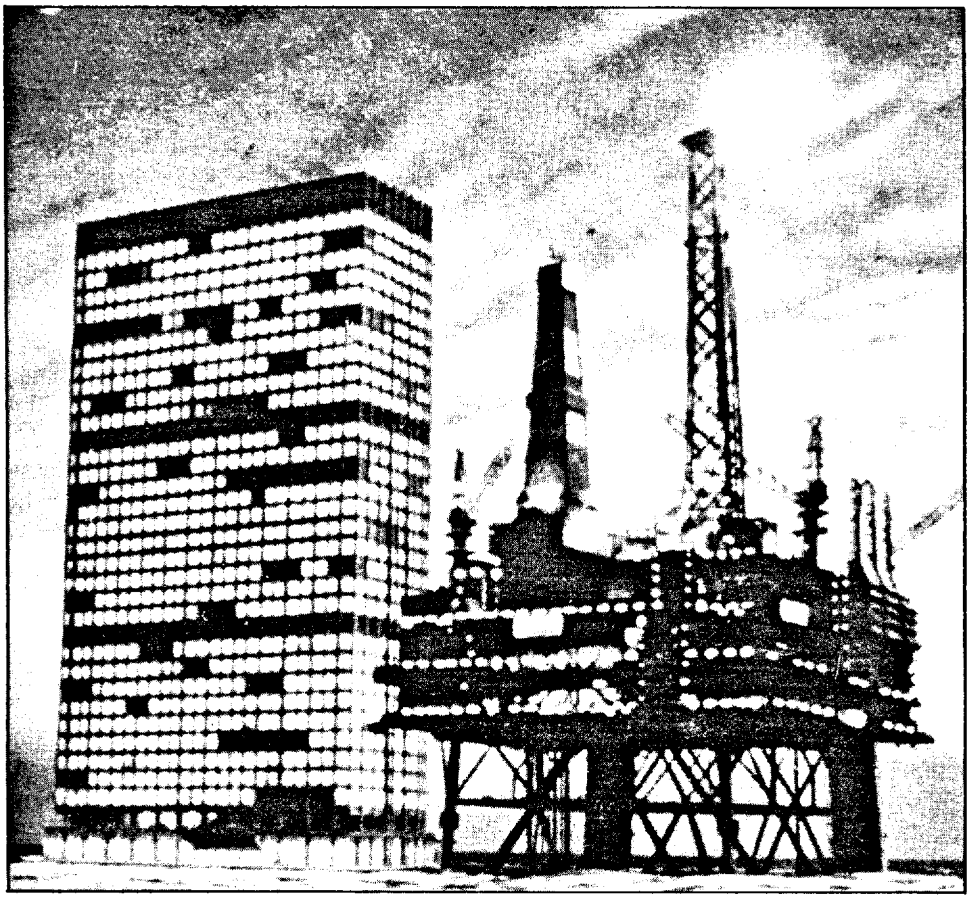
The Institution of Structural Engineers
The Institution of Civil Engineers
International Association for Bridge and Structural Engineering

MARCH 1989

Soil–structure interaction

The real behaviour of structures



Published by the Institution of Structural Engineers

The Institution of Structural Engineers
The Institution of Civil Engineers
International Association for Bridge and Structural Engineering

Soil–structure interaction

The real behaviour of structures

MARCH 1989

The Institution of Structural Engineers

11 UPPER BELGRAVE STREET, LONDON SW1X 8BH

Constitution

S. Thorburn, OBE, FEng, FIStructE, FICE, FASCE, FIHT, FGS *Chairman*
Professor J. B. Burland, MSc(Eng), PhD, FEng, MIStructE, MICE (1/4) *Vice-Chairman*
R. J. Adam, BSc(Eng), CEng, MICE (3/7)
J. Billam, PhD, CEng, MICE (resigned 1985) (1/7)
J. B. Boden, MSc, CEng, MICE (7/8)
K. W. Cole, MSc, CEng, FICE (2)
R. W. Cooke, BSc(Eng), CEng, MICE (1)
D. J. Curtis, BSc, MSc(Eng), DIC, CEng, MICE (3/8)
David J. Dowrick, BSc(Eng), BE, CEng, FICE, FIENZ – corresponding member
R. Driscoll
H. Faulkner – corresponding member
H. B. Gould, CEng, FIStructE, FICE
D. R. Green, MSc, PhD, CEng, MIStructE (1/7)
P. A. Green, ACGI, BSc, DIC, FGS, CEng, FICE (1/4)
J. M. Head. BSc
J. A. Hemsley, DIC, MEng, PhD, CEng, MIStructE, MICE (1/8)
W. J. Larnach, BSc(Hons), MSc, PhD, CEng, FIStructE, FICE
Professor G. Macchi – corresponding member
K. C. Mead, MA(Cantab), CEng, MICE (5)
R. T. Murray, BSc, PhD, CEng, MICE, MIHT (7/8)
Professor Roy Olson – corresponding member
J. F. S. Pryke, MA(Cantab), CEng, FIStructE, FICE (1/2)
M. F. Randolph, MA, PhD (5/6)
W. J. Rigden, MSc, CEng, MICE (5/6)
R. M. Semple, BSc, MSc, PhD, CEng, MICE (5/6)
Brian Simpson, MA(Cantab), PhD, CEng, MICE (4)
J. E. Spindel, BSc(Eng), PhD, ACGI, CEng, MICE (3)
Professor I. M. Smith, DSc, CEng, MICE (5/6)
W. J. R. Smyth, BA, BAI, CEng, FIStructE, FICE, FIHT (3)
I. F. Symons, MSc, BSc, CEng, MICE, MIHT (4)
W. R. Varley, BScTech, CEng, FICE (3)
B. P. Wex, OBE, BSc, FCGI, FEng, FIStructE, FICE, FIHT
L. A. Wood, BSc, PhD, CEng, MICE (2/4)
R. J. W. Milne, BSc *Secretary*

Note on method of preparation

Following the symposium on soil–structure interaction held on 5 December 1984 a decision was made to extend the 1978 report to cover other than building structures. The Committee divided into a number of working groups:

1 Building structures	(Mr. P. A. Green, Convener)
2 Underpinning	(Mr. J. F. S. Pryke, Convener)
3 Bridge structures	(Mr. W. R. Varley, Convener)
4 Earth retaining structures	(Dr. Brian Simpson, Convener)
5 Offshore structures	(Mr. W. J. Rigden, Convener)
6 Storage tank structures	(Dr. R. M. Semple, Convener)
7 Earthworks and buried structures	(Dr. R. T. Murray, Convener)
8 Tunnels and caverns	(Mr. J. B. Boden, Convener)

The reference number against the member's name indicates which Working Group(s) the member joined. Two of the Working Groups augmented their membership as indicated below:

A Steering Committee guided the Working Groups and finally edited their output into the present report. The Steering Committee consisted of:

Mr. Sam Thorburn, OBE, FEng *(Chairman)*
Professor J. B. Burland, FEng *(Vice-Chairman)*
Mr. R. W. Cooke
Mr. H. B. Gould
Dr. W. J. Larnach
Mr. B. P. Wex, OBE, FEng

Assistance to the Working Group on Tunnels and caverns was given by Mr. B. L. Bubbers, Mr. L. M. Lake and Mr. R. E. Williams, to the Working Group on Tank structures by Mr. A. F. Abbs, Dr. D. A. Greenwood, Dr. A. D. M. Penman, Mr. M. A. B. Steel, Mr. M. Sweeney and Mr. Z. Witkowski, and to the Working Group on Offshore structures by Mr. J. Clarke, Professor T. J. Poskitt, Mr. V. A. Mirza and Mr. M. J. Reardon. In addition Mr. C. Laird assisted in the drafting of Section 12 (Buried structures).

Contents

Foreword

Prior to 1970 design practice tended to consider the ground and the structure in relative isolation. The Institution of Structural Engineers, in support of the need for recognition to be given to interactive effects, formed a Special Study Group in 1971 to study the matter and make recommendations. This led to the setting-up of an *ad hoc* committee which prepared the state-of-the art report – *Structure–soil interaction* – published in 1978. In accordance with Institution procedures the relevance of the 1978 document to current practice was reviewed, and the need for revision and extension was identified.

The Institution, with the cooperation of the Institution of Civil Engineers and the International Association for Bridge and Structural Engineering, has responded to the current demand for adequate definition of the problems presented by interactive effects and has initiated the preparation of this comprehensive guidance covering most types of structure.

Sam Thorburn, *Chairman, Joint Committee*

Preamble

1 Introduction

1.1 General

The real behaviour of structures in contact with ground involves an interactive process beginning with the construction phase and ending with a state of balance after a period of adjustment of stresses and strains within the structure and within the ground influenced by the structure.

Building structures, storage tanks, bridges adjacent to high embankments on soft ground, buried pipes and culverts, retention systems, tunnels, and offshore platforms all experience interactive effects.

A retaining structure is a classic example of the problem of strain and time-dependent effects causing variations in ground pressures, and of the response of a structure to these changes.

A subjective decision may be made by designers to ignore the mechanism of structural behaviour known as soil-structure interaction, but interaction will occur and its effects may be more than envisaged. A decision to design a structure in isolation can result in a satisfactory solution provided either:

- the ground can sustain the loading with acceptable displacements, or
- the ground is treated by some suitable technique to provide appropriate stiffness and strength.

Piled foundations often have been employed to provide relatively rigid foundations and have permitted structures to be designed in isolation. Piled foundations however, although reliable, are not necessarily economic and may result in over-conservative designs in many situations.

A sympathetic treatment of problems of interaction is required except where either the stiffness and strength of the ground or of the structure are clearly dominant.

There are situations where interaction results from the existence of a structure at a particular location rather than from its weight on the ground. Ground displacements and accelerations arising from actions such as ground subsidence caused by mineral extraction, major landslips or seismic events are typical instances.

The actual behaviour of structures relates to the inherent spatial variations in the ground, and it should be appreciated that these variations are not always readily identifiable by occasional and local boring, sampling and testing.

1.2 Categories of interaction

The contents of this report are presented in two parts in order to reflect the two main categories of interaction. Part I provides guidance for the design of different types of structure supported by ground, and Part II deals with situation where ground is supported by structures.

1.2.1 Category I – Structures supported by ground

General

It is important to distinguish between two broad objectives in carrying out soil–structure interaction analyses: first, and perhaps of most concern to the engineer, is the need to estimate the form and magnitude of the relative deflections; this information is used to assess the likelihood of damage and to investigate the merits of different foundation and structural solutions: secondly, is the much more specialized requirement of calculating the distribution of forces and stresses within the structure. The second requirement entails a degree of sophistication and complexity many times greater than the first.

Golder (1969) has pointed out that engineers could estimate the settlements for a perfectly flexible load or they could estimate the average settlement of a rigid load, but in between these limits the engineers could say *nothing.* During the past few years progress has been made, but simple practical techniques are urgently required. Until this is achieved the knowledge that is being accumulated on the observed behaviour of structures will be difficult to apply. De Mello (1969) has emphasized the lack of logic in relating such information to computed differential settlements that neglect the stiffness of the structures.

Flexible building structures

For many sites underlain by ground that has been subjected to past loading the structure could be designed in relative isolation after adopting the following simple approach to the prediction of ground–structure compatibility.

The structure is considered to be flexible and to apply loads in a uniformly distributed manner over specific areas. If conventional geotechnical calculations predict ground displacements that a basic structure, its cladding, its partitions and its finishes can accommodate then no further consideration need be given to interactive effects. If, however, the calculations indicate movements that cannot be accommodated then care must be taken to ensure that the design and details of construction recognize the situation.

The decision that a building structure can accommodate the movements that are anticipated can be taken only with reference to previous experience in similar situations or to published criteria such as that presented by Burland & Wroth (1975). The limitations on the use of empiricism in design practice have been demonstrated by situations in which problems have arisen. Frequently poor structural performance has arisen from significant departures from traditional structural design, routine loading, and familiar ground conditions. The interaction of cladding and partition walls within a basic structure must not be forgotten since the response of the basic structure to loading can be modified significantly by the incorporation of these secondary elements.

In circumstances where the decision is taken that a building structure cannot accommodate the movements that are anticipated from conventional geotechnical calculations – assuming a flexible structure – movement joints may be introduced to permit articulation and to provide global flexibility. Care has to be taken in the detailing of joints in the basic structure, its foundations, and its cladding and partition walls to permit relative displacements without impairment of appearance, durability, weathertightness, and acoustic and thermal performance.

Many structures can be modified to accommodate large movements within the limits of function and aesthetics by introducing separation joints and using suitable construction materials.

Rigid building structures

Alternatively, structures having similar functional requirements can be designed to redistribute load and so achieve an acceptable reduction of differential settlement. In these instances structural design is relatively complex, and practical treatment of the subject requires reasonable assumptions to be made regarding physical models for analysis.

If the stiffness of a structure can be evaluated adequately, bearing in mind the modifying influence of progressive stages of construction, and if the ground and its stiffness moduli can be defined sufficiently by a proper investigation of the site, reasonable predictions can be made of forces and displacements.

Powerful analytical techniques are now available to designers, but at present, there is a paucity of information derived from carefully conducted full-scale tests on all types of structure to permit the differences between idealization and reality to be defined with complete confidence.

Underpinning

Successful underpinning requires an awareness of the hidden distributions of strain and stress and of the particular ground support conditions. Paths of load transfer, both primary and secondary, need to be fully defined within any structure to be underpinned, as do the probable concentrations of strain and stress in the building while in its passive condition. Further, it is important to establish the cause of settlement in a building that suddenly displays movement and, in particular, to know whether the event can be arrested by underpinning alone. Structural strengthening measures may be an essential requirement in conjunction with underpinning to arrest movements. Underpinning may be unnecessary if the event is initiated by a short-term phenomenon and the movements do not disturb significantly the natural state of balance of a building structure.

The transfer of load from a structure to its underpinning components needs to be carefully executed, and the mechanism of load distribution has to be identified and controlled to an extent commensurate with either the simplicity of the operation or its complexity and the need to restrict movements.

Bridge structures

Bridge structures are platforms capable of supporting dynamic loads, and their serviceability limits are different from those required for building structures. Buildings are containment structures providing not only structural support but also an ambience suitable for occupants or for the storage of materials.

Piled foundations for bridges do not obviate problems where soft compressible soils exist, since the major asymmetric loading imposed by high embankments behind bridge abutments induce high shear stresses in the soft soils and cause significant lateral movements of piled abutments.

Consideration has to be given to the particular problems of interaction presented by bridge structures, and the assumption of rigid supports at abutments and piers should not be made on the grounds of simplicity and ease of calculation.

Offshore structures

The cyclic nature of the environmental loading imposed on offshore platforms requires consideration to be given to the effects of cyclic stresses on soils, to the potential for liquefaction, to the possibility of seismicity, and to the fatigue of structural components.

The installation of fixed or floating platforms in offshore localities resting on or anchored to the seabed creates structures whose behaviour is interactive. Safe and economic design of these large structures involving major capital expenditure requires that dynamic and interactive responses be recognized in their designs. The economics of a design are relative since cost depends on structural provisions that may vary depending on the levels of risk accepted by the owner of the installation and by the certifying authority. The principal design engineer for an installation should be aware of the global interactive effects between an offshore structure, its foundations, and the soils that support it.

Storage-tank structures

Tanks are used for the storage of liquids having different properties and wide range of temperature. Steel and concrete are generally used in the construction of tanks. The ductility of the former material and the relative brittleness of the latter enforce the adoption of distinctly different serviceability limits and structural forms.

Steel storage tanks are often cylindrical with their thin steel plate bases resting on very soft soils. The loading intensities applied by the tank bases approach the limiting soil stresses, and large plastic strains are experienced. Severe distortion of the steel plates forming the walls and bases of tanks is often experienced.

The design of tank structures requires careful consideration to be given to the magnitude and rate of settlement, and to the distortion to which the tank elements can be subjected. The problem is often one of interaction between thin shell structures and soft soils, and interactive effects cannot be avoided if economic designs are to be evolved.

Category II – Ground supported by structures

Earth-retaining structures

Earth-retaining structures are unique in that the walls are integral components of soil–structure systems deriving both loading and support from the soil. Strain and time-dependent forces and movements cause variations in ground pressures, and retaining structures respond to these changes in order to maintain a state of balance.

For many traditional gravity or cantilever retaining walls the magnitudes of the movements required to mobilize full active pressures behind retaining walls are relatively small. This phenomenon has encouraged the use of statics in more complex designs of modern retaining structures where interactive effects have a major influence.

It is important to take into account in the design of retention systems the initial *in situ* stresses, together with the modifying effects of structural movements on lateral soil pressures and, in particular, the effects of construction.

Tunnels

The interaction between tunnel linings and the ground within the effective fields of stress around tunnels demands recognition in order to comprehend and make allowance for the real behaviour of these man-made cavities. The same recognition needs to be given in the design of unlined tunnels or caverns in competent rocks.

Considerable experience is required for the economic and safe design of tunnels, and empiricism based on careful field measurements is still used by engineers. The stress relief permitted by construction methods modifies the ground pressures, and interaction is inevitable.

Buried structures

Pipes and culverts interact with the ground, and the stresses generated both in the ground and these structures are controlled and modified by the strains that occur. Time-dependent phenomena contribute largely to the variations in stress that are experienced during the service life of the structures.

Thin-walled corrugated steel pipes and culverts depend on interactive effects for their strength and structural behaviour, and stresses are both applied and resisted by the ground surrounding them.

1.3 Ground behaviour

1.3.1 General

Ground is the generic term used to describe the basic elements of soil and rock. Codes of Practice refer frequently to these main categories of ground as superficial and solid deposits. The expression *soil–structure interaction* does not completely represent the subject since structures founded on, or retaining, weak rocks can experience interaction effects.

Inorganic soils are composed of discrete mineral particles, water and gas in solution, and exist in fully and partially saturated states. The soil particles vary in shape depending on origin and attrition and have sizes ranging generally from large gravel (60 mm) to clay fraction ($< 2\mu m$).

1.3.2 Effective stress

The strength of the discrete particles comprising the soil is generally large relative to the strength of the mass. Thus failure takes place at the grain contacts rather than through the grains. The dependence of the mechanical properties on the forces acting between the discrete particles is unique in the science of material behaviour.

The cornerstone of soil mechanics is the effective stress concept first proposed by Terzaghi (1943). He defined the effective stress on any plane through the soil as the total stress on the plane minus the porewater pressure. Since water cannot carry shear, a shear stress will always be an effective stress. The effective-stress concept states that the mechanical properties of a soil, and in particular its strength, are dependent only on the effective stresses acting in the soil.

It is evident that in order to define the effective stress on any element of soil it is necessary to know not only the total stress but also the porewater pressure. That is why groundwater conditions play such a vital role in most ground engineering problems. Changes in groundwater pressure without changes in total pressure can take place because of seepage, groundwater-table fluctuations, consolidation or swelling. All these effects will give rise to changes in effective stress and result in important, sometimes catastrophic, soil behaviour.

Fine-grained soils are relatively impermeable, and hence any tendency to change volume will take place slowly because of the length of time taken for the porewater to flow into or out of the soil pores. Therefore changes in effective normal stress will take place only slowly even though large rapid changes in total stresses might take place. Thus in the short term, the strength of the clay will be controlled by the initial effective stresses giving what is called the undrained strength, which is sometimes thought of as an apparent cohesion.

In the longer term, drainage into or out of the soil takes place giving rise to changes in strength that will be directly related to the changes in effective stress. The strength of a soil in terms of effective stresses is defined by the equation:

$$\tau_f = c' + \sigma_v' \tan \phi'$$

where ϕ' is the effective angle of friction of the soil, and c' is the effective cohesion. Both ϕ' and c' relate to the soil in its undisturbed state of stress history and should be determined for the range of stresses applicable to the particular problem. Thus ϕ' and c' are not soil constants but depend on stress history and stress level.

1.3.3 Stress history

Soils often have been affected by past loading, and the expression overconsolidation ratio is the ratio of the past maximum vertical effective stress to the present *in situ* vertical effective stress.

A soil is described as normally consolidated when an equilibrium state has been attained under the present *in situ* vertical effective stress, being the maximum vertical effective stress to which the soil has been subjected.

Estimation of overconsolidation is an essential step during the investigatory phase of a project and is complementary to the determination of the variations of the strength and of the compressibility of the ground. Overconsolidation implies the possible existence of high lateral *in situ* stresses relative to the effective vertical stresses in the ground, and the design of retention systems, tunnels and buried structures should recognize this situation.

General stress history involves overconsolidation of soils over such a large area of ground surface that the dimensions of a new structure are generally insignificant by comparison. The geographical extent of geological events causing overconsolidation is generally of sufficient dimensions as to control district, if not regional, geology. Overconsolidation related to general stress history can result from groundwater-table movements, soil erosion, glaciation, chemical weathering, cementation, and secondary compression.

Local stress history is the result of events either natural and involving desiccation of near-surface layers, or artificial and related to past loading by former buildings, road and railway embankments, etc.

1.3.4 Influence of non-homogeneity of soil

The existence of varying stiffness has a very important influence on the form and extent of the 'settlement bowl' around a loaded area. For example, Terzaghi (1943) showed that an underlying rigid stratum concentrated the surface movements around the loaded area. Gibson (1967 & 1974) noted a similar effect for increasing stiffness with depth. Conversely, a stiff overlying layer will disperse the settlements further from the loaded area. The sensitivity of surface settlements to non-homogeneity has to be taken into account in any soil–structure interaction analysis. Lateral variations of compressibility are clearly significant, but little investigation work has been carried out on the influence of this form of non-homogeneity on stress distributions beneath loaded areas.

1.3.5 Theoretical and real behaviour

The previous brief descriptions of the principles of soil behaviour are intended to demonstrate that soil variability is the rule rather than the exception and that the stress histories of soils and the dependence of mechanical behaviour on the effective stresses between the discrete particles demand recognition.

Theoretical models may not be in exact conformity with reality but may be sufficient for engineering purposes. A clear distinction must be made between adequacy and accuracy, and the reliability of analytical models depends not only on extensive use but also that the probability of failure provided by a theoretical model is a true measure.

Terzaghi (1943) expressed the opinion that the difference between the theoretical and the real behaviour of ground could be ascertained only by field experience. In every branch of applied mechanics the researcher or theoretician considers the behaviour of an ideal material, and Terzaghi emphasized it was necessary to be aware that theory has to be combined with a thorough knowledge of the physical characteristics of real ground with the additional awareness of the difference between the behaviour of soils in the laboratory and in the field.

1.4 Site and ground investigation works

1.4.1 Desk studies

Site investigations and ground investigations are separate but equally important phases of the work required to provide proper and adequate information for the design and construction of foundations. The former embraces the comprehensive investigation of a site, including past use and environmental constraint, and much of this information can be obtained from a desk study and a search of local archives. The latter is an exploratory and geotechnical investigation of the ground conditions to determine the geological structure and the characteristics of superficial and solid deposits. Both investigatory phases should be carried out in accordance with BS 5930 (1981). A useful guide to investigation procedures and equipment is provided by Weltman & Head (1983).

A site investigation generally involves the acquisition of information on the following:

- historical use of the site
- ground conditions
- groundwater regime
- mineral support conditions
- type and condition of adjacent buildings.

It is important to identify any buried features associated with the historical use of sites, and proper and extensive searches of old records, plans and memoirs should be made.

In large industrial cities, extensive areas of land surface are artificial, and have resulted from the deposition of a wide variety of materials to elevate low-lying ground and to backfill old stone quarries and clay pits. Fill materials often comprise boiler ash, steelworks slag, coarse discard from former mineral workings, chemical waste, demolition debris and excavation spoil. Household refuse can also be found within landfill sites. Ancient watercourses have been placed in culverts throughout the past few centuries and now exist as buried features. All such historical hidden features can remain unknown until exposed by the unwary.

1.4.2 Soil sampling and testing

It is generally appreciated that lack of definition of variations in ground conditions can result in completely misleading predictions of the performances of foundations. Proper methods of sampling, soil description, laboratory testing and field testing are essential if the characteristics of superficial and solid deposits are to be determined with acceptable accuracy for rigorous analyses of foundation behaviour.

In cohesive soils having soft-to-firm consistencies, continuous piston sampling is now recognized as the best method of recovering samples in a reasonably undisturbed condition for testing. The U100 open-drive sampler causes serious disturbance to the fabric of clay soils, although its use for recovery of samples of stiff-to-hard overconsolidated clays will probably continue because of the ability of the sampling equipment to withstand the hard driving stresses in such soils.

In situations where there are wide variations of results from routine testing, which are suspected to arise mainly from sampling disturbance, it is recommended that the probable undrained strength of the clay soil be assessed from correlations between the liquidity index and strength.

1.4.3 Field tests

Cone penetration tests (CPT) and self-boring or push-in-pressuremeter tests are currently used to determine the undrained strengths of clay soils, and greater use may be expected to be made of *in situ* testing to obviate the effects of sampling disturbance on laboratory tests. Information on the *in situ* condition of fine-grained non-cohesive soils can be obtained both from the standard penetration test (SPT) and the CPT. However, good drilling and cleaning techniques and careful execution of the SPT are essential in order to ensure that typical values of penetration resistance are obtained from this form of testing.

1.4.4 Rock strata

A qualitative assessment of the strength of rock strata can be made from visual examination of rock cores and from the rock quality designations (RQD). Compressive strengths obtained from uniaxial compression should be compared with the subjective assessment of strengths as defined by the Geological Society Engineering Group Working Party (1970) and subsequently referred to in BS 5930 (1981).

In order to assess intact rock strength, point load tests may be carried out on small portions of rock cores. Point load tests should be carried out axially in order to avoid simply measuring bedding-plane separation failures, and also because axial testing involves a loading direction that is more representative of the essentially vertical stresses imposed by foundations.

Considerable use has also been made of the SPT as a means of assessing the strength and stiffness of rocks. An extensive study has been made by Stroud (1974) concerning the SPT in insensitive clays and soft rocks. Stroud demonstrated that the SPT can be used to estimate the properties of clays *in situ*, and extended the correlations for stiff fissured London clay to a wide variety of clays and weak rocks.

1.4.5 Groundwater regime

The existence of a groundwater regime and the seasonal movements of the groundwater table has to be defined during the ground-investigation stage of the work. Information on the behaviour of the groundwater table is required both to aid the selection of the most suitable form of construction and to ensure that no adverse change is made to the groundwater regime during and after construction. Artificial lowering of the groundwater table to facilitate subsurface construction can cause significant increases in effective vertical soil stresses, which may result in the settlement of adjoining buildings. It should be a requirement of the investigation works to establish the real groundwater conditions within a site in relation to sources, piezometric heads, hydraulic gradients, and the influence of climatic change. The real situation can rarely be determined during the relatively short period of time afforded by boring operations, and open-type piezometers should be installed in boreholes selected because of their advantageous location within a site in respect of lateral and vertical variations in the ground conditions.

1.4.6 Mineral situation

It is important carefully to assess the mineral support conditions under a site by reference to such bodies as British Coal and the British Geological Survey. The current state of the art is defined by Healy & Head (1984) and ICE (1977). The behaviour of old mine shafts and their treatment is described by the National Coal Board (1982). The ground displacements caused by mining subsidence are unrelated to the stresses imposed by foundations unless the bases are founded unknowingly immediately above voids caused by mineral extraction.

It is now recognized that although ground displacements arising from modern active mining may be predicted with reasonable accuracy, the local and severe ground movements caused by the collapse of old pillar-and-stall workings can be estimated only roughly by empirical relationships.

1.5 Allowable movements

Compared with the literature on the prediction of settlement, the question of allowable settlements and the influence on the performance and serviceability of structures has received little attention. This is remarkable when it is considered that large sums of money are spent on soils investigations aimed at assessing probable settlement, and that the foundations of many large structures are designed specifically to limit total and differential settlement.

The problem of limiting settlements and soil–structure interaction forms a part of the much wider problem of serviceability and structural interaction. There are many obvious reasons why so little progress has been made on this universal problem. Some of them are:

- serviceability is subjective and depends both on the function of the structure and the reaction of the users
- structures vary so much one from another, both in broad concept and in detail, that it is difficult to lay down general guidelines as to allowable movements
- structures, including foundations, seldom perform as designed because construction materials display different properties from those assumed in design. Moreover, a 'total' analysis including the ground and the cladding would be impossibly complex and would still contain a number of questionable assumptions
- as well as depending on loading and settlement, movements in structures can be attributed to a number of factors such as creep, shrinkage and temperature. There is as yet little quantitative understanding of these factors, and there is a lack of careful measurements of the performance of actual structures.

There is a tendency among foundation engineers to believe that movements of foundations are the major cause of distress in structures and that by controlling these the satisfactory performance of the structures is guaranteed. The symposium on design for movement in structures (Concrete Society, 1969) clearly demonstrated that this is far from being true. The proceedings of this symposium suggest that engineers are in no better position to calculate relative movements of structural members in working conditions than they are for calculating settlements. Many cases are quoted of damage to finishes that result from movements of structural members rather than foundations. Moreover, the problem of movement in structures is becoming more important because of the modern trend towards longer spans, higher permissible stresses, greater brittleness of walls and facing materials, and larger non-structural units.

Another aspect of the problem that engineers may overlook is that a certain amount of cracking is unavoidable if the structure is to be economic (Peck *et al*, 1956). It is said that it is impossible to build a structure that does not crack because of shrinkage, creep, etc. Little (1969) has estimated that in one particular type of structure the cost of preventing any cracking could exceed 10% of the total structure cost.

In the symposium mentioned above, numerous examples are quoted of simple design and construction expedients that permit the accommodation of movement without damage, and it appears that the majority of these are relatively inexpensive. It could be argued that effort should be placed in developing and applying better design and construction details rather than in attempting to control serviceability by limiting movements. It may well be the case, but if design and construction are to be improved it is necessary to develop a clearer understanding of the relationship between movement and damage in various types of structure, and of methods of estimating such movements.

Simple guidelines are given in Section 4 which may be helpful in assessing the limiting settlement of buildings. However, these guidelines must not be treated as general rules and should never be used as a substitute for a more detailed evaluation of special features that may affect the performance of a particular structure.

1.6 Serviceability limits

It is important to differentiate between damage to the primary support elements of a structure and damage to cladding, partitions and finishes. Ground movements affect visual appearance as well as function and serviceability, but it is essential to recognize the relative unimportance of purely aesthetic considerations. Classifications of visible damage to building structures in relation to widths of structural cracks vary considerably. The relationship between serviceability and amount of visible damage is not simple, and the structural engineer has to make a decision based on assessment of the particular circumstances. It should be appreciated that slight damage may be unacceptable for a hospital in contrast to an industrial building where moderate damage is acceptable since it probably would not affect serviceability or function. Table 1 is based on BRE Digest 251 (1978) and uses ease of repair of brickwork and masonry as a measure of the category of damage. In contrast Table 2 presents a classification of damage to walls of buildings in relation to their use in service (Thorburn & Hutchinson, 1985).

Damage should not be related only to the widths of cracks, and any proper assessment of damage should take into account the means by which a structure is supported (i.e. frame or shear wall), its state of balance, the nature of the cracking (i.e. tensile or shear or a combination of both), and whether ground movements may be expected to continue. Differential movements can cause cracking and separation of walls into units that are then capable of articulation without experiencing failure provided that the structure is capable of maintaining its state of balance. Unfortunately, experience has shown that once cracking develops, from whatever source, it is probable that movements from other sources will be concentrated at these lines of weakness. Cracking that is initiated by one cause and is initially negligible may become excessive and unacceptable when other movements are superimposed.

1.7 Definitions of ground and foundation movement

Complete description of the settlement of a structure requires a large number of observation points so that detailed contours and profiles of foundation movement can be plotted. Detailed graphical presentation of observations becomes cumbersome when correlating a number of studies, and it is necessary to categorize the various types of movement that can occur.

A study of the literature on allowable settlements reveals a wide variety of symbols and terminology describing

Table 1 Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry

category of damage	description of typical damage* (ease of repair is italicized)	approximate crack width, mm
	Hairline cracks of less than about 0.1mm width are classed as negligible	$\ngtr$ 0.1**
1	*Fine crack that can easily be treated during normal decoration.* Perhaps isolated slight fracturing in building. Cracks in external brickwork visible on close inspection.	$\ngtr$ 1.0**
2	*Cracks easily filled. Redecoration probably required.* Several slight fractures showing inside of building. Cracks are visible externally and *some repointing may be required externally to ensure weathertightness.* Doors and windows may stick slightly.	$\ngtr$ 5.0**
3	*The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.* Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15** or a number of cracks >3.0
4	*Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.* Window and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15 to 25** but also depends on number of cracks
5	*This requires a major repair job* involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually >25** but depends on number of cracks

*It must be emphasized that in assessing the category of damage account must be taken of the location in the building or structure that it occurs.
**Crack width is one factor in assess category of damage and should not be used on its own as a direct measure of it.

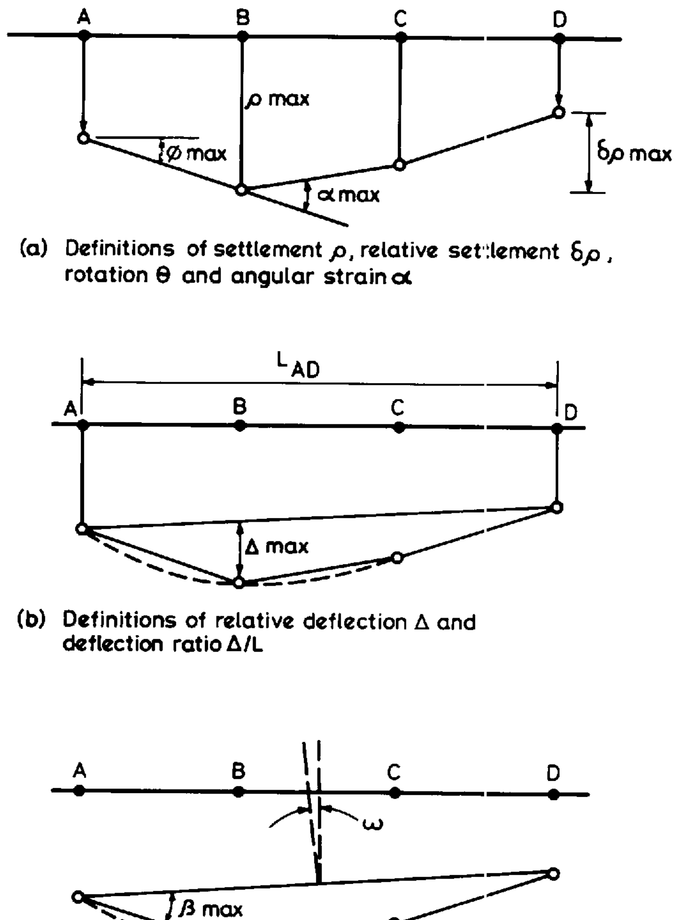
Table 2 Serviceability limits

crack width mm	degree of damage			effect on structure and building use
	dwelling	commercial or public	industrial	
$\ngtr$0.1	insignificant	insignificant	insignificant	none
0.1 to 0.3	very slight	very slight	insignificant	none
0.3 to 1	slight	slight	very slight	aesthetic only.
1 to 2	slight to moderate	slight to moderate	very slight	accelerated weathering to external features
2 to 5	moderate	moderate	slight	serviceability of the building will be affected, and towards the upper bound, stability may also be at risk
5 to 15	moderate to severe	moderate to severe	moderate	
15 to 25	severe to very severe	moderate to severe	moderate to severe	
>25	very severe to dangerous	severe to dangerous	severe to dangerous	increasing risk of structure becoming dangerous

foundation movements, much of it confusing. For example, the term 'angular distortion' has been used to describe at least four different modes of deformation.

In order to tackle the problem of allowable settlements and criteria of damage successfully it is necessary to have a clear and consistent set of definitions describing the types of movements and deformations experienced by foundations. It is important that the terms should in no way prejudice concepts about the behaviour of the associated superstructure since this will depend on a large number of other factors such as size, details of construction, materials, time, etc. The list of definitions and symbols below has been put forward by Burland & Wroth (1975). In presenting these it is assumed that the settlement of a number of discrete points is known (see Fig. 1a). However, the details of the foundation and structure are deliberately not specified, and the precise deformed shape between the observation points is not necessarily known.

Fig. 1 Definitions of foundation movement



(a) Definitions of settlement ρ, relative settlement $\delta\rho$, rotation θ and angular strain α

(b) Definitions of relative deflection Δ and deflection ratio Δ/L

(c) Definitions of tilt ω and relative rotation (angular distortion) β

The definitions and symbols for the deformation of foundation are:

- A change of length δL over a length L gives rise to an average strain $\varepsilon = \delta L/L$. A shortening of $-\delta L$ over a length L gives rise to a *compressive* strain $\varepsilon = \delta L/L$.
- *Settlement* (see Fig. 1a) is denoted by the symbol ρ and implies that the displacement is downward. If the displacement is upwards it is termed *heave* and denoted by ρ_h.

- *Differential* or *relative settlement* (or heave) is denoted by $\delta\rho$ (or $\delta\rho_h$). In Fig. 1a the settlement of C relative to D is denoted ρ_{CD} and is taken as positive. (The settlement of D relative to C is denoted by $\delta\rho_{DC}$ which equals $-\delta\rho_{CD}$). Maximum differential settlement is denoted by $\delta\rho_{max}$.
- *Rotation* is denoted by ϕ (see Fig. 1a) and is used to describe the change in gradient of the straight line joining two reference points embedded in the foundation or ground.
- *Tilt* is denoted by ω (see Fig. 1c) and normally describes the rigid body rotation of the whole superstructure or of a well defined part of it. Normally it is not possible to ascertain the tilt unless details of the superstructure and its behaviour are known. Even then it can be difficult when the structure itself flexes
- *Relative rotation* (angular distortion) is denoted by β and describes the rotation of the straight line joining two reference points relative to the tilt (see Fig. 1c). Note that the 'angular distortion' defined by Skempton & MacDonald (1956) is identical to the relative rotation
- *Angular strain* is denoted by α. From Fig. 1a it can be seen that the angular strain at B is given by

$$\alpha_B = \frac{\delta\rho_{BA}}{L_{AB}} + \frac{\delta\rho_{BC}}{L_{BC}}$$

Angular strain is positive if it produces sag or upward concavity as at B in Fig. 1a, and negative if it produces hog or downward concavity. Angular strain is useful for predicting crack width in buildings in which movement occurs at existing cracks or lines of weakness. Note that if the deformed profile between the three reference points ABC is smooth the average curvature is given by $2\alpha_B/L_{AC}$.
- *Relative deflection* (relative sag or relative hog) is denoted by Δ (see Fig. 1b) and is the maximum displacement relative to the straight line connecting two reference points a distance L apart. Relative sag produces upward concavity (as at B) for which Δ is positive. Relative hog produces downward concavity for which Δ is negative.
- *Deflection ratio* (sagging ratio or hogging ratio) is denoted by Δ/L (see Fig. 1b). The sign convention is the same as in the previous definition. The deflection ratio is identical to the 'relative deflection' quoted by Polshin & Tokar (1957). When $L_{AB} = L_{BD}$ or the deformed profile is approximately circular, $\alpha = 4\Delta/L_{AD}$.

The definitions above should be adequate to describe most types of in-plane deformation, although additions could be made, e.g. 3-dimensional behaviour such as warping.

The list of symbols and definitions relates to foundation and ground movements. Description of the behaviour of the superstructure has not been attempted since standard terminology and sign conventions in structural engineering are widely used and understood.

1.8 Dynamic response

Dynamic soil–structure interaction occurs when a structure is subject to dynamic excitations. In analysing the response of structures to dynamic excitations in the ground, soil–structure interaction is represented by the difference in calculated response assuming that the motion experienced by the base of the structure is that which would occur (i) if the structure were present, and (ii) if the structure were not present.

The difference between results obtained from assumptions (i) and (ii) is a function of the inter-relationship between the properties of the structure and the soil, and may give an increase or decrease in response compared with that obtained using the commonly adopted second assumption. In many cases the soil–structure interaction effect is negligible so that assumption (ii) may safely be adopted, although it is strictly correct only if the soil is effectively rigid. In some cases, especially under earthquake loadings, allowing for soil–structure interaction may significantly reduce the calculated responses and is thus a desirable feature to include in the analyses. To take account of dynamic soil–structure interaction the analytical model has to take due account of:

- the stiffness and damping properties of the soil as well as of the structure; and
- the travel paths of the waves involved in the dynamic excitation.

These ingredients are required to model the energy distribution in the soil–structure system, which requires adequate modelling of the manner in which energy from the source enters the system, its behaviour within the system (including scattering, concentration, dissipation), and how much energy radiates out of the system.

The creation of rigorous analytical models of the above type is possible in a few simple situations, but in general such models rapidly become highly complex within the limits of what is theoretically possible. Hence designers need in the first instance to be able to assess by simple means (e.g. see Appendix A5) whether dynamic soil–structure interaction is likely to be worth considering, and then to choose the simplest way of adequately analysir g the problem.

Energy sources that cause dynamic response in structures include:

earthquakes, wind, waves
explosions, blasting
mine collapses
machinery in factories
machinery in construction, demolition.

It is now accepted as good practice to make allowance for dynamic soil–structure interaction in major structures such as power plants, offshore platforms and tall buildings, the energy sources considered mostly being limited to the major natural causes (i.e. earthquakes and waves, as appropriate). Wind may also be included in the soil–structure interaction analysis, but this would normally be done only if the analytical model has already been set up for earthquakes or waves. If a structure was considered to be very wind-sensitive, it might be considered appropriate to analyse soil–structure interaction in the full dynamic sense, but normally it would suffice simply to model the soil stiffness and perhaps soil material damping.

The other energy sources noted above (i.e. those caused by human activity) sometimes require dynamic soil–structure interaction studies, the most common probably being foundations for vibrating machines. In other cases the nature of the energy sources and the physical situations in which they arise are so varied or so little researched that reliable theoretical modelling is usually impossible. Obviously in such cases normal empirical problem-solving techniques in dynamic response are required, which may or may not include some implicit allowance for soil–structure interaction. Information on the effects of vibratory machinery has been given by Littlejohn (1972), while response of structures to blasting and underground explosions is discussed in two reports by the American Society of Civil Engineers (1974 & 1975).

While it is not therefore possible or appropriate to assemble a set of numerical criteria for all types of loading, some qualitative design guidance pertinent to dynamic problems in general can be given. First, the engineer should attempt to assess qualitatively the vulnerability of the structure to the relevant dynamic loadings. The engineer should decide whether the dynamic response is likely to be excessive in terms of either stress or motion. Factors worth examination include resonance (or high dynamic amplification) and undue flexibility of the system.

High dynamic amplification may occur when there is a close match between the forcing frequency and a dominant structural frequency. This may occur either for the whole system or locally within the system. It may also be undesirable to have closely matching frequencies for a part of a system and for its support.

Undue flexibility either of soil or of structure can lead to excessive displacements, even in the absence of high amplification effects. For example, slender buildings or bridges sometimes are uncomfortable for users during wind or traffic loading, and flexible buildings are liable to incur excessive non-structural damage because of lateral drift in earthquakes. If a structure is to be sited on soft or loose soil, and is likely to be subjected to significant ground vibrations, it is desirable to have an integrated foundation structure with sufficient stiffness to prevent excessive differential movement of column bases.

An outline of the theoretical basis of dynamic soil–structure interaction is given in Appendix A5.

References

American Society of Civil Engineers (1974): *Study of damage to a residential structure from blast vibrations*

American Society of Civil Engineers (1975): *Structural response to explosion-induced ground motions*

BRE Digest 251 (1978): *Assessment of damage in low-rise buildings,* Building Research Station, Watford

BS 5930 (1981): *Code of practice for site investigation,* British Standards Institution, London

Burland, J. B., & Wroth, C. P. (1975): 'Settlement of buildings and associated damage', state of the art review *Proc conf., settlement of structures,* Cambridge, Pentech Press, London p. 611

Concrete Society (1969): *Proceedings of symposium on design for movement in buildings,* London

de Mello, V. F. B. (1969): 'Foundations of buildings in clay', State of the art report, *Proc. 7th int. conf. SM&FE,* Mexico, special volume, 1969, p. 49

Geological Society Engineering Group Working Party (1970): 'Report on the logging of rock cores for engineering purposes', *Quarterly Jnl Eng. Geol.,* **3,** 14

Gibson, R. E. (1967): 'Some results concerning displacements and stresses in a non-homogeneous elastic half-space', *Geotechnique,* **17,** no. 1, p. 58

Gibson, R. E. (1974): 'The analytical method in soil mechanics'. 14th Rankine Lecture, *Geotechnique,* **24,** no. 2, p. 115

Golder, H. O. (1969): Panel discussion – Session 2, *Proc. 7th Int. Conf. SM&FE,* Mexico, **3,** p. 227

Healy, P. R., & Head, J. M. (1984): *Construction over abandoned mine workings,* CIRIA special publication 32, London

Institution of Civil Engineers (1977): *Ground subsidence,* London

Little, M. E. R. (1969): Discussion – Session 6, *Proc. symp. on design for movement in buildings,* Concrete Society, London

Littlejohn, G. S. (1972): *Effects of vibrations on people, structures and ground with particular reference to vibratory equipment,* Report no. V/I, Department of Engineering, University of Aberdeen

National Coal Board (1982): *The treatment of disused mine shafts and adits,* London

Peck, R. B., Deere, D. U., & Capacete, J. L. (1956): Discussion on paper by Skempton, A. W., & MacDonald, D. H., 'The allowable settlements of buildings'. *Proc. ICE,* part 3, **5,** p. 778

Polshin, D. E., & Tokar, R. A. (1957): 'Maximum allowable non-uniform settlement of structures'. *Proc. 4th int. conf. SM&FE,* **1,** p. 402

Skempton, A. W., & MacDonald, D. H. (1956): 'Allowable settlement of buildings', *Proc. ICE,* part 3, **5,** p. 727

Stroud, M. A. (1974): 'The standard penetration test in insensitive clays and soft rocks', *Proc. Eur. symp. on penetration testing,* Stockholm

Terzaghi, K. (1943): *Theoretical soil mechanics,* John Wiley & Sons, New York

Thorburn, S., & Hutchinson, J. F. (1985): *Underpinning,* Surrey University Press, Glasgow & London

Weltman, A. J., & Head, J. M. (1983): *Site investigation manual,* CIRIA special publication 25, London

2 Design philosophy

Perhaps the most important statement that can be made about designing for the effects of soil–structure interaction is:

'Any design that relies for its success on precise analysis is a *bad* design'.

There is a feeling among many geotechnical specialists that it is necessary to convey to the structural engineer and client the same degree of apparent analytical precision that underlies much structural design. Successful design for soil–structure interaction requires an objective and realistic assessment of the bounds and confidence limits of calculations without feelings of guilt or inferiority on the part of the geotechnical engineer. He has, after all, to deal with by far the most complex and variable material, composing the *total* structure and usually has no 'say' in its specification, manufacture or placement.

2.1 Importance of the soil profile

The prime requirement for successful geotechnical design is, and always will be, a good ground investigation carried out with a knowledge of the requirements of the proposed structure. In order of importance this entails:

(i) a knowledge of the soil profile and groundwater conditions across the site set in the context of the local geology and tied in with local experience. This can usually be achieved only by the engineer visiting the site
(ii) a detailed and systematic description of the soil in each stratum in terms of its visual and tactile properties. It is important that the engineer should himself handle the various soil types and satisfy himself about their descriptions
(iii) the determination of the relevant mechanical properties of each stratum by means of laboratory and/or *in situ* tests.

The order of importance of these three requirements is significant. It is not an overstatement to say that in 95 cases out of 100 the decisions as to the type and depth of foundations can be made primarily on the basis of (i) and (ii) above. Moreover, the planning of construction procedures depends heavily on this information. Put another way, the majority of costly delays and failures result from deficiencies in the knowledge about the soil profile and groundwater conditions.

No amount of soil testing or sophisticated analysis can compensate for a lack of knowledge about the soil profile. Yet there is an increasing tendency to design on the basis of numbers contained in soils reports in the mistaken belief that these give a faithful representation of the properties of the ground. A sound understanding of the factors influencing the mechanical properties of the ground is essential. However this must be coupled with an awareness of the limitations of theories, testing techniques and information about the ground conditions.

2.2 Idealization and reality

Analytical methods have been developing so rapidly over the last few years that it is now possible to obtain solutions to many complex problems which a few years ago would have been quite out of reach. If used sensibly and with discernment these powerful analytical methods can be of considerable assistance in enabling a designer to gain a feel for the behaviour of a soil–structure system. However, if used blindly, such methods are a menace and can be extremely misleading.

The key to successful use is to gain a clear understanding of the idealizations that are being made and to be aware of how far they may be from reality. To carry out an analysis requires knowledge about the *geometry,* the *material properties* and the *loading.* These may be considered in relation to the soil and then the structure.

2.2.1 Soil geometry

Every geotechnical problem needs a site investigation, and on the basis of limited data, judgments and idealizations have to be made about the continuity and thickness of the various strata. In most cases the cost of drilling sufficient boreholes to define the exact *geometry* of the ground is prohibitive, and it is seldom that the engineer has more than an approximate model.

2.2.2 Soil properties

The difficulties of predicting appropriate values of compressibility, undrained stiffness and permeability are considerable. Approximate properties may be adequate for settlement calculations, but detailed behaviour, such as local pressure distributions and relative displacements, is much more sensitive to the form of the stress–strain/time properties of the soil and their local variations. The task of accurately ascertaining realistic *in situ* properties of most natural soils and the vertical and horizontal variations is formidable.

2.2.3 Resultant loads

The resultant externally applied loads acting on a structure supported by ground are usually reasonably well defined. The greatest difficulties arise for structures subject to dynamic forces, e.g. earthquakes, wave loading, etc.

For ground supported by structures the loads develop primarily as a result of structure–soil interaction. These depend significantly on the soil properties and structural properties but also on construction procedures. These are frequently outside the detailed control of the designer and even the contractor since the weather and other uncertainties play their part.

2.2.4 Structural geometry

The final geometry is usually accurately specified. However the geometry at any given time during construction is usually not known with any certainty.

2.2.5 Structural loading

The structural loading usually cannot be ascertained accurately, and individual members have to be designed to withstand any likely magnitude and distribution of loads. Often all the attention in structural design is devoted to the design of individual members with little or no analysis of the total structure.

2.2.6 Structural properties

The materials composing the structure are probably somewhat easier to model than the ground. Nevertheless, the stress deformation properties of the various components that comprise a building are complex, particularly with regard to creep, thermal and moisture effects. Moreover, the actual properties 'as built' undoubtedly differ significantly from those that are specified. In practice the degree of fixity at joints is uncertain, and the cladding and infill panels of buildings have varying degrees of fit. The overall stiffness of the structures is therefore difficult to assess with any accuracy.

2.3 Conclusion

It is evident from the foregoing that even if engineers were in possession of unlimited analytical power the uncertainties in the soil, the structure and the precise excavation/construction procedure are so great that precision in the prediction of behaviour would be unlikely to improve significantly. Analysis is only one of the facilities required in designing for soil–structure interaction. In most circumstances the real value of analysis will be in assisting the engineer to place bounds on overall behaviour or in assessing the influence of various construction features, e.g. a local stiffening because of a deep beam or a shear wall. Recognition of these inevitable uncertainties can lead only to improved designs.

Part I:
Structures supported by ground

3 Historical note

The 1978 report dealt essentially with building structures. It has been extended to cover many other types of structure, and Part 1 covers not only building structures but also bridge structures, offshore structures and cylindrical tanks.

Building structures

4.1 The construction sequence

Fig. 2 (Burland *et al.*, 1977) is a simple diagrammatic representation of the net loading and settlements of a simple framed building founded on a raft during and subsequent to construction. During excavation some heave of the soil will occur. The raft will then be constructed and will be influenced by the differential settlements thereafter. As the structural load is applied short-term settlements take place, the part of the structure in existence distorts and the overall stiffness gradually increases. The cladding is then added and may substantially increase the stiffness of the building. Finally, the imposed load is applied. It should be noted that not all the components of the building are subject to the same relative deflections.

The relative deflections experienced by the raft will be the largest. Those experienced by the structural members will vary with location and elevation in the building. The hatched portion in Fig. 2 represents the relative deflections, affecting the cladding, partitions and finishes, which are the cause of any architectural damage.

It is evident from Fig. 2 that the likelihood of damage will diminish the larger the proportion of immediate- to long-term settlement ρ_i/ρ_t the smaller the ratio of imposed/dead load, and the later the stage at which the finishes etc. are applied. It should be noted that the proportion of immediate- to long-term settlement is influenced by the net increase in effective stress and the amount of consolidation taking place during construction. It is frequently stated that building materials are less prone to damage when distortions develop over a long period and this appears reasonable, although Grant *et al* (1972) found little evidence to support it.

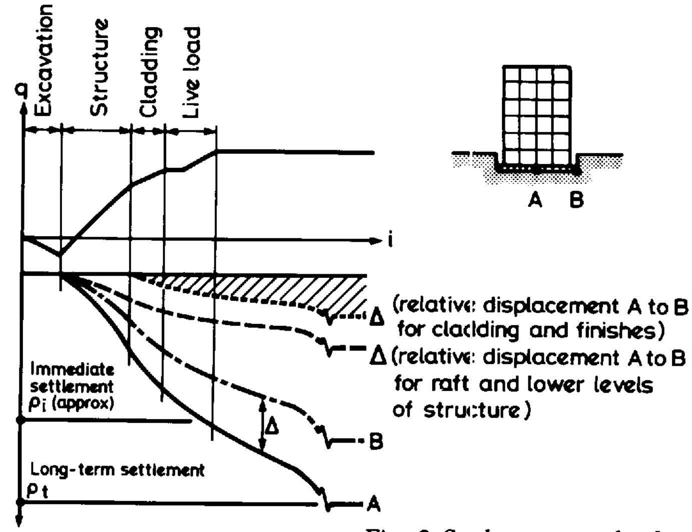
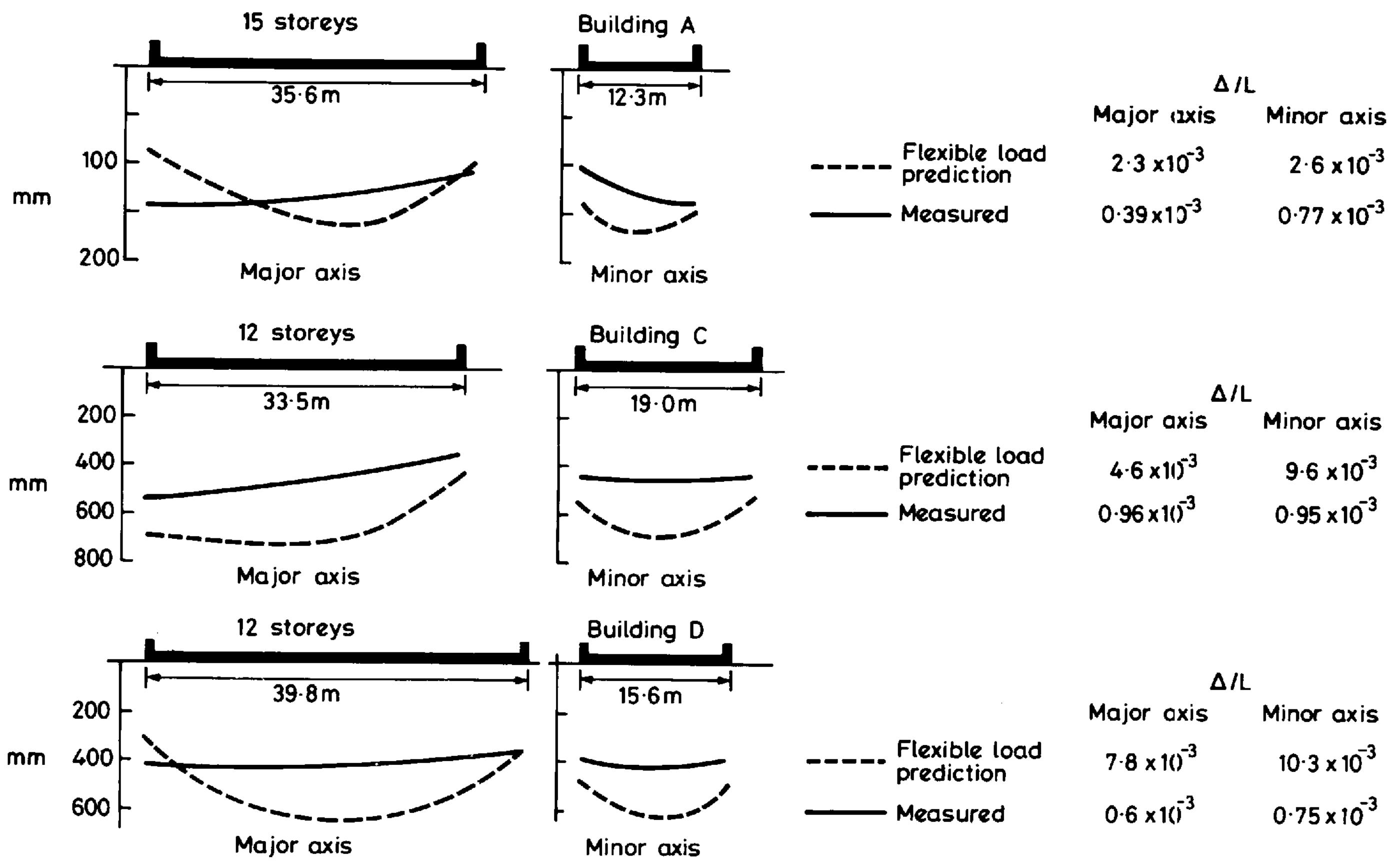


Fig. 2 Settlements and relative deflections during and subsequent to construction.



4.2 Analysis of soil–structure interaction

4.2.1 General

Charts of the type developed by Fraser & Wardle (1976), which show the relationship between stiffness and differential settlement of structures (see Appendix and Fig. A9), should prove valuable for routine design purposes or for preliminary design prior to a complete analysis. The stiffness of the superstructure can be included in this type of simple analysis using approximate methods outlined by Meyerhof (1953) for estimating the equivalent flexural rigidity of a frame superstructure, including panels and shear walls. This method was endorsed by the American Concrete Institute Committee no. 436 (de Simone, 1966).

The value of a simple approach of this type is illustrated in Fig. 3 where observations on four buildings in the city of Santos, Brazil, presented by Machado (1961) are shown. The buildings were of reinforced concrete framed construction, 12 to 15 storeys in height, founded on sand overlying a soft clay layer. Detailed estimates of the total and differential settlements were made using traditional methods assuming a flexible loaded area. It is evident that the predicted average total settlements are in reasonable agreement with the observed values, but the differential settlements are seriously overestimated.

Comparison of the *predicted* values of deflection with routine limits would have led to the conclusion that serious damage would occur. However, the *measured* relative deflections were all within tolerable limits. Unfortunately the structural details of the buildings were not given by Machado (1961) so that estimates of the relative stiffness could not be made with any accuracy (Tsytovich, 1961). Further field studies of this type are required to study the influence of superstructure stiffness on relative deflections (Rabinovici, 1970).

4.2.2 Detailed analysis

A high order of sophistication is needed if detailed analysis of forces and stresses acting on foundations and structural members is required. The Appendix gives a full discussion of the technique of interactive analysis.

