Foundations ON ROCK

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Duncan C. Wyllie

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Second edition

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Contents

Foreword to first edition

Duncan Wyllie has given us a complete, useful textbook on rock foundations. It is complete in its coverage of all parts of this important subject and in providing reference material for follow-up study. It is eminently useful in being well organized, clearly presented, and logical.

Rock would seem to be the ultimate excellent reaction for engineering loads, and often it is. But the term 'rock' includes a variety of types and conditions of material, some of which are surely not 'excellent' and some that are potentially dangerous. Examples of frequently hazardous rock masses are those that contain dissolved limestones, undermined coal-bearing sediments, decomposed granites, swelling shales and highly jointed or faulted schists or slates. Moreover, the experience record of construction in rocks includes numerous examples of economic difficulties revolving around mistaken or apparently malevloent behavior of rock foundations. Such cases have involved excavation overbreak, deterioration of prepared surfaces, flooding or icing by ground water seepage, accumulation of boulders from excavation, gullying or piping of erodible banks, and misclassification or misidentification of materials in the weathered zone. Another class of difficult problems involve the forensic side of siting in evaluating potentialities for rock slides, fault movement, or long-term behavior.

Problems of investigating and characterizing rock foundations are intellectually challenging; and it may require imagination to tailor the design of a foundation to the particular morphological, structural and material properties of a given rock site. Thus the field of engineering activity encompassed in this book is interesting and demanding. The subject is worthy of a book on this subject and of your time in studying it.

Richard E. Goodman
Berkeley, California

Introduction

The first edition of *Foundations on Rock* was written during the period 1988 to 1990. In the decade that has passed since the initial material was collected on this subject, there has been steady development in the field of rock engineering applied to foundations, but no new techniques that have significantly changed design and construction practices. Consequently, the purpose of preparing this second edition, which has been written between 1996 and 1998, has been to update the technical material, and add information on new projects where valuable experience on rock foundations has been documented.

The following is a summary of the material that has been added:

- Chapter 1: expanded discussion on acceptable reliability levels for different types of structures in relation to the consequences of failure, as well as methods of risk analysis;
- Chapter 2: new material has been added on typical probability distributions for discontinuity lengths and spacing, and methods of collecting data on these features;
- Chapter 3: information is included on the deformation behavior of very weak rock that has been determined from *in situ* testing;
- Chapter 4: the procedures for mapping geological structure has been extensively revised to conform to the procedures drawn up by the International Society of Rock Mechanics, and has now been consolidated in Appendix II. It is intended that this information will help in the production of standard mapping results that are comparable from project to project;
- Chapter 5: a list of projects with substantial foundations bearing on rock has been included describing the rock conditions and the actual bearing pressures that have been successfully used. Also, the section on the detection of karstic features and the design of foundations in this geological environment has been greatly expanded. With respect to prediction of foundation performance, an example of numeric analysis of the stability of jointed rock masses has been included;
- Chapter 6: an example has been prepared of probabilistic stability analysis to calculate the coefficient of reliability of a foundation. Also, a technique for assessing scour potential of rock is presented in detail;
- Chapter 7: with the increasing need to rehabilitate existing dams either to meet new design standards, or to repair deterioration, a section on foundation improvement, scour potential and tie-down anchors has been added;
- Chapter 8: for the design of laterally loaded rock socketed piers, new information is provided on *p-y* curves for very weak rock;
- Chapter 9: the testing procedures and acceptance criteria for tensioned anchors has been updated to conform with 1990's recommended practice;
- Chapter 10: new information has been added on contracting procedures, and in particular Partnering.

It is believed that this is still one of the few books devoted entirely to the subject of rock foundations. As with the first edition, it is still intended to be a book that can be used by practitioners in a wide range of geological conditions, while still providing a sound theoretical basis for design.

The preparation of this edition has drawn extensively on the knowledge of many of the author's colleges in both the design and construction fields, all or which are gratefully acknowledged. In addition, Glenda Gurtina has provided great assistance in the preparation of the manuscript and Sonia Skermer has prepared all the new artwork to her usual high standard. Finally, I would like to thank my family for supporting me through yet another book project.

Duncan Wyllie
Vancouver, 1998

Introduction to first edition

Foundations on Rock has been written to fill an apparent gap in the geotechnical engineering literature. Although there is wide experience and expertise in the design and construction of rock foundations, this has not, to date, been collected in one volume. A possible reason for the absence of a book on rock foundations is that the design and construction of soil foundations is usually more challenging than that of rock foundations. Consequentially, there is a vast collection of literature on soil foundations, and a tendency to assume that any structure founded on 'bedrock' will be totally safe against settlement and instability. Unfortunately, rock has a habit of containing nasty surprises in the form of geological features such as solution cavities, variable depths of weathering, and clay-filled faults. All of these features, and many others, can result in catastrophic failure of foundations located on what appear to be sound rock surfaces.

The main purpose of this book is to assist the reader in the identification of potentially unstable rock foundations, to demonstrate design methods appropriate for a wide range of geological conditions and foundation types, and to describe rock construction methods. The book is divided into three main section. Chapters 1–4 describe the investigation and measurement of the primary factors that influence the performances of rock foundations. Namely, rock strength and modulus, fracture characteristics and orientation, and ground water conditions. Chapters 5–9 provide details of design procedures for spread footings, dam foundations, rock socketed piers, and tension foundations. These chapters contain worked examples illustrating the practical application of the design methods. The third section, Chapter 10, describes a variety of excavation and stabilization methods that are applicable to the construction of rock foundations.

The anticipated audience for this book, which has been written by a practising rock mechanics engineer, is the design professional in the field of geotechnical engineering. The practical examples illustrate the design methods, and descriptions are provided of investigation methods that are used widely in the geotechnical engineering community. It is also intended that the book will be used by graduate geotechnical engineers as a supplement to the books currently available on rock slope engineering, geological engineering and rock mechanics. *Foundations on Rock* describes techniques that are common to a wide selection of projects involving excavations in rock and these techniques have been adapted and modified, where appropriate, to rock foundation engineering.

Much of the material contained in this book has been acquired from the author's experience on projects in a wide range of geological and construction environments. On all these projects there have, of course, been many other persons involved: colleagues, owners, contractors and, equally importantly, the construction workers. The author acknowledges the valuable advice and experience that have been acquired from them all.

There are many people who have made specific contributions to this book and their assistance is greatly appreciated. Sections of the book were reviewed by Herb Hawson, Graham Rawlings, Hugh Armitage, Vic Milligan, Dennis Moore, Larry Cornish, Norm Norrish and Upul Atukorala. In additon a number of people contributed photographs and computer plots and they are acknowledged in the text. Important contributions were also made by Ron Dick who produced all the drawings, and Glenys Sykes who diligently searched out innumerable references. Finally, I appreciate the support of my family who tolerated, barely, the endless early-morning and late-night sessions that were involved in preparing this book.

D. C. Wyllie

Notation

The following symbols are used in this book.

A	Cross-sectional area (m^2, $inch^2$)
B	Width of footing, diameter of pier, burden (blasting) (m, ft)
b	Radius of footing (m, ft)
C_d	Dispersion coefficient (structural geology); influence factor for foundation displacement
C_f	Correction factor for foundation shape
CR	Coefficient of reliability
c	Cohesion (MPa, p.s.i.)
D	Diameter, depth of embedment (m, ft)
d	Diameter (m, ft)
$\bar{d}$	Mean value of displacing force (MN, lbf)
E_m	Deformation modulus of rock mass (MPa, p.s.i.)
E_r	Deformation modulus of intact rock (MPa, p.s.i.)
$E_{m(b)}$	Deformation modulus of rock mass in base of pier (MPa, p.s.i.)
$E_{m(s)}$	Deformation modulus of rock mass in shaft of pier (MPa, p.s.i.)
e	Eccentricity in foundation bearing pressure
FS	Factor of safety
F	Foundation factor (seismic design); shape factor (falling head tests)
f_r	Resisting force (MN, lbf)
f_d	Displacing force (MN, lbf); factor in limit states design
$G_{r,m}$	Shear modulus: intact rock (r), rock mass (m) (MPa, p.s.i.)
$G_{1,2}$	Viscoelastic constants defining creep characteristics of rock (MPa, p.s.i.)
H	Height (m, ft); horizontal component of force(s) (MN, lbf)
h	Head measurement in falling head test (m)
I	Importance factor in seismic design
I_s	Point load strength (MPa, p.s.i.)
i_h	Pressure gradient
K	Bulk modulus (MPa, p.s.i.)
K_s	Factor for construction type in seismic design
k	Permeability (m/s); blast vibration attentuation factor
$k_{n,s}$	Stiffness, normal and shear (GPa/m, p.s.i./in)
L, l	Length of foundation, outcrop, socket (m, ft)
$\boldsymbol{l}$, $\boldsymbol{m}$, $\boldsymbol{n}$	Unit vectors of direction cosines (structural geology)
m	Rock mass strength factor (Hoek–Brown strength)
N	Normal force (MN, lbf); number (of analyses) bearing capacity factor
P	Probability; rate of energy dissipation (kW/m^2)

p	Pressure (MPa, p.s.i.)
PF	Probability of failure
Q	Foundation load (MN, lbf)
Q_s	Seepage rate (l/s, ft^3/s)
q	Flow rate (l/s, gal/s); foundation bearing pressure (MPa, p.s.i.)
q_a	Allowable foundation bearing pressure (MPa, p.s.i.)
R	Force modification factor in seismic design
$\lvert R \rvert$	Resultant unit vector
R_e	Reynolds number
r	Radius (m, ft)
S	Spacing (m, ft); shear force (MN, lbf); seismic response factor
S	Siemen (unit of conductivity)
SD	Standard deviation
s	Rock mass strength factor (Hoek–Brown strength)
T	Basic time lag (s); rock bolt tension (MN, lbf)
U	Water uplift force (MN, lbf)
u	Water uplift pressure (MPa, p.s.i.)
V	Water force in tension crack (MN, lbf); vertical component of force(s) (MN, lbf); base shear
v	Zonal velocity ratio in seismic design
W	Weight of sliding block; weight factor in seismic design
$\overline{x}$	Mean value
Z	Factor for seismic intensity
α	Dip direction of plane, or trend of force (degrees); adhesion factor of pier side-walls
β	Settlement angular distortion, dip (degrees); blast vibration attenuation factor
γ	Unit weight (kN/m^3, lbf/ft^3)
γ_w	Unit weight of water (kN/m^3, lbf/ft^3)
δ	Settlement; displacement (mm, in)
Δ	Settlement relative deflection; displacement (mm, in)
ε	Strain (%)
η	Dynamic viscosity – rock creep (MPa min., p.s.i. min., poise (cgs units))
θ	Apex angle of rock cone (degrees)
ν	Poisson's ratio
σ	Normal stress (MPa, p.s.i.)
$\sigma_{u(m)}$	Uniaxial compressive strength of rock mass (MPa, p.s.i.)
$\sigma_{u(r)}$	Uniaxial compressive strength of intact rock (MPa, p.s.i.)
τ	Shear stress (MPa, p.s.i.)
ϕ	Friction angle (degrees)
ψ	Dip of plane or force (degrees)
ω	Settlement tilt (degrees)
Ω	Factor in rock anchor bond strength calculation
▼	Water table

Note

The recommendations and procedures contained herein are intended as a general guide and prior to their use in connection with any design, report or specification they should be reviewed with regard to the full circumstances of such use. Accordingly, although every care has taken in the preparation of this book, no liability for negligence or otherwise can be accepted by the author or the publisher.

