conditions can vary from one geographic location to another depending upon climatic conditions. For example, the equilibrium suction below an asphalt pavement surface has been related with the Thornthwaite Moisture Index. On many small, engineered structures, however, it is often artificial causes such as leaky water lines and poor drainage that control the final pore-water pressures in the soil.

The heave of an individual soil layer can be written in terms of a change in void ratio as follows:

$$\Delta h_i = \frac{\Delta e_i}{1+e_{oi}} h_i \tag{15.6}$$

where:

Δh_i	=	heave of an individual layers
h_i	=	thickness of the layer under consideration
Δe_i	=	change in void ratio of the layer under consideration (i.e., $e_{fi} - e_{oi}$)
e_{oi}	=	initial void ratio of the soil layer, and
e_{fi}	=	final void ratio of soil layer.

The change in void ratio, Δe_i, in Equation 15.6 can be computed using Equation 15.3 to give the following form:

$$\Delta h_i = \frac{C_s}{1+e_{oi}} h_i \log\frac{P_{fi}}{P_{oi}} \tag{15.7}$$

where:

P_{fi}	=	final stress state in the soil layer, and
P_{oi}	=	initial stress state of the soil layer.

The total heave from several soil layers, ΔH, is equal to the sum of the heave for each soil layer.

$$\Delta H = \sum \Delta h_i \tag{15.8}$$

15.3.2 Example of Heave Calculations

Figure 15.14 illustrates the calculations required to predict the potential heave from a 2-meter layer of expansive soil. The initial void ratio is 1.0, the total unit weight is 18 kN/m³ and the swelling index, C_s, is 0.1. Only one oedometer test was performed on a soil sample taken from a depth of 0.75 m and the measured, 'corrected' swelling pressure was 200 kPa. It is assumed that the 'corrected' swelling pressure is constant throughout the 2-meter layer and that the ground surface will be covered with an impermeable layer such as asphalt. The suction in the soil below the asphalt will decrease with time due to the discontinuance of evaporation and evapotranspiration from the ground surface. It is assumed that the final pore-water pressures will eventually go to zero at all depths.

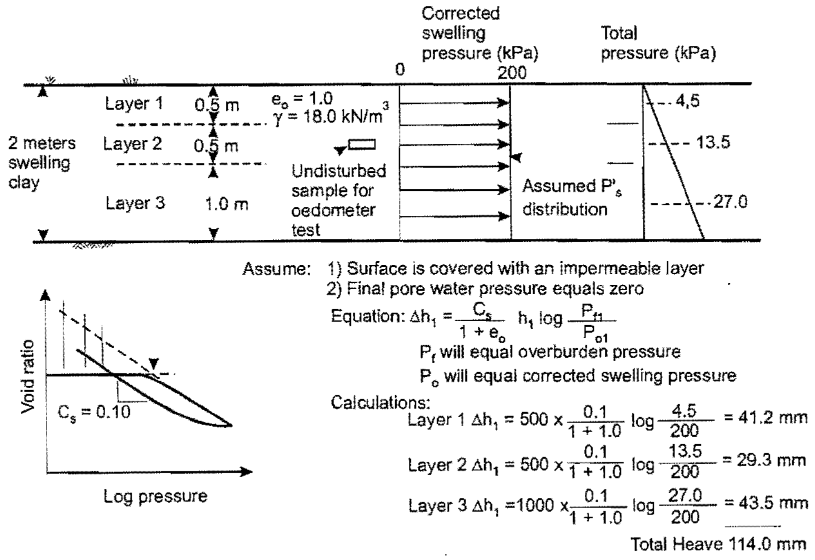


FIGURE 15.14 *Example illustrating heave calculations for 2-meter layer of expansive soil when matric suction becomes zero*

The 2-meter layer is subdivided into three layers, the top layer being the thinnest. (Normally more than 3 layers would be used to obtain an accurate solution). The amount of heave in each layer is computed by considering the mid-point of each of the three layers. The initial stress state, P_o, is equal to the 'corrected' swelling pressure at all depths. The final stress state, P_f is equal to the overburden pressure. Equation 15.7 is used to calculate the heave for each layer. The calculations shown on Figure 15.14 reveal a total heave of 114 mm. About 36 % of the total heave occurs in the upper quarter of the clay strata.

15.3.3 Closed-Form Heave Calculations

The calculation of the amount of heave depends primarily on the swelling pressure and the swelling index of the soil. It is possible to compute closed-form solutions for problems with specific geometric boundary conditions using Equations 15.7 and 15.8. The assumption is made that the final soil suction will be zero. Figure 15.15 shows the general layout of the geometry under consideration, divided into a number of equally spaced layers.

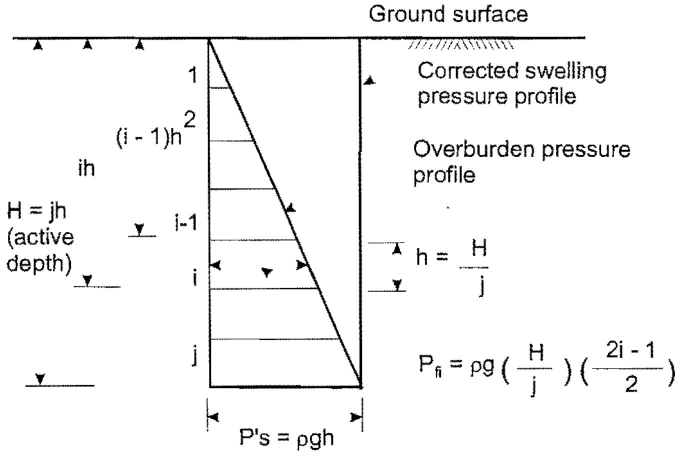


FIGURE 15.15 *Idealized geometry profile used for the "closed-form" solution for the amount of potential heave if the soil suction becomes zero*

The expansive clay layer is assumed to start at the ground surface. The soil is assumed to become wet (i.e., the soil suction goes to zero) to a depth where the overburden becomes equal to the swelling pressure. The total heave for an expansive soil can then be computed as shown in Figure 15.16. The total heave increases significantly as the swelling pressure of the soil increases. However, it must be possible for water to enter the entire soil profile in order for the potential heave to be realized.

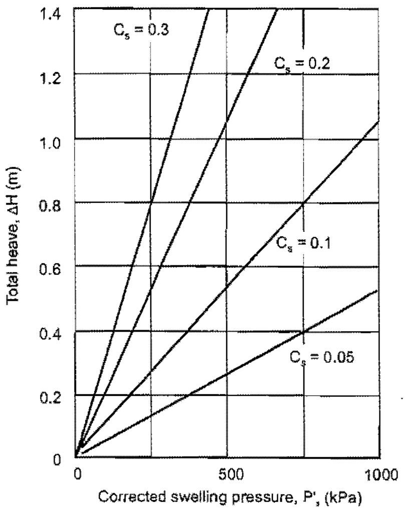


FIGURE 15.16 *Closed-form calculation of total heave when the soil suction becomes zero*

15.4 Design Alternatives, Treatment and Remediation

Following are some general guidelines regarding the design of foundations on expansive soils and the control of the 'active zone.' The basic concept behind the design of a foundation system on expansive soils involves giving detailed attention to control of the environment (e.g., moisture movement) or to isolation of the structure from soil movement. In general, it is not prudent to attempt to resist movement imposed by swelling soils. Rather, it is better to attempt to control the environment (i.e., moisture control) surrounding the structure. Suggestions for moisture control are given following a description of possible foundation designs for expansive soils.

15.4.1 Basic Types of Foundations on Expansive Soils

There are three general foundation alternatives for expansive soils:

- shallow spread footings
- a pier and beam system, or
- stiffened slab-on-grade.

Shallow spread footings are the most common type of foundation for light structures. Generally there is little engineering design associated with these foundations and consequently these structures suffer distress when placed on expansive soils. It is often difficult to convince owners that additional funds should be initially invested in an adequate foundation that is placed on expansive soils. Generally, an initial investment in engineering consultation will prove to be a wise investment after a few years.

15.4.2 Shallow Spread Footings for Heated Buildings

Shallow foundations may be economical and give adequate service for certain structures on soils of low-to-moderate volume-change potential in humid to sub-humid regions. The foundations should be reinforced to minimize effects of seasonal edge movements and non-uniform bearing surfaces, such as over service trenches. The spread footing foundation should perform satisfactorily provided there are no deep-seated or long-term effects such as major changes in the water table (i.e., pore-water pressure conditions) or vegetation conditions. Shallow spread footing foundations will not likely perform well under severe environmental conditions.

Good engineering design practices include giving consideration to the following issues:

- Positive surface drainage should be provided away from the structure by carefully selecting the slab surface and the outside grade elevations;
- Placing the slab on a granular, free-draining fill;
- Ensuring stable and uniform moisture conditions under and around the foundation;
- Excluding deep root penetration under the foundation and protecting against undetected leakage from underground piping;
- Preventing the back-up of water through poorly backfilled trenches; and
- Providing adequate perimeter insulation around the foundation to eliminate steep thermal gradients through reactive soils under and around the foundations.

Other precautions worth consideration as part of the 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 differential movements; and
- Providing free-spanning of floors and roofs between load-bearing exterior walls and frames, wherever possible.

15.4.3 Crawl Spaces Near or Slightly Below Grade on Shallow Foundations

In addition to the recommendations given above, crawl-space designs require that special attention be given to the following issues:

- Provision of adequate drainage slopes to sump areas and drainage-tile beds within the crawl space;
- Provision of adequate ground cover in the crawl space to control evaporation of moisture from the soil;
- Provision of adequate heat supply and insulation to prevent frost penetration below footings and to control extreme thermal gradients in soils below and around foundation units. This is necessary to prevent excessive accumulation of moisture or the drying in the underlying soils; and
- Provision of adequate ventilation of the crawl space throughout all seasons to prevent condensation on or within structural materials in the crawl space.

The magnitude of total, differential, and tilt movements of shallow foundations will depend on the many factors related to the active zone and the reactivity of the soils on the site. Even for soils of relatively low volume-change potential, some differential movement of the perimeter spread footing units relative to central units will occur. Relative movement should be anticipated and provision should be 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.

The magnitude of total and differential movements experienced by structures on shallow foundations is influenced by the net unloading of the soils. This is the case even with a typical full-basement excavation and a lightweight one or two-storey building. Although central footings may be designed to carry equal structural loads and to have similar dimensions to ensure similar stress increases in the underlying soils, the net area-unloading effect of the excavation has a much deeper influence. Consequently, deep-seated heaving tends to affect central footings much more than perimeter footings. The provision of adjustable columns in important for these situations.

Serious attention must also be given by designers to stacks, chimneys, heating ducts, furnaces, and other equipment placed on ground-supported basement floors. On moderate-to-high volume-change soils, differential heaving of basement floors will likely become excessive and objectionable to many occupants over a period of a few years after construction. This problem can best be addressed at the design stage by providing a structural basement-floor system that spans between foundation supports. It is also possible to provide an adjustable flooring system that can easily be maintained by the occupant or owner. All shallow foundations may be subject to tilt deformations or localized settlement caused by non-uniform soil 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 an equivalent fluid-pressure.

15.4.4 Pile and Grade-Beam System

A pile and grade-beam foundation system generally provides a superior foundation to that of a spread footing system. The piles are generally of the cast-in-place concrete type but other types of piles can also be used. The piles need to be extended below the depth of seasonal ground movement. A grade-beam system supports the loads between the piers. A structural floor slab system tied into the grade-beam generally performs well. However, a floating slab resting on the grade beams can also prove to be a satisfactory system. Compacted sand or gravel is generally placed below the floor slab but sometimes the floating slab is placed directly on the soil Figure 15.17.

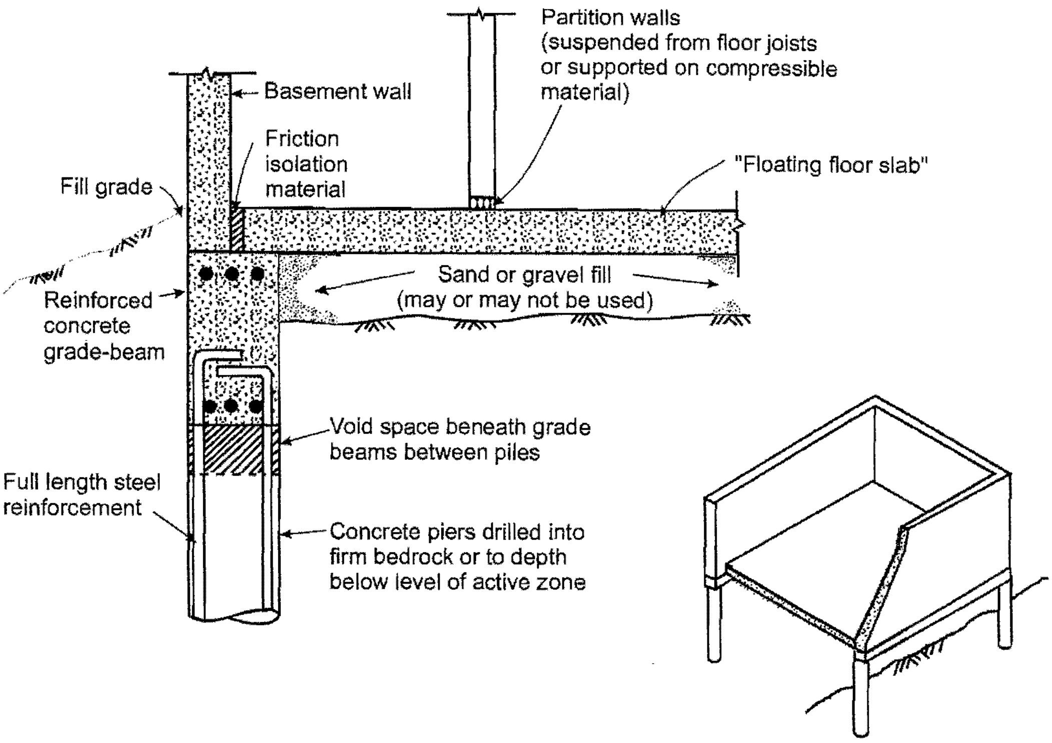


FIGURE 15.17 *Typical layout for a drilled pile and grade beam foundation system (Nelson and Miller, 1992)*

Good engineering design practices include giving consideration to the following issues:

- The piles need to be extended well below the depth of seasonable movement and have sufficient depth to resist uplift resulting from the expansion of the soil,
- The piles may be straight shafts or may be belled at the bottom, as deemed most suitable for the structure under consideration,
- The piles need to be reinforced to resist the potential uplift forces associated with the expansion of the soil in the upper portion of the profile,
- Consideration may be given to the possibility of using a material along the upper portion of the pile that reduces the adhesion of the soil to the pile in the swelling portion of the profile,
- The grade-beams need to be tied into the grade-beams,
- A space must be left below the grade-beams (i.e., between the locations of the piles) in order to accommodate potential upward swelling of the soil below the grade-beam. The amount of space that must be left below the grade-beam varies depending upon the soil conditions but will commonly be in the order of 150 mm (6 inches) or more, and
- Precautions previously mentioned related to surface drainage need to be respected for pile and grade-beam systems as well.

15.4.5 Stiffened Slabs-on-Grade

Stiffened slabs-on-grade (Figure 15.18) are not a common type of foundation system in Canada because of the adverse weather conditions. Frost penetration further accentuates the potential for foundation movements, over and above that due to expansive soils. In situations where a stiffened slab-on-grade might be considered as a potential foundation type, a competent structural and geotechnical engineer should be retained to design a system that can be ensured to perform satisfactorily.

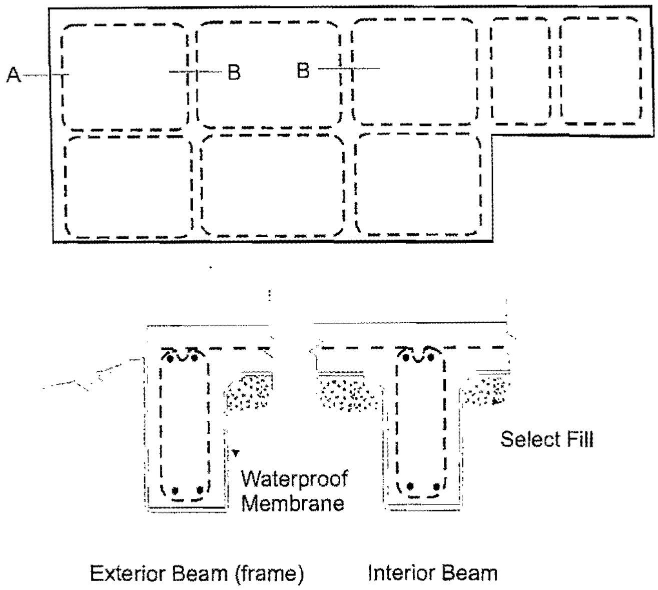


FIGURE 15.18 *Typical layout for a reinforced slab-on-grade (Nelson and Miller 1992)*

The Building Research Advisory Board Recommendations for the design of residential stiffened slabs takes hogging and sagging ground movement conditions into consideration. A qualified structural engineer must be retained to design the necessary reinforcement that must be included in the slab.

15.4.6 Moisture Control and Soil Stabilization

Measures that ensure a control on the movement of moisture in and out of the foundations soils should be made a part of the foundation design. Numerous procedures have been used in various parts of the world. Some of the procedures have proven to be successful in some countries while not providing a successful solution in other countries. It is important that a qualified geotechnical engineer be retained to ensure that moisture control and soil stabilization techniques are assessed and applied in an appropriate manner for the situation at-hand. There are a number of details that can be added to the design to ensure the successful performance of the foundation system. Figure 15.19 shows a concrete apron placed around a foundation, tied into the foundation system. Low permeability aprons have been found to perform quite well in reducing differential heave in expansive soils.

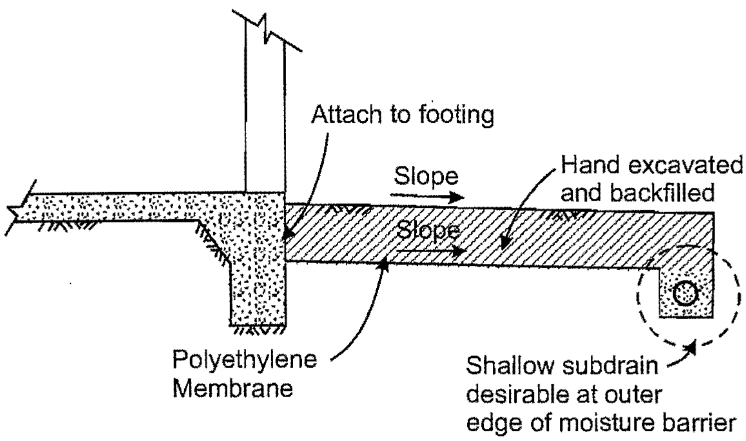


FIGURE 15.19 *Details such as a (concrete) membrane tied to the foundation system can assist in controlling infiltration (Nelson and Miller 1992)*

Some other possible soil stabilization techniques are as follows:

Soil Stabilization: Many expansive soils can be rendered essentially inert through the addition of lime. Lime stabilization designs can be considered; however, in most situations it will be sufficient to use a lime modification procedure. Lime modification usually requires that only 6 to 8 percent lime be mixed with the soil. Soil testing is required in each situation to determine the amount of lime that should be added. A decision regarding the appropriate amount of lime to add can be based the reduction in the plasticity index as a result of adding lime. It should be noted that the addition of lime to the soil may not be a potential option in many situations because of the toxic nature of lime.

Remove and Replace: In some situations where the expansive soil is relatively shallow, it may be possible to excavate and replace the expansive soil. The cost-effectiveness of this option will usually control whether or not it is an option that should be considered.

Mixing for Homogenization: It is the highly heterogeneous nature of expansive soils deposits that give rise to differential ground movements that are essentially equal to the total ground movements. However, the excavation of a soil deposit followed by the subsequent recompaction of the soils will result in reduced and more uniform ground movements. The use of the mixing and recompaction of a soil deposit should be used under the supervision of a qualified geotechnical engineer. A laboratory-testing program should be undertaken to verify that excavation and recompaction will produce the anticipated results.

Pre-wetting: The swelling potential of a soil can theoretically be eliminated by soaking the soil prior to construction. However, this practice may not produce satisfactory results. It would appear that this practice has been used successfully in some parts of the world but there are probably more situations where it has been unsuccessful. The problem appears to arise with the difficulty in obtaining a uniform wetting of the soil. If the soil is cracked near to the ground surface, it appears that the expansive soils in the upper portion of the profile swell closed and then it is not possible for further wetting to occur in a reasonable period of time.

After construction has been completed, the moisture in the soil often undergoes a slow redistribution process with the result that the structure suffers distress. The pre-wetting technique should only be used after thorough study under the supervision of a qualified geotechnical engineer.

Chemical Stabilization: There are chemicals other than lime that can be used to stabilize an expansive soil. These chemicals may be salts, enzymes or other chemicals. It is important that the effect of the addition of any particular chemical on the behavior of the soil be thoroughly studied and appraised by a qualified geotechnical engineer prior to it use.

Surcharging: Placing a load, such as an inert fill, overtop of an expansive soil may significantly reduce the potential for volume change. The amount of load applied will depend upon the swelling pressure of the soil. The greatest amount of swelling generally occurs near to the ground surface. Consequently, an inert fill can be quite effective in reducing swelling even though it will not likely eliminate the total amount of swelling. The amount of load that would need to be applied for a particular situation should be assessed by a qualified geotechnical engineer.

Capillary Barriers: A capillary barrier is a more coarse-grained material such as a silt, sand or gravel, placed over the expansive soil. Normally a coarse-grained material is thought of as a highly permeable material that will merely allow water to reach the expansive soil with the result that swelling will occur. However, the coefficient of permeability can be extremely low when a coarse-grained material has a low degree of saturation. The water storage capacity of a finer/coarser series of soil layers can be made quite large. This form of capillary barrier can be extremely effective in reducing the amount of water that reaches the expansive soil by storing infiltrating moisture near the surface where it can be released back into the atmosphere by evapotranspiration. The capillary barrier needs to be designed such that it has the appropriate air entry value and storage properties for the situation-at-hand. The design of the capillary barrier must be consistent with the climate and drainage conditions at the site. Capillary barriers have been effectively used around light-engineered structures to reduce the amount of distress to the structure.

Each of the above potential solutions to handling an expansive soil should be reviewed and studied by a qualified geotechnical engineer. The need for input from a qualified geotechnical engineer cannot be over-stated because case histories reveal that often steps taken to remedy the expansive soils problem merely aggravate the situation.

16 Site and Soil Improvement Techniques

16 Site and Soil Improvement Techniques

16.1 Introduction

A number of techniques can be used to improve the strength and compressibility of subsoils that are too weak to support conventional shallow foundations. These include preloading, vertical drains, dynamic consolidation, vibro processes, lime treatment, ground freezing, blast densification, compaction grouting, chemical grouting, vacuum preloading, and electrical strengthening methods.

A state-of-the-art report by Mitchell (1981) presents a comprehensive review of soil improvement techniques that complements the techniques highlighted in this section. Additional information is provided by Bell (1993) and Moseley (1993).

16.2 Preloading

16.2.1 Introduction

The preloading technique was developed in the 1940s, mainly in connection with highway construction. Since that time, it has been applied to a wide variety of projects, including buildings, storage tanks, airfields, flood control structures and land reclamation projects (Johnson, 1970a,b). The technique has been used to improve all types of natural cohesive soils (including peat), deposits of loose sand and silt, and fills, including waste materials. It is uneconomical and impractical for structures with heavy, concentrated loads.

16.2.2 Principle of Preloading

Ground treatment by preloading implies placing a load on top of the ground to be treated well in advance of the construction of the proposed structure. The magnitude of the pressure exerted by the load would usually be greater than the maximum pressure imposed by the proposed structure.

Temporary surcharge loads (defined as loads in excess of the final loading) are frequently employed to decrease the time required for preloading. Such surcharge is needed when the preloading is intended to minimize the effects of secondary compression.

Methods of applying the preload are by earth fills, water loading, vacuum under impervious membranes, and groundwater lowering by well-points or deep wells.

After removal of the preload, including any surcharge, a slight rebound is to be expected. However, the rebound is, usually, very small and negligible. Construction of the final structure may start over the precompressed soils immediately after removal of the preload. The principle of the precompression technique is illustrated in Figure 16.1.

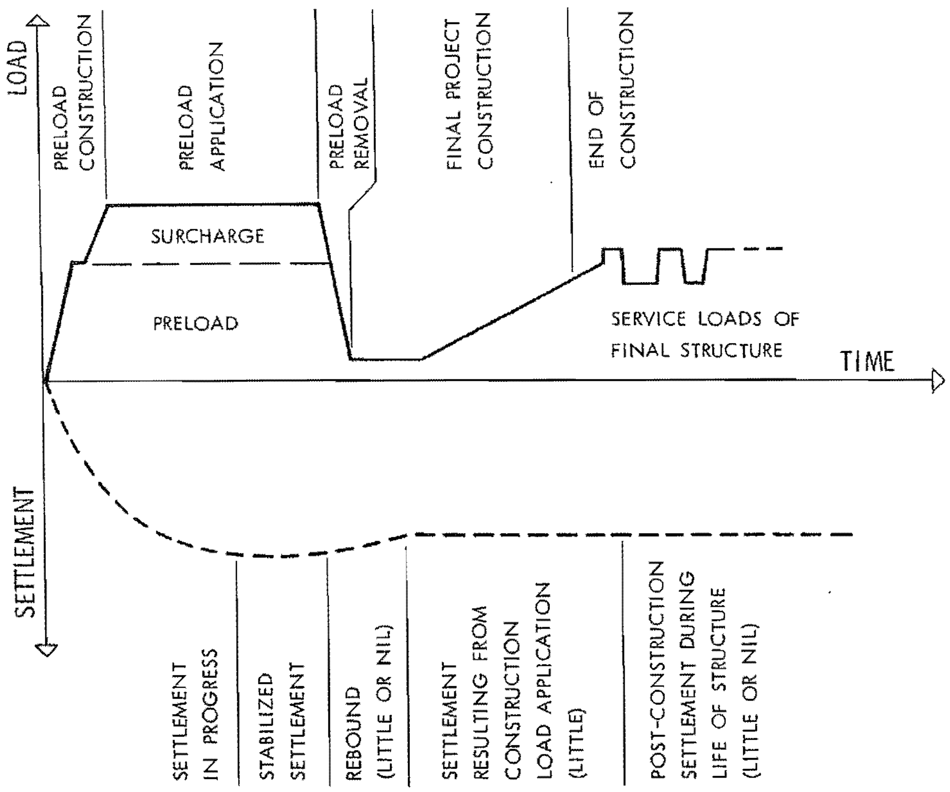


FIGURE 16.1 *Principle of preloading technique*

16.2.3 Design Considerations

16.2.3.1 Evaluation of Settlement

In the planning of a preloading program, the magnitude and duration of consolidation under preload need to be evaluated. This can be done using the methods in Chapter 11 of this Manual. Settlement calculation requires that the stratigraphy and properties of the subsoil be determined through a soil investigation of the site. Parameters such as in-situ shear strength, preconsolidation pressure, compression index, swelling index, coefficient of consolidation or modulus number, stress exponent, and permeability may be required. In coarser soils having a permeability exceeding about 1 x 10^{-6} m/s, settlement will occur rapidly, and a preloading time of a few months is normally sufficient. In more impervious soils, vertical drains (Section 16.3) may be employed to accelerate the consolidation.

16.2.3.2 Stability

If the foundation soils are weak, the design of a preloading program must also consider stability. This may require stability berms and the use of a controlled rate of loading to enable gain in strength of the foundations soils during preloading (Tavenas et al., 1978). Vertical drains also serve the purpose of accelerating the increase in strength of the foundation soils.

16.2.3.3 Size of Preloading Area

The preloading area must exceed the limit of the final structure in such a way that the stresses induced at any depth in the foundation soils by the preload under the edge of the proposed structure are uniform and at least equal to or, preferably, greater than the final stresses at that location. In addition, it is desirable to extend the preload area to allow for possible future extension of the proposed structure.

16.2.3.4 Instrumentation

A proper instrumentation program should be mandatory for all preloading schemes to provide a continuous monitoring of the results. The instrumentation should be designed to monitor in representative locations and depths the magnitude and duration of settlement during preloading, during removal of the preload, and during the construction period of the structure. Monitoring of the final structure for several years after construction is a recommended practice.

In pervious soils, instrumentation may be limited to a number of settlement points installed at final foundation level to monitor the overall settlement of the compressible subsoil. In clayey soils, the instrumentation may include settlement gauges at variable depths below ground surface and piezometers to monitor the rate of pore pressure dissipation and degree of consolidation. Inclinometer may be added to measure horizontal deformations at the edge of a preload fill and to monitor settlement distribution with depth, if required.

16.2.3.5 Foundation Design for the Final Structure

Once the preloading technique has been applied on a compressible ground to make it capable of supporting the foundations for the final structure, design of these foundations may be done using usual procedures as recommended for spread footings or rafts. In sizing footings, particular attention must be paid to those shallow foundations that rest at the surface of a thin layer of fill over soils with little or no confining pressure around the footing.

16.2.3.6 Advantages and Disadvantages

The preloading technique offers several advantages over other ground improvement methods, in particular, when time restrictions are not critical and materials used to apply the preload are available at low costs. The main advantages are:

- Post-construction settlement is reduced to relatively small values, in particular for foundations over heterogeneous soils.
- The preload material may be re-used as general backfill of a site after the completed preloading. This may represent an important economic factor in the selection of a ground improvement method.
- The preloading technique is a 'quiet' one, free of vibrations or noise usually accompanying other techniques of ground improvement, and should be considered when environmental restrictions are imposed.

The main disadvantages of the preloading technique are:

- settlements may take longer than expected, causing delays that may be economically unacceptable;
- disposal of fill material used for preloading may represent a costly item, unless it can be reused on the site; and
- future extensions of the proposed structure need to be considered in the preloading program, which may impose an undesirable initial investment for the foundations of the future structures.

16.3 Vertical Drains

16.3.1 Introduction

Settlements in clayey soils take a long time to develop. The time required depends on two main factors - linearly on the permeability of the soil, and exponentially on the drainage path, i.e., the thickness of the settling soil layer (see Section 11.11). The time can be reduced appreciably if the drainage path is shortened by means of vertical drains. The spacing between the drains controls the length of the drainage path. For instance, drains installed at a spacing that is a tenth of the thickness of a soil layer that is drained on both sides could accelerate the settlements about 25 times. Furthermore, as the permeability of the soil in the horizontal direction is generally several times larger than the permeability in the vertical direction and the drainage when using vertical drains occurs in the horizontal direction, the time for completion of the primary settlement is further shortened.

The potential benefits of using vertical drains became obvious very soon after Terzaghi in 1926 published his theory of consolidation. Thus, vertical drains have been used in engineering practice for more than 50 years. At first, vertical drains were made of columns of free draining sand (sand drains) installed by various means. In about 1945, premanufactured band-shaped drains were invented (Kjellman, 1948) and, since about 1970, the technical and economical advantages of the premanufactured band-shaped drains have all but excluded the use of sand drains.

16.3.2 Theoretical Background

For the analysis of the acceleration of pore pressure dissipation in fine-grained soils (drainage) and subsequent settlement (consolidation), the theory developed by Barron (1948) and Kjellman (1948) is used (Hansbo, 1979). The theory is summarized in the Barron and Kjellman formula as follows (see Figure 16.2):

$$t = \frac{D^2}{8c_h}\left[In\frac{D}{d} - 0.75\right]In\frac{1}{1-\bar{U}_h} \tag{16.1}$$

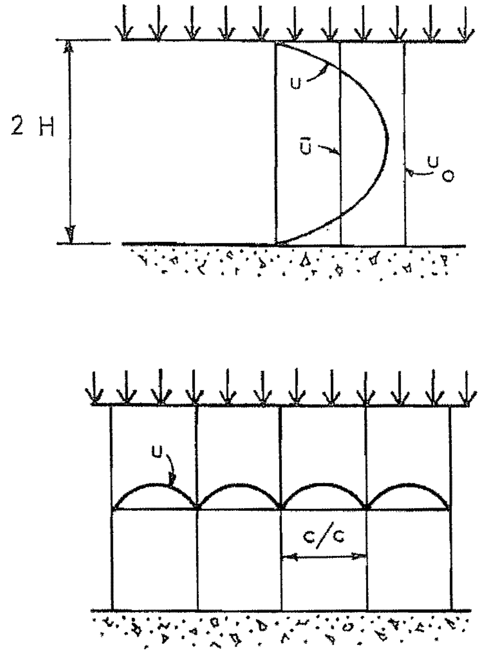
where

- t = time from start of consolidation
- D = zone of influence of a drain
- d = equivalent diameter of a drain
- $\bar{U}_h$ = average degree of horizontal consolidation
- c_h = coefficient of horizontal consolidation

The zone of influence of a drain is the diameter of a circle having the same area as the area influenced by the drain, i.e., if in a given large area of size A there are n drains placed at some spacing and in some grid pattern, each drain influences the area A/n. Thus, as shown in Figure 16.2, for drains with a centre- to-centre spacing, c/c, in a square and triangular pattern, the zone of influence D, is 1.13 c/c and 1.05 c/c, respectively.

In the case of sand drains, the equivalent diameter, d, is often taken as equal to the nominal diameter of the sand drain. In the case of bandshaped drains, there is no agreement on what to use as the equivalent diameter of the drain. One approach used is simply to equalize the outside surface area of the bandshaped drain with a circular sand drain of the same surface. However, this approach does not recognize the difference between the usually open surface of the premanufactured drain and the rather closed surface of the sand drain, nor the differences between various makes of bandshaped drains.

Strictly speaking, the equivalent diameter of a bandshaped drain should be termed 'the equivalent circle diameter' to separate it from 'the equivalent sand drain diameter'. The equivalent circle diameter is the diameter of a circle having the same free or unobstructed surface to the surrounding soil as the drain. It has been suggested that the equivalent circle diameter of a sand drain is the sand drain nominal diameter multiplied by the porosity of the sand in the drain. The porosity of loose, free-draining sand is normally about 0.4 to 0.5. Thus, the equivalent circle diameter of a sand drain is about half of the nominal diameter. However, the consolidation time is not very sensitive to variations of the equivalent diameter. The spacing is important, however, as is also the total length of drains used at a site.



WITHOUT VERTICAL DRAINS

$$t = T_v \cdot \frac{1}{c_v} \cdot H^2$$

$$T_v = -0.1 - \lg\,(1 - \bar{U})$$

$$\bar{U} = 1 - \frac{\bar{u}}{u_o}$$

WITH VERTICAL DRAINS

$$t = T_h \cdot \frac{1}{c_h} \cdot D^2$$

$$T_h = \frac{1}{8}\left(\ln\frac{D}{d} - \frac{3}{4}\right)\ln\frac{1}{1 - \bar{U}}$$

□ SQUARE GRID ⟶ $D = \sqrt{\frac{4}{\pi}} \cdot c/c = 1.13\ c/c$

△ TRIANGULAR GRID ⟶ $D = \sqrt{\frac{2\sqrt{3}}{\pi}} \cdot c/c = 1.05\ c/c$

FIGURE 16.2 *Principle of vertical drains*

For bandshaped drains of, commonly, 100-mm width, values proposed as the equivalent circle diameter have ranged between 30 mm and 80 mm, and full-scale studies have indicated that the performance of such drains have equalled the performance of sand drains of 200 mm to 300 mm in nominal diameter.

The degree of consolidation at a certain time, $\bar{U}$, is defined as the ratio between the average increase of effective stress, $\Delta\sigma'$, in the soil over the applied surcharge causing the consolidation process, i.e., $\Delta\sigma'/q$. In practice, it is determined from measurements of either pore pressure increase or settlement and, alternatively, defined as 1 minus the ratio between the average pore pressure increase in the soil over the total pore-pressure increase resulting from the applied surcharge, i.e., $1 - \bar{u} / \bar{u}_o$, or, the amount of settlement obtained over the final amount of settlement at completed consolidation, $\Delta S/S$. The consolidation ratio is generally based on pore pressure increase, because pore pressures can be determined at the start of a project, whereas the value of the final settlement is not obtained until after the project is completed. However, as pore pressures and pore-pressure dissipation vary with depth and, in particular, with the distance to the drains, pore-pressure observations can be unreliable measures of the degree of consolidation.

The horizontal coefficient of consolidation, c_h, is critical for the design of a vertical drain project, because the dissipation time calculated according to the Barron and Kjellman formula (given above), is inversely proportional to the c_h value. The c_h value is not usually determined in a soils investigation program, but the c_v value is. In a homogeneous soil layer, the horizontal coefficient of consolidation, c_h, is generally about two to six times greater than the vertical coefficient of consolidation, c_v. The extent of the mobilization of a coefficient higher than the c_v value depends on the disturbance to the soil caused by the installation of the drains. For sand drains, in particular displacement-type sand drains, a c_h value that is greater than the c_v value can rarely be mobilized.

The coefficient of consolidation varies widely in natural soils. In normally consolidated clays, the c_v value usually ranges from 1×10^{-8} to 30×10^{-8} m²/s. In silty clays and clayey silts, the c_v value can range from 5×10^{-8} to 50×10^{-8} m²/s.

The coefficient of consolidation is normally determined from laboratory testing of undisturbed soil samples or, preferably, in-situ by determining the pore-pressure dissipation time in pore-pressure sounding (piezocone - CPT). The actual value to use requires considerable judgement in its selection, and it cannot be determined more closely than within a variation range of three to five times. This means that an engineering design of a project requires supporting data for selection of the c_h value to avoid having to employ a very conservative approach.

16.3.3 Practical Aspects to Consider in Design

16.3.3.1 Sand Drains

The sand used in a sand drain must be free draining, which means that the portion of fine-grained soil in the sand must not exceed 5% by weight and preferably be less than 3%.

As indicated by Casagrande and Poulos (1969), the installation of full-displacement sand drains (driven drains) in soils that are sensitive to disturbance is not advisable. Jetted sand drains will eliminate much of the undesirable effect associated with driven sand drains, but at the cost of creating a muddy site, and, potentially the destruction of the drainage blanket on the ground.

Furthermore, before pouring the sand into the water-filled, jetted hole, the water must be flushed clear so that fines suspended in the water do not mix with the sand, rendering it non-free-draining. It is more difficult to control the risk of fine-grained soil sloughing off, or being flushed off the side of the hole and mixing with the stream of sand during the pouring procedure changing the sand into the non free-draining kind.

Sand drains are apt to neck and become disrupted during the installation work, or as a consequence of lateral movements in the soil.

Despite the stated disadvantages, sand drains can be useful where large amounts of water are expected, in soils less sensitive to disturbance by the installation, and where the ratio of length to nominal diameter is not greater than 50, and the ratio of spacing to the nominal diameter is larger than 10.

16.3.3.2 The Premanufactured Bandshaped Drain

The bandshaped drain consists in principle of a channelled (grooved or studded) core wrapped with a filter. The filter serves the purpose of letting water freely through while preventing fine soil particles from entering the channels. The channels lead the water up to the ground surface, or to above the groundwater table, or down to a draining layer below.

The filter must be able to receive water not only from clay soil, but also from coarser soils, such as silty, fine sand typically found in lenses, or layers in most fine-grained soils. Furthermore, while the drain is receiving water over its full length, it must be capable of discharging this water through a very short distance of its length, as discharge through the end of the drain is a rather special case. Consequently, the filter must have a permeability coefficient no smaller than that of fine sand, i.e., approximately 1×10^{-6} m/s.

The premanufactured drain is often manhandled on the construction site: it is dragged on a truck floor and on the ground, it is left in the sun and in the rain, it gets soaked and is then allowed to freeze, it is stepped on, etc. This puts great demands on strength, in particular wet strength, on the filter and the glue used to hold the longitudinal filter seam together. A rip or tear in one spot of the filter of an installed drain can reduce the drain performance considerably.

The drain core must provide a free volume (free cross-sectional area) large enough for the water flow not to be impeded, i.e., the well resistance must be small. The water flow in a chain used for accelerating settlement is very small and the required free volume is small. Typically, the water flow is smaller than about 5 litres per day or 3 cm^3/min, which is about what a dripping tap produces.

The drain core must be flexible enough to deform both by folding (due to lateral soil movement) and by axial compression when the soil settles around the drain. The settlement, or strain in compression, can be greater than 20 %. While a drain cannot be sufficiently soft that it compresses this amount axially, it must be able to 'microfold' because of the imposed compression strain, without breaking or blocking the passage of water, i.e., creating an excessive well resistance.

At the same time, it must be strong enough to resist large, lateral, soil pressures without collapsing and effectively closing the longitudinal drainage path in its channels. For instance, at a depth of 20 m in a clay soil underneath an embankment 10 m high, the effective lateral soil stress is 200 kPa to 300 kPa, and it is desirable that the drain be able to resist this pressure without developing excessive well resistance.

16.3.3.3 General Aspects

If the settling soil contains thin layers, bands, or lenses of permeable soil, this will have little effect on the vertical drainage - the case of no drains. On the other hand, when vertical drains are used, the permeable layers will drain the consolidating soil and lead the water toward the drains. Such bands or lenses (even if very thin) can be quite effective in channelling water. Normally, therefore, as stratified or banded clays occur in many places, the assumption of the Barron and Kjellman formula of homogeneous soil is commonly not valid. Furthermore, the consolidation time will not be governed by the spacing of the drains but by the distance between the permeable layers (on condition, of course, that the horizontal layering has not been broken down by the drain-installation procedure used and that the filter permeability is not too small).

Figure 16.3 illustrates the acceleration of settlement by means of vertical drains underneath an embankment on compressible soil. The upper sketch indicates the back pressure in the drain created by use of a filter with too low coefficient of permeability forcing water to rise in the drain inside the filter to a height above the water table, where balance is achieved between the inflow and the discharge of water.

The lower sketch in Figure 16.3 illustrates the similar condition created by the ponding of rain water and melting snow in the depression created by the initial amount of settlement. The ponding is due to insufficient horizontal drainage on the ground surface. In a design of a vertical drain project, the expected amount of settlement must be calculated and a drainage scheme designed that ensures a horizontal gradient from the treated area at all times.

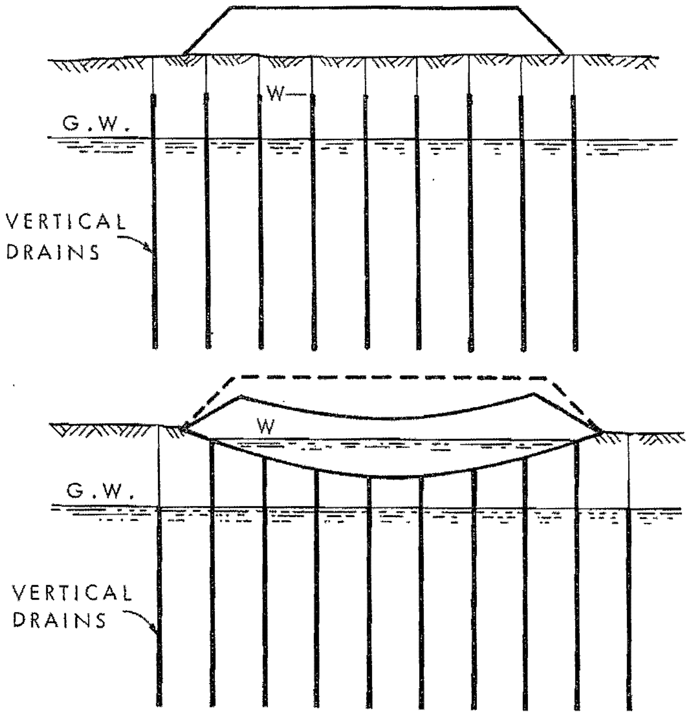
The build-up of back pressure will have a temporary effect on the time development of the settlement. Anyone unfamiliar with this phenomenon will observe a flattening out of the time-settlement curve and draw the false conclusion that all of the primary settlement has been obtained. However, eventually the back pressure will disappear, and the settlement, delayed due to the back pressure, will recur.

The acceleration of settlement by means of vertical drains is only efficient where the applied surcharge creates a final effective stress in the soil that exceeds the preconsolidation pressure in the soil. This requirement often governs the installation depth of drains.

Often, it is unnecessary to install the drains beneath an embankment of width B beyond a depth that is greater than B/2 to B.

The minimum width of the installation of the vertical drains should extend to the foot of the embankment. To reduce the magnitude of differential settlement causing bowing of the surface, it is recommended that drains be installed to a distance outside the embankment equal to about half the height of the embankment.

The theoretical analysis is sensitive to the parameters used as input, in particular to the coefficient of consolidation. Unless prior experience is available from or a nearby site where a similar installation took place, any design using a theoretical analysis, whether the Barnon and Kjellman formula or a more involved one, has only qualitative value. When data are available from similar sites, the design analysis can be assisted by a thorough site investigation, which is aimed at establishing the presence of bands or lenses of permeable soil at the site. This requires pore-pressure sounding (Chapter 4) and continuous sampling of undisturbed soil with subsequent laboratory identification and testing.



G.W. = groundwater level; W = water level in drains.

FIGURE 16.3 *Typical cases of back pressure in vertical drains*

In most cases, spacing of the drains will have to be estimated and the project monitored by means of observations of the development of settlement and pore pressures over time. The settlement should be monitored not just at the ground surface, but also as to its distribution with depth. Piezometers need to be carefully installed in relation to the drains. Naturally, the data will be of limited value unless coupled with a thorough site investigation.

Often a predesign testing program is carried out to determine the parameters to use in the design and the spacing and type of drain to use. Typically, more than one spacing are used. Equally important is to arrange for a reference in the form of an area with no drains, so that the positive effect of the drains can be correctly established. The literature contains many comparisons between a theoretical calculation of what the time development of the settlement would have been without use of vertical drains and observed development underneath a drained area. Whereas such a comparison has the 'advantage' of always 'proving' the desired positive effect of the drain installation, it is of limited engineering value.

When monitoring the effect of a drain installation, it is important that the observation period be long enough, preferably up to the end of the primary consolidation. Experience has shown that large potential errors can be associated with a value of achieved degree of consolidation determined before about 75 % of the settlement has been obtained.

16.3.3.4 Other Uses of Drains

The use of premanufactured band-shaped drains is not limited to strengthening of clay soils and the acceleration of settlement underneath embankments, fill areas such as airports, and reclaimed land. Other applications have been to release pore pressures in tailings dams and in slopes, and to relieve pore pressures behind retaining walls. The drains have also been used in combination with load application by the vacuum method described in Section 16.11

(see also Holtz and Wager, 1975).

16.4 Dynamic Consolidation

16.4.1 Introduction

Dynamic consolidation, also known as dynamic compaction or heavy tamping, is a method of ground improvement that was developed in the early 1970s by Louis Menard.

In essence, the technique consists of the application of high intensity impacts over the surface of the ground to be treated by means of a free-falling, heavy steel or concrete weight. The strain waves generated by these impacts travel to considerable depths and rearrange the soils into a denser, more compact state.

Dynamic compaction is used to increase bearing capacity and decrease total and differential settlement within a specified depth of improvement to allow the use of shallow footings for different types of civil-engineering projects, including runways, coal facilities, dockyards, etc. It has been used to reduce the liquefaction potential of loose soils in seismically active regions. Some unique applications include compaction under water, displacing unsuitable materials such as peat, and collapsing sinkholes and abandoned mine workings.

The main advantages offered by the process are its low cost, rapidity of execution, applicability to a large variety of constructed fills and loose natural soils, and usefulness for improving sites underlain by peat and landfills.

16.4.2 Methodology

Dynamic compaction involves the use of heavy steel or concrete block tamper weighing typically 100 kN to 200 kN and dropped in free-fall from heights of up to 30 m using heavy crawler cranes. Under such conditions, compressible soils have been compacted to depths of as much as 15 m. With special equipment, it is possible to drop heavier weights and improve soils at greater depths.

The distribution of the impacts and the sequence of the application are critical in achieving successful compaction, particularly if deeper zones are to be treated. The impacts are normally applied in increments, each complete coverage of the working surface being referred to as a phase. The early phases, also called the high-energy phases, are designed to improve the deeper layers with impact points at a spacing dictated by the depth of the compressible layer. Generally, the phase is followed by a low-energy phase with contiguous impacts (hence, the name 'ironing phase'), which is mainly designed to densify the surficial layers.

Although the process is effective on saturated coarser grained soils and can be used even on sites where the water table is near the surface, it is nevertheless complicated and possibly ineffective in fine-grained soils by the creation of increased porewater pressures during compaction. This phenomenon will reduce the effectiveness of the subsequent phases, unless it is recognized and measures taken to promote and accelerate the dissipation of pore pressures. If not, remoulded soil conditions can develop.

During its execution, the process must be continuously monitored, first, to evaluate the degree of soil improvement being achieved, and, second, for environmental considerations such as potential damage to nearby structures and annoyance to people. Earthworks to level the site after each phase and to replace uncompactable materials with suitable soils are also part of the operation. Final verification testing to ensure that the specification requirements have been fulfilled must be performed upon completion of the treatment.

16.4.3 Ground Response

16.4.3.1 Ground Deformation

The impact of falling weight upon the earth compacts the natural soils and collapses voids in fill soils, causing deformation in both vertical and horizontal directions. The induced settlement is significant inasmuch as it provides an indication of the efficiency of the process. The magnitude of settlement depends on the initial compactness of the soils, the applied energy per unit area, and the adequacy of the compaction plan. Generally, the induced subsidence amounts to between 5 % and 7 % of the thickness of the loose soils being treated. Several sites are reported to have subsided as much as 2 m as a result of the treatment.

The horizontal deformations, although accompanied by some degree of compaction, are important mainly because of the potential displacement of adjacent structures. In the case of fine-grained soils, noticeable swelling of the ground surface generally occurs, as much as 0.3 m in certain instances, but diminishing quickly to become undetectable at a distance of 4 m to 5 m from the treated area.

16.4.3.2 Ground Vibration

The impact of the tamper on the ground generates compression, shear, and Rayleigh waves. Rayleigh waves, which travel at surface, generate vibrations that may affect nearby structures as well as people living and working in them. These vibrations normally have a frequency of about 5 Hz to 8 Hz, and the shock accompanying each blow of the falling weight is felt for about one second.

Peak particle velocities are generally used to define damage criteria for building structures and annoyance levels to persons. The peak particle velocities increase significantly with the densification of the soil mass.

16.4.3.3 Pore Pressures

Where the water table is within the depth of influence of the process, the densification is accompanied by an increase of porewater pressures. In the case of sands and gravels, these pressures dissipate quickly. In less pervious soils, the induced pore pressures may take days or even weeks to dissipate fully.

16.4.3.4 Soil Improvement

The engineering properties of soils densified by the process are improved to a depth and degree that depend largely on the proper assessment of the several variables and parameters characterizing each project. The variable parameters include the weight of the tamper, its height of fall and impact surface area, the grid spacing, the number of phases, the total compactive energy, and the time delay between phases. The non-variable, or given, parameters include the existing soil types, the initial soil conditions, groundwater levels, and the environment.

A convenient approximation of the maximum depth of influence is (d_{max}) in metres given by the following empirical formula:

$$d_{\max} = \alpha \sqrt{WH} \qquad (16.2)$$

where

W = weight of tamper (N)
H = height of fall (m)
α = a factor usually taken as 5×10^{-3} to 7×10^{-3} (dimension $\left(\sqrt{m/N}\right)$)

Improvement achieved by means of dynamic consolidation has been observed to increase with depth to a maximum at a specific depth and then diminish with depth until reaching a depth, d_{max}, below which the soil properties remain unchanged. The specific depth is approximately between one third and one-half of the maximum depth.

16.4.3.5 Control Testing

Quality-control measures must be undertaken to ensure that improvement does indeed occur and that the engineering characteristics of the soil have been attained as specified.

Control testing may be divided into three types: production, environmental and specification. Production control includes quality-assurance aspects, such as logging the impacts, elevation survey of the working surface, and monitoring the changing soil characteristics during treatment using in situ geotechnical testing methods. Environmental control consists of measuring ground vibration levels and carrying out boundary surveys to minimize the effects of the tamping operations on adjacent properties. When compacting in close proximity to existing structures, it may include instrumentation designed to detect potential movement and deformation. Specifications or verification controls are carried out after the treatment is completed to certify that the objectives of the treatment have been attained.

The most frequently used in-situ geotechnical methods for production or specification control have been the pressuremeter, the standard penetration test (SPT), the static cone penetrometer (CPT), and the dynamic cone penetrometer. Geophysical surveys have proven useful in soils that are difficult to test with conventional methods, such as rockfills. Other types of field measurements include observation of pore pressure, measurement of peak particle velocities, and subsurface settlement. Field vanes, dilatometer and plate test loading are also used.

16.4.3.6 Limitations of the Process

The safe use of dynamic consolidation presupposes the knowledge and understanding of its limitations. The main limitations are depth, soil type and soil conditions, the environment, the engineering requirements, and the climate. Reviewing these various factors, the following guidelines are suggested:

16.4.3.6(1) Depth

Using conventional lifting equipment, it is possible to treat free-draining granular soils to depths of 15 m and fine silty sands and silts to depths of 10 m to 12 m. Greater depths of improvement have been achieved with special equipment, but the efficiency of the process beyond depths greater than 15 m remains unproven, except for coarse-grained material, and any such application should be approached with caution.

16.4.3.6(2) Soil Types and Conditions

The efficiency of the process for improving clays and clayey alluvials remains unproven. Such applications should be considered only for projects where the potential economy is sufficiently important to justify a full-scale field test.

Because of their loose state and the presence of numerous voids, most types of constructed fills, including clay fills, can be successfully improved by the process. However, secondary settlement due to loss of volume accompanying the decomposition of organic matter remains a phenomenon that is difficult to assess.

The application of the treatment may be complicated if the water table is closer than 2 m below ground surface. Remedial measures will generally consist of raising the grade with imported materials but may also consist of pumping to lower the groundwater level. High pore pressures generated in fine-grained soils can adversely influence the results of the process.

16.4.3.6(3) Environment

The application of the process is accompanied by noise, vibrations, gusts of air, and permanent soil deformations. It may therefore be disadvantageous when used in an urban area.

Noise generated by the impact is generally muffled and not objectionable. By contrast, the roar of the lifting crane engine can be quite loud and may have to be abated by suitable equipment. Typically, the noise level (impact + engine noise) will reach 130 dB at a distance of 12 m, decreasing below 100 dB at 100 m.

Air gusts will displace materials around the edges of impact points, sometimes projecting chunks of earth and mud to considerable distances creating a risk of damage to property and injury to persons. Suitable precautions are required.

Vibrations generated by the process are not normally damaging, unless peak particle velocities exceed 50 mm/s, but will at a much lower value cause annoyance to persons nearby. It should be stressed, however, that the reaction of people to vibrations is generally unpredictable.

When working in close proximity to existing structures, the intensity of vibrations can be reduced by the use of a lighter tamper, or a lower height of fall, or a combination of both. The observance of the following formula should ensure a safe operation.

$$D > \frac{\sqrt{WH}}{80} \tag{16.3}$$

where

D = distance from impact (m)
W = weight of tamper (N)
H = height of fall (m)

Notwithstanding the above, an experienced specialty contractor may work safely as close as 3 m from underground services and 6 m from sound structures.

Soil deformation that is permanent can occur as much as 6 m away from the limits of compaction. At the ground surface swelling occurs, which could raise and crack pavements and sidewalks; at depth, lateral displacement occurs, which could affect underground structures. Permanent horizontal displacements of 20 mm at a distance of 4 m, and of 6 mm at a distance of 6 m, have been recorded.

16.4.3.7 Engineering Requirements

Although many types of soils will be improved by the process, the attainable engineering characteristics vary considerably. As a general guideline, the limits in Table 16.1 are proposed, where the presumed design bearing pressures are given. The design bearing pressures correspond to serviceability limit states not exceeding 25mm total settlement.

TABLE 16.1 *Presumed Design Bearing Pressures for Soils Treated by Dynamic Consolidation*

Type of Soil	Design bearing pressure (kpa)
Fine-grained alluvials, silty fills	100 - 150
Heterogeneous fills	100 - 200
Fine silty sand, hydraulic fills	200
Coarse sand, gravel	300
Well-graded gravel, rockfill	400 – 500

16.4.3.8 Climate

Adverse weather conditions such as heavy rainfalls, snow, and frost do not preclude the use of the process; they may however, have a considerable influence on its costs. Dynamic compaction has been carried out in Canada in temperatures as low as -15°C to -25°C.

16.5 In-Depth Vibro Compaction Processes

16.5.1 Introduction

Improving soils by using depth vibrators started in the early 1930s, when the concept was developed that deep deposits of soil could be compacted by means of a machine that would carry the source of vibration to the desired depth (Steuermann, 1939). Since then, depth vibrators have been used extensively throughout the world for the densification of granular deposits. The process uses elongated vibrators, and, when combined with water jetting, it is generally known as Vibro compaction.

In the early 1960s, the use of more technologically advanced depth vibrators led to their use for the improvement of fine- grained soils and fill materials by replacing the fines in the soil, which are washed out during the jetting, with coarse materials, which then are compacted by the process. This application of the process is called Vibro replacement or, somewhat incorrectly, the 'stone column' method, although it is essentially no different from the Vibro compaction method.

The Vibro process provides an economically attractive and technically feasible basis for the treatment of soils that exhibit (in their natural state) insufficient strength to support anticipated building loads.

16.5.2 Equipment

The essential element of the depth vibrator is a heavy tubular steel body, approximately 300 mm to 400 mm in diameter and 4.5 m to 5.0 m long, within which are housed eccentric discs that rotate on an axial shaft. In order to reach depths greater than 5 m, the vibrating unit (called the vibrator) is connected to simple extension tubes of approximately the same diameter. The complete assembly is suspended from a conventional crane.

Two types of vibrator are in use: an electrically driven vibrator with a frequency of 60 Hz, and a hydraulically driven vibrator with variable frequency. The power demand of the electrically powered vibrators is generally about 35 kJ to 50 kJ, although vibrators of up to 100 kJ effect are also available.

16.5.3 Vibro Processes

16.5.3.1 Vibro Compaction of Loose Cohesionless Soils

The vibrator is allowed to penetrate the soil under its own weight (approximately 30 kN to 60 kN, depending upon the total length of the unit) with the help of water or airjetting from the nose cone, and the induced vibrations. After penetration to the required depth, the water flow is reduced and the vibrator is withdrawn in small incremental lifts ensuring uniform compaction of the soil from depth to grade. Vibro compaction will cause a reduction in volume of the soil up to 10 %, often leading to substantial reductions in the level of the site surface. If the elevation of the site is to be maintained, granular material (either imported or from other areas of the site) can be added around the vibrator. As illustrated in Figure 16.4, the added material gravitates down around the vibrator to the base of the hole, where it is compacted and integrated into the natural subsoil by the action of the vibrator.

Since the vibrations produced at depth emanate from a point close to the bottom end of the vibrator, and since these vibrations radiate in the horizontal plane, there is little difficulty in achieving uniform densification with increase in depth. The radial densification of granular soils (even though the vibrations are produced in the horizontal plane) is limited, however. In well-graded sands, centre-to-centre spacings approaching 3 m to 3.5 m may be sufficient to achieve a density index in the order of 70 %. Closer spacings can produce density indexes of approximately 90 % (D'Appolonia, 1953).

The centre-to-centre spacings used for individual sites depend not only on the degree of compaction required, but also on the material to be densified. While correlations between spacings and compactness condition achieved have been undertaken, these can only be related to sites having identical soil conditions and where the same type of vibrator is used. Normally, a spacing of about 1.5 m is required for fine sand. Most of the compaction takes place within the first five minutes at any given treatment depth.

16.5.3.2 Vibro Replacement in Soft Cohesive Soils and Inorganic Fills

The equipment used in this process is identical to that for Vibro compaction. The vibrator sinks under its own weight, assisted by vibration (and water or air as a flushing medium) into the ground until it reaches the predetermined depth. Water is generally used as a flushing medium in fully saturated soils, and compressed air is used in partially saturated soils. For work in saturated fine sand or silt, it is essential that the water level in the hole is kept at a level that is at least equal to and preferably higher than the groundwater table throughout the advancement of the hole and installation of the gravel.

During the penetration of the vibrator, the water flowing up along the side of the vibrator washes out the fines in the soil, leaving the coarser material in the hole. The lost material is replaced incrementally from depth to grade with charges of coarse-size fill, usually well-graded gravel of size between 10 mm and 80 mm. The vibrator is repeatedly withdrawn and reinserted to ensure a uniform result. With each charge of gravel, the vibrator displaces the backfill horizontally into the native soils, while at the same time, compacting underneath its bottom edge. Repetition of this procedure forms an irregular, cylindrical gravel column between the bottom of penetration and working grade (see Figure 16.4).

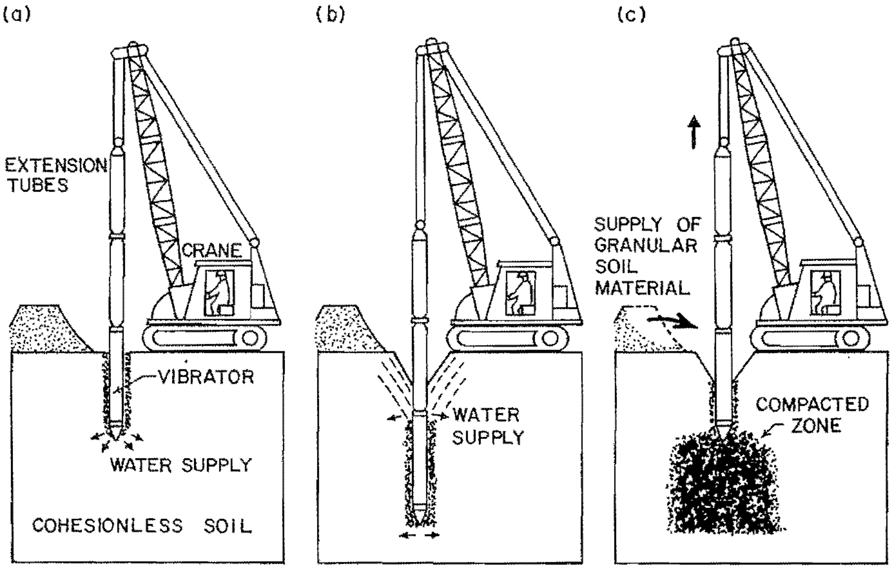


FIGURE 16.4 *The principle of the Vibro process*

The diameter of the compacted column ranges normally between 0.6 m and 1 m and depends mainly on the strength of the native soil, the sort of flushing medium used to create the hole, and on the time spent to compact and displace the backfill. Columns are generally installed in a square or triangular grid pattern. The spacing between the column centres ranges from about 1 m to 3 m and is mainly determined by the degree of improvement required to achieve the following four basic objectives:

1. to limit total settlements;
2. to reduce differential settlements;
3. to achieve higher bearing capacity; and
4. to increase shear strength.

Various theories have been proposed for the design and failure criteria of compacted columns (Hughes et al., 1975). One of these theories considers the column as an axially loaded member of frictional material supported by the passive resistance of the surrounding native soil. Accordingly, the ratio of applied stress on the column to passive restraint is a maximum at the depth of maximum effect. The resulting settlement depends upon the induced radial strain in the soil when passive resistance develops. Hughes et al., (1975) indicated that the vertical displacement of the stone column, within the range of service stresses, is smaller than half the maximum radial strain in the column. Settlement of a treated area may be estimated by determining a stiffness - 'elastic' modulus or modulus number (stress exponent usually equal to 0.5) - of the untreated soil and the columns in combination, and performing a settlement calculation as presented in Chapter 11.

16.6 Lime Treatment

16.6.1 The Action of Lime in Soil

Soil improvement by means of mixing lime into fine-grained soil is probably the oldest of all site-improvement methods. It was used, for instance, in Roman roads 2,000 years ago. When unslaked lime is mixed into moist soil, the following four reactions take place:

- hydration;
- ion exchange;
- cementation (pozzolanic reaction); and
- carbonation.

Hydration reduces the water content and raises the temperature of the soil. In the process, the shear strength of the soil increases. The hydration starts immediately and is finished within a short time.

Ion exchange also starts immediately and finishes early. As a result of this process, water-stable aggregates are formed, which have low compressibility and high permeability compared to the original soil.

The pozzolanic reaction is comparatively slow and continues for a long time. The resulting cementation of the soil particles results in a considerable shear-strength increase and reduction of compressibility. Carbonation is a reaction between the lime and air and results in a reduced strength. When the mixing of lime takes place below the groundwater table, its influence is minimized.

The amount of lime necessary to achieve a maximum improvement of strength and compressibility is about 3 % to 6 % of dry lime per dry weight of soil. The lime has to be mixed thoroughly with the soil and quickly, or the reaction will be incomplete.

16.6.2 Surface Lime Treatment

Surface lime treatment consists of spreading lime on a soil to be stabilized and mixing it with a rotary tiller. The optimum water content and the liquid limit of the soil will increase and the lime-treated fine-grained soil can be compacted using a sheepfoot roller or similar equipment. The method is used for wet and soft sites where the soil is very silty and difficult for construction equipment to travel on. The lime treatment and compaction creates about a 0.2-m-thick layer of soil, which, in addition to being strengthened, has become more pervious. It should be noted, however, that unslaked lime is dangerous to inhale, and powdered lime spread on the ground may constitute a health hazard.

16.6.3 Deep Lime Treatment

Lime can be mixed into the soil by means of special equipment, which will produce a column of treated soil (Broms and Boman, 1977, 1979; Holm et al., 1983). The lime column will be capable of supporting point loads much greater than those that the untreated soil can support. When lime columns are placed in a grid pattern over an area, they will have the combined beneficial effect of both increasing shear strength (bearing capacity) and reducing settlement, particularly the differential settlement. Furthermore, because of the increased permeability of the lime-treated soil, the lime columns may act as vertical drains and accelerate the settlement.

Lime columns have been used to support embankments and spread footings; they have been used in trenches both to retain the trench walls and to support sewage pipes placed in the trench; they have been used in combination with pile foundations for buildings, where the piles support the structure and the lime columns the ground floor, as well as the immediate area outside the building; and they have been used to stabilize areas damaged by landslides.

16.7 Ground Freezing

16.7.1 The Freezing Process

Controlled ground freezing for mining and construction applications has been in use for more than a century. This method may be used in most soil or rock formations, but it is better suited to soft ground than to rock conditions and is not suitable in coarse gravel, boulder soils, or expansive soils. Freezing may be used for any size, shape, or depth of excavation, and the same physical plant can be used from job to job, despite wide variation in these factors.

Freezing is normally used to provide structural underpinning, or temporary support for an excavation or to prevent groundwater flow into an excavated area. As the low permeability frozen earth barrier is constructed prior to excavation, it generally eliminates the need for compressed air, dewatering, or the concern for adjacent ground subsidence during dewatering or excavation. However, lateral groundwater flows may result in failure of the freezing program, if not properly taken into consideration during the planning process. Furthermore, though subsidence may not be of concern, ground movements resulting from frost expansion of the soil during freezing may occur under certain conditions, and this potential hazard must be considered in the planning.

Freezing can be completed rapidly if necessary, or desirable, although the freezing rate is directly related to overall costs, and rapid freezing is relatively more costly than slower freezing.

Frozen ground behaves as a visco-plastic material (exhibits creep), whose strength properties depend primarily on the ice content, the duration of applied load, and the temperature of the ground.

The refrigeration plant and refrigerant or coolant distribution system may represent as much as 45 % to 60 % of the direct costs of a freezing project. Furthermore, the direct costs, as well as the time required to complete adequate freezing, depend to some extent on the type of freezing approach used.

The thermal energy required to freeze ground is directly proportional to the water content of the soil. For coarse-grained soils, the energy requirements are relatively low, provided no lateral groundwater flow occurs. In fine-grained silt and clay soils, the energy requirement will normally be higher. As a rule of thumb, the energy requirements in megajoules per cubic meter (MJ/m^3) of soil frozen will be between 9 and 12 times the percentage of the water content.

16.7.2 Exploration and Evaluation of Formations to be Frozen

One of the most important factors in devising a freezing system is to thoroughly explore the subsurface formation to be frozen. If the nature of the subsurface structure is not well known, an adequate and efficient freezing system, no matter how well it is designed, may not accomplish its purpose.

In order to determine what freezing facilities should be provided to stabilize the subsurface structure, the characteristics of the materials to be frozen should be ascertained as accurately as possible. This requirement cannot be stressed too strongly.

The exploration is generally accomplished by the drilling of boreholes, taking samples of material from all zones, and observing water conditions below ground. A sufficient number of exploratory holes should be drilled so that the entire mass to be frozen is covered, and a complete record of the exploratory operations should be kept.

The exploration should supply the following information:

- surface conditions of the freezing site;
- location and depth of boreholes (maintain borehole logs for inspection and analysis);
- location, depth, and extent at which competent and incompetent strata occur;
- location and extent of aquifers;
- nature of materials: rock-shale-clay-anhydrites etc. at all depths;
- water content in all strata;
- static water levels of each acquifer;
- ground temperatures in different strata;
- analysis of water or brine in each aquifer;
- evidence and amount of occluded air or gas in water or air in aquifers;
- evidence of vertical water flows between aquifers having different static levels; and
- evidence of horizontal water flows in aquifers through mass to be frozen.

16.7.3 References

For additional information on ground freezing reference may be made to Gail (1973), Ladanyi and Johnston (1973), Ladanyi (1982), Sanger and Sayles (1978), and Shuster (1972).

16.8 Blast Densification

In favourable circumstances, deep compaction by blasting can be an effective and economic means of achieving densification of loose saturated sandy soils. Blast densification has been used to reduce seepage quantities, increase bearing capacity, and reduce susceptibility to both static- and seismic-induced liquefaction.

The general procedure consists of:

- advancing a cased hole by jetting, vibration or other means; a borehole 150 mm in diameter is usually sufficient; uncased holes with heavy drilling mud for hole support, and subsequently stemming, have also been used;
- installing explosives at appropriate depths as drilling casing is withdrawn, or left in disposable plastic casing;
- backfilling the hole, or stemming above the charges; and
- detonation of charges in a pattern, and with appropriate delays between charges.

Blast densification can often offer considerable economic advantages, as the major piece of site equipment is a suitable drilling rig. Such equipment is often more readily available at remote sites for less mobilization costs than the equipment required for other densification techniques.
A real impediment to the applications of blast densification is the lack of contractors who are prepared to bid to a predetermined specification for site improvement. Guidelines for blast densification are largely empirical and trials are usually required to determine the optimum configuration of charge size, depth and detonation sequence. However, real cost savings are available to owners willing to underwrite an observational approach.

Blast densification, as in other techniques such as vibroflotation, works by localized liquefaction of loose saturated sand. When the sand is liquefied, excess pore pressures are created and the sand settles to a more compact condition. Criteria are available to estimate the charges required to achieve full liquefaction (that is, a zero effective stress condition). Blasting is particularly effective if a loose sand layer is overlain by dense sand which provides a containment effect. Charges are set within the loose layer and a stand-off distance established so as not to weaken the surface dense layer. Charges are often set at the one-third to two-thirds points within the loose zone. Charge densities of 10 g/m^3 to 30 g/m^3 of soil to be densified are commonly used.

An interesting feature of blast densifications, and one which is also observed in vibroflotation, is that while surface settlements occur almost immediately after blasting, results of Cone Penetration or Standard Penetration Tests often require several weeks to reflect ground improvement in terms of the in-situ test measurements.

Safety, particularly in urban environments, is often a perceived, rather than a real concern. The charge per delay is often relatively small, allowing off-site ground vibrations to be held to acceptable limits using the same type of criterion common to pile driving or dynamic compaction.

16.9 Compaction Grouting

Compaction grouting is the injection of very stiff, low slump (0.25 cm to 3.0 cm slump) mortar type grout under relatively high pressures to displace and compact soils in place. Recently advances in grout techniques have allowed the use of injection using silty soils without the addition of cement. This requires that the silty soils be well graded and that pumping trials be carried out. The technique is used for strengthening loose disturbed or soft soils under existing structures, for reduction of settlement during soft ground tunnelling, compaction of soils for earthquake liquefaction resistance, and for sinkhole filling and densification.

Grout pipes are usually installed in a predetermined pattern (usually vertically but frequently angled) to the required depth. Grout is pumped until one of the following criteria is reached

- refusal at a predetermined pressure,
- a maximum grout volume (or 'take') is reached, and
- ground heave is observed.

The resultant injection consists of a homogenous grout bulb or a series of linked grout bulbs, which are formed around the end of the grout pipes. The injection of the grout displaces the in-situ soil and compacts the adjacent ground.

The process is most effective in cohesionless soils but can also be effective in finer grained soils where disturbance has occurred.

QA/QC control during construction is achieved by:

- recording pressure and grout volumes
- monitoring of ground heave
- slump tests where cement is used
- control mix when non-cement mixes are used

16.10 Chemical Grouting

Chemical grouting is the permeation of sands and gravels with fluid grouts to produce sandstone like masses or to "fill" the voids and thereby reduce water flow. Grout types consist of sodium silicate, acrylates, polyurethane and microfine cement. More toxic chemicals have environmental restrictions, which would preclude their use.

Sleeve port grout pipes are installed in a predetermined pattern (vertical, horizontal or horizontal) in a grouted borehole. Grout is injected through the ports at specific designed intervals and rates to fully treat the area. A variety of the process is termed "tube a manchette" in Europe.
The process is suitable for cohesionless soils particularly clean sands and gravels although some effect can be achieved in silty sands. The approach is particularly useful in trundling, utility support, and groundwater control.

QA/QC during construction is achieved by monitoring:

- gel time records,
- flow rates,
- pressure,
- volume of injection at each injection port,
- core samples,
- shear wave velocity measurements using cross hole techniques,
- water pressure testing, and

- intrusive testing methods such as Standard Penetration Tests.

16.11 Preloading by Vacuum

The principles of using vacuum for preloading of soft clayey soils were first introduced in the early 1950s (Kjellman 1952). When a vacuum is applied to a soil mass, it generates a negative pore water pressure. If the total stress remains unchanged, the negative pore pressure increases the effective stress in the soil and this leads to consolidation. A schematic of the vacuum preloading method is shown in Figure 16.5.

The working platform consists of a sand layer through which vertical drains are placed in the soil. The vertical drains must be above any sand layer to sustain the vacuum pressure. A flexible geomembrane (polyethylene) liner covers the area and keys into an anchor trench that provides a watertight seal. A perforated pipe system is placed beneath the liner to collect water. Specially prepared vacuum pumps capable of pumping water as well as air are connected to the collection system. It is essential that the area be consolidated is totally sealed and isolated from any surrounding permeable soils to avoid the loss of vacuum. Leaks must also be avoided. Since pinholes or cracks in the sealing membrane are difficult to locate and repair, the membrane should be covered with water, which will also minimize potential damage from foot traffic and wildlife. When the required preloading pressure is higher than the capacity of the vacuum pumps, a surcharge fill may be used in conjunction with the vacuum method, as shown in Figure 16.5. The fill must be free from stones or sharp objects. If a fill is placed on the membrane liner during the vacuum period, it may be necessary to add a leak detection system under the liner to help locate leaks.

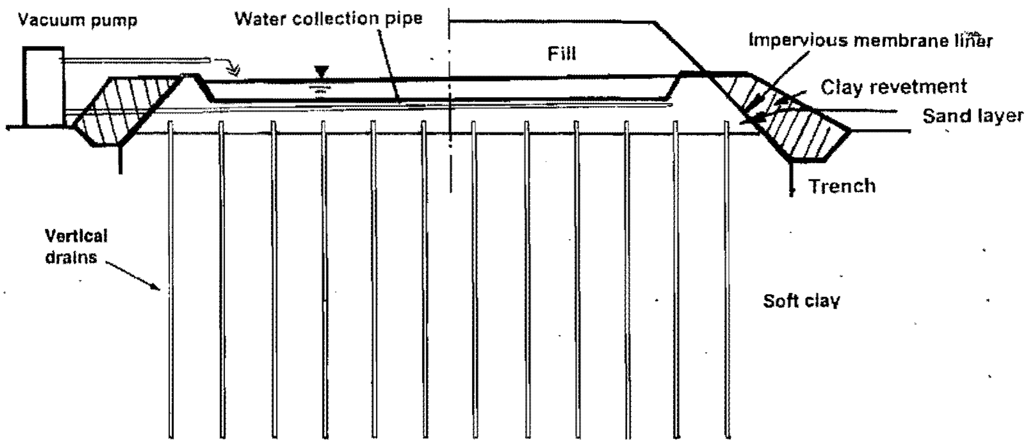


FIGURE 16.5 *Schematic of vacuum preloading system (modified from Shang et al. 1998)*

The vacuum method has the following characteristics (Shang et al. 1998): (1) a vacuum pressure of more than 80 kPa (600 mm Hg) can be achieved in practice using available vacuum equipment, which is equivalent to a fill 4 m to 4.5 m in height; (2) the lateral deformation of soil is inward due to the suction generated by the vacuum (instead of soil " squeeze-out" encountered in a surcharge preloading process, tensile cracks may develop adjacent to the treated area); and (3) there is no need to control the rate of vacuum application to prevent a bearing capacity failure because applying a vacuum pressure leads to an immediate increase of the effective stress and hence strengthening of the soil.

Despite a relatively good understanding of the principles of the vacuum method (Holtz and Wager 1975), the technique was not used widely in geotechnical engineering practice until the early 1980s, due mainly to high cost. The technology gained the attention of the Asian geotechnical community in the late 1980s (Qian et al. 1992) due to advances in geosynthetics and the shortage of land along shorelines. Prefabricated vertical (wick) drains that are effective, cost efficient and easy to install compared to sand drains, have made the cost of the vacuum method acceptable.

A number of projects have been undertaken in the Netherlands, France, Malaysia, Sweden and China. One of the largest projects was the East Pier Project in Xingang Port, Tianjin, China (Shang et al. 1998). The soil improvement project was conducted on 480,000 m^2 of reclaimed land using the vacuum preloading method. After 29 months including a preloading period of 135 to 247 days, the average consolidation settlement reached 2.0 m, corresponding to increases in undrained shear strength of two to four folds, as shown in Figure 16.6. The study showed that the vacuum method was an effective tool for the consolidation of very soft, highly compressible clayey soils over a large area. The technique is especially feasible in cases where there is a lack of suitable materials for use as a surcharge and extremely low shear strength. Access to a power supply for the vacuum pumps is necessary.

16.12 Electro-Osmotic and Electro-Kinetic Stabilization

Electro-osmosis is a technique used for the consolidation and strengthening of soft, saturated clayey soils. When a direct current (DC) voltage is applied to soil via electrode poles, the soil pore water will be attracted towards the direction of the negative terminal (cathode) due to the interaction of the electric field, the ions in the pore water and the soil particles. If drainage is provided at the cathode and prohibited at the anode, consolidation will be induced by electro-osmosis, resulting in the lower soil water content, higher shear strength and lower compressibility. In addition, electrochemical reactions associated with an electro-osmotic process alter the physical and chemical properties of the soil and lead to a further increase in shear strength (Mitchell 1993).

Casagrande (1937, 1959) first applied the technique of electro-osmosis to strengthen and stabilize soft silty clays in the middle 1930s. Since then, successful field tests have been reported that used electro-osmosis to strengthen silty clays and soft sensitive clays; to stabilize earth slopes and to reinforce steel piles installed in clayey soils (Bjerrum et al. 1967; Casagrande 1983; Lo et al. 1991). Electro-osmotic consolidation has been considered for projects requiring a rapid improvement in the properties of soft clayey soils.

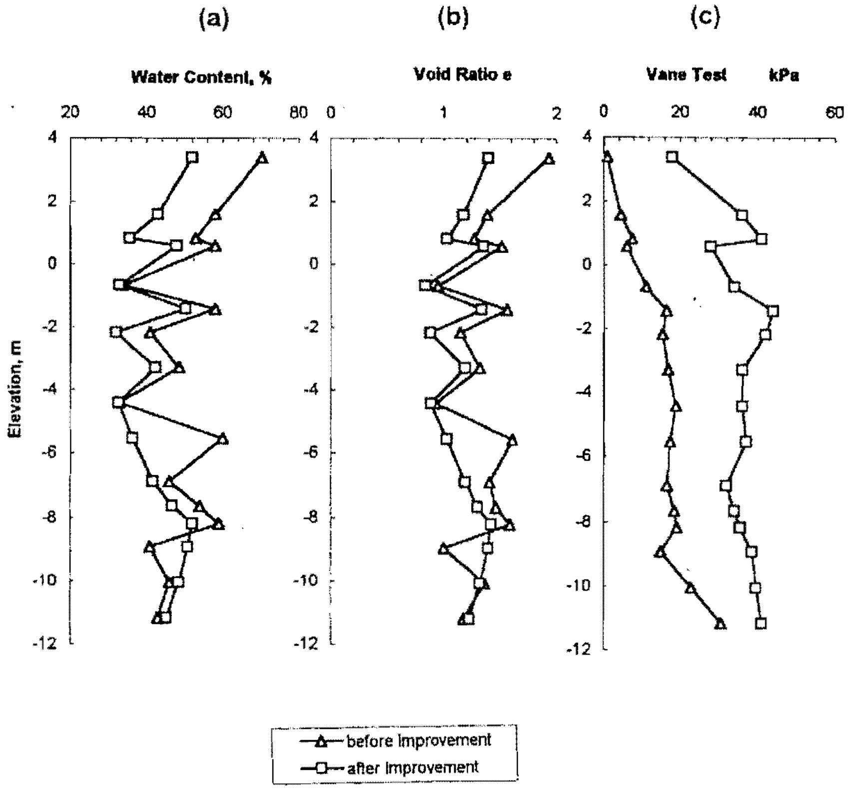


FIGURE 16.6 *Soil properties before and after vacuum preloading consolidation, East Pier Project, Tianjin, China (modified from Shang et al. 1998)*

When an open cathode and sealed anode condition is present, a negative pore water pressure is generated upon the application of a direct current (DC) electrical field. In one dimension, the pore pressure generated by electro-osmosis, $u_{eo}(x, t \rightarrow \infty)$ (kPa), at a distance x (m) to the cathode is given by (Esrig 1968):

$$u_{eo}(x) = -\frac{k_e}{k_h}\gamma_w U(x) \tag{16.4}$$

where

k_e (m²/sV) = electro-osmotic permeability
k_h (m/s) = hydraulic conductivity
γ_w = 9.81 kN/m³ = unit weight of water
$U(x)$ (V) = electrical potential at distance x to the cathode

Additional information on vacuum preloading can be found in Thevanayagam et al. (1994), Thevanayagam and Nesarajah (1996).

The equation shown above indicates that the pore pressure induced by electro-osmosis is negative and proportional to the electrical potential (i.e., it has a maximum magnitude at the anode and zero at the cathode). The negative pore pressure results in an increase in the effective stress in the soil, leading to consolidation, as described in the conventional consolidation theory. Knowing the pore pressure generated by electro-osmosis, the time rate of electro-osmotic consolidation can be estimated by conventional consolidation theory.

The electro-osmotic permeability, k_e, governs the water flow in a soil mass under an electrical gradient in the similar way as the hydraulic conductivity governs the flow in soil under a hydraulic gradient. When both the anode and cathode are open to drainage and the hydraulic gradient is set to zero, k_e can be determined by measuring the flow velocity across a soil plug using an empirical relation (Mitchell 1993)

$$q_e = k_e E \tag{16.5}$$

where

q_e = water flow vector due to an electrical gradient (m/s)
E = electric field intensity vector, defined as

$$E = -\nabla U \tag{16.6}$$

The power consumption per cubic metre of soil mass per hour is calculated from:

$$p = \kappa E^2 \tag{16.7}$$

where

p = unit power consumption (kW /m³)
κ = electrical conductivity of the soil (1/Ωm)

Equation (16.7) indicates that the power consumption of electro-osmotic treatment increases with the soil electrical conductivity and applied electric field. Table 16.2 summarizes the typical ranges of soil and electrical properties that are suitable and have been used for electro-osmotic consolidation.

TABLE 16.2 *Design Parameters and Common Soil Properties in Electro-osmotic Consolidation*

Parameter	Unit	Typical Range
k_h, Hydraulic Conductivity	m/s	10^{-10}-10^{-8}
k_e, Electro osmotic Permeability	m²/sV	~ 10^{-9}
κ, Electrical Conductivity of Soil	simens/m (1/Ωm)	0.01-0.5
E, Electric Field Intensity	V/m	20-100
c_v, Coefficient of Consolidation	m²/s	10^{-8}-10^{-7}
p, Hourly Power Consumption	kWh/m³	0.01-1

A two-dimensional electro-osmotic consolidation model was developed by Shang (1998) that can take the effects of both preloading and electro-osmotic consolidation into account. The most predominant electrochemical effects during an electro-osmotic process include the development of a pH gradient, the generation of gases and heating. The pH of soil water will increase rapidly to as high as 11or 12 at the cathode and decrease to almost two at the anode. Consequently, metallic anodes will corrode. Oxygen gas is generated at the anode and hydrogen gas at the cathode due to hydrolytic reactions. The electrical current also generates heating. The seriousness of these effects is directly related to the applied voltage and current. It is usually desirable to minimize heating effects to reduce power consumption. It has been found that applying polarity reversal and intermittent (pulse) current can reduce pH gradient and corrosion and increase the electro-osmotic permeability of the soil, thus improving the efficiency of electro-osmotic treatment (Shang et al. 1996).

The evaluation of electro-osmotic consolidation on a specific soil can be conducted according to the following procedure.

Determination of parameters

In addition to conventional soil properties such as the grain size, preconsolidation pressure, shear strength, water content, hydraulic conductivity, k_v, and coefficient of consolidation c_v, the parameters required for a treatability analysis include the electro-osmotic permeability, k_e; electrical conductivity, K; intensity of electric field, E; and power consumption, p. All these parameters can be determined from laboratory tests prior to field application. Table 16.2 lists the typical ranges of the major parameters for soils that are suitable for electro-osmotic treatment.

Electrical Operation System in Field Applications

The electrical operation system can be designed based on the parameters obtained from laboratory tests and from the geotechnical investigation of the site. Typically, the electrode poles consist of metallic rods or pipes installed vertically into the ground with prefabricated vertical drains installed at the cathode, as shown in Figure 16.7. The depth of the electrode insertion should be equal to the thickness of the soil layer to be treated. The upper portion of the electrodes in contact with the ground surface crust or top drainage layer should be electrically insulated using a dielectric coating to avoid short-circuiting due to the presence of surface water (Lo et al. 1991).

The material, layout and spacing of electrodes and the applied voltage are of utmost importance to a successful field application. Among the most commonly used conducting metallic materials used, the best results were reported using electrodes made of iron and copper rather than aluminium (Sprute and Kelsh 1980, Mohamedelhassan and Shang 2001). Electrodes made of carbon-coated steel rods and graphite have been used in laboratory studies to prevent electrode corrosion.

The typical spacing between anodes and cathodes reported in the literature ranged from 1 m to 3 m (Casagrande 1983; Lo et al. 1991). In general, an approximately uniform electric field gives the best results (Casagrande 1983).

To achieve an approximately uniform electric field, the spacing between electrode rods of the same polarity should be much less than spacing of the opposite polarity.

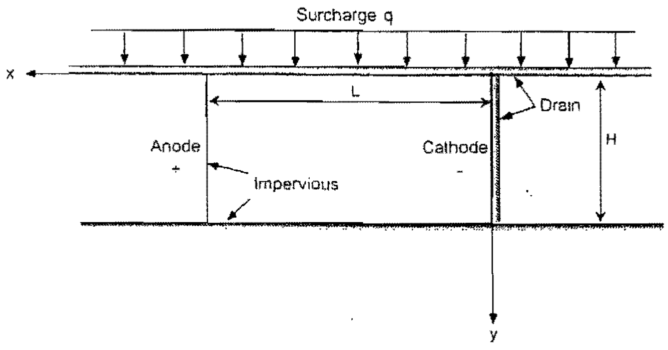


FIGURE 16.7 *Schematic of electro-osmotic consolidation (modified from Shang 1998)*

Power supply capacity can be estimated based on the soil's electrical conductivity and electrode layout. It has been found that a more dramatic voltage drop takes place at the soil-electrode contacts at a higher applied voltage, which made the treatment less efficient (Casagrande 1983; Shang et al. 1996). It was also observed that the voltage drop at the soil-electrode interface is affected by the electrode materials (Mohamedelhassan and Shang 2001). Therefore, a lower voltage applied across smaller anode-cathode spacing is desirable to generate the required electric field and special attention should be made for the electrode materials and configurations. However, the cost of electrodes and installation should also be considered. The final design will be based on a balance of the cost of electrodes and electrode installation as well as the treatment efficiency. For additional information, see Arman (1978), Broms (1979), Mitchell (1981, 1993), U.S. Navy (1983), Van Impe (1989) Hausmann (1990), Micic et al. (2003a, 2003b).

Electro-kinetic stabilization is a hybrid between electro-osmosis and chemical grouting. The infusion of certain stabilization chemicals into silty and sandy soils is made more efficient by the application of an electrical potential difference to the soil mass. The procedure is more effective in silty soils that are otherwise difficult to grout ordinarily. Information on this technique can be found in Broms (1979) and Mitchell (1981). More recently, electrokinetic assisted chemical stabilization has been applied to offshore calcareous soils (silts and sands) for stabilization of petroleum platforms (Mohamedelhassan and Shang, 2003, Shang et al. 2004a and 2004b).

17 Deep Foundations - Introduction

17 Deep Foundations - Introduction

17.1 Definition

A deep foundation is a foundation that provides support for a structure by means of toe resistance in a competent soil or rock at some depth below the structure, and/or by shaft resistance in the soil or rock in which it is placed. Piles are the most common type of deep foundation.

Piles are usually installed to support compression, uplift, or lateral loads from a structure. Although capacity aspects may be emphasized in design, the foremost reason for using piles is to reduce deformation, normally settlement. Piles are also used to densify granular soils and so stiffen the soil and/or change the natural frequency of soil under foundations for machinery, and are essential in situations in which water may scour foundation soils.

Piles can be pre-manufactured or cast-in-place; they can be driven, jacked, jetted, screwed, bored, or excavated. They can be made of wood, concrete, or steel, or a combination thereof. Bored piles of large diameter are frequently referred to as drilled piers in Canada.

17.2 Design Procedures

The quality of a deep foundation depends on the installation or construction technique, on equipment, and on workmanship. Such parameters cannot always be quantified nor taken into account in normal design procedures. Consequently, it is often desirable to design deep foundations on the basis of test loading of actual foundation units and to monitor construction to ensure that design requirements are fulfilled.

However, only a few projects are large enough to warrant full-scale testing during the design phase, and, in most cases, tests (proof-tests) are performed only during or even after construction of the foundation. Therefore, it is necessary to provide the engineer with appropriate design methods. Chapters 17 through 21 of the Manual present methods applicable to the various types of deep foundations encountered in practice.

17.3 Pile-Type Classification

The classification of pile types is governed by a number of factors (see Table 17.1), most of which must be considered before finalizing a design.

17.4 Limitations

Because of the influence of construction procedures on the behaviour of deep foundations, inspection should be considered as an integral part of the design of deep foundations and should be carried out by the engineer responsible for the design.

TABLE 17.1 *Pile-Type Classification*

Factor	Subgroup
1. Installation	Driven; bored; cast in-situ; jetted; excavated; augered.
2. Displacement	Displacement; low-displacement; non- displacement.
3. Material	Concrete; steel; wood.
4. Function	Shaft bearing; toe bearing; combination.
5. Capacity	High; moderate; low,
6. Shape	Square; round, hexagonal; octagonal; H-section; Tapered.
7. Environment	Land; marine; off-shore.
8. Inclination	Vertical; battered.
9. Length	Long; short.
10. Structure	Bridges; buildings; platforms; towers; machinery; etc.

18 Geotechnical Design of Deep Foundations

18 Geotechnical Design of Deep Foundations

18.1 Introduction

The design method used for a particular deep foundation will depend on the soil in which it lays, whether it is cohesive (clay) or cohesionless (sand), and whether the pile toe bears on soil or rock. In addition, each pile design should be based on considerations of both ultimate limit states (load capacity) and serviceability limit states (expected deformations or settlements). In the sections that follow, consideration is given to the geotechnical axial capacity (Section 18.2) and settlement (Section 18.3) of piles in soil, the lateral capacity (Section 18.4) and lateral movement (Section 18.5) of piles in soil, and the geotechnical axial capacity (Section 18.6) and settlement (Section 18.7) of piles bearing on rock. Both single pile behaviour for isolated piles and multiple pile behaviour for pile groups are examined.

18.2 Geotechnical Axial Resistance of Piles in Soil at Ultimate Limit States

18.2.1 Single Piles - Static Analysis

This section considers the geotechnical axial capacity of piles embedded in soil. Piles derive their load-carrying capacity from both toe and shaft resistance. The relative contribution of each to the total capacity of the pile depends, essentially, on the density and shear strength of the soil and on the characteristics of the pile.

The geotechnical axial capacity of a single pile, R, can be estimated by summing the shear stresses along the shaft, q_s, adding the bearing capacity of the pile toe, q_t, and subtracting the pile weight, viz.

$$R = \sum_{z=0}^{L} C q_s \Delta z + A_t q_t - W_P \qquad (18.1)$$

where

the pile of circumference, C, and embedded length, L, is subdivided into segments of length, Δz, and the pile toe has area, A_t, and pile weight, W_P.

The factored geotechnical axial resistance at ultimate limit states is taken as the ultimate axial capacity (R) multiplied by the geotechnical resistance factor (Φ) of 0.4 for compression and 0.3 for uplift (see Tables 8.1 and 8.2 in Chapter 8).

18.2.1.1 Cohesionless Soils

For cohesionless soil, the unit shaft friction at any depth z along the pile is given by

$$q_s = \sigma'_v K_s \tan \delta = \beta \sigma'_v \qquad (18.2)$$

and the bearing capacity of the pile toe is

$$q_t = N_t \sigma'_t \tag{18.3}$$

where

β = a combined shaft resistance factor
K_s = coefficient of lateral earth pressure
σ'_v = vertical effective stress adjacent to the pile at depth z
δ = the angle of friction between the pile and the soil
N_t = bearing capacity factor
σ'_t = vertical effective stress at the pile toe

The value of K_s is influenced by the angle of shearing resistance, the method of installation, the compressibility, degree of overconsolidation and original state of stress in the ground, as well as the material, size and shape of the pile. It increases with the in-situ density and angle of shearing resistance of the soil and with the amount of displacement. It is higher for displacement- type piles than for low-displacement-type piles such as H-piles. For bored piles, K_s is usually assumed equal to the coefficient of earth pressure at rest, K_0. For driven displacement-type piles, K_s is normally assumed to be twice the value of K_0.

The value of δ depends on the surface roughness of the pile, which depends on the pile material (steel, concrete. wood), the mean particle size of the soil, the normal pressure at the pile-soil interface and method of installation. It ranges from 0.5 to 1.0 ϕ.

The combined shaft resistance coefficient β generally ranges from 0.20 to 1.5 as indicated in Table 18.1 - see Fleming et al. (1992) for further discussion.

TABLE 18.1 *Range of β Coefficients*

Soil Type	Cast-in-Place Piles	Driven Piles
Silt	0.2 - 0.30	0.3 - 0.5
Loose sand	0.2 - 0.4	0.3 - 0.8
Medium sand	0.3 - 0.5	0.6 - 1.0
Dense sand	0.4 - 0.6	0.8 -1.2
Gravel	0.4 - 0.7	0.8 - 1.5

O'Neill and Reese (1999) indicate that β decreases as the bored (cast-in-place) pile length increases in sands and gravels. The values in Table 18.1 could be considered average values for rather long piles.

The toe bearing capacity factor N_t depends on soil composition in terms of grain size distribution, angularity and mineralogy of the grains, natural soil density, and other factors. Typical ranges of values for N_t are given in Table 18.2.

TABLE 18.2 *Range of N_t Factors*

Soil Type	Cast-in-Place Piles	Driven Piles
Silt	10 – 30	20 – 40
Loose sand	20 – 30	30 – 80
Medium sand	30 – 60	50 – 120
Dense sand	50 – 100	100 – 120
Gravel	80 - 150	150 - 300

Remarks

i. The toe response of bored piles is certainly softer than for driven piles. However, it may be argued that this is a serviceability issue and not a capacity issue. The toe capacity is only governed by the geological nature of the deposit near the pile toe rather than the method of installation. Thus, the N_t value for both cast-in-place and driven piles should be the same and equal to those given in Table 18.2 for driven piles. In the absence of test loading, a factor of safety of at least three should be applied to any theoretical computation.

ii. Both q_s and σ'_t may continue to increase with increasing depth, but at a decreasing rate. For practical design purposes, it is advisable to adopt limiting values of both q_s and σ'_t for long piles in cohesionless soils (Poulos et al., 2001). Jardine and Chow (1996) and Jardine et al. (1998) provide a method to estimate q_s in cohesionless soils based on the use of the cone penetrometer, in which the cone resistance is used to estimate radial effective stresses after pile installation and accounts for effects of soil dilation at the pile-soil interface and pile compressibility. The method also accounts for the effect of pile depth. This method should be used whenever CPT tests can be conducted.

18.2.1.1(1) Tapered Piles

For tapered piles, the skin friction at any station along the pile shaft can be calculated by (Wei & El Naggar 1998):

$$q_s = K_t K_s \sigma'_v \tan \delta \tag{18.4}$$

The taper coefficient K_t is introduced to capture the taper effect and in the case of cylindrical piles K_t=1. The taper coefficient for cohesionless soil is a function of the internal friction of the soil, pile-soil interface friction angle, δ, taper angle, θ, the pile geometry, settlement level and the effective overburden pressure. The taper coefficient K_t is given by (El Naggar and Sakr 2000):

$$K_t = \frac{\tan(\theta+\delta)\cot(\delta)}{1+2\zeta\tan(\theta)\tan(\theta+\delta)} + \frac{4G\tan(\theta)\tan(\theta+\delta)\cot(\delta)S_r}{(1+2\zeta\tan(\theta)\tan(\theta+\delta))K_s\sigma_v} \tag{18.5}$$

where

G = the shear modulus of the sand, $\zeta = ln\,(r_l/r_m)$ in which r_m = the average pile radius and r_l is a radius at which the shear stress becomes negligible and is taken to be equal to 2.5L (1-ν) where ν = Poisson's ratio of the soil, and S_r is the pile settlement at the ultimate load as a ratio of its diameter = 0.1. The effective overburden pressure has a profound effect on K_t as it decreases quickly with an increase in σ_v. For practical tapered piles length, K_t may be taken as two.

18.2.1.1(2) Layered Soils under the Pile Toe

For piles bearing on layered soils, the toe capacity should be estimated with due consideration. Meyerhof (1976) and Meyerhof and Sastry (1978) considered three cases of layered soil profiles. In the first case, where a weak soil layer over lies a dense sand layer, the full toe capacity is not developed until the pile penetrates six diameters into the dense sand (Meyerhof and Sastry 1978). The toe capacity can be assumed to decrease linearly from the value for the dense sand layer to the value for the weak layer for a penetration distance less than six pile diameters. In the second case, a weak layer underlies a dense sand layer and the toe capacity would be affected if the pile toe is less than three times the pile toe diameter above the weak layer (Matsui 1993). The toe capacity can be assumed to decrease linearly from the value for the dense sand layer to the value for the weak layer for a distance less than three pile diameters.

In the third case, the dense sand layer is sandwiched between two weak layers and their effects must be considered together.

18.2.1.2 Cohesive Soils

Design methods for piles in fine-grained soils are in some cases of doubtful reliability. This is particularly so for the bearing capacity of shaft-bearing piles in clays of medium-to-high shear strength. Because of this, pile test loading should be carried out where economically justified or, alternatively, an adequate factor of safety should be used.

Piles in cohesive soils and bearing on stiff soils may mobilize substantial toe resistance, which, for large-diameter bored piles, may represent the usable capacity of the pile.