Numerous studies of this type have been carried out, often using springs to represent the soil but recently using more realistic models. The finite-element idealization is particularly suited to the solution of plane or axisymmetric problems (Smith, 1970; and Hooper, 1973). However, only the simplest of structures can be analysed in this way, and resort must usually be made to a 3-dimensional analysis. Examples are given by King & Chandrasekaran (1974 & 1975) and Majid & Cunnell (1976), who have studied the influence of soil–structure interaction on bending moments in framed structures.

The use of elastic half-space or layer theory to represent the ground, coupled with a suitable idealization of the structure offers many advantages (Fraser & Wardle, 1976). Meyerhof (1947) obtained results for a simple plane frame using this approach, and recently studies of increasing sophistication have been reported including time effects, non-linearity and changes of stiffness during construction (see, for example, Sommer, 1965; Heil, 1969; Larnach, 1970; de Jong & Morgenstern, 1971; Larnach & Wood, 1972; Klepikov *et al*, 1973; Binder & de Ortigosa, 1975; and Brown, 1975). Very general computer programs have been written employing these methods (Fraser & Wardle, 1975; and Wood & Larnach, 1975 a & b) which can handle rafts and footings of arbitrary shape and rigidity, and superstructures comprising plate and beam elements. It is to be hoped that in the near future the influence of pile groups will be included, perhaps by means of equivalent rafts that take into account shear deformations as well as bending.

Programs of this type should prove useful to the engineer wishing to investigate special soil–structure interaction problems. However, in doing so the engineer should always bear in mind the limitations in knowledge about the ground and structure. Whenever possible, sensitivity studies should be carried out so that realistic upper and lower bounds can be placed on the problem. So often papers are published showing pressure distributions or bending moment distributions with no indication of the sensitivity of these to the various assumptions. It is not infrequent that a foundation that is expected to sag actually experiences hogging, and an example of such a case is given by Erb (1963).

4.3 Limiting movements

There are basically three considerations that have to be satisfied when dealing with the question of limiting movements. These are movements affecting:

- visual appearance
- serviceability and function
- stability and structural damage.

This subsection is concerned primarily with the first two, although the third is discussed briefly.

4.3.1 Relative movements affecting visual appearance

Visible deviation of members from the vertical or horizontal will often cause subjective feelings that are unpleasant and possibly alarming. Persons vary in their appraisal of relative movement and are often guided by neighbouring or adjacent buildings or members. In general, deviations from the vertical in excess of about 1/250 are likely to be noticed. For horizontal members it is suggested that a local slope exceeding 1/100 would be clearly visible, as would a deflection ratio of more than 1/250.

4.3.2 Visible damage

An important criterion of serviceability is that relating to visible damage. As mentioned in subsection 1.5 damage is difficult to quantify as it depends on subjective criteria. It is probable that if a simple classification of degrees of damage were widely adopted some of the subjective element in judging serviceability might be eliminated.

Table 1 presents a classification of damage recommended by BRE Digest 251 (1978) based on ease of repair and is derived from the work of Jennings & Kerrich (1962). Approximate crack widths are listed but intended as indicators rather than as a direct measure of degree of damage. It must be emphasized that the classification in Table 1 relates only to visible or aesthetic damage. In situations where cracking may permit corrosion of reinforcement, or allow penetration or leakage of liquids or gases, the criteria will be much more stringent. Similarly, the criteria of cracking of structural members could be much more stringent (e.g. see BS 8110, 1985, for concrete members).

The acceptable width of cracking is, to some extent, related to the scale of the structure. Thus slight crack widths (3 to 5mm) seen at eye level in brickwork in a low-rise dwelling might be regarded as unacceptable and would need repair. On the other hand it is likely that no action would be taken to repair cracks of the same width occurring at a high level in, say, a multi-storey warehouse or power station, provided of course that the weathertightness of the structure is not significantly affected.

4.3.3 Relative movements affecting serviceability and function

There is often no clear distinction between movements affecting visual appearance and movements affecting serviceability and function. Often the particular function of the building or one of its services will dictate limiting movements, e.g. overhead cranes, lifts, precision machinery, etc. The engineer should question carefully limiting movements laid down for services as these movements may be stipulated arbitrarily by the manufacturer and if adhered to may have a profound influence on the cost and construction of foundations.

There is also no simple relationship between serviceability and degree of visible damage. For example, in a hospital degree of damage worse than 'very slight' may be regarded as unacceptable, whereas for many industrial buildings 'moderate' damage may not in any way affect the serviceability of function of the building. Most residential buildings will be serviceable with 'slight' or even 'moderate' damage, although in the latter case the value of the property may be affected, and this is an important factor. The difference in cost between foundations aiming to avoid any visible cracking and those that might lead to some damage can be considerable.

4.3.4 Limiting relative settlements

Perhaps the best known study of limiting settlements of structures is that of Skempton & MacDonald (1956), and guidance for design has been based largely on their work. There is a tendency to follow these guidelines blindly with little or no account being taken of the limited range of structures studied or the criterion that was used to define limiting relative settlements. Three important points should be noted about Skempton & MacDonald's studies:

- they were confined to traditional mill-type steel-framed industrial buildings, reinforced-concrete framed buildings with traditional cladding, and some loadbearing masonry wall buildings
- the criterion for limiting deformation was the 'angular distortion' $\delta\rho/L$, which is the same as relative rotation β defined in subsection 1.7
- no classification of degree of architectural or visible damage was used.

The full significance of the choice of deformation criterion is seldom appreciated. By its very definition the implication is made that the building will tend to distort in shear. While this may well be true for framed buildings it is not necessarily the case for structures in general. Both Meyerhof (1956) and Polshin & Tokar (1957) recognized that unreinforced loadbearing walls have a different mode of deformation from that of framed structures. In recognition of this difference in behaviour Polshin & Tokar (1957) recommended that the deflection ratio Δ/L should be used as the limiting criterion for masonry and loadbearing walls.

With the distinction between framed structures and loadbearing walls the limiting deformations recommended by various investigators are summarized in Table 3.

The following points should be noted about the recommendations in Table 3:

- For framed buildings all the investigators have remarkably similar recommendations. Burland & Wroth (1975) and Grant *et al* (1974) presented data from some modern buildings that appeared to confirm these recommendations
- For loadbearing walls the L/H ratio is significant. The larger the L/H ratio the higher the limiting value of Δ/L
- Burland & Wroth (1975) have drawn attention to the fact that unreinforced walls subjected to hogging are much more susceptible to damage than similar walls undergoing sagging. Hogging modes of deformation are likely to occur adjacent to neighbouring works such as tunnels or excavations and downdrag by an adjacent building undergoing settlement. It is also a common mode of deformation for foundations on swelling and shrinking clays
- None of the investigators has attempted to correlate deformation with degree of damage. The limiting values given in Table 3 probably relate to damage not exceeding 'very slight' to 'slight' in Table 1
- The limiting values of angular distortion for structural damage in framed buildings are for structural members of average dimensions. They do not apply to exceptionally large and stiff beams or columns where the limiting values of angular distortion may be much less and should be evaluated by structural analysis.

Table 3 Summary of limiting deformations

(a) Framed buildings and reinforced loadbearing walls

Limiting values of relative rotation (angular distortion) β

	Skempton & MacDonald (1956)	Meyerhof (1956)	Polshin & Tokar (1957)	Bjerrum (1963)
structural damage	1/150	1/250	1/200	1/150
cracking in walls and partitions	1/300 (but 1/500 recommended)	1/500	1/500 (0.7/1000 to 1/1000 for end bays)	1/500

(b) Unreinforced loadbearing walls

Limiting values of deflection ratio Δ/L for the onset of visible cracking

	Meyerhof (1956)	Polshin & Tokar (1957)	Burland & Wroth (1975)
sagging	1/2500	$L/H < 3$; 1/3500 to 1/2500 $L/H < 5$; 1/2000 to 1/1500	1/2500 at $L/H = 1$ 1/1250 at $L/H = 5$
hogging (unreinforced)	—	—	1/5000 at $L/H = 1$ 1/2500 at $L/H = 5$

4.4 Fundamental damage criteria

4.4.1 General

The limiting damage criteria discussed in the previous subsection may be useful general guides but are unsatisfactory for a number of reasons. The criteria are based on observations and are therefore essentially empirical and offer no insight into the cause of damage. The criteria cannot be used for unusual structures or unusual materials. Most important of all, the criteria do not encourage the engineer to examine the details of the structures and finishes with a view to checking serviceability.

4.4.2 Limiting tensile strain

With these limitations in mind Burland & Wroth (1975) suggested that a more fundamental criterion for damage was required and put forward the idea that a criterion related to visible cracking would be useful since tensile cracking is so often associated with settlement damage. Following the work of Polshin & Tokar (1957) they assumed that the onset of visible cracking in a given material was associated with a limiting tensile strain ε_{lim}.

The application of the concept of limiting tensile strain can be illustrated by applying it to the cracking of a simple beam, which may be thought of as representing a building (see Fig. 4a). It is assumed that the deflected shape of the beam is known. The problem is to define the deflection criteria for initial cracking when the limiting tensile strain is reached at some point within the beam. Two possible extreme modes of deformation, bending only and shearing only, are shown in Figs. 4b and 4c. It is immediately obvious that the limiting deflection for initial cracking of a simple beam will depend on the ratio of L/H and on the relative stiffness of the beam in shear and in bending.

It can be shown that for a given deflection the maximum tensile strains are not very sensitive to the precise form of loading. Timoshenko gave the expression for the central deflection of a centrally loaded beam of unit thickness in both shear and bending as:

$$\Delta = \frac{PL^3}{48EI}\left[1 + \frac{18}{L^2}\cdot\frac{I}{H}\cdot\frac{E}{G}\right] \text{..............(1)}$$

where E is Young's modulus; G is the shear modulus; and I is the moment of inertia.

Eqn. 1 may be written in terms of the maximum extreme fibre strain $\varepsilon_{b(max)}$ as follows*

$$\frac{\Delta}{L} = \varepsilon_{b(max)}\cdot\frac{L}{12y}\left[1 + \frac{18}{L^2}\cdot\frac{I}{H}\cdot\frac{E}{G}\right] \text{........(2)}$$

Similarly for the maximum diagonal strain $\varepsilon_{d(max)}$ eqn. 1 becomes:

$$\frac{\Delta}{L} = \varepsilon_{d(max)}\left[1 + \frac{L^2}{18}\cdot\frac{H}{I}\cdot\frac{G}{E}\right] \text{.........(3)}$$

By setting $\varepsilon_{(max)} = \varepsilon_{(lim)}$, eqns. 2 and 3 define the limiting values of Δ/L for cracking of simple beams in bending and in shear. It is evident that for a given value of ε_{lim} the limiting value Δ/L (whichever is the lowest from eqns. 2 and 3) depends on L/H, E/G and the position of the neutral axis (and hence I).

For an isotropic beam ($E/G \doteqdot 2.5$) with neutral axis in the middle, the limiting relationship between $\Delta/L\varepsilon_{lim}$ and L/H is given by curve 1 in Fig 5.

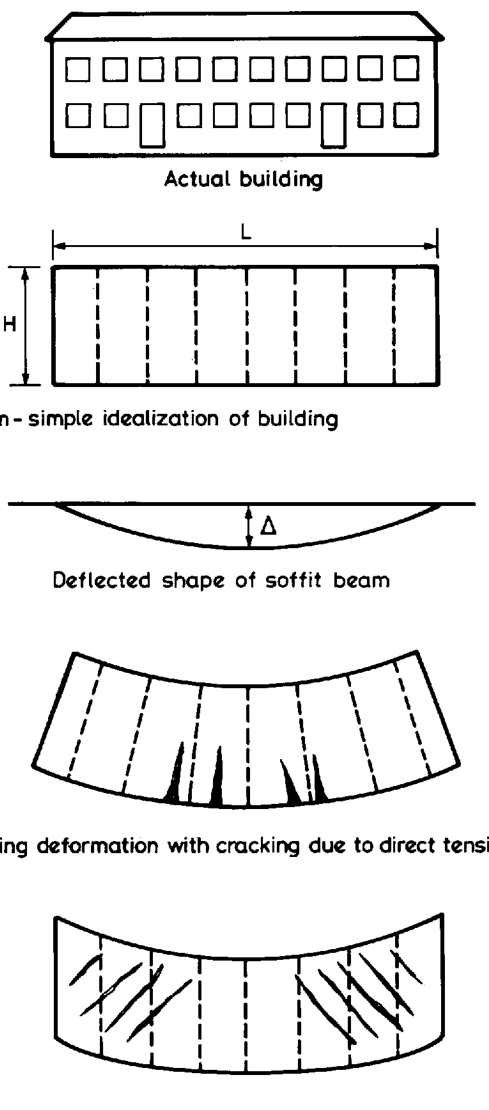


Fig. 4 Cracking of a simple beam in bending and in shear

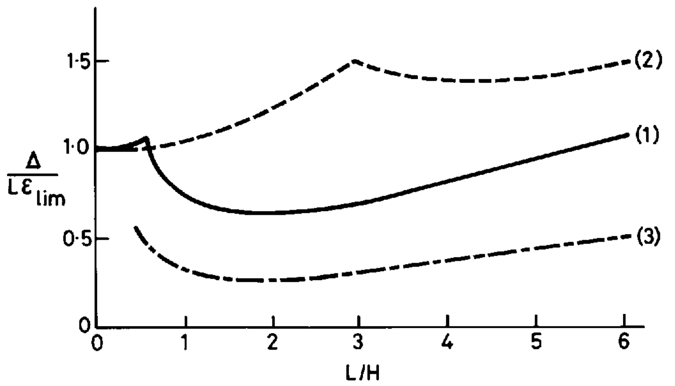


Fig. 5 Influence of E/G on the cracking of a simple rectangular beam

For a beam that has a relatively low stiffness in shear ($E/G = 12.5$) the limiting relationship is given by curve 2. A particularly important case is that of a beam that is relatively weak in bending and which is subjected to hogging such that its neutral axis is at the bottom. Curve 3 shows the limiting relationship for such a beam ($E/G = 0.5$). These curves serve to illustrate that even for simple beams the limiting deflection ratio causing cracking can vary over wide limits.

Burland & Wroth (1975) carried out a preliminary survey of data for cracking of infill frames and masonry walls and concluded that the range of values of average tensile strain at the onset of visible cracking for a variety of common building materials was remarkably small. For brickwork and blockwork set in cement mortar ε_{lim} lies between 0.05% and 0.1%, while for reinforced concrete having a wide range of strengths the values lie between 0.03% and 0.05%.

In order to assess the potential value of the limiting tensile strain approach in estimating the onset of cracking in buildings, Burland & Wroth (1975) compared the limiting criteria obtained from the analysis of simple beams with observations of the behaviour of a number of buildings, many of them of modern construction. For this comparison a value of limiting tensile strain $\varepsilon_{lim} = 0.075\%$ was used. The buildings were classified as framed, loadbearing wall undergoing sagging and loadbearing wall undergoing hogging. Figs. 6a, b and c (Skempton & MacDonald, 1956; Polshin & Tokar, 1957; Fjeld, 1963; Thorburn & McVicar, 1975; Vargas & Silva, 1973; Wood, 1952; Burhouse, 1969; Breth & Chambosse, 1975; Morton & Au, 1975; Horn & Lambe, 1964; Tschebotarioff, 1938; Cheney & Burford, 1975; Samuels & Cheney, 1975; Rigby & Dekema, 1952; and Littlejohn, 1975) show the comparison with curves 2, 1 and 3, respectively, from Fig. 5. Also shown is the criterion of limiting relative rotation $\beta = 1/300$ and the limiting relationship proposed by Polshin & Tokar (1957) for loadbearing walls. In spite of its simplicity the analysis based on tensile strain reflects the major trends in the observations. In particular, the prediction is borne out that loadbearing walls, especially when subjected to hogging, are more susceptible to damage than framed buildings, which are relatively flexible in shear. Clearly there is scope for more realistic analysis of actual structures using numerical methods of analysis. It is hoped that the success of the present over-simplified approach will stimulate further work along these lines.

At this point it is necessary to emphasize that limiting tensile strain is not a fundamental material property like tensile strength. Mainstone (1975) has pointed out that local strains during the early stages of crack development are much smaller than the values of ε_{lim} used by Burland & Wroth (1975). Hence 'limiting tensile strain' should be regarded as a measure of serviceability that, when used in conjunction with an elastic analysis, aids the engineer in deciding whether his building is likely to develop visible cracks and where the critical localities of these might be. The advantages of the approach over traditional empirical rules limiting deformation are that:

- it can be applied to complex structures employing well established stress analysis techniques
- it makes explicit the fact the damage can be controlled by paying attention to the modes of deformation within the building structure and fabric
- the limiting value can be varied to take account of differing materials and serviceability limit states, e.g. long experience has shown that the use of soft bricks and lean mortar can substantially reduce cracking, i.e. it raises the value of ε_{lim} (Girault, 1964).

Limiting strain is preferred to a 'notional' tensile strength as its value does not appear to vary a great deal for a wide range of types and strengths of common building materials. Moreover, it retains a physical significance after cracking which 'strength' does not.

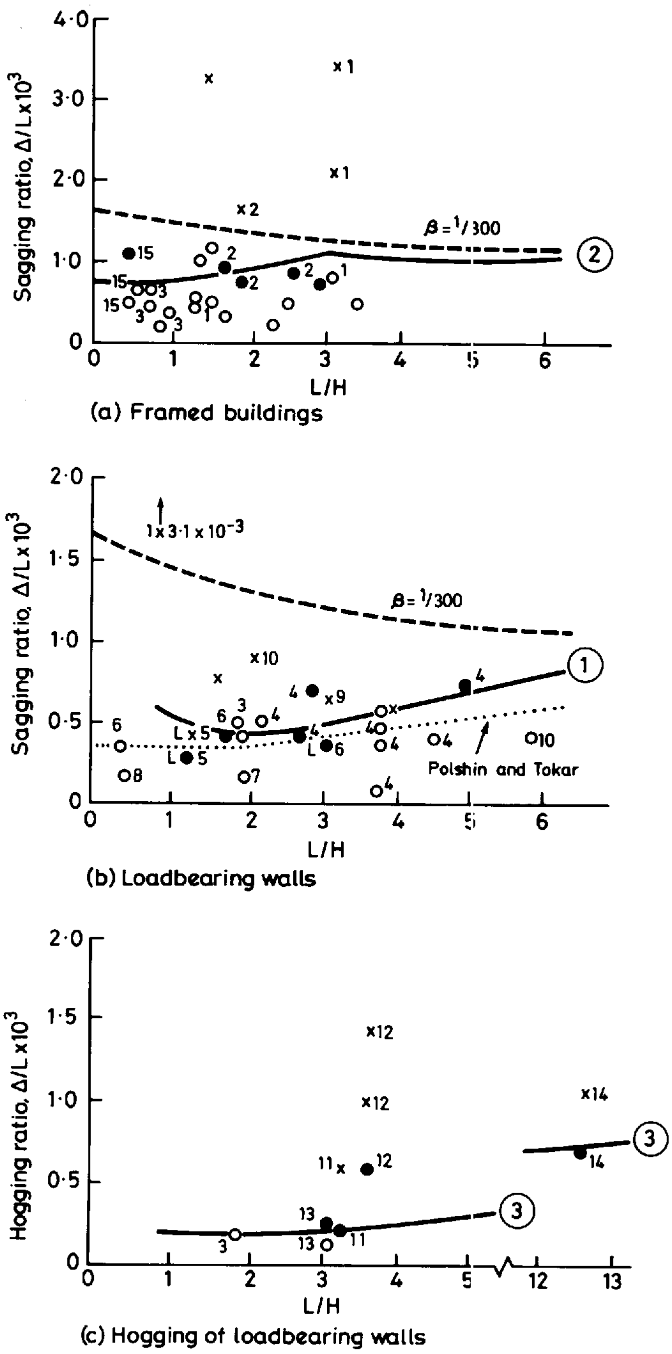
4.4.3 Crack propagation

The onset of visible cracking does not necessarily represent a limit of serviceability. Provided that the cracking is controlled, as in a reinforced concrete beam, it may be acceptable to allow deformation to continue well beyond the initiation of cracking. Cases where the propagation of initial cracks may be fairly well controlled are reinforced loadbearing structures and framed structures with panel walls. Unreinforced loadbearing walls undergoing sagging under restraining action of the foundations may also fall into this category. However, Ward (1956) has drawn attention to such a case where slip along the bitumen dampproof course resulted in extensive cracking in the overlying brickwork.

An important mode of deformation where uncontrolled cracking can occur is that of hogging of unreinforced loadbearing walls. Once a crack forms at the top of the wall there is nothing to stop it propagating downwards.

Kerisel (1975) has drawn attention to the growing problems of old buildings near tunnels, excavations or new heavy buildings. The examples he quoted emphasized the vulnerability of old buildings to the convex deformations that occurred. He suggested that the critical radius of curvature for old buildings subject to hogging was four times that for framed buildings. This is in agreement with the results given in Fig. 6. D'Appolonia (1971), Döllerl *et al* (1976) and Burland & Hancock (1977) gave detailed measurements of convex deformations alongside deep excavations. In these circumstances tensile strains in the ground may be just as significant in contributing to damage.

Green *et al* (1975) analysed cracking of brick structures employing a finite-element method incorporating a brittle limiting tension material. While such an approach is far too complex for routine design purposes, it offers a useful adjunct to future research on the relationships between movement and damage in buildings. Littlejohn (1975) described some important experiments on the cracking of brickwalls subject to mining subsidence. Such studies are essential to a proper understanding of the mechanisms of cracking arising from foundation movement.



Key
○ No damage
● Slight damage
X Severe damage
1. Skempton and MacDonald (1956)
2. Fjeld (1963)
3. Thorburn and McVicar (1975)
4. Polshin and Tokar (1957)
5. Wood (1952)
6. Burhouse (1969)
7. Breth and Chambosse (1975)
8. Morton and Au (1975)
9. Horn and Lambe (1964)
10. Tschebotarioff (1938)
11. Cheney and Burford (1975)
12. Samuels and Cheney (1975)
13. Rigby and Dekema (1952)
14. Littlejohn (1975)
15. Vargas and Silva (1973)

Fig. 6 Relationship between △/L and L/H for buildings showing various degrees of damage

Numbered points on the diagram are references, unnumbered points are from data given by Grant *et al* (1974) and Burland & Wroth (1975).

4.4.4 Discussion

The studies referred to in this subsection have served to emphasize the complexity of the problem of allowable movements and associated damage. The simple analogue of a uniform rectangular beam demonstrates that the limiting relative deflection will depend on the brittleness of the building material, the length/height ratio, the relative stiffness in shear and bending, and the mode of deformation (sagging or hogging). In addition, the propagation of cracks will depend on the degree of tensile restraint built into the structure and its foundation. All these factors point to framed buildings with panel walls being able to sustain much larger relative deflections without severe damage than unreinforced loadbearing walls. The evidence presented in Fig. 6 supports these conclusions.

It is evident from a study of this subject that there is a paucity of well documented case histories of damage. Until an adequate number of case histories becomes available for a variety of building types the temptation to lay down definitive rules on limiting deformation should be resisted as these will tend to inhibit future developments. It is more important that the basic factors are identified and appreciated by engineers.

4.5 Routine guides on limiting settlement

4.5.1 Introduction

The assessment of limiting settlements of structures is even more complex than that of limiting deformations as it brings in the behaviour of the ground and its interaction with the structure. The problem is essentially one of estimating the maximum relative deflections and rotations likely to be experienced by the structure. Analytical methods of doing this are discussed in the Appendix. Nevertheless, the practising engineer needs to know when it is reasonable for him to proceed in a routine manner, and for this he uses simple guidelines based on previous experience.

All too often such guidelines are interpreted as providing rigid rules for 'allowable maximum settlements'. Terzaghi (1956) issued a stern warning against such proposals. The problem is to provide safe simple guides without inhibiting the search for optimum solutions when appropriate. It is therefore suggested that the term 'routine limits' be used when such guidelines are proposed.

Following Terzaghi & Peck (1948) foundations on sand will be treated separately from those on clayey soils. Such a division does, of course, leave out a wide range of types of ground for which the engineer must use his judgment and experience.

4.5.2 Sands

Terzaghi & Peck (1948) suggested that for footings on sand the differential settlement is unlikely to exceed 75% of the maximum settlement, and since most ordinary structures can withstand 20mm of differential settlement between adjacent columns, a limiting maximum settlement of about 25mm was recommended. For raft foundations the limiting maximum settlement was increased to 50mm. Skempton & MacDonald (1956) correlated measured maximum relative rotation (angular distortion) with total and differential settlement for 11 buildings founded on sand. They concluded that for a safe limit of β = 1/500 the limiting maximum *differential* settlement is about 25mm and the limiting *total* settlements are about 40mm for isolated foundations and 40 to 65mm for raft foundations. The following features should be noted:

- in sands settlement takes place rapidly under load. For framed buildings, where often a significant proportion of the load is applied prior to the application of the cladding and finishes, the above guides may therefore be conservative
- no cases of damage to buildings founded on sand were reported by Skempton & MacDonald (1956) or Grant *et al* (1972)
- Terzaghi (1956) stated that he knew of no building founded on sand that had settled more than 75mm. Of the 37 settlement instances reported by Bjerrum (1963) only one exceeded 75mm, and the majority were less than 40mm. None of the cases reported by Meyerhof (1965), or Schultze & Sherif (1973) exceeded 35mm. A comprehensive review of case histories by Burland & Burbidge (1985) has confirmed that very few cases exist where buildings have settled more than 75mm on sand.

Therefore few problems should be encountered with routine buildings founded on deep layers of sand. Difficulties have occurred when vibration has taken place because of machinery and traffic or nearby construction. Also, significant settlements can occur because of large fluctuations in load as with silos (Nonveiller, 1963). Finally, it should be noted that even small quantities of organic matter, silt or clay increase the compressibility and variability significantly.

4.5.3 Clay soils

Using similar procedures to those described previously, Skempton & MacDonald (1956) concluded that for foundations on clay the design limit for maximum *differential* settlement is about 40mm. The recommended design limits for *total* settlements are about 65mm for isolated foundations and 65 to 100mm for rafts. These recommendations were criticized by Terzaghi (1956) on the grounds that the relationship between maximum relative rotation β and maximum settlement in clays is dependent on too many factors for a single value to be assinged to it. Grant *et al* (1972) have added a number of case records to the original data. These confirmed that there was no simple correlation between maximum relative rotation and maximum settlement in clays. Nevertheless, the engineer should consider whether the recommendations by Skempton & MacDonald (1956) are acceptable as routine limiting values.

Fig. 7 shows the maximum differential settlements $\delta\rho_{max}$ plotted against maximum settlements ρ_{max} for framed buildings on isolated foundations and for buildings with raft foundations. Much of the data has been taken from Skempton & MacDonald (1956) and Grant *et al* (1972) and the remainder from recent papers. As far as possible, cases have been excluded where the thickness of the compressible strata varied or where the loading intensity was significantly non-uniform. A distinction has been drawn between buildings founded directly on clayey soils and those founded on a stiff layer overlying the clay stratum.

In Fig. 7b (raft foundations) framed buildings are distinguished from buildings of loadbearing wall construction. The figures against some of the points refer to the number of storeys. Buildings showing slight to moderate damage are indicated by full points and those showing severe damage by crosses. Fig. 7 is similar to one given by Bjerrum (1963), and his suggested upper limit curves for flexible structures and rigid structures have been incorporated. The following features are particularly noteworthy:

- in both Figs. 7a and 7b the ratio between maximum differential settlement and the maximum settlement ($\delta\rho_{max}/\rho_{max}$) is less for building founded on a stiff overlying layer than for those founded directly on clay
- Bjerrum's upper limit curves for flexible and rigid structures appear to be confirmed for undamaged buildings, but it is of interest to note that many of the results for damaged buildings lie above the curve

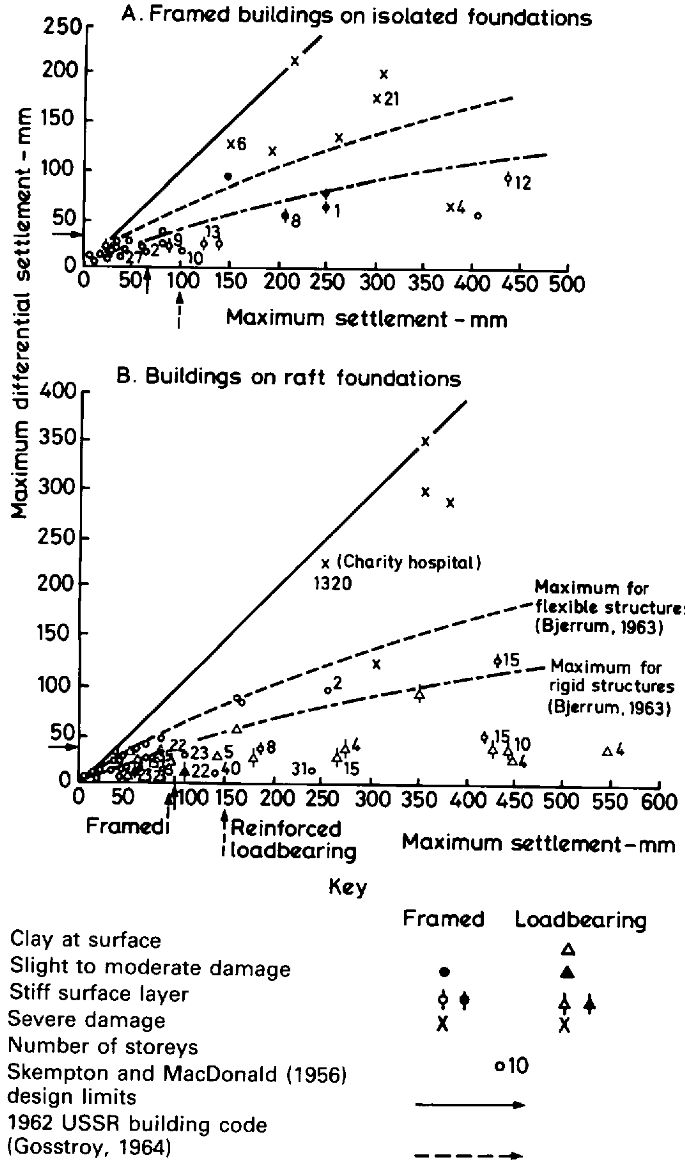


Fig. 7 Performance of buildings on clayey soils

- in Fig. 7a some cases of slight damage to buildings on isolated foundations are reported for *differential* settlements in excess of 50mm and *total* settlements in excess of 150mm
- in contrast, damage to buildings on rafts (Fig. 7b) has not been reported for differential settlements and total settlements less than 125mm and 250mm, respectively. Even these are not truly representative as one building is reported as being founded on fill and the Charity Hospital (Skempton & MacDonald, 1956) has distinctly non-uniform loading. What is clear from Fig. 7b is that many buildings on rafts have undergone substantial total settlements with no reported damage.

It must be emphasized that the diagrams are based on limited data for *uniformly* loaded buildings founded on *uniform* clayey strata. They indicate some of the factors influencing performance for these conditions. The full arrows represent the design limits suggested by Skempton & MacDonald (1956). It is not the purpose of this report to suggest alternative guides. Fig. 7 shows that there are many examples of undamaged buildings that have settled more than the limits given by Skempton & MacDonald (1956). The recommendations made by Skempton & MacDonald particularly as regards differential settlements, are probably reasonable as 'routine limits'. However, provided that it can be demonstrated that the deflection ratios Δ/L or relative rotation β (see subsection 1.7) will be within tolerable limits there appears to be no reason why larger total and differential settlements should not be accepted. Methods of calculating Δ/L, making due allowance for the stiffness of the superstructures, are discussed in the Appendix. For many stiff buildings on uniform ground the limiting settlements are likely to be governed more by considerations of tilt, damage to services entering the buildings or the influence on adjacent structures than of damage to the building itself.

4.5.4 General remarks

The discussion has covered only limiting settlements on sand and uniform clay soils. Clearly this does not cover the majority of ground conditions, including silts, peat, organic soils, residual deposits and unmade ground. For most of these there is no short cut to estimating the probable maximum distortions of the structure. Estimates have to be made of the degree of heterogeneity of the ground and its influence on the structure using such techniques as are expedient, including borings, probing and *in situ* testing. It is also necessary to take account of the proposed foundation construction method, particularly if excavation is envisaged, as it will often radically affect the compressibility of the underlying ground. Cases of damage have resulted from the induced vertical stresses in the ground locally exceeding the preconsolidation pressure (e.g. Vargas, 1955). A case history of such an instance is given by Burland & Davidson (1976). In such cases the stiffness and strength of the structure need to be sufficient to resist the local increase in compressibility of the ground.

This discussion on limiting settlements has also been confined to simple routine structures. The routine guides described above should never be applied indiscriminately to buildings and structures that are in any way out of the ordinary or for which the loading intensity is markedly non-uniform. Finally, it must always be borne in mind that the foundations and underlying ground are a part of the structure and often an economic solution to a differential settlement problem can be found by suitable design and detailing of the structural members and finishes.

4.6 Criteria for design for dynamic loading

Because of the diversity of types of dynamic loading it would not be appropriate to attempt to assemble a set of numerical criteria for all types of loading. However it is possible to give some qualitative design guidance pertinent to dynamic problems in general.

First, the designer should attempt to assess qualitatively the vulnerability of the structure to the relevant dynamic loadings. He should decide whether the dynamic response is likely to be excessive in terms of either stress or motion. Factors worth examination include resonance (or high dynamic amplification) and undue flexibility of the system.

High dynamic amplification may occur when there is a close match between the forcing frequency and a dominant structural frequency. This may occur either for the whole system or locally within the system. It may also be undesirable to have closely matching frequencies for a part of a system and for its support.

Numerical design relating to the above considerations are reasonably well developed only for earthquakes, as reflected in the codes of practice applicable in seismic regions such as California (?, 1976) or New Zealand (?, 1976), and basic guidance on structural form for earthquake resistance is given by Dowrick (1977).

Definitive criteria for damage control for vibrations of many sources other than earthquakes, such as blast vibrations and ground motions induced by explosions, pile driving and various machines, are difficult to develop. Useful information appropriate to some problems may be found from Littlejohn (1972) and two reports of the American Society of Civil engineers (1974 & 1975).

A useful summary of vibration criteria for human discomfort has been given by Littlejohn (1972). The classical work of Reiher & Meister (1931) as illustrated in Fig. 8 remains a valuable source for discomfort criteria, the subsequent recommendations of Postlethwaite (1944) and Dieckmann (1958) and the German DIN 4025 having similar results, but with an attempt to define the degree of discomfort more usefully. Dieckmann's findings (1958) are summarized in Table 4.

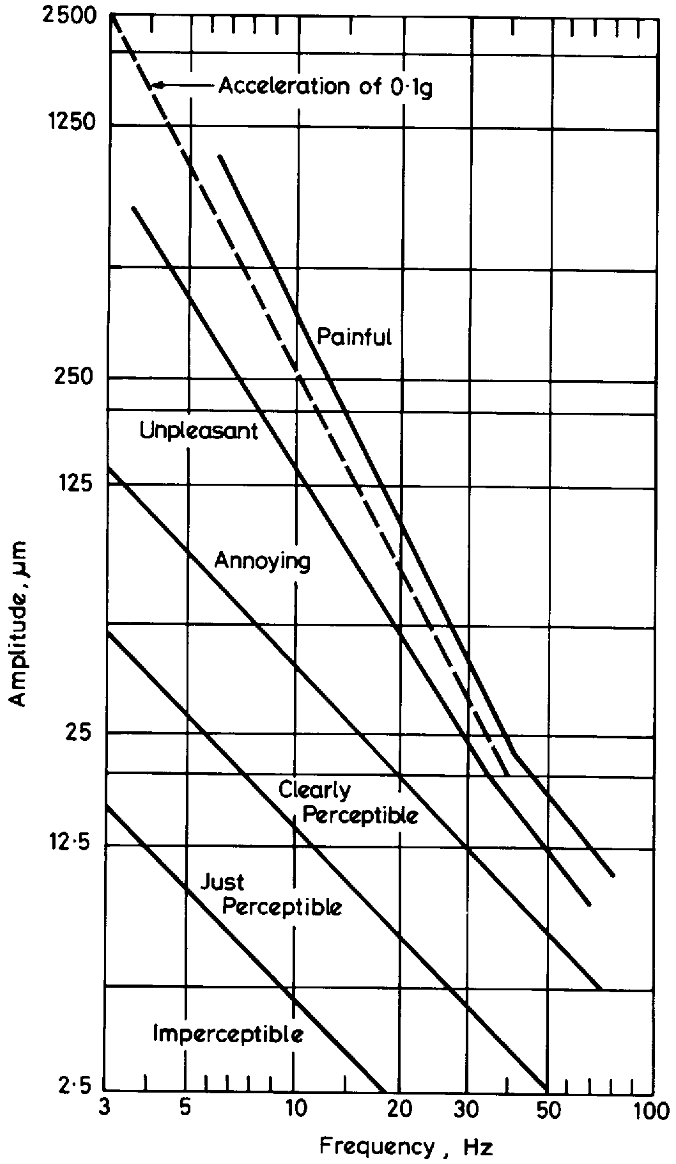


Fig. 8 Human sensitivity to vibration
Based on Reiher & Meister (1931)

Table 4 Relationship between vibration amplitude and frequency for human discomfort (after Dieckmann, 1958)

vertical vibrations		horizontal vibrations	
up to 5Hz:	$K = df^2$	up to 2Hz:	$K = 2df^2$
from 5-40Hz:	$K = 5df^2$	from 2-25Hz:	$K = 4df^2$
above 40Hz:	$K = 200d$	above 25Hz:	$K = 100d$

K-value	description of discomfort level
0.1	lower limit of human perception
1.0	allowable in industry for any period of time
10.0	allowable only for a short time
100.0	upper limit of strain or endurance for the average person

(d is the amplitude in millimetres, and f is the frequency in hertz)

4.7 Case histories

Well documented case histories provide a vital link between theory and practice. Good examples are rare and more are needed, but some have been published during recent years.

In the area of surface foundations, Holland *et al* (1979) have studied housing slabs on expansive clays, and Burland *et al* (1983) have investigated problems associated with pad footings on chalk. Cooke & Thorburn (1984) have presented field data for brickwork housing blocks on strip foundations in alluvial soil, while Toh *et al* (1985) compared the computed and observed behaviour of a raft founded on a variable sequence of soil and weathered rock. The fact that appreciable settlements can continue for decades, even for structures founded on firm interbedded clay and sand strata, has been demonstrated by Semple & Fenske (1984) who summarize the settlement records for a stiff cellular raft obtained during the period 1936-80.

In the case of piled foundations, the work of Cooke *et al* (1981) has added considerably to the understanding of field behaviour for large buildings on London clay by measuring pile loads, raft contact pressures and settlements. Similar investigations have been undertaken by Ishihara *et al* (1977) for blast furnace foundations in alluvial soil, and by Leung & Radhakrishnan (1985) for a piled raft in weak rock. Settlement records for three low-rise buildings on sand and clay strata have been presented by Kishida & Tsuji (1979), and an example of foundation failure has been described by Chin (1979). Comments on the combined effect of temperature and settlement movements have been given by Aschieri & Uliana (1984) in a field study of a thick piled raft.

Burland & Kalra (1986) described an important case history in which a limited number of piles were successfully used purely to limit the stresses in a raft foundation. This novel concept can also be used to control settlements and bending moments in raft foundations. The use of piles in this way offers considerable cost benefits.

4.8 Underpinning

4.8.1 General

Foundations need to be underpinned either when a structure is being distorted by foundation movements or because proposed new works are likely to cause foundation failure if the foundations are not improved. If investigations show that movements are continuing or that the risk that recent movements will continue is unacceptably high then underpinning should be considered.

Thorburn (1985) identified three main categories of structure and classes of underpinning:

Categories of structure
(i) ancient – greater than 150 years since completion
(ii) recent – 50 to 150 years since completion
(iii) modern – less than 50 years

Classes of underpinning
(i) conversion works
(ii) protection works
(iii) remedial works

Shoring generally is used to provide temporary support to structures while the underpinning works are being executed. The interaction between shoring and underpinning should be appreciated, and great care must be taken during the final phase of the operations involving the removal of the temporary shoring and the acceptance of all structural loads by the underpinning.

Changes to the state of balance and to the pattern of load distribution within the structure take place during all phases of the underpinning operations, and it is important to identify the mechanisms of load distribution and load sharing. An awareness and knowledge of the effects of age on the durability and performance of the materials and of the fabric of structures is essential.

Earlier and recent forms of shoring and underpinning have been described by Prentis & White (1950), Hunter (1952), Tomlinson (1986) and Thorburn & Hutchison (1985).

The usual reason for underpinning is to protect a structure alongside which, or beneath which, substantial excavations are planned. However, underpinning may also be required:

- if heavy loads are to be placed on the ground alongside an existing structure causing settlement of the ground beneath the foundations of the existing building
- if the loads on a structure are to be increased beyond the capacity of the existing foundations
- if a building is being extensively refurbished and investigation has shown that the foundations are significantly substandard by comparison with current design criteria
- if the foundation of a building needs to be strengthened before some relatively unusual operation such as raising or moving part or the whole of it.

4.8.2 Design considerations

Detailed design cannot be undertaken until all possible approaches to the underpinning process have been examined, and the engineer should look at each solution in terms of the interactions between the structure, the modified foundation and the supporting soil.

If only parts of a foundation are to be underpinned then the engineer should be satisfied that any movements between those parts being underpinned and the remainder will be acceptable.

If the problem is one of shallow foundations on shrinkable clay then failure to remove all the soil immediately beneath the existing foundations may pose a threat as the clay may be expanding, or likely to expand, causing uplift. The underpinning design should ensure that the lifting force will not exceed the load on the foundations or cause unacceptable distortion.

Some forms of underpinning involve excavations for the installation of deeper or wider foundations. The excavations will remove support from part of the foundation while the work is in progress, and care must be taken to ensure that the structure remains safe. The structure and existing foundations should be able to arch safely over the excavations. If this is not the case because the wall is too weak or too fragmented at foundation level, such as poorly jointed random rubble stone masonry, then additional work should be carried out before the underpinning commences to strengthen the foundations.

Special care should be taken when constructing traditional underpinning segments at building corners or beneath, or partly beneath, existing piers or isolated foundations. At such points work above cannot arch over and is thus more likely to need additional support. Needling, shoring or the prior construction of an underpinning beam will normally be required before the underpinning can be constructed.

There is generally no need for temporary support when piling methods of underpinning are employed. Conventional piles for underpinning are either formed *in situ* in bored holes, driven in by vibratory hammers, or jacked in. Bored cast *in situ* piles are the most common form and have a minimum diameter of 300mm. A more usual size is 450mm diameter. Piles that are jacked or vibrated in may be of a smaller size and need not be circular.

Small diameter piles may be installed close to or beneath the loads to be supported, and very small diameter mini-piles are frequently drilled through existing foundations or the bases of thick walls. Although the bearing capacities of individual mini-piles are small, these piles have proved quick and economic to install and increasingly cost effective as improved drilling and driving equipment has been developed.

4.8.3 Soil–structure interaction aspects

To predict the performance of underpinned structures the engineer needs to consider the state of the supporting soil and the effect that the underpinning technique will have on it. Additional loads will cause further settlement of the loaded surface, while other changes such as lowering ground levels or removing trees may cause heave of that surface. If it is necessary to control precisely the performance of the underpinning operation then the whole construction sequence and timing need to be specified comprehensively.

Before a building is underpinned it will have consolidated the ground directly beneath the building footings. Underpinning will remove some of this consolidated ground and apply load to less consolidated ground. All walls of a building are not always underpinned. The success of partial underpinning, where properly designed and executed, has been demonstrated by experience. However geotechnical analysis is more complicated for partial underpinning. Unless the condition of the structure and the characteristics of the ground are known, underpinning cannot be properly designed and detailed. The heavier, the older or the more unusual the structure, the more important and relatively expensive will be the investigation.

References

Aschieri, F., & Uliana, F. A. (1984): 'Seasonal temperature effects on a foundation', *Proc. ASCE*, J Geot. Engg. **110,** June, no. 6, p. 796

Binder, K., & de Ortigosa, P. (1975): 'Influencia de asentamientos diferenciales en un marco esopacial: parte II', *Revista del IDIEM,* **14,** 1, p. 1

Bjerrum, L. (1963): Discussion *Proc. Eur. conf. SM&FE,* Wiesbaden, **2,** p. 135

BRE Digest 251 (1978): *Assessment of damage in low-rise buildings,* Building Research Station, Watford

Breth, H., & Chambosse, G. (1975): 'Settlement behaviour of buildings above subway tunnels in Frankfurt clay', *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 329

Brown, P. T. (1975): 'The significance of structure–foundation interaction', *Proc. 2nd Australian–New Zealand Conf. on Geomech.,* Brisbane, p. 79

Burhouse, P. (1969): 'Composite actions between brick panel walls and their supporting beams', *Proc. ICE,* **43,** p. 175

Burland, J. B., & Thorburn, S. (1983): 'A case history of a foundation problem on soft chalk', *Geotechnique,* Dec, **33,** no. 4, p. 385

Burland, J. B., Broms, B., & de Mello, V. F. B. (1977): 'Behaviour of foundations and structures', State-of-the-art review for Session II, *Proc. 9th int. conf. SM&FE,* Tokyo

Burland, J. B., & Davidson, W. (1965): 'A case of cracking of columns supporting a silo due to differential foundation settlement', *Proc. conf. on performance of building structures,* Glasgow, Pentech Press, London

Burland, J. B., & Hancock, R. J. R. (1977): 'Geotechnical aspects of the design for the underground car park at the Palace of Westminster, London', *Structural Engineer,* **55,** no. 2, Feb, p. 87

Burland, J. B., & Davidson, W. (1976): 'A case of cracking of columns supporting a silo due to differential foundation settlement', *Proc. conf. performance of building structures,* Glasgow, Pentech Press, London, 1976

Burland, J. B., Hancock, R. J., & May, J. (1983): 'Case history of a foundation problem of soft chalk', *Geotechnique,* **33,** no. 4, Dec. 1983, p. 385

Burland, J. B., & Wroth, C. P. (1975): 'Settlement of buildings and associated damage', State-of-the-art review, *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 611

Burland, J. B., & Burbidge, M. C. (1985): 'Settlement of foundations on sand and gravel, *Proc. ICE,* **78,** Pt 1, Dec. 1985, p. 1325

Cheney, J. E., & Burford, D. (1975): 'Damaging uplift to a three-storey office block constructed on a clay soil following removal of trees', *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 337

Chin, F. K. (1979): 'A case study of soft clay–foundation–structure interaction', *Proc. 6th Asian reg. conf. SM&FE,* Singapore, July, **1,** p. 277

Cooke, R. W., & Thorburn, S. (1984): 'Observations of settlement and foundation loading on four and five-storey housing blocks on alluvial soils near the river Clyde', *Proc. symp. soil–structure interaction,* London, Dec, p. 9

Cooke, R. W., *et al* (1981): 'Some observations of the foundation loading and settlement of a multi-storey building on a piled raft foundation in London Clay', *Proc. ICE,* Pt. 1, **70,** Aug, p. 433

D'Appolonia, D. J. (1971): 'Effects of foundation construction on nearby structures', *Proc. 4th Panam conf. SM&FE,* **1,** 1971, p. 189

de Jong J., & Morgenstern, N. R. (1971): 'The influence of structural rigidity on the foundation loads of the CN tower, Edmonton', *Canadian Geotech. Jnl.,* **8,** no. 4, 1971, p. 527

de Simone, (1966): 'Suggested design procedures for combined footings and mats', *Jnl. ACI,* Oct. 1966, p. 1041

Dieckmann, D. (1958): 'A study of the influence of vibration on man', *Ergonomics,* **1,** no. 4, p. 347

Döllerl, A., Hondl, A., & Proksch, E. (1976): 'Der U-Bahn Bau und die Massnahmen zum Schutz des Stephansdomes', *Proc. 6th Eur. conf. SM&FE,* **1.1,** p. 119

Dowrick, D. J. (1977): *Earthquake resistant design—a manual for engineers and architects,* Wiley, London

Erb, Ch. (1963): 'Setzungsmessungen an einem Hochhaus', *Proc. Eur. conf. SM&FE,* Wiesbaden, **1,** p. 241

Fjeld, S. (1963): 'Settlement damage to a concrete-framed structure', *Proc. Eur. conf. SM&FE,* Wiesbaden, **1,** p. 391

Fraser, R. A., & Wardle, L. J. (1976): 'Numerical analysis of rectangular rafts on layered foundations', *Geotechnique,* **26,** no. 4, p. 613

Fraser, R. A., & Wardle, L. J. (1975): 'A rational analysis of shallow foundations considering soil–structure interaction', *Austr. Geomech. Jnl.,* **G5,** no. 1, p. 20

Girault, P. (1964): Discussion on a paper: 'Tolerance of structures to settlement', by J. Fjeld, *ASCE Conf. on design of foundation for control of settlement,* Evanston

Gosstroy (1964): *Construction Regulations,* SNiP-II-B.2-62, Chap. 2, *Bases and foundations of buildings and structures on collapsible soils,* Design Norms, Moscow, 1964 (in Russian)

Grant, R., Christian, J. T., & Vanmarcke, E. R. (1972): *Tolerance of buildings to differential settlements,* Soils Publication no 315, Dept. of Civ. Eng., MIT, 1972

Grant, R., Christian, J. T., & Vanmarcke, E. R. (1974): 'Differential settlement of buildings', *Proc. ASCE,* **100,** no. GT9, p. 973

Green, D. R., MacLeod, I. A., & Stark, W. G. (1975): 'Observation and analysis of brick structures on soft clay', *Conf. performance of building structures,* Glasgow, Pentech Press, London, **1,** p. 321

Heil, H. (1969): 'Studies on the structural rigidity of reinforced concrete building frames on clay', *Proc. 7th int. conf. SM&FE,* Mexico, **2,** p. 115

Holland, J. E., *et al* (1979): 'Behaviour of experiment housing slabs on expansive clays', *Proc. 6th Asian reg. conf. SM&FE,* Singapore, July, **1,** p. 289

Hooper, J. A. (1973): 'Observation on the behaviour of a piled-raft foundation on London clay', *Proc. ICE,* 55, Pt 2, Dec. 1973, p. 855

Horn, H. M., & Lambe, T. W. (1964): 'Settlements of buildings on the MIT Campus', *Proc. ASCE,* **90,** no. SM5, p. 181

Hunter, L. E. (1952): 'Underpinning and strengthening of structures', *Contractors Records & Municipal Engineering,* p. 00

Ishihara, K., *et al* (1977): 'Blast furnace foundations in Japan', *Proc. 9th int. conf. SM&FE,* Tokyo, July, Case History vol. 1981, p. 157

Jennings, J. E., & Kerrich, J. E. (1962): 'The heaving of buildings and the associated economic consequences, with particular reference to the Orange Free State Goldfields', *Civ. Eng. in Sth. Africa,* 5:5:112

Kerisel, J. (1975): 'Old structures in relation to soil conditions', 15th Rankine Lecture, *Geotechnique,* **25,** no. 3, p. 433

King, G. J. W., & Chandrasekaran, V. S. (1974): 'Interactive analysis of a rafted multistorey space frame resting on an inhomogeneous clay stratum', *Proc. conf. finite element methods in engineering,* Univ. NSW., p. 493

King, G. J. W., & Chandrasekaran, V. S. (1975): 'An assessment of the effects on interaction between a structure and its foundation', *Conf. settlement of structures,* Cambridge, Pentech Press, London, p. 368

Kishida, H., & Tsuji, S. (1979): 'Settlement of three buildings on piled foundations', *Proc. 6th Asian Reg. Conf. SM&FE,* Singapore, July, **1,** p. 305

Klepikov, S. N., *et al* (1973): 'Analysis of the foundation slabs and upper structure interaction', *Proc. 8th int. conf. SM&FE,* Moscow, **1.3,** p. 127

Larnach, W. J. (1970): Computation of settlements in building frames', *Civ. Eng. & Pub. Wks. Rev.,* **65,** p. 1040

Larnach, W. J., & Wood, L. A. (1972): 'The effect of soil–structure interaction on settlements', *Int. symp. comp. design,* Dept. of Eng., Warwick Univ.

Leung, C. F., & Radhakrishnan, R. (1985): 'The behaviour of a pile-raft foundation in weak rock', *Proc. 11th int. conf. SM&FE,* San Francisco, Aug, **3,** p. 1429

Littlejohn, G. S. (1972): *Effects of vibrations on people, structures and ground with particular reference to vibratory equipment,* Report no. V/1, Dept. of Eng., University of Aberdeen

Littlejohn, G. S. (1975): 'Observation of brick walls subjected to mining subsidence', *Proc. conf. on settlement of structures,* Cambridge, Pentech Press, London, p. 384

Machado, J. (1961): 'Settlement of structures in the city of Santos, Brazil', *Proc. 5th int. conf. SM&FE,* Paris, **1,** p. 719

Mainstone, R. J. (1975): Discussion – Session 5, *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 771

Majid, K. I., & Cunnell, M. D. (1976): 'A theoretical and experimental investigation into soil–structure interaction', *Geotechnique,* **26,** no. 2, p. 331

Meyerhof, G. G. (1947): 'The settlement analysis of building frames'. *Structural Engineer,* **25,** no. 9, Sept., p. 369

Meyerhof, G. G. (1953): 'Some recent foundation research and its application to design', *Structural Engineer,* **31,** June, p. 151

Meyerhof, G. G. (1956): Discussion paper by Skempton *et al* 'Settlement analysis of six structures in Chicago and London', *Proc. ICE,* **5,** no. 1, p. 170

Meyerhof, G. G. (1965): 'Shallow foundations', *Jnl. Soil Mech. Found. Div.,* ASCE, **91,** no. SM2, p. 21

Morton, K., & Au, E. (1975): 'Settlement observations on eight structures in London', *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 183

Nonveiller, E. (1963): 'Settlement of a grain silo on fine sand', *Eur. conf. SM&FE,* Wiesbaden, **1,** p. 285

Polshin, D. E., & Tokar, R. A. (1957): 'Maximum allowable non-uniform settlement of structures', *Proc. 4th int. conf. SM&FE,* **1,** p. 402

Postlewaite, F. (1944): 'Human susceptibility to vibration', *Engineering,* **157,** no. 4072, p. 61

Prentis, E. A., & White, L. (1950): *Underpinning — its practice and applications,* Columbia University Press, New York

Rabinovici, A. (1970): 'Action reciproque entre la structure et le sol de foundation', *Bull. Techn. Suisse Rom,* **96,** no. 9, p. 131

Reiher, H. J., & Meister, F. J. (1931): 'Human sensitivity to vibration', *Forsch, auf dem Geb. des Ing.,* **2,** no. 11, p. 381

Rigby, C. A., & Dekema (1952): 'Crack-resistant housing', *Pub. Wks. of South Africa,* **2,** no. 95

Samuels, S. G., & Cheney, J. E. (1975): 'Long-term heave of a building on clay due to tree removal', *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 212

Schulze, E., & Sherif, G. (1973): 'Prediction of settlements from evaluated settlement observations for sand', *Proc. 8th int. conf. SM&FE,* Wiesbaden, **1,** p. 285

Semple, R. M., & Fenske, C. W. (1984): 'Settlement of the San Jacinto monument', *Proc. symp. soil–structure interaction,* London, Dec, p. 65

Skempton, A. W., & MacDonald, D. H. (1956): 'Allowable settlement of buildings', *Proc. ICE,* part 3, **5,** p. 727

Smith, I. M. (1970): 'A finite element approach to elastic soil–structure interaction'. *Canad. Geotech. Jnl.,* **7,** no. 2, p. 95

Sommer, H. (1965): 'A method for the calculation of settlements, contact pressure, and bending moments in a foundation including the influence of the flexural rigidity of the superstructure', *Proc. 6th int. conf. SM&FE,* Montreal, **2,** p. 197

Terzaghi, K. (1956): Discussion on paper by Skempton & MacDonald 'The allowable settlement of buildings', *Proc. ICE,* part 3, **5,** p. 775

Terzaghi, K., & Peck, R. B. (1948): *Soil mechanics in engineering practice,* Wiley, New York

Thorburn, S., & McVicar, R. S. L. (1975): 'The performances of buildings founded on rivel alluvium', *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 425

Thorburn, S. (1985): *The philosophy of underpinning: book on underpinning,* Surrey University Press

Thorburn, S., & Hutchison, J. F. (1985): *Underpinning,* Surrey University Press

Timoshenko, S. (1947): *Strength of materials, Part 1,* van Nostrand, London

Toh, C. T., *et al* (1985): 'Behaviour of a raft foundation on improved soil and rock', *Proc. 8th SE Asian geotech. conf.,* Kuala Lumpur, March, **1,** p. 3-64

Tomlinson, M. J. (1986): *Foundation design and construction,* 5th ed., Pitman, London

Tschebotarioff, G. P. (1938): 'Settlement studies of structures in Egypt', *Proc. ASCE,* **64**

Tsytovich, N. A. (1961): General report — Division 3/A. *Proc. 5th int. conf. SM&FE,* **2,** p. 824

Vargas, M. (1955): 'Foundation of structures on over-consolidated clay layers in Sao Paulo', *Geotechnique,* **5,** p. 253

Vargas, M., & Silva, F. P. (1973): 'Settlement of tall buildings in Sao Paulo and Santos', *Proc. Panam. reg. conf. on tall buildings,* Porto Alegre

Ward, W. H. (1956): Discussion on paper by Skempton & MacDonald 'The allowable settlement of buildings', *Proc. ICE,* part 3, **5,** p. 782

Wood, L. A., & Larnach, W. J. (1975a): 'The effects of soil–structure interaction on raft foundations', *Proc. conf. settlement of structures,* Cambridge, Pentech Press, London, p. 460

Wood, L. A., & Larnach, W. J. (1975b): 'The interactive behaviour of a soil–structure system and its effect on settlement', *Symp. analysis soil behaviour & its application to geotechnical structures,* Univ. of NSW

Wood, R. H. (1952): *Studies in composite construction,* Part 1, National Building Studies, Research paper 13, London, HMSO

5 Bridge structures

5.1 Structural purpose

Whatever the purpose of the structure the basic principles of soil mechanics apply, and the behaviour of the structure in response to the action of the soil is controlled by its shape and form. A bridge is different in purpose, shape and form from a building, and hence its response to the strain and time-dependent effects of soil–structure interaction will not be the same as that of a building; its location is imposed as a consequence of route selection rather than adopted as commercial choice; its manner of construction is different; and also its *raison d'être*.

The cost of a bridge is largely the cost of the structure with its foundations. The only fittings are parapets, light columns, public utilities of a type that are not normally affected by minor settlements, and expansion joints and bearings whose purpose is to allow the bridge to follow a prescribed movement as it responds to all structural interaction.

A bridge is for loads moving in a single, or two opposing, directions. It provides a horizontal plane of high aspect ratio in comparison to the more squarely proportioned polygonal plan form of a building. The combination of high aspect ratio, moving loads and linear direction gives rise to dynamic and horizontal effects being significant features of design, a low capacity to resist lateral forces, and a high demand to maintain continuity of profile with a minimum number of joints. Furthermore, for highway bridges, the moving load can represent half the total load for which the structure is designed, and two-thirds of that for railway bridges. These loads are transient; a changing load is the rule, full loading a relatively rare event, but there is a need for the structure and foundation to resist the cyclical loading and the full factored loading without excessive deformation. Long-span bridges are subjected to wind loading, which may give significant and irregular movements and forces.