1

Characteristics of rock foundations

1.1 Types of rock foundation

There are two distinguishing features of foundations on rock. First, the ability of the rock to withstand much higher loads than soil, and second, the presence of defects in the rock which result in the strength of the rock mass being considerably less than that of the intact rock. The compressive strength of rock may range from less than 5 MPa (725 p.s.i.) to more than 200 MPa (30 000 p.s.i.), and where the rock is strong, substantial loads can be supported on small spread footings. However, a single, low strength discontinuity oriented in a particular direction may cause sliding failure of the entire foundation.

The ability of rock to sustain significant shear and tensile loads means that there are many types of structures that can be constructed more readily on rock than they can be on soil. Examples of such structures are dams and arch bridges which produce inclined loads in the foundation, the anchorages for suspension bridges and other tie-down anchors which develop uplift forces, and rock socketed piers which support substantial loads in both compressive and uplift. Some of these loading conditions are illustrated in Fig. 1.1 which shows the abutment of an arch bridge. The load on the footing for the arch is inclined along the tangent to the arch, while the loads on the column and abutment are vertical; the load capacity of these footings depends primarily on the strength and deformability of the rock mass. The wall supporting the cut below the abutment is anchored with tensioned and grouted rock bolts; the load capacity of these bolts depends upon the shear strength developed at rock–grout interface in the anchorage zone.

If the material forming the foundations of the bridge shown in Fig. 1.1 was all strong, massive, homogeneous rock with properties similar to concrete, design and construction of the footings would be a trivial matter because the loads applied by a structure are generally much less than the rock strength. However, rock almost always contains discontinuities that can range from joints with rough surfaces and cohesive infillings that have significant shear strength, to massive faulted zones containing expansive clays with relatively low strength. Figure 1.1 shows how the geological structure can affect the stability of the foundations. First, there is the possibility of overall failure of the abutment along a failure plane (a–a) passing along the fault, and through intact rock at the toe of the slope. Second, local failure (b) of the foundation of the vertical column could occur on joints dipping out of the slope face. Third, settlement of the arch foundation may occur as a result of compression of weak materials in the fault zone (c), and fourth, poor quality rock in the bolt anchor zone could result in failure of the bolts (d) and loss of support of the abutment.

Foundations on rock can be classified into three groups – spread footings, socketed piers and tension foundations – depending on the magnitude and direction of loading, and the geotechnical conditions in the bearing area. Figure 1.2 shows examples of the three types of

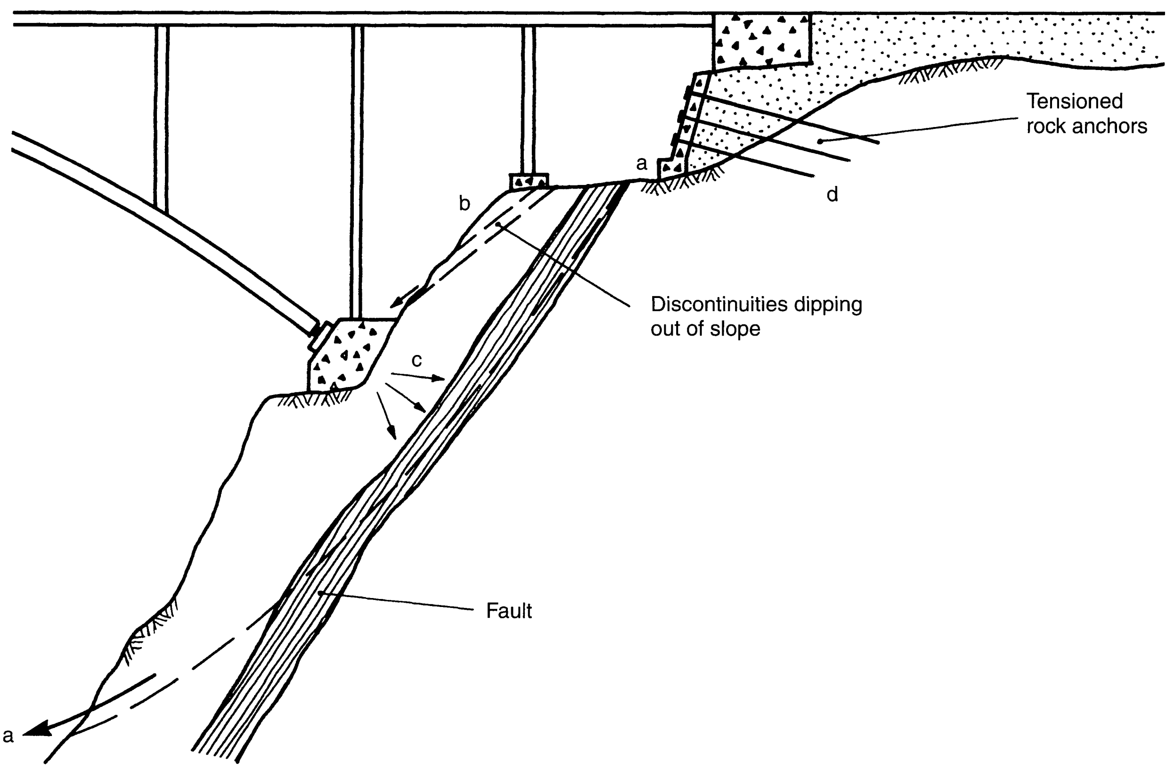

Figure 1.1 Stability of bridge abutment founded on rock: (a–a) overall failure of abutment on steeply dipping fault zone; (b) shear failure of foundation on daylighting joints; (c) movement of arch foundation due to compression of low-modulus rock; and (d) tied-back wall to support weak rock in abutment foundation.

foundations and the following is a brief description of the principal features of each. The basic geotechnical information required for the design of all three types of foundation consists of the structural geology, rock strength properties, and the ground water conditions as described in Chapters 2–4. The application of this data to the design of each type of foundation is described in Chapters 5–9.

1.1.1 Spread footings

Spread footings are the most common type of foundation and are the least expensive to construct. They can be constructed on any surface which has adequate bearing capacity and settlement characteristics, and is accessible for construction. The bearing surface may be inclined, in which case steel dowels or tensioned anchors may be required to secure the footing to the rock. For footings located at the crest or on the face of steep slopes, the stability of the overall slopes, taking into account the loads imposed by the structure, must be considered (Fig. 1.2(a)).

Dam foundations, which fall into the category of spread footings, are treated as a special case in this book. Loads on dam foundations comprise the weight of the dam together with the horizontal water force which exert a non-vertical resultant load (Fig. 1.2(b)). Furthermore, uplift forces are developed by water pressures in the foundation. These loads can be much larger than the loads imposed by structures such as bridges and build-

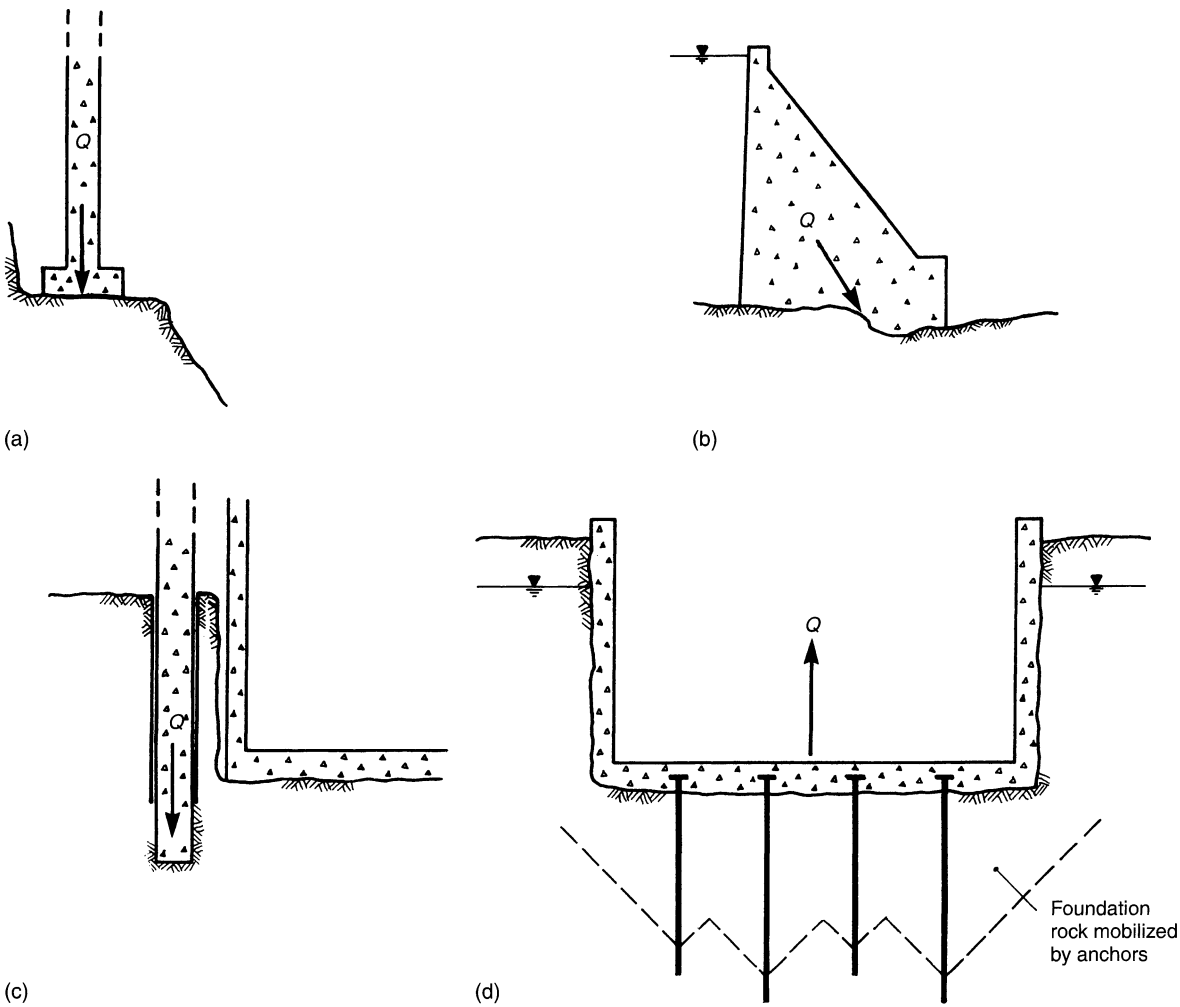

Figure 1.2 Types of foundations on rock: (a) spread footing located at crest of steep slope; (b) dam foundation with resultant load on foundation acting in downstream direction; (c) socketed pier to transfer structural load to elevation below base of adjacent excavation; and (d) tie-down anchors, with staggered lengths, to prevent uplift of submerged structure.

ings. In addition there is the need for a high level of safety because the consequences of failure are often catastrophic. Dams must also be designed to withstand flood conditions, and where appropriate, earthquake loading. The design of dam foundations, excluding foundations for arch dams, is discussed in Chapter 7.