18.2.1.2(1) Total Stress versus Effective Stress Approach

Until recently it was the general practice to evaluate the capacity of piles in clay from a total stress approach, i.e., on the basis of the undrained shear strength, s_u, of the clay. Empirical correlations between s_u and the toe-and-shaft resistance on a pile have been developed, but these have not proved reliable, particularly for s_u in excess of about 25 kPa. Therefore, analysis in terms of effective stresses is more rational, i.e., the same method as used for piles in cohesionless soils applies in all details. Burland (1973) provides a detailed discussion on relevant values of β; Skempton (1951) and Ladanyi (1963) present discussion and values of N_t. The relationship in Subsection 18.2.1.1 may be used in design with the following values:

$$\beta = 0.25 - 0.32 \qquad (18.6)$$

$$N_t = 3 - 10 \qquad (18.7)$$

For tapered piles, Blanchet et al. (1980) suggest using $\beta = 0.5$ to 0.6.

18.2.1.2(2) Shaft Resistance in Clays with s_u < 100 kPa

A pile driven in clay with undrained shear strength smaller than 100 kPa derives its capacity almost entirely from shaft resistance. It is still common practice to determine the ultimate shaft resistance of a single pile using total stress analysis from the formula:

$$q_s = \alpha s_u \qquad (18.8)$$

where α = adhesion coefficient ranging from 0.5 to 1.0.

Figure 18.1 shows the adhesion coefficient as a function of the undrained shear strength of the clay. However, the actual resistance depends significantly on the geometry of the foundation, the installation method and sequence, the properties of the clay, and time effects. The capacity of piles determined from the above formula should be confirmed by test loading.

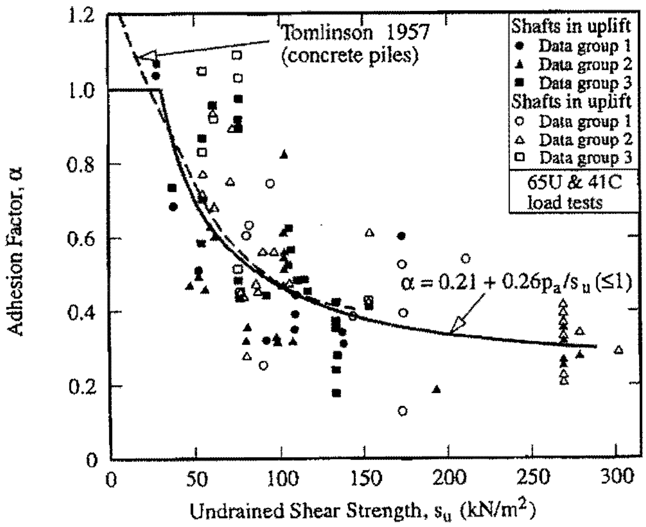


FIGURE 18.1 *Adhesion as a function of undrained shear strength*

18.2.1.2(3) Shaft Resistance in Clays where s_u > 100 kPa

A pile driven in clay with an undrained shear strength in excess of 100 kPa derives its capacity from both shaft and toe resistance. However, the shaft resistance of such a pile cannot be predicted with any degree of reliability because little is known of the effect of driving on the resistance and on the final effective contact area between clay and pile. For preliminary design, the relationship given in Section 18.2.1.2 can be used. For final design purposes, however, it is suggested that the pile capacity be determined by test loading.

Large-diameter bored piles (with or without enlarged belled bases, or under-reamed shafts) are successfully used in clays or cohesive soils where s_u > 100 kPa. Present design methods have been derived from extensive studies on bored piles in London clays. Considering the special properties of these soils, the generalization of empirical design parameters to other types of soils should be made with caution. Bored piles are also used in argillaceous intermediate geomaterials (cohesive earth materials), such as hard clays and clay-based rock (e.g., Queenston shale formations). Hassan et al. (1997) provide a method to estimate q_s and q_t that accounts for the pile-geomaterial interface roughness and the initial effective stress at the interface. For bored piles in porous sandstone, the methods provided by Seidel (1993) and McVay et al. (1992) are more suitable.

18.2.1.2(4) Toe Resistance

The ultimate toe resistance may be estimated from:

$$R_t = N_t s_u A_t \qquad (18.9)$$

where

R_t = toe resistance
A_t = cross-sectional area of pile at toe
s_u = minimum undrained shear strength of the clay at pile toe
N_t = a bearing capacity coefficient that is a function of the pile diameter, as follows:

Pile toe diameter	N_t
smaller than 0.5m	9
0.5 m to 1 m	7
larger than 1m	6

In very stiff clays and tills, where samples are difficult to retrieve and the undrained shear strength is not easily measured, a pressuremeter may be used to evaluate the strength of the soil.

18.2.1.3 Stratified Deposits

The relative contribution of the various strata penetrated by a pile to the capacity of that pile is primarily a function of the relative stiffness of these layers and of the type of pile. Static analysis for total axial capacity essentially involves calculating contributions of various unit shaft resistance values, q_s, associated with the different strata that the pile penetrates and the end-bearing associated with the stratum containing the pile toe.

Furthermore, it is important to install the top of the pile a distance of at least four diameters into any stiffer clay stratum so that the full value N_t = 9 can be used, and to watch for the presence of a weaker stratum below the toe which could reduce the toe resistance.

18.2.1.4 Helical (Screw) Piles

The basic form of a helical pile or anchor for construction applications consists of a helically shaped bearing plate or multiple plates attached to a central shaft. Historically, helical piles or anchors have been used in relatively light load applications, with shaft diameters and helix diameters typically less than 100 mm and 400 mm respectively.

Recently however, through the development of high-capacity torque drives (in excess of 50,000 ft-lbs) that are used for helical pile installation, larger diameter shafts and helixes have been constructed and installed.

When installed to proper depth and torque, the helical plates act as individual bearing elements to support a load. The helical pile is therefore a deep, end-bearing foundation that can be used to resist both compressive and tension loads. Installation of helical piles is accomplished by hydraulic torque drives that can be mounted to just about any type of machine (e.g. bed-mounted drill rigs, rubber-tired backhoes, skid-steer loaders, mini-excavators, and track-hoe excavators).

The total capacity of the helical pile or anchor equals the bearing capacity of the soil applied to the individual helical plate(s) and, in some instances, the skin friction of the shaft. This is:

$$R = Q_t + Q_f \tag{18.10}$$

where

Q_t = Total multi-helix pile capacity
Q_f = Capacity due pile shaft skin friction

The evaluation of these components is described further below.

The factored geotechnical axial resistance at ultimate limit states is taken as the ultimate axial capacity (R) multiplied by the geotechnical resistance factor (Φ) of 0.4 for compression and 0.3 for uplift (Tables 8.1 and 8.2 in Chapter 8).

18.2.1.4(1) Helical Plate(s) Bearing Capacity

The total capacity of an end-bearing helical pile is evaluated as the sum of the capacities of each individual helical plate(s). The helical pile capacity is thus determined by calculating the unit bearing capacity of the soil and applying it to the individual helical plate(s) areas, i.e,

$$Q_h = A_h(s_u N_c + \gamma D_h N_q + 0.5\gamma B N_\gamma) \tag{18.11}$$

Q_h = Individual helix bearing capacity
A_h = Projected helix area
s_u = undrained shear strength of the soil
γ = Unit weight of the soil
D_h = Depth to helical bearing plate
B = diameter of the helical plate
N_c, N_q and N_γ = Bearing capacity factors for local shear conditions

The total helical plates capacity, Q_t, can be expressed as:

$$Q_t = \Sigma Q_h \tag{18.12}$$

The bearing capacity equation is applicable only when the helical bearing plates are spaced far enough apart, at least three times the diameter of the largest helix, to avoid overlapping of their stress zones. In cases involving overlapping stress zones, the multi-helix capacity can be determined by computing the bearing capacity of the bottom plate, and the cylindrical shear capacity developed between the upper and lower plate(s). The formulation provided below with revision of pile shaft diameter to effective helix diameter may be considered.

18.2.1.4(2) Capacity Due to Skin Friction

The skin friction along the pile shaft typically is not considered in the total capacity unless the shaft is at least 100 mm in diameter (or equivalent diameter). The capacity due to skin friction can be calculated as follows:

$$Q_f = \sum[\pi D f_s \Delta L_f] \tag{18.13}$$

Q_f = Frictional resistance of pile
D = Diameter of pile shaft
f_s = Sum of friction and adhesion between soil and pile
ΔL_f = Incremental pile length over which πD and f_s are taken as constant

18.2.1.4(3) Relationship of Load Capacity to Installation Torque

An estimate of the helical pile ultimate capacity may be achieved through monitoring of installation torque. Recording of installation torque can also serve as a quality control step, identifying piles that did not achieve the expected installation torque and may require load testing. The relationship between the load capacity and installation torque, which was developed based on pullout tests on helical piles, can be described using the following empirical equation:

$$Q_u = K_T \times T \tag{18.14}$$

where
R = Ultimate capacity of screw pile
K_T = Empirical torque factor
T = Average installation torque

The value of K_T may range from 3/ft to 20/ft if T is recorded in ft-lbs, or 10/m to 33/m if T is recorded in N-m. The selection of K_T is dependent upon the soil conditions and anchor design including plate and shaft diameter. For small sized square shaft anchors (less than 90 mm diameter), the K_T value has been found to range from 10/ft to 12/ft with 10/ft (33/m) being the recommended default value. For pipe shaft anchors (90 mm O.D. pipe), the recommended default value is 7/ft (23/m), with this value decreasing to 3/ft (10/m) for shaft diameters approaching 200 mm.

Torque monitoring tools are available to provide a suitable method of production control during installation. As a quality assurance measure, it is recommended that the engineer specifies a required torque during construction.

Installation torque is primarily a function of the frictional resistance along the shaft, the frictional resistance along the top and bottom surfaces of the helical plate(s), and the passive resistance along of the leading edge(s) of the plate(s). Although soft zones at depth may not influence the recorded torques, they may adversely impact the load carrying capacity of the helical pile. As a result, a good understanding of the ground conditions around pile(s), within and extending beyond the zone that is expected to be stressed as a result of loads on the pile(s) is important.

18.2.2 Pile Groups - Static Analysis

It is common practice to define the axial capacity of a pile group relative to the sum of the capacities of the individual piles in the group. Group 'efficiency' is defined as the ratio of the group capacity to this sum of the individual pile capacities.

18.2.2.1 Cohesionless Soils

Driven piles in cohesionless soils develop larger individual capacities when installed as a group (group efficiency > 1) since lateral earth pressure and sand density increase with the driving of additional piles. Therefore, it is conservative to use the sum of the individual pile capacities as an estimate of the pile group capacity.

For bored pile groups, the individual pile capacity is reduced by the addition of the extra piles, since the boring process reduces sand density and lateral earth pressures (efficiency is < 1).

For bored pile groups, a reduction factor (Meyerhof (1976) suggests 0.67 for piles in clean sand) may need to be applied to the sum of individual pile capacities. However, for piles in sands with some fines (e.g., silty sands) and if the cap is firmly in contact with the soil and spacing of the piles is less than 4.5d (where d = pile diameter) the group efficiency is 1.0.

18.2.2.2 Cohesive Soils

In addition to the possibility that individual piles in a pile group act independently to support the applied load, a closely spaced pile group, can act as a 'block' whereby the soil between adjacent piles is dragged down between them, shaft resistance develops around the perimeter of the group only, and end-resistance develops under the whole of the pile-soil block. A rational approach to estimating pile group capacity is to use the minimum of a) the sum of the individual pile capacities and b) the capacity of the pile- soil block analysed as an equivalent single pile. For this block capacity calculation, an average unit shaft resistance, q_s, must be calculated since for zones on the perimeter where there is soil-soil contact, $q_s = s_u$ and for zones where there is soil-pile contact, $q_s = \alpha\, s_u$. The block perimeter is the circumference, C, of the equivalent pile, and the area of the block base is taken as the base area, A_t, of the equivalent pile.

18.2.3 Single Piles - Penetrometer Methods

18.2.3.1 Limitations

Field test data are often available in the form of static or dynamic penetration resistance. Clearly, it is appealing to generate predictions of axial capacity directly from penetration resistance, rather than from more fundamental soil shear strength parameters. Caution must be exercised however, given that this attempted simplification may disregard the complexity of both the penetration tests themselves and the axial pile response.

18.2.3.2 Cone Penetration Test

The axial capacity of deep foundations in soils can be computed from the results of a static cone penetration test (CPT). The test is suitable for a large range of soils provided adequate pushing force is available for sufficient depth of penetration.

The ultimate geotechnical axial capacity of a single pile can be estimated using the basic equation given in Section 18.2.1 and estimating the unit base resistance, q_t, and unit shaft resistance, q_s, from

$$q_t = k_c\, q_{ca} \tag{18.15a}$$

$$q_s = \frac{1}{\alpha} q_c \tag{18.15b}$$

where

q_c = cone penetration resistance (units of stress) from CPT
q_{ca} = equivalent cone penetration resistance at pile base according to Figure 18.2
k_c = bearing capacity factor based on soil type and pile type (Table 18.3)
α = friction coefficient (Table 18.4)

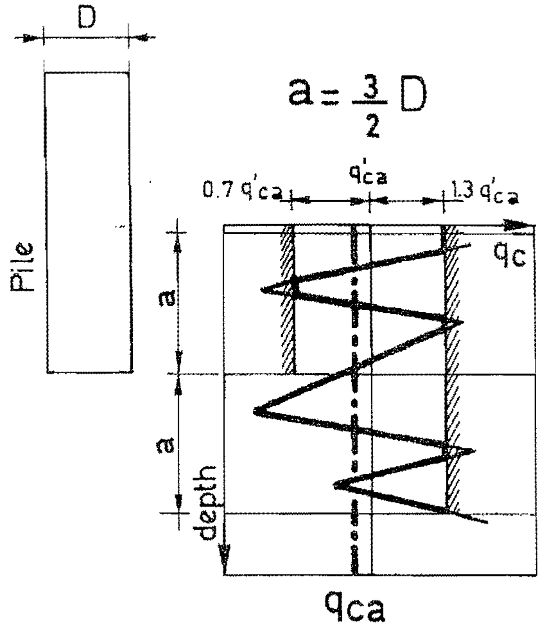


FIGURE 18.2 *CPT method to determine equivalent cone resistance at pile base (after Bustamante and Gianeselli, 1982)*

This approach is based on extensive full scale pile load test data from France (Bustamante & Gianeselli, 1982) and supported by pile load test data in North America (Robertson et al., 1988; Briaud & Tucker, 1988). The scaling effect to account for the difference in size between the cone penetrometer and the pile and the method of installation is accounted for in the selection of k_c and α using Tables 18.3 and 18.4.

The method developed by Lehane and Jardine (1994) and Jardine and Chow (1996) is especially useful in estimating q_s for piles driven in cohesionless soils using the cone penetrometer measurement. The method accounts for effects of soil dilation at the pile-soil interface and pile depth and compressibility. It should be used whenever CPT tests are conducted. The ultimate axial capacity for design is influenced by the number of CPTs performed, the observed variability of the test results and the local experience available. Caution should be exercised when designing piles in sensitive clays.

TABLE 18.3 *Bearing Capacity Factors, k_c*

Soil Type	Q_c (MPa)	Factors k_c	
		Group I*	Group II*
Soft clay and mud	< 1	0.4	0.5
Moderately compact clay	1 to 5	0.35	0.45
Silt and loose sand	≤ 5	0.4	0.5
Compact to stiff clay and compact silt	> 5	0.45	0.55
Soft chalk	≤ 5	0.2	0.3
Moderately compact sand and gravel	5 to 12	0.4	0.5
Weathered to fragmented chalk	> 5	0.2	0.4
Compact to very compact sand and gravel	> 12	0.3	0.4

* Note:

i. Group I: Plain bored piles, mud bored piles, micro piles (grouted under low pressure), cased bored piles, hollow auger bored piles, piers and barrettes.

ii. Group II: Cast-in-place screwed piles, driven precast piles, prestressed tubular piles, driven cast piles, jacked metal piles, micropiles (grouted under high pressure with diameters < 250 mm).

The factored geotechnical axial resistance at ultimate limit states is taken as the predicted ultimate capacity multiplied by the geotechnical resistance function (Φ) of 0.4 for compression and 0.3 for uplift (Table 8.1 in Chapter 8).

TABLE 18.4 *Friction Coefficient, α*

Soil Type	q_c (MPa)	Coefficient α				Maximum Limit of q_c (MPa)					
		Category*									
		I		II		I		II		III	
		A	B	A	B	A	B	A	B	A	B
Soft clay and mud	< 1	30	90	90	30	0.015	0.015	0.015	0.015	0.035	-
Moderately compact clay	1-5	40	80	40	80	0.035 (0.08)	0.035 (0.08)	0.035 (0.08)	0.035	0.08	≥ 0.12
Silt and loose sand	≤ 5	60	150	60	120	0.035	0.035	0.035	0.035	0.08	-
Compact to stiff clay and compact silt	> 5	60	120	60	120	0.035 (0.08)	0.035 (0.08)	0.035 (0.08)	0.035	0.08	≥ 0.20
Soft chalk	≤ 5	100	120	100	120	0.035	0.035	0.035	0.035	0.08	-
Moderately compact sand and gravel	5-12	100	200	100	200	0.08 (0.12)	0.035 (0.08)	0.08 (0.12)	0.08	0.12	≥ 0.2
Weathered to fragmented chalk	> 5	60	80	60	80	0.12 (0.15	0.08 (0.12)	0.12 (0.15)	0.12	0.15	≥ 0.2
Compact to very compact sand and gravel	> 12	150	300	150	200	0.12 (0.15)	0.08 (0.12)	0.12 (0.15)	0.12	0.15	≥ 0.2

Note: Bracketed values of maximum limit unit skin friction, q_s, apply to careful execution and minimum disturbance of soil due to construction.

* Category:

IA	Plain bored piles, mud bored piles, hollow auger bored piles, micropiles (grouted under low pressure), cast-in-place screwed piles, piers and barrettes.
IB	Cased bored piles, driven piles.
IIA	Driven precast piles, prestressed tubular piles, jacked concrete piles.
IIB	Driven metal piles and jacked metal piles.
IIIA	Driven grouted piles and driven rammed piles.
IIIB	High pressure grouted piles with diameters > 250 mm and micropiles grouted under high pressure.

18.2.3.3 Standard Penetration Test

The ultimate geotechnical axial capacity of a single pile in granular soils can be estimated from the results of the Standard Penetration Test (SPT) as suggested by Meyerhof (1976).

$$R = mNA_t + NA_s \tag{18.16}$$

where

R = pile capacity
M = an empirical coefficient equal to 400 for driven piles and to 120 for bored piles
N = SPT index at the pile toe
A_t = pile toe area
n = an empirical coefficient equal to two for driven piles and to one for bored piles
N = average SPT index along the pile
A_s = pile embedded shaft area

Decourt (1995) developed a more comprehensive correlation of the shaft and toe resistance of piles with the SPT value. He suggested the following expressions:

$$q_s = \alpha\,(2.8N_{60} + 10)\ (kPa) \tag{18.17a}$$

$$q_t = K_b\overline{N}_b\ \ (kPa) \tag{18.17b}$$

where

α = 1 for displacement piles in any soil and non-displacement piles in clays, and 0.5 to 0.6 for non-displacement piles in granular soils.
N_{60} = average SPT index (normalized to 60 % energy efficiency) along the pile shaft
$\overline{N}_b$ = average of SPT index in the vicinity of the pile toe
K_b = is a base factor given in Table 18.5.

TABLE 18.5 *Base Factor, K_b (Decourt, 1995)*

Soil Type	Displacement Piles	Non-Displacement Piles
Sand	325	165
Sandy silt	205	115
Clayey silt	165	100
Clay	100	80

The Standard Penetration Test has significant limitations (see Chapter 4), and care must be exercised when using the test results. For this reason and when using working stress design, a minimum factor of safety of four should be applied to the calculated capacity unless local experience indicates otherwise. For factored geotechnical axial resistance of ultimate limit states, it is suggested that the ultimate axial capacity be multiplied by a geotechnical resistance factor of 0.3.

18.2.4 Single Piles - Dynamic Methods

18.2.4.1 Introduction

The objective when dealing with the dynamic methods of pile design is to relate the dynamic pile behaviour (acceleration or driving resistance) to the ultimate static pile resistance. Care should be taken when using these methods, since they may ignore the effects of 'set up' in soft clays (dynamic methods usually provide estimates of pile capacity just after driving), downdrag (see next section) and serviceability issues (whether expected pile settlement is acceptable).

18.2.4.2 Axial Capacity Based on Dynamic Monitoring

The capacity of a single pile can be estimated by means of dynamic measurements. The reliability of this estimate of the capacity, under favourable conditions, can be almost equal to that of a routine static loading test. The measurements and the evaluation of the data must be carried out by a person competent in this field. For more details, see Chapters 19 and 20.

18.2.4.3 Geotechnical Axial Capacity Based on Wave-Equation Analysis

The wave-equation analysis (which is discussed in Chapter 19) is a tool for determining pile bearing capacity, pile driveability, and for hammer selection. The wave equation requires accurate input of several hammer and soil parameters that can vary widely from case to case. Hammer-rated energy can differ substantially from actual measurements, and the soil parameters are 'model-dependent' empirical values and not rational properties that can be measured independently. Unless there is calibration to field measurements, the analysis can only be used to provide general guidance.

18.2.4.4 Dynamic Formulae

The assumptions made in the dynamic formulae are oversimplified, and the results cannot always be related to actual pile capacity. One reason is that the dynamic formulae input hammer-rated energy and not the actually delivered energy and this results in considerable error. Nevertheless, when used by competent persons and related to local experience, a dynamic formula can still serve as a guide to engineering judgement. However, dynamic formulae are best replaced by other techniques.

18.2.5 Negative Friction and Downdrag on Piles

When piles have been installed in or through a clay deposit that is subject to consolidation, the resulting downward movement of the clay around the piles, as well as in any soil above the clay layers, induces downdrag forces on the piles through negative skin friction. The magnitude of settlement needed to cause the negative skin friction is very small. For instance, observations by Fellenius and Broms (1969) and Fellenius (1972) of negative skin friction on piles in a 40 m thick clay layer indicate that the relative movement required can be smaller than a millimetre. Such small relative movements occur easily as a result of the large stiffness difference between the pile and the soil. Therefore, with time, small movements or strains will occur in any portion of a pile and positive resistance along a lower portion of a pile are the norm rather than the exception.

The simplest method for computing the negative skin friction is to assume that it is proportional to the undrained shear strength of the soil (Terzaghi & Peck, 1967).

$$q_n = \alpha\, s_u \tag{18.18}$$

where

- q_n = unit negative skin friction
- α = a reduction coefficient ranging from 0.5 to 1.0
- s_u = the undrained shear strength after the soil has consolidated under the new load and therefore should be estimated from CU tests representative of the expected overburden pressure.

Field observations on instrumented piles have shown that the negative skin friction is a function of the effective stress acting on the pile and may be computed in the same way as the positive shaft resistance, as detailed in Subsection 18.2.1.1. In most clays and silts, the magnitude of the negative skin (shaft) friction approximates to a β factor of about 0.2 to 0.3.

The total drag load, Q_n, for a single pile is:

$$Q_n = q_n C D_n \tag{18.19}$$

where

C = shaft circumference or perimeter length
D_n = length of pile embedded in settling soil.

Alternatively, elastic methods can be used. These methods suggest how downdrag relates to settlement (for example Poulos & Davis, 1972), and provide a means for estimating the maximum downdrag force and its development with time. Various theoretical solutions are available for single piles (Poulos & Davis, 1980).

18.2.5.1 Design Considering Downdrag

The design must consider the structural axial capacity, the settlement and the geotechnical axial capacity of the pile. The downdrag increases the structural loads in the pile and thus has to be accounted for when evaluating the structural ultimate limit state of the pile. The downdrag also increases the pile settlement and therefore should be accounted for when evaluating the serviceability limit state of the pile. However, the downdrag has no effect on the geotechnical axial capacity of the pile. It is important to realize that drag load and transient live load do not combine, and that two separate loading cases must be considered: permanent load plus drag load, but no transient live load; and permanent load and transient live load, but no drag load. Furthermore, a rigid, strong pile will have a large drag load, but small settlement, whereas a less rigid and less strong pile will have a smaller drag load, but larger settlement. Also, no pile subjected to downdrag condition will settle more than the ground surface nearest the pile.

As a first step in the design of the pile, the neutral plane must be determined. The neutral plane is located where the negative skin friction changes over to positive shaft resistance. It is determined by the requirement that the sum of the applied dead load plus the drag load is in equilibrium with the sum of the positive shaft resistance and the toe resistance of the pile. The location of the neutral plane governs both the maximum load in the pile and the settlement of the pile.

18.2.5.1(1) Neutral Plane

The neutral plane is found as the intersection of two curves. First, as illustrated in Figure 18.3, a load distribution curve is drawn from the pile head and down with the load value starting with the applied dead load and increasing with the load due to negative skin friction acting along the entire length of the pile. Second, a resistance distribution curve is drawn from the pile toe and up, starting with the value of ultimate toe resistance and increasing with the positive shaft resistance.

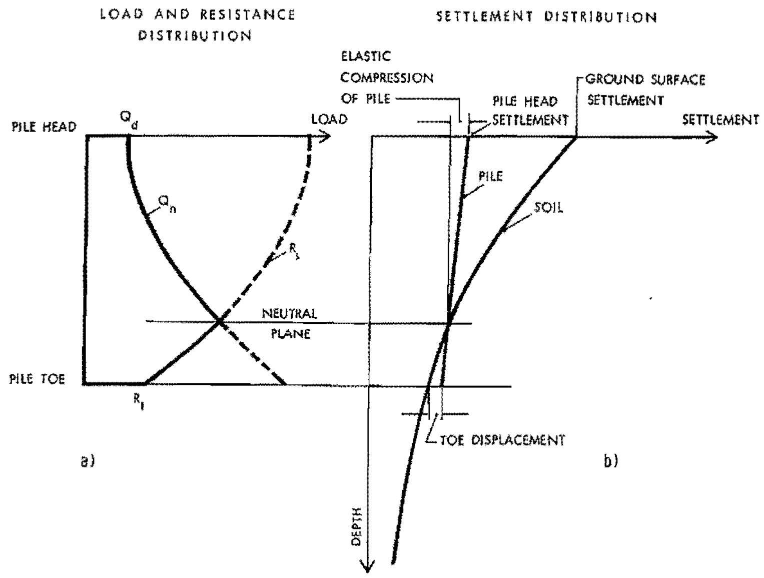


FIGURE 18.3 *Calculation of the location of the neutral plane and the settlement of a pile or a pile group (after Fellenius, 1984a)*

The determination of the load distribution in a pile is subject to uncertainty. Reliable information on the soil strength is required when determining the load distribution. It is recommended that the theoretical analysis adopting the method in Section 18.2.1.1 be used. The analysis should be supplemented with information from penetrometer tests, such as the SPT and the static cone penetrometer. For driven piles, the analysis should be combined with results from the analysis of dynamic monitoring data.

18.2.5.1(2) Structural Axial Capacity

The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the drag load; transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but drag load is to be excluded.

At the neutral plane, the pile is confined, and the maximum combined load may be determined by applying a safety factor of 1.5 to the pile material strength (steel yield and/or concrete 28-day strength and long-term crushing strength of wood).

If the negative skin friction and the positive shaft resistance as well as the toe resistance values are determined, assuming soil-strength values 'err' on the strong side, the calculated maximum load on the pile will be conservative.

18.2.5.1(3) Settlement

As illustrated in Figure 18.3b, the settlement of the pile head is found by drawing a horizontal line from the neutral plane, as determined according to the foregoing method, to intersect with the curve representing the settlement distribution in the soil surrounding the pile. The settlement of the pile head is equal to the settlement of the soil at the elevation of the neutral plane plus the elastic compression of the pile due to the applied dead load and the drag load (Fellenius, 1984a).

One condition for the analysis is that the movement at the pile toe must be equal to or exceed the movement required to mobilize the ultimate toe resistance of the pile. In most soils, this required movement is equal to about 1 % to 2 % of the pile toe diameter of driven piles and about 5 % to 10 % of the toe diameter for bored piles. If the movement is smaller than this required magnitude, the neutral plane will move higher up in the settlement diagram and the settlement will increase correspondingly. If this occurs, the magnitude of the settlement will normally be negligible and correspond to the elastic compression of the pile.

The settlement calculation should be carried out according to conventional methods (see Chapter 11) for the effective stress increase caused by dead load on the pile(s), surcharge, groundwater lowering, and/or any other aspect influencing the stress in the soil. The dead load applied to the pile cap should be assumed to act at an equivalent footing located at the level of the neutral plane and the load distributed from this plane. The settlement of the pile cap is the sum of the settlement of the equivalent footing and the compression of the piles above the neutral plane. Note that Figure 18.3 does not show the settlement due to the dead load acting on the equivalent footing at the neutral plane.

The accuracy of the calculation of the distribution of settlement depends on the reliability of the input data, which in turn depends on the completeness of the site investigation program. It is imperative that representative samples be obtained from all soil layers, including those below the pile toe, and that the strength and compressibility properties of the soil be determined in the laboratory. In-situ testing methods such as vane tests and static cone-penetrometer tests will enhance the laboratory testing.

For the case in which the structure is built before the pore pressures induced by the pile installation have dissipated, it is necessary to estimate the additional settlement caused by the pore pressure dissipation.

18.2.5.1(4) Geotechnical Axial Capacity

The last part of the design is to check the safety against plunging failure of the pile. In this case, the pile moves down along its entire length and the downdrag is eliminated. Therefore, the load is the combination of the dead load and the live load, no drag load, and the case is similar to that of designing the allowable load of a pile not in a downdrag condition.

As stated by Fellenius (1984a), when the capacity has been determined using the static loading test or the dynamic testing method, a factor of safety of 2.0 or larger ensures that the neutral plane is located below the mid-point of the pile. When the capacity is calculated from soil-strength values, the factor of safety should not be smaller than 3.0.

18.2.5.1(5) Special Considerations

Downdrag on piles caused by negative skin friction is most often a settlement problem and rarely a capacity problem. According to the method recommended in this Section, the service load should not be reduced by any portion of the drag load unless required by insufficient structural strength of the pile at the location of the neutral plane, or in order to lower the location of the neutral plane (reducing settlement).

When settlement occurs around a pile or a pile group, piles that are inclined will be forced to bend by the settling soil. For this reason, it is advisable to avoid inclined piles in the foundation, or, at least, to limit the inclination of the piles to values that can follow the settlement without excessive bending being induced in the piles. Furthermore, piles that are bent, doglegged, or damaged during installation will have a reduced ability to support the service load in a downdrag condition. Therefore, a design carried out according to this section postulates that the pile installation will be subjected to stringent quality control to ensure that the installation is sound.

18.2.5.1(6) Downdrag in Groups of Vertical Piles

Briaud and Tucker (1997) examined downdrag effects on groups of vertical piles. They indicated that downdrag effects may be approximated by considering the downdrag stresses on the perimeter of the group, unless the piles are very widely spaced.

18.2.5.2 Means for Reducing Downdrag

When the pile settlement is excessive and the solutions, such as those of increasing the pile length or decreasing the pile diameter, are not practical or economical, the downdrag acting on the piles can be reduced by the application of bituminous or other viscous coatings to the pile surfaces before installation (Fellenius, 1975a, 1979). For cast-in-place piles, floating sleeves have been used successfully. Briaud and Tucker (1997) provide some useful provisions for reducing downdrag forces in piles.

18.2.6 Uplift Resistance

Pile foundations must sometimes resist uplift forces and should be checked both for resistance to pullout and their structural ability to carry tensile stresses. The ultimate uplift resistance of a pile is equal to the shaft resistance that can be mobilized along the surface area of the shaft. For bored piles in clay soils, the uplift resistance is commonly assumed to be equal to that contributing to the bearing capacity of the pile as described in Section 18.2.1 (O'Neill & Reese, 1999).

For either bored or driven piles in cohesionless soils, q_s in the uplift is about 75 % to 80 % of its value in compression (El Naggar & Sakr, 2001; O'Neill, 2001). However, for piles with high residual stresses as a result of the driving the actual shaft resistance in uplift (pile in tension) is considerably smaller (about half) compared to the apparent resistance in compression. In such cases, the applied factors of safety should be double those applied in the case of compression. The uplift resistance of tapered piles in cohesionless soils is comparable to the uplift resistance of cylindrical piles with the same average embedded pile diameter (El Naggar & Wei, 2000).

When piles are built primarily to resist uplift forces, the pullout resistance can be increased by providing one or more sections whose diameter is larger than the average pile diameter. Expanded base piles, underreamed and multi-underreamed piles, and screw- piles are typical.

The most reliable way of designing piles subjected to uplift loads is by means of uplift testing. The tests should be designed and carried out in accordance with ASTM designation D3689.

The uplift resistance of a pile group is the lesser of the two following values:

- the sum of the uplift resistance of the piles in the group; and
- the sum of the shear resistance mobilized on the surface perimeter of the group plus the effective weight of soil and piles enclosed in this perimeter.

18.2.7 Other Considerations

18.2.7.1 Axial Capacity Based on Test Loading

The design of piles based on theoretical or empirical methods, as described above, is subjected to some uncertainty including:

- soil properties that cannot be measured with great accuracy and are variable within a building site;
- the correlation between the soil parameters and the bearing capacity of a pile includes a margin of error; and
- the actual driving or installation 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 test-load typical units. General considerations on the use of load tests, the recommended methods of testing, and interpreting the test results are given in Chapter 20.

18.2.7.2 Compacted Concrete (Expanded-Base) Piles

Compacted concrete piles in granular soils derive their bearing capacity from the densification of the soil around the base due to the installation process. The bearing capacity of such piles is, therefore, dependent on the construction method, and the capacity value used should be supported by documented local experience and/or static test loading.

18.2.7.3 Piles Installed by Vibration

Piles may be installed in soils with little cohesion using a vibratory device attached to the top of the pile. This method has two advantages over conventional driving: it is relatively quiet and produces less excessive vibration levels. Installing piles by vibration is facilitated by weakening the soil strength along the pile shaft (likely due to liquefaction) and no densification effect is realized due to the installation.

The capacity of piles installed using vibration can be established using static analysis and using the provisions for bored piles. The capacity of these piles cannot be estimated from driving records and thus, their capacity has to be verified by dynamic analysis of restrike blows to all or a specified percentage of the piles.

18.2.7.4 Augured Cast-In-Place-Piles

The augured cast-in-place (ACIP) or continuous flight auger (CFA) pile system was developed in the USA in the late 1940s. Today, the method is in wide use throughout the world, including Canada. ACIP piles must be installed by an experienced contractor who is familiar with the augercast process and local geology and soil conditions. ACIP piles can be designed as bored piles. At least one pile load test should be conducted to confirm the pile capacity.

18.2.7.5 Soil Set-Up and Relaxation

In some soils, the capacity of driven piles is subject to change with time during or following driving. In dense, saturated, fine-grained soils, such as non-cohesive silts and fine sands, the ultimate capacity may decrease after initial driving. This is known as relaxation. In this case, the driving process is believed to cause the soil to dilate, thereby generating negative pore pressures and a temporary higher strength. When these pore pressures return to normal, the resistance reduces.

On the other hand, temporary liquefaction, which causes a reduced resistance to pile penetration, may also occur in saturated fine sands or silts. The probability of liquefaction is greater in loose sands, but liquefaction can occur even in dense material, if there is a sufficient number of stress cycles, if the magnitude of the stress is large enough, or if the confining pressure is low. After the temporary pore pressures dissipate, long-term capacity is indicated by the return to a higher resistance to pile penetration.

Because the resistance to pile penetration may increase (due to soil set-up), or decrease (due to relaxation), it is essential that re-striking be carried out once equilibrium conditions in the soil have been re-established. The need for re-striking should be recognized in the contract specifications. The time for the return of equilibrium conditions can be determined by trial and error or from pore pressure dissipation tests performed during a pause in the penetration of a cone penetration test where pore pressures are measured (piezo-cone test) (Robertson et al., 1990). The resistance developed in the first five blows of re-striking is generally indicative of the equilibrium resistance.

However, conclusions on soil set-up from re-striking without simultaneous measurement of developed energy and stresses are highly unreliable, and test loading may be required to appraise the final capacity. The effects of soil set-up should be treated with great caution in large pile groups. Also, soil set-up cannot be quantified by re-striking piles that have been driven to a penetration resistance greater than about 2 mm to 3 mm/blow in initial driving.

Piles driven into cohesive soils induce some disturbance, which is a function of:

- the soil properties, in particular its sensitivity to remoulding;
- the geometry of the pile foundation (diameter of piles, number, and spacing of piles in the groups); and
- the driving method and sequence.

The disturbance results in a temporary loss of strength in some soils and a corresponding reduction of support provided by the piles (see Fellenius & Samson, 1976; Bozozuk et al., 1978a; Clark & Meyerhof, 1972a/b). In some cases, such as in soft sensitive clays, complete remoulding of the clay may occur. The effect of the remoulding diminishes with time following driving, as the soil adjacent to the pile consolidates. This results in an increase in the capacity of the pile occurring at a slower rate around a concrete or steel pile as opposed to a wooden pile.

Test loading of a pile in fine-grained soil should not be carried out without knowledge of these processes. It is advisable to delay testing for at least two weeks after driving.

18.2.7.6 Porewater Pressures Induced by Driving

Pile driving in clay generates high porewater pressures, the effects of which are to:

- temporarily reduce the bearing capacity of the piles (and of adjacent piles);
- affect the process of reconsolidation of the clay around the pile, thereby making it necessary to delay the application of the load. Delays of 30 days and more are not unusual (Blanchet et al., 1980);
- drastically alter the natural stability conditions in sloping ground. (There have been a few examples of major landslides triggered by pile-driving operations.)

If necessary, stability can be monitored with instrumentation of the clay layer for measurement of porewater pressures and soil displacements during driving. Alternatively, porewater pressures can be reduced by the use of proper driving techniques and sequences (preboring is an efficient way to reduce porewater pressures and soil displacements); and the use of vertical pre-manufactured drains attached to the surface of the piles, or preferably, installed at the site prior to the pile driving (see Holtz & Bowman, 1974).

18.2.7.7 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, and could result in a reduction in the capacity of these piles. This problem is of particular significance when large pile groups are driven.

18.2.7.8 Construction Effects for Bored Pier

In deep large-diameter excavations for cast-in-place piles, or when the concreting is delayed, significant strength reductions may occur as a result of heave and lateral flow within the excavation. Also, poor slurry construction techniques that leave a thick layer of slurry between the pile and surrounding soil can have a detrimental effect on shaft capacity. These factors should be considered during the design.

18.2.7.9 Penetration Resistance

The penetration per blow (the set) decreases rapidly after a resistance of 5 mm/blow for shaft-bearing piles and 3 mm/blow for toe-bearing piles. There is little justification in requiring sets smaller than 3 mm/blow for a end-bearing pile that may only be warranted if driving is easy in the soil above the bearing stratum, or under special circumstances.

18.3 Settlement of Piles in Soil

18.3.1 Settlement of Single Piles

Many factors influence the settlement of single piles, so it is difficult to make precise estimates of settlement of single piles or pile groups. In general, the shaft resistance is mobilized with very little movement, typically 5 mm to 10 mm, whereas the toe resistance when embedded in soil requires longer movements typically between 5 % and 10 % of the pile diameter. Hence, the actual load-settlement response of a single pile is a function of the relative contributions of shaft and toe resistance, the ground conditions and the method of pile installation. However, a number of empirical and theoretical solutions have been developed that can be used to make reasonable estimates of pile response.

18.3.1.1 Empirical Method

For normal load levels, the settlement of a pile may be estimated from the empirical formula (Vesic, 1970, 1977):

$$S = S_p + S_s \tag{18.20}$$

in which

$$S_s = S_{ss} + S_{st} \tag{18.21a}$$

where

S_p = elastic deformation of pile shaft
S_s = settlement of ground in which the pile is embeded
S_{ss} = settlement of pile caused by load transmitted along the pile shaft
S_{st} = settlement of pile toe caused by load transmitted at the toe

The elastic deformation of the pile shaft is given by:

$$S_P = (Q_{ta} + \alpha_S Q_{Sa})\frac{L}{A_P E_P} \qquad (18.21b)$$

where

Q_{ta} = actual load transmitted to the pile toe (due to applied load)
Q_{sa} = actual shaft load (due to applied load)
α_s = depends on distribution of skin friction = 0.5 for uniform or parabolic distribution and 0.67 for linear distribution
L = total length of the pile
A_p = average cross-section area of the pile
E_p = modulus of elasticity of the pile material

Alternatively, the pile shaft compression can be approximated by:

$$S_P = 0.75\frac{QL}{A_P E_P} \qquad (18.22)$$

where
QL = applied pile load.

The settlement components due to soil deformation are given by;

$$S_{SS} = C_S \frac{Q_{Sa}}{L\, q_t} \qquad (18.23)$$

$$S_{St} = C_t \frac{Q_{ba}}{d\, q_t} \qquad (18.24)$$

where
C_t = empirical coefficient (typical values given in Table 18.6)
d = pile diameter

$$C_S = 0.93 + 0.16\,(L/d)^{0.5} \qquad (18.25)$$

TABLE 18.6 *Typical Values of Coefficient C_t (Vesic, 1977)*

Soil Type	Driven Piles	Bored Piles
Sand (dense to loose)	0.02-0.09	0.09-0.18
Clay (stiff to soft)	0.02-0.03	0.03-0.06
Silt (dense to loose)	0.03-0.05	0.09-0.12

18.3.1.2 Elastic Continuum Solutions

Poulos and Davis (1980) provide a comprehensive set of results for both floating and end-bearing piles. For example, the settlement of a pile in a deep layer of uniform elastic material is expressed as:

$$S = \frac{Q}{E_s d} I_0 R_K R_\upsilon \qquad (18.26)$$

where

E_s = soil modulus
I_0 = settlement influence factor, Figure 18.4
R_K = compressibility correction factor, Figure 18.5
R_v = Poisson's ration correction factor, Figure 18.6

The factors I_0, R_K and R_v are obtained using analysis based on Mindlin's solution for a vertical point load applied within an elastic half-space. They are dependent on pile length to diameter ratio L/d, base diameter d_b, area ratio R_A (ratio of pile section to area bounded by outer pile circumference), pile modulus E_P, and compressibility $K = R_A E_P / E_S$.

Further factors are available to correct the settlement for the effects of end-bearing onto a stiffer soil as well as finite thickness of the soil stratum in which the pile is floating. The nonlinear pile response can also be modelled by taking into account pile-soil slip.

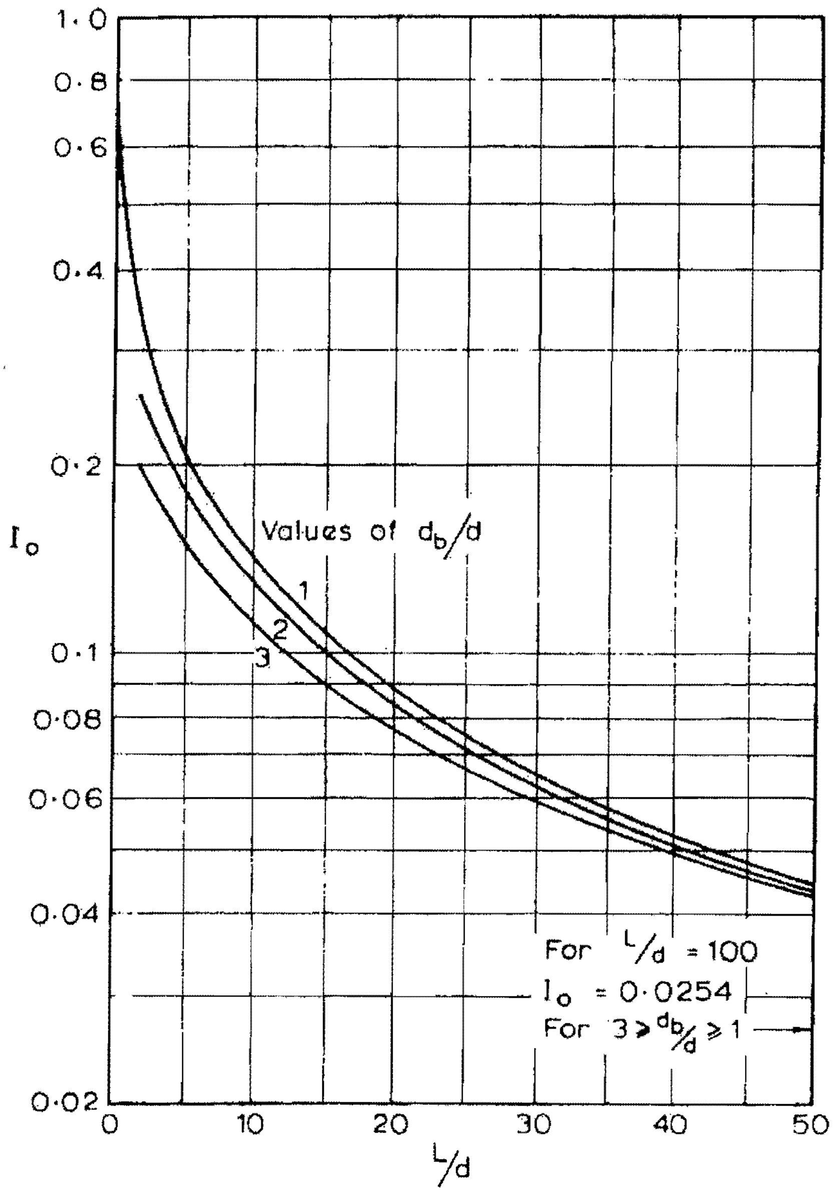


FIGURE 18.4 *Settlement-influence factor, I_0 (after Poulos and Davis, 1980)*

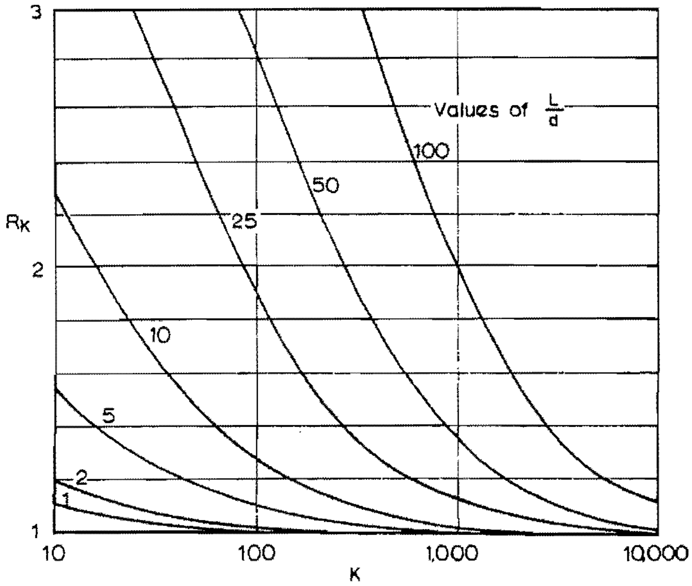


FIGURE 18.5 *Compressibility correction factor for settlement, R_K (after Poulos and Davis, 1980)*

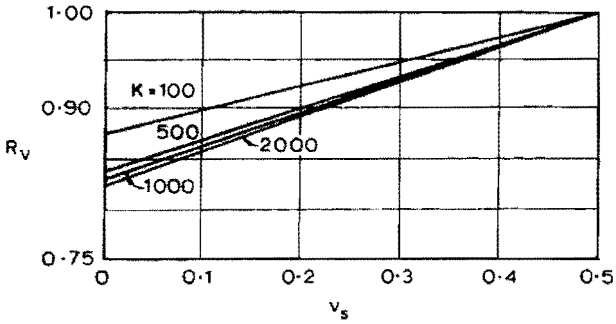


FIGURE 18.6 *Poisson's ratio correction factor for settlement, R_v (after Poulos and Davis, 1980)*

Randolph (Fleming et al., 1992) developed a closed form solution for I_0 for piles in a soil with a modulus that increases linearly with depth, given by

$$I_0 = 4(1+\nu)\frac{\left[1+\frac{1}{\pi\lambda}\frac{8}{(1-\nu)}\frac{\eta_1}{\zeta}\frac{\tanh(\mu L)}{\mu L}\frac{L}{d}\right]}{\left[\frac{4}{(1-\nu)}\frac{\nu_S}{\xi}+\frac{4\pi\rho}{\zeta}\frac{\tanh(\mu L)}{\mu L}\frac{L}{d}\right]} \quad (18.27)$$

in which

$\eta_1 = d_b/d$, d_b is the diameter of the pile toe; $\xi = E_{SL}/E_b$ where E_{SL} is the soil modulus at the pile toe and E_b is the modulus of the bearing stratum underneath the pile toe; $\rho = \bar{E}/E_{SL}$ and

$$\lambda = 2(1+\nu)\frac{E_P}{E_{SL}} \quad (18.28)$$

$$\zeta = \ln\left\{\left[0.25+(2.5\rho(1-\nu)-0.25)\xi\right]\frac{2L}{d}\right\} \quad (18.29)$$

$$\mu L = 2\left(\frac{2}{\zeta\lambda^{0.5}}\right)\frac{L}{d} \tag{18.30}$$

When applying the above elastic solutions, the immediate or undrained settlement (for pile in clay) is calculated with undrained E_{SL} values and $\nu = 0.5$. For total final settlement calculations in sand or clay, drained values of E_{SL} and ν_s are used.

To employ an elastic continuum solution of this type, the soil profile must be simplified appropriately and elastic properties for the soil must be estimated, in particular the secant modulus E_S for working load levels. Poulos and Davis (1980) suggest average values of E_S for driven piles in sands, a value of ν of 0.3 (where no test data are available) and, for driven piles, a value for soil modulus below the pile toe of $5E_S$ to $10E_S$. For clays, Callanan and Kulhawy (1985) indicate that E_S/s_u ranges from 200 to 900, with an average of 500. Greater values may occur for shorter piles where $L < 15d$. Poulos and Davis (1980) also provide an empirical correlation between E_S and s_u for piles in clay. Alternatively, the pile settlement theory can be used to back-calculate representative soil parameters using results from field tests on model or prototype piles. Kulhawy and Mayne (1990) provide a great deal of information regarding the estimation of soil parameters for foundation design.

For layered soil profiles, it is adequate for most practical purposes to replace the layered soil along the pile shaft with an equivalent homogeneous soil, using a weighted average, i.e.:

$$E_S(ave) = \frac{1}{L}\sum_{i=1}^{n} E_i h_i \tag{18.31}$$

where

n is the number of layers and E_i and h_i are the elastic modulus and thickness of layer i, respectively. The modulus of the soil at the pile base may be taken as the average of the soil modulus within a distance equal to $2d_b$ below the pile toe.

It is important to note that the relevant mechanical properties of the soil are modified as a result of pile installation, in particular for driven piles. Consequently, the values of E_S used in design are not equal to values obtained from laboratory tests on intact specimens; typical values derived from experience as mentioned above should be used in the absence of local experience.

Nonlinear Analysis: For floating piles (which derive most of their resistance from shaft friction), linear elastic solutions are generally adequate. However, for end-bearing piles (which derive a substantial proportion of their resistance from the toe), the load-settlement behaviour is strongly nonlinear even at normal working loads. For such cases, Poulos and Davis (1980) developed an approximate procedure that involves the construction of a tri-linear load-settlement curve. In this procedure, the shaft and toe ultimate resistances are estimated and used to construct the load-settlement curve of the pile.

18.3.1.3 Load-Transfer Method

Soil data are measured from field and laboratory tests and presented in the form of curves relating the ratio of adhesion to soil shear strength and to the soil movement. Coyle and Reese (1966) developed the method to estimate load settlement response for the pile. This method accounts for the continuity of the soil mass in an approximate manner as the curves are established from field measurements, which inherently contain the continuity effects. The load transfer method is particularly useful in modeling the load-deformation performance of piles that display strong nonlinear behaviour such as very long compressible piles. O'Neill et al. (1977) extended the method to model the performance of pile groups. A disadvantage of this method is the difficulty in obtaining load- transfer curves at a particular site.

18.3.2 Settlement of a Pile Group

18.3.2.1 Introduction

In groups of closely spaced piles, individual piles interact so that loads applied to any particular pile will lead to the settlement of other piles in close proximity. This interaction leads to an overall increase in pile group settlement and the redistribution of loads on individual piles. Elastic analysis of the pile interaction can be used to establish to what extent the shear resistance of the soil causes an unloaded pile to settle as a result of loads applied to an adjacent pile (e.g., Poulos & Davis, 1980; Randolph, 1987; El Sharnouby & Novak, 1990). These solutions can be used to predict pile group response taking into account the pile cap stiffness and its influence on load distribution within the group.

It is also useful to approximate the pile group as an equivalent single pier, particularly when there is a large number of piles in the group or the influence of an underlying compressible stratum is to be estimated, (e.g., see Terzaghi & Peck, 1967; Poulos & Davis, 1980). However, this has generally been found to predict settlement that greatly overestimates the actual values (uneconomical pile lengths will then result where settlement governs the design).

18.3.2.2 Empirical Methods for Piles in Sand

The settlement of a pile group is evaluated on an empirical basis and it has been found that the methods are less reliable than those used for single piles because of the limited reference data available. For pile groups in cohesionless soil, two empirical methods are available:

Vesic's Method
The ratio of the settlement of the pile group with width, B, to that of the individual pile with diameter, d, (Vesic, 1970) is:

$$\frac{S_{group}}{S_{individual}} = \sqrt{\left(\frac{B}{d}\right)} \tag{18.32}$$

Meyerhof's Method
The settlement of a pile group, S_{group} in millimetres, may be related to the standard penetration N of the soil (Meyerhof, 1976) by:

$$S_{group} = 0.92q\frac{\sqrt{B}}{N}I \tag{18.33}$$

where

- q = equivalent net vertical foundation pressure, in kPa, determined from $q = Q/LB$, where Q is total load transferred to piles, and L and B are the length and width respectively of the plan area of the pile group
- B = pile group width, in metres
- I = an influence factor ranging from 0.5 to 1.0, (refer to Meyerhof, 1976).

18.3.2.3 Empirical Method for Piles in Clay

For the evaluation of the settlement of pile groups in homogeneous clay, Terzaghi and Peck (1967) assumed that the load carried by the pile group is transferred to the soil through an equivalent footing located at one third of the pile length up from the pile toe (Figure 18.7). The load is assumed to spread into the soil at a slope of 2V:lH under the assumption that the equivalent footing is the top of the frustum of a pyramid. The settlement calculation for the equivalent footing then follows the methods described in Chapter 11. The Terzaghi and Peck method usually results in settlement values that greatly overestimate the actual values. Therefore, where settlement considerations govern the design, the method may result in uneconomical pile lengths.

Field tests and long-term settlement observations of piles in the sensitive clays of the St. Lawrence Valley suggest that the assumption of an equivalent footing placed at the lower third-point is not representative of the actual settlement behaviour of a pile group. Blanchet et al., (1980) report that the settlement of a pile group was due mainly to reconsolidation of the clay after driving and to shear creep deformation with little if any consolidation settlement. However, for large pile groups and pile groups supporting bridge abutments the consolidation settlement may become the main source of settlement.

All piles have a neutral plane located at some level in the soil, where an equilibrium exists between the loads on the pile above the neutral plane and the shaft-and-toe resistance below the neutral plane. The loads consist of the service load (dead load, only) and downdrag due to negative skin friction. The negative skin friction is caused by shear creep deformation in combination with the large stiffness difference between the soil and the pile (Fellenius, 1984a). Accordingly, the settlement calculation of a pile group, or of a single pile, in a soil not undergoing consolidation settlement from causes other than from the service load, follows the same approach as given for piles in soil where consolidation settlement from other causes does occur in the soil around the piles.

In clay soils, reconsolidation can take an appreciable time, i.e., more than a year for large pile groups, and the pore-pressure dissipation occurring during the reconsolidation will cause settlement. Therefore, the settlement analysis must include the effect of the reconsolidation of the soil around the piles after the pile driving.

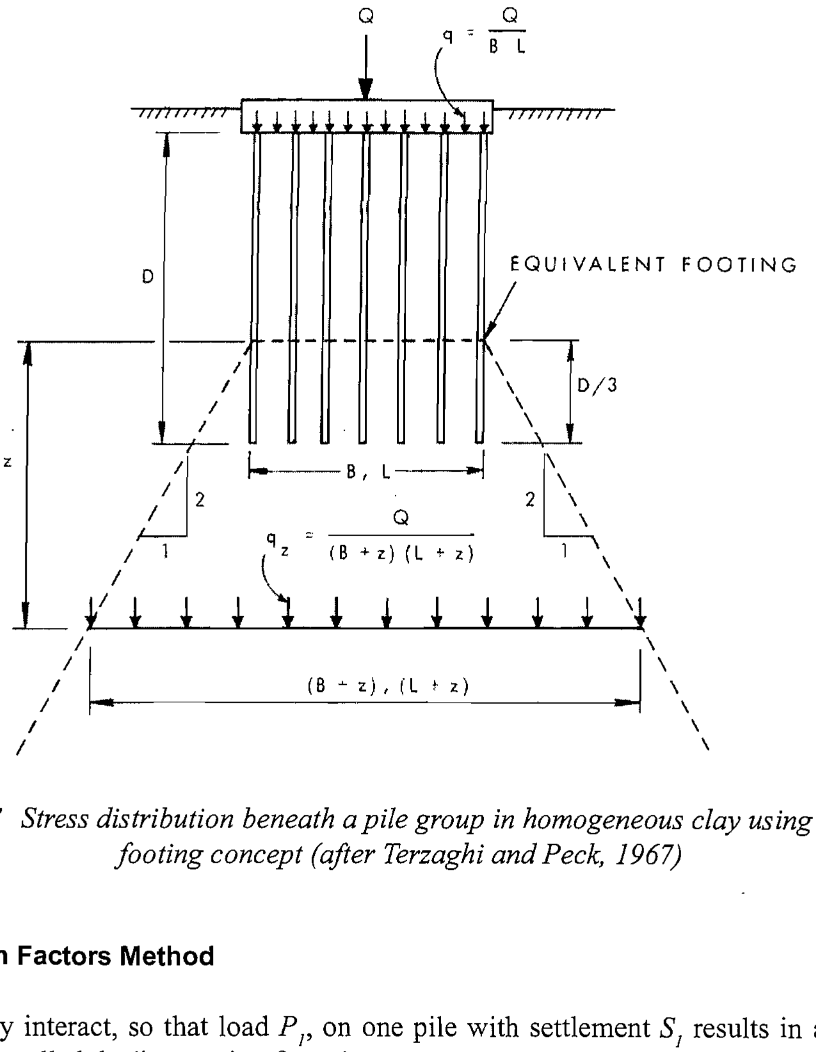


FIGURE 18.7 *Stress distribution beneath a pile group in homogeneous clay using the equivalent footing concept (after Terzaghi and Peck, 1967)*

18.3.2.4 Interaction Factors Method

Piles in close proximity interact, so that load P_I, on one pile with settlement S_I results in a settlement αS_I of an adjacent pile where α is called the 'interaction factor'.

Total settlement of a pile j in a group of n piles:

$$S_j = S_u \sum_{i=1}^{n} P_i \alpha_{ij} \tag{18.34}$$

where

S_j = the settlement of pile j

S_u = the settlement of a pile under unit load, evaluated using one of the procedures from Section 18.3.1

P_i = the load on pile i

α_{ij} = the interaction factor relating settlement of pile j to load on pile i. They are found using elastic theory, provided in Figure 18.8 for floating piles from Poulos and Davis (1980). Other solutions are available for end-bearing piles.

An accurate analysis of settlement of pile groups, based on elastic theory has to be done using a suitable computer program, i.e., Poulos and Randolph (1982); El Naggar and Novak (1990). The methods based on elastic theory, however, should not be used in situations involving downdrag, creep or significant deep-seated settlement. Furthermore, it is only applicable within the working load level.

18.3.2.5 Pile Cap Conditions

Two simplified pile cap conditions can be examined using the general settlement equation shown above:

- A rigid pile cap, where all piles settle an equal amount but loads on individual piles are not known.
- A flexible pile cap, where the loads on each pile are known and each pile has different settlement.

The flexible pile cap problem is solved by using the settlement equation directly. The rigid pile cap problem is solved using the n general equations (one for each pile) and the known total load applied to the pile group, which is the sum of the individual loads:

$$P_{tot} = \sum_{i=1}^{n} P_i \tag{18.35}$$

There are then n+l equations with n+l unknowns (P_1, P_2, ---- P_n, S). In addition to the group settlement S, the individual pile loads are evaluated.

18.4 Lateral Capacity of Piles in Soil

Vertical piles resist lateral 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 stiffness of the pile and the strength of the soil.

The horizontal load capacity of vertical piles may be limited in three different ways:

- the capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
- the bending moments and/or shear may generate excessive bending or shear 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. There is much room for improvement of these design methods, and often the best method is still the one based on well-planned and well-executed lateral test loading.

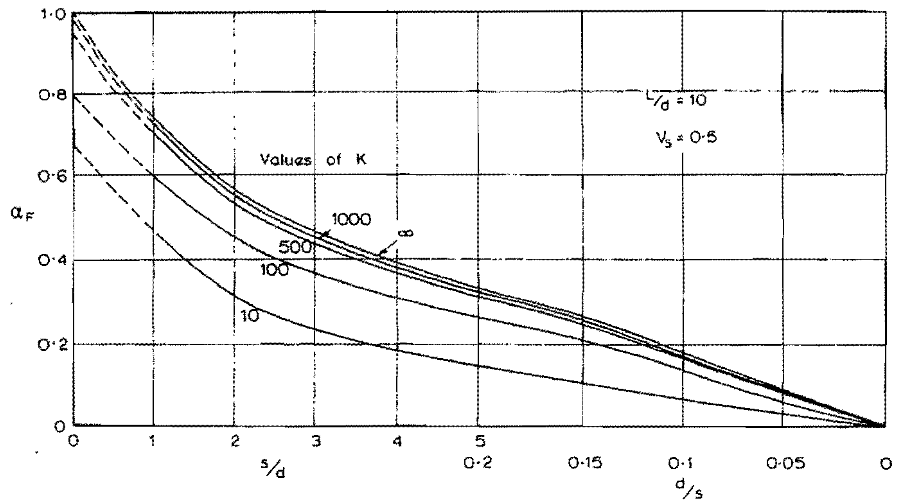


FIGURE 18.8a *Interaction factors for floating piles, L/d=10 (after Poulos and Davis, 1980)*

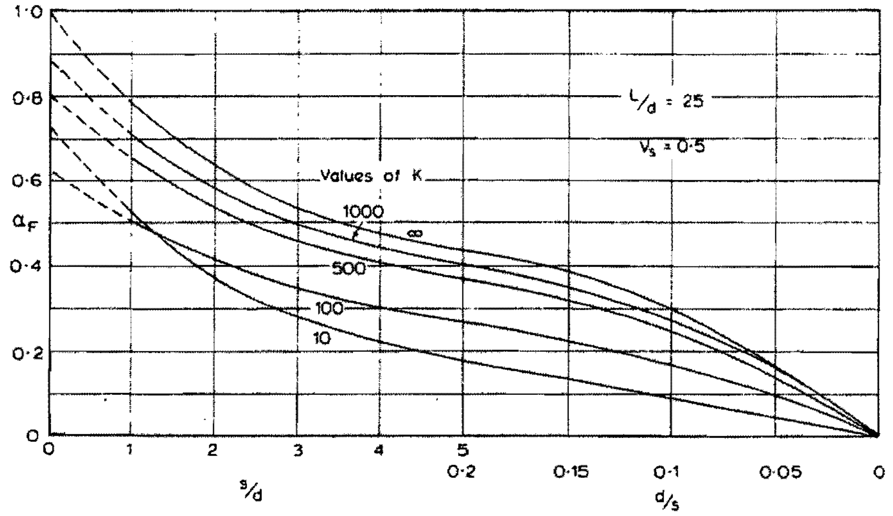


FIGURE 18.8b *Interaction factors for floating piles, L/d=25 (after Poulos and Davis, 1980)*

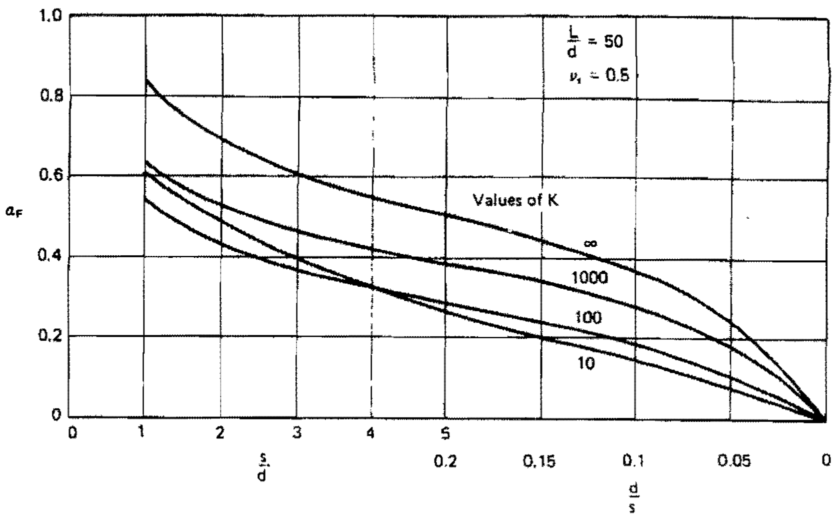


FIGURE 18.8c *Interaction factors for floating piles, L/d=50 (after Poulos and Davis, 1980)*

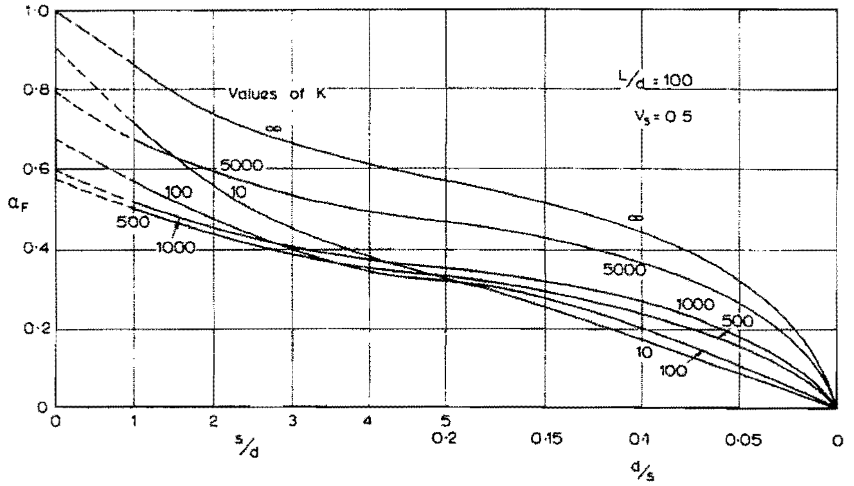


FIGURE 18.8d *Interaction factors for floating piles, L/d=100 (after Poulos and Davis, 1980)*

18.4.1 Broms' Method

Various static analyses of lateral load capacity have been reported, including those of Brinch-Hansen (1961). Broms (1964a,b) has presented solutions in graphical form (see Figures 18.9 and 18.10) for uniform clay and sand strata. In each case, two types of pile failure are examined:

- 'short' pile failure where the lateral capacity of the soil adjacent to the pile is fully mobilized; and
- 'long' pile failure where the bending resistance of the pile is fully mobilized.

Solutions are based on a number of simplifying assumptions that cover the magnitude of lateral soil pressures and their distribution along the pile. Results are given for: a pile of diameter d and embedded length, L; lateral load capacity H_u; yield moment of pile, M_{yield}; clay cohesion, c_u; coefficient of passive sand resistance, K_p; height of lateral load above groundline, e; and soil unit weight, γ.

Poulos (1985) has extended Broms' solutions to consider lateral load capacity for piles in layered clay soils.

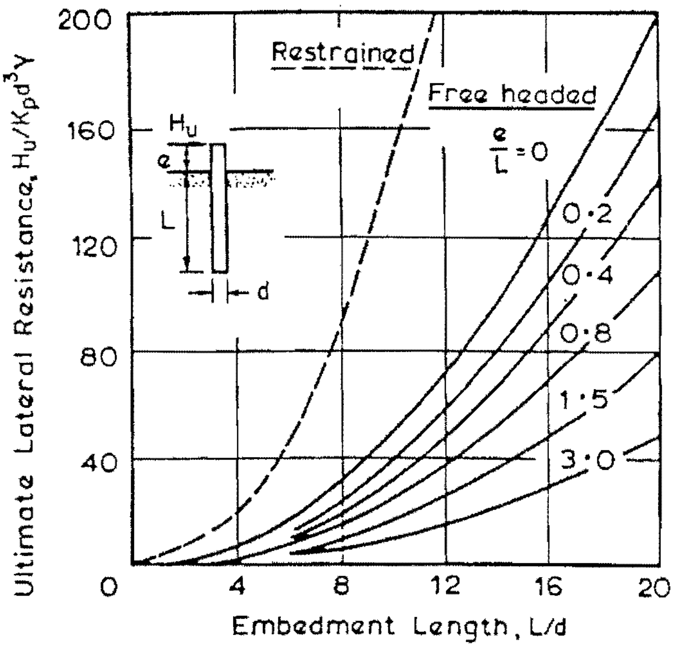
18.4.2 Pressuremeter Method

Considering the close analogy between the behaviour of soils around a horizontally loaded pile and around a pressuremeter probe, an empirical method for determining horizontal resistance R_h from pressuremeter test results has been proposed by Menard (1962). According to this method, the ultimate horizontal resistance of a short head-restrained pile may be expressed by:

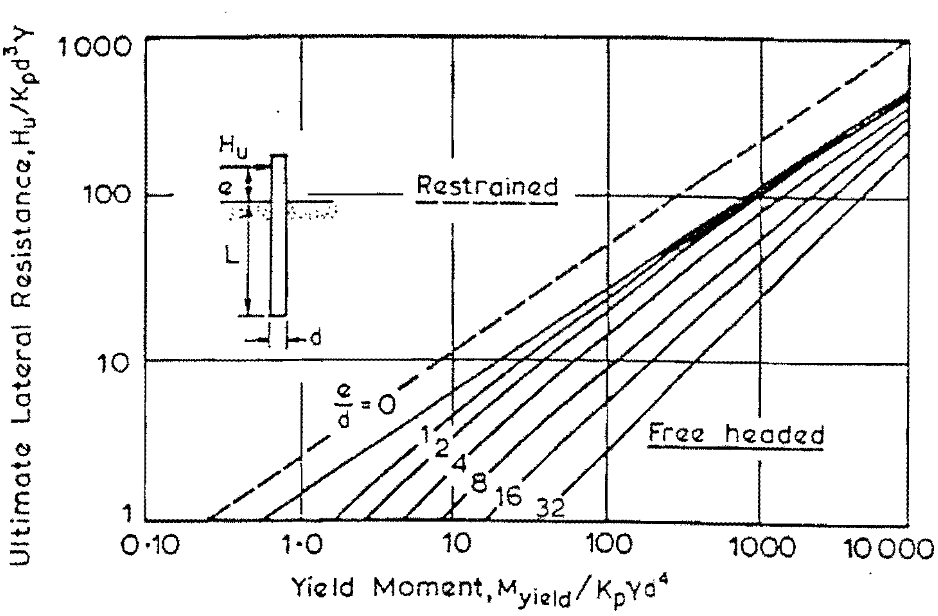
$$R_h = P_l d\,(D-d) \tag{18.36}$$

where

R_h = ultimate horizontal resistance of pile
P_l = limit pressure from pressuremeter test
D = embedment depth of pile
d = pile diameter



a) Short Pile



b) Long Pile

FIGURE 18.9 *Ultimate lateral resistance of piles in cohesionless soils (after Broms, 1964b)*

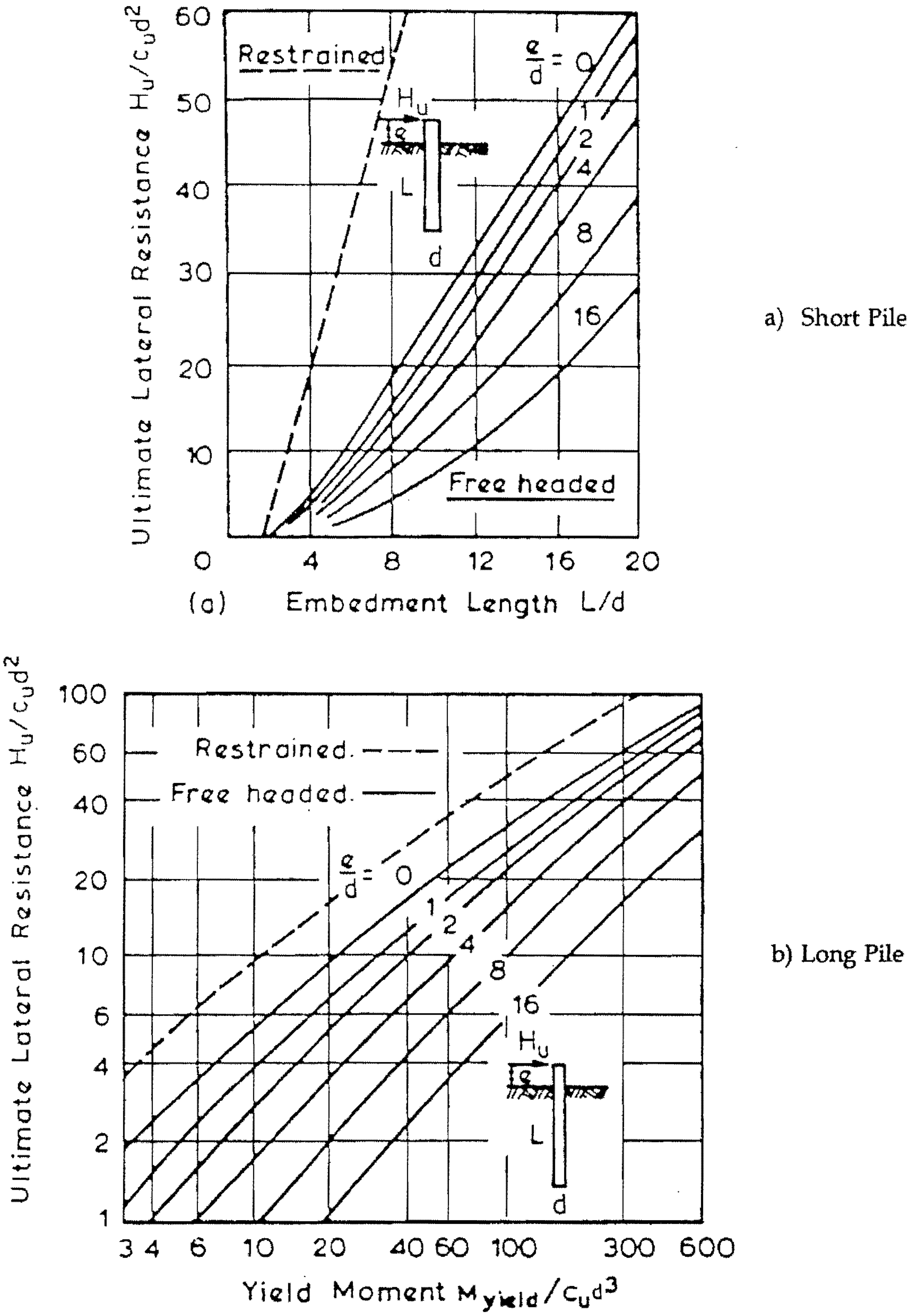


FIGURE 18.10 *Ultimate lateral resistance of piles in cohesive soils (after Broms, 1964a)*

18.5 Lateral Pile Deflections

The response of a pile to lateral loads is highly nonlinear and methods that assume linear behaviour (e.g., theory of subgrade reaction and theory of elasticity) are appropriate only where maximum pile deflections are small (less than 1 % of the pile diameter), where the loading is static (no cycling) and where the pile material is linear (e.g., steel). In most practical applications, one or more of these conditions are not met and methods that can model the pile and soil non-linearity are called for.

The *p-y* curves (unit load transfer curves) approach (see Reese et al., 1974) is a widely accepted method for predicting pile response under static loads because of its simplicity and practical accuracy. The method allows the analysis of a pile's response to lateral static, cyclic or even transient loads (El Naggar and Bentley 2000). The method is briefly described in the following section.

18.5.1 The *p-y* Curves Approach

Based on model tests, *p-y* curves relate pile deflections to the corresponding soil reaction at any depth (element) below the ground surface. The *p-y* curve represents the total soil reaction to the pile motion. It represents the relationship between the static soil reaction, p, and the pile deflection, y, for a given *p-y* curve at a specific load level. The *p-y* curves are established using empirical equations (Matlock, 1970; Reese & Welch, 1975; Reese et al., 1975). The shape of the *p-y* curve can be estimated based on laboratory results and back calculation of field performance data (Matlock, 1970; Murchison & O'Neill, 1984; Gazioglu & O'Neill, 1984) or based on in-situ test results (Baguelin, et al., 1978; Briaud, et al., 1983; Robertson, et al., 1986) or curve fit to measured strain data using an accepted method such as the modified Ramberg-Osgood model (Desai & Wu, 1976).

The general procedure for computing *p-y* curves in clays both above and below the groundwater table and corresponding parameters are recommended by Matlock (1970) and Bhushan et al. (1979), respectively. The *p-y* relationship was based on the following equation:

$$\frac{p}{P_u} = 0.5\left(\frac{y}{y_{50}}\right)^n \tag{18.37}$$

where

- p = soil resistance
- y = deflection corresponding to p
- n = a constant relating soil resistance to pile deflection
- y_{50} = corrected deflection at one-half the ultimate soil reaction determined from laboratory tests.
- P_u = ultimate soil resistance, is the minimum of:

$$P_u = 3s_u + \gamma xd + Js_u x \tag{18.38a}$$

or

$$P_u = 9s_u d \tag{18.38b}$$

where

- s_u = the undrained shear strength
- γ = the effective unit weight of the soil
- J = an empirical coefficient dependent on the shear strength. A value of $J = 0.5$ is typically used for soft clays (Matlock 1970) and $J = 1.5$ for stiff clays (Bhushan et al. 1979).

The most commonly used criteria for development of *p-y* curves for sand were proposed by Reese et al. (1974) but tend to give very conservative results. Bhushan et al. (1981) and Bhushan and Askari (1984) used a different procedure based on full-scale load test results to obtain nonlinear *p-y* curves for saturated and unsaturated sand. Bhushan and Haley (1980) and Bhushan et al. (1981) developed *p-y* curves for different sands below and above the water table. The secant modulus approach is used to approximate soil reactions at specified lateral displacements. In this approach, the soil resistance in the static *p-y* curve model can be calculated using the following equation:

$$p = (k)(x)(y)(F1)(F2) \tag{18.39}$$

where

k = a constant that depends on the lateral deflection y (i.e., k decreases as y increases) and relates the secant modulus of soil for a given value of y to depth ($E_s=kx$)

x = is the depth at which the p-y curve is being generated

$F1$ and $F2$ = are density and groundwater (saturated or unsaturated) factors, respectively, and can be determined from Meyer (1979)

The main factors affecting k are the relative density of the sand (loose or dense) and the level of lateral displacement. The secant modulus decreases with increasing displacement and thus the nonlinearity of the sand can be modeled accurately. This analysis assumes a linear increase of the soil modulus with depth (but varies nonlinearly with displacement at each depth) that is typical for many sands.

The actual soil response is a function of the pile installation and soil type. Methods used to estimate the non-linear p-y curves do not always account for changes in ground conditions due to pile installation. Some techniques have been proposed whereby a pressuremeter is installed in a manner that simulates the pile installation and the non-linear p-y curve determined from the subsequent pressuremeter test (Robertson, et al., 1986).

Various methods for modeling laterally loaded piles that employ the p-y curve method or the strain wedge method (Ashour et al., 1998; Ashour & Norris, 2000) are encoded in computer programs that are available on the market and are efficiently used to analyse the nonlinear lateral response of piles. Most of these computer programs account for soil and pile nonlinearity and can handle static, cyclic or transient loading. Furthermore, they calculate the bending moment and shear forces along the pile shaft, which are required for the structural design of the pile. Some of the available programs are LPILE (Reese & Wang, 1997), SWM (Ashour et al., 1998) and FLPIER (McVay et al., 1992). For cases where the load is transient (impact loading, seismic loading, etc.), PYLAT (El Naggar & Bentley, 2000) can be used.

18.5.2 Elastic Continuum Theory

Poulos and David (1980) present solutions for the lateral deflection of a single pile floating within an elastic continuum responding to a lateral load, H, applied at distance, e, above the groundline. These solutions make use of soil modulus, E_s and are presented in Figures 18.11 to 18.13 for Poisson's ratio of the ground $\nu = 0.5$. Groundline displacement, ρ, and groundline rotation, θ, are expressed as:

$$\rho = \frac{H}{E_S L}\left(I_{\rho H} + \frac{e}{L} I_{\rho M}\right) \tag{18.40a}$$

$$\rho = \frac{H}{E_S L^2}\left(I_{\theta H} + \frac{e}{L} I_{\theta M}\right) \tag{18.40b}$$

where

the pile has embedded length, L, and the influence factors $I_{\rho H}$, $I_{\rho M}$, $I_{\theta H}$ and $I_{\theta M}$ are given in Figs. 18.11 to 18.13. These particular solutions are for a uniform soil and elastic pile, and use the pile flexibility factor, $K_R = \frac{E_P I_P}{E_S L^4}$ where the pile has modulus, E_p, and second moment of area, I_p. The soil modulus used in these solutions should be calibrated for a given pile type, magnitude of load, and nature of load (static, cyclic or trasient) through site-specific loading tests whenever possible.

There are other solutions for a pile that yields and for a non uniform soil profile (Poulos & Davis, 1980). Nonlinear pile response has been examined by Poulos (1982).

18.5.3 Group Effects

The solutions presented in the preceding sections can be used to estimate the lateral response of single piles. When piles are installed as a group, interaction occurs between the individual piles so that the lateral pile deformations are increased. This effect can be quantified using theoretical solutions of Poulos and Davis (1980) so that the pile group response can be estimated. (See also the work of Randolph, 1981 and El Sharnouby & Novak, 1985). A number of computer programs that employ mainly linear elastic pile and soil models are available for the response analysis of pile groups such as PGROUP, DEFPIG and PIGLET (Fleming et al., 1992).

For laterally loaded pile groups, the direction of the applied load relative to the group becomes important, particularly for groups driven in a rectangular configuration where the rectangle length is substantially greater than the rectangle breadth.

The proper evaluation of the lateral performance of pile groups requires an approach that accounts for the soil nonlinearity, especially near the ground surface. Budhu and Davies (1987, 1988) and El Naggar and Novak (1996) have examined the nonlinear pile group response. The most common design method for laterally loaded pile groups is based on the *p-y* curve approach. In this method, piles within the group are analysed for lateral loading per single piles except that the *p* values are multiplied by a reduction factor termed the *p*-multiplier (Brown et al., 1988; Brown et al., 2000; Mostafa & El Naggar, 2002). Computer programs are available to facilitate the analysis and design of laterally loaded pile groups (FLPIER (McVay et al., 1996); GROUP (Reese & Wang, 1996; PYLATG (El Naggar & Mostafa, 2001)).

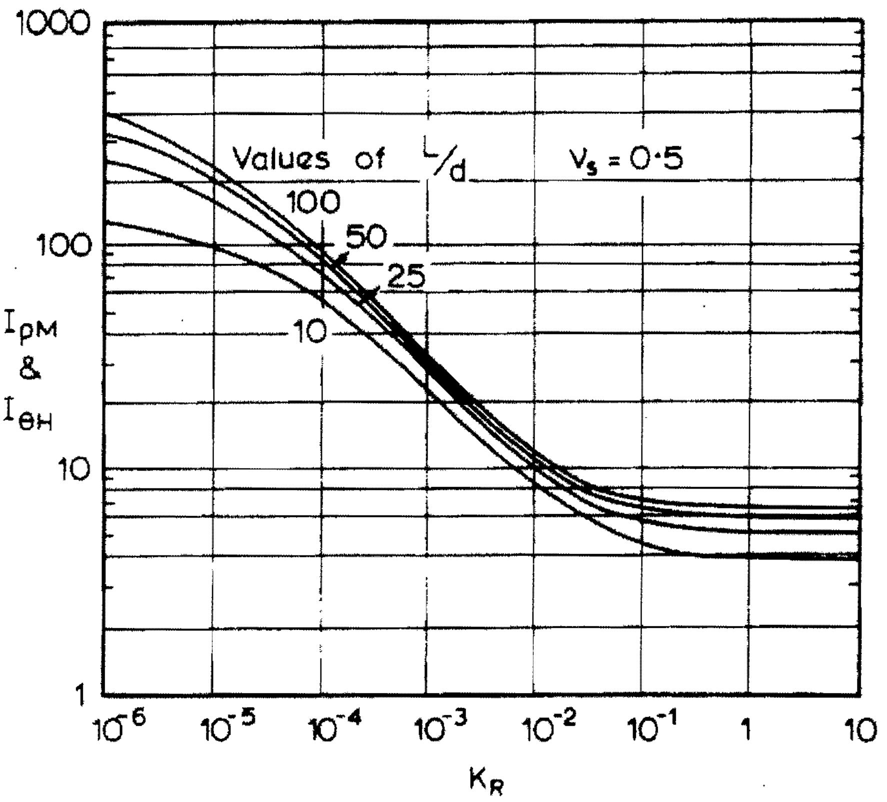


FIGURE 18.11 *Values of* $I_{\rho M}$ *and* $I_{\theta H}$ *– free-head floating pile, constant soil modulus (after Poulos and Davis, 1980)*

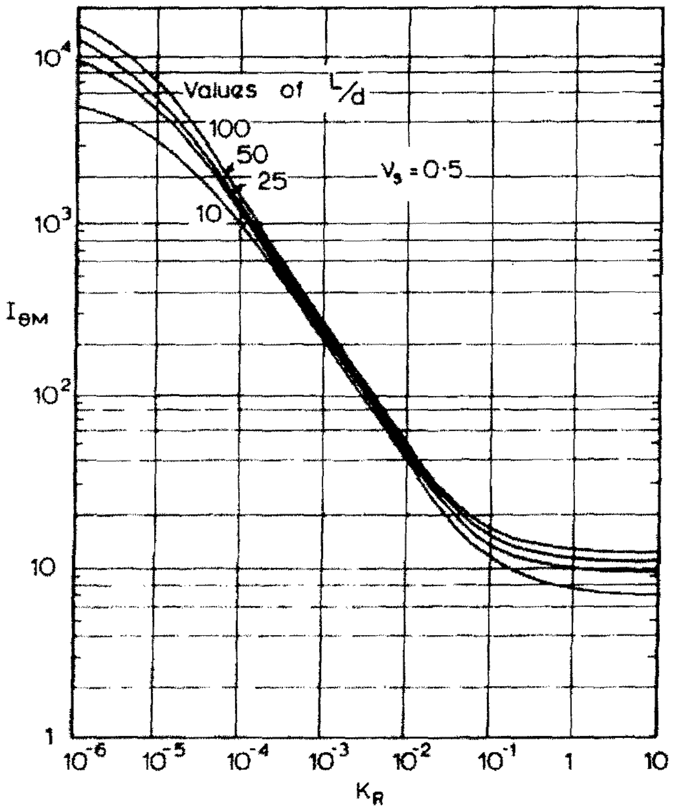


FIGURE 18.12 *Values of* $I_{\theta M}$ *– free-head floating pile, constant soil modulus (after Poulos and Davis, 1980)*

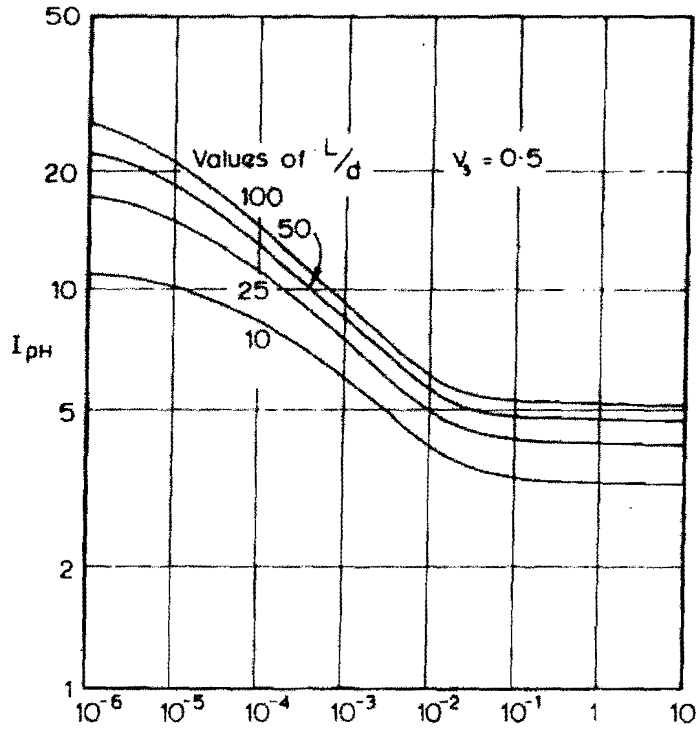


FIGURE 18.13 *Values of* $I_{\rho H}$ *– free-head floating pile, constant soil modulus (after Poulos and Davis, 1980)*

18.6 Geotechnical Axial Capacity of Deep Foundations on Rock

18.6.1 Introduction

Deep foundations placed on or socketed into rock normally carry heavy loads. They may be used when the quality of the rock mass at the surface is poor. They may be driven, drilled, or cast-in-place. Carter and Kulhawy (1988) and Lo and Hefny (2001) provide a useful review of analysis and design methods for piles socketed into rock.

Piles can be driven onto or into rock. However, the exact area of contact with rock, the depth of penetration into the rock, and the quality of the rock at the foundation level are largely unknown. Consequently, the determination of the capacity of such foundations using theoretical or semi-empirical methods cannot be made with certainty. Therefore, the capacity should be confirmed on the basis of driving observations, local experience and test loading.

18.6.2 Drilled Piers or Caissons - Design Assumptions

Deep foundations can be drilled, bored or excavated, and cast-in-place. In this case, the area of contact with the rock, the depth of penetration into the rock, and the quality of the rock at the foundation level can be verified. Therefore, the capacity of these foundations may be determined with a reasonable degree of confidence using various design methods. The following discussion relates to the axial capacity of the socketed piers. The behaviour of foundations under lateral load is discussed by Poulos and Davis (1980), Kulhawy and Carter (1992), Carter and Kulhawy (1992) and Wyllie (1992).

In most cases, where cast-in-place deep foundations are socketed into the rock, the depth of the socket is typically one to three 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:

1. The capacity is assumed to be derived from toe resistance only. This assumption can be considered to be safe, since the capacity of the rock is available, regardless of the construction procedure. However, if the bottom of the excavation is not properly cleaned, the capacity may not be mobilized before large settlements occur owing to the compression of any debris remaining in the bottom of the socket.

2. The capacity is assumed to be derived from the "bond" between concrete and rock along the surface perimeter of the socket. However, theoretical considerations indicate that the load distribution is not necessarily uniform, but depends upon the modulus of elasticity of both concrete and the surrounding rock (Coates, 1967; Williams et al., 1980). Furthermore, the magnitude of shaft resistance, or "bond", is highly dependent on the quality of the rock surface on the walls of the socket and on the roughness of the rock face.

3. The capacity is assumed to be derived from both toe resistance and shaft resistance. In this case, consideration must be given to the load transfer behaviour of the pier-socket system. Verification of the design load by full-scale test and/or well-documented local experience is recommended.

18.6.3 End-Bearing

18.6.3.1 Introduction

Toe or end-bearing resistance is the area of the socket base multiplied by the bearing pressure. The socket base capacity may be considered to provide the whole socket capacity (Approach 1 above) or to provide one component of the socket capacity (Approach 3).

18.6.3.2 Bearing Pressure from Pressuremeter Results

In situ pressuremeter tests may be useful in the determination of rock mass properties. The pressuremeter limit pressure, p_l, serves as a strength index of the rock mass. The ultimate capacity of a socketed pile in rock, R_u is given by the following equation:

$$R_u = K_b (p_l - p_o) + \sigma_o \quad (18.41)$$

where

p_l = limit pressure as determined from pressuremeter tests in the zone extending two pile diameters above and below the pile toe

p_o = at rest horizontal stress in the rock at the elevation of the pile toe

σ_o = total overburden pressure at the toe of the pile

K_b = an empirical non-dimensional coefficient, which depends on the socket diameter-depth ratio as follows:

TABLE 18.7 *Bearing Capacity Coefficient K_b as a Function of Normalized Depth*

Depth / Diameter	0	1	2	3	5	7
K_b	0.8	2.8	3.6	4.2	4.9	5.2

The allowable bearing pressure in working stress design is usually taken as the bearing capacity, R_u, divided by a safety factor of 3.

The factored geotechnical axial resistance at ultimate limit states is taken as the ultimate axial capacity (R_u) multiplied by the geotechnical resistance factor (Φ) of 0.4 for compression and 0.3 for uplift (Tables 8.1 and 8.2 in Chapter 8).

18.6.3.3 Bearing Pressure from Strength of Rock Cores

The method described in Chapter 9 of this Manual is applicable to deep foundations. According to Ladanyi and Roy (1971) the effect of depth is included and the formula becomes:

$$q_a = \sigma_c K_{sp} d \quad (18.42)$$

where

q_a = allowable bearing pressure

σ_c = average unconfined compressive strength of rock core, from ASTM D2938

K_{sp} = empirical factor, as given in Section 9.2 and including a factor of safety of 3

d = depth factor = $1 + 0.4\frac{L_s}{B_s} \leq 3$

L_s = depth (length of the socket)

B_s = diameter of the socket

For limit states design, it is suggested that the ultimate axial capacity be calculated as multiplying the allowable value by three. The factored geotechnical resistance at ultimate limit states would then be obtained by multiplying the ultimate capacity by the geotechnical resistance factor of 0.4 and 0.3 for compression and uplift conditions respectively (Tables 8.1 and 8.2 in Chapter 8).

The uniaxial compression strength is not representative of the in-situ mechanical properties of the rock mass because of the absence of discontinuities in the laboratory test specimens. For such a rock mass, the conventional bearing capacity equation may be used, provided relevant strength parameters have been evaluated from in-situ tests or estimated on the basis of a rock mass classification system as discussed in Chapter 3 of this Manual. Note that the bearing capacity equation usually furnishes an upper bound capacity value.

This method is generally not applicable to soft stratified rocks, such as shales or limestones.

18.6.4 Shaft Capacity of Socket

18.6.4.1 Introduction

Analytical studies of measurements from test loading of drilled piers socketed into bedrock have indicated that socket shear can account for a large portion of the total capacity. The ultimate socket shear load, Q_s, is approximately given by the following relationship:

$$Q_s = \pi B_s L_s q_s \tag{18.43}$$

where
- B_s = diameter of the socket
- L_s = length of the socket
- q_s = average unit shear resistance along the socket

This shaft capacity may be taken as the whole of the socket capacity (Approach 2 in Section 18.6.2) or as part of it (Approach 3). The mechanism of shear strength development is complex, depending upon the frictional and adhesive strength of the rock-concrete bond, as well as any changes in normal stresses acting between rock and concrete due to dilation associated with interface slip or Poisson's ratio effects. Empirical data are currently used to assess the shear strength of the rock-concrete interface. The factored geotechnical axial resistance at ultimate limit states is obtained by multiplying the ultimate capacity by an appropriate value of the geotechnical resistance factor (Tables 8.1 and 8.2 in Chapter 8).

18.6.4.2 Conventional Piers

Piers that are excavated and constructed using conventional methods have a relatively smooth concrete-rock interface. Horvath (1982), Horvath et al (1983), and Rowe and Armitage (1984) have examined the relationship between unit socket shear and the compressive strength of the rock. An approximate relationship has been developed:

$$\frac{q_s}{P_a} = b\left(\frac{q_u}{P_a}\right)^{0.5} \tag{18.44}$$

where
- q_s = unit socket shear
- q_u = unconfined compressive strength of rock
- b = an empirical factor
- P_a = atmospheric pressure

If, the concrete compressive strength, f_c', is lower than the unconfined compressive strength of the rock, q_u, then:

$$q_a = 0.05 f_c'$$

Values for the empirical factor, b, have been proposed, as follows:

TABLE 18.8 *Proposed b Values for Different Design Treatments*

Proposed Value for *b*	Comment	Reference
1.41	Expected average value, for use in limit states design approach	Rowe and Armitage (1984)
0.63 to 0.94	Conservative lower bound value, for use in a working stress design approach	Horvath et al (1983)
0.63	Conservative lower bound value	Carter and Kulhawy (1988)

The range in proposed values for the empirical factor reflects the wide variability in test results. Lo and Hefny (2001) note that the differences between the proposed factors are in part due to the difference in the design approach proposed by the authors as noted in the preceding table. Other methods for estimating side shear resistance are discussed by Lo and Hefny (2001).

Given the large variability in the test data used to determine the empirical method discussed above, it is important that in situ testing for direct measurement of side shear resistance be made for projects where this factor is of critical importance.

18.6.4.3 Grooved Piers

Grooves can be made in the socket wall to increase the roughness of the pier-rock interface and thus, increase the shaft resistance. Using the expression from the preceding section, a best-fit to data as assessed by Rowe and Armitage (1984) is $b = 1.9$ for grooves of depth and width greater than 10 mm, at spacings between 50 mm and 200 mm.

18.6.5 Design for Combined Toe and Shaft Resistance

If both toe and shaft resistance are to be used for estimating socket capacity, then the proportions of load carried at the sides and base must be estimated. This requires some analysis of the socket/rock system.

Pells and Turner (1979) have analysed the socket/rock system, assuming the concrete and rock materials respond as elastic isotropic solids, and that the bond along the rock-concrete boundary must not be broken. A proportion, n, of load reaching the socket base can be estimated from Figure 18.15. Assuming uniform shear over the shaft of the socket, socket shear q_s is:

$$q_s = \frac{(1-n)Q}{L_s b_s} \tag{18.45}$$

where

n = proportion of Q_u that reaches the socket base, from Figure 18.14
Q = total load applied at the top of the socket
L_s = socket length
b_s = socket diameter

This side shear, q_s, must be compared with allowable values to ensure stability. Generally, the base load, nQ, will be easily supported at the toe, but this can be checked using q_a in the formula in Section 18.6.3.3 and the base area.

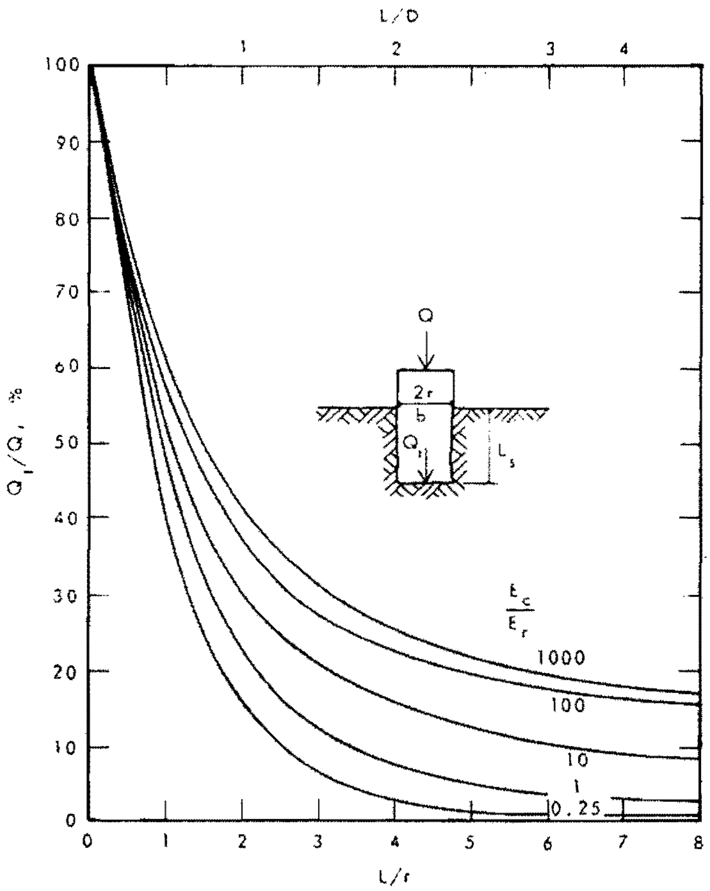


FIGURE 18.14 *Load distribution in a rock socket (after Pells and Turner, 1979)*

18.6.6 Other Failure Modes

In addition to the failure modes discussed, the designer may need to consider the axial uplift capacity of the rock-socketed pile, its lateral capacity or its torsional capacity. Alternatively, design of the socket allowing for both shaft and tip resistance but permitting slip along the shaft-rock boundary may be contemplated. These situations are examined by Carter and Kulhawy (1988).