Soil–structure interaction begins with the first disturbance of the existing soil, and it is the problems arising from the construction and presence of the bridge that provide the initial, and probably the most marked difference, from a building in the context of soil–structure interaction. There is often a major topographical or geological feature at the bridge site, a feature that is possibly the reason for the presence of the bridge. The horizontal forces and movements referred to in the previous paragraph seldom exist on their own. They will be accompanied by vertical forces and movements, and together they act in a 3-dimensional space. In the soil the directions of movement will be controlled by soil characteristics – discontinuities, shear planes – which may translate into several directions of movement. The principal axes of the soil strata may not lie in the same direction as the principal axes of the structure.

5.2 Interface between bridge and soil

The action and reaction, the interplay between soil and structure takes place at the interface of soil and bridge foundation. The effect of differences between building and bridge in matters of purpose, shape and form have already been identified, and in addition the interface provided by a bridge structure differs from that provided by a building structure:

- the foundation loads (vertical and horizontal) are transmitted to the soil through small bearing areas, located at discrete points, that are spaced widely apart in comparison to the size of individual footings
- the bridge foundations may be considered individually but should also be considered collectively. Ground conditions often vary considerably between foundations, but movements between them, both in plan and level, have to be kept within limits that can be tolerated by the structure. Ground movements at one location can be impeded or encouraged by the presence of nearby foundations or earthworks
- a bridge designer has more freedom of choice to select a form of structure that will respond to the soil movement
- solid abutments constitute rigid beams that cannot accept local settlement but can span local ground features
- the points of support are often in water; they may be deep and may require caissons or heavy piling, and the use of raking piles, with high lateral pressures coming onto these foundations
- the bridge usually begins and ends in an embankment (manmade) or cutting side-slope (natural ground), which can cause a load on the structure. The embankment may be formed before or after construction of the bridge and may give both a vertical and horizontal interface.

5.3 Features of interaction

The design considerations that are the features of interaction are those of stiffness and movement.

Stiffness of the components and of the whole bridge structure will change during construction, and may be modified by the movements that the structure undergoes, either during construction or on completion. For example, Table 11, BS 5400: Part 4 lists effective length of columns with factors ranging from 0.7 to 2.3 depending on the restraints, while clause 5.5.1.2 remarks that the accommodation of movements will influence the degree of restraint, a factor to be assessed as accurately as possible taking account of the foundation flexibility.

Movement may be linear, variable or reversible, and may be continuous, intermittent or terminable. Changes in reactions brought about by minor movements will affect only the serviceability of the structure, and not seriously, provided that allowance is made in the design for that movement. Such allowance can be quite large in the case of simply-supported structures, more restricted in the case of continuous structures, and may be severely limited where geometrical considerations apply.

Continuous structures would include portal frames, or others with redundant reactions including arches. Geometrical limitations are imposed by considerations of, particularly, rail traffic and more seriously in the case of movable bridges, which have to return exactly to precise locations (Brown, 1937).

5.4 The reality

The reality of soil–structure interaction is illustrated in many published papers (Hambly, 1979; Cole, 1980; Marche & Lacroix, 1972; Clements, 1984; Knox *et al,* 1984; McGibbon & Booth, 1984; Cullen Wallace & Nissen, 1984; SM&FE, 1977; Wolde-Tinsae *et al,* 1980; Smyth *et al* 1980; and Brown & Mead, 1973). Hambly (1979) concentrates on the foundations and substructures of small- and medium-sized bridges which, by their number, present the majority of site problems. It summarizes the different approaches to foundation design adopted by bridge engineers in the UK. Marche & Lacroix (1972) is a study of 15 North American bridges founded on piles through a soft layer and whose abutments have experienced horizontal movements.

For major bridges, soil–structure interaction is related to the type of structure chosen. It can affect both permanent and temporary work. Examples are:

- movement and rotations of arch foundations. If the founding material is homogeneous the movements can be calculated and allowed for in the design. In rock that contains joints or fissures, however, these must be grouted since movements arising from closing the gaps cannot be determined
- friction resistance of ground to caisson sinking, which depends on the type of ground and the speed of sinking
- transmission of vertical and horizontal loads acting on a caisson to the surrounding ground and to the underside of the base, which depends on the type of ground and the geometry of the caisson
- resistance to lateral and longitudinal forces action on barrette or vertical pile foundations of viaducts. In particular the movements required to mobilize sufficient ground resistance to withstand these forces
- effects of heave at the base of large deep excavations particularly in fissured clays
- transfer of horizontal pull from a suspension bridge cable via an anchorage to the surrounding ground, in particular the relation between the load carried beneath, in front and along the sides of the anchorage and the associated movements as the cable pull increases during construction
- effects of earthquakes on pier foundations abutments – there is no universally accepted method of design against earthquake effects
- resistance of large pile groups to horizontal loading
- effects of mining subsidence on the bridge structure as a whole.

Although the existence of soil–structure interaction should be recognized and its effects analysed, it is usually possible to do this only in general terms since neither the soil parameters nor the interaction mechanism are known with any degree of accuracy.

The simple descriptions of features and interfaces that have been given in subsections 5.1 to 5.3 are enhanced or disguised by the nature of the bridge and of the ground conditions at the site. Many bridges are sited in river valleys with some foundations not merely in water but in high velocity and tidal rivers. Other conditions present at the same site may be a thick overburden of soft alluvial deposits with changing water table in, or through which, some of the supports are founded, and approach embankments that have to be founded on soft material. Soil behaviour is not amendable to precise calculation, and soil effects and their consequences over the long life expectancy of a bridge cannot be predicted with exactness.

Foundations in water can produce construction problems, but the main interaction, in the general sense, is scour. These foundations may be piled (Clements, 1984; Knox *et al,* 1984; McGibbon & Booth, 1984; and Cullen Wallace & Nissen, 1984) and may have to cope with substantial horizontal forces from the structure or the soil. Long piles can deviate from straightness because of small variations in the soil characteristics, variations not detected or incorrectly interpreted in the soil survey. Long piles, unless slip coated, mobilize large downdraw forces. The soil can be analysed as an elastic 3-dimensional medium with elastic piles embedded in it, and to analyse this some artificial boundary conditions have to be assumed. The effect of these assumptions being modified by soil–structure interaction needs to be examined.

The compensated foundation is another way of dealing with the soft ground situation (Pike & Saurin, 1952; and Hunt). Hunt describes the design and construction, alongside a main line railway, of a cellular foundation in weakly bonded sandstone overlying laminated glacial alluvial clays and peat, with a watertable at ground level. In this type of foundation, customarily the soil displaced by the foundation is nearly equal in weight to the dead load, including the foundation of the bridge, and the buoyancy effect is mobilized to support the imposed load. This type of foundation is necessarily large, and the large excavation can disturb the soil properties. It also raises the problem of what happens at the interface during the passage of the imposed load, and the effect of changes in groundwater level, short term and long term. (An increase in level of the measured water table has been noticeable in traditional industrial areas, e.g. London, Birmingham, with the reduction in water usage by industry.)

Generally for the bridge as a whole, the main interaction problems that occur, apart from differential settlement, are at the abutments and are usually caused by the weight of the approach embankment on a soft foundation (Bastin, 1983). This weight produces vertical and horizontal movements in the soft material, and if the abutment is founded on piles there can be dragdown effects (negative skin friction) and horizontal displacement of the piles. Guidance for designers confronted by this situation is given in the proceedings of the 9th international conference on soil mechanics and foundation engineering (1977). In addition, the abutment may be tilted in such a way that the ground pressure on the stem may be substantially greater than the active pressure. Cole (1980) describes a case where conventional consideration of the forces indicated outward movement and forward tilt of the abutment. However when the ground displacement arising from the fill behind the abutment was taken into account inward and backward tilting movement was forecast. Abutments are often skew, and consequently a lateral horizontal movement is introduced that may be translated into rotation about the vertical axis and transmitted through the bearings to longitudinal and lateral movements in the deck.

The actual timing, proximity and sequence of construction of the approach embankment, piles and abutment wall are crucial to the behaviour of the abutment and of the piles. The geometry and time of placing of the soil wedge between embankment and abutment is important. Customarily this is a third phase of construction, coming after the abutment, and embankment granular fill is often used to reduce compaction problems. Time has to be allowed for the porewater pressures to dissipate, yet the porewater pressures and channels under the soil wedge will have been modified by the soil's reactions to the two earlier construction phases. Allowance must be made for forward movement of the abutment, which tends to close the expansion gap, or backward movement, which may unseat bearings.

The importance of the abutment lies in the fact that not only is it the major interface between bridge and soil, but it is also the interface between the bridge and embankment, which is an earlier and different form of construction, and which has its own interface with the soil. Hence the abutment is responding to the soil–structure interaction of another construction. The practice of considering the behaviour of the completed bridge and adjacent earthworks is described by Hambly (1979), who notes that the true behaviour can be very different from the simplifying assumptions of calculations. Figs. 9 to 12 illustrate movements that can be experienced by ground and structure at bridge sites (Hambly, 1979).

Fig. 9 shows the large vertical and horizontal displacements that can be caused by the construction of an embankment or soft ground. Fig. 10 shows how the construction sequence may affect the movements experienced by a bridge deck. The settlement of the embankment on soft ground can cause large differential movements of the bankseat relative to the bridge piers.

Fig. 11 shows a piled bridge abutment retaining an embankment. Large ground movements are experienced if the ground is soft because of the high asymmetric loading imposed by the embankment. The ground strains impose downdrag and lateral loads on the piles and can cause significant differential movements of the abutment unless the ground is surcharged so that consolidation can take place before the structure is built.

Where ground is removed to form a cutting, the ground heave that results reflects the relief of vertical stress (see Fig. 12). These movements can be significant for a stiff structure such as an arch.

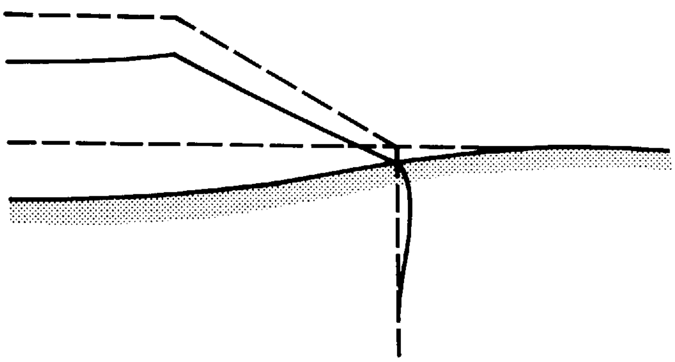


Fig. 9 Vertical and horizontal displacements caused by construction of embankment on soft ground

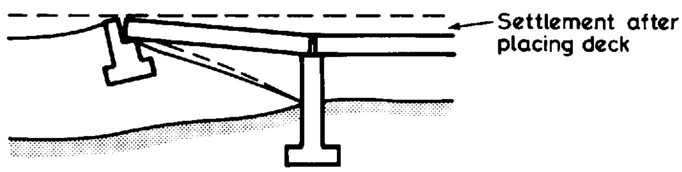


Fig. 10 Effect of construction sequence

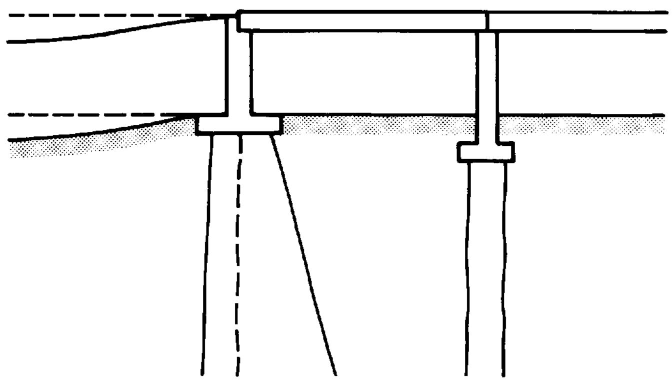


Fig. 11 Piled bridge abutment

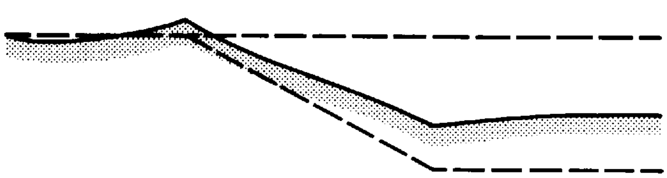


Fig. 12 Ground heave resulting from relief of vertical stress

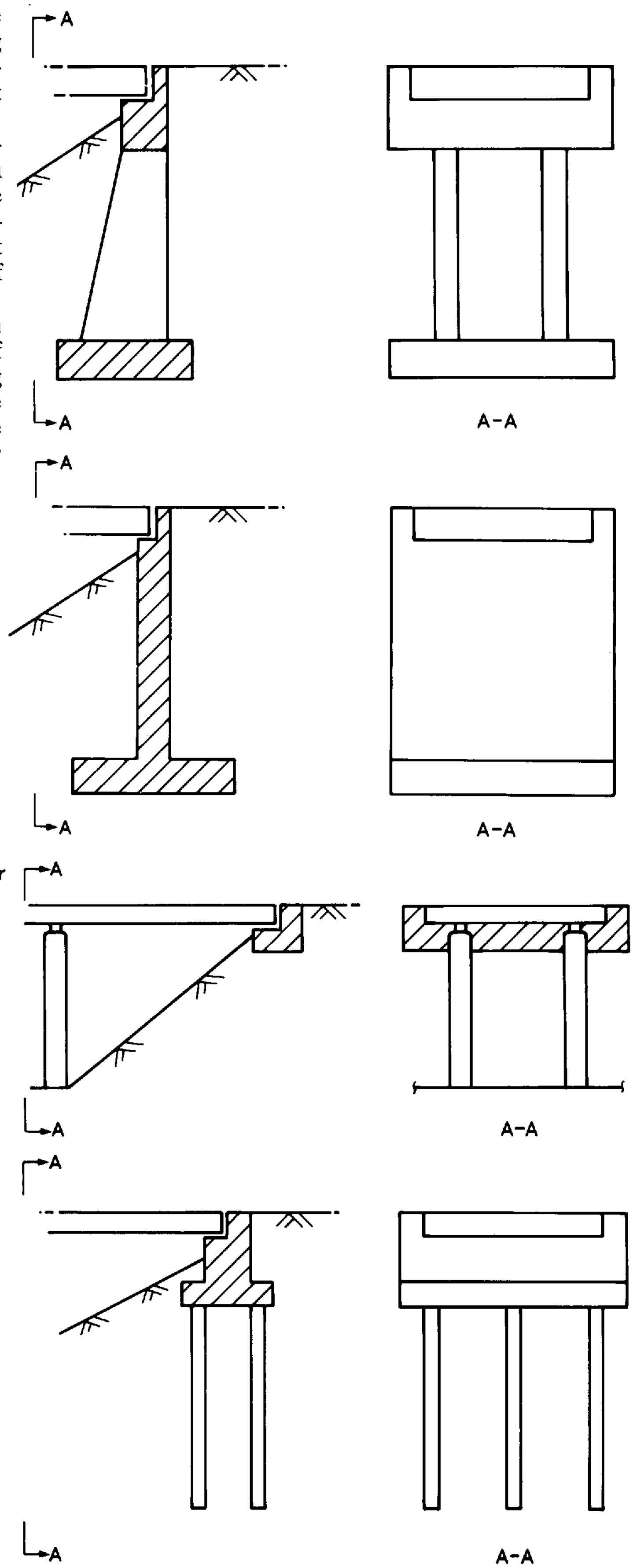


Fig.13 Main types of open abutment

a burned skeleton
b bank seats
c buried wall
d bank seats on piles

The majority of abutment forms are well catalogued. The references quoted above detail current practice in design and construction. The experiences described, although often very different, are seldom contradictory. A possible exception is the spill-through abutment. Spill-through abutments have had considerable application, but their method of analysis and design is unclear and ambiguous such that there is considerable scope for incorrectly assessing the lateral pressures applied to the piers.

5.5 Spill-through abutments

5.5.1 Introduction

Spill-through abutments have been developed with the objective of reducing the thrust from the earth fill. There are essentially four main types of open abutment as shown in Fig. 13. The buried skeleton type is most commonly used as the forces applied to the foundations are smallest and hence problems of settlement are reduced. In essence the bridge deck is supported by a series of columns or piers with sufficient space between to permit the soil to be placed at a slope corresponding to its natural angle of repose. Thus the piers are required to support only the thrust associated with the difference in earth pressure between the two faces of the piers in the direction of the slope.

The rather arbitrary design rules (Hambly, 1979) that currently apply to spill-through abutments are as follows:

(*a*) In stable embankments with slopes of 1 in 2 or shallower it is assumed that no lateral pressures are applied to the piers.

(*b*) The most common design approach, based on proposals by Chettoe & Adams (1938), assumes that full active pressures are applied to the embankment side of the piers allowing no reduction for the fill on the down-slope side. In addition, to allow for the effect of soil arching, it was suggested that the true width d, be replaced by an effective width *Ed* (see Fig. 14). The value of *E* could lie between 1 and 2 but no explicit guidance on the value to choose for particular circumstances was given.

(*c*) A proposal by Huntington (1957) was that, providing the openings were less than twice the width of the piers, they should be designed to support the full active pressure over the gross width, as for a conventional retaining wall, but taking account of the active pressures on the down-slope side with due allowance for the descending slope.

(*d*) Full active pressures over the gross width of the abutment as for a conventional retaining wall.

It is apparent that, on the basis of the above rules, rather different designs could be produced depending on the method selected. This unsatisfactory situation has stimulated further interest in the topic and has led to renewed efforts to obtain additional experimental evidence (Randolph *et al* (1985)) on which a more realistic assessment of the behaviour of spill-through abutments can be based (Lindsell, 1984). A further stimulus has been the increasing requirements to introduce limit-state concepts into all aspects of geotechnical engineering. This is to improve the compatibility of design between substructure and superstructure, as limit-state methods are currently employed for the latter situation. Although research is still continuing on spill-through abutments, the considerations given below are based mainly on these recent studies.

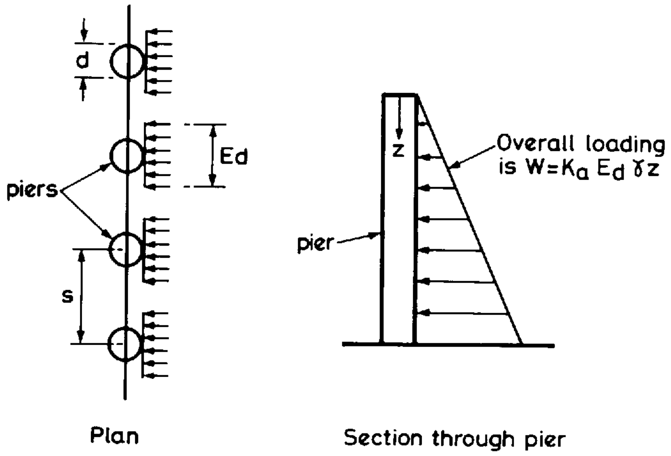


Fig. 14 Chettoe and Adams design approach

5.5.2 Design considerations

The interaction between the piers and soil in a spill-through abutment is complex being essentially a 3-dimensional problem with associated arching behaviour in the fill surrounding the piers. It would thus be unrealistic to attempt to account fully for such behaviour, and recent research has been mainly concerned with establishing how the horizontal thrusts and bending moments acting on the piers are influenced by their diameter, spacing and height for a prescribed limit condition.

The most common causes of damage or failure of spill-through abutments are associated with differential settlements of the foundations, toe wash-out or slip failure of the embankment. The limit condition invoked in the recent studies was considered to be representative of these various potential collapse mechanisms (Ah-Teck, 1983).

Design, in terms of the serviceability aspect of spill-through abutments, appears to receive little attention. Nonetheless, it is an equally important consideration and generally more relevant to the performance in the working condition. For example, rotation or translation of the piers could seriously impair the proper function of the abutment in supporting the bridge deck by reducing the expansion joints and permitting greater thrusts to be developed in the system overall.

Lateral movement of the piers may result from the same mechanisms that were referred to in the collapse assessment given above. However, because at the working condition any movements of the fill in the slope would be relatively small, the distributions of lateral pressure on the piers could be rather different from that postulated in the collapse analysis. For example, an outward rotation of the piers, resulting from differential settlement of the base, could reduce the pressures on the embankment side to the active condition. However, the pressures on the down-slope side would increase towards the passive condition. As previously for the collapse situation, the pressures would be increased by soil arching on to the piers.

Perhaps the bending moments would be most severe in association with movements of the pier towards the embankment side as this would engender the maximum passive resistance. Such a situation could again be induced by differential settlement of the base slab to the piers.

Movements of the piers may also result from the construction process and in particular the forces induced by placement of the concrete in the deck and associated beams and by subsequent thermal and shrinkage stresses in setting. Evidence for the significant influence that the construction process can have on the stresses and deformations is provided by a recent study (Lindsell, 1984) of a full-scale structure. It was apparent from observations that the most severe conditions experienced by the piers were associated with casting the deck.

5.5.3 Ultimate lateral resistance of piers

The analysis is based on the assumption that ultimate limiting conditions are induced as a result of partial removal of support to the piers on the down-slope side. Thus, as shown by the postulated rupture planes in Fig. 15, a potential wedge failure develops in the slope, the lateral pressures on the down-slope faces of the piers reduce and the differential forces and moments across the piers increase. Equilibrium analysis indicates that the critical angle of the wedge, θ, conforms closely with that of 45° − ½φ given by the Coulomb wedge analysis (Ah-Teck, 1983). The coefficient of earth pressure in the horizontal direction, K_s, on the down-slope side is then given by:

$$K_s = \frac{\cos \phi' \cos (45° + \tfrac{1}{2}\phi')}{[\tan (45° + \tfrac{1}{2}\phi') + \tan \beta] \sin (45° + \tfrac{3}{2}\phi')}$$

A further aspect of the analysis is concerned with the variation in pressure between adjacent piers on the embankment side. As a result of soil arching, the postulated distributions are shown in Fig. 16. As shown on the figure, the lateral earth pressure coefficient is assumed to vary from a maximum value of K_m directly behind the piers to the minimum value of K_s between piers, provided that the piers are sufficiently far apart. A recent parametric study employing centrifuge models indicated that the reduction in horizontal stress with distance from the pier was proportional to a ⅔ power law. The maximum value of lateral earth pressure coefficient, K_m, will be governed by the placement and compaction procedures for the fill, as well as by the flexibility of the piers. Because the construction process cannot be properly simulated in centrifuge model testing, the upper limiting lateral earth pressure coefficient in this study was the at-rest value K_o. On the

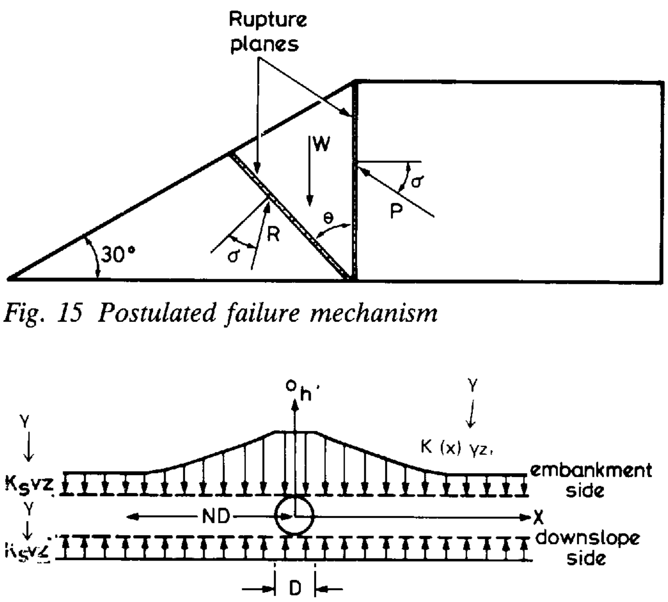
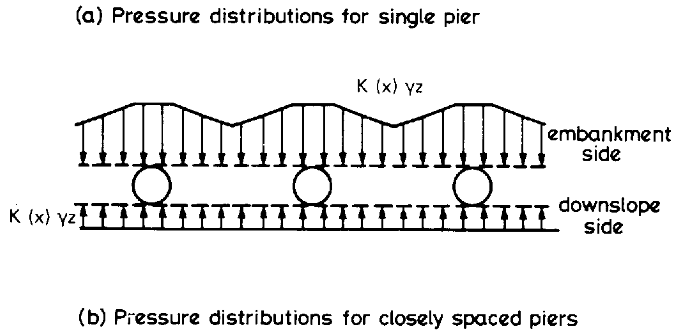


Fig. 15 Postulated failure mechanism



(a) Pressure distributions for single pier

(b) Pressure distributions for closely spaced piers

Fig. 16 Illustration of pressure distributions each side of piers in a spill-through abutment

above basis, therefore the earth pressure coefficient ($K(x)$) is assumed to vary with distance from the pier in the following manner:

$$K(x) = K_m \qquad o \leq x \leq d/2$$

$$K(x) = K_m - (K_m - K_s)\,[(x/d - \tfrac{1}{2})/3.5]^{2/3} \qquad d/2 \leq x \leq 4d$$

$$K(x) = K_s \qquad x \geq 4d$$

where d is the diameter (or width) of the piers and x is the distance from the centre of a pier. The horizontal stress distribution at depth z on the embankment side may therefore be obtained from the following expression:

$$\sigma_{h\,(\text{embankment})} = K(x)\gamma z$$

while on the down-slope side the horizontal stress is given by:

$$\sigma_{h\,(\text{down-slope})} = K_s\gamma z$$

The net force is the difference between the forces on the embankment and down-slope sides. Therefore, net force on a pier, W, is given by the following expression:

$$W = \int_o^{h} 2 \int_{d/2}^{s/2} [K(x) - K_s)\gamma z]\, dx.dz$$

leading to:

$$W = (K_m - K_s)\frac{d\gamma h^2}{2}\left[\frac{s}{d} - 0.164\left(\frac{s}{d} - 1\right)^{5/3}\right]$$

where s is the spacings between centres of the piers and h is the depth of fill at the piers. This expression applies for piers in the spacing to diameter range up to a value of 8. Above this value the expression to employ is:

$$W = 3.8(K_m - K_s)d\,\frac{\gamma h^2}{2}$$

The foregoing expressions may be used for determining the limiting force acting directly on the piers, but an alternative procedure is to recast the expression into a form equivalent to the original Chettoe & Adams method in which an effective width Ed is used.

This is achieved by dividing the expressions by that employed for conventional active conditions ($\tfrac{1}{2}K\gamma h^2$) to give:

$E = [s/d - 0.164\,(s/d - 1)^{5/3}](K_m - K_s)/K_a$.........for $s/d \leq 8$

$E = 3.8(K_m - K_s)/K_a$..................................for $s/d \geq 8$

It is apparent that, with greater deflection of the piers, larger internal friction angles will be mobilized in the fill behind the piers. Thus the force applied to the piers, and hence the effective width E, will vary with mobilized friction.

The stress conditions investigated in the centrifuge study varied between at-rest and active conditions. This behaviour can be expressed in terms of a variable coefficient, k, which ranges between unity and zero for at-rest and fully active conditions, respectively, as follows:

$$k = \frac{K_m - K_a}{K_o - K_a}$$

The variation of this coefficient with normalized pier head displacement for both dense and loose soils is shown in Fig. 17. The effective width of the piers related to the spacing to diameter ratio is presented in Fig. 18 in terms of the coefficient k.

The results from these most recent studies into spill-through abutment behaviour are clearly limited in scope in view of the fact that only one soil was investigated in association with two different pier geometries. Moreover, because the data were obtained from centrifuge model tests, the upper limiting earth pressure coefficient was the

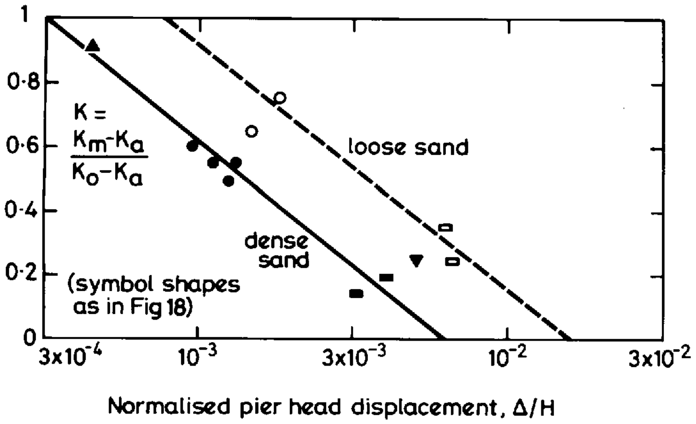


Fig. 17 Variation of k with pier movement

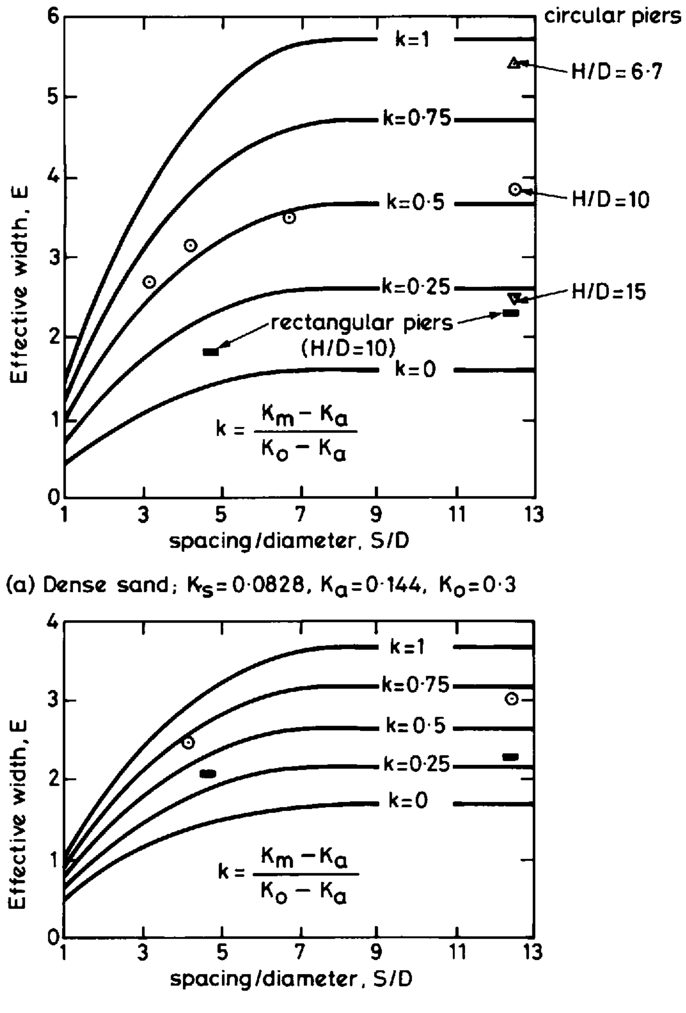


Fig. 18 Variation of effective width with pier spacing and maximum pressure, $K_m\nu z$, behind pier

K_o condition, whereas the influence of compaction could significantly increase the coefficient above this value. The results, however, provide a more rational basis for the ultimate design of spill-through abutments than has been previously available.

5.5.4 Spill-through abutment piers at the working condition

Although there is a lack of information on the behaviour of spill-through abutments at the working condition, a considerable amount of effort has been applied to the study of the associated problem of lateral loads on piles. The earliest and most commonly used approach for tackling this latter problem is based on the method of subgrade reaction originally proposed by Winkler . The method involves the use of a relation between pressure, p, and displacement ρ on the basis of a modulus of subgrade reaction, k_h, having units of force per cubic metre.

The method, which is widely employed in soil–structure interaction problems, is based on the following two assumptions in considerations of pile behaviour (Elson, 1984) or, as in this case, the behaviour of piers:

- the response of the soil both in front and behind a laterally loaded pile, or pier, may be simulated by closely spaced isolated springs at close intervals at those intervals along its length
- only the soil associated with the loaded area is subject to deformation, i.e. springs simulating the soil beyond the loaded area are not influenced by loads on adjacent springs.

The governing differential equation corresponds to that of a beam on elastic foundations, and in suitably modified form is as follows:

$$E_p I_p \frac{d^4\rho}{dx^4} + k_h d\rho = 0$$

Although a closed-form solution to this equation is possible for the more simple boundary conditions, for the typical situation of a cohesionless soil with elastic modulus increasing with depth, the solution is most conveniently achieved by a numerical approach (Poulos & Davis, 1980). Other more complex conditions have also been solved employing numerical techniques (Poulos & Davis, 1980).

Clearly a major difficulty with the method of subgrade reaction is the determination of the appropriate soil properties. The relation between pressure and displacement of the soil should be ideally determined for the anticipated conditions of overburden pressure, density, anistropy of the fill, principal stress direction and frictional conditions of the soil–pier interface. In practice, this approach would introduce insurmountable difficulties, and the moduli are most commonly determined on the basis of empirical correlations with other soil properties or on the basis of large-scale loading tests. Some consideration of the procedures for determining the modulus of subgrade reaction are presented by Elson (1984). In view of the difficulties of obtaining reliable data on soil properties and also because of the simplifying assumptions inherent in the analysis, the subgrade reaction method may be subject to inaccuracies.

An alternative procedure involves treating the soil as a continuum with elastic or elasto–plastic properties. The analysis is then carried out on the basis of a finite-difference, finite-element or boundary-element procedure after discretization of both the pile and soil and normally performing the calculations in terms of plane strain (Poulos & Davis, 1980). The elastic continuum method of evaluating the lateral forces on piles appears to offer a satisfactory means of assessing the behaviour of spill-through abutments. However, in this latter situation the influence of the base slab on which the piers are generally founded will also have to be considered in the analysis.

The application of the elastic continuum method to pile behaviour has been extensively studied and appears to be generally reliable, particularly in relation to single piles, provided that the properties of the soil are reliably determined. The closely associated problem of a pile embedded in soil undergoing lateral movement has been considered by Poulos (1973) in terms of the elastic continuum method. The analysis involves the treatment of the pile (or pier) as a thin vertical strip of width d, length L and constant flexibility E_pI_p. The pile is subdivided into elements and the solution obtained in numerical form on the basis of a finite-difference method employing the governing differential equation expressed numerically as follows (Poulos, 1973):

$$[D]\{\rho\} = \frac{dL^4}{E_pI_p}\{p\}$$

where $\{\rho\}$ = pile-displacement vector
$\{p\}$ = horizontal pressure vector
$[D]$ = matrix of finite-difference coefficients

In the method developed by Poulos, the relationship between load and displacement in each element is determined by integration of Mindlin's equations over the element, assuming that the soil is a homogenous elastic half-space. An inconsistency of the method is that no account is taken of friction or arching between the piles or piers. Although such effects may be small initially when the movements are small, as discussed above in relation to ultimate conditions, it would be expected that significant arching develops as the displacements increase. Such arching could be dealt with as previously by increasing the effective width d given in the above expression.

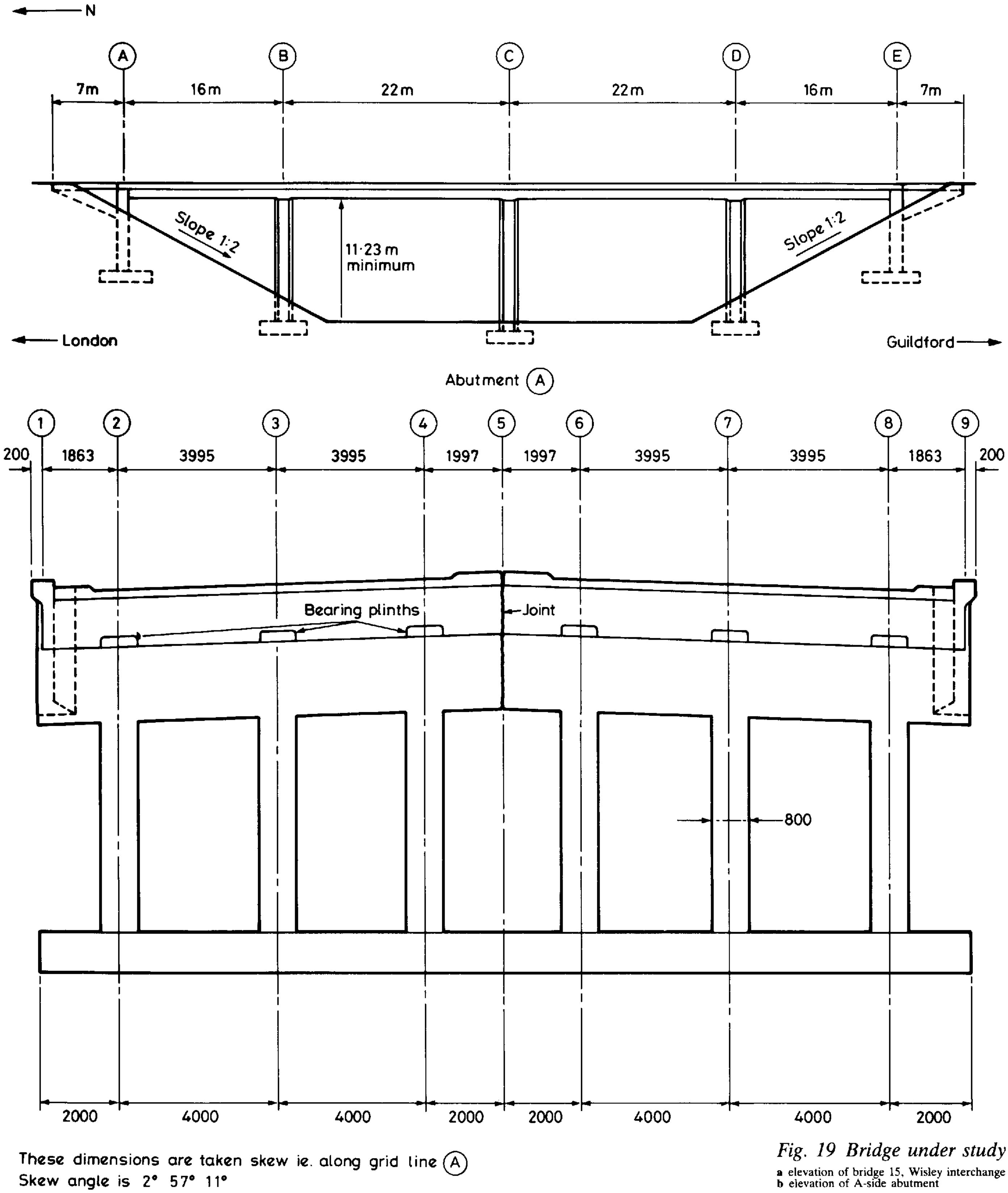


Fig. 19 Bridge under study
a elevation of bridge 15, Wisley interchange
b elevation of A-side abutment

5.5.5 A study of performance

During 1981 a bridge, consisting of a four-span continuous voided slab with spill-through abutments at either end, was constructed to carry the A3 trunk road over the M25 motorway and its associated slip roads. These bridge abutments were the subject of a full-scale investigation carried out jointly by TRRL and the University of Surrey (Lindsell, 1984).

The spill-through abutments consisted of 3m deep capping beams carried on six rectangular columns spaced at 4m intervals. The columns were 0.8m wide in the direction of the slope and 1.3m deep (see Fig. 19). The columns were founded on a base slab having a width of 5.5m.

Instrumentation for the scheme consisted of earth pressure cells in the fill and mounted flush with the columns and foundation slab. In addition a number of the columns had vibrating-wire strain gauges attached to the primary reinforcement. Temperature coils were included in selected gauges to monitor the changes occurring at the top and bottom of each column. The movements of the abutments were monitored, employing precise surveying techniques.

The investigation is still in progress, and only a small selection of the results obtained are presented herein relating to the strain gauge and pressure cell observations.

The compressive strains recorded on the embankment side of two columns (A3 and E3 in Fig. 19) are shown in Fig. 20 at various stages during and after construction. The results indicate that following the development of some initial strains, principally arising from shrinkage and creep of the concrete, very large strains were induced during the deck pour and subsequent setting of the concrete. These large strains were sustained for a few days but eventually reduced to about the same values as had existed prior to forming the deck. Thereafter the trend of the results indicated that subsequent backfilling and construction of the road pavement tended to reduce the compressive strains.

The influence of the deck construction on the lateral pressure acting on the columns is shown in Fig. 21. As shown in the Figure the pressure distributions were approximately symmetrical on the two sides of the column prior to forming the capping beam. Significant passive pressures were developed on the embankment side of the column after constructing the capping beams, with an associated reduction in lateral pressure on the down-slope side indicating some tendency for the construction to displace the column towards the embankment. Some further observations of lateral pressure acting on the columns are presented in Fig. 22 corresponding to different stages of

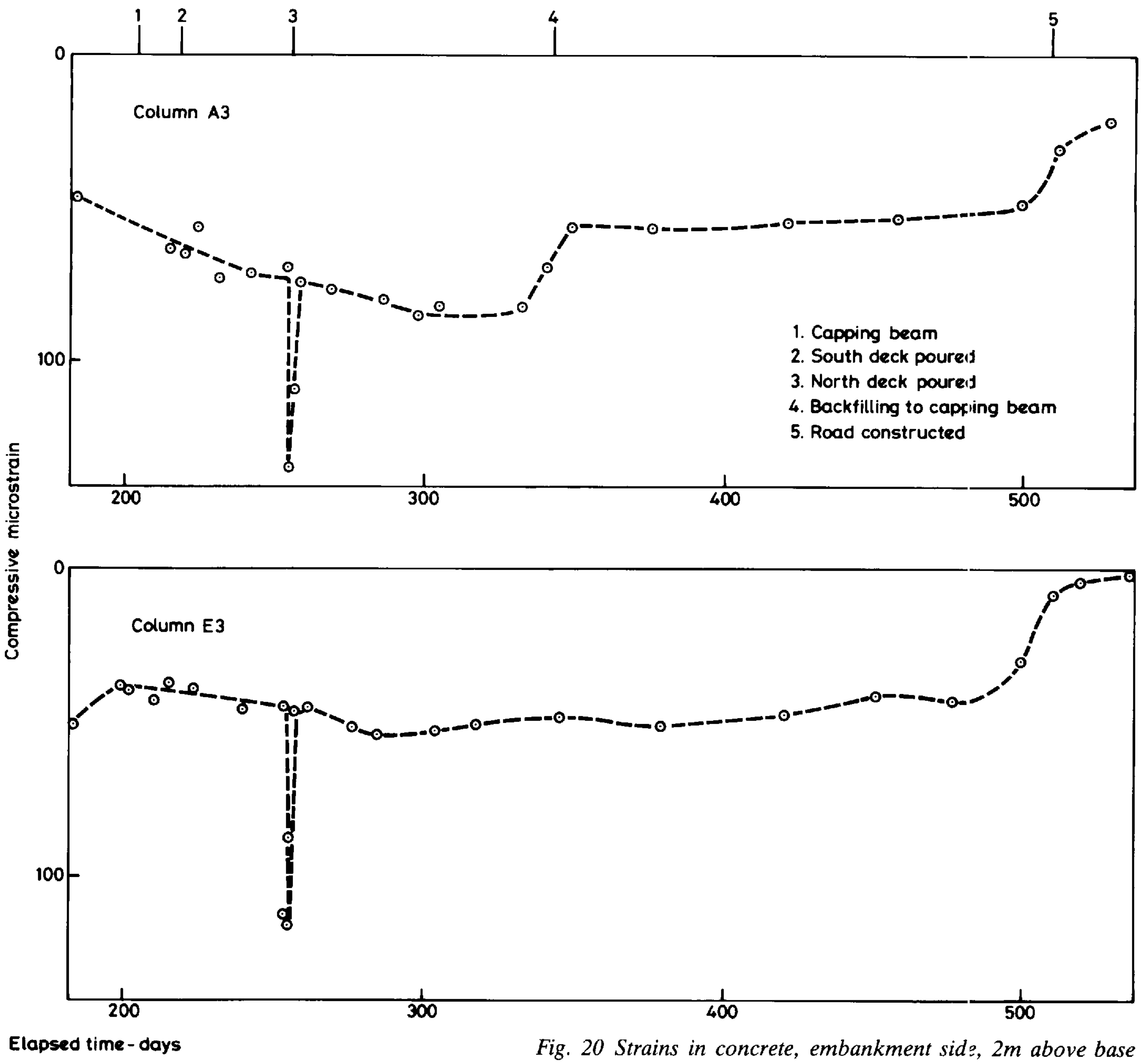


Fig. 20 Strains in concrete, embankment side, 2m above base

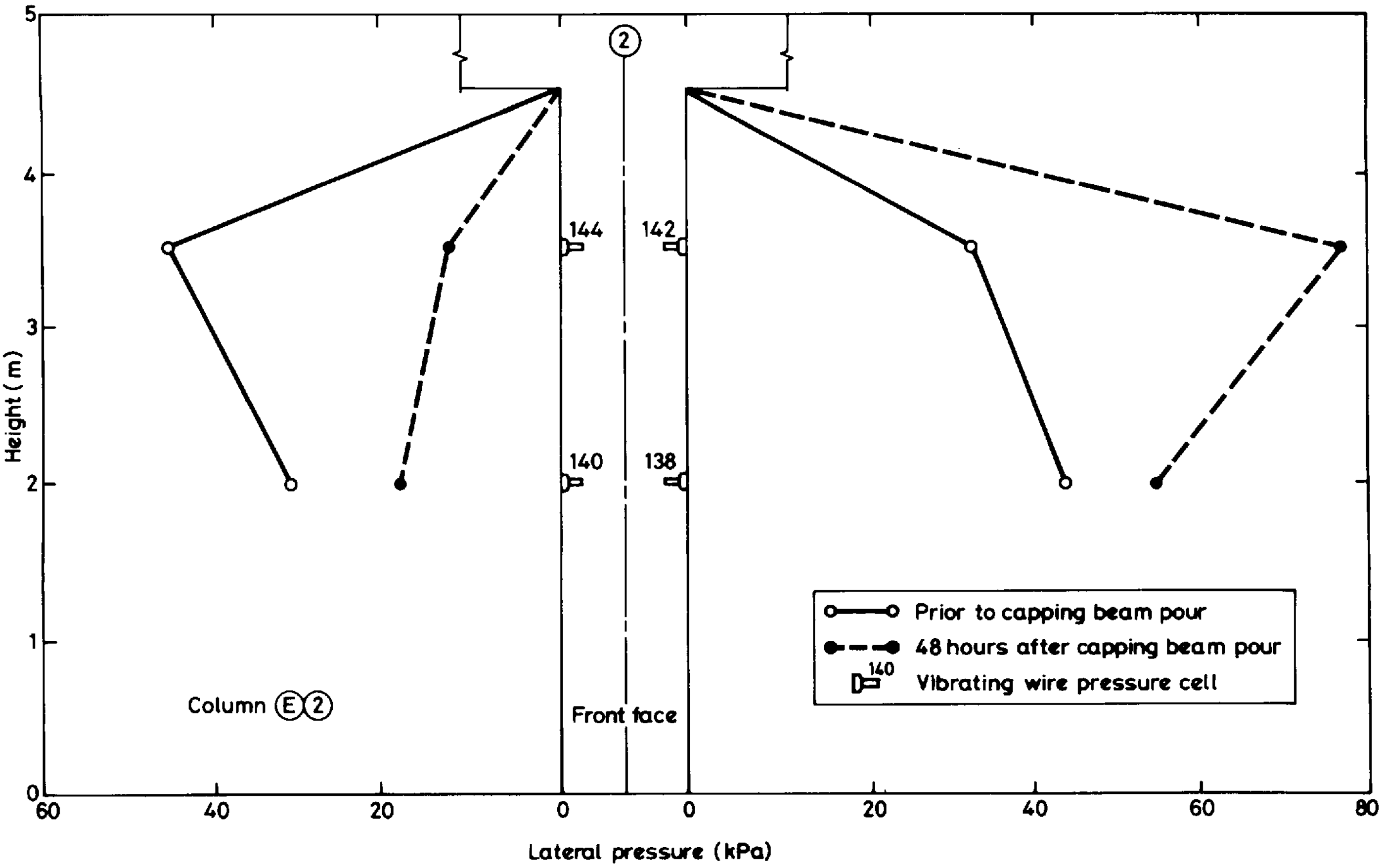


Fig. 21 Changes in side pressures on the E2 column

construction. The values corresponding to the end of construction have been used to calculate the effective width, E, as described for ultimate collapse analysis and assuming the K_0 condition. As can be seen from Fig. 23, the value is in reasonable agreement with the relevant curve.

In conclusion the study has shown that the most severe conditions encountered by the columns were associated with construction of the bridge deck. The lateral forces acting on the columns were much larger than had been considered for conventional design and are difficult to allow for properly in any case. However, it may be that careful attention to the details of the formwork will assist in reducing these lateral forces.

It is of interest to note that the pressures observed on the embankment side of the columns are in reasonable agreement with the relations between effective width and spacing/diameter ratio established in the centrifuge model study (Randolph *et al,* 1985) (see Fig. 23).

5.6 Construction

Construction methods and practices for bridges are different from those for buildings. Bridges are single plane; in comparison to a building this plane is high above ground level and access underneath may be difficult, but some forms of construction call for support from the ground. When a new piece of deck is concreted, the ground under the falsework settles and invokes a soil–structure interaction. The effect on the soil may be permanent, but more immediately the response of the soil to the temporary construction loads has to be assessed. Ground support near to a pier where consolidation of soil can have taken place from the compacting effect of construction traffic may be relatively 'hard', in between piers 'soft'. The sequence of the *in situ* concrete construction, its change from fluid to solid state, and the influence of any prestressing will change the loading impositions on the soil. The resulting movement may affect the stresses in the structure at a time when the structure has not developed adequate strength to deal with them.

The stiffness of the bridge will change during its construction, and the effect of the sequence of construction should be examined. This examination should embrace the whole time construction activity on site. Brown & Mead (1973) described the construction (with all its attendant considerations) of a new bridge on soil already subject to the nearby presence of an existing heavy bridge, which has been in position 100 years, but which is removed after completion of the new bridge. The foundation soil will have sustained several phases from virgin, responding to construction of original bridge, adjusting to presence of original bridge (long term), responding to construction of new bridge, adjusting to presence of two bridges (short term), responding to removal of old bridge, adjusting to presence of new bridge. Soil response on removal of a load may not be immediate.

It is seldom that a bridge is designed with account being taken of the possibility of another being built alongside. Smyth *et al* (1980) described a situation in which it was impossible to repeat the arch form of the existing bridge, as a new arch bridge constructed alongside would have caused damage to the existing arch through spreading of the abutments.

The majority of published papers on bridges describe the horizontal forces that can arise and how these are transmitted to the foundations, and also how the bridge is designed to cope with permanent horizontal forces coming onto the foundations. Papers on mining subsidence deal with the passage of slow-moving ground-originated waveforms. Gifford *et al* (1969) describes a bridge on a 3-point support and making provision for differential settlement of 0.45m and overall settlement of 1.2m. Knox *et al* (1984) describes the rare (for UK) case of provision being made for a ground shock wave (earthquake).

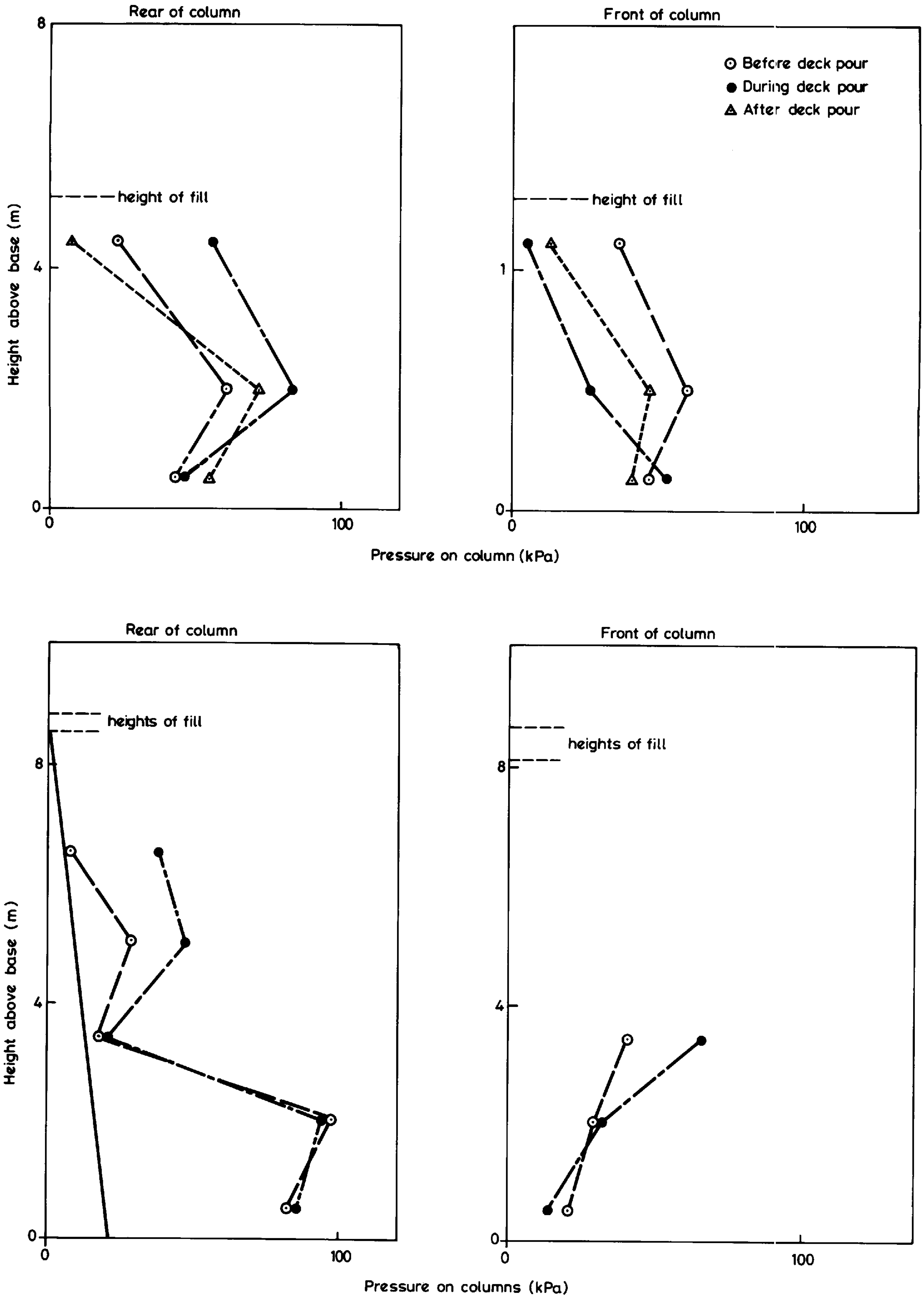


Fig. 22 Pressures on column A3 during and after construction

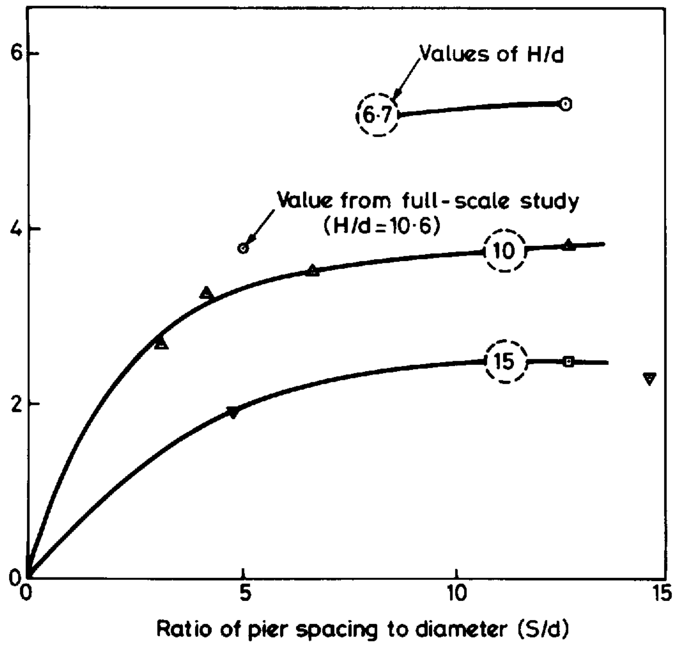


Fig. 23 Relations between E and s/d from centrifuge study with value obtained from full-scale study shown for comparison

References

Ah-Teck, C. Y. (1983): *The behaviour of spill-through abutments,* MPhil. thesis, Dept. of Engineering, University of Cambridge, Cambridge

Bastin, R. D. (1983): 'The Selby diversion of the east coast main line: part 3 bridges', *Jnl. ICE,* Nov., p. 733

Brown, J. G. (1937): 'Kincardine-on-Forth bridge', *Jnl. ICE,* April, no. 6, 1936-37, p. 687

Brown, C. D., & Mead, P. F. (1973): 'London bridge: planning, design demolition and construction', *Proc. ICE,* **54,** Feb., p. 25

Chettoe, C. S., & Adams, H. C. (1938): *Reinforced concrete bridge design,* 2nd ed, Chapman & Hall, p. 298; John Wiley & Sons, New York, p. 387

Clements, L. (1984): 'The Kessock Bridge design-and-build contract, and proposals for managing similar contracts', *Proc. ICE,* part 1, **76,** Feb, p. 23

Cole, K. W. (1980): 'The south abutment at Kessock Bridge, Scotland', IABSE, 11th Congress, Final Report, p. 1099

Cullen Wallace, A. A., & Nissen, J. (1984): 'Kessock Bridge: joint engineers' role', *Proc. ICE,* part 1, **76,** Feb, p. 67

Elson, W. K. (1984): *Design of laterally loaded piles,* CIRIA Report 103, London

Gifford, E. W. H., Woolley, M. V., & Butler, A. A. W. (1969): 'The design and construction of the Calder bridge on the M1 motorway', *Jnl. ICE,* Aug, p. 527

Hambly, E. C. (1979): *Foundations and substructure,* Building Research Establishment, HMSO, London

Hunt, K: Compensated foundation at Loversall Carr Bridge, Doncaster (unpublished)

Huntington, W. C. (1957): *Earth pressures and retaining walls,* John Wiley & Sons, London

Knox, H. S. G., Homberg, H., & Deason, P. M. (1984): 'Kessock Bridge: design by contractor', *Proc. ICE,* part 1, **76,** Feb, p. 35

Lindsell, P. (1984): *Monitoring spill-through abutments – report to the Transport & Road Research Laboratory,* Dept of Civil Engineering, University of Surrey, Guildford

McGibbon, J. I., & Booth, G. W. (1984): 'Kessock Bridge: construction', *Proc. ICE,* part 1, **76,** Feb, p. 51

Marche & Lacroix (1972): 'Stabilite des culees de ponts etablies sur des pieux traversand une couche molle', (Stability of bridge abutments founded on piles through a soft layer), *Canadian Geotech. Jnl.,* 9/1 Feb

Pike, C. W., & Saurin, B. F. (1952): 'Buoyant foundations in soft clay for oil-refinery structures at Grangemouth', *Proc. ICE,* part III, **1,** Dec., p. 301

Poulos, H. G. (1973): 'Analysis of piles in soil undergoing lateral movement, ASCE, *J. Soil Mech & Found. Div.,* **99,** SM5, p. 391

Poulos, H. G., & Davis, E. H. (1980): *Pile foundation and analysis design,* John Wiley & Sons, New York

Randolph, M. F., Ah-Teck, C. Y., & Murray, R. T. (1985): 'Centrifuge study of spill-through abutments', *Proc. 11th int. conf. SM&FE,* **2,** Session 2C, San Francisco, p. 1117

SM&FE (1977):

Smyth, W. J. R., Benaim, R., & Philpott, D. H. (1980): 'The new Runneymede bridge' *Struct. Eng,* **58A,** no. 1, Jan., p. 5

Wolde-Tinsae, Greimann & Johnson (1980): 'Performance of integral bridge abutments', *Proc. IABSE,* P 58/83

The effect of horizontal loads on piles due to surcharge or seismic effects, Proc. 9th Int. Conf. SM&FE, Tokyo July 1977 (particularly the papers by de Beer (Belgium); Hughes & Goldsmith (New Zealand); Ingold (UK); Marche & Schueeburger (Canada)).