1.1.2 Socketed piers

Where the loads on individual footings are very high and/or the accessible bearing surface has inadequate bearing capacity, it may be necessary to sink or drill a shaft into the underlying rock and construct a socketed pier. For example, in

Fig. 1.2(c) a spread footing could not be located on the edge of the excavation made for the existing building, and a socketed pier was constructed to bear in sound rock below the adjacent foundation level. The support provided by socketed piers comprises the shear strength around the periphery of the drill hole, and the end bearing on the bottom of the hole. Socketed piers can be designed to withstand axial loads, both compressive and tensile, and lateral forces with minimal displacement. Design methods for socketed piers are discussed in Chapter 8.

1.1.3 Tension foundations

For structures that produce either permanent or transient uplift loads, support can be provided by the weight of the structure and, if necessary, tie-down anchors grouted into the underlying rock (Fig. 1.2(d)). The uplift capacity of an anchor is determined by the shear strength of the rock–grout bond and the characteristics of the rock cone that is developed by the anchor. The dimensions of this cone are defined by the developed anchor length, and the apex angle of the cone. The position of the apex is usually assumed to be at mid-point of the anchor length, and the apex angle can vary from about 60° to 120°. An apex angle of about 60° would be used where there are persistent discontinuities aligned parallel to the load direction, while an angle of about 120° would be used in massive rock, or rock with persistent discontinuities at right angles to the load direction.

In calculating uplift capacity, a very conservative assumption can be made that the cone is 'detached' from the surrounding rock and that only the weight of the cone resists uplift. However, unless the anchor is installed in a rock mass with a cone-shaped discontinuity pattern, significant uplift resistance will be provided by the rock strength on the surface of the cone. The value of the rock strength depends on the strength of the intact rock, and on the orientation of the geological structure with respect to the cone surface. As shown in Fig. 1.2(d), the lengths of the anchors can be staggered so that the stresses in the rock around the bond zones are not concentrated on a single plane.

Design methods for tension anchors, including testing procedures and methods of corrosion protection, are described in Chapter 9.

1.2 Performance of foundations on rock

Despite the apparently favorable stability conditions for structures founded on strong rock, there are, unfortunately, instances of foundation failures. Failures may include excessive settlement due to the presence of undetected weak seams or cavities, deterioration of the rock with time, or collapse resulting from scour and movement of blocks of rock in the foundation. Factors that may influence stability are the structural geology of the foundation, strength of the intact rock and discontinuities, ground water pressures, and the methods used during construction to excavate and reinforce the rock.

The most complete documentation of foundation failures has been made for dams because the consequences of failure are often catastrophic. Also, the loading conditions on dam foundations are usually more severe than those of other structures so study of these failures gives a good insight on the behavior and failure modes of rock foundations. The importance of foundation design is illustrated by Gruner's examination of dam failures in which he found that one third could be directly attributed to foundation failure (Gruner, 1964, 1967). The following is a review of the stability conditions of rock foundations.

1.2.1 Settlement and bearing capacity failures

Settlement and bearing capacity type failures in rock are rare but may occur where large structures, sensitive to settlement, are constructed on very weak rock (Tatsuoka *et al.*, 1995), and where beds of low strength rock or cavities formed by weathering, scour or solution occur beneath the structure (James and Kirkpatrick, 1980). The most potentially hazardous conditions are in karstic areas where solution cavities may form under, or close to, the structure so that the foundation consists of only a thin shell of competent rock (Kaderabek and Reynolds, 1981). Rock types susceptible to solution are limestone, anhydrite, halite, calcium carbonate and gypsum. The failure mechanism of the foundation under these conditions may be punching and shear failure, or more rarely bending and tensile failure. Lowering of the water table may accelerate the solution process and cause failure long after construction is complete. A related problem is that of a thin bed of competent rock overlying a thick bed of much more compressible rock which may result in settlement as a result of compression of the underlying material (mechanism (c) in Fig. 1.1).

Loss of bearing capacity with time may also occur due to weathering of the foundation rock. Rock types which are susceptible to weathering include poorly cemented sandstones, and shales, especially if they contain swelling clays. Common causes of weathering are freeze–thaw action, and in the case of such rocks as shales, wetting and drying cycles. Foundations which undergo a significant change in environmental conditions as a result of construction, such as dam sites where the previously dry rock in the sides of the valley becomes saturated, should be carefully checked for any materials that may deteriorate with time in their changed environment.

1.2.2 Creep

There are two circumstances under which rocks may creep, that is, experience increasing strain with time under the application of a constant stress. First, creep may occur in elastic rock if the applied stress is a significant fraction (greater than about 40%) of the uniaxial compressive strength (σ_u). However, at the relatively low stress level of 40% of σ_u the rate of creep will decrease with time. At stress levels greater than about 60% of σ_u, the rate will increase with time and eventually failure may take place. At the stress levels usually employed in foundations it is unlikely that creep will be significant.

A second condition under which creep may occur is in ductile rocks such as halite and some sediments. A ductile material will behave elastically up to its yield stress but is able to sustain no stress greater than this so that it will flow indefinitely at this stress unless restricted by some outside agency. This is known as elastic–plastic behavior and foundations on such materials should be designed so that the applied stress is well below the yield stress. Where this is not possible, the design and construction methods should accommodate time-dependent deformations.

Time-dependent behavior of rock is discussed in more detail in Section 3.6.

1.2.3 Block failure

The most common cause of rock foundation failure is the movement and collapse of blocks of rock formed by intersecting discontinuities (mechanism (b) in Fig. 1.1). The orientation, spacing and length of the discontinuities determines the shape and size of the blocks, as well as the direction in which they can slide. Stability of the blocks depends on the shear strength of the discontinuity surfaces, and the external forces which can comprise water, structural, earthquake and reinforcement loads. Analysis of stability conditions involves the determination of the factor of safety or coefficient of reliability, and is described in more detail in Section 1.6.4 and Chapter 6.

An example of a block movement failure occurred in the Malpasset Dam in France where a wedge formed by intersecting faults moved when subjected to the water uplift forces as the dam was filled (Londe, 1987). The failure resulted in the loss of 400 lives. Bridge foundations also experience failure or movement as a result of instability of blocks of rock (Wyllie, 1979, 1995). One cause of these failures is the geometry of bridge foundations, with the frequent construction of abutments and piers on steep rock faces from which blocks can slide. Other causes of failure are ground water effects which include weathering, uplift pressures on blocks which have a potential to slide, river scour and wave action which can undermine the foundation, and traffic vibration which can slowly loosen closely fractured rock. It is standard practice on most highways and railways to carry out regular bridge inspections which will often identify deteriorating foundations and allow remedial work to be carried out. It is the author's experience that rock will usually undergo observable movement sufficient to provide a warning of instability before collapse occurs.

An example of the influence of structural geology on stability is shown in Fig. 1.3 where a retaining wall is founded on very strong granite containing sheeting joints dipping at about 40° out of the face. Although the bearing capacity of the rock was ample for this loading condition, movement along the joints and failure of a block in the foundation resulted in rotation of the wall. Fortunately, early detection of this condition allowed remedial work to be carried out. This consisted of concrete to fill the cavity formed by the failed rock and the installation of tensioned bolts to prevent further movement on the joints.

1.2.4 Failure of socketed piers and tension anchors

The failure of socketed piers is usually limited to unacceptable movement which may occur as a result of loss of bond at the rock–concrete interface on the side walls, or compression of loose material at the base of the pier. A frequent cause of movement is poor cleaning of the sides and base of the hole, or in the case of karstic terrain, collapse of rock into an undetected solution cavity. In

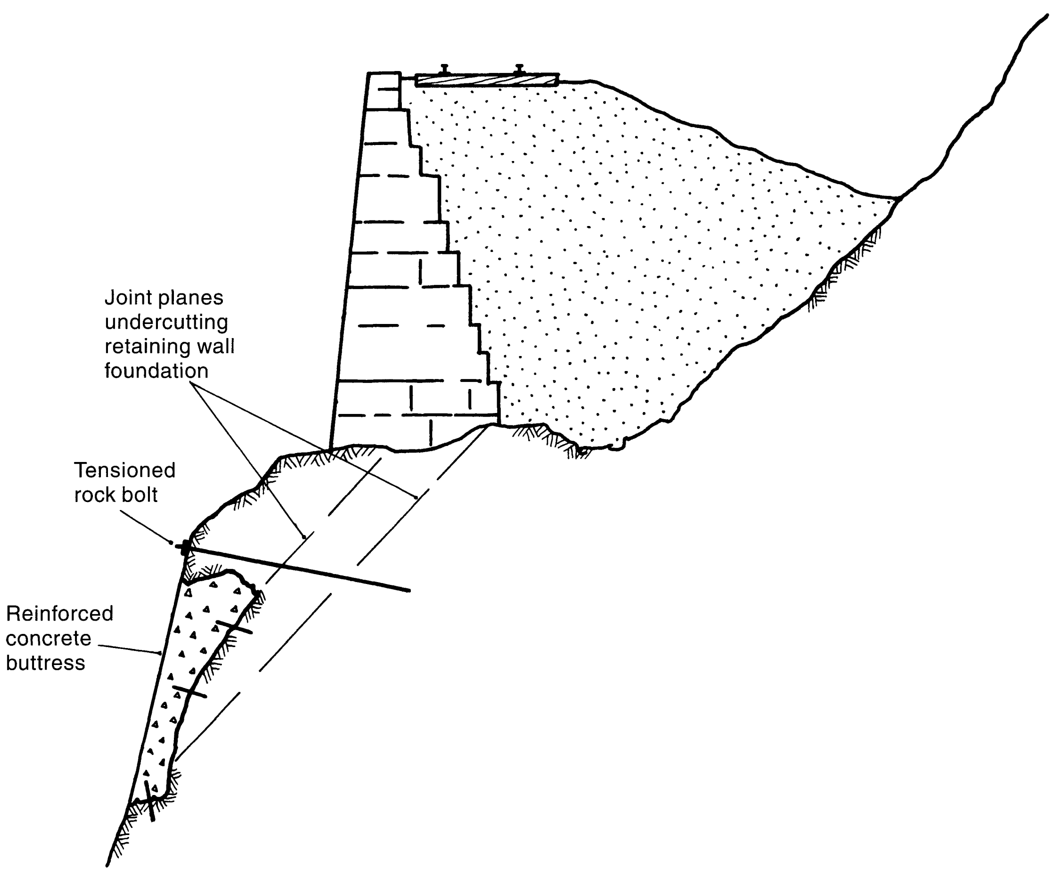

Figure 1.3 Retaining wall foundation stabilized with reinforced concrete buttress and rock bolts.

the case of tensioned anchors, loss of bond at the rock–grout interface on the walls of the hole may result in excessive movement of the head, while corrosion failure of the steel may result in sudden failure long after installation. The long term reliability of tensioned anchors depends to a large degree on the details of fabrication and installation procedures as discussed in Chapter 9.