18.7 Settlement of Piers Socketed into Rock

18.7.1 Fundamentals

The settlement for a pier founded on sound rock is generally negligible. Settlement may be significant for piles on soft rock. Significant settlement of foundations on rock is often associated with the presence of open joints, or seams of compressible material. Because of the discontinuous nature of a rock mass, settlement analysis of drilled pier foundations placed on, or socketed into, rock is difficult. Where such conditions are anticipated, special investigations and analysis and/or test loading are required.

Settlement may also result from the presence of mud or 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.

Elastic moduli measured on rock core samples have little relation to the settlement behaviour 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.

18.7.2 Settlement Estimated from Pressuremeter Testing

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 the pressuremeter modulus of the rock mass, including some measure of the influence of joints and other discontinuities. However, the effect of occasional thin horizontal joints and compressible seams cannot be taken into account using this method, and the results may be misleading if such joints or seams do occur. On the other hand, in highly fractured rock, pressuremeter tests may be the best to provide reliable results.

18.7.3 Settlement from Plate Test Loading

The in-situ plate test loading can 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 must be recognized. Ideally, the plate should be of the same diameter as the foundation unit; however, for practical reasons, this is seldom possible and smaller plates are normally used. The results obtained from loading smaller plates may be considered representative of the actual foundation behaviour, provided the diameter of the plate is not smaller than half the diameter of the foundation unit, and always larger than 0.3 m.

The results of plate load tests are frequently variable. The evaluation of the test results can be performed using the three-dimensional elastic displacement approach (see Section 11.3). To obtain a reliable evaluation of the foundation behaviour, a series of tests has to be carried out (see Rowe, 1982). The cost of such tests and of the resulting design work is high. It is only justified for large projects, or where the structure to be supported is very sensitive to settlement.

18.7.4 Settlement using Elastic Solutions

18.7.4.1 Introduction

In cases where settlement is important, design methods based on elastic solutions have been proposed by Ladanyi (1977), Pells and Turner (1979), Horvath et al (1983), Rowe and Armitage (1987) and others. An excellent summary of elastic design is given by Lo and Hefny (2001).

18.7.4.2 Determination of Material Properties

The subsurface conditions at the proposed site should be thoroughly investigated. The material properties of the concrete and rock should be carefully determined using appropriate laboratory and in-situ testing methods. Representative values of rock mass modulus, E_r, and average shaft resistance at yield are necessary.

Based on back analysis of pile load test data, Rowe and Armitage (1987) proposed the following approximate relationship for rock mass modulus for use in settlement calculations:

$$\frac{E_r}{P_a} = b\left(\frac{q_u}{P_a}\right)^{0.5} \qquad (18.46)$$

where

- E_r = rock mass modulus
- q_u = unconfined compressive strength of rock
- b = an empirical factor
- P_a = atmospheric pressure

The best fit to available data was obtained for $b = 680$. Based on a statistical study, Rowe and Armitage (1987) concluded that the probability of exceeding design settlement could be 30 % if a value of $b = 475$ was used, and 11 % if a value of $b = 340$ was used.

The most direct method of determining value of rock mass properties for design calculations is to perform field tests on full-scale or small-scale socketed piers. It is important that the roughness factor of the test sockets be comparable to the actual pier sockets. Carter and Kulhawy (1988) discuss various aspects of field tests and their interpretation.

18.7.4.3 Estimation of Settlement of the Pier

Once the pier dimensions have been determined, the settlement of the pier can be calculated using Figures 18.15 to 18.17. If the calculated settlement values exceed the allowable settlement, the diameter and/or length of the pier socket should be adjusted. These solutions assume that the rock and concrete remain bonded together along the socket shaft.

18.7.4.4 Other Methods

Carter and Kulhawy (1988) provide a comprehensive review of methods for prediction of load-displacement response of axially loaded piers. The nonlinear solutions of Rowe and Armitage (1987) can be used to predict axial deformations in soft rock, both before, and after, slip occurs along the pier shaft. Alternatively, Carter and Kulhawy (1988) provide analytical solutions that give pier response before first slip, and with full slip along the rock-pier boundary.

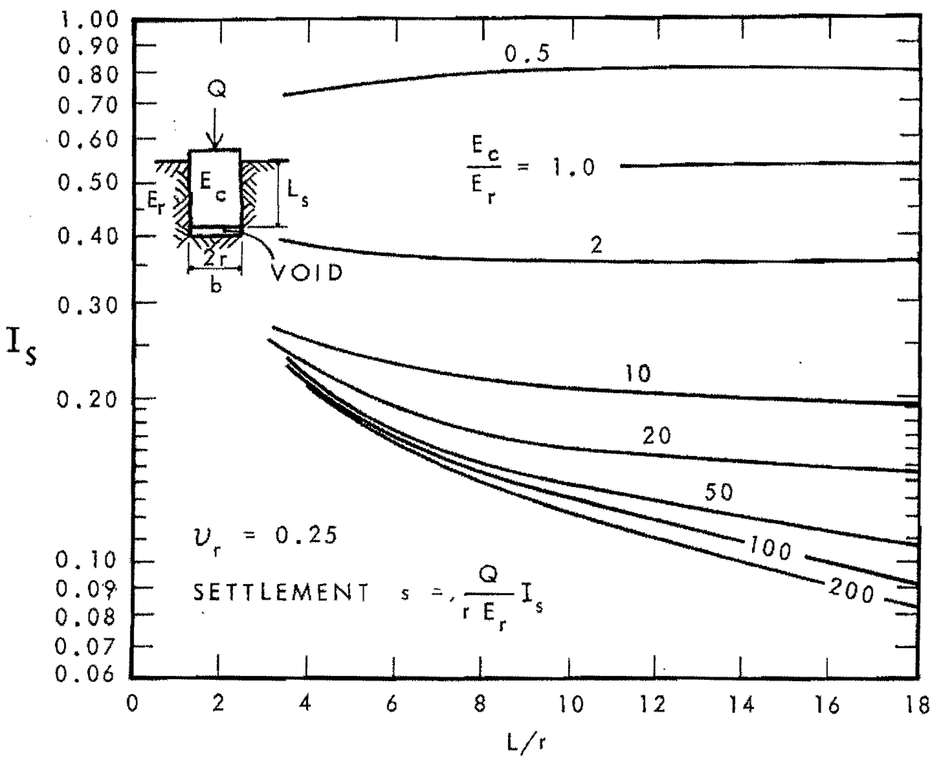


FIGURE 18.15 *Elastic settlement of shear socket (after Pells and Turner, 1979)*

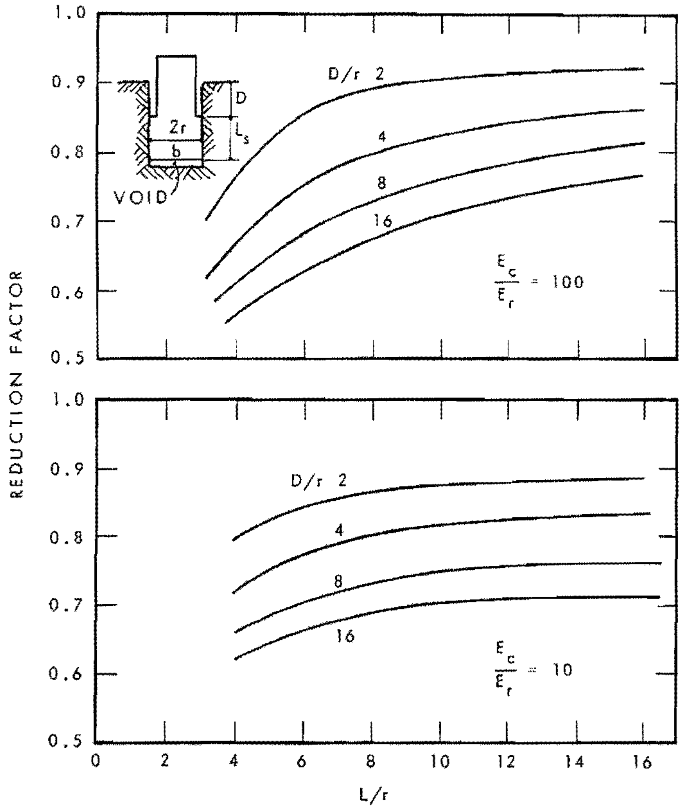


FIGURE 18.16 *Embedment reduction factor for shear sockets. E_c = elastic modulus of concrete, E_r = elastic modulus of rock (after Pells and Turner, 1979)*

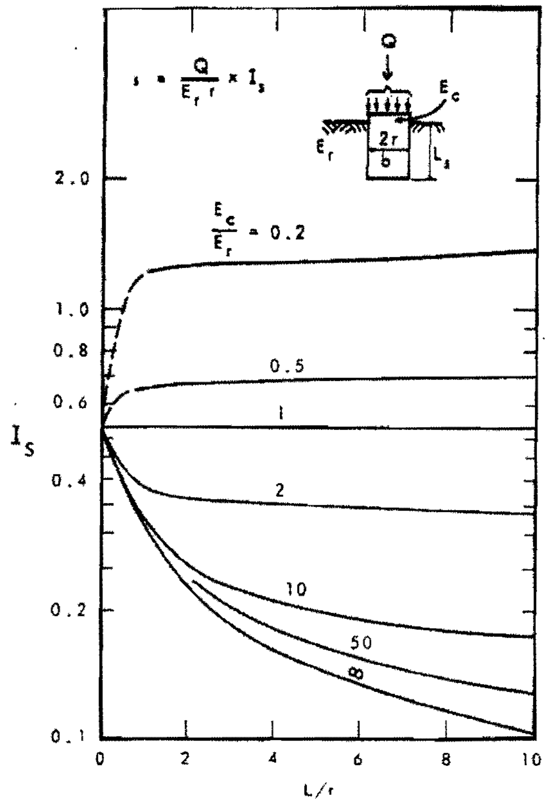


FIGURE 18.17 *Elastic settlement of a complete rock socket (after Pells and Turner, 1979)*

19 Structural Design and Installation of Piles

19 Structural Design and Installation of Piles

19.1 Introduction

This chapter gives information on the use of different types of deep foundations, including special features of structural design and considerations regarding the installation of such foundations.

19.1.1 Resistance of Deep Foundations

19.1.1.1 Structural Resistance

The structural resistance of a deep foundation unit, determined in accordance with the National Building Code of Canada, represents the load that the unit can support as a structural member. In most cases, the bearing capacity of a deep foundation unit is governed by geotechnical considerations rather than by the structural resistance of the unit. The installation and inspection of a deep foundation unit are generally less controllable than for a similar superstructure member. Moreover, the environment of the deep foundation unit may be potentially more damaging structurally than the environment of the superstructure member.

It is important to note that permissible deviations in alignment and location of the unit can be established and considered in the design when determining the structural resistance. Normally, it is not possible to install deep foundations closer than 70 mm to the specified position and, therefore, the design should allow for this location limitation. When the off-location effect is considered, the restraining influence of the pile cap, tie beams and soil should be included. The effects of moments and lateral loads must also be considered in the design.

19.1.1.2 Geotechnical Resistance of a Driven Pile

The geotechnical resistance of a driven pile is a function of the dynamic response of the pile, the so-called dynamic impedance $E_p A_p/c_p$, where E_p is the modulus of elasticity, A_p is the cross- sectional area of the pile and c_p is the speed of the strain wave in the pile. The strength of the pile material has no influence beyond a minimum value, which mostly is smaller than about 250 MPa. Therefore, the geotechnical resistance of a driven pile differs from that of the structural resistance. The potential geotechnical resistance of two piles with the same impedance is the same, whether the piles are of the same material or are different, e.g., steel or concrete, whereas the structural resistance may differ.

The allowable geotechnical stress of a driven pile should be limited to a factor times E_p/c_p of the pile material. In the absence of field verification of the existence and magnitude of soil set-up or soil relaxation, the factor is suggested to be 2 (units = m/s). Field verification by means of test loading or dynamic monitoring (Subsection 19.1.3) will supersede this suggestion.

The value of 2 E_p/c_p for steel piles is relatively constant and equal to 80 MPa. For precast piles, ordinary reinforced piles, and prestressed piles, the elastic modulus, E_p, and the wave speed, c_p, vary and E_p/c_p is not constant. However, 20 E_p/c_p is usually within the relatively narrow range of 12 to 15 MPa and 12 MPa is suggested for use in design. For further discussion see Fellenius (1984b).

The dynamic impedance of a closed-toe steel pile can be substantially improved by concreting the pile before restriking. The resulting increased dynamic impedance (new combined value of $E_p A_p/c_p$) will enable the pile to be driven to a higher geotechnical resistance and/or to verify the existence of soil set-up. By finishing the concrete with a slightly convex upper surface that protrudes above the steel tube, the tube avoids impact of the hammer. Also, it is advisable to add some reinforcing bars to the concrete within a zone of four pile diameters from the pile head.

Soil set-up can be verified in the field by a load test or by dynamic measurements during restriking. While restriking alone is a highly recommended method of quality control and will verify soil relaxation, it does not provide sufficiently reliable information on soil set-up on a pile driven to refusal, unless the pile impedance is increased or a heavier hammer is used that can develop more force and driving energy per blow (as opposed to the hammer used in initial driving) and, therefore, move the pile to a penetration larger than about 3 mm to 4 mm per blow. Ideally, when driving composite piles the design should ensure that the impedance, $E_p A_p/c_p$, of the sections of the pile is the same. If the impedance differs by more than a factor of 2, serious damage or driving difficulties can arise. Composite piles are concrete piles with long steel H-pile ends, or steel pipe and steel H-piles combined, or two sections of different size concrete piles combined, etc.

When driving a pile with a follower made of the same material as the pile, the areas should be equal. If the pile and the follower are of different material, e.g., a concrete pile and a steel follower, the impedances should be equal. This means that the steel area should be about 20 % that of the concrete area. For additional comments, see Fellenius, (1980a).

19.1.2 Wave-Equation Analysis

The one-dimensional wave-equation analysis is the application of longitudinal wave transmission to the pile driving process, which provides a mathematically accurate expression describing the mechanics of strain wave travel along a pile after it has been hit by the ram of the pile hammer. This method takes into account the characteristics of:

- the hammer (mass of the ram or piston, height of fall of the ram, rated energy and impact velocity);
- the driving cap or helmet (mass, stiffness and coefficient of restriction of the hammer cushion or capblock, and the pile cushion, when used);
- the pile (material, dimensions, mass and stiffness); and
- the soil (assumed deformation characteristics represented by quake and damping factors for shaft and toe resistance).

The wave-equation analysis can be used to great advantage when assisting in the selection of hammers and capblocks, in the design of cushions, in the prediction of driving stresses and bearing capacities, and in the choice of driving criteria.

The wave-equation analysis is fundamentally correct. It can provide qualitative information to use in, for instance, the comparison between two hammers. However, the results of the analysis are only as accurate as the data used as input in the analysis.

When no direct measurements or observations are available for reference (calibration), it will be fortuitous if the results are quantifiably relevant to the real situation. In the absence of calibration data from the analysis of dynamic monitoring (Subsection 19.1.3), the wave-equation analysis should be limited to use for providing a range of results established with due consideration to the possible variations of the hammer-pile-soil system.

The factored geotechnical axial compression resistance at ultimate limit states is taken as the ultimate predicted capacity multiplied by a geotechnical resistance factor of 0.4 (Tables 8.1 and 8.2).

The wave-equation analysis should be recognized as one of the major advances of the current state-of-the art. Its use is highly recommended. However, it should be considered as a tool among many others and should be used by a person well experienced not only in wave-equation analysis, but also in the overall art of pile installation and pile-soil analysis.

19.1.3 Dynamic Monitoring

The effect of the hammer impact on a pile in terms of force (stress, strain) and velocity (acceleration) can be monitored using special instrumentation and analyzing the obtained force and velocity 'wave traces'. Information can be obtained as to the proper functioning of the hammer, the impact force, the transferred energy and the soil response to the impact on the pile.

The dynamic monitoring method has been used in Canada since 1976 and is well established. For details on the instrumentation and method see Goble et al. (1970), Rausche et al. (1972), Fellenius et al. (1978), and Authier and Fellenius (1983).

The soil response may be related to the pile static capacity by a method called Case Method Estimate (CMES). This method is fast and produces a value for each impact as the driving proceeds. For more accurate capacity determination, a more time-consuming computer treatment of the data is required, called CAPWAP. Representative blows are selected for analysis when required. The CAPWAP analysis provides not only a value of the static capacity that closely agrees with the capacity obtained by means of a static load test, but also provides data suitable for input in a wave equation analysis.

The advantage of dynamic monitoring is that several piles can be tested for the cost of only one static load test to account for the natural variability of capacity between piles. Apart from the capacity, the method also provides a control of hammer efficiency and determines the energy and driving stresses developed in the pile. In addition, the integrity of a pile can be ascertained by means of dynamic monitoring. No pre-design pile test driving should be performed without inclusion of dynamic monitoring in the program.

The factored geotechnical axial resistance at ultimate limit states is taken as the ultimate capacity obtained from the monitoring results of dynamic testing multiplied by the geotechnical resistance factor of 0.5 (Tables 8.1 and 8.2). Dynamic monitoring and analysis should be carried out by an experienced person and the data should be related to other important geotechnical information from the site.

19.1.4 Dynamic Pile Driving Formulae

The dynamic driving formulae, e.g., Hiley, Engineering News, Janbu formulae and more than 100 others, are derived by equating the nominally available energy, that is, the rated energy, not the actual energy, with work performed by the pile calculated as the static capacity of the pile times the penetration for the blow.

The approach is fundamentally incorrect. However, the static pile capacity predicted by dynamic formulae in particular cases and in local areas can be close to the real values because the smaller the penetration of a pile for a hammer blow, the larger the static capacity. Nevertheless, quantitative agreement is only accidental and cannot be relied upon.

Since the wave-equation analysis is superior and as easy to perform, there is now little reason to continue using the dynamic formulae.

19.2 Wood Piles

19.2.1 Use of Wood Piles

Wood piles are best suited for use as friction piles in sands, silts and clays. They are not recommended for driving through dense gravel or till, or for end-bearing piles to rock since they are vulnerable to damage both at the head and the toe in hard driving. They are difficult to splice. Wood piles are commonly used for depths of 6 m to 15 m, for diameters of 200 mm to 400 mm, corresponding to the natural dimensions of available tree trunks, and for design loads of 100 kN to 500 kN.

19.2.2 Materials

Wood piles must conform to the requirements of the National Building Code of Canada (2005). They may be used untreated where they are entirely located below the permanent water table. In this condition, they are resistant to decay, irrespective of the quality of groundwater.

Where untreated wood piles are exposed to soil or air above the permanent water table, and, in particular, when they are subjected to intermittent submergence, they are vulnerable to decay. In such an environment, only treated piles should be used for permanent structures.

19.2.3 Structural Design

The structural design of wood piles must conform with the requirements of Section 4 of the National Building Code of Canada (2005). No special consideration needs to be given to handling stresses, but special precautions must be taken to protect the pile toe and head from damage due to driving stresses.

19.2.4 Installation of Wood Piles

When driving wood piles, low-velocity hammer blows should be used. For example, drop hammers and single-acting steam/air hammers should have relatively small heights of fall, and incorporate a soft cushion in the capblock. The size of the hammer used for the driving depends on a number of factors, among them the weight of the pile, its size (diameter of head and toe), impedance and the soil properties. As an approximate guide, and in the absence of local experience, the hammer-rated energy should not exceed a value equal to 160 000 J (Newton metre) times the pile head diameter.

Where hard driving occurs, the pile head should be provided with protection in the form of a steel ring and the pile toe should be protected with a special steel shoe. Wood piles cannot withstand hard driving. Over-driving will generally lead to the destruction of the pile. To avoid this, driving must be stopped when high resistance to penetration is encountered (set of about 1 mm to 2 mm per blow).

19.2.5 Common Installation Problems

The potential problems associated with driving of wood piles are the splitting and brooming of the pile toe or head, the splitting or bowing of the body of the pile, and the breaking of the pile during driving.

19.3 Precast and Prestressed Concrete Piles

19.3.1 Use of Precast and Prestressed Concrete Piles

Because of their structural strength and wide choice of possible dimensions, precast and prestressed concrete piles can have a wide range of loading. They are:

- suitable for use as shaft bearing piles when driven in sand, gravel, or clays;
- suitable for use as end-bearing piles;
- suitable for resisting uplift forces, when designed for it; and
- suitable for driving in soils containing boulders, when correctly designed.

Precast concrete piles have been used for depths up to 15 m and up to 40 m for prestressed concrete piles without splicing devices, and to greater depths with splicing devices.

Typical cross-sections are square, hexagonal and octagonal, with face-to-face diameters of 200 mm to 600 mm, or cylindrical, with diameters up to 1400 mm. (The larger diameter cylinders are usually hollow and prestressed.) Design loads vary over a wide range depending on geometry, concrete strength and reinforcing.

19.3.2 Materials and Fabrication

Concrete piles must conform to the requirements of Section 4 of the National Building Code of Canada (2005). High-quality concrete piles, whether normally reinforced or prestressed, should utilize concrete with a 28-day strength of at least 50 MPa. For additional comments on materials and fabrication, see 'Recommendations for Design, Manufacture, and Installation of Concrete Piles' prepared by American Concrete Institute Committee 543.

19.3.3 Pile Splices

Since the length of precast concrete piles is limited by handling conditions, special mechanical 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 controlling influence that splices have on the integrity and drivability of concrete piles. Pile splices are now produced by specialized manufacturers and have been subjected to extensive design review and testing. General requirements for splices are as follows:

- the strength of the splice must be comparable to that of the pile in compression, tension and bending;
- the splice must be designed and positioned so as to ensure and maintain the alignment of the joined pile segments; therefore, the splices must be cast square with the pile segment, and the maximum permissible deviation (out of squareness) of a pile segment end is 1:100; and
- the splice must be designed so that the tolerance play (slack) between two joined pile segments is less than 0.5 mm in either compression or tension (values in excess of this amount will impair the drivability of the pile).

Additional requirements have been imposed as to the bending stiffness, etc., of a spliced pile.

Splices can be obtained with a centre tube, usually with an I.D. of 40 mm, cast in each pile segment, and providing inspection access through the entire pile after driving (see Section 21.3.3).

19.3.4 Structural Design

The structural design of precast and prestressed concrete piles must conform with the requirements of Section 4 of the National Building Code of Canada (2005). The effects of moments or lateral loads must be considered in the structural analysis of the pile.

Temporary stresses resulting from handling and driving may be significant factors in the structural design. For driven piles, common practice is to select a pile having an adequate factor of safety against structural and/or geotechnical failure under service loading, and then to select the driving equipment, hammer and pile cushion on the basis of the structural resistance of the selected pile.

The tolerance on placing reinforcing steel and thickness of concrete cover are important because too-wide tolerances will result in a too-thick concrete cover on one side (and, potentially, a too-thin cover on the other). The concrete cover for high-quality piles should be as thin as possible and equal to 1.5 times the diameter of the largest reinforcing bar or prestressing strand, or equal to the specified largest stone aggregate, whichever value is the largest.

The longitudinal reinforcing bars or prestressing strands should have a minimum cross-sectional area. This is determined by the condition that the maximum axial tension stress in the pile, when calculated on the steel area alone, must not exceed a value of 0.6 times the steel yield strength. This value has been shown to be adequate against tension failure, including additional tension stresses caused by forces due to bending and torsion during driving. Increasing the prestress will not increase the tensile capacity of the pile, unless the steel area and/or the steel yield is increased also.

19.3.5 Installation

Care is required when driving precast or prestressed concrete piles. Tensile stresses are high when the penetration resistance is low. On the other hand, when the penetration resistance is high, the compression force reflected at the toe is superimposed on the impact force and the combined stress may exceed the compression strength of the pile.

19.3.5.1 Required Quality of Pile

19.3.5.1(1) Structural Integrity before Driving

Piles should be designed and fabricated according to the recommendations in Sections 19.3.2 to 19.3.4. However, all piles should be carefully inspected before driving and those that have become severely fissured, sparred, or otherwise damaged, should be rejected. Minor fissures with widths of 0.3 mm or less may be acceptable.

19.3.5.1(2) Pile Head

It is essential that the pile head be perpendicular to the pile axis in order to avoid the uneven distribution of impact forces. For anticipated hard driving, it is good practice to protect the pile head by means of a steel plate at least 13 mm thick. The plate should be anchored into the pile by means of separate reinforcing bars. 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 needs only to be chamfered at the edges and corners. It is important to ensure that no reinforcing steel or prestressed strands protrude from the head.

19.3.5.1(3) Pile Toe

In most cases, the pile toe needs only to be chamfered at the edges and comers. However, when hard driving conditions are expected and, in particular, where piles are driven to toe bearing, a special steel shoe should be attached to the pile toe.

Generally, pile shoes should be used if there is no previous experience available from the site or from a representative nearby site, or from a special study made before finalizing the design.

19.3.5.1(4) Centre Tubes for Inspection

When the length of precast or prestressed piles is more than 25 m, the integrity of the pile after driving can be very difficult to assess from driving records alone. However, it is easy to cast a centre tube in the pile through which access to the full length of the pile is achieved for inspection purposes. The cost of the centre tube is offset by the quality assurance gained, and by the increase of allowable load that this increased assurance justifies (see Section 21.3.3). However, precast piles with centre tubes are normally not available from stock and may require a substantial lead time to supply as opposed to piles without centre tubes.

19.3.5.2 Driving Hammers

19.3.5.2(1) Types of Hammers

Drop hammers and diesel hammers are commonly used for driving precast or prestressed concrete piles. Single acting and differential acting air/steam hammers may also be used. Vibratory hammers are not recommended for precast or prestressed concrete piles because of the high-tension stresses they generate.

19.3.5.2(2) Size of Hammer

The selection of the appropriate hammer is important. In the absence of experience, a wave equation analysis should be used to guide the selection of the hammer.

19.3.5.2(3) Height of Fall and Impact Velocity

To avoid the formation of tension cracks, the ram velocity, or drop height, should 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 toe can be kept within acceptable limits.

19.3.5.3 Driving Cap

19.3.5.3(1) 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.

19.3.5.3(2) Capblock

A capblock must be placed inside the driving helmet to eliminate the damage that would be caused by direct impact. The most common material for a capblock is a hardwood block, whose grain runs parallel to the pile axis, and which is enclosed in a tightly fitting steel sleeve. A typical thickness is 150 mm. The hardwood changes its properties during driving and should not be used once it is crushed or burned since damage to the pile may result.

19.3.5.3(3) Cushion

To avoid damage to the head of concrete piles and to assist in controlling tensile stresses as the result of direct impact from the steel driving helmet, it is essential that a cushion be provided between the helmet and the pile head. A typical cushion is made of compressible material such as plywood with a minimum thickness of 50 mm. The cushion should be changed before excessive deformation or charring takes place. The cushion must fit snugly inside the helmet. A fit that is too loose will result in rapid destruction of the cushion and an undesirable increase in its stiffness.

19.3.6 Common Installation Problems

Two installation problems commonly arise. Regular horizontal tension cracks may form in the early stages of driving when the resistance to penetration is low. These cracks that are visible above ground frequently indicate severe damage below ground, sometimes even loss of a portion of the pile. In hard driving, the pile toe or pile head may be crushed in compression.

19.4 Steel H-Piles

19.4.1 Use of Steel H-Piles

Steel H-piles are:

- suitable for use as shaft-bearing piles, end- bearing piles, and combinations of these two;
- commonly used for any depth, since splicing is relatively easy, and for loads of 350 kN to 1800 kN. Optimum mill lengths are 12 m to 21 m.

19.4.2 Materials

Steel H-piles must conform to the requirements of Section 4 of the National Building Code of Canada (2005). To minimize damage during driving, it is advantageous to use a pile with high-yield strength. However, it is not always possible to utilize the high-yield stress to obtain higher pile resistance (see Section 19.1.1.2). Experience indicates that corrosion is seldom a problem for steel piles driven into natural soil. However, in fill at/or above the ground water table moderate corrosion may occur. Steps should be taken to protect the piles when these conditions occur. Solutions include the application of coatings (such as coal tar epoxy) before driving, encasement by cast-in-place concrete jackets, cathodic protection, specification of copper content in the steel, or combinations of these. A common practice is to increase the steel section, typically by 1.5 mm, to provide an allowance for corrosion.

19.4.3 Splices

Common Canadian practice is to splice by making full strength butt welds on H-piles. Although this is not always necessary to carry the design loads, it is good practice in order to accommodate the high driving stresses that can develop in steel. Sufficient time should be allowed for heating the pile segment ends before welding and for the welded pile to cool below 300 ^{0}C before driving is resumed. Special pile splices available from industry greatly reduce the splicing (welding) time in the field.

19.4.4 Structural Design

The structural design of steel H-piles must conform to the requirements of Section 4 of the National Building Code or Canada (2005) and to Section 19.1.1 of this Manual. Because of the high strength of steel, handling conditions are usually not considered in the design of steel H-piles. Uneven stresses of up to about 10 MPa can exist between the flanges and the web, owing to the different cooling conditions for the flanges as opposed to the web in the manufacturing process.

19.4.4.1 Driving Conditions

Experience indicates that the maximum stresses developed in a pile occur during the driving process. The impact stress delivered by any hammer does not normally exceed a value of about 200 MPa, which is smaller than the yield strength of ordinary steel used in steel piles. However, because of additional stresses imposed by bending, eccentric blows and, above all, reflected forces from the pile toe superimposed on the impact force, higher-than-ordinary steel yield strength may be needed. This increased structural axial resistance, however, does not increase the geotechnical axial resistance of the pile (see Section 19.1.1.2).

Figure 19.1 shows measurements of impact stress developed in a large number of steel piles driven to refusal by different hammers under different conditions in different parts of Canada. The measurements illustrate the high variability of driving conditions and the necessity of quantifiable control of the developed stresses in actual pile driving.

19.4.4.2 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. For piles subjected to moments or horizontal loads in addition to vertical loads, the effects of such loads, as described in Chapter 18, must be taken into account in the structural design of piles.

19.4.5 Installation and Common Installation Problems

Driving of steel H-piles is generally easy. However, problems can arise when driving H-piles through dense gravel, or through tills containing boulders. If driven unprotected under these conditions, the pile toe may deform to an unacceptable extent, and separation of the flanges and web may occur. To avoid problems, when hard driving is expected, the toes of the H-pile should be protected using cast steel drive shoes. Chamfering of flanges is also used to prevent toe damage, when driving H-piles through or into hard material. Welding steel plates to the flanges at the toe of the pile can be an inadequate method.

Pile shoes should be used if there is no previous experience available from the site or from a representative nearby site, or from a special study made before finalizing the design.

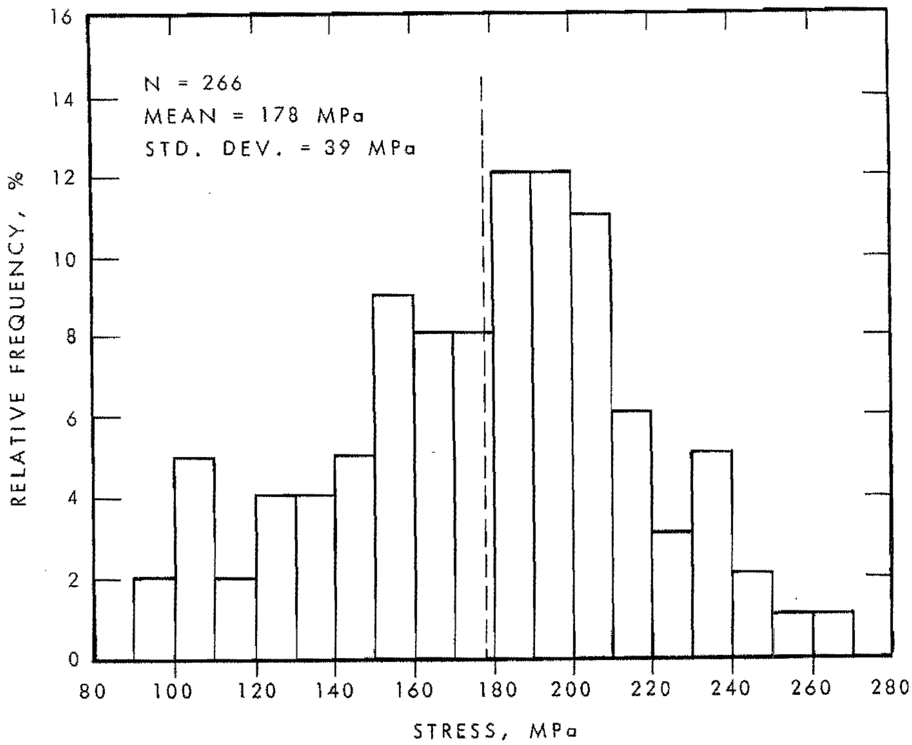


FIGURE 19.1 *Measurements of impact stress in driven steel piles (after Fellenius et al., 1978)*

Long H-piles are prone to bending and doglegging, and the straightness of the H-pile cannot be inspected after driving (Hanna, 1968). Therefore caution is recommended when using long H-piles.

All kinds of driving hammers may be used to drive steel H-piles. A wave equation analysis is recommended for use when selecting the appropriate hammer. As a guide and in the absence of local experience, the rated energy of the hammer should be limited to a value of 6 x 10^6 J (Newton metre) times the cross-sectional area of the pile. (For recommendations for the driving cap and capblock, see Section 19.3.5.3). Pile cushions are not used when driving steel H-piles.

Driving H-piles with poorly fitting helmets, or with helmets misaligned with the pile and/or non-concentric, may damage the pile head. The damaged pile head may then act as a cushion on the pile and adversely affect the penetration resistance.

Commonly, installation problems with H-piles originate when the section of the pile used is too small.

19.5 Steel Pipe Piles

19.5.1 Use of Steel Pipe Piles

Steel pipe piles may be driven with the lower end of the pipe open or closed. They may be left open or filled with concrete. They can be used as shaft-bearing piles, end-bearing piles, or socketed piles.

Steel pipe piles are commonly used for variable lengths, since cuts and splices are easily made. Piles with diameters up to 600 mm are used to carry loads up to 1800kN.

19.5.2 Materials

19.5.2.1 Steel

The materials to be used for steel pipe piles must conform to the requirements of the National Building Code (2005). Experience indicates that corrosion is not normally a problem for steel piles driven into natural soil. However, in fill at/or above the groundwater table moderate corrosion may occur. Where these conditions exist, steps should be taken to protect the piles. These steps include the application of coatings such as coal tar epoxy before driving, encasement by cast-in-place concrete jackets, cathodic protection, inclusion of copper content in the steel, or any combination of these. Increasing the wall thickness to provide a margin for corrosion is also a common practice.

For detailed information on corrosion of steel piles, see National Bureau of Standards Monograph 127 (1972) and Bjerrum (1967).

19.5.2.2 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 of the National Building Code of Canada (2005). However, the concrete must have a slump of 120 mm or greater, and it should be installed through a short funnel ('elephant trunk') that causes the concrete to fall down the centre of the pile, allowing air to escape and eliminating the risk that voids will develop due to entrapped air.

Concrete in a confined state such as when it is encased by a heavy wall steel tube develops a higher compressive strength than it does without confinement. Allowance is seldom made for this phenomenon. Similarly, the modulus of elasticity of confined concrete is higher than that of unconfined concrete as in the case of a precast pile or in a thin-wall pipe pile (confined modulus is higher than Young's modulus).

19.5.3 Structural Design

The structural design of steel pipe piles must conform to the requirements of Section 4 of the National Building Code of Canada (2005). Because of the properties of steel, handling conditions are not normally considered in design.

19.5.3.1 Driving Conditions

Experience indicates that the maximum stresses developed in a pile occur during the driving process. The impact stress delivered by any hammer does not normally exceed a value of about 200 MPa, which is less than the yield strength of ordinary steel used in steel piles.

However, because of additional stresses imposed by bending, eccentric blows and, above all, reflected forces from the pile toe superimposed on the impact force, higher-than-ordinary steel yield strength may be needed. This increased structural axial resistance, however, does not increase the geotechnical axial resistance of the pile (see Section 19.1.1.2).

Figure 19.1 shows measurements of impact stress that developed in a large number of steel piles driven to refusal by different hammers under different conditions in different parts of Canada. The measurements illustrate the high variability of driving conditions and the necessity of quantifiable control of the developed stresses in actual pile driving.

19.5.3.2 Working Conditions

The structural resistance of steel pipe piles is determined according to the requirements of the National Building Code of Canada (2005) and Section 19.1.1.1 of this Manual.

19.5.4 Installation

Installation of steel pipe piles is generally easy. Problems can arise when driving pipe piles that are closed at the toe through materials containing obstructions, or when driving open pipe piles through dense material, or when using thin-wall pipes. In the first case, piles may deflect and deviate from their design alignment to an unacceptable extent. In the second case, the pipe may be deformed, in particular, at the pile toe, but also could be deformed by local buckling along the pile.

19.5.4.1 Protection of the Pile

19.5.4.1(1) Piles Driven with Closed Toe

A float plate of 10 mm to 20 mm thickness is normally used to close the toe of the pile. The diameter of the toe plate is normally 20 mm larger than that of the pile. When pipe piles are driven to weathered rock or through soil containing boulders, a special shoe of cast iron is often needed.

Pile shoes should be included in the design of all piles if no previous experience is available from the site or from a representative nearby site, or unless a special study made before finalizing the design indicated that pile shoes are not necessary.

Pile shoes are not normally necessary, but the damage and the difficulties arising from not using them when they are necessary more than justify the foregoing requirement. On projects large enough to make an index pile-driving program economical, piles may be pulled for visual inspection of the pile toe. Sometimes, dynamic monitoring observations can clarify the need for pile shoes. If pile shoes are found not to be necessary, they can then be omitted from the piles still to be installed. To take advantage of this approach, appropriate financial conditions need to be included in the contract specifications.

19.5.4.1(2) Piles Driven with Open Toe

No special protection is necessary for soft or medium driving. When hard driving is expected, as in dense gravel, a special driving shoe should be provided that is made of special steel or steel alloy. Regular checks on the level of soil within the pile are necessary to recognize the formation of a soil plug at the pile toe.

If the soil within the pipe is cleaned out, the integrity and the straightness of the pile can be inspected but it should be noted that cleaning out the soil using jetting might compromise the pile shaft and toe resistance. If jetting is used, it must be assured that the process does not degrade the geotechnical axial resistance of the pile.

19.5.4.2 Driving Equipment

All kinds of driving hammers may be used to drive steel pipe piles. A wave-equation analysis should be used in selection of the appropriate hammer. As a guide and in the absence of local experience, the rated energy of the hammer should be limited to a value of 6×10^6 J (Newton metre) times the cross-sectional area of the pile. For recommendations for the driving cap and capblock, see Section 19.3.5.3. Pile cushions are not used when driving steel pipe piles.

A pile head damaged by local buckling will act as a cushion and reduce the effect of the driving. Such damage is normally not caused by too large an impact force but from misalignment of the pile in the helmet, non-concentric impacts, and/or a poorly fitting helmet.

19.5.5 Common Installation Problems

Common installation problems are:

- High porewater pressure may develop in fine-grained soils due to pile driving. This pressure may cause thin-walled piles to buckle locally;
- The impedance of thin-walled pipes may not be adequate to allow the development of sufficient force in the pile to achieve acceptable pile resistance; and
- Failure to use backing rings when splicing and to ensure that full butt welds are properly made may cause rupture or leaks.

19.6 Compacted Expanded-Base Concrete Piles

19.6.1 Use of Compacted Concrete Piles

Compacted expanded-base concrete piles (also called 'pressure injected footings') require the use of special installation equipment handled by persons experienced in the installation work. Compacted-base concrete piles develop their bearing capacity primary from the densification of soil around the expanded base. They are:

- 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;
- usually unsuited in loose granular soils containing more than about 15 % to 20 % of fine-grained soil, where special measures are needed to ensure the integrity of the base and shaft; and
- commonly used with shaft diameters of 300 mm to 600 mm, for loads of 500 kN to 1600 kN, and for lengths of 3 m to 18 m.

19.6.2 Materials

Materials used for compacted concrete piles must conform with the requirements of Section 4 of the National Building Code of Canada (2005). Because of the installation technique, however, damp concrete must be used in the compacted base and in the compacted shaft when an encased shaft is not used. (Damp concrete is concrete with zero slump, and containing about 15 litres of water per cement bag.)

The strength of the concrete should be checked on special compacted samples, although there is currently no standard method for such sampling and testing. When test cylinders are made in a conventional way, water may be added to alleviate difficulties in ensuring a homogeneous product. However, adding water may prevent the discovery of cases where too dry concrete was used in the base.

Compacted concrete piles are sometimes built with an encased shaft. The casing is generally made of light-gauge steel tubing and is intended only to provide protection against intrusion of water or soil during concreting operations. Temporary casing is recommended to protect shafts in soft soil.

19.6.3 Structural Design

The structural design of compacted concrete piles must conform with the requirements of Section 4 of the National Building Code of Canada (2005) and Section 19.1.1 of this Manual.

19.6.3.1 Working Conditions

19.6.3.1(1) Piles with Compacted Shaft

Piles with compacted shafts are made of damp concrete compacted against the soil and may be reinforced. The area of concrete effective in load carrying is limited to a value equal to the nominal area of pile shaft corresponding to the outside diameter of the driving tube, unless local experience and measurement of test piles indicate that a larger diameter is achieved on compaction of the shaft.

19.6.3.1(2) Piles with Encased Shaft

Piles with encased shafts may be reinforced. Where compacted concrete piles have to resist uplift forces, the structural strength of the shaft must be determined accordingly. Consideration must be given to a proper continuity of reinforcing at the junction of the shaft with the base.

19.6.4 Installation

Installation of compacted concrete piles requires the use of special equipment and should be done by an experienced contractor.

For piles with compacted shafts, 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 gap in the concrete shaft. To avoid this problem, constant control must be exercised on the quantities of concrete placed, the evaluation of the base of the tube, and the elevation of the top of the compacted concrete.

Under some soil conditions, such as where 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. Typical cases are discussed by Brzezinski et al. (1973) and Clark (1978).

The 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 when such piles are used in areas or soil conditions where little or no experience is available.

19.6.5 Common Installation Problems

Poor quality concrete resulting from inadequate design of the mix or from using concrete that is too dry, frequently causes problems. Segregation of aggregates or loss of cement by adherence to the walls and fins of a drum-type mixer may occur. Premature setting of concrete is of concern, particularly, when heated concrete or casting is used in hot weather.

Serious problems have been caused by poorly made reinforcing cages or by the use of brittle steel. A reinforced compacted shaft can have its concrete badly disturbed by the motion of a broken cage as subsequent charges of concrete are rammed.

Attempts to form compacted shafts under conditions where high pore-water pressures exist, or have been induced in the soil by the driving may lead to necking or contamination of the shaft. Inadequate compaction may result from having too much concrete in the drive tube at the time of the ramming. Heave and displacement caused by driving of nearby piles, cause many failures of compacted shafts and should be checked carefully during the installation process.

19.7 Bored Piles (Drilled Shafts)

19.7.1 Use of Bored Piles (Drilled Shafts)

Bored piles can be made in different shapes and dimensions. Although cylindrical piles are the most commonly used type, in recent years elements of diaphragm walls have been used in various combinations (I, H, X) as deep-foundation units. Bored piles called "barettes" are being used increasingly because of their high load capacities.

Bored piles are:

- well suited for end-bearing, high-capacity piles to rock or dense till;
- successfully used in stiff clays, primarily using shaft resistance; and
- commonly used for variable lengths (bored piles drilled with bentonite slurry have been installed at depths in excess of 100 m, for diameters in excess of 1 m and up to 3 m, using up to 4 m bells, and for loads up to 18,000 kN).

Where deposits of loose cohesionless materials have to be penetrated, or where groundwater conditions prevail, it may be necessary to resort to the use of bentonite slurry or temporary casing.

19.7.2 Materials

Materials to be used for bored piles must conform to the requirements of Subsection 4 of the National Building Code of Canada (2005), except where concrete is placed by tremie or pump, when the requirements of CSA A23.1 concerning maximum slump cannot be met. Slumps of about 180 mm are normally used. A person competent in this field of work should design the concrete mix.

When bored piles are provided with structural steel casings, the appropriate considerations discussed in Sections 19.5 of this chapter also apply.

When bored piles are excavated with bentonite slurry (premixed), the quality of the slurry (density, viscosity, etc.) should be determined and documented, and it should be kept under constant control to enhance satisfactory performance.

19.7.3 Structural Design

The structural design of bored piles must conform to the requirements of Section 4 of the National Building Code of Canada (2005).

19.7.4 Installation

19.7.4.1 Excavation (Drilling)

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, rotating 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 auger or bucket mounted on a Kelly bar to remove the soil, and by keeping the excavation open by use of a bentonite slurry; and
- by drilling, coring, or dropping when penetration into rock is specified (blasting could adversely affect the properties of surrounding rock and soil).

Selection of the excavation procedure depends upon the soil and prevailing groundwater conditions. O'Neill and Reese (1999) give extensive descriptions of construction methods and problems with bored piles (drilled shafts).

Regardless of the procedure used for excavation, it is essential that the base be cleaned to the sound founding material. Groundwater must 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 bonding the concrete to the walls of the socket.

19.7.4.2 Placing Concrete

After the excavation has been completed, inspected and accepted, concrete may be placed during one continuous operation. Steel reinforcement, steel stubs, 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. Where the excavation is dry, concrete may be placed using buckets, funnels, chutes, or 'elephant trunks', so that segregation does not occur. Free-falling concrete must be poured through a centring chute, making it fan down the centre of the hole, well clear of the walls of the shaft. This results in adequate compaction below the upper 1.5 m. Vibration of the concrete in the upper 1.5 m is then required to produce uniform strength concrete. Concrete slumps equal to or exceeding 120 mm must not be vibrated but gently rodded.

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, pinched off by earth movement, or contaminated by the entry of water or mud. The level of the concrete should be maintained at a height above the bottom of the casing sufficient to counter the head of any water or mud outside the casing. If the method of drilling has created an annulus around the casing, mud may fill this annulus and rise with the casing as it is withdrawn, creating a temporary head that may cause intrusion if the cut-off level of the pile is below ground level.

Pumping is the best method for placing concrete, although a tremie may be used with adequate safeguards. With either method, the pour must be fast and continuous. Use of a tremie is often restricted to piles shorter than 15 m concreted length that can be handled with a single length of pipe, i.e., with no joint to be uncoupled. A tremie should be withdrawn slowly and should have a uniform and smooth cross-section to minimize disturbance after placement. Employment of a retardant is desirable.

19.7.5 Common Installation Problems

Some common problems associated with the installation of bored piles are:

- discontinuous supply of concrete, especially when bentonite slurry is used to keep the excavation open. When the concrete flow stops, some cuttings in suspension in the slurry settle to the top of the concrete resulting in a defect;
- inadequate precautions to control groundwater flow during excavation, resulting in loss of ground;
- the flow of concrete through water when the tremie pipe is pulled out of the concrete during placing (the result is a layer or pocket of sand and gravel and a concentration of cement or laitance at cut-off level);
- the intrusion of soil in the theoretical concrete section (necking), caused by too-rapid withdrawal of the temporary casing;
- the temporary casing becomes stuck and is withdrawn after partial set of concrete has taken place causing cracking of the shaft;
- use of concrete that is too old when placed (a retarder should be specified when delays are possible) ; and
- use of low-slump concrete without vibration causing the formation of voids.

20 Load Testing of Piles

20 Load Testing of Piles

20.1 Use of a Load Test

Test loading of piles is the most positive method of determining load capacity and forms a fundamental part of pile foundation design. The results of a pile load test can be used to evaluate the ultimate load capacity of the pile and its load-settlement (serviceability) behaviour or to verify the design assumptions. They can also be used to confirm the acceptability of the performance of the pile or to examine its structural integrity. Depending upon the type and size of the foundation, such tests may be performed at different stages during design and construction.

The method of load testing is selected and the load test is designed to provide relevant information regarding:

- the axial geotechnical load capacity in compression and in uplift (an ultimate limit state);
- the lateral geotechnical load capacity (an ultimate limit state);
- the performance of the pile under both axial and lateral working load conditions (a serviceability limit state); or
- contractibility of the piles.

Different load tests may be needed to provide all the above information and the selection of the test method and the number of tests should be made by the engineer responsible for the design depending on the circumstances of the specific project. The common types of test that may be employed are discussed in the following section.

20.1.1 Common Pile Load Test Procedures

20.1.1.1 Static Axial Load Test

This type of test is the most common and most fundamental pile load test. It involves the application of axial static load directly to the pile head. Static axial load tests of piles should be carried out according to ASTM D-1143. This standard is reviewed and revised about every five years and the latest standard should be used. It should be noted that ASTM D-1143 specifies what is required and how to arrange and document a test. It does not specify a particular method, but gives several options.

The static axial load test can be expensive and time consuming, and in some cases practically impossible to perform (e.g., for very large capacity piles). The results of the test may be influenced by the test setup required to provide the load at the pile head. Interpretation of the test results should account for the effects of the interaction of the elements of the test setup and the pile under consideration.

The factoral geotechnical axial resistance at ultimate limit states is obtained by multiplying the measured ultimate load by the geotechnical resistance factor of 0.6 (Tables 8.1 and 8.2).

20.1.1.2 Static Lateral Load Test

This type of test is required when the pile is expected to support a significant lateral load component transmitted from the superstructure. The test involves a lateral load at the pile head by jacking the pile against one or more other piles, or against a concrete block. ASTM Standard D3966 outlines procedures for pile lateral load testing.

The static lateral load test is expensive and time consuming. Using other piles as reaction bodies in the test may influence the test results. The interaction between the piles and the flexibility of the support system should be accounted for when interpreting the test results.

20.1.1.3 Dynamic Load Test

This type of test involves the application of a small mass accelerated to a multitude (about 20 times) of gravity acceleration to impact the pile head over a very short period of time (3-4 milliseconds). There are several well established approaches for the interpretation of the test results using wave equation analysis and a curve matching procedure (see Rausche et al. (1985); El Naggar & Novak (1994) and Middendorp & Van Weele (1986)). This test is now gaining popularity because it is fast and relatively inexpensive. It has become a routine procedure for quality control and design confirmation purposes. For driven piles, the test should be performed by restriking the pile some time after initial installation (minimum two weeks).

The dynamic pile load test has a number of potential disadvantages:

- The load-settlement behaviour obtained is not unique, but rather is a best-fit estimate;
- The load and settlement are not measured directly. Instead, the strain and acceleration are measured and used to calculate the load and settlement, and the distribution of the soil resistance along the pile; and
- The load is applied very rapidly, which affects the results in two ways. The pile stiffness is likely to be overestimated and the time dependent settlement (consolidation and creep) is not developed during the test.

20.1.1.4 Osterberg Cell Test

This type of test was developed by Osterberg (1989) and has been used increasingly since then, especially for large capacity piles. In this type of test, pressure is applied through a special cell that is cast in or near the pile toe. As a result of this pressure, the pile toe (or the soil underneath it) is jacked downwards and the pile shaft is jacked upwards. Load tests of up to 150 MN have been performed using the Osterberg cell.

The Osterberg cell test has some limitations:

- The test is terminated when the element with the smaller capacity reaches its ultimate capacity. The ultimate capacity of the entire pile may not be estimated properly.
- The test is applicable to bored piles only.
- The test pile must be pre-selected as the cell must be pre-installed prior to the test.
- The pile shaft and toe interact with each other during the test, and each will tend to move less than the real movement. Thus, the shaft and toe stiffnesses evaluated from the test are overestimated.
- The zone just above the load cell may not yield unit shaft friction values that are equivalent to those that would occur in an ideal static axial load test.

20.1.1.5 Rapid Load Test

A rapid load test denotes an impulse or push-load on a pile. The rapid load test is characterized by a relatively long duration (80ms to 300ms) compared with a dynamic load test. This load-duration is from 10 to 1000 times the natural period of a foundation, given by

$$f = \frac{2L}{V_c} \tag{20.1}$$

where

L = the pile length

$V_c = \sqrt{E/\rho}$ = primary wave velocity of the pile material

E = elastic modulus of the pile material

ρ = mass density of the pile material.

The rapid load test is also characterized by low pile velocity and acceleration (below 2 m/s and 1 g (9.81 m/s^2), respectively) compared with the dynamic load test. Two methods of rapid load testing are currently in use: Statnamic (STN) and the Pseudo-static Pile Load Tester (PLT). Both methods were developed in the late 1980s and have gained wide acceptance. In both methods, the applied load is measured using a load cell, and the displacement is measured using an optical displacement transducer.

The strengths and weaknesses of Rapid Load Testing can be summarized by the following:

Strengths

- direct measurements of load and displacement;
- quick set-up time;
- minimal physical space required (less than static test);
- easy to apply in over-water situations;
- less costly than static testing; and
- standard static failure criteria can be applied to the predicted static load-movement behaviour after removing the dynamic effects.

Weaknesses

- duration of applied load too fast to observe time-dependent movement of the foundation;
- 'calibration' with static test may be necessary in soils that are sensitive to loading-rate (clay) – can over-predict pile capacity in clay; and
- unable to separate skin-friction and end bearing unless the pile is instrumented with strain gauges

20.1.1.4 (1) Statnamic Load Test (STN)

The STN was jointly introduced in Canada and The Netherlands in the late 1980s (Middendorp et al., 1992, El Naggar & Novak, 1992). The STN device uses a solid fuel propellant to generate gas pressure. This gas pressure acts upward against a reaction mass in order to apply a rapid downward load on the pile.

Using the Statnamic device, load tests ranging from 600 kN to 30 MN can be performed. Most Statnamic tests can be assembled within one working day, with smaller devices taking even less time. Multiple loading cycles are easily performed. Assembly requires the use of a crane. The Statnamic device has also been used to perform lateral load tests successfully (El Naggar, 1998).

20.1.1.4 (2) Pseudo-Static Load Tester (PLT)

The Pseudo-static pile load tester (PLT) is used to drop a mass of 25,000 kg on the pile head. In order to soften the impact and spread the transmitted energy over a longer period, heavy coiled springs are attached to the bottom side of the mass. In this way, a slow-rising, long period blow is introduced to the pile without causing wave propagation effects.

The dynamic effects that are present during dynamic load testing are minimized by spreading the impact wave over a longer period of time (effective up to 200 milliseconds). A load cell and a displacement transducer are mounted on the pile.

Although similar in procedure to a high-strain dynamic test (which also uses a drop-mass), the PLT's special mechanical springs create a loading pulse that is of significantly longer duration, making the PLT a rapid load testing technique rather than a high-strain dynamic technique. The PLT is currently capable of testing loads up to 3,500 kN. Set-up and testing of multiple piles can be performed in a single day; multiple loading cycles are easily performed. Assembly and movement of the PLT rig can be accomplished without the need for a crane.

20.1.2 Load Tests during Design

Results of load tests are invaluable aids in the design of pile foundations. The engineer responsible for the design should select the test method, the number of tests, type of piles tested, method of driving or installation and loading. The following points should be considered:

- a detailed soil investigation should be carried out at the test location;
- the piles, and the installation equipment and procedure should be those intended to be used in the construction of the foundation;
- the pile installation must be observed and documented in detail;
- the piles should be loaded to at least twice the proposed working load, and preferably to failure;
- the arrangement, execution, and reporting of the test should closely follow the appropriate ASTM Standard (D-1143, push test; D-3689, pull test; and D-3966, lateral test); and
- where possible, measurements using telltales should be made of the toe movement of the pile to allow for a separate evaluation of shaft and toe resistance.

20.1.3 Load Test during Construction

It is recommended practice to perform proof tests on representative units at early stages of construction. The purpose of such tests is to ascertain that the allowable loads used for design are appropriate, and that the installation procedure is satisfactory.

The engineer responsible for the design should make the selection of the test piles. The selection should be made on the basis of observed installation behaviour.

20.1.4 Routine Load Tests for Quality Control (Inspection)

If the engineer is to take full advantage of Sentences 4.2.4.1.(1)(c) and 4.2.7.2.(2) of the National Building Code (2005), a sufficient number of tests must be carried out on representative units to assess and verify the uniform safety of the allowable loads and the proper behaviour of the constructed foundation. Test loading for control should be performed on one of every 250 piles, or portion thereof, of the same type and capacity. 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 tested is the responsibility of the design engineer.

Static load testing is expensive, and while it is not practical to attempt a statistically representative number of tests, usually more than one test is necessary.

However, the necessary number of static tests can be significantly reduced if combined with dynamic testing and monitoring, where the static testing will serve essentially as a calibration of the dynamic testing. More STN or PLT load tests can be performed for the quality control for the same or less cost than static load tests.

20.2 Test Arrangement

20.2.1 Static Load Test

A static load test must be arranged in conformity with the ASTM D-1143. If the minimum distance and accuracy values recommended by the ASTM standard are reduced, the reliability and usefulness of the test results could be impaired. For instance, the specified clear distances between the measuring beams, the platform supports, etc., and the test pile, are minimum values, which actually mean that some, usually negligible, erratic influence on the test data is accepted. When performing other than routine tests, it is advisable to increase these values.

In a routine test, the load is generally applied by means of a hydraulic jack that is also used as a load gauge to measure the applied load. This system may have an apparent high accuracy, because of the use of a high-precision manometer. Nevertheless, because of many influencing factors not evident in a laboratory calibration, an actual hidden error, which can be as high as 20 % of the applied load, often affects the load values. This error is usually on the unsafe side. When higher accuracy and confidence in the test results are needed, i.e., where potential errors of up to 20% cannot be accepted, a separate load cell has to be used as the main gauge for determining the load. The jack pressure gauge should then be kept as a back-up.

The load cell must be suitable for field use, i.e., have a low sensitivity toward inclined and eccentrically applied loads, and toward temperature variations. In order to ensure reasonably accurate load values in the field, the ASTM D-1143 recommendation that uses a thick steel plate on both sides of the jack (and load cell), and the use of a spherical bearing plate (swivel plate) must be followed.

Load test value is enhanced by measuring the pile compression using a telltale to the pile toe, and also the movement of the pile toe. Whenever possible, it should also be considered for routine tests.

20.2.2 Statnamic Test

The Statnamic apparatus consists of two main components, a piston and a cylinder. The piston is mounted directly on the test pile, which is instrumented with a load cell and displacement transducer. The piston also holds the STN solid fuel. The Statnamic cylinder is mounted on top of the piston thus creating an enclosed combustion chamber. The cylinder also holds the reaction masses (generally about 5% of the desired test load). A silencer is bolted to the top of the cylinder and is used to reduce the noise when the gases are released.

The Statnamic test is initiated using an electric ignition circuit. As the solid fuel burns, the pressure (and consequently the applied load) on the pile increases until the fuel supply is exhausted. The cylinder holding the reaction masses is accelerated upwards during this process. The gas pressure is released gradually as the cylinder moves past an exhaust opening. After the loading event has finished, the reaction masses continue to move upward, usually to a height of about 3 m to 5 m. The masses are prevented from falling on the pile using various types of hydraulic and/or mechanical 'catching mechanisms'.

The simplest type of catching mechanism involves surrounding the Statnamic reaction masses with a steel container and filling the annular space with gravel. As the masses 'jump', the gravel falls below them and cushions their subsequent fall to the ground.

20.2.3 Pseudo-Static Load Test

The PLT consists of a crawler with outriggers and a lead for guiding the drop-mass. The mast is equipped with a hydraulic cylinder for lifting the drop mass. The base of the lead contains large mechanical springs for decelerating the fall of the drop-mass. At the completion of the downward movement, the mass (with springs) hits an anvil that rests on top of the test pile and the energy is transmitted to the test pile. As the mass bounces upward, it is caught in its highest position by hydraulic clamps to avoid a direct second blow.

During the test, the pile is driven into the soil depending on the resistance. The load-settlement curve is obtained by directly measuring the force and settlement at the pile head. The force introduced on the pile during the blow is measured by means of a calibrated (dynamic) load cell and the absolute displacement of the pile during the blow is measured using an optical technique. In this technique, an optical transmitter is attached to the pile and an optical receiver is placed at a distance of 15 m to 20 m. This distance between the pile and the optical receiver should be maintained to ensure minimal ground vibrations.

The force and settlement time histories are recorded using a computer attached to the load cell and the optical displacement sensor. The results are plotted on a load-settlement curve. The test can be repeated with different drop heights and several load-settlement curves are established, each featuring a different peak load and corresponding settlement. Accompanying software is then used to establish the pile load-settlement performance using the peak load and settlement points from the different tests.

20.3 Static Load Testing Methods

20.3.1 Methods According to the ASTM Standard

The ASTM Designation D-1143 contains seven separate methods for performing a load test. Reference to the ASTM method, therefore, implies acceptance of any one method unless the desired one is specified. The methods are as follows:

1. 'Standard Loading Procedure'--a slow maintained-load method using eight equal-load increments to twice the design load. Total test duration is 48 to 72 hours, or more.

2. 'Cyclic loading'--the 'Standard Loading Procedure' method with unloading and reloading cycles added.

3. 'Loading in Excess of Standard Test Load'--after finishing the 'Standard Loading Procedure' method, the pile is reloaded until failure or to a predetermined maximum load.

4. 'Constant Time Interval Loading'--a maintained-load method in ten equal increments of load until twice the design load. The increments are applied every 60 min. regardless of settlement. The method is similar in all other aspects to the 'Standard Loading Procedure' method.

5. 'Constant Rate of Penetration Method' (CRP)- requires the use of a special pump that can provide a constant flow of oil to the jack. Usual penetration rate is between 0.25 and 0.5 mm/min. Total test duration is 2 to 3 hours.

6. 'Quick Load Test Method'--a maintained-load method using many small load increments applied at constant short time intervals. The test is carried out to failure, or to a predetermined maximum load. Total test duration is 3 to 6 hours.

7. 'Constant Settlement Increment Loading Method'--a special method, where the applied load increments are varied so as to achieve approximately equal settlements per load increment. The settlement increment is chosen to be about 1% of the pile head diameter.

The slow testing methods, (1) to (4), and (7) above, are very time-consuming. When the objective of the test is to determine the bearing capacity of the pile, these methods can actually make the test data difficult to evaluate and disguise the pile's true load movement behaviour, thereby counteracting the objective of the test. The benefit of the slow methods lies in the additional soil-pile behaviour information, occasionally obtained, which the interpreting engineer can use in an overall evaluation of the piles.

Settlement cannot be predicted from the results of a static load test. The settlement of a pile group, in particular, cannot be predicted from the results of a test on a single pile, even if the pile test was a long-term one of weeks or months. However, the long-term test on a single pile can occasionally provide valuable information on the pile behaviour and the distribution of load between the shaft and the toe that can be used as a guide when predicting the group behaviour.

For routine testing and proof testing purposes, the quick methods, (5) and (6) above, are sufficient. The quick test is technically preferable to the slow methods when the objective is to determine the bearing capacity of the pile for a limit states design.

On occasion, there could be conflicting reasons for choosing a quick or a slow test loading method. The conflict is best resolved by first carrying out a quick test to soil failure, or a maximum test load, and, thereafter, a slow test, usually according to an elaborate slow-method testing programme (if the slow method is performed first, the value of the quick-method test is impaired).

An additional aspect in favour of the quick-testing method is that it is the easiest and also the least costly testing method of the seven listed.

When a quick-test is performed, the duration of each load increment should not exceed 15 minutes, nor should it be shorter than 5 minutes unless a well trained crew is at hand. When manually recorded telltale measurements are included, the duration should be no shorter than 10 minutes. The number of load increments to failure, or maximum load, should be more than 25, preferably 35 to 40.

When the pile is unloaded, the oil in the hydraulic pump is first leaked a small amount. This normally shows up as a significant pressure drop on the manometer, indicating a reduced jack load. However, the actual load reduction is small, as can be observed when a load cell is used to obtain the true load on the pile. The load difference between the jack load and the load cell is an indication of the friction in the jack. Then, the oil is leaked a bit at a time and, after a wait period of about two minutes, readings are taken of load, manometer pressure and deformation at approximately five to seven levels. Note that the pressure in the jack is constantly being reduced. The pump must not be activated during the unloading of the pile to load the pile up (e.g., to bring the load up to a level that was 'missed').

In particular, when telltale measurements are included, the data from the first couple of load increments are extremely important for the complete evaluation of a test. Once a test is completed, additional valuable data can be quickly and cheaply obtained by quickly reloading the pile to within three increments of the previous maximum load, followed by the regular quick test method from this load onwards.

20.3.2 Other Testing Methods

It may be necessary to test piles under loading conditions other than the usual axial compressive load; for example, pullout tests (ASTM D-3689) and horizontal (lateral) tests (ASTM D-3966) may be specified.

Pullout testing is similar to compressive testing (push test) but does not require as elaborate a reaction arrangement. It should be recognized that the pile shaft capacity in pull is smaller than the shaft capacity in push. Therefore, in a combination test -- push and pull -- the difference in capacity will not indicate the correct toe resistance of the pile.

When one designs and performs a lateral test, the following points should be considered:

- The method of applying horizontal loads, for instance, by inserting horizontal jacks between the heads of two adjacent piles in a group or a row, may not be acceptable in very stiff clay or dense granular soil unless the spacing between the piles is larger than 10 pile diameters;
- In most cases, it is not sufficient to measure the horizontal displacement of the pile head versus the applied load. To allow for an appropriate evaluation of the elastic behaviour of the pile-soil system, and in particular of the coefficient of subgrade reaction, k_s, or p-y curves, it is also necessary to measure head rotation and highly desirable to instrument the pile for the measurement of bending stress and curvature; and

- Since horizontal loads applied by the structures are generally of a transient nature (wind loads, earthquake, etc.) it may be necessary to provide similar transient loading conditions in the tests.

20.4 Presentation of Test Results

20.4.1 Static Load Test Results

The results of tests performed according to any of the methods described in Subsection 20.3.1 should be presented in a report conforming to the requirements of ASTM D-1143. Presentation of the results should include a load-movement curve and a time-movement curve.

20.4.1.1 Load-Movement Curve

The load-movement readings taken should be presented in a diagram with the loads on a linear scale on the ordinate, and the observed movements (at both the beginning and the end of each load increment) on a linear scale on the abscissa. To facilitate the interpretation of the test results, the scales for the loads and the movements should be selected so that the line representing the calculated elastic deformation of the pile will be inclined at an angle of about 20 degrees to the load axis. The elastic shortening is computed from:

$$\delta = \frac{QL}{AE} \tag{20.2}$$

where

δ = calculated elastic compression
Q = applied load
L = pile length
A = cross-sectional area of the pile
E = elastic modulus of the pile material.

20.4.1.2 Time-Movement Curve

The time-movement readings should be presented in a diagram with the time on a linear scale on the abscissa and the observed movements on a linear scale on the ordinate.

20.4.2 Rapid Load Test Results

The force and settlement time histories are recorded during the test using a computer attached to the load cell and the optical displacement sensor. Computer software provides immediate load and displacement time histories as well as load versus displacement plots. In many cases, the pile head is fitted with an accelerometer that allows for measurement of acceleration at the pile head during the test. In this case, the computer software will plot the acceleration time history. The acceleration can be integrated with respect to time to obtain the velocity time history.

20.5 Interpretation of Test Results

20.5.1 Interpretation of Static Load Test Results

There is a wide variety of criteria for interpreting test-loading results, which can be divided into two groups:

- Criteria governing the acceptance of the tested pile. Typical of these is the method that once was specified in the National Building Code (1970 edition). In this method, no consideration is given to the failure load of the pile. In most cases, a pile is deemed acceptable if the observed settlement of the pile head is within specified limits, which are selected independently of the type and length of the pile. These methods overestimate the capacity of a short pile and underestimate the capacity of a long pile and should not be used.

- Criteria defining the failure load of the tested pile, from which the allowable load may be computed by applying a factor of safety. Such methods should be used because they provide a better understanding of pile capacity and behaviour.
- Many different failure criteria have been proposed in the technical literature, a number of which have been discussed by Fellenius (1975b, 1980). The failure loads as evaluated from the different criteria show a range of about 30% from the lowest to the highest. The person responsible for the evaluation of the results of a test must choose the criterion of his or her own technical preference. As a guide, three methods are briefly described and commented on below.

20.5.1.1 The Offset Limit Load

The offset limit load, (Davisson, 1973), of a pile is the load that produces a movement of the pile head, which is equal to:

$$A = \delta + (4 + 8d)10^{-3}\ (\mathrm{m}) \qquad (20.3)$$

where

A = the movement of the pile head at the offset limit load elastic shortening of the pile (m)
d = diameter of the pile at the toe, or the base of an expanded base pile (mm)
δ = the elastic shortening of the pile calculated from Equation 20.2 (mm)

When using the offset limit for expanded base piles, the theoretical diameter of the base should be used as the pile diameter. The offset limit load is intended for driven piles.

When applied on bored piles, it becomes impractically conservative. O'Neill and Reese (1999) define the failure load for bored piles as the load that produces a movement of the pile head equal to 5% of the toe diameter. For large-diameter piles, this definition of failure may occur at such large settlement that the settlement at the design load will govern the design.

The offset limit load criterion is represented by a straight line on the load-movement curve (Figure 20.1). The load-movement curve of the test intersects the line at point F, the ordinate of which is the limit load of the test.

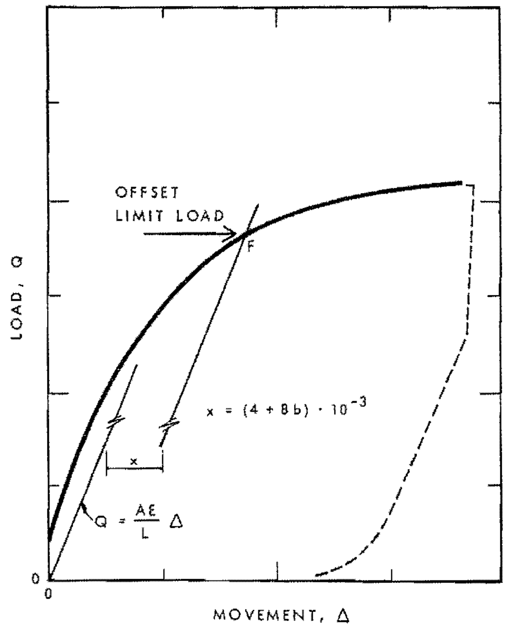


FIGURE 20.1 *Construction of the offset limit load (after Fellenius, 1980)*

The offset limit load provides a failure value that is always conservative. A primary advantage is that the actual limit line can be drawn in the load-movement diagram before the test is started. The limit load can, therefore, be used as an acceptance criterion for proof-tested piles in contract specifications; for instance, 'the pile head movement for 180 % of the design load shall not exceed the elastic free-standing column compression of the pile plus (4 + 8d) x 10^{-3}, where d is the pile toe diameter'.

The disadvantage of the offset limit load lies in the difficulty in determining Young's modulus, E, for concrete piles and concreted pipe piles. Also, the criterion is very sensitive to errors, or inaccuracies in both load and movement values. Furthermore, in an actual case, the interpreted limit load is often more a value that represents the boundary between semi-elastic and semi-plastic ranges of pile-movement behaviour. In other words, it is sometimes too conservative and can deviate significantly from the plunging failure, which, when it occurs, is then conceived as the true ultimate failure load.

The offset limit load criterion is primarily intended for interpreting quick testing methods, but it can also be used when interpreting results from the slow methods. It is not suitable for testing methods that involve unloading and loading cycles.

20.5.1.2 The Brinch-Hansen Failure Criterion

The Brinch-Hansen criterion (Brinch-Hansen, 1961) assumes that the shape of the pile load-movement curve is such that when the movements, Δ, are plotted along the abscissa in a diagram with $\sqrt{\Delta}/Q$ along the ordinate, the data points lie on a straight line having a slope, C_1, and a y-intercept, C_2 (see Figure 20.2). The criterion is as follows: the load, Q_u, for the movement, Δu, is the failure load, if the load 0.80 Q_u gave the movement 0.25 Δu.

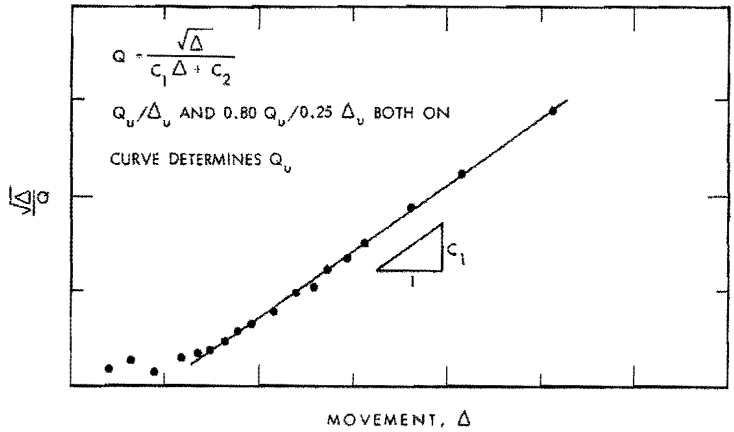


FIGURE 20.2 *Ultimate failure according to the 80 % - Criterion (after Fellenius, 1980)*

After determining the slope, C_1, and y-intercept, C_2 of the plotted line, the ultimate failure load and movement at failure are:

$$Q_u = \frac{1}{2\sqrt{C_1 C_2}} \tag{20.4a}$$

$$\Delta_u = \frac{C_2}{C_1} \tag{20.4b}$$

The Brinch-Hansen criterion has the advantage that it agrees well with the plunging failure and is subjectively conceived as a true failure value.

The criterion is applicable to both quick- and slow-testing methods. However, it is not suitable for methods that include unloading and loading cycles.

The disadvantage of the Brinch-Hansen 80 % criterion, as opposed to the offset limit load criterion, is that plotting and calculations involved cannot be performed in advance of the test loading.

The Brinch-Hansen failure load is only 'true' if the failure load was reached during the test and, in particular, if the point $0.80Q_u/0.25\Delta u$ plots on the test curve. If these conditions are not fulfilled, plunging failure has not been obtained in the particular test. When plunging failure has not been obtained in a test, then for design purposes, the failure load must be assumed to be equal to the maximum load applied. The test-loading curve must not be extrapolated to a failure value.

20.5.1.3 The Chin Failure Criterion

The Chin criterion (Chin, 1970) uses a plot similar to that of Brinch-Hansen, plotting Δ /Q along the ordinate and Δ along the abscissa. When failure is approached, these data points plot in a straight line, and the inverse slope of the line is defined as the failure load.

The Chin criterion always results in a failure load that is greater than the maximum test load applied and is therefore less useful. The value of the 'Chin line' lies in that it can be used to indicate potential damage to the pile, as can be shown by sudden changes (i.e., curves or kinks) in the line.

20.5.2 Interpretation of Rapid Load Test Results

There are two approaches to analyse the rapid load test (RLT) data. In the first approach, the unloading point method, the pile is assumed to behave as a rigid body during the rapid load test. The second approach, which involves a signal matching process is more rigorous and accounts for the pile flexibility. Both approaches are described here.

20.5.2.1 Unloading Point Method (UPM)

The bearing capacity and the load-deflection curves of the pile may be interpreted from the RLT results using the Unloading Point Method (UPM) (Middendorp et al., 1992). The UPM assumes that the pile moves as a rigid body during the RLT and consequently, the inertial force can be evaluated based on the mass of the pile and the acceleration measured at the pile head during the test. The governing equilibrium equation for a rigid body in a one-degree of freedom model under dynamic loads is given by

$$F(t) = kw + c\dot{w} + m\ddot{w} \tag{20.5}$$

where

$w, \dot{w}, \ddot{w}$ represent displacement, velocity and acceleration, respectively. The k, c and m represent the stiffness, damping and mass, respectively. The pile mass is known, displacement time history and force, $F_{RLT} = F(t)$, are measured and the acceleration, velocity, are measured or computed (using integration and/ or differentiation) from the load test. Therefore, the damping, c, and stiffness, k, are the only unknowns in Equation 20.5. The velocity is zero at the maximum displacement, w_{max}. In this instance, $t = t_{wmax}$, the equilibrium equation becomes

$$F_{RLT} = (t_{wmax}) = kw\ (t_{wmax})\ m\ddot{w}\ (t_{wmax}) \tag{20.6}$$

Equation 20.6 can be used to calculate the static component of the soil resistance, $F_{wstatic}$, at this particular moment in time, when the velocity is zero:

$$F_{wstatic} = kw(t_{wmax}) = F_{RLT}(t_{wmax}) - m\ddot{w}\ (t_{wmax}) \tag{20.7}$$

Assuming that $F_{wstatic}$ equals the maximum soil resistance and its value remains constant for the duration from the time step of maximum force (unloading point, $F_{RLT} = F_{max}$), t_{Fmax}, to the time step of maximum displacement, t_{wmax} (Figure 20.3), the damping constant may be approximated by

$$c = \sum_{t_{w\max}}^{t_{F\max}} \frac{F_{RLT} - F_{wstatic} - m\ddot{w}}{\dot{w}} \quad (20.8)$$

The damping constant, c, is averaged over the duration between the instants of maximum force and displacement and assumed to be constant throughout the entire RLT. The RLT force, pile mass, acceleration and velocity are all measured from the actual load test while $F_{wstatic}$ is calculated from Equation 20.7. After c is calculated, the original governing equation of a rigid body to dynamic loads is rearranged and the Derived Static Force (DSF) is calculated along the entire test, i.e.:

$$F_{static} = kw = F_{RLT} - m\ddot{w} - c\dot{w} \quad (20.9)$$

A failure criterion can be implemented (e.g., Davisson, 1973) to predict the bearing capacity of the pile from the derived static force-displacement curve, similar to the static load test case.

The UPM is successful for rigid piles. However it lacks accuracy for flexible piles where the main assumption of rigid body motion does not hold. For the evaluation of the applicability of the UPM to a specific pile, its flexibility is characterized by the parameter N_w. This parameter is calculated as

$$N_w = \frac{c_p t}{L} \text{ and } c_p = \sqrt{\frac{E}{\rho}} \quad (20.10)$$

where

t is the duration of RLT force, E is the Young's modulus of the pile, ρ is the density of the pile, L represents the length of the pile and c_p represents the primary wave velocity of the pile. The accuracy of UPM deteriorates and the method becomes unreliable for flexible piles with $N_w \leq 12$. Therefore, an alternative method should be employed for the analysis of flexible piles under RLT loads.

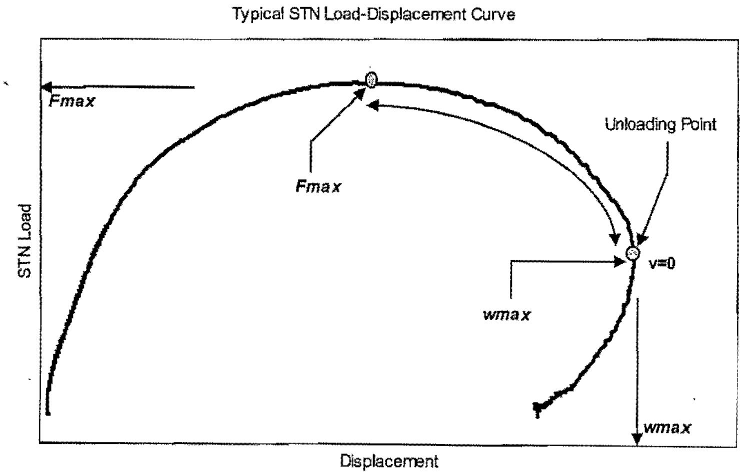


FIGURE 20.3 *Typical Statnamic Load-Displacement Curve*

20.5.2.2 Dynamic Analysis and Signal Matching Approach

El Naggar and Novak (1992, 1994) developed an approach to analyse the pile response during the STN test and used the results of the analysis to predict the pile capacity. In this approach, a one-dimensional model is used to represent the pile-soil system that is subjected to a transient dynamic load (due to the STN or PLT). The model accounts for soil non-homogeneity and non-linearity and slippage at the pile-soil interface as well as energy dissipation through damping. The soil parameters are adjusted iteratively until a best match between the measured and calculated signals is achieved. The model uses standard geotechnical soil parameters to model the soil behaviour during the RLT. The model was used to analyse several Statnamic load tests and to predict pile capacity. The results illustrated that the model predictions compared well with static load test results.

El Naggar and Baldinelli (2000) further improved the model to account for the post-peak behaviour displayed by some soil types and other numerical refinements. They also developed an automatic matching technique (AMT) to facilitate the signal matching process for the analysis of pile response during the RLT. The AMT has several advantages including: reducing the bias in the results due to the initial selection of the soil parameters; increasing the accuracy and reliability of computed pile capacity; and reducing computational time thus allowing for the analysis of the test results in the field.

It is then possible to compute the pile capacity and make a timely decision on its suitability. This approach was used to analyse the results of six STN load tests and the comparison between the measured and computed results was satisfactory.

21 Inspection of Deep Foundations

21 Inspection of Deep Foundations

21.1 Introduction

The quality of a deep foundation is governed by the methods used to install it. The appropriate choice of installation procedure and equipment, good workmanship, and tight control of all installation work are essential to the construction of a good deep foundation. Consequently, inspection (review) is of utmost importance. Sentence 4.2.2.3(1) of the National Building Code of Canada (2005) requires that:

> "A field review shall be carried out by the designer or by another suitably qualified person to ascertain that the subsurface 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;
- interpret pile driving data properly, particularly when piles are driven into rock; and
- evaluate actual soil conditions in bored piles.

21.2 Documents

Good inspection practices begin prior to actual construction, with the examination of all design documents. The following should be provided to the inspector for review before the start of construction and then should be kept for reference at the site:

- soil investigation report;
- all drawings pertaining to the foundation; specifications; contract; and
- any other documents on special design features or assumptions, such as design briefs, hammer data sheet, etc.

On the foundation drawings, the exact location of each deep foundation unit should be indicated, and each unit should be identified by a unique designation: pile number, column number, or structure designation followed by the pile number. This designation should be used for reference throughout the construction and the inspection process.

If any of the documents contain unclear or contradictory matter, this should be reported by the inspector and clarified immediately.

21.3 Location and Alignment

21.3.1 Location

The location of each deep foundation unit should be staked in advance and checked immediately prior to installation of each unit. After the installation is complete, the location of each unit should be checked against the design location and permissible deviations, as indicated on the design documents.

Article 4.2.7.4 of the National Building Code of Canada (2005) states

> "Where a deep foundation unit has not been placed within the permissible deviations referred to in Article 4.2.7.3, the condition of the foundation shall be assessed by the designer, any necessary changes made and action taken as required in Article 2.2.4.7 of Division C."

It is usually impractical to limit the location deviations to values smaller than 70 mm for deep foundation units. A deviation of 50 mm would be the limit even under favourable conditions. In marine construction, where piles are driven into or through water, the practical value of tolerance of location is 150 mm. The foundation design should allow for this expected variation.

Units constructed in the wrong locations will result in: modified load distribution on the different units in a group; and modified stress distribution in the cross-section of single acting units and, potentially, a reduction of the structural capacity of the unit.

Subsection 4.2.7 of the National Building Code of Canada (2005) requires that when a deep foundation unit is wrongly located, the condition of the foundation shall be assessed by the person responsible for the design and the necessary changes made. However, it should be recognized that as a result of a stringent inspection (quality control), the actual safety has been improved by the assurance gained through the inspection. Therefore, off-locations not giving rise to more than 15 % increase of load on a pile can normally be accepted.

21.3.2 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.

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;
- practically all piles, particularly when driven, are more or less out of design alignment; (a perfectly straight pile is a theoretical concept, seldom achieved in practice);
- the 2 % alignment limit refers to the direction or slope of the pile at cut-off elevation, because alignment problems affect the pile cap and the superstructure and not the pile-soil behaviour below the pile cap; and
- only the curvature of a pile is of importance for its structural and geotechnical behaviour.

Although the mechanism is not fully understood at present, vibrations generated at the toe of the pile during driving in weak soils may cause the pile to deflect from its vertical path. This deflection may cause the pile toe to move toward adjacent piles in a group, or to induce unsafe curvatures in the pile column. H-piles are inherently more prone to this behaviour than circular sections.

21.3.3 Curvature

When long piles are driven into any type of soil, or shorter piles are driven through soils containing obstructions, the piles can bend, dogleg, and even break without this being recognized by the usual inspection means after the driving. Pipe piles that are closed at the toe provide the possibility of inspection of the curvature and integrity given by the open pipe. It is normally not possible to inspect a precast concrete pile for binding. However, by casting a centre tube in the precast concrete pile, access is provided for inspection down the pile.

Often, the inspection down the pile is only carried out by lowering a flashlight into the pipe or centre tube to check that the pile is sound, which it is considered to be if the flashlight can reach the bottom of the pile while still being seen from above. However, dust and water can obstruct the light, and if the light disappears because the pile is bent, there is no possibility to determine whether the pile is just gently sweeping, which is of little concern, or whether the pile is severely bent or doglegged. In such a case, a specially designed, but simple, plumb-bob probe can be used to confirm undamaged piles and to provide data that aids in judging and evaluating a suspect pile.

The plumb-bob probe consists of a staff pipe with dimensions chosen so that it, theoretically, will 'jam' inside the pipe or centre tube at a predetermined limiting bending radius. For obvious reasons, both the probe and the centre tube should be made from standard pipe sizes. A suitable size for a centre tube in precast concrete piles is 1.5 inch schedule 40 (inside diameter 40.9 mm), with a corresponding size of pipe for the probe of 1.0 inch schedule 80 (outside diameter 33.4 mm). The probe must be made from heavy, stiff pipe. The probes used in steel pipe piles should have about the same ratio between the outside diameter of the probe and the inside diameter of the pipe, i.e., about 0.8.

The length of the probe is determined by the desired limiting bending radius and the annulus between the probe and the tube, as given by the following relation (see Figure 21.1):

$$R = \frac{L^2}{8t} \tag{21.1}$$

where

R = bending radius
L = length of the plumb-bob probe
t = annulus = $D_1 - D$
D_1 = inside diameter of the pipe or centre tube
D_2 = outside diameter of the plumb-bob probe

When the plumb-bob probe is passed down the pile it is affected by numerous irregularities, such as an oval shape and diameter tolerances of pipes used, unavoidable 'snaking' of the centre tube cast in a concrete pile, offsets when splicing the pile, etc. However, the plumb-bob is not intended to be an exact instrument for determining bending. Instead, it is a refinement of the slow, crude, and imprecise inspection by eye and flashlight. Its main purpose is to save piles, which otherwise may have to be rejected.

Consequently, when inspecting the limiting bending radius, one should not use the approach according to calculations of the bending moment, M (M = EI/R), and work from the calculated bending stress. If so, limiting radii in the order of 400 m, and more, would result. Probes designed according to such strict values are impractical and cause more problems than they solve.

Practice has shown that the most suitable probes are those designed for limiting radii of 200 m and 100 m, the 100-m probe being used only if the 200-m probe 'jams'. Any 'jamming' (inability of the probe to reach the bottom of the pile) would then be evaluated, considering the location of the 'stop', the pile driving records, the behaviour of the neighbouring piles, the intended use of the pile, etc.

Centre tubes used in concrete piles must be seamless and smooth inside. They can consist of steel tubes or PVC pipes. The steel tubes are preferred for practical reasons, as they are stiffer and heavier. The PVC pipes are cheaper but more apt to snake laterally, to float in the fresh concrete, and to be dislocated by the vibrator.

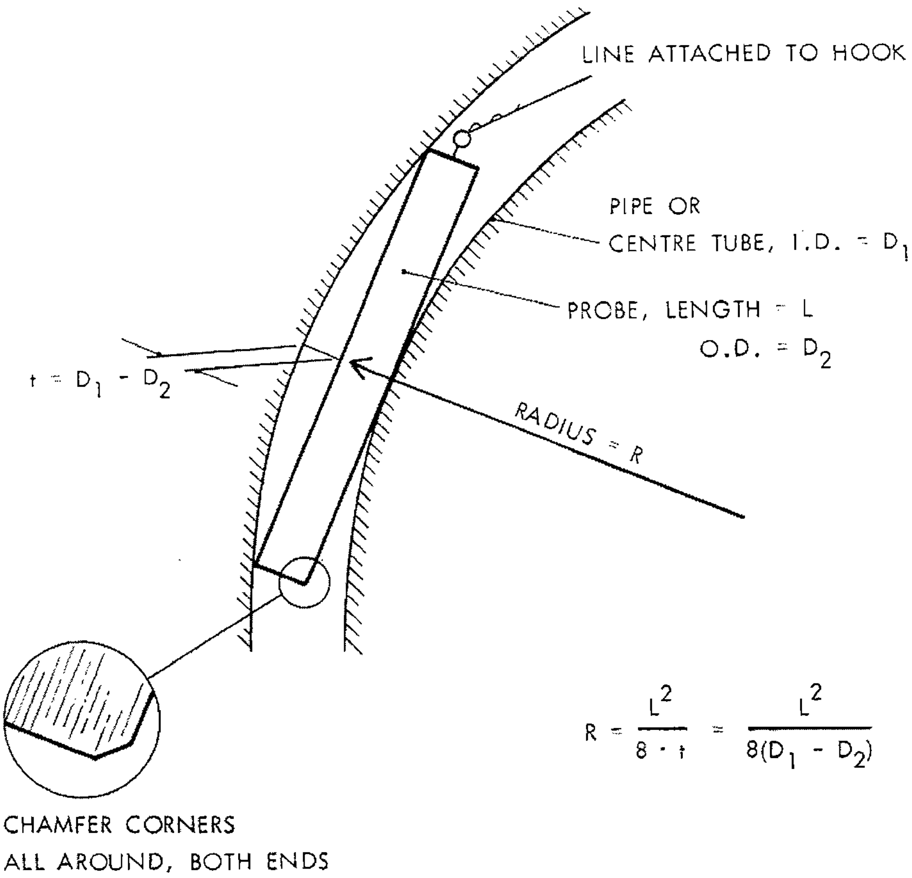


FIGURE 21.1 *Design of the plumb-bob probe used for inspection down pipe piles, or centre tubes in precasts concrete piles (after Fellenius, 1983)*

The splicing of the tubes must be made square and with outside couplings to ensure that no inside lips or edges obstruct the passage of the probe. Conical couplings are unsuitable because the tubes (particularly the PVC tube), expand thermally from the heat generated by the concrete when curing and arising from steam curing when used. The subsequent longitudinal force is substantial and would squeeze the tube end into the conical coupling and deform it, causing an obstruction inside the centre tube. Naturally, all couplings must be almost water tight to prevent the cement solution from entering the tubes.

To ensure a straight centre tube, it must be supported in the casting form and tied to the longitudinal reinforcement. A centre tube is considered straight in the casting form before pouring the concrete if the maximum deviation of the tube as measured over a distance of 4 m is 5 mm. This deviation tolerance corresponds to a calculated bending radius of 400 m. Thus, the limit is quite liberal. Experience has shown that there is no difficulty in having the tubes cast straight in the piles.

Piles with centre tubes are usually equipped with pile shoes. Where this is the case, it is necessary to supply the base plate of the shoes with a receiving pipe to centre the tube in the pile, and to ensure positively that the tube at the toe of the pile (the zone of particular importance in the inspection) is straight.

If splices are used in the pile, a similar centring of the tube is necessary to enable the probe to pass through the splices without encountering difficulties due to the offset of centres, 'knees', etc.

It is advisable to check that the tubes are straight and unobstructed after casting by pushing the probe into and through the centre tube while the pile lies on the ground in the casting yard (the probe has to be attached to the end of a standard pipe of small diameter or pulled through by a line blown ahead through the tube).

21.4 Inspection of Pile Driving Operations

21.4.1 Introduction

Dynamic monitoring of the pile driving should always be considered. The dynamic measurements will not only enable an estimate of the pile-bearing capacity of a large number of piles for the equivalent cost of one test loading, but will also provide valuable information on the hammer-and-pile performance including the energy actually delivered to the pile, stresses in the pile, and variations of behaviour between hammers and between piles at the site.
If it is recognized during the design phase that dynamic monitoring will be used at a site as a means of quality control, the allowable load can often be increased due to the improved assurance gained.

Items of importance in driving different types of piles are discussed in Chapter 19. The following checklists are given for the guidance of inspection personnel.

21.4.2 Driving Equipment

Items to be checked and recorded include the following:

1. Type of hammer is as specified.

2. For drop hammers:
   - mass of the hammer
   - type of crane and trip mechanism drop height
   - sliding condition in the leads.

3. For steam hammers:
   - type (single or double acting), make, serial number
   - mass of the hammer and ram
   - positions of the valves, trips, and resulting stroke
   - steam pressure
   - energy rating
   - blows per minute when driving piles at regular intervals and when experiencing hard driving
   - general condition of the hammer.

4. For diesel hammers:
   - type, make, serial number
   - mass of the hammer and ram stroke
   - energy rating
   - blows per minute when driving piles at regular intervals and when experiencing hard driving.

5. For driving cap:
   - mass of the cap
   - dimensions as related to pile, hammer, and lead dimensions
   - type of capblock
   - thickness of capblock
   - condition of the capblock (this should be checked regularly, and burned, crushed, or broomed capblocks should be replaced immediately)
   - type of hammer cushions used
   - thickness of hammer cushion
   - condition of hammer cushion.

6. Type and characteristics of other equipment such as drive heads, followers, etc.

21.4.3 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 for each shipment meets the specifications (cut off about 0.4 m of the pile, weigh the cut piece and calculate the exact size of the pile from the known unit weight of the material);
   - that the condition of the piles is satisfactory (not damaged or bent);
   - that toe and head protections, if any, are as specified;
   - that proper handling and storage procedures are followed;
   - that the head of the pile is perpendicular to the longitudinal axis;
   - 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.

And for prestressed piles:

- that there is a certificate indicating that the prestressing cables meet specifications;
- that the prestressing procedure and forces used are as specified;
- that strand slippage is minimal (about 2 mm maximum when releasing the wire pretension).

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 the 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, sparred, etc.);
- that splices, if any, conform to specifications;
- whether strands appear recessed at the segment ends.

c) For wood piles:
- that there is a certificate indicating the species and grade of timber;
- that there is a certificate regarding protective treatment, where specified;
- that length and dimensions at toe, mid-height, and head of piles are recorded;
- that the piles are straight within the specified tolerances;
- that proper handling and storage procedures are followed;
- that points or shoes, if any, conform to specifications and are properly installed;
- that protective treatment is intact over the full surface of the pile where specified.

21.4.4 Driving Procedures

Items to be checked or noted include:

- general information such as date, weather conditions, pile identification;
- the exact location of the pile;
- the stability and alignment of the driving rig and leads and the number of blows;
- deformations of the pile under blows at various depths;
- the position and quality of splices;
- the location, time, and duration of any interruption in driving;
- elastic deformations, permanent set (millimetre per blow) determined from the penetration for the final five or ten blows;
- the elevations of ground surface, pile toe and cut off;
- any erratic or unusual pile behaviour, together with a record of the time of observation and corresponding toe elevation; possible heave of adjacent piles; and
- other pertinent information.

21.5 Inspection of Compacted Concrete Piles

21.5.1 Introduction

The construction of compacted concrete piles requires the use of special equipment and specialized technique. It should be undertaken only by contractors experienced in the construction of this particular type of deep foundation.

21.5.2 Equipment

The equipment should be checked to ensure that it conforms to the specifications or to good work practices. Of particular importance are:

- the size and mass of the ram;
- the dimensions of the driving tube; and
- the condition of the clamping equipment to hold the driving tube when forming the base.

21.5.3 Installation

Items to be checked or noted include:

- general information such as 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; mass of the hammer; number of blows per 300-mm penetration;
- the elevation of the bottom of the driving tube before forming the base;
- specifics of the concrete for the base: the mix used, strength determined from the compacted samples;
- the formation of the base: number of buckets and number of blows per bucket; hammer mass, drop height, and resulting energy per blow; final volume of the base; and final driving energy for the last bucket;
- elevation of the bottom of the ram when forming the base;
- placement of reinforcement, if any;
- seating into the base of the permanent liner, if any;
- quality of concrete for the shaft: mix, slump, freshness; and that there are test cylinders for each day of the pour, of each 30 m, 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 backfiring of the annular space around the permanent liner.

21.6 Inspection of Bored Deep Foundations

21.6.1 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; 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, especially pH, hardness, chlorides and sulphates content plus such compounds as may be considered hazardous by the authorities.

21.6.2 Boring/Drilling

Items to be checked or noted include:

- general information such as date, weather conditions, unit identification, time excavation was started and completed;
- location of the unit;
- conformity of the boring/drilling technique to the specifications or to good practice;
- alignment and dimensions of the drilled shaft at regular intervals;
- the technique and equipment used to penetrate water-bearing strata, if any;
- the technique and equipment used to penetrate large obstructions, if any;
- log of sediments and rock penetrated during drilling;
- 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 should be done);
- cleanliness of the bottom and sides of the drilled shaft and permanent liner, if any;
- rate of seepage into the drilled shaft;
- quality of the bentonite slurry, if any; and
- losses of bentonite slurry, if any, (time, elevation and quality).

21.6.3 Concreting

After the drilled shaft has been inspected and accepted, placing of reinforcement and concrete may proceed. Items to be checked or noted include:

- general information such as 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;
- the placing method: 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 the position of the reinforcing cage conform to the drawings and specifications;
- that the weight of the concrete is adequate to balance the existing groundwater pressure;

- quantity of concrete compared to the height of shaft;
- concrete level in the casing during casing withdrawal;
- vibration of the top of the concrete;
- 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.

21.6.4 General

In most projects, the contractor performs the work as the contractor thinks best and takes responsibility for it. The inspector should not direct the contractor, but keep good and complete records, as indicated above. Copies of these records should be provided to the contractor on a regular basis. If the inspector discovers methods and events that are not in accordance with the contract or good practice, the inspector should immediately bring this to the attention of the supervising engineer and inform the contractor about this action. Availability of digital cameras makes it straightforward to take one or more photographs of the site each day to visually document the progress of the work, location of equipment, etc. It is now also possible to employ web-cams to provide off-site review of construction activities.

Additional comments and discussion that may benefit the inspector are given by Davisson (1972), the Deep Foundation Institute (1979a,b), Hunt (1979) and O'Neill and Reese (1999).

22 Control of Groundwater

22 Control of Groundwater

22.1 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 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 under full control 24 hours a day, 7 days a week, to avoid fluctuations likely to cause instability of the excavation;
- effective filters be provided where necessary to prevent loss of ground;
- adequate pumping capacity as well as standby pumping and power capacity be provided;
- pumped water be discharged in a manner that will not interfere with the excavation and comply with environmental requirements;
- 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 0.5 m to 1.5 m below the bottom of the excavation to ensure dry satisfactory 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.

22.2 Gravity Drainage

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

22.3 Pumping From Inside the Excavation

Frequently, groundwater levels are controlled by pumping inside the excavation. However, pumping from outside the excavation is often a safer approach.

The location of drainage channels leading to the sumps should be a matter for careful consideration 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 that is surrounded by graded gravel filter material or wrapped in a geotextile cloth.