6 Offshore structures

6.1 Introduction

Soil–structure interaction effects on three main structural types of offshore structure are discussed, namely piled structures, gravity-base structures and jack-up units.

The subsection on piled structures is the largest of the three, reflecting the fact that this is the most common type of structure used to support offshore operating facilities. Standard foundation design methods and analyses for single piles and pile groups are described, and the importance of correctly modelling foundation stiffness is outlined. Shortcomings associated with current design methods are identified. Included in this subsection is a discussion on the interaction of the temporary seafloor support on the overall structure.

Subsection 6.4 on gravity structures discusses loading regimes, environmental loading, and displacements associated with quasi-static and cyclic loading. The effects of cyclic loading on displacements and soil degradation are also covered.

Subsection 6.5 includes a brief review of the types of jack-up unit, their installation/operation on site and the moment-rotation behaviour of the 'spudcan' foundation under combined load.

6.2 Site investigation

Offshore site evaluations comprise two primary components: a geophysical survey and a geotechnical investigation. The special requirements of a marine site investigation (borehole frequency, sampling and testing methods, potential hazards such as pockmarks and shallow gas, etc) are detailed in St. John (1980), De Ruiter *et al* (1983) and SUT (1985).

6.3 Analysis of offshore pile foundations

6.3.1 Introduction

On the early offshore oil platforms in the shallow waters of the Gulf of Mexico the deck structure was supported at a safe height above the sea surface on tubular pipe piles. These piles were given guidance during installation, and subsequently derived lateral support from a 'jacket'. The jacket was placed on the seabed and the piles were driven through the hollow tubular legs. Prior to receiving the deck, the top of the piles were welded to the jacket.

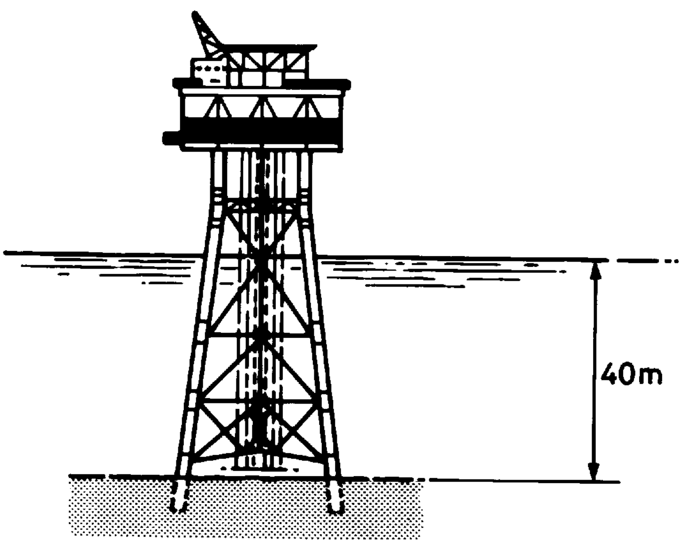


Fig. 24 Shallow-water platform with through the leg piles

It was jackets of this form that were installed in the southern North Sea to support platforms in the relatively shallow waters of the gas fields (see Fig. 24). As hydrocarbon exploration moved into deeper waters the Gulf of Mexico template-type jacket was extended with additional piles driven around the jacket perimeter to augment the capacity of the main leg piles. However, there comes a point, with greater water depth (giving more severe wave loading), where it is more efficient to arrange the pile in clusters around the main legs of the jacket, and to terminate them in sleeves at the jacket base rather than continue them to deck level (see Fig. 25). The main differences therefore between shallow-water and deep-water jackets are that the former have single piles that carry the topside weight directly, while the latter tend to have piles grouped around the main legs. For enhanced stability (offshore platforms are subjected to unusually high horizontal loads) it is normal for jackets to have larger plan dimensions at the bottom than the top. This has two benefits, first the larger base dimensions reduce the overturning moment-induced axial loads on the piles. Secondly, because they are battered, the horizontal component of the axial load in the piles carries a significant part of the horizontal wave loading.

There are two principal methods of installing pipe piles – by driving or by drilling and grouting. Driving is chosen wherever possible because of the speed, and therefore low cost of installation. However, in highly calcerous sands, because of the uncertain influence of cementation on driving and resultant low skin friction that may be mobilized, grouted piles are generally considered more efficient. Before the advent of the very large offshore pile driving hammers, drilling and grouting was used where hard pile driving was anticipated in order to give higher capacity piles.

In more recent years, with the development of reliable underwater hammers, the use of vertical piles has gained favour. The main advantage is one of costs arising from ease of installation. However, this advantage must be weighed against increased lateral pile deflections (arising from the pile groups now seeing the full horizontal load) and hence greater pile-head bending moments. When development moves into very deep waters of the North Sea (200m plus) where the structural design will be dominated by fatigue and where the natural period of the structure will be of prime importance, it may be that this softer lateral response will make vertical piles untenable.

Piles on the larger North Sea platforms are commonly 2.134m diameter with future possibilities of 2.438 and 2.591m piles as have been installed offshore Brazil and which are planned for deepwater jackets in the Mediterranean.

Because of the large forces that would be involved to perform them, the literature contains no full-scale test data for ultimate axial capacity, lateral capacity or load/deflection behaviour for such piles. Unlike land practice, where a test pile will be driven and its capacity verified, the design of offshore piles involves drivability studies to predict achievable penetrations for the available hammer sizes, followed by theoretical predictions of the ultimate axial capacity. Offshore pile design relies heavily therefore on extrapolation from small-scale model tests, as described in more detail below.

6.3.2 Approach to design

In general, the number of piles required on an offshore platform is governed by the vertical loading regime. Lateral considerations can become critical if soft surface soils prevail or if the lateral forces are particularly severe.

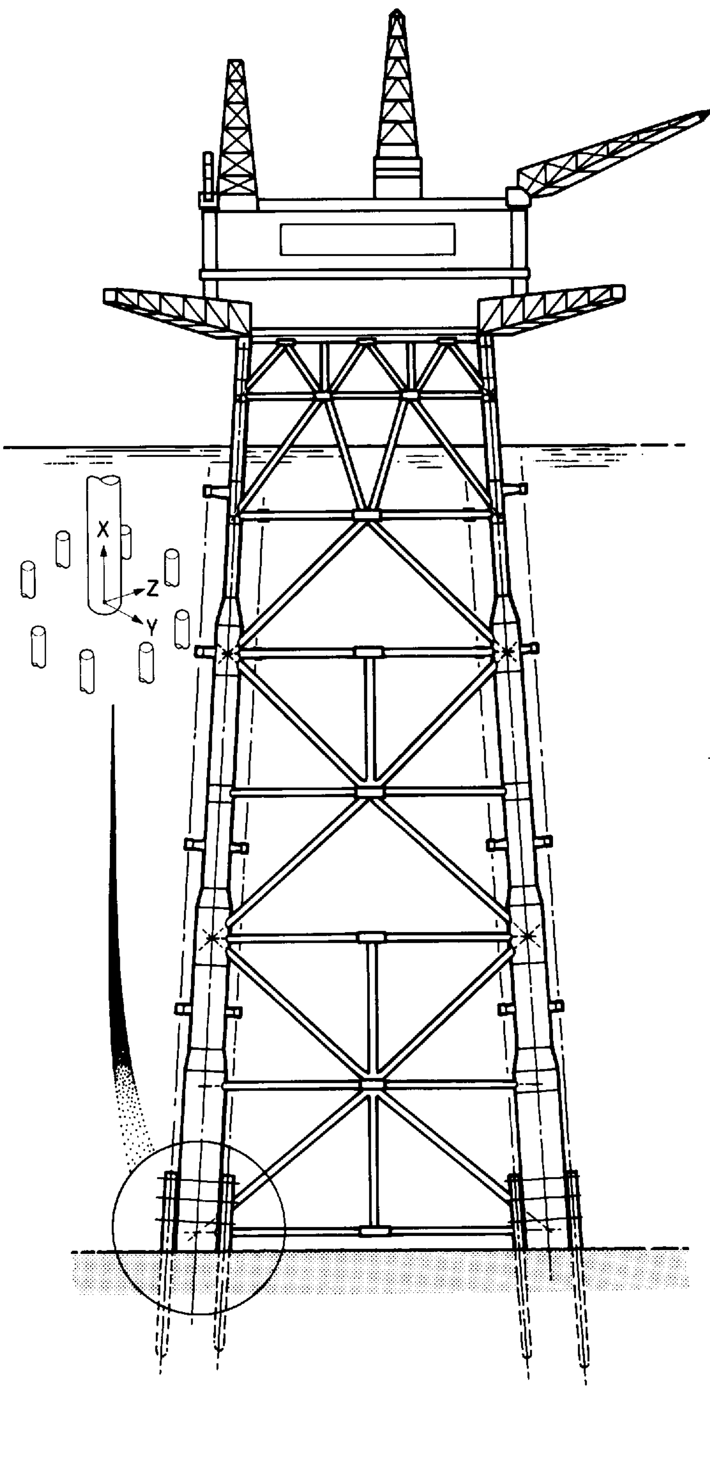


Fig. 25 Deepwater platform with a pile group at each corner

On a jacket with piles arranged in groups at the four corners, having determined the appropriate pile diameter, the depth to which the pile can be driven and its ultimate axial capacity at target penetration, it is relatively straightforward to compute the number of piles required for a given set of gravity loads and environmental forces. In so doing it is usual to size the pile group on average axial loads and to ignore axial variation between piles that can occur as a result of moments in the jacket leg. The axial loads on individual piles can be checked at a later stage following the computer structural analysis of the jacket and its foundations.

Where it is decided to arrange piles either singly or in small groups around the perimeter of the jacket (as may be done where gravity loads predominate over environmental loads) the system, i.e. the jacket and piles, becomes less determinate than a jacket with four discrete pile groups. As a result the forces on each pile/pile group become more heavily dependent on the stiffness of the pile groups relative to each other and relative to the jacket.

6.3.3 Prediction of foundation response

Axial-load deflection behaviour

Axial-load analyses are performed to evaluate pile-head deflections under working load level. This is required for evaluation of compatibility between the jacket structure and its foundations. Various schemes are available to compute load transfer from pile to soil; the soil may be idealized as a continuum, as in elastic analyses and finite- and boundary-element methods, or may be replaced by a set of non-linear independent springs supporting the pile.

For soil profiles that show only a gradual variation of stiffness with depth, elastic analysis has the advantage of offering solutions in closed form (Randolph & Wroth, 1978), or expressions in terms of influence factors available in chart form (Poulos & Davis, 1980). Allowance may be made for slip between pile and soil, adopting a simple elastic perfectly-plastic response, at the soil–pile interface. Elastic analysis is particularly appropriate when considering group effects, as discussed later.

For less uniform soil deposits, or where the soil response is markedly non-linear (including possible strain softening after peak shear stress), the flexibility offered by the discrete-spring approach becomes more attractive. This method, known as the load-transfer function method (also as the subgrade reaction method) was originated by Seed & Reese (1957) and extended by Coyle & Reese (1966). It is widely used because of its simplicity, and the coded version that utilizes a finite-difference scheme can be executed on most microcomputers. Input into the numerical solution consists of load-transfer curves, t–z for side friction, and q–z for end bearing and pile properties. The load-transfer curves describe mobilization of shaft resistance, t, and end bearing, q, as a function of pile movement, z. The ultimate values of t and q are those adopted in ultimate pile capacity computations. The deformation response of a pile will be governed by its compressibility and the coordinates and shape of the load-transfer curves.

Several procedures are available to construct t–z curves for clays (Coyle & Reese, 1966), for sand (Coyle & Sulaiman, 1967) and for both clays and sands (Vijayvergiya, 1977). The criteria proposed by Coyle & Reese (1966) and Coyle & Sulaiman (1967) are based on results from load tests on piles much smaller in size than those used to support offshore jackets. Experimental data (Aurora, 1981) and theoretical analyses (Poulos & Davis, 1968; Randolph & Wroth, 1978; and Kraft *et al*, 1981), show that the displacement at which ultimate skin friction is mobilized is generally greater than those recommended by Coyle & Reese (1966) and Coyle & Sulaiman (1967). While the ultimate skin friction can be fully mobilized at very small displacements, probably of the order of 0.5% of pile diameter, relatively large deflections are required to mobilize ultimate end-bearing resistance. The magnitude of these deflections is known to be related to the dimension and type of foundation (Skempton, 1951; Vesic, 1975; Vijayvergiya, 1977).

Current pile design utilizes static criteria; however cyclic loading and rate of loading effects have a significant influence on pile performance (Poulos, 1983; and Randolph, 1983). Available criteria to account for those effects are not well established and need refinement before they can be applied on a routine basis. The net effects of cyclic loading in degrading soil response (Karlsrud & Haugen, 1985) and of the relatively rapid loading rate in increasing soil stiffness (Kraft *et al,* 1981) offset each other to varying degrees.

Lateral load deflection behaviour

The behaviour of the soil surrounding a laterally loaded pile is normally described in terms of *p–y* curves, which relate the soil resistance, *p,* per unit pile length to the pile deflection, *y,* at various depths below the surface. In general, these curves are non-linear and depend on several parameters including depth, shear strength of the soil, and nature of loading. The soil response is characterized as a set of discrete mechanisms based on the subgrade reaction approach (Terzaghi, 1955; and McClelland & Focht, 1958).

Procedures currently used in developing *p–y* data for offshore piles are based on analyses of lateral-load test data of instrumented full-scale pipe piles in soft clay (Matlock, 1970), stiff clay (Reese *et al,* 1975) and sand (Reese *et al,* 1974). These procedures have been developed to include the effects of short-term static loading and of cyclic loading representative of the ocean environment, and are recommended by the American Petroleum Institute (1986). The criteria were developed from tests in relatively homogeneous soil conditions but are applied to a layered soil profile. The stiff clay criteria of Reese *et al* (1975) are generally regarded as unsuitable for most North Sea stiff clays because of differences in the data base. A *p–y* criterion applicable to clays regardless of their consistency was evolved by Sullivan *et al* (1980). The *p–y* criteria in clays were recently reviewed by Gazioglu & O'Neill (1984) who suggested, as an improvement, the 'integrated clay method', which accounts for relative pile–soil stiffness and scale effects. Evaluations of *p–y* criteria in sand are given by Murchison & O'Neill (1984).

Most offshore piles are long and flexible, and the active length of pile that resists lateral load is probably no greater than 5 to 10 diameters below the seafloor. To assess the lateral response, the pile is considered as a beam–column on non-linear supports as represented by the *p–y* springs, and a pile-head boundary condition is selected based on the restraint provided by the jacket. The solution is conveniently obtained using a finite-difference program known as BMCOL (Haliburton, 1971). In sandy seabed conditions with high current velocities, erosion of the sand, or scour, may occur around the pile head leading to an unsupported pile length near the seafloor. This effect, together with reduction in the stiffness of the *p–y* springs, has to be accounted for in beam–column analyses.

In addition to the lateral stiffness of the pile foundation, the bending-moment distribution down the pile needs to be calculated in order to assess the required pile section. Elementary mechanics may be used to compute the longitudinal stress in the pile wall under combined axial and lateral loading. The pile wall may be increased in thickness in the upper few diameters in order to carry the high bending moments.

Pile responses under axial and lateral loads are generally assumed uncoupled. However, large lateral displacements of the pile head, together with the applied axial load, will induce an additional moment resulting in an increase in the lateral displacements. This effect can be investigated using the BMCOL program. In addition, cyclic lateral loading can lead to the formation of a softened zone of soil (or even a gap) around the upper part of the pile, with a resulting loss in axial load-carrying capacity.

Behaviour of pile groups

The response of a pile group differs from that of a single isolated pile because of interaction of the zones of influence of each pile. The capacity of a group of piles may differ from the sum of the individual pile capacities, partly because of effects of installation (Rigden & Semple, 1983) and partly because of different modes of failure of the group. Installation effects are particularly difficult to assess and as a result are generally ignored in design. Several empirical equations have been proposed to relate group efficiency to pile spacing and number of piles in square or rectangular groups based on model tests. For typically circular offshore pile groups, the equivalent-pier concept proposed by Terzaghi & Peck (1967) is used in evaluating group efficiency. O'Neill (1983) presents a comprehensive review of group efficiency in sands and in clays.

In designing the piles, it is necessary to determine the load distribution within the group and the pile deformation response. The load deformation response of a pile group will be less stiff than the sum of the individual pile stiffnesses because of the interaction between piles (Cooke *et al,* 1980). Elastic methods are generally used to quantify interaction between piles in a group.

Procedures for estimating group response may be considered in two categories:

- consistent elastic methods (Banerjee & Davies 1977; Poulos, 1980; and Randolph, 1980) and
- hybrid methods, combining load-transfer analysis (subgrade-reaction methods) for the single-pile response together with elastic interaction (Focht & Koch, 1973; Clausen *et al,* 1981; and O'Neill & Ha, 1982).

Interaction between piles through the soil is mainly attributed to superposition of strains set up in the soil by pile loading. Installation effects are not explicitly considered. The degree of interaction between a pile pair is normally expressed as a factor or a coefficient. The interaction factors are computed based on elastic theory, and solutions have been developed for axially loaded piles in floating and end-bearing conditions, and for laterally loaded piles with pile heads fixed and free (Poulos & Davis, 1980). Closed-form expressions to evaluate interaction effects are given in Randolph & Wroth (1978) and Randolph (1981) for axial and lateral loading, respectively.

For laterally loaded pile groups, Focht & Koch (1973) proposed combining the subgrade reaction, *p–y* method and elastic half-space procedure of Poulos (1971) to predict group action considering both the plastic deformation of the soil near the pile and the elastic deformation in the overall soil mass. The analyses lead to a modification of *p–y* curves for an isolated pile to produce *p–y* curves representative of an equivalent single pile in a group.

Soil moduli are required in the methods above to perform interaction analyses. Guidance on selection of moduli for axial and lateral-load analyses is given in Banerjee & Davies (1978), Poulos (1980) and Randolph

(1980). Whether the soil modulus in the analysis is assumed to be constant or linearly increasing with depth could have a significant impact on the computed pile-head deformation and distribution of load among the piles in the groups. For sands and soft normally consolidated clays, a linearly increasing modulus for lateral-load analysis may provide a satisfactory representation of soil stiffness. Selection of soil modulus and its pattern of variation must also consider the type of the analytical method (O'Neill & Ha, 1982; and O'Neill 1983). For example, predictions of group settlement and pile loads in clays using the program DEFPIG (Poulos, 1980) and PILGPI (O'Neill *et al,* 1981) against field behaviour show the best-fit soil moduli in PILGPI to be about two to five times those of DEFPIG. Clausen *et al* (1981) used their computer program SPLICE to compute the lateral response of the pile group example analysed by Focht & Koch (1973). Using the same soil modulus, the predictions using SPLICE showed considerably less group action than the prediction by Focht & Koch.

Results from the methods described above show that the behaviour of a pile group is dependent on several factors including the number of piles, pile size, spacing and stiffness, soil stiffness, magnitude and direction of load, pile-head restraint and group configuration. Poulos (1971) has shown that the deflection of a pile group subjected to a lateral load is greater than that of an isolated pile carrying the average shear load of the group, i.e. the subgrade reaction experienced by an isolated pile is reduced by group effects. In addition, the maximum bending moment of a pile in the group will be greater than that for an isolated pile. Similarly, under axial loading, the settlement of a group-pile under an average load is greater than that of a single isolated pile.

Of particular interest to the designer is the bending-moment diagram and hence the bending stresses of a pile in the group. This may be derived from the *y*-modifier approach of Focht & Koch (1973) or the procedure by Bogard & Matlock (1983). Available algorithms (Poulos, 1980; and O'Neill *et al,* 1981) give the distribution of moments and loads in any pile in the group.

6.3.4 Cyclic loading

One of the major differences between offshore and onshore pile groups, apart from scale, lies in the pattern of loading applied to the piles. In the offshore situation, the ratio of lateral to axial load is generally high, and a much higher proportion of the load is live or cyclic in nature.

The flexibility of offshore piles has a significant bearing on both the monotonic response and the cyclic response. Under cyclic loading, highly stressed soil in the upper part of the pile may become degraded, throwing load further down the pile. Modelling of this interaction of soil and pile is the principal attraction of load-transfer analysis (Randolph, 1983). Load-transfer curves may be adapted simply to simulate the response of soil elements to monotonic and cyclic loading, e.g. as deducted from scale-model or laboratory tests. The effects of different assumptions regarding soil response may then be explored with minimal computational costs.

As an alternative to load-transfer analysis, Poulos (1983 & 1985) has discussed approaches whereby effects of cyclic loading can be incorporated into boundary-element analysis of single piles and pile groups. Empirically derived parameters are used to relate degradation of soil strength or stiffness to the level and number of load cycles. Hence the post-cyclic response of the pile group may be estimated. Poulos (1983) demonstrated that group effects lead to significant reduction in the level of cyclic load that can be sustained by the pile. However, results of cyclic lateral load tests on pile groups in soft clay, reported by Matlock *et al* (1980), show that cyclic degradation initiated at a threshold deflection related to the single-pile diameter and was unaffected by the group action.

6.3.5 Structural modelling

Having determined the pile group size and layout, the response of the foundations to forces in the various degrees of freedom is determined, as discussed above, for use in the stiffness analysis of the jacket. There are two basic methods by which the behaviour of the foundations may be incorporated in a stiffness analysis:

- piles modelled separately and individually connected to the jacket leg with constraint equations between adjacent piles to model interaction effects
- a lumped model (which incorporates interaction effects) to represent the combined behaviour of all the piles in a group.

Because foundations response is rarely linear, especially at high load levels, and because the stiffness analysis usually assumes a linear-elastic structure and foundation, it is necessary to input the foundation stiffness as the secant stiffness at the expected working load level. This approximation is usually adequate, but more sophisticated analyses may be performed using programs that accept non-linear foundations and which iterate, using substructuring techniques, until a solution is found (Clausen *et al,* 1981).

The most convenient way in which to include the lumped model is by way of a 6 × 6 stiffness matrix (see Fig. 26).

$$\underbrace{\begin{bmatrix} Fx \\ Fy \\ Fz \\ Mx \\ My \\ Mz \end{bmatrix}}_{\text{Forces at jacket foundation interface}} = \underbrace{\begin{bmatrix} K_{11} & 0 & 0 & 0 & 0 & 0 \\ 0 & K_{22} & 0 & 0 & 0 & K_{26} \\ 0 & 0 & K_{33} & 0 & K_{35} & 0 \\ 0 & 0 & 0 & K_{44} & 0 & 0 \\ 0 & 0 & K_{53} & 0 & K_{55} & 0 \\ 0 & K_{62} & 0 & 0 & 0 & K_{66} \end{bmatrix}}_{\text{Foundation stiffness}} \underbrace{\begin{bmatrix} \delta x \\ \delta y \\ \delta z \\ \theta x \\ \theta y \\ \theta z \end{bmatrix}}_{\text{Displacement of rigid pile cap}}$$

Fig. 26 Foundation matrix at centroid of pile group

Kriger (1980) provides a set of guidelines for constructing a suitable stiffness for use in structural analysis. The stiffness coefficients are developed with reference to a predefined sign convention and relate the six degrees of freedom (three orthogonal displacements and three rotations) to the six loads and moments applied to the pile cap. Coupling between lateral deflections and rotations is assumed (Kriger, 1980; and Randolph & Poulos, 1982) and can be evaluated from a subgrade-reaction approach or elastic methods by applying various boundary conditions to the pile head. Other coupling terms (such as those relating lateral and torsional effects, axial deflections and rotations, and axial and lateral deflections) can be evaluated but with a lesser degree of accuracy, and are usually of no great significance under working load levels. Nonsymmetric pile groups have very complicated coupling, which can be simplified in modelling by treating the pile group through its centroid (Kriger, 1980) and including any offset in the structural model.

On completion of the structural analysis a compatible set of forces and deflections will be output at the pile–structure interface. In an analysis in which the piles are modelled individually these forces (and bending moments) will be sufficient to design the pile structurally. In the analysis that has a lumped foundation model it is necessary to back analyse the results to obtain the pile-head forces on each pile.

One advantage of the substructuring technique available in most structural analysis programs (Bathe & Wilson, 1976) is that it allows an economic investigation of the structural effects of varying soil stiffness. This is of particular use where data are sparse and soil stiffness cannot be estimated with any confidence, or at highly non-uniform sites where it is possible that the soil properties could vary significantly between piles/pile groups. A further use is where pile groups of differing sizes are proposed and where uncertainty over the relative stiffness between the groups becomes a potential problem. Thus substructuring allows a variety of foundation stiffnesses to be examined to determine the most onerous condition for the foundations and the most onerous condition for the superstructure.

6.3.6 Temporary seafloor support for fixed offshore platforms

Objectives

Temporary seafloor support for a jacket structure prior to piling may be provided by mudmats, the lowest level of horizontal bracing members, or a combination of both. Which combination of these is adopted will depend on the nature of the surface soils at a particular site and on structural considerations. For example, at a site with weak soils both options may be used, while at a site with firm surface deposits a simple mudmat incorporated within the pile-sleeve cluster may be utilized. In the latter case, the length of the sleeves extending beneath the mudmat may be minimal, and the support derived from them may then be negligible. When sleeve or leg extensions are used it is necessary that they fully penetrate the surface soils.

Whatever the structural arrangement adopted for temporary support it must perform the following functions:

- limit penetration of the structure
- prevent sliding
- limit differential settlement to ensure verticality.

Design

For jacket structures the mudmats can be considered as rigid footings and can be designed using bearing-capacity procedures. The loads acting on the footing that derive from gravity and environmental loadings will have horizontal and vertical components. The horizontal components of force induce shear stresses at the interface between the mudmat and the soil, and this affects the bearing capacity (Hansen, 1970; Kezdi, 1961; Meyerhof, 1963; and Vesic, 1975).

The stability of the structure will depend on the shallow sea-bottom soil conditions beneath the mudmats. Therefore the properties of the soil to a depth that depends on the dimensions of the mudmat should be known. If the footing is circular, the depth should be equal to the diameter of the footing.

If the surface clay is significantly weaker than the underlaying material there is a possibility that a significant part of it may be squeezed out laterally during placing of the structure. For soft layers that are relatively thin by comparison with the width of the mudmat the results of Meyerhof & Chaplin (1953) may be used to compute the bearing capacity.

For clays that show a distinct linear increase in undrained shear strength with depth the method of Davis & Booker (1973) can be used to compute the ultimate bearing pressure. For layered soils a procedure devised by Brown & Meyerhof (1969) can be used.

For granular soils the bearing capacity factors recommended are those given by Vesic (1975). The possibility of the mudmat punching through granular soils into underlying soft clays should be investigated.

An adequate factor of safety against bearing capacity failure is required for the worst combination of vertical and lateral forces (normally 1 year summer storm conditions). A factor of safety of 1.5 is usually applied.

Lateral resistance has to be investigated to ensure an adequate factor of safety against sliding. This has to take account of skin friction and adhesion on the base of the mudmats. If the mudmats have skirts an allowance must be made for these. Additional resistance will arise because of resistance from sleeve or leg extensions. This is complex and may arise either from passive resistance (which can be calculated using Rankine earth pressure theory) or from a flow-induced phenomenon. Where the level of horizontal bracings come into contact with the seafloor soils the resistance of these to sliding has to be included.

The settlement of a mudmat can be divided into two components. The first is immediate settlement arising from elastic deformation and plastic yielding and may be calculated using D'Appolonia *et al* (1971), while the second is time-dependent because of consolidation and creep. The second component need be checked only when delays are expected between touchdown and grouting of the piles. Settlements on cohesive soils may be significant, while those on cohesionless soils are generally small.

In-service interaction of the temporary support system

Once the piles have been grouted the temporary seafloor support system has served its purpose. Where the temporary seafloor support system comprises a mudmat, its presence may have an important influence on the behaviour of the structure in normal working load conditions. In particular, it may have a beneficial effect on the natural frequencies of the structure, and this will in turn improve the fatigue life. In assessing the performance of a structure in service conditions these effects are generally conservatively ignored in design as it cannot be ensured that the surface soils will not be eroded from beneath the mudmat. As a result, the actual elastic behaviour of the foundations will be more stiff than is theoretically predicted. Measurements of the natural frequencies of structures in the Forties field and elsewhere tend to confirm this, although not conclusively as the natural period of the structure is also a function of mass. It has been found that assumptions about the added mass of the structure have more significant effects than foundation stiffness.

The reason for ignoring the temporary support system is based on the assumption that contributions to the stiffness from this source may not be present for the whole life of the structure. Apart from concern about possible undermining of the mudmat, the argument to support this is that when the structure is initially placed the load is carried entirely by the mudmats. Following insertion and grouting of the piles any subsequent load is imagined to be carried by the piles. If the structure is then subjected to a severe storm a redistribution of load from the mudmats to the piles takes place.

Generally with tower structures, the lowest natural frequencies are associated with tower sway. Therefore it is the lateral response of the soil that is the most significant in fatigue studies. If the shear stiffness of the temporary support system contributes to this it should be included. Little is known of this form of soil–structure interaction, and research is needed in order that it may be taken account of realistically.

6.4 Gravity base response in service conditions

6.4.1 Introduction

Until now, gravity platforms have mainly been founded at the surface of 'strong sites', i.e. on dense sands or heavily overconsolidated clays. Design studies are however well advanced for structures founded on normally consolidated clays in the North Sea. In that case, long 'skirts' are envisaged that make the foundation at least partially dependent on lateral resistance at depth for its support. This report is mainly restricted to surface-founded structures. Their number is not large: only 15 exist at present in the North Sea.

6.4.2 Loading regimes

Loads applied to the base are:

- *Vertical gravity load from the weight of the structure.* This is controllable by ballasting; and designers have chosen to operate in diverse ways. For example, for structures of weight W and base are A founded on clays with an undrained strength C_u, some designers have worked in the region W/AC_u of 1.5 and others in the region W/AC_u of 3. (The ultimate value is 6 for vertical loading.)
- *Vertical alternating wave load.* This is a relatively small component that may be out of phase with the horizontal load.
- *Horizontal alternating wave load.* This is a large force, ultimately typically one-third or one-quarter of the structure's weight. The point of action of this force is some way up the base resulting in a fairly large moment. It is paramount in gravity-base design for three reasons:
  - it cannot be altered very significantly by 'designing out'
  - because of its alternating character it causes fatigue in the foundation
  - depending on its frequency components it could cause dynamic amplification.

  On the positive side, the wave load is believed to be accurately calculable by simple wave theory (which is not true for piled or semi-buoyant structures).
- *Predominantly-horizontal alternating-earthquake load.* In certain regions these forces would dominate.

6.4.3 Waves and earthquakes

Earthquake engineering is well developed, and its philosophies are widely propounded. However, any attempt to equate these two cyclic processes is misconceived. An earthquake originates deep within the ground and excites the whole mass of soil in the vicinity of a (gravity) structure. Cyclic shear strains are reasonably uniformly distributed throughout the ground. Wave loading imparts *localized* shear to the foundation, whereas earthquakes impart *generalized* shear.

6.4.4 Quasi-static loads and displacements

For desk-study purposes, elastic-layer or half-space analyses are adequate using assumed soil stiffness (e.g. Poulos & Davies, 1974). Both vertical and lateral modes of deformation can be analysed. For analysis of real structures, computing costs are negligible relative to the capital investment, and numerical studies (finite element, finite difference, boundary element, etc) are justified, taking into account the true structural geometry and soil layering. Undrained, drained and partially drained analyses are feasible today, the degree of complexity of the soil model depending only on the amount and quality of site-investigation data. Axisymmetric or plane-strain idealizations are usual for preliminary studies, with a few 3-dimensional checks. Rate effects are generally discounted.

6.4.5 Displacements arising from cyclic loading

For real structures (i.e. beyond desk-study stage) the minimum complexity of soil model that is acceptable involves elasto–plasticity, however simple. When the cyclic-loading behaviour of a structure on such a soil foundation is computed, the main features that will emerge are either essentially elastic (recoverable) response or a progressive settlement of the structure into the seabed (shakedown). This arises from plastic redistribution of shear stresses beneath the base (Smith & Molenkamp, 1980). The amount of this permanent vertical displacement can govern the acceptability of the design. Remarks on acceptability of displacements, both cyclic horizontal and permanent vertical, are given by Anderson *et al* (1982).

6.4.6 Cyclic degradation arising from porewater pressures

The above cyclic phenomena would occur in any plastifying material. Analysis of soil is further complicated by the potentially damaging effects of generation of excess porewater pressure arising from cyclic phenomena. These phenomena are virtually incalculable because

- computer times are excessive for the tens of thousands of load cycles involved (contrast earthquakes where the load cycles are in the order of hundreds) and
- stress and pore pressure equalization conditions in test apparatus are suspect.

Computations can be made, but the conservative approach is to subject specimens of the foundation soil to cyclic loading tests (undrained) and to use the resulting moduli in quasi-static analyses. Because of sharp strain gradients beneath platforms, pore-pressure dissipation rates are almost certainly higher than commonly assumed at present. A further alternative for real structures is to conduct model tests using a large centrifuge.

6.4.7 Dynamic amplification of displacements

Natural periods of platform–foundation systems will not coincide with major wave energies by design; however, in deeper water locations, dynamic amplification will become more important. Nevertheless, smaller waves can still have a disproportionate effect, particularly those having periods one-third, one-fifth, one-seventh, etc. of the system's fundamental. The damage potential of such waves should be accentuated in any inference from laboratory tests. For earthquakes, the state of the art is to use quasi-linear frequency domain analyses similar to those in long use in the nuclear power industry. For the purpose of desk studies, frequency-dependent dynamic stiffness for half-spaces and layers are readily available.

6.5 Jack-up units

Before operating a jack-up platform (see Fig. 27) at a specific location, the ability of the unit to withstand the envisaged 50 year storm conditions is usually assessed to ensure that structural failure resulting in either damage or catastrophic accident does not occur (Ridehalgh & Edwards, 1982). The structural strength of the unit is checked, and the performance of the foundations is evaluated. During the structural analysis it is apparent that in certain cases excessive bending moments can occur in the legs at the position of the lower guide.

When arriving at this conclusion it is traditionally assumed that the foundations behave as a pin joint and cannot therefore sustain any bending moment. Intuition and practical experience suggest that this may be a conservative approach, and some sectors of the offshore industry take a view that consideration should be given to the rotational restraint, or foundation fixity, mobilized below the footing (see Fig. 28). The significance of allowing for foundation fixity is realized when assessing the structural strength of the legs of the unit, as inclusion of rotational

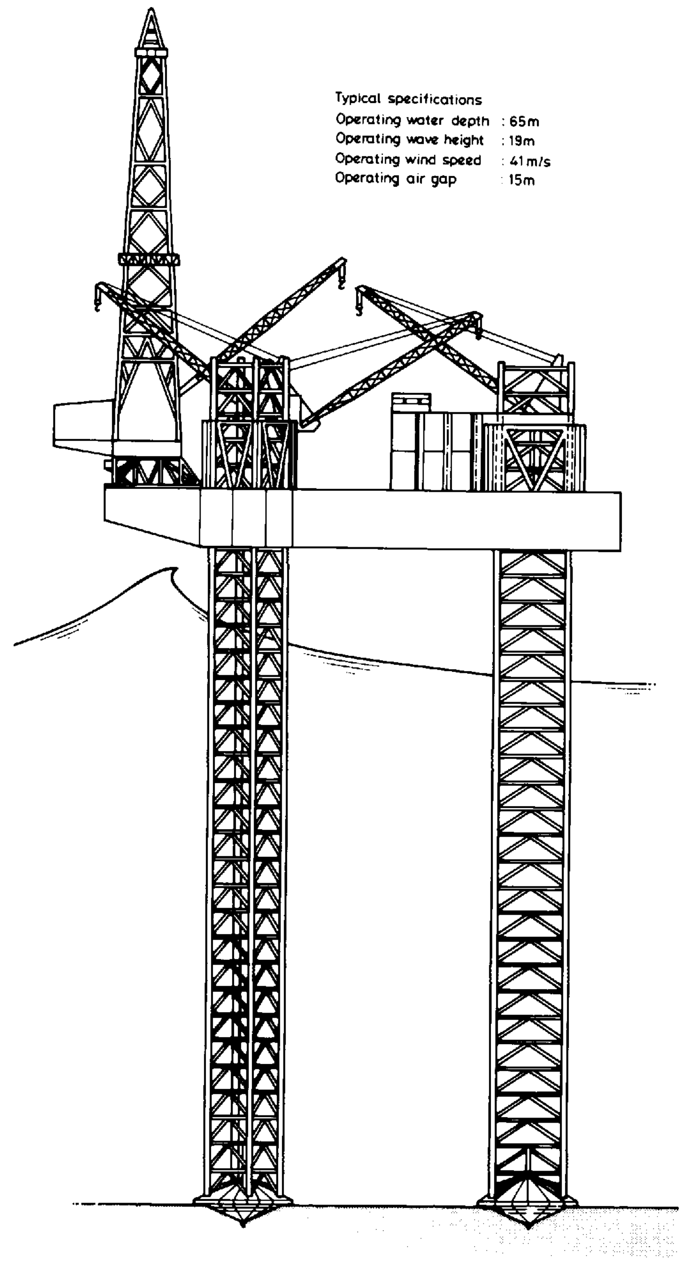


Fig. 27 The self-elevating jack-up platform

restraint results in a less onerous distribution of leg bending moments. This may lead to the view that a given unit may be suitable for operations in considerably harsher environments than if fixity is discounted.

A review of the published literature on the subject of rotational fixity of foundations indicates that essentially researchers to date have considered only the elastic displacement of a rigid circular footing on an isotropic homogeneous elastic soil. The solution for this situation is:

$$\theta = \frac{3(1 - v)\,M}{8Gr}$$

where θ = rotation

v = Poisson's ratio of the soil
G = elastic shear modulus of the soil
r = radius of the base
M = overturning moment.

This solution is cited in API RP 2A (1986) but is derived from the work of Borowicka (1936) and Gerrard & Harrison (1970).

In assessing the magnitude of foundation fixity that may be mobilized at a jack-up unit footing, it is essential to address the following parameters:

- foundation shape
- soil state below the footing
- transient storm loading
- lateral loading
- cyclic loading.

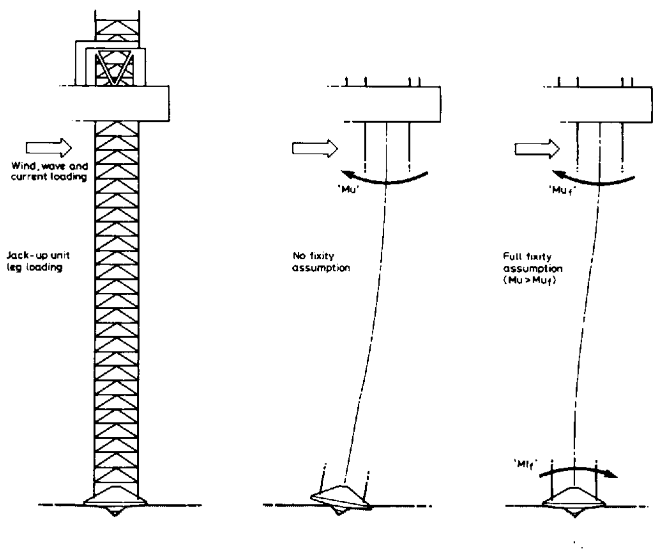


Fig. 28 The concept of foundation fixity

In particular, misleading results will be obtained unless the following points are noted:

- in stiff or dense soil, small penetrations of the spudcan will occur. For spudcans having conical bearing areas small penetrations may result in a reduced effective diameter. This must be accommodated in analyses so as to model the resulting reduction in fixity
- in preloading conditions, footings will penetrate until the bearing capacity of the soil below the foundation equals the applied vertical load. Accordingly, some assumptions may be made on the soil parameters, such as shear strength and shear modulus, that give rise to the equilibrium between bearing capacity and vertical load. The resulting values are considered the most appropriate for inclusion in the analyses
- changes in boundary conditions during operation of the unit should be considered. The occurrence of scour around a footing will reduce its effective diameter, and a reduction in fixity will arise.

A significant point regarding elastic solutions is that they involve stress concentrations at the edges of the footings, and these concentrations cause the soil to deform plastically. The elastic solutions are thus never strictly applicable to real soil, but for relatively small loads are useful for practical purposes since the region of plastic deformation will be small. As the loads increase, however, the plastic region becomes larger and the elastic solutions become less relevant. Eventually, a sufficiently large plastic region forms for failure to occur.

In the case of jack-up foundations it can be shown that during the design storm, the footing may be close to, or at, plastic failure (Hansen, 1970; Meyerhof, 1953; and Vesic, 1975). Accordingly, the use of elastic solutions to the problem of rotational fixity of jack-up units during the design storm is incorrect. At present it is reasonable to suppose that a typical spudcan, when acted on by a gradually increasing moment, will behave as follows:

- at low moment levels the moment rotation behaviour is likely to be nearly linear with no significant zones of plasticity in the soil
- as the moment increases, zones of plasticity will extend leading to a non-linear moment rotation response
- at the limit, continued rotation of the spudcan will occur without any further increase in the moment, possibly followed by a decrease. The value of the limit is likely to be governed by the vertical and horizontal load components.

The behaviour described is shown diagrammatically in Fig. 29 as a 3-stage profile. The exact form of the transition from elastic to plastic behaviour is not known; neither is the moment rotation response after plastic failure of the soil.

In an attempt to determine more accurately these unknowns, research is being conducted (Knott, 1985) using model and centrifugal tests and finite-element techniques. However, until the results of this research are available, industry will be obliged to adopt the simplified approach as described above, with a result that the true factor of safety against foundation failure in association with structural failure cannot be assessed.

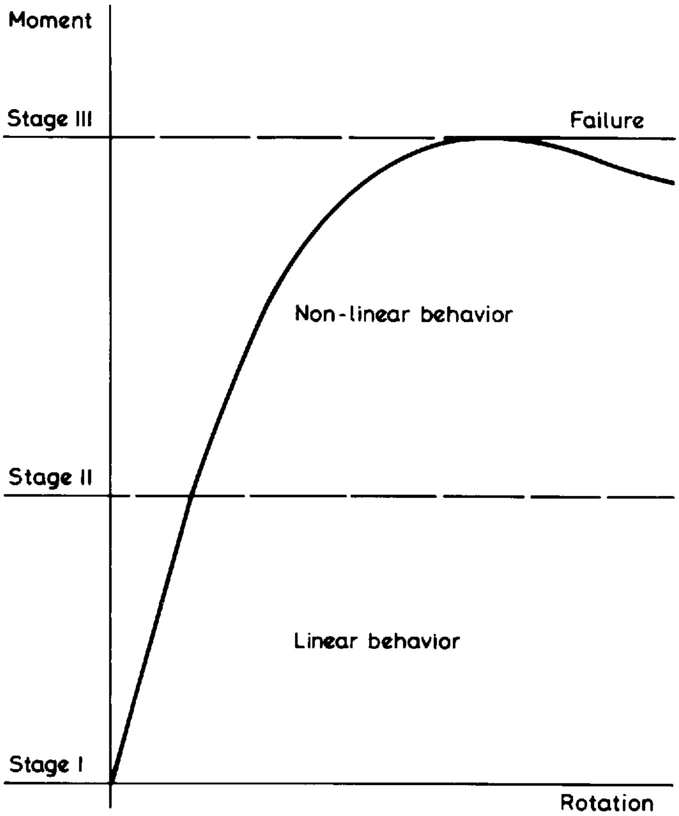


Fig. 29 Possible 3-stage profile of spudcan rotational behaviour

References

American Petroleum Institute (1986): *Recommended practice for planning, designing and constructing offshore platforms,* API RP 2A, 15th ed.

Andersen, K. H., Lacasse, S., Aas, P. M., & Andenas, E. (1982): 'Review of foundation design principles for offshore gravity platforms', *Proc. 3rd int. conf. on behaviour of offshore structures,* Massachusetts Institute of Technology, Massachusetts, **1**, p. 243

Aurora, R. P. (1981): Model studies of long piles in clay: *Proc. 13th offshore technology conf.,* Houston, **4**, p. 331

Banerjee, P. K., & Davies, T. G. (1978): 'The behaviour of axially and laterally loaded piles embedded in nonhomogenous soils', *Geotechnique,* **28**, no. 3, p. 309

Bathe & Wilson (1976): *Numerical methods in finite element analysis,* Prentice-Hall, p. 263

Bogard, D., & Matlock, H. (1983): 'Procedures for analysis of laterally loaded pile groups in soft clay', *Proc. symposium on geotechnical practice in offshore engineering,* ASCE, S. G. Wright (ed), p. 499

Borowicka, H. (1936): 'Influence on rigidity of a circular foundation slab on the distribution of pressures over the contact surface', *Proc. 1st int. conf. SM&FE,* **2**, p. 144

Brown, J. D., & Meyerhof, G. G. (1969): 'Experimental study of bearing capacity in layered clays', *Proc. 7th int. conf. SM&FE* Mexico, **2**, p. 45

Clausen, C. J. F., Aas, P. M., & Hasle, E. (1981): 'SPLICE – a computer program for analysing structure-pile-soil interaction problem', *Proc int. symposium on offshore engineering,* Brasil Offshore 1981, Pentech Press

Cooke, R. W., Price, G., & Tarr, K. (1980): 'Jacked piles in London clay: interaction and group behaviour under working conditions', *Geotechnique,* **30**, no. 2, p. 97

Coyle, H. M., & Reese, L. C. (1966): 'Skin friction for steel piles in sand', *Jnl. Soil Mech. & Found. Div. ASCE,* **92**, no. SM2, p. 1

Coyle, H. M., & Sulaiman, I. H. (1967): 'Skin friction for steel piles in sand', *Jnl Soil Mech. & Found. Div. ASCE,* **93**, no. SM6, p 261

D'Appolonia, D. J., Poulos, H. G., & Ladd, C. C. (1971): 'Initial settlement of structures on clay', *Jnl. Soil Mech. & Found. Div. ASCE,* **93**, SM 10, p. 1357

Davis, E. H., & Booker, J. R. (1973): 'The effect of increasing strength with depth in bearing capacity', *Geotechnique,* **23**, no. 4, p. 551

De Ruiter, J., & Richards, A. F. (1983): 'Marine geotechnical investigations – a mature technology', *Proc. conf. on geotechnical practice in offshore engineering,* ASCE, Austin, Texas, p. 1

Focht, J. A., Jr., & Koch, K. J. (1973): 'Rational analysis of the lateral performance of offshore groups', *Proc. 5th offshire technology conference,* Houston, **2**, p. 702

Gasioglu, S. M., & O'Neill, M. W. (1984): 'Evaluation of *P–Y* relationships in cohesive soil', *Proc. symp. on analysis and design of pile foundations,* ASCE, p. 192

Gerrard, C. M., & Harrison, W. J. (1970): 'Circular loads applied to a cross-anisotropic half space, *CSJRO, Aust. Div. App. Geomech.,* Tech paper no. 8

Haliburton, T. A. (1971): *Soil structure interaction: numerical analysis of beams and beam columns.* School of Civil Engineering, Oklahoma State University, Stillwater, Oklahoma, Tech. pub. no. 14

Hansen, J. B. (1970): *A revised and extended formula for bearing capacity,* DGI Bulletin no. 28

Karlsrud, K., & Haugen, T. (1985): 'Behaviour of piles in clay under cyclic axial loading – results of field model tests', *Behaviour of offshore structures (BOSS '85),* Delft, Netherlands, p. 589

Kezdi, A. (1961): 'The effect of inclined loads on the stability of a foundation', *Proc. 5th int, conf. SM&FE,* Paris, **1**, p. 669

Knott, T. (1985): 'Down amongst the spud cans something is stirring', *Offshore engineer,* Oct. 1985, p. 24

Kraft, L. M., Jr., Cox, W. R., & Verner, E. A. (1981): 'Pile load tests: cyclic loads and varying load rates', *Jnl. Geotech. Eng. Div. ASCE,* **107**, no. GT, p. 1

Kriger, G. A. (1980): 'Modelling of piled foundation', *Proc. 12th offshore technology conf.,* **II**, p. 117

Matlock, H. (1970): 'Correlations for design of laterally loaded piles in soft clay, *Proc. 2nd offshore technology conference',* Houston, **1**, p. 577

Matlock, H., Ingram, W. B., Kelley, A. E., & Bogard, D. (1980): 'Field tests of the lateral-load behaviour of pile groups in soft clay', *Proc. 12th offshore technology conference,* Houston, **4**, p. 163

McClelland, B., & Focht, J. A. (1958): 'Soil modulus for laterally-loaded piles', *Journal of ASCE,* paper 2954, p. 1049

Meyerhof, G. G. (1953): 'The bearing capacity of foundations under eccentric and inclined loads', *Proc. 3rd ISCME,* Zurich, **1,** p. 440

Meyerhof, G. G. (1963): 'Some recent research on the bearing capacity of cohesive layers', *Brit. Jnl. of Appl. Physics,* **4,** no. 1, p. 16

Meyerhof, G. G., & Chaplin, T. K. (1953): 'The compression and bearing capacity of cohesive layers', *British Journal of Applied Physics,* **4,** no. 1, p. 20

Murchison, J. M., & O'Neill, M. A. (1984): 'Evaluation of *P–Y* relationships in cohesionless soils', *Proc. conf. on the analysis and design of pile foundations,* ASCE, San Francisco, California, p. 174

O'Neill, M. W. (1983): 'Group action in offshore piles', *Proc. conf. on geotechnical practice in offshore engineering,* ASCE, S. G. Wright (ed.), p. 25

O'Neill, M. W., & Ha, H. B. (1982): 'Comparative modelling of vertical pile groups', *Proc. 2nd int. conf. on numerical methods in offshore piling,* Austin, Texas, p. 399

O'Neill, M. W., Hawkins, R. A., & Mahar, L. J. (1981): *Field study of pile group action: final report,* Fed. Highway Admin., US, DOT report nos FHW.RD-81.002-FHWA, RD-81/008, March

Poulos, H. G. (1971): 'Behaviour of laterally-loaded piles: 1. Single piles', *Jnl Soil Mech. and Found. Div. ASCE,* **97,** no. SM5, p. 733

Poulos, H. G. (1980): 'An approach for the analysis of offshore pile groups', *Proc. 1st conf. on numerical methods in offshore piling,* ICE, London, p. 119

Poulos, H. G. (1983): 'Cyclic axial pile response – alternative analyses', *Proc. conf. on geotechnical practice in offshore engineering, ASCE,* Texas, Austin, p. 403

Poulos, H. G. (1985): 'Ultimate lateral pile capacity in two-layer soil', *Geotech. eng.,* **16,** no. 1, p. 25

Poulos, H. G., & Davis, E. H. (1968): 'The settlement behaviour of single axially loaded incompressive piles and piers', *Geotechnique,* **18,** p. 351

Poulos, H. G., & Davis, E. H. (1974): *Elastic solutions for soil and rock mechanics,* Wiley

Poulos, H. G., & Davis, E. H. (1980): *Pile foundations analysis and design,* Wiley

Randolph, M. F. *(PIGLET: a computer program for the analysis and design of pile groups under general loading conditions,* Soil report TR91 SUED/D, Cambridge University

Randolph, M. F. (1981): 'Pile subjected to torsion', *Jnl. Geotech. Eng. Div. ASCE,* 107, no GT8

Randolph, M. F. (1983): 'Design consideration for offshore piles', *Proc. conf. on geotechnical practice in offshore engineering,* ASCE, Austin, Texas, p. 422

Randolph, M. F., & Poulos, H. G. (1982): 'Estimating the flexibility of offshore pile groups', *Proc. 2nd int. conf. on numerical methods in offshore piling,* Austin, Texas, p. 313

Randolph, M. F., & Wroth, C. P. (1978): 'An analysis of the deformation of vertically loaded piles', *Jnl. Geotech. Eng. Div. ASCE,* **104,** no. GT12f, p. 1465

Reese, L. C., Cox, W. R., & Koop, F. D. (1974): 'Analysis of laterally loaded piles in sand', *Proc. 6th annual offshore technology conference,* Houston, **2,** p. 473

Reese, L. C., Cox, W. R., & Koop, F. D. (1975): 'Field testing and analysis of laterally loaded piles in stiff clay', *7th annual offshore technology conference,* Houston, p. 671

Ridehalgh, J., & Edwards, A. J. (1982): 'Design operation and towage of jack-up rigs', *Royal Institution of Naval Architects,* 18 Feb. 1982

Rigden, W. J., & Semple, R. M. (1983a): 'Design operation and towage of jack-up rigs', *Royal Institution of Naval Architects.* 18 Feb. 1982

Rigden, W. J., & Semple, R. M. (1983b): 'Design and installation of the Magnus foundation: prediction of pile behaviour', *Proc. conf. on developments in the design and construction of offshore structures,* ICE, London, p. 37

St. John, H. D. (1980): *A review of current practice in the design and installation of piles for offshore structures,* Dept. of Energy, Offshore technology paper 5

Seed, H. B., & Reese, L. C. (1957): 'The action of soft clay along friction piles', *Trans. ASTM,* **122,** p. 731

Skempton, A. W. (1951): 'The bearing capacity of clays', *Proc. Building Research Congress, Division 1,* ICE, London, p. 180

Smith, I. M., & Molenkamp, F. (1980): 'Dynamic displacements of offshore structures due to low frequency sinnsoidal loading', *Geotechnique,* **30,** no. 2, p. 179

SUT (1985): Proc. conf. on advances in underwater technology and offshore engineering, 3, Offshore site investigation, Society for Underwater Technology

Sullivan, W. R., Reese, L. C., & Fenske, C. W. (1980): 'Unified method of analysis of laterally loaded piles in clay', *Proc. int. conf. on numerical methods in offshore piling,* ICE, London, p. 107

Terzaghi, K. (1955): 'Evaluation of coefficients of subgrade reaction', *Geotechnique,* **5,** no. 4, p. 297

Terzaghi, K., & Peck, R. B. (1967): *Soil mechanics in engineering practice,* 2nd ed., Wiley, p. 217

Vesic, A. S. (1975): 'Bearing of capacity of shallow founcations', *Foundation Engineering Handbook,* ed. Winkerkorn, H. F., & Fang, H. F., Van Nostrand

Vijayvergiya, V. N. (1977): 'Load movement characteristics of piles', *4th symposium Waterway, Port, Coastal and Ocean Div. ASCE,* Long Beach, Ca., **2,** p. 269

7 Cylindrical storage-tank structures

7.1 Introduction

It is interesting to note that if a welded steel tank is scaled down to the size of a paper bag, the thickness of its base and walls do not exceed the paper thickness. The flexibility of steel tanks can be, and often is exploited, for example to make use of economically advantageous soft estuarine sites providing foundation conditions that are unsuitable for any kind of building structure. Tanks normally have fixed or floating roofs but occasionally are open-top. Today, concerns over excessive settlement tend to be about distortion of the tank shell impairing the function of floating roofs. Problems of overstressing the tank fabric are a less frequent occurrence for modern tanks. Hence, the emphasis in this section is on predicting settlements and the distortion of tank shape rather than the induced structural stresses, which nonetheless are considered.

Although steel tanks are very flexible, there are limits to the differential settlements that can be tolerated. Tanks are simple structures, and it is common knowledge that they can be realigned or repaired should settlements become excessive. This facility may not always be available, however, depending on the importance of the tank being brought into, and remaining in, service from an early date. Further, tanks that operate at low temperatures are more sensitive to differential settlement.

Conventional steel tanks on soft ground sites often have low computed safety factors for bearing under the full tank load. Consequently, inelastic soil response may have to be predicted, and its effects subsequently monitored in the field as the tank is prepared for service. As weak normally consolidated soils can develop relatively large inelastic strains prior to failure, evaluating the foundation response can present a formidable challenge. Unlike many structures, tank foundations are loaded to the maximum design value and experience many cycles of virtually zero to maximum load. The reliability of predicting large settlements, and their effects on tanks, could be improved if more comprehensive performance data were obtained.

Many of the difficulties that arise with steel tanks do so because of weaknesses in project management. These are discussed herein, and the need for better liaison between the tank designer and foundation engineer is emphasized.

Concrete tanks can be considered essentially as conventional building structures. As they often are used as secondary containment structures for storage of noxious liquids, criteria for allowable differential settlements can often be stringent.

This section begins with brief descriptions of tanks, both steel and concrete, their foundations and general design criteria. Problem soil conditions are then discussed, together with methods of ground improvement. Subsection 7.4 discusses modes of tank distortion and gives permissible values, while subsection 7.5 provides methods for settlement prediction. This is followed by information on ways of improving tank performance, performance monitoring, and site investigation. The final subsections comment on tank design codes and project management.

7.2 General description

7.2.1 Tank dimensions

The provisions of this report are intended to apply to the complete range of vertical cylindrical storage tank sizes and types. Floating-roof tanks have been built as small as 7m diameter or as large as 125m, with heights up to 23m. Fixed-roof tanks have been from 3 to 100m diameter, and up to 30m high.

7.2.2 Concrete tanks

Types and applications

Concrete tanks are widely used as primary containment structures in the water and public-health sectors. In the petroleum, gas and chemical industries, concrete tanks are mainly used as secondary containment for safety purposes, and the stored liquid does not come into direct contact with the concrete in normal operating conditions.

Both reinforced and prestressed concrete are used depending on the intended purpose and economy. Large cylindrical tanks for refrigerated and cryogenic liquids generally have a prestressed wall. If a reinforced concrete wall is used for such tanks then a surrounding fill embankment would normally be required to prevent overstressing of the wall. In either case a reinforced concrete base is common.

Foundation considerations

Concrete tanks can be supported directly on the ground using raft or slab foundations, or by piled foundations. Ground conditions generally have to be good for economic use of ground-supported foundations, as differential settlements have to be limited to prevent overstressing of the structure and damage to liquid-proof joints and seals. Predicted differential settlements exceeding 25 to 50mm may require the use of piled foundations.

As in the design of building structures, considerable liaison is needed between the structural engineer (tank designer) and the foundation engineer in order to limit settlements to permissible values. Settlement estimates should be made for all loading cases including empty, under full hydrotest, and after long-term service. Unlike steel tanks, concrete tanks cannot be readily relevelled in order to correct excessive settlement. Where there is a surrounding fill embankment to a liquified gas tank, the effects on settlement have to be taken into account.