1.2.5 Influence of geological structure

The illustrations of foundation conditions shown in Figs 1.1 and 1.3, and the analysis of foundation failures, show that geologic structure is often a significant feature influencing the design and construction of rock foundations. Detailed knowledge of discontinuity characteristics – orientation, spacing, length, surface features and infilling properties – are all essential information required for design. The examination of the structural geology of a site usually requires a three-dimensional analysis which can be most conveniently carried out using stereographic projections as described in Chapter 2. This technique can be used to identify the orientation and shape of blocks in the foundation that may fail by sliding or toppling.

It is also necessary to determine the shear strength of discontinuities along which failure could take place. This involves direct shear tests, which may be carried out in the laboratory on pieces of core, or *in situ* on undisturbed samples. Methods of rock testing are described in Chapter 4.

1.2.6 Excavation methods

Blasting is often required to excavate rock foundations and it is essential that controlled blasting methods be used that minimize the damage to rock that will support the planned structure. Damage caused by excessively heavy blasting can range from fracturing of the rock with a resultant loss of bearing capacity, to failure of the slopes either above or below the foundation. There are some circumstances, when, for example, existing structures are in close proximity or when excavation limits are precise, in which blasting is not possible. In these situations, non-explosive rock excavation methods, which include hydraulic splitting, hydraulic hammers and expansive cement, may be justified despite their relative expense and slow rate of excavation (see Section 10.3.6).

A typical effect of geological conditions on foundation excavations is shown in Fig. 1.4 where the design called for a notch to be cut in strong granite to form a shear key to resist horizontal forces generated in the backfill. However, the bearing surface formed along pre-existing joints and it was impractical to cut the required notch; it was necessary to install dowels to anchor the wall. Only in very weak rock is it possible to 'sculpt' the rock to fit the structure, and even this may be both expensive and ineffective.

Methods of rock excavation are discussed in Chapter 10.

1.2.7 Reinforcement

The reinforcement of rock to stabilize slopes above and below foundations, or to improve bearing capacity and deformation modulus, has wide application in rock engineering. Where the intact rock is strong but contains discontinuities which form potentially unstable blocks, the foundation can be reinforced by installing tensioned cables or rigid bolts across the failure plane. The function of such reinforcement is to apply a normal stress across the sliding surface which increases the frictional resistance on the surface; the shear strength of the steel bar provides little support in comparison with the friction component of the rock strength. Another function of the reinforcement is to prevent loosening of the rock mass, because reduction in the interlock between blocks results in a significant reduction in rock mass strength.

Where the rock is closely fractured, pumping of cement grout into holes drilled into the foundation can be used to increase the bearing capacity and modulus. The effect of the grout is to limit interblock movement and closure of discontinuities under load, both of which increase the strength of the rock mass and reduce settlement. Where it is

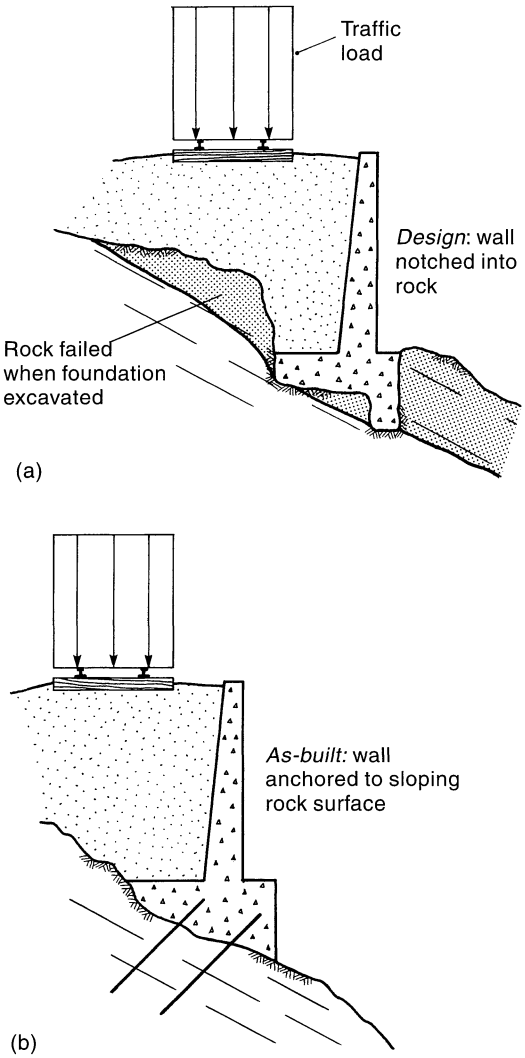

Figure 1.4 Construction of rock foundation: (a) attempted 'sculpting' of rock foundation to form shear key; and (b) 'as-built' condition with footing located on surface formed by joints.

required to protect closely fractured or faulted rock faces from weathering and degradation that may undermine a foundation, shotcrete can often be used to support the face. However, shotcrete will have no effect on the stability of the overall foundation.

Methods of construction and rock reinforcement are discussed in Chapter 10.

1.3 Structural loads

The following is a summary of typical loading conditions produced by different types of structures based on United States' building codes and design practices (Merritt, 1976). The design information required on loading conditions consists of the magnitude of both the dead and live loads, as well as the direction and point of application of these loads. This information is then used to calculate the bearing pressure, and any overturning moments acting on the foundation.

An important aspect in foundation design is communication between the structural and foundation engineers on the factors of safety that are incorporated in each part of the design. If the structural engineer calculates the dead and live loads acting on the foundation and multiplies this by a factor of safety, it is important that the foundation engineers do not apply their own factors of safety. Such multiplication of factors of safety can result in overdesigned and expensive foundations. Conversely, failure to incorporate adequate factors of safety can result in unsafe foundations. A description of methods of calculating loads imposed by structures on their foundations is beyond the scope of this book; this is usually the responsibility of structural engineers. The following four sections provide a summary of the design methods, and the appropriate references should be consulted for detailed procedures.

1.3.1 Buildings

Loads on building foundations consist of the dead load of the structural components, and the live load associated with its usage, both of which are closely defined in various building codes. For dead loads, the codes describe a wide range of construction materials such as various types of walls, partitions, floors finishes and roofing materials and the minimum loads which they exert. An option that may be suitable for poor foundation conditions is the use of lightweight aggregate in concrete which reduces the dead load for concrete slabs from 24 Pa per millimeter of thickness

(12.5 p.s.f. per inch) for standard concrete, to 17 Pa per millimeter of thickness (9 p.s.f. per inch).

A special case is the dead load on buried structures in which a considerable load is exerted by the backfill – granular fill has a density of about 19 kN/m^3 (120 lb/ft^3), and a 3 m thick backfill will exert a dead load equal to about seven floors of an office building. A very significant reduction in the foundation loads can be achieved by using lightweight fills such as styrofoam which has a density of 0.3 kN/m^3 (2 lb/ft^3) and is used in road fills on low strength soils. The disadvantage of styrofoam is that it is flammable and soluble in oil, so must be carefully protected.

The live loads, which are determined by the building usage, are defined in the codes and range from 12 kN/m^2 (250 lb/ft^2) for warehouses and heavy manufacturing areas, 7.2 kN/m^2 (150 lb/ft^2) for kitchens and book storage areas, and 1.9 kN/m^2 (40 lb/ft^2) for apartments and family housing. Live loads are generally uniformly distributed, but are concentrated for such usage as garages and elevator machine rooms.

Additional loads result from snow, wind and seismic events, which vary with the design of the structure and the geographic location. Wind, snow and live loads are assumed to act simultaneously, but wind and snow are generally not combined with seismic forces.

Ground motion in an earthquake is multidirectional and can induce forces in the foundation of a structure that can include base shear, torsion, uplift and overturning moments. The magnitude of the forces depends, for a single-degree-of-freedom structure, on the fundamental period and damping characteristics of the structure, and on the frequency content and amplitude of the ground motion. The resistance to the base shear, torsion forces and overturning moments is provided by the weight of the structure, the friction on the base, and if necessary, the installation of tie-down anchors.

The total base shear at the foundation, which can be used as measure of the response of the structure to the ground motion, is the sum of the horizontal forces acting in the structure and is given by (Canadian Geotechnical Society, 1992; National Building Code of Canada, 1990):

$$\text{Base shear, } V = (V_e/R)U_e \tag{1.1}$$

where V_e is the equivalent lateral seismic force representing elastic response, R is a force modification factor and U_e is a calibration factor with a value of 0.6. The lateral seismic force V_e is defined by:

$$V_e = vSIFQ \tag{1.2}$$

The following is a discussion on each of these factors.

- R, **force modification factor**, is assigned to different types of structure reflecting design and construction experience, and the evaluation of the performance of structures during earthquakes. It endeavors to account for the energy-absorption capacity of the structural system by damping and inelastic action through several load reversals. A building with a value of R equal to 1.0 corresponds to a structural system exhibiting little or no ductility, while construction types that have performed well in earthquakes are assigned higher values of R. Types of structures assigned high values of R are those capable of absorbing energy within acceptable deformations and without failure, structures with alternate load paths or redundant structural systems, and structures capable of undergoing inelastic cyclic deformations in a ductile manner.
- v, **zonal velocity ratio**, which varies from 0.0 for seismic zone 0 located in areas with low risk of seismic events, to 0.4 for seismic zone 6 where there is active seismic activity resulting from crustal movement. For example, in North America, zone 0 lies in the central part of the continent, while zone 6 lies along the east and west coasts.
- S, **seismic response factor**, which depends on the fundamental period of the structure, and the seismic zone for a particular geographic location.
- I, **importance factor**, has a value of 1.5 for buildings that should be operative after an earthquake. Such buildings include power generation and distribution systems, hospitals, fire and police stations, radio stations and towers, telephone ex-

changes, water and sewage pumping stations, fuel supplies and civil defense buildings. Schools, which may be needed for shelter after an earthquake, are assigned an I value of 1.3, and most other buildings are assigned a value of 1.0.

- F, **foundation factor**, accounts for the geological conditions in the foundation. As earthquake motions propagate from the bedrock to the ground surface, soil may amplify the motions in selected frequency ranges close to the natural frequencies of the surficial layer. In addition, a structure founded on the surficial layer and having some of its natural frequencies close to that of the layer, may experience increased shaking due to the development of a state of quasi-resonance between the structure and the soil. For structures founded on rock, the foundation factor F is usually taken as 1.0. However, in steep topography there may be amplification of the ground motions related to the three-dimensional geometry of the site. For example, at the Long Valley Dam in California, the measured acceleration on the abutment at an elevation of 75 m (250 ft) above the base of the dam was a maximum of 0.35g compared with the maximum acceleration at the base of 0.18g (Lai and Seed, 1985). The amplification of ground motion in canyons has been studied extensively for dam design and both three-dimensional and two-dimensional models have been developed to predict these conditions (Gazetas and Dakoulas, 1991).
- Q, **weight factor**, is the weight of the structure.