Loss of ground from around the sump must be prevented. A good 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.

22.3.1 Pumping From Unsupported Excavations

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. This must be prevented by filtered drains. Alternatively, the face of the excavation should be cut back to a stable slope, or the water level lowered by pumping.

22.3.1.1 Heave Due to Artesian Pressure at Depth

Where an excavation is dug into a clay deposit underlain by a pervious stratum under artesian pressure, pressure and seepage may result, leading to instability of the excavation. Where an excavation is underlain by an impermeable layer, such as a stratum of silt or clay, which is, in turn, underlain by an aquifer in a pervious stratum of sand under artesian pressure, upward seepage from the deeper stratum may keep the bottom of the excavation wet, even though drainage pumps may be in use. If either of these situations exist, it may be necessary to lower the water pressure in the sand stratum below the bottom of the excavation by means of relief wells.

The above-mentioned case is illustrated in Figure 22.1. The hydrostatic head in the deep sand below the impervious clay layer can be somewhat higher than the bottom of the excavation. When the effective stress at point A approaches zero, the situation becomes unstable and dangerous. Therefore, the pore pressure at point A should not exceed 70 % of the total stress at this point. Otherwise, heave may occur in the bottom of the excavation. This requirement may be relaxed somewhat for narrow excavations, because the shear strength of the clay becomes a significant contributor to uplift resistance.

Bottom heave in excavations in clay underlain by pervious strata under artesian conditions is generally sudden and catastrophic. Appropriate care should be exercised to avoid such failures.

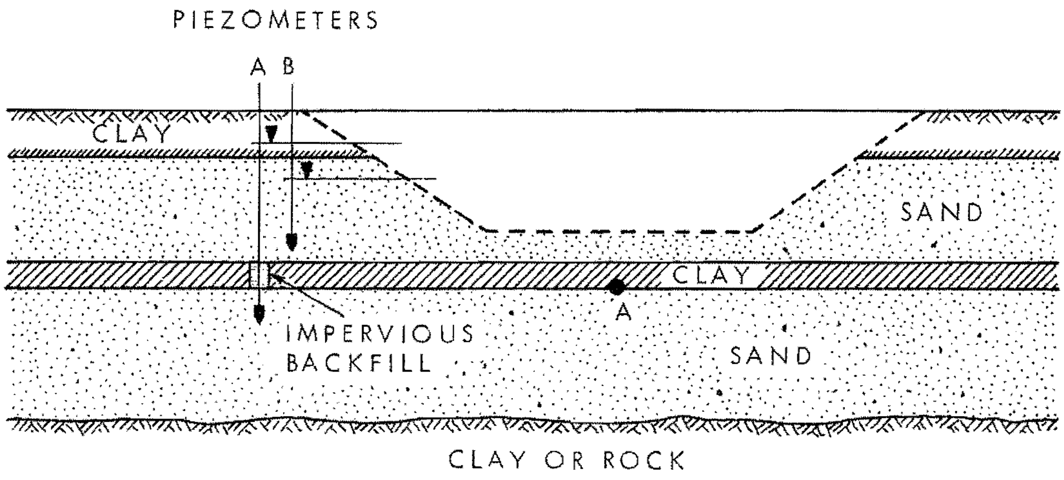


FIGURE 22.1 *Artesian groundwater condition below excavation*

22.3.1.2 Use of Relief Wells

If relief wells are installed within the excavation, 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 to control upward seepage and achieve a dry, stable bottom. Stratification of the soil will also affect the allowable uplift pressure. For additional information, see NAVFAC (1971).

22.3.2 Pumping From Sheeted Excavations

If an excavation is made using sheet piling or an impermeable diaphragm taken into an impermeable stratum, the flow of water in the overlying pervious ground will be substantially reduced and the dewatering of the area enclosed by the cofferdam is simplified.

22.3.2.1 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 referred to as piping, heave, or boiling. Instability occurs if the vertical seepage exit gradient at the base of the excavation equals about 1. To prevent piping or heave, sheeting must penetrate a sufficient depth below the base of the excavation. Figure 22.2 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 personnel and equipment operating on the subgrade. To avoid this, the sheeting penetration should be sufficient to provide an adequate safety factor against piping or heave.

The sheeting penetration required in layered subsoils is given in Figure 22.3.

22.3.2.2 Heave Due to Artesian Pressure at Depth

See Subsection 22.3.1.1.

22.3.2.3 Use of Relief Wells

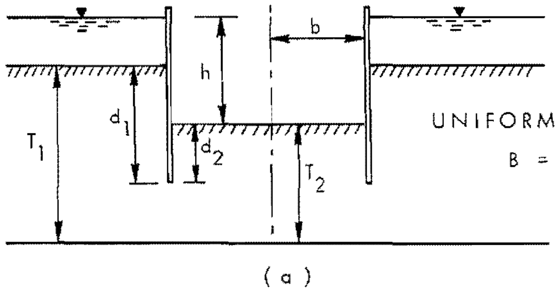
See Subsection 22.3.1.2.

22.4 Pumping From Outside the Excavation

The objective 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. Some methods used for lowering the groundwater level outside an excavation are as follows (a pump test is often advisable before selecting the method to use):

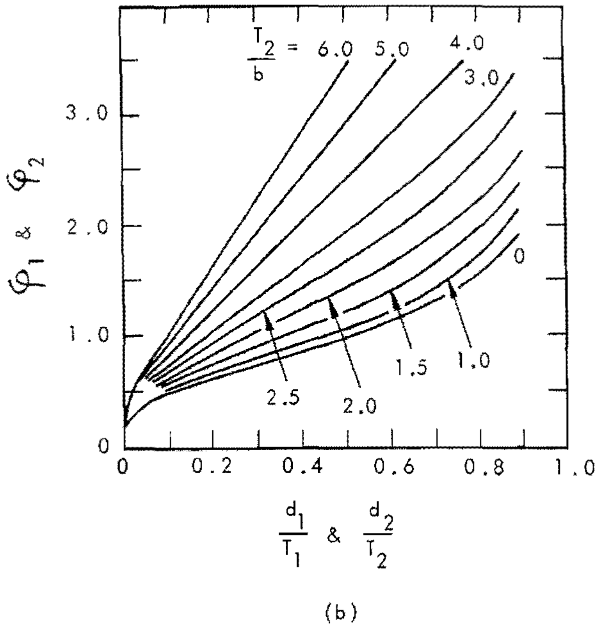
- excavated wells or sumps with independent pumps.
- a number of small diameter well points (the well-point system);
- multiple bored filter wells with independent submersible pumps in each well (the deep-well system), or where the quantities of water to be pumped are small, well point in jet eductors (the eductor system);
- multi-stage installations of (ii) and (iii) above; and
- vacuum well methods.

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



NOTE:

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



1) FOR TWO PARALLEL WALLS

$$i_{exit} = \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2}$$

$$Q = \frac{kh}{\varphi_1 + \varphi_2} \quad \text{PER UNIT LENGTH}$$

φ_1 AND φ_2 FROM FIG. (b)

2) FOR A CIRCULAR EXCAVATION

$$i_{exit} = 1.3\ \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2}$$

$$Q = 0.8\ \frac{kh}{\varphi_1 + \varphi_2}\ 2\pi R$$

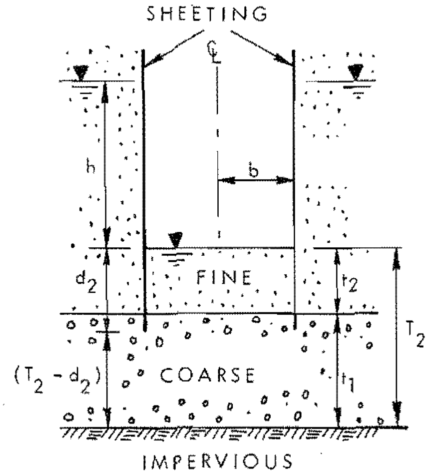
3) FOR A SQUARE EXCAVATION

$$i_{exit} = 1.3\ \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2} \quad \text{(MIDDLE SECTION OF THE SIDES)}$$

$$i_{exit} = 1.7\ \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2} \quad \text{(IN THE CORNERS)}$$

$$Q = 0.7\ \frac{kh}{\varphi_1 + \varphi_2}\ 4B$$

FIGURE 22.2 *Penetration of sheeting and exit gradient for isotropic sand*

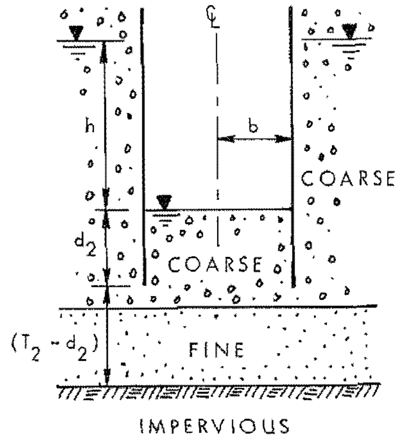


Coarse Sand Underlying Fine Sand

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 Figure 22.2.

If top of coarse layer is at a depth below sheeting tips greater than width of excavation, exit gradients of Figure 22.2 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 10 times that of fine layer, failure head (h) = thickness of fine layer (t_2).

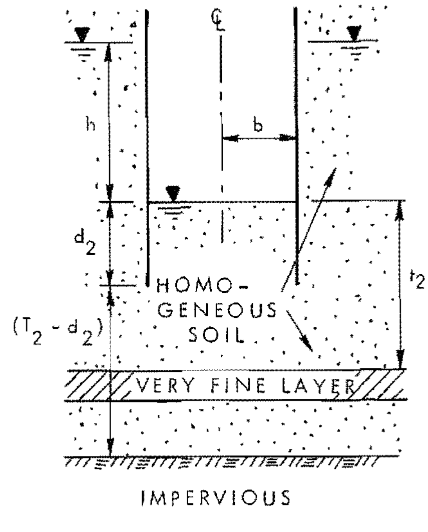


Fine Sand Underlying Coarse Sand

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 Figure 22.2.

If top of the fine layer lies above sheeting tips, the exit gradients of Figure 22.2 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 Figure 22.2 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.

FIGURE 22.3 *Penetration of sheeting required to prevent piping in stratified sand*

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 (Figure 22.4) 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.

The details of these methods and their design are given in various textbooks and references. (See Mansur and Kaufman, 1962; Terzaghi and Peck, 1967; Cedergren, 1977; Delleur, 1999; Loughney, 2001 for a comprehensive description.)

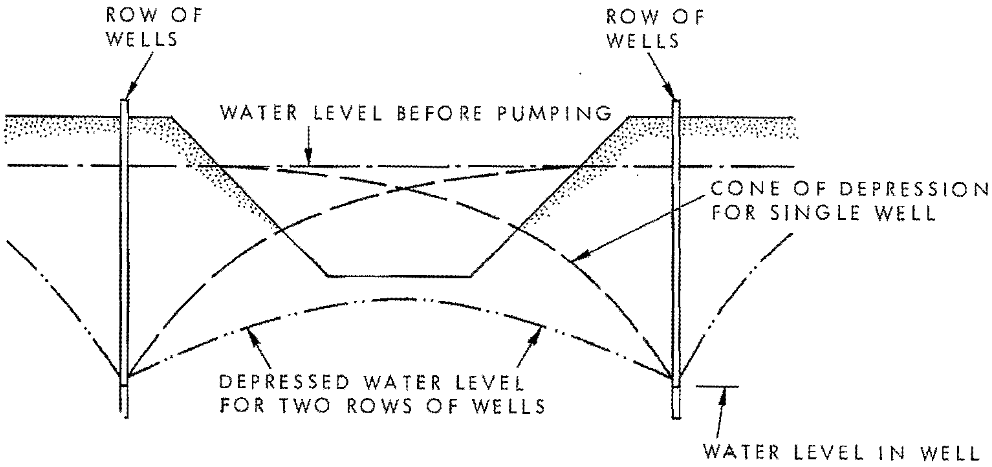


FIGURE 22.4 *Reduction of water levels below an excavation by means of a well groundwater-lowering system*

23 Geosynthetics

23 Geosynthetics

23.1 Introduction

Geosynthetics, for the purposes of this chapter, include a variety of synthetic polymer materials that are specially fabricated to be used in geotechnical, geoenvironmental, hydraulic and transportation engineering applications. The choice of a particular type of geosynthetic is largely dictated by its design function. It is convenient to identify the primary function of a geosynthetic as being one of: separation, filtration, drainage, reinforcement, fluid/gas containment, or erosion control. In some cases the geosynthetic may serve dual functions.

Separation: The geosynthetic acts to separate two layers of soil that have different particle size distributions. For example, geotextiles are used to prevent road base materials from penetrating into soft underlying soft subgrade soils, thus maintaining design thickness and roadway integrity. Separators also help to prevent fine-grained subgrade soils from being pumped into permeable granular road bases.

Filtration: The geosynthetic acts similar to a sand filter by allowing water to move through the soil while retaining all upstream soil particles. For example, geotextiles are used to prevent soils from migrating into drainage aggregate or pipes while maintaining flow through the system. Geotextiles are also used below rip rap and other armour materials in coastal and river bank protection systems to prevent soil erosion.

Drainage: The geosynthetic acts as a drain to carry fluid flows through less permeable soils. For example, geotextiles are used to dissipate pore water pressures at the base of roadway embankments. For higher flows, geocomposite drains have been developed. These materials have been used as pavement edge drains, slope interceptor drains, and abutment and retaining wall drains. Prefabricated vertical drains (PVDs) have been used to accelerate consolidation of soft cohesive foundation soils below embankments and preload fills.

Reinforcement: The geosynthetic acts as a reinforcement element within a soil mass or in combination with the soil to produce a composite that has improved strength and deformation properties over the unreinforced soil. For example, geotextiles and geogrids are used to add tensile strength to a soil mass in order to create vertical or near-vertical changes in grade (reinforced soil walls). Reinforcement enables embankments to be constructed over very soft foundations and to build embankment side slopes at steeper angles than would be possible with unreinforced soil. Geosynthetics (usually geogrids) have also been used to bridge over voids that may develop below load bearing granular layers (roads and railways) or below cover systems in landfill applications.

Fluid/Gas (barrier) containment: The geosynthetic acts as a relatively impermeable barrier to fluids or gases. For example, geomembranes, thin film geotextile composites, geosynthetic clay liners (GCLs) and field-coated geotextiles are used as fluid barriers to impede flow of liquid or gas. This function is also used in asphalt pavement overlays, encapsulation of swelling soils and waste containment.

Erosion control: The geosynthetic acts to reduce soil erosion caused by rainfall impact and surface water runoff.

For example, temporary geosynthetic blankets and permanent lightweight geosynthetic mats are placed over the otherwise exposed soil surface on slopes. Geotextile silt fences are used to remove suspended particles from sediment-laden runoff water. Some erosion control mats are manufactured using biodegradable wood fibres.

Geotextiles are also used in other applications. For example, they are used for asphalt pavement reinforcement and as cushion layers to prevent puncture of geomembranes (by reducing point contact stresses) from stones in the adjacent soil, waste or drainage aggregate during installation and while in service. Geotextiles have been used as daily covers (ASTM D6523, D7008) to prevent dispersal of loose waste by wind or birds at the working surface of municipal solid waste landfills. Geotextiles have also been used for flexible concrete formworks and for sandbags. Cylindrical geotubes are manufactured from double layers of geotextiles that are filled with hydraulic fill to create shoreline embankments (Gilbert and Fowler 1997) or to dewater sludge (Fowler et al. 1996).

Geosynthetics can be broadly classified into categories based on method of manufacture.

Geotextiles are continuous sheets of woven, nonwoven, knitted or stitch-bonded fibres or yarns. The sheets are flexible and permeable and generally have the appearance of a fabric. Geotextiles are used for separation, filtration, drainage, reinforcement and erosion control applications.

Geogrids are geosynthetic materials that have an open grid-like appearance. The principal application for geogrids is the reinforcement of soil.

Geonets are open grid-like materials formed by two sets of coarse, parallel, extruded polymeric strands intersecting at a constant acute angle. The network forms a sheet with in-plane porosity that is used to carry relatively large fluid or gas flows.

Geomembranes are continuous flexible sheets manufactured from one or more synthetic materials. They are relatively impermeable and are used as liners for fluid or gas containment and as vapour barriers.

Geocomposites are geosynthetics made from a combination of two or more geosynthetic types. Examples include: geotextile-geonet; geotextile-geogrid; geonet-geomembrane; or a geosynthetic clay liner (GCL). Prefabricated geocomposite drains or prefabricated vertical drains (PVDs) are formed by a plastic drainage core surrounded by a geotextile filter.

Geosynthetic clay liners (GCLs) are geocomposites that are prefabricated with a bentonite clay layer typically incorporated between a top and bottom geotextile layer or bonded to a geomembrane or single layer of geotextile. Geotextile-encased GCLs are often stitched or needle-punched through the bentonite core to increase internal shear resistance. When hydrated they are effective as a barrier for liquid or gas and are commonly used in landfill liner applications often in conjunction with a geomembrane.

Geopipes are perforated or solid-wall polymeric pipes used for drainage of liquids or gas (including leachate or gas collection in landfill applications). In some cases the perforated pipe is wrapped with a geotextile filter.

Geocells are relatively thick, three-dimensional networks constructed from strips of polymeric sheet. The strips are joined together to form interconnected cells that are infilled with soil and sometimes concrete. In some cases 0.5 m- to 1 m-wide strips of polyolefin geogrids have been linked together with vertical polymeric rods used to form deep geocell layers called geomattresses.

Geofoam blocks or slabs are created by expansion of polystyrene foam to form a low-density network of closed, gas-filled cells. Geofoam is used for thermal insulation, as a lightweight fill or as a compressible vertical layer to reduce earth pressures against rigid walls (Horvath 1995).

Additional terminology for geosynthetics can be found in the standards ASTM D4439 and the Canadian General

Standards Board (CGSB 148.2-M89). The Specifier's Guide (IFAI 2004) published annually by the Industrial Fabrics Association International is a useful reference for geosynthetic products available in North America together with material property and performance values reported by the manufacturers. Additional references for design using geosynthetics are the books by Koerner (1997), Holtz et al. (1997), Shukla (2002) and chapters in the book by Rowe (2001). The collection of 127 "Geosynthetics Case Histories" (Raymond and Giroud 1993) is another valuable resource.

23.2 Geotextiles

The most common constituent polymers used in the manufacture of geotextiles for separation, filtration, drainage and reinforcement applications are high-density polyethylene (HDPE), polypropylene (PP) or polyester (PET). Properties associated with geotextiles, typical ranges for property values, and related test methods are summarized in Table 23.1.

TABLE 23.1 *General Properties of Geotextiles, Test Methods, Reference and Guidance Documents*

	Range[1]	Test method, reference or guidance document
Terminology		D4439-92[2]; CGSB 148.2-M89[3]
Physical Properties		
Type and construction		D123-03a
Polymer type	Typically HDPE, PP, PET	Koerner (1997)
Identification of polyester fibres		D276
Carbon black content for polyolefins (%)	2-3	D5596-03; D1603-01
Mass per unit area (g/m²)	135-1000	D5261-92; CGSB 148.1 NO. 2-M85
Thickness (mm)	0.25 - 7.5	D5199-01; CGSB 148.1 NO. 3-M85
Roll length, width, weight, diameter	variable	D3774-96
Specific gravity	0.9 – 1.4	D792-00; D1505-03
Stiffness (Mg/cm)	0 – 25,000	CGSB 148.1 NO. 14-93
Thread diameter/linear density		D204-02; D 861-01
Pore size distribution		D6767-02
Identification, storage and handling		D4873-02
Sampling		D4354-99; CGSB 148.1 NO. 1-94
Conditioning for testing		D 1776; CGSB 148.1 NO. 100-95
Specification Conformance		D4759-02
Mechanical Properties		
Compressibility	Nil to high	D6364-99
Grab tensile strength (kN)	0.45 – 4.5	D4632-91; CGSB 148.1 NO. 6.1-93
Wide-width strip strength (kN/m)	9 – 180	D4595-86
Confined tensile strength (kN/m)	18 – 180	FHWA[4] (1998)
Seam strength (% of tensile strength)	50 - 100	D4884-96
Cyclic fatigue strength (kN/m)	50 - 100	Koerner (1997)

	Range[1]	Test method, reference or guidance document
Burst strength (kPa)	350 - 5200	D3786-01; CGSB 148.1 NO. 6.1-93
Tear strength (N)	90 - 1300	D4533-91
Impact strength (J)	14 - 200	Koerner et al. (1986)
Puncture strength (N)	45 - 450	D4833-00; D6241-99
Interface friction (% of soil friction coefficient)	60 - 100	D5321-02
Pullout behavior (% of geotextile strength)	50 - 100	D6706-01
Bond strength (for geocomposites)		D7005-03
Hydraulic Properties		
Porosity (nonwovens) (%)	50 – 95	Equation 26.1
Opening size (sieve size) (mm) (No 10-200)	2 – 0.075	D4751-99a; CGSB 148.1 No. 10-94
Open area (wovens) (%)	0 – 36	Section 26.2.1
Permittivity (1/s)	0.02 – 2.2	D4491-991; CGSB 148.1 NO. 4-94
Permittivity under load (1/s)	0.01 – 3.0	D5493-93
Transmissivity (m^2/min)	1 x 10^{-5} to 2 x 10^{-3}	D4716-03; D6574-00
Sediment Control		
Turbidity and silt curtains		U.S. Army Corps of Engineers (1977)
Silt fences		D5141-96; Richardson and Middlebrooks (1991); Trow (1996)
Durability Properties		
Selection of test methods for durability testing		D5819-99; D5970-96
Installation damage (% of geotextile strength)	0 - 70	D5818-95; FHWA (1997); WSDOT[5] (2004)
Unconfined tensile creep		D5262-02a; D6992-03
Confined tensile creep		FHWA (1998); WSDOT (2004)
Abrasion (% of geotextile strength)	50 - 100	D4886-88; D3884-01
Gradient ratio clogging		D5101-01
Hydraulic conductivity ratio		D5567-94
Chemical resistance to liquids		D6388-99; D5322-98; D5496-98
Resistance to UV/Outdoor weathering		D4355-02; G155-00; D5208-01; D1435-99; D5970-96
Biological clogging and resistance to fungi		D198-95; G21-96
Temperature		D4594-96; D1042-01; D2136-02
Oxidation induction time (polyolefins)		D5885-97
Oxidation and hydrolysis		FHWA (1999)
Fibre shrinkage		D2102-02; D5104-02

1 Koerner (1997), Koerner and Hsuan (2001)
2 American Society for Testing and Materials, 100 Barr Harbour Drive, West Conshohocken, PA 19428-29593
3 Canadian General Standards Board, Gatineau, Quebec, Canada K1A 1G6
4 Federal Highway Administration, Washington, D.C. (available from National Technical Information Services, VA 22161)
5 Washington State Department of Transportation, State Materials Laboratory, PO Box 47365, Olympia, WA 98504-7365, USA

23.2.1 Hydraulic Properties of Geotextiles, Geonets and Drainage Geocomposites

Porosity, permeability and filtration performance of a geotextile are important properties in separation, drainage and filtration applications. For geonets and drainage geocomposites, the in-plane flow capacity, which is related to the transmissivity of the product, is required for design (Section 23.2.4). A comprehensive reference that addresses and links together conventional soil filter and geotextile filter design is the paper by Giroud (2002).

The porosity 'n' of a nonwoven geotextile is not measured directly but calculated as follows:

$$n = 1 - \frac{M_A}{\rho_f \, t_{GT}} \tag{23.1}$$

where

M_A = mass per unit area (g/m^2); ρ_f = density of the geotextile fibre (g/m^3); and t_{GT} = thickness of the geotextile (m).

The characteristic pore size of a geotextile is the primary parameter used to select the product that best fulfils the filtration function. To obtain an apparent pore size in a geotextile, different techniques have been developed to measure the pore size distribution (Bhatia et al. 1996). Indirect measurements can be performed using a dry sieving method (Apparent Opening Size - AOS) or a hydrodynamic sieving method (Filtration Opening Size - FOS). Current geotextiles are not capable of filtering fine-grained soils composed completely of particles less than No. 200 sieve (75 μm) in size when dynamic, pulsating or cyclic flow is involved. Such soils should be separated from geosynthetics with at least continuously graded sand grading down to 1 % of particles passing the No. 200 sieve (e.g. concrete sand meeting recommendations in ASTM C-33).

The Apparent Opening Size (AOS) is determined from the granulometric curve of glass beads that have been transported through a geotextile specimen under the action of dry sieving. Its value in mm or μm is evaluated from the O_{95} of the granulometric curve where O_{95} refers to the opening size in the geotextile for which 95 % of the openings are smaller (ASTM D4751).

The Filtration Opening Size (FOS) is determined in a similar fashion. However, a gradation of glass beads is used and is forced through the geotextile specimen under hydrodynamic forces rather than just a shaking action (CGSB 148.1 No. 10-94) (Mlynarek et al. 1993). Above 150 μm both AOS and FOS values are similar. Below this value the FOS value is recommended.

The Percent Open Area (POA) is used for woven geotextiles to quantify the relative portion of the open area between yarns to the total area of the geotextile.

The geotextile normal permeability (or hydraulic conductivity) k_n, in m/s, for water at 20°C is a measure of the water flow through the geotextile Q (in m^3/s), per unit area A and hydraulic gradient H/t_{GT} of unity:

$$k_n = (Q \times t_{GT})/(A \times H) \tag{23.2}$$

where H = hydraulic head in (m). The normal permeability of a geotextile can be determined using constant-head or falling-head permeameter devices. Because of the difficulty in measuring thickness, the permittivity ψ of a geotextile normal to the plane of the geotextile is commonly quoted (ASTM D4491). Permittivity is reported in s^{-1}, for flow of water at 20°C, and is expressed as follows:

$$\psi = k_n / t_{GT} = Q / (A \times H) \tag{23.3}$$

23.2.2 Filtration and Separation

The use of geotextiles in filtration and drainage applications is based on the proper selection of geotextile to minimize migration of soil particles as well as to prevent clogging. For this reason, it is difficult to separate filtration and drainage functions. For example, to function as a drain, any geotextile in contact with the soil must act as an effective filter.

A common drainage application using geotextiles is the design of trench drains to replace conventional soil filters. The tendency of fine particle sizes to migrate with seepage flow is great, and the reduction of particle movement through the filtering material is critical to the performance of the drain.

The selection of the most appropriate set of geotextile properties is based on the criteria outlined in this section. The two critical design properties that will affect the design of the drain are the normal permeability and the distribution of opening sizes in the geotextile.

The natural soil to be drained is represented by a particle size distribution and characterized by an indicative particle size D_I (Lafleur et al. 1989), uniformity coefficient C_u and permeability k_n. The problem is to select the appropriate geotextile based on permeability k_n and pore size (AOS or FOS value) to satisfy the design function of the system and to resist the passage of the soil particles without clogging.

23.2.2.1 Soil Retention Criteria (steady state flow)

For soil with less than 50 % passing the No. 200 sieve, calculate $C_u = D_{60}/D_{10}$ for the entire sample. The maximum recommended geotextile O_{95} value (AOS or FOS), in mm, is evaluated using the following relationship:

$$\text{AOS or FOS} < B \times D_I \tag{23.4}$$

where

$$\begin{array}{ll} \text{for } C_u < 2 & B = 1 \text{ and } D_I = D_{85} \\ \text{for } 2 < C_u < 4 & B = 0.5 \times C_u \text{ and } D_I = D_{85} \\ \text{for } 4 < C_u < 8 & B = 8/C_u \text{ and } D_I = D_{85} \end{array} \tag{23.5}$$

For broadly graded cohesionless soils with $C_u > 8$, use Equation 23.4 (Lafleur 1999) where:

$$\begin{array}{ll} B = 1 \text{ and } D_I = D_{50} & \text{for linearly graded soils} \\ B = 1 \text{ and } D_I = D_{30} & \text{for soils with concave upward gradation curves} \\ B = 1 \text{ and } D_I = D_G & \text{for gap-graded soils, where } D_G \text{ is the minimum gap size} \end{array} \tag{23.6}$$

For soils with more than 50 % passing the No. 200 sieve, the maximum AOS or FOS value should not exceed 0.3 mm. A summary of proposed retention criteria from many different sources can be found in the paper by Gardoni and Palmeira (2002).

23.2.2.2 Permeability Criteria

The permeability of a geotextile can be evaluated using the following relationships:

$$\begin{array}{l} \text{for retention of fines: } k_n > 10 \times k_s \\ \text{for retention of clean medium to coarse sands: } k_n > k_s \end{array} \tag{23.7}$$

where k_n and k_s are the normal permeability of the geotextile and retained soil, respectively.

Recommended permittivity requirements are:

$$\begin{aligned} &\psi \geq 0.5\ s^{-1} \text{ for 15 \% passing No. 200 sieve} \\ &\psi \geq 0.2\ s^{-1} \text{ for 15 \% to 50 \% passing No. 200 sieve} \\ &\psi \geq 0.1\ s^{-1} \text{ for} > 50 \text{ \% passing No. 200 sieve} \end{aligned} \tag{23.8}$$

23.2.2.3 Clogging Criteria

For well-graded or uniform soils with $C_u > 3$, and low hydraulic gradients under steady flow conditions the following criterion is recommended:

$$\text{AOS or FOS} > 3 \times D_{15} \tag{23.9}$$

For $C_u < 3$ use the criteria in Section 23.2.2.1 to select the maximum AOS or FOS value. For all applications, the percent opening area (POA) of woven geotextiles should be greater than 4 % and the porosity of nonwoven geotextiles should be greater than 50 %. For severe applications a soil/geotextile filtration test should be performed for prequalification of candidate geotextiles. Holtz et al. (1997) recommend that this performance test be the gradient ratio test (ASTM D5101). The gradient ratio GR should not exceed one unless it can be demonstrated that no impediment to seepage flow will result (Fannin et al. 1994). It is also recommended that the internal stability of the filtered soil particles be checked (Kenney and Lau 1985, 1986).

23.2.2.4 Other Considerations

It is also necessary to ensure that the granular material contained within the drainage trench is sufficiently permeable to carry the anticipated flow. The drainage material should have a permeability value sufficiently in excess of that of the geotextile to allow the system to perform properly. This drainage material should have a permeability value at least 10 times that of the geotextile. Where drainage distances are large or the grade is relatively flat a perforated pipe should be placed in the drain with cleanouts located every 100 metres or less.

After all the geotextile requirements have been identified, it is necessary to examine the product literature in order to select a geotextile or drainage geocomposite that satisfies the needs of the project. The designer must be careful to correctly interpret manufacturers data in terms of the selection criteria recommended in this section.

One of the best ways to minimize the migration of fines is to confine them as tightly as possible. For example, care must be taken to ensure that there are no gaps between the geotextile and the sides of the trench excavation. If there are voids or loosened soils in this zone, then the flow of water towards the geotextile will inevitably initiate the transport of fines resulting in plugging of the drainage system or reduction of the permeability of the geotextile due to cake formation on the filter.

23.2.3 Dynamic, Pulsating and Cyclic Flow

For applications with a dynamic, pulsating or cyclic flow, different soil retention criteria must be used. Dynamic flow conditions may occur in pavement edge drain applications. Geotextiles placed below slope protection or embankment riprap layers in tidal areas or other shoreline applications can be subjected to cyclic water flows.

Where the pulsating flow is large, the geotextile should be sufficiently open to prevent blow up and should be weighted down. The opening size of the geotextile should satisfy the lesser of:

$$\text{AOS or FOS} < 0.5 \times D_{85} \tag{23.10}$$

$$\text{AOS or FOS} < 0.3 \text{ mm} \tag{23.11}$$

Limited gradient ratio testing, in cyclic flow, indicates these criteria are conservative (Fannin and Pishe 2001). Nevertheless, these criteria should not be used for pulsating loads on horizontal geotextile layers placed at the base of highway and railway granular base materials where the flows are small and cyclic loads large. For these conditions the criteria described by relationships in Section 23.2.2 should be used.

23.2.4 In-Plane Drainage

The purpose of a geotextile or drainage geocomposite in many applications is to provide a relatively high permeability path along which liquid (typically water) can flow in order to dissipate excess pore water pressures or to minimize the development of hydrostatic or seepage pressures in slopes, embankments, and retaining wall structures. These applications include geocomposite drainage boards, pavement edge drains or geonets located above geomembrane layers in fluid containment liner systems and prefabricated vertical drains (PVDs). Selection of test methods for PVDs can be found in ASTM D6917. A useful reference on the use of PVDs is the journal Geotextiles and Geomembranes, "Special Issue on Prefabricated Vertical Drains", (Vol. 22, Nos. 1 & 2, 2004).

The critical parameter for the passage of fluid within the plane of the geosynthetic is transmissivity T. It is defined as the product of the planar permeability k_t of the material and its thickness t, as follows:

$$T = k_t \times t \tag{23.12}$$

The thicker the drainage product with a given planar permeability, the higher its transmissivity. The designer must review the manufacturers' test data to determine if normal pressures acting on the geotextile or geocomposite can reduce product transmissivity. Experience shows flow in geonets to be semi-turbulent rather than laminar at relatively low hydraulic gradients (< 0.1) (Fannin et al. 1998), hence calculation of flow capacity cannot be made using Darcy's law unless a relative permeability factor is used as determined from laboratory testing (ASTM D4716).

The transmissivity of geocomposite drainage products is considerably greater than for geotextiles alone. An advantage of drainage geocomposites constructed with a variety of plastic cores is that they have a greater open area and are less compressible than geotextiles. It is important when using any drainage systems (particularly geocomposites) that low invert elevations are provided with discharge outlets. Geosynthetic drainage installations must be constructed with inverts at elevations less than the soil to be drained and have sufficient drainage slopes to remain self-cleaning. Recommendations for edge drain design and installation in highway applications can be found in the paper by Raymond et al. (2000).

23.3 Geogrids

Geogrids fall into two main categories based on structure and are used in soil reinforcement applications. One category includes extruded polyolefin sheets that are punched and drawn to form uniaxial (HDPE) or biaxial (PP) products that have some flexural stiffness. The second category is comprised of high tenacity PET yarns that are knitted or woven together and coated for dimensional stability and durability. These geogrids have effectively no flexural stiffness.

23.4 Strength and Stiffness Properties of Geotextiles and Geogrids

The strength and stiffness properties of a geotextile or geogrid are a concern primarily in reinforcement applications but also in separation applications where the geotextile may be subjected to tensile load. The most common laboratory index test for strength and stiffness properties of geotextiles is the Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method (ASTM D4595).

The corresponding test standard for geogrids is (ASTM D6637). The ASTM D4595 test involves gripping a 200 mm-wide by 100 mm-long specimen and applying a constant axial strain rate of 10 %/minute until rupture. The ultimate strength T_{ult} of the geosynthetic specimen at rupture should be recorded in units of force per unit width (e.g. N/m) along with specimen elongation at rupture (expressed as percent axial strain). In addition, the tensile load of the specimen at 2 and 5 % elongation should be recorded since the secant stiffness at working load levels will vary between different geosynthetics largely as a result of the constituent polymer type and method of manufacture (e.g. woven geotextiles are generally less extensible than nonwoven geotextiles).

Geosynthetics in conventional design practice are expected to creep under load in the field. The results of constant load tests (ASTM D5262) carried out at temperatures representative of field conditions are used to ensure that the reinforcement will not strain excessively or creep to rupture over the design life of the structure. For long-term rupture of the geosynthetic as a limit-state, current practice is to plot "stress-rupture" curves as load at rupture (from constant load tests) versus log time to rupture. A creep reduction factor is then calculated as the ratio $RF_{CR} = T_{ult} / T_l$ where T_l is the rupture load at the design time of interest. The results of constant load tensile tests (carried out at a single constant temperature) may be extrapolated to not more than one log cycle of time for a particular product. For greater extrapolations a larger creep reduction factor should be used. Manufacturers normally supply the designer with this data. Alternatively, temperature accelerated creep testing (ASTM D6992) and more recently the Stepped Isothermal Method (SIM) (Thornton et al. 1998, 2002) have been used to provide equivalent creep data at elapsed times matching the design life of geosynthetic reinforced soil systems (75 years) without the need for excessively long test times in the laboratory (WSDOT 2004). Temperature will also influence the load-strain-time behaviour of geosynthetics particularly for polyolefin materials. In retaining wall applications, current US practice is to consider the wall temperature for design as the mean of the average yearly air temperature and the normal daily air temperature for the hottest month of the year (AASHTO 2002).

The tensile capacity of the reinforcement determined from constant load laboratory testing must also be adjusted using reduction factors to account for site-specific potential loss of strength due to chemical and biological degradation (RF_D) and mechanical damage during installation (RF_{ID}). The allowable tensile strength of the reinforcement T_{allow} is then calculated as (AASHTO 2002):

$$T_{allow} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_D \times RF_{ID} \times RF_{CR}} \qquad (23.13)$$

All reduction factors must be based on product-specific testing. In no case should values for RF_D and RF_{ID} be less than 1.1. A protocol for field installation damage testing can be found in FHWA (1997) and WSDOT (2004). In the absence of such data, AASHTO (2002) recommends that RF not be less than 7 or 3.5 for permanent and temporary wall structures, respectively. The magnitude of creep reduction factor (RF_{CR}) will vary with design life. Typical values may range from 1.5 to 3.0 with the lowest value corresponding to short life times. Manufacturers can advise the designer on the appropriate reduction factors for a given application based on the soils to be used, method of construction and chemical environment. The value selected for T_{ult} is the minimum average roll value (MARV) defined as the average ultimate strength value for a roll which can be expected to be exceeded by 97 % of product rolls.

The maximum design load for a geosynthetic layer in a permanent reinforced wall application is typically reduced to a long-term allowable design load T_{des} where:

$$T_{des} = \frac{T_{allow}}{FS} \qquad (23.14)$$

Here FS is an overall factor-of-safety to account for uncertainty in problem geometry, soil variability and applied loads and has a minimum value of 1.5. For reinforced slopes, FS = 1 since the overall factor-of-safety is accounted for in the stability analyses (Section 23.9.1.1). Finally, long-term field observations have confirmed that post-construction creep strain in reinforced soil structures is typically very small, and well-described by standard laboratory load-strain-time data (Fannin 2000a, Allen and Bathurst 2002).

23.5 Geosynthetics in Waste Containment Applications

Geosynthetics are now used routinely in municipal solid waste and hazardous landfill applications. Geomembranes are used as a primary barrier to prevent the off-site migration of leachate, which is the liquid by-product of infiltrated precipitation and waste decomposition. Failure of a geosynthetic liner system may seriously impact the quality of local groundwater and possibly surface water. Conceptual examples of barrier systems at the base of landfills that incorporate geosynthetics are illustrated in Figure 23.1.

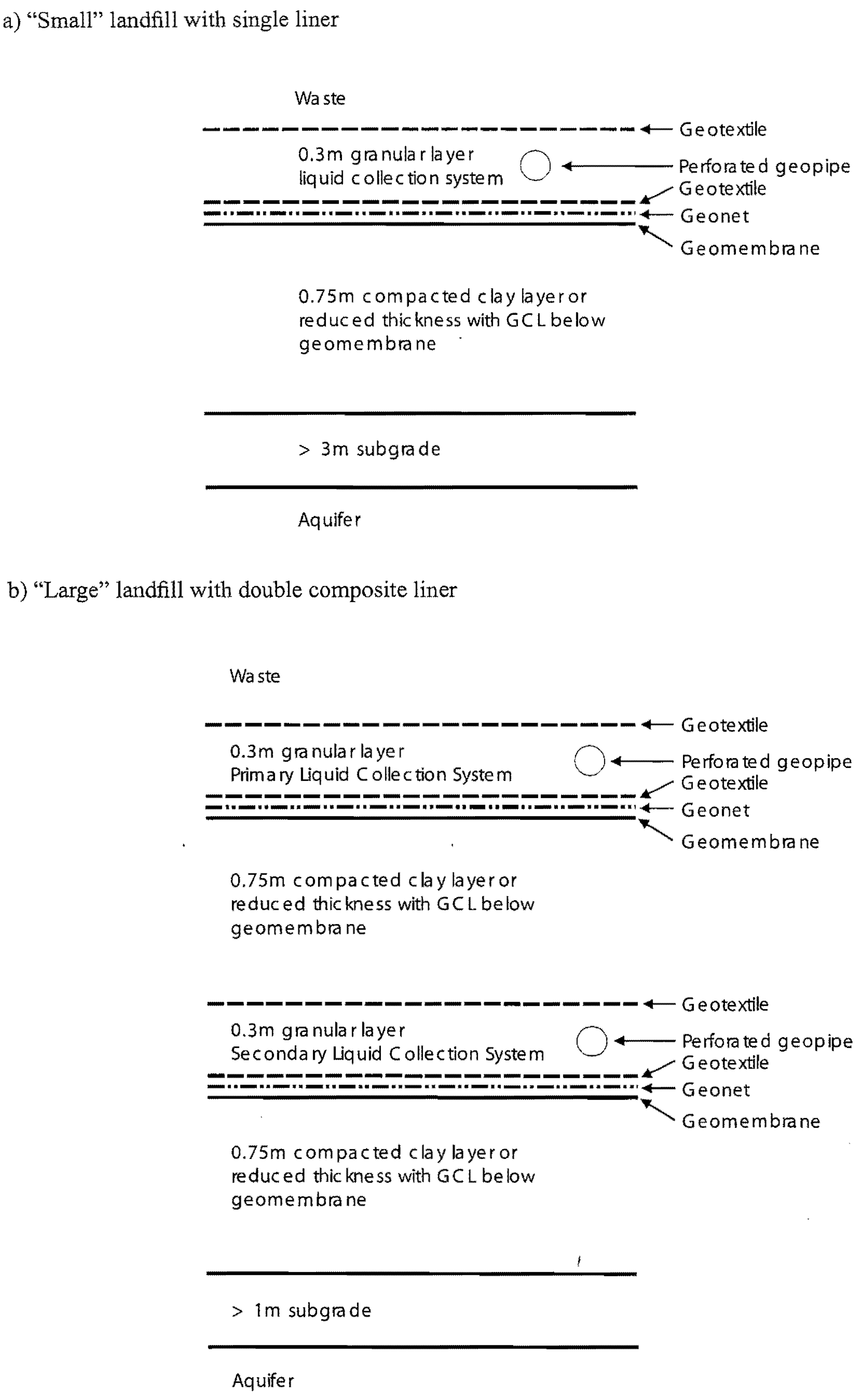


FIGURE 23.1 *Example liner systems at base of landfill illustrating the use of geosynthetics*

Once the landfill has reached its capacity, it should be properly capped. This process often involves a geomembrane barrier that performs a double function: (i) containment of landfill gas (methane and carbon dioxide), and (ii) exclusion of infiltrating precipitation. Methane is generally exhausted to the atmosphere using vents through the cover, or collected for subsequent flaring to atmosphere. In some cases the gas has been recovered as a fuel. A conceptual example of a cover system at the top of a landfill that incorporates geosynthetics is illustrated in Figure 23.2.

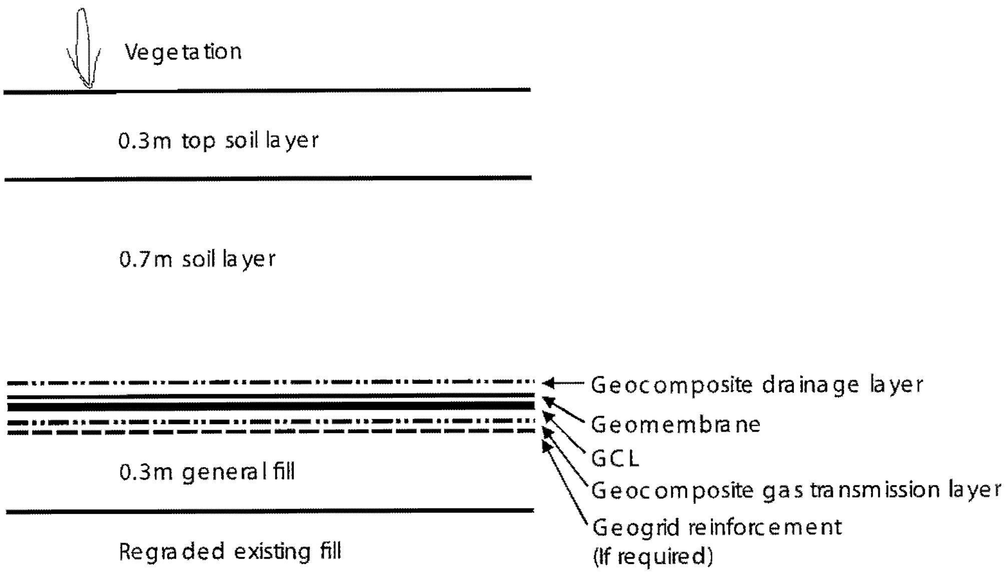


FIGURE 23.2 *Example cover system at top of landfill illustrating the use of geosynthetics*

Design guidance for geosynthetics in landfill applications can be found in the books by Qian et al. (2000), Rowe et al. (2004), Rollin et al. (2002) and special issues of the journal Geosynthetics International: "Design of Geomembrane Applications", (Vol. 2, No. 6, 1995); "Liquid Migration Control Using Geosynthetic Liner Systems", (Vol. 4, Nos. 3-4, 1997); and "Special Issue on Liquid Collection Systems", (Vol. 7, Nos. 4-6, 2000).

23.6 Geomembranes

Geomembranes are manufactured from a great variety of primary resins and other ingredients. They include thermoplastic, semi-crystalline plastics or vulcanized materials. The most widely used geomembranes are manufactured from high-density polyethylene (HDPE), very flexible polyethylene (VFPE), including very low-density polyethylene (VLDPE) and linear low-density polyethylene (LLDPE), and polyvinyl chloride (PVC) (Koerner and Hsuan 2001). Most products have a smooth surface but some may have a textured surface to improve interface shear resistance with adjacent materials. Geomembranes are manufactured in thicknesses from 0.25 mm to 7.5 mm and roll widths from 1.5 m to 10 m. PVC geomembranes are often prefabricated (seamed) into large panels. Geomembranes are relatively impermeable to water (1×10^{-12} to 1×10^{-15} m/s) but are permeable in varying degrees to gases, vapours, and liquids on a molecular scale in a three-step process: (1) by dissolution in or absorption by the geomembrane on the upstream side; (2) diffusion through the geomembrane; and (3) desorption on the downstream side of the barrier. Various methods to assess the permeability of geomembranes to single component permeants, such as individual gases, vapours, and liquids are described in ASTM D5886. Current test methods for geomembranes are summarized in Table 23.2. A useful reference on identification and performance testing of geomembranes is RILEM Report 4 (1991).

TABLE 23.2 *Geomembranes*
(1) Identification, Terminology, and Selection of Test Methods

	Test Method/Reference
Selection of test methods	D6434-99[1]; D6455-99; D5886-95
Sampling	D4354-99
Terminology	D4439-92; CGSB 148.2-M89[2]
Thickness	D5994-98; D5199-01; D 3767-03
Density	D792-00; D1505-03; D297-93
Mass per unit area	D3776-96
Specific gravity	D792-00; D1505-03
Tensile test	D638-03; D882-02; D412-98
Brittleness at low temperature	D1790-02; D746-98;
Oxidation inductance time (OIT)	D3895-03
Infra-red spectroscopy (IR)	Koerner (1997)
Differential scanning calorimetry (DSC)	D3417-99; D3418-03
Thermogravimetric analysis (TGA)	Koerner (1997)
Thermomechanical analysis (TMA)	D648-01; E 831-01
Chromatography (CG and HPLC)	Koerner (1997)
Melt Index (MI)	D1238
Intrinsic viscosity	D4603-03
Carbon black content for polyolefins (%)	D5596-03; D1603-01
Dehydrochlorination (for PVC)	RILEM REPORT 4 (1991)
Surface texture	Dove and Frost (1996)

(2) Performance Tests on Geomembranes

	Test Method/Reference
Tear	D5884-04; D1004-94a; D624-00; D1938-02
Tensile properties	D4885-01; D5323-92; D5617-99; D6693-03; D7003-03; D7004-03
Bursting	Bray and Merry (1999)
Puncture	D5494-93; D5514-94
Impact	D1709-03; D1822-99; D746-98
Interface shear	D5321-02;
Creep	RILEM REPORT 4 (1991)
Thermal expansion	D696-03; E228-95
Dimensional stability	D1204-02
Water permeation	D5886-95
Vapor permeation	D5886-95; E 96-00
Attachment to penetrations or structures	D6497-02; Giroud et al. (1995b)
Leak detection	D6747-1; D7002-03; D7007-03
Ultrasonic	D7006-03

(3) Performance Tests on Seams

	Test Method/Reference
Standard practice/Terminology (field seams)	D4437-99; D6214-98
Standard practice/Terminology (Factory seams)	D4545-86
Peal test	D6214-98; D6392-99; D6636-01; D413-98
Non-destructive tests (Spark test)	D6365-98
Mechanical point stressing	Koerner (1997)
Air channel	D5820-95
Vacuum box	D5641-94
Ultrasonic methods	Koerner (1997)

(4) Durability

	Test Method/Reference
Standard practice for test selection	D5747-95a; D5819-99
Volatiles	RILEM REPORT 4 (1991)
Thermal ageing	D3045-92; D1042-01
UV exposure/Outdoor weathering	G155-00; D4798-01; D1435-99; D5970-96
Chemical resistance	EPA 9090[3]
Biological resistance	G21-96[4]
SO_2 ageing (for PVC)	RILEM REPORT 4 (1991)
Stress-cracking	D1693-01; D5397-99

1 ASTM
2 Canadian General Standards Board
3 USA Environmental Protection Agency
4 Geosynthetic Research Institute, Drexel University, PA, USA

23.6.1 Other Geomembrane Applications

Geomembranes are also used for water reservoirs, canal liners, containment of spills, industrial effluents, fuels/hydrocarbons, mine tailings leachate pads, and for floating covers on liquid impoundments.

23.6.2 Selection

Geomembrane in-situ life expectancy is dependent upon its chemical and biological resistance, temperature and creep stability. Other factors affecting longevity are construction damage, wind and water erosion, wave action, vegetation, underlying granular puncture and excessive tension from slippage or bank failure/settlement.

Candidate geomembranes and any other geosynthetic materials that come in contact with chemical compounds must be evaluated for chemical resistance by laboratory (ASTM D5322) or field immersion tests (ASTM D5496) followed by a range of physical and mechanical tests described in ASTM D5747. This may be an arduous task since there are instances (e.g. hazardous waste landfills) where any number of substances may be present. Product property sheets obtained from the manufacturer should be consulted to determine the geomembrane resistance to chemical agents. Careful attention should also be paid to the properties of the constituent material in candidate geosynthetics when they are to be placed in biologically active environments or under conditions where they may be exposed to extremes of temperature.

23.6.3 Seaming

Geomembranes are generally manufactured to a width that is less than that finally required and thus have to be joined at panel edges. Seaming methods depend upon the liner material. The most common types are thermal processes (extrusion or fusion welding), chemical fusion and adhesive seaming (ASTM D4437, D4545). Seaming techniques include single bond, double or overlapping bond, and dual hot edge bond with continuous air channel between two sealed seams. The integrity of the seams is critical. Non-destructive methods for testing field seams include mechanical point stress, electrical sparking, air lance, vacuum chamber, ultrasonic impedance plane (UIP), and ultrasonic pulse echo (UPE).

23.6.4 Installation

Geomembranes must be installed without incurring construction damage. This may be a difficult task since winds (e.g. Giroud et al. 1995a, 1999), rain and extremes in temperature affect the laying, seaming and field testing of the geomembrane. Construction equipment, used to place cover materials over the liner, may also puncture the product. Adequate anchorage at the crest of a slope should be provided which typically involves installing the geomembrane in an anchor trench. Quality assurance of production and delivery to site are important. Quality control of the installation is critical.

23.7 Geosynthetic Clay Liners

Geosynthetic clay liners (GCLs) introduced in Section 23.1 are now used routinely in liquid containment applications typically in conjunction with geomembranes to form a composite liner with the GCL placed below the geomembrane. They are typically thin (~10 mm) but can be used to perform the same barrier function as much thicker compacted clay liners at both the base of landfill (Figure 23.1) and as part of the cover system (Figure 23.2). GCLs are typically manufactured into panels 4 m to 5 m in width and 30 m to 60 m in length and delivered as rolls at the job site. The bentonite core hydrates on contact with fluids (e.g. water or leachate) to create a low permeability and low diffusion barrier. Typical permeability values with respect to water for GCLs are in the range of 1×10^{-11} m/s to 5×10^{-11} m/s. Guidance on the use of GCLs can be found in the books by Rowe et al. (2004), Rollin et al. (2002) and the journal Geosynthetics International: "Special Issue on Geosynthetic Clay Liners", (Vol. 11, No. 3, 2004).

23.8 Walls

Geosynthetics are widely used in reinforced soil walls. This topic is discussed in Chapter 27: Reinforced Soil Walls.

23.9 Slopes and Embankments over Stable Foundations

This section addresses the use of geosynthetic reinforcement to stabilize slopes and embankments over stable foundations. For problems in which the foundation soils may fail the reader is referred to Section 23.10. Useful references for design of slopes on firm foundations are FHWA (1993, 2001).

23.9.1 Internal Stability

Layers of geosynthetic reinforcement are used to stabilize slopes against potential deep-seated failure using horizontal layers of primary reinforcement. It is usually necessary to stabilize the face of the slope (particularly during fill placement and compaction) by using relatively short and more tightly spaced secondary reinforcement (Figure 23.3). In most cases the face of the slope must be protected against erosion. This may require geosynthetic materials including thin soil-infilled geocell materials or relatively lightweight geomeshes that are often used to temporarily anchor vegetation. The figure shows that an interceptor drain may be required to eliminate seepage forces in the reinforced soil zone.

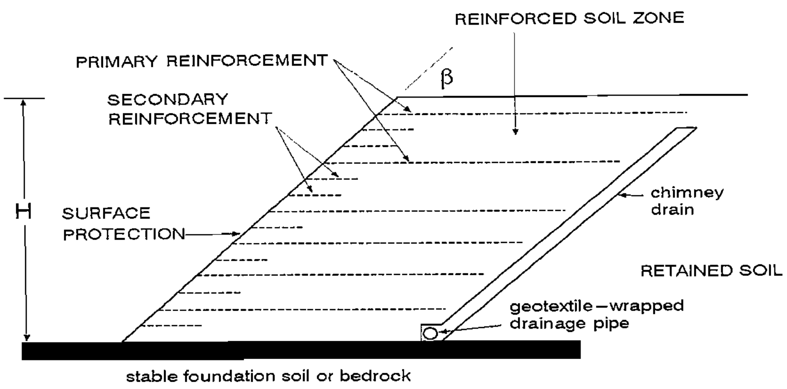


FIGURE 23.3 *Geosynthetic reinforced soil slope over stable foundation*

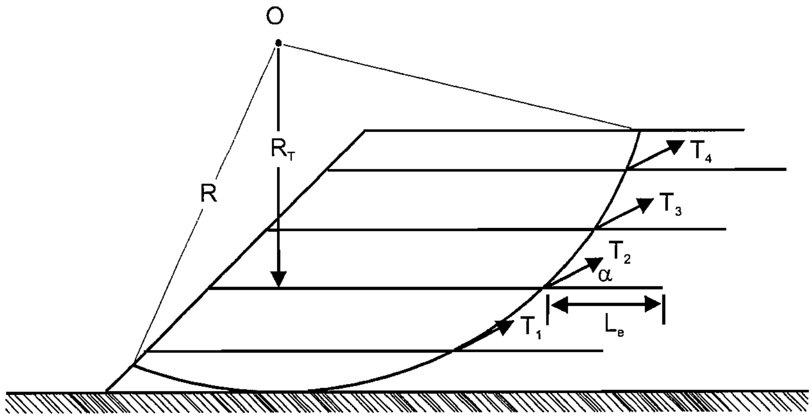


FIGURE 23.4 *Example circular slip analysis of reinforced soil slope over stable foundation*

23.9.1.1 Primary Reinforcement

The location, number, length and strength of the primary reinforcement required to provide an adequate factor-of-safety against slope failure is determined using conventional limit-equilibrium methods of analysis modified to include the stabilizing forces available from the reinforcement.

The designer may use a "method of slices" approach together with the assumption of a circular failure surface, composite failure surface, two-part wedge or a multiple wedge failure mechanism. The reinforcement layers are assumed to provide a restraining force at the point of intersection of each layer with the potential failure surface being analysed. The potential failure surfaces must also include those passing partially through the reinforced soil mass and into the soil beyond the reinforced zone as well as those completely contained by the reinforced soil zone. An example circular slip analysis is illustrated in Figure 23.4. A solution for the factor-of-safety using conventional Bishop's Method of analysis can be carried out using the following equation:

$$FS = \left(\frac{M_R}{M_D}\right)_{\text{unreinforced}} + \frac{\sum T_{allow} \times R_T \cos\alpha}{M_D} \qquad (23.15)$$

where M_R and M_D are the resisting and driving moments for the unreinforced slope, respectively, and α is the angle of tensile force in the reinforcement with respect to the horizontal. Since geosynthetic reinforcement is extensible and can elongate at incipient collapse of the slope, the designer can assume that the reinforcement force acts tangent to the failure surface in which case $R_T \cos \alpha = R$ in Equation 23.15.

The maximum tensile force assumed for the reinforcement should not exceed the allowable tensile strength of the reinforcement T_{allow} or the pullout capacity (see Equation 23.13). Seismic-induced inertial forces are easily accommodated in slope stability methods by introducing additional outward body forces calculated as the product of soil slice or wedge weight and the peak horizontal ground acceleration value (Bathurst and Alfaro 1996, Shukla 2002).

Commercial slope stability packages are available that explicitly include the stabilizing forces from reinforcement layers. For preliminary design purposes and for simple slope geometries the design charts by Jewell (1991) may be used.

23.9.1.2 Secondary Reinforcement

Secondary reinforcement should be placed at not more than 0.6 m vertical spacing and should extend 1.3 m to 2 m into the slope. The secondary reinforcement need not have the same strength as the primary reinforcement.

23.9.2 External Stability

Slopes and embankments over stable foundations must also be analysed for sliding along the base of the reinforced soil mass. This potential mechanism is similar to that assumed for the retaining wall case described in Chapter 27. Similarly, this mechanism may control the length of primary reinforcement. The sliding mass may be treated as a monolithic block (i.e. gravity structure) with a vertical back face located at the free end of the base reinforcement layer. Active Coulomb earth forces can be used to calculate the destabilizing horizontal earth force.

23.10 Embankments on Soft Ground

Embankments over soft ground may fail due to: (i) bearing capacity failure of the underlying soils; (ii) a circular slip failure extending through the embankment materials and into the subsoils; (iii) lateral spreading of the embankment materials due to excessive shear stresses developed at the surface of the underlying soils; or (iv) failure due to excessive displacement of the embankment (e.g. embankments on highly compressible peat deposits).

The primary function of geosynthetics for embankments is reinforcement. In some cases the geosynthetic may act initially as a separator and to facilitate construction. The use of a layer of relatively strong and high tensile stiffness geosynthetic reinforcement (typically geotextile, but in some cases combined with an overlying geogrid layer) at the base of the fill can increase the factor-of-safety against catastrophic collapse due to the first three failure modes identified above. The use of a geosynthetic for reinforcement will not influence the magnitude of settlements generated at the surface of the subsoils and there is little evidence that the differential settlements that would be expected for an unreinforced embankment are modified by the presence of the geosynthetic. Geomattresses (deep granular-infilled geocells) have also been used to support embankments over soft ground (Bush et al. 1990). Base reinforcement spanning pile caps has been used to transfer embankment loading to piles placed in soft ground (BS8006 1995).

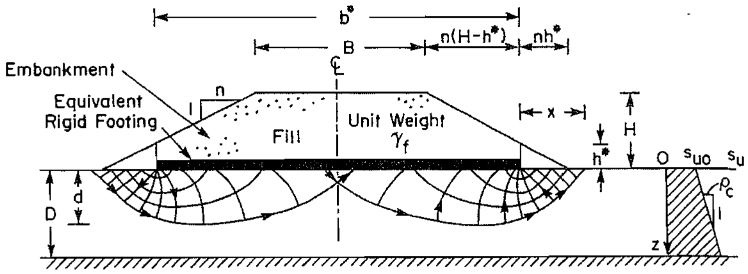


FIGURE 23.5. *Definition of variables used to estimate the collapse height for a perfectly reinforced embankment (Rowe and Soderman 1987)*

23.10.1 Bearing Capacity

Rowe and Soderman (1987) have proposed a simple method of estimating the maximum undrained stability that can be achieved with reinforcement for embankments on soft cohesive soils, considering the effect of increasing undrained shear strength with depth and the effect of the relative thickness of the deposit (Figure 23.5). Their method places an upper limit on the improvement in stability which can be achieved using high strength/stiffness reinforcement since a reinforced embankment can never be reinforced beyond the point of being perfectly rigid. Since an embankment will generally be trapezoidal in shape and the plasticity solutions are for a rigid footing of width b, an approximation must be made to determine the equivalent width of embankment. From plasticity considerations, the pressure at the edge of a rigid footing at failure is $(2+\pi)s_{uo}$, where s_{uo} is the undrained shear strength directly beneath the footing. It is assumed here that the effective width of the footing b will extend between the points on either side of the embankment when the applied pressure γh is equal to $(2+\pi)s_{uo}$. Thus:

$$h = (2 + \pi)\, s_{uo}/\gamma \tag{23.16}$$

and hence from Figure 23.5;

$$b = B + 2n\,(H - h) \tag{23.17}$$

where B is the crest width, H is the embankment height and n is the cotangent of the slope angle.

The bearing capacity q_u of the equivalent rigid footing of width b is given by:

$$q_u = N_c\, s_{uo} + q_s \tag{23.18}$$

where q_s is a uniform surcharge pressure applied to the foundation soil surface outside of the footing width. (The bearing capacity factor N_c is determined using Figure 23.6). Inspection of Figure 23.5 shows that the triangular edge of the embankment provides a surcharge that increases stability and hence, the estimate of q_s in terms of the pressure applied by this triangular distribution is required.

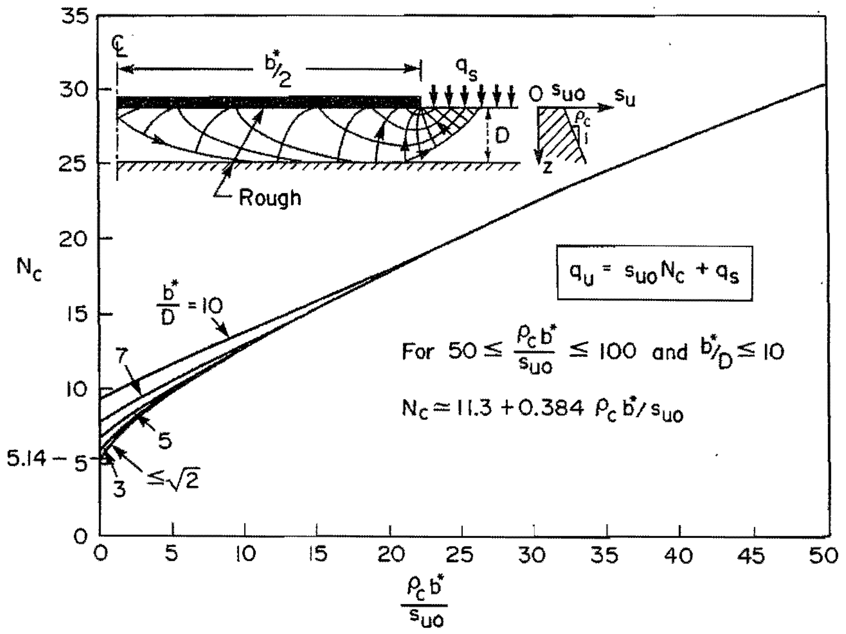


FIGURE 23.6 *Bearing capacity factors for embankments over for non-homogeneous cohesive foundations (Rowe and Soderman 1987)*

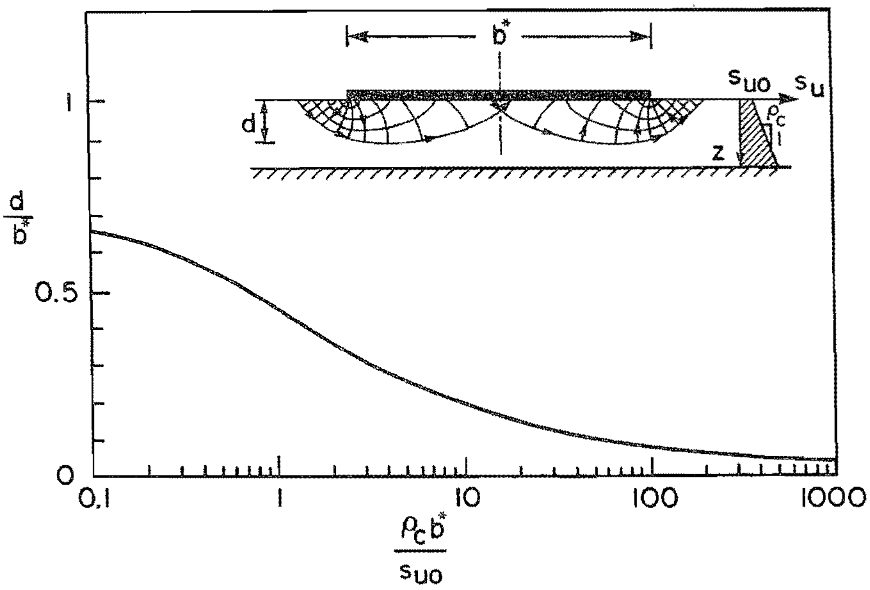


FIGURE 23.7 *Effect of non-homegeneity on depth of failure beneath a rough rigid footing (modified from Matar and Salencon 1977)*

Figure 23.7 shows the depth d to which the failure mechanism is expected to extend. The lateral extent of the plastic region involved in the collapse of a rigid footing extends a distance x from the footing where x is approximately equal to the minimum of: d as determined from Figure 23.7; and the actual thickness of the deposit D, i.e.:

$$x = \min(d, D) \tag{23.19}$$

Thus distributing the applied pressure due to the triangular distribution over a distance x gives:

$$q_s = n\gamma h^2/2x \qquad \text{for } x > nh \tag{23.20}$$

and;

$$q_s = (2nh - x)\gamma h/2nh \quad \text{for } x \leq nh \tag{23.21}$$

This value may then be compared with the average applied pressure q_a due to the embankment width b, according to:

$$q_a = \gamma[BH + n(H^2 - h^2)]/b \tag{23.22}$$

For the purposes of estimating the maximum possible factor-of-safety (defined here as FS = q_u/q_a) for a given embankment geometry and soil profile, q_u and q_a can be determined directly from Equations 23.18 to 23.22. If the calculated factor-of-safety exceeds the desired factor-of-safety then the selection of an appropriate reinforcement can allow construction of the embankment to the desired height H.

If the calculated factor-of-safety is less than the desired value then reinforcement alone is not sufficient and the use of staged construction, berms or lightweight fill may be necessary, particularly over muskeg (Raymond 1969) or soft landfills (Holtz 1990).

Once it has been established that reinforcement can provide the desired factor-of-safety, it is then necessary to select a particular geosynthetic reinforcement and check that it provides a satisfactory margin of safety against circular slip and lateral spreading modes of failure as described below.

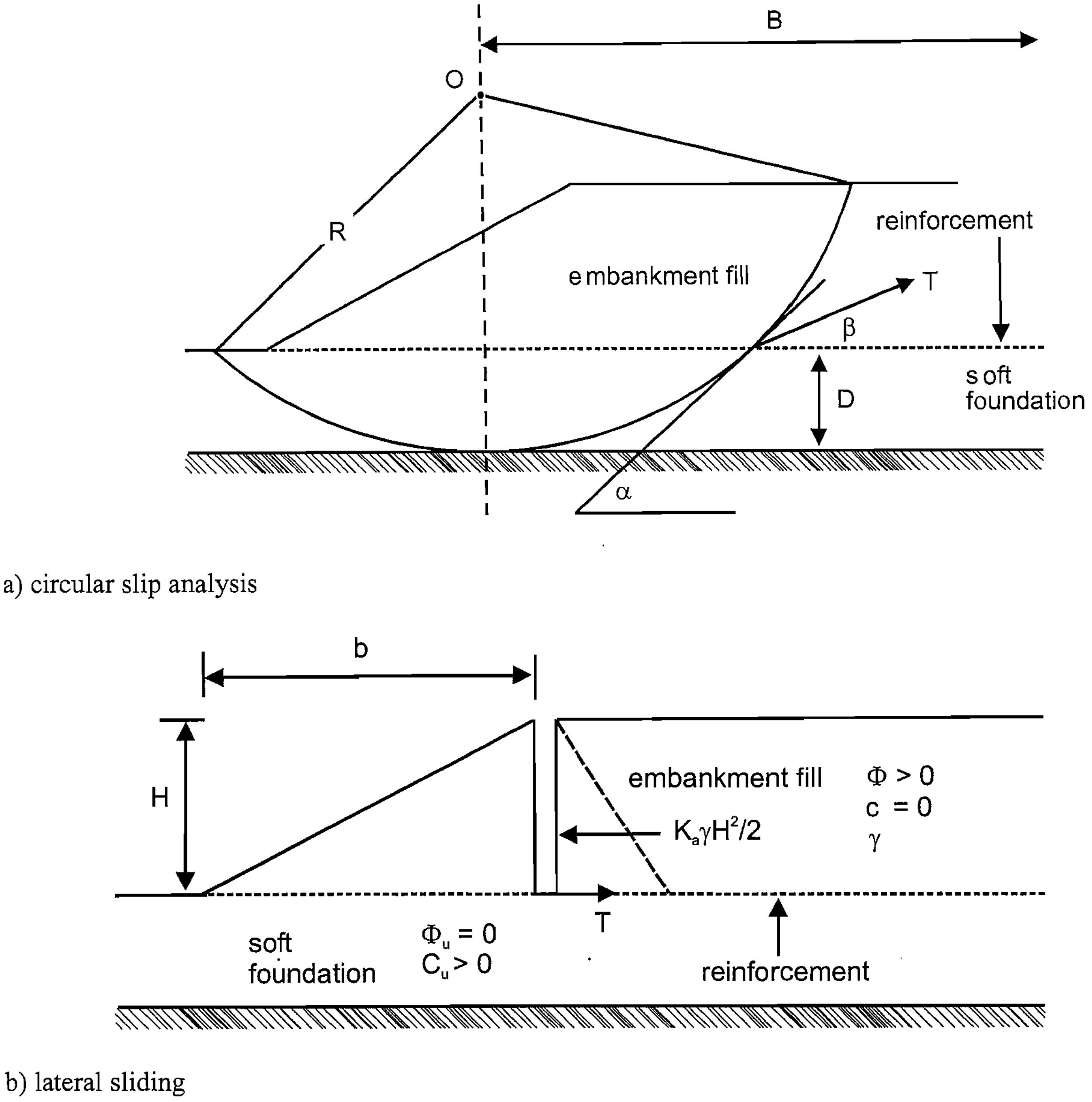


FIGURE 23.8 *Stability analyses for reinforced embankments over soft foundations (after Bonaparte and Christopher 1987)*

23.10.2 Circular Slip Failure

The calculation of factor-of-safety for circular slip surfaces is carried out in a similar manner to that described for embankments over firm foundations (Section 23.9.1.1). For example, if a modified Bishop's Method is used, the tensile force available in the reinforcement at the intersection of the circular slip surface and the reinforcement provides an additional stabilizing moment. The assumed orientation of the tensile force vector in stability calculations can be assumed to act at $0 \leq \beta \leq \alpha$ (see Figure 23.8a). Holtz et al. (1997) recommend the following assumptions with respect to β:

$\beta = 0$	for brittle, strain-sensitive foundation soils (e.g. leached marine clays);
$\beta = \alpha/2$	for D/B < 0.4 and moderate to highly compressible soils (e.g., soft clays);
$\beta = \alpha$	for D/B ≥ 0.4 and highly compressible soils (e.g., very soft clays); reinforcement with the elongation potential ($\varepsilon_{design} \geq 10$ %) and large tolerable deformations.

Slip circle analyses are not recommended for embankments on fibrous peat. Rather, the restoring force provided by the geosynthetic should be taken as the minimum of:

1. The force that can be mobilized in the geosynthetic considering the horizontal earth pressure of the embankment and the potential shear resistance that can develop between the embankment and the underlying soil (see next section);
2. The pullout capacity of the reinforcement; or
3. The allowable force (i.e. allowable strain times secant tensile stiffness).

A number of recommendations can be found in the literature regarding what strain (or force) should be used in limit equilibrium calculations. Rowe and Soderman (1985a) and Hinchberger and Rowe (2003) have proposed a technique that may be used to estimate an "allowable compatible strain", ε_a, for use in limit equilibrium calculations for reinforced embankments constructed on soft clayey foundations with a constant strength and strength varying with depth, respectively. These approaches limit the reinforcement strain depending on the foundation properties. Bonaparte and Christopher (1987) also suggest limiting strain values that are dependent on foundation soil type. However, caution should be exercised when selecting an allowable strain, particularly for use in designing reinforced embankments on brittle cohesive soils that are susceptible to progressive failure (see Rowe and Mylleville (1990) and Mylleville and Rowe (1991)). As for all reinforced soil structures, the potential reduction in strength due to installation damage and creep of the geosynthetic should be considered.

23.10.3 Lateral Embankment Spreading

Horizontal instability of the embankment can occur if: (1) the embankment slides on the reinforcement, or (2) the reinforcement fails in tension and the fill slides along the foundation soils (Figure 23.8b). Bonaparte and Christopher (1987) recommend the following factor-of-safety relationships:

$$FS = b \tan \phi_{sg} / K_a H \qquad \text{embankment slides on the reinforcement} \qquad (23.23)$$

$$FS = 2\,(b\,C_g + T) / K_a \gamma H^2 \qquad \text{reinforcement fails in tension and the fill slides along the foundation soils} \qquad (23.24)$$

Term ϕ_{sg} refers to the peak angle of sliding friction developed at the reinforcement soil interface and C_g is the mobilized adhesion. A factor-of-safety of 2 is typically used in these calculations.

23.11 Reinforced Embankments on Soft Foundations with Prefabricated Vertical Drains (PVDs)

Li and Rowe (2001) have developed a method of analysis that considers the combined effect of geosynthetic reinforcement and vertical drains (e.g. PVDs) on embankment stability. The accelerated consolidation of a soft soil foundation afforded by the installation of (i.e. increase in rate of shear strength gain) is coupled with a circular slip analysis. The method reduces the excessive conservatism that results from the conventional assumption of constant undrained shear strength of the underlying cohesive soil deposit.

23.12 Embankments on Fibrous Peats

Rowe and Soderman (1985b) have developed design charts which may be used to select an appropriate geosynthetic reinforcement for embankments constructed on fibrous peat underlain by a firm base as shown in Figure 23.9. This design chart assumes appropriate construction control and that the maximum excess pore water pressure in the peat $\Delta u_{max} \leq 0.34\, \Delta\sigma_v$. Rowe and Soderman (1986) have also provided recommendations for embankments constructed on peat and underlain by a soft cohesive stratum as shown in Table 23.3. (The reader is referred to the original paper for details regarding limitations).

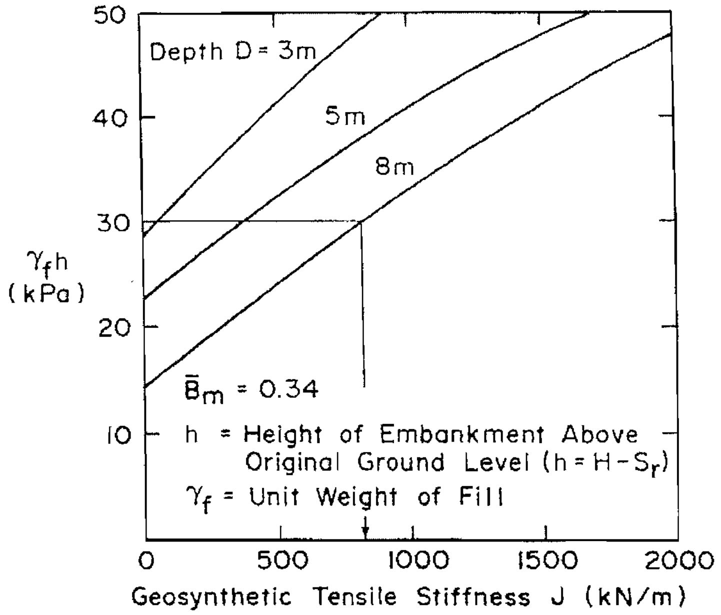


FIGURE 23.9 *Design chart for peat underlain by a firm base (Rowe and Soderman 1985b)*

TABLE 23.3 *Geosynthetic Stiffness Values for Embankments on Fibrous Peat underlain by Soft Cohesive Strata (Rowe and Soderman 1986)*

Peat thickness (m)	Underlying strata	Strength of underlying strata s_u (kPa)	Maximum height of fill above original ground level (m)			
			1.0	1.5	2.0	2.5
3	2m clay	15	NRR[a]	NRR	500 (5%)[b]	1000 (6%)
		10	NRR	NRR	500 (6%)	1000 (6.5%)
3	2m clay	7.5	NRR	150 (10%)	500 (8%)	1000 (7%)
		5	NRR	150 (14%) to 500 (8%)[c]	2000 (4%)	PF[d]
3	2m clay	2.5-5	150 (10%)	2000 (5.5%)	PF	PF
5	3m clay	15	NRR	150 (14%) to 500 (6.5%)	500 (9.5%) to 1000 (5.5%)	1000 (6.5%) to 2000 (4%)
		10	150 (5%)	500 (8.5%) to 1000 (7.5%)	2000 (5.5%)	PF
5	3m clay	7.5	500 (5%)	1000 (8.5%) to 2000 (5%)	PF	PF

a NRR = no reinforcing geosynthetic required.
b 500 (5%) = geosynthetic with stiffness J = 500 kN/m is recommended. Under the assumed conditions a maximum geosynthetic strain of approximately 5 % is anticipated.
c 150 (14%) to 500 (8%) = for the assumed conditions the embankment could be constructed using J = 150 kN/m but the expected strain of 14 % is large. If conditions are likely to be as assumed, a higher stiffness geosynthetic is recommended. J = 500 kN/m would give 8 % strain under the assumed conditions.
d PF = potential failure for the assumed conditions: do not construct.

23.13 Unpaved Roads over Soft Ground

Unpaved, unbound and water bound roads are designed for mining and drill site access, forest roads, and temporary construction roads. They have no bituminous or Portland cement surface layer but consist entirely of one or more layers of unbound granular material found at the site or imported and then compacted (often by truck traffic alone). The strength of the pavement structure is derived entirely from the careful selection of the granular materials and in their subsequent treatment (i.e. compaction). Such roads must be constructed at minimum cost. They may carry very high wheel loads, and often encounter particularly poor subgrades. Stabilization may be achieved more economically through the use of geosynthetics.

With increasingly thicker granular depths, a "road" begins to behave more like an embankment when the dead load exceeds the live load on the subgrade. With wheel loads in the range 5 to 10 tonnes/wheel (usually applied by a wheel with a dual tire arrangement), subgrade pressures due to embankment dead loads begin to exceed subgrade pressure resulting from live wheel loads at a depth of around 1 m. The discussion in this section is limited to unpaved road structures of about one metre in thickness or less, lying on a geosynthetic layer placed on the subgrade. For depths greater than one metre the design should follow the procedures given in Sections 23.10 and 23.12.

23.13.1 Reinforcement Mechanisms and Geosynthetic Requirements

The geosynthetic must first withstand construction operations. The survivability criteria given in Section 23.15 must be checked for candidate geosynthetics. The geosynthetic in this application should effectively separate the subgrade material from the fill and minimize migration of the subgrade soils. The applicable criteria for these functions are given in Section 23.2.2.

There is still some question as to the nature of the fundamental reinforcement mechanism, but there is general agreement that the geosynthetic "reinforces" the road structure. The mechanisms that have been proposed include:

1. Membrane action by the displaced geosynthetic;
2. Restraint of the granular fill through shear stresses acting on the geosynthetic; and
3. For geogrids, interlocking of the fill particles with the geogrid.

Numerous design methods have been proposed. A review of many of these methods can be found in the paper by Hausmann (1987). To date, many of the methods have been based on a rut depth criterion, rather than road stiffness or working load behaviour. Milligan et al. (1989) have proposed a design method that attributes the primary mechanism of reinforcement to be the reduction of outward-acting shear stress on the subgrade surface below the loaded area, which otherwise causes the bearing capacity of the subgrade soil to diminish.

23.13.2 Design Methods for Unpaved Roads over Cohesive Soils

23.13.2.1 Proposed Design Method Using Geotextiles

A widely used method is that proposed by Giroud and Noiray (1981) and summarized in the book by Koerner (1997). The following assumptions are made:

1. Undrained behaviour of the subgrade;
2. Wheel loads applied at the road structure surface spread in a truncated pyramid (Figure 23.10); and
3. Parabolic geotextile deformation in cross section (one parabola under each wheel, and an inverted parabola between them).

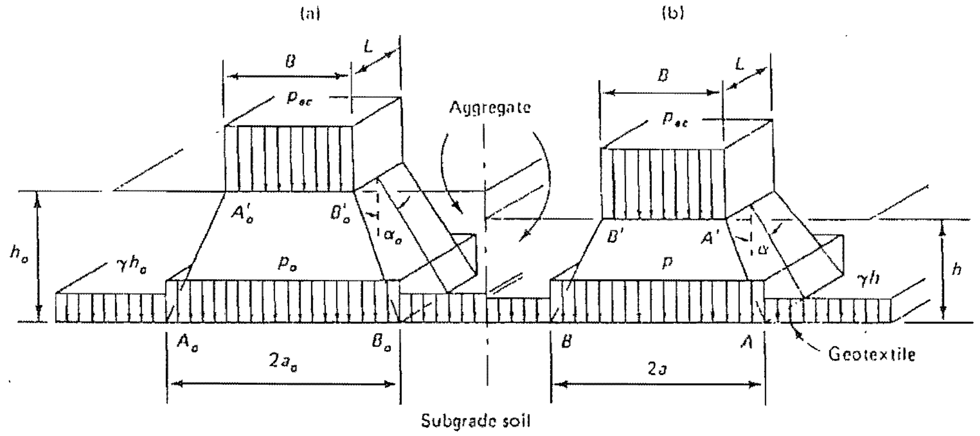


FIGURE 23.10 *Assumed load distribution through unpaved road structure: (a) without geotextile; and (b) with geotextile (Giroud and Noiray 1981)*

The scheme requires the determination of the depth of fill required with and without a geotextile taking into account the appropriate amount of traffic. It is then assumed that road structures that do not include a geotextile layer fail when the pressure imposed on the subgrade using uniform elastic theory reaches:

$$p_o = \pi\, s_u + \gamma\, h_o \tag{23.25}$$

(see Figure 23.10 for the definition of variables). For road structures that include a geotextile layer, it is assumed that the pressure on the subgrade to initiate failure is:

$$p^* = (2 + \pi)\, s_u + \gamma\, h \tag{23.26}$$

The decrease in the pressure caused by the presence of the geotextile, is given by substituting p - p_g for p* with p_g being the contribution made by the geotextile:

$$p_g = \frac{E\,\varepsilon}{a\sqrt{1+(a/S)^2}} \tag{23.27}$$

where

E = geotextile modulus (stiffness in units of force per unit width); ε = strain in geotextile; a = geometric property (see Figure 23.11); and S = settlement, or rut depth.

Combining Equations 23.25 to 23.27, together with assumptions about the geometry of the stress distribution, the difference in fill thickness Δh is determined and is given in Figure 23.11. Finally, the fill thickness required for an analysis which takes traffic into account, but where no geotextile is provided, gives an estimated fill thickness (h'_o) according to:

$$h'_o = \frac{b \log N_s}{CBR^{0.63}} \tag{23.28}$$

where

h'_o = fill thickness required, with no geotextile, but taking traffic into account; b = constant; N_s = traffic volume, in equivalent single axles; and CBR = subgrade strength. The results are plotted as three sets of curves (Figure 23.11) representing:

1. The fill thickness required for a road without geotextile (Equation 23.28);
2. The difference in fill thickness, Δh; and
3. The implied strain in the geotextile for the given subgrade CBR and geotextile modulus (stiffness).

The determination of the reduced fill thickness (h'_o − Δh) is left to the designer to calculate.

Charts such as Figure 23.11 are valid only for the assumed axle load P, rut depth r, and tire contact pressure p_c. For Figure 23.11 and axle loadings other than the standard 80 kN axle load, the use of equivalent single axle loads is recommended (e.g. TAC 1997).

Another method that has been used in a similar application is the US Forest Service Method (Steward et al. 1977) which is described in the book by Holtz et al. (1997). Observations from vehicle loading trials confirm a significant improvement with inclusion of a geosynthetic, and reasonable agreement between field performance and analytical predictions (Fannin and Sigurdsson 1996).

23.13.2.2 Proposed Design Method using Geogrids

A similar approach to the design of unpaved roads built with geogrids is proposed by Giroud et al. (1984) and is summarized by Koerner (1997). The design charts illustrate that fill heights may be reduced to a maximum of 40 % of the unreinforced thickness for foundation CBR values less than 3.

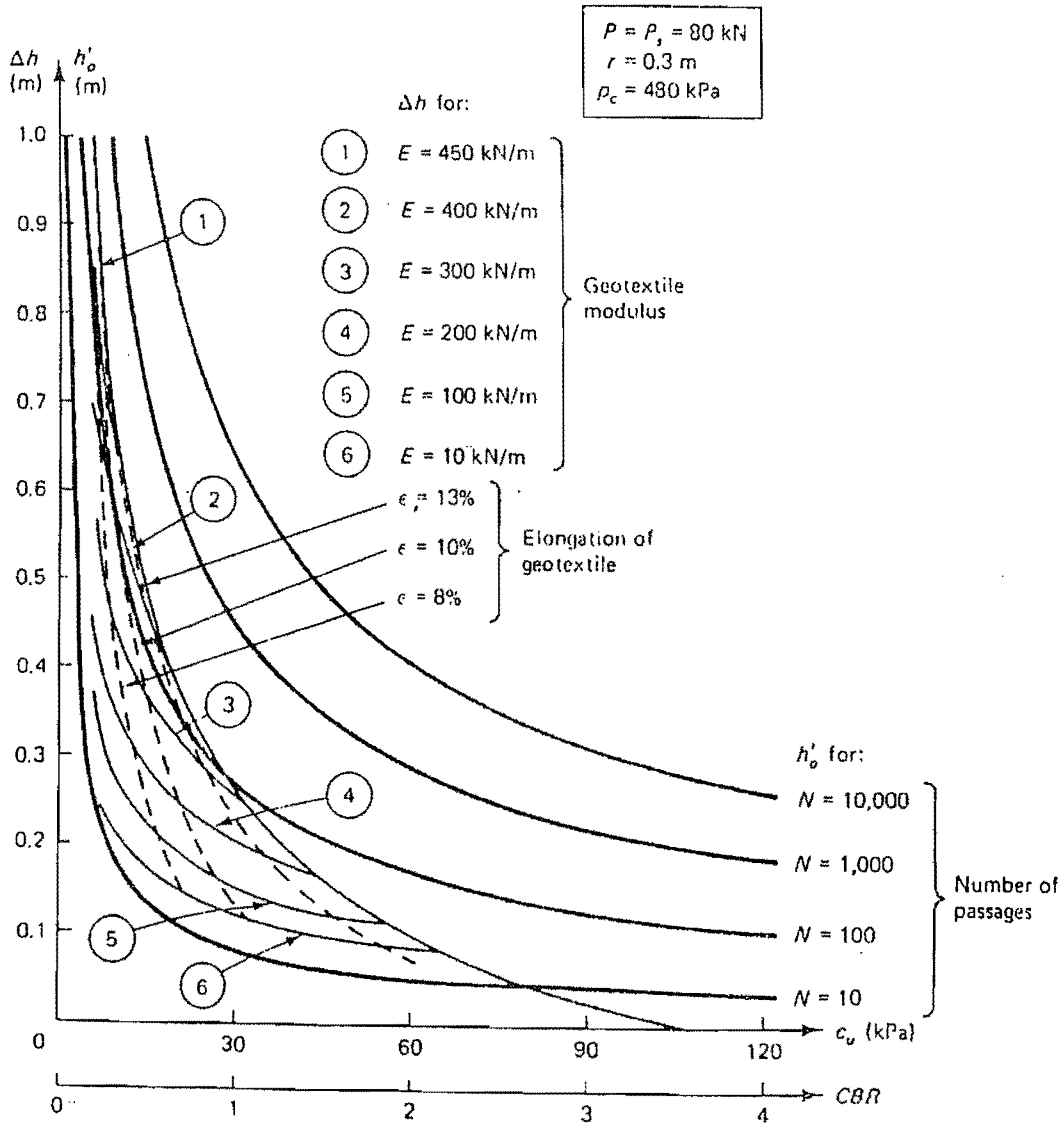


FIGURE 23.11 *Unreinforced fill thickness, fill reduction, and geotextile strain for roads with geotextiels: axle load P = 80 kN, rut depth r = 0.3 m and tire presssure p_c = 480 kPa (after Giroud and Noiray 1981)*

23.13.3 Unpaved Roads over Peat Soils

At the time of writing there are no simple analytical methods available for the design of unpaved granular bases over peat subgrades. Peat soils are highly compressible and road bases can be expected to develop deep channelized ruts that will require filling and grading before the road base is stabilized. The introduction of a geotextile or geogrid prior to fill placement serves to act as a separator as well as reinforcement. The geosynthetic is particularly effective in bridging voids that may be present in the surface vegetative mat. The results of large-scale laboratory modelling of geosynthetic-reinforced granular bases has shown that savings of the order of 30 % of unreinforced aggregate depths may be anticipated by incorporating a geotextile or geogrid at the base of thin layers of unpaved road base over peat (i.e. 0.3 m to 0.45 m thick). Where possible, ruts should be filled with soil rather than re-graded flat to avoid damaging the geosynthetic and to progressively mobilize the reinforcing effect of the geosynthetic.

23.14 Paved Roads, Container Yards and Railways

The use of geosynthetics in highway design and construction (as opposed to unpaved roads) generally relates to two main functions: (i) The construction of the first granular lift over soils that may soften on saturation; and (ii) reinforcement of the upper granular layers. Similar functions may be assumed for stabilization of ballasted railway track.

23.14.1 Geotextiles for Partial Separation

Softening of fine-grained soils can occur when the water table is within 600 mm of the surface for several days of the year even if the soil was originally compacted at its optimum moisture content to maximum density. An example would be ponded ditch water adjacent to road fills. The exact amount of softening depends on many factors but the Plasticity Index (PI) of the soil is generally considered a reliable indicator of the susceptibility of a soil to softening. Typically soils with PI $\geq$ 50 may be expected to soften to less than CBR = 2. Those in the range of 30 to 50 may be expected to soften to less than CBR = 3 (e.g. Black 1962). Guidelines are contained in Transportation Road Research Laboratory Road Notes No. 29. General experience has shown that granular soils containing gravel-size particles placed on such softened soil and subjected to repeated loads will penetrate these soils even when graded to meet filter requirements. The estimated penetration may be empirically expressed as a fraction of the total design granular thickness where the design thickness has been obtained from a recognized design method (e.g. AASHTO 1986) as:

$$\text{Fractional loss} = [1 - \text{CBR} / 4]^2 \quad \text{for CBR} \leq 4 \tag{23.29}$$

Note that where design thicknesses are less than normal design requirements the loss may be expected to be greater, while for greater thicknesses the loss may be expected to be less.

In general the decision whether to use a geotextile is based on comparing the cost of the geotextile with that of the extra granular material required to replace the loss due to penetration. After penetration the working thickness is, of course, reduced adding to arguments in favour of using a geotextile. A design methodology for selection of a geotextile separator in a permanent road application is reported by Christopher and Holtz (1991) and reproduced in the book by Holtz et al. (1997). A review of current design methods to select geosynthetics for granular base reinforcement in flexible paved road applications can be found in the paper by Perkins and Ismeik (1997).

Where the first lift of granular soil fails to meet conventional soil filter requirements, geotextiles (generally needle-punched nonwovens) are often used to retard penetration of fines into the granular base. In general, presently available geotextiles do not have sufficiently small filter opening sizes (FOS) to prevent minus 75 μm particles from migrating under pumping action. Under these conditions, preference should be given to the use of a capping sand placed over the compacted subgrade (typically graded similar to the concrete sand particle size distribution given in ASTM C-33 plus between 1 to 5% passing the 75 μm size as determined by ASTM C-117, i.e. by washing). Research geotextiles with fibre sizes smaller than 6 denier (i.e. 0.7 Mg/m length of yarn) may solve this problem in future years.

23.14.2 Geosynthetics for Granular Base Reinforcement

Geosynthetics are used for the reinforcement of granular bases within the base layer. Geogrids are often used for this purpose since the apertures of the grid allow the geosynthetic to interlock with gravel-sized particles. Reinforcement of granular bases to stabilize the aggregate against lateral deformations has also been accomplished through the use of three-dimensional cellular confinement systems where the walls of the cellular network increase the stiffness and passive resistance of the soil layer by laterally confining the aggregate. Geotextiles (as opposed to geogrids) for reinforcement of load bearing granular soil subject to repeated loading are not recommended since it is difficult to ensure interlock of the geotextile and the soil. To ensure interlock the geogrid should be embedded within the granular layer being reinforced and not placed at an interface with a differently graded soil. In general, reinforcement of a granular soil where the horizontal reinforcement layer is placed close to the trafficked surface increases the effectiveness of the granular layer into which the reinforcement is embedded by a factor of about 1.5 to a maximum additional thickness of 150 mm (i.e. for depths of aggregate less than 300 mm, reinforcement produces an equivalent thickness of 1.5 times the granular thickness). For aggregate depths greater than 300 mm the reinforcement increases the equivalent thickness by 150 mm (see Carroll et al. 1987, Walls and Galbreath 1987, Raymond 1992). For granular thicknesses greater than 300 mm the reinforcement should be placed between 100 and 150 mm below the upper surface of the granular base (e.g. Raymond 1992, Bathurst and Raymond 1987, Binquet and Lee 1975, Guido et al. 1986). At present there is no exact method of estimating the strength of the reinforcement, required. However, an empirical guide for granular bases in railway track support applications involves calculating the additional resistance to failure above that to support the track on unreinforced aggregate reduced by an appropriate factor-of-safety. This gives the design strength of the reinforcement as:

$$T = \frac{(B-2e-2D_r)(\Psi+\sin\Psi)}{(B-2e)\pi}(Q_r - Q_u) \leq \frac{Q_r \tan\varphi}{2} \tag{23.30}$$

where

$$\Psi = 2\tan^{-1}\left(\frac{B-2e}{2D_r}\right) \tag{23.31}$$

Here, B is the tie length (footing width), e is the eccentricity at the base of the tie from the loading at the top of the rail, D_r is the depth of the reinforcement below the base of the rail and must be less than $(B-2e)/2$, Q_r is the total resultant design load including live and impact loading and Q_u is the resistance offered by the unreinforced soil including any reduction for factor-of-safety. In most cases the strength for survivability will dominate strength requirements and pullout will control the maximum value of T that can increase Q_r. Maximum T will generally be achieved if the reinforcement length exceeds the tie length by 15 % on each side of the track (i.e. total length equals 1.3B). Where lateral loads are high and no positive wide gauge resistance exists, sufficient shoulder resistance should be provided to prevent excessive lateral track movement. If necessary, geocells may be used to reinforce the shoulder ballast (Raymond 2001). In any event, since any geosynthetic would be installed within the large size aggregate, suitable reinforcement material should be selected to survive installation, in-service abrasion, and later ballast tamping during track maintenance.

Prior to compaction, the geosynthetic should have a minimum 100 mm and preferably 150 mm cover to prevent damage. In the design of heavily loaded track structures typical of gantry cranes and the like, placement of a reinforcement layer in the upper layer of the granular support is generally very beneficial in increasing the ability of the granular soil to support the eccentric and inclined loadings generated from the lateral forces of the wheel flanges on the rail head. Reinforcement at depth is generally of little value in resisting these forces. Reinforcement of railway tracks with geogrids placed 150 mm below the tie base have been used with major success in track rehabilitation of electrified lines and the like where increasing ballast depths is economically prohibitive. While the same reinforcement benefits would occur elsewhere and have been used (Walls and Galbreath 1987), economics has generally dictated increased aggregate depths.

Heavy needle-punched geotextiles placed at least 300 mm below the tie base have been used with major success in track rehabilitation of hard-to-maintain areas where internal drainage is a major concern. In such cases it is essential to ensure that the geotextile facilitates drainage from the load bearing area (Raymond 1999). In such cases, the geotextile has been installed as a separator rather than as reinforcement and must facilitate internal drainage from the load bearing area to the side ditches.

23.15 Construction Survivability for Geosynthetics

In order to perform their design function, geosynthetics must survive construction activities. In many instances soil placement and compaction may impose the most severe loading conditions experienced by the geosynthetic. In the absence of field data the guidelines contained in Tables 23.4 and 23.5 should be used to ensure that a candidate geotextile is sufficiently robust to survive construction in the applications identified. Fannin (2000b) reports forensic observations from case studies that confirm the suitability of the AASHTO standard specifications for unpaved roads, erosion control works, subsurface drains and riprap revetments. For geogrids there are no similar guidelines available and the reader should consult with the manufacturer. However, field installation damage testing can be carried out using the construction method and equipment anticipated in the field structure. Mechanical degradation can then be quantified by comparing the results of standard testing (e.g. tensile testing) for exhumed and control geosynthetic products (FHWA 1997, WSDOT 2004). Recommended properties for geotextiles in railway rehabilitation applications are summarized in the papers by Raymond and Bathurst (1990) and Raymond (1999).

TABLE 23.4 *Construction survivability ratings (after AASHTO 2003)*

Site soil CBR at installation[1]	< 1		1 to 2		> 3	
Equipment ground contact pressure (kPa)	> 350	< 350	> 350	< 350	> 550	< 550
Cover thickness[2] (compacted, mm)						
100[3,4]	NR[6]	NR	H[6]	H	M[6]	M
150[5]	NR	NR	H	H	M	L[6]
300	NR	H	M	M	M	L
450	H	M	M	M	M	L

NOTES:

1. Assume saturated CBR unless construction scheduling can be controlled.
2. Maximum aggregate size not to exceed one-half the compacted cover thickness.
3. For low-volume, unpaved roads (ADT < 200 vehicles).
4. The 100 mm minimum cover is limited to existing road bases and is not intended for use in new construction.
5. Maximum aggregate size ≤ 30 mm.
6. NR = NOT RECOMMENDED; L = LOW; M = MODERATE; and H = HIGH.

TABLE 23.5 *Physical property requirements[1] (after AASHTO 2003)*

Survivability level	Grab Strength[4] ASTM D4632 (N)		Puncture Resistance[4] ASTM D4833 (N)		Tear Strength[4] ASTM D4533 (N)	
	<50% Geotextile elongation[2,3]	>50% Geotextile elongation[2,3]	<50% Geotextile elongation	>50% Geotextile elongation	<50% Geotextile elongation	>50% Geotextile elongation
High (Class 1)	1400	900	500	350	500	350
Moderate (Class 2)	1100	700	400	250	400	250
Low (Class 3)	800	500	300	180	300	180

Additional Requirements	**Test Method**
Apparent Opening Size < 50 % soil passing 0.075 mm sieve, AOS < 0.6 mm > 50 % soils passing 0.075 mm sieve, AOS < 0.3 mm	ASTM D4751
Permeability k of the geotextile > k of the soil (permittivity x the nominal geotextile thickness)	ASTM D4491
Ultraviolet Degradation At 500 hours of exposure, 50 % strength retained	ASTM D4355
Geotextile Acceptance	ASTM D4759

NOTES

1. For the index properties, the first value of each set is for geotextiles that fail at less than 50 % elongation, while the second value is for geotextiles that fail at greater than 50 % elongation. Elongation is determined by ASTM D4632.
2. Values shown are minimum average roll values. Strength values are in the weakest principal direction.
3. The values of the geotextile elongation do not relate to the allowable consolidation properties of the subgrade soil. These must be determined by a separate investigation.
4. AASHTO classification.