7.2.3 Steel tanks

Structural stiffness

A conventional welded-steel storage tank is a flexible structure that transmits the weight of its liquid contents to the foundation as a uniformly distributed load. The tank can tolerate significant total and differential settlement without functional impairment. In particular, the differential settlement between the centre and edge of the tank bottom can be large. Allowable differential settlements beneath the tank shell are more restrictive, especially for a floating-roof tank where the clearance between roof and shell is critical.

With a cryogenic tank, there are the possibilities of brittle fracture in the chilled steel and malfunction of the brittle base insulation layer. These considerations place greater restrictions on allowable differential settlements.

Foundation types

The foundations that are built for tanks can be divided into a number of classes, including built-up earth, concrete raft and ringwall. The selection of foundation type depends on ground conditions and on tank design requirements. In many cases, there may be more than one type of foundation suitable for a particular tank design.

A tank foundation is required to provide total support for the tank bottom, which, for all practical purposes, has no structural strength, being only a liquid-tight membrane. Tank foundations are normally coned up to the centre of the tank to assist drainage of the product and water. However, the tank bottom may be flat, or coned down to the centre, depending on process considerations.

Refrigerated tanks

Foundations of refrigerated storage tanks require special consideration, as they have to combine loadbearing and thermal-insulation properties. Frost heave in the ground, which would cause damage to the tank, has to be prevented. Concrete rafts are either built at ground level incorporating heating elements, or raised on piles with natural air circulation below the concrete to provide bottom heating. Permissible differential settlements imposed on the tank-bottom insulation are usually more restrictive than those for the tank structure itself.

Tank roof

In general, a floating roof is employed if the product is volatile and is to be stored in large volumes. Fixed-roof storage is used for refrigerated liquids and non-volatile products. In cold climates both volatile and non-volatile products may be stored in fixed-roof tanks as the seals of a floating-roof tank may not operate in severe frosts and heavy snowfalls.

7.2.4 Hydrotest

It is normal for bulk-storage tanks to be designed for a minimum product specific gravity of 1.0, and tested to their maximum capacity with water, even though in service they may be storing liquids with significantly lower specific gravities. This procedure acts as a safeguard against overstressing the tank because of a change in stored product. Certain pressure tanks, notably refrigerated tanks, are designed using the specific gravity of the product as they will never see any alternative service. Frequently, such tanks have been partially hydrotested to a height that gives a foundation hydraulic load not greatly in excess of the service condition. There is a tendency to move away from this procedure towards full height testing, to acquire the fullest benefit of preloadings, so that foundation loads at test will greatly exceed service conditions.

7.3 Foundation considerations for steel tanks

7.3.1 Soil conditions

It is not uncommon for tanks to be required on sites near rivers or estuaries. Often there are considerable thicknesses of recent deposits, and in many cases land has recently been reclaimed by filling with soft dredgings. A dried crust a metre or so thick may have an average shear strength of about $50 kN/m^2$, and a misleading impression of the site may be formed until borings are put down. The strength of the underlying soil may be much lower and may gradually increase with depth from $10 kN/m^2$ just below the crust, not reaching a strength equal to that of the ground surface until about 20m depth.

Once the true nature of the ground is revealed, the designer may decide to use piles to transfer the weight of proposed tanks through the soft strata to bedrock below. This is not always a technically satisfactory solution, and it may be expensive. Alternatively, the soil may be improved by one of several methods described below.

7.3.2 Consolidation under tank loading

Clarke (1971) indicated that it is the policy of one major oil company to strengthen the ground by consolidation if the predicted maximum tank-edge settlement is greater than 300mm. Clarke recommended preconsolidation by surcharge, but this approach has not been widely adopted. When the project schedule permits, a simple and cheap method is to use the weight of the hydrotest load in the tank itself. This method requires the most careful monitoring of tank behaviour during consolidation in order to avoid excessive settlements with respect to tank integrity. Success depends on soil type and thickness, and the drainage conditions. If the site is an estuarine silty clay containing sand layers and resting on sand, the time required to achieve a sufficient strength gain through consolidation may be only days (Penman & Watson, 1967). However, if the site is on a considerable thickness of soft clay supported by an impervious bedrock, some months or years might be required for strength improvement by consolidation only, and some other method may have to be used.

Clearly the greater the safety factor against shear failure, either at the initial undrained soil strength or after slow test loading in drained conditions, the smaller the excess settlements. The owner has to balance cost savings of the simple pad foundation and faster test loading against costs incurred through tank distortion and movement of connections. A cost allowance has to be made for the design and control of test loading that requires high-grade supervision.

7.3.3 Preloading with surcharge

Rather than use the tank test load to consolidate and strengthen the ground, a fill surcharge can be used. Commonly, fill is twice as dense as water so that a mound about half tank height should provide test loading. With this arrangement, the opportunity is usually taken to make the mound somewhat higher, thereby prestressing the foundation with a load in excess of that caused by the tank. Good practice is to extend the area of full-height surcharge to cover the tank-shell position in order to minimize subsequent bottom-plate rotations near the shell. While failure should be avoided, to prevent breaking the surface crust and loosing fill, its consequences are usually not so serious as if a tank had been in position. When sufficient consolidation has taken place, the fill can be removed down to the desired level for the tank floor. The lowest remnant of the fill can be shaped and coated with asphalt to form the base pad for the tank.

Fill can also be used to form a stabilizing bund around the periphery of a tank during test loading, and to reduce ground heave and hence lateral flow of soil from under the tank (e.g. Liu & Dugan, 1975).

7.3.4 *In situ* compaction and stone columns

Loose sands and silty sands or sandy silts can be improved by *in situ* compaction. Dynamic compaction can be used, but the depth to which this method is effective depends on the energy applied. In general, the upper layers are the ones most compacted. Deeper compaction can be achieved by use of laterally vibrating pokers equipped with waterjets. The cylindrical holes so formed are filled from the surface with additional granular material. By using stone as fill over a vibrating poker, a stone column can be formed in weak cohesive soil. Such columns can sometimes be formed so as to extend through the whole thickness of the weak soil to stiffen it, and so reduce and speed consolidation settlement. The columns then act as compressible piles and transmit some of the foundation load to the underlying strata. The poker compacts the stone tightly against the soil. Dilation and expansion of the stone columns under foundation load can help to densify the soil further and to provide more restraint for the columns. Further discussion of these procedures can be found in Greenwood & Kirsch (1983).

7.3.5 Piles

If a layer of weak soil is considered unsuitable for any form of improvement, conventional piling may be used to transmit tank weight to lower strata. A large number of piles is usually required to reduce bending moments in slabs to economic values. Slab deformations can be complex, necessitating pile-head connections that allow for movements. Caution is required if piles are to be raked when there is also a possibility of large consolidation settlements under the wide loaded area. Care needs to be taken when driving piles to avoid damage by lateral displacement to those piles already installed. Difficulties of displacement damage, driving resistance, and to some extent negative skin friction can be overcome by placing the piles in predrilled holes. However, care also has to be taken with bored piles (cast *in situ*) to avoid displacements of soft soil by fluid concrete, and other damage, as the piles are being formed (Leonards, 1982).

7.3.6 Underbase preparation

Earth construction

A simple pad foundation usually is made of a well graded granular material that can be compacted with a vibrating smooth roller to a dense state in layers. It is extended beyond the tank edge sufficiently to distribute concentrated loads such as those arising from the weight of the tank shell. The pad has to be made sufficiently thick to allow for settlements, and it is usual to raise it in the centre and provide a uniform slope towards the edges. Detailed recommendations can be found in BS 2654 (1984). Since it is known that maximum settlements often occur not at the tank centre, but at about one-third of the tank diameter from the centre, there is an argument for profiling the pad surface to mirror predicted settlement. It is desirable that floor settlements do not induce large tensile stresses in the tank plates (Green & Hight, 1975).

When relatively weak tank-pad materials are used, a composite section including high-strength materials (e.g. crushed rock) below the tank shell will be required. Some examples are given in API Standard 650 (1980) and by Roberts (1961) and Clarke (1971). A minimum computed safety factor of 1.5 is recommended for the concentrated shell loading to provide sufficient assurance against local edge cutting (Roberts, 1961).

The tank pad may be provided with drainage layers and/or an impermeable membrane in order to provide early warning of bottom-plate leakage. If the membrane is incorrectly laid it may lead to water ponding thus causing bottom corrosion. The upper surface of the pad is usually finished in soft asphalt to give a smooth surface to accept the tank plates, to provide corrosion protection, and also to allow some slight subsequent sliding movement when settlements occur.

Ringwall support

For pressurized tanks, it may be necessary to prevent uplift of the shell by means of an anchorage built into a reinforced-concrete ring beam. A concrete beam or wall may also be used with the purpose of limiting lateral flow and differential settlement beneath the tank perimeter. The depth of the ringwall is usually limited by the depth that can be trenched in a soft soil. This trench may cut through any hard crust and lead to a greater immediate settlement of the tank walls than of the floor. There is a danger that use of a ringwall will accentuate differential settlement at the edge of the floor plates. A ringwall should be of some advantage on loose sand in limiting migration of sand from under the tank edge, but it may be of questionable use with weak clay soils. Tank pads with ringwalls are discussed in API Standard 650 (1980).

7.4 Limiting tank distortions

7.4.1 General

There is a considerable amount of empirical data on cylindrical steel tanks that, in the context of normal structures, have settled substantially and may have failed as a consequence. There are fewer examples of large reinforced-concrete tanks. These have been used mainly for storage of cryogenic or noxious liquids, and there is apparently little information on their settlements. Such large concrete tanks are usually prestressed to preclude tensile cracking. Compared with the flexibility of thin steel tanks they are relatively stiff structures which, taken together with risk of spillage of harmful contents, generally leads to selection of piled or naturally stiff foundations.

The subject of interaction between a structure and a relatively compressible soil is complex. Generally, it is not practical to resolve major uncertainties concerning the effective stiffness of the structure and its variation during construction, the effect of the construction rate on the ground, and the precise ground behaviour especially if it is weak.

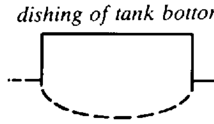
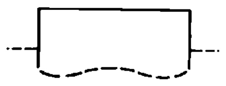
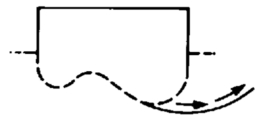
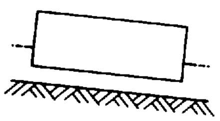
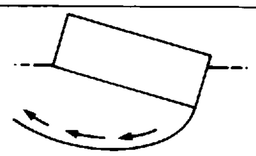
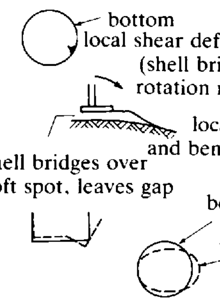
7.4.2 Deformation criteria for steel tanks

The following discussion of deformation modes and criteria for their limitations is based on reported observations and applies to the range of steel tank sizes and types stated earlier (see clause 7.2.1).

Based mainly on a recent summary of published experience (D'Orazio & Duncan, 1982) Table 5 indicates that types of deformation commonly experienced, designated as A to F, and their consequences. Experience indicates that the ascending order of concern is F, C, A/B, D and E. Tilting (types D and E) can occur simultaneously with perimeter distortion (type F) and distortion of the tank bottom (types A/B and C). Edge shear failure (type C) may simply be a gross form of type F since both are initiated through a preferential local weakness.

Table 5 provides 'common design limits' for the various types of deformation. The basis for these values is discussed in subsequent paragraphs. It is important to recognize that these values, often quoted in the literature as tolerable limits, are not in fact operational limits for tanks. Instead, they represent no more than a conservative interpretation of successful experience that may be useful to the designer. There are undamaged tanks that have substantially exceeded these criteria, and to do so should not result in unserviceability if hydrotest performance is carefully monitored in the field. Some organizations would also distinguish between different limits and use criteria that relate specifically to design, fabrication acceptance and operational limits. Literature relevant to the various design limits is identified in the clauses below. Study of these publications discloses the limited information from which the criteria have been developed, and the extent to which they cannot be said to represent distortions causing functional impairment or failure of tanks.

Table 5 Deformation of steel bulk liquid storage tanks

type	type of deformation	common design limits	effect on tank	potential consequence
A	*dishing of tank bottom*  consolidation	$\left(\dfrac{\delta_{\mathbb{C}} - \delta_{edge}}{\text{diameter}}\right)$ $\dfrac{1}{75}$	excess tensile stresses in bottom	tank rupture
B	 consolidation and shear deformation	$\dfrac{1}{125} - \dfrac{1}{75}$	excess stresses in the region of the shell–sole plate connection with tendency to rotate joint	tank rupture – bottom plate yields or welds tear in floor or at sole plate
C	 edge shear failure	$\dfrac{1}{250} - \dfrac{1}{125}$	differential movement of roof columns development of undrainable area on tank bottom	buckling of cone roof tank is difficult to empty and clean
D	*tilting*  consolidation	slope of maximum tilt on any diameter $\dfrac{1}{200}$ (visible tilt)	excess stresses in shell wall separates from floating roof	tank rupture evaporation of product–air pollution
E	 base shear failure	$\dfrac{1}{200}$ (unless associated with type F)	excess stresses in reinforced concrete mat (if present)	failure of mat
F		deformation measured from 'best fit' tilt plane 0.03m to 0.05m for diameters 15m to 100m, respectively	excess local bending stresses in bottom when wall bridges over a 'gap' excess local shell stresses on fixed roof tanks distortion of tank walls and jamming of floating roof	tank rupture – plastic yield of base of walls tank rupture damage to roof seal and roof

Differential perimeter settlement

Criteria vary for distortion limits along the edge of the tank shell. Limits are usually quoted as angular distortion measured along the tank circumference with respect to the tilted plane through the shell base that best fits the measured settlement data. However, a criterion of this type is incomplete without standardization of methods of assessing the distortion in respect of the length over which it is measured, and therefore averaged, and its relationship to tilting measured across a tank diameter.

Tanks normally are monitored during proving water tests immediately after construction by means of perimeter settlement measurements at a minimum of 8 points. For the largest tanks in current use, separation between 8 points approaches 40m, while for smaller ones it is less than 6m. In relation to the scale of significant soil strength variation that can reflect through the foundation pad, the former is clearly too great and the latter more appropriate. Since larger tanks are more flexible and more likely to deform with the ground it is prudent to limit the length over which such measurements are made to about 6m.

Experience documented in the literature indicates that angular distortions* up to 1/500 can safely be tolerated (de Beer, 1969; Langeveld, 1974; Belloni *et al* 1975; Sullivan & Nowicki, 1975; Greenwood, 1975; Penman, 1977; Bell & Iwakiri, 1980; and Rosenberg & Journeaux, 1982). The commonly used design values given in Table 5 allow greater angular deformations in small tanks.

*Difference in level from the average base tilt plane divided by distance between measurement points

Average tilt plane

Concern is with differential settlements – the magnitude of tank distortion from its original shape. Settlement measurements on the perimeter usually will indicate some overall tilt. In order to compute differential settlements, it is necessary to determine and account for settlements resulting from tilt. The deformations that give rise to the least change of stresses will be those that yield the minimum distortion in relation to a tilted basal plane. This is the average or 'best fit' tilt plane, not necessarily the plane of maximum tilt that is measured across a diameter (Marr *et al*, 1982).

The average tilt plane is obtained by ascertaining absolute levels at equidistant points on the tank perimeter. Commonly, perimeter distortion will broadly indicate that the tank edge has bent about two diametrically opposed nodes, as in Fig. 30 where the tank has folded and twisted about the 4–8 diameter. In this case, the average tilt plane can be determined using the graphical procedure described by de Beer (1969). The diameter having the maximum difference in measured settlement is selected, and the elevation of all points is plotted in relation to their location along it (see Fig. 30b). Pure tilt appears as a straight line in such a diagram, and as a sine curve when represented as in Fig. 30c. The regression line of level on location in Fig. 30b gives the required tilt plane. The average plane used as a datum for the type F distortion criterion is not the same as the maximum diametric tilt plane, which is the basis for criteria types D and E (see Table 5).

The development of exaggerated multiple node distortions is unusual. If they do occur, one way to determine the average tilt plane is from a Fourier analysis of settlement data as recommended by Marr *et al* (1982). The use of a Fourier analysis was initially suggested by Malik *et al* (1977), who gave an example application.

Sometimes only a segment of tank perimeter will coincide with a planar tilt, other segments fitting no clear pattern. In such cases, a graphical trial-and-error fitting process may provide the most effective way of judging the 'best-fit' tilt plane (Bell & Iwakiri, 1980). There remains, however, much confusion in the literature over determination of the average tilt plane, which reflects the difficulties that can arise in practice (Craig, 1974).

Tilting

Fig. 30 illustrates how the plane of maximum diametric tilt can differ from that of the 'best-fit' plane. Before the influence of tilting can significantly change the stresses arising from liquid loading, the amount of tilt becomes unacceptable. Furthermore, it is unusual for tilting to result in such excessive ovality in the plane of the floating roof of tanks that the seal tolerance is overcome. Effects on pipe connections and aesthetic considerations therefore are likely to control the criterion for maximum tilt. Tilts become readily visible at about 1 in 200 measured as the tangent of the slope of maximum tilt (Greenwood, 1975; and Bell & Iwakiri, 1980).

Tank-bottom deformations

The bottom plates of steel tanks are relatively thin, and tank floors are very flexible. Deformations associated with edge shear failure (type C) or with incipient shear failure (type B) usually lead to the greatest curvature of base plates. In these cases maximum settlements are recorded at about one-quarter to one-third of the radius from the perimeter (Penman & Watson, 1967; de Beer, 1969; and Bell & Iwakiri, 1980).

Settlements occurring during water test are rarely measured except at the tank centre and perimeter. Distortions of 300mm over 3m have been measured after entering a tank on completion of test loading (de Beer, 1969), representing considerably greater curvature of the floor than the centre-to-edge average. Concentration of this curvature close to the weld at the annular plate is common (Penman, 1977; and Rosenberg & Journeaux, 1982). Moreover, it occurs in a contrary sense to the outward rotation of the base at the wall under hydrostatic pressure. As discussed later, tolerances on annular welds and plastic yielding of bottom plates need to be checked independently of the criteria listed in Table 5.

D'Orazio & Duncan (1982) analysed reported tank performance data (Fig. 31) from which they derived characteristic tank-bottom profiles (Fig. 32), and proposed limits appropriate to each settlement type. These proposed limits are shown in Table 5 and are centre-to-edge averages representing satisfactory performance even if maximum base settlements do not occur at the centre. This is possible because of the usual geometric relationships of the structure and soft foundation strata that allow classification of settlement behaviour into three fairly well defined types. Criteria based on initial base shape and stress calculations only are discussed in a later subsection. Experience of bottom deformations has been documented by Saurin (1949), Carlson & Fricano (1961), Rinne (1963), Penman & Watson (1967), Clarke (1971), Guber (1974), Langeveld (1974), Belloni *et al* (1975), Esrig *et al* (1975), Green & Hight (1975), Jamiolkowski (1975), Sullivan & Nowicki (1975), Penman (1977), Bell & Iwakiri (1980), Rosenberg & Journeaux (1982) and Hegg *et al* (1983).

Shell deformation

The roof-seal tolerance to radial deformation is the limiting criterion for shell ovality in floating-roof tanks. The allowance is normally ±100mm from the neutral position at the shell lower course. At the shell upper courses this

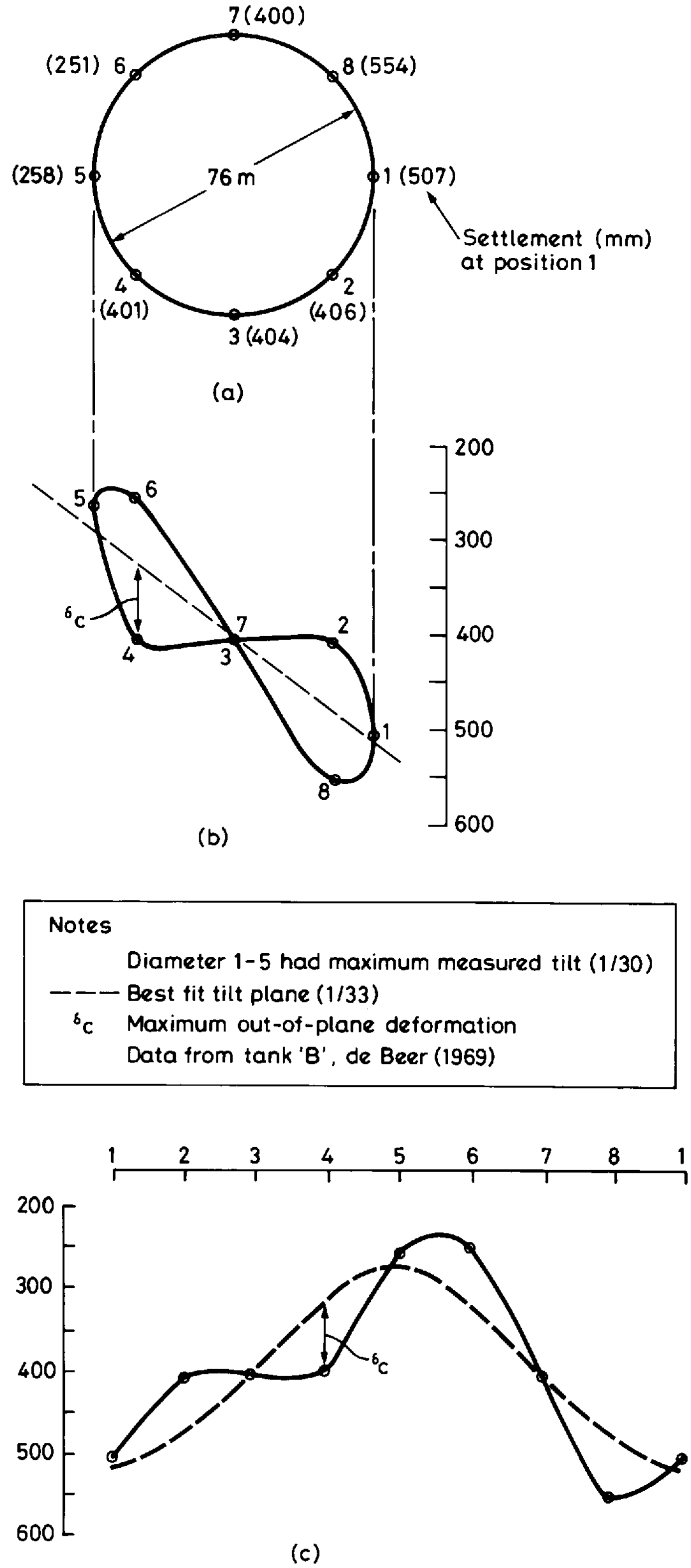


Fig. 30 Determination of basal tilt plane

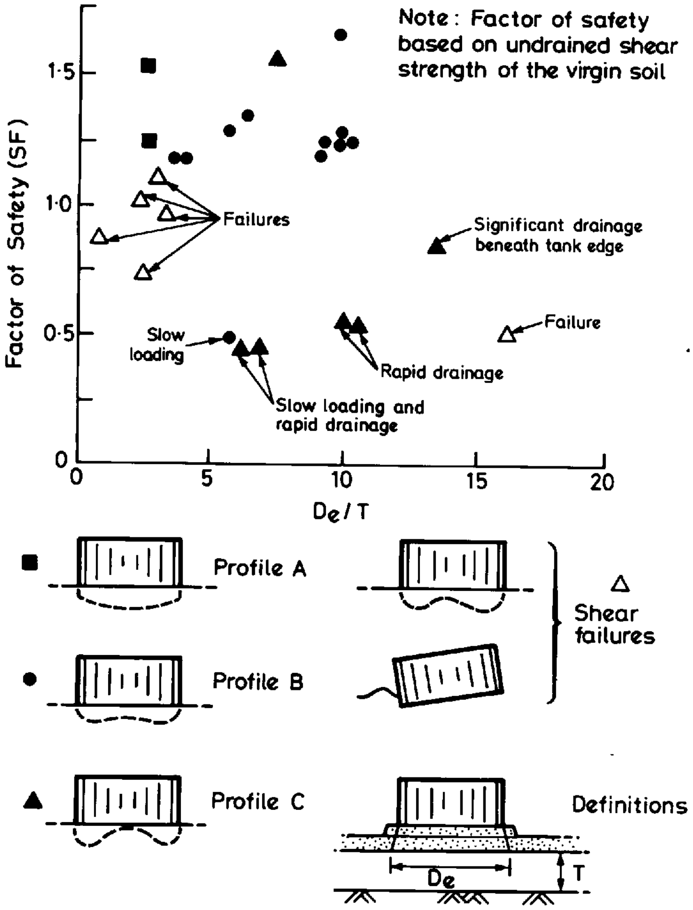


Fig. 31 Factors of safety and settlement shapes
Based on D'Orazio & Duncan (1982)

allowance can be less because of tank construction tolerances. Seals are most at risk with bi-nodal foundation deformations. Excessive outward movements render the seal ineffective, whereas inward movement carries the danger of seizing of the roof against the shell.

Tests performed on an open-top tank model (Malik *et al*, 1977), correlated with field site surveys, have provided a useful relationship between the maximum increase in diameter at the shell top ($\triangle D$), the maximum out-of-plane settlement at the circumference (δ_c), and the height/diameter ratio of the tank (H/D). Specifically,

$$\triangle D/\delta_c = n^2.H/D$$

where n is $2\pi/\phi$, and ϕ is the angle in the horizontal plane over which the settlement occurs. Various formulae of this type available in the literature have been compared by Marr *et al* (1982) who recommend the relationship proposed by Malik *et al* (1977). As ϕ reduces below one radian there is a tendency for shell bridging over the settling area, and the predicted radial movements become progressively pessimistic. In such cases, more detailed computer solutions are necessary if accurate prediction is required of shell behaviour in specific foundation settlement conditions (e.g. Guber, 1974).

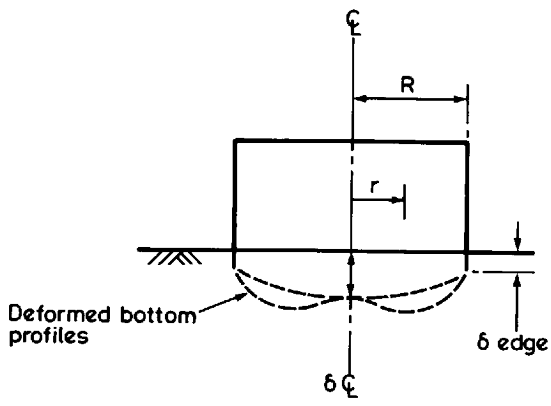
Experience indicates that the various methods of predicting shell deformations developed and published over the last 10 to 15 years are of considerable assistance in certain situations. However, no single method is all-embracing, and judgment is required in selecting the method that most suits the problem under consideration.

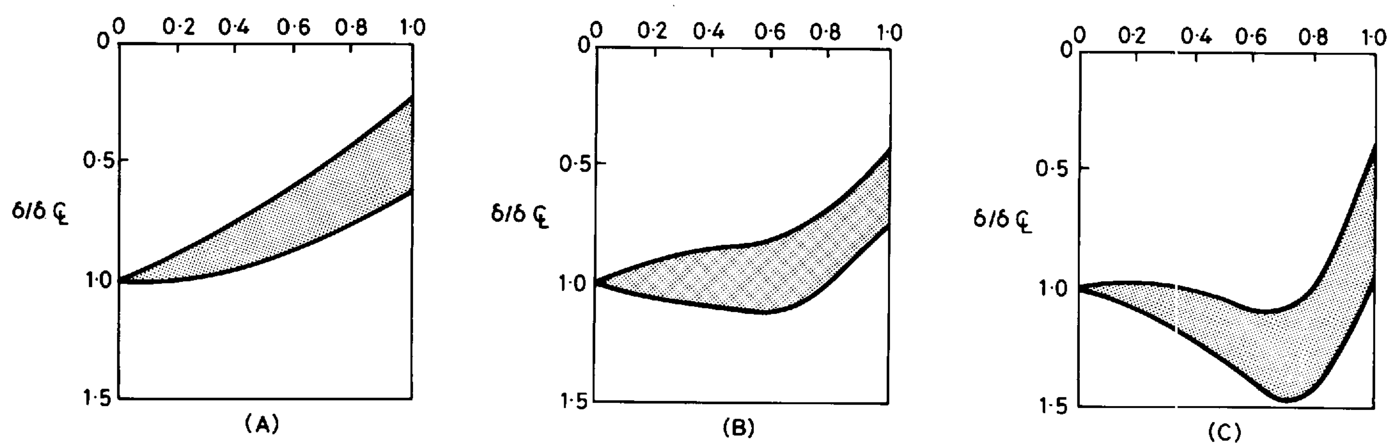
Fig. 32 Normalized settlement of tank bottom
Based on D'Orazio & Duncan (1982)

SF	D_e/T	Most Likely Profile Shape
>1·15	≤4	A
>1·15	>4	B
≤1·15	any	C

See fig.16 for De/T

See Fig. 31 for D_e/T





7.4.3 Steel overstress criteria

Tank bottom

Stresses in the tank base plates arising from settlement are influenced by the initial shape to which the base was constructed: either coned up or down. Coned-up tanks bases may be free of tensile stress in the initial stages of settlement, but are susceptible to the formation of wave buckles in the course of settlement. Typically, these may be 150mm high and 300mm wide.

In coned-down tanks, the deeper the initial cone the more rapidly the tensile stress in the plates increases as settlement occurs. The criteria in Table 6 have proved to be safe, based on consideration of stress calculations only. These limiting values indicate the allowable change in slope of the base profile. They are deformations from the initial position represented by differential edge-to-centre settlement expressed as a fraction of the tank radius. These data were based on an assumed triangular rather than parabolic settlement distribution, full edge fixity, and an allowance for 'slack' plate. The plate thicknesses, materials, welding procedures and allowable stresses are those typical of current codes and standards. Observed tank settlements indicate that considerably greater deformations may be acceptable, although the German code DIN 4119 (1979) indicates more conservative criteria may be required for larger tanks.

Table 6 Limiting values of base profile slope

form of base	coned-up	flat	coned-down			
initial slope	1/120	0	1/100	1/75	1/50	1/25
limiting value	1/20	1/25	1/33	1/50	1/70	1/100

The above rules exclude the effects of local weak spots, which should be investigated individually. Weak spots under tank foundations are known to occur because of inadequate backfill in excavated areas. Typically, such areas are located adjacent to a concrete ringwall supporting the tank shell, or the cellar housing the bottom outlet pipe. The steel plating in these areas normally can tolerate considerable deformations resulting from soil settlement. However, for low-temperature tanks it may be prudent to incorporate concrete transition slabs over the critical backfilled areas to protect the tank bottom plating from the combined effect of differential settlement and low temperature.

Shell-to-bottom junction

The junction of the shell walls to the bottom plates is a critical zone. Current rules in tank design standards stipulate a maximum fillet weld size at the junction equal to the thickness of the annulus, without the requirement for the designer to check the actual local stress arising from discontinuity forces and moments in both the weld and the plate. This criterion has proved satisfactory where the junction is not subjected to additional rotation because of settlement. In the vertical radial plane, the junction rotates because of dilation on filling, causing the nominally 90° angle between the shell and the bottom to open by about 2°. As a general guide, the design of the foundation should restrict foundation settlement so that the additional opening of the junction arising from settlement is limited to a maximum of 5°. Occasionally, because of imbalanced welding of the junction, the annulus may lift upwards which, on filling, may cause additional opening of the junction by up to 2°. In this case, opening arising from settlement of up to 3°, additional to the nominal rotation arising from dilation, should be acceptable. Note that the additional slope arising from foundation settlement should be measured over a base length of 1m from the junction. This limitation can be rationally applied only after the tank fabrication, and so it is more valuable as an inspection aid rather than a design objective.

In cases where there is a possibility of substantial rotation because of settlement and welding distortion, a finite-element analysis may be performed to assess the acceptability of the resulting steel stresses, considering both static and fatigue conditions. The analysis is not straightforward as particular attention should be paid to modelling the annular plate lift-off just inside the shell, local squeezing of the asphalt layer, and non-linear behaviour of the tank pad. The criteria for acceptability would be hinge formation in the bottom annulus with yielding through, say, 40% of plate thickness, and the fatigue limit stated in BS 5500 (1985) for pressure vessels.

Shell

For floating-roof tanks, it is shell deformation criteria relating to roof seals (see clause 7.4.2 – shell deformation) rather than overstressing that are of importance. With fixed-roof tanks there is a tendency for rotation at the roof–shell connection, which can induce excess local stressing because of restraining ovality. The influence of settlement on shell stresses can be assessed by investigating forces and moments required to restrain free shell deformations at the perimeter of its roof elements.

Shell nozzles

Piping connected to the shell nozzles will exert loading if the tank settles relative to the first pipe support. Tolerable differential settlement can be evaluated by estimating local stresses generated in the shell by the piping moments and forces caused by tank settlements. These stresses can be estimated as indicated in BS 5500 (1985) for pressure vessels, or more accurately by the method proposed by Adams (1985). The effect of pipe loading on a settling tank can be minimized by providing inherent pipe flexibility, spring supports, bellow units, or simply by periodic pipe cutting and rewelding.

7.4.4 Reinforced-concrete tanks

The comparative stiffness and need for crack control in concrete tanks requires a more stringent set of criteria for tolerable settlements. As a result, such structures are generally built on naturally solid ground or on piled foundations taken to such depths as to limit settlements largely to the elastic deformation of the piles themselves. Consequently, empirical criteria based on failures have not been established.

Generally, the height/diameter ratio of concrete tanks is rather greater than that of steel tanks. Accordingly, they are stiffer and more likely safely to bridge local soft spots.

It is suggested that the concept of critical tensile strain proposed by Burland & Wroth (1975) be employed to limit cracking in accordance with requirements for liquid-retaining structures. Differential settlement at the perimeter will then depend on the tank height/diameter ratio, and on the provision of shear reinforcement in relation to bending stiffness and deformation mode. The data collected by Burland & Wroth for loadbearing walls suggest that a safe figure for out-of-plane perimeter distortion would be about 1/1000. Such values should be viewed in the context of the consequences of failure.

7.4.5 Cryogenic tanks

Refrigerated contents further complicate design and choice of tolerances as both insulation materials and metals or concrete may be subject to abnormally low working temperatures. It is assumed that foundations will be adequately designed to prevent potential frost heave.

Having regard to these circumstances, BS 4741 (1971) for low-temperature steel tanks recommends a diametric tilt limit of 1/1200, maximum centre-to-edge differential settlement of 1/600 of the tank diameter, and a maximum angular distortion at the perimeter of 1/700 over a 9m length. There is also a requirement to avoid excessive local distortions near any ringwall. These appear to be overcautious criteria that may be helpful to the designer, but should not be viewed as proven operational limits.

7.5 Stability and settlement of steel tanks

7.5.1 Foundation loads

Three loading actions should be considered in foundation design:

- the concentrated weight of the shell on a strip at the perimeter, including the shell-supported roof when applicable and uplift loads in pressurized tanks
- the uniformly distributed bearing pressure imposed by the stored product or water test load plus the additional stress arising from the tank pad, and
- the loaded tank as a unit.

For large tanks, those of more than about 50000m^3 capacity, the second aspect will normally control settlement behaviour and govern soil strength requirements.

7.5.2 Stability considerations

Effect of safety factor on settlement

The magnitude and pattern of tank settlements are affected by the degree of soil shear-stress mobilization, which is governed by the safety factors against general and local bearing-capacity failures provided by the design. As the mobilized shear stresses approach the shear strength of the soil, the increasing non-linear response results in progressively larger ground movements. Zones of stress concentration at the tank edge typically are more affected, so that inadequate bearing capacity usually results in excessive settlements or a failure near the perimeter of the tank.

The tendency for edge-stability problems in weak soils is exacerbated by the typical profile of increasing shear strength with depth that occurs in normally or lightly overconsolidated clays. In such soils, a shallow localized failure at the edge of the flexible tank bottom is more likely than a general base failure, which requires a larger failure surface that must penetrate deeper and stronger soils. Further, the soils influenced by a large tank are seldom homogeneous. Weak and heterogeneous layers may not affect the general stability of the tank but may result in a local failure at the tank edge, particularly if these layers are present at shallow depths near the tank bottom. Assessment of edge stability is discussed by Bjerrum & Overland (1957).

General bearing capacity

Experience has shown that the general bearing capacity of a foundation is governed by the average shear strength existing to a depth below the foundation equal to two-thirds of the foundation width, provided that the shear strength within this depth does not vary by more than 50% from the average strength (Skempton, 1951). The bearing capacity then may be taken as six times the average shear strength. Recommendations for applying bearing capacity theory to tanks on more variable layered soils are given by Duncan & D'Orazio (1984).

Compressible ground

The conventional welded steel tank is suffienciently flexible that settlement is unlikely to be a problem if the ground provides a safety factor of two or more against local and general bearing capacity. Significant settlements can occur in cohesive soils when the increment of vertical stress, q, arising from the loaded tank and tank-pad construction above surrounding grade exceeds three times the undrained shear strength, s_u, of the weakest soils in the profile. When $q > 5s_u$, the subgrade is too weak to carry the load of the filled tank without large and probably excessive settlement. In such conditions, shallow or superficial construction immediately beneath the tank bottom is unlikely to produce significant improvements. It is then necessary carefully to investigate foundation stability, and to consider the methods of subgrade strengthening discussed earlier (see subsection 7.3).

Detailed stability assessment

In weak soils where the computed safety factor is less than 1.5 (i.e. $q/s_u>4$), the ground will be loaded into the virgin compression range, resulting in relatively large volumetric strains after drainage of the initial excess porewater pressures. In addition, the initial application of loading can stress the soils such that inelastic strains or plastic flow may occur in undrained conditions in some zones beneath the tank. For important tanks on weak soils ($q/s_u>4$) it is usually appropriate to evaluate stability in some detail.

In addition to the bearing-capacity methods cited above, stability can be assessed by considering various potential slip surfaces (e.g. Penman & Watson, 1965) or by computing shear-stress levels at various points beneath the tank. In the latter approach, shear-stress levels can be obtained by comparing the distribution of shear stresses (from stress-distribution theory or a finite-element analysis) with that of soil shear strength. Simple linear elastic-stress distribution coupled with an estimate of the initial *in situ* stress state (Brooker & Ireland, 1965; Mayne & Kulhawy, 1982) and shear strength data from the site can provide insight into the likely development of yield or plastic deformations in the ground. For sites with marginal or deficient stability, such predictions provide a rational basis for interpreting pore-pressure measurements and tracking effective-stress states for various critical soil elements to guide preloading or stage loading operations aimed at strengthening the ground. Useful information on stress conditions induced by tank loading, and their effects, may be found in Darragh (1964), Penman & Watson (1965), Davis & Poulos (1968), D'Appolonia *et al* (1971), Wood (1980), Clausen *et al* (1984), and Watson *et al* (1984).

7.5.3 Settlement prediction

Tank bottom

In relatively weak ground ($q/s_u>3$), where settlements are likely to be of significance, comparison of tank settlement data with theoretical predictions indicate that an order-of-magnitude estimate for settlement at the tank centre can be obtained from conventional one-dimensional consolidation theory (Foott & Ladd, 1981). The increment of vertical stress at various depths below the tank can be estimated from standard elastic-stress-distribution theory (e.g. Poulos & Davis, 1974) considering conditions beneath the tank centre. Characteristic tank-bottom settlement profiles based on measurement data are given in Fig. 32 (D'Orazio & Duncan, 1982). The profiles are axisymmetric, and depend on the factor of safety against bearing-capacity failure and the thickness of compressible soils relative to the tank size. From the calculated centre settlement, values of settlement along a tank diameter may be obtained from the estimated profile shape. Variable soil-layer thicknesses or strengths beneath the tank will affect estimated settlement profiles. Such variations could significantly change the profile given in Fig. 32.

Perimeter distortion

Reliable prediction of differential settlement along the tank perimeter is very difficult, and in practice often may not be possible because of the variable nature of the ground or insufficient soils information. However, de Beer (1969) describes an extensive site-investigation programme, including eight points around the proposed tank perimeter, and how the results were used to predict edge settlements by one-dimensional consolidation theory. Measurements indicated that differential settlements were overpredicted, probably because the bending stiffness of the tank shell with respect to in-plane movement was ignored. There is no simple relationship of general applicability between the centre settlement of a tank, whose approximate magnitude can be forecast, and differential settlements at the tank edge (e.g. Jamiolkowski, 1975). D'Orazio & Duncan (1982) indicate that Japanese engineers have concluded, from statistical analysis of tank behaviour, that perimeter distortion is likely to cause problems for large conventional tanks when the average perimeter settlement exceeds 600mm at clay sites. Edge settlements normally are measured during the initial filling of the tank and simultaneously compared with tolerance criteria such as those described earlier. This allows loading to be stopped and corrective measures to be taken before excessive settlement occur.

7.5.4 Immediate settlement

Undrained shear displacements

That one-dimensional consolidation theory provides an order-of-magnitude estimate of tank centre settlement in weak ground is in many cases fortuitous. Actual soil behaviour beneath a tank can depart significantly from that of one-dimensional consolidation theory, which in this regard may be viewed as a convenient practical model. Unless the compressible soil is present only as a relatively thin layer at some depth below ground surface, there will be immediate (or undrained) settlements resulting from 3-dimensional strains, i.e. from lateral movements. For sites with relatively weak clays ($q/s_u>3$) these settlements can be viewed as resulting not only from elastic strains but also from plastic movements in the more highly stressed zones within the foundation soils. The consequence can be relatively large undrained settlements that occur simultaneously with tank loading in a manner not implied by the use of one-dimensional consolidation theory for settlement prediction.

Using *in situ* displacement measurements, Penman & Watson (1965) showed that the volume of lateral movement from beneath a tank on weak soil correspond to the undrained settlement. Other notable lateral deformation measurements beneath tanks in weak ground have been made by the Italian Geotechnical Institute (Belloni *et al*, 1975; and Hegg *et al*, 1983).

Settlement prediction

Approximate forecasts of immediate tank settlements for weak ground conditions can be obtained using the procedure given by Foott & Ladd (1981), which accounts for the initial *in situ* stress state and the average shear-stress level induced by tank loading. The method can, in principle, provide estimates of immediate settlement for shear-stress levels ranging from very low (where only small elastic movements occur in relatively competent soils) up to values producing fully plastic soil response (corresponding to failure). Although inelastic movements are predicted, the key soil input parameter is its elastic modulus and low stress level. Accurate determination of initial soil stiffness is difficult, but recent advances in laboratory measurement technique are reported by Jardine *et al* (1984). Such relatively sophisticated testing may be appropriate for important tanks on weak soils in order to avoid unduly pessimistic settlement predictions.

The purpose of the foregoing discussion is to emphasize that immediate settlements are not necessarily elastic or small. They may be significant for weak soils, particularly highly plastic clay with some organic content. Nonetheless, measurements of total settlement have indicated that it is reasonably predicted by one-dimensional consolidation theory (see clause 7.5.3 – tank bottom). Specifically, it is not necessary to add the immediate and one-dimensional settlement values (Foott & Ladd, 1981), or to modify the total settlement calculated from consolidation theory in the manner recommended for footings by Skempton & Bjerrum (1957).

7.5.5 Long-term settlement

Clause 7.5.4 provides guidance on predicting settlements based on experience and the body of observational data available in the literature. However, virtually all of the published tank-settlement records represent the early stage of soil consolidation. Some tank owners have long-term settlement data gathered over the lifetime of tanks which indicate continuing settlement for 10 or 20 years, well beyond the time for completion of primary consolidation. Bjerrum (1966) presented long-term settlement records for different containment structures on sand and clay, and attributed the continuation of settlement to cyclic imposed load variation, one of the cases reported being a tank that typically was unloaded and reloaded between 7 and 10 times a year. On some occasions when measurements have been made, settlements over twice the value predicted by one-dimensional consolidation theory have occurred by the end of a tank's service life, and tank design for settlement needs to be robust.

7.6 Contingency and remedial measures

7.6.1 General

This subsection outlines the various actions, other than the ground improvements or reinforcement discussed in subsection 7.3, that can be taken during the design, construction and testing of welded steel tanks in order to minimize settlement effects (Rinne 1963; and Clarke, 1971). These measures can create their own problems, and they should be carefully appraised before they are included in the project specification.

The timing and extent of remedial actions will be influenced by the tank type, safety considerations and cost. Although monitored settlement should not be allowed to continue to a stage where the tank itself is damaged, repeated small corrections for settlement may impose unnecessary stresses on the tank.

7.6.2 Tank dimensions

The simplest method of reducing settlement is to reduce the tank height, and increase its diameter to maintain the required capacity. There are economic penalties with this approach, including land cost, tank construction cost, and the increased volume of deadstock in the tank, which may be significant.

7.6.3 Bottom slope

An increased tank bottom slope can be employed to allow base consolidation to take place, perhaps during an extended water-test period. In cases for which reliable settlement predictions can be made, this method may be attractive, but may result in the need for remedial work on the tank bottom plating to correct the resultant local distortion after settlement has taken place.

7.6.4 Bottom plating

An increase in thickness of bottom plating, or an enhanced welding procedure, such as multi-pass welding, may be specified in an attempt to minimize the incidence of bottom leakage (e.g. Clarke, 1971). The normal tank bottom is intended to act as a liquid-tight membrane, fully supported by the foundation, and has insignificant structural strength. Increasing the thickness will increase this strength to some degree, but bottom plates may still fail if subjected to large local distortions because of foundation settlement.

7.6.5 Floating-roof tanks

In cases where significant differential settlement is thought likely, it can be beneficial to specify a greater than normal range for the seal movement to compensate for excessive shell ovalling. A wide-range seal may have a cost penalty, and may not perform as well as a more conventional unit.

7.6.6 Attached pipework

Tanks are generally very sensitive to external loads from pipework, and these have always to be kept to a minimum. It follows that if extensive settlement is predicted, increased flexibility of the piping system should be incorporated to minimize the risk of damage to the tank shell (see clause 7.4.3 – shell nozzles).

7.6.7 Tank jacking

When jacking of the tank shell ultimately may be required, it can be advantageous to install jacking brackets around the tank shell at the construction stage. The additional cost is small, but care needs to be taken to ensure that the detail used will not induce accelerated corrosion of the tank shell, or of the bracket itself.

Jacking can be used to repair settlement around the tank perimeter, or more general settlement across the tank base. In the former case, the shell can usually be jacked by a small amount to allow packing of the foundation material. For the general settlement condition on a large tank, jacks may be placed in locally stiffened internal holes and used to lift the bottom. It may sometimes be necessary to release the bottom plating from the shell to permit jacking to a height that will allow access for equipment and materials. The bottom plating itself would then be cut away where necessary to facilitate foundation repair.

7.6.8 Grouting

When localized settlement has taken place beneath the tank bottom plates, grout can be injected beneath the plates to fill the voids. Overpressurization, which could result in severe damage to the plates, has to be avoided. Future settlement should also be considered before grouting is undertaken. If further local settlement is thought likely, it may be better to cut away the bottom plates in the affected areas, and reinstate the foundation, rather than risk creating a foundation with widely varying stiffness characteristics, which could increase damage in the long term. Grouting introduces moisture and increases the possibility of bottom corrosion.

7.6.9 Tank removal and replacement

When gross settlement or a total foundation failure has taken place, it may be necessary to move the complete tank to allow rebuilding of the foundation (e.g. Leggatt & Bratchell, 1973). A small fixed-roof tank can often be lifted away as one unit. Larger tanks can be moved on air, rails or water, depending on the terrain to be traversed and the location of bunds suitable for flooding.

7.6.10 Tank rectification

It may be considered necessary to rectify damage to the tank, as differential settlement can produce localized plastic deformations in the shell plates. Ruptured bottom plates may also be required to be replaced. It is normal to perform a full hydrotest on the tank after such rectification work, which means that for most stored products the foundation will be subjected to stresses considerably in excess of those that caused the settlement damage. This fact should be borne in mind when the foundation repair specification is being prepared.

7.7 Performance monitoring

7.7.1 Purpose of monitoring

Monitoring is required to check that tank behaviour complies with design assumptions, so the position and type of measurement should be selected with due regard to the critical aspects of the design. Monitoring may be reduced to a minimum, or even in some cases omitted, if tank behaviour can be predicted accurately and the deformations are expected to be small. Satisfactory behaviour of other nearby tanks reduces the need for monitoring.

7.7.2 Measuring settlements

Traditionally, tank settlements are measured around their perimeter by surveyor's level and staff. Accurate results can be achieved only if brackets are provided on the tank to support the staff, and the levels can be related to a stable reference such as a deep datum. Simple steel brackets are often welded to the side plates within 300mm of the floor. Depending on tank diameter, between 8 and 32 measurement points normally are equally spaced along the perimeter. As recommended in clause 7.4.2, the spacing between settlement measurement points ideally should not exceed 6m. If the foundation response to test loading is critical, up to three surveyor's levels may be required so that measurements of all positions can be made essentially simultaneously to avoid errors during rapid settlement.

Floor settlements can be measured by a variety of remote reading devices. Water overflow settlement gauges of the type described by Penman & Watson (1965) can be installed in the base pad at a number of discrete points.

Levels along the length of several separate diameters can be measured by laying access tubes in the base pad along the desired diameters. A remote reading probe (McKenna & Roy, 1974) can be passed through the empty access tube and levels measured at appropriate distances. Alternatively, the ends of the access tubes can be elevated, the tubes filled with water, and the type of probe described by Bozozuk (1969) used to measure levels. To save time and avoid the labour of pulling probes through the access tubes, units have been placed at predetermined positions in access tubes (Penman & Charles, 1982).

Ground surface levelling external to the tank foundation has been useful in detecting the onset of plastic soil deformation during water testing. Such data also are useful in assessing interaction between closely spaced tanks, or between a tank and adjacent structures.

7.7.3 Shell ovality

Shell ovality of a steel floating-roof tank can be assessed by measuring the gap between the tank and roof. Usually the measurements are made at the same perimeter positions as the levelling brackets at the bottom. More detailed information about shell shape can be obtained by 3-dimensional surveying. Theodolite target points are welded to the tank walls, and proper reference stations set up for accurate positioning.

7.7.4 Excess pore pressures

Those responsible for the foundation design should decide where piezometers are to be placed to obtain the information required to ensure stability and assist in the study of foundation performance. The various types of piezometer and their installation are discussed by Hanna (1985). Where air or gas could be present in the soil, it may be best to use a fine-pored intake filter and a system that can be flushed. Twin-tube hydraulic piezometers (Bishop *et al,* 1960) have been used successfully under tanks. However, pneumatic and electrical piezometers that cannot be flushed should be satisfactory for typical soft-ground tank sites where there is a high water table.

Piezometric monitoring for water-fill control during tank testing is not always successful. The case history described by Sinclair & Cundall (1977) is particularly relevant. Despite detailed site investigation and extensive instrumentation, the instruments gave ambiguous readings, and the water test had to be extended by several months.

7.7.5 Lateral soil movements

Lateral spread of soil from under a tank can be measured using inclinometers placed vertically through the thickness of the weak soil around the tank perimeter. At least four should be used, and greater numbers will reveal a more detailed picture of foundation behaviour, particularly for large tanks. The instruments should be oriented for maximum sensitivity to movements on the line of tank radius.

Initial readings should be repeated several times at an early stage to ensure that the zero condition is accurately established. Usually it is not practical to install the access tubes until after tank construction because of danger of damage. However, sufficient time needs to be allowed for the tubes to be grouted in and settle down, and to be measured numerous times before test loading begins. Details on inclinometers and their use can be found in Hanna (1985).

7.8 Site investigation

It is usual to investigate the foundation soils to a depth at least 1.5 times the tank diameter. Strength and deformation parameters are required for the soil strata investigated over a radius equal to the tank diameter about the position for its centre. Where weak soil overlies firmer strata, sufficient information should be obtained to define accurately the shape of the surface of the lower strata, and to reveal any buried valley or large variations in the thickness of the weaker soil under the tank site. Heterogeneities in the soil profile should also be identified.

Sufficient laboratory and/or *in situ* tests should be performed on the weaker soils encountered within the stated area and depth to define reasonably the soil plasticity characteristics, maximum past vertical stress, compression and recompression indexes, and the depth profile of undrained shear strength. For soft ground sites where the computed safety factor on bearing is less than 1.5, laboratory testing should include appropriate means of accounting for sample disturbance. A method of evaluating soil strength is given by Ladd & Foott (1974).

For important tanks on potentially difficult sites, a competent engineer or engineering geologist should be present during the site investigation in order to assure the quality of the work.

7.9 Design codes

7.9.1 General

There are various guidance documents for tank design and construction. For the petroleum industry, believed representative of good practice, conventional steel tanks are covered by BS 2654 (1984). BS 4741 (1971) and BS 5387 (1976) provide guidance for single- and double-wall low-temperature steel tanks. BS 8007 (1987) deals with reinforced and prestressed concrete tanks, while FIP/2/3 (1978) and FIP/2/6 (1982) give recommendations for ambient and low-temperature prestressed concrete tanks. The equivalent American codes are API Standards 620 and 650 for steel tanks and ACI 67-40 (1970) for prestressed concrete. The Engineering Equipment and Materials Users Association (1986) document gives design and safety recommendations for storage of refrigerated and liquefied gases.

The guidance provided for foundations reflects their importance for the successful performance of the type of tank being considered. The treatment varies in the amount of information provided, and in the degree to which tank and foundation design considerations are integrated. The clauses below give a brief resume and critique of the salient points of the codes with respect to soil–structure interaction.

7.9.2 Concrete tanks

Ambient temperature

The documents for ambient-temperature tanks points out that the ground will yield and that the resulting stress redistribution should be accounted for in the structural design. FIP/2/3 (1978) details some highly idealized tank deformation modes for which there is no supporting evidence in the literature. The associated recommendations relating shell movement to settlement are necessarily simplistic and unlikely to be correct. There is, however, a paucity of information from which to develop more realistic criteria.

Low temperature

FIP/2/6 (1986) strongly emphasizes the need to restrict differential settlements within tolerable bounds for tank insulation systems. While no firm recommendations are made, it is stated that a maximum change in slope of 1/500 between any two points on the base has been specified in some cases. Resort to piling or ground stabilization is recommended to limit differential settlements, and it is advised that ground movements be considered in the structural design.

7.9.3 Steel tanks

Ambient temperature

Reflecting industry practice for conventional tanks, BS 2654 (1984) has traditionally separated the tank and foundation design considerations. Little discussion on foundations has been included in the main text, with most of the commentary being placed in an appendix. The standard states that the tank bottom is to be fully supported by a foundation provided by the tank purchaser. In a significant improvement, the 1984 revision introduced into the foundation appendix the concept that limits for permissible settlement are to be agreed between the purchaser and tank manufacturer. The appendix outlines in a general way tank performance considerations affecting permissible settlements, but no specific settlement criteria are indicated.

BS 2654 gives only minimum requirements for structural design of the tank bottom, which is the part of a conventional tank most vulnerable to overstress by settlement. Some measures to enhance tank-bottom performance are indicated, but there is no adjustment of the bottom fabrication specification for tanks that will be placed on highly compressible ground. There is no mention of contingency measures such as fabricating shell-bottom course plates with provision for jacking lugs, or of overbuilding the tank pad to accommodate large settlements.

Low temperature

In contrast with BS 2654 for conventional tanks, BS 4741 (1971) for low-temperature tanks established some time ago the need to consider the tank and foundation designs jointly. This difference reflects the relative sensitivity to ground movements of conventional and insulated tanks. The BS 4741 foundation appendix identifies varying types of differential settlement and specifies limiting values. Allowable angular distortions are given for the whole tank (tilting), the bottom (edge-to-centre) and along the perimeter. The criteria are stringent but perhaps not unreasonably so, given the importance of limiting the differential settlements of insulated tanks. However, specification of out-of-plane perimeter settlements rather than absolute values would be more rational. Moreover, as tank technology changes, the value of establishing settlement criteria in such detail might be questioned.

7.10 Project management

7.10.1 Project organization

The way in which tank projects are realized depends on the purchaser's organization and the tank function, type and size. Most tanks of any significant size store purchaser's products, and the purchasers have a continuing need for new tankage. Hence, organization of tank projects is intrinsically different from, say, that of building structures. Tank purchasers are, to varying degrees, knowledgeable about tanks, often directly managing the project rather than employing the services of a consulting engineer. This arrangement is most likely in the common situation of a project involving a small number of conventional steel tanks.

The purchaser managing a project employs separate specialist firms to design the tank and the foundation. This is reflected in BS 2654 (1984), which states that the tank foundation is normally to be provided by the purchaser. Tank suppliers essentially design a container, are not expert in foundation matters and typically have no contractual responsibility for the foundation.

7.10.2 Functions of participants

With the arrangement outlined above, the burden of project management, including the anticipation of problems, rests with the tank purchaser. Problems involving differential settlement are often associated with large tanks that are built for large organizations in the petrochemical industry. As these companies have both mechanical and civil specialists, responsibilities for the tank and for its foundation still tends to be separated. Smaller organizations with fewer, if any, specialist personnel may not have difficulties with fragmentation of responsibility, but there is a reduced awareness of potential problems to a level that may be incompatible with the project-management role.

The role of purchaser as project manager is reflected in the literature where typically it is client engineers who have proposed measures to improve tank performance on compressible ground (e.g. Rinne, 1963; Clarke, 1971; Guber, 1974; and Langeveld, 1974). Consultants generally have confined their published contributions to studies of tank behaviour, particularly the interpretation of measurement data, reflecting their role as analysts rather than engineers-in-charge. Manufacturers, with minimal responsibility for foundations, have not contributed to the literature on settlement behaviour of tanks. Their views on tolerable settlements, as seen in project documents, are conservative compared with published recommendations based on performance observations.

7.10.3 Cryogenic projects

A consulting engineer or a managing contractor usually is appointed for large refrigerated liquid developments. However, process plant considerations dominate selection of facilities from the designs being offered, and the tankage may have a low priority. Lump-sum contracts can inhibit adjusting foundations and tank details to improve performance on compressible ground.

7.10.4 Construction aspects

Foundation-related difficulties have arisen from an over-reliance on accuracy in geotechnical predictions, over-optimism of ground-improvement specialists, and poor construction supervision, as well as weakness in project organization and management. Quality in construction supervision and good liaison between design office and site are important. The installation of minor tank equipment intruding into the body of the foundation has led to problems. The designer should approve even minor site changes to the foundation detailing.

The separation of responsibility for the tank and its foundation is a problem that can be overcome by close interaction between engineers of differing disciplines, particularly within purchaser's organizations. The foundation engineers' input can come too late, the tank design being established in isolation from foundation considerations. Measures may then have to be adopted to achieve tank-site compatibility, assuming that potential problems are indeed recognized. Design codes could encourage integration of civil and mechanical engineering input; guidance on the effect of foundation settlements on the tank should be integrated into the discussion of bottom, shell and roof design as appropriate.