1.3.2 Bridges

Loads that bridge foundations support consist of the dead load determined by the size and type of structure, and the live load as defined in the codes for a variety of traffic conditions. For example, an HS20-44 highway load, representing a truck and trailer with three loaded axles, is a uniform load of 9.34 kN per lineal meter of load lane (0.64 kips per lineal foot) together with concentrated loads at the wheel locations for moment and shear. For railway bridges, the live load is specified by the E number of a 'Cooper's train', consisting of two locomotives and an indefinite number of freight cars. Cooper's train numbers range from E10 to E80, with E80 being for heavy diesel locomotives with bulk freight cars.

For both highway and railway bridges, impact loads are calculated as a fraction of the live load, with the magnitude of the impact load diminishing as the span length increases. Methods of calculating impact loads vary with the span length, method of construction and the traffic type. Other forces that may affect the foundations are centrifugal forces resulting from traffic motion, wind, seismic, stream flow, earth and ice forces, and elastic and thermal deformations. The magnitude of these forces is evaluated for the particular conditions at each site.

1.3.3 Dams

Loads on dam foundations are usually of much greater magnitude than those on bridge and building foundations because of the size of the structures themselves, and the forces exerted by the water impounded behind the dam. The water forces are usually taken as the peak maximum flood (PMF), with an allowance for accumulations of silt behind the dam, as appropriate. Any earthquake loading can be simulated most simply as a horizontal pseudostatic force proportional to the weight of the dam. The resultant of these forces acts in a downstream direction, and the dam must be designed to resist both sliding and overturning under this loading condition. There may also be concentrated compressive stresses at the toe of the dam and it is necessary to check that these stresses do not cause excessive deformation.

A significant difference between dams and most other structures is the water uplift pressures that are generated within the foundations. In most cases there are high pressure gradients beneath the heel of the dam where drain holes and grout curtains are installed to relieve water pressures and control seepage. The combination of these load conditions, together with the high degree of safety required for any dam, requires that the investigation, design and construction of the foundation be both thorough and comprehensive.

1.3.4 Tension foundations

Typical tension loads on foundations consist of bouyancy forces generated by submerged tanks, angle transmission line towers and the tension in suspension bridge cables. Foundations may also be designed to resist uplift forces generated by overturning moments acting on the structure resulting from horizontal loads such as wind, ice, traffic and earthquake forces.

1.4 Allowable settlement

Undoubtedly the most famous case of foundation settlement is that of the Leaning Tower of Pisa which has successfully withstood a differential settlement of 2 m and is now leaning at an angle of at least 5°11′ (Mitchell *et al.*, 1977). However, this situation would not be tolerated in most structures, except as a tourist attraction! The following is a review of allowable settlement values for different types of structures.

1.4.1 Buildings

Settlement of building foundations that is insufficient to cause structural damage may still be unacceptable if it causes significant cracking of architectural elements. Some of the factors that can affect settlement are the size and type of structure, the properties of the structural materials and the subsurface soil and rock, and the rate and uniformity of settlement. Because of these complexities, the settlement that will cause significant cracking of structural members or architectural elements, or both, cannot readily be calculated. Instead, almost all criteria for tolerable settlement have been established empirically on the basis of observations of settlement and damage in existing buildings (Wahls, 1981).

Damage due to settlement is usually the result of differential settlement, i.e. variations in vertical displacement at different locations in the building, rather than the absolute settlement. Means of defining both differential and absolute settlement are illustrated in Fig. 1.5, together with the terms defining the various components of settlement.

Study of cracking of walls, floors and structural members shows that damage was most often the result of distortional deformation, so 'angular distortion' β has been selected as the critical index of settlement. These studies have resulted in the following limiting values of angular distortion being recommended for frame buildings (Terzaghi and Peck, 1967; Skempton and McDonald, 1956; Polshin and Tokar, 1957):

$\beta > 1/150$ – structural damage probable;
$\beta > 1/300$ – cracking of load bearing or panel walls likely;
$\beta < 1/500$ – safe level of distortion at which cracking will not occur.

In the case of load bearing walls, it is found that the deflection ratio Δ/L is a more reliable indicator

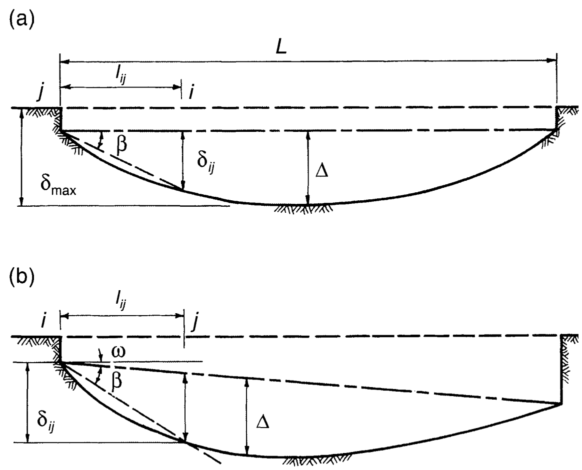

Figure 1.5 Definition of settlement terminology for buildings (Wahls, 1981): (a) settlement without tilt; (b) settlement with tilt. δ_i is the vertical displacement at i; δ_{max} is the maximum displacement; δ_{ij} is the displacement between two points i and j with distance apart l_{ij}; Δ is the relative deflection which is the maximum displacement from a straight line connecting two reference points; ω is the tilt, or rigid body rotation; $\beta_{ij} = [(\delta_{ij}/l_{ij}) - \omega]$ is the angular distortion; and Δ/L is the deflection ratio, or the approximate curvature of the settlement curve.

of damage because it is related to the direct and diagonal tension developed in the wall as a result of bending (Burland and Wroth, 1974). The proposed limiting values of Δ/L for design purposes are in the range 0.0005–0.0015.

1.4.2 Bridges

Extensive surveys of horizontal and vertical movement of highway bridges have been carried out to assess allowable settlement values (Walkinshaw, 1978; Grover, 1978; Bozozuk, 1978). It is concluded that settlement can be divided into three categories depending on its effect on the structure:

1. tolerable movements;
2. intolerable movements resulting only in poor riding characteristics; and
3. intolerable movements resulting in structural damage.

It is not feasible to specify limiting settlement values for each of these three categories because of the wide variety of bridge designs and subsurface conditions. For example, Walkinshaw reports of tolerable vertical movements that ranged from 13 to 450 mm (0.5 to 17.7 in), although the average value was about 85 mm (3.3 in). Intolerable vertical movements causing only poor riding quality averaged about 200 mm (7.9 in), while vertical movements causing structural damage varied from 13 to 600 mm (0.5 to 23.6 in) with an average value of about 250 mm (10 in). As a comparison with these results, Fig. 1.6 shows the results of the survey carried out by Bozozuk of bridge abutments and piers on spread footings with lines giving the limits of tolerable, harmful but tolerable, and intolerable movements.

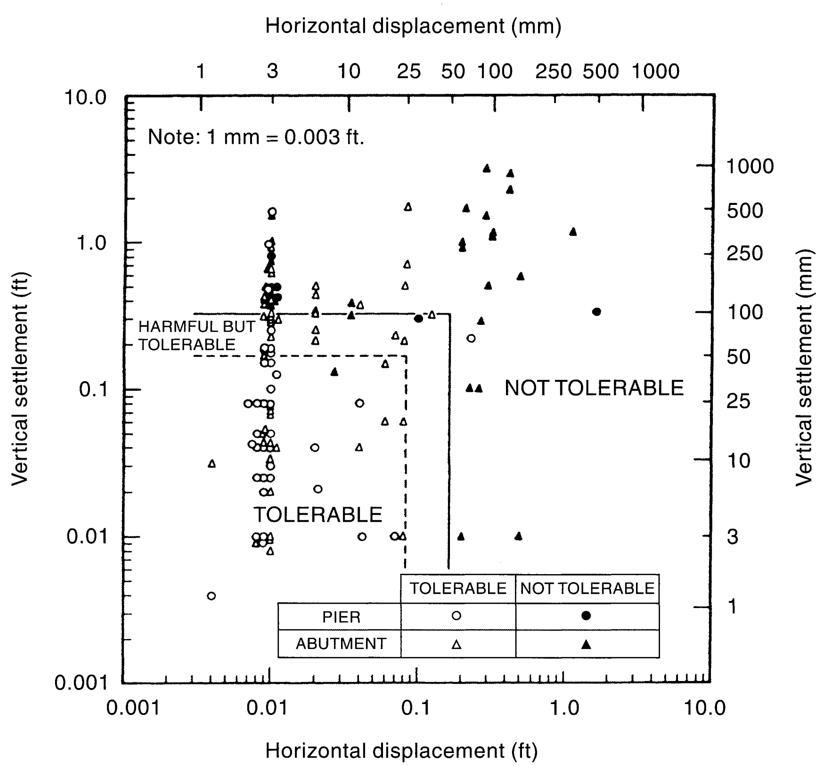

Figure 1.6 Engineering performance of bridge abutments and piers on spread footings (Bozozuk, 1978).

The conclusions that can be drawn from these studies are that tolerable movements can be as great as 50–100 mm (2–4 in), and that structural damage may not occur until movements are in excess of 200 mm (8 in). Also, differential and horizontal movements are more likely to cause damage that vertical movements alone. One possible reason is that vertical settlement of simply supported spans can readily be corrected by lifting and shimming at the bearing points (Grover, 1978). In comparison, horizontal movements are more difficult to correct, with one of the most important effects being the locking of expansion joints.

1.4.3 Dams

Allowable settlement of dams is directly related to the type of dam: concrete dams are much less tolerant of movement and deformation than embankment dams. There are no general guidelines on allowable settlements for dams because the foundation conditions for each structure should be examined individually. However, in all cases, particular attention should be paid to the presence of rock types with differing moduli, or seams of weathered and faulted rock that are more compressible than the adjacent rock. Either of these conditions may result in differential settlement of the structure.

1.5 Influence of ground water on foundation performance

The effect of ground water on the performance of foundations should be considered in design, particularly in the case of dams and bridges. These effects include movement and instability resulting from uplift pressures, weathering, scour of seams of weak rock, and solution (Fig. 1.7). In almost all cases, geological structure influences ground water conditions because most intact rock is effectively impermeable and water flow through rock masses is concentrated in the discontinuities. Flow quantities and pressure distributions are related to the aperture, spacing and continuous length of the discontinuities: tight, impersistent discontinuities will tend to produce low seepage quantities and high pressure gradients. Furthermore, the direction of flow will tend to be parallel to the orientation of the main discontinuity set.

1.5.1 Foundation stability

Typical instability caused by water uplift forces acting on potential sliding planes in the foundation is illustrated in Fig. 1.7(a). The uplift force U acting on the sliding plane reduces the effective normal force on this surface, which produces a corresponding reduction the shear strength (see Chapter 3). For the condition shown in Fig. 1.7(a), the greatest potential for instability is when a rapid draw down in the water level occurs ($V = 0$), and there is insufficient time for the uplift force to dissipate.