24 Lateral Earth Pressures & Rigid Retaining Structures

24 Lateral Earth Pressures & Rigid Retaining Structures

24.1 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.

$$K = \frac{\sigma'_h}{\sigma'_v} \tag{24.1}$$

24. 2 Earth Pressure at-Rest

The horizontal effective stress that 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:

$$K_o = 1 - \sin \phi' \tag{24.2}$$

This equation considers the case of zero lateral strain during deposition of the soil. K_o is known to increase, however, with overconsolidation of the soil resulting from stress changes. As a first approximation, in soils with an overconsolidation ratio, OCR, the following expression may be used:

$$K_{oOC} = (1 - \sin \phi')OCR^m \tag{24.3}$$

where m is an exponent related to the soil type and can be estimated as approximately equal to $1 - \sin\phi'$ (Mayne and Kulhawy 1982). Though this expression has been considered reasonably representative of horizontal stress conditions arising from changes in vertical stress (loading and unloading), apparent overconsolidation can be reflective of other natural conditions (e.g. weathering, cementation, desiccation).

The magnitude and use of K_o should be considered carefully as the construction of retaining structures can not occur without some change in the horizontal stress-state through excavation, temporary support (if needed), and backfilling. Few retaining structures are designed to consider K_{oOC} from natural overconsolidation using the above expression. In . placed against rigid retaining walls or rigid foundation walls, compaction increases the earth pressure, and values of K in excess of 1.0, and even close to the passive condition, have been observed (see Section 24.8).

24.3 Active and Passive Earth Pressure Theories

A number of theories have been developed and used to evaluate the lateral earth pressures on rigid retaining structures. The two most common are those developed by Coulomb (1776) and Rankine (1856). Coulomb's approach to the lateral earth problem generally included the following assumptions:

1. soil is isotropic and homogeneous;
2. soil shear strength is best characterized using the angle of internal friction;
3. the failure surface is planar;
4. the backfill surface is planar;
5. friction resistance is distributed uniformly along the failure surface;
6. the mass of soil between the wall and failure surface is a translating rigid body;
7. friction develops between the wall and soil mass (see Section 24.5); and
8. the resultant earth force acts at an angle parallel to the angle of wall friction as measured normal to the back face of the wall (see Figure 24.1 and Figure 24.3)

The assumptions forming the basis for Rankine's solution are nearly identical except that Rankine did not include friction along the back of the wall and the resultant active force was considered to act at an angle parallel to the backfill slope. Mathematical expressions for both approaches are provided in this Chapter with commentary on their application.

In general, the active and passive earth pressure coefficients that are presented in this Chapter are resolved for the horizontal direction in keeping with the direction of forces fundamental to the ideas presented in Sections 24.1 and 24.2. However within Figure 24.1 and Figure 24.3 solutions are also provided to assess the magnitude and direction of forces consistent with the theories of Coulomb and Rankine. Where the coefficients and forces are to be found in the direction assumed by either the Coulomb or Rankine methods, these are noted by additional subscripts relevant to the force direction (see Figure 24.1 and Figure 24.3).

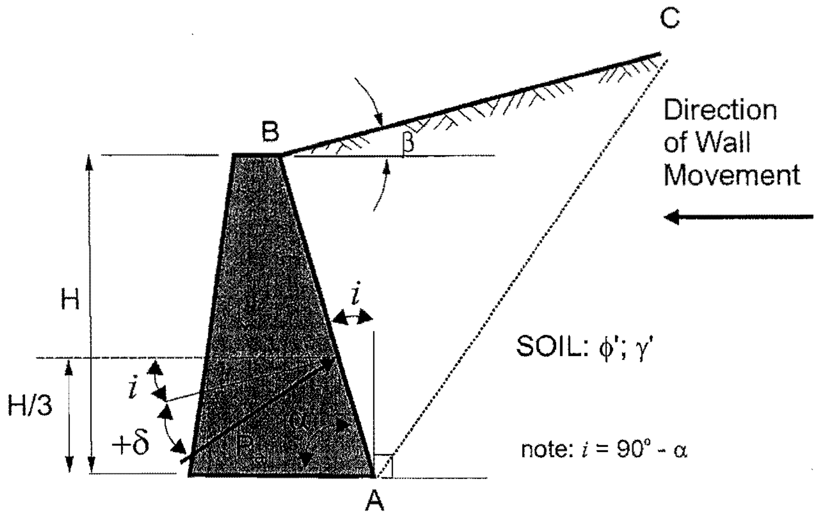
For active earth pressure calculations, both approaches can yield reasonable results, although the Rankine solution will generally be more conservative. For passive earth pressure solution, however, the failure surfaces are not planar and are more realistically described by logarithmic spirals or curved surfaces. For passive earth pressure calculations the Coulomb solutions can be unsafe depending on the degree of wall friction assumed to exist as discussed below.

24.3.1 Active Earth Pressure

The active earth pressure is the minimum value of lateral earth pressure that a soil mass can exert against a 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 that are not rigidly restrained from rotation or translation.

The ratio of lateral to vertical effective stress in a granular soil under active failure conditions, K_a, can be obtained from the formulae given in Figure 24.1. Wall friction can only mobilize by movement between the wall and the soil. In practice, the effect of wall friction is often ignored and the Rankine solutions are applied. If the effect of wall friction is to be included, the relative movement between the wall and backfill should be considered in detail and this may be appropriate for relatively high or flexible and semi-flexible walls. In general, the Coulomb solutions for active pressure conditions are not more than about 12 % different than the Rankine solutions where the back-slope of the wall is less than plus or minus 15° and the interface friction angle is less than $1/2\phi'$.

For stratified soils, K_a can be determined for each soil type. In general, the lateral earth pressure at any depth is equal to $K_a\sigma'_z$ where σ'_z is the vertical effective stress at depth z. When calculating the earth pressure, the most practical way to perform the calculations is to determine the total vertical stress, σ_z, and then by deducting the pore-water pressure, u, to determine the effective stress, σ'_z



Coulomb solution for active earth pressure coefficient:

$$K_a = \cos(\delta + i)\frac{\sin^2(\alpha + \phi')}{\sin^2\alpha \sin(\alpha - \delta)\left[1 + \sqrt{\frac{\sin(\delta + \phi')\sin(\phi' - \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}}\right]^2}$$

Note: K_a in *horizontal* direction

P_a in direction of δ as shown above = $(1/2\gamma H^2)K_a/\cos(\delta + i)$, or $K_{a\delta} = K_a/\cos(\delta + i)$

Rankine solution for active earth pressure coefficient:

$$K_a = \cos\beta\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}}$$

Note: K_a in *horizontal* direction

P_a in direction parallel to $\beta = (1/2\gamma H^2)K_a/\cos\beta$, or $K_{a\beta} = K_a/\cos\beta$

Note that if $\delta = 0$, $\beta = 0$ and $\alpha = 90°$ then both Coulomb and Rankine solutions produce:

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2(45 - \phi'/2)$$

FIGURE 24.1 *Active Earth pressure coefficients*

In cohesive soil with $\phi' = 0$, $\delta = 0$, $\alpha = 90°$, and $\beta = 0$, the total active earth pressure, p_a, is equal to the total vertical stress minus twice the undrained shear strength, s_u. as follows:

$$p_a = \sigma_z - 2\,s_u \tag{24.4}$$

In a soil exhibiting both cohesion and friction with $\delta = 0$, $\alpha = 90°$, and $\beta = 0$, the active effective earth pressure is as follows:

$$p_a = K_a\sigma'_z - 2c'(K_a)^{1/2} \tag{24.5}$$

These expressions, however, indicate that the pressures near the top of the wall can be negative. In practice, such calculated negative pressures are disregarded because of the propensity for cohesive soils to form tension cracks in such situations. Figure 24.2 illustrates theoretical and possible pressure distributions for design. Two assumed pressure distributions are shown as Figures 24.2b and 24.2c. Figure 24.2c is the most conservative design case. If it is likely that the tension crack could fill with water, hydrostatic pressures for the height of the crack should be added to the pressure distribution.

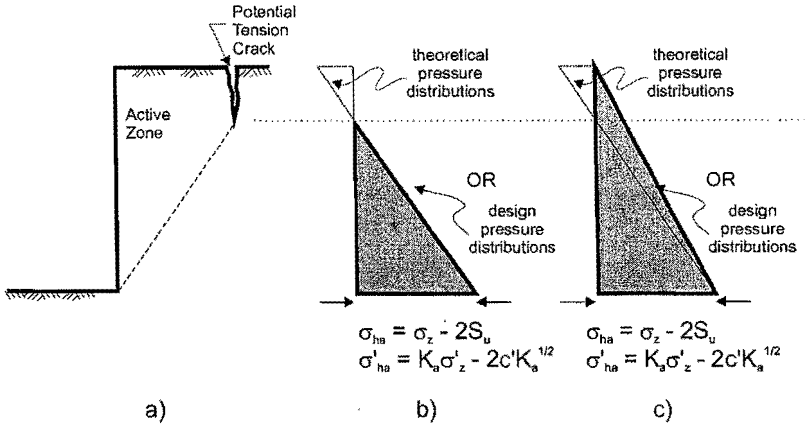


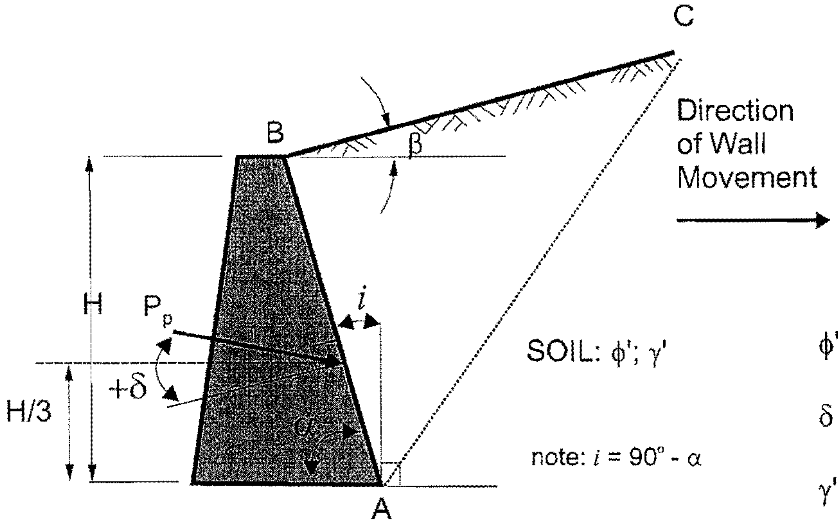
FIGURE 24.2 *Calculated and assumed active earth pressure distributions for cohesive and cohesive-frictional soils.*

The solutions provided above consider cohesion for simple cases. For more complex problems, including sloping backfill and wall surfaces reference should be made to Bell (1915).

24.3.2 Passive Earth Pressure

The passive earth pressure is the maximum value of lateral earth pressure that can be mobilized by the relative motion of 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 required to mobilize the soil strength can be large and the ability of the wall to move the required distance should be checked (see Section 24.4). If movement is restricted, lower pressure can be expected. On the other hand, compacted backfill (having already been 'prestressed' by the compacting work) can require very little movement to produce pressures approaching the full lateral passive value.

The ratio of lateral to vertical effective stress in a granular soil under passive failure conditions, K_p, can be obtained from the formulae given in Figure 24.3 for rigid retaining structures. Wall friction can only mobilize by relative movement between the wall and the soil. To fully mobilize the passive resistance, movement on the order of 2 % strain (dense sand) to 15 % (loose sand) is required. In practice, the effect of wall friction is often ignored and the Rankine solutions are applied, especially related to passive earth pressure calculations. During passive failure, the failure surface is not the planar geometry assumed by both Coulomb and Rankine theories.



φ' = effective angle of internal soil friction
δ = angle of interface friction between wall and soil
γ'= effective unit weight of

Coulomb solution for passive earth pressure coefficient:

$$K_p = \cos(\delta - i)\frac{\sin^2(\alpha - \phi')}{\sin^2\alpha \sin(\alpha + \delta)\left[1 - \sqrt{\dfrac{\sin(\delta + \phi')\sin(\phi' + \beta)}{\sin(\alpha + \delta)\sin(\alpha + \beta)}}\right]^2}$$

Note: K_p in *horizontal* direction

P_p in direction of δ as shown above = $(1/2\gamma' H^2)K_p/\cos(\delta - i)$, or $K_{p\delta} = K_p/\cos(\delta - i)$
P_p = total passive load

Rankine solution for passive earth pressure coefficient:

$$K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}$$

Note: K_p in *horizontal* direction

P_p in direction parallel to β = $(1/2\gamma H^2)K_p/\cos\beta$, or $K_{p\beta} = K_p/\cos\beta$

Note that if δ = 0, β = 0 and α = 90° then both Coulomb and Rankine solutions produce:

$$K_p = \frac{1 + \sin\phi'}{1 - \sin\phi'} = \tan^2(45 + \phi'/2)$$

FIGURE 24.3 *Passive earth pressure coefficents*

For this reason, Coulomb's solutions can be unrealistic and unsafe where the interface friction angle is assumed to be greater than about 14° or $1/3\phi'$. Charts for higher values of wall friction angle in relation to wall back slope and backfill surface slope can be found using the charts for curved failure surfaces provided in Caquot and Kerisel (1949), Tschebotarioff (1973), and NAVFAC (1988).

Table 24.1 and Table 24.2 provide passive earth pressure coefficients for common values of β and ϕ' for a rigid retaining wall with a vertical back face based on the Caquot and Kerisel (1949) approach to passive earth pressure problems. This method assumes that the resultant force acts in the direction of δ. Table 24.1 provides the maximum passive earth pressure coefficients where $\delta = \phi'$. To find values of the passive earth pressure coefficients for other values of δ, multiply the value of $K_{p\delta}$ from Table 24.1 by the reduction factor, R, based on the ratio of the interface friction angle and effective angle of soil friction as provided in Table 24.2. To find the values of K_p in the horizontal direction, multiply the Caquot and Kerisel value by $\cos\delta$. Note that for a wall friction angle of $\delta/\phi' = 0$ and $\beta = 0$, the resulting passive earth pressure is nearly identical to the Rankine solution.

For stratified soils, K_p, can be determined for each soil type. In general, the lateral earth pressure at any depth is equal to $K_p\sigma'_z$, where σ'_z is the effective stress at depth z.

When $\delta = 0$, $\alpha = 90°$, and $\beta = 0°$, the ratio of lateral to vertical effective stress under passive failure conditions, K_p, is the inverse of the active earth pressure coefficient, K_a.

TABLE 24.1 *Values of $K_{p\delta}$ for a Wall with a Vertical Back Face ($\alpha = 90°$, $i = 0$) and $\delta = \phi'$ (after Caquot and Kerisel, 1949)*

ϕ'	Flat $\beta = 0.0°$	Positive β 4H:1V $\beta = 14.0°$	Positive β 3H:1V $\beta = 18.4°$	Positive β 2H:1V $\beta = 26.6°$	Negative β 4H:1V $\beta = -14.0°$	Negative β 3H:1V $\beta = -18.4°$
20	3.0	4.3	4.5	---	1.6	1.2
25	4.3	6.2	7.0	---	2.3	1.8
30	6.4	9.0	12.0	15.0	3.3	2.8
35	10.2	17.5	20.3	29.0	5.0	4.3
40	17.5	35.0	40.0	60.0	8.0	6.0
45	33.5	65.0	85.0	130.0	14.3	12.0

TABLE 24.2 *Reduction Factor R for $K_{p\delta}$ Wall Friction between $\delta/\phi' = 1$ and $\delta/\phi' = 0$ (after Caquot and Kerisel, 1949)*

ϕ'	Reduction Factor, R where $K_{p\delta} = RK_{p\delta=\phi'}$							
	$\delta/\phi' = 0.7$	$\delta/\phi' = 0.6$	$\delta/\phi' = 0.5$	$\delta/\phi' = 0.4$	$\delta/\phi' = 0.3$	$\delta/\phi' = 0.2$	$\delta/\phi' = 0.1$	$\delta/\phi' = 0.0$
20	0.939	0.901	0.862	0.824	0.787	0.752	0.716	0.678
25	0.912	0.860	0.808	0.759	0.711	0.666	0.620	0.574
30	0.878	0.811	0.764	0.686	0.627	0.574	0.520	0.467
35	0.836	0.752	0.674	0.603	0.536	0.475	0.417	0.362
40	0.783	0.682	0.592	0.512	0.439	0.375	0.316	0.262
45	0.718	0.600	0.500	0.414	0.339	0.276	0.221	0.174

In cohesive soil with $\phi' = 0$, $\delta = 0$, $\alpha = 90°$, and $\beta = 0$, the unit passive total earth pressure, p_p, is equal to the total vertical stress plus twice the undrained shear strength, s_u, as follows:

$$p_p = \sigma_z + 2s_u \tag{24.6}$$

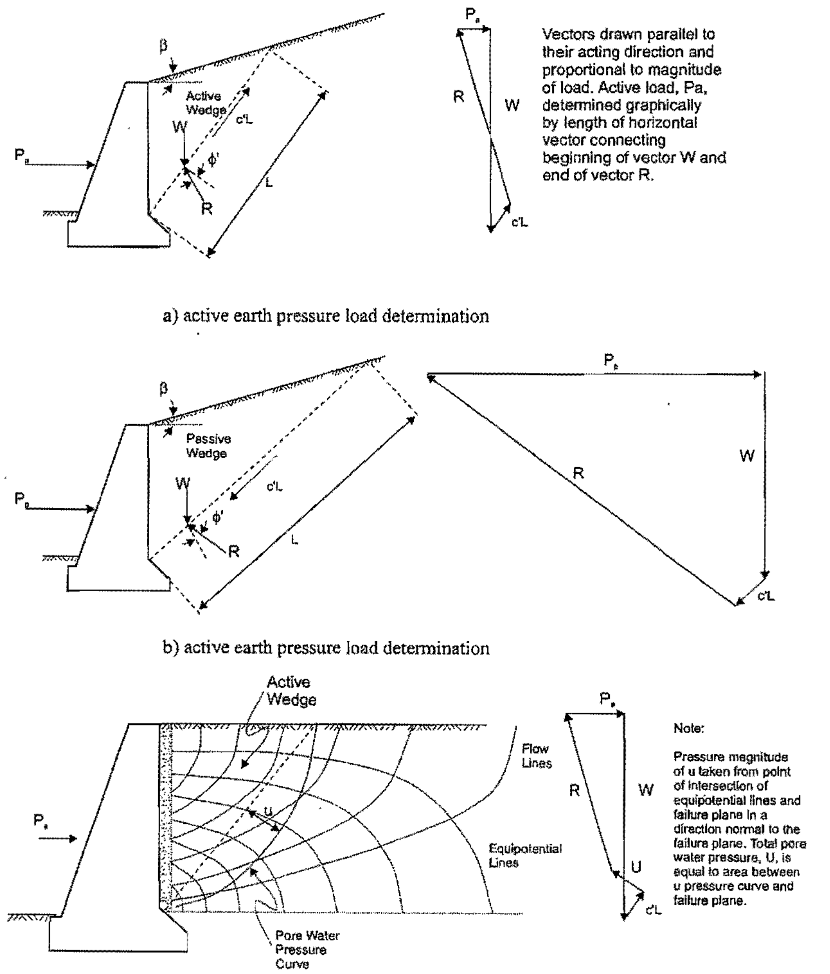
In a soil exhibiting both cohesion and friction with $\delta = 0$, $\alpha = 90°$, and $\beta = 0$, the passive effective earth pressure is as follows:

$$p_p = K_p\sigma'_z + 2c'(K_p)^{1/2} \tag{24.7}$$

The solutions provided above that consider cohesion are for simple cases. For more complex problems, including sloping backfill and wall surfaces reference should be made to Bell (1915).

24.3.3 Graphical Solutions for Determination of Loads due to Earth Pressures

For a homogeneous backfill material, the resultant active and passive loads, P_a and P_p, can be determined graphically according to Figure 24.4. These graphical solutions assume that sufficient movement has occurred so that the full shear strength of the soil along the shear plane is mobilized. When using the graphical solutions, several trial failure surfaces should be examined to find the maximum value of P_a and the minimum value of P_p. No wall friction is considered in the graphical solution provided in Figure 24.4 and it is also considered that the wall back face is vertical. If any pore water pressures are present (see also Section 24.6) these must be considered in determination of the resultant load. Surcharge loads must be added separately.



a) active earth pressure load determination

b) active earth pressure load determination

c) active earth pressure and water pressure load determination
for case of backfill saturated by rainfall seeping to a back drain

FIGURE 24.4 *Graphical solutions for active and passive pressures for rigid and frictionless walls with a vertical back face*

24.4 Earth Pressure and Effect of Lateral Strain

Lateral strain in the soil will alter its horizontal stress condition. Depending on the magnitude and direction of the strain involved, the final horizontal stress can lie anywhere between two limiting (soil failure) conditions. The limiting stresses occur at the active state when the wall moves away from the soil mass, and at the passive failure state when the wall moves into the soil mass.

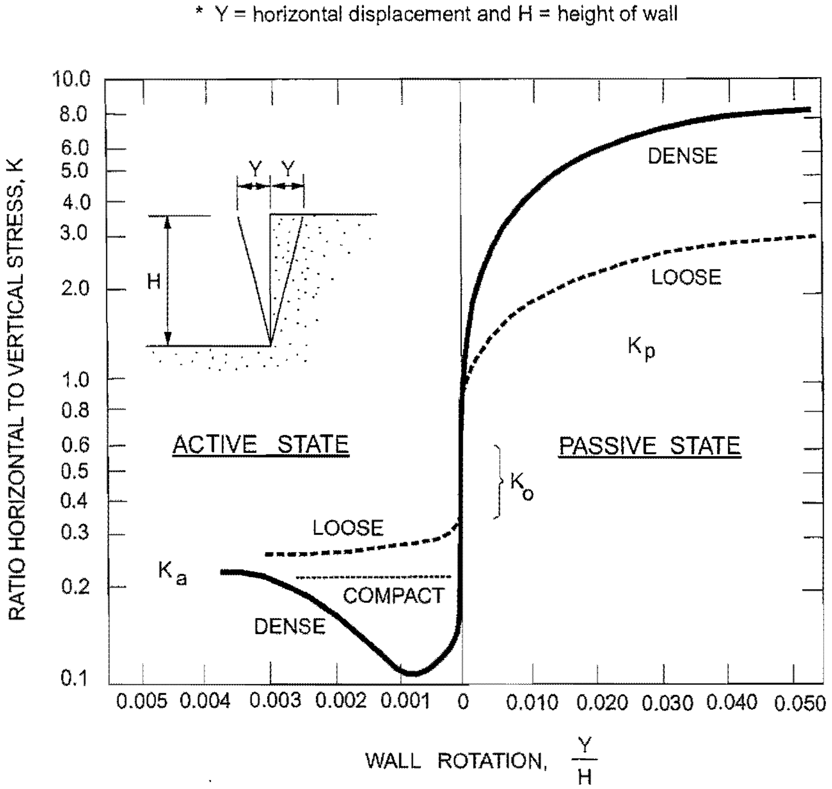


FIGURE 24.5 *Effect of deformation on earth pressures in cohesionless materials*

Figure 24.5 illustrates the dominant role of soil strain in determining the horizontal stress acting on the supporting structure. The magnitude of wall rotation, Y/H, required to achieve active and passive earth pressure conditions in various soil types is indicated in Table 24.3.

TABLE 24.3 *Magnitude of Wall Rotation to Reach Active and Passive Earth Pressures*

Soil Type and Condition	Rotation, Y/H[1]	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

[1] where Y = amount of horizontal translation or rotational displacement of the wall top relative to the wall bottom.

Soft cohesive materials will continue to creep at pressures close to active and passive earth pressures and with time the deformations will increase if the walls are sufficiently flexible to allow full development of active and passive earth pressures.

Table 24.3 and Figure 24.5 describe the effects of simple rotation or translation of a rigid wall. Other deformation patterns will induce different earth pressure magnitudes and distributions. Such deformation-dependent stress changes are most pronounced on flexible retaining structures and are discussed in more detail in Chapter 26.

24.5 Wall Friction

Unless a wall is settling, friction on its back acts upward on the active wedge, reducing active pressures. Wall friction acts downward against the passive wedge, resisting its upward movement and increasing passive pressures. In general, the effect of wall friction on active pressure is small and is often disregarded. The effect of wall friction on Coulomb's 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 assumed to be in the range of one-half to two-thirds of ϕ'. Values of δ greater than ½ of ϕ' should not be used in calculations of passive earth pressure using the Coulomb approach since unrealistically large estimates will result. Values for friction factors between dissimilar materials are provided in Table 24.4.

TABLE 24.4 *Angles and Coefficients of Friction and Adhesion between Dissimilar Materials*

Structure Materials	Ground or Backfill Materials[1]	Interface Friction Angle, δ	Friction Factor, $\tan\delta$
Frictional Materials			
Mass concrete or masonry	Clean sound rock	35	0.7
	GW, GP	29 to 31	0.55 to 0.60
	GM, GC, SW, SP	24 to 29	0.45 to 0.55
	SM, SC	19 to 24	0.35 to 0.45
	ML (non-plastic)	17 to 19	0.30 to 0.35
	CL (stiff to hard), residual soils	22 to 26	0.40 to 0.50
Steel sheet piles	Well-graded rock fill	22	0.40
	Uniform size rock fill	17	0.30
	GW, GP	22	0.40
	GM, GC, SW, SP	14	0.25
	SM, SC	14	0.25
	ML (non-plastic)	11	0.20
Formed or pre-cast concrete	Well-graded rock fill	22 to 26	0.40 to 0.50
	Uniform size rock fill	17 to 22	0.30 to 0.40
	GW, GP	22 to 26	0.40 to 0.50
	GM, GC, SW, SP	17	0.30
	SM, SC	14	0.25
	ML (non-plastic)	14	0.25

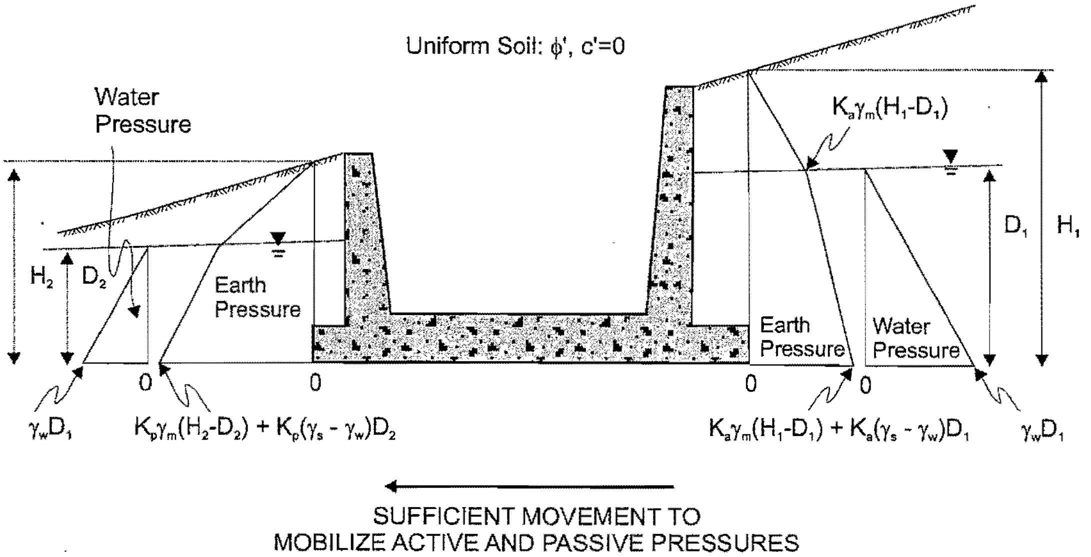
Adhesion in Cohesive Soils		
Consistency	Undrained Shear Strength, s_u, kPa	Adhesion (kPa)
Very Soft	0 to 12	0 to 12
Soft	12 to 25	12 to 25
Medium Stiff	25 to 50	25 to 38
Stiff	50 to 100	38 to 48
Very Stiff	100 to 200	48 to 65

1 two-letter designations refer to USCS soil classifications (see Chapter 3).

24.6 Water Pressure

The effect of the greatest unbalanced water pressure that will act across the wall must be included in the design of rigid retaining structures (see Figure 24.6). For instance, in cohesionless soils, the total active force on a wall with static water level at the top of the backfill is more than double that for dry backfill against the same wall. To obtain the total pressure, or thrust, on the wall, the net water pressure must be added. Flow nets or computer analyses should be used to assess potential pore-water pressure conditions and their effect on active and passive pressures for all conditions in cases of seepage behind and around retaining structures. For design of rigid retaining structures, it is commonly considered that a free-draining backfill or a drainage system will be constructed during backfilling (see Figure 24.7). Seepage effects around retaining structures are more common for design and construction of flexible walls and bulkheads and such seepage effects are discussed in greater detail in Chapter 26. When the wall is intended to prevent all leakage of groundwater, maximum exterior groundwater pressures should be used.

Where the phreatic surface of a pore water pressure profile intersects the potential failure plane, pore water pressures must be accounted for even when the backfill is not saturated beneath a horizontal and static groundwater level. Saturation of only a part of the backfill can occur with a source of seepage through the native or backfill soils and when only partial drainage is provided (see Figure 24.7). Such pore water pressures are not generally considered in the calculation of the active and passive earth pressure coefficients. In general, a graphical and iterative approach to finding the resultant earth loads should be performed taking into account the pore water pressures derived from flow net or computer-aided seepage analyses. A simple case is illustrated in Section 24.3.3 and Figure 24.4 where a wedge analysis and a flow net are used for determination of resultant active earth pressures considering a saturated active zone.



K_a = active earth pressure coefficient
γ_m = moist or wet soil unit weight
γ_s = saturated soil unit weight
γ_w = unit weight of water
K_p = passive earth pressure coefficient

FIGURE 24.6 *Effects of groundwater on active and passive pressures*

24.7 Surcharge Loading

24.7.1 Uniform Area Loads

Where the surcharge behind a wall consists of a large uniformly loaded area, with an intensity that is small compared to the total backfill forces (total force on wall from surcharge is less than 30 % of the active force), the additional wall pressure can be calculated using:

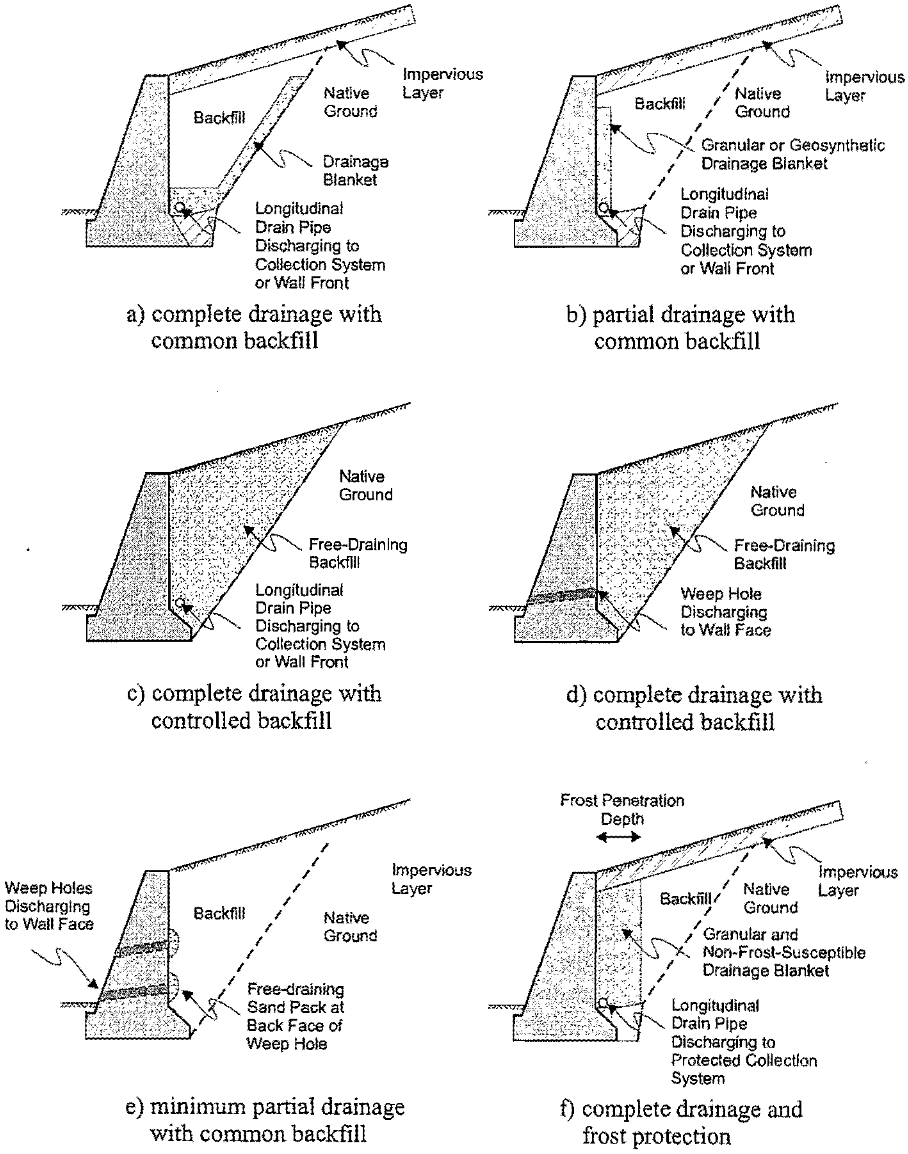
$$\sigma'_{hs} = q\,K \tag{24.8}$$

where

σ'_{hs} = horizontal pressure due to surcharge;
q = uniform surcharge pressure; and
K = applicable earth pressure coefficient (i.e. K_o or K_a)

24.7.2 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 pressure may be calculated using the formulae in Figure 24.8. For heavy surcharges, a wedge analysis should be used.



Note: All drainage materials, including backfill, weep hole sand packs, drainage blankets, and all geosynthetics must be designed to limit loss of soil according to proper filtration criteria.

FIGURE 24.7 *Schematic drainage designs for rigid retaining walls*

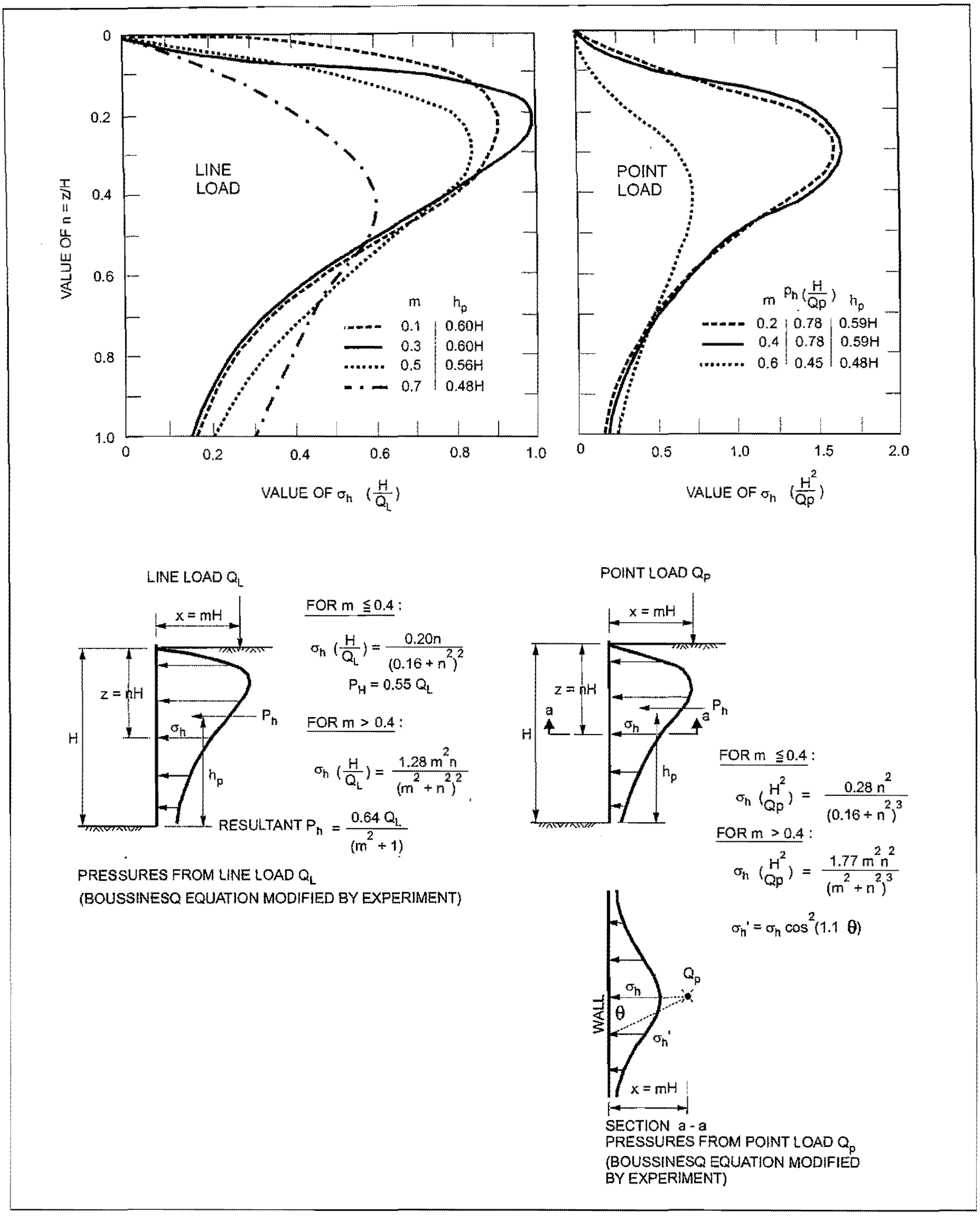
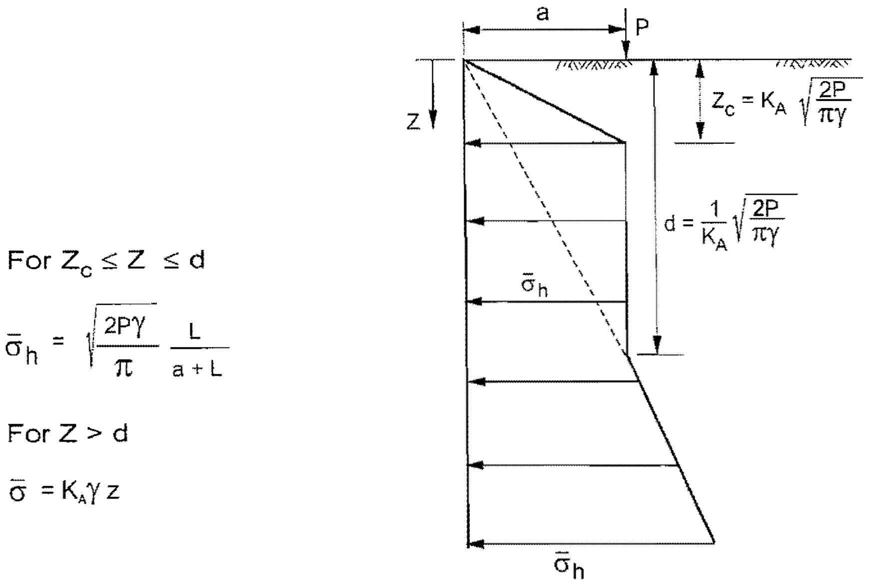


FIGURE 24.8 *Horizontal pressure on a wall due to point and line load surcharges*

24.8 Compaction-Induced Pressures

Compaction of backfill behind retaining structures can induce loads greater than the active or at-rest earth pressures calculated using the formulae presented above. Compaction-induced pressures can be such that the structural capacity of walls designed to withstand K_o or K_a can be exceeded. Figure 24.9 illustrates the general distribution and calculation of stresses from compaction. For reference purposes, Table 24.5 provides some typical values of P for various small roller compaction equipment types.



$$P\text{ (roller load)} = \frac{\text{dead weight of roller + centrifugal force}}{\text{weight of roller}}$$

a = distance of roller from wall

L = length of roller

FIGURE 24.9 *Horizontal pressure on walls from compaction effort*

TABLE 24.5 *Typical Compaction Equipment Data for Estimating Compaction-Induced Loads*

Equipment Type	Dead Weight (kN)	Centrifugal Force (kN)	Roller Width (mm)	P (kN/m)
Single-drum walk-behind	2.3	8.3	560	18.9
Dual-drum walk-behind	1.6	10.1	560	20.9
Dual-drum walk-behind	12.1	8.8	760	27.5
Dual-drum walk-behind	9.2	19.8	750	38.7

Small vibratory or hand-operated ram compaction equipment can exert less compaction-induced stresses than the equipment noted in Table 24.5. Analysis, design, and construction specifications for retaining structures backfilled with compacted materials should consider in detail the physical requirements for compaction equipment or the necessary limitations thereof.

24.9 Earthquake-Induced Pressures

Earthquakes will induce additional pressures on retaining structures. The magnitude and distribution of earthquake-induced loads is often approximated using the Mononobe-Okabe equations (Mononobe and Matsuo 1929, Okabe 1926). In general, these equations are extensions of Coulomb's analysis for active forces on retaining walls and are provided below.

For active earth pressure loads:

$$P_{ae} = \tfrac{1}{2}\gamma H^2(1-k_v)K_{ae} \qquad (24.9)$$

where

P_{ae} = resultant active lateral earth load including static and dynamic loads;
γ = unit weight of the soil behind the wall;
k_v = vertical component of the earthquake acceleration (as a decimal fraction of the acceleration due to gravity);
k_h = horizontal component of the earthquake acceleration (as a decimal fraction of the acceleration due to gravity); and
K_{ae} = horizontal component of active earth pressure coefficient including effects of earthquake loading;

and

$$K_{ae} = \frac{\cos(\delta + \alpha\cos^2(\phi' - \varphi - i)}{\cos^2 i \cos\delta \cos(\delta + i + \varphi)(1 + X_a^{1/2})^2} \qquad (24.10)$$

$$X_a = \frac{\sin(\delta + \varphi')\sin(\phi' - \varphi - \beta)}{\cos(\delta + i + \varphi)\cos(\beta - i)} \qquad (24.11)$$

$$\varphi = \tan^{-1}[k_h/(1 - k_v)] \qquad (24.12)$$

$$i = 90 - \alpha \qquad (24.13)$$

and all other notation are as above and in Figure 24.1 and Figure 24.3.

The total active earth pressure coefficient considering earthquake loading acting along the line parallel to the wall friction angle measured normal to the plane of the wall back face is equal to $K_{ae}/\cos(\delta + i)$, see Figure 24.1.
For passive earth pressure loads:

$$P_{pe} = \tfrac{1}{2}\gamma H^2(1-k_v)K_{pe} \qquad (24.14)$$

where

P_{pe} = resultant passive lateral earth load including static and dynamic loads;
K_{pe} = active earth pressure coefficient including effects of earthquake loading;

and

$$K_{pe} = \frac{\cos(\delta - i)\cos^2(\phi' + i - \varphi)}{\cos^2 i \cos\varphi \cos(\delta - i + \varphi)(1 - X_p^{1/2})^2} \qquad (24.15)$$

$$X_p = \frac{\sin(\delta - \phi')\sin(\phi' + \beta - \varphi)}{\cos(\delta - i + \varphi)\cos(\beta - i)} \qquad (24.16)$$

The value of φ reduces to 0 when no earthquake loads exist, so $K_{ae} = K_a$ and $K_{pe} = K_p$.

The total passive earth pressure coefficient considering earthquake loading acting along the line parallel to the wall friction angle measured normal to the plane of the wall back face is equal to $K_{pe}/\cos(\delta - i)$, see Figure 24.3. As with static passive earth pressure calculations, the effect of wall friction should be used with caution as unrealistically high values may result if values of δ greater than about $\phi'/3$ or $\phi'/2$ are used.

The locations of the resultant forces need to be defined to calculate moments for completion of retaining structure design. In general, the incremental loads induced by earthquake forces only, ΔP_{ae} and ΔP_{pe}, can be defined as:

$$\Delta P_{ae} = P_{ae} - P_a \qquad (24.17)$$

$$\Delta P_{pe} = P_{pe} - P_p \qquad (24.18)$$

where P_a and P_p equals the static active and passive earth loads derived using the methods provided previously.

The Mononabe-Okabe determination of the active and passive earth pressures does not provide any indication of the distribution of the loads. It is considered that the increases in active and passive incremental earthquake pressures are greater near the top of the wall. Therefore, it is common to apply the resultant incremental earthquake loads at a height of 0.6H where H is the height above the bottom of the wall.

If earthquake forces are to be considered in retaining wall design it is also reasonable to utilize a lower factor of safety of about 1.2.

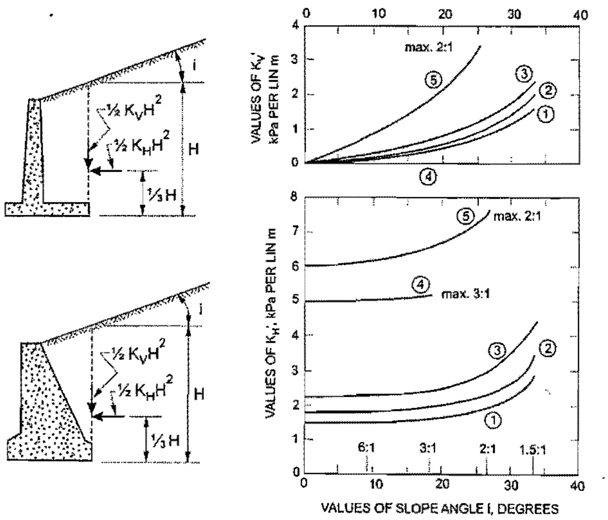
24.10 Frost-Induced Loads

Retaining walls, if backed by frost susceptible soils, can be subjected to forces many times the active, at-rest, or compaction-induced earth and hydrostatic forces for which they were designed. Guidance on soil types susceptible to frost action is provided in Chapter 13. The pressures generated by frost action are a complex combination of factors including the temperature gradient, moisture supply within the soil, overburden stresses, and soil composition. In general, walls are not designed to withstand pressures induced by frost. Rather, actions are taken to eliminate or minimize the potential for frost development behind the wall. Such actions include:

- providing sufficient drainage and surface barriers to isolate the wall and backfill from sources of water and moisture (see also Figure 24.7);
- providing a non-frost-susceptible backfill; and
- providing thermal insulation.

24.11 Empirical Pressures for Low Walls

For walls less than 6 m in height and where the type, placement and control of backfill material are not clearly specified, the backfill pressures should be estimated from the charts on Figures 24.10 and 24.11. Uniform surcharge should be considered an equivalent weight of backfill; the force on the wall from a line load surcharge can be estimated using Figure 24.8.



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 tosoft 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 1.2 m; resultant acts at a height of (H-1.2)/3 above base.

FIGURE 24.10 *Design loads for low retaining walls and straight sloping backfill (after Terzaghi and Peck, 1967)*

The backfill pressures in Figures 24.10 and 24.11 include the effects of seepage and time-dependent 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. Pervious backfill consisting of soil types should be covered with a surface layer of impervious soil to limit infiltration of precipitation and storm run-off. Interface stability between the dissimilar backfill layers should also be checked during design.

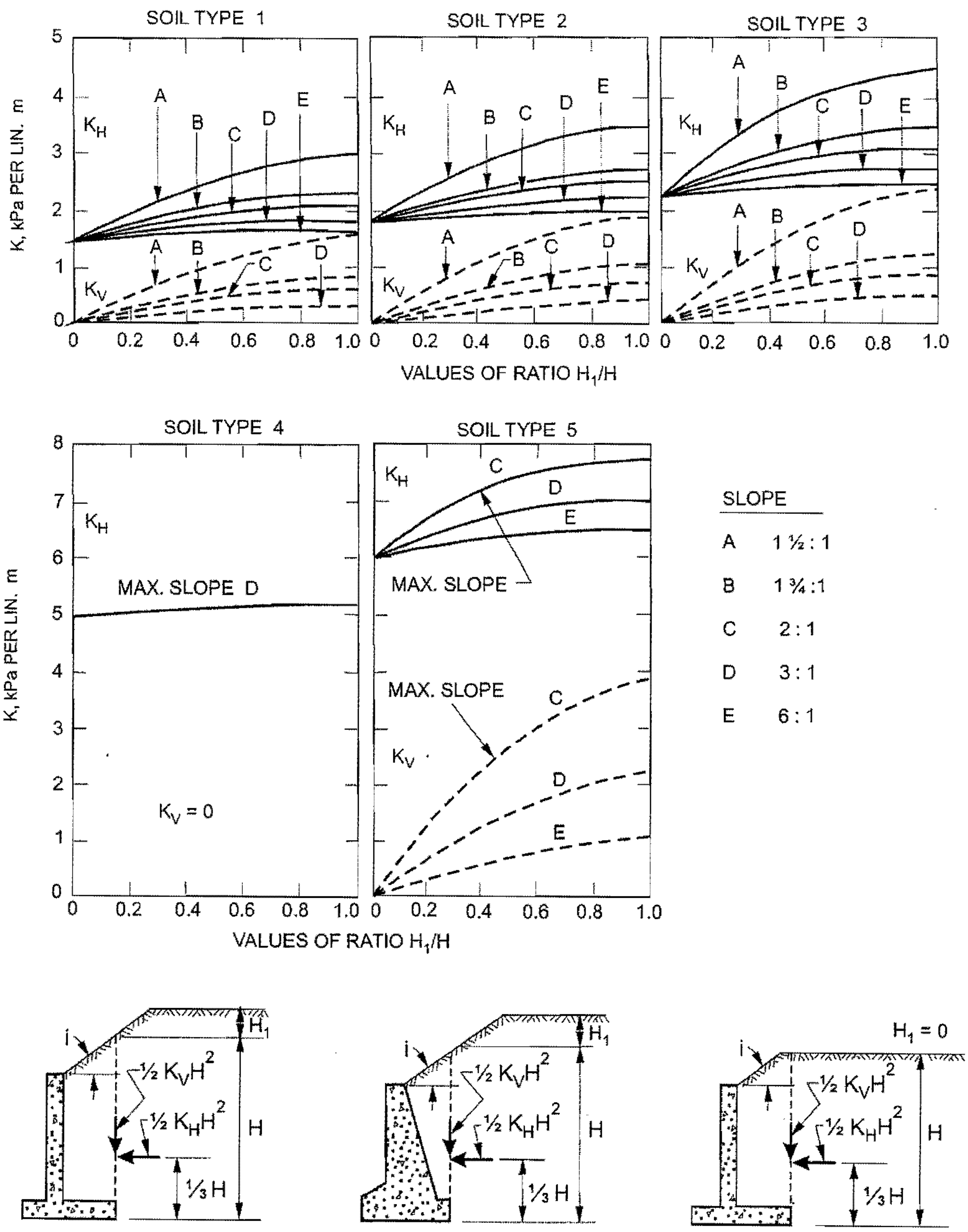


FIGURE 24.11 *Design loads for low retaining walls with broken slope backfill (after Terzaghi and Peck, 1967)*

For further guidance on the use of Figures 24.10 and 24.11 refer to Terzaghi et al. (1996).

24.12 Design of Rigid Retaining Walls

Rigid retaining walls develop their lateral resistance primarily from their self-weight and the weight of backfill placed above structural elements behind the wall face. Rigid retaining walls include concrete gravity, cantilever and counterfort retaining walls as well as bin or crib walls. They must be designed to ensure adequate security against failure in sliding and bearing.

"Unrestrained" walls are walls that are free to move sufficiently to allow active pressures to develop behind the wall in the limiting condition.

"Restrained" walls are walls that are prevented from moving sufficiently for active pressures to develop behind the wall in the limiting condition, when bearing or sliding failure is occurring.

24.12.1 Design Earth Pressures

Since equations for determining factors of safety against sliding and bearing assume limit conditions, the active pressure condition is appropriate for determining the stability of unrestrained walls. Active earth pressures are appropriate for the calculation of sliding and bearing of all free-standing retaining walls and most bridge abutments with flexible bearings. At-rest pressures should be used for the design of restrained walls including backfilled basement walls, rigid portals and box culverts.

Design of permanent single-level or multiple-level basement or foundation walls that are constructed within a shored excavation do not conform to the above general conditions of "restrained" walls. Though the permanent structure may be restrained from movement at various points, the process of excavation and shoring will induce some permanent deformations and stress relief in the ground. The appropriate magnitude and distribution of stresses for design of the permanent structure should likely lie somewhere between the at-rest earth pressure and the pressure exerted on the shoring system. For restrained basement walls constructed within very flexible shoring (e.g. sheet pile or soldier-pile and lagging walls) the earth pressure on the permanent structure will likely not exceed that exerted on the shoring. When the temporary shoring system is design to be integral with the permanent structure (e.g. permanent concrete diaphragm walls) the final pressure will likely be closer to at-rest pressure conditions, though the distribution of this load may differ. In any such cases, however, the total resultant thrust on restrained walls should be greater than the active pressures as full mobilization of the soil strength will likely not be achieved if deflections are to be controlled. Selection of wall design pressures for permanent restrained basement walls built within shored excavations may be facilitated by careful use of computer models. Selection of shoring design pressures and use of computer models for such purposes is discussed further in Chapter 26. The final choice of design pressures should be made based on experienced judgement considering the shoring, construction processes, ground and water conditions along with design life, deformation, and final performance goals.

Design of the structural components of rigid retaining walls should be based on the maximum load on the wall. Thus, the stem of a cantilever wall should be designed for the actual earth and water pressures likely to occur, taking account of deflection and rotation of the wall during construction and the effect of compaction.

24.12.2 Effects of Backfill Extent

When designing a retaining wall for a design load based on active pressure, the design should take into consideration the extent of the backfill. The maximum active pressure may develop either from limiting conditions occurring entirely within the backfill material or from potential failure surfaces within the material present behind the zone of backfill.

24.12.3 Backfill Types

24.12.3.1 Cohesionless Soils

Soils classified as GW, GP, SW, or SP, Unified Soil Classification System, USCS (USACE WES, 1967) are excellent backfill materials. When using such soils, theoretical earth pressures are valid for design purposes.

24.12.3.2 Sandy Clays and Clayey Sands

Soils classified as SC, SM, GC or GM, according to the USCS, may be suitable as wall backfill if kept dry, but are subject to frost action when wet. Where drainage is adequate, theoretical earth pressures are valid for design purposes. These soils cannot be properly compacted when wet.

24.12.3.3 Silts and Clayey Silts

Soils classified as CH, CL, MH, ML or OL, according to the USCS, are often subject to excessive frost action and swelling when used as wall backfill. Wall movement is likely to be excessive; the use of these materials should be avoided. Where they must be used, frost protection should be provided and an earth-pressure coefficient of 1.0 used in design.

24.12.4 Stability of Rigid Retaining Structures

Rigid retaining walls can fail under several different conditions:

1. Overturning Stability: If the earth pressures behind the wall are great enough to overcome the deadweight of the wall (including any backfill over the wall base) and any passive resistance in front of the wall, the wall can overturn as a whole.

2. Sliding Stability: If the earth pressures behind the wall are great enough to overcome the frictional resistance along the base of the wall, and any passive resistance in front of the wall, the wall can slide along its base as a whole.

3. Bearing Capacity: The bearing capacity of the supporting ground must be sufficient to resist the distribution of overturning and gravity loads imposed on the wall foundation. Progressive bearing capacity failure of the ground can lead to a shift in the center of gravity and rotation of the wall and subsequent overturning instability.

4. Settlement: Though the bearing capacity beneath the wall may be adequate, excessive settlement under the more highly loaded areas of the wall footing can also lead to overturning instability as for the bearing capacity case above.

5. Overall Stability: The overall stability of the retaining structure and the surrounding ground must be satisfactory or the wall and the retained soil can fail as a mass with the failure surface passing behind the wall backfill and below the wall. The wall itself may be adequately designed to resist the earth pressures, but if the stresses resulting from the change in grade levels exceeds the strength within the surrounding ground, the wall will not remain stable.

Figure 24.12 presents the general cases to be evaluated for the stability of rigid retaining structures. Additional discussion on each of these stability cases is provided below.

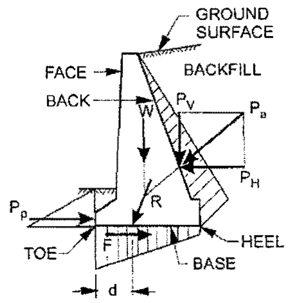
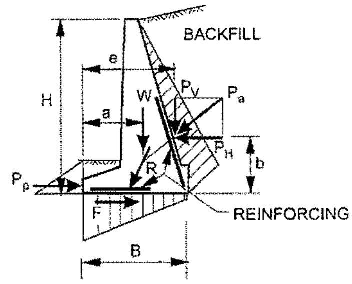
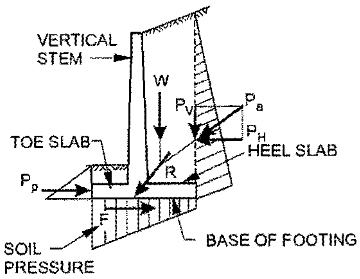
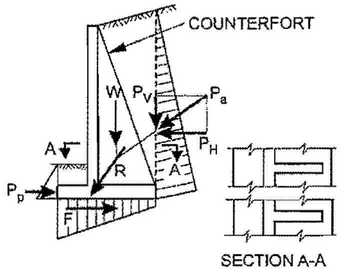
TYPE OF WALL	LOAD DIAGRAM	DESIGN FACTORS
GRAVITY		LOCATION OF RESULTANT Moments about toe: $d = \dfrac{Wa + P_V e - P_H b}{W + P_V}$ Assuming $P_p = 0$
SEMI-GRAVITY		RESISTANCE AGAINST SLIDING $F_s = \dfrac{(W + P_V)\tan\delta}{P_H} \geqq 1.5$ $F_s = \dfrac{(W + P_V)\tan\delta + P_p}{P_H} \geqq 2.0$ $F = (W + P_V)\tan\delta$
CANTILEVER		$\tan\delta$ = Friction between soil and base W = Buoyant weight of wall. Includesweight 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. Must be reduced by any uplift force due to water pressures, including seepage pressures.
COUNTERFORT		Pp = Force due to passive pressure below frost depth. Must take account of water pressure including seepage pressures. OVERALL STABILITY, CONTACT PRESSURE(S) For analysis of overall stability and contact pressures, see Chapter 8 through 12 of the manual.

FIGURE 24.12 *Design criteria for rigid retaining walls*

24.12.4.1 Stability against Overturning

To adequately resist overturning, the wall can include additional deadweight positioned such that its center of gravity produces an overturning moment opposite in direction to the soil pressures acting to overturn the wall. The deadweight can be provided simply by the wall mass or by extending the rear of the wall footing (heel) such that the overlying backfill provides the vertical load. Alternatively, the toe (front of the wall foundation) can be extended to provide additional resistance to overturning by mobilizing the bearing capacity of the soil in front of the wall. Extending the wall toe, however, is typically less efficient than extending the heel and is generally only used when insufficient space allows additional backfill placement.

24.12.4.2 Stability against Sliding

The base of a retaining wall should be placed at a sufficient depth below ground surface to allow for frost action, seasonal volume changes, and scour. In no case should the base be less than 1m below ground surface. Sliding stability must be adequate without including the passive resistance above the frost depth as frost action can degrade the strength of the natural or backfill materials. If insufficient sliding resistance is available, piles or anchors can be provided, or the depth increased. If weak layers are present in the soil or rock, the mechanism of sliding along the weak layer must be considered in the analysis.

Seepage under a wall can cause a significant reduction in sliding and passive resistance. Sliding resistance and passive pressures must take due account of water pressures on the base or pore pressures in the soil or rock, including any seepage effects. Seepage pressures should be determined through use of graphical flow nets or suitable numerical modeling techniques (see also Chapter 22).

A shear key may be provided to increase the sliding stability of walls. The effect of a shear key can be considered to either:

- ensure that sliding failure takes place within the soil or rock and not at the interface the base of the retaining wall footing; or
- mobilize passive resistance from below the frost depth to the base of the key, in which case loading from the active pressures should be considered from the bottom of the footing to the base of the shear key.

24.12.4.3 Stability against Bearing Failure

Allowable bearing pressure at the base of the wall should be checked by the methods described in Chapters 9 and 10 of this Manual, taking into account the eccentricity and inclination of the resultant load applied by the wall foundation to the supporting ground. If the eccentricity and inclination of the resultant load are accounted for a separate check on overturning is not required. The factored geotechnical bearing resistance at ultimate limit states should be checked as described in Chapters 8 and 10. In general, the footing dimensions should be selected such that the resultant load acts within the middle third of the footing when using working stress methods loads to avoid potentially large toe edge pressures. The Canadian Highway Bridge Design Code (2000) requires that the resultant of all factored loads remains within a distance to the footing centre-line of 0.3 times the footing dimension in the direction of the eccentric load being considered.

24.12.4.4 Settlement

If the foundation soil is compressible, the settlement should be computed and the tilt of the rigid wall due to this settlement estimated. If the consequential tilt exceeds acceptable limits, alternative foundations should be provided. In general, the footing dimensions should be selected such that the resultant load acts within the middle third of the footing to avoid potentially large toe edge pressures and consequent differential settlement or tilting.

22.12.4.5 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 and well behind the backfill. The stability of the soil mass containing the retaining wall should be checked with respect to the most critical potential failure surface according to the guidelines provided in Chapter 25. A minimum factor of safety of 1.5 should be used when designing for overall stability.

25 Unsupported Excavations

25 Unsupported Excavations

25.1 General

Excavations are frequently left unsupported when they are of limited depth, or when the dimensions of the construction site permit the provision of sufficiently flat slopes to ensure slope stability. Slope failures in unsupported excavations are often catastrophic. Consequently, the design and construction of unsupported excavation should be carried out with great care. Consideration should be given to local safety regulations.

25.2 Excavation in Rock

Because of their high strength, rock masses that are free of continuous joints or fractures are usually suitable for unsupported excavations of moderate height. Exceptions may occur in cases where the lithology and/or the degree of weathering contribute to a reduced compressive or shear strength. Considering only massive rock slopes under 50 m in height, problems are likely when slope height (in m) divided by the rock UCS (in MPa) exceeds a rough limit of 10 (reduces to less than 5 for moderately-weathered / moderately-jointed rockmasses and even less for undrained conditions). For ratios higher than this limit, for slopes greater than 50 m, or for persistently jointed ground, some form of analysis may be necessary. Unfavourably oriented joint patterns may result in the presence of unstable rock wedges along parts of the excavation slopes. For excavations to the depths generally encountered in civil-engineering works, the evaluation of the stability of rock slopes may be limited to an examination of the pattern of joints and other discontinuities of the rock mass, and to consideration of the impact of disturbance and drainage efficiency on the stability of blocks formed by these structures, provided the rock is sound.

Where rock discontinuities are unfavourably oriented, local support must be provided by means of rock bolts or rock anchors, depending on the size of the potentially unstable wedges. Where discontinuities are spaced so closely as to permit the loosening and fall of small blocks, protection of the work area must be provided, for instance, by means of steel wire mats attached to the face of the slope by rock bolts or by the application of sprayed concrete or polymer linings (with appropriate drainage) combined with bolting. It is a common practice to provide for a series of benches in the excavated slope. This is particularly desirable where rockfall hazard is likely and where vertical bench faces are desired to reduce lateral velocity of falling rocks. The vertical spacing of these benches is about 6 m to 10 m; their width and the inclination of the slopes between the benches are determined according to the features of the rock mass and on the method of drilling and blasting. For a detailed treatment on the design and construction of unsupported excavations in sound rock, see Hoek and Bray (1981).

25.3 Excavation in Granular Soil

Unsupported excavation may be performed in granular soil only in situations where the natural groundwater table is deeper than the bottom of the excavation, or where drainage (groundwater control) has been carried out prior to the excavation (see Chapter 22).

In dry granular soil, the slopes of the excavation must be inclined at an angle smaller than the friction angle of the soil. Provision should be made to minimize infiltration of rainwater and to divert surface water run-off away from excavated slopes.

Underwater excavations in granular soil represent a special problem to be investigated by an engineer with experience in this type of work.

25.4 Excavation in Clay

Excavation in clay presents very complex problems and must be approached with great caution. Slope failures may occur from a few days to a few years after the end of the excavation, at times when construction personnel may be at work on the excavation floor, or when the structures have already been erected. In sensitive clays, retrogressive slides may develop following initial local failures, resulting in damage not only to the construction site, but, also, to neighbouring sites or structures. The design of unsupported excavations in clays should be made by specialist geotechnical engineers, who should also inspect the construction work. Some design problems and methods are briefly outlined in the following subsections.

25.4.1 Behaviour of Clays in Excavated Slopes

When overburden stresses are released from a clay mass during an excavation, negative pore pressures are generated. These pore pressures dissipate with time after the end of the excavation to reach steady-state conditions in the long term. As a result, effective stresses decrease, leading to a progressive reduction of the shear strength of the clay. Therefore, a cut slope in clay might be stable for an initial period and then fail. The amount of strength reduction depends on the clay properties, particularly, the overconsolidation ratio. The rate at which the strength reduction occurs depends on the geometry of the problem and on the stiffness and the permeability of the clay. Experience, although limited, shows that about 50 % of the difference between the short- and long-term factors of safety is lost in 20 % of the time required to reach complete pore pressure equilibrium. Detailed discussion is presented by Leroueil (2001).

In all cases, the designer of a cut slope should consider both the short and the long-term stability conditions. Generally, more stringent requirements are applied for the analysis of long-term conditions, as opposed to analysis of the stability of temporary slopes.

The possibility of bottom heave due to excessive pore pressures acting underneath the bottom of the excavation has to be carefully checked. This requires, in particular, a good knowledge of the local stratigraphy and groundwater regime.

25.4.2 Short-Term Stability

The short-term stability of cut slopes in clay is analysed using undrained parameters. Methods selected should be appropriate to the particular problem. The undrained shear strength of clay may be determined from in-situ vane-shear tests. Continued sampling and laboratory testing may also be employed. Although Chapter 4 describes the use of a correction for plasticity of the in-situ vane shear strength, a review of available data (Leroueil et al., 1990; Leroueil, 2001) indicates that such a correction may not be necessary for soft clays. On the other hand, the undrained shear strength measured for stiff and fissured clays generally overestimates the strength that can be mobilized in an in-situ clay mass (Lo, 1970; Chandler, 1984; Leroueil, 2001)

Clay deposits usually present a stiff fissured crust near the ground surface. As a result of the excavation, the upper part of a slope is subjected to horizontal tensile stresses. Therefore, these fissures will open and no strength should be assumed in the fissured crust. Also, unless drainage or protection from water infiltration is provided at the crest of the slope, full hydrostatic pressure may act in the fissures and should be considered.

The stability of cut slopes should be evaluated by means of limit equilibrium analysis or numerical methods. The cut slope inclination should be determined to ensure a minimum factor of safety of 1.5. A lower factor of safety may be acceptable when

- A particularly detailed soil investigation has been carried out ; or
- Where the analysis is supported by well documented local experience; or
- Where geotechnical instrumentation to measure pore pressure and movement is provided and monitored at regular intervals to check the slope behaviour; or
- Where slope failure would have only limited consequence.

25.4.3 Long-Term Stability

Analysis should be carried out considering effective stresses reached after complete pore pressure equilibrium and representative strength parameters. For details, see, for example, Fredlund and Krahn (1977), Skempton (1977), Lefebvre (1981), Tavenas and Leroueil (1981), Chandler (1984) and Leroueil (2001).

25.4.4 Construction Measures

Slopes in clays should be excavated in such a manner as to ensure stability at all stages. Heavy surface loads caused by construction equipment, or fill material, should not be placed near the crest of the slope, unless the stability of the slope is proven acceptable. Care should also be taken so that clay will not be destructured or disturbed by the passage of heavy equipment or excessive vibrations.

Excavated slopes should be protected against erosion, ponding, infiltration of surface water, and frost action. The crest of a slope may be sealed against entrance of water, or alternatively, drainage may be provided. Permanent slopes may be mulched and seeded or sodded to bind the surface of the slope.

Major slopes should be instrumentated to monitor pore pressures and slope movements. The instrumentation may include piezometers, survey points at various locations, and inclinometer tubes.

26 Supported Excavations & Flexible Retaining Structures

26 Supported Excavations & Flexible Retaining Structures

26.1 Introduction

For rigid retaining walls, such as free-standing gravity walls, earth and water pressures can be computed from theory for many real situations. For flexible and semi-flexible walls, such as those commonly used for the temporary support of vertical faces of excavations and for permanent bulkheads, and having a variety of support conditions, no satisfactory general theoretical solutions for earth pressures are available. Design of such systems is, therefore, based on a combination of theoretical methods, empirical methods, and experienced judgment.

There are two basic approaches to design of excavation support and flexible retaining structures:

1. Design for the minimum requirements to satisfy load carrying capacity (those loads that the soil itself does not carry) and system stability; or

2. Design for control of deformations.

In general, design for control of deformations will produce a support system or wall that is stiffer than one designed based on an estimation of the loads imparted on the wall. Temporary excavation support systems designed for control of deformation may be capable of carrying loads equal to or larger than those for which the permanent structure is designed, depending on the choice of walls and design conditions.

Design of supported excavations and flexible retaining structures requires detailed consideration of the following load and stability cases:

Load Considerations
- earth pressures
- water pressures
- surcharge loads from equipment, structures, or spoil piles
- earthquake loading
- loads from frost action
- temperature-induced stresses in structural members
- stresses from swelling ground
- pre-stressing loads
- loads on buried portions of walls
- loads from sloping ground

Stability Considerations
- structural stability of the support system (loading)
- stability of the excavation base related to shear failure in the soil

- stability of the excavation base related to groundwater uplift forces
- deep-seated failure encompassing wall and any ground anchors or rakers
- stability of slopes above the excavation
- stability of any internal berms

Discussion of these loading and stability conditions is provided in this chapter with references made, where appropriate, to the theoretical solutions or guidance provided in Chapter 24. Since the loading conditions and stability considerations are often directly related to the retaining system, this chapter addresses these design issues as related to each different general type of wall system (anchored or braced) with discussions on topics common to all walls in separate sections.

Flexible earth retaining structures can take many forms. Flexible retaining systems are available including walls formed using soil mixing and/or jet grouting, small diameter drilled piles and soil nails. These walls are not considered in detail in this chapter, but the more common wall types are described in Table 26.1

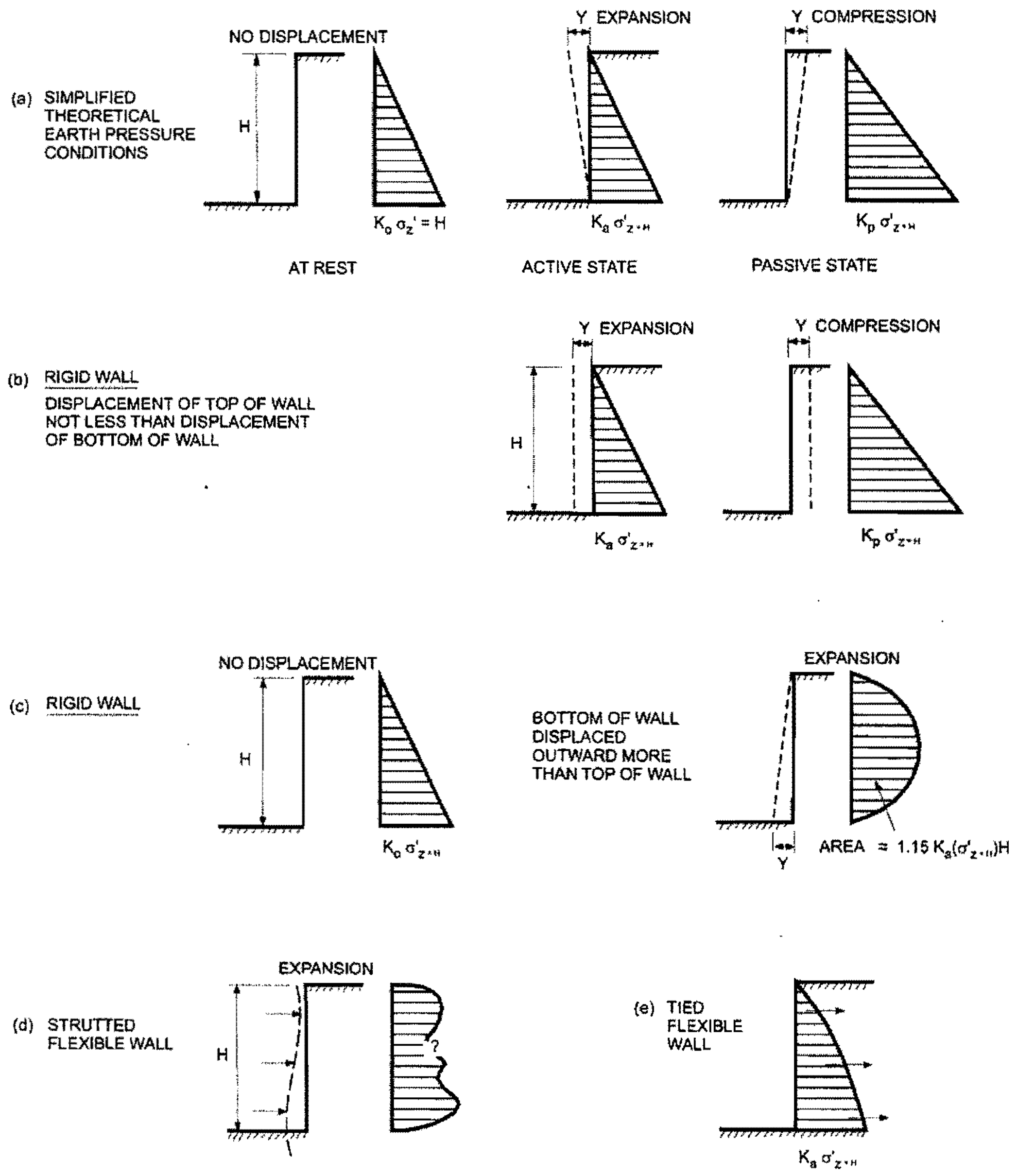
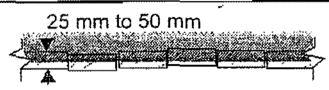
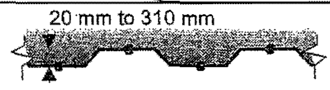
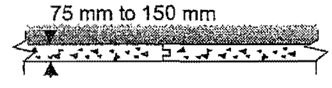
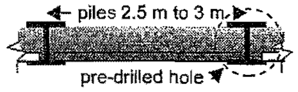
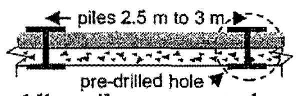
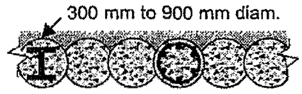
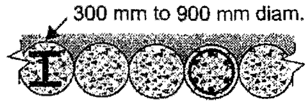
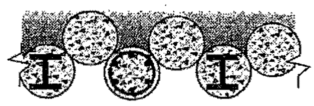
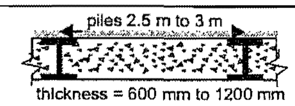
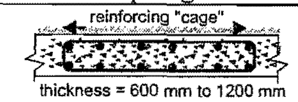


FIGURE 26.1 *Effect of deformation on earth pressures in cohesionless materials*

TABLE 26.1 *General characteristics of various flexible retaining structure types (modified after ASCE 1997).*

SYSTEM	RANGE OF STIFFNESS (EI MPa/m)	DEPTH RANGE (m)	ADVANTAGES	DISADVANTAGES
 wood sheeting	15 to 160	1.5 to 5	low cost, ease of installation in good ground, conventional equipment, simple skills	discontinuous, low strength, limited soil conditions, limited depth
 steel sheet piles	450 to 7850	5 to 21	continuous, high strength, readily available, effective in soft ground	limited by soil conditions, hampered by obstructions
 pre-cast concrete panels	230 to 1550	3 to 10	durable, cost effective, assists in minimizing seepage	limited availability, can be damaged during driving, limited depth, hampered by obstructions
 soldier-piles, wood lagging	450 to 6250	5 to 25	ease of installation in good ground, readily available, adaptable to poor ground, low cost	loss of ground common, pre-drilling may be required in hard ground
 soldier-piles, concrete lagging	450 to 6250	5 to 25	ease of installation in good ground, readily available, adaptable to poor ground, low cost, stiffer than wood lagging	loss of ground common, pre-drilling may be required in hard ground
 tangent piles	11,000 to 94,000	10 to 25	readily available, adaptable to poor ground, stiffness can be large, can assist in limiting seepage	poor alignment tolerance at depth limit, gaps may allow ground loss and seepage
 secant piles	12,500 to 110,000	10 to 18	readily available, adaptable to poor ground, improved stiffness and groundwater control by interlock	poor alignment tolerance at depth limit, gaps may allow ground loss and seepage, concrete strength critical for pile overlaps
 staggered drilled piles	15,000 to 130,000	10 to 18	readily available, adaptable to poor ground, stiffness can be large, improved stiffness	poor tolerance for alignment at limit of depth range, gaps may allow ground loss and seepage intermediate piles
 concrete diaphragm	55,000 to 250,000	10 to 30	high strength, durable, can be integrated into permanent structures, good tolerances	high cost, special equipment and field control required
 concrete diaphragm	47,000 to 160,000	10 to 30	high strength, durable, can be integrated into permanent structures, good tolerances	high cost, special equipment and field control required

26.2 Earth Pressures and Deformation

The earth pressure acting on an earth-supporting structure depends strongly on the lateral deformations of the soil (Figure 26.1). Consequently, unless the deformation conditions can be estimated with reasonable accuracy, no single theoretical 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 smaller than the displacement of the bottom of the wall. As shown in Figure 26.1b, the pressure distribution remains close to a triangular form and ranges between the failure limits of the 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 outwards farther than the top of the wall, a parabolic pressure distribution results, as shown in Figure 24.1c. The corresponding force on the wall for this condition is generally about 10 % to 15 % greater than the force under active failure conditions.

For flexible walls, the deformation and, hence, the earth pressure are much more complex. The yield of one part of a flexible wall redistributes pressure onto the more rigid parts due to the internal shear strength of the soil – a process often called "arching." Therefore, the pressures in the vicinity of supports are higher than in unsupported areas, and the loads on or between individual supports vary, depending largely on the stiffness characteristics of the various wall components themselves (e.g. piles, struts, anchors, lagging, etc.). The earth pressure distribution on flexible walls and their final deformation also depend on deformation that occurs below the base of the cut, before the installation of supports. They also depend on details in the construction technique and procedure. The final average deflection condition can vary widely depending on ground conditions, support locations, support flexibility, and construction methods.

The deflection characteristics and pressure distribution differ for anchors and strutted walls. Once installed and stressed, struts can be considered to be nearly fixed-deflection supports. Anchors, on the other hand, can be considered to provide nearly constant-load supports, and anchored walls come nearer than strutted walls to having triangular pressure distributions when the anchors are not heavily pre-stressed to a pre-determined design load. However, in the calculations for anchored walls it may be desirable to assume a trapezoidal or rectangular distribution to ensure more positive support of adjacent footing or buried services (i.e. the alternative distribution can result in higher design loads and a stiffer wall system near the top).

Pre-stressing of anchors or struts tends to reduce wall movements, although the reduction of the lateral movement may be small in some soils (see Clough and Tsui 1974, Mana and Clough 1981, and Boone et al. 1999). In general, when the largest component of deformation occurs below the base of the excavation, pre-stressing is less effective than in more competent soils.

Since pressure distributions on flexible walls are not uniform and depend on deformations, a number of "load reduction factors" have been applied to flexible wall design to account for the "arching" effect induced by the ground's internal strength. Reduction factors must be used with caution because they were developed to address particular wall types and design or construction cases. Further guidance on the use of reduction factors is provided within this chapter as appropriate to design of each individual wall type.

26.3 Earth Pressures and Time

When calculating earth pressures, assumptions will be made regarding whether or not the soil behaves in an "undrained" or "drained" mode of failure. The choice of failure mode can affect the outcome of the design. The use of the undrained strength parameters is generally associated with "short-term" construction. Although many excavation support systems are temporary structures, the length of time that these systems remain in service depends on the structure being built and the particular project schedule. Likewise, whether a soil exhibits truly undrained behaviour depends on its particular properties. The balance between pore-water pressure dissipation and the anticipated construction schedule must be considered carefully when making the choice regarding whether to evaluate any particular excavation support system utilizing total stress (undrained) or effective stress (drained) geotechnical analyses, as the first condition (undrained) will transition to the later condition (drained) given sufficient time.

26.4 Effects of Seepage and Drainage

Groundwater pressures estimated in design should be consistent with the required or permissible drawdown levels in the groundwater around the wall. Where soldier piles with wood lagging are used, groundwater is normally assumed to be at, or below, the base of the interior of the excavation; however, where dewatering or drainage is not complete, additional groundwater pressures may exist and these should be evaluated through flow nets or computer analyses. When the wall is intended to prevent all leakage of groundwater, maximum exterior groundwater pressures should be used.

The effect of the greatest unbalanced water pressure that will act across the wall must be included in the pressure computations (see Figure 26.2). For instance, in cohesionless soils, the active force on a wall with static water level at the top of the backfill is more than double that for dry backfill against the same wall. To obtain the total pressure, or thrust, on the wall, the net water pressure must be added. Net water pressures against a bulkhead wall are illustrated by Figure 26.2a where no seepage occurs beneath the wall. If there is seepage below the wall, the pore pressure must be calculated considering the seepage gradients. Figure 26.2b illustrates seepage around a bulkhead in uniform soil where the active and passive earth pressures are reduced as a result of the seepage pressures. The use of flow nets or computer analyses should be used to assess potential pore-water pressure conditions and their effect on active and passive pressures for all conditions in cases of seepage around retaining structures.

Where the phreatic surface of a pore water pressure profile intersects the potential failure plane, such pore water pressures must be taken into account. Such pore water pressures are not accounted for in the calculation of the active and passive earth pressure coefficients. In general, a graphical and iterative approach to finding the resultant earth loads should be performed taking into account the pore water pressures derived from flow net analyses where the critical failure plane crosses the phreatic surface. A wedge analysis utilizing flow nets for determination of water pressures is illustrated in Chapter 24. For flexible wall design, however, alternative analyses may be required. Guidance on the use of generalized limit equilibrium approaches ("method of slices") to evaluate active and passive earth pressures is provided in Noshin et al. (1999).

26.5 Surcharge Pressures

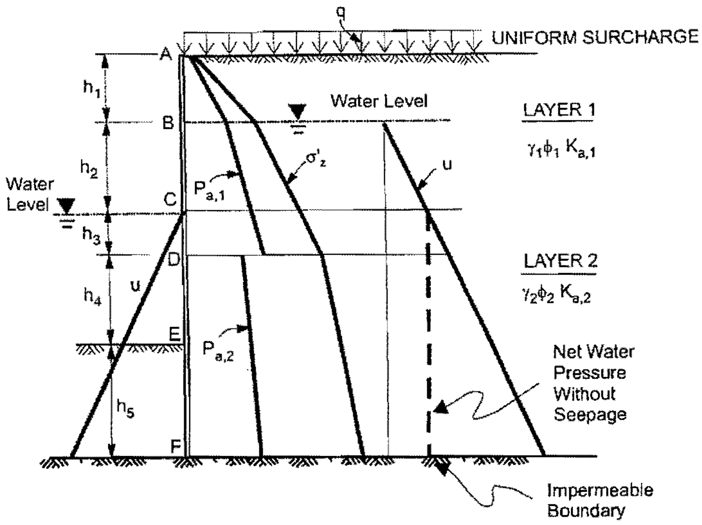
As with lateral earth pressures, the distribution of surcharge loads may also be affected by wall deformation. It is customary, however, to apply the theoretical surcharge pressures according to the guidance provided in Chapter 24.

26.6 Frost Pressures

In the winter, freezing of the soil behind the wall may impose forces on the flexible wall, particularly if the freezing alternates with thawing periods. Frost pressures are not usually considered for design of flexible retaining structures. Design is generally focused on limiting the potential freezing of soils adjacent to the walls. Under such conditions, draining and moisture control of the soils behind the wall is important. Potential frost pressures, however, can be an important consideration for flexible walls installed through frost-susceptible soils that permit migration of freezing temperatures through their structure. Drainage and control of moisture often cannot readily be controlled for flexible walls installed in-situ (e.g. sheetpile, tangent pile, or concrete diaphragm walls). In such cases, insulation or other frost protection or temperature control measures may be necessary (Chapter 13 provides further discussion regarding frost effects).

26.7 Swelling/Expansion Pressures

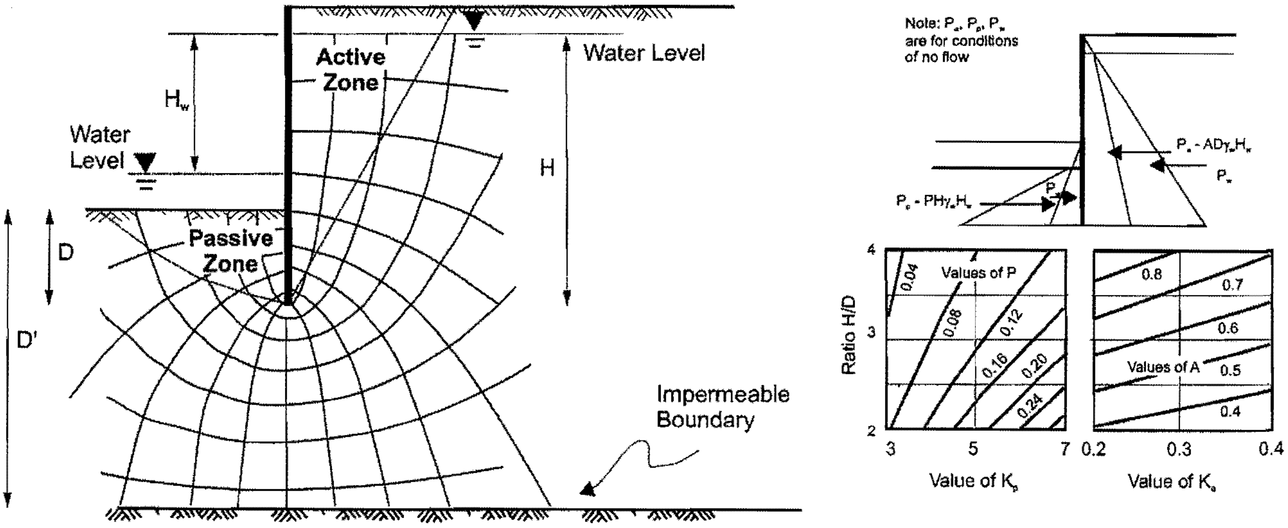
Swelling or expansion of clay soils can produce loads well in excess of the active earth loads. Walls can be designed to either withstand swelling pressures using laboratory tests to estimate the design pressures. Alternatively, ground improvement or isolation of expansive soils from moisture can be carried out. Further guidance on swelling soils is provided in Chapter 15.



RIGHT HAND SIDE

ELEV.	γ	$q + \Sigma\gamma h = \sigma_z$	u	σ'_z	K_a	P_a
A	γ_1	$q + 0\,\gamma_1 = \sigma_A$	0	σ'_A	$K_{a,1}$	$K_{a,1}\sigma'_A$
B		$\sigma_A + h_1\gamma_1 = \sigma_B$	0	σ'_B		$K_{a,1}\sigma'_B$
C		$\sigma_B + h_2\gamma_1 = \sigma_C$	u_C	σ'_C		$K_{a,1}\sigma'_C$
D		$\sigma_C + h_3\gamma_1 = \sigma_D$	u_D	σ'_D		$K_{a,1}\sigma'_D$
D	γ_2	σ_D	u_D	σ'_D	$K_{a,2}$	$K_{a,2}\sigma'_D$
F		$\sigma_D+(h_4+h_5)\,\gamma_2 = \sigma_F$	u_F	σ'_F		$K_{a,2}\sigma'_F$

a) example bulkhead in two soil layers without seepage



b) example bulkhead in uniform soil conditions with seepage (after NAVFAC, 1988)

FIGURE 26.2 *Effects of water and seepage on flexible wall pressures, example bulkhead wall without and with seepage conditions*

26.8 Cantilevered (Unbraced) Walls

26.8.1 Cantilevered Walls – Loading Conditions

Cantilevered walls (Figure 26.3) are sometimes used to support excavations, cuts, or fills up to about 5 m in height. They are generally considered to act as rigid or semi-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 the active and passive failure conditions. Since the wall is assumed to rotate about some point above its base, the dominant mode of loading will change along the embedded portion of the wall as illustrated in Figure 26.4.

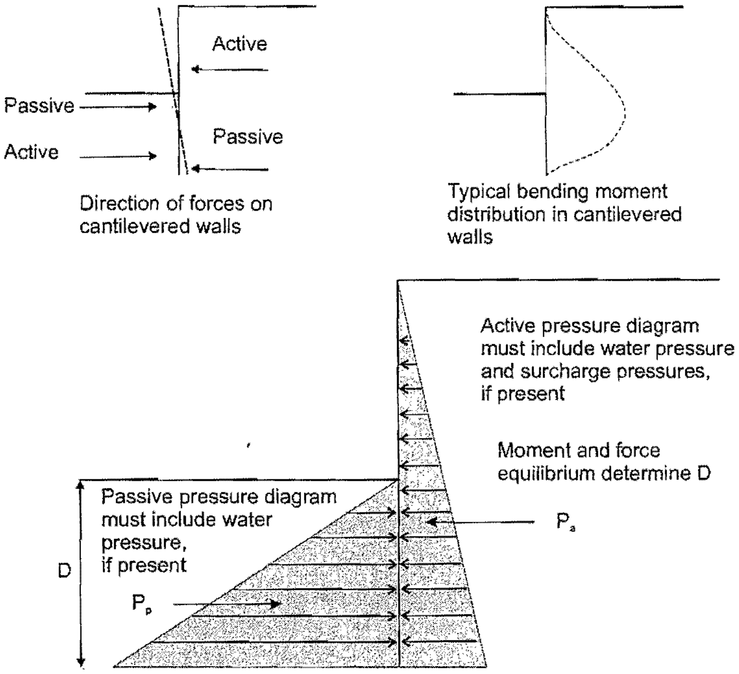


FIGURE 26.3 *General loading conditions for cantilever walls*

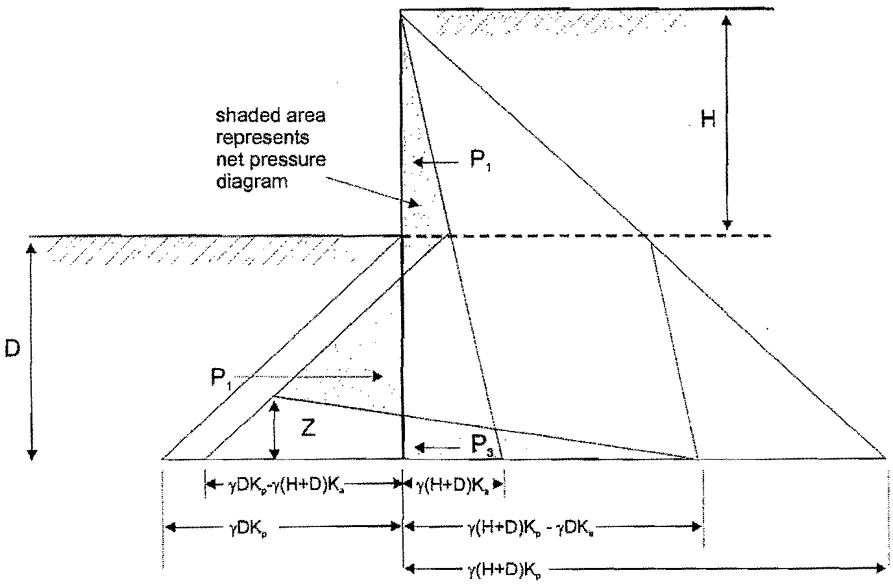


FIGURE 26.4 *Calculation of penetration depth and bending moment for cantilever wall in uniform soil (general case)*

Cantilevered walls may be used for permanent support of granular soils and low-compressibility clay soils. For permanent support in these soils, the walls should be analysed on the basis of effective stresses, using 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 and the computed earth pressures may be negative, see Chapter 24. Such negative pressures should not be assumed to contribute to support.

Where water occurs behind the wall, the relevant hydrostatic or seepage pressures must be added to earth pressures in all effective stress analyses.

26.8.2 Cantilevered Walls – Determination of Penetration Depth

Calculation of the vertical penetration for a cantilever retaining structure is generally an iterative process. Below the excavation or dredge level, the wall will rotate about a point and derive its lateral support through both active and passive earth pressures as shown on Figure 26.3. Using Figure 26.4:

1. Assume a trial depth of penetration D estimated using the empirical estimates provided in Table 26.2.
2. Calculate active, passive, surcharge and water pressures to develop an appropriate pressure distribution diagram following the guidance of Figure 26.4.
3. Satisfy requirements of static equilibrium where the sum of all horizontal forces must equal zero and the sum of all moments about any point must equal zero. By trial and error determine the distance z. For a uniform dry granular soil, this may be found by:

$$z = \frac{K_p D^2 - K_a (H+D)^2}{(K_p - K_a)(H+2D)} \tag{26.1}$$

   If the sum of the moments about the bottom of the wall is not zero, increase or decrease D until the sum of the moments is zero.
4. Increase D by 20 % to 40 % to result in an appropriate factor of safety between 1.5 and 2.0.

TABLE 26.2 *Empirical Estimate of Penetration Depth for Cantilever Walls*

Standard Penetration Resistance, N (blows/0.3 m)	Depth of Penetration
0 to 4	2.0H
5 to 10	1.5H
11 to 30	1.25H
31 to 50	1.0H
> 50	0.75H

26.8.3 Cantilevered Walls – Determination of Structural Design Bending Moments

Structural design of the vertical members of a cantilever wall should be completed after developing a balanced earth pressure distribution. The maximum design moment should be calculated at the point of zero shear. Load reduction factors should not be applied to design of vertical members for cantilever retaining structures.

26.9 Single-Anchor and Single-Raker Retaining Structures

26.9.1 Loading Conditions

Some excavations and bulkheads are supported by flexible wall systems constructed using only one level of horizontal support. For flexible walls a single support level can be provided by soil anchors, or ties to "deadmen", or raking braces.

The pressure distribution on anchored walls is generally considered to be triangular in form consistent with the theoretical distributions provided in Chapter 24, while some account is made of stress redistribution arising from the support conditions as discussed below. For all soil, it is generally preferable that pressures be computed on the basis of effective stresses, using the effective angle of shearing resistance and neglecting effective cohesion. Where water seepage exists or could occur, behind the wall, the relevant water pressures and their effect on active and passive earth pressures must be considered in all effective stress analyses.

26.9.2 Penetration Depth and Structural Bending Moments

The pressure distribution on flexible walls with large unsupported spans, such as in flexible bulkheads, differs from both cantilever walls and walls with multiple support levels and is discussed in detail by Rowe (1952 and 1957) and Bjerrum et al. (1972).

Two different methods can be used for design of single-anchor and single-raker wall systems. These methods are commonly referred to as the "free-earth" and "fixed-earth" methods:

1. The "free-earth" approach assumes that the wall acts as a beam spanning two supports, these being the top anchorage and the passive pressure of the earth below the excavation or dredge line (i.e. the wall is free to rotate or translate horizontally at its bottom end);
2. The "fixed-earth" approach assumes that the wall extends sufficiently into the ground to develop fixity at some point below the excavation or dredge line and the wall cannot rotate or translate at this point.

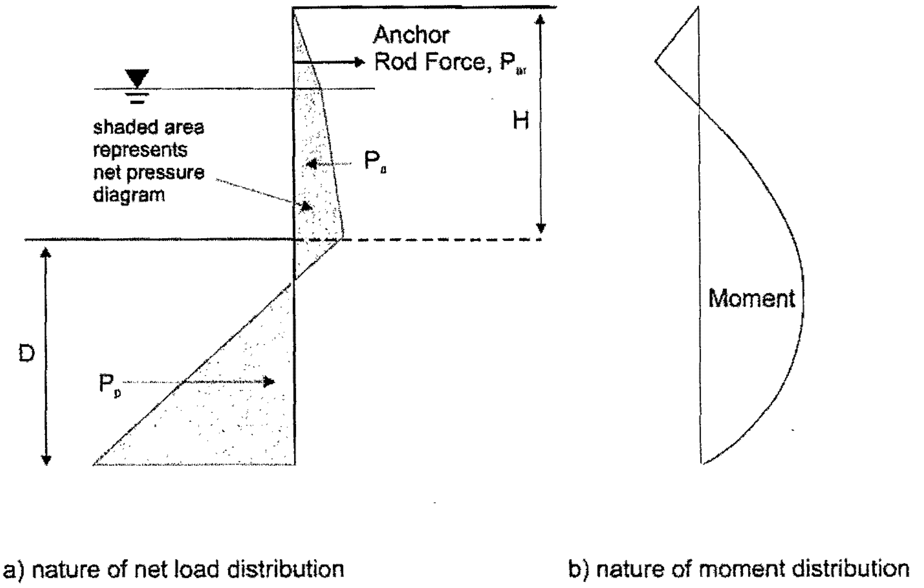


FIGURE 26.5 *Free-earth method for design of retaining structure supported by single anchor (general case)*

The "fixed-earth" support method can sometimes produce more economical designs because bending moments are generally lower and the resulting required section modulus of the steel is less even though the depth of penetration is generally greater. However, in soft or loose ground conditions, such generalities cannot be relied upon. Design methodology for the general case is provided in Figure 26.5 in which the "free-earth" support method is used whereby:

1. Compute active, passive, surcharge and water pressures using the methods provided in Chapter 24.
2. Calculate the net pressure diagram.
3. Satisfy moment equilibrium to solve for D taking moments about the support point.
4. Calculate the anchor pull, P_a, by satisfying horizontal force equilibrium.
5. Determine the maximum moment utilizing the "free-earth" assumption where the wall acts as a beam spanning two supports, these being the top support and the passive pressure of the earth below the excavation or dredge line (i.e. the wall is free to rotate at its bottom end), see Figure 26.5.
6. Apply moment reduction factors provided in Figure 26.6 to determine the design moment. Moment reduction factors should not be used for loose granular or soft to medium stiff cohesive soils.
7. Increase the depth of penetration by 30 % to 40 % to provide an adequate factor of safety of 2 or more.
8. Check factor of safety (FS) by calculating the ratios of the resisting and acting moments, and resisting and acting horizontal forces, i.e.:

   FS (moment equilibrium) = $\Sigma M_{passive} / \Sigma M_{active}$
   FS (horizontal force equilibrium) = $\Sigma P_{passive} / \Sigma P_{active}$

9. Utilize design moment and methods provided for in the Canadian Handbook of Steel Construction to determine appropriate sheeting section.