References

ACI 67-40 (1970): *Design and construction of circular prestressed concrete structures,* American Concrete Institute

Adams, N. J. I. (1985): 'A theoretical and experimental investigation of stress and stiffness coefficients for low entry nozzles in cylindrical storage tanks', *Int. Jnl. of Pressure Vessels & Piping,* **19**, no. 2

API 620 (1982): *Recommended rules for design and construction for large welded low-pressure storage tanks,* 7th ed., revision 1, American Petroleum Institute, 1985

API 650 (1980): *Welded steel tanks for oil storage,* 7th ed., revision 1, American Petroleum Institute, 1984

Bell, R. A., & Iwakiri, J. (1980): 'Settlement comparison used in tank failure study', *Journal, Geotechnical Engineering Division, ASCE,* **106**, no. GT2, p. 153

Belloni, L., Barassino, A., & Jamiolkowski, M. (1975); 'Differential settlements of petroleum steel tanks', *Proc. conference on settlement of structures,* British Geotechnical Society, p. 323

Bishop, A, W., Kennard, M. F., & Penman, A. D. M. (1960): 'Pore pressure observations at Selset dam', *Conf. on pore pressure and suction in soils,* British Geotechnical Society, p. 91

Bjerrum, L. (1966): 'Secondary settlements of structures subjected to large variations in live load', *Proc. int. symposium on rheology & soil mechanics,* Grenoble, Springer, p. 460

Bjerrum, L., & Overland, A. (1957): 'Foundation failure of an oil tank in Fredrikstad, Norway', *4th int. conf. SM&FE,* London, **1**, p. 287

Bozozuk, M. (1969): 'A fluid settlement gauge', *Canadian Geotech. Jnl.,* **6**, no. 3, p. 362

Brooker, E. W., & Ireland, H. O. (1965): 'Earth pressure at rest related to stress history', *Canadian Geotech. Jnl.,* **2**, no. 1, p 1

BS 2654 (1984): *Specification for vertical steel welded storage tanks with butt-welded shells for the petroleum industry,* British Standards Institution, London

BS 4741 (1971): *Specification for vertical cylindrical welded steel storage tanks for low-temperature service: single wall tanks for temperatures down to −50°C,* British Standards Institution, London

BS 5387 (1976): *Specification for vertical steel welded storage tanks for low-temperature service: double wall tanks for temperatures down to −196°C*, British Standards Institution, London

BS 5500 (1985): *Unfired fusion welded pressure vessels*, British Standards Institution, London

BS 8007 (1987): *British Standard Code of Practice for the design of concrete structures for retaining aqueous liquids*, British Standards Institution, London

Burland, J. B., & Wroth, C. P. (1975): 'Settlement of buildings and associated damage', *Proc. conference on settlement of structures*, British Geotechnical Society, p. 611

Carlson, E. D., & Fricano, S. P. (1961): 'Tank foundations in eastern Venezuela', *Jnl. Soil Mechanics & Foundations Division, ASCE*, **87**, no. SM5, p. 69

Clarke, J. S. (1971): 'How to handle tank bottom and foundation problems', *Oil & gas journal*, **5**, p. 82

Clausen, C. J. F., Graham, J., & Wood, D. M. (1984): 'Yielding in soft clay at Mastemyr, Norway', *Geotechnique*, **34**, no. 4, p. 581

Craig, H. A. (1975): Discussion on storage tanks, *Proc. conference on settlement of structures*, British Geotechnical Society, p. 789

Darragh, R. (1964): 'Controlled water tests to preload tank foundation', *Jnl. Soil Mechs & Found. Div., ASCE*, **90**, no. SM5, p. 303

Davis, E. H., & Poulos, H. G. (1968): 'The use of elastic theory for settlement prediction under three-dimensional conditions', *Geotechnique*, **18**, p. 67

D'Appolonia, D. J., Poulos, H. G., & Ladd, C. C. (1971): 'Initial settlement of structures on clay', *Jnl. Soil Mechanics & Foundations Division, ASCE*, **97**, no. SM10, p. 1359

de Beer, E. (1969): 'Foundation problems of petroleum tanks', *Jnl. Belgian Petroleum Insititute*, no. 6, p. 25

DIN 4119 (1979): *Above ground cylindrical flat-bottom tank installations of metallic materials*, Deutsches Institut für Normung

D'Orazio, T. B., & Duncan, J. M. (1982): *The behaviour of steel oil storage tanks on compressible foundations*, Department of Civil Engineering, University of California, Berkeley, Report no. UCB/GT/82-06

Duncan, J. M., & D'Orazio, T. B. (1984): 'Stability of steel oil storage tanks', *Jnl. Geotech. Eng. Division, ASCE*, **110**, no. 9, p. 1219

Engineering Equipment & Materials Users' Association (1986): *Recommendations for the design and construction of refrigerated liquified gas storage tanks*, publication no. 147

Esrig, M. I., Ahmad, S., & Mayo, H. C. (1975): *Settlement of ammonia storage tanks*, Ammonia Plant & Safety, American Institute of Chemical Engineers, **17**, p. 93

FIP/2/3 (1978): *Guide to good practice; basic reinforced concrete and prestressed concrete construction*, FIP

FIP/2/6 (1982): *Guide to good practice: demolition of reinforced and prestressed concrete structures*, FIP

Foott, R., & Ladd, C. C. (1981): 'Undrained settlement of plastic and organic clays', *Jnl. Geotech. Eng. Division, ASCE*, **197**, no. GT8, p. 1079

Green, P. A., & Hight, D. W. (1975): 'The failure of two oil-storage tanks caused by differential settlement', *Proc. conf. settlement of structures*, Cambridge, Pentech Press, London, p. 353

Greenwood, D. A. (1975): 'Differential settlement tolerance of cylindrical steel tanks for bulk liquid storage', *Proc. conference on settlement of structures*, British Geotechnical Society, p. 35

Greenwood, D. A., & Kirsch, K. (1983): 'Specialist ground treatment by vibratory and dynamic methods', *Conf. on piling and ground treatment*, ICE, p. 17

Guber, F. H. (1974): 'Design engineering contributions to quality tankage', *Proc. annual meeting of the Institute of Welding*, Section on welding of storage tanks for the oil and gas industries, p. 99

Hanna, T. H. (1985): *Field instrumentation in geotechnical engineering*, Trans. Tech. Publications

Hegg, U., Jamoilkowski, M. B., Lancellotta, R., & Parvis, E. (1983): 'Performance of large oil tanks on soft ground', *Proc. conference on piling & ground treatment*, ICE, p. 125

Jamiolkowski, M. (1975): Discussion in *Proc. conference on settlement of structures*, British Geotechnical Society, p. 796

Jardine, R. J., Symes, M. J., & Burland, J. B. (1984): 'The measurement of soil stiffness in the triaxial apparatus', *Geotechnique*, **34**, p. 323

Ladd, C. C., & Foott, R. (1974):'New design procedure for stability of soft clays', *Jnl. Geotech. Eng. Div., ASCE*, **100**, no. GT7, p. 763

Langeveld, J. M. (1974): 'The design of large steel storage tanks for crude oil and natural gas', *Proc. annual meeting of the International Institute of Welding*, Section on welding of storage tanks for the oil and gas industries, p. 35

Leggatt, A. J., & Bratchell, G. E. (1973): 'Submerged foundations for 100,000 ton oil tanks', *Proc. ICE*, part 1, **54**, p. 291; Discussion, p. 699

Leonards, G. A. (1982): 'Investigation of failures', *Jnl. Geotech. Eng. Div., ASCE*, **108**, no. GT2, p. 187

Liu, T. K., & Dugan, J. P. (1975): 'Elastic settlement behaviour of three steel storage tanks, *Proc. conf. on settlement of structures*, British Geotechnical Society, p. 110

Malik, Z., Morton, J., & Ruiz, C. (1977): 'Ovalisation of cylindrical tanks as a result of foundation settlement', *Jnl. of Strain Analysis*, **12**, no. 4, p. 339

Marr, W. A., Ramos, J. A., & Lambe, T. W. (1982): 'Criteria for settlement of tanks', *Jnl. Geotechnical Engineering Division, ASCE*, **18**, no. GT8, p. 1017

Mayne, P., & Kulhawy, F. (1982): 'K_o-OCR relationships in soil', *Jnl. Geotech. Eng. Div., ASCE*, **108**, no. GT6, p. 851

McKenna, J. M., & Roy, M. (1974): 'Performance of the instruments used to monitor the M5 motorway embankment built on soft ground between Edithmead and Huntworth', *Proc. conf. on field instrumentation*, British Geotechnical Society, p. 362

Penman, A. D. M. (1977): 'Soil–structure interaction and deformation problems with large oil tanks', *Proc. int. symposium on soil–structure interaction*, University of Roorkee, India, **1**, p. 521

Penman, A. D. M., & Charles, J. A. (1982): 'An improved horizontal plate gauge', *Geotechnique*, **32**, no. 3, p. 277

Penman, S. D. M., & Watson, G. H. (1965): 'The improvement of a tank foundation by the weight of its own test load', *Proc. 6th int. conf. on SM&FE*, Montreal, **2**, p. 169

Penman, A. D. M., & Watson, G. H. (1967): 'Foundations for storage tanks on reclaimed land at Teesmouth', *Proc. ICE*, **37**, p. 19

Poulos, H. G., & Davis, E. H. (1974): *Elastic solutions for soil and rock mechanics*, John Wiley

Rinnie, J. E. (1963), 'Tanks on soft soils are economic challenge', *Petroleum management*, **35**, no. 10, p. 206

Roberts, D. V. (1961): 'Foundations for cylindrical storage tanks', *Proc. 5th int. conf. SM&FE*, Paris, **3**, p. 785

Rosenberg, P., & Journeaux, N. L. (1982): 'Settlement limitations for cylindrical steel storage tanks', *Canadian Geotech. Jnl.*, **19**, no. 3, p. 232

Saurin, B. F. (1949): 'Tests on a large surface foundation', *Geotechnique*, **1**, no. 4, p. 272

Sinclair, T. J. E., & Cundall, P. A. (1977), 'The prediction of soil displacements and design of controlled water test programme for ground supported tanks on soft soils', *Proc. conf. on large ground movements and structures*, Cardiff, Pentech Press, p. 272

Skempton, A. W. (1951): 'The bearing capacity of clays', *Proc. Building Research Congress*, ICE, Division 1, p. 180

Skempton, A. W., & Bjerrum, L. (1957): 'A contribution to settlement analysis of foundations on clay', *Geotechnique*, **7**, no. 4, p. 168

Sullivan, R. A., & Nowicki, J. F. (1975): 'Differential settlements of cylindrical oil tanks', *Proc. conference on settlement of structures*, British Geotechnical Society, p. 402

Watson, G. H., Crooks, J. H. A., Williams, R. S., & Yam, C. C. (1984): 'Performance of preloaded and stage-loaded structures on soft soils in Trinidad, *Geotechnique*, **34**, no. 2, p. 239

Wood, D. M. (1980): 'Yielding in soft clay at Backebol, Sweden', *Geotechnique*, **30**, no. 1, p. 49

Part II:
Ground supported by structures

8 Fundamentals

Soil–structure interaction is particularly important to ground supported by structures because the soil generates the loading as well as providing the resistance to load. The structure transfers the load generated by the retained soil so that it is resisted elsewhere in the soil mass. The manner in which this transfer occurs depends, among other things, on the type of support structure, its relative stiffness and the method by which it is constructed.

In many cases, the actual working stresses in the ground and the supporting structural members depend significantly on the relative stiffnesses and displacements of these elements. Often it is possible to carry out satisfactory design calculations without regard for this fact: however, consideration of the interaction of stiffness and deformation is sometimes essential to an adequate assessment of stresses, forces and bending moments in the supporting structure.

Ground support structures may be involved in interaction with the ground in various other ways such as:

- interaction of permeability if the structure acts as a dam or a drain (this may in turn lead to additional ground movements)
- chemical interaction, especially where chemicals in the ground attack the structure
- heat interaction, usually caused by abnormal temperature in the structure.

However, the considerations in this report are limited to situations in which stiffness, deformation or movement of the structural system and ground are important in determining the behaviour of the structure in working conditions.

Two aspects of ground behaviour are of major importance to soil–structure interaction of ground support structures:

- the presence of porewater in the soil, and its pressure and movement through the soil mass
- the deformation characteristics of the soil skeleton, including its ultimate failure in shear.

8.1 Porewater pressure

The behaviour of saturated soil is governed by the principle of effective stress enunciated in standard soil mechanics texts. Generally, increases in water pressure or water content lead to a reduction in stiffness and loss of shear strength. In saturated soils the time-scale for such changes is governed by the bulk permeability of the ground. Thus in soils of low permeability, such as silts and clays, the period for full adjustment may extend over many years or decades.

8.2 Deformation characteristics

For design purposes, it is sometimes unnecessary to consider the stress/strain characteristics of a soil, apart from its shear strength.

Shear strength properties can normally be represented in effective stress terms by cohesion, c', and angle of shearing resistance ϕ'. In clays, a total undrained shear strength may be used. However, some aspects of soil–structure interaction are sensitive to the details of deformation behaviour, the more important of which are summarized here.

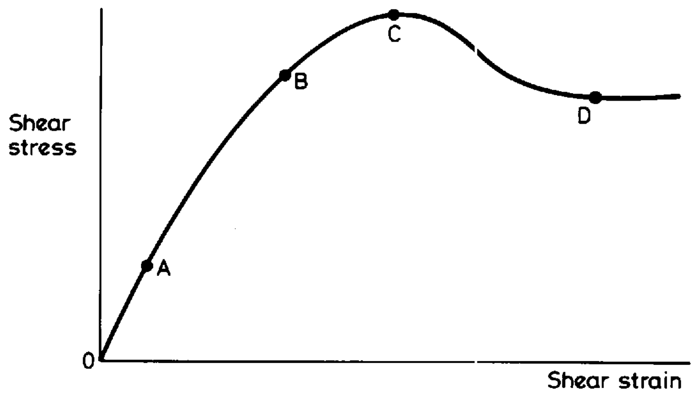


Fig. 33 Typical stress/strain curve in shear

Fig. 33 shows a stress/strain curve for a soil deformed in shear at constant pore pressure. Initially, along the path OAB, the soil deforms in a manner that can be approximated using linear elasticity. At point B, plastic yield becomes significant, and a peak shear stress is attained at point C. As strains increase further, reduction in strength may occur to point D, a 'critical state' condition, and at very large strains the strength may fall further to a residual value. Residual strengths are particularly relevant to clays that have previously failed in shear. Many soils exhibit some but not all of the features outlined here.

In the working state, much of the soil experiences strains limited to the range OAB. Although this is approximately linear in some cases, it is more generally a continuous curve, and the strains are not completely elastic. Recent research has indicated that many soils exhibit a very high stiffness in the range of very small strains OA, and this can have major implications in analysis of soil–structure interaction (Jardine *et al*, 1984).

In zones of soil that are highly stressed, the peak strength may be exceeded locally, and this can lead to progressive failure as the zone of soil strained beyond peak gradually extends.

Volumetric deformations are also significant in the working state, but in clays they may be inhibited or delayed by porewater pressures. Elastic models are sometimes adequate to represent volumetric behaviour, but in soft clays, volumetric plastic yield is also significant (Atkinson & Bransby, 1978).

8.3 *In situ* stresses

The earth pressures action on ground support structures depend on the initial lateral stresses acting in the ground, the stiffness of the structure and its method of construction.

A knowledge of the initial stress state in the ground is therefore required for predictions of behaviour in working conditions (Potts & Burland, 1983; and Pappin *et al*, 1986). The horizontal stresses are governed by the previous stress history of the deposit as discussed by Burland *et al* (1979) and Simpson *et al* (1981).

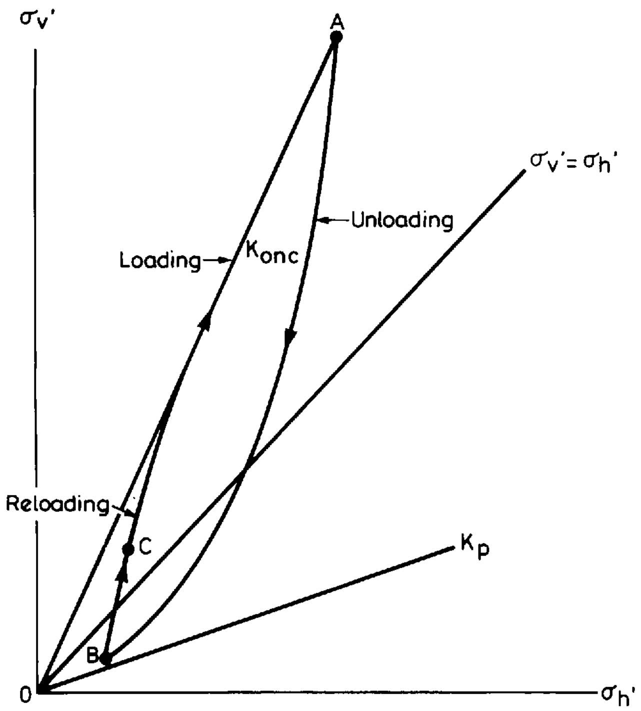


Fig. 34 Effective stress paths during deposition, erosion and reloading
Based on Henkel (1970)

Fig. 34 shows the stress path followed by a clay soil during loading, unloading and reloading. During deposition of the overlying deposits along path OA, the material is normally consolidated and the earth pressure coefficient K_o has a constant value K_{nc}. The value of this coefficient is given by the approximate equation $K_{nc} = 1 - \sin\phi'$ derived from the work of Jaky (1944). During erosion of the overlying deposits the vertical effective stress decreases more rapidly than the horizontal effective stress so that the value of K_o increases towards the passive value K_p along path AB. The net result is that in heavily overconsolidated deposits the value of K_o lies close to the passive value at shallow depths and then decreases progressively with depth towards the normally consolidated value (Skempton, 1961; and Bishop *et al* 1965). Estimation of the K_o value in such deposits can be obtained from a knowledge of the overconsolidation ratio (OCR) using an equation of the form $K_o = K_{nc}\,\mathrm{OCR}^n$ where n depends on the plasticity of the clay and OCR is the ratio of the maximum past to the current vertical effective stress (Ladd *et al*, 1977; Brooker & Ireland, 1965; and Wroth, 1975).

Use of this equation is not valid if any subsequent reloading of the deposit has occurred, since the horizontal stress then increases more slowly than the vertical stress as indicated by the path BC. The presence of even relatively thin layers of more recent deposits or reductions in the groundwater level therefore have a pronounced effect on the value of K_o at the shallow depths of relevance in the design of earth retaining structures (Burland *et al*, 1979). However, since the horizontal stress changes by only small amounts during reloading, its value may be assessed with greater confidence from the loading, unloading and reloading sequence in deposits where the geological history is well defined.

An estimation of the *in situ* horizontal stresses can also be obtained from field measurements (Tedd & Charles, 1983; and Windle & Wroth, 1977) or from laboratory tests on undisturbed samples (Burland & Maswoswe, 1982).

In many situations reliable assessment of the *in situ* stress conditions is nevertheless difficult to achieve, and this may in turn limit the accuracy of predictions.

References

Atkinson & Bransby (1978): *The mechanics of soils*, McGraw Hill, London

Bishop, A. W., Webb, D. L., & Lewin, P. I. (1965): 'Undisturbed samples of London clay from the Ashford Common shaft: strength effective stress relations', *Geotechnique*, **15**

Brooker, E. W., & Ireland, H. O. (1965): 'Earth pressures at rest related to stress history', *Canadian Geotech. Journal*, **2**, 1

Burland, J. B., & Maswoswe, J. (1982): Discussion, *Geotechnique*, **32**, p. 285

Burland, J. B., Simpson, B., & St. John, H. D. 'Movements around excavations in London clay'. *Proc. 7th Euro. conf. SM&FE*, Brighton, **1**, p. 13

Jaky, J. (1944): 'The coefficient of earth pressure at rest', *Jnl. of the Soc. of Hungarian Architects & Engineers*, **78**, 22

Ladd, C. C., Foott, R., Ishihara, K., Schlosser, F., & Poulos, H. G. (1977): 'State of the art report: stress-deformation and strength characteristics', *Proc. 9th int. conf. SM&FE*, Tokyo, **2**, p. 421

Pappin, J. W., Simpson, B., Felton, P. J., & Raison, C. (1986): 'Numerical analysis of flexible retaining walls', *Proc. symp. computer applications in geotech. eng.*, Midland Geotechnical Soc., Birmingham

Potts, D. M., & Burland, J. B. (1983): *A numerical investigation of the retaining walls of the Bell Common tunnel*, TRRL suplementary report 783

Simpson, B., Calabresi, G., Sommer, H., & Wallays, M. (1981): 'Design parameters for stiff clays', General report, *Proc. 7th Euro. conf. SM&FE*, Brighton (1979), **5**, p. 91

Skempton, A. W. (1961): Horizontal stresses in an overconsolidated Eocene clay', *Proc. 5th int. conf. SM&FE*, Paris 1

Tedd, P., & Charles, J. A. (1983): 'Evaluation of push-in pressure cell results in stiff clay', *Int. symp. on soil and rock investigations by* in situ *testing*, Paris, 2

Windle, D., & Wroth, C. P. (1977): '*In situ* measurements of properties of stiff clays', *Proc. 9th int. conf. SM&FE*, Tokyo, 1

Wroth, C. P. (1975): '*In situ* measurements of deformation characteristics', *Proc. speciality conf. on* in situ *measurements of soil properties*, ASCE, Raleigh

9.1 Introduction

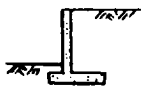
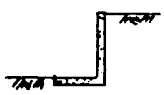
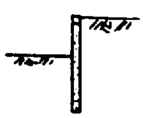
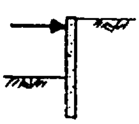
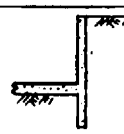
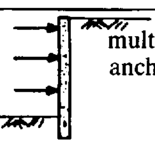
The purpose of an earth retaining wall is generally to withstand the lateral forces exerted by a vertical or near vertical surface in natural ground or fill. The structural system usually includes a wall, which may be supported by other structural members such as props, floor slabs or ground anchors. Alternatively, or additionally, the wall may be supported by ground at its base or into which it penetrates. The scope of this Section includes all types of walls retaining natural ground or fill used in situations that may be regarded as either temporary or permanent. Walls used in bridge abutments are not included, however, but are discussed in Section 5.

9.2 Types of retaining structure requiring consideration of soil–structure interaction

9.2.1 General

The complexity and uncertainty involved in design increases with the degree of soil–structure interaction and thus depends on the type of structure to be employed. It is therefore appropriate to categorize the types of retaining structure on the basis of the soil–structure interaction problems that arise in design. In Table 7 the main wall types are listed in order of increasing complexity of soil–structure interaction.

Table 7 Effects of soil–structure interaction in various types of retaining structure

types	construction	typical use	effects of soil–structure interaction
 gravity wall	stone walls concrete walls with soil backfilled behind	retention of fill, embankments, etc.	Compaction of fill leading to higher earth pressures and structural stresses
 counterfort or buttress wall	crib walls gabbion walls	small excavations	ground movements due to compressible soil beneath wall leading to rotation of wall
 cantilever wall			ground movements during construction when soil may be temporarily unsupported
 reverse cantilever wall			
 embedded cantilever wall	steel sheet piles (either driven)	small excavations	↕ **minor** Effect of soil–structure interaction on structural stresses **major**
 propped or anchored cantilever wall	box piles or secant piles	shallow basements cut and cover tunnels	structural stresses affected by: • relative stiffness of wall, props and soil
 cantilever wall propped at excavation level	bored/ hand dug contiguous piles	depressed road cuttings	• development of active and passive pressures in relation to wall movement • initial (*in situ*) earth pressures
 multi-propped anchored wall	or piles with intermediate panels excavated diaphragm walls	deep basements, road cuttings, etc.	ground movements sensitive to stiffness of structure and soil both adjacent to and beneath excavation

9.2.2 Non-embedded walls

Gravity, counterfort and cantilever walls are stiff structures for which the soil–structure interaction is relatively simple. For overall stability, the earth forces on the back of a wall have to be balanced by normal and shear stresses at its base. Movement arising from deformation of the ground beneath the wall is often negligibly small, but exceptions occur if the wall and backfill are constructed on a deep layer of compressible soil (e.g. Cole, 1980). Rotation of the base may lead to large movements at the top of the wall.

For counterfort and cantilever walls account has to be taken of stresses induced in the structural elements by any mechanical compaction of the backfill. If they are used to provide long-term support for excavations, it is likely that the excavated face or slope will be temporarily unsupported during construction. This may lead to significant ground movements that could affect adjacent structures.

Reverse cantilever walls are sometimes used to provide permanent support to excavations. To facilitate contruction some form of temporary support, such as a sheet pile wall, is often required during construction. Thus the engineer may be faced with the design of two different types of retaining structure.

If props or anchors are used to aid stability of gravity, counterfort or cantilever walls, the soil–structure interaction increases in complexity, and some of the comments given below for embedded cantilever walls are relevant.

9.2.3 Embedded walls

When soil movements are important and/or construction space is limited, embedded cantilever walls may be used, with or without props or anchors. To maintain stability these walls rely on the resistance of the ground below excavation level and on the forces provided by any props or anchors employed.

The flexibilities of embedded walls vary within a wide range, and this has considerable effect on the distribution of earth pressures. A flexible wall may be constructed using driven steel sheet piles or box piles, for example, and much stiffer walls may be formed from bored piles or by using the diaphragm-wall technique. The more flexible walls will often have smaller bending moments in the structural elements but may also lead to significantly larger deformations, particularly for embedded cantilevers with no props or anchors.

The complexity of the soil–structure interaction increases with the number of rows of props or anchors and hence with increasing structural redundancy.

Embedded walls are often used in conjunction with deep excavations for which ground movements around and beneath the excavation may be critical. The effects of these movements on adjacent structures must be considered. Heave beneath the excavation may also be significant, and in extreme cases may lead to instability.

9.2.4 Short- and long-term conditions

For permanent structures in clayey soils it is essential that account is taken in design of conditions in both the short-term, during and immediately after construction, and in the long-term when full equilibrium has been achieved. Which of these proves the more critical depends on whether the ground has been subjected to a net increase or decrease in stress by the construction of the retaining wall. For example, the critical condition of stability of a cantilever or gravity wall retaining granular backfill and founded on a soft clay subsoil is likely to occur in the short-term, while vertical deformations and settlements will be greater in the long term. Conversely, for an *in situ* wall embedded in stiff clay the stability is likely to decrease, and lateral deformations are likely to increase with time. This is associated with softening and swelling of the ground that occurs in response to the reduction in stress caused by excavation in front of the wall.

In cohesive soils the earth pressures and deformations occurring in the short term are frequently assessed from a total stress calculation using undrained soil properties. An inherent assumption of this approach is that no change in these properties occurs during the construction period. For many cohesive soils that contain discontinuities or more permeable seams within their mass, this condition is unlikely to prevail in practice. Particular care is therefore necessary in the use of such methods where a deterioration in soil properties with time is anticipated, such as in the case of temporary works involving an *in situ* wall in stiff fissured clay (Skempton & La Rochelle, 1965).

The long-term earth pressures and deformations in cohesive soil are calculated from an effective stress analysis. Such analyses can also be used to assess conditions in the short term but require information on the porewater pressure regime in the ground, which is more difficult to assess in the transitory state prior to the establishment of an equilibrium pattern of groundwater flow.

In cohesionless soils of high permeability, such as sands and gravels, the response of the ground to stress changes induced by the construction is substantially immediate, and effective stress analyses are therefore appropriate for the design of both permanent and temporary structures.

For many earth-retaining structures an important component of the loading is derived from groundwater. A rise in the water table generally has a pronounced and adverse effect on wall stability by increasing the water loading and decreasing the available shear resistance of the soil. The designer has therefore to identify the most unfavourable combination of flow conditions likely to be experienced by the structure during its life. Such conditions would include, for example:

- the presence of artesian water pressures beneath a cohesive layer
- restrictions in natural groundwater flow paths introduced by wall penetration into less permeable strata
- rapid drawdown conditions for waterfront structures retaining cohesive soil
- rises in water levels behind retaining structures caused by flooding or fractures of water mains.

Where stiff cohesive backfills are used behind walls, suctions developed within the soil during placement and compaction can lead to reduced earth pressures in the short term. Account has to be taken in design of the increase in total thrust that may take place in the longer term as porewater pressures within the fill rise towards an equilibrium condition.

Soil cannot sustain significant tension. Thus, in cohesive soils, any tendency for movement of a wall away from the retained ground or for shrinkage of the soil is likely to lead to the formation of cracks within the potential zone of tensile stress. The design therefore needs to consider the possibility of enhanced water pressures acting on the structure because of ingress of surface water into such cracks.

9.3 Earth pressures

9.3.1 Limiting active and passive pressures

As a wall moves towards a collapsed state, zones will be formed in the adjoining ground within which the soil is in a state of plastic equilibrium. Within these zones the direc-

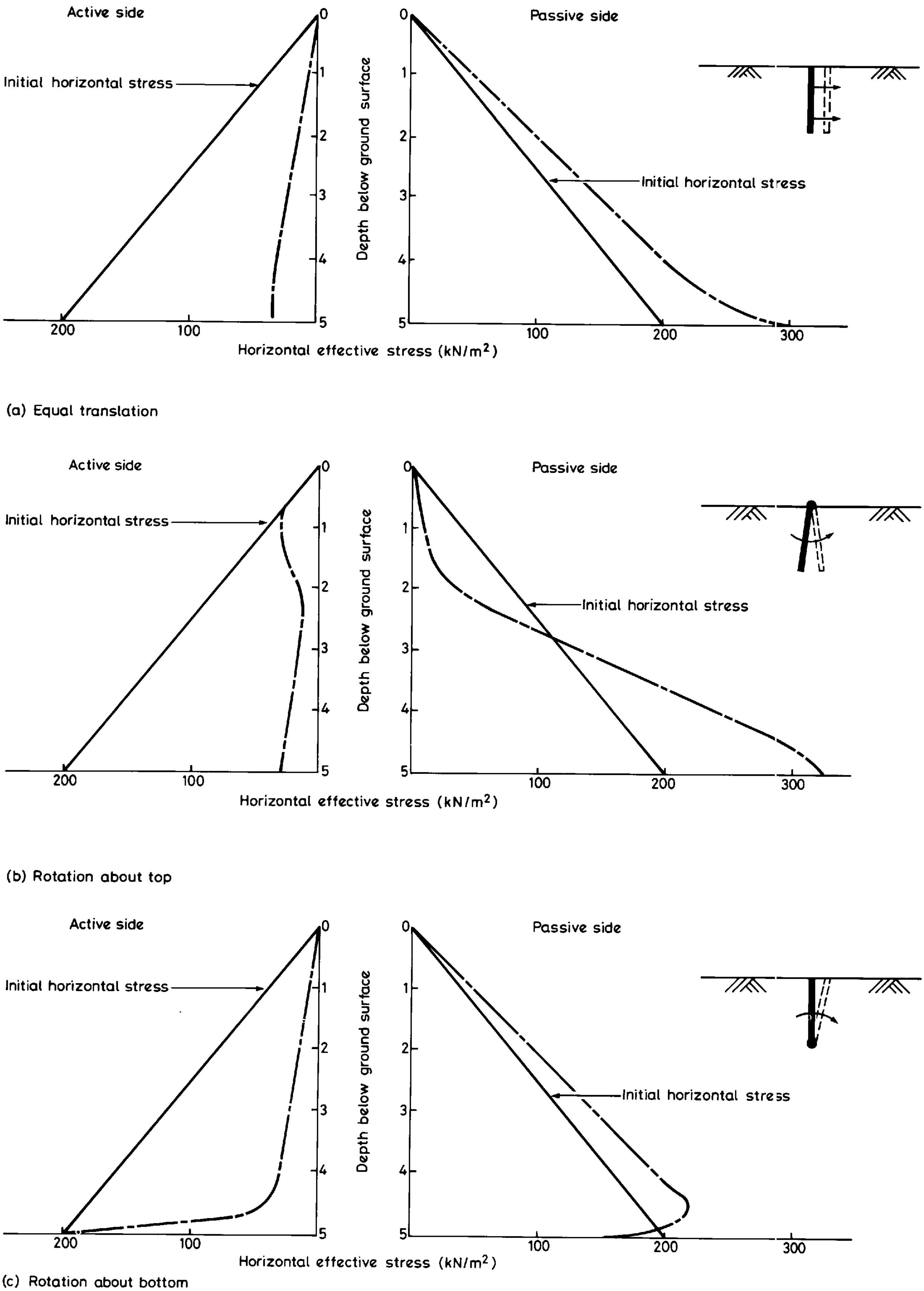


Fig. 35 Active and passive stress distribution for a smooth wall

Based on Potts and Fourie (1986)

tion of shear is such that the soil thrust on the wall is a minimum within the active zones where movement of the wall is away from the soil, and a maximum within the passive zones where movement of the wall is towards the soil. The shapes and extent of these failure zones are greatly influenced by adhesion or friction developed between the soil and the wall, by the inclination of the wall and the slopes of the ground.

For uniform soils in simple situations it is often assumed in design that active and passive earth pressures increase linearly with depth. In this case the effective stresses acting normal to a wall at depth z can be calculated using equations of the following form:

$$p'_a = K_a(\gamma z + q - u) - c' K_{ac} \text{ for } p_{a'} > 0 \quad \text{.......(1)}$$

$$p'_p = K_p(\gamma z + q - u) + c' K_{pc} \quad \text{...................(2)}$$

where γ is the bulk unit weight of the soil
q is a uniformly distributed surcharge pressure
u is the porewater pressure at depth z
K_a, K_{ac} are the active pressure coefficients
K_p, K_{pc} are the passive pressure coefficients

On this basis, the limits on total stress normal to the wall may be written as:

$$p_a = p'_a + u \quad \text{..............................(3)}$$

$$p_p = p'_p + u \quad \text{.............................(4)}$$

Values of the earth pressure coefficients for use in these equations are given in standard references and codes of practice for a range of boundary and geometric conditions (Caquot & Kerisel, 1948; CP 2, 1951; and NAVFAC, 1971). The values are dependent on soil properties and also on the relative vertical movements between soil and wall which govern wall adhesion and friction.

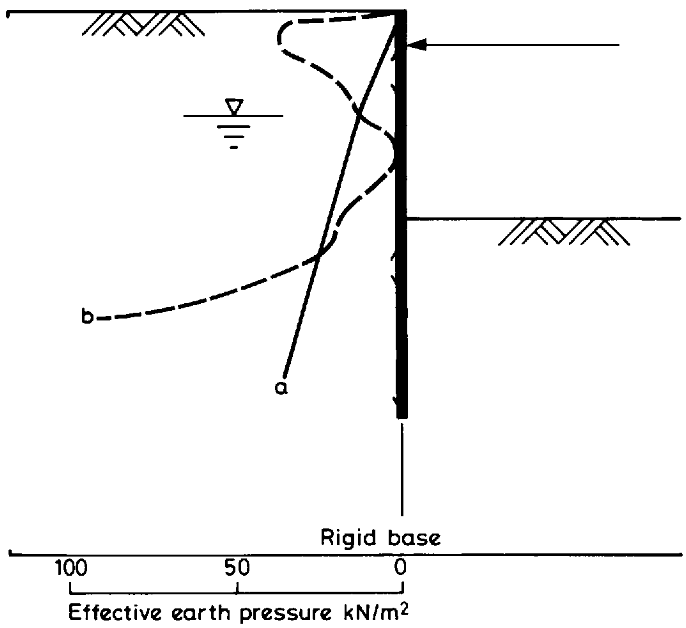


Fig. 36 Active pressures

a eqn. (1)
b as computed with allowance for deformation

Integration of these equations may be carried out to give the active and passive thrusts acting on the structures:

$$P_a = \int_0^z p_a \, dz \quad \text{..............................(5)}$$

$$P_p = \int_0^z p_p \, dz \quad \text{.............................(6)}$$

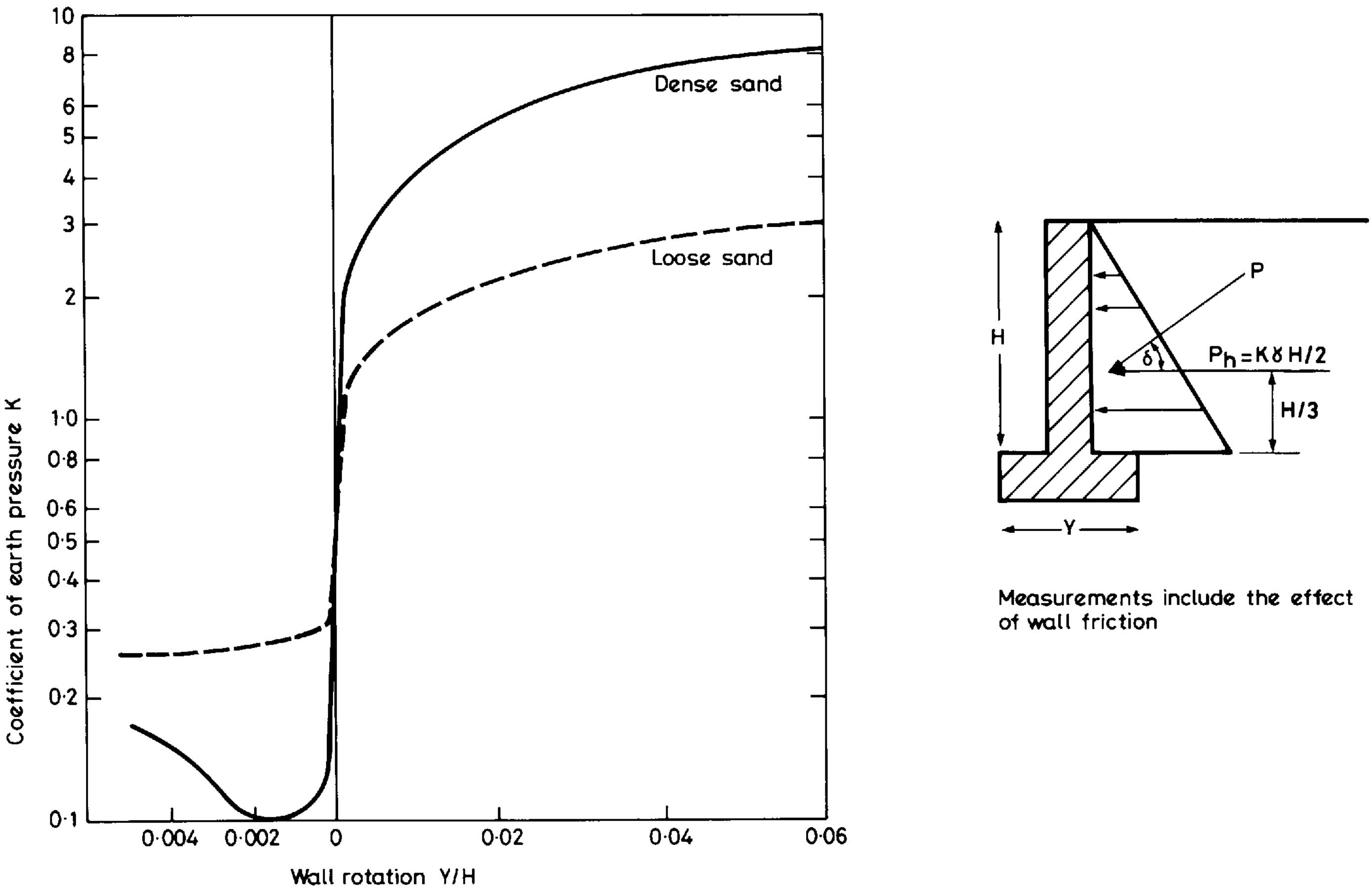


Fig. 37 Experimental results for a rigid wall rotating about the base

Based on NAVFAC (1971)

Field observations and analytical studies indicate that earth pressures may lie outside the limits p_a to p_p set by eqns. 1 to 4. This is a result of stress transfer or arching within the soil, and the resulting earth pressure distribution is greatly affected by the flexibility of the wall and its mode of deformation. However, the total earth forces may not lie outside the range P_a to P_p given by eqns. 5 and 6, but these equations alone give no indication of the limiting pressure distributions and hence do not provide a sufficient restriction to guarantee equilibrium. A conservative approach, which provides some allowance for realistic earth pressure distributions, is therefore necessary in assessing bending moments and propping forces. Simpson (1984) has suggested additional conditions that give a better approximation to plastic equilibrium.

Examples of the distributions of earth pressure corresponding to the limiting active and passive thrust for a rigid wall subject to different modes of movement are illustrated in Fig. 35. For flexible walls, the limiting stress distributions may be much more complex, especially when stiff props are used to support the walls. Pressure distributions for a variety of situations are shown by Padfield & Mair (1984).

Fig. 36 shows the stress distribution computed for a flexible wall with a single prop. In the supported soil, passive pressures are computed to occur close to the prop. Lower down the wall the pressures lie below the values calculated using eqn. 3 but comply with eqn. 5, together with other limits suggested by Simpson (1984).

9.3.2 Relationship between earth pressures and wall movements

In the proceeding clause no consideration was given to the magnitude of soil strains or wall movements necessary to achieve the active or passive conditions. Fig. 37 shows the results of tests in which a rigid wall was rotated about its base into and away from a dry sand backfill (NAVFAC, 1971). The results are presented in terms of the coefficient of horizontal earth pressure K, derived from the horizontal thrust P_h, plotted against wall rotation. Relationships of similar form have also been obtained from more recent pilot-scale tests (Carder *et al*, 1977).

The relative movements required to achieve the limiting active and passive condition will depend on the initial stress state in the ground prior to movement of the wall. This is illustrated in Fig. 38 which shows the development of active and passive coefficients obtained from a numerical study of a 5m high rough wall translated into a clay with values of K_o of 2.0 and 0.5. Also shown on this Figure is the effect of different modes of wall movement.

In cases where excavation takes place in front of the wall, passive pressures will be generated with much smaller movements. This occurs because, as excavation takes place, the horizontal stresses in the soil decrease by only a small proportion of the decrease in vertical stress. The ratio of horizontal to vertical stress therefore tends to increase until the passive limit is approached. The results of Potts & Fourie (1984) indicate the formation of passive wedges beneath an excavation surface, even when the factor of safety against failure is high or when movement of the wall is completely prevented.

Consideration of the relative movements required to achieve the limiting soil thrust are of great importance in terms of the bending moment acting on the wall in working conditions. This is illustrated diagrammatically in Fig. 39, which shows two possible equilibrium distributions of thrust acting on a propped wall, giving the same overall

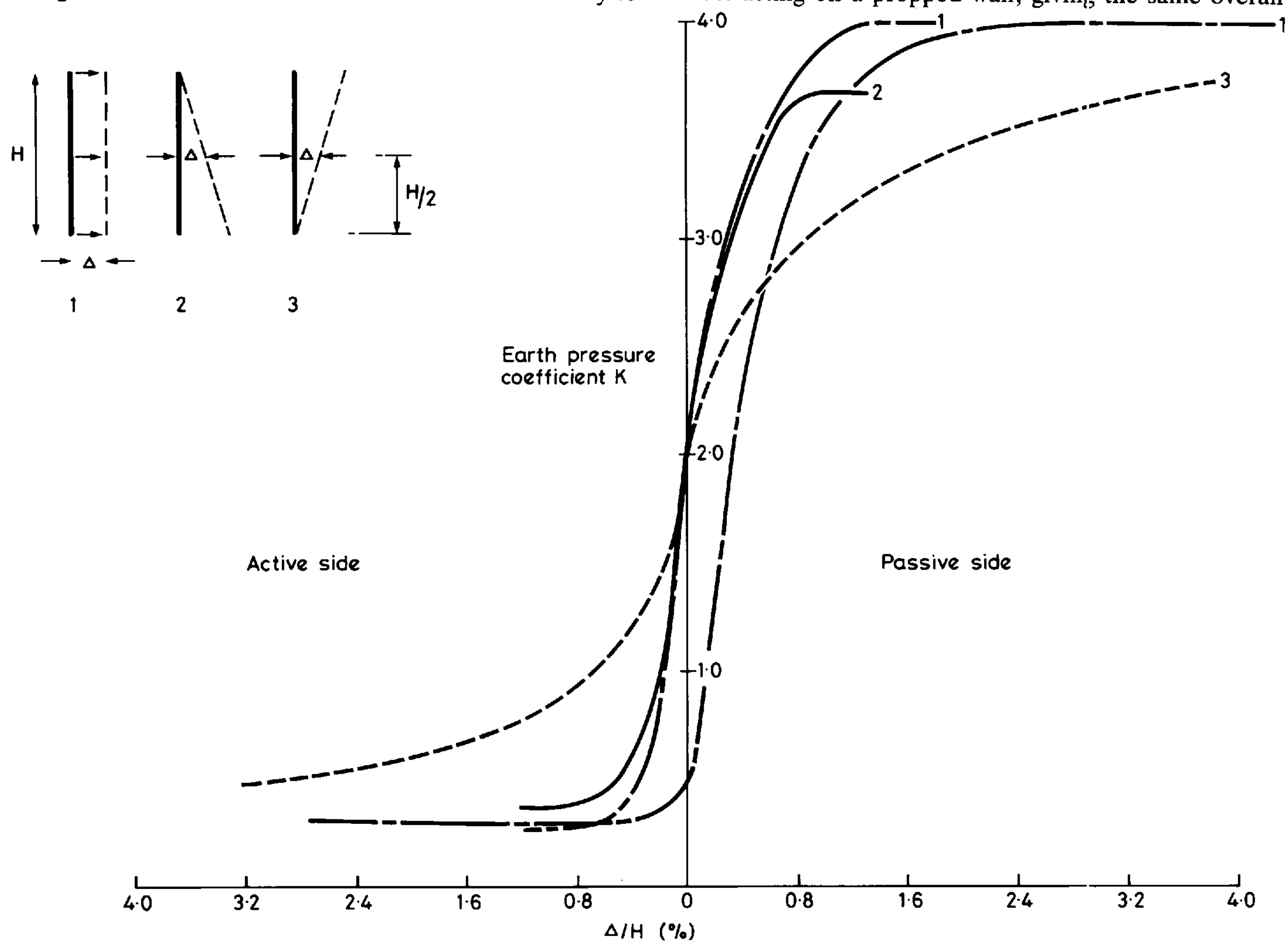
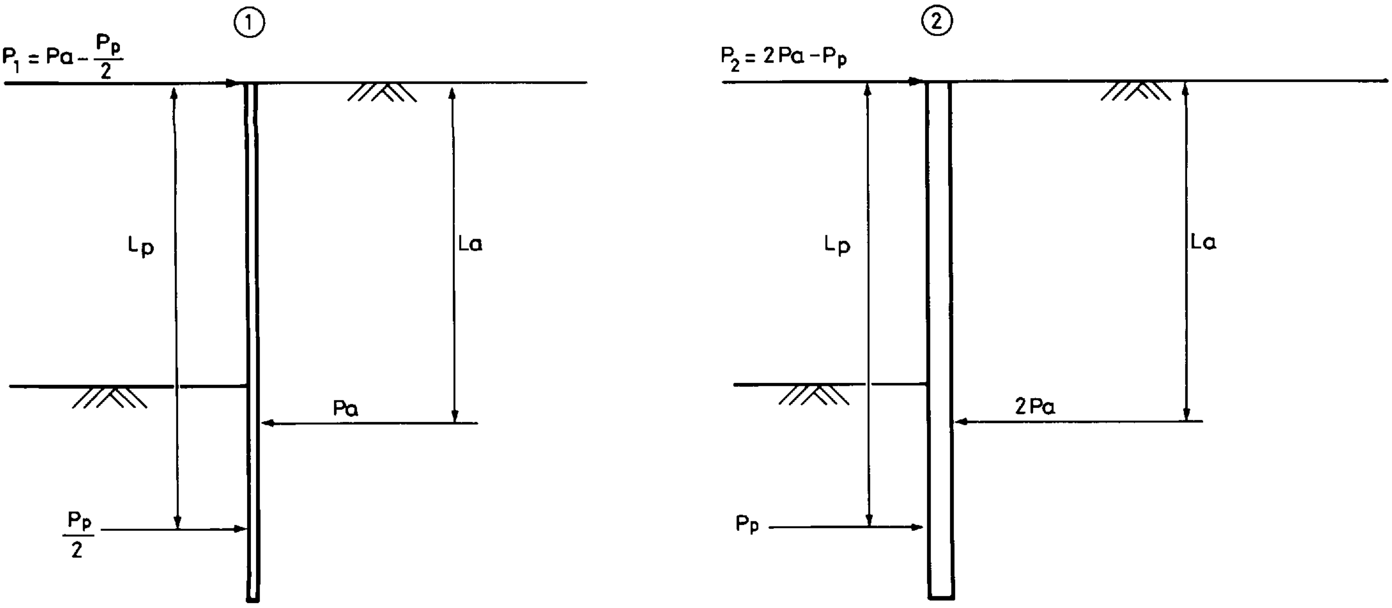


Fig. 38 Development of active and passive pressure coefficients for a rough wall

Based on Potts & Fourie (1986)



$$F = \frac{Mp}{Ma} = \frac{Pp/2\ Lp}{Pa\ La} \equiv F = \frac{Pp\ Lp}{2Pa\ La}$$

Moment in wall 2 = 2 x moment in wall 1

Fig. 39 Influence of distribution of thrusts on walls with the same factor of safety

factor of safety but with bending moments differing by a factor of 2.

9.3.3 Earth pressures arising from surcharges

The ground surface behind a retaining wall may be subject to surcharges. These may be permanent in character, such as the shallow foundations of an adjacent building, or temporary, such as traffic or construction loads caused by plant, storage of materials, etc.

The applied surcharges are usually vertical forces, although they may also have a horizontal component, and will in either case result in an increase in the horizontal earth pressure acting on the wall. For a given magnitude and distribution of the surcharge, the horizontal stresses that act on the wall depend on the properties of the soil and the stiffness of the wall and its supports.

When the surcharge can be modelled as a uniformly distributed vertical pressure of infinite extent, its effects on horizontal earth pressures are readily calculated. Elastic or plastic (usually active) equilibrium should be assumed, as appropriate.

For surcharges of limited extent, it is convenient to consider the two extreme cases of

- a completely restrained wall and
- a wall that moves sufficiently to allow active yield in the soil.

In the case of a rigid wall supporting soil that is not in a state of active yield it is reasonable to assume that the soil will behave in an approximately elastic manner. In this case, the stress distribution may be calculated on the basis of elastic theory, using equations developed from the work of Boussinesq (1885). With the simplifying assumptions that the wall is completely restrained and frictionless, it can be shown that the horizontal stresses exerted on the wall are double the horizontal stresses calculated for the same position relative to the surcharge in an elastic half-space. If the wall deflects slightly as a result of application of the surcharge, the horizontal stress imposed on the wall because of the soil and surcharge will usually be reduced. Elastic analyses are available for surcharges of limited area and for various geometric arrangements (e.g. Poulos & Davis, 1974).

It has been proposed by Jarquio (1981) and others that the same approach may be applied when the wall is at active or passive yield. However, Steenfelt & Hansen (1983) have pointed out that this assumption is invalid and may in some cases be unsafe. This is particularly important since consideration of active pressures is most commonly required for design purposes.

There is no theoretical justification for the assumption that earth pressures calculated from linear elastic theory can be used when the ground is in the plastic state of active yield. Although the total stresses will be lower in the active case than for an unyielding wall, the additional active pressure caused by the surcharge may be greater than indicated by elastic calculations. This is illustrated in Fig. 40. The alternative of adopting the product $2K_a p_v$ has been suggested, where p_v is the elastic vertical stress arising from the surcharge. However, this is equally invalid.

Civil Engineering Code of Practice no. 2 (1951) proposes

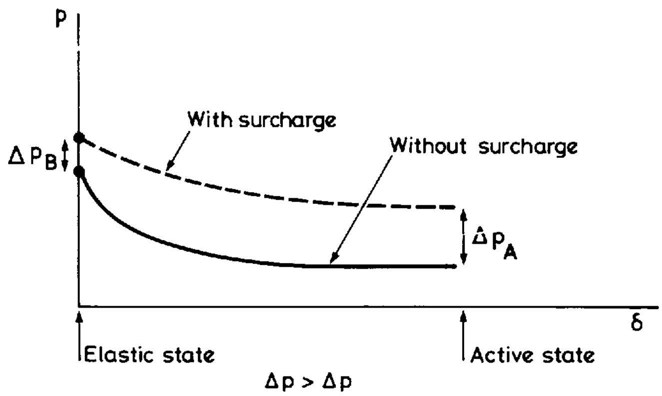


Fig. 40 Effect of increasing movement δ until wall pressure p tends to active limit

Δp_B = double the calculated horizontal stress due to the loaded area using the Boussinesq equations
Δp_A = effect of surcharge at active yield
Note: Δp_A may exceed Δp_B

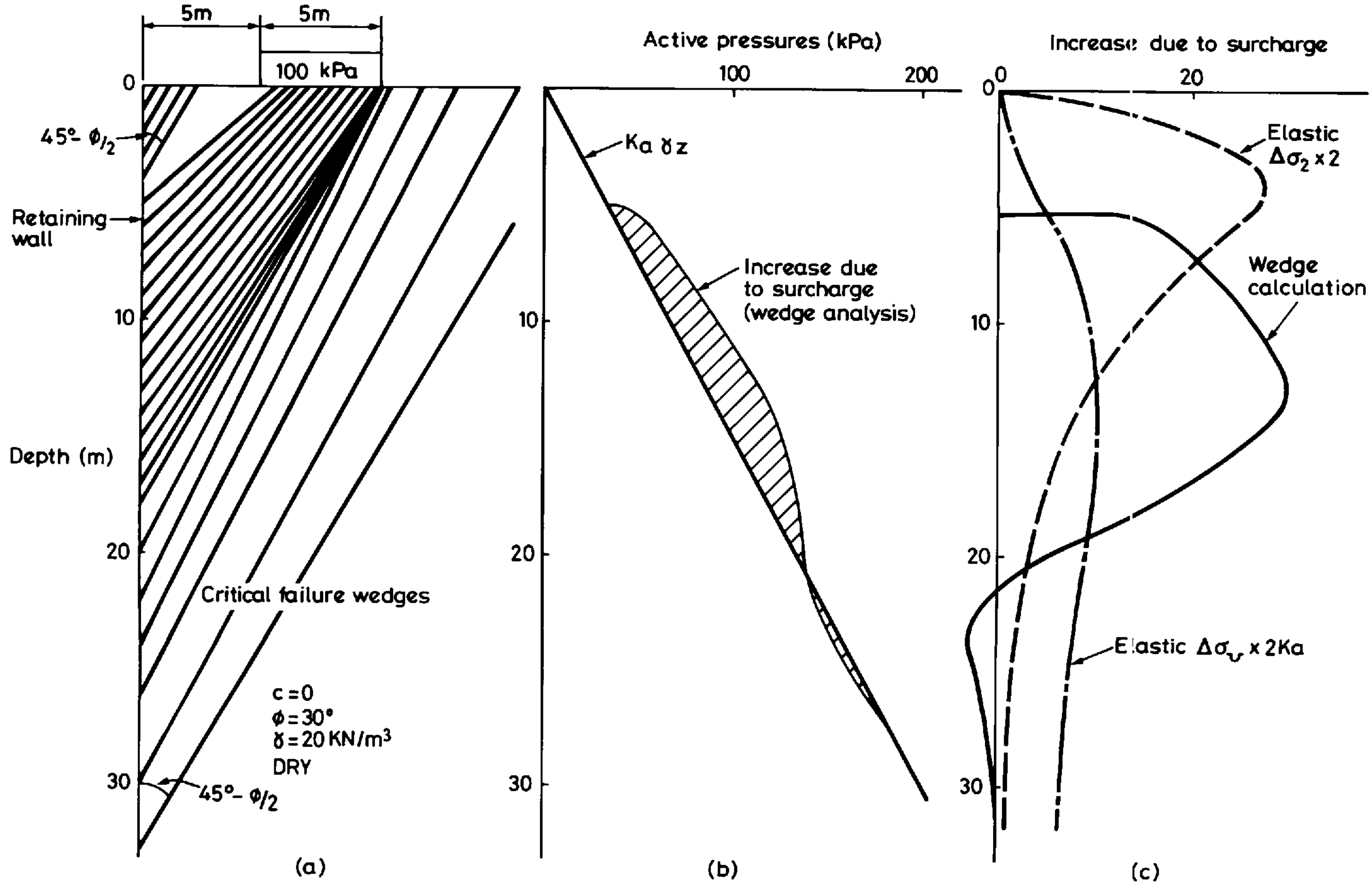


Fig. 41 Effects of a surcharge on earth pressure at active yield

a critical slip planes
b, c calculated earth pressures

that active pressures because of strip surcharges should be calculated by analysis of trial wedges. This approach is supported by Steenfelt & Hansen (1983). Using this approach, it has been found that the calculated additional active earth pressure caused by a surcharge is not very sensitive to the properties of the soil.

Fig. 41 shows an example in which a surcharge is imposed behind a frictionless wall supporting a dry soil with an angle of shearing resistance of 30°. It can be seen that the wedge solution produces results markedly different from the stress distributions derived from elastic theory.

For active pressure, theoretical solutions are not available for the case of surcharges of limited length, but a tentative method is proposed by CP 2.

9.4 Ground movements

Any excavation induces stress changes and consequent ground movements that depend on the ground properties, the geometry of the excavation and the support system. The general form of movement is for the base of the cut to heave while the retained soil moves inwards over depths extending below the base of the excavation. As a result the ground around the excavation subsides (Burland *et al*, 1979). Within this general framework the magnitude and pattern of movement and stress change is greatly dependent on the mode of wall deformation, which is controlled by its stiffness and by the type and position of the supports.

The patterns of movement associated with a free and rigidly propped flexible cantilever wall are illustrated in Fig. 42. For soils in undrained conditions, deformations occur at constant volume, and equal horizontal and vertical movement will take place at all points on the ground surface (Milligan, 1983). Provided that no movement occurs at the base, the settlement profile behind the wall will be approximately equal to the deflected shape of the wall for all modes of deformation. The magnitude of the surface movements relative to those of the wall will then alter as volume changes take place within the retained ground. For the cantilever mode, the horizontal ground

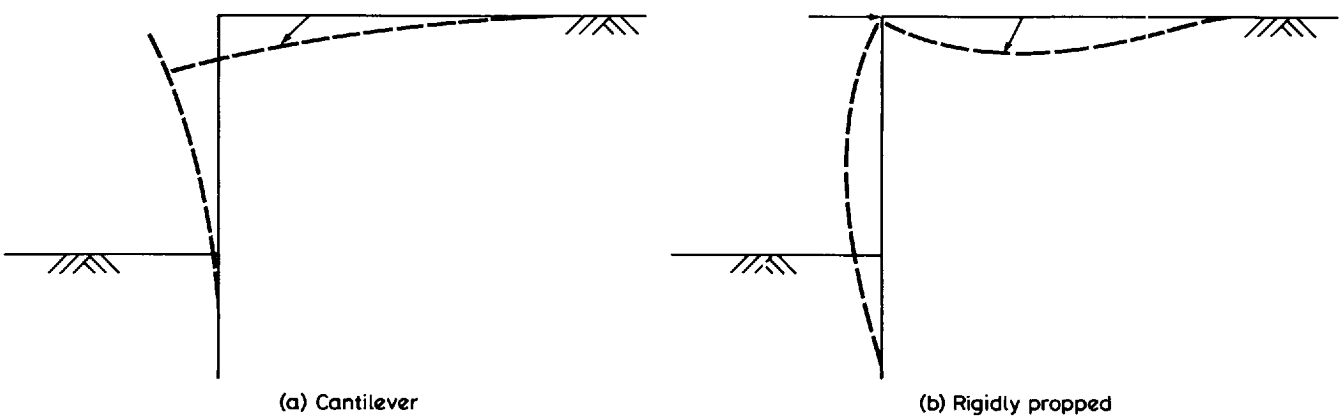


Fig. 42 Modes of deflection of a flexible wall

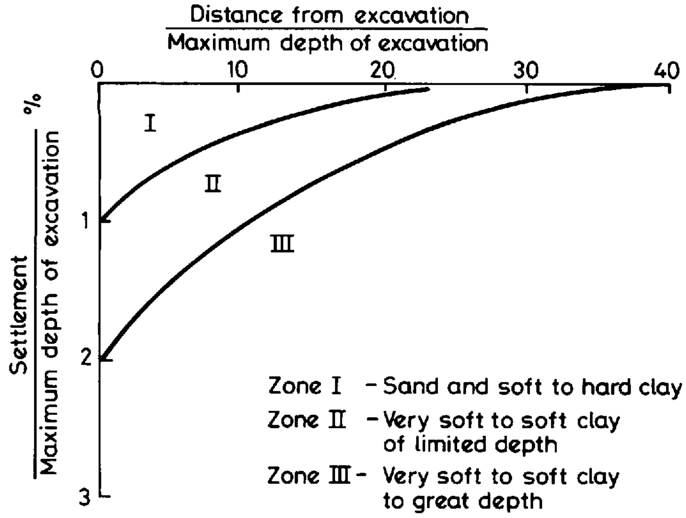


Fig. 43 Settlement adjacent to open cuts in various soils
Based on Peck (1969)

movements are likely to be greater at any point than the corresponding vertical movements, while for a wall rigidly propped at the surface the reverse is likely to be true (O'Rourke, 1981).