The flow of water through and around a foundation can have a number of effects on stability apart from reducing the shear strength. First, rapid flow can scour low strength seams and infillings, and develop openings that undermine the foundation (Fig. 1.7(a)). Second, percolation of water through soluble rocks such as limestone can cause cavities to develop. Third, rocks such as shale may weather and deteriorate with time resulting in loss of bearing capacity. Such weathering may occur either so rapidly that it is necessary to protect bearing surfaces as soon as they are excavated, or it may occur a considerable time after construction causing long term settlement of the structure. Fourth, flow of water into an excavation can make cleaning and inspection of bearing surfaces difficult (Fig. 1.7(b)) and result in increased construction costs.

1.5.2 Dams

In dam foundations it is necessary to control both uplift due to water pressures to ensure stability, and seepage to limit water loss (Fig. 1.7(c)). Control measures consist of grout curtains and drains to limit seepage and reduce water pressure as described in Chapter 7. The rock property that determines seepage quantities and head loss is permeability, which relates the quantity of water

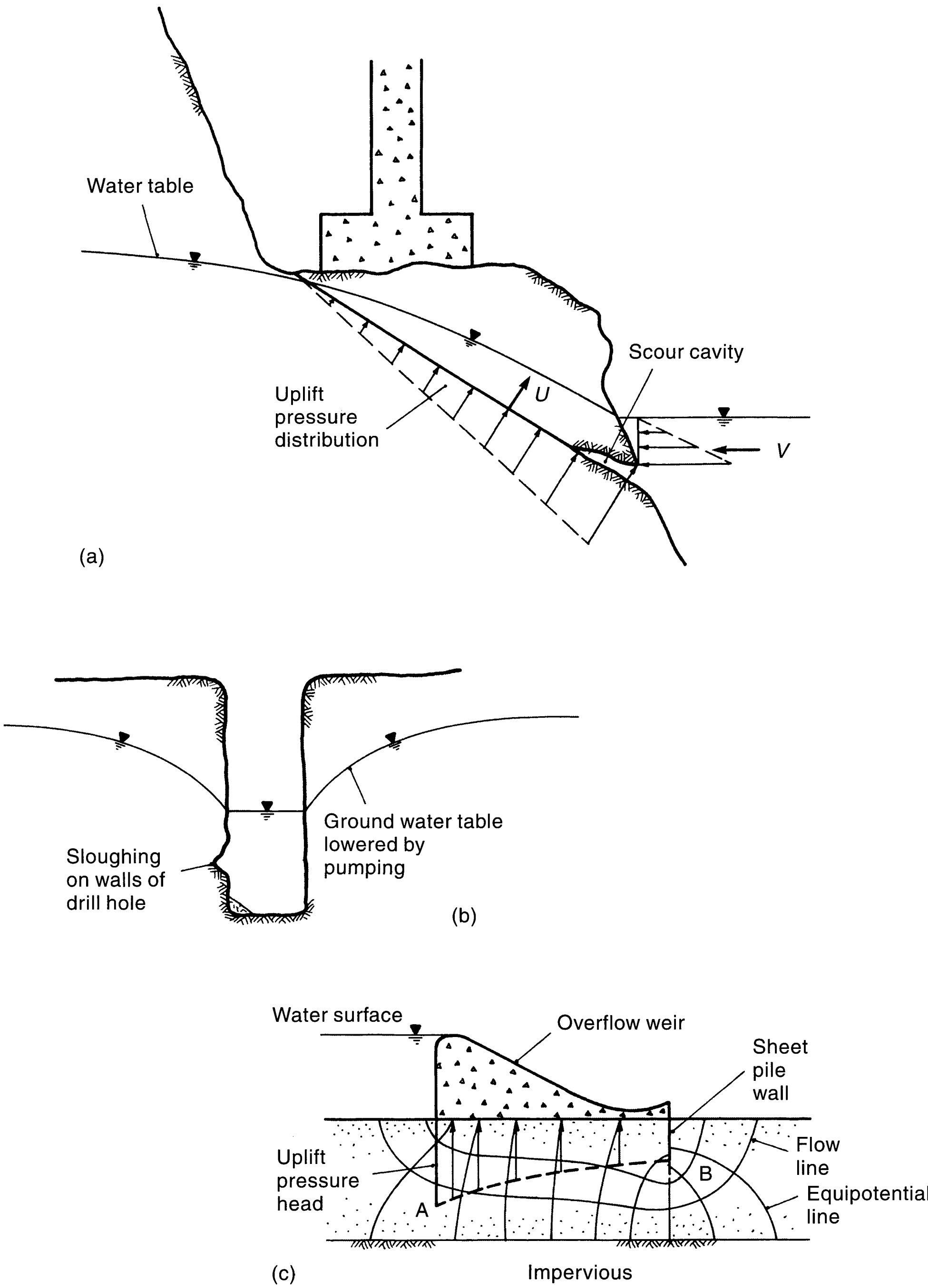

Figure 1.7 Typical effects of ground water flow on rock foundations: (a) uplift pressures developed along continuous fracture surface; (b) water flow into hole drilled for socketed pier; and (c) typical flow net depicting water flow and uplift pressure distribution in dam foundation (after Cedergren, 1989).

flow through the rock to the pressure gradient across it. As discussed at the start of this section, water flow is usually concentrated in the discontinuities, so seepage quantities will be closely related to the geological structure. For example, seepage losses may be high where there are continuous, open discontinuities that form a seepage path under the dam, while a clay filled fault may form a barrier to seepage. The study of seepage paths and quantities, and calculation of water pressure distributions in the foundation is carried out by means of flow nets (Cedergren, 1989). A flow net comprises two sets of lines – **equipotential lines** (lines joining points along which the total head is the same) and **flow lines** (paths followed by water flowing through the saturated rock) – that are drawn to form a series of curvilinear squares as shown in Fig. 1.7(c). The distribution of equipotential lines can also be used to determine the uplift pressure under a foundation which is also shown in Fig. 1.7(c).

1.5.3 Tension foundations

Where tension foundations are secured with anchors located below the water table, it is necessary to use the buoyant weight of the rock in calculating uplift resistance provided by the 'cone' of rock mobilized by the anchor. Figure 1.2(d) shows an example of such an installation where the rock in which the tie-down anchors is located below the water table and the effective unit weight of the rock is about 16 kN/m^3 (100 lb/ft^3). Another important factor in design is provision for protection of the steel against corrosion, with corrosion occurring most rapidly in low-pH and salt-water environments. Protective measures for 'permanent' installations consist of plastic sheaths grouted on to the anchors and full grout encapsulation which produces a crack resistant, high-pH environment around the steel (see Chapter 9).

1.6 Factor of safety and reliability analysis

Structural design and geotechnical analysis are usually based on the following two main requirements. First, the structure and its components must, during the intended service life, have an adequate margin of safety against collapse under the maximum loads and forces that might reasonably occur. Second, the structure and its components must serve the designed functions without excessive deformations and deterioration. These two service levels are the ultimate and serviceability limit states respectively and are defined as follows. Collapse of the structure and foundation failure including instability due to sliding, overturning, bearing failure, uplift and excessive seepage, is termed the ultimate limit state of the structure. The onset of excessive deformation and of deterioration including unacceptable total and differential movements, cracking and vibration is termed the serviceability limit state (Meyerhof, 1984).

The following is a discussion on a number of different design methods for geotechnical structures. Factor of safety analysis is by far the most widely used technique and factor of safety values for a variety of structures are generally accepted in the engineering community. This provides for each type of structure to be designed to approximately equivalent levels of safety. Adaptations to the factor of safety analysis include the limit states and sensitivity analysis methods, both of which examine the effect of variability in design parameters on the calculated factor of safety. An additional design method, reliability analysis, expresses the design parameters as probability density functions representing the range and degree of variability of the parameter. The theory of reliability analysis is well developed and its major strength is that it quantifies the variability in all the design parameters and calculates the effect of this variability on the factor of safety (Harr, 1977). However, despite the analytical benefits of reliability analysis, it is not widely used in geotechnical engineering practice (as of 1998).

1.6.1 Factor of safety analysis

Design of geotechnical structures involves a certain amount of uncertainty in the value of the input parameters which include the structural geology, material strengths and ground water pressures. Additional uncertainties to be considered in design are extreme loading conditions such as floods and seismic events, reliability of the analysis procedure, and construction methods. Allowance for these uncertainties is made by including a factor of safety in design. The factor of safety is the ratio of the total resistance forces – the rock strength and any installed reinforcement, to the total displacing forces – downslope components of the applied loads and the foundation weight. That is,

$$\text{Factor of safety}, FS = \frac{\Sigma(Resisting\ forces)}{\Sigma(Displacing\ forces)} \tag{1.3}$$

The ranges of minimum total factors of safety as proposed by Terzaghi and Peck (1967) and the *Canadian Foundation Engineering Manual* (1992) are given in Table 1.1.

The upper values of the total factors of safety apply to normal loads and service conditions, while the lower values apply to maximum loads and the worst expected geological conditions. The lower values have been used in conjunction with performance observations, large field tests, analysis of similar structures at the end of the service life and for temporary works.

The factors of safety quoted in Table 1.1 are employed in engineering practice, and can be used as a reliable guideline in the determination of appropriate values for particular structures and conditions. However, the design process still requires a considerable amount of judgment because of the variety of geological and construction factors that must be considered. Examples of conditions that would generally require the use of factors of safety at the high end of the ranges quoted in Table 1.1 include:

1. a limited drilling program that does not adequately sample conditions at the site, or drill core in which there is extensive mechanical breakage or core loss;
2. absence of rock outcrops so that detailed mapping of geological structure is not possible;
3. inability to obtain undisturbed samples for strength testing, or difficulty in extrapolating laboratory test results to *in situ* conditions;
4. absence of information on ground water conditions, and significant seasonal fluctuations in ground water levels;
5. uncertainty in failure mechanisms of the foundation and the reliability of the analysis method. For example, planar type failures can be analyzed with considerable confidence, while the detailed mechanism of toppling failures is less well understood;
6. uncertainty in load values, particularly in the case of environmental factors such as wind, water, ice and earthquakes where existing data is limited;
7. concern regarding the quality of construction, including materials, inspection and weather conditions. Equally important are contractual matters such as the use of open bidding rather than pre-qualified contractors, and lump sum rather than unit price contracts;
8. lack of experience of local foundation performance; and
9. usage of the structures; hospitals, police stations and fire halls and bridges on major transportation routes are all designed to higher factors of safety than, for example, residential buildings and warehouses.

Table 1.1 Values of minimum total safety factors

Failure type	*Category*	*Safety factor*
Shearing	Earthworks	1.3–1.5
	Earth retaining structures, excavations	1.5–2.0
	Foundations	2–3

1.6.2 Limit states design

In order to produce a more uniform margin of safety for different types and components of earth structures and foundations under different loading conditions, the limit states design method has been proposed (Meyerhof, 1984; *Ontario Highway*

Bridge Design Code, 1983; *National Building Code of Canada*, 1985). The two Canadian codes are based on unified limit states design principles with common safety and serviceability criteria for all materials and types of construction.