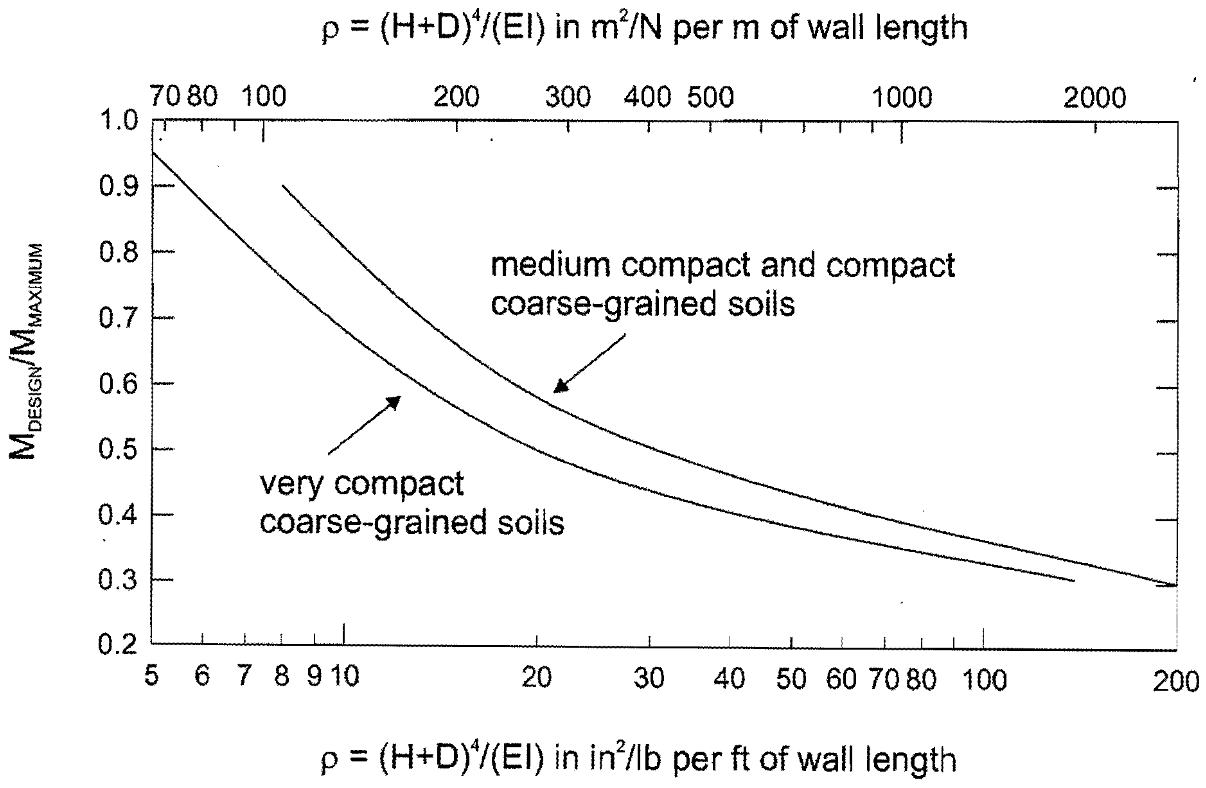


FIGURE 26.6 *Bending moment reduction factor for retaining structures designed using the free earth method*

For bulkhead structures that can tolerate deformation of the sheeting (on the order of 0.5 % of the wall height or more), the bending moments should be derived as described above using Figure 26.5 and Figure 26.6. The use of moment reduction factors illustrated in Figure 26.6 is restricted to design moments developed using the "free-earth" design method as discussed above. Movement of the tie-rods, deadmen, rakers, or anchors can, however, reduce the beneficial effects of soil arching and load redistribution. Therefore, these factors should only be used with caution where deformation can be tolerated.

26.10 Multiple-Anchor, Multiple-Raker and Internally Braced (Strutted) Retaining Structures

26.10.1 Loading Conditions

Since ground anchors approximate constant-load supports, rather than nearly fixed-deformation supports, the design of walls supported by multiple anchors can be carried out using either triangular or apparent earth pressure diagrams (See Section 26.10.3 through 26.10.7), depending on permissible deformations. In general, the use of triangular earth pressure diagrams and anchors stressed to match such loads will result in a measured load distribution that is close to the design distribution by virtue of elastic elongation of the anchors and development of active pressures. However, if it is desired to control movements near the ground surface, the apparent earth pressure diagrams as described below for braced structures should be used such that the higher anchors are further restrained against movement as a result of their higher design loads.

For walls designed using multiple anchors and a triangular earth pressure distribution, the individual anchor loads can be solved through calculation of horizontal force equilibrium. Walls supported with multiple raker levels typically experience larger deformations at their top than at their bottom. Therefore, it is common to utilize conventional triangular distributions of earth pressures according to the methods provided in Chapter 24. Terminology for the method outlined below is provided in Figure 26.7. Note that all horizontal loads should be applied including those from active and passive earth pressures, surcharges, unbalanced water pressures, seepage pressures, and seismic loads as appropriate. To calculate individual anchor/raker loads, assume triangular active and passive earth pressure distributions and:

1. Assume that the highest load on the nth level anchor/raker occurs just before placing the next anchor, and draw the excavation cross section for that condition (Figure 26.7).
2. For the first anchor/raker level, calculate the depth of penetration to result in moment equilibrium taken about the first anchor/raker level, and the first anchor/raker load will be equal to the load required for horizontal force equilibrium.
3. For all anchors/rakers other than the lowest, determine the depth of penetration of the wall required to establish a factor of safety of 1.0 against rotation about the wall top, using the pressure diagrams previously established, and taking into account the design forces in previously installed anchors;
4. Determine the required force in the n^{th} anchor/raker for stability of the wall, based on equilibrium of all horizontal forces;
5. For the next to lowest anchor/raker, check that the intermediate depth of penetration, as indicated by the analysis described in 26.11.3 is adequate to allow safe excavation to the lowest anchor level;
6. For the lowest anchor/raker, take the depth of penetration at the proposed design value and calculate the anchor force from horizontal force equilibrium; and
7. In general, where the lowest anchor/raker is more than 1m 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/raker may increase as a result of stress redistribution.

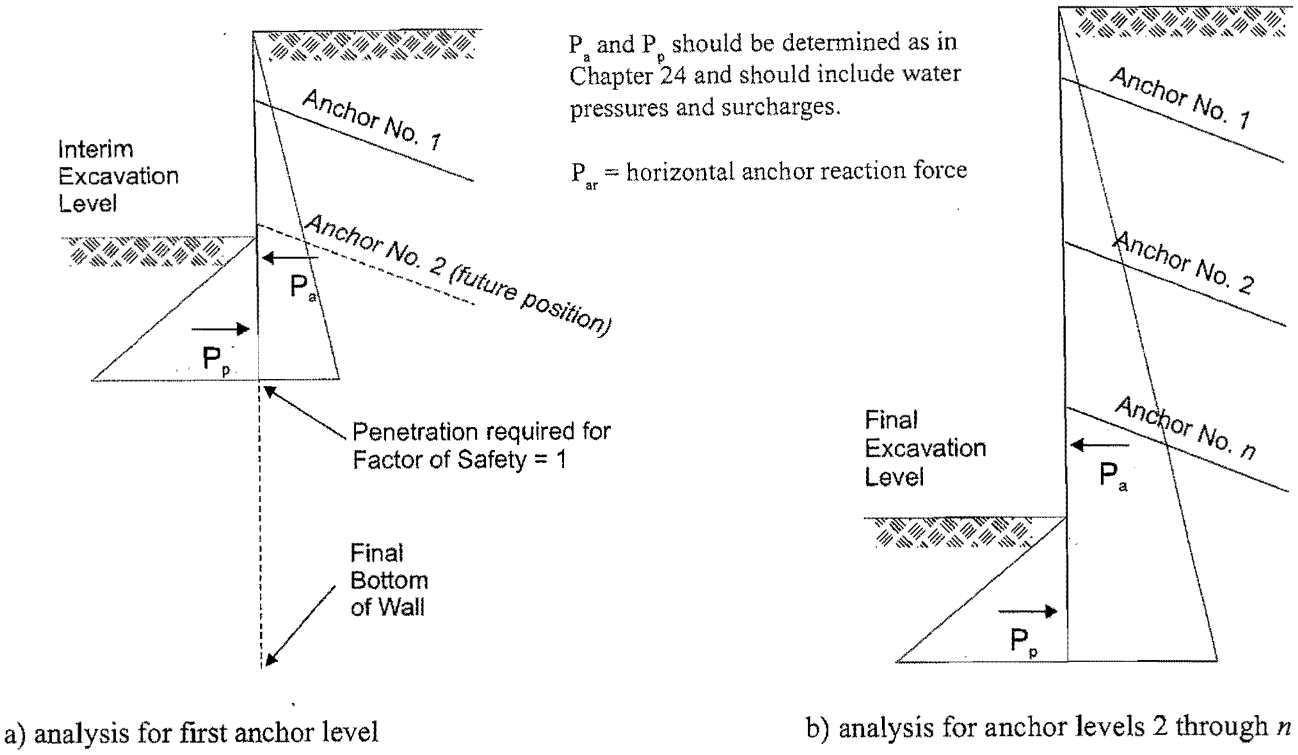


FIGURE 26.7 *Calculation of anchor forces and conditions for multiple-anchor retaining systems*

The bending moments that will develop in the wall at each stage of construction should be checked. Critical conditions will occur immediately before each anchor is installed.

26.10.2 Effect of Anchor Inclination

Anchors are usually inclined downward 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. Downward movement induced by inclined anchor loads is resisted by friction/adhesion along the wall and reaction at the base or toe of the vertical member. With soldier pile and lagging systems, the available shaft resistance is reduced during the excavation process; additional toe capacity may be required to limit vertical deformations. Although shaft resistance is developed, particularly where shoring is supported by anchors, the magnitude is unpredictable because of the softening or weakening imposed by freeze-thaw cycles or lubrication of the soldier pile hole by wet silt or sand prior to the installation of concrete backfill.

A conservative approach to retaining structure design is to ignore friction or adhesion along the back of the wall. Such vertical forces must therefore be supported in bearing at the toe of the support system. The toe capacity of the wall must be checked, or unacceptable vertical and horizontal deformations may take place. Toe capacity of excavation support systems composed of steel sheet pile walls and inclined anchors can be a critical factor in design as the end area of the sheeting is typically not great with respect to the vertical component of anchor loads. Careful consideration should be given to toe capacity, friction/adhesion along the back of the wall, and anchor inclination for such walls founded in loose granular or soft to medium stiff cohesive soils.

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

26.10.3 Braced Retaining Structures – Loading Conditions

The distribution of stress against the walls of braced excavations cannot be adequately predicted from theory. Field measurements have shown that the actual stress distribution varies from section to section, depending on many construction variables. Based on measured strut loads, envelopes of maximum design stresses were developed for different soil types by distributing the measured strut loads over a "tributary area". Such design load envelopes developed from empirical load data are referred to as "apparent" earth pressures since their magnitude is deduced from point loads distributed over an assumed design area (Figure 26.8).

Apparent earth pressures incorporate many factors including construction staging, soil type, and some implicit effects of temperature. Excavation support designs based on such apparent earth pressure distributions have been generally shown to be safe; however, since many factors affect the safety of excavations, each factor must be considered according to its own merit.

Because movements are restricted to varying degrees and the excavation and support process is sequential, fully active conditions may not develop for every stage of the excavation. Therefore, it is typical that the shoring experiences lateral earth pressures that exceed the theoretical active earth pressures calculated according to the methods provided in Chapter 24.

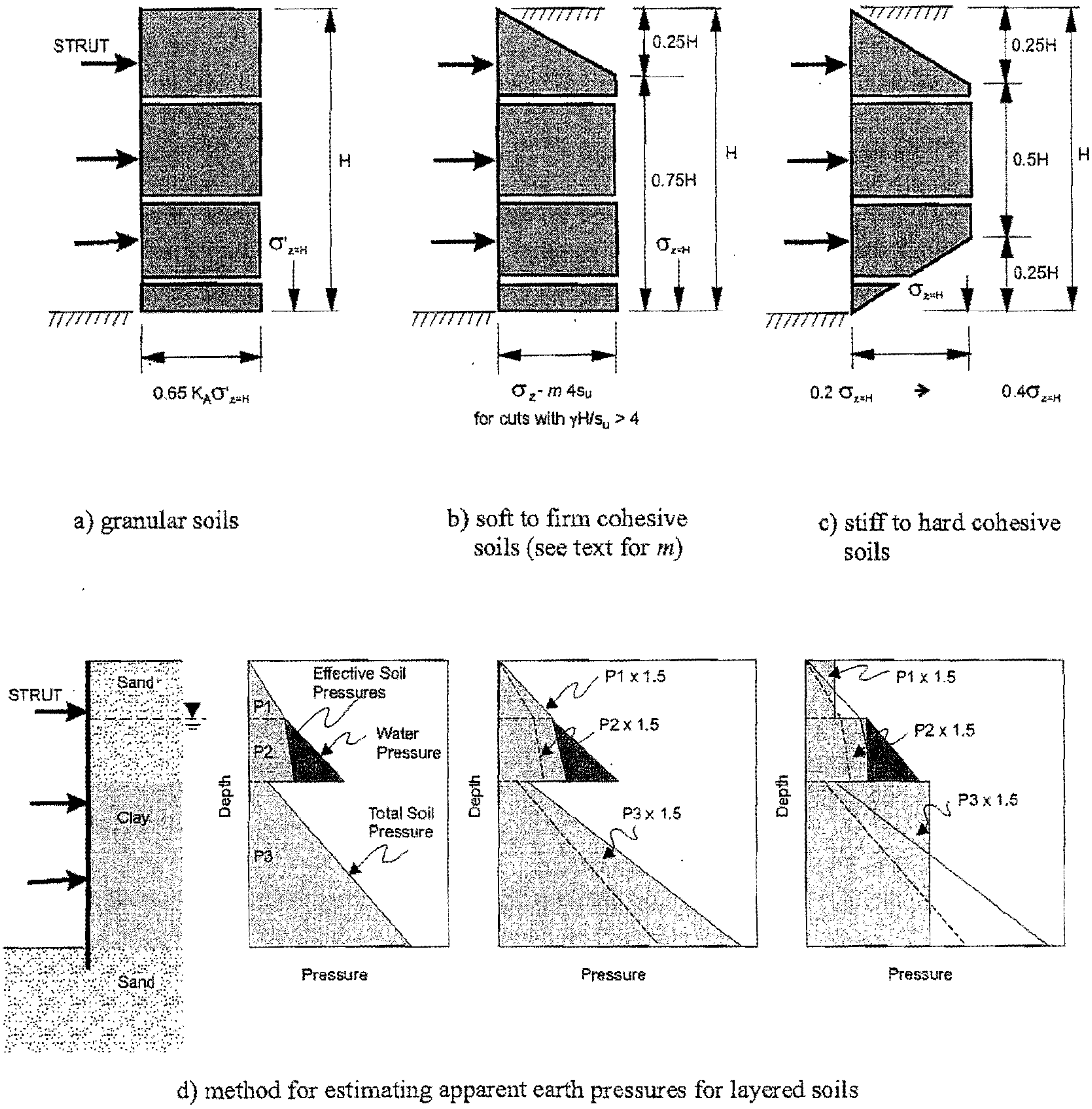


FIGURE 26.8 *Apparent earth pressure distributions*

Strut loads are determined by:

1. Choosing the appropriate apparent earth pressure diagram (Figure 26.8);
2. Calculating the tributary area as the cumulative height halfway to the struts above and below multiplied by the horizontal distance halfway to each of the neighbouring struts on the same level; and
3. Applying the apparent earth pressure diagram specific to the tributary height to the tributary area to develop a design strut load.

26.10.4 Coarse-Grained Soils

For cohesionless soils, a possible distribution of the earth-pressure coefficient to be used in design is shown in Figure 26.8a. The area of this rectangular pressure distribution envelope produces a lateral thrust about 30 % greater than the Rankine active value. Hydrostatic or seepage pressures must be added to the apparent earth pressure where groundwater is not fully lowered by dewatering or drainage to below the excavation level.

26.10.5 Soft to Firm Clays

For soft to firm saturated clays, the pressure envelopes to be used are given in Figure 26.8b. The diagram postulates total stress design, i.e., water pressure is included. However, an apparent earth pressure smaller than the hydrostatic water pressure should not be used. Where a great depth of soft clay exists below the excavation, the value of the apparent earth pressure at the base should be modified by using $m = 0.4$. Where a more resistant layer exists at or near the base of the excavation, use $m = 1.0$.

26.10.6 Stiff to Hard Clays

For stiff clays, the pressure envelope is shown in Figure 26.8c. The indicated variation in the value of maximum stress level depends on the character of the clay, the degree of jointing or fissuring, and the reduction in strength of the clay with time. The choice can only be made on the basis of experience and detailed knowledge of the clay deposit and may lie outside the given range. The apparent earth pressure is given as related to the total stress analysis. Therefore, if there is a risk of fissures in the clay becoming filled with water, hydrostatic water pressure must be added to the earth pressure determined according to the diagram.

26.10.7 Layered Strata

The apparent earth pressure diagrams represent simplified ground conditions and are not readily adapted to complex stratigraphy or groundwater conditions. Many approximations have been suggested to address such situations. Theoretical earth pressure (utilizing the Rankine or Coulomb approaches) can be utilized provided that modifications to the final pressure distribution are made. Because the theoretical earth pressure calculations assume full mobilization of the soil strength it is customary to multiply the resultant theoretical earth pressures by a factor ranging between 1.1 and 1.5 for the design of strutted systems. Such factors are applied to provide some account of field installation and base stability effects. It is also customary to redistribute the calculated pressures from the primarily triangular distributions to rectangular distributions. In general, approximations of apparent earth pressure distribution for layered strata can be developed as follows:

1. Calculate the active earth pressures for each strata;
2. Calculate the total load of the active earth pressure diagram for each strata (area under the triangular distribution);
3. Multiply the total earth load of each individual strata by a factor of 1.1 to 1.5 to account for partial mobilization of soil strength (water and surcharge pressures remain the same); and
4. Redistribute the total earth load in each strata in a rectangular distribution (water and surcharge pressures remain the same).

An example of this approach is provided in Figure 26.8d.

26.11 Stability of Flexible Retaining Systems

26.11.1 Excavation Base Stability

The base of a supported excavation can fail in three general modes including:

1. Shear failure within the ground from inadequate resistance of the loads imposed by the differences in grades inside and outside the excavation;
2. Piping or quick conditions from water seepage through granular soils at the excavation bottom; and
3. Heave of layered soils due to water pressures confined by intervening low permeability soils.

The first mechanism primarily occurs in soft to medium-stiff clays. However, depending on the excavation depth, the overburden pressure may be so great as to exceed the strength of stiff clays, and therefore the terms "soft" to "medium-stiff" must be considered relative. Shear failure of excavation bases is generally rare in granular soils and if adequate lateral support is provided, the factor of safety against bottom instability is typically satisfactory. Inadequate dewatering or pressure-relief of groundwater can, however, cause instability in excavations made through granular or layered soils.

26.11.1.1 Cohesive Soils

Deep excavations in soft to firm clays are subject to base heave failures which result from overstressing the soil in shear (Figure 26.9). The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z \tag{26.2}$$

where

N_b = stability factor dependent upon the geometry of the excavation and given in Figure 26.9.

s_u = undrained shear strength of the soil below the base level (corrected for plasticity, test method, and anisotropy as appropriate).

σ_z = total overburden and surcharge pressures at the bottom of the excavation.

As the potential for bottom instability increases, the heave in the base of the excavation and movement surrounding the excavation increase or FS_b decreases. In the case of soft clays underlying the base of an excavation where FS_b is smaller than 2, substantial deformations may occur. Where FS_b is less than 1.5, the depth of penetration of the support system must extend below the base of the excavation. The force on the buried section of wall, P_H, can be calculated according to Figure 26.9 and Section 26.11.3.

26.11.1.2 Granular 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 or 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.

26.11.1.3 Layered Soils

Layered soils present complex problems for excavation support. If the layered soils consist of soft to medium-stiff cohesive soils and loose sand, the factor of safety against base stability should be calculated using the more critical undrained shear strength approach using the strength data and methodology for cohesive soils as discussed above. When the layered soils at and below the base of the excavation include granular soils under water pressure, the factor of safety against base instability for "blow-out" should be calculated though simple vertical force equilibrium considering all cohesive layers as potential barriers to upward flow of water. In these cases, the saturated unit weight of the soil layers that are cut-off from groundwater recharge (by the wall penetration) above the excavation base must off-set the vertical upward seepage or bouyant forces. A minimum factor of safety of 1.2 should be satisfied.

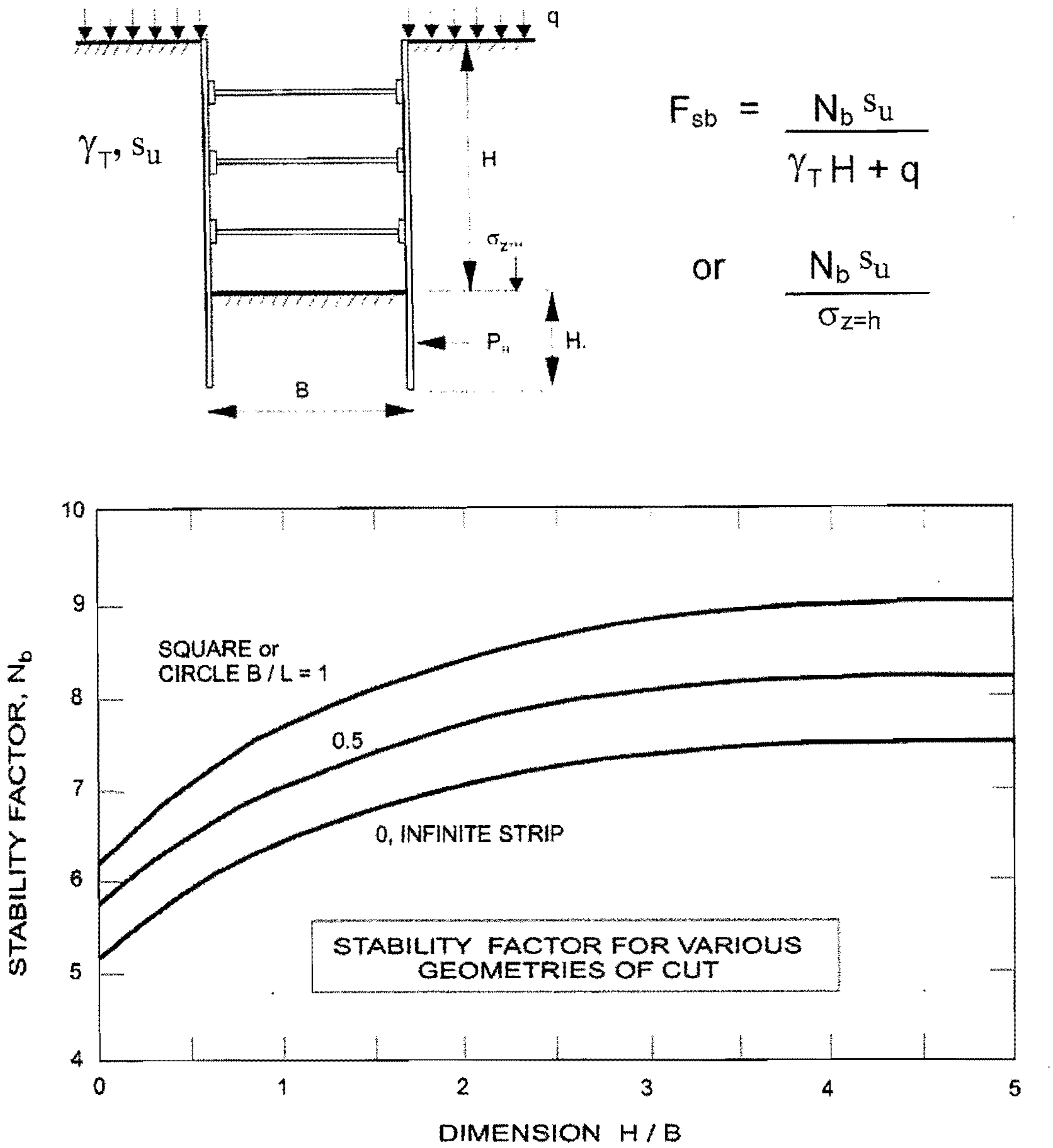


FIGURE 26.9 *Factor of safety with respect to base failure in cohesive soils (after Janbu 1954)*

26.11.2 Overall Stability of Anchored Systems

Even though appropriate retaining system pressures and anchor design criteria can be satisfied, an excavation support system or retaining structure supported by anchors can fail if the entire block encompassing all wall components is not stable. The overall stability of the anchor system is checked by analysing the stability of the block of soil lying between the wall and the mid-point of the anchors.

26.11.2.1 Single-Level Anchor Systems

The anchoring boring mass equilibrium, considering that the loads from the active masses (bounded by BFH and CDI) are evaluated separately. Figure 26.10 illustrates the approach for uniform ground. Figure 26.11 illustrates stability calculation of an anchor through a two zone soil mass.

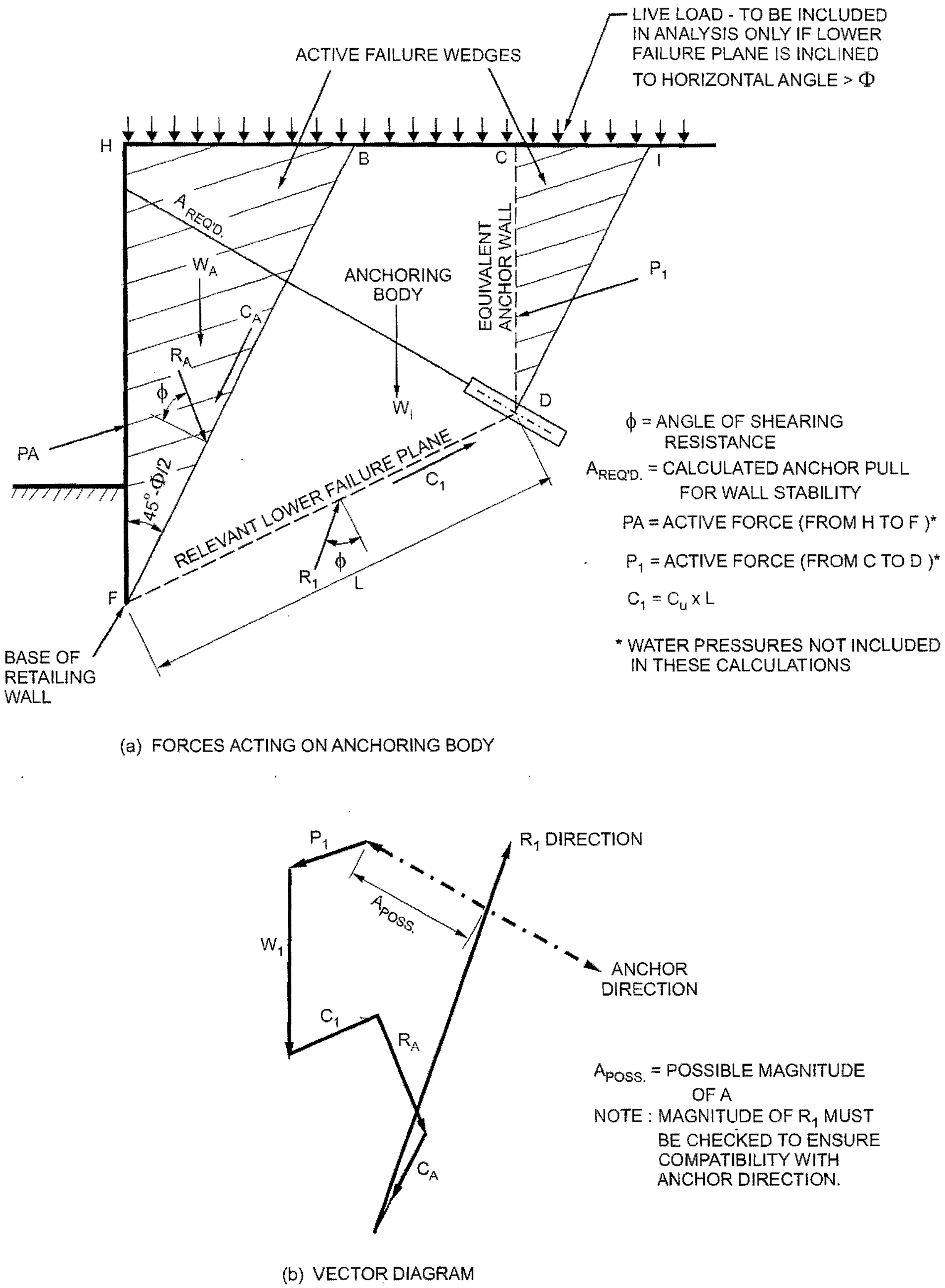


FIGURE 26.10 *Graphic analysis of anchored wall in uniform soil*

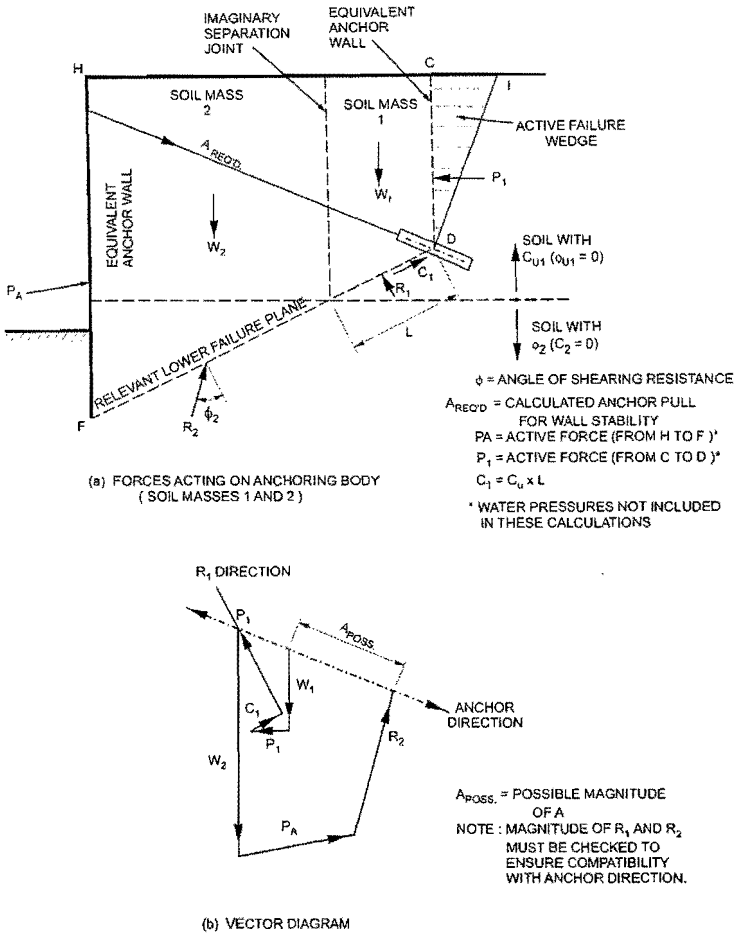


FIGURE 26.11 *Graphic analysis of anchored wall in two soil layers*

26.11.2.2 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 typical possible cases for the location of anchors with respect to the base of the retaining wall are shown in Figure 26.12. The failure planes requiring stability analysis are indicated in each case. The method of analysis for each anchoring body is the same as that indicated for the single anchor system.

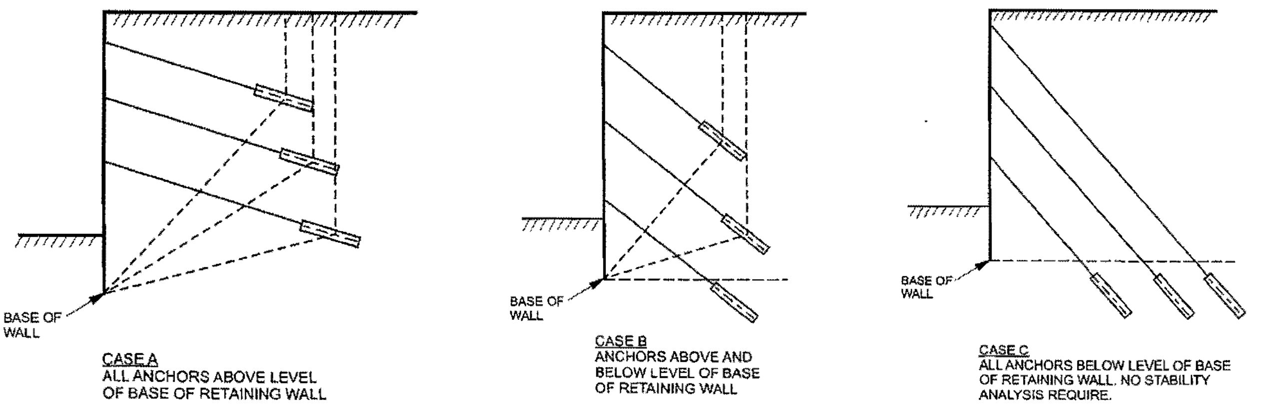


FIGURE 26.12 *Typical multiple-level anchor systems showing potential failure planes requiring analysis*

26.11.3 Overall Stability of Anchored Systems

When there are no struts or anchors near the bottom of the excavation, the depth of penetration in homogeneous materials may be about 1.5 times the depth required for moment equilibrium about the lowest strut or anchor. In this calculation, it is assumed that the vertical member is hinged at the lowest support point and no consideration is given to the moment resistance provided by the structural member. The resistance provided to the portion of wall penetrating below the base of the excavation is generally computed using the theoretical passive earth pressure and ignoring wall friction. For soldier piles, the maximum horizontal force on the soldier pile below the bottom of the excavation may be conservatively taken as three times the value computed for the width of the pile provided that the center-to-center spacing of the piles is more than three times the pile width. For individual piles the width may be taken as the pile flange width for driven piles, or the drilled diameter for concrete-filled sockets for pre-bored piles.

In soft to medium stiff cohesive soils, the embedded part of the wall may be subject to a force, P_H, potentially larger than calculated using the apparent earth pressure distribution diagrams as the apparent pressure distribution terminates at the base of the cut. Where the base stability factor of safety is less than about 1.5, the wall must penetrate below the base of the excavation to maintain stability. The force on the buried part of the wall can be calculated as follows (see also Figure 26.9):

$$\text{For } H_I > \frac{2}{3}\frac{B}{2^{1/2}},\quad P_H = 0.7(\gamma_T HB - 1.4 s_u H - \pi s_u B) \tag{26.3}$$

$$\text{For } H_I < \frac{2}{3}\frac{B}{2^{1/2}},\quad P_H = 1.5\, H_I\, (\gamma_T H - 1.4 s_u H/B - \pi s_u) \tag{26.4}$$

The force, P_H, will act at the mid-height of the buried part of the wall.

26.11.4 Structural Design of Vertical Members

In practice, a wide variety of assumptions have been made regarding the design of sheeting, soldier piles, and other excavation support systems. Since deformation and load are inversely related, load "reduction factors" have sometimes been applied to the design of various components of excavation support systems. Where deformation is of little concern, the design loads for the walls of an excavation support system might also be selected presuming some redistribution of load to the struts as a result of pile deformation and soil "arching" similar to cantilever and single-support systems as described previously. When sizing the vertical members of retaining structures, careful attention must be given to the assumptions regarding assumed load distributions, permissible deformations, design moment calculation method, structural assumptions (simple spans or continuous spans), and application of load reduction factors.

For walls supported by anchors or struts designed using the apparent earth pressure diagram "tributary area approach" (essentially assuming hinges at all support points), bending moments should be calculated by:

1. Applying the appropriate apparent earth pressure diagram from Figure 26.8;
2. For the cantilever section above the top strut or below the bottom strut, assume that the vertical element (sheeting, piles) is fixed at the upper support level and calculate the maximum moment in the cantilever section; and
3. For the interior spans between the struts, assume that the vertical element is hinged at each support point and calculate the maximum simple-span moment.

The larger of the bending moments calculated at either of the cantilever or simple spans should be used for design. Table 26.3 provides moment calculation formulae for some simple design cases. The maximum moments for other load distributions should be determined by development of appropriate shear and bending moment diagrams. Further guidance on calculation of maximum bending moments for complex load distributions can be found in the

Handbook of Steel Construction, Canadian Institute of Steel Construction, latest edition.

The formulae provided in Table 26.3 represent simple and cantilever spans and will generally result in the highest value for design moment. In reality, the vertical members are not hinged at the support levels unless plastic deformation takes place. Therefore, it is common practice to consider the spans continuous over the supports and reduce the maximum moments by about 20 % to account for beam (vertical member) continuity. It is not recommended that such a reduction be applied to cantilever span sections where moments are calculated assuming fixity at the support locations.

TABLE 26.3 *Moment Calculation Formulae for Simple Beam Cases*

Span Type	Load Distribution	Maximum Moment
Cantilever	triangular, increasing linearly to fixed end	$M_{max} = (1/6)p_{amax}l^2$
	uniform load	$M_{max} = (1/6)p_{amax}l^2$
Simple Span	triangular, increasing linearly to one end	$M_{max} = (1/7.8)p_{amax}l^2$
	uniform load	$M_{max} = (1/8)p_{amax}l^2$

It has been suggested that the apparent earth pressure loads may be reduced for design of the vertical members of shoring systems (e.g. sheet piles, soldier piles) considering the "arching" that redistributes loads from flexible vertical members to the support points. Earth load reduction factors as low as 2/3 have been applied to shoring design (e.g. Peck, Hanson, and Thornburn 1974). These load reduction factors have been applied in addition to considering the simple-span vertical members to be continuous. The combination of load reduction factors and considering the wall continuous across the supports results in design moments that are more than 40 % less than those calculated using the simple-span formulae in Table 26.3. Although a structurally stable design may result from the lower design moments, these load reduction factors should be used with caution as they result in design of a more flexible excavation support system. If load reduction factors are applied, the potential loads and deformations should be considered in detail. In general, where close control of ground deformations is required, such earth load reduction factors should not be used.

26.12 Horizontal Supports – Anchors, Struts and Rakers

26.12.1 Struts

26.12.1.1 Temperature Loading of Struts

Struts may be subjected to temperature-induced stresses, and an allowance in design for this effect may be necessary (Boone and Crawford, 2000). Though the apparent earth pressure diagrams implicitly include some effect of temperature (Peck 1969, Goldberg et al. 1976), this effect is not quantified in such diagrams. For braced excavations in soft or loose soil, the effect of temperature on struts may not be great since the end restraint provided by the ground may be small. However, for excavations in dense or hard soils and loose rock, the struts may respond dramatically to temperature and the loads induced by temperature fluctuations may approach those that would be calculated assuming that the ends of the strut are perfectly fixed. The ratio between the temperature-induced load for a real strut and the load that could be experienced by a strut with perfectly-fixed ends is called the load ratio, LR. The load on a strut due to temperature fluctuations may be calculated using the equations provided in Table 26.4.

The value of the soil elastic modulus used in the expressions provided in Table 26.4 should be chosen appropriately for the level of strain induced in the soil by the expanding and contracting strut. It has been shown that reasonable estimates of the secant elastic modulus for this level of strain can be based on values resulting from unload-reload curves of pressuremeter or plate load tests at strains less than approximately 1/4 to 1/3 of the failure or limit strain.

TABLE 26.4 *Temperature-Induced Loads in Struts*

Temperature-Induced Load, P_T, =	$\dfrac{\alpha_s \Delta TL}{(2I)/(sE_{s(m)}) + L/(A_S E_S)}$
Load Ratio, LR, =	$\dfrac{L}{(2A_S E_S)/(sE_{s(m)}) + L}$
Mobilized Elastic Modulus, $E_{s(m)}$, =	$\dfrac{2IP_T}{s[\alpha_s \Delta TL + P_T L/(A_s E_s)]}$

where

α_s = coefficient of thermal expansion of the strut (units of strain per degree)
ΔT = change in temperature (degrees)
L = length of the strut
I = influence factor for area loaded by strut (pile width and length between struts or area of continuous wall between supports) assuming that the deformation is analogous to settlement of a footing on the surface of an elastic half-space
s = average vertical spacing between subject strut and struts above and below
E = mobilized secant elastic modulus of supported ground
A_S = end area of steel strut
E_S = elastic modulus of steel strut

26.12.1.2 Pre-Loading Struts

Struts are often pre-loaded to help obtain a tight fit between the struts, wales, and wall, and to impart a load on the wall, strut and ground such that some proportion of the final arching effects are induced prior to further excavation. The principal goal of these effects is to reduce the overall movement of the shoring system and the retained ground.

A wide variety of systems have been used for pre-stressing struts including:

- wedging with steel shims;
- jacking against special flanges welded to the strut and filling the resulting gap between the strut end and the wall; and
- constructing telescoping sections of strut that are welded together following jacking.

Additional detail on some construction procedures can be found in FHWA (1976). Careful attention should be given to the method of load transfer since after removing pre-stressing jacks, compression can occur in previously unstressed parts of the connection and the pre-stressing load can be lost (e.g. Boone et al. 1999). Particular attention should be given on structural and shop drawings to procedures for pre-stressing, wedging, or jacking to maintain tight contact for all bracing members and to provide for distribution of load to struts and wales.

The amount of pre-loading necessary for any particular project should be determined considering the need to control ground movements, strut design, construction methods, and the presence of nearby structures. Pre-loads should not induce passive failure of the ground behind the retaining system. If existing structures (buildings, utilities) are near the shoring system, the magnitude of the pre-load should be examined in detail so that additional and potentially damaging stresses are not exerted on the nearby structure. Typical strut pre-load values are often in the range of 40 % to 70 % of the final design load.

26.12.2 Rakers and Raker Footings

Raker footings should be designed in accordance with the design principals for shallow foundations subject to inclined loading. 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 for the piles, where D = depth of penetration of the piles below the base of the excavation. No excavation should be made within two footings widths of the raker footings on the side opposite the rakers.

When sloping berm excavation procedures are employed, the rakers should be installed in trenches in the berm to minimize movement of the retaining wall being supported (Figure 26.13). The trenching procedure illustrated in Figure 26.13 should be used with caution since the passive pressure in front of the wall is greatly reduced in comparison to the pressure available with a horizontal grade at the top of the berm and must be evaluated in detail (see also Clough and Denby 1977). Since berms are used during their installation, and because of the difficulties in pre-stressing the inclined supports, rakers should generally not be used where control of wall deformations is critical.

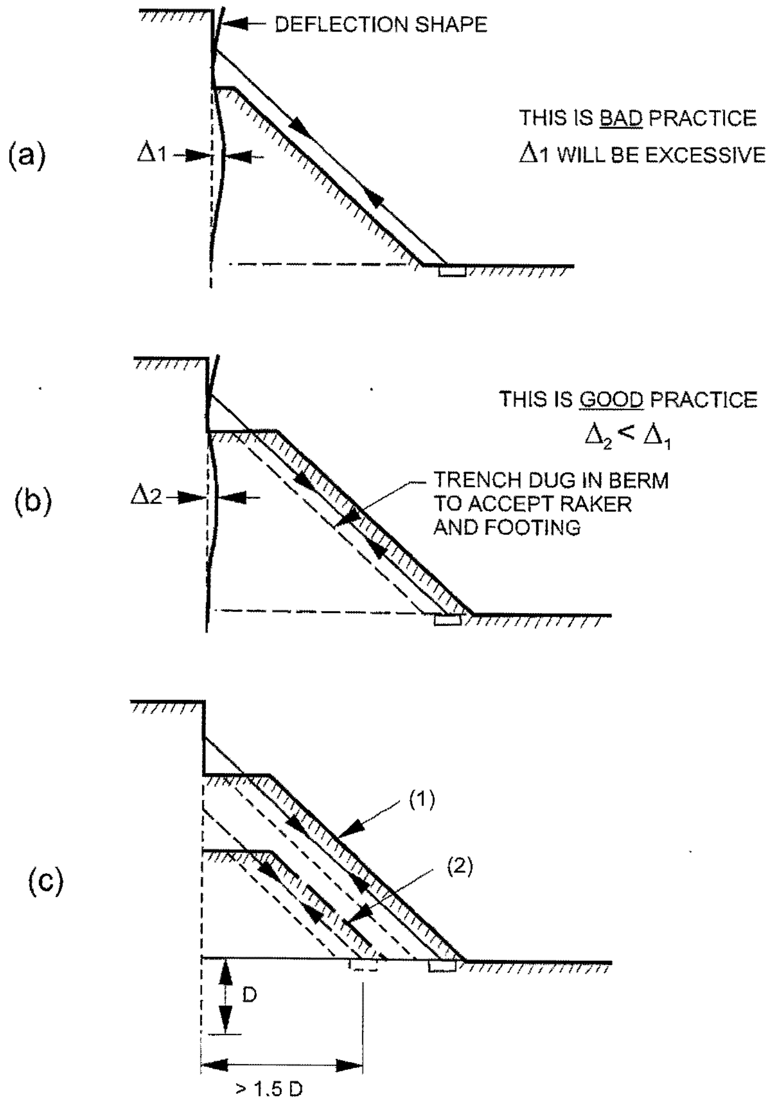


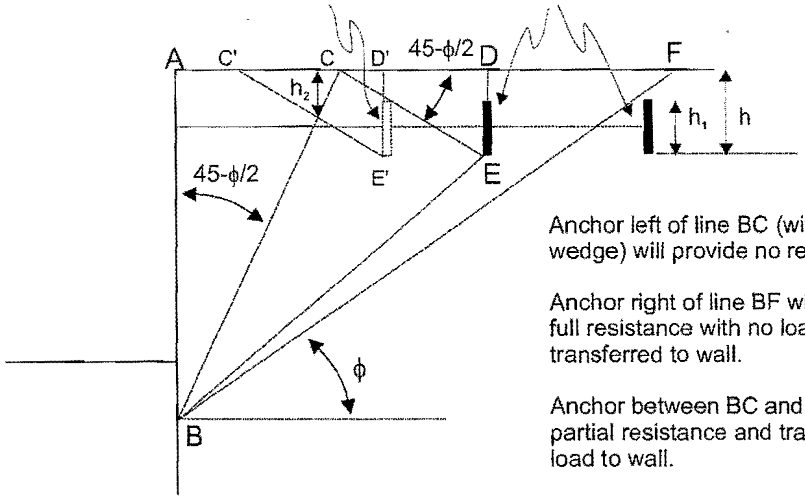
FIGURE 26.13 *Placing of rakers*

26.12.3 Buried Anchors

Marine bulkheads and retaining structures supported by a single anchor level are often supported by a tie-rod connecting the wall and a buried mass or "deadman" at some distance behind the wall. Such anchors develop their resistance by virtue of their deadweight and the passive resistance of the ground between the anchor and the wall. Such anchors can be constructed of mass concrete, sheetpiles, a parallel wall structure, or piles (driven or drilled). The load capacity of a "deadman" anchor is highly dependent on its placement relative to the wall. Criteria for design are illustrated in Figure 26.14.

SECTION THROUGH WALL, BACKFILL, AND ANCHORS

Possible Anchor Positions



Anchor left of line BC (within active wedge) will provide no resistance.

Anchor right of line BF will provide full resistance with no load transferred to wall.

Anchor between BC and BF provides partial resistance and transfers some load to wall.

Anchor Left of CE

$P_P = 1/2K_p\gamma h^2$ against front (wall side) face of anchor
$P_A = 1/2K_A\gamma h^2$ against rear (backfill side) face of anchor

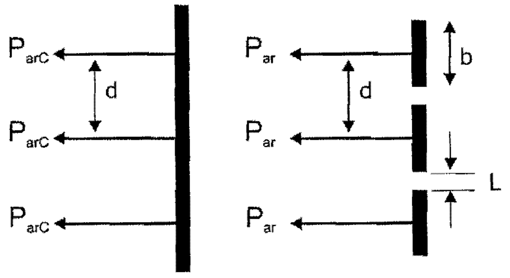
Anchor Right of CE

For h_1 greater than or equal to h_2

$P_P = 1/2K_p\gamma h^2 - (1/2K_P\gamma h_2^2 - 1/2\ K_A\gamma h_2^2)$ against front (wall side) face of anchor
$P_A = 1/2K_A\gamma h^2$ against rear (backfill side) face of anchor

PLAN OF ANCHORS

Continuous Anchor Wall | Individual Anchors



For h_1 greater than or equal to h/2:

1. Continuous Wall $P_{arC} = d(P_P - P_A)$

2. Individual Anchors

If $d > b+h$, $P_{ar} = d(P_P - P_A) + 2P_o\tan\phi$
where $P_o = 1/2K_o\gamma h$ for area CDE or C'D'E'
If $d = b+h$, $P_{ar} = 0.7P_{arC}$, $L = L' = h$ for this condition
where $P_o = 1/2K_o\gamma h$ for area CDE or C'D'E'
If $d < b+h$, $P_{ar} = P_{arC}/d - L/h(0.3P_{arC}/d)$

For h_1 less than h/2, P_{arC} and P_{arC} are equivalent to the bearing capacity of footing of width h1 and surcharge load $\gamma(h - h_1/2)$

FIGURE 26.14 *Criteria for design of buried anchors*

Design of the tie-rods and their connections should consider corrosion protection. Tie-rod design should also consider potential settlement of the backfill and measures to protect the tie-rods from loading by the overlying fill and potential sag if vertical support is removed from beneath the rod.

The allowable anchor rod force should be taken as the calculated value of P_{ar} from Figure 26.14 divided by a factor of safety of 2.0. The tie-rod, anchor and wall connections should be designed for a load 20 % greater than the allowable load calculated as above. The tie-rod should also be connected to the anchor at the location of the resultant earth pressure acting in the vertical plane of the anchor. If the anchor is constructed by compaction of backfill around the anchor, all backfill should be compacted to at least 95 % of its maximum dry density.

26.12.4 Soil and Rock Anchors

The anchors discussed in this section 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 anchor or head. In cohesionless soils and rock, the fixed anchors are often formed by pressure grouting techniques, while in stiff cohesive soils, grout or concrete can also be placed using tremie methods, except where the inclination of the hole to the horizontal is not very great. Typical anchor details are shown in Figure 26.15.

The performance of soil and rock anchors depends not only on minor variations in soil and groundwater conditions, but also on construction techniques and details. Anchor capacities calculated using the procedures outlined are considered to represent a rational basis for design, but must be verified by test or proof loading during construction.

26.12.4.1 Estimated Capacity of Soil Anchors

Computation of the pull-out resistance, P_{ar}, for tremie-grouted anchors in cohesionless soils can be estimated from the following equation:

$$P_{ar} = \sigma'_z A_s L_s \alpha_g \tag{26.5}$$

where

σ'_z = effective vertical stress at the midpoint of the load carrying length (Figure 26.15)
A_s = effective unit surface area of the anchor bond zone
L_s = effective length of the anchor bond zone (limited to about 8 m)
α_g = anchorage coefficient dependent on the soil type and conditions as given in Table 26.5

TABLE 26.5 *Anchorage Coefficient α_g*

Soil Type	Relative Density		
	Loose	Compact	Dense
Silt	0.1	0.4	1.0
Fine sand	0.2	0.6	1.5
Medium sand	0.5	1.2	2.0
Coarse sand, gravel	1.0	2.0	3.0

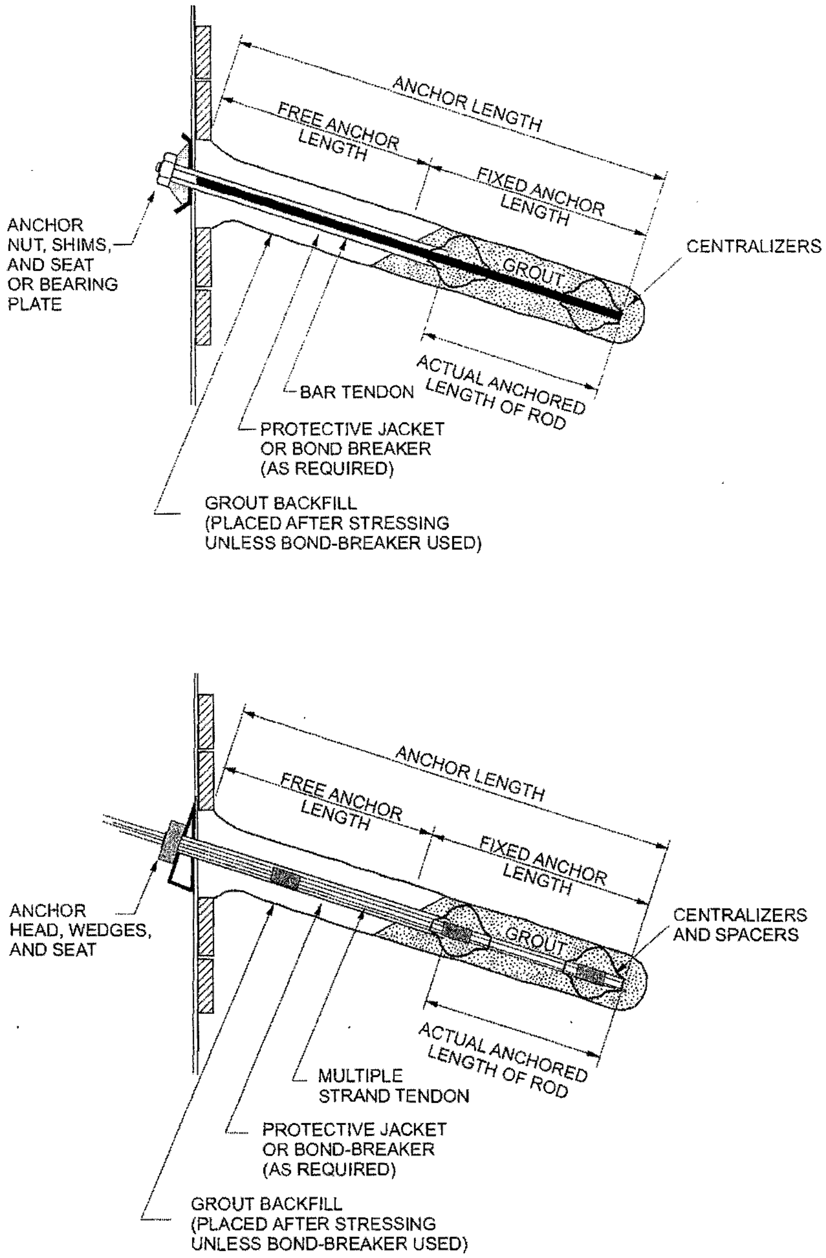


FIGURE 26.15 *Schematic of typical bar and multiple strand anchors*

Computation of the pull-out resistance, P_{ar}, for tremi-grouted anchors in stiff to very hard clay soils can be estimated from the following equation:

$$P_{ar} = \alpha_c A_s L_s s_u \tag{26.6}$$

where

α_c = reduction factor related to the undrained shear strength (Figure 26.17)
A_s = effective unit surface area of the anchor bond zone
L_s = effective length of the anchor bond zone (limited to about 8 m)
s_u = average undrained shear strength of the clay over the anchor length

Normally, anchors should not be formed in soft or firm clays, sensitive clays, or in uncontrolled fill materials because of the large deformations that can occur, both at and subsequent to loading.

Small diameter, pressure-grouted anchors can develop higher capacities than those that could be calculated using the methods above for an equivalent diameter tremie-grouted anchor. Preliminary capacities of pressure-grouted anchors may be calculated according to the values provided in Table 26.6 provided that:

- the nominal diameter of the anchor is between 150 mm and 200 mm;
- grout is injected using a positive pressure of about 1 MPa;
- and the center-to-center spacing of the anchors in the bond zone is more than 4 times the anchor diameter or 20 % of the bond zone length (see also Figure 26.16).

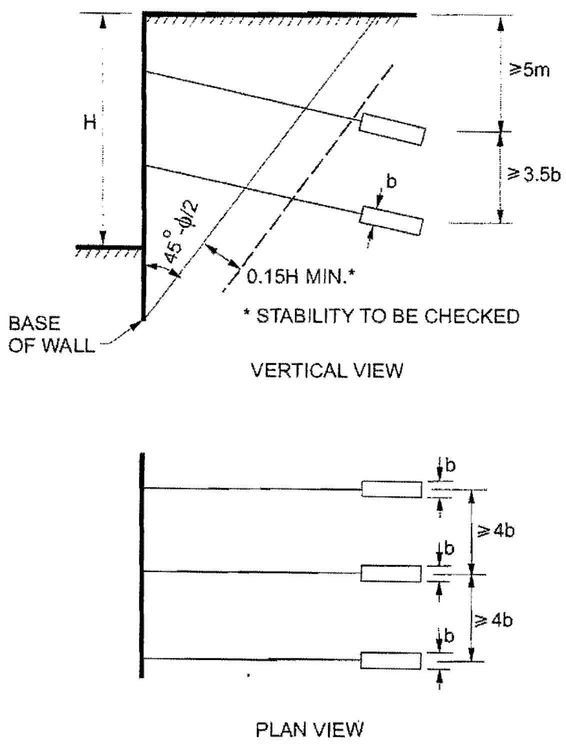


FIGURE 26.16 *Minimum spacing and depth for soil anchors*

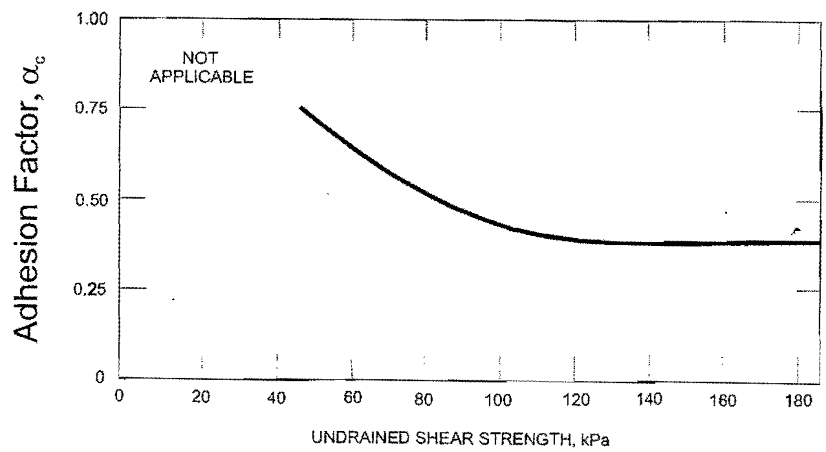


FIGURE 26.17 *Adhesion factor for anchors installed in cohesive soils*

TABLE 26.6 *Estimation of Capacity for Pressure-Grouted Anchors (after FHWA 1984)*

Ground Type	Relative Density/Consistency (SPT "N" range)	Estimated Ultimate Load Transfer (kN/m)
Sand and Gravel	loose (4 – 10) medium compact (10 – 30) compact (30 to 50)	145 220 290
Sand	loose (4 – 10) medium compact (10 – 30) compact (30 to 50)	100 145 190
Sand and Silt	loose (4 – 10) medium compact (10 – 30) compact (30 to 50)	75 100 130
Low-Plasticity Silt & Clay	stiff (10 – 20) hard (20 – 40)	30 60

The ultimate load transfer values provided in Table 26.6 are generally suitable for anchors constructed using a single stage of grouting. Some anchors, however, are designed and constructed such that they can be post-grouted in one or more stages subsequent to initial grouting. During secondary and later stages of grouting, the initial grout around the anchor zone is fractured and the secondary ground forces the fractured grout against the soil, and the secondary grout can also further permeate or compact the surrounding ground. Multiple stages of grouting can increase the load transfer values above those provided in Table 26.6, however, the use of higher values for such anchors should be based on detailed experience with local ground conditions and construction practices. When considering pressure-grouted anchors, consideration should be given to the actual grouting pressures, pressure dissipation around the anchor, and the potential for heaving or fracturing the ground.

The capacity of anchors estimated using the methods above presumes a relatively linear increase of capacity with a corresponding increase in bond zone length. However, anchor capacities generally do not increase once the length of the bond zone increases beyond about 8 m. The capacity of an anchor in soil can also be established by a pull-out test.

The allowable anchor load is determined by dividing the ultimate capacity of the anchor by a factor of safety. Where no pull-out tests are carried out, the allowable anchor load is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3 or more.

26.12.4.2 Soil Anchor Capacity Established From Pull-Out Tests

Where the capacity of anchors is to be determined by pull-out tests, at least one anchor in 100 of those actually used in the project should be tested — with a minimum of one in each soil or rock type. See also 26.12.4.7.

If the anchor movement is appreciable, the capacity of the anchor is defined as that load at which the anchor begins to pull out of the ground (plastic displacement). If the load is not clearly apparent from the test data, the capacity is taken as the maximum load at which the observed movement is still tolerable for the structure. If the ultimate resistance is not reached, or no plastic displacement is observed in the test loading, the greatest applied test load should be assumed as the capacity for calculation of the allowable anchor.

26.12.4.3 Estimated Capacity of Rock Anchors

Anchorage design in rock is based on an allowable grout-to-rock bond stress acting over the fixed anchorage length. The allowable bond stress should be smaller than 1/30 times the unconfined compressive strength of the grout. It should not exceed 1300kPa. Whenever possible, the capacity of an anchor in rock should be established by means of a pull-out test. Table 26.7 provides estimates of ultimate load transfer values for a variety of rock types.

TABLE 26.7 *Estimation of Capacity for Pressure-Grouted Anchors (after FHWA 1984)*

Rock Type	Estimated Ultimate Load Transfer (kN/m)
Granite or Basalt	730
Dolomitic Limestone	580
Soft Limestone	430
Sandstone	430
Slates and Hard Shales	360
Soft Shales	145

The allowable anchor load is determined by dividing the ultimate capacity of the anchor by a factor of safety. Where no pull-out tests are carried out, the allowable anchor load is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3 or more.

26.12.4.4 Location of Anchors

Where multiple anchors are used, the interaction of anchors must be considered if spaced more closely than 4b, where b is the anchor diameter, or if spaced closer than one-fifth of the anchor length (see Figure 26.16). Anchors should also extend beyond the theoretical active earth pressure failure plane as illustrated in Figure 26.16 so that the anchors are not placed within a potentially deforming soil mass. Depending on the results of overall stability analyses (see Section 26.11.2), anchors may need to extend further beyond potential planes of deep-seated (rotational) failure.

26.12.4.5 Installation of Anchors

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 water-bearing zones or wet soil are encountered, holes must be temporarily cased, or supported by other means. In some rock formations and in soil, small diameter anchors are often installed using flush-joint steel casing with an internal drill bit and a cutting shoe at the advancing end of the casing. Anchors in soil are sometimes installed by drilling a hole using a hollow-stem auger to the full anchor depth. Where the hole is 200 mm diameter or less, grout is injected through the casing or hollow auger stem (often at a pressure greater than 10 MPa) to achieve a grouted anchor length.

Care must be taken to ensure that high grout pressure will not cause damage to adjacent structures or services. Where the hole is up to 300 mm diameter, concrete rather than grout is often pumped through the casing or hollow stem auger as it is withdrawn from the hole. Normally, if a hole is of this larger diameter, a lesser pressure to inject the concrete is used. Depending on the nature of the ground and the inclination of the hole, the use of hollow stem augers for anchor construction may result in significant disturbance to the ground surrounding the hole. If auger rotation, penetration, and extraction rates are not well controlled, excess ground may be withdrawn around the auger stem and wet, loose, or soft soils may cave toward the auger and be removed during anchor drilling. Such disturbance can result in low capacities, excessive deformation, or anchor failure and installation methods must be chosen appropriate to the ground conditions.

26.12.4.6 Stressing and Proof Loading of Anchors

The method of tensioning, the sequence of stressing, and the procedures to be adopted for each stressing operation should be specified at the planning stage of a project. For grouts based on Portland cement, stressing should not commence until the primary grout forming the fixed anchor has attained a uniaxial compression strength of at least 30 MPa, as verified from tests on 100 mm or 150 mm cubes. No tendon that is intended to form part of any temporary or permanent work should be stressed at any time beyond 80 % of the guaranteed ultimate strength.

The stressing equipment should have duplicate reading capability, e.g., both a load cell and pressure gauge. If a discrepancy occurs between the two that exceeds 5 % that cannot be eliminated by re-centering or reseating the equipment, the instruments should be re-calibrated. To provide accurate measurements of deformation, an independent reference frame must be used. Measurements of deformation of both the anchor head and wall should be made. It is generally insufficient to measure the extension of the jack ram since the wall may deform and permit the jack to extend while the anchor head does not move. Such measurements could suggest unrealistic, adverse, or unreasonable deformations.

26.12.4.7 Design Tests

Design tests are carried out before the installation of working (production) anchors. They provide criteria to substantiate the design parameters used to define acceptable performance of service anchors during proof testing. In general, a cyclic loading pattern for design tests should be adopted as follows:

- Apply a small "seating" load (smaller than 5 % of the maximum test load) to allow the jack to stabilize, then check all gauges and readings of load and corresponding displacements. Increase the applied load in increments in the following manner:

  T – Working Load of Anchor
  0.00T, 0.25T, 0.50T, 0.25T, 0.50T, 0.75T, 0.50T, 0.25T,
  0.50T. 0.75, 1.00T*
  0.75T, 0.50T, 0.25T
  0.50T, 0.75T, 1.00T, 1.25T, 1.50T*
  1.25T, 1.00T, 0.75T, 0.50T, 0.25T
  0.50T, 0.75T, 1.00T, 1.25T, 1.50T, 1.75T, 2.00T**
  1.75T, 1.50, 1.25T, 1.00T, 0.75T, 0.50T, 0.25T
  0.00T***

  Each load, except the maximum for each cycle, shall be maintained for a minimum time of 1 minute. Deformation readings should be taken at the beginning and end of this time.

  * At 1.00T and 1.50T, the load should be maintained for 1 hour and load and deformation readings taken at 1, 2, 5, 10, 20, and 60 minutes.
  ** At 2.00T, the load should be maintained for 10 hours if a creep test is to be performed with load and deformation readings taken as above for the first hour and then at hours 2, 4, 5, and 10.
  *** At this point, an additional loading cycle, directly to the ultimate load can also be utilized as a check of all the deformation readings.

- Hold the applied loads steady within a variation of no more than 2 % of the incremental load – variation in the load can make interpretation difficult and will compromise the test results; and
- On completion of each loading step, reduce the load to the seating load; hold this steady and take a reading of permanent deformation before proceeding to the next loading step.

In general, an anchor should be acceptable when:

- Deformation during the final log cycle of time does not exceed about 2 mm; and
- The recorded elastic movement of the tendon exceeds 0.80 of the theoretical elongation of the free-length.

Creep-deformation criteria are subject to judgement and depend on the use of the anchors (i.e. for short-duration construction, long-duration construction, or as permanent anchors). Further guidance may be found in FHWA (1984).

If failure occurs, continue the loading process only as desirable to determine the reasons for failure. Record the failure load and the nature of the failure.

26.12.4.8 Proof Tests

Proof tests, carried out on all working (servicing) anchors, can employ procedures similar to design tests, but are normally abbreviated and simpler. Proof tests are not intended to be destructive.

A proof test should be taken to the maximum test load of 1.33 times the working (service) load. The general procedure given above for design tests is recommended with the following guidance:

- The number of equal load increments is reduced to four, each load level being 33 % of the working load;
- A single cycle is used up to the maximum 1.33 T; and
- At 1.33 T, the load is held for 10 minutes with load and deformation readings taken at 0.5, 1, 2, 5, and 10 minutes.

Upon reaching and having maintained the maximum load, the tension is released to the specified "lock-off Load" and the tension load is transferred from the jack to the nut/anchor head, whereupon the jack and equipment are removed. If an anchorage fails to meet the specified acceptance criteria, further stages of testing may be required.

26.12.4.9 Lift-Off Tests

Lift-off testing is the technique of using the stressing jack straddling over the stressing head to lift it clear of its seat and recording the tendon load with the jack pressure gauge or load cell. The distance the anchor head is raised is usually 1 mm, although this may be as low as 0.1 mm. The distance should be predetermined and the method of measurement should ensure that all sides of the stressing head are clear of the distribution plate.

26.12.4.10 Plotting and Reporting Anchor Test Results

The results should be plotted as shown in Figure 26.18 and Figure 26.19 and show all points of measurement. The load-displacement graph should include the theoretical elastic line of the tendon and its elastic limit, calculated using the nominal free length of the tendon. Any significant difference between the slope of the elastic line and the slope of the rebound portions of the load-displacement curves could indicate a discrepancy between the actual free length of the tendon and the nominal free length intended in the design, or a progressive loss of bond. Records should be taken and kept of the following:

- Data on soil conditions pertinent to the anchorage performance, including locations and characteristics of soil and rock strata intersected by the anchor drill hole.
- Details of drilling including hole diameter and length, method of drilling, peculiarities of the drill hole, type of drill and cutting head, and the method of flushing or supporting the hole.
- Type and composition of the grout, date of grouting, volume injected, and grouting pressure.
- Type, diameter, cross-sectional area, elastic modulus, elastic limit, strength of the tendon steel. Special features, such as corrosion protection systems, should be fully described.
- Details of the test equipment and procedures, including a diagram showing details of method used and

accuracy of force and displacement measurement. A copy of the calibration certificate should be included.

- For each test, graphs corresponding to the tabulation (as per Figure 26.18 and Figure 26.19), and a report with sketches/photographs showing the test arrangement and (should anchorage failure occur) the nature of the failure.

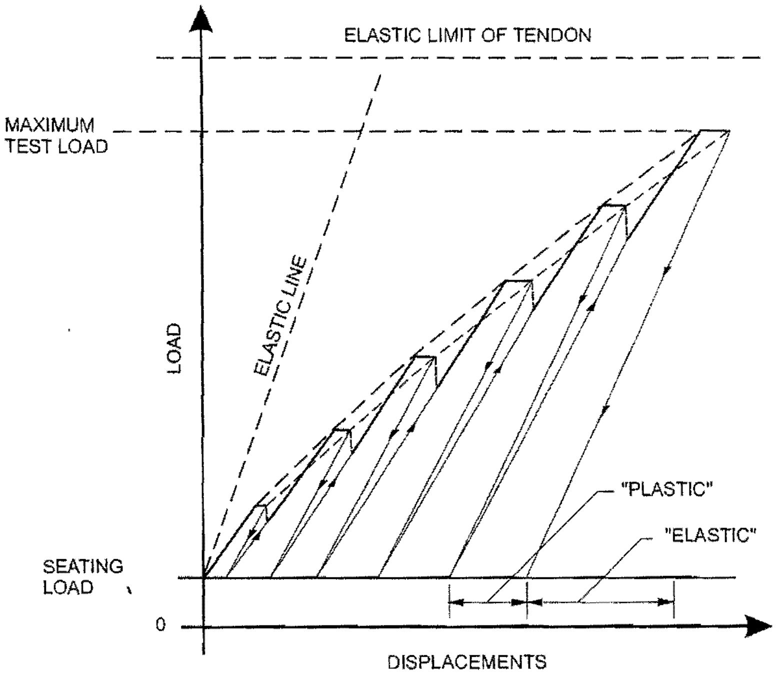


FIGURE 26.18 *Load-displacement curve for anchor test*

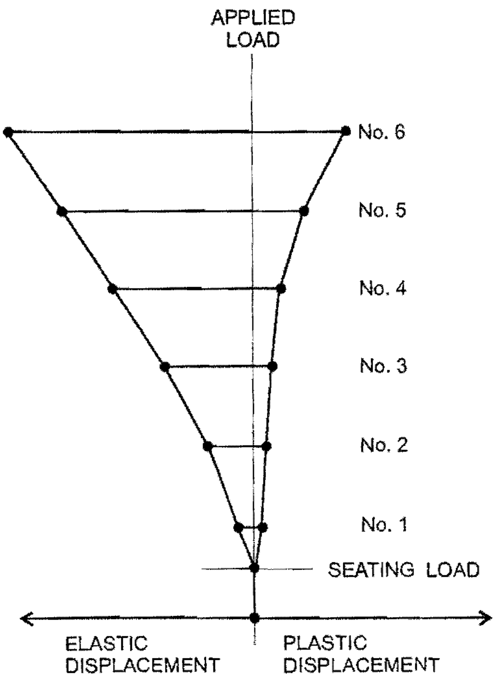


FIGURE 26.19 *Diagram of elastic and plastic displacements*

26.13 Other Design and Installation Considerations

26.13.1 Installation of Sheeting

Use of steel sheet piles is limited by loading conditions, movement control considerations (stiffness) as well as installation considerations. Sheeting is typically driven into place using well-established methods and sequences. Since sheeting is driven into the ground, boulders, cobbles, very dense or very hard soil, or other obstructions can significantly inhibit installation. Vibrations from sheet pile driving can also be prohibitive in urban areas due to disturbance and the potential for vibration-related settlement damages to nearby facilities. After construction, sheeting can be extracted. However, sheeting extraction can leave voids within the soil because of adhesion to the sheet as it is extracted. Disturbances from sheeting extraction and any resulting voids can be particularly problematic where the sheeting has extended below the base of the excavation and the new structure.

26.13.2 Horizontal Spacing and Installation of Soldier Piles

These piles provide intermittent vertical support and are installed, before excavation commences. The supports induce arching in the soil behind the pile, which provides the primary support to the retained soil. The spacing of the piles should be chosen to suit the arching ability of the soil and the proximity of any structures sensitive to settlement. Spacing of 2 m to 3 m is commonly used in strong soils, where no sensitive structures are present. The spacing is closed-up to 1 m to 2 m in weaker soils or near sensitive buildings. Because of the intermittent nature of the primary support system, the soldier pile and lagging method is not normally suitable in very soft clays or in flowing soils, where continuous support is required.

In most cases, it is essential that the soldier piles are maintained in full contact with the soil. For this reason, they should be either driven or placed in predrilled holes that are back-filled to the ground surface with soil-cement, grout, or concrete. Installation of driven piles may be problematic as discussed above for driven sheeting.

Installation of pre-drilled soldier piles must consider the influence of ground and groundwater conditions on the potential for maintaining a stable drill hole. Collapse of the hole can result if saturated sand, loose soils, or upward seepage gradients at the hole base are not adequately controlled. Temporary casings (drilled in or driven in with a vibratory hammer), a water level within the hole to balance groundwater pressures, a stabilizing drilling slurry, or a combination of these may be required to maintain a stable drill hole for placement of the pile. Construction of soldier pile holes utilizing water-balancing or slurry methods requires careful control and cleaning of the hole bottom and placement of concrete using tremie methods, particularly if the wall is to be supported by tie-backs with a component of vertical load.

26.13.3 Installation of Secant or Tangent Pile (Caisson) Walls

Where contiguous drilled shafts are used to compose the wall, they may be designed and constructed to overlap ("secant") or to abut one another ("tangent"). Such walls are often selected to provide greater structural stiffness than either sheeting or soldier-pile and lagging walls. Secant or tangent pile walls also offer the advantage that they can assist in minimizing the flow of groundwater into the excavation. However, such walls are rarely "watertight' as the overlap or continuity of the joint between successive piles is subject to considerable construction variability. It may be necessary to grout or otherwise fill gaps in the walls during excavation to satisfactorily limit the ingress of water and soil particles. Installation of contiguous pile walls is subject to drill-hole stability and concrete placement considerations as discussed above for soldier pile walls.

26.13.4 Installation of Concrete Diaphragm (Slurry) Walls

Concrete diaphragm walls are typically used where greater control over ground deformations or water flow is required or where the temporary and permanent walls are to be integral. These walls are constructed by excavating short sections of a trench in which the trench walls are supported by a viscous fluid, or slurry. Structural reinforcement is provided by reinforcing bars tied in "cages", soldier piles, pre-cast concrete panels lowered into the slurry. The slurry is then displaced by concrete placed using tremie methods. Since these walls are constructed using slurry and tremie techniques, control over the properties of the slurry (trench stability) and adequate cleaning of the trench are critical to successful wall construction. Typical trench widths range from 0.6 m to 1 m.

26.13.5 Lagging Design and Installation

The lagging serves as a secondary support to the soil face and prevents progressive deterioration of the soil arching between the piles. Lagging can consist of steel sheeting, concrete panels, wood boards, or sprayed and reinforced concrete ("shotcrete"). By far, the most common method in North America is to use wood lagging.

Lagging is often installed in lifts of 1 m to 1.5 m, depending on the soil being supported and on the convenience of working. If there is risk of groundwater building up behind a lagged wall, or of washing in of soil particles, gaps or weep holes should be left in the lagging to allow for drainage, and these should be packed with straw or synthetic filter material to prevent loss of soil.

Any voids behind the lagging should be backfilled with granular material packed in-place if drainage behind and through the lagging is required. To limit the potential for the backpacking to ravel out from behind the lagging during successive lifts, it may be beneficial to backfill the bottom lagging board in each lift with cohesive materials or grout. However, consideration should be given to the need for vertical drainage continuity. In addition, it is good practice to drive hardwood wedges between the piles and the lagging (boards, or sheets) in particular, at the top of each lift to maintain a tight fit between the lagging, retained soil and backfill and piles.

There is no established method for the design of lagging and its design is largely empirical. Experience on a large number of excavations indicates that the wood thickness given in Table 26.8 may be safely used for any depth of excavation less than about 16 m (White 1973, FHWA 1976). However, the thickness of the lagging may have an effect on the ground deformation that occurs between piles, and consequently the deformation that occurs in the ground behind the support systems depending on pile spacing, horizontal support spacing, soldier-pile flexibility and other related factors.

Local experience may indicate modifications to the spacing and lagging sizes indicated in Table 26.8. It is important to consider all local conditions, such as the duration of excavation, seasonal variations in the groundwater and the possibility of ice loading.

TABLE 26.8 *Thickness of Wood Lagging*

Thickness of Lagging[1]	Maximum Spacing of Soldier Piles (m)	
	Soft to Medium Clays[2]	All Other Soils
50 mm	--	1.5
75 mm	2.00	2.50
100 mm	2.25	3.00

1. Construction grade lumber
2. Where $\gamma H/s_u > 5$ the use of lagging is questionable

For design of alternative lagging materials (sheeting, shotcrete), it is customary to use approximate one-half of the earth and surcharge pressures used for design of the remaining wall elements. The use of such reduced pressures, however, depends in large measure on the degree of lagging deformation that occurs between soldier-piles and the load-deformation relationships of any particular lagging material should be checked carefully.

26.13.6 Excavation Sequences

The design of all members including struts, wales, sheetpiling, walls, and soldier piles should be checked for all stages of partial excavation where the excavation has been taken to the next lower support level but prior to installation of the support. Where horizontal supports are to be removed during excavation, and the wall is to be supported by partially completed permanent structures, the spans resulting from support removal should also be checked for structural adequacy. Either of these situations may result in the maximum loading on any one particular member of the excavation support system.

26.13.7 Design Codes and Drawings

Structural members such as walls, struts, soldier piles, and sheeting should be designed in accordance with the structural requirements of the National Building Code of Canada and the Canadian Handbook of Steel Construction. The effects of combined axial and flexural loading, unsupported span lengths, and lateral stability of the members must be considered in the design.

Details on contractors' shop drawings should show:

1. Appropriate means for positioning of struts and walers and for bracing of struts in both vertical and horizontal planes to provide lateral stability;
2. Details on web and connection stiffeners, and brackets; and
3. Provisions for wedging and jacking of struts to prevent horizontal movement

Details are important for the adequacy of earth-retaining structures and should be shown on shoring drawings along with the installation sequence of all elements of the structure.

26.14 Alternative Design Methods

Excavation support and retaining systems can be designed taking into account deformations in the soil in order to calculate wall deflections and stresses. Such analyses can be done by finite-element or finite-difference methods, although a simplified and common design method makes use of soil-spring analyses, based on beam-on-elastic support theory and structural analysis computer programs.

Figure 26.20 shows typical beam-on-elastic support analyses. In the method illustrated by Figure 26.20, springs are used to model the soil on both active and the passive sides. On the active side, as the wall moves away from the soil, the load drops off from some initial value (analogous to an at-rest pressure in the soil), to a minimum value consistent with active pressure. On the passive side, as the wall pushes into the soil, resistance, proportional to deflection, is mobilized in the springs. This spring stiffness can be chosen to model the soil stiffness in passive compression. When a resistance develops corresponding to the ultimate passive resistance available from the zone of soil represented by that spring, the spring resistance is maintained at a constant value, simulating the plastic phase of the elasto-plastic soil model. The strains required to mobilize the passive resistance are much greater than the strains required to mobilize active pressure. Since active conditions are mobilized at very small strains, the analysis can be simplified. Rather than using active soil springs to create the appropriate loading, the active pressure can be assumed as a loading on the wall. This is shown in Figure 26.20. The passive soil zone and the struts are modeled as springs as in the method shown in Figure 26.20. Struts or anchor supports can be also be modeled as springs, with spring values chosen appropriate to the type of support.

With the beam-on-elastic foundation method of analysis, several fundamental problems need to be considered:

1. Using a single model to represent the final excavation and bracing support will not adequately represent the staged excavation conditions and both loads and deformations will probably be underestimated (see Figure 26.20);
2. For each stage of a staged-excavation analysis, a separate model is typically required to simulate the changed

excavation and support levels;

3. Prior to the installation of each support, deformation will have occurred at this point – these deformations must be added to the final deformation profile if the model is intended to estimate deformations of the structural members;
4. If the supports are to be removed, these stages must also be considered in the analysis
5. Beam-on-elastic-foundation models will not account for vertical stress-redistribution effects;
6. Beam-on-elastic-foundation models cannot account for the deformations and stress-redistribution effects that occur between discontinuous support points (e.g. soldier-piles); and
7. Deformations that occur beneath the toe of the wall (structural elements) are not considered.

While numerical methods such as beam-on-elastic supports, finite-elements, or finite-differences can be used to assist structural design, such programs should be used with caution. The use of spring constants presumes a linear stress-deformation response that is not consistent with the non-linear behaviour of soils. Therefore, the choice of such a constant requires an iterative approach to assure an appropriate degree of strain compatibility. The use of computer-based numerical methods to simulate the construction process and soil-structure interaction should be undertaken only by personnel with demonstrated experience in numerical modeling since the choice of soil behaviour model, structural behaviour model, interfaces, and other boundary conditions can all significantly affect the analysis results.

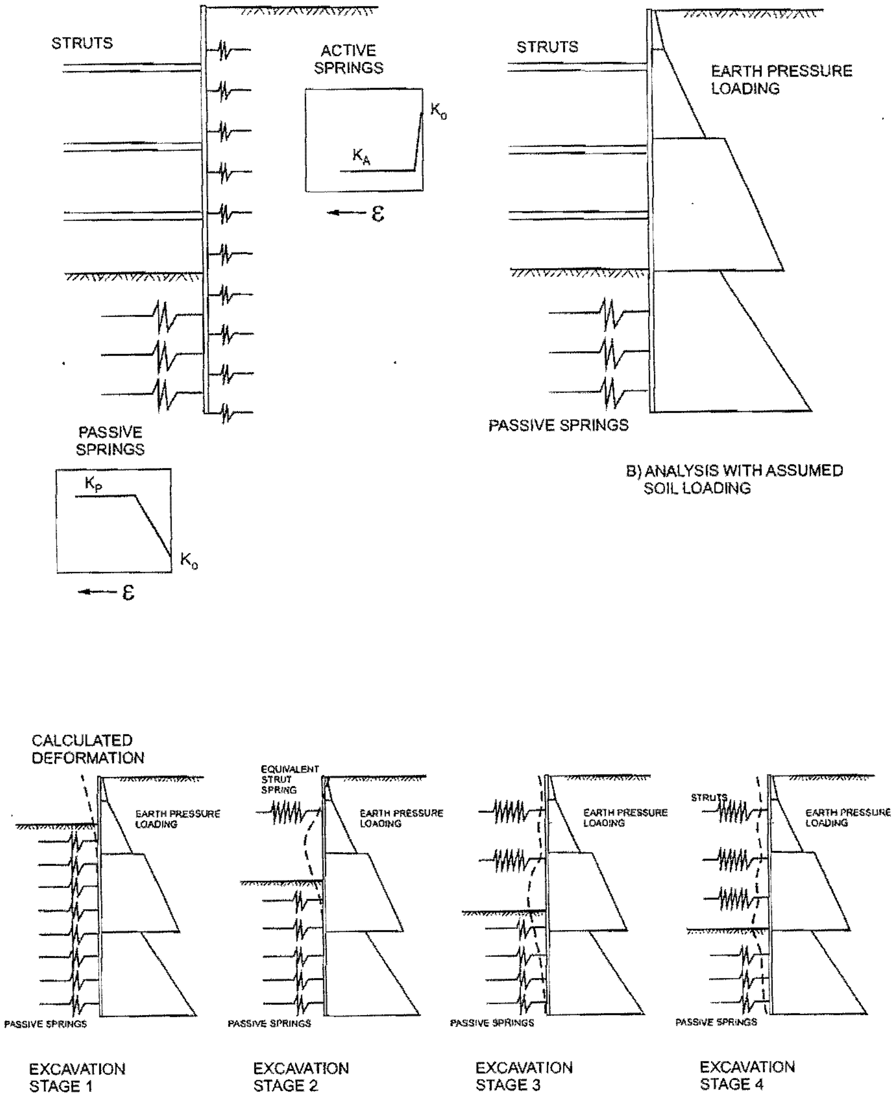


FIGURE 26.20 *Beam on elastic support analysis for diaphragm wall design*

In general, numerical models do not account for the effects of construction unless a number of assumptions are made and included in the models. Estimates of horizontal and vertical deformation made using numerical models should consider the sequential nature of the construction process and, depending on the particular models, deformations arising from each stage of analysis may need to be combined to produce a reasonable result (as noted above). Numerical modeling, however, can be a powerful tool for parametric evaluation of variables (soil parameters, structure configuration and materials, construction sequence, excavation geometry) for complex soil-structure interaction problems. When numerical methods are used to evaluate complex soil-structure interaction problems, they should also be calibrated to empirical observations of local projects in similar ground conditions.

26.15 Movements Associated with Excavation

Movements associated with excavations are related to a number of factors including:

- Base stability
- Soil type
- Consolidation of loose sands and soft clays arising from pore-water pressure changes
- Wall type
    - sheet piles
    - soldier piles and lagging
    - soil mixed walls
    - secant/tangent piles
    - concrete diaphragm walls
- Structural stiffness of vertical support elements
- Horizontal support type
    - rakers
    - struts
    - anchors
- Horizontal and vertical spacing of horizontal supports
- Construction procedures
- Workmanship

Any one of the above factors may control the overall movement of a supported excavation. Direct and quantitative analysis of ground movements associated with excavation support is difficult since the total deformation is a complex interaction of the above factors in three dimensions. Structural modeling of excavation support systems seldom produces a reliable prediction of deformation since:

- Deformation can occur during excavation to each support level prior to installation of the supports (struts, anchors, etc.);
- Deformation can occur below the base of the excavation;
- Deformation occurs in a horizontal plane between the horizontal supports (e.g. from wale or wall deflection between struts); and
- Deformation occurs in a vertical plane between the horizontal supports (e.g. soldier-pile deformation);

Because of these factors, and depending on the point of measurement, the ground behind an excavation support system can move more than the structural system itself. Therefore, estimation of ground movements associated with supported excavations is generally based on a combination of analytical and empirical methods combined with judgement and experience. Guidance regarding potential magnitudes of deformation considering some of the above factors is provided below.

In general, the movements of anchored walls can be less than the movements of excavations supported by struts for several reasons:

- Anchors are typically fully stressed to the design load prior to excavation below the anchor level;
- Typically little excavation occurs below the level of the anchor since anchor installation equipment needs

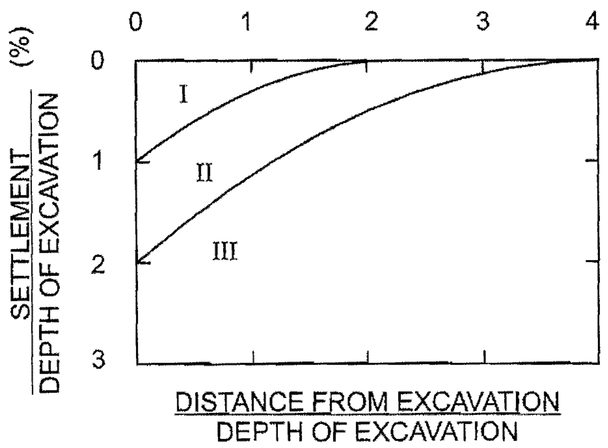
sufficient level ground to work from;

- Because struts interfere with excavation, berms and trenches may be used to facilitate equipment access and the reduced passive resistance can lead to additional deformation;
- The connection between struts, wales, and walls is often imperfect and without pre-loading the struts to the full design loads compression of these connections can lead to additional deformation; and
- When completing structures within the excavations, struts are often removed when anchors are often left in place, thus an additional stage of potential deformation is experienced with strutted excavations.

Movements of excavations supported by rakers are often greater than those experienced by either braced or anchored excavations. The necessity of cutting berms prior to raker placement and the time required for raker footing construction contribute to deformations. Pre-loading of rakers is often more difficult than pre-loading of struts or anchors because of the connection geometry. In general, rakers are not suitable for support of excavations where deformation control is critical.

26.15.1 Magnitude and Pattern of Movements

For well-constructed support systems, the general magnitudes of ground deformations associated with excavation support can be categorized according to the predominant soil type in which the excavation is made. Deformation of supported excavations will cause both vertical and lateral movements within the adjacent ground. In general, the maximum lateral and vertical movements are of the same order of magnitude. Depending on the density of the adjacent ground and the magnitude of shear strains induced during excavation, the vertical movements can be nearly equal to the maximum lateral movements. If significant dilation of the ground occurs during shear (dense sand and stiff to hard clay), the maximum vertical movements may be as little as 0.33 to 0.5 of the maximum lateral movements. Reliance on reduction of settlements due to dilation however, should only be considered following careful and detailed analyses. If the excavation permits pore-water pressure losses within the supported ground, excavations in compressible soils could also induce additional vertical consolidation settlements in the surrounding area such that the maximum vertical settlements are greater than the maximum lateral movements. Excluding consolidation settlements and provided that excavation and construction are well controlled, it can be reasonably assumed that the vertical deformations and lateral deformations may be nearly equal.

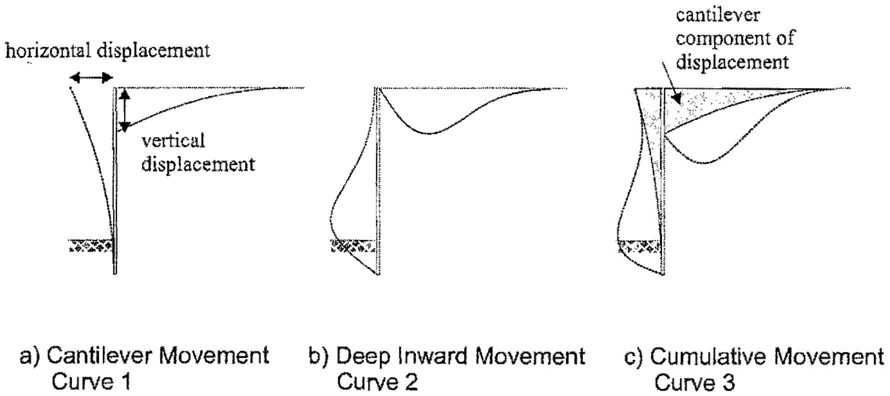


ZONE I	Sand and soft-to-hard clay
ZONE II	Very soft to soft clay when the depth of clay below the excavation is limited, or when $FS_b > 1.3$
ZONE III	Very soft to soft clay when $FS_b < 1.3$

FIGURE 26.21 *Settlement adjacent to an open cut (after Peck 1969)*

The general magnitude and pattern of settlements adjacent to supported excavations was first practically illustrated by Peck (1969) as illustrated by Figure 26.21 that suggested the deformation behaviour was primarily dependent upon the ground type through which the excavation was made. The magnitude of vertical ground deformation near supported excavations, however, depends on a number of inter-related factors as described previously.

The pattern of deformation depends largely on the wall system type (i.e. whether or not there is significant friction along the wall back) and the horizontal support locations. For walls with little friction along the wall back (e.g. sheet pile walls in sand or soldier-pile and lagging walls) where wall deformation can be generally considered rotation about the toe or lateral translation, vertical ground deformation is often shaped like a parabolic spandrel (Figure 26.22). Where significant wall friction is present (e.g. secant/tangent pile or concrete diaphragm walls), or where the lateral deformation shape is characterized by an outward bulging of the wall at depth, the vertical deformation profile is often shaped like an inverted exponential probability distribution curve (Figure 26.22). Many excavations induce both patterns of deformation to varying degrees.



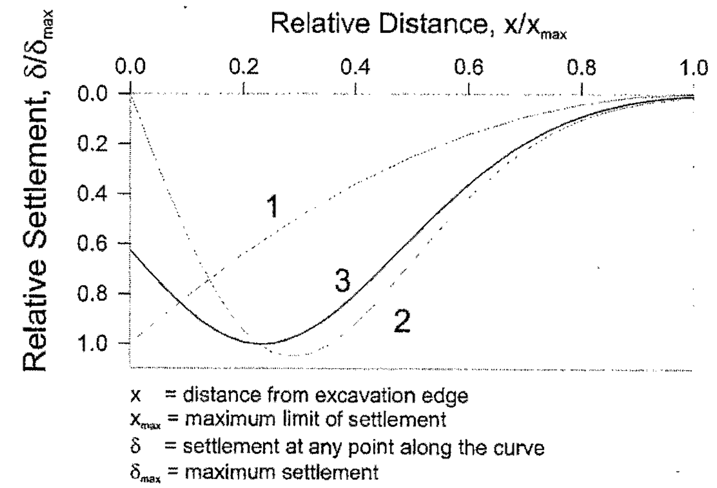


FIGURE 26.22 *Patterns of movement behind excavation support systems and their mathematical approximations*

Simplified mathematical approximations for the generalized patterns of wall movement are provided below as Equations 26.7 to 26.9 (see also Figure 26.22) as these may assist with assessing the effects of the excavation on adjacent facilities:

For a spandrel settlement trough:

$$\delta_{s,I} = \delta_{max}((x_{max}-x_i)/x_{max})^2 \tag{26.7}$$

For a concave settlement trough

$$\delta_{c,I} = \delta_{max}6(x_i/x_{max})e^{-6(x_i/x_{max})^2} \tag{26.8}$$

If the maximum values of the individual spandrel and concave settlement parts of the cumulative trough can be made, a combined settlement trough can be estimated as

$$\delta_i = \delta_{s,I} + \delta_{c,I} \tag{26.9}$$

where the subscripts i and max relate to the settlements at a single point and the maximum within the profile, respectively.

While Figures 26.21 and 26.22 and the above expressions for $\delta_{s,i}$, $\delta_{c,i}$ and δ_i illustrate the general characteristics of settlement troughs adjacent to excavations, the trough shape and magnitude will be dependent upon the complex interaction of many factors. Figure 26.23 through Figure 26.26 illustrate the bounds (envelopes) of measured deformation for a variety of ground conditions. In some cases, the pattern of deformation may match the pattern indicated by the envelope; however, the envelopes as shown do not discriminate between wall types or construction conditions. It should also be recognized that ground deformations arising from excavation include both a vertical (settlement) component and a horizontal component. The use of any generalized trough or maximum settlement envelope should be based on anticipated excavation systems design, ground conditions, local experience, and judgement.

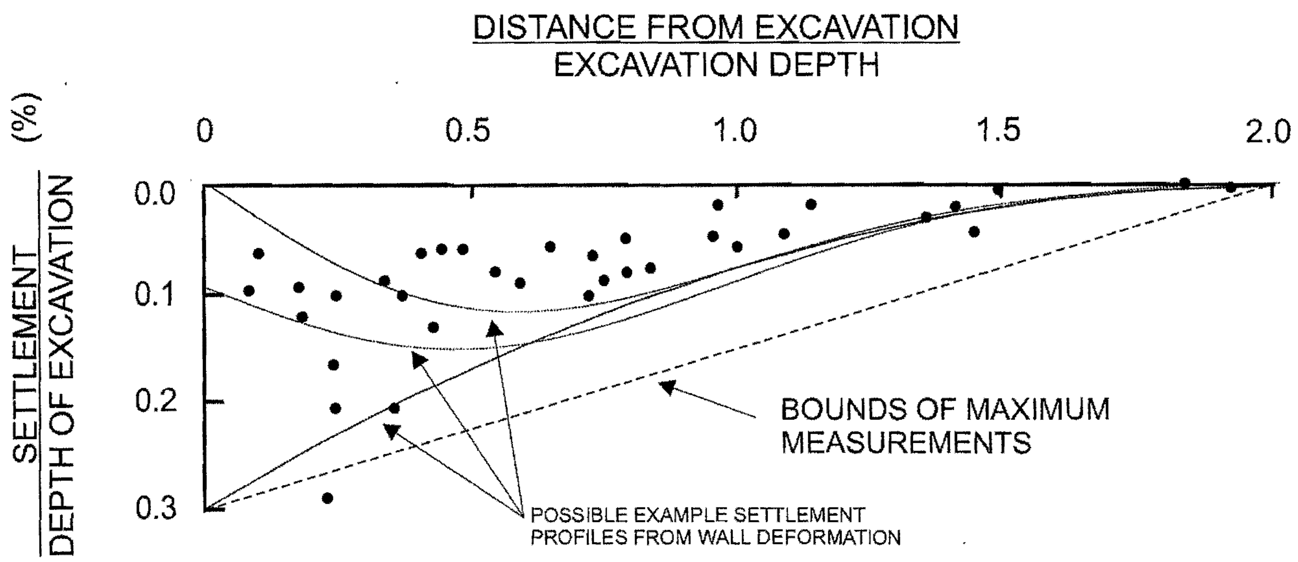


FIGURE 26.23 *Summary of measured settlements adjacent to excavations in sand (after Clough and O'Rourke 1990).*

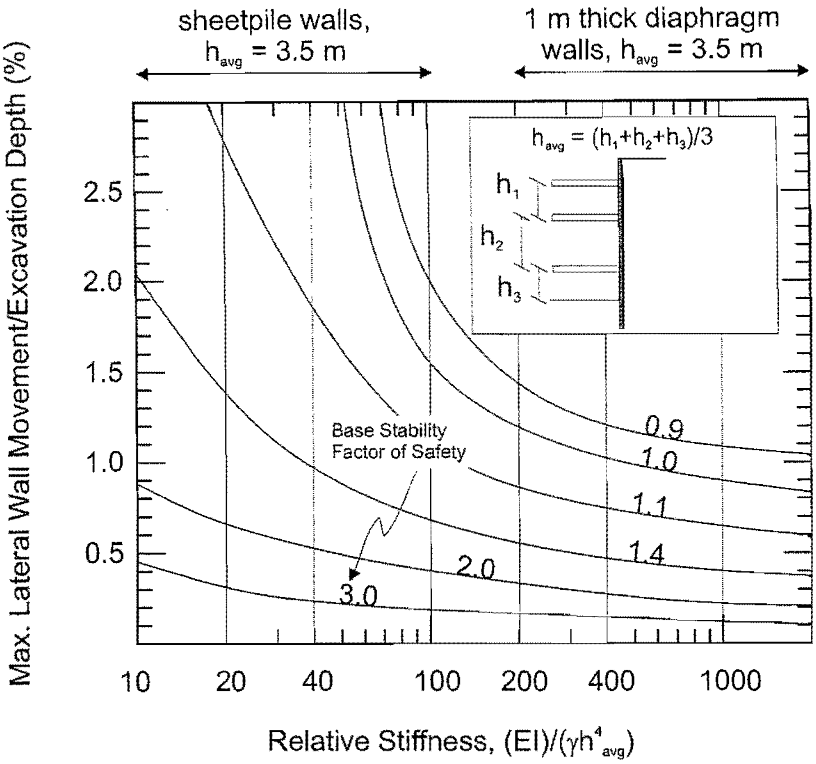


FIGURE 26.24 *Relationship between maximum lateral movement and excavation base stability actor of safety (after Clough et al. 1989)*

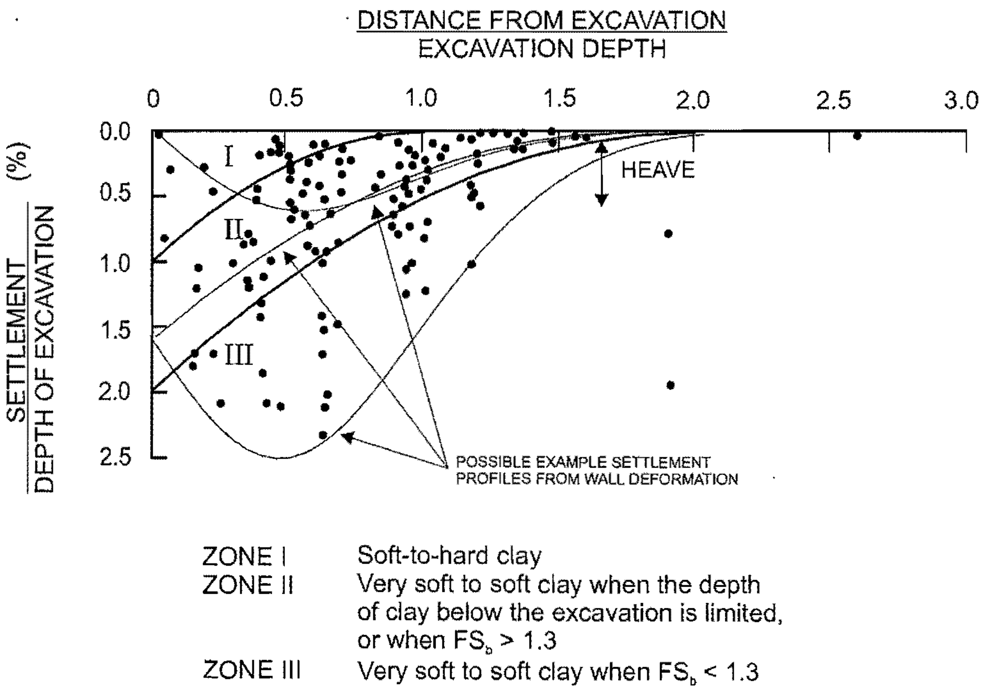


FIGURE 26.25 *Summary of measured settlements adjacent to excavations in soft to medium stiff clay (after Clough and O'Rourke 1990)*

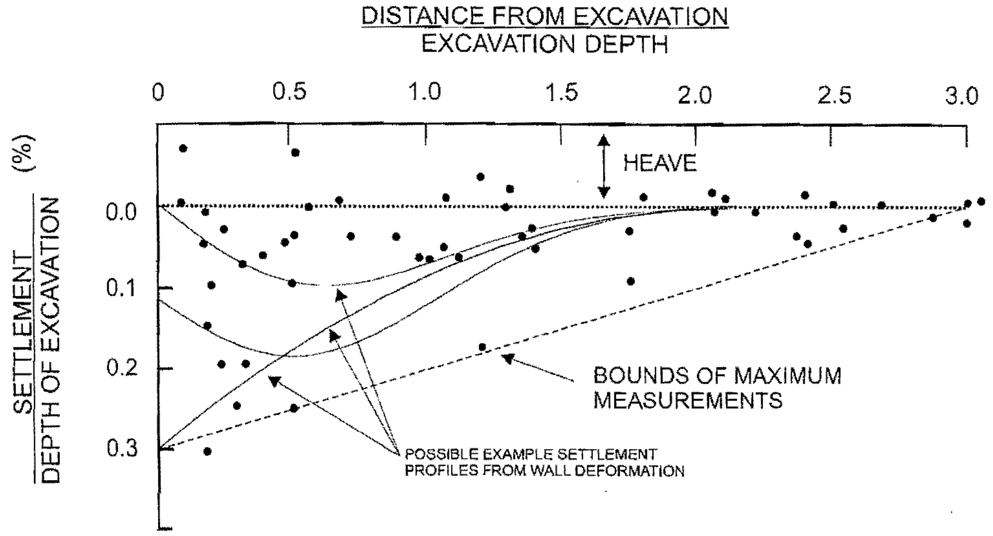


FIGURE 26.26 *Summary of measured settlements adjacent to excavations in stiff clay (after Clough and O'Rourke 1990)*

26.15.2 Granular Soils

In general, for equivalent excavation support systems, the magnitude of vertical movements in granular soils of most densities is typically less than lateral movements in soft to stiff cohesive soils. If horizontal supports are installed as soon as the support level is reached the maximum vertical ground movements can be expected to be in the range of about 0.2 % to 0.3 % depending on wall stiffness, workmanship, and the degree of support pre-stressing (Figure 26.23). For excavations in granular soil, the maximum zone of influence, x_{max}, is typically about 2.0 times the value of the excavation depth as illustrated by Figure 26.23.