These simple patterns of movement will be altered by more global movements of the ground that occur around the structure. Increases in settlement in the retained ground, in heave of the base and in inward movement of the wall are likely as the excavation in front of the wall is deepened (Bjerrum & Eide, 1956).

For multi-propped walls, changes in the mode of deformation are likely during construction as supports are added and removed and as support stiffness changes. In this context the quality of workmanship, the presence and type of packing and preloading of the struts have an important effect on movements and earth pressures (O'Rourke, 1981).

Observations of vertical ground movements around a number of excavations in various soil types have been summarized by Peck (1969), as shown in Fig. 43. The movements relate to average workmanship, and appreciably larger movements are likely if loss of ground occurs during construction, for example by migration of sand into an excavation because of inadequate groundwater control. Field observations of wall and ground movements around deep excavations in London clay are discussed by Burland *et al* (1979).

For trenching operations, the magnitude and distribution of ground movements are particularly susceptible to construction operations such as trafficking by heavy plant. Relatively large ground movements can develop when trenching systems are employed which, although meeting safety requirements, provide little restraint to movement (Symons, 1980). Field observations of ground movements and strut loadings during trenching in a range of ground conditions are presented by Ryley *et al* (1985), Rumsey *et al* (1983) and Chard *et al* (1983).

9.5 Effect of stiffness of the structural system

9.5.1 General

The behaviour of an earth-retaining structure is significantly affected by the stiffness and strength of all the structural and soil components involved. Thus consideration has to be given to the relative stiffness of the wall and soil, the penetration of the wall and the presence of any berms, together with the stiffness and distribution of supports such as props and ground anchors. Another important factor is the method of construction and sequence of excavation.

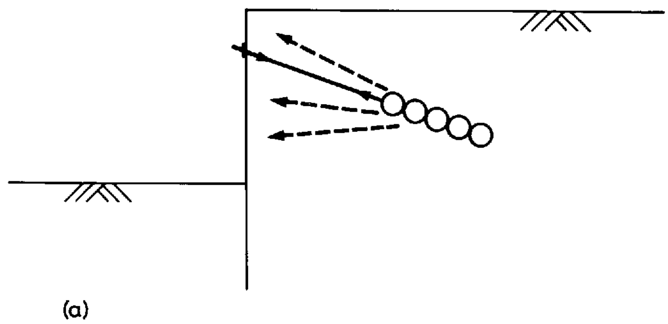
9.5.2 Props and ground anchors

Retaining walls are commonly supported by either props or ground anchors. Props may take the form of temporary steel frames or struts, or the permanent reinforced concrete floor slabs of a basement may be used. Props usually play a passive role, providing a reaction as the wall advances towards them. In contrast, ground anchors are often prestressed and provide an approximately constant force despite movement of the wall.

The stiffness of propping systems affect both the movements of walls and prop forces. Where individual struts are used, the workmanship associated with the installation and packing of each strut may lead to significant variations between the forces in adjacent struts. In extreme cases this may mean that some struts become dangerously overloaded before adjacent struts take up load. In large excavations, elastic compression of steel or concrete supports may contribute significantly to the total movements, and further movement may be allowed by creep and shrinkage of reinforced concrete. St. John (1975) has observed seasonal variations in prop forces that probably relate to temperature changes in floor slabs.

Ground anchors may be used to improve the stability of retaining walls and to reduce ground movements. During prestressing, the wall will move backwards towards the supported soil. Besides providing a beneficial force at the wall, ground anchors also exert an equal and opposite force in the ground at the fixed anchor length. Some of the force transmitted into the ground at the fixed anchor length may be transferred back to the wall itself as indicated by the dashed arrows in Fig. 44a. This latter force generally has an adverse effect and so reduces the benefit derived from the anchors to some extent. The importance of this factor is diminished by increasing the distance of the fixed anchor length from the wall. Furthermore, in considering active failure, slip surfaces of the type shown in Fig. 44b should be analysed. In assessing movements of the wall, ground movements at the fixed anchor length should be considered (Sills *et al*, 1977), together with possible yield or creep of the fixed anchor length relative to the surrounding soil.

For multi-propped or anchored walls consideration has also to be given to the excavation and prestressing sequence



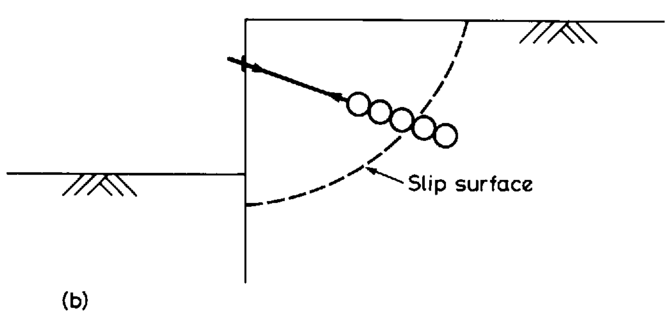
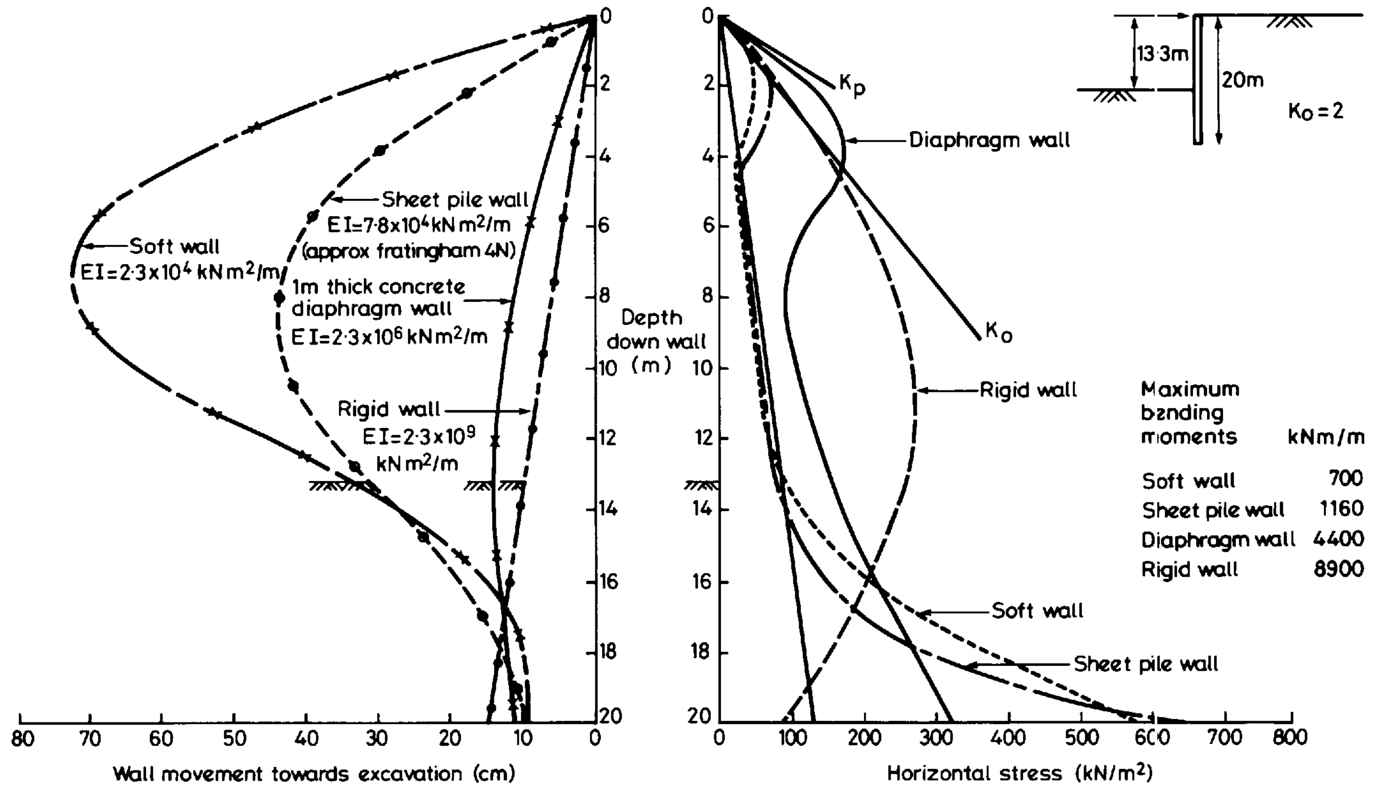


Fig. 44 Interaction between ground anchor and wall
a quasi-elastic state
b slip surface to be considered in assessing active pressures



(a) Wall displacements

(b) Earth pressures down back of wall

Fig. 45 Computed effect of wall stiffness on wall displacements and earth pressure Based on Potts & Fourie (1985)

and, if appropriate, subsequent removal of temporary supports and transfer of load to the permanent works. All of these factors will influence the behaviour of the wall (Wood & Perrin, 1984).

Where props are employed to brace between walls, particular attention should be paid to the construction sequence. With all but the simplest of walls it is essential that a behaviour envelope based on the possible range of construction and excavation sequences is used in the design.

The degree of moment connectivity between the props and the wall will have a marked effect on the bending moments developed in the wall itself. In practice, the assessment of such connectivity is difficult, and the assumption of a pinned joint is usual but by no means always representative of the true situation. Where appropriate, analyses based on the extreme pinned and rigid assumptions may provide the design envelope.

9.5.3 Wall penetration and stiffness

When designing embedded cantilever walls, either propped or unpropped, the depth of embedment required to maintain overall stability has to be determined. This depth is then increased to provide an adequate factor of safety. In the working condition, the wall is not on the verge of instability, and therefore limiting earth pressures will not be mobilized simultaneously in front of and behind the wall.

Numerical work by Potts & Fourie (1984) and Simpson (1984) indicates that increased penetration has little effect on bending moments, shear forces and movements unless the wall is abnormally stiff. When limitation of movement is of prime concern, the introduction of additional levels of support is likely to provide a more positive solution. Sufficient penetration should be provided to prevent failure of the base of the excavation, but this may be minimal in some cases.

The stiffness of the wall can have a large effect on wall movements and on distributions of earth pressures. This arises because more flexible walls deform more readily and cause a redistribution of the earth pressures, which generally leads to reduced bending moments and thrusts. However, these reductions are achieved at the expense of somewhat larger movements that are of practical significance for cantilever walls. For propped walls, however, a three- to five-fold change in wall stiffness has little effect on deformations (Simpson, 1981; and Hubbard *et al*, 1984). The degree of earth pressure redistribution depends on the number of support levels, the type of soil and the initial state of stress in the soil prior to construction.

On the basis of results from model tests, Rowe (1955 & 1957) prepared design charts for sheet pile walls that allowed for the effects of flexibility on bending moments and prop forces. Similar effects have been demonstrated from numerical work by Bjerrum *et al* (1972) and Pappin *et al* (1986).

Fig. 45 shows the results of finite element computations published by Fourie & Potts (1985) for a propped, embedded cantilever wall in stiff clay. The effects of varying the wall stiffness on wall displacement and earth pressures are shown, together with the computed bending moments. These authors also show that the effects of varying wall stiffness are dependent on the magnitude of the initial horizontal stress in the ground before construction.

9.5.4 Berms

For a propped retaining wall embedded in deep deposits of clay an important component of horizontal wall movement during construction is likely to result from deep-seated movements within the clay. Since such movements are essentially a response of the ground to the relief in vertical stress in front of the wall, the introduction of props to reduce this component of movement is likely to be effective only if they are installed in advance of excavation. The use

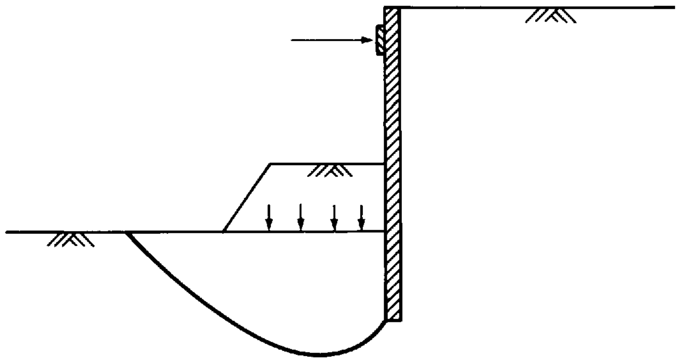


Fig. 46 Effect of a berm

of berms located in front of the wall as illustrated in Fig. 46 can provide an effective means of temporarily improving stability and limiting wall deformations during construction. Construction of permanent struts at excavation level can then be achieved by rapid removal of the berm over short lengths. The principal effects of such a berm are to extend the length of potential failure surfaces in front of the wall, to add a surcharge to the ground at excavation level, and to inhibit swelling and softening of the underlying clay. The beneficial effects of the use of berms for propped embedded walls in overconsolidated clay has been demonstrated from computational studies and field monitoring (Burland *et al*, 1979; Cole & Burland, 1972; and St. John, 1975).

9.6 Effect of type and method of wall construction

9.6.1 Walls retaining backfill

Adequate compaction of fill behind retaining walls is generally necessary to limit settlement. The operation of plant induces transient horizontal stresses on the structure during placing and compaction of the fill and residual horizontal pressures on completion. The magnitude of these transient stresses and residual pressures is dependent on the type of fill and state of compaction, on the type and size of plant and its proximity to the structure and on the flexibility of the wall and the restraints on its movement.

For rigid unyielding walls the effect of compaction of backfill is to induce pressures over the upper part of the structure appreciably greater than the at-rest pressures of normally consolidated fills. For walls that can move or deflect during backfilling, such as unpropped cantilever walls, the local earth pressure at any depth is related to the wall movements that occur subsequent to compaction of the fill at that depth rather than to total wall movement from the start of backfilling. On completion of filling, therefore, pressures appreciably in excess of the active values are likely to be present over the upper part of the structure, although only small additional deformations are generally required to relieve these excess pressures (Carder *et al*, 1977; and Broms & Ingelson, 1971). Where no such yield of the wall can take place, account has to be taken in the design of the stresses induced by compaction, or limitations have to be placed on the type of compaction plant used and its proximity to the structure (Jones & Sims, 1975; and Symons & Wilson, 1972).

Simplified methods of assessing earth pressures during and on completion of filling are given for rigid walls by Broms (1971) and Carder *et al* (1980) and for yielding walls by Ingold (1979). These methods make use of elastic analyses to calculate the stresses induced by compaction and take into account the differences in response of the soil to loading and unloading similar to that illustrated in Fig. 34.

9.6.2 *In situ* walls

The principal stages in the construction of an *in situ* wall comprise installation of the wall in the natural ground, excavation in front of the wall and the insertion of temporary and permanent supports. For such structures the method of forming the wall is likely to lead to changes in the initial stress state in the adjoining ground and to accompanying ground movements. Thus for walls of diaphragm or bored-pile type in ground with high K_o, some reduction in horizontal stress and consequent inward movement of the ground may be expected (Burland & Hancock, 1977; Tedd *et al*, 1984; and Cowland & Thornley, 1985). Similarly for driven-pile walls, a local increase in earth pressure to above the at-rest condition is likely.

Only limited field data are available on the ground movements and pressure changes occurring during wall installation, but these suggest that ground behaviour is heavily dependent on soil type and on the details of the construction method (Davies & Henkel, 1970; and Raison, 1985). For diaphragm or bored-pile walls, ground movements are likely to result from the following effects:

- disturbance or loss of ground during excavation or boring
- temporary loss of support caused by a reduction in the excess head of slurry in diaphragm construction or from premature extraction of casing from pile bores
- the formation of voids or cavities during concreting.

Delays during the installation process are likely to enhance ground movements.

9.7 Calculation methods

9.7.1 General

Use of the finite-element method provides the most comprehensive picture of soil–structure interaction available from current procedures. However, simpler forms of calculations also provide considerable insight and are often adequate for design purposes.

9.7.2 Simple calculations

Retaining walls are often designed taking no account of the relative stiffnesses of soil and structure. The calculations are concerned solely with the provision of an adequate 'factor of safety' in terms of stability. Once this has been established, the bending moments in the wall may be determined from the assumed earth- and water-pressure diagrams (i.e. active and passive earth pressures). If an assessment of wall deformation is required then the assumption of a point of fixity at some depth below the dredge level gives rise to a tractable solution. While these methods of calculation form a prerequisite to any more sophisticated analyses that take account of the interaction effects, they do not attempt to model the true behaviour of the wall.

Various calculation methods have been proposed that make allowance for interaction effects without requiring them to be analysed explicitly. For example, the design charts published by Rowe (1955) indicate the effect on bending moments of the relative stiffness of sheet-pile walls and the ground. Similarly Peck's (1969) trapezoidal envelopes of earth pressure allow for redistribution of force onto the supports of a multi-propped wall.

9.7.3 Elasticity calculations

An estimate of the overall movements of a wall can sometimes be made by using solutions derived for the rotation and displacement of a rigid wall in homogeneous elastic continuum (Poulos & Davis, 1974). These take account of deep-seated movements within the soil, but the effect of curvature in the wall itself on earth pressures and bending moments is not modelled. Alternatively, if the changes of earth pressure are considered to be known, the displacements associated with them may be calculated using the equations of Vaziri *et al* (1982). These assume a linear elastic half-space, and allowance must therefore be made for the effects of a rigid base at an appropriate depth. A small computer program is required to evaluate these equations.

9.7.4 Complete methods

The remaining methods of analysis, discussed below, require a knowledge of the stiffness characteristics of the wall, the soil and the props or ground anchors, in addition to the shear strength parameters and water conditions necessary to determine stability. Information is also needed on the initial *in situ* soil stresses and the stress changes induced by the installation of the wall. The difficulty of assessing these input parameters reliably serves to limit the accuracy of predictions obtained from numerical methods and their use as a direct design tool at present. Qualitative studies using such methods nevertheless provide a useful design aid in situations where deformations and movements are of prime concern, particularly if linked to field monitoring during construction (Symons *et al*, 1985).

When undertaking any analysis it is prudent to give consideration to the degree of certainty with which the various parameters may be quantified. It should be remembered that different values of strength parameters may apply to the stability and deformation calculations. For example, stability may be based on the worst credible set of parameters, while the computation of ground movements may be computed for the most likely service condition.

Where the prime concern is the behaviour of the wall and its supports, methods of analysis may be adopted that take account of the wall–soil interaction but do not compute the general ground movements. Such methods include the so-called 'beam-on-springs' approach and the more recent continuum soil models that use interaction coefficients derived from finite element analyses or boundary integral equations (Wood, 1979, 1981 & 1984; Simpson, 1984; and Pappin *et al*, 1986). All these methods may incorporate limiting active and passive pressures in order to obtain a more realistic analysis of the working state.

Although the beam-on-springs method leads to simpler computations, it employs an unrealistic model of the ground. It is therefore difficult to provide appropriate values for the necessary spring constants. Its use is not recommended.

Computation methods using continuum models are used mainly for embedded walls. They often provide an adequate analysis for structural design purposes and also indicate the movement of the wall itself. This type of computation is particularly effective during preliminary design studies and is probably suitable for most everyday design work. Its cost is a small fraction of that of an equivalent non-linear finite-element study.

9.7.5 Finite-element method

Of the calculation methods that are available, only the finite-element method takes account of the interaction between all the components within the retaining system. Use of this approach will yield results not only for the behaviour of the wall but also for general ground movements, anchor and prop forces and movements, effects of surcharges and so on.

For overconsolidated clay, linear-elastic finite-element methods have yielded good predictions of overall ground and wall movements (Burland & Hancock, 1977; Cole & Burland, 1972; and Burland *et al*, 1979). If predictions of the likely magnitude and extent of ground movements behind the wall are required, however, more sophisticated models, such as those proposed by Simpson *et al* (1979) and Potts & Fourie (1984), may be necessary. For soft clays and sands, yield in shear has a dominant effect on earth pressures and movements. This should therefore be included when the soil is modelled by finite elements.

Finite-element computations may be used to analyse any type of structure. At a research level, their use has provided many insights into the likely behaviour of retaining walls (e.g. Potts & Burland, 1983). However, their use in design is currently limited to major constructions, particularly where the presence of existing structures necessitates a prediction of ground movements.

References

Bjerrum, L., Clausen, C. J. F., & Duncan, J. M. (1972): 'Earth pressures on flexible structures', state-of-the-art report, *5th Euro. conf. SM&FE,* Madrid

Bjerrum, L., & Eide, O. (1956): 'Stability of strutted excavations in clay', *Geotechnique,* **6**, no. 1, p. 32

Boussinesq, J. (1885): *Application des potentials à l'etude de l'equiline et du mouvement des solides elastiques,* Gauthier-Villars, Paris

Broms, B. B. (1971): 'Lateral earth pressures due to compaction of cohesionless soil', *Proc. 4th Budapest conf. on SM&FE*

Broms, B. B., & Ingelson, I. (1971): 'Earth pressures against the abutments of a rigid frame bridge', *Geotechnique,* **21**, p. 15

Burland, J. B., & Hancock, R. J. R. (1977): 'Underground car park at the House of Commons: geotechnical aspects', *Struct. Eng.,* **55**, p. 87

Burland, J. B., Simpson, B., & St. John, H. D. (1979): 'Movements around excavations in London clay', *Proc. 7th Euro. conf. SM&FE,* Brighton, **1**, p. 13

Carder, D. R., Murray, R. T., & Krawczyk, J. V. (1980): *Earth pressures against an experimental retaining wall backfilled with silty clay,* TRRL Laboratory report 946

Carder, D. R., Pocock, R. G., & Murray, R. T. (1977): *Experimental retaining wall facility – lateral stress measurements with sand backfill.* Dept. of Environment, TRRL report 766, Crowthorne, Transport & Road Research Laboratory.

Caquot, A., & Kerisel, J. (1948): *Tables for the calculation of passive pressure, active pressure and bearing capacity of foundations,* Gauthier-Villars, Paris

Chard, B. M., Symons, I. F., Toombs, A., & Nagarkatti, A. (1983): *Trial trench construction in alluvium: a ground movement study at Avonmouth,* TRRL report LR 1077

Cole, K. W. (1980): 'The south abutment at Kessock Bridge, Scotland', *Proc. 11th Congress IABSE,* Vienna, p. 1099

Cole, K. W., & Burland, J. B. (1972): 'Observations of retaining wall movements associated with a large excavation', *Proc. 5th Euro. conf. SM&FE,* Madrid, **1**, p. 445

Cowland, J. W., & Thornley, C. B. B. (1985): 'Ground and building settlement associated with adjacent slurry trench excavation', *Proc. 3rd int. conf. ground movements & structures,* UWIST, Cardiff, 1984, Pentech Press

CP 2 (1951): Civil Engineering Code of Practice no. 2. *Earth retaining structures,* IStructE, London

Davies, R. V., & Henkel, D. J. (1980): 'Geotechnical problems associated with the construction of Chater Station, Hong Kong', *Proc. conf. mass transportation in Asia,* Hong Kong, paper 13

Fourie, A. B., & Potts, D. M. (1985): private communication

Henkel, D. J. (1970): 'Geotechnical considerations of lateral stresses', *Proc. ASCE speciality conf. lateral stresses in the ground and design of earth-retaining structures,* Cornell, 1-50

Hubbard, H. W., Potts, D. M., Miller, D., & Burland, J. B. (1984): 'Design of the retaining walls of the M25 cut and cover tunnel at Bell Common', *Geotechnique,* **34**, no. 4, p. 495

Ingold, T. S. (1979): 'The effects of compaction on retaining walls', *Geotechnique,* **29,** 3

Jarquio, R. (1981): 'Total lateral surcharge pressure due to strip load', *Jnl. Geo. Eng. Div. ASCE,* **107,** 10, p. 1424

Jones, C. J. F. P., & Sims, F. A. (1975): 'Earth pressures against abutments and wing walls of standard motorway bridges', *Geotechnique,* **25,** p. 731

Milligan, G. W. E. (1983): 'Solid deformations near anchored sheet piled walls', *Geotechnique,* **33,** 1

NAVFAC (1971): *Design manual – soil mechanics, foundations and earth structures,* Report DM-7, Washington DC, US Naval Facilities Engineering Command

O'Rourke, T. D. (1981): 'Ground movements caused by braced excavations', *Jnl. Geo. Eng. Div. ASCE,* **107,** GT 9

Padfield, C., & Mair, R. (1984): *Design of retaining walls embedded in stiff clay,* CIRIA report 104, London, CIRIA

Pappin, J. W., Simpson, B., Felton, P. J., & Raison, C. (1986): 'Numerical analysis of flexible retaining walls', *Proc. symp. computer applications in geotech. eng.,* Midland Geotechnical Soc., Birmingham

Peck, R. B. (1969): 'Deep excavations and tunnelling in soft ground', *Proc. 7th int. conf. SM&FE,* Mexico

Potts, D. M., & Burland, J. B. (1983): *A numerical investigation of the retaining walls of the Bell Common Tunnel,* TRRL supplementary report 783

Potts, D. M., & Fourie, A. B. (1984): 'The behaviour of a propped retaining wall – the results of a numerical experiment', *Geotechnique,* **34,** no. 3, p. 383

Poulos, H. C., & Davis, E. H. (1974): *Elasticity solutions for soil and rock mechanics,* Wiley

Raison, C. A. (1985): Discussion, *Geotechnique.* **35,** p. 540

Rowe, P. W. (1955): 'A theoretical and experimental analysis of sheet piled walls', *Proc. ICE,* **4,** p.32

Rowe, P. W. (1957): 'Sheet pile walls in clay', *Proc. ICE,* **7,** p. 629

Rumsey, P. B., Cooper, I., & Kryou, K. (1982): 'Ground movement and pipe strain associated with trench excavation', *Proc. int. conf. on restoration of sewerage systems,* Thomas Telford, London, p. 104

Ryley, M., McCaul, C., & Symons, I. F. (1985): *Trial trench construction in boulder clay: a ground movement study at Cowden,* TRRL Report (in press)

Sills, G. C., Burland, J. B., & Czechowski, M. K. (1977): 'Behaviour of an anchored diaphragm wall in stiff clay', *Proc. 9th int. conf. SM&FE,* Tokyo, **2,** p. 147

Simpson, B. (1981): 'Finite elements in design with particular reference to deep basements in London clay', in *Finite elements in geotechnical engineering* by D. J. Naylor, G. N. Pande, B. Simpson & R. Tabb, Pineridge Press

Simpson, B. (1984): 'Computational analysis of flexible retaining walls', *Proc. symp. soil–structure interaction,* Institution of Structural Engineers, London

Simpson, B., Calabresi, G., Sommer, H., & Wallays, M. (1981): 'Design parameters for stiff clays', General report, *Proc. 7th Euro. conf. SM&FE,* Brighton (1979), **5,** p. 91

Skempton, A. W., & La Rochelle, P. (1965): 'The Bradwell slip: a short term failure in London clay', *Geotechnique,* **15,** p. 221

Steenfelt, J. S., & Hansen, B. (1983): Discussion of Jarquio (1981): *Jnl. Geotech. Eng. ASCE,* **109,** 2, p. 271

St. John, H. D. (1975): *Field and theoretical studies of the behaviour of ground around deep excavations in London clay.* PhD thesis, University of Cambridge

Symons, I. F. (1980): 'Ground movements and their influence on shallow buried pipes', *Public health engineer*

Symons, I. F., Potts, D. M., & Charles, J. A. (1985): 'Predicted and measured behaviour of a propped embedded retaining wall in stiff clay', *Proc. 11th int. conf. SM&FE,* San Francisco

Symons, I. F., & Wilson, D. S. (1972): 'Measurement of earth pressures in pulverized fuel ash behind a rigid retaining wall', *Proc. 5th Euro. conf. SM&FE,* Madrid, **1,** p. 569

Tedd, P., Chard, B. M., Charles, J. A., & Symons, I. F. (1984): 'Behaviour of a propped embedded retaining wall in stiff clay at Bell Common Tunnel', *Geotechnicque,* **34,** no. 4, p. 513

Vaziri, H., Simpson, B., Pappin, J. W., & Simpson, L. (1982): 'Integrated forms of Mindlin's equations', *Geotechnique,* **32,** no. 3, p. 372

Wood, L. A. (1979): 'LAWPILE – program for the analysis of laterally loaded pile groups and propped sheetpile and diaphragm walls', *Advances in eng. soft.,* **1,** p. 173

Wood, L. A. (1981): The implementation of boundary element codes in geotechnical engineering, in *Boundary element methods,* ed. C. A. Brebbia, Springer-Verlag, p. 310

Wood, L. A. (1984): *LAWWALL user manual,* SIA, London

10 Reinforced-soil structures

10.1 Introduction

It has been long known that soils and rocks are weakest in tension. A favoured approach for ensuring structural stability has been, therefore, to employ techniques that produced net increases in compressive stress in the structure and foundations, as exemplified by the widespread use of arch construction. Nonetheless, the concept of improving the tensile strength of soils and rocks by the inclusion of reinforcement is not new, although prior to 1963 when Vidal (1963) introduced a proprietary system referred to as La.Terre Armée, the principles of reinforced soil had not attracted serious attention.

The earth reinforcement system initially developed by Vidal (1969) involved the use of metal strips, typically of galvanized mild steel, in conjunction with a semi-elliptical form of metal facing and sand or gravel earth fills (see Fig. 47). Since the early work of Vidal many alternative soil reinforcement systems have been developed (see Figs. 48 to 51). To some extent such developments have resulted from the timely introduction of relatively new materials such as high-strength polymers and fibre-reinforced plastics, both in strip and sheet form, although traditional reinforcement forms such as welded steel mesh have also found application.

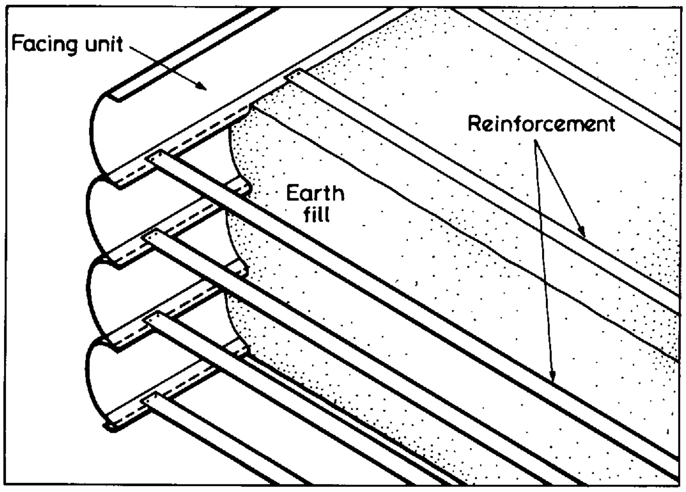


Fig. 47 Early form of reinforced earth system

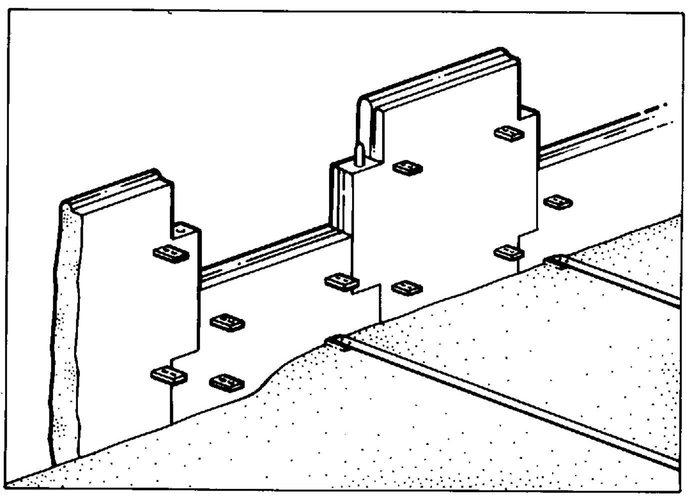


Fig. 48 Precast concrete facing units

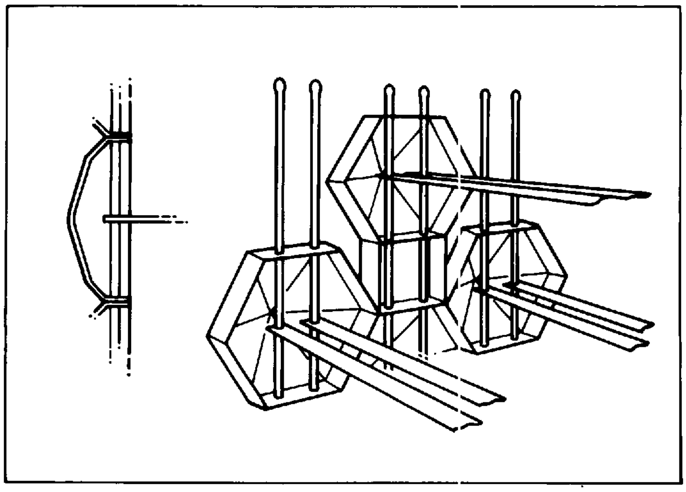


Fig. 49 York system with sliding connections

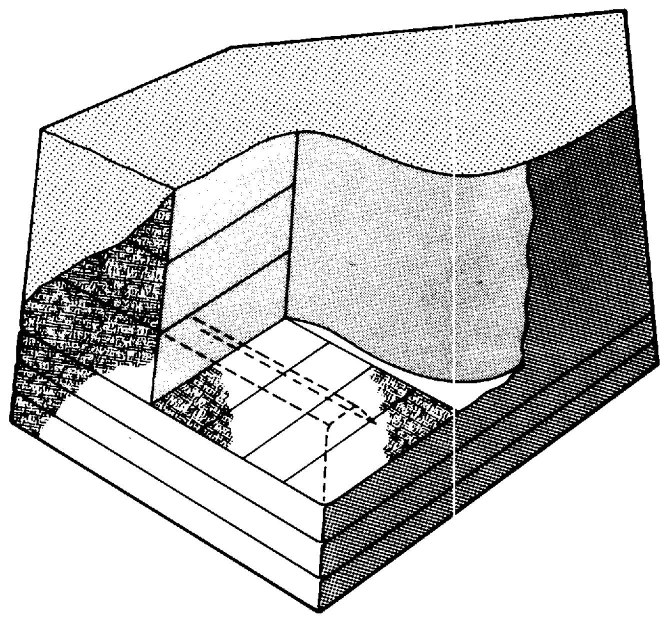


Fig. 50 Reinforced earth system based on geogrid reinforcement

There is clearly an analogy between reinforced concrete and reinforced soil in that a matrix material that is strong in compression but weak in tension is combined with tensile elements to produce a material with enhanced properties. There are significant differences, however, both in functional requirement and detailed behaviour. In contrast to reinforced-concrete structures, an important attribute of reinforced-soil structures, which is of particular relevant to soil–structure interaction behaviour, is that they are considered to be flexible. Reinforced-concrete structures are

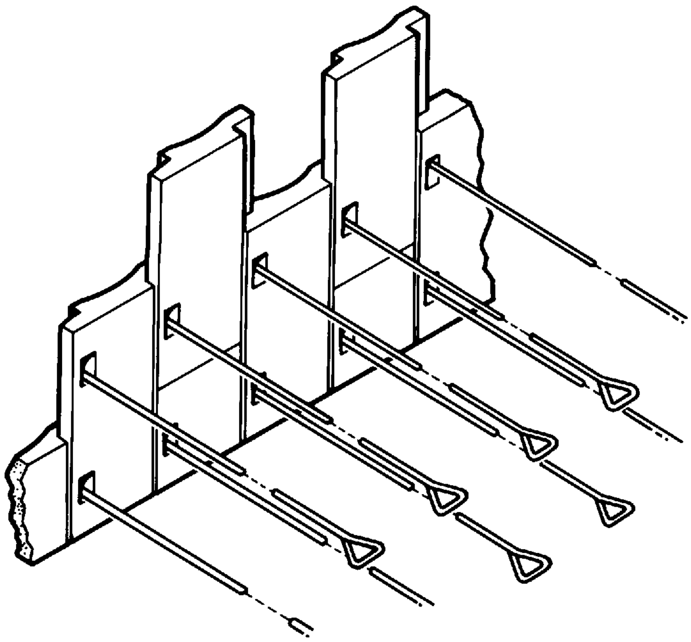


Fig. 51 Schematic arrangement of TRRL anchored earth system

also designed and expected to support tensile forces. However, reinforced-soil structures are normally required to support only a minimum compressive force corresponding to a condition of active earth pressure that is associated with lateral expansion and failure of the soil. Such failure is often manifest by blocks or wedges of soil physically separating from the main mass in the form of slip zones. The reinforcement is introduced to prevent failure by imposing constraints on the soil to maintain the system intact.

It is usual to assume a perfect chemical bond between concrete and reinforcement. The interaction between soil and reinforcement most commonly is developed through surface friction, and an ideal bond would correspond to the development of the same shear strength at the interface as that of the soil; a situation which is not usually achieved. As an alternative to this method of soil–reinforcement interaction some systems develop passive strength by the use of anchors (Murray & Irwin, 1981) or projections in association with both strip and grid-type reinforcements (Schlosser & Guillaux, 1979; Peterson, 1980; and Bell *et al*, 1984).

The improved flexibility of reinforced soil should lead to reduced complexity in interpreting or predicting behaviour associated with soil–structure interaction problems, and moreover, such structures can be expected to tolerate differential settlements more effectively than conventional structures. Nonetheless, this improved flexibility would not prevent damage to imposed superstructures in the event of differential settlements, although it is to be expected that a more uniform deformation profile would result such that the effects are less severe.

Reinforced soil has a wide range of applications (Jones, 1985) including retaining structures, slopes, foundations and subsurface vaults. The most common application has been in relation to earth-retaining structures, and it is this aspect of the topic which is considered in this section, although the other applications have much in common with the principles employed for retaining walls.

10.2 Design considerations

The design of reinforced soil-retaining structures in the UK is generally based on a limit-equilibrium approach following the rules contained in a technical memorandum (Department of Transport, 1978). For purposes of a design, both external and internal stability must be considered. The assessment of external stability described in the technical memorandum is based on the use of ultimate properties divided by an appropriate factor of safety. In contrast, internal stability is based mainly on a permissible-stress approach, apart from the design calculations relating to interface friction where the former method, involving ultimate properties, is employed.

As the tensions mobilized are associated with differential movements, the detailed interaction behaviour between reinforcement and soil is complex as the movements may vary both radically and along the length of the reinforcements. Furthermore, the facing panels are articulated to permit a degree of flexibility that further increases the complexity of the displacement and stress field within the system. Thus a realistic stress/strain analysis of a reinforced soil system would be extremely difficult and expensive to carry out. The current approach to design therefore ignores the detailed stress/displacement behaviour and treats reinforced soil as an anchor system.

Although such an approach appears over-simplistic, experience has shown that the performance of reinforced-soil structures designed on this basis is generally satisfactory (Hollinghurst & Murray, 1986; and Murray & Hollinghurst, 1986), particularly with the commonly used high-modulus reinforcement and granular fill. With reinforcing elements that are more strain susceptible, such as polymer reinforcements, this approach is also employed but is clearly less satisfactory as stress-transfer mechanisms may be operating over relatively long periods associated with the time-dependent properties of these materials that are not considered in the design procedure.

The approach to the design of reinforced-soil retaining walls in France (French Ministry of Transport, 1979) and the USA (Goughnour & DiMaggio, 1979), where large numbers of structures have been erected, has many similarities with the procedures described in the DTp technical memorandum. The main differences occur in relation to the assessment of overall internal stability. It is assumed in the technical memorandum that a plane surface of failure will develop allowing a design assessment to be carried out on the basis of the Coulomb wedge approach (Fig. 52a). In contrast, the design manual published by the Ministry of Transport in France (1979) prescribes the use of bilineal failure surface for purposes of design, as shown in Fig. 52b. The location of the bilineal slip surface has been determined from observations of peak tension on structures in their working condition. It is apparent that the potential slip surface determined by this method does not relate to a true collapse condition and may not provide a reliable indication of failure.

Various other forms of failure surfaces which have been proposed include a logarithmic spiral (Juran & Schlosser, 1978), a parabolic curve (Borg *et al*, 1980) and composite linear surfaces (Smith & Wroth, 1978). Although there are likely to be a significant number of occasions when curved or composite linear failure surfaces are more appropriate, most specifying authorities seem to favour the use of either the Coulomb wedge method or the analysis-based bilineal surfaces. This may be justified to a considerable extent in view of the generally satisfactory performance achieved with current methods and also by the fact that the savings resulting from design based on more complex failure surfaces are unlikely to be significant.

The application of the finite-element method to the analysis and design of reinforced-soil structures provides more scope for determining the detailed internal stress/strain behaviour and is being employed with greater

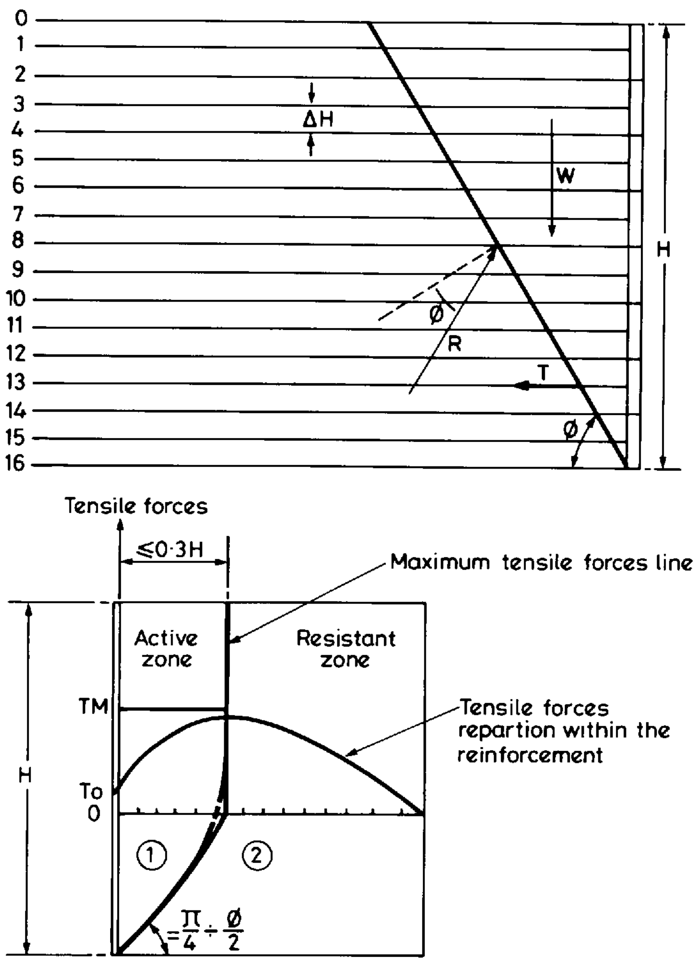


Fig. 52 Assessment of collapse conditions

a Coulomb wedge analysis (Department of Transport, UK)
b Bilineal analysis (Ministry of Transport, France)

frequency (Jones, 1985). However, there are particular difficulties in establishing appropriate constitutive modelling laws for describing the behaviour of reinforcement and soil. Thus such analyses are unlikely to be representative of actual behaviour.

In applying the finite-element method, the most common approach has been to analyse the structure in two dimensions. For purposes of this analysis, strip reinforcements are treated as sheets with equivalent tensile and frictional characteristics. An alternative approach has been to assume that the soil and reinforcement are intimately combined to produce a single material with properties representative of the two components involved. As reinforcement spacings are typically of the order of 0.7m, this latter method is unlikely to provide reliable data on detailed behaviour.

A further difficulty of the finite-element method is that it is not ideally suited for collapse analysis by virtue of the fact that it is a discretized representation of a continuum. Thus the physical separation and slip of soil regions, which is frequently a manifestation of collapse in soil structures, is at variance with the requirement to maintain a system of interconnected nodes and elements. Nonetheless, collapse analysis associated with finite elements is receiving increasing attention and may offer greater scope in future (Szalwinski, 1982). Perhaps the most effective use of the finite-element method for reinforced-soil applications concerns those situations where the analysis would be virtually intractable by conventional methods. One example would be where a reinforced-soil structure is in combination with an imposed superstructure and piled, or similarly treated, foundation. Another might be where both reinforcement and soil have time-dependent properties. Although the finite-element analysis is unlikely to produce reliable detailed behaviour because of the difficulties referred to previously, for purposes of making relative assessments of different designs the method could prove very useful.

The requirements for a satisfactory design of structure have been defined in the report on structural safety produced by the Institution of Structural Engineers (1955). The report stipulates that 'within a reasonable degree of probability' the following conditions should be fulfilled:

- the structure shall retain throughout its life the characteristics essential for fulfilling its purpose without abnormal maintenance cost
- the structure shall retain throughout its life an appearance not disquieting to the user and general public, and shall neither have nor develop characteristics leading to concern as to its structural safety
- the structure shall be so designed that adequate warning of danger is given by visible signs; and that none of these signs shall be evident under design working loads.

The required life of structures can vary significantly, but probably the maximum period is 120 years as specified by the Department of Transport. Such a long period imposes particular difficulties. In addition to the need to make reliable predictions of the extreme variation in environmental and other factors, such as groundwater, loading and climatic conditions, it is essential to have an accurate knowledge of material behaviour, particularly any potential for deterioration. The stability of reinforced-soil structures is dependent on the integrity of the reinforcing elements, and their durability is thus an important consideration in design. Unfortunately, this aspect of reinforced soil raises most controversy and concern over the reliability of the technique.

With metallic reinforcement the risks of corrosion are reduced by selecting fills for the reinforced region that are considered to be non-aggressive on the basis of established corrosion criteria (Peterson, 1980; and French Ministry of Transport, 1979). In addition, a corrosion allowance is provided by increasing the thickness of the reinforcement beyond that needed for tensile-strength requirements and perhaps by also including a protective or sacrificial coating. It is also important to ensure that retained soils are 'isolated' from the selected fills by the use of drainage measures behind and beneath the reinforced earth fill.

The problem of corrosion is avoided by the use of polymers or other forms of plastic reinforcements. However, as such materials have been introduced relatively recently, their performance over long periods has not been reliably established. As the deformations resulting from time-dependent strains can be much more significant with polymeric reinforcement, this can have important consequences for the design.

Thus, because of the difficulties of predicting the performance of metallic and other types of reinforcement in a soil environment over very long periods, there may be problems in ensuring that all requirements for a satisfactory design referred to above are fulfilled. This makes it particularly important to comply with the need to design the structure to show adequate warning of danger and so avoid a catastrophic collapse. In this respect the larger strains associated with polymer reinforcement may be an advantage, as it would be expected that such materials would provide adequate warning of imminent collapse, except in those circumstances where excessive strains were limited to a very short length of material.

This aspect of the design of reinforced soils walls has been discussed by Bolton & Pang (1982), who point out that the present approach may not always provide adequate warning, particularly for the case of metallic reinforcement. As a brittle or rapid failure would be generally associated with rupture of the elements, they have proposed the use of a plastic-yield criterion to ensure adequate plastic distortion prior to collapse.

The greater flexibility of reinforced soil when compared to conventional walls enables the technique to be more widely applicable to soil–structure interaction problems where relatively large movements could occur. Examples of such applications are associated with areas of mining subsidence, construction over compressible soils and in regions of seismic activity.

It has been pointed out by Jones (1985) that reinforced-soil structures are particularly suited to areas of mining subsidence as their susceptibility to damage from compressive strain is very small, although care is needed to prevent tensile strain inducing a tear mechanism of failure (see Fig. 53). A similar argument would apply to construction over compressible soils where the solution for a conventional wall is to install piled foundations or, if not excessively deep, to excavate the compressible materials. Both of these expedients are generally costly, and the reinforced-soil structure will usually offer a much cheaper alternative.

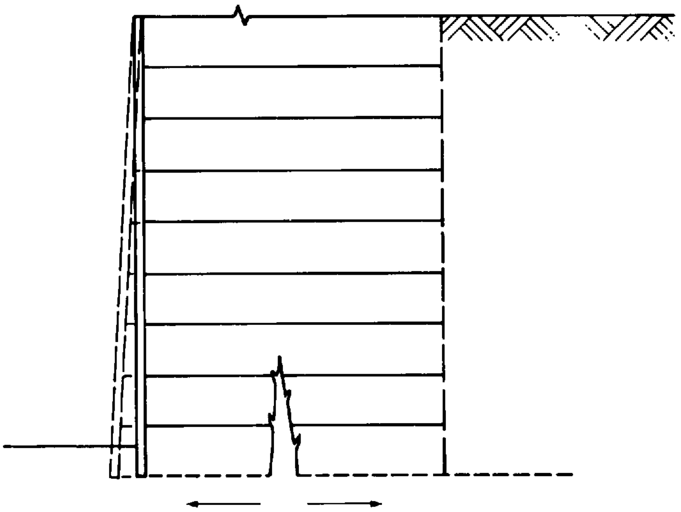
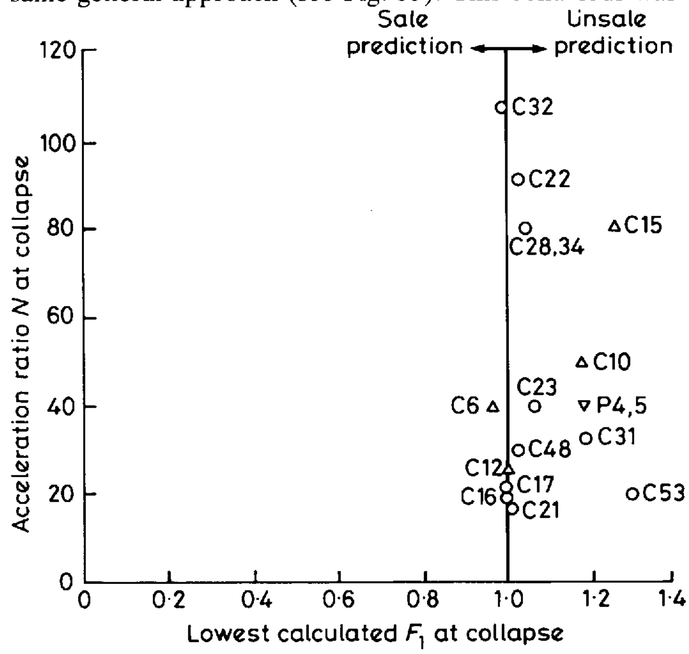


Fig. 53 Mechanism of tear failure resulting from subsidence

Studies of seismic behaviour of reinforced-soil structures in the US have indicated that such structures have performed well, although the importance of ensuring a ductile mode of failure was emphasized (Richardson & Lee, 1974). To ensure this mode of failure it was proposed that the factor of safety against pull-out of the reinforcing elements was smaller than for their tensile rupture. It should be noted that this may occur in any case, however, as the additional metal incorporated to provide a corrosion allowance will significantly increase the tensile strength above the design value until such time as this has been eroded.

At present, superstructure design is generally based on a limit-state approach, whereas the design of substructures is normally based on a limit-equilibrium method employing lump factors of safety, as is the case for reinforced soil. Thus, at the interface between superstructure and substructure, incompatibilities in the design procedures occur that may lead to differences in relative stiffness and thereby accentuate problems of soil–structure interaction.

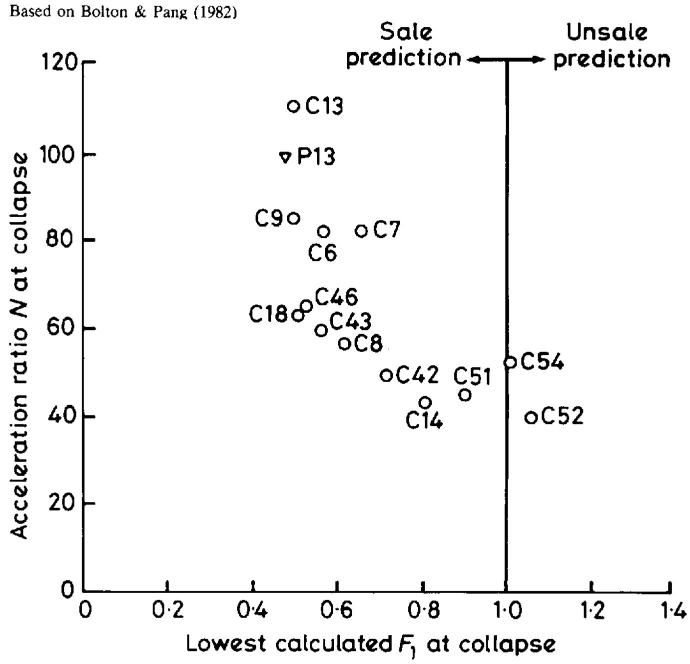
The Transport & Road Research Laboratory has been engaged on research to develop a limit-state method of design of reinforced-soil retaining walls and, in association with this project, commissioned a programme of centrifuge testing to obtain data on collapse behaviour which was carried out by Bolton & Pang (1982) and Bolton *et al* (1978). Prior to carrying out these tests, available information on collapse was almost entirely based on small-scale static model tests at failure and extrapolated data from full-scale structures in their working condition. The study demonstrated that, as regards internal stability in terms of adherence, the collapse limit state could be reasonably represented by a stress equation given in the DTp technical memorandum for the assessment of local stability (see Fig. 54). However, internal stability related to the tensile rupture of metal reinforcing elements significantly underestimated the strength of the system on the basis of the same general approach (see Fig. 55). This behaviour was



Mark I tests (C) ○ SS: L/H = 1·00 (C31) to 0·43 (C48)
△ MS: L/H = 0·75
○ AL: L/H = 0·50
Mark II tests (P) ▽ SS: L/H = 0·80

Fig. 54 Friction failures of walls with flexible facing

Based on Bolton & Pang (1982)



Mark I tests (C) ○ AL: L/H = 1·5 (C6,7,8) to 0·5 (C14,18,51,52,54)
Mark II tests (P) ▽ AL: L/H = 0·5

Fig. 55 Tension failures of walls with flexible facing

Based on Bolton & Pang (1982)

attributed to friction being developed at the base and behind the facing which, in association with soil arching, reduced the tension in the lower layers of reinforcing elements where it is assumed in design that the maximum values are developed. However, because such effects cannot always be relied on, it was proposed that the equation be retained as previously.

This method of assessing a collapse limit state is essentially a lower-bound approach as it involves the determination of a stress field that is in equilibrium and does not violate the failure criteria. Confirmation of this latter condition could prove difficult but, as discussed by Bolton & Pang (1982), may not be such a major drawback as first appears.

It is of interest to note that the present method of designing reinforced-soil walls corresponds to an upper-bound approach and is therefore unconservative, as the mechanism of collapse assumes the simultaneous attainment of full strength of all components at failure.

With regard to the serviceability limit state, the greater flexibility of reinforced-soil walls has been previously referred to, and such structures are therefore less susceptible to damage. However, the construction process may produce large apparent distortions of the unpropped articulated facing. It is usually necessary to compensate for such distortion by constructing with a batter, and this should normally be sufficient to satisfy the serviceability requirements of walls constructed with metal, or other high modulus, reinforcing elements. An exception would be where time-dependent deformations occurred in the foundation soils, and in such circumstances further action, specific to the particular situation, may be required to alleviate the difficulties. Possible actions might include:

- preloading of the foundation soils to reduce their compressibility
- surcharge loading of the reinforced-soil wall prior to constructing the superstructure to reduce post-construction settlement
- use of reinforced-soil foundation mats to reduce differential settlements
- treatment of the foundation soils to improve their performance; such treatment could include the use of stone or lime columns, vertical drainage measures, dynamic compaction, or piled foundations.

Where relatively low-modulus reinforcements are employed such as polymers, there may be additional serviceability problems associated with both immediate and time-dependent strains. At present, there is no generally accepted approach for serviceability-limit-state design, but research on this topic (McGown *et al*, 1986) appears to indicate that the deformation of the structure may not be simply related to the strains in the reinforcement. One explanation is that, because of the significant time-scales involved, there is opportunity for the earth fill to redistribute some, or all, of any additional forces resulting from the creep strain. In some situations both fill and foundation soils may change their properties, particularly with cohesive soils, whereby any loss of stiffness of the reinforcement may be to some extent compensated by the increased stiffness of the soil. However, as no suitable design procedure is available for taking account of any load-transfer mechanisms, the most appropriate solution is to limit the loads carried by the polymer reinforcement to a suitable proportion of their ultimate strength and to see that the anticipated strains comply with the requirement for serviceability.