Limit states design uses partial factors of safety which are applied to both the loads, and the resistance characteristics of the foundation materials. The procedure is to multiply the loads by a load factor f_d and the resistances, friction and cohesion, by resistance factors f_ϕ, f_c as shown in Table 1.2. The values given in parenthesis apply to beneficial loading conditions such as dead loads that resist overturning or uplift.

In limit states design the Mohr–Coulomb equation for the shear resistance of a sliding surface is expressed as

$$\tau = f_c c + (\sigma - f_U U) f_\phi \tan \phi \quad (1.4)$$

The cohesion c, friction coefficient, $\tan \phi$, and water pressure U are all multiplied by partial factors with values less than unity, while the normal stress σ on the sliding surface is calculated using a partial load factor greater than unity applied to the foundation load.

1.6.3 Sensitivity analysis

Another means of assessing the effects of the variability of design parameters on the factor of safety is to use sensitivity analysis. This procedure consists of calculating the factor of safety for a range of values of parameters, such as the water pressure, which cannot be precisely defined. For example, Hoek and Bray (1981) describe the stability analysis of a quarry slope in which sensitivity analyses were carried out for both the friction angle (range 15°–25°) and the water pressure – fully drained to fully saturated (Fig. 1.8). This plot shows that water pressures have more influence on stability than the friction angle. That is, a fully drained, vertical slope is stable for a friction angle as low as 15°, while a fully saturated slope is unstable at an angle of 60°, even if the friction angle is 25°.

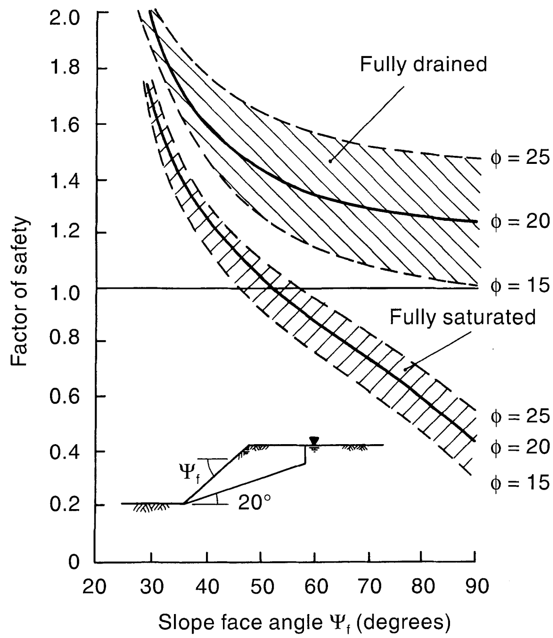

Figure 1.8 Sensitivity analysis showing the relationship between factor of safety and slope angle for range of water pressures and friction angles (Hoek and Bray, 1981).

Table 1.2 Values of minimum partial factors (Meyerhof, 1984)

Category	*Item*	*Load factor*	*Resistance factor*
Loads	Dead loads	(f_{DL}) 1.25 (0.8)	
	Live loads, wind, earthquake	(f_{LL}) 1.5	
	Water pressure (U)	(f_U) 1.25 (0.8)	
Shear strength	Cohesion (c) – stability, earth pressure		(f_c) 0.65
	Cohesion (c) – foundations		(f_c) 0.5
	Friction angle (ϕ)		(f_ϕ) 0.8

1.6.4 Coefficient of reliability

The factor of safety and limit states analyses described in this section involves selection of a single value for each of the parameters that define the loads and resistance of the foundation. In reality, each parameter has a range of values. A method of examining the effect of this variability on the factor of safety is to carry out sensitivity analyses as described in Section 1.6.3 using upper and lower bound values for what are considered to be critical parameters. However, to carry out sensitivity analyses for more than three parameters is a cumbersome process and it is difficult to examine the relationship between each of the parameters. Consequently, the usual design procedure involves a combination of analysis and judgment in assessing the influence on stability of variability in the design parameters, and then selecting an appropriate factor of safety.

An alternative design method is reliability analysis, which systematically examines the effect of the variability of each parameter on the stability of the foundation. This procedure calculates the coefficient of reliability *CR* of the foundation which is related to the more commonly used expression probability of failure *PF* by the following equation:

$$CR = (1 - PF) \tag{1.5}$$

The term coefficient of reliability is preferred for psychological reasons: a coefficient of reliability of 99% is more acceptable to an owner than a probability of failure of 1%.

Reliability analysis was first developed in the 1940's and is used in the structural and aeronautical engineering fields to examine the reliability of complex systems. Among its early uses in geotechnical engineering was in the design of open pit mine slopes where a certain risk of failure is acceptable and this type of analysis can be readily incorporated into the economic planning of the mine (Canada DEMR, 1978; Pentz, 1981; Savely, 1987). Examples of its use in civil engineering are in the planning of slope stabilization programmes for transportation systems (Wyllie *et al.*, 1979; McGuffey *et al.*, 1980), landslide hazards (Cruden and Fell, 1997) and in design of storage facilities for hazardous waste (Roberds, 1984, 1986).

There is sometimes reluctance to use probabilistic design when there is a limited amount of design data which may not be representative of the population. In these circumstances it is possible to use subjective assessment techniques that provide reliable probability values from small samples (Roberds, 1990). The basis of these techniques is the assessment and analysis of available data, by an expert or group of experts in the field, in order to arrive at a consensus on the probability distributions that represent the opinions of these individuals. The degree of defensibility of the results tends to increase with the time and cost that is expended in the analysis. For example, the assessment techniques range from, most simply, informal expert opinion, to more reliable and defensible techniques such as Delphi panels (Rohrbaugh, 1979). A Delphi panel comprises a group of experts who are each provided with the same set of data and are required to produce a written assessment of this data. These documents are then provided anonymously to each of the other assessors who are encouraged to adjust their assessments in light of their peer's assessments. After several iterations of this process, it should be possible to arrive at a consensus that maintains anonymity and independence of thought.

The use of reliability analysis in design requires that there be generally accepted ranges of reliability values for different types of structure, as there are for factors of safety. To assist in selecting appropriate reliability values, Athanasiou-Grivas (1979) provides charts relating factor of safety and probability of failure. Also, Fig. 1.9 gives a relationship between required levels of annual probability of failure for a variety of engineering projects, and the consequence of failure in terms of lives lost. For example, for structures such as low rise buildings and bridges with low traffic density where failure could result in less than about five lives lost, the range of annual probability of failure should not excced about 10^{-2}–10^{-3} (*CR* = 99%–99.9%). In comparison, for dams where failure could result in the loss of

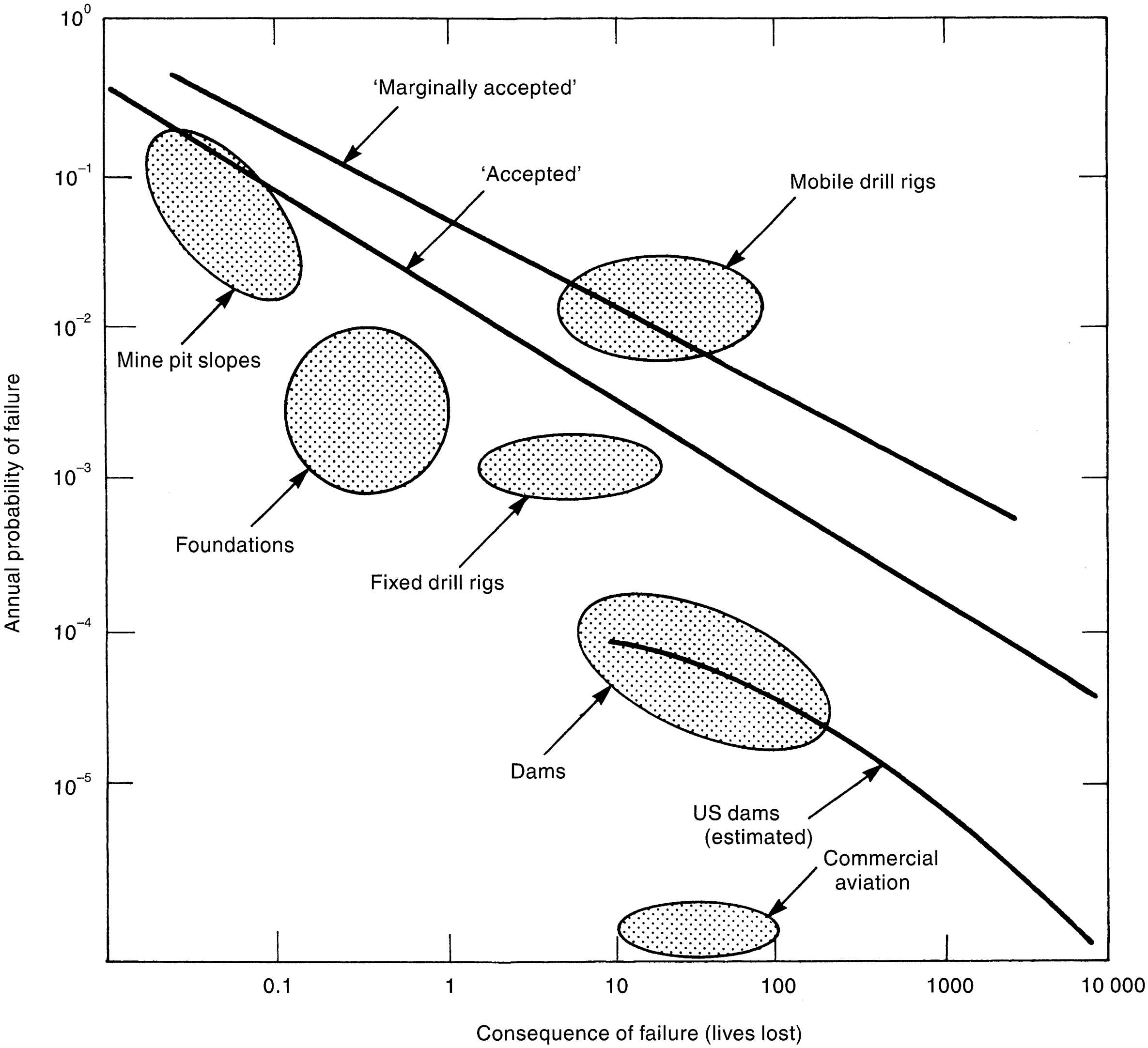

Figure 1.9 Risks for selected engineering projects (Whitman, 1984).

several hundred lives, annual probability of failure should not exceed about 10^{-4}–10^{-5} (CR = 99.99%–99.999%). Despite the wide range of values shown in Fig. 1.9, this approach provides a useful benchmark for the ongoing development of reliability based design (Salmon and Hartford, 1995).