26.15.3 Soft to Firm Clays

Substantial movements often occur when vertical cuts are made in soft clays. These movements occur in spite of well-constructed support systems. Measurements have shown that 60 % to 80 % of the total lateral yield at each support level occurs below the level of the excavation. Maximum lateral deformations have been shown to be directly related to the factor of safety against base heave and the wall system stiffness (Figure 26.24). Even if the system is properly installed and appropriate pre-stressing of supports is carried out, the maximum lateral and vertical movements of the ground are likely to be between 1 % to 2 % of the excavation depth depending on the wall stiffness and factor of safety against base heave. For excavations in soft to firm clays, the maximum zone of influence, x_{max}, is typically about 1.0 to 2.0 times the value of the excavation depth as illustrated by Figure 26.25. Consolidation settlements, however, can increase this zone of influence to values in excess of 3.0 times the excavation depth depending on changes induced in the groundwater patterns.

26.15.4 Stiff Clay

The lateral movements of temporary support systems decrease sharply as the shear strength of the soil increases (as implied by Figure 26.24). Movements are generally small if horizontal supports are installed as soon as the support level is reached and can be expected to be in the range of 0.1 % to 0.3 % of the excavation depth (Figure 26.26), depending on workmanship, the wall stiffness, and degree of support pre-stressing. For excavations in stiff clays, the maximum zone of influence, x_{max}, is typically about 2.0 times the value of the excavation depth as illustrated by Figure 26.26. Depending on the factor of base stability and wall deformation patterns, the zone of influence of vertical deformations can extend to 3.0 times the excavation depth as illustrated in Figure 26.26.

26.15.5 Hard Clay and Cohesive Glacial Till

Movements are generally small if horizontal supports are installed as soon as the support level is reached and can be expected to be in the range of 0.1 % to 0.2 % of the excavation depth depending on workmanship, the wall stiffness, and degree of support pre-stressing. For excavations in hard clay and cohesive glacial till, the maximum zone of influence, x_{max}, is typically about 1.0 to 2.0 times the value of the excavation depth.

26.15.6 Means of Reducing Movements

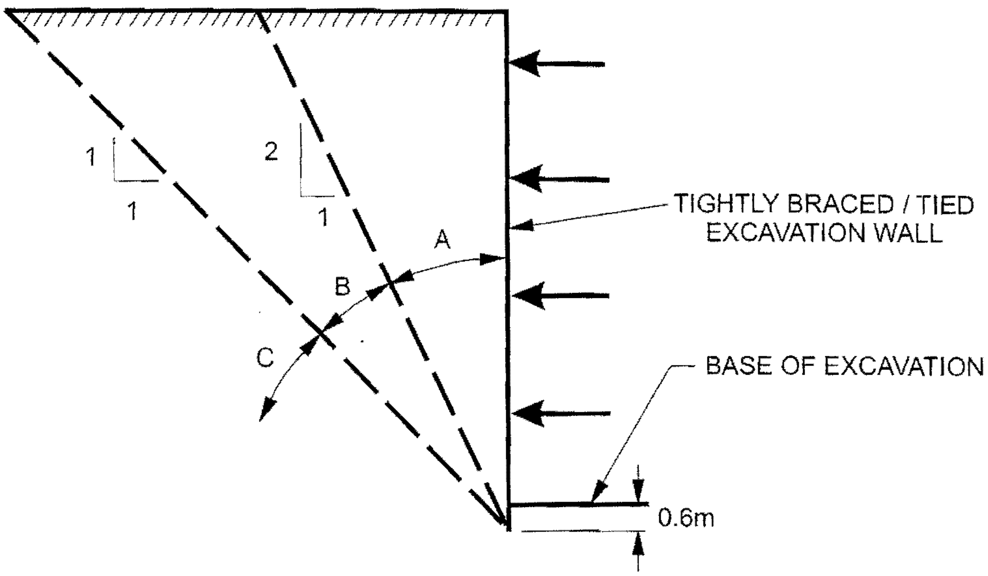
To reduce the magnitude of movements, it is necessary to alter the pattern and magnitude of shear strains induced in the ground by excavation. Several methods are available to effect this as described below.

- The overall excavation support system can be made stiffer by shortening the horizontal and/or vertical distance between horizontal supports. In general, the top supports should be placed as high as possible. Typically, a vertical spacing between rakers, struts, and anchors of 2.5 m is considered a minimum, with 4 m to 5 m being preferred. The maximum spacing for closely controlling deformation is generally close to 4 m, but where underpinning of small or light adjacent structures is omitted, a smaller spacing should be used. Vertical spacing between rakers, struts, and anchors greater than about 6 m is rare.
- The vertical component of the support system can be made stiffer (e.g. increasing pile size).
- The use of berms and trenching should be used with caution, however, as the passive resistance in front of the excavation support system is greatly reduced while the berm is in place and prior to the support installation.
- Larger pre-loading stresses can be used for rakers, struts, and anchors; however, the potential passive deformation or failure of the ground behind the wall must also be considered.
- Alternative wall types can be considered to optimise both ground movement control and cost and construction considerations.

26.16 Support for Adjacent Structures

Though underpinning can be used to reduce movements caused by excavations, underpinning itself can cause more movement than a well-designed and well-constructed excavation. The zones illustrated in Figure 26.27 are provided as general guidance, however, comparisons should be made between the anticipated patterns of deformation behind the supported excavation and the planned geometry of underpinning.

Providing underpinning only near the excavation edge or only within Zone A could result in unacceptable structure deformation if the pattern of deformation extends beyond the underpinned area and is of sufficient magnitude to cause damage. Guidelines for assessing the damage potential for structures adjacent to excavations, based on the characteristics of the building and deformation profile are provided in Chapter 11.



Zone A : Foundations located within this zone may require underpinning. Horizontal and vertical pressures on the excavation wall of non-underpinned foundations must be considered. Horizontal and vertical deformations of foundations within this zone must be considered relative to underpinned and non-underpinned foundations.

Zone B : Foundations located within this zone do not normally require underpinning. Horizontal and vertical forces on the excavation wall for non-underpinned foundations must be considered. Horizontal and vertical deformations of foundations within this zone must be considered relative to underpinned and non-underpinned foundations.

Zone C : Underpinning to structures is normally founded in this zone. Lateral pressure from underpinning is not normally considered.

FIGURE 26.27 *Guidelines for underpinning soils*

27 Reinforced Soil Walls

27 Reinforced Soil Walls

27.1 Introduction

Horizontal layers of reinforcement materials such as steel and various geosynthetics can be included within retaining wall backfills to provide a reinforced soil mass that acts as a gravity structure to resist the earth and surcharge forces developed behind the reinforced soil zone. The local stability of the backfill at the front of the reinforced soil mass is assured by attaching the reinforcement to facing units constructed with concrete, timber, or polymeric materials comprised of various shapers (Figure 27.1). Reinforced soil walls are commonly used in Canada as flexible retaining structures, ranging in height from approximately one metre to tens of metres.

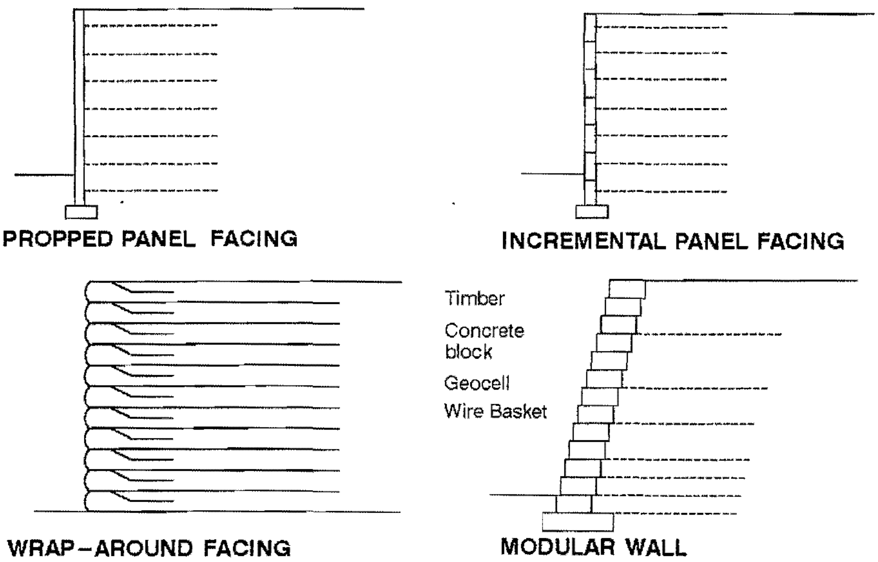


FIGURE 27.1 *Examples of reinforced soil wall types.*

Reinforced soil walls are frequently referred to as Mechanically Stabilized Earth (MSE) structures in many parts of Canada and the United States. These types of structures are generally proprietary in terms of design and construction methods. In normal state-of-practice, the internal stability design considerations of the reinforced soil wall are handled by the various proprietary design methodologies. The external stability design considerations (e.g. bearing capacity or resistance of the foundation soils, overall global stability and the like) are provided or confirmed by the geotechnical engineer.

Reinforced soil walls and MSE systems represent an alternate method for retaining and supporting earth pressures and surcharges as compared to conventional cast-in-place cantilevered or pile type walls made of concrete, steel, wood or other engineered materials. The components of reinforced soil walls need to be engineered for the specific intended conditions and environment. The compacted soil portion is engineered geotechnically. By taking into account the appropriate interaction with the reinforcement, the non-soil portion (reinforcement and facing) is engineered structurally to achieve the required or desired performance in terms of capacity and durability over the design life.

Most reinforced soil walls built to date have been constructed with the length of soil reinforcement a minimum of 70 % of the height of the wall. However, for specific design situations, the reinforcement length could vary from this general proportional relationship with wall height. The height of wall considered in determining reinforcement length and other calculations should include any surcharge on the wall.

27.2 Components

The three main components of reinforced soil walls are Reinforcement, Soil Backfill and Facing.

27.2.1 Reinforcement

Generally inextensible and extensible types of soil reinforcement are used. Inextensible reinforcements are elements typically made of metal, usually steel. High modulus stiff reinforcement allows very little strain of the soil to occur, which results in the internal failure wedge becoming modified from the classical (Coulomb/Rankine) soil failure wedge. Consequently, higher earth pressures, and hence loads, are generally developed for this type of low strain (more rigid) reinforcement than for less rigid reinforcement. Metallic reinforcement is typically made of hot-rolled steel, either in strips (both deformed/ribbed and flat), or in mesh or ladder type sheets (welded or rolled). Metal reinforcement design requires special consideration of corrosion from the aggressiveness of soil, moisture and time.

Extensible reinforcements are typically made of geosynthetics. The strain of geosynthetic reinforcement material is larger than that of steel reinforcement and allows the soil to deform to a greater extent, which results in the classical Coulomb or Rankine earth pressure theory and soil failure wedge being developed. Geosynthetics type reinforcement is made of polymers that are extruded, woven or non-woven, and come in various shapes as discussed in Chapter 23.

The design of reinforcement needs to take into account the tensile rupture, elongation and creep characteristics of the reinforcement material. In this regard, geosynthetics materials tend to have lower strength and higher elongation and creep characteristics than metallic reinforcement.

The connection between the soil reinforcement and the wall facing is an important design consideration. Connections need to be sufficiently strong and robust.

Tolerances for facing movement are usually given by specifying authorities and in relevant codes.

Design life and durability are key design issues. Design life varies depending on the jurisdictional agency and the application, but common values are: 100 years for bridge abutment walls and rail supporting walls; 75 years for retaining walls supporting or adjacent to highways; and the 10 to 30 years that are often specified for mining or temporary structures.

For steel soil reinforcement, achieving a specified design life is usually accomplished by applying a corrosion allowance to the thickness of the steel section. The thickness of steel section used in design is calculated as the Initial Thickness minus the Corrosion Allowance Thickness. Geosynthetics reinforcement design needs to account for the effects of creep, environmental degradation and construction damage. Chapter 23 provides additional discussion of these issues.

27.2.2 Soil Backfill

There are four regions of soil that influence the design and function of reinforced soil walls.

Reinforced Soil Zone

The Reinforced Soil Zone is the region of soil that surrounds the soil reinforcement. The selection of this soil material is important in the proper design and functioning of a reinforced soil wall. Most walls in Canada have been constructed using a clean (i.e. little to no fines) sand, or clean sand and gravel mixture. These materials provide good structural and frictional properties, good drainage and compact well under various water contents. They also avoid concerns about frost action concerns behind hard facing units. For some walls, a clean backfill material has been used behind the facing for the depth of frost penetration, beyond which a finer native material was used. This, however, is the exception as most reinforced soil walls are specified with granular material throughout. Walls constructed with clean granular backfill are the easiest to construct, avoid hydrostatic pressures and result in the best facing alignment and long term assurance of stability and safety.

Nevertheless, the cost advantage of using on-site fill materials has led to the successful use of soils with a large percentage of fines. However, caution must be exercised in the construction of reinforced soil walls that are not constructed with "free-draining" granular soils. Such soils may lead to backfill settlements that put additional loads on the wall facing-geosynthetic connections or that lead to the development of hydrostatic or seepages forces behind the facing that are difficult to quantify at the design stage. Porewater pressure could also develop along the reinforcement, which would reduce the interface friction. This aspect needs to be carefully considered and evaluated in design.

Properties such as unit weight and internal friction angle are required for design. In some cases, the parameters are assumed by the wall designer and then verified by the geotechnical engineer reviewing the design.

Most of the reinforced soil walls constructed in Canada have had backfill with the following recommended characteristics:

i. grain sizes – granular, less than 12 % passing 75 μm and 100 % passing 150 mm
ii. friction angle greater than 35 degrees unless tested otherwise
iii. plasticity index less than 6
iv. water content at time of compaction at or below 2 % dry of optimum
v. compaction – minimum 95 % Standard Proctor for retaining walls and 100 % under bridge abutments

Other aspects to consider in the selection of the Reinforced Soil Zone are:

i. chemical and electrochemical properties such as resistivity, pH, sulphates and chlorides
ii. biochemical and degradation damages, including hydrolysis, likelihood of extreme heat (fire), bacteria and construction damages

Criteria on the above can be found in jurisdictional and regulatory codes such as AASHTO Bridge Design Specifications, CSA and Building Codes. Additional information is provided in Section 27.3.2 and in Chapter 23.

External Backfill

The External Backfill is the region of soil that is placed immediately behind the Reinforced Soil Zone and just beyond the end of the soil reinforcement. This region of soil applies horizontal earth pressure to the Reinforced Soil Zone. This material may be an engineered fill with specified geotechnical parameters such as unit weight and internal friction angle, or it can be native soil such as in a cut or excavation situations.

Foundation Soil

The foundation soils under the Reinforced Soil Zone and wall facing units should be reviewed by a geotechnical engineer familiar with MSE wall design. The geotechnical engineer will confirm that the soils are suitable to adequately and safely support the applied loads, and will provide input for design and tender specifications such as minimum soil reinforcement length to achieve adequate global stability. This review is recommended to be carried out prior to tendering of bids for the wall systems to help avoid post tender delays, claims or errors in design.

Should the geotechnical engineer anticipate problems with bearing capacity, global stability or settlement, then options should be reviewed to either improve the foundation soils or reduce the loads.

To improve the global stability, the following three techniques are commonly used:

i. embedding the wall deeper below finished grade,
ii. lengthening the soil reinforcement, or
iii. foundation improvement such as placing a shear key below the Reinforced Soil Zone.

All of these techniques essentially cause potential global slip surfaces to go deeper, which usually results in a higher factor of safety.

Estimates of settlements should be provided to the wall designer to confirm that the proposed wall system can tolerate the anticipated settlements.

Fill Above the Reinforced Soil Zone

For the case of a sloping embankment fill or a structure placed above the Reinforced Soil Zone, the loading and restoring effects of this zone needs to be included in the design of the reinforced soil wall.

27.2.3 Facing

Facing Types

Facing types depend on the intended use of the wall, tolerance to settlement (total and differential), required durability, level of security and aesthetics. Examples of types of facing are shown on Figure 27.1 and include:

i. Precast Reinforced Concrete Segmental Panels
   The size of the facing panels varies from system to system. Small panel sizes do not attract large bending moments and can be constructed with light lifting equipment. Facings usually have a smooth surface finish, but can be accented with architectural relief. Segmental panel systems can have compressible pads in the horizontal joints that can accommodate differential settlements of up to 1 % along the face of the wall.

ii. Precast Full Height Panels
   Precast panels extend the full height of the wall. These panels require very careful aligning and bracing during installation to avoid misalignments. If excessive misalignment occurs, a complete rebuilding of the wall may be required to fix the problem. Some jurisdictions limit the maximum height of full height panel systems. Full height panel systems generally do not accommodate differential settlements as well as segmental and modular block systems.

iii. Dry-cast Un-reinforced Segmental (Modular) Blocks
   This facing is made with a "dry-cast" moulding technique, similar to that used for concrete blocks used for building. The blocks are typically 200 mm in height and of various widths. They can be provided with smooth or rough surfaces and in different colours. Dry-cast blocks are normally unreinforced and can be subject to cracking due to freeze-thaw and susceptible to damage (spalling) from point loads that develop

during movements of the modular blocks as a result of settlement.

iv. Wire mesh facing
Reinforced soil walls can be constructed with a wire mesh facing with a geotextile backing. To enhance service life, wire mesh should be made from hot dipped galvanized wire and backed with coarse rockfill. This type of facing is commonly used for short-term design lives or temporary applications.

v. Geosynthetic wrapped facing
For a short-term design life or temporary application, a geosynthetic wrap wall can often be used.

Facing Considerations

The following factors should be considered in the design or selection of facing types for reinforced soil walls:

i. design life
ii. connection type
iii. bending stresses and flexural strength
iv. loads generated during handling, transportation and construction
v. tolerances to movements during and after construction settlement, deflection, rotation
vi. temperature and shrinkage effects
vii. porosity, freeze-thaw
viii. drainage
ix. seismic movements
x. lifting anchors
xi. aesthetics
xii. durability for given exposure classes as specified by governing codes
xiii. life cycle cost considerations and future maintenance

27.3 Design Considerations:

27.3.1 Site Specific Design Input

The following general characteristics of the site, loads and grades should be considered for reinforced soil wall design.

i. Structure geometry – alignment and grades (top and bottom at front of wall), top slopes, lateral offsets, embedment, existing facilities, interface with other structural components, obstructions etc.
ii. Loadings – surcharge, its magnitude and configuration, dead and live loads
   - dead loads, concentrated or uniform (bridge seats, building, columns/footings or slopes),
   - live loads - uniform, line, eccentric, cyclic, impact, bollard and vibration loadings etc. (vehicular traffic, rail, dump trucks, cranes, machinery, or surge piles etc.)
iii. Site Characteristics - foundation stratigraphy, soil bearing capacity, allowable ground movement tolerances (during and after installation, lateral and longitudinal)
iv. Site Conditions – temperature ranges, freeze-thaw, hydraulic conditions (water levels, drawdown, submergence/flood), seismic influence and impact to adjacent structures
v. Soil Parameters - friction angle and unit weight

27.3.2 Design Methodology and Approval

The recommended approach for analysing and designing reinforced soil wall structures is based on concepts of classical earth pressure theory which provide an analytical framework that is familiar to geotechnical engineers. The relative stability of these structures is quantified based on conventional geotechnical concepts of factor-of-safety against failure of the soil and reinforcement at limit equilibrium. As noted in Section 27.2.1, classical

Rankine or Coulomb earth pressure theories apply to geosynthetics reinforced soil walls. For metallic (inextensible) reinforcement, the applied earth pressure and loads are higher than those based on Rankine/Coulomb earth pressure theory.

A detailed description of recommended design and analysis calculations for geosynthetics reinforced soil walls can be found in guidelines produced by the Federal Highways Administration (FHWA 2001) and AASHTO (2002) and the Geotechnical Engineering Office of Hong Kong (Geoguide 6 2002). Guidance for the analysis, design, construction and specification of modular concrete block (segmental) walls can be found in NCMA (National Concrete Masonry Association 1996). Computer-based codes are available from many suppliers of reinforcement materials and facing components that implement current North American design methods. A generic program for the design and analysis of modular block retaining walls is available from the National Concrete Masonry Association. An empirical-based working stress design method has also been proposed by Allen et al. (2003). A limit-state design approach to the design of these structures can be found in BS 8006 (1995) and Geoguide 6 (2002).

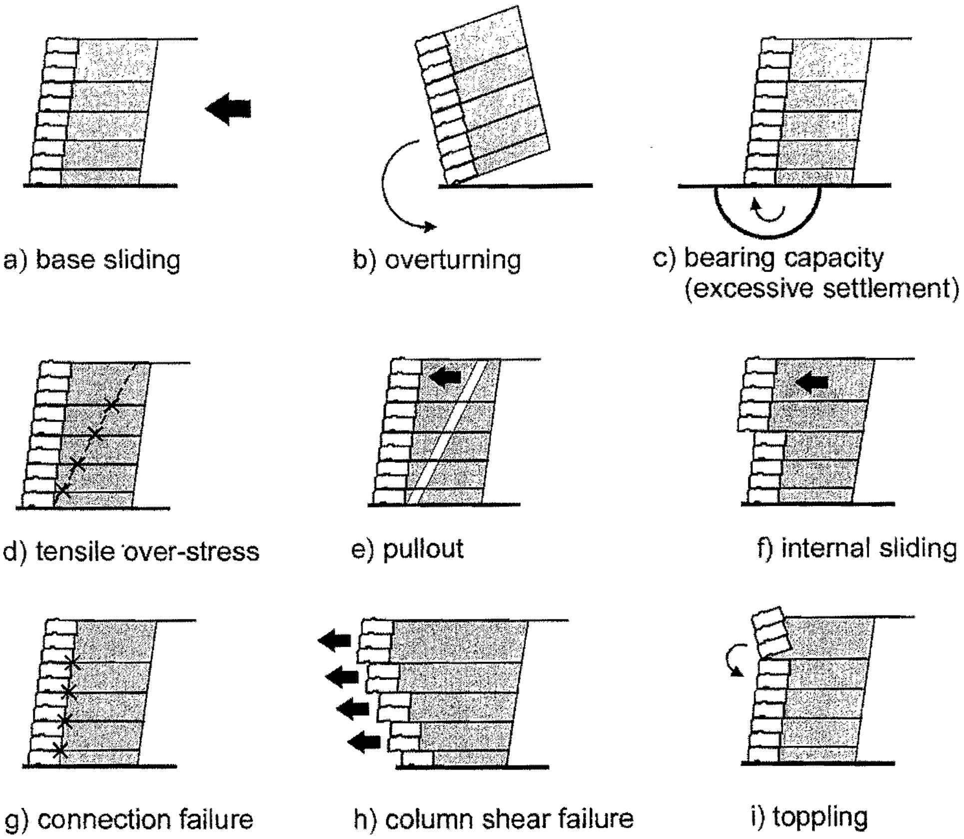


FIGURE 27.2 *Modes of failure for reinforced soil walls: a), b), c) External; d), e), f) Internal; g), h), i) Facing*

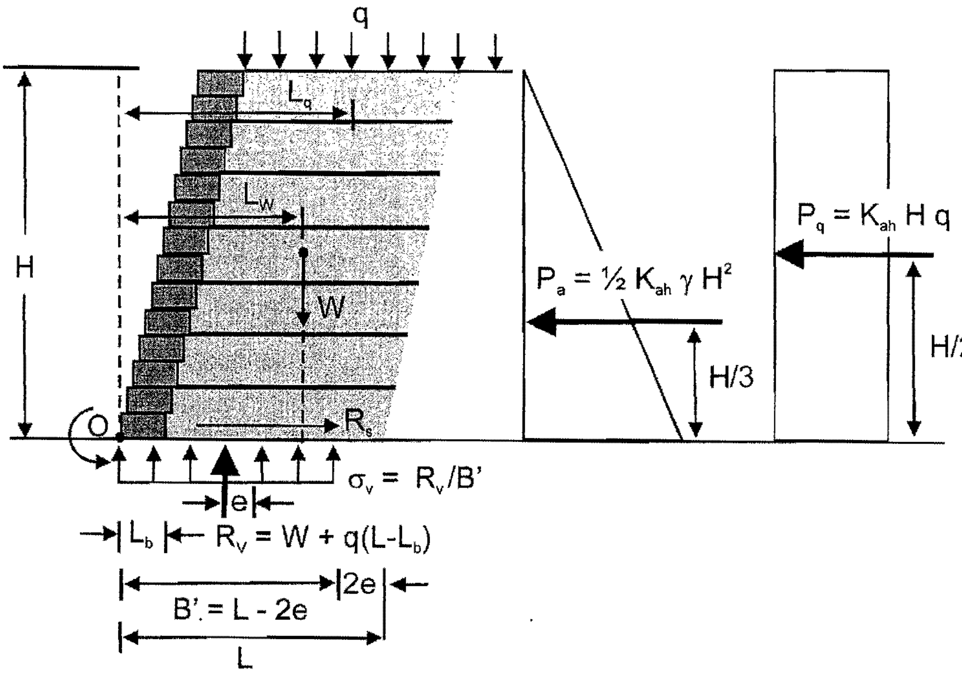
27.3.3 External, Internal, Facing and Global Stability

Analysis and design calculations for reinforced soil walls are related to external, internal and facing modes of failure (Figure 27.2), and to global instability. Global modes of failure refer to failure mechanisms that pass beyond the composite reinforced soil structure. In other words, the structure must not be located so that it is part of a larger global instability. These analyses are routinely handled using conventional slope stability methods of analysis.

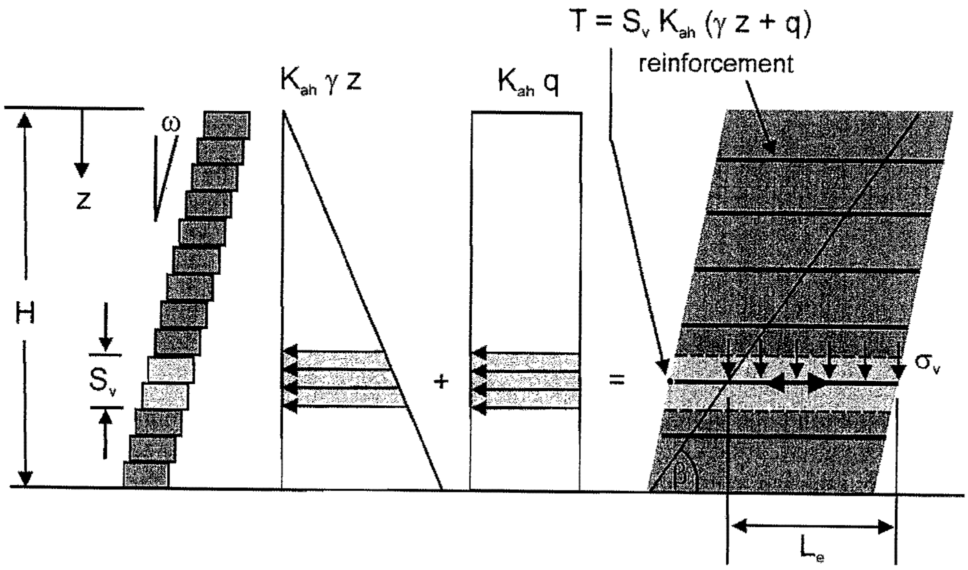
27.3.3.1 External stability

The volume of retained soil that is reinforced by horizontal layers of geosynthetic and the facing column can be imagined to act as a monolithic block of material. This homogenization of the reinforced zone for modular facing systems is assured by keeping the spacing between layers to not more than twice the width of the facing units. Regardless of the facing type, the reinforcement spacing should not exceed 1 m.

The composite mass must be stable against sliding along the base of the structure at the foundation/reinforced soil interface, overturning about the toe and bearing capacity failure of the supporting foundation soils (Figure 27.2 a, b, c). The geometry and forces used in external stability analyses are illustrated in Figure 27.3 for a modular block wall with simple geometry and cohesionless uniform backfill soils. Parameter K_{ah} refers to the horizontal component of the active earth pressure when using Coulomb earth pressure theory. Typical ratios of length of reinforcement zone to height of wall L/H are 0.5 to 0.7.



a) external



b) internal

FIGURE 27.3 *Free body diagrams for external and internal stability calculations*

The factor-of-safety against base sliding can be expressed as:

$$FS = \frac{R_s}{P_a + P_q} = \frac{(W + q(L - L_b))\mu}{P_a + P_q} \geq 1.5 \tag{27.1}$$

where μ is the friction coefficient at the base of the reinforced soil mass and q is any permanent uniformly distributed dead load surcharge. Live loads that contribute to the resistance terms in any stability calculations are commonly neglected. Note that W includes the weight of the reinforced soil zone and the facing column.

The factor-of-safety against overturning about the toe can be expressed as:

$$FS = \frac{W \times L_w + q(L - L_b) \times L_q}{P_a \times \frac{H}{3} + P_q \times \frac{H}{2}} \geq 2 \tag{27.2}$$

where L_w and L_q are moment arms associated with the total weight of the composite reinforced soil mass and the total surcharge load, respectively.

Bearing capacity calculations assume that the base of the reinforced zone acts as an eccentrically loaded footing with an equivalent footing width of L − 2e where e is base eccentricity (i.e. the conventional Meyerhof approach). The factor-of-safety against bearing capacity failure for a c-ϕ foundation soil can be expressed as:

$$FS = \frac{cN_c + \frac{1}{2}\gamma B' N_\gamma}{R_v / B'} \geq 2 \tag{27.3}$$

The designer must also ensure that the settlements occurring at the foundation surface are also within tolerable limits for the structure. In general, the longer the base reinforcement length, the greater the factor-of-safety against external modes of failure.

27.3.3.2 Internal Stability

Potential internal failure mechanisms are illustrated in Figure 27.2 d, e, f.

The recommended method of analysis to determine the arrangement and number of reinforcement layers is based on the limit equilibrium "tie-back wedge method" (AASHTO 2002). In this approach an internal active failure plane propagating from the heel of the wall face at an angle of $\beta = 45 + \phi/2$ degrees to the horizontal (i.e. an active wedge) is assumed to develop where ϕ is the peak friction angle of the reinforced soil. Alternatively, β can be calculated from Coulomb wedge theory. The essential features of this method of analysis are summarized in Figure 27.3b.

The design tensile load T in each layer of reinforcement based on the contributory area S_v around each reinforcement layer must not exceed the design tensile strength T_{des} of the reinforcement (refer to Section 23.4 in Chapter 23). Hence for a reinforcement layer located at depth z:

$$T = S_v K_{ah} (\gamma z + q) \leq T_{des} \tag{27.4}$$

The calculation of the pullout capacity of these materials is dependent on the geometry of the reinforcement and reinforcement-soil interaction. For example; geogrid inclusions, which allow the soil to penetrate through the grid openings, develop a portion of their pullout capacity from passive resistance of the transverse grid members against the soil. Geotextiles that are essentially continuous sheets derive most of their pullout capacity from friction between the geotextile and the surrounding soil. Coated polyester straps have also been used as soil reinforcement. The factor-of-safety expression for pullout of continuous geosynthetic sheets can be expressed as (NCMA 1996):

$$FS = \frac{2\,L_e C_i \sigma_v \tan\phi}{S_v K_{ah}(\gamma z + q)} \geq 1.5 \tag{27.5}$$

Here, L_e is the anchorage length between the point of intersection of the internal failure plane and the free end of the reinforcement and C_i is the coefficient of soil-geosynthetic interaction, which can be determined using a laboratory test procedure (ASTM D6706). To ensure adequate pullout capacity, L_e should not be less than 1 m.

Because of the discrete facing construction of modular block walls, internal sliding mechanisms as illustrated in Figure 27.2f should be assessed. The calculation of factor-of-safety against internal sliding at each interface can be carried out in a similar manner to Equation 27.1, but using the horizontal forces and mass of soil (and surcharge) located above the sliding surface. In this calculation, the interface shear resistance between the block units is required which can be determined using the test protocol described in NCMA (1996) or ASTM D6916. The coefficient of direct sliding between the soil and geosynthetic ($\mu = \mu_{sg}$) can be determined using ASTM D5321. Internal sliding does not apply to full-height concrete panel walls. The factor-of-safety against internal sliding should not be less than 1.5 (NCMA 1996).

Modified slope stability methods described in Section 23.9.1.1 (Chapter 23) are also used when one or more reinforcement layers are intersected by a critical failure surface that passes through both the reinforced and retained soil zones or into the foundation soils, or a critical failure mechanism develops between or along reinforcement layers.

27.3.3.3 Facing Stability

The connection capacity between the reinforcement layers and the facing system is often a critical design issue, particularly for modular block walls (Figure 27.2g). The connection capacity must be at least 1.5 times the design tensile load T calculated using Equation 27.4. A test protocol for connection capacity for modular block systems is described in ASTM D6638. Possible local shear failure between the facing units (Figure 27.2h) and toppling failure (Figure 27.2i) of the unreinforced facing column near the top of the wall must also be considered in the design of these systems. Details of these calculations and recommended factors-of-safety can be found in NCMA (1996) and Bathurst and Simac (1993, 1994, 1997).

27.3.4 Wall Deformations

Geosynthetic reinforced soil walls can be expected to develop outward movements due largely to the extensible properties of the geosynthetic reinforcement, soil creep and facing construction. The outward movement is beneficial (if performance criteria are not exceeded) since lateral earth pressures can be reduced to minimum values consistent with the notion of an active earth pressure condition. However, the mechanical behaviour of these structures is complex and no simple analytical models are available to predict wall deformations in advance. A review of a large number of wall case studies by Bathurst et al. (2002) showed that for typical structures, maximum post-construction movements are in the range of 0.4 to 0.7 % of the height of the wall. Estimated post-construction movements can also be made using recommendations found in AASHTO (2002). Geosynthetic-reinforced walls are often constructed with a small facing batter for aesthetic reasons and to accommodate outward wall deformations.

As discussed in Section 27.1.2, high modulus (stiff) reinforcement, such as steel, allows very little strain of the soil to occur, but results in higher earth pressures and loads than that developed for geosynthetic (extensible) reinforcement. The magnitude and pattern of wall deformation for soil walls with inextensible reinforcement can be provided by the designers and suppliers of steel (inextensible) reinforced soil walls.

27.3.5 Seismic Design

The analytical approach for design and analysis of retaining walls under static loading conditions can be modified to consider additional seismic-induced loads using pseudo-static methods. These methods are based on Mononobe-Okabe theory to calculate destabilizing earth forces. An overview of seismic design and analysis methods is given by Bathurst and Alfaro (1996) and an updated version of this paper appears in the book edited by Shukla (2002). Guidance for seismic analyses can be found in AASHTO (2002), Holtz et al. (1997) and FHWA (2001). For modular block wall systems, the reader is directed to Bathurst (1998). As a general rule, for peak horizontal ground accelerations less than 0.3g, pseudo-static design methods can be used. For higher peak ground accelerations, Newmark sliding block methods of analysis are recommended (e.g. Cai and Bathurst 1996).

References

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. Proceedings 6th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Montreal, Vol. 1, pp. 141-145.

Aas, G., Lacasse, S., Lunne, I., and Hoeg, K. (986). Use of In Situ Tests for Foundation Design in Clay. Proceedings, In Situ '86, American Society of Civil Engineers, pp. 1-30.

AASHTO (1986). Guide for Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO (1997). AASHTO Standard Specifications for Highway Bridges. 16th Ed. (1996 with 1997 interims), American Association of State Highway and Transportation Officials, Washington, DC, USA.

AASHTO (1998). LRFD Bridge Design specifications. American Association of State Highway and Transportation Officials, Second Edition, Washington DC, USA.

AASHTO (2002). Standard Specifications for Highway Bridges. American Association of State Highway and Transportation Officials, Seventeenth Edition, Washington, D.C., USA.

AASHTO (2003). The Materials Book - Standard Specifications for Transportation Materials and Methods of Sampling and Testing. 23rd Edition, American Association of State Highway and Transportation Officials, Washington, D.C., USA.

AASHTO-AGC-ARTBA Joint Committee (1991). Design Guidelines for Use of Extensible Reinforcements (Geosynthetic) for Mechanically Stabilized Earth Walls in Permanent Applications. prepared by Task Force 27. American Association of State Highway and Transportation Officials, Washington, D.C., USA.

AASHTO-AGS-ARTBA (1989). Joint Committee Interim Specifications'. American Association of State Highway and Transportation Officials, Washington, D.C., USA.

Adams J., and Halchuk S. (2004). Fourth generation seismic hazard maps for the 2005 National Building Code of Canada. In Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada. Paper 2502 on CD-ROM. 1-6 August 2004,Vancouver.

Aldrich, H.P. (1956). Frost Penetration Below Highway And Airfield Pavements. U.S. Highway Research Board, Bull. 135, pp. 124-149.

Allen, D.E. (1975). Limit States Design – A Probabilistic Study. Canadian Journal of Civil Engineering. Vol. 2, pp. 36-49.

Allen, T.M. and Bathurst, R.J. (2002). Observed Long-Term Performance Of Geosynthetic Walls, And Implications For Design. Geosynthetics International, Vol. 9, Nos. 5-6, pp. 567-606.

Allen, T.M., Bathurst, R.J., Lee, W. F., Holtz, R. D. and Walters, D.L. (2003). A new working stress method for prediction of reinforcement loads in geosynthetic walls. Canadian Geotechnical Journal, Vol. 40, pp. 976-994.

Altaee, A., Fellenius, B. H., and Evgin, E. (1993). Load Transfer for Piles in Sand and the Critical Depth. Canadian Geotechnical Journal, 30(3): 455-463.

American Concrete Institute Committee 543. Recommendations for Design, Manufacture, and Installation of Concrete Piles.

Andersland, O.B. and Anderson, D.M. (1978) (Editors). Geotechnical Engineering for Cold Regions. McGraw-Hill Book Company, New York, 566 p.

Andersland, O.B. and Ladanyi, B. (2004). Frozen Ground Engineering Second Edition. The American Society of Civil Engineers (ASCE) and John Wiley & Sons, Inc. Published by John Wiley & Sons, Inc. Hoboken, New

Jersey.

Anderson, L.G. (1968). A Modern Approach to Overburden Drilling. Western Miner, November.

Anderson, W.F. (1979). Discussion: Design parameters for stiff clays. Proceedings 7th European Conference on Soil Mechanics and Foundation Engineering, Brighton, Vol. 4, pp. 149-152.

Arman, A. (1978). Current Practices in the Treatment of Soft Foundations, Soil Improvement, History, Capabilities, and Outlook, report by Committee on Placement and Improvement of Soils, Geotechnical Engineering Division, ASCE, pp. 30-51.

Arya, S., O'Neil, M., and Pincus, G. (1979). Design of Structures and Foundations for Vibrating Machines. Gulf Publishing Co., Houston, Texas.

ASCE (1976). Subsurface Investigation for Design and Construction of Foundations of Buildings American Society of Civil Engineers (ASCE), Manuals and Reports on Engineering practice, No. 56, 61 pages.

ASCE. (1996). Rock Foundations. Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 16, New York: ASCE Press, 129 pages.

ASCE (1997). Guidelines of Engineering Practice for Braced and Tied Back Excavations. Geotechnical Special Publication No. 74, ASCE.

Ashour, M., and Norris, G. (2000). Modeling lateral soil-pile response based on soil-pile interaction. Journal of Geotechnical and Geoenvironmental Engineering, 126(5): 420-428.

Ashour, M., Norris, G., and Pilling, P. (1998). Lateral loading of a pile in layered soil using the strain wedge model. Journal of Geotechnical and Geoenvironmental Engineering, 124(4): 303-315.

ASTM (1988). ASTM Standards on Geotextiles. Philadelphia, PA 19103, USA.

ASTM D 2435-03 Standard Test Method for One-Dimensional Consolidation Properties of Soils. ASTM International.

ASTM D 4428 / D 4428M-00 Standard Test Methods for Crosshole Seismic Testing. ASTM International.

ASTM D1194-72 (1987). Standard Test Method for Bearing Capacity of Soil for Static Load and Spread Footings. (Reapproved 1987). Annual book of ASTM Standards, 1991, Vol. 04.08, p. 200-202, Philadelphia, PA.

ASTM D1586-84 (1984). Standard Method for Penetration Test and Split-Barrel Sampling of Soils. (Reapproved 1984). Annual book of ASTM Standards, 1991, Vol. 04.08, pp. 232-236, Philadelphia, PA.

ASTM D2573-72 (1978). Standard Test Method for Field Vane Shear Test in Cohesive Soil; (Reapproved 1978). Annual book of ASTM Standards, 1991, Vol. 04.08, pp. 330-332.

ASTM D2664-86. Test method for triaxial compressive strength of undrained rock core specimens without pore pressure measurements. American Society for Testing and Materials, West Conshohocken, PA.

ASTM D2936-84. Test method for unconfined compressive strength of intact rock core specimens. American Society for Testing and Materials, West Conshohocken, PA.

ASTM D3441-86 (1986). Standard Test Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil. (Reapproved 1986). Annual book of ASTM Standards, 1991, Vol. 04.08, pp. 439-444, Philadelphia, PA.

ASTM D4633-86 (1986). Standard Test Method for Stress Wave Energy Measurement for Dynamic Pentrometer Testing Systems. American Society of Testing and Materials, Philadelphia, PA.

ASTM D4945-89. Standard Test Method for High-Strain Dynamic Testing of Piles. American Society of Testing and Materials, Philadelphia, PA.

ASTM Standard D422-63 (1998). Standard test method for particle-size analysis of soils. American Society for Testing and Materials, Philadelphia, Pa.

ASTM Standard D4546-90 (1996). Standard test methods for one-dimensional swell or settlement potential of cohesive soils. American Society for Testing and Materials, Philadelphia, Pa.

ASTM STP 1014 (1988). Vane Shear Strength Testing in Soils: Field and Laboratory Studies. American Society of Testing and Materials, Philadelphia, PA.

Aubery, D. and Chapel, F. (1981). 3-D dynamic analysis of groups of piles and comparisons with experiments. Transactions, 6th International Conference on Structural Mechanics in Reactor Technology, Paris, 9 p.

Authier, J. and Fellenius, B .H. (1983). Wave Equation Analysis and Dynamic Monitoring of Pile Driving. Civil Engineering for Practising and Design Engineers, Pergamon Press, Vol. 2, No. 4, pp. 387-407.

Baguelin, F., Jézéquel, J.F., LeMee, E., and LeMelhaute, A. (1972). Expansion of Cylindrical Probes in Cohesive Soils. ASCE Journal of Soil Mechanics and Foundations Division, 98, (SM11), pp. 1129-1142.

Baguelin, F., Jézéquel, J.F., and Shields, D.H. (1978). The Pressuremeter and Foundation Engineering. Trans Tech publications, 617 p.

Baldi, G. Bellotti, R., Ghionna, V.N., Jamiolkowski, M., and Lo Presti, D.C.F. (1989}. Modulus of sands from CPT's and DMT's. In Proceedings XII International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Rio de Janeiro.

Barkan, D. D. (1962). Dynamics of Bases and Foundations (translated from Russian). McGraw-Hill Book Co., Inc., New York, N.Y.

Barron, R.A. (1948). Consolidation of Fine-Grained Soils by Drain Wells. American Society of Civil Engineers (ASCE) Transactions, Vol. 113, pp. 718-742.

Barsvary, A.K., Klym, T.W., and Franklin, J.A. (1980). List of Terms, Symbols, and Recommended Si-Units and Multiples for Geotechnical Engineering. Canadian Geotechnical Journal, Vol. 17, No. 1, pp. 89-96.

Barton, N.R. (1973). Review of a new shear strength criterion for rock joints. Engineering Geology, 7, 287–332.

Barton, N.R., and Choubey, V. (1977). The shear strength of rock joints in theory and practice. Rock Mech., 10, 1–54.

Barton, N.R., Lien, R., and Lunde, J. (1974). Engineering classification of rockmasses for the design of tunnel support. Rock Mech., May, 189 – 236.

Bates, R.L. and Jackson, J.A. (1980). Glossary of Geology. American Geological Institute, Falls Church, Virginia, 749 p.

Bathurst, R.J. (1998). National Concrete Masonry Association (NCMA) Segmental Retaining Wall Seismic Design Procedure. Supplement to Design Manual for Segmental Retaining Walls (Second Edition 1997) published by the National Concrete Masonry Association (NCMA), Herndon, VA, USA.

Bathurst, R.J. and Alfaro, M.C. (1996). Review of seismic design, analysis and performance of geosynthetic reinforced walls, slopes and embankments. Invited keynote paper, IS Kyushu'96, 3'rd International Symposium on Earth Reinforcement, Fukuoka, Kyushu, Japan, 12-14 November 1996, Balkema, Vol. 2, pp. 887 918.

Bathurst, R.J. and Raymond, G.P. (1987). Geogrid Reinforcement of Ballasted Track. Transportation Research Board, Transportation Research Record 1153. pp. 8-14.

Bathurst, R.J. and Simac, M.R. (1994). Geosynthetic reinforced segmental retaining wall structures in North America. Keynote paper, 5th International Conference on Geotextiles, Geomembranes and Related Products, 6-9 September 1994, Singapore, Vol. 4, pp. 1275-1298.

Bathurst, R.J. and Simac, M.R. (1997). Design and performance of the facing column for geosynthetic reinforced segmental retaining walls. International Symposium on Mechanically Stabilized Backfill, Denver Colorado, 6-8 February 1997, Balkema, J. Wu (ed), pp. 193-208.

Bathurst, R.J., Allen, T.M. and Walters, D.L. (2002). Short-term strain and deformation behavior of geosynthetic walls at working stress conditions. Geosynthetics International, Vol. 9, Nos. 5-6, pp. 451-482.

Bathurst, R.J., Simac, M.R. and Berg, R.R. (1993). Review of the National Concrete Masonry Association (NCMA) segmental retaining wall design manual for geosynthetic-reinforced structures. Transportation Research Record 1414, pp. 16-25.

Becker, D.E. (1996a). Limit states design for foundation Part 1. An overview of the foundation design process. Canadian Geotechnical Journal, 33(6), pp. 956-983.

Becker, D.E. (1996b). Limit states design for foundation. Part II. Development for National Building Code of Canada. Canadian Geotechnical Journal, 33(6), pp. 984-1007.

Becker, D.E. (2001). Site Characterization. Geotechnical and Geoenvironmental Handbook. R.K. Rowe, Ed., Chapter 4, Kluwer Academic Publishers, pp. 69-105.

Becker, D.E. (2003). Limit States Foundation Design Code Development in Canada. Proceedings of LSD2003. International Workshop on Limit State Design in Geotechnical Engineering Practice - Phoon, Honjo and Gilbert (Editors). Cambridge, Massachusetts, USA. June 26, 2003. World Scientific Publishing Company.

Becker, D.E., Crooks, J.H.A., Been, K., and Jefferies, M.G. (1987). Work as a criterion for determining in situ and yield stresses in clays. Canadian Geotechnical Journal, 24: 549-564.

Becker, D.E., Crooks, J.H.A., Been, K. (1988). Interpretation of the Field Vane Test in Terms of In-Situ and Yield Stresses. In Vane Shear Strength Testing in Soils: Field and Laboratory Studies. A.F. Richards Editor. American Society for Testing and Materials ASTM STP 1014. Philadelphia. pp. 71-87.

Begemann, H.K. (1965). The Friction Jacket Cone as an Aid in Determining the Soil Profile. Proceedings 6th

International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Montreal, Vol. 1, pp. 17-20.

Bell, A.L. (1915). The lateral pressure and resistance of clay and the supporting power of clay foundations. In A Century of Soil Mechanics (ICE), London, 93 – 134.

Bell, F.G. (1993). Engineering Treatment of Soils. Published by E&FN Spon (Chapman & Hall), London, England.

Berardi, R., and Lancellotta, R. (1991). Stiffness of granular soil from field performance. Géotechnique, 41(1): 149–157.

Beredugo, Y. O. and Novak, M. (1972). Coupled horizontal and rocking vibration of embedded footings. Canadian Geotechnical Journal, 9 (4), 477-497.

Berezantsev, V.G., Kristoforov, V.S. and Golubkov, V.N. (1961). Load Bearing Capacity and Deformation of Pile Foundations. Proceedings 5th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Paris, Vol. 2, pp. 1145.

Bertram, G.E. (1940). An experimental investigation of protective filters. Graduate School of Engineering, Harvard University, Cambridge, MA. Soil Mechanics Series No. 7:1-21.

Bhatia, S.K., Smith, J.L. and Christopher, B.R. (1996). Geotextile characterization and pore-size distribution: Part III. Comparison of methods and application to design, Geosynthetics International. Vol. 3, No. 3, pp. 301-328.

Bhushan K., and Haley, S. C. (1980).Development of Computer Program Using P-Y Data from Load Test Results for Lateral Load Design of Drilled Piers a research report prepared for Woodward-Clyde Consultants Professional Development Committee, San Francisco, California.

Bhushan, K., and Askari, S. (1984). Lateral Load Tests on Drilled Pier Foundations for Solar Plant Heliostats Laterally Loaded Piles, ASTM STP 835, James A. Langer, Ed., American Society of Testing and Materials, pp. 141-155.

Bhushan, K., Haley, S. C., and Fong, P. T. (1979). Lateral Load Tests on Drilled Piers in Stiff Clays Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT8, Proc. Paper 14789, pp. 969-985.

Bhushan, K., Lee, L. J., and Grime, D. B. (1981). Lateral Load Tests on Drilled Piers in Sand Proceedings of a Session on Drilled Piers and Caissons, sponsored by the Geotechnical Engineering Division at the ASCE National Convention, St. Louis, Missouri, pp. 131-143.

Bieniawski, Z.T. (1973). Engineering classification of jointed rock masses. Trans. South African Institute Civil Eng., 15(12), 335–344.

Bieniawski, Z.T. (1976). Rock Mass Classifications in Rock Engineering. Proceedings Symposium on Exploration for Rock Engineering, Cape Town, Balkema, pp. 76-106.

Bieniawski, Z.T. (1979). The Geomechanics Classification in rock engineering classifications. Proc. 4th Int. Congr. On Rock Mech., ISRM, Montreux, Rotterdam: A.A. Balkema, 2, 41–48.

Bieniawski, Z.T. (1989). Engineering rock mass classifications. New York: Wiley.

Binquet, J. and Lee, K.L. (1975). Bearing Capacity Analysis of Reinforced Earth Slabs. J. Geot. Div., ASCE, 101 (12), pp. 1257-1276.

Biot, M.A. (1941). General theory of three-dimensional consolidation. Journal of Applied Physics. 12: 155.

Bishop, A.W. and Morgenstern, N.R. (1960). Stability Coefficients for Earth Slopes. Geotechnique, Vol. 10, No. 1, pp. 129-150.

Bjerrum, L. (1963). Allowable Settlement of Structures. Proceedings 3rd European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Vol. 2, pp. 135-137.

Bjerrum, L. (1967). Norwegian Experience with Steel Piles to Rock. Geotechnique, Vol. 7, No. 2, pp. 73-96.

Bjerrum, L. (1972) Embankments on Soft Ground, Proceedings of the ASCE Specialty Conference on Earth and Earth-Supported Structures, Purdue University, Vol. 2, pp. 1-54.

Bjerrum, L. (1973). Problem of Soil Mechanics and Construction on Soft Clays. Proceedings 8th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Moscow, State of the Art Report, Vol. 3, pp. 111-159.

Bjerrum, L., Frimann-Clausen, C.J., Duncan, J.M. (1972). Earth pressures on flexible structures – a state-of-the-art report. Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, Vol. 2, pp. 169 – 196.

Bjerrum, L., Johannesson, I.J. and Eide, O. (1969). Reduction of Skin Friction on Steel Piles to Rock. Proc. 7th Int.

Conf. Soil Mechanics and Foundation Engineering, Vol. 2, pp. 27-34.

Bjerrum, L., Moum, J., and Eide, O. (1967) Application of Electro-Osmosis on a Foundation Problem in a Norwegian Quick Clay. Geotechnique, Vol. 17, No. 3, pp. 214-235.

Black, W.P.M. (1962). A Method of Estimating the California Bearing Ratio of Cohesive Soils from Plasticity Data. Geotechnique, Vol. 12, pp. 271-282.

Blanchet, R., Tavenas, F.A. and Garneau, R. (1980). Behaviour of Friction Piles in Soft Sensitive Clays. Canadian Geotechnical Journal, Vol. 17, No. 2, pp. 203-224.

Blaney, G. W., Kausel, E. and Roesset, J. M. (1976). Dynamic stiffness of piles. 2nd International Conference on Numerical Methods in Geomechanics. ASCE, New York, N.Y., 2, 1001-1012.

Bonaparte, R. and Christopher, B.R. (1987). Design and construction of reinforced embankments over weak foundations. Transportation Research Record 1153, pp. 26-39.

Bond, D.W. (1961). Influence of Foundation Size on Settlement. Geotechnique, Vol. 11, No.2, pp. 121-143.

Boone, S.J. (1996). Ground-movement-related building damage. Journal of Geotechnical Engineering. 122(11): 886-896.

Boone, S.J. and Crawford, A.M. (2000). Braced Excavations: Temperature, Elastic Modulus, and Strut Loads. Journal of Geotechnical and Geoenvironmental Engineering. ASCE, 126(10), 870 – 881.

Boone, S.J., Bidhendi, H., Westland, J., and Grabinsky, M. (1999). Rationalizing the practice of strut preloading for braced excavations. Geo-Engineering for Underground Facilities, Geotechnical Special Publication No. 90, ASCE, pp. 393 - 404.

Boulanger, R. W., Meyers, M. W., Mejia, L. H., Idriss, I. M. (1998). Behavior of a fine-grained soil during the Loma Prieta earthquake. Canadian Geotechnical Journal 35(1): pp. 146-158.

Boulanger, R.W., and Idriss, I.M. (2004). Evaluation of potential for liquefaction or cyclic failure of silts and clay. Report No. UCD/CGM-04/01, Department of Civil & Environmental Engineering, University of California at Davis.

Bowles, J.E. (1988). Foundation Analyses and Design. 4th Edition, McGraw-Hill International Editions, New York, USA.

Bowles, J.E. (1996). Foundation Analysis and Design. 5th Edition, McGraw-Hill, New York.

Boyd, D.W. (1973). Normal Freezing and Thawing Degree-Days for Canada 1931-1960. Environment Canada, Atmospheric Environment, Report CL14-73.

Bozozuk, M. (1981). Bearing Capacity of a Pile Preloaded by Down Drag. Proceedings 10th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE) , Stockholm, Vol. 2, pp. 631-636.

Bozozuk, M. and Burn, K.N. (1960). Vertical Ground Movements Near Elm Trees. Geotechnique, Vol. 10, No. 1, pp. 19-32.

Bozozuk, M., Fellenius, B.H. and Samson, L. (1978a). Soil Disturbance from Pile Driving in Sensitive Clay. Canadian Geotechnical Journal, Vol. 15, No. 3, pp. 346-361.

Bozozuk, M., Keenan, G.H. and Pheeney, P.E. (1978b). Analysis of Load Tests on Instrumented Steel Test Piles in Compressible Silty Soil ASTM Symposium on the Behaviour of Deep Foundations, R. Lundgren, Editor, Special Technical Publication, SPT 670, 1979, pp. 153-180.

Bray, J.D. and Merry, S.M. (1999). A comparison of the response of geosynthetics in the multi-axial and uniaxial test device. Geosynthetics International, Vol. 6, No. 1, pp. 19-40.

Bray, J.D., Sancio, R.B., Riemer, M.F. and Durgunoglu, T. (2004). Liquefaction susceptibility of fine-grained soils. In Proceedings of the 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd International Conference on Earthquake Geotechnical Engineering, Berkeley, CA, Jan. 7-9, pp. 655-662.

Briaud, J.L. Smith, T.O. and Meyer, B.J. (1983). Laterally Loaded Piles and the Pressuremeter: Comparison of Existing Methods, ASTM Special Technical Publication 835 - Design and Performance of Laterally Loaded Piles and Pile Groups.

Briaud, J.L., and Tucker, L.M. (1988). Measured and Predicted Axial Response of 98 Piles, ASCE Geot. Journal, Vol. 114, No. 9, Sept., pp. 984-1001.

Briaud, J.L., Tucker, L.M. (1997). Design and construction guidelines for downdrag on uncoated and bitumen-coated piles. NCHRP Report 393, Transportation Research Board, National Academy Press, Washigton DC.

Brinch-Hansen, J. (1956). Limit Design and Partial Safety Factors in Soil Mechanics. Danish Geotechnical Institute, Copenhagen, Bulletin No. 1, 4 p.

Brinch-Hansen, J. (1961). The Ultimate Resistance of Rigid Piles against Transversal Forces. Geoteknisk Instit., Bull. No. 12, Copenhagen.

Brinch-Hansen, J. (1970). A Revised and Extended Formula for Bearing Capacity. Danish Geotechnical Institute, Bulletin No. 28, Copenhagen, pp. 5-11.

British Standards Institution Code of Practice, BS 5930 (1981). Code of Practice for Site Investigations.

Broch, E. and Franklin, J.A. (1972). The Point-Load Strength Test. International Journal of Rock Mechanics and Mining Science, Vol. 9, pp. 669-697.

Broms, B.B. (1964a). Lateral Resistance of Piles in Cohesive Soils. Journal for Soil Mech. and Found. Engrg., ASCE, Vol. 90, SM2, pp. 27-64.

Broms, B.B. (1964b). Lateral Resistance of Piles in Cohesionless Soils. Journal for Soil Mech. and Found. Engrg., ASCE, Vol. 90, SM3, pp. 123-156.

Broms, B. B. (1979) Problems and Solutions to Construction in Soft Clay, Proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Singapore, Guest Lecture, Vol. II, pp. 3-38.

Broms, B.B. and Boman, P. (1977). Lime Columns — A New Type of Vertical Drain. Proceedings 9th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Vol. 1, pp. 427-432.

Broms, B.B. and Boman, P. (1979). Lime Columns — A New Foundation Method. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 105, GT4, pp. 539-556.

Brown, D. A. and Bollman, H. (1996). Pile group design using COM 624. Conference on The Design of Bridges for Extreme Events, U.S. Department of Transportation, Federal Highway Administration.

Brown, D. A., Morrison, C., and Reese, L. C. (1988). Lateral load behavior of pile group in sand Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 11, pp. 1326-1343.

Brown, D. A., O'Neill, M.W., Hoit, M., El Naggar, M.H., and Chakiaberty, S. (2001). Static and dynamic lateral loading of pile groups. Research report prepared for National Cooperative Highway Research Program, Highway Research Center, Auburn University, Alabama.

Brown, E.T. (1981). Rock Characterization, Testing, and Monitoring. ISRM Suggested Methods, Pergamon Press, 211 p.

Brown, Roger J.E. (1970). Permafrost in Canada. It's Influence on Northern Development. University of Toronto Press.

Brzezinski, L. S., Shector, L., Macphi, H. L. and Vander Noot, H. J. (1973). An Experience with Heave of Cast In Situ Expanded Base Piles. Canadian Geotechnical Journal, Vol. 10, No. 2, pp. 246-260.

BS8006 (1995). Code of practice for strengthened/reinforced soils and other fills. British Standards Institution, London, UK.

Budhu, M., and Davies, T. G. (1987). Nonlinear analysis of laterally loaded piles in cohesionless soils. Canadian Geotechnical Journal, 24: 289-296.

Budhu, M., and Davies, T. G. (1988). Analysis of laterally loaded piles in soft clay. Journal of Geotechnical Engineering. ASCE, 114(1): 21-39.

Building Research Advisory Board (BRAB) (1968), Criteria for Selection and Design of Residential Slabs-on-grade. Publication 1571, National Academy of Sciences Report No. 33 to Federal Housing Administration, NTIS, No. PB-261 551.

Bullen, K. E. and Bolt, B. A. (1985). An Introduction to The Theory of Seismology. Cambridge University Press, Cambridge.

Burbidge, M.C. (1982). A Case Study Review of Settlements on Granular Soils. M.Sc. Dissertation, Imperial College of Science and Technology, London University.

Burland, J., Broms, B.B. and deMello, V.F.B. (1977). Behaviour of Foundations and Structures. Proceedings, 9th International Conference Soil Mechanics and Foundation Engineering (ICSMFE), Tokyo, Vol. 2, pp. 495-546.

Burland, J.B. (1973). Shaft Friction of Piles in Clay; A Simple Fundamental Approach. Ground Engineering, Foundation Publications Ltd., London, Vol. 6, No. 3, pp. 30-42.

Burland, J.B. (1989). Ninth Laurits Bjerrum Memorial Lecture: Small is Beautiful – the Stiffness of Soils at Small Strains. Canadian Geotechnical Journal, 26(4): 499-516.

Burland, J.B. and Burbidge, M.C. (1985). Settlement of Foundations on Sand and Gravel. Proc. Instn. Civil Engrs. December, 78 (part 1), pp. 1325-1381.

Burland, J.B. and Wroth, C.P. (1974). Settlement of buildings and associated damage. Settlement of Structures, Pentech Press, London pp. 611-654.

Burr, J.P., Pender, M.J and Larkin, T.J. (1997). Dynamic response of laterally excited pile groups. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 1, pp. 1- 8.

Bush, D.L., Jenner, C.G. and Bassett, R.H. (1990). The design and construction of geocell foundation mattress supporting embankments over soft ground. Geotextiles and Geomembranes, Vol. 9, pp. 83-98.

Bustamente, M. and Gianeselli, L. (1982). Pile Bearing Capacity Prediction by Means of Static Penetrometer CPT. Proceedings of Second European Symposium on Penetration Testing (ESOPT II), Amsterdam, A.A. Balkema, Vol. 2, pp. 493-500.

Bycroft, G. N. (1956). Forced vibrations of a rigid circular plate on a semi-infinite elastic space and on an elastic stratum. Philosophical Transactions of the Royal Society, London, Ser. A., 248, 327-368.

Cai, Z. and Bathurst, R.J. (1996). Seismic-induced permanent displacement of geosynthetic reinforced segmental retaining walls. Canadian Geotechnical Journal, Vol. 31, pp. 937-955.

Calembert, L. (1973). Engineering Geological Problems in Karstic Regions. Bulletin of the International Association of Engineering Geology, Vol. 12, pp. 93-132.

Callanan, J.F. and Kulhawy, F.H. (1985). Evaluation of Procedures for Predicting Foundation Uplift Movements. Report E1-4107, Electric Power Research Institute, Palo Alto, Calif.

Campanella, R.G. and Robertson, P.K. (1981). Applied Cone Research. American Society of Civil Engineers (ASCE)Symposium on Cone Penetration Testing and Experience, St.Louis, pp. 343-362.

Campanella, R.G. and Robertson, P.K. (1982). State of the Art in In Silt Testing of Soils. Proceedings Engineering Foundation Conference on Updating Subsurface Sampling of Soils and Rocks, and Their In Situ Testing. Santa Barbara, California, 23 p.

Campanella, R.G. and Robertson, P.K. (1991). Use and Interpretation of a Research Dilatometer. Canadian Geotechnical Journal, Vol. 28, No. 1. pp. 113-126.

Canadian Standards Association (CSA). (1992). Codes for Offshore Structures, CAN/CSA S471-92 General Requirements, Design Criteria, the Environment, and Loads. S472-92 Foundations; S471-92 Steel Structures; S474-94 Concrete Structures; and S475-93 Sea Operations. CSA International, Rexdale, Ontario, Canada.

Caquot and Kerisel (1949). Tables for the Calculation of Passive Pressure, Active Pressure and Bearing Capacity of Foundations. Gauthier-Villars, Paris.

Carroll Jr., R.G., Walls, J.C. and Haas, R. (1987). Granular Base Reinforcement of Flexible Pavements Using Geogrids. Geosynthetics 1987. Industrial Fabrics Association International, St. Paul, MN, Vol. 1, pp. 46-57.

Carter, J.P. and Kulhawy, F.H. (1988). Analysis and Design of Drilled Shaft Foundations Socketed into Rock. Report El-5918, Electric Power Research Institute, Palo Alto, Calif.

Carter, J.P. and Kulhawy, F.H. (1992). Analysis of laterally loaded shafts in rocks. Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 6, pp. 839-855.

Analysis of laterally loaded shafts in rocks. Journal of Geotechnical Engineering.

Carter, J.P., Booker, J.R. and Small, J.C. (1979). The analysis of finite elasto-plastic consolidation. International Journal for Numerical and Analytical Methods in Geomechanics. 3: 107-129.

Casagrande, A. (1932). A New Theory on Frost Heaving. Discussion, Highway Research Board, (HRB), Proceedings, No. 11, pp. 168-172.

Casagrande, A. (1948). Classification and Identification of Soils. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Transactions, Vol. 113, pp. 901-930.

Casagrande, A. and Poulos, S. (1969). On the Effectiveness of Sand Drains. Canadian Geotechnical Journal, Vol. 6, No. 3, pp. 287-326.

Casagrande, L. (1937). Method of hardening soils. U.S Patent no. 2,099,328.

Casagrande, L. (1959) Review of Past and Current Work on Electro-Osmotic Stabilization of Soils, Harvard Soil Mechanics Series, No. 45.

Casagrande, L. (1983). Stabilization of soils by means of electro-osmosis - state of the art. Journal of Boston Society of Civil Engineers ASCE, 69 (2): 255-302.

Cedergren, H.R. (1977). Seepage, Drainage, and Flow Nets. Second Edition. John Wiley & Sons, New York.

Cetin, K.O., Seed, R.B., Kiureghian, A.D., Tokimatusu, K., Harder, L.F. Jr., Kayen, R.E., and Moss, R.E.S. (2004). Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 130(12): 1314-1340.

Chandler, R.J. (1984). Recent European experience of landslides in over-consolidated clays and soft rocks. Proc. 4th Int. Symp. on Landslides, Toronto, 1: 61-81.

CHBDC – Canadian Standards Association. (2000). Canadian Highway Bridge Design Code (CHBDC). A National Standard of Canada. CAN/CSA Standard S6-00. CSA International, Rexdale, Ontario, 752 p.

Chin, F.K. (1970). Estimation of the Ultimate Load of Piles not carried to Failure. Proceedings, 2nd Southeast Asian Conference on Soil Engineering, Singapore, pp. 81-90.

Christopher et al., editors (1989). Reinforced Soil Structures, Vol. 1 - Design and Construction Guidelines. U.S. Federal Highway Administration.

Christopher, B.R. and Holtz, R.D. (1991). Geotextiles for subgrade stabilization in permanent roads and highways. Geosynthetics'91, Industrial Fabrics Association International, pp. 701-713.

Clark, J. I. 1978. Failure During Construction and Subsequent Rehabilitation and Performance of Dynamically Cast-In-Place Concrete Pile Foundation. ASTM, Symposium on The Behaviour of Deep Foundations, R. Lundgren, Editor, Special Technical Publication (SPT) 670, 1979, pp. 207-230.

Clark, J.I. (1978). Failure During Construction and Subsequent Rehabilitation and Performance of a Dynamically Cast-in-Place Concrete Pile Foundation. American Society for Testing and Material (ASTM) Symposium on the Behaviour of Deep Foundations, R. Lundgren, Editor, Special Technical publication, STP 670, 1979, pp. 207-230.

Clark, J.I. and Meyerhof, G.G. (1972a). The Behaviour of Piles Driven in Clay, Part I: An Investigation of Soil Stress and Porewater Pressure as Related to Soil Properties. Canadian Geotechnical Journal, Vol. 9, pp. 351-373.

Clark, J.I. and Meyerhof, G.G. (1972b). The Behaviour of Piles Driven in Clay, Part II Investigation of the Bearing Capacity of Using Total and Effective Strength Parameters. Canadian Geotechnical Journal, Vol. 10, No. 1, pp. 86-102.

Clough, G.W. and Duncan, J.M. (1991). Earth pressures, Chapter 6, Foundation Engineering Handbook. H.F. Winterkorn and H.Y. Fang, eds., Van Nostrand Reinhold, New York, NY.

Clough, G.W. and O'Rourke, T.D. (1990). Construction induced movements of insitu walls. Design and Performance of Earth Retaining Structures, Geotechnical Special Publication 25, ASCE, pp. 439 - 470.

Clough, G.W. and Tsui, Y. (1974). Performance of Tied Back Walls in Clay. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 100, GT12, pp. 1259-1273.

Clough, G.W., Smith, E.M., and Sweeney, B.P. (1989). Movement control of excavation support systems by iterative design. Foundation Engineering, Current Practices and Principles, Geotechnical Special Publication 22, F.H. Kulhawy, ed., ASCE, Vol. 2, pp.869 – 884.

Clough,G.W. and Denby, G.M. (1977). Stabilizing berm design for temporary walls in clay. Journal of the Geotechnical Engineering Division, ASCE, 103(2), 75 – 90.

Coates, D.F. (1967). Rock Mechanics Principles (Revised Edition). Mines Branch Monograph 874, Ottawa, Canada, 358 p.

Coon, R.F., and Merritt, A.H. (1970). Predicting in situ modulus of deformation using rock quality indexes. ASTM Special Publication 477, ASTM, Philadelphia, 154–173.

Cording, E.J., and Deere, D.U. (1972). Rock tunnel supports and field measurements. Proc. North American Rapid Excavation Tunnelling Conf., Chicago (eds. K.S. Lane and L.A. Garfield), 1, 601–622.

Coulomb, C.A. (1776). Essai sur une Application des Régles des Maximis et Minimis à Quelques Problèmes de Statique Relatifs à L'Architecture, Mem. Acad. Roy. Pres. Div. Sav., Vol. 7, Paris, France.

Coyle, H.M. and Reese, L.C. (1966). Load Transfer for Axially Loaded Piles in Clay. J. Soil Mech. and Foundations Div., ASCE, Vol. 92, SM2, pp. 1-26.

Crawford, C.B. (1963). Interpretation of the Consolidation Test. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 90, SM5, pp. 87-102.

Crawford, C.B. (1965). The Resistance of Soil Structure to Consolidation. Canadian Geotechnical Journal, Vol. 2, pp. 90-97.

Crawford, C.B. (1968). Frost Action Construction Hazard. Engineering Contract Records, Vol. 81, No. 1, pp. 51-57.

Crocker, C.R. (1965). Moisture Problems During Winter Construction Operations. Technical paper 224, National Research Council, Division of Building Research.

Crouse, C.B. and Cheang, L. (1987). Dynamic testing and analysis of pile-group foundation. Geotechnical Special

Publication No. 11, ASCE, pp. 79-98.

D'Appolonia, D.J., D'Appolonia, E. and Brisette, R.F. (1968). Settlements on Spread Footings on Sand. American Society of Civil Engineers (ASCE), Journal of Soil Mechanics and Foundation Engineering, Vol. 94, SM3, pp. 735-760.

D'Appolonia, D.J., Poulos, H.G. and Ladd, C.C. (1971). Initial settlement of structures on clay. JSMFD, ASCE, 97(SM10): 1359-1397.

D'Appolonia, E. (1953). Loose Sands and their Compaction by Vibroflotation. American Society for Testing and Materials, ASTM, Special Technical publication, SW 156, pp. 138-154.

Dahlberg, R. (1974). Penetration, Pressuremeter, and Screw Plate Tests in a Preloaded Natural Sand Deposit. Proceedings of the European Symposium on Penetration Testing, ESOPT I, Stockholm, Vol.2.2.

Danish Code of Practice (1977). Code of Practice for Foundation Engineering. Danish Association of Civil Engineers, Danish Geotechnical Institute, Bulletin No. 32, 49 p.

Danish Geotechnical Society (DGS). (1993). Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, Denmark, May 26-28, 1993. 748 p.

Das, B. M. (1995). Principles of Soil Dynamics. PWS-Kent Publishing Co., Boston, Massachusetts.

Das, Braja, M. (1990). Principles of Foundation Engineering. 2nd Edition, PWS - Kent Publishing Company.

Davies, T. G., Sen, R. and Banerjee, P. K. (1985). Dynamic behavior of pile groups in inhomogeneous soil. Journal of Geotechnical Engineering, ASCE, 111 (12), 1365-1379.

Davis, E.H. and Booker, J.R. (1973). The Effect of Increasing Strength with Depth on the Bearing Capacity of Clays. Geotechnique, Vol. 23, No. 4, pp. 551-563.

Davis, E.H., and Booker, J.R. (1971). The bearing capacity of strip footings for the point of view of plasticity theory. In Proceedings. First Australia-New Zealand Conference on Geomechanics, Institution of Engineers, Melbourne, Australia, Vol. 1, pp. 276-282.

Davis, E.H., and Poulos, H.G. (1968). The use of elastic theory for settlement prediction under three-dimensional conditions. Géotechnique, 18(1): 67-91.

Davis, E.H., and Poulos, H.G. (1972). Rate of settlement under three-dimensional conditions. Géotechnique, 22(1): 95-114.

Davisson, M.T. (1970). Lateral Load Capacity of Piles. Highway Research Board, Washington, D.C., Highway Research Record No. 333, pp. 104-112.

Davisson, M.T. (1972). Inspection of Pile Driving Operations. Technical Report M-22, U.S. Corps of Engineers.

Davisson, M.T. (1973). High Capacity Piles. Proceedings, Lecture Series, Innovations in Foundation Construction, ASCE, Illinois Section, 52 p.

De Mello, V. F. B. (1969). Foundations of building on clay State of the Art Report, Proc. 7th International Conference of Soil Mechanics and Foundation Engineering, Mexico City, Vol. 2, pp. 49-136.

Decourt, L. (1989). The Standard Penetration Test State-of-the-Art Report. Proceedings 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janiero, Brazil.

Decourt, L. (1995). Prediction of load-settlement relationships for foundations on the basis of the SPT-T. Ciclo de Conferencias Internationale, Leonardo Zeevaert, UNAM, Mexico, pp. 85-104.

Deep Foundation Institute (1979a), Inspector's Manual for Deep Foundations. Deep Foundations Institute, New Jersey, 47p.

Deep Foundation Institute (1979b). Inspector's Guide to Hammers. Deep Foundation Institute, Springfield, New Jersey, 47 p.

Deere, D.U. and Deere, D.W. (1988). The rock quality designation (RQD) index in practice. Rock classification systems for engineering purposes, (ed. L. Kirkaldie), ASTM Special Publication 984, 91–101.

Deere, D.U. and Miller, R.P. (1966). Engineering classification and index properties for intact rock. Technical Report No. AFWL-TR-65-116, Albuquerque, NM: Air Force Weapons Laboratory.

Deere, D.U., Hendron, A.J., Patton, F.D. & Cording, E.J. (1967). Design of surface and near surface construction in rock. In Eighth US Symposium on Rock Mechanics, Minneapolis, pp. 237-303.

Delleur, J.W. (1999). The Handbook of Groundwater Engineering. CRC Press, New York, USA

Desai, C.S. and Wu, T.H. (1976). A General Function for Stress-Strain Curves. Proc. 2nd Int. Conf. On Numerical Methods in Gomechanics, ASCE, 1, June, pp. 306-318.

Dobry, R. and Gazetas, G. (1985). Dynamic stiffness and damping of foundations by simple method. Vibration

Problems in Geotechnical Engineering, editors Gazetas, G. and Selig, E. T., ASCE, 77-107.
Dobry, R. and Gazetas, G.(1988). Simple method for dynamic stiffness and damping of floating pile groups. Geotechnique, 38 (4), 557-574.
Douglas, B.J. and Olsen, R.S. (1981). Soil Classification Using Electric Cone Penetrometer. American Society of Civil Engineers (ASCE) Geotechnical Engineering Division, Symposium on Cone penetration Testing and Experience, St.Louis, pp. 209-227.
Dove, J.E. and Frost, J.D. (1996). A method for measuring geomembrane surface roughness. Geosynthetics International, Vol. 3, No. 3, pp. 369-392.
Duncan, J.M., Tan, C.K., Barker, R.M., and Rojiani, K.B. (1989). Load and resistance factor design on bridge structures. In Proceedings of the Symposium on Limit States Design in Foundation Engineering. Canadian Geotechnical Society – Southern Ontario Section, Toronto, May 26-27, pp. 47-63.
Dunnicliff, C.J. (1982). Geotechnical Instrumentation for Monitoring Field Performance. National Cooperative Highway Research Program, Synthesis of Highway Practice No. 89, Transportation Research Board, National Research Council, Washington, D.C. 46 p.
Dunnicliff, J. (1988). Geotechnical Instrumentation for Monitoring Field Performance. Wiley-Interscience Publication, John Wiley & Sons, Inc.
Eden, W.J. and Jarrett, P.M. (1971). Landslide at Orleans, Ontario. National Research Council, Division of Building Research (NRCC)paper 11856, 23 p.
Eisenstein, Z. and Morrison, N.A. (1973). Prediction of Foundation Deformations in Edmonton Using an In Situ Pressure Probe. Canadian Geotechnical Journal, Vol. 10, No. 1, pp. 193-210.
El Naggar, M.H. (1998). Interpretation of Lateral Statnamic Load Test Results. Geotechnical Testing Journal, ASTM, Vol. 21, No. 3, pp. 169-179.
El Naggar, M.H and Baldinelli, M.J.V. (2000). Interpretation of axial Statnamic load test using an automatic signal matching technique. Canadian Geotechnical Journal, Vol. 37, No. 5, pp.927-942.
El Naggar, M.H. and Bentley, K.J. (2000). PYLAT Computer program for the lateral response of single piles to dynamic loads. Geotechnical Research Centre, The University of Western Ontario, London, Canada.
El Naggar, M.H. and Mostafa, Y.E. (2001). PYLATG Computer program for the lateral response of pile groups to dynamic loads. Geotechnical Research Centre, The University of Western Ontario, London, Canada.
El Naggar, M.H. and Novak, M. (1990). GROUPSET. A Computer Code for Pile Group Settlement. Research Report, University of Western Ontario, London, GEOP-90-02.
El Naggar, M. H. and Novak, M. (1992). Analytical model for an innovative pile test. Canadian Geotechnical Journal, Vol. 29, 569-579, February 1992.
El Naggar, M. H. and Novak, M. (1994). Non-Linear model for dynamic axial pile response. Canadian Geotechnical Journal, Vol. 120, No. 2, 308-329, February 1994.
El Naggar, M. H. and Novak, M. (1995a). Nonlinear lateral interaction in pile dynamics. Journal of Soil Dynamics and Earthquake Engineering, 14 (2), 141-157.
El Naggar, M. H. and Novak, M. (1995b). Effect of foundation nonlinearity on Modal properties of offshore towers. Journal of Geotechnical Engineering, ASCE, 121 (9), 660-668.
El Naggar, M. H. and Novak, M. (1996). Influence of foundation nonlinearity on offshore tower response. Journal of Geotechnical Engineering, ASCE, 122 (9), 717-724.
El Naggar, M.H. and Novak, M. (1996). Nonlinear analysis for dynamic lateral pile response. Journal of Soil Dynamics and Earthquake Engineering, Vol. 15, No. 4, pp. 233-244.
El Naggar, M.H., and Sakr, M. (2001). Evaluation of axial performance of tapered piles from centrifuge tests. Canadian Geotechnical Journal, 37(6): 1295-1308.
El Naggar, M.H. and Wei, J.Q. (2000). Uplift behaviour of tapered pile established from model tests. Canadian Geotechnical Journal, Vol. 37, No. 1, pp.56-74.
El Sharnouby, B. and Novak, M. (1985). Static and Low Frequency Response of Pile Groups. Canadian Geotechnical Journal, Vol. 22, No. 2, pp. 79-84.
El Sharnouby, B. and Novak, M. (1990). Stiffness Constants and Interaction Factors for Vertical Response of Pile Groups. Res. Report GEOT-8-90, Faculty of Engrg. Science, University of Western Ontario.
El-Marsafawi, H., Han, Y.C. and Novak, M. (1992). Dynamic experiments on two pile groups. Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 4, pp. 576-592.

Elsabee, F. and Morray, J. P. (1977). Dynamic behavior of embedded foundations. Research Report R77-33, Civil Engineering Department, Massachusetts Institute of Technology, Cambridge, Massachusetts.

ENV 1991-1 (1994). Basis of design and actions on structures-Part 1. Basis of design, CEN (European Committee for Standardization), Brussels.

ENV 1997-1 (1994). Eurocode 7 Geotechnical Design-Part. General rules, CEN (European Committee for Standardization), Brussels.

ENV 1997-2 (1999). Eurocode 7 Geotechnical Design – Part 2. Design Assisted by Laboratory Testing, CEN (European Committee for Standardization), Brussels.

ENV 1997-3 (1999). Eurocode 7 Geotechnical Design-Part 3. Design Assisted by Field Testing, CEN (European Committee for Standardization), Brussels.

EPA 9090 (1986). Compatibility Test for Wastes and Membrane Liners. September.

ESOPT II (1982). Proceedings of the Second European Symposium on Penetration Testing. Amsterdam, A.A. Balkema 18.

Esrig, M.I. (1968). Pore pressures, consolidation and electrokinetics. Journal of Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 94, SM4, 899-921.

Eurocode 7 (1987). Draft Model code for Eurocode 7. Geotechnics Design, Unpublished Report prepared for the Commission of the European Communities by representatives of the Geotechnical Societies within the European Communities.

Eurocode 7 (1990). Eurocode 7. Geotechnics, Preliminary draft prepared for the Commission of the European Communities in 1989, published in Geotechnik in 1990/1, pp. 1-40.

Fahey, M., and Carter, J.P. (1993). A finite element study of the pressuremeter test in sand using a nonlinear elastic plastic mode. Canadian Geotechnical Journal, 30: 348–362.

Fang, Hsai-Yang (1991). Foundation Engineering Handbook. 2nd Edition, Van Nostrand Reinhold, publishers.

Fannin R.J. and Pishe, R. (2001). Testing and specifications for geotextile filters in cyclic flow applications. Geosynthetics 2001, Portland, Oregon, USA, Industrial Fabrics Association International, pp. 423-435.

Fannin, R.J. (2000a). Long-term variations of force and strain in a steep geogrid-reinforced soil slope. Geosynthetics International, Vol.8, No. 1, pp. 81-96.

Fannin, R.J. (2000b). Basic geosynthetics: a guide to best practices. BiTech Press, Richmond, B.C., 86 p.

Fannin, R.J. and Hermann, S. (1992). Strength of Polymeric Reinforcement — Design to Ultimate and Serviceability Limit States. ASCE Speciality Conference on Stability and Performance of Slopes and Embankments — II, Berkeley, California.

Fannin, R.J. and Sigurdsson, O. (1996). Field observations on the stabilization of unpaved roads with geosynthetics. Journal of Geotechnical Engineering, ASCE, Vol.122, pp. 544-553.

Fannin, R.J., Choy, H.W. and Atwater, J.W. (1998). Interpretation of transmissivity data for geonets. Geosynthetics International, Vol. 5, No. 3, pp. 265-285.

Fannin, R.J., Vaid, Y.P. and Shi, Y.C. (1994). Filtration behaviour of nonwoven geotextiles. Canadian Geotechnical Journal, Vol. 31, pp. 555-563.

Farouki, O.T. (1986). Thermal Properties of Soils. Trans. Tech. Publications, Series on Rock and Soil Mechanics, Vol. 11, 136 p.

Faure, Y., Gourc, J.P., Brochier, P. and Rollin, A.L. (1986). Soil-Geotextile System Interaction in Filter Systems. 3rd International Conference on Geotextiles, Vienna, pp. 1207-12.

Feld, J. (1965). Tolerance of Structures to Settlement. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 91, SM3, pp. 63-67.

Fellenius, B.H. (1972). Downdrag on Long Piles in Clay Due to Negative Skin Friction. Canadian Geotechnical Journal, Vol. 9, No. 4, pp. 323-337.

Fellenius, B.H. (1975a). Reduction of Negative Skin Friction With Bitumen Slip Layers. Discussion. Journal of Geotech. Engrg. Div., ASCE, Vol. 101, GT4, pp. 412-414.

Fellenius B.H. (1975b). Test Loading of Piles - Methods, Interpretation, and Proof Testing. Journal of the Geotech. Engrg. Div., ASCE, Vol. 101, GT9, pp. 855-869.

Fellenius, B. H. (1975). Test load of piles and new proof testing procedure. Journal of Geotechnical Engineering Div., ASCE, Vol. 101, No. GT9, pp. 855-869.

Fellenius, B.H. (1979). Downdrag on Bitumen Coated Piles. Discussion. Journal of the Geotech. Engrg. Div.,

ASCE, Vol. 105, GT10, pp. 1262-1265.
Fellenius, B.H. (1980). The Analysis of Results from Routine Pile Test Loading. Ground Engineering, Foundation publications Ltd., London, Vol. 13, No. 6, pp. 19-31.
Fellenius, B.H. (1983). The Design, Installation, and Inspection of Precast Prestressed Concrete Piles. Lecture notes, University of Ottawa, 43 p.
Fellenius, B.H. (1984a). Negative Skin Friction and Settlement of Piles. Proceedings Second International Geotechnical Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 18 p.
Fellenius, B.H. (1984b). Geotechnically Allowable Stress for Driven Piles. American Society of Civil Engineers (ASCE) Civil Engineering, Vol. 54, No. 11, pp. 72-75.
Fellenius, B.H. and Broms, B.B. (1969). Negative Skin Friction for Piles Driven in Clay. Proceedings 7th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Mexico City, Vol.2, pp. 93-98.
Fellenius, B.H. and Samson, L. (1976). Testing of Drivability of Concrete Piles and Disturbance to Sensitive Clay. Canadian Geotechnical Journal, Vol. 13, No. 2, pp. 139-160.
Fellenius, B.H., Samson, L., Thompson, P.E. and Trow, W. (1978). Dynamic Behaviour of Foundation Piles and Driving Equipment. Terratech Ltd. and the Trow Group Limited., Final Report, Department of Supply and Services, Canada, Research project, Contract No. 1ST77.00045, Vol. I and Vol. II, 580 p.
FHWA (1976) Lateral Support Systems and Underpinning. Vol. I, Vol. II, Report FHWA-RD-75-130. U.S. Department of Transportation, Federal Highway Administration.
FHWA (1984). Permanent Ground Anchors, Report FHWA-DP-68-1R. U.S. Department of Transportation, Federal Highway Administration.
FHWA (1993). Guidelines for design, specification, and contracting of geosynthetic mechanically stabilized earth slopes on firm foundations (Berg, R.R.). FHWA-SA-93-025, Federal Highway Administration, Washington, D.C., USA.
FHWA (1997). Corrosion/degradation of soil reinforcements for mechanically stabilized earth walls and reinforced soil slopes (Elias, V.). FHWA-SA-96-072, Federal Highway Administration, Washington, D.C., USA.
FHWA (1998). Development of protocols for confined extension/creep testing of geosynthetics for highway applications (Elias, V., Yuan, Z, Swan, R.H. and Bachus, R.C.). FHWA-RD-97-143, Federal Highway Administration, Washington, D.C., USA.
FHWA (1999). Testing protocols for oxidation and hydrolysis of geosynthetics (Elias, V., Salman, A., Juran, I., Pearce, E. and Lu, S.). FHWA-RD-97-144, Federal Highway Administration, Washington, D.C., USA.
FHWA (2001). Mechanically stabilized earth walls and reinforced soil slopes (Elias, V., Christopher, B.R. and Berg, R.R.). FHWA-NHI-00-043, Federal Highway Administration, Washington, D.C., USA.
Finn, W.D.L. (1993). Seismic Safety Evaluation of Embankment Dams. In Proceedings, International Workshop on Dam Safety Evaluation, 26-28 April, Grindewald, Switzerland, Vol. 4, pp. 91-135.
Finn W.D.L., and Wightman A. (2003). Ground motion amplification factors for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, 30: 272-278.
Finn, P.S., Nisbet, R.M. and Hawkins, P.G. (1984). Guidance on Pressuremeter, Flat Dilatometer and Cone Penetration Tests in Sand. Geotechnique, Vol. 34, No. 1, pp. 81-97.
Finn, W. D. L., Ledbetter, R. H. and Wu, G. (1994). Liquefaction in silty soils: Design and analysis. Ground failures under seismic conditions. Geotechnical Special Publication No. 44, ASCE: pp. 51-76.
Fleming, W.G.K., Weltman, A.J., Randolph, M.F. and Elson, W.K. (1992). Piling Engineering, 2nd Edition, New York. Halsted Press.
Focht, J. A., Jr. and Koch, K. J. (1973). Rational analysis of the lateral performance of offshore pile groups. Offshore Technology Conference, Dallas, Paper No. OTC. 1896.
Fowler, J., Bagdy, R. and Trainer, E. (1996). Dewatering sewage sludge with geotextile tubes. Proceedings of the 49th Canadian Geotechnical Conference, St. John's, Newfoundland, Canada.
Franklin, J.A. and Chandra, R. (1972). The Slake Durability Test. International Journal of Rock Mechanics and Mining Science, Vol. 9, pp. 325-341.
Franklin, J.A., and Dusseault, M. (1989). Rock Engineering. McGraw Hill.
Franklin, J.A., and Gruspier, J.E. (1983). Evaluation of shales for construction projects. Ministry of Transportation and Communications, Ontario, Research and Development Branch, 98 pages.
Fredlund, D.G. (1969). Consolidometer Test Procedural Factors Affecting Swell Properties. Proceedings of the

Second International Conference on Expansive Clay Soils, Texas A and M, College Station, Texas, pp. 435-456.
Fredlund, D.G. (1979). Appropriate Concepts and Technology for Unsaturated Soils. Second Canadian Geotechnical Colloquium, Canadian Geotechnical Journal, Vol. 16, No. 1, pp. 121-139.
Fredlund, D.G. (1983). Prediction of ground movements in swelling clays. presented at the 31st Annual Soil Mechanics and Foundation Engineering Conference (ASCE) Invited Lecture, Minneapolis, MN
Fredlund, D.G. (1987). The Prediction and Performance of Structures on Expansive Soils. Keynote Address, International Symposium on Prediction and Performance in Geotechnical Engineering, Calgary, Canada, June 17-19, Ed. by R.C. Joshi and F.J. Griffiths, pp. 51-60.
Fredlund D.G. and Krahn, J. (1977). Comparison of Slope Stability Methods of Analysis. Canadian Geotechnical Journal, Vol. 14, No.3, pp. 429-439.
Fredlund, D.G. and Morgenstern, N.R. (1977). Stress State Variables for Unsaturated Soils. Journal Geotechnical Engineering Division, ASCE, Vol. 103, No. GT5, pp. 447-466.
Fredlund, D.G. and Rahardjo, H. (1993). Soil Mechanics for Unsaturated Soils. John Wiley & Sons, New York, N.Y., 517 p.
Fredlund, D.G., Hasan, J.U., & Filson, H.L. (1980). The Prediction of Total Heave. Proceedings of the 4th International Conference on Expansive Soils, Denver, CO., ASCE, pp. 1-17.
Gail, C.P. (1973). Freezing as an Aid to Soft Ground Tunnel Construction. American Society of Civil Engineers (ASCE)Conference on Soft Ground Tunnelling, University of Wisconsin, W.T. Painter, Editor.
Gardoni, M.G. and Palmeira, E.M. (2002). Microstructure and pore characteristics of synthetic filters under confinement. Géotechnique, Vol. 52, No. 6, pp. 405-418.
Garneau, R. and Lebihan, J.P. (1977). Estimation of Some Properties of Champlain Clays with the Swedish Fall Cone Canadian Geotechnical Journal, Vol. 14, No. 4, pp. 571-581.
Gazetas, G. (1991). Foundation vibratins. Foundation Engineering Handbook, 2nd Edition, Fang, H. Y. Editor, Van Nostrand Reinhold, New York, N.Y.
Gazetas, G. and Dobry, R. (1984). Horizontal response of piles in layered soils. Journal of Geotechnical Engineering, ASCE, 110 (1), 20-40.
Gazetas, G. and Makris (1991). Dynamic pile-soil-pile interaction, part I: Analysis of axial vibration. Earthquake Engineering and Structural Dynamics, 20, 115-132.
Gazetas, G., Fan, K. and Kaynia, A.M. (1993). Dynamic response of pile groups with different configurations. Journal of Soil Dynamics and Earthquake Engineering, Vol. 12, No. 4, pp. 239-257.
Gazioglu, S.M. and O'Neill, M.W. (1984). Evaluation of P-Y Relationships in Cohesive Soils. ASCE Symposium on Analysis and Design of Pile Foundations, San Francisco, pp. 192-213.
GEO (1993). Review of granular and geotextile filters. GEO Publication No. 1/93, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong, p. 141.
GEOGUIDE 6 (2002). Guide to reinforced fill structure and slope design. Geotechnical Engineering Office, The Government of Hong Kong.
Geosynthetics International (1995). Special Issue on Design of Geomembrane Applications. Vol. 2, No. 6.
Geosynthetics International (1997). Special Issue on Liquid Migration Control Using Geosynthetic Liner Systems. Vol. 4, Nos. 3-4.
Geosynthetics International (2000). Special Issue on Liquid Collection Systems. Vol. 7, Nos. 4-6.
Geosynthetics International (2004). Special Issue on Geosynthetic Clay Liners. Vol. 11, No. 3.
Geotextile Design and Construction Guidelines (1989). (Pub. No. FHWA-HI-90-001).
Geotextiles and Geomembranes (2004), Special Issue on Prefabricated Vertical Drains. Vol. 22, Nos. 1-2.
Gibbs H.J., and Holtz, W.G. (1957). Research on Determining the Density of Sands by Spoon Testing. Proceedings 4th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), London, Vol. 1, pp. 35-39.
Gilbert, P.A. and Fowler, J. (1997). Development and demonstration of dredged material containment systems using geotextiles. Technical report; CPAR-GL-97-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss. USA, 84 p.
Giroud, J.P. (2002). Filter Criteria. Jubilee Volume, 75th Anniversary of K. Terzaghi's Erdbaumechanik (Soil Mechanics), H. Brandl, Editor, Reports of the Institute for Soil Mechanics and Geotechnical Engineering, Technical University of Vienna, Austria.

Giroud, J.P. and Noiray, L. (1981). Geotextile-reinforced unpaved road design. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT9, pp. 1233-1254.

Giroud, J.P., Ah-Line, C. and Bonaparte, R. (1984). Design of Unpaved Roads and Trafficked Areas with Geogrids. Proc. Symposium on Polymer Grid Reinforcement in Civil Engineering, London, March 22, 23 1984, Paper 4.1, Swindon: Science and Engineering Research Council, Blackburn: Netlon Limited.

Giroud, J.P., Gleason, M.H. and Zornberg, J.G. (1999), Design of geomembrane anchorage against wind action. Geosynthetics International, Vol. 6, No. 6, pp. 481-507.

Giroud, J.P., Pelte, T. and Bathurst, R.J. (1995a). Uplift of geomembranes by wind, Geosynthetics International. Vol. 2, No. 6, pp. 897-952.

Giroud, J.P., Soderman, K.L., Pelte, T. and Beech, J.F. (1995b). Design method to prevent geomembrane failure in tank corners. Geosynthetics International, Vol. 2, No. 6, pp. 971-1018.

Goble, G.G., Rausche, F. and Moses, F. (1970). Dynamic Studies on the Bearing Capacity of Piles. Phase III. Report No. 48, Vol. 1 and 2, Division of Solid Mechanics, Structures and Mechanical Design, Case Western Reserve University, Cleveland, 325 p.

Goddard, E.N. (1979). Rock Colour Chart, Geological Society of America, Boulder, Colorado.

Goldberg, D.T., Jaworski, W.E., and Gordon, M.D. (1976). Lateral Support Systems and Underpinning. FHWA-RD-75-130, Vol. III. Federal Highway Administration, Washington D.C., pp 149-153.

Goodrich, L.E. (1982). An Introductory Review of Numerical methods for Ground Thermal Regime Calculations. National Research Council, Division of Building Research, Ottawa, Paper No. 1061, 32 p.

Green, R. (1989). Limit states design: some thoughts. In Proceedings of the Symposium on Limit States Design in Foundation Engineering. Canadian Geotechnical Society-Southern Ontario Section, Toronto, May 26-27, pp. 91-116.