References

Bell, J. R., Szymoniak, T., & Thommen, C. R. (1984): 'Construction of a steep-sided geogrid retaining wall', *Proc. symp. on polymer grid reinforcement in civil eng.*, paper no. 6.3

Bolton, M. D., Choudhury, S. P., & Pang, P. L. R. (1978): 'Modelling reinforced earth', *Ground Eng.*, **11**, no. 6, p. 19

Bolton, M. D. & Pang, P. L. R. (1982): 'Collapse limit states of reinforced earth retaining walls', *Geotechnique*, **32**, no. **4**, p. 349

Borg, S., Shen, C. K., & Romstad, K. M. (1980): *Analysis of an earth reinforcement system for deep excavation*, Transportation research record no. 769, Transportation Research Board, Washington DC

Department of Transport (1978): *Reinforced earth retaining walls and bridge abutments for embankments*, Technical memorandum (bridges), BE 3/78

French Ministry of Transport, (1979): *Reinforced earth structures – recommendations and rules of the art*, LCPC & SETRA, Paris

Goughnour, R. D., & DiMaggio, J. A. (1979): 'Application of reinforced earth in highways throughout the United States', *Proc. int. conf. on soil reinforcement*, **1**, p. 287

Hollinghurst, E., & Murray, R. T. (in press): Reinforced earth retaining wall at A3/A322 interchange: design, construction and cost, (to be published by Institution of Civil Engineers, London)

Institution of Structural Engineers (1955): 'Report on structural safety', *Struct. Eng.*, **33**, May, p 141

Jones, C. J. F. P. (1985): *Earth reinforcement and soil structures*, Butterworths advanced series in geotech. eng., London

Juran, I., & Schlosser, F. (1978): 'Theoretical analysis of failure in reinforced earth structures', *Proc. symp. on earth reinforcement*, Pittsburgh, ASCE, p. 528

McGown, A., Andrawes, K., & Murray, R. T. (1986): 'The load-strain-time temperature relationship of geotextiles', *3rd int. conf. on geotextiles*, Vienna

Murray, R. T., & Hollinghurst, E. (in press): 'Reinforced earth retaining wall at A3/A322 interchange: instrumentation, site observation and performance and performance' (to be published by Institution of Civil Engineers, London)

Murray, R. T., & Irwin, M. J. (1981): *A preliminary study of TRRL Earth*, TRRL supplementary report 674

Peterson, L. M. (1980): *Pullout resistance of welded wire mesh embedded in soil*, MSc thesis, Utah State University

Richardson, C. H., & Lee, K. L. (1974): *Response of model reinforced earth walls to seismic loading conditions*, National Science Foundation, UCLA – Eng – 7412

Schlosser, F., & Guillaux, A. (1979): 'Friction between soil and strips in reinforced earth structures', *Int. conf. on soil reinforcement*, **1**, Paris, p. 151

Smith, A. K. C., & Wroth, C. P. (1978): 'The fatigue of model reinforced earth walls', *Symp. on earth reinforcement*, Pittsburgh, ASCE, p. 794

Szalwinski, C. M. (1982): *Finite element modelling of kinematic history for particulate media*, PhD thesis, University of Cambridge

Vidal, H. (1969): *The principle of reinforced earth*, Highway research record no. 282, National Research Council, USA

Vidal, H. (1963): Diffusion restpeinte de la terre armée

11 Tunnels and underground openings

11.1 Introduction

In this report an underground opening is defined as one that is excavated from within, as distinct from a buried structure or one that is formed externally. It may be a cavern or tunnel, but excludes immersed tubes, cut-and-cover tunnels, jacked and buried pipes. The concept of 'structure', when applied to underground openings, has a broader sense than is inferred in other situations. For example, an unlined cavern in competent rock is undoubtedly a structure, yet it involves no fabrication or erection of linings.

Further, ground–structure interaction controls almost every aspect of construction and performance in service of underground openings, and the preponderance of load effects on the structure derive from it. The material whose behaviour predominates in tunnelling is obviously the ground within which the opening is formed. There are many types of ground, and its properties vary enormously even at one site, making attempts to codify design procedures extremely difficult. The International Tunnelling Association (1982) has compiled a list of standards.

It is not surprising that tunnelling is widely thought of as an art, in which scientifically based design methods that dominate in the cases of building and bridge structures, for example, are considered inappropriate. However, 'methods' abound, ranging from unashamedly empirical to extremely elaborate theoretical analyses. Some methods are better founded than others, but none can be applied with confidence without understanding.

The objective of this section is to identify aspects of ground–structure interaction and their effect on the construction and design of underground openings and supports.

11.2 The tunnel system

In order to design a tunnel attention has to be paid to:

- operating requirements
- the nature of the ground
- the alignment and profile of the tunnel
- the method of excavation
- the means of support.

The decisions taken on any one of the groups of factors depend on the others. Megaw & Bartlett (1981) discuss the concept of 'systems engineering' to describe the integration of design and construction as a unified process. Acceptable levels of safety, economy, and sometimes feasibility can satisfactorily be assessed only if such a concept is followed.

As the nature of ground in the region of the tunnel is the single most significant factor in the construction of a tunnel, the geotechnical investigation in the proposed locations is of immense importance. It must provide information for the planning, design, and construction and on environmental considerations.

It may be helpful to consider the tunnelling process at this stage. The ground in which the opening is to be constructed is initially effectively in equilibrium under a system of loads resulting from gravity, its formation and its previous history. The geotechnical investigation might reveal that the present equilibrium is only apparent, or it may be unstable, because of the presence of active faults, or potential slip surfaces; in this event very serious problems need to be overcome.

If construction is to proceed, it can be achieved only by destressing and removal of ground material that was part of an equilibrating system. It is rarely, if ever, possible to replace fully, or even substantially, the support to the remaining ground that was provided by the excavated material, and certainly not before excavation. Two important consequences of this are:

- the excavation will reduce the support to the remaining ground by changing its stresses, and
- a new state of equilibrium must be achieved by some means or another.

As well as inducing a concentration of stress around the perimeter of the excavations, an opening:

- will form a drain for groundwater, and
- may induce changes in the properties of the surrounding ground, e.g. the use of explosives in rock will loosen the ground, or the ground might shrink because of drying out or swell in the presence of free water.

The changes arising from excavation and support will generally be time-dependent. Tunnelling in ground whose response to excavation all takes place virtually immediately may be good (in a very strong competent rock) or bad (in loose or weak ground). In bad conditions it may be necessary to use compressed air or a shield that provides continual support to the face and behind it to the point where permanent support can be installed. Alternatively, the ground can be treated in advance of excavation to improve its properties. Usually, time-dependent behaviour allows the constructor to plan a relatively economic procedure, and the support designer can take advantage of the stress redistribution that will occur before the supports are installed.

11.3 Geotechnical investigations

Areas of special concern in geotechnical investigations are listed by the ASCE (1984) under the headings:

- selection of tunnel alignment and profile
- prediction of ground behaviour in tunnelling
- selection of tunnel cross-section and lining
- decision between water-pressure resistance or relief
- excavation and disposal of waste
- ground stabilization
- groundwater problems
- variations in ground conditions
- hazardous conditions
- effects on adjacent structures
- changes in groundwater regime.

The first four headings are for planning and design, the next four for construction methods and the remainder for environmental effects.

The classification of soil and rock is an important part of the investigations. However, a classification system cannot be all things to all men. A description of classification systems is given by the Ontario Ministry of Transportation & Communications (1976). The classification by Terzaghi is very descriptive: hard, firm, slow ravelling, fast ravelling, squeezing, swelling, running, cohesive running, very soft squeezing, flowing and bouldery. The rock-quality designation RQD by Deere *et al* (1969) is based on particle size and fracture spacing (see Fig. 56). Deere

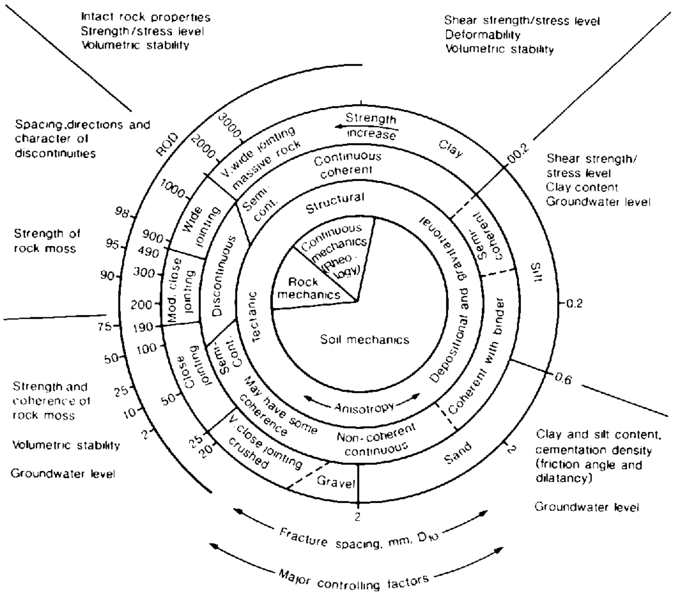


Fig. 56 Unified classification of geologic materials
Based on Deere *et al* (1969)

relates the field of study with type of ground, its structure, particle size and fracture spacing, and outlines the major controlling factors. The effective grain size D_{10} is defined as the mesh size through which 10% of the particles will fall. RQD is the sum of the lengths of all pieces of a recovered core, each over 102mm long divided by an appropriate cored length. Bieniawski (1973) and Barton *et al* (1974) develop this approach to include block size, inter-block shear strength and active stress.

In shales, 'slatey' rocks and softer ground, RQD is less meaningful. The stability ratio, developed by Broms & Bennermark (1967), provides a measure of the behaviour of fine-grained materials. The stability ratio is the vertical overburden stress divided by undrained shear strength. Allowance can be made for the use of compressed air or other measures to reduce the ratio. When the stability ratio exceeds 1 immediate ground support is required.

Classifications help in the planning stages, but are insufficient for design. The limitations of classification and indexing systems are demonstrated by Rutledge (1977), and the requirements of investigations by Cording & Maher (1978). Their importance lies in the insight that a systematic review will give the designer to the likely behaviour of the ground. Geotechnical investigations should be broadly based to provide information for the evaluation of the ground by several classification systems.

The speed and method of excavation will strongly influence the interpretation to be placed on the results of the geotechnical investigation. Another important parameter is the size of the opening. It affects the 'standup' time available for the installation of supports as shown in Fig. 57. Moreover, the effective properties of the ground, by which it interacts with the support system, will be affected by the size of the opening in relation to the spacing and pattern of discontinuities. The value of the geotechnical investigation

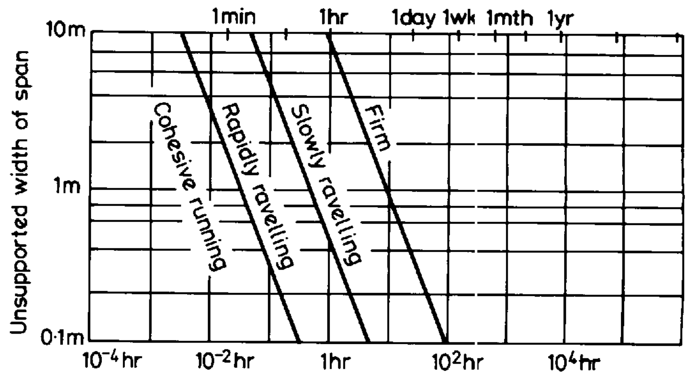


Fig. 57 Standup time
Based on Deere *et al* (1969)

is much enhanced if trial shafts or adits of a size comparable with the planned opening can be undertaken, as appropriate *in situ* tests can give more meaningful results than laboratory tests on core samples.

The art of tunnelling lies in avoiding sudden effects. It follows that the designer seeks from the geotechnical investigation and tests as much information as possible that will help to achieve this objective. This might include creep and consolidation tests, as well as *in situ* tests to estimate water flows. Any of the ground properties may, and often will, be different in different directions.

11.4 Ground–support interaction

The effects of interaction in very shallow tunnels arise mostly from gravity loads on the upper part of the support. In traditional terms the ground above the tunnel loosens and rests on the support (see Fig. 58). The loosening may result from disturbance because of excavation or from volumetric changes in soft material arising from stress changes. Provided that the support does not fail in shear,

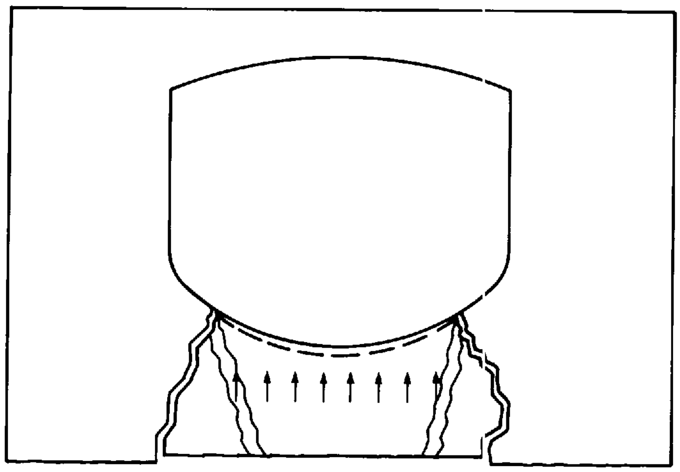


Fig. 58 Loosening loads on a shallow opening

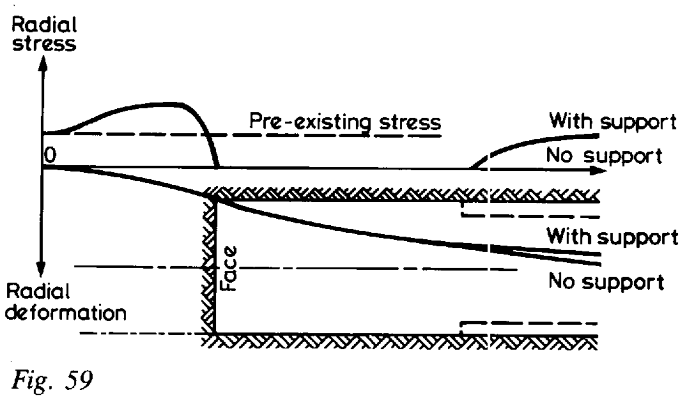


Fig. 59

the load will be resisted by deformation depending on the shape of the opening and the relative stiffness of ground and support.

There is immediate response of ground to excavation. In front of the face, the pre-existing radial stresses, along a line extended forwards from the edge of the excavation, will increase from the pre-existing level, but will be zero at the face. This stress will increase again from the point at which the support is installed (see Fig. 59). Inward radial deformations will increase from nothing at some distance ahead of the face to a significant value at the face, and will continue to increase to a final value at a distance into the support region.

The face does not fully support the surrounding ground. The radial movement ahead of the face is accompanied by a longitudinal movement into the face. As a consequence more material is excavated than is calculated by considering the nominal cut volume of the opening.

11.5 Time-dependent effects

Underground openings that are inherently unstable can often be constructed because the ground will fail only at some time after excavation. It is common to refer to the 'standup' time of the ground (Fig. 57). Several time-dependent effects can contribute to the behaviour of an opening.

In ground beneath the water table, the excavation of an opening forms a drain. The initial movement of the ground may arise because of the change in total stress (effective ground plus porewater pressures), but a flow regime will be established in time as consolidation takes place. The hydrostatic head before excavation is likely to be the depth below the water table. After excavation, the head will be zero or negative, and the head at a distance away from the excavation has to be dissipated through the ground. If the support is relatively impermeable compared with the surrounding ground, the full hydrostatic head will eventually be carried by the support, but the proportions of the effective ground stress it carries might be reduced. The effective ground stresses can themselves be increased by the changing flow regime. In clays and some other soft grounds, consolidation may take place. Harder ground materials may be more or less permeable, but the presence of fissures will govern flow rates and the speed with which a new flow regime is achieved.

Water pressure at the interface between support and ground is shared between them. Supports that are themselves permeable will tend to carry less load than impermeable supports, as the final interface water pressure is less than the full hydrostatic head. The inflow of water needed to reduce the active head can be very small, but arrangements need to be made to accommodate it. The possibility that high water pressures could lead to separation of ground and support over part of the periphery should also be considered.

Both ground and supports may undergo creep. The behaviour of the ground is considered in two parts: an immediate (quasi-elastic) response, followed by a creep response. Any portion of the immediate response carried by the support is found in the same way as has already been discussed, but modifying the modulus values to allow for creep. Some of the creep response may also have occurred by the time the support is installed. For this part, the load carried by the lining is reduced by a factor depending on the creep rate and the elapsed time since excavation. Visco-elastic creep is discussed by Curtis (1976) and Lo & Yuen (1981). Lo & Yuen (1981) contains back-analysis examples for actual tunnels. In some grounds it is difficult to obtain separate quantities for consolidation and creep; in practice it may not be necessary to do so, provided that distinction is made in the geotechnical testing and in the design between total and effective stress behaviour.

Other time-dependent effects include:

- shrinkage of concrete supports
- temperature variations
- changes in behaviour properties of the ground because of the taking-up of water (swelling) or drying-out by evaporation. Swelling occurs as a result of negative pore-water pressures near the periphery of an excavation. Some clays exhibit an ability to take up considerable quantities of water if it is available, and the resulting volume increase can prove troublesome. Tunnels tend to be hot and ventilated during construction, and soft rocks particularly tend to shrink if they are dried. In this case incipient fracture planes will open, and blocks may fall out or water channels be created
- subsequent construction of nearby openings, earthworks or buildings on the surface of the ground.

11.6 Elastic interaction

In an elastic ground that is not affected by time-dependent changes, a support installed sufficiently far behind the face would not carry any load at all. A support placed at the face would not carry all the load that used to be sustained by the excavated ground, however massively stiff it is made. The load the support eventually carries depends on the proportion of the full immediate deformation of the ground that occurs when the support is installed, and on the relative stiffness of ground and support. Although the assumptions of plane-strain elastic behaviour rarely exist in reality, they allow some useful indications of interactive behaviour to be illustrated.

The elastic interaction for a circular opening in a homogeneous ground is discussed by Muir Wood (1975) and Curtis (1976). Such an analysis shows:

- the ratios and radial and tangential stiffness of ground and support are not the same. The ground and support will not in general share an applied load according to a single stiffness ratio.
- the support is far stiffer when resisting radial loadings than under distortional loads. This suggests that the main function of a support is to maintain or control the *circumferential length* of the opening, and that it is likely to be of limited value in maintaining its *shape*.

Consider first the radial part of the load. A given uniform radial pressure P_r applied at the interface between ground and support will be shared between them by simple proportion.

'Convergence-confinement' lines (AFTES, 1978) can be drawn for this case as in Fig. 60. The convergence shows the inward movement of the ground because of the gradual release of the original *in situ* stress P_o. When the deformation is U_o the support is installed and the confinement line

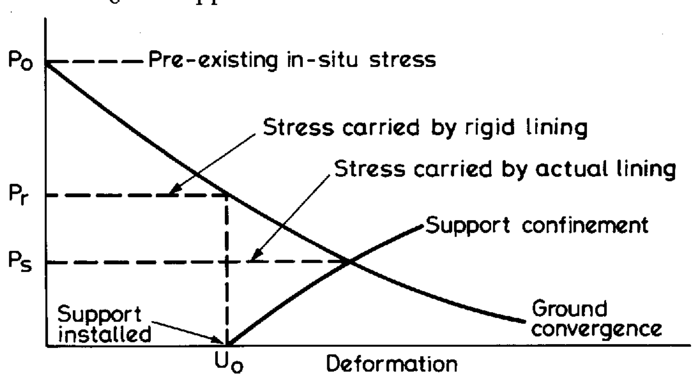


Fig. 60 'Convergence-confinement' lines drawn. The intersection of the lines occurs at stress P_s, which represents the load on the support.

The representation of ground–support interaction in this form is not limited to linear elastic method and is applicable to all kinds of tunnelling.

The distortional effect is more difficult to apportion. In order to ensure compatibility of deformations radially and tangentially between ground and support, the normal and shear components of the distortional stress P_d are shared in different proportions. Fig. 61 shows the proportion of load

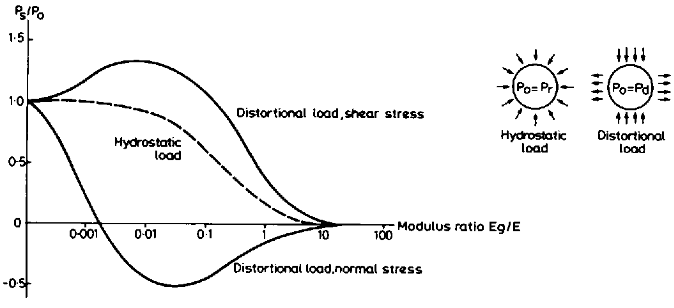


Fig. 61 Applied load carried by the support

carried by the support for a range of ratios of ground and support moduli. The relationship for radial hydrostatic loads is also shown for comparison.

Many real openings are made in the region of Fig. 61 where the shear stress at the interface is greatest. Slip might occur, which would cause a different distribution of the load. Fig. 62 shows the variation of hoop thrust N and

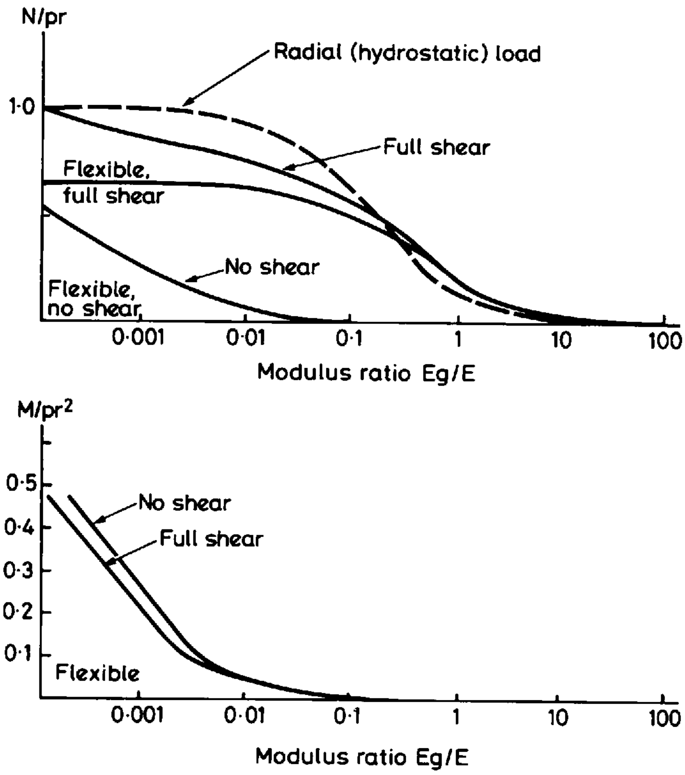


Fig. 62 Hoop thrust, N, and bending moment, M, in support

Distortional loading component, except as noted

bending moment M in the support for the cases full and no-shear interaction. It also shows the effect of supports with no flexural stiffness. In practical terms it might be observed:

- for ground-support modulus ratios of 0.01 or less, the support will carry as a hoop load nearly all of the overburden
- for greater ratios, the reduction because of interaction may be substantial
- if shear cannot be transmitted across the interface, the maximum circumferential force in the support will be reduced
- a flexible support that cannot carry bending moments will attract smaller circumferential forces than one that can carry them
- except at very small modulus ratios the distortion of the opening will be determined by the properties of the ground: a support that is required to limit distortions may need to be very substantial to achieve this purpose.

11.7 Elasto–plastic interaction

A supported opening confines the ground around it so that tangential stresses in the ground tend to be compressive at higher stress levels than existed beforehand. Many types of ground will yield plastically in this situation. It is difficult to obtain reliable field data with which to carry out a meaningful analysis in these cases, but two possible effects of plastic behaviour of the ground should be noted:

- the flow of the ground material will tend towards an equalization of normal pressures on the lining, rather like the 'no-shear' case in elastic interaction
- volumetric changes may lead to increased pressure on the support.

The support system may deform plastically. In an elastic ground, this has the effect of increasing the modulus ratio, which reduces the circumferential force.

11.8 Ground-movement prediction

Tunnelling in soft ground is accompanied by all-round ground movements that are manifest as a surface trough in the form shown in Fig. 63. This Figure indicates the notation adopted for the movements.

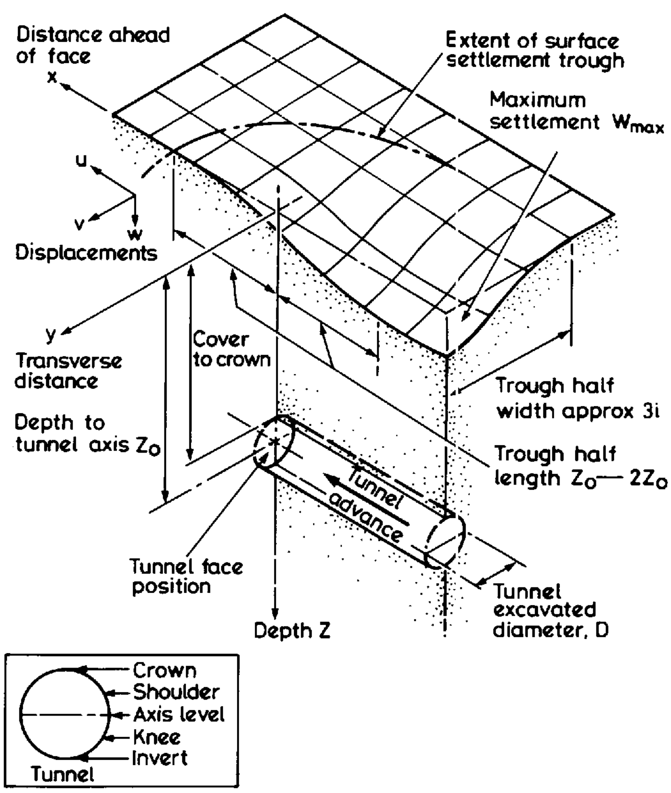


Fig. 63 3-dimensional shape of surface trough and tunnel coordinate system

Based on Yeates (1984)

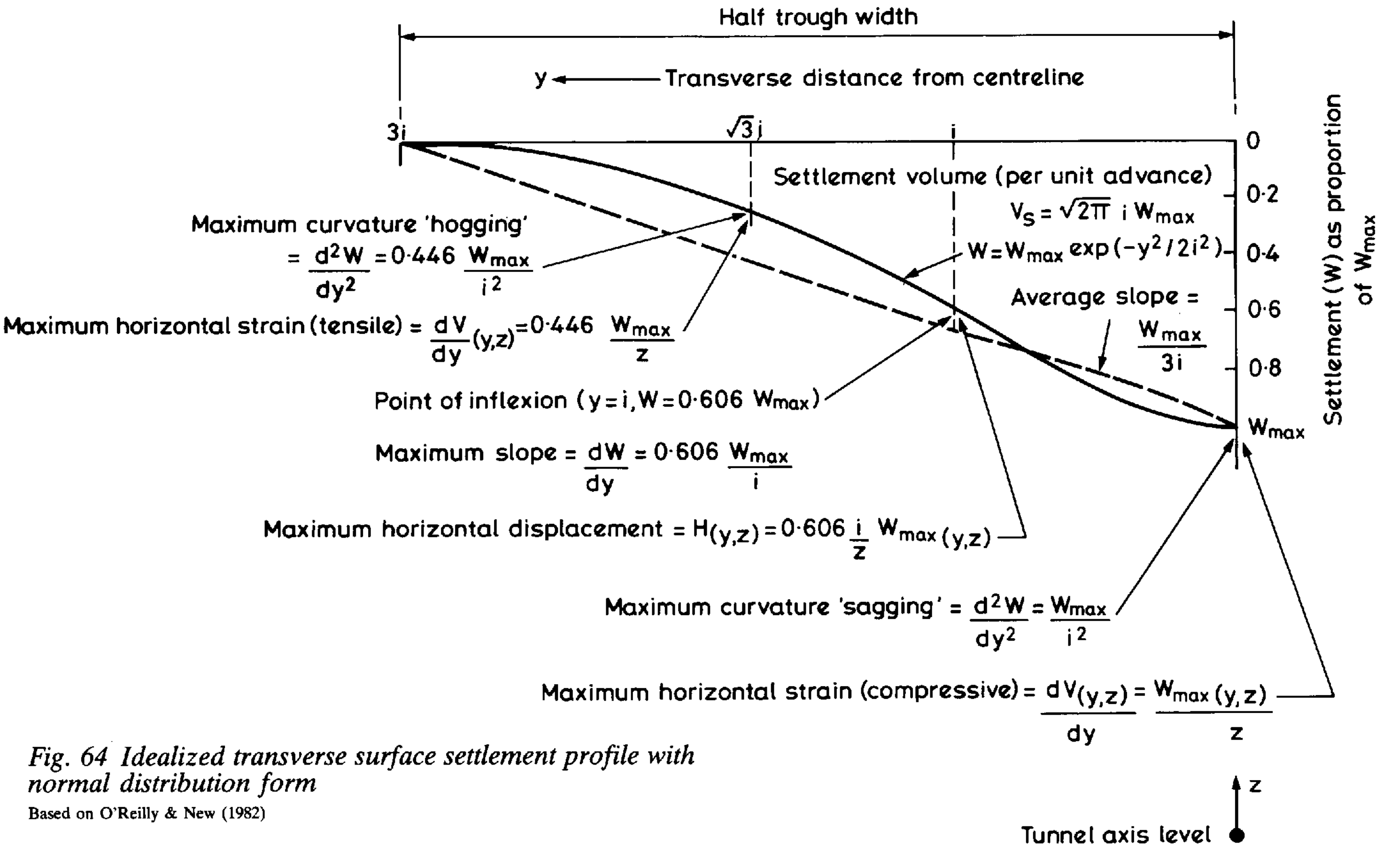


Fig. 64 Idealized transverse surface settlement profile with normal distribution form
Based on O'Reilly & New (1982)

Two phases of movement are recognized: an immediate phase that accompanies excavation and tunnel construction; and a post-construction phase. The latter embraces time-dependent movements arising from consolidation because of porewater pressure changes and creep, but also other factors such as recompaction. The immediate movements can be predicted with reasonable accuracy (by empirical methods based on case histories) but not so the subsequent movements, although as this is generally less damaging to buildings, accuracy is less crucial.

The likely nature and causes of the two phases of movement are given in Table 8.

Some typical values of maximum settlements (w_{max}) for tunnels driven in clay, which include a portion of time-dependent movement (derived from case histories by O'Reilly & New, 1982), are given in Table 9.

It has been observed that at right-angles to the tunnel axis the shape of most surface settlement troughs in the initial stages correspond approximately to an inverted normal Gaussian distribution curve (see Fig. 64).

The surface settlement at any point at a distance, y, from the axis of the tunnel can be calculated from the expression:

$$w = w_{max} \exp^{(-y^2/2i^2)}$$

where w = settlement at point y from the axis of the tunnel
w_{max} = maximum settlement above axis
i = the distance from tunnel centre-line to point of inflection, and for most practical purposes equals half the depth to the tunnel axis, z_o.

Table 9 Some typical values of maximum settlements (w_{max}) for tunnels driven in clay
(from case histories by O'Reilly & New, 1982)

diameter D, m	depth to tunnel axis z_o, m	maximum settlement w_{max}, mm	undrained shear strength C_u, kPa	remarks
4.15	34	5	230	hand excavated and lined with concrete segments within shield in London clay
4.15	20	7	230	
1.26	6.3	44	50	hand excavated and lined (3 segment concrete) within mini-tunnel shield in soft silty clay with sand lenses.
1.26	5.9	56	50	
3.4	6	20	18	hand excavated within shield with compressed air in alluvium
2.7	5.5	60	12	hand excavated and lined with concrete segments within shield with compressed air in marine silty clay (measurement 300 days after excavation)

Table 8 Causes of ground movement

nature of movement	nature of loss	cause of loss
initial	loss of material into the face	elastic and/or plastic yielding or flows of soil
	loss over the shield	poling plates, overcutters, or beads
		over-excavation, ploughing, yawing or negotiating curves
		pushing aside boulders
		build up of grout on tailskin
	loss on or after erection of lining (at tail)	when soil void not completely filled
		delays in erection of lining or in grouting
	loss with time as heading advances	void collapse
		lining deflecting
time-dependent	additional loss with time	recompaction of soil
		consolidation of soil because of reduced pore pressures

Generally, the overall trough width, to the detectable limits of surface settlement, is approximately equal to 6 times the trough width parameter *i*, or approximately equal to 3 times the depth to the tunnel axis.

The above relationships apply most closely to tunnelling clay, and generally the effects of a number of tunnels may be predicted by summation of the individual settlement troughs. In the case of close parallel tunnels, station enlargements or complex interlinking tunnels, predictions based on this approach are less reliable.

In granular soils the settlement trough also exhibits an inverted normal distribution shape, but often with a central enlargement of variable character, caused by loosening or other ground losses concentrated over the crown. The position of the tunnel with respect to the water table is important and will often determine the method of construction, extent of predrainage and type of ground treatment. Hence the position will have a major influence on the settlement experienced.

In any underground construction there is the possibility of catastrophic failure of the face or losses of ground that can occur for a variety of reasons. These are characterized by singular, large, sudden and unrestrained ground movements. Case studies suggest that the highest potential for such losses occurs when unforeseen geological conditions arise (e.g. interfaces between soil types) or the contingency measures provided are inadequate. The importance of assessing the implications and minimizing the risk of catastrophic loss at the design stage cannot be overemphasized.

The horizontal and vertical ground movements are directed towards the centre of the transverse trough and an idealized relationship between these movements and horizontal strain is shown in Fig. 65. The maximum horizontal movements are developed near the point of inflexion *(i)* on the settlement trough, and the ratio of maximum horizontal movement to maximum settlement (v_{max}/w_{max}) is commonly between 0.25 and 0.40.

Towards the margin of the transverse trough the magnitude of lateral movement may be similar or greater than the vertical movement, a factor of considerable significance in assessing risk of damage to structures or services. Discernible lateral movements may have been recorded outside the limit of discernible vertical movements.

Lateral movements at the surface develop corresponding lateral strains, as shown in Fig. 65. The maximum tensile strain occurs at a point about $\sqrt{3}i$ from the trough centre, which is also the point of maximum hogging curvature on the vertical movement profile. Tensile strain and hogging are potentially the most damaging movements. The maximum compressive strain (often two or three times the magnitude of maximum tensile strain) occurs over the trough centre-line.

For the longitudinal profile (see Fig. 66), the limit of detectable ground-surface movement is about 2.5 *i* ahead of the tunnel face, and in some instances a small heave has been noted at this extremity. The maximum slope, curvatures and horizontal strains are significantly less for the longitudinal profile than for the transverse trough. By implication, when considering risk of damage to structures, the transverse trough should represent the worst case. Examination of longitudinal effects may need evaluation where the location or orientation places a particular structure at special risk, for instance, because of torsional effects.

The present evidence from tunnels in clay suggests that the time-dependent phase of ground movements are predominantly vertical, and additional potentially damaging lateral ground movements are unlikely.

11.9 Initial risk assessment

During the development and design of a project involving tunnelling it is necessary to assess the damage that could be caused to existing or planned structures. An early initial assessment of possible effects may enable a scheme to be modified and the risk to structures minimized or avoided. For a large or complex scheme in an urban area with many structures in the area of influence, a simple means is required to assess which structures may be affected and to what degree. Appropriate action can then be determined.

In the first instance it is convenient to adopt the conservative assumption that shallow foundations to structures follow the slope of the ground settlement trough, neglecting any restraint created by the structure. Thus, only a knowledge of the depth, shape and position of the settlement trough in relation to the position of the structure is required.

The actual settlements vary from those predicted, because of such factors as variations in ground properties, method of working and workmanship; and non-uniform settlement results. It is this that causes structural distortion and may be assumed to be proportional to the magnitude of predicted settlement at any given point.

As a basis for classifying structures and services at risk, a combination of predicted maximum settlement and slope can be adopted. The values adopted depend on ground conditions and on the type and condition of the buildings in the area under consideration. Table 10 provides typical values used for planning and design purposes and to optimise the tunnel alignment. However, these values should be considered tentative as there are insufficient case records to confirm their general validity.

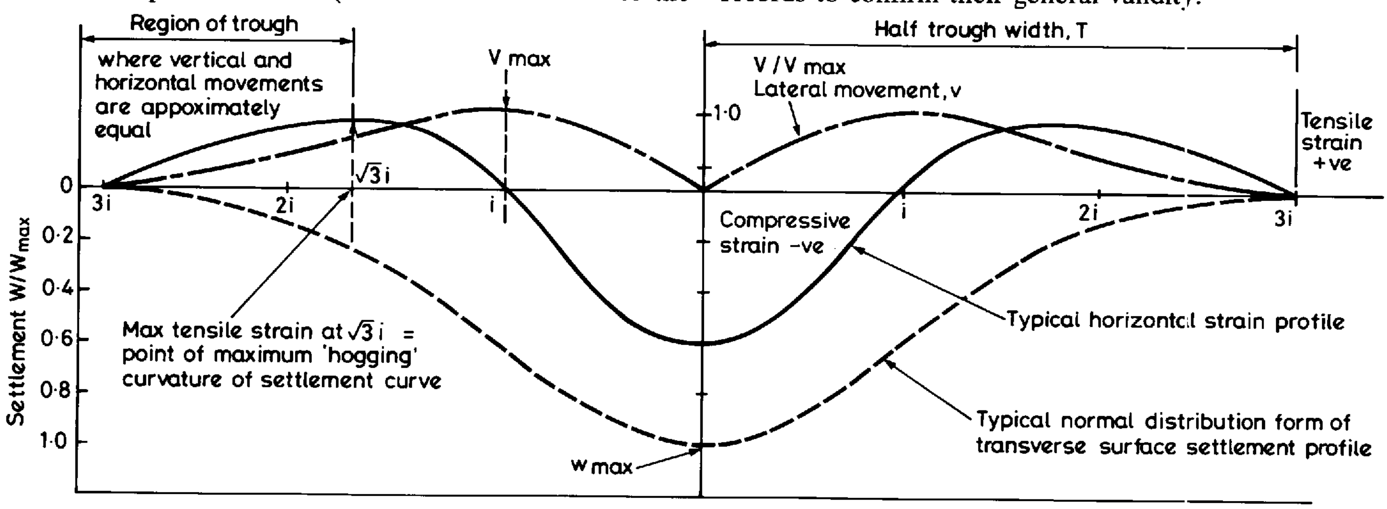


Fig. 65 Idealization of surface displacements shortly after tunnelling

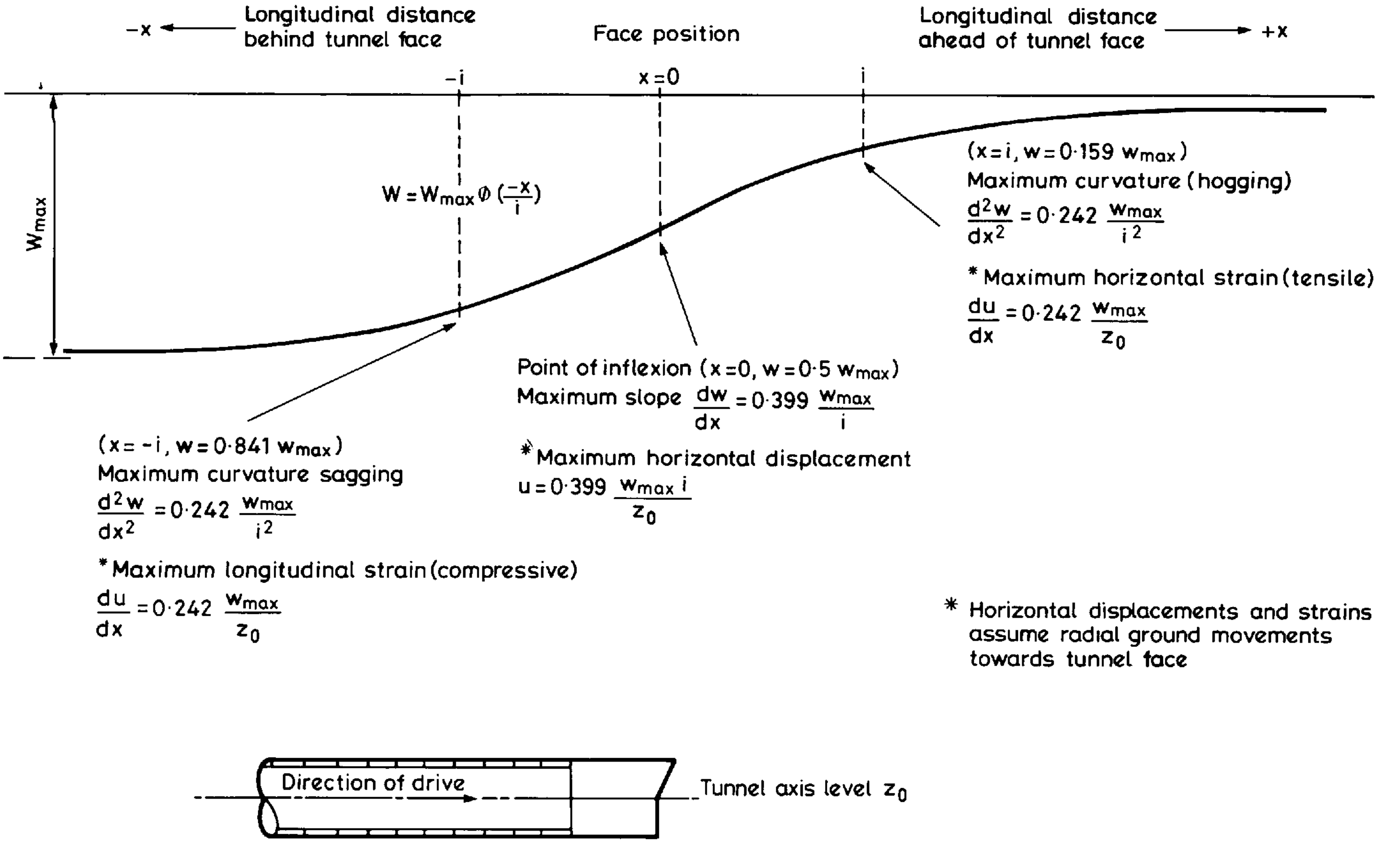


Fig. 66 Idealized longitudinal surface settlement profile with cumulative normal distribution form

The foregoing simplified assessment has been based on the deformation classifications published by Burland & Wroth (1974), Skempton & MacDonald (1956), Meyerhof (1956), Polshin & Tokar (1957), Bjerrum (1963) and O'Rourke *et al* (1976). For a detailed risk assessment, recourse should be to these sources.

Finite-element and other numerical techniques have been used to predict deformations and ground–structure interaction, but with limited success. These techniques have not yet been reliably developed for this purpose and have limited data against which they can be calibrated. Their current application is probably limited to parametric studies.

Factors influencing the allowable additional movement of existing structures include:

- type of movement
- rate of movement
- magnitude and distribution of movement
- type, construction and condition of structure
- interactive soil–structure effects (either reducing of concentrating effects).

Recognizing that ground movements are an inevitable consequence of underground construction, the movements that can be allowed within the structure have to be assessed. The allowable movements can be considered under the following headings:

- safety
- architectural or aesthetic damage
- functional damage
- structural damage
- prevention of repair

The assessment of some of these factors can be subjective, depending for example on personal perception, the geographical/geological setting or the use and status of the structure.

Table 10 Typical values of maximum building slope or settlement for damage risk assessment

risk category	maximum slope of building	maximum settlement of building, mm	description of risk
1	< 1/500	< 10	negligible, superficial damage unlikely
2	1/500 to 1/200	10 to 50	possible superficial damage that is unlikely to have structural significance
3	1/200 to 1/50	50 to 75	expected superficial damage and possible structural damage to buildings, possible damage to relatively rigid pipelines
4	> 1/50	> 75	expected structural damage to buildings and expected damage to rigid pipelines or possible damage to other pipelines

NB The higher risk category for either slope or settlement dominates.

11.10 Stability

The discussion to this point has centred on the primary cause of load distributions – the removal of stressed ground. Interaction between ground and support occurs because of the redistribution of stresses. The strength of the ground and support system directly affects the stability of the opening. A sensibly proportioned opening, properly supported, is essentially a conservative system. The predominant stress field is compressive, and the structure (the ground) vast in extent. In other structures, loads required to propagate failure fall off from those that initiated failure, and their ability to absorb energy is finite. Materials failing in compression can continue to carry high loads, subject only to the need to offer support to prevent a sudden change in geometry; the support force might be only a few percent of the forces in the structure. The large extent of the ground also offers the possibility that local failure could be halted when sufficient ground becomes involved to regain equilibrium. The practical description of this behaviour is known as the 'arching effect'.

Ground materials possess a tensile strength that is much lower than their compressive strength. This is true even in strong rock, as fissures or planes of weakness will be found. The shape of the excavation will be chosen to avoid tensile regions. However, a tensile region of small extent, giving rise to a simple crack, may be of little significance to overall stability.

The existence of a compressive region at failure will be important, but if the region is small, the load may be thrown away from the excavation to a region where some confining radial pressure exists to restore stability. The conflicting requirements of stress and stability design are discussed by Daemen & Fairhurst (1978). This describes how the optimum tunnel shape in a predominantly vertical stress field is, for stress design, an ellipse with its major axis vertical. This results in a substantial region in which the compressive ground stress is large. For stability, if the ellipse were orientated with its major axis horizontal, the peak compressive ground stress would be larger, but of much smaller extent. The practical consequences of this illustration are that large radius excavations, particularly those with flat sides, should be avoided in highly stressed ground.

There is a trade-off between the ability of the ground to form a stable arch and the consequence that this will in general increase the size of the opening. Ground remote from the plastic region is affected by this.

Although stress analysis shows that sress concentrations around tunnels are little affected by tunnel size (in an iostropic elastic medium the stress change is entirely independent of diameter and elastic properties), it is common experience that the risk of failure increases with excavated diameter. The crown, or roof, of an opening is more at risk than any other part of the excavation, as gravity reduces the confining effect of the stress concentration arising from the opening. Although gravity assists stability at inverts, it will not necessarily prevent failure there, as a flat invert may be inappropriate in many cases.

Even a fractured or broken ground can carry considerable loads. Its ability to do so depends on the amount of movement that is permitted or caused by the tunnelling process.

Design for stability is chiefly a matter of providing just sufficient support to prevent a shear failure around the excavation. An interaction analysis can be undertaken to establish beforehand the amount of support required, but often, as in the 'new Austrian tunnelling method' (Rabcewice & Golser, 1973), convergence is monitored *in situ*, and the support requirements (thickness of shotcrete, for example) determined as construction progresses. Care is needed to ensure that the support will not itself fail in shear.

11.11 Analytical methods

The complexity of the tunnelling system, and the variability of the ground, render the use of analytical methods less reliable than for most other types of structure. Empirical methods have been developed to cover a wide range of circumstances. In softer grounds, the assumption is made that the support will carry all or nearly all the total overburden pressure, and the design is completed by an estimate of the deformations to be expected. In harder grounds, the various rock quality indexes are employed to derive a volume of rock to be supported. Considerable sophistication has been developed, and methods are described in some of the listed references.

Continuum analysis methods are now in common use, especially in linear elasticity. The methods range from simple beam-and-spring models to finite-element analysis incorporating many elaborate features to model bedding, fracture planes and other features. Closed-form solutions of the kind employed in the examples of this report exist only for a limited number of cases, but they are useful for parametric studies. The extension of finite-element methods to 3-dimensions, the time domain and plasticity is mostly limited to research, owing to the difficulty of obtaining values for ground behaviour parameters with sufficient accuracy. The importance of the 3-dimensional history of stress changes caused by tunnelling is discussed by Lo (1984).

The advantage of analytical methods is the possibility of estimating realistically the factor of safety in any situation. The disadvantage is that undue credibility can be given to the results without first appreciating the limitations of the assumptions inherent in the modelling of the problem, and then taking sufficient care in the interpretation of the results. The state of the art seems to be that back-analysis of actual tunnels can be achieved with the ever-increasing catalogue of tools, but prediction remains the prerogative of the brave. For this reason the usual design procedure is:

- to employ more than one predictive method
- to obtain as much information by exploration and testing at the site as can be afforded
- to compare the proposed design with previous experience.

11.12 Support types

The type of support selected for an underground opening will be influenced by the operational and construction requirements, as well as considerations of ground–structure interaction. The shape of the opening is particularly important. As far as possible, excavated surfaces should be curved to provide stability. A circular cross-section for tunnels is the best arrangement in many situations as it offers both stability and the design of efficient supports, and can be advanced rapidly with mechanized excavation methods.

Types of support include:

- *In situ concrete.* Development of efficient concreting techniques can be expected to overcome some of the disadvantages of this type. From the design aspect, stability of the excavation before placement of the concrete may require an initial support system. Advantage may be taken of shrinkage and creep of the concrete to delay or reduce the load carried in the lining
- *Segmental linings.* These may be concrete (often reinforced, especially at the radial joints), spheroidal-graphite cast-iron, or steel. Segmental linings can be bolted together or articulated, and installed by expanding them against a smooth-bore excavation. It is often desirable to grout behind the linings to ensure uniform contact with the ground. If it is necessary to make this lining watertight, special seals or caulking add to the costs
- *Steel ribs, blocked against the ground.* This form is adaptable to many excavation methods and ground conditions but requires some form of support to the ground between the ribs. The ribs may later be encased in sprayed or cast *in situ* concrete

- *Sprayed concrete.* Techniques for spraying concrete are improving and the system is well suited for initial support, except behind full-face tunnelling machines. It may be applied in several passes, allowing deformation to occur before the final thickness is achieved
- *Rock bolting.* Several forms of bolting are employed, from simple dowelling to prevent blocks of rock from becoming detached, to elaborate means of applying radial pressure to increase the stability of the excavation. Bolting may be employed as an initial support to be supplemented by other systems
- *Unsupported rock.* In very-good-quality rock, no permanent support system may be necessary. The surface of the excavation may be sealed to prevent long-term deterioration.

There are many variations and combinations of support systems (Craig & Muir Wood, 1978). Whatever other factors are involved, the designer has to provide a system that allows the opening to be constructed safely in the first instance, and which will not be forced to carry more load than is necessary to meet operational requirements.

11.13 Design methods

Design has usually involved the estimation of some loads, possibly in conjunction with assumed deformations, followed by the sizing of supports using an arbitrary factor of safety. The true factor of safety is then impossible even to guess. It appears that this approach leads to nominally rather conservative support systems, which is just as well since principles employed in the detailed design of other types of structure are often ignored. Supports dimensioned on this basis are justified if they perform successfully, and when designed by those whose experience of similar tunnelling situations is wide, they probably will. More rational approaches are possible. Designers like to carry out trials before committing themselves irrevocably. Since a trial can provide a great deal of information on suitable excavation methods, and gives access to the ground for geotechnical testing as well, it is sensible to undertake one whenever doubts exist. Another approach is to monitor the behaviour of the opening as it is excavated, and to adapt the support system to suit (as in the new Austrian tunnelling method). Sometimes openings have to be fully designed before any substantial excavation can be made, however, but rationality need not be abandoned entirely. Two principles can be adopted:

- determine limit states for the load effects in the support, using interaction methods and having regard to stability requirements
- design the support system as an engineering structure. The largest load that could possibly occur can be estimated; it will represent a factor on the largest load that probably would occur (the working load). Factors can be allowed for uncertainties of calculations, fabrication and erection tolerances, and also for material strengths. The design is then based on the collapse of failure mechanisms, rather than working stresses. The behaviour of the system at working loads has to also be investigated to ensure that unacceptable cracks and deformations will not be generated. A design based on rational procedures may not be safer than another, but it should be more efficient. Its safety still depends on the adequacy of the information available to the designer, and the quantity and quality required can be greater than for the application of empirical methods.

11.14 Caverns

The major differences between a cavern and a tunnel are:

- a cavern is generally much larger in section; the size effects are much more pronounced, so that there is less margin for error in design
- the excavation and support are usually carried out in several stages; there is a redistribution of stress at each stage, and the ground at a point on the periphery may be disturbed at each stage
- mechanization is more suited to a long excavation of a constant section than for cavernous shapes; methods employed tend to be slow and to disturb the ground more.

11.15 Multiple openings

The excavation of an opening close alongside another has two effects on the first:

- it increases the load carried by the support, and
- it increases its distortion.

The amount of these increases depends on the distance apart of the openings and the ground properties. A clearance of half a tunnel diameter will limit the increases to about 20% or 30% in firm clays, and less in stronger material. Special measures are required if openings are to be closer, and attention has to be paid to the stability of the ground remaining between them.

An opening may be shielded from another if they lie along an axis in the direction of the principal compressive stress in the ground.

11.16 Intersections

Square intersections between tunnels of significantly different diameters are relatively straightforward to build. Some load will be relieved from the support of the major tunnel, and this will have to be carried by adjacent sections. Where the tunnels are of similar size, the intersections will effectively create a much larger span, and it may be difficult to redistribute loads to permit safe construction. The ground in the vicinity of the first constructed tunnel will then be more highly stressed than initially; construction of the opening will cause further redistribution, increased ground stresses and a complex stress field.

11.17 Commentary

The objective of this section will have been met if it provides an insight into the complex interactions between an underground opening, its manner of construction, and the surrounding ground. The broad subject-matter precludes the inclusion of design charts and rules, but the selection of the design method appropriate for a given case requires an understanding of all the factors that could influence the choice. Most of the given references themselves contain excellent bibliographies. In particular, Megaw & Bartlett (1981), ASCE (1984), Ontario Ministry of Transportation & Communications (1976) and Daemen & Fairhurst (1978) provide summaries and extensive overviews of the problems encountered and the means of overcoming them.

Descriptions and a comparison between two particularly interesting tunnels, the Anglo-French Channel Tunnel proposal and the Japanese Seikan Tunnel, are given in Chapter 10 of Megaw & Bartlett (1981). Few tunnels are completed without unexpected difficulties, and the Seikan Tunnel was no exception. Megaw & Bartlett (1981) contains bibliographies of accounts of the design and construction of many tunnels. Examples of case histories are also given by Cording & Maher (1978), Lo & Yuen (1981), Daemen & Fairhurst (1978), Lo (1984) and Craig & Muir Wood (1978).

References

AFTES (1978): 'Analysis of tunnel stability by the convergence-confinement method', *Proc. Paris conference* (English translation , in *Underground space*, Pergamon Press, **14,** nos. 4, 5 & 6, 1980)

ASCE (1984): *Guidelines for tunnel lining design*, ed. T. D. O'Rourke

Barton, N. *et al*, (1974): 'Engineering classification of rock masses for the design to tunnel support, *Rock mechanics*, **6,**

Bieniawski, Z. T. (1973): 'Engineering classification of jointed rock masses', *The civil engineer in South Africa*, Dec.

Bjerrum, L. (1963): 'Evaluation of allowable settlements', *Proc. European conf. on SM & FE*, **2,** p. 135

Broms, B. B., & Bennermark, H. (1967): 'Stability of clay at vertical openings', *Jnl. of Soil Mech. and Found. Div.*, ASCE, SM 1

Burland, J. B., & Wroth, C. P. (1974): Settlement of buildings and associated damage, *Proc. conference on settlement of structures*, University of Cambridge, p. 611

Cording, E. J., & Maher, J. W. (1978): 'Index properties and observations for design of chambers in rock', *Eng. Geology*, **12,** no. 2

Craig, R., & Muir Wood, A. M. (1978): *A review of tunnel lining practice in the United Kingdom*, TRRL supplementary report 335

Curtis, D. J. (1976): Discussion on reference 9, *Geotechnique*, **26,** no. 2

Daemen, J. J. J., & Fairhurst, C. (1978): *Ground/support interaction – fundamentals and design implications*, Appendix K of *Tunnels and shafts in rock*, US Dept. of the Army Corps of Engineers, Report EM 110-2-2901

Deere, D. U. *et al* (1969): *Engineering classification of in situ rock*, Tech. rep. no. AFWL-TR-67-144, Air Force Weapons Laboratory, Albuquerque, NM, USA

International Tunnelling Association (1982): *Views on structural design models for tunnelling: advances in tunnelling technology and subsurface use*, **3,** no. 2, Pergamon Press

Lo, K. Y. (ed.) (1984): 'Tunnelling in soil and rock', *Proc. of geotech. III speciality conference*, Atlanta, Georgia, ASCE

Lo, K. Y., & Yuen, C. M. K. (1981): 'Design of tunnel lining in rock for long term time effects', *Canadian Geotech. Jnl.*, **18,** no. 1

Megaw, T. W., & Bartlett, J. V. (1981): *Tunnels: planning, design, construction*, Ellis Horwood

Meyerhof, G. G. (1956): Discussion on paper by A. W. Skempton & D. H. MacDonald. *Proc. ICE*, pt. II, **5,** p. 774

Muir Wood, A. M. (1975): 'The circular tunnel in elastic ground', *Geotechnique*, **26,** no. 2

Ontario Ministry of Transportation & Communications, Canada (1975): *Tunnelling technology*, Golder Associates & James F. Maclaren Ltd.

O'Reilly, M. P., & New, B. M. (1982): 'Settlements above tunnels in the United Kingdom – their magnitude and prediction', *Tunnelling '82*, Inst. Min. & Met., London, p. 173

O'Rourke, T. D. *et al* (1976): *The ground movements related to braced excavation and their influence on adjacent buildings*, Report to US Dept of Transportation, Washington DC, by University of Illionois, Dept. of Civil Eng.

Polshin, D. E., & Tokar, R. A. (1957): 'Maximum allowable non-uniform settlement of structures', *Proc. 4th int. conf. on SM&FE*, **1,** p. 402

Rabcewice, L., & Golser, J. (1973): 'Principles of dimensioning the support system for the new Austrian tunnelling method', *Water power*, March

Rutledge, J. C. (1977): 'Engineering classifications of rock for the determination of tunnel support', *Proc. of the Technical Groups, Tunnelling in New Zealand*, NZIE

Skempton, A. W., & MacDonald, D. H. (1956): 'Allowable settlement of buildings', *Proc. ICE*, **5,** p. 727

12 Buried structures

The contribution of soil–structure interaction of the loading on, and behaviour of, buried structures was recognized in design since research began in the early 1900s (Marston & Anderson, 1913; Marston, 1930; Spangler, 1933; and Spangler, 1941). Other design methods (White & Layer, 1960; and Kloppel & Glock, 1970) were developed in the interim period before modern analytical techniques (Katona *et al*, 1976; Duncan, 1979; CIRIA, 1978; and Murray, 1974) provided rigorous solutions.

Research has concentrated largely on rigid concrete culverts/pipes and flexible corrugated metal culverts, and this section explains the effect of the stiffness of those types of buried structure on performance in service.

12.1 Stiffness

The two extremes of structural stiffness – rigid and very flexible – are typified in buried structures by thick-walled reinforced-concrete culverts and thin-walled corrugated-steel culverts, respectively.

Rigid structures are designed such that permissible material stresses are not exceeded, deflections generally being acceptable. Conversely, one of the main design criteria for flexible culverts is deflection under working load. The ability of flexible culverts to deflect under load alters the magnitude and distribution of soil pressure on the culvert, and prediction of behaviour requires an understanding of interactive effects.

Although rigid structures do not distort significantly under load, their presence in the soil causes interaction in the form of a redistribution of stresses around a buried structure from that which would be expected in a free-field situation. The determination of the stress field around a rigid culvert is relatively simple compared with that of flexible culverts where the complication of significant distortions of the culvert results in associated changes in the distribution of pressure. There are two basic reasons for soil–structure interaction:

- the introduction of a culvert into a soil mass. This results in an altered stress field around the culvert because of the difference in stiffness and weight of the culvert from the soil that it replaces, and
- the distortional behaviour of culverts, especially of flexible types, which results in a redistribution of pressure as an equilibrium condition between the surrounding soil mass and the culvert is achieved.

Other factors affecting the interaction of a culvert–soil system are:

- shape of culvert
- loading condition
  - dead loads
  - imposed loads
  - temperature effects
- construction techniques
- relative stiffness of foundation soil, embankment fill, etc.
- bedding condition
- longitudinal settlement effects.

The complexity of the problem provided by a combination of these factors results in a soil–structure system that cannot be analysed properly using traditional theories of earth pressures and structural analysis.

Since one of the main criteria governing soil–structure interaction is the stiffness of the culvert, it would appear convenient to classify structures as either rigid or flexible. However, there is no sharp division between the two classifications, and it is necessary to introduce an intermediate classification. A brief definition of each class of structure is given below.

12.1.1 Rigid structures

Reinforced-concrete structures with relatively thick walls and roof slabs are in this category. The standard approach to design criteria is to provide a structural section sufficient to resist axial, flexural and shear forces rather than