(a) Distribution functions

In reliability analysis each parameter for which there is some uncertainty is assigned a range of values which is defined by a probability density function. Some types of distribution functions that are appropriate for geotechnical data include the normal, beta, negative exponential and triangular

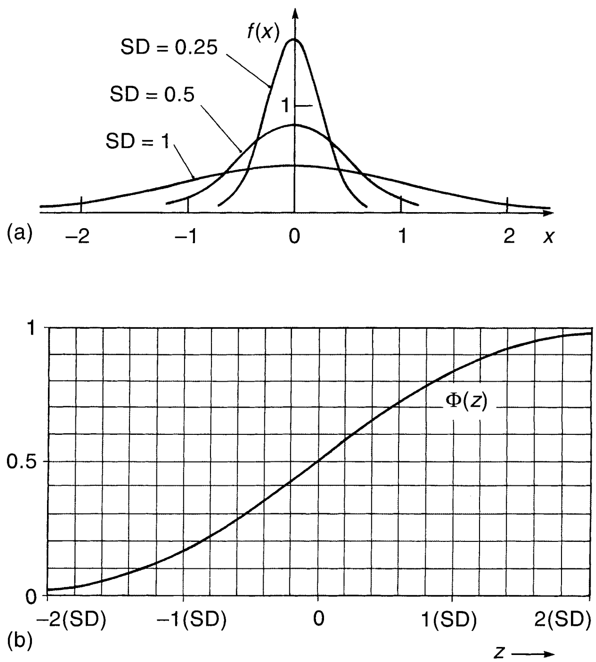

Figure 1.10 Properties of the normal distribution (Kreyszig, 1976): (a) density of the normal distribution with mean $x = 0$ and various standard deviations (SD); and (b) distribution function $\Phi(z)$ of the normal distribution with mean 0 and standard deviation 1.

distributions. The most common type of function is the normal distribution in which the mean value is the most frequently occurring value (Fig. 1.10(a)). The density of the normal distribution is defined by:

$$f(x) = \frac{1}{SD\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x - \bar{x}}{SD}\right)^2\right] \tag{1.6}$$

where $\bar{x}$ is the mean value given by

$$\bar{x} = \frac{\sum_{x=1}^{n} x}{n} \tag{1.7}$$

and SD is the standard deviation given by

$$SD = \left[\frac{\sum_{x=1}^{n} (x - \bar{x})^2}{n}\right]^{\frac{1}{2}} \tag{1.8}$$

and is the number of samples.

As shown in Fig. 1.10(a), the scatter in the data, as represented by the width of the curve, is measured by the standard deviation. Important properties of this function are that the total area under the curve is equal to 1.0. That is, there is a probability of unity that all values of the parameter fall within the bounds of the curve. Also, 68% of the values will lie within a range of one standard deviation either side of the mean and 95% will lie within two standard deviations either side of the mean.

Conversely it is possible to determine the value of a parameter defined by a normal distribution by stating the probability of its occurrence. This is shown graphically in Fig. 1.10(b) where $\Phi(z)$ is the distribution function with mean 0 and standard deviation 1. For example, a value which has a probability of being greater than 50% of all values is equal to the mean, and a value which has a probability of being greater than 16% of all values is equal to the mean minus one standard deviation.

The normal distribution extends to infinity in both directions which is often not a realistic expression of geotechnical data in which the likely upper and lower bounds of a parameter can be defined. For these conditions, it is appropriate to use the beta distribution which has finite maximum and minimum points, and can be uniform, skewed to the left or right, U-shaped or J-shaped (Harr, 1977). For conditions in which there is little information on the distribution of the data, a simple triangular distribution can be used which is defined by three values: the most likely and the minimum and maximum values. Examples of probability distributions are shown in the worked example in Section 6.2.

(b) Coefficient of reliability calculation

The coefficient of reliability is calculated in a similar manner to that of the factor of safety in that the relative magnitude of the displacing and resisting forces in the foundation are examined (see Section 1.6.1). Two common methods of calculating the coefficient of reliability are the margin of safety method and the Monte Carlo method as discussed below.

The **margin of safety** is the difference between the resisting and displacing forces, with the foundation being unstable if the margin of safety is negative. If

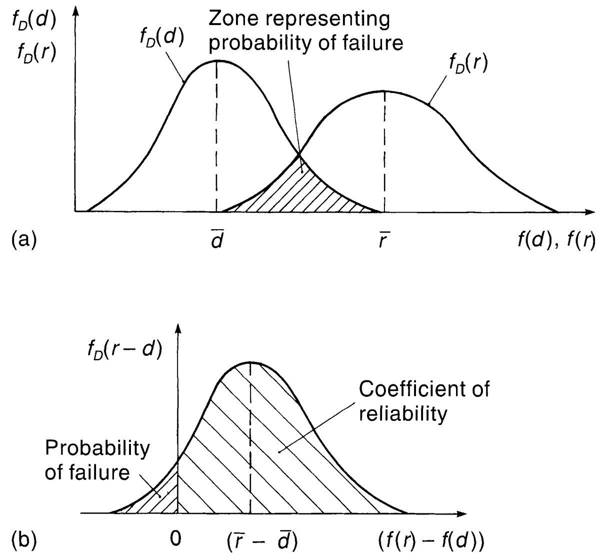

Figure 1.11 Calculation of coefficient of reliability using normal distributions: (a) probability density functions of the resisting force f_r and the displacing force f_d in a foundation; and (b) probability density function of difference between resisting and displacing force distributions $f_D(r-d)$.

the resisting and displacing forces are mathematically defined probability distributions – $f_D(r)$ and $f_D(d)$ respectively in Fig. 1.11(a) – then it is possible to calculate a third probability distribution for the margin of safety. As shown in Fig. 1.11, there is a probability of failure if the lower limit of the resisting force distribution $f_D(r)$ is less than the upper limit of the displacing force distribution $f_D(d)$. This is shown as the shaded area on Fig. 1.11(a), with the probability of failure being proportional to the area of the shaded zone. The method of calculating the area of the shaded zone is to calculate the probability density function of the margin of safety: the area of the negative portion of this function is the probability of failure, and the area of the positive portion is the coefficient of reliability (Fig. 1.11(b)). If the resisting and displacing forces are defined by normal distributions, the margin of safety is also a normal distribution, the mean and standard deviation of which are calculated as follows (Canada DEMR, 1978):

$$\text{Mean, margin of safety} = \bar{f}_r - \bar{f}_d \tag{1.9}$$

Standard deviation, margin of safety

$$= (SD_r^2 + SD_d^2)^{1/2} \tag{1.10}$$

where $\bar{f}_r$ and $\bar{f}_d$ are the mean values, and SD_r and SD_d are the standard deviations of the distributions of the resisting and displacing forces respectively. Note that the definition of the conventional factor of safety is given by $\bar{f}_r/\bar{f}_d$.

Having determined the mean and standard deviation of the margin of safety, the coefficient of reliability can be calculated from the properties of the normal distribution. For example, if the mean margin of safety is 2000 MN and the standard deviation is 1200 MN, then the margin of safety is zero at 2000/1200, or 1.67 standard deviations. From Fig. 1.10(b), where the margin of safety distribution is represented by $\Phi(z)$, the probability of failure is 5%, and the coefficient of reliability is 95%.

Note that the margin of safety concept discussed in this section can only be used where the resisting and displacing forces are independent variables. This condition would apply where the displacing force was the structural load, and the resisting force was the installed reinforcement. However, where the resisting force is the shear strength of the rock, then this force and the displacing force are both functions of the weight of the foundation, and are not independent variables. Under these circumstances, it is necessary to use Monte Carlo analysis as described below.

Monte Carlo analysis is an alternative method of calculating the coefficient of reliability which is more versatile than the margin of safety method described above. Monte Carlo analysis avoids the integration operations which can become quite complex, and in the case of the beta distribution cannot be solved explicitly. The particular strength of Monte Carlo analysis is the ability to work with any mixture of distribution types, and any number of variables, which may or may not be independent of each other.

The Monte Carlo technique is an iterative procedure comprising the following four steps (Fig. 1.12).

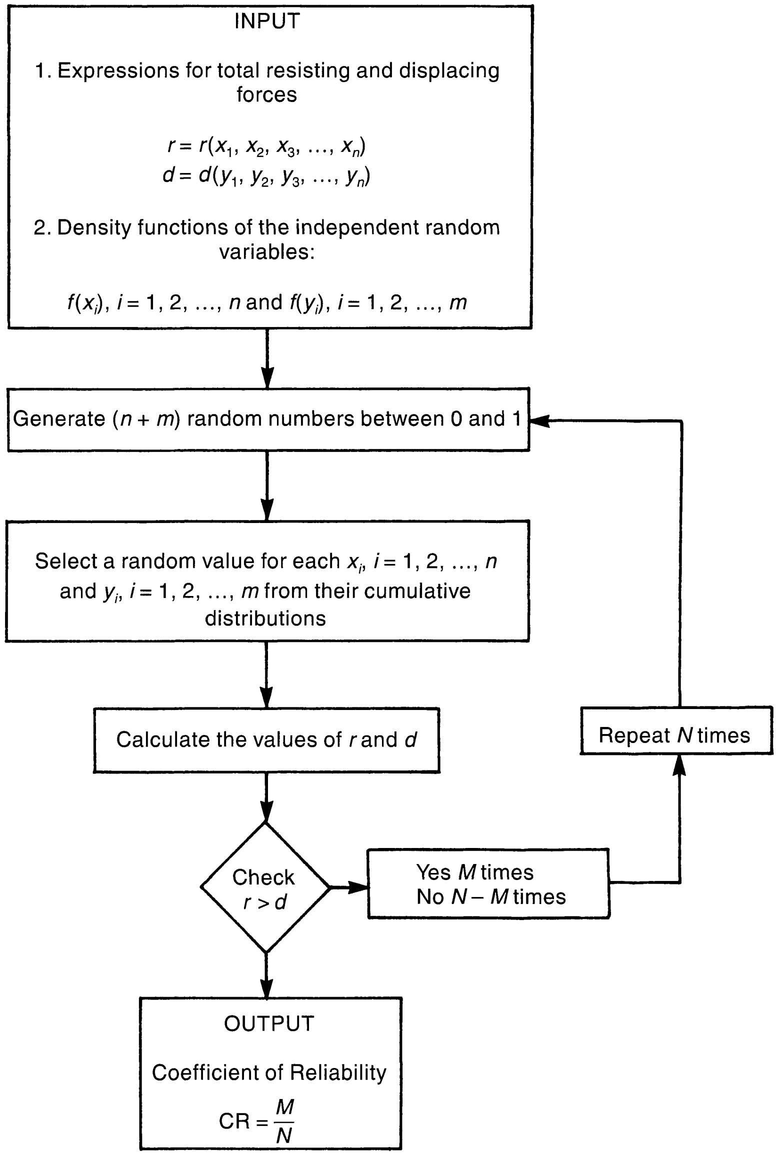

Figure 1.12 Flow chart for Monte Carlo simulation to calculate coefficient of reliability of a structure (Athanasiou-Grivas, 1980).

1. Estimate probability distributions for each of the variable input parameters.
2. Generate random values for each parameter; Fig. 1.10(b) illustrates the relationship for a normal distribution between a random number between 0 and 1 and the corresponding value of the parameter.
3. Calculate values for the displacing and resisting forces and determine if the resisting force is greater than the displacing force.