Green, R. (1991). The development of a LRFD code for Ontario bridge foundations. Proceedings of the Geotechnical Engineering Congress 1991. American Society of Civil Engineers, Geotechnical Special Publication No. 27, Vol. II, pp. 1365-1376.

Green, R. (1993). LSD code for bridge foundations. In Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering. Copenhagen, Denmark, May 26-28. Sponsored by the Danish Geotechnical Society. Vol. 2, pp. 459-468.

Green, R. and Becker, D.E. (2000). National Report on Limit State Design in Geotechnical Engineering: Canada. Proceedings of LSD 2000: International Workshop on Limit State Design in Geotechnical Engineering, Melbourne, Australia, November 18, 2000.

Green, R. and Becker, D.E. (2001). National report on limit state design in geotechnical engineering. Canada. Geotechnical News, 19(3), pp. 47-55.

Guido, V.A., Chang, D.K. and Sweeney, M.A. (1986). Comparison of geogrid and geotextile reinforced slabs. Canadian Geotechnical Journal, Vol. 23, No. 4, pp. 435-440.

Gutenberg, B. and Richter, C. F. (1944). Frequency of Earthquakes in California. Bulletin of the Seismological Society of America, 46: 105-145.

Hakulinen, M. (1991). Measured full-scale dynamic lateral pile responses in clay and in sand. Proceedings of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri at Rolla, Rolla, MO, pp. 201-206.

Hamilton J.J. (1965). Shallow Foundations on Swelling Clays in Western Canada. Proceedings International Research Conference on Expansive Soils, Texas A&M University, Vol. 2.

Hamilton, J.J. (1969). Effects of Environment on the Performance of Shallow Foundations. Canadian Geotechnical Journal, Vol. 6, No. 1, pp. 65-80.

Hamilton, J.J. (1977), Foundations on Swelling or Shrinking Subsoils. Canadian Building Digest, (CBD) 184, Division of Building Research, National Research Council, Ottawa, ON, Canada, 4 p.

Hamilton, J.J., and Handegord G.O. (1964). House Basements on Prairie Clays. Canadian Builder, Vol. 14, No. 9.

Hanna, T. H. (1968). The Bending Of Long H-Section Piles. Canadian Geotechnical Journal, Vol. 5, No. 3, pp. 150-172.

Hansbo, S. (1979). Consolidation by Bandshaped Prefabricated Drains. Ground Engineering, Foundation Publications Ltd., London, Vol. 12, No. 5, pp. 16-25.

Hansen, J.B. (1970). A revised and extended formula for bearing capacity. Danish Geotechnical Institute Bulletin

No. 28.
Hansen, W.R. (1965). Effects of the Earthquake of March 27, 1964, at Anchorage Alaska. United States Geological Survey, Professional Paper 542A, 68 p.
Harder, L.F. and Seed, H.B. (1986). Determination of Penetration Resistance for Coarse-Grained Soils Using the Becker Hammer Drill. Report No. UCB/EERC-86/06, University of California, Berkeley, 118 pages. May 1986.
Hardin, B.O. and Black, W.L. (1968). Vibration modulus of normally consolidated clay. Journal of the Soil Mechanics Division, ASCE, Vol. 94, No. SM2, pp. 353-369.
Hardin, B.O. and Drnevich, V.P. (1972). Shear modulus and damping in soils: design equations and curves. Journal of the Soil Mechanics Division, ASCE, Vol. 98, No. SM7, pp. 667-692.
Hassan, M.H., O'Neill, M.W., Sheikh, S.A. and Ealy, C.D. (1997). Design method for drilled shfts in soft arillaceous rock. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, vol. 123, No. 3, pp. 272-280.
Hatanaka, M., and Uchida, A. (1996). Empirical correlation between penetration resistance and N of sandy soils. Soils & Foundations, 36 (4): 1-9.
Hausmann, M.R. (1987). Geotextiles in unpaved roads, a review of design procedures. Geotextiles and Geomembranes, Vol. 5, No. 3, pp. 201-234.
Hausmann, M.R. (1990). Engineering Principles of Ground Modification. McGraw Hill.
Hayley, D.W. (1988). Maintenance of a Railway Grade Over Permafrost in Canada. The Northern Engineer, University of Alaska, Volume 21, No. 3, pp. 4-10.
Hight, D.W., and Leroueil, S. (2002). Characterisation of soils for engineering purposes. *In* Characterisation and Engineering Properties of Natural Soils. *Edited by* T.S. Tan, Phoon, K.K., Hight, D.W. and Leroueil, S. Singapore. 2003. A.A. Balkema, Vol.1, pp. 255-360.
Hinchberger, S.D. and Rowe, R.K. (2003). Geosynthetic reinforced embankments on soft clay foundations. Predicting reinforcement strain at failure, Geotextiles and Geomembranes, Vol. 21, No. 3, pp. 151-175.
Hoek, E. and Bray, J.W. (1977). Rock Slope Engineering, 2nd Edition. Institution of Mining and Metallurgy, London, 402 p.
Hoek, E. and Bray, J.W. (1981). Rock Slope Engineering, 3rd Edition., Institute of Mining & Metallurgy, London.
Hoek, E. and Brown, E.T. (1980). Empirical Strength Criterion for Rock Masses. Journal of the Geotech. Engrg. Div., ASCE, Vol. 106, GT9, pp. 1013-1035.
Hoek, E., and Brown, E.T. (1997). Practical estimates of rock mass strength. Int. J. Rock Mech. Min. Sci. Geomech. Abstr., 34(8): 1165–1186.
Hoek, E., and Marinos, P. (2000). Predicting Squeeze. Tunnels and Tunnelling International. November, 45–51.
Hoek, E., Kaiser, P.K., and Bawden, W.F. (1995). Support of Underground Excavations in Hard Rock. Rotterdam. A.A. Balkema, 215 pages.
Holm, G., Trank, R. and Ekstrom, A. (1983). Improving Lime Column Strength with Gypsum. Proceedings 8th European Conference on Soil Mechanics and Foundation Engineering, Helsinki, Vol. 2, pp. 903-907.
Holtz, R. (1986). Use of Engineering Fabrics in Transportation. Federal Highway Administration Instruction Manual.
Holtz, R.D. (1990). Design and construction of geosynthetically reinforced embankments on very soft soils. (State of the Art Paper) International Reinforced Soil Conference, Glasgow, June 1990, Thomas Telford, London, UK.
Holtz, R.D. and Boman, P. (1974). A New Technique of Reduction of Excess Pore Pressures During Pile Driving. Canadian Geotechnical Journal, Vol. 11, No. 3, pp. 423-430. Discussion in Vol. 12, No. 1, 1975, pp. 157-159
Holtz, R.D. and Kovacs, W.D. (1981). An Introduction to Geotechnical Engineering. Prentice-Hall, Englewood Cliffs, New Jersey, 733 p.
Holtz, R.D. and Wager, O. (1975). Preloading by Vacuum – Current Prospects. National Research Council, Transportation Research Board, Record, TRR548, pp. 26-29.
Holtz, R.D. and Wennerstrand, J. (1972). Embankments on Soft Ground. Discussion. American Society of Civil Engineers (ASCE)Proceedings Conference on Earth and Earth Supported Structures, Vol.3, pp. 59-60.
Holtz, R.D., Christopher, B.R. and Berg, R.R. (1997). Geosynthetic Engineering. BiTech Publishers Ltd., Richmond, BC, Canada.
Holtz, W.G. and Gibbs, H.J. (1956), Engineering Properties of Expansive Soils. Transactions of ASCE, Vol. 121, pp. 641-663.
Horne, W.T. (1987). Prediction of Frost Heave Using the Segregation Potential Theory. University of Alberta,

Department of Civil Engineering Unpublished M.Sc. Thesis, 194 p.
Horvath R.G. (1982). Drilled Piers Socketed into Weak Shale: Methods of Improving Performance. Ph.D Thesis, University of Toronto, Department of Civil Engineering.
Horvath R.G., Kenney, T.C. and Kosicki, P. (1983). Method of Improving the Performance of Drilled Piers in Weak Rock. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 758-772.
Horvath, J.S. (1995). Geofoam Geosynthetic, Horvath Engineering, P.C., Scarsdale, New York, NY, USA.
Hsieh, P-G and Ou, C-Y. (1999). Shape of ground surface settlement profiles caused by excavation. Canadian Geotechnical Journal, Vol. 35, pp. 1004 – 1017.
Huang, A-B., Hsueh, C-K, O'Nieill, M. W., Chern, S., and Chen, C. (2001). Effects of construction on laterally loaded pile groups. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 5, pp. 385-397.
Hughes J.M.O. and Robertson P.K. (1985). Full Displacement Pressuremeter Testing in Sand. Canadian Geotechnical Journal, Vol. 22, No. 3, pp. 298-307.
Hughes, M.M.O., Withers, N.J. and Greenwood D.A. (1975). A Field Trial of the Reinforcing Effect of a Stone Column in Soil. Geotechnique Vol. 25, pp. 31-44.
Hunt, H.W. (1979). Design and Installation of Driven Pile Foundations. Associated Pile and Fitting Corporation, Clifton, New Jersey, 217 p.
Hunt, R.E. (1986). Geotechnical Engineering Analysis and Evaluation. New York: McGraw-Hill Book Company.
Hutchinson, D.J., and Diederichs, M.S. (1996). Cablebolting in Underground Mines. Vancouver, BiTech Publishers, 406 pages.
Hvorslev, M.J. (1949). Subsurface Exploration and Sampling of Soil for Civil Engineering Purposes. The Engineering Foundation, New York.
Hwang, C.T. (1976). Predictions and Observations on the Behaviour of a Warm Gas Pipeline on Permafrost. Canadian Geotechnical Journal Volume 13, No. pp. 452-480.
Idriss, I. M. and Sun, J.I. (1992). SHAKE91: A computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits. Program modified based on the original SHAKE program published by Schnabel, P.B., Lysmer, J. and See, H.B. (1972). University of California, Earthquake Engineering Research Center, November.
IFAI 2004. Specifier's Guide (2004). Industrial Fabrics Association International. 1801 County Road, B.W., Roseville, MN 55113-4061, USA.
Industrial Fabrics Association International (1990). A Design Printer: Geotextiles and Related Materials. St. Paul, MN.
Ingold, T.S. (1979). Retaining wall performance during backfilling. Journal of the Geotechnical Engineering Division, ASCE, 105(5), 6 13 – 626.
ISRM - International Society for Rock Mechanics (1974). Suggested methods for determining shear strength. Final Draft, February, 1974: Committee on Field Tests.
ISRM - International Society for Rock Mechanics (1978). Suggested methods for the quantitative description of discontinuities in rock masses. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 15(6): 319–368.
ISRM - International Society for Rock Mechanics (1978). Suggested methods for determining the strength of rock materials in triaxial compression. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 15: 47-51.
ISRM - International Society for Rock Mechanics (1979). Suggested methods for determining the uniaxial compressive strength and deformability of rock materials. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 16(2): 135-140.
ISRM - International Society for Rock Mechanics (1979). Suggested methods for determining water content. porosity, density, absorption and related properties and swelling and slake durability index properties. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 16(2): 141-156.
ISRM - International Society for Rock Mechanics (1979). Suggested methods for determining in situ deformability of rock. Part I. Suggested method for deformability determination using a plate test (superficial loading). Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 16: 197-202.
ISRM - International Society for Rock Mechanics (1979). Suggested methods for determining in situ deformability of rock. Part II. Suggested method for field deformability determination using a plate test down a borehole. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 16: 202-208.

ISRM - International Society for Rock Mechanics (1981). Rock Characterisation, Testing and Monitoring. ISRM Suggested Methods, (ed. E.T. Brown), Pergamon Press, 221 pages.

ISRM - International Society for Rock Mechanics (1985). Suggested Method for Determining Point Load Strength. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 22(2): 53–60.

Ireland H.Q., Moretto, O. and Vargas, M. (1970). The Dynamic Penetration Test: a Standard that is not Standardized. Geotechnique, Vol. 20, pp. 185-192 and pp. 452-456.

ISOPT (1988). Penetration Testing 1988. Proceedings of the First International Symposium on Penetration Testing, ISOPT-1 Orlando Florida USA, A.A. Balkema.

ISSMFE (1977a). List of Symbols, Units, and Definitions. Proceedings 9th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Tokyo, Vol. 3, pp. 156-170.

ISSMFE (1977b). Report of the Subcommittee on Standardization of Penetration Testing in Europe. Proceedings 9th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Tokyo, Vol. 3, Appendix 5, pp. 95-152.

ISSMFE (1989). Report of the ISSMFE Technical Committee on Penetration Testing of Soils-TC16 with Reference Test Procedures CPT-SPT-DP-WST.

IWS Kamakura (2002). Foundation Design Codes and Soil Investigation in view of International Harmonization and Performance Based Design. Editors: Y. Honjo, O. Kusakabe, K. Matsui, M. Kouda and G. Pokharel. Proceedings of International Workshop, Japan, April 10-12, 2002. A.A. Balkema Publishers, The Netherlands, pp. 31-48. 459p.

Jamiolkowski M., Ladd, C.C., Germaine, J.T. and Lancellotta, R. (1985). New Developments in Field and Laboratory Testing of Soils. Theme Lecture No. 2, Proceedings of 11th Internat. Conl, on Soil Mech. and Found. Engineering, San Francisco, Vol. 1, pp. 57-154.

Janbu, N. (1954). Application of Composite Slip Surfaces for Stability Analysis. Proceedings of the European Conference on Stability of Earth Slopes, Stockholm Vol. 3, pp. 43-49.

Janbu, N. (1963). Soil Compressibility as Determined by Oedometer and Triaxial Tests. European Conference on Soil Mechanics and Foundation Engineering Wiesbaden, Vol. 1 and Vol. 2.

Janbu, N. (1965). Consolidation of Clay Layers Based on Non-Linear Stress-Strain. Proceedings 6th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Montreal, Vol. 2.

Janbu, N. (1967). Settlement Calculations Based on the Tangent Modulus Concept. University of Trondheim, Norwegian Institute of Technology, Bulletin No. 2, 57 p.

Janbu, N. and Senneset, K. (1973). Field Compressometer; Principles and Applications. Proceedings 8th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Moscow, Vol.1.1, pp. 191-198.

Jardine, R.J. and Chow, F.C. (1996). New design methods for offshore piles. Report No. 96-103, Marine Technology Directorate Ltd., London.

Jewell, R.A. (1991). Application of Revised Design Charts for Steep Reinforced Slopes. Geotextiles and Geomembranes Vol. 10, No. 3, pp. 203-234.

Johnson, S. (1970). Foundation Precompression with Vertical Sand Drains. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 96, SM1, pp. 145-175.

Johnson, S. (1970). Precompression for Improving Foundation Soils. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering Vol. 96, SM1, pp. 111-144.

Johnston, G.H. (Editor) (1981). Permafrost Engineering, Design and Construction. Associate Committee on Geotechnical Research, National Research Council of Canada, John Wiley & Sons, Toronto, New York, 540 p.

Jones, D.E. and Holtz, W. G. (1973), Expansive Soils – Hidden Disaster. Civil Engineering, ASCE, New York, N.Y., pp. 87-89.

Kana, D. D., Boyce, L. and Blayney, G. W. (1986). Development of a scale model for the dynamic interaction of a pile in clay. Journal of Energy Resources Technology, ASME, Vol. 108, pp. 254-261.

Kany, M. (1959). Beitrag Zur Berechnung Von Flachengrundlagen. Wilhelm Ernst und Sohn, Berlin.

Karabalis, D. L. and Beskos, D. E. (1985). Dynamic response of 3-D embedded foundations by the boundary element method. 2nd Joint ASCE/ASME Conference, Albuquerque, p. 34.

Karpoff, K.L. (1955). The Use of Laboratory Tests to Develop Design Criteria for Protective Filters. American Society for Testing and Materials, ASTM Proceedings, Vol. 55, pp. 1183-1198.

Kausel, E. and Ushijima, R. (1979). Vertical and torsional stiffness of cylindrical footing. Civil Engineering

Department Report R79-6, MIT, Cambridge, Massachusetts.

Kausel, E., Roesset, J. M. and Waas, G. (1975). Dynamic analysis of footings on layered media. Journal of the Engineering Mechanics Division, ASCE, 101 (EM5), 679-693.

Kaynia, A. M. and Kausel, E. (1982). Dynamic behavior of pile groups. 2nd International Conference on Numerical Methods in Offshore Piling, Austin, Texas, 509-532.

Kenney, T.C. and Lau, D. (1985). Internal instability of granular filters. Canadian Geotechnical Journal, Vol. 22, pp. 215-225.

Kenney, T.C. and Lau, D. (1986). Internal instability of granular filters: reply. Canadian Geotechnical Journal, Vol. 23, pp. 420-423.

Kersten, M.S. (1949). Thermal Properties of Soils. University of Minnesota, Engineering Experiment Station, Bull. 28, 227 p.

Kezdi, A. (1964). Bodenmechanik. Verlag fur Bauwesen, Berlin, Band 1.

Kezdi, A. (1979). Soil Physics. Amsterdam: Elsevier

Kjellman, W. (1948). Accelerating Consolidation of Fine-Grained Soils by Means of Cardboard Wicks. Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, pp. 302-305.

Kjellman, W. (1952) Consolidation of Clay Soil by Means of Atmospheric Pressure, Proceedings of the Conference on Soil Stabilization, M. I. T., pp. 258-263.

Klute, A. (1986), Methods of Soil Analysis, Part 1 – Physical and Mineralogical Methods. Second Edition, Agronomy, No. 9, Part 1, American Society of Agronomy, Soil Science Society of America, Madison, Wisconsin, U.S.A., 1188 p.

Kobayashi, K.Yao, S. and Yoshiada, N. (1991). Dynamic compliance of pile group considering nonlinear behaviour around piles. Proceedings of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri at Rolla, Rolla, MO, pp. 785-792.

Kobayashi, S. and Nishimura, N. (1983). Analysis of dynamic soil-structure interactions by boundary integral equation method. Proceedings of 3rd International Symposium on Numerical Methods in Engineering, Paris, 353-362.

Kobori, T., Minai, . R. and Baba, K. (1977). Dynamic behaviour of a laterally loaded pile. 9th International Conference of Soil Mechanics, Tokyo, Session 10 (6), 175-180.

Kobori, T., Minai, R. and Suzuki, T. (1971). The dynamical ground compliance of a rectangular foundation on a viscoelastic stratum. Bulletin Disaster Prevention Research Institute, Kyoto University, 20, 289-329.

Kobori, T., Nakazawa, M., Hijikata, K., Kobayashi, Y., Miura, K., Miyamoto, Y. and Moroi, T. (1991). Study on dynamic characteristics of a pile group foundation. Proceedings of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri at Rolla, Rolla, MO, pp. 853-860.

Koerner, R.M. (1990). Designing with Geosynthetics. 2nd ed. Englewood Cliffs, N.J.:

Koerner, R.M. (1997). Designing with Geosynthetics. 4th edition, Prentice Hall Canada Inc., Toronto, ON, Canada.

Koerner, R.M. and Hsuan, Y.G. (2001). Geosynthetics: characteristics and testing. Geotechnical and Geoenvironmental Engineering Handbook, (edited by R.K. Rowe), Kluwer Academic Publishing, Norwell, MA, USA.

Koerner, R.M., Monteleone, M.J., Schmidt, R.K. and Roethe, A.T. (1986). Puncture and impact resistance of geosynthetics. Proceedings of the 3rd International Conference on Geosynthetics, Vienna, pp. 677-682.

Koester, J.P. (1992). The influence of test procedure on correlation of Atterberg limits with liquefaction in fine-grained soils. Geotechnical Testing Journal. 15 (4): pp. 352 – 361.

Konrad, J.M. (1987). Piezo-Friction-Cone Penetrometer Testing in Soft Clays. Canadian Geotechnical J. 24, pp. 645-652.

Konrad, J.M. (1999) Frost susceptibility related to soil index properties Canadian Geotechnical Journal, Vol.36, No 3, pp 403 – 417.

Konrad, J.M. (2000). Chapter 20 Cold Region Engineering in Geotechnical and Geoenvironmental Engineering Handbook edited by R.K. Rowe. Publisher: Chapman & Hall, New York, London.

Konrad, J.M. and Law, K.T. (1987a). Preconsolidation Pressure from Piezocone Test in Marine Clay. Geotechnique 37, No. 2, pp. 177-190.

Konrad, J.M. and Law, K.T. (1987b). Undrained Shear Strength from Piezocone Tests. Canadian Geotechnical J. 24, pp. 392-405.

Konrad, J.M. and Morgenstern, N.R. (1980). A Mechanistic Theory of Ice Lensing in Soils. Canadian Geotechnical Journal, Volume 17, pp. 473-486.

Konrad, J.M. and Morgenstern, N.R. (1981). The Segregation Potential of a Freezing Soil. Canadian Geotechnical Journal, Volume 18, pp. 473-486.

Kovacs, W.D. and Salomone, L.A. (1982). SPT Hammer Energy Measurements. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 108, GT4, pp. 599-620.

Kovacs, W.D., Salomone, L.A. and Yokel, F.Y. (1981). Energy Measurements in the Standard Penetration Test. United States National Bureau of Standards, Building Science Series 135.

Kovacs, W.D., Yokel, F.Y., Salomone, L.A. and Holtz, R.D. (1984). Liquefaction Potential and the International SPT. Proceedings of the 8th World Conference on Earthquake Engineering, San Francisco, CA.

Krohn, J.P. and Slosson, J.E. (1980), Assessment of Expansive Soils in United States. Proceedings of the 4th International Conference on Expansive Soils, ASCE, Denver, CO, pp. 596-608.

Kuhlemeyer, R. L. (1979). Static and dynamic laterally loaded floating piles. Journal of the Geotechnical Engineering Division, ASCE, 105 (GT2), 289-304.

Kulhawy, F.H. (1991). Drilled Shaft Foundations. Chapter 14 in Foundation Engineering Handbook (2nd Edition), Editor: H-Y Fang, Van Nostrand Reiner, New York.

Kulhawy, F.H. and Carter, J.P. (1992). Socketed foundations in rock masses. In Bell, F.G,(Ed.) Engineering in Rock Masses, Butterworth-Heinemann, Oxford, pp. 509-529.

Kulhawy, F.H. and Goodman, R.E. (1980). Design of Foundations on Discontinuous Rock. Proceedings International Conference on Structural Foundations on Rock, Sydney, Australia, J.P.N. Pells, Editor, Vol. 1, pp. 209-222.

Kulhawy, F.H. and Mayne, P.W. (1990). Manual on Estimating Soil Properties for Foundation Design. Report El-6800, Electric Power Research Institute, Palo Alto, Calif.

Kulhawy, F.H. and Phoon, K.K. (2002). Observations on geotechnical reliability-based design developments in North America. Proceedings of International Workshop on Foundation Design Codes and Soil Investigation in view of International Harmonization and Performance Based Design. Editors: Y. Honjo, O. Kusakabe, K. Matsui, M. Kouda and G. Pokharel. Proceedings of International Workshop, Japan, April 10-12, 2002. A.A. Balkema Publishers, The Netherlands, pp. 31-48.

Ladanyi, B. (1963). Expansion of a Cavity in a Saturated Clay Medium. J. Soil Mech. Foundn. Division, ASCE, Vol. 89, SM4, pp. 127-161.

Ladanyi, B. (1972). In Situ Determination of Undrained Stress - Strain Behaviour of Sensitive Clays with the Pressuremeter. Canadian Geotechnical Journal, Vol. 9, No. 2, pp. 313-319.

Ladanyi, B. (1977). Friction and End Bearing Tests on Bedrock for High Capacity Socket Design. Discussion. Canadian Geotechnical Journal, Vol. 14, pp. 153-155.

Ladanyi, B. (1982). Determination of Geotechnical Parameters of Frozen Soil by Means of the Cone Penetrometer Test. Proceedings European Symposium on Penetration Testing, ESOPT II, Amsterdam, Vol. 2, pp. 671-678.

Ladanyi, B. and Johnston, G.H. (1973). Evaluation of In Situ Creep Properties of Frozen Soils with the Pressuremeter. Proceedings Second International Conference on Permafrost, Yakutsk, pp. 310-318.

Ladanyi, B. and Roy, A. (1971). Some Aspects of Bearing Capacity of Rock Mass. Proc., 7th Canadian Symposium on Rock Mechanics, Edmonton, pp. 161-190.

Ladanyi, B., Dufour, R., Larocque, G.S., Samson, L., and Scott, J.S. (1974). Report of the subcommittee on foundations and near-surface structures to the Canadian Advisory Committee on Rock Mechanics. 55 pages.

Lade, P.V. (2001). Engineering Properties of Soils and Typical Correlations. Geotechnical and Geoenvironmental Handbook, R.K. Rowe, Ed., Chapter 4, Kluwer Academic Publishers, pp. 43-67.

Lafleur J., Mlynarek J. and Rollin A.L. (1989). Filtration of broadly graded cohesionless soils. Journal of Geotechnical Engineering, ASCE, Vol.115, No.12, pp. 1747-1768.

Lafleur, J. (1999). Selection of geotextiles to filter broadly graded cohesionless soils. Geotextiles and Geomembranes. Vol. 17, Nos. 5 & 6, pp. 299-312.

Lambe, T.W., Wolfskill, L.A. and Wong, J.H. (1970). Measured Performance of Braced Excavations. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 96, SM3, pp. 817-836.

Lamé, G. (1852). Lećons sur la théorie mathematique d'élasticité des corps solides. Bachelier, Paris, France.

Lefebvre, G. (1981). Strength and Slope Stability in Canadian Soft Clays. Fourth Canadian Geotechnical Colloquium.

Canadian Geotechnical Journal, Vol. 18, No. 3, pp. 420-442.
Legget, R.F. (1962). Geology and Engineering. 2nd Edition, McGraw-Hill Book Company, 884 p.
Legget, R.F. (1965). Soils in Canada, Geological, Pedological and Engineering Studies. The Royal Society of Canada Special Publications, No. 3. Published by the University of Toronto Press.
Legget, R.F. (1976). Glacial Till, An Inter-disciplinary Study. The Royal Society of Canada Special Publications, No. 12. Published by the University of Toronto Press.
Legget, R.F. (1979). Geology and Geotechnical Engineering. The Thirteenth Terzaghi Lecture, October 20, 1977, American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 105, GT3, pp. 339-391.
Legget, R.F., and Crawford, C.B. (1965). Trees and Buildings. Canadian Building Digest 62, February, National Research Council, Division of Building Research.
Legget, R.F. and Karrow, P.F. (1983). Handbook of Geology in Civil Engineering. McGraw Hill Book Company, New York, 1,308 p.
Lehane, B. and Fahey, M. (2002). A simplified nonlinear settlement prediction model for foundations on sand. Canadian Geotechnical Journal, J. 39: 293-303.
Lehane, B.M. and Jardine, R.J. (1994). Shaft capacity of driven piles in sand: a new design approach. Proceedings BOSS 94. 7th International Conference on the Behaviour of Off-Shore Structures, Volume 1, pp. 23-36, Boston, USA.
Lemasson, H. (1976). Une Nouvelle Methode pour la Mesure en Place de l'Anistropie des Argiles. Stabilité des Talus, Tome II, Laboratoire Central des ponts et Chaussees, Paris, pp. 107-116.
Leroueil, S. (1996). Compressibility of clays: Fundamental and practical aspects. Journal of Geotechnical Engineering, 122(7): 534-543.
Leroueil, S. (2001). 39th Rankine Lecture–Natural slopes and cuts. movement and failure mechanisms. Géotechnique, 51(3): 197-243.
Leroueil, S., La Rochelle, P., Tavenas, F. & Roy, M. (1990). Remarks on the stability of temporary cuts. Canadian Geotechnical Journal, 27(5): 687-692.
Leroueil, S., Tavenas, F.A., Samson, L. and Morin, P. (1983). Preconsolidation Pressure of Champlain Clays. Part II: Laboratory determination. Canadian Geotechnical Journal, Vol. 20, pp. 803-816.
Li, A.L. and Rowe, R.K. (2001). Combined effects of reinforcement and prefabricated vertical drains on embankment performance, Canadian Geotechnical Journal, Vol. 38, No. 6, pp. 1266-1282.
Liao, S.S.C. and Whitman, R.V. (1986). Overburden Correction Factors for the SPT in Sand. Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 3, pp. 373-377.
Linell, K.A., Hennion, F.B. and Kobacz, E.F. (1963). Corps of Engineers Pavement Design in Areas of Seasonal Frost. Highway Research Record No. 33, Highway Research Board, pp. 76-128.
Lo, K.Y. (1972). The Operational Strength of Fissured Clays Geotechnique, Vol. 20, pp. 57-74.
Lo, K.Y. and Cooke, B.H. (1989). Foundation design for the Skydome Stadium, Toronto. Cnadian Geotechnical Journal, Vol. 26, No. 1, pp. 22-33.
Lo, K.Y., and Hefny, A.M. (2001). Foundations on Rock. In Geotechnical and Geoenvironmental Handbook, (ed. R.K. Rowe), Kluwer Academic Publishers, Chapter 11: 305 – 335.
Lo, K.Y., Inculet, I.I., and Ho, K.S. (1991). Field test of electro-osmotic strengthening of soft sensitive clay. Canadian Geotechnical Journal. 28, 74-83.
Loughney, R.W. (2001). Seepage, Drainage and Dewatering. Chapter 8 in Geotechnical and Geoenvironmental Engineering Handbook. Edited by R.K. Rowe. Klawer Academic Publishers, Boston, Massachusetts, USA.
Lpile Plus 4. (2000). A program for the analysis of piles and drilled shafts under lateral loads. Ensoft Inc.
LSD (2000). Proceedings of International Workshop on Limit State Design in Geotechnical Engineering (ISSMGE Technical Committee TC23). Melbourne, Australia, November 18, 2000. GeoEng (2000). An International Conference on Geotechnical & Geological Engineering. Melbourne, Australia, November 19-24, 2000.
Luco, J. E. and Hadjian, A. H. (1974). Two-dimensional approximations to the three-dimensional soil-structure interaction problem. Nuclear Engineering and Design, 31 (2), 195-203.
Luco, J. E. and Westmann, R. A. (1971). Dynamic response of circular footings. Journal of the Engineering Mechanics Division, ASCE, 97 (EM6), 1381-1395.
Lunne, T., Lacasse, S., and Rad, N.S. (1989). SPT, CPT, pressuremeter testing and recent developments in in-situ

testing – Part 1: All tests except SPT. In Proceedings, 12th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE) Rio de Janeiro, Volume 4, A.A. Balkema, Rotterdam, pp. 2339-2403.

Lunne, T., Robertson, P.K., and Powell, J.J.M. (1997). Cone Penetration Test in Geotechnical Practice. Spon Press. London.

Lysmer, J. and Kuhlemeyer, R. L. (1969). Finite dynamic model for infinite media. Journal of the Engineering Mechanics Division, ASCE, 95 (EM4), 859-877.

MacFarlane, I.C. (1969). Muskeg Engineering Handbook. The Muskeg Subcommittee of the National Research Council of Canada (NRC) Associate Committee on Geotechnical Research. University of Toronto Press.

MacGregor, J.G. (1976). Safety and limit states design for reinforced concrete. Canadian Journal of Civil Engineering, Vol. 3, pp. 484-513.

Mair, R.J. and Wood, D.M. (1987). Pressuremeter Testing Methods and Interpretation. Construction Industry Research and Information Association (CIRIA). Ground Engineering Report: In-situ testing. Butterworths, London, UK. 160 p.

Major, A. (1962). Dynamics in Civil Engineering. Akademical Kiado, Budapest, Vol. I-IV.

Makris, N. and Gazetas, G. (1992). Dynamic pile-soil-pile interaction. Part II: Lateral and seismic response. Earthquake Engineering and Structural Dynamics, Vol. 21, pp. 145-162.

Mana, A.I. and Clough, G.W. (1981). Prediction of movements for braced cuts in clay. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. 6, 759 – 777.

Mansur, C.I. and Kaufman, R.I. (1962). Dewatering. In Leonards, G.A. (Ed.) Foundation Engineering, McGraw-Hill, New York, pp. 241-350

Marchetti, S. (1980). In Situ Tests by Flat Dilatometer. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 106, GT3, pp. 299-321.

Marchetti S., Monaco P., Totani G., and Calabrese M. (2001). The Flat Dilatometer Test (DMT) in Soil Investigations A Report by the ISSMGE Committee TC16. In Proceedings, IN SITU 2001, International Conference on In-Situ Measurement of Soil Properties, Bali, Indonesia, May 2001, 41 pp.

Marinos, P., and Hoek, E. (2001). Estimating the geotechnical properties of heterogeneous rock masses such as flysch. Bull. Eng. Geol. Env., 60(2): 85–92.

Masuda, K., Saseki, F., Urao, K. Veno, K. and Miyamoto, Y. (1986). Simulation analysis of forced vibration test of actual pile foundation by thin layer method. Proceedings of Annual Meeting of Arch. Inst. Of Japan, Architectural Institute of Japan.

Matar, M. and Salencon, J. (1977). Capacité Portante a une Semelle Filante sur sol Purement Cohérent d'épaisseur Limitée et de Cohésion Variable avec la Profondeur. Annales de l'Institute Technique du Bâtiment et des Travaux Publics, Supplement No. 352, Juillet-Aout, Serie: Sols et Fondations, No. 143, pp. 95-107.

Matlock, H. (1970). Correlations for Design of Laterally Loaded Piles in Soft Clay. 2nd Offshore Technology Conference, Houston, Paper No. OTC 1204. Vol. 1, pp. 577-588.

Matlock, H. and Reese, L.L. (1960). Generalized Solutions for Laterally Loaded Piles. J. for Soil Mech. and Found. Engrg., ASCE, Vol. 86, SM5, pp. 63-81.

Matlock, H., Foo, H. C. and Bryant, L. M. (1978). Simulation of lateral pile behaviour under earthquake motion. Proceedings of the ASCE Speciality Conference on Earthquake Engineering and Soil Dynamics, Pasadena, California, 2, 1065-1084.

Matsui, T. (1993). Case studies on cast-in-place bored piles and some considerations for design. Proceedings BAP II, Ghent, Balkema, Rotterdam, pp. 77-102.

Mayne, P.W. and Kulhawy, F.H. (1982). Ko-OCR Relationships in Soil. Journal of the Geotechnical Engineering Division, ASCE, 108(6), 851 – 852.

Mayne, P.W. and Mitchell, J.K. (1988). Profiling of Overconsolidation Ratio in Clays by Field Vane. Canadian Geotechnical Journal, Vol. 25, No. 1, pp. 150-158.

Mayne, P.W. and Poulos, H.G. (1999). Approximate displacement influence factors for elastic shallow foundations. Journal of Geotechnical and Geoenvironmental Engineering, 125(6): 453-460.

McGown, A., Andrawes, K.Z., Yeo, K.C. and Dubois, D. (1984). Load-Strain-Time Behaviour of Tensar Geogrids. Symposium on Polymer Grid Reinforcement in Civil Engineering, ICE, London.

McKeen, R.G. and Hamberg, J. (1981), Characterization of expansive soils. Research Record No. 790, Transportation Research Board, pp. 73-78.

McKeen, R.G. and Nielsen, J.P. (1978), Characterization of Expansive Soils for Airport Pavement Design. U.S. Dept. of Transportation, Federal Aviation Administration, Report No. FAA-120-78-59.

McVay, M.C., Townsend, F.C. and Williams, R.C. (1992). Design of socketed drilled shafts in limestone. Journal of Geotechnical Engineering, ASCE, ASCE, Vol. 118, No. 10, pp. 1626-1637.

Meek, J. W. and Veletsos, A. S. (1974). Simple models for foundations in lateral and rocking motions. Proceedings of the 5th World Conference on Earthquake Engineering, Rome, 2, 2610-2613.

Meek, J. W. and Wolf, J. P. (1992a). Cone models for homogeneous soil. Journal of Geotechnical Engineering, ASCE, 118 (4), 667-685.

Meek, J. W. and Wolf, J. P. (1992b). Cone models for soil layer on rigid rock. Journal of Geotechnical Engineering, ASCE, 118 (4), 686-703.

Meek, J. W. and Wolf, J. P. (1994). Cone models for an embedded foundation. Journal of Geotechnical Engineering, ASCE, 120 (1), 60-80.

Meigh, A.C. and Nixon, I.K. (1961). Comparison of In Situ Tests for Granular Soils. Proc. 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. 1, pp. 499-507.

Menard, L. (1956). An Apparatus for Measuring the Strength of Soils in Place. M.Sc. Thesis, University of Illinois, Urbana.

Menard, L. (1962). Comportement d'une Fondation Profonde Soumise a des Efforts de Renversement. Sols Soils, Vol. 3, No. 4, pp. 9-23.

Menard, L. (1965). Regle pour le Calcul de la Force Portante et du Tassement des Fondations en Fonction des Resultats Pressiometri ques. Proceedings 6th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Montreal, Vol. 2., pp. 295-299.

Mesri, G. and Godlewski, P.J. (1977). Time and Stress Compressibility Interrelationship. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 103, GT5, pp. 417-430.

Meyer, B.J. (1979). Analysis of single piles under lateral loading. Ph.D. Thesis, University of Texas, Austin.

Meyerhof, G.G. (1956). Penetration Tests and Bearing Capacity of Cohesionless Soils. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 82, SM1, pp. 1-19.

Meyerhof, G.G. (1957). The Ultimate Bearing Capacity of Foundations on Slopes. Proceedings 4th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), London, Vol. 1, pp. 384-389.

Meyerhof, G.G. (1963). Some Recent Research on Bearing Capacity of Foundations. Canadian Geotechnical Journal, Vol. 1, No. 1, pp. 16-26.

Meyerhof, G.G. (1965). Shallow foundations. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 91, SM2, pp. 21-31.

Meyerhof, G.G. (1970). Safety Factors in Soil Mechanics. Canadian Geotechnical Journal, Vol. 7, No. 4, pp. 349-355.

Meyerhof, G.G. (1976). Bearing Capacity and Settlement of Pile Foundations. The Eleventh Terzaghi Lecture, Nov. 6, 1975. J. of Geotech. Engrg. Div., ASCE, Vol. 102, GT3, pp. 195-228. Discussions in Vol. 103, GT3 and GT4, Closure in Vol. 103, GT9.

Meyerhof, G.G. (1982). Limit States Design in Geotechnical Engineering. Structural Safety Journal, Vol. 1, pp. 67-71.

Meyerhof, G.G. (1984.) Safety factors and limit states analysis in geotechnical engineering. Canadian Geotechnical Journal, Vol. 21, pp. 1-7.

Meyerhof, G.G. (1995). }Development of geotechnical limit state design. Canadian Geotechnical Journal, Vol. 32, pp. 128-136.

Meyerhof, G. G. and Sastry, V. V. R. N. (1978). Bearing capacity of piles in layered soils. Part I and II. Canadian Geotechnical Journal, vol. 15, No. 2, 171-189.

Meyerhof, G. G., Sastry, V. V. R. N., and Yalcin, A. S. (1988). Lateral resistance and deflection of flexible piles. Canadian Geotechnical Journal, 25(3): 511-522.

Micic S., Shang, J.Q. and Lo, K.Y. (2003). Electrocementation of a Marine Clay Induced by Electrokinetics. International Journal of Offshore and Polar Engineering, Vol.13, No.4.

Micic S., Shang, J.Q. and Lo, K.Y. (2003). Load carrying capacity enhancement of skirted foundation element by electrokinetics. International Journal of Offshore and Polar Engineering, Vol.13, No.3, 182-189.

Middendorp, P. and van Weele, P.J. (1986). Application of characteristic stress wave method to offshore practice.

Proceedings of 3rd International Conference on Numerical Methods in Offshore Piling. Nantes, France.

Middendorp. P., Bermingham, P. and Kuiper, B. (1992). Statnamic testing of foundation pile. Proceedings of the 4th International Conference on the Application of Stress-Wave Theory to Piles, The Hague, pp. 585-588.

Miller, R.D. (1978). Frost Heaving in Non-Collided Soils. Proc 3rd Int. Conf. on Permafrost, Edmonton, Alberta, Canada, National Research Council, Vol. 1, pp. 707-713.

Milligan, G.W.E., Jewell, R.A., Houlsby, G.T. and Burd, H.J. (1989). A new approach to the design of unpaved roads – part I. Ground Engineering. Vol. 22, No. 3, pp. 25-29.

Mitchell, J.K. (1981) Soil Improvement-State-of-the-Art Report, Session 12, Proceedings of the Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 4, pp. 506-565.

Mitchell, J.K. (1988). New Developments in Penetration Tests and Equipment. International Symposium on Penetration Testing (ISOPT-1). Orlando, Florida, USA, Proc. Vol. 1, pp. 245-262, A.A. Balkema.

Mitchell, J.K. (1993), Fundamentals of Soil Behavior. John Wiley and Sons, New York, N.Y., 422 p.

Mitchell, J.K., Guzikowski, F. and Villet, W.C.B. (1978). The Measurement of Soil Properties In-Situ. Report prepared for U.S. Department of Energy, Contract W-7405-ENG-48, Lawrence Berkeley Laboratory, University of California, Berkeley, CA, 67 p.

Mizuno, H. and Iiba, M. (1992). Dynamic effects of backfill and piles on foundation impedance. Proceedings of 10th World Conference on Earthquake Engineering, Madrid, Spain, Vol. 3, pp. 1823-1828.

Mlynarek, J., Lafleur, J., Rollin, R. and Lombard, G. (1993). Filtration opening size of geotextiles by hydrodynamic sieving, Geotechnical Testing Journal. ASTM, Vol. 16, No. 1, 61–69.

Mohamedelhassan, E. and Shang, J.Q. (2001). Effects of Electrode Materials and Current Intermittence in electroosmosis. Ground Improvement. Vol.5, No.1. 3-11.

Mohamedelhassan, E. and Shang, J.Q. (2003). Electrokinetics generated pore fluid and ionic transport in an offshore calcareous soil. Canadian Geotechnical Journal, Vol.40, 1185-1199.

Mononobe, N. and Matsuo, H. (1929). On the determination of earth pressures during earthquakes. Proceedings. World Engineering Congress, 9 pp.

Morgenstern, N.R. and Eigenbrod, K.D. (1974). Classification of Argillaceous Soils and Rocks. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 100, GT10, pp. 1137-1156.

Morgenstern, N.R. and Sego, D.C. (1981). Performance of Temporary Tie-Backs Under Winter Conditions. Canadian Geotechnical Journal, Volume 18, No. 4, pp. 566-572.

Morin, P., Leroueil, S. and Samson, L. (1983). Preconsolidation Pressure of Champlain Clays; Part I: In Situ Determination. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 782-802.

Morrison, K.I. and Watts, B.D. (1985). Soil Modulus, Friction and Base Resistance from Simple Pile Load Tests on End-bearing Piles. 38th Canadian Geotechnical Conference, Edmonton, Alberta, pp. 273-281.

Moseley, M.P. (1993). Ground Improvement. Blackie Academic & Professional, Chapman & Hall. CRC Press Inc., Boca Raton, Florida, USA.

Moseley, M.P., and Priebe, H. J. (1993) Vibro Techniques. Ground Improvement, M. P. Moseley, Editor, Blackie Academic & Professional, pp. 1-19.

Mostafa, Y.E. and El Naggar, M.H. (2002). Dynamic analysis of laterally loaded pile groups in sand and clay. Canadian Geotechnical Journal, Vol. 39, No. 6, pp. 1358-1383.

Murchison, J.M. and O'Neill, M.W. (1984). Evaluation of p-y relationships in cohesionless soils. Analysis and design of pile foundations, Ed. J.R. Meyer, ASCE, 174-191.

Musser, S.C. (1996). Utah DOT's testing program to determine the soil-structure interaction of pile groups under lateral loads. Proceedings of 4th Caltrans Seismic Research Workshop, California Department of Transportation, Sacramento, 1996.

Mylleville, B.L.J. and Rowe, R.K. (1991). On the design of reinforced embankments on soft brittle clays, Geosynthetics'91. Industrial Fabrics Association International, Atlanta, GA, pp. 395-408.

Mylonakis, G., Nikolaou, A. and Gazetas, G. (1997). Soil-pile-bridge seismic interaction: kinematic and inertial effects. Part I: soft soil. Journal of Earthquake Engineering and Structural Dynamics, Vol. 26, No. 3, pp. 337-359.

National Building Code of Canada (1990). Associate Committee on the National Building Code. National Research Council of Canada.

National Bureau of Standards (1972). Corrosion on Steel Piling in Soils. Monograph 127, U.S. Department of Commerce, Washington.

National Concrete Masonry Association (1996). Design Manual for Segmental Retaining Walls. (2nd Edition) (Collin, J.G. editor), 2302 Horse Pen Road, Herndon, VA 20171-3499, USA.

NAVFAC (1971), Soil Mechanics, Foundations and Earth Structures, Design Manual. NAVFAC DM-7, Naval Facilities Engineering Command, Department of the Navy, Bureau of Yards and Docks, Washington, DC.

NAVFAC (1982). Soil Mechanics. Design Manual 7.1. Department of the Navy Naval Facilities Engineering Command, Alexandria, VA, US Government Printing Office, Washington, DC, USA.

NAVFAC (1986). Design Manual 7.2, Foundations and Earth Structures. Naval Facilities Engineering Command, U.S. Department of the Navy, Alexandria, VA.

NAVFAC (1988). Design Manual 7.2, Naval Facilities Engineering Command. U.S. Department of the Navy, Alexandria, VA.

NAVFAC DM 7.01 (1986). Soil Mechanics - Design Manual 7.01. United States Department of the Navy, Naval Facilities Engineering Command.

NBCC (1977). Commentaries to Part 4, Supplement to National Building Code of Canada (NBCC). Canadian Commission on Building and Fire Codes. National Research Council of Canada, Ottawa, Canada.

NBCC (1995). National Building Code of Canada (NBCC). Canadian Commission on Building and Fire Codes. National Research Council of Canada, Ottawa, Canada. 571 p.

NBCC (1996). User's Guide – NBC 1995, Structural Commentaries (Part 4). Institute for Research in Construction (ICR), Ottawa, Canada, 135 p.

NBCC (2005). National Building Code of Canada (NBCC) Volumes 1 and 2. 12th Edition 2005, National Research Council of Canada, Ottawa, Canada.

NBCC (2005). User's Guide – NCB 2005, Structural Commentaries (Part 4 of Division B). Canadian Commission on Building and Fire Codes. National Research Council of Canada, Ottawa, Canada.

Nelson, J.D. and Miller, D.J. (1992), Expansive Soils: Problems and Practice in Foundation and Pavement Engineering. John Wiley & Sons, Inc., New York, N.Y.

Newmark, N. (1965). Effects of Earthquakes on Dams and Embankments. Geotechnique, Vol. 15, No. 2, pp. 139-160.

Newmark, N.M. (1942). Influence Charts for Computation of Stresses in Elastic Foundations. University of Illinois, Engineering Experiment Station, Bulletin 338, 28 p.

Nixon, J.F. (1983). Practical Applications of a Versatile Geothermal Simulator. American Society of Mechanical Engineers, Journal of Energy Resources, Technology Volume 105, No. 4, pp. 442-447.

Nixon, J.F. (1987). Ground Freezing and Frost Heave - A Review. Northern Engineer, Volume 19, 3 & 4, Fall/ Winter, pp. 8-18.

Nixon, J.F. (1991). Discrete Ice Lens Theory for Frost Heave in Soils. Canadian Geotechnical Journal, Volume 28, No. 6, pp. 843-859.

Nogami, T. and Novak, M. (1976). Soil-pile interaction in vertical vibration. International Journal of Earthquake Engineering and Structural Dynamics, 4 (3), 277-293.

Nogami, T. (1980). Dynamic stiffness and damping of pile groups in inhomogeneous soil. Proceedings of Session on Dynamic Response of Pile Foundations: Analytical Aspects, ASCE National Convention, October, Florida, 31-52.

Noshin, Z., Fredlund, D.G. and Pufahl, D.E. (1999). Interslice force functions for computing active and passive earth forces. Canadian Geotechnical Journal, Vol. 36, 1015 - 1029.

Novak, M. (1974). Dynamic stiffness and damping of piles. Canadian Geotechnical Journal, 11, 574-598.

Novak, M. (1979). Soil pile interaction under dynamic loads. Proceedings of International Symposium on Numerical Methods in Off-Shore Piling, London, England, pp. 41-50.

Novak, M. and Aboul-Ella, F. (1978). Impedance functions of piles in layered media. Journal of the Engineering Mechanics Division, ASCE, 104 (EM3), 643-661.

Novak, M and Beredugo, Y. O. (1972). Vertical vibration of embedded footings. Journal of the Soil Mechanics and Foundations Division, ASCE, 98 (SM12), 1291-1310.

Novak, M. and El Hifnawy, L. (1983). Vibration of hammer foundations. Journal of Soil Dynamics and Earthquake Engineering, Vol. 2, No. 1, pp. 43-53.

Novak, M. and El-Sharnouby, B. (1983). Stiffness and damping constants of single piles. Journal of the Geotechnical Engineering Division, ASCE, 109 (GT7), 961-974.

Novak, M. and El-Sharnouby, B. (1984). Evaluation of dynamic experiments on pile group. Journal of Geotechnical Engineering, ASCE, 110 (6), pp. 738-756.

Novak, M. and Grigg, R.F. (1976). Dynamic experiments with small pile foundations. Canadian Geotechnical Journal, Vol. 13, No. 4, pp. 372-385.

Novak, M. and Mitwally, H. (1987). Random response of offshore towers with pile-soil-pile interaction. Proceedings of 6th International Symposium on Offshore Mechanics and Arctic Engineering, Houston, 1, 329-336.

Novak, M. and Nogami, T. (1977). Soil-pile interaction in horizontal vibration. International Journal of Earthquake Engineering and Structure Dynamics, 5 (3), 263-282.

Novak, M. and Sachs, K. (1973). Torsional and coupled vibrations of embedded footings. International Journal of Earthquake Engineering and Structure Dynamics, 2 (1), 11-33.

Novak, M. and Sheta, M. (1980). Approximate approach to contact problems of piles. Proceedings of Geotechnical Engineering Division, ASCE National Convention Dynamic Response of pile Foundations: Analytical Aspects. October, Florida, 53-79.

Novak, M., Nogami, T. and Aboul-Ella, F. (1978). Dynamic soil reactions for plane stain case. Journal of the Engineering Mechanics Division, ASCE, 104 (EM4), 953-959.

Novak, M., El Naggar, M. H., Sheta, M., El-Hifnawy, L., El-Marsafawi, H., and Ramadan, O. (1999). DYNA5 a computer program for calculation of foundation response to dynamic loads. Geotechnical Research Centre, The University of Western Ontario, London, Ontario.

O'Brien, A.J. and Lovell, D.W. (1983). Pile Testing for Waterfront Structures. American Society of Civil Engineers (ASCE) Proceedings Speciality Conference on Port Modernization, New Orleans, pp. 495-511.

OHBDC – Ministry of Transportation, Ontario. (1983). Ontario Highway Bridge Design Code and Commentary. (Two Volumes). Second Edition, Ministry of Transportation Ontario, Downsview, Ontario. 634 p.

OHBDC – Ministry of Transportation, Ontario. (1992). Ontario Highway Bridge Design Code and Commentary (Two Volumes). Third Edition, Ministry of Transportation Ontario, Downsview, Ontario.

Okabe, S. (1926). General theory of earth pressures. Journal of the Japan Society of Civil Engineering. Vol. 12, No. 1.

Olson, R.E. and Daniel D.E. (1981). Measurement of the Hydraulic Conductivity of Fine-Grained Soils. American Society for Testing and Materials (ASTM) Special Technical Publication, STP 746, pp. 18-64.

Olson, S.M. and Stark, T.D. (2002). Liquefied strength ratio from liquefaction flow failure case histories. Canadian Geotechnical Journal, 39(3): 629-647.

O'Neill, M.W. (2001). Side resistance in piles and drilled shafts. The Thirty-Fourth Karl Terzaghi Lecture, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, N0. 1, pp. 3-16.

O'Neill, M.W. and Poormoayed, N. (1980), Methodology for Foundations on Expansive Soils. Journal of Geotechnical Engineering Division, ASCE, Vol. 106, GT12, pp. 1345-1367.

O'Neill, M.W. and Reese, L.C. (1999). Drilled shafts: Construction, procedures and design methods. ADSC-TL-4, FHWA-IF-99-025.

O'Neill, M.W., Ghazzaly, O.I. and Ha, H.B. (1977). Analysis of three-dimensional pile groups with nonlinear soil response and pile-soil-pile interaction. Proceedings of 9th Offshore Technology Conference, Houston, paper OTC2838, pp. 245-256.

Osterberg, J. (1989). New Device for Load Testing Driven and Drilled Shafts Separates Skin Friction and End Bearing. Proceeding International Piling and Deep Foundations, London, pp.421-427.

Ovesen, N.K. (1981). Towards an European code for foundation engineering. Ground Engineering. Vol. 14(7), pp. 25-28.

Ovesen, N.K. (1993). Eurocode 7: An European code of practice for geotechnical design. In Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, Denmark, May 26-28. Sponsored by the Danish Geotechnical Society, Vol. 3, pp. 691-710.

Ovesen, N.K., and Orr, T. (1991). Limit states design – the European perspective. Proceedings of Geotechnical Engineering Congress 1991. American Society of Civil Engineers, Special Publication No. 27, Vol. II, pp. 1341-1352.

Peck, R.B. (1962). Art and science in subsurface engineering. Geotechnique, 12(1): pp.60-66.

Peck, R.B. (1969). Deep excavations and tunneling in soft ground, State-of-the-Art Report. Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, pp. 225 - 290. Note: the section on tunneling has not been included in this collection.

Peck, R.B. and Bazaraa, A.S. (1969). Discussion on Settlement of Spread Footings on Sand. Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 95, No. SM3, pp. 905-909.

Peck, R.B., Hanson, W.E. and Thornburn, T.H. (1974). Foundation Engineering. Second Edition, John Wiley and Sons, 514 p.

Pells, P.J.N. and Turner, R.M. (1979). Elastic Solutions for Design and Analysis of Rock Socketed Piles. Canadian Geotechnical Journal, Vol. 16, No. 3, pp. 481-487.

Penner, E. (1974). Uplift Forces on Foundations in Frost Heaving Soils. Canadian Geotechnical Journal, Vol. 11, No. 3, pp. 323-338.

Penner, E. and Crawford, C.B. (1983). Frost Action and Foundations. National Research Council, Division of Building Research, Paper No. 1090, 53 p.

Penner, E. and Gold, L.W. (1971). Transfer of Heaving Forces by Adfreezing to Columns and Foundation Walls in Frost-Susceptible Soils. Canadian Geotechnical Journal, Vol. 17, No. 2, pp. 261-285.

Penner, E. and Goodrich, L.E. (1983). Adfreeze Stresses on Steel Pipe Piles, Thompson, Manitoba. Proceedings, 4th International Conference on Permafrost. National Academy Press, Washington, D.C. pp. 979-983.

Penzien, J., Scheffey, C. F. and Parmelee, R. A. (1964). Seismic analysis of bridges on long piles. Journal of the Engineering Mechanics Division, ASCE, 90 (EM3), 223-254.

Perkins, S.W. and Ismeik, M. (1997). A synthesis and evaluation of geosynthetic-reinforced base layers in flexible pavements. Part II, Geosynthetics International, Vol. 4, No. 6, pp. 605-621.

Phoon, K.K., Becker, D.E., Kulhawy, F.H., Honjo, Y., Ovesen, N.K. and Lo, S.R. (2003). Why consider reliability analysis for geotechnical limit state design. Proceedings of LSD2003: International Workshop on Limit State Design in Geotechnical Engineering Practice - Phoon, Honjo and Gilbert (Editors). Cambridge, Massachusetts, USA. June 26, 2003. World Scientific Publishing Company.

Poulos, H. G. (1968). Analysis of settlement of pile groups. Geotechnique, 18, 449-471.

Poulos, H. G. (1971). Behaviour of laterally loaded piles: II- pile groups. Journal of the Soil Mechanics and Foundations Division, ASCE, 97 (SM5), 733-751.

Poulos, H. G. (1979). Group factors for pile-deflection estimation. Journal of the Geotechnical Engineering Division, ASCE, 105 (GT12), 1489-1509.

Poulos, H.G. (1982). Developments in the Analysis of Static and Cyclic Lateral Loading of Piles. Res. Rep. No. 425, Univ. of Sydney, Nov.

Poulos, H.G. (1982). Single pile response to cyclic lateral load. Journal of Geotechnical Engineering, ASCE, Vol. 108, No. 3, pp. 355-375.

Poulos, H.G. (1985). Ultimate Lateral Pile Capacity in a Two-Layer Soil. Geotechnical Engineering, Vol. 16 (1), pp. 25-37.

Poulos, H.G. (2000). Foundation settlement analysis – Practical procedures versus modern research. The Eighth Spencer J. Buchanan Lecture, College Station, Texas, 34 pp.

Poulos H.G. and Davis, E.H. (1972). The Development of Negative Friction with Time in End-Bearing Piles. Australian Geomechanics Journal, Vol. G2, No. 1, pp. 11-20.

Poulos, H.G. and Davis, E.H. (1974). Elastic Solutions for Soil and Rock Mechanics. John Wiley and Sons, New York.

Poulos, H.G. and Davis, E.H. (1980). Pile Foundations Analysis and Design. New York: John Wiley and Sons.

Poulos, H.G. and Randolph, M.F. (1983). A study of two methods for pile group analysis. Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 3, pp. 355-372.

Poulos, H.G., Carter, J.P. and Small, J.C. (2001). Foundations and retaining structures- Research and practice. Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, Istanbul, Vol. 4, pp. 2527-2606.

Prakash, S. and Puri, V. K. (1988). Foundations for Machines: Analysis and Design. John Wiley and Sons, Inc., New York, N. Y.

Prakash, S. and Sharma, H.D. (1990). Pile foundations in engineering practice John Wiley & Sons, Inc., New York.

Prevost, J. H. and Scanlan, R. H. (1983). Dynamic soil-structure interaction: centrifugal modelling. Journal of Soil Dynamics and Earthquake Engineering, Vol. 2, No. 4, pp. 212-221.

Priest, S.D (1985). Hemispherical Projection Methods for Rock Engineering. George Allen and Unwin, London.

Priest, S.D. (1993). Discontinuity Analysis for Rock Engineering. London: Chapman and Hall, 473 pages.

Priest, S.D., and Hudson, J.A. (1976). Discontinuity spacings in rock. Int. J. of Rock Mech. Min. Sci. & Geomech. Abstr., 13, 135–148.

Qian, J.H., Zhao, W.B., Cheung, W.B. Cheung, Y.K., and Lee, P.K.K. (1992). The theory and practice of vacuum preloading. Computers and Geotechnics, 13, 103-118.

Qian, X., Koerner, R.M. and Gray, D.H. (2000). Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, Pearson Education Canada Ltd, Toronto, Canada.

Quigley, R.M. (1980). Geology, Mineralogy and Geochemistry of Canadian Soft Soils: A Geotechnical Perspective. Canadian Geotechnical Journal, Vol. 17, No. 2, pp. 261-285.

Radforth, N.W. and Brawner, C.O. (1977). Muskeg and the Northern Environment in Canada. The Muskeg Subcommittee of the National Research Council of Canada (NRC) Associate Committee on Geotechnical Research. University of Toronto Press.

Randolph, M.F. (1981). Response of flexible piles to lateral loading. Geotechnique, Vol. 31, No. 2, pp. 247-259.

Randolph, M.F. (1987). PIGLET, A Computer Program for the Analysis and Design of Pile Groups. Department of Civil Engineering., University of Western Australia, Research Report GEO 87036.

Randolph, M.F. and Wroth, C. P. (1978). Analysis of deformation of vertically loaded piles. Journal of Geotechnical Engineering, ASCE, 114(12): 1465-1488.

Randolph, M.F. and Wroth, C.P. (1982). Recent Developments in Understanding the Axial Capacity of Piles in Clay. Ground Engineering, Vol. 15, No. 7, pp. 17-25, 32.

Rankine, W.J.M. (1856). On The Mathematical Theory of The Stability of Earth-work and Masonry. Proceedings of Royal Society, Vol. 8.

Rausch, E. (1950). Maschinen Fundamente. VDI-Verlag, Dusseldorf, (in German), Chapter 6, 107-232.

Rausche, F. Moses, F. And Goble, G. G. (1972). Soil Resistance Predictions from Pile Dynamics. ASCE, Journal For Soil Mechanics And Foundation Engineering, Vol. 95, SM9, pp. 917-925.

Rausche, F., Goble, G.G. and Likins, G.E. (1985). Dynamic determination of pile capacity. Journal of Geotechnical Engineering Division, ASCE, Vol. 111, pp. 367-383.

Rausche, F., Moses, F. and Goble, G.G. (1972). Soil Resistance Predictions from Pile Dynamics. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 95, SM9, pp. 917-925.

Raymond, G.P. (1969). Construction Method and Stability of Embankments on Muskeg. Canadian Geotechnical Journal, Vol. 6, pp. 81-96.

Raymond, G.P. (1982). Geotextiles for Railroad Bed Rehabilitation. Proceedings, 2nd International Conference on Geotextiles, Las Vegas, USA, August 1982, Vol. 2, pp. 479-484.

Raymond, G.P. (1984). Research on Geotextiles for Heavy Haul Railways. Canadian Geotechnical Journal, May 1984, Vol. 21, pp. 259-276.

Raymond, G.P. (1986a). Installation Factors Affecting Performance of Railroad Geotextiles. Transportation Research Record, Transportation Board, Washington, D.C., Record 1071, 1986, pp. 64-71.

Raymond, G.P. (1986b). Performance Assessment of a Railway Turnout Geotextile. Canadian Geotechnical Journal, Vol. 23, 1986, pp. 472-480.

Raymond, G.P. (1986c). Geotextile Application for a Branch Line Upgrading. Geotextiles and Geomembranes, Vol. 3, 1986, pp. 91-104.

Raymond, G.P. (1988). Railway Track Geotextiles. 3rd Canadian Symposium on Geosynthetics, Kitchener, BiTech Publications Ltd., Vancouver, pp. 23-28.

Raymond, G.P. (1992). Reinforced sand behaviour overlying compressible subgrades. Journal of Geotechnical Engineering Division, ASCE, Vol. 118, No. 11, pp. 1663-1680.

Raymond, G.P. (1999). Railway rehabilitation geotextiles, Geotextiles and Geomembranes. Vol. 17, pp. 213-230.

Raymond, G.P. (2001). Failure and reconstruction of a gantry crane ballasted track. Canadian Geotechnical Journal, Vol. 38, No. 3. pp. 507 - 529.

Raymond, G.P. and Bathurst, R.J. (1990). Test results on exhumed railway track geotextiles. 4th International Conference on Geotextiles and Geomembranes, The Hague, Holland, Vol. 1, pp. 197-202.

Raymond, G.P. and Giroud, J.P. (editors) (1993). Geosynthetics Case Histories. International Society for Soil Mechanics and Geotechnical Engineering, Bi-Tech Publishers, Richmond, B.C., Canada.

Raymond, G.P., Bathurst, R.J. and Hajek, J. (2000). Evaluation and suggested improvements to highway edge drains incorporating geotextiles, Geotextiles and Geomembranes. Vol. 18, No. 1, pp. 23-46.

Reddi, L.N. (2003). Seepage in soils: Principles and Applications. Wiley, p. 416.

Reese, L.C. and Wang, S.T. (1996). Technical manual for documentation for program group. Ensoft Inc., Austin, Tx.

Reese, L. C., and Welch, R. C. (1975). Lateral Loading of Deep Foundations in Stiff Clay, Journal of the Geotechnical Engineering Div., ASCE, Vol. 101, No. GT7, July, pp. 633-649.

Reese, L. C., Cox, W. R., and Koop, F. D. (1975). Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay, Proceedings, 7th Annual Offshore Technology Conference, Houston, Texas, Vol. 2, pp. 671-690.

Reese, L. C., Cox, W. R., and Koop, F. D.,(1974). Analysis of Laterally Loaded Piles in Sand, Paper No. OTC 2080, Proceedings, 6th Annual Offshore Technology Conference, Houston, Texas, Vol. 2, pp.473-483.

Reese, L.C. (1977). Laterally Loaded Piles: Program Documentation. ASCE Journal of the Geotech. Eng. Division, Vol. 103, No. GT4, pp. 287-305.

Reinforced Soil Structures Vol. 1 Design and Construction Guidelines (1989). Federal Highway Administration Report (Christopher et al. Eds.).

Reissner, E. (1936). tationare, axialsymmetrische durch eine schuttelnde masseerregte schwigungen eines homogenen elastischen halbraumes. (in German) Ingenieur-Archiv, 7 (6), 381-396.

Richard, R. and Elms, D. (1979). Seismic Behaviour of Gravity Retaining Walls. Journal of Soil Mechanics and Foundations Division, ASCE, Reston, VA, 105(4): 449-464.

Richardson, G.N. and Koerner, R.M. (1987). Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments. Final Report U.S. E.P.A. Contract No. 68-033-3338, CR1, Philadelphia, PA, U.S.A.

Richardson, G.N. and Middlebrooks, P. (1991). A simplified method for silt fences, Geosynthetics'91. Industrial Fabrics Association International, pp. 879-888.

Richart, F.E. (1962). Foundation Vibrations. American Society of Civil Engineers (ASCE) Transactions, Vol. 127, part 1, pp. 863-925.

Richart, F. E. (1975). Foundation Vibrations. Foundation Engineering Handbook, 1st Edition. Fang, H. Y. Editor, Van Nostrand Reinhold, New York, N.Y.

Richart, F. E., Hall, J. R. and Woods, R. D. (1970). Vibrations of soils and foundations. Pentice-Hall, Inc., Englewood Cliffs, N.J.

Richter, C. F. (1958). Elementary Seismology. W. H. Freeman, San Francisco, CA.

Rilem Report 4 (1991). Geomembranes: Identification and Performance Testing. (Rollin, A.L. and Rigo, J.M., editors) Van Nostrand Reinhold.

Robertson, P.K. (1986). In-Situ Testing and Its Application to Foundation Engineering. Canadian Geotechnical Journal, Vol. 23, No. 4, pp. 573-594.

Robertson, P.K. (1990). Soil Classification by the Cone Penetration Test. Canadian Geotechnical Journal, Vol. 27, pp. 151-158.

Robertson, P.K. (1997). Risk Based Site Characterization. Graduate course in Site Investigation, University of Alberta, Edmonton, Alberta.

Robertson, P.K. and Campanella, R.G. (1983a). Interpretation of Cone Penetration Tests — Part I: Sand. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 718-733.

Robertson, P.K. and Campanella, R.G. (1983b). Interpretation of Cone Penetration Tests — Part II: Clay. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 734-733.

Robertson, P.K. and Wride, C.E. (1998). Evaluating cyclic liquefaction potential using the cone penetration test. Vol. 35, No. 3, pp. 442-459.

Robertson, P.K., Campanella, R.G. and Wightman, A. (1983). SPT—CPT Correlations. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 109, GT11, pp. 1449-1459.

Robertson, P.K., Hughes, J.M.O., Campanella, R.G., Brown, P. and McKeown, S. (1986). Design of Laterally Loaded Piles using the Pressuremeter. ASTM Special Technical Publication 950, The Pressuremeter and its Marine Applications.

Robertson, P.K., Campanella, R.G., Brown, P.T., Grof, I. and Hughes, J.M.O. (1988). Design of Axially and Laterally

Loaded Piles. Canadian Geotechnical Journal, Vol. 22, No. 4, pp. 518-527.

Robertson, P. K., Davies, M. P., and Campanella, R. G. (1989). Design of laterally loaded driven piles using the flat dilatometer. ASTM Geotechnical Testing Journal. Vol. 12, No. 1, pp. 30-38.

Robertson, P.K., Woeller, D.J. and Gillespie, D. (1990). Evaluation of Excess Pore Pressure and Drainage Conditions around Driven Piles using the CPTU. Canadian Geotechnical Journal, Vol. 27, pp. 249-254.

Robinsky, E.I. and Bespflug, K.E. (1973). Design of Insulated Foundations. ASCE Journal of Geotechnical Engineering. Volume 99, No. SM9, pp. 649-667.

Rollin, A.L. (1984). Geomembranes. Proceedings of the Canadian Symposium on Geotextiles, Geogrids and Geomembranes, Toronto.

Rollin, A.L., Lafleur, J., Marcotte, M., Dascal, O. and Akber, Z. (1984). Selection Criteria for the Use of Geomembranes in Dams and Dykes in Northern Climate. Proc. Int. Conf. on Geomembranes, Denver, Colorado, June.

Rollin, A.L., Pierson, P. and Lambert, S. (2002). Géomembranes – Guide de choix. Presses Internationales Polytechnique. 274 p.

Rowe, P.W. (1952). Anchored sheet-pile walls. Proceedings of the Institution of Civil Engineers. Vol. 1, No. 1, pp. 27 – 70. Note that a discussion was also published in September of the same year, pp. 616 – 647.

Rowe, P.W. (1956). The Single Pile Subject to Horizontal Force. Geotechnique, Vol. 6, No. 2, pp. 70-85.

Rowe, P.W. (1957). Sheet-pile walls in clay. Proceedings of the Institution of Civil Engineers. Vol. 7, No. 3, pp. 629 – 654.

Rowe, R.K. (1982). The Determination of Rock Mass Modulus Variation with Depth for Weathered or Jointed Rock. Canadian Geotechnical Journal, Vol. 19, No. 1, pp. 29-43.

Rowe, R.K. (editor) (2001). Geotechnical and Geoenvironmental Engineering Handbook 2001. Kluwer Academic Publishing, Norwell, MA, USA.

Rowe, R.K. and Armitage, H.H. (1984). The Design of Piles Socketed into Weak Rock. Research Report GEOT-11-84, Faculty of Engineering. Science, University of Western Ontario, London, Ont.

Rowe, R.K. and Armitage, H.H. (1987a). Theoretical Solutions for Axial Deformation of Drilled Shafts in Rock. Canadian Geotechnical Journal, Vol. 24, pp. 114-125.

Rowe, R.K. and Armitage, H.H. (1987b). A Design Method for Drilled Piers in Soft Rock. Canadian Geotechnical Journal, Vol. 24, pp. 126-142.

Rowe, R.K. and Booker, J.R. (1981a). The behaviour of footings on a non-homogeneous soil mass with a crust - Part I - strip footings. Canadian Geotechnical Journal, 18(2): 250-264.

Rowe, R.K. and Booker, J.R. (1981b). The behaviour of footings on a non-homogeneous soil mass with a crust - Part II - circular footings. Canadian Geotechnical Journal, 18(2): 265-279.

Rowe, R.K. and Mylleville, B.L.J. (1990). Implications of Adopting an Allowable Geosynthetic Strain in Estimating Stability. 4th International Conference on Geotextiles, Geomembranes and Related Products, The Hague, The Netherlands, pp. 131-136.

Rowe, R.K. and Soderman, K.L. (1985a). An Approximate Method for Estimating the Stability of Geotextile-Reinforced Embankments. Canadian Geotechnical Journal Vol. 22, No. 3, pp. 392-398.

Rowe, R.K. and Soderman, K.L. (1985b). Geotextile Reinforcement of Embankments on Peat. Geotextiles and Geomembranes, Vol. 2, No. 4, pp. 277-298.

Rowe, R.K. and Soderman, K.L. (1986). Reinforced Embankments on Very Poor Foundations. International Journal of Geotextiles and Geomembranes, Vol. 4, No. 1, pp. 65-81.

Rowe, R.K. and Soderman, K.L. (1987). Stabilization of Very Soft Soils Using High Strength Geosynthetics: the Role of Finite Element Analyses. Geotextiles and Geomembranes, Vol. 6, No. 1, pp. 53-80.

Rowe, R.K., Quigley, R.M. and Booker, J.R. (2004). Clayey Barrier Systems for Waste Disposal Facilities. E&FN Spon.

Rowley, R.K., Watson, G.H. and Ladanyi, B. (1975). Prediction of Pile Performance in Permafrost Under Lateral Load. Canadian Geotechnical Journal, Vol. 12, No. 4, pp. 510-523.

RTAC (1977). Pavement Management Guide. Roads and Transportation Association of Canada (RTAC, now TAC), Ottawa.

Sage, R. (editor) (1976). CANMET Pit Slope Manual, Chapter 1- Summary'; CANMET Report 76-22.

Sanger, F.J. (1963). Degree-Days and Heat Conduction in Soils. Proceedings International Conference on Permafrost, Lafayette, Indiana, U.S. National Academy of Sciences, Publ. 1287, pp. 253-263.

Sanger, F.J., and Sayles, F.H. (1978). Thermal and Rheological Computations for Artificially Frozen Ground Construction. Engineering Geology, Elsevier, Vol. 13, pp. 311-337.

Sanglerat, G. (1972). The Penetrometer and Soil Exploration. Elsevier Publishing Company, Amsterdam.

Sanin, M.V. and Wijewickreme, D. (2004). Applicability of the empirically based criteria for liquefaction susceptibility of Fraser River silt. In Proc. 57th Canadian Geotechnical Conference, Quebec City, PQ, Canada, October 2004.

Sanin, M.V. and Wijewickreme, D. (2005). Response of a natural deltaic silt under level ground cyclic shear loading. Manuscript in preparation for submission to Soil Dynamics and Earthquake Engineering.

Sanin, M.V. and Wijewickreme, D. (2006). Cyclic shear response of channel-fill Fraser River Delta silt. Soil Dynamics and Earthquake Engineering, 26(9): 854-869.

Schapp, L.H.J. and Zuidberg, H.M. (1982). Mechanical and Electrical Aspects of the Electric Cone Penetration Point. Proceedings Second European Symposium on Penetration Testing (ESOPTII) Amsterdam, Vol. 2, pp. 841-851.

Schmertmann, J.H. (1970). Static Cone to Compute Static Settlement Over Sand. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 96, SM3, pp. 1011-1043.

Schmertmann, J.H. (1975). Measurement of In Situ Shear Strength. American Society of Civil Engineers (ASCE) Geotechnical Engineering Division, Proceedings of the Specialty Conference on In Situ Measurement of Soil Properties, Raleigh, Vol. 2, pp. 57-138.

Schmertmann J.H. (1978). Guidelines for Cone Penetration Test, Performance and Design. Federal Highway Administration (FHWA) Report TS-78-209, Washington, 145 p.

Schmertmann, J.H. (1979). Statics of SPT. American Society of Civil Engineers (ASCE) Journal of Geotechnical Engineering, Vol. 105, GT5, pp. 655-670.

Schmertmann, J.H. (1986). Suggested Method for Performing the Flat Dilatometer Test. ASTM Subcommmittee D.18.02. Geotechnical Testing Journal, ASTM Vol. 9, No. 2, pp. 99-101.

Schmertmann, J.H. and Palacios, A. (1979). Energy Dynamics of SPT. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 105, GT8, pp. 909-926.

Schmertmann, J.H., Hartman, J.P., and Brown, P.R. (1978). Improved strain influence factor diagrams. Journal of Geotechnical Engineering Division, ASCE, 104(8), 1131-1135.

Schnabel, P., Lysmer, J. and Seed, H. B. (1972). SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Schultze, E. and Melzer, K.J. (1965). The Determination of the Density and the Modulus of Compressibility on Non-Cohesive Soils by Sounding. Proceedings 6th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Montreal Vol. 1, pp. 354-358.

Scott, R. F., Liu, H. P. and Ting, J. (1977). Dynamic pile tests by centrifuge modelling. Proceedings of 6th World Conference on Earthquake Engineering, New Delhi, India, Vol. 4, pp. 199-203.

Scott, R. F., Ting, J. and Lee, J. (1982). Comparison of centrifuge and full-scale dynamic pile tests. Proceedings of International Conference on Soil Dynamics and Earthquake Engineering, Southampton, Vol. 1, pp. 299-309.

SDS Drilling Ltd. Becker Denseness Test. technical literature.

Seed, H.B. (1970), Soil Problem and Soil Behaviour. Earthquake Engineering, R.L. Wiegel, Editor, Chapter 10, pp. 227-251.

Seed, H.B. and Idriss, I. M. (1971). Simplified procedure for the evaluation of soil liquefaction potential. Journal of Soil Mechanics and Foundation Division, ASCE, vol. 107, SM9, pp. 1249-1274

Seed, H.B. and Idriss, I. M. (1982). On the importance of dissipation effects in evaluating pore pressures due to cyclic loading. Soil Mechanics – Transient and Cyclic Loads, Eds. G. N. Pande and O. C. Zienkiewicz, pp. 53-70, John Wiley & Sons

Seed, H.B. and Whitman, R.V. (1970). Design of earth retaining structures for dynamic loads. Proceedings, Lateral Stresses in the Ground and Design of Earth-Retaining Structures, ASCE, 103 - 147.

Seed, H.B., Woodward, R.J. Jr., and Lundgren, R. (1962), Prediction of swelling potential for compacted clays. Journal of Soil Mechanics and Foundation Division, ASCE, No. 88, (SM3), pp. 53-87.

Seed, H. B., Idriss, I. M., Makdisi, F. and Banerjee, N. (1975). Representation of irregular stress time-histories by equivalent uniform stress series in liquefaction analysis. Report No. UCB/EERC 75-29, University of California,

Berkeley.
Seed, H. B., Martin, P. P. and Lysmer, J. (1976). Pore-water pressure changes during soil liquefaction. Journal of Geotechnical Engineering Division, ASCE, vol. 102, GT4, pp. 323-345.
Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M. (1984). Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations. Report No. UCB/EERC-84/15, Berkeley. Reprinted in Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 12 pp. 1425-1445.
Seed, H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K. (1986). Moduli and damping factors for dynamic analyses of cohesionless soils. Journal of Geotechnical Engineering, ASCE, vol. 112, No. GT11, pp. 1016-1032.
Seed, R.B. and Harder, L.F., Jr. (1990). SPT-based analysis of cyclic pore pressure generation and undrained residual strength. Proceedings of the H.B. Seed Memorial Symposium, Bi-Tech Publishing Ltd., 2: 351-376.
Seed, R.B., Cetin, K. O., Moss, R. E. S., Kammerer, A. M., Wu, J., Pestana, J. M., Riemer, M. F., Sancio, R.B., Bray, J.D., Kayen, R. E. and Faris, A. (2003) Keynote Presentation. 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Long Beach, California
Seidel, J.P. (1998). Program ROCKET, Dept. of Civil Engineering, Monash University, Clayton, Victoria, Australia.
Semple, R.M. and Rigden, W.J. (1984). Shaft Capacity of Driven Pipe Piles in Clay. Analysis and Design of Pile Foundations, ed. J.R. Meyer, ASCE, New York, pp. 59-79.
Shang, J.Q. Mohamedelhassan, E. and Ismail, M. (2004a) Electrochemical Cementation of Offshore Calcareous Soil. (2004a) Canadian Geotechnical Journal, Vol. 41, No. 5, 877-893.
Shang, J.Q., Lo, K.Y. and Huang, K.M. (1996). On influencing factors in electro-osmotic consolidation. Journal of Geotechnical Engineering, South East Asia Geotechnical Society, 27:2, 23-26.
Shang, J.Q., Mohamedelhassan, M. Ismail, M.A., and Randolph, M.F. (2004b). Electrochemical Cementation of Calcareous Sand for Offshore Foundations Proceedings. the 14th International Offshore and Polar Engineering Conference, Toulon, France, May 22-27 2004.
Shang, J.Q., Tang, M. and Z. Miao (1998). Vacuum Preloading Consolidation of Reclaimed land: a Case Study. Canadian Geotechnical Journal. Vol. 35, No.5, in press.
Sharma, H. D., Sengupta, S. and Harron, G. (1984), Cast-in-place bored piles on soft rock under artesian pressures Canadian Geotechnical Journal, Vol. 21, No. 4, pp. 684-698.
Sherard, J.L., Dunnigan L.P., and Talbot J.R. (1984a). Basic Properties of Sand and Gravel Filters. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 110, No. 6, pp. 684-700.
Sherard, J.L., Dunnigan L.P., and Talbot J.R. (1984b). Filters for Silt and Clay. American Society of Civil Engineers (ASCE) Journal of the Geotechnical Engineering Division, Vol. 10, No. 6, pp. 701-718.
Sherard, J.L., Woodward, R.J., Gizienski, S.F. and Clevenger, W.A. (1963). Earth and Earth-Rock Dams. John Wiley and Sons, New York.
Sheta, M. and Novak, M. (1982). Vertical vibration of pile groups. Journal of the Geotechnical Engineering Division, ASCE, 108 (GT4), 570-590.
Shi, X. and Richards, Jr. R. (1995). Seismic Bearing Capacity with Variable Shear Transfer. In Kramer, S. and Siddharthan, R. (Eds.) Earthquake Induced Movements and Seismic Remediation of Existing Foundations and Abutments. Geotechnical Special Publication No. 55, ASCE, Reston, VA, pp. 17-32.
Shields, D.H., Scott, J.D., Bauer, G.E., Deschenes, J.H. and Barsvary, A.K. (1977). Bearing Capacity of Foundations Near Slopes. Proceedings 9th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Tokyo, Vol. 2, pp. 715-720.
Shukla, S.K. (editor) (2002). Geosynthetics and Their Applications. Thomas Telford Ltd., London, UK.
Shuster, J.A. (1972). Controlled Freezing for Temporary Ground Support. Proceedings 1st North American Rapid Excavation and Tunnelling Conference, Chicago, pp. 863-894.
Simac, M.R., Bathurst, R.J. and Goodrum, R.A. (1991). Design and Analysis of Three Reinforced Soil Retaining Walls. Geosynthetics '91, Atlanta, Georgia.
Skempton, A.W. (1951). The Bearing Capacity of Clays. Building Research Congress, London, Institute of Civil Engineers, Div. I, p. 180.
Skempton, A.W. (1953), The Colloidal Activity of Clays. Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Switzerland, Vol. 1, pp. 57-61.
Skempton, A.W. (1961). Horizontal Stresses in an Over-consolidated Eocene Clay. Proceedings, 5th International

Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. 1, pp. 351-358.
Skempton, A.W. (1977). Slope stability of cuttings in Brown London Clay. Proc. 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 3: 261-270.
Skempton, A.W. (1986). Standard Penetration Test, Procedures and Effects in Sands of Overburden, Relative Density, Particle Size, Aging and Over-consolidation. Geotechnique, Vol. 36, No. 3, pp. 425-447.
Small, J.C. (2001). Shallow Foundations. Geotechnical and Geoenvironmental Handbook. R.K. Rowe, Ed., Chapter 9, Kluwer Academic Publishers, pp. 223-259.
Small, J.C., Booker, J.R. and Davis, E.H. (1976). Elasto-plastic consolidation of soil. International Journal of Solids and Structures, 12: 431-448.
Sowers, G.B. and Sowers, G.F. (1970). Introductory Soil Mechanics and Foundations, 3rd ed. MacMillan Co., New York, N.Y., 1970, 556 pp.
Sprute, R.H., & Kelsh, R.H. (1980). Dewatering fine-particle suspensions with direct current. Proceedings of International Symposium of Fine Particle Process. Las Vegas, Nevada, vol. 2, pp. 1828-1844.
Stas, C.V. and Kulhawy, F.H. (1984). Critical Evaluation of Design Methods for Foundations Under Axial Uplift and Compression Loading. Report EL-3771, Electric Power Research Institute, Palo Alto, Calif.
Steinbrenner, W. (1936). A Rational Method for Determination of Vertical Normal Stress Under Foundations. Proceedings 1st International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Cambridge, Vol., 2, pp. 142-143.
Steuermann, S. (1939). A New Soil Compaction Device. Engineering News-Record, Vol. 122, July 20, pp. 63-64.
Stewart, J., Williamson, R. and Mohney, J. (1977). Guidelines for use of fabrics in construction and maintenance of low-volume roads. USDA, Forest Service, Portland, OR., USA.
Sy, A. and Campanella, R.G. (1991a). An Alternative Method of Measuring SPT. Energy. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics.
Sy, A. and Campanella, R.G. (1991b). Wave Equation Modelling of the SPT. ASCE Geotechnical Engineering Congress.
Sy, A. and Campanella, R.G. (1992a). Dynamic Performance of the Becker Hammer Drill and Penetration Test. Proceedings 45th Canadian Geot. Conference, Toronto.
Sy, A. and Campanella, R.G. (1992b). Dynamic Measurements of the Becker Penetration Test with Implications for Pile Driving Analysis. Proceedings 4th International Conference on the Application of Stress-Wave Theory to Piles, The Hague, Netherlands.
TAC (1997). Pavement Design and Management Guide. Transportation Association of Canada, 323 St. Laurent Blvd., Ottawa, ON K1G 4J8, Canada.
Tajimi, H. (1969). Dynamic analysis of a structure embedded in an elastic stratum. Proceedings of 4th World Conference on Earthquake Engineering, Chile, III, 53-69.
Tan, C.K. and Duncan, J.M. (1991). Settlement of Footings on Sands – Accuracy and Reliability In Proceedings Geotechnical Engineering Congress, ASCE, Geotechnical Special Publication No. 27. Vol. 2, pp. 446-455.
Tavenas, F.A. (1971). The Standard Penetration Test. Discussion, Proceedings 4th PanAmerican Conference on Soil Mechanics and Foundation Engineering, Puerto Rico, Vol. 3, pp. 64-70.
Tavenas, F.A. (1973). Difficulties in the Use of Relative Density as a Soil Parameter. American Society for Testing and Materials (ASTM) Special Technical publication, STP 523, pp. 18-60.
Tavenas, F.A. (1981). Advantages and Limitations of the Piezocone. Proceedings 10th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), Stockholm, Vol. 4, pp. 769-773.
Tavenas, F.A. (1984). Landslides in Canadian Sensitive Clays — A state-of-the art. Proceedings 4th International Symposium on Landslides, Toronto, Vol. 1, pp. 141-153.
Tavenas, F.A. and Leroueil, S. (1981). Creep and Failure of Slopes in Clay. Canadian Geotechnical Journal, Vol. 18, No. 1, pp. 106-120.
Tavenas, F.A., Garneau, R., Blanchet, R. and Leroueil, S. (1978). The Stability of Stage-Constructed Embankments on Soft Clays. Canadian Geotechnical Journal, Vol. 15, No. 2, pp. 283-305.
Tavenas, F.A., Ladd, R.S. and Larochelle, P. (1973). The Accuracy of Relative Density Measurements. American Society for Testing and Materials, ASTM, Special Technical Publication, STP523, pp. 18-60.
Tavenas, F.A., Leblond, P., Jean, P. and Leroueil, S. (1983a). The Permeability of Natural Soft Clays, Part I: Methods of Laboratory Measurement Canadian Geotechnical Journal, Vol. 20, No.4, pp. 629-644.

Tavenas, F.A., Leblond, P., Jean, P. and Leroueil, S. (1983b). The Permeabillity of Natural Soft Clays. Part II: Permeability Characteristics. Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 645-6 60.

Taylor, D.W. (1948). Soil Mechanics. John Wiley and Sons, New York, 700 p.

Teng, W.C. (1975). Soil Stresses for Design of Drilled Caisson Foundations Subjected to Lateral Loads. American Society of Civil Engineers (ASCE) Conference on Analysis and Design in Geotechnical Engineering

Terzaghi, K. (1943). Theoretical Soil Mechanics, Wiley, New York.

Terzaghi, K. (1953). Anchored Bulkheads. American Society of Civil Engineers (ASCE) Separate Publication 262.

Terzaghi, K. (1955). Evaluation of Coefficients of Subgrade Reaction. Geotechnique, Vol. 5, No. 4, pp. 297-326. Discussion in Vol. 6, No. 2, pp. 94-98.

Terzaghi, K. (1965). Sources of error in joint surveys. Geotechnique, 15(3): 287-304.

Terzaghi, K., and Peck, R.P. (1948). Soil Mechanics In Engineering Practice. 1st edition John Wiley & Sons Inc., New York.

Terzaghi, K., and Peck, R.P. (1967). Soil Mechanics In Engineering Practice. 2nd edition, John Wiley & Sons Inc., New York, 729 p.

Terzaghi, K., Peck, R.B. and Mesri, G. (1996). Soil Mechanics in Engineering. Practice 3rd edition, John Wiley & Sons, New York.

Thevanayagam, S., and Nesarajah, S., Editors (1996). International Workshop on Technology Transfer for Vacuum-Induced Consolidation. Engineering and Practice, report of a workshop sponsored the National Science Foundation, Port of Los Angeles, and ISSMFE TC-17, Los Angeles.

Thevanayagam, S., Kavazanjian, E., Jacob, A., and Juran, I. (1994) Prospects of Vacuum-Assisted Consolidation for Ground Improvement of Coastal and Offshore Fills. Geotechnical Special Publication No. 45 ASCE, pp. 90-105.

Thornburn S. (1970). Discussion. Proc. Conf. on Behaviour of Piles, Institution of Civil Engineers, London, pp. 53-54.

Thornthwaite, C.W. (1948), An Approach Towards a Rational Classification of Climate. Geographical Rev., Vol. 38, pp. 55-94.

Thornton, J.S. and Baker, T.L. (2002). Comparison of SIM and conventional methods for determining creep-rupture behavior of a polypropylene geotextile. 7th International Geosynthetics Conference, Nice, France, Vol. 4, pp. 1545-1550.

Thornton, J.S., Paulson, J. and Sandri, D. (1998). Conventional and stepped isothermal methods for characterizing long-term creep strength of polyester geogrids. 6th International Conference on Geotextiles, Geomembranes, and Related Products, Atlanta, GA, USA, pp.691-698.

Tiedemann, D.A. (1973). Variability of Laboratory Relative Density Tests. American Society for Testing and Materials (ASTM) Special Technical Publication, STP 523, pp. 61-73.

Ting, J. M. and Scott, R. F. (1984). Static and dynamic lateral pile group action. Proceedings of 8th World Conference on Earthquake Engineering, San Francisco, Vol. III, pp. 641-648.

Tomlinson, M.J. (1957). The Adhesion of Piles Driven in Clay Soils. Proceedings 4th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE), London, Vol. 2, pp. 66-71.

Transportation Road Research Laboratory (1970). A Guide to the Structural Design of Pavements for New Roads. Road Note 20, Department of the Environment, U.K.

Trochanis, A. M., Bielak J., and Christiano, P. (1988). A Three-Dimensional Nonlinear Study of Piles Leading to the Development of a Simplified Model. A Technical Report of Research Sponsored by The NSF, Grant No. ECE-86/1060, Carnegie Mellon University.

Trow Consulting Engineers Ltd. (1996). Instream sediment control techniques field implementation manual. Ontario Ministry of Natural Resources, Northeast Science and Technology, FG-007, 109 p.

Tschebotarioff, G.P. (1973). Foundations, Retaining and Earth Structures. 2nd Edition. McGraw-Hill, New York.

Tuzuki, M., Inada, O. and Yamagishi, M. (1992). Field testing and analysis of dynamic loaded pile group. Proceedings of 10th World Conference on Earthquake Engineering, Madrid, Spain, Vol. 3, pp. 1787-1790.

U.S. Army Corps of Engineers (1977). Civil Works Construction Guide Specification for Plastic Fabri. Corps of Engineer Specification No. CW-02215, Office, Chief Engineers, U.S. Army Corps of Engineers, Washington, D.C., USA.

U.S. Bureau of Reclamation (1974). Earth Manual. (2nd Edition), 810 p.

U.S.Navy (1983) Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction, Design Manual 7.3, Naval Facilities Engineering Command.

Ulrich, C. M. and Kuhlemeyer, R. L. (1973). Coupled rocking and lateral vibrations of embedded footings. Canadian Geotechnical Journal, 10 (2), 145-160.

USACE WES (1967). The Unified Soil Classification System. Technical Memorandum No. 3-357. U.S. Army Corps of Engineers, Waterways Experiment Station (USACE WES) Vicksburg, Miss.

Van der Merwe, D.H. (1964), The Prediction of Heave from the Plasticity Index and Percent Fraction of Soils. Civil Engineering in South Africa, Vol. 6, No. 6, pp. 103-107.

Van Impe, W. F. (1989) Soil Improvement Techniques and their Evolution, Balkema.

Veletsos, A. S. and Nair, V. V. D. (1974). Torsional vibration of viscoelastic foundation. Journal of the Geotechnical Division, ASCE, 100 (GT3), 225-246.

Veletsos, A. S. and Verbic, B. (1973). Vibration of viscoelastic foundations. Earthquake Engineering and Structural Dynamics, 2, 87-102.

Veletsos, A. S. and Wei, Y. T. (1971). Lateral and rocking vibrations of footings, Journal of the Soil Mechanics and Foundations Division, ASCE, 97 (SM9), 1227-1248.

Vesic, A.S. (1970). Tests on Instrumented Piles, Ogeechee River Site. American Society of Civil Engineers (ASCE) Journal for Soil Mechanics and Foundation Engineering, Vol. 96, SM2, pp. 561-584.

Vesic, A.S. (1975). Bearing Capacity of Shallow Foundations. Foundation Engineering Handbook, H.F. Winterkorn and H. Fang, Eds, Chapter 3, Van Nostrand Reinhold, New York, pp. 121-147.

Vesic, A.S. (1977). Design of Pile Foundations. Transportation Research Board, TRB, National Cooperative Highway Research Program, Washington, Synthesis of Highway Practice No. 42, 68 p.

Waas, G. and Hartmann, H. G. (1981). Pile foundations subjected to dynamic horizontal loads. European Simulation Meeting Modelling and Simulation of Large Scale Structural Systems, Capri, Italy, 17 p.

Walls J.C. and Galbreath L.L. (1987). Railroad ballast reinforcement using geogrids. Geosynthetics'87, International Fabrics Association International, St. Paul, MN, USA, Vol. 1, pp. 38-45.

Wang, W. (1979). Some findings in soil liquefaction. Water Conservancy and Hydroelectric Power Research Institute, Peking, China.

Wei, J. and El Naggar, M.H. (1999). Experimental study of axial behaviour of tapered piles: Reply. Canadian Geotechnical Journal, Vol. 36, No. 6, pp. 1204-1205.

White, E.E. (1973). Sheeting and Bracing Systems for Deep Excavations. World Construction, Vol. 26, No. 1, 20 –24.

White, R.E. (1962). Caissons and Cofferdams. In Foundation Engineering G.A. Leonards, Editor, McGraw-Hill Book Company, New York, pp. 894-964

Whitman, R. and Liao, S. (1985). Seismic Design of Retaining Walls. Miscellaneous Paper GL-85-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Wickham, G.E., Tiedemann, H.R., and Skinner, E.H. (1972). Support determination based on geologic predictions. Proc. North American Rapid Excavation and Tunnelling Conf., Chicago, 43–64.

Wijewickreme, D., Sriskandakumar, S., Byrne, P.M. (2005). Cyclic loading response of loose air-pluviated Fraser River sand for validation of numerical models simulating centrifuge tests. Canadian Geotechnical Journal, 42(2): 550-561.

Williams, A.A.B (1958). The Prediction of Total Heave from the Double Oedometer Test. Discussion: Transactions of the South African Institution of Civil Engineers, Vol. 8, No. 6, pp. 123-124

Williams, A.F., Donald I.B. and Chiu, H.K. (1980) Stress Distributions in Rock Socketed Piles. Proceedings of the International Conference on Structural Foundations on Rock, Sydney Australia, PP. 317-326

Williams, P.J. and Smith, M.W. (1989) The Frozen Earth. Cambridge University Press, Cambridge, U.K. 306 p.

Wilson, G.W., Fredlund, D.G. and Barbour, S.L. (1991), The Evaluation of Evaporative Fluxes from Soil Surfaces for Problems in Geotechnical Engineering. Proceedings of the Canadian Geotechnical Conference, Calgary, AB., Sept. 29-Oct. 2, pp. 68.1-68.9.

Windle D. and Wroth, C.P. (1977). In-Situ Measurement of the Properties of Stiff Clays. Proc. 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 1977, Vol. 1, pp. 347-352.

Winterkorn, H.F. and Fang, H-Y. (1975). Foundation Engineering Handbook. Chapter 16 – Combined and Special

Footings by J.E. Bowles, pp. 516-518. Van Nostrand Reinhold Company, New York, U.S.A.

Withers N.J., Schapp L.H.J. and Dalton J.C.P. (1986) The Development of a Full Displacement Pressuremeter. ASTM STP 950. The Pressuremeter and Its Marine Application, Second International Symposium, pp. 38-56.

Wolf, J. P. (1995). Foundation Vibration Analysis Using Simple Physical Models. Prentice-Hall, Inc., Englewood Cliffs, NJ.

Wolf, J. P. and Darbre, G. R. (1984). Dynamic-stiffness matrix of soil by the boundary-element method: embedded foundations. Earthquake Engineering and Structural Dynamics, 12, 401-416.

Wolf, J.P. and Darbre (1986). Nonlinear soil-structure interaction analysis based on the boundary-element method in time domain with application to embedded foundation. Earthquake Engineering and Structural Dynamics, Vol. 14, pp. 83-101.

Wolf, J. P. and von Arx, G. A. (1978). Impedance functions of a group of vertical piles. Proceedings of ASCE Speciality Conference on Earthquake Engineering and Soil Dynamics, Pasadena, Ca, II, 1024-1041.

Wolf, J. P., von Arx, G. A., de Barros, F. C. P. and Kakubo, M. (1981). Seismic analysis of the pile foundation of the reactor building on the NPP Angra 2. Nuclear Engineering and Design, 65 (3), 329-341.

Wolf, J.P., Meek, J.W. and Song, Ch (1992). Cone models for a pile foundation. Piles Under Dynamic Loads. Edited by S. Prakash, Geotechnical Special Publication No. 34, ASCE, pp. 94-113.

Wroth, C.P. (1975). In Situ Measurement of Initial Stresses and Deformation Characteristics. American Society of Civil Engineers (ASCE) Geotechnical Engineering Division, Proceedings of the Specialty Conference on In situ Measurements of the Properties of Soft Clays, Raleigh, Vol. 2, pp. 181-230.

Wroth, C.P. (1982). British Experience with the Self-Boring Pressuremeter. Proc. Symp. on The Pressuremeter and its Marine Application Institut du Petrole, Laboratories des Ponts et Chaussées, Paris; Editions Technip, Collections Colloques et Seminaires 37, pp. 143-164.

Wroth, C.P. (1984). The Interpretation of In-Situ Soil Tests. Geotechnique Vol. 34, No. 4, pp. 449-489.

Wroth, C.P. and Hughes, J.M.O. (1973). An Instrument for the In Situ Measurement of the Properties of Soft Clays. Proceedings 8th International Conference On Soil Mechanics And Foundation Engineering (ICSMFE) Moscow, Vol. 1.2, pp. 487-494.

WSDOT (2004). Standard Practice T 925, Standard Practice for Determination of Long-Term Strength for Geosynthetic Reinforcement. Washington State Department of Transportation, USA.

Wu, J. (2002). Liquefaction Triggering and Post Liquefaction Deformations of Monterey 0/30 Sand Under Uni-Directional Cyclic Simple Shear Loading. PhD Dissertation, University of California, Berkeley, Calif.

Wyllie, D.C. (1992). Foundations on Rock. London: E & FN Spon.

Yoshimi, Y. and Tokimatsu, K. (1983). SPT Practice Survey and Comparative Tests. Technical Note, Soils and Foundations Japanese Society of Soil Mechanics and Foundation Engineering Vol. 23, No. 3. pp. 106-111, September.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, PP.K., Seed, R.B. and Stokoe II, K.H. (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(10): 817-833.

Youd, T.L. and Noble, S. K. (1997). Magnitude scaling factors, Proceedings of NCEER Workshops on Evaluation of Liquefaction Resistance of soils. National Centre For Eqrthquake Engineering, Res., State Univ. of New York at Buffalo: 149-165.

Zeevaert, L. (1959). Reduction of Point Bearing Capacity Because of Negative Skin Friction. Proceedings First Panamerican Conference on Soil Mechanics and Foundation Engineering Vol. 3, pp. 1145-1152.

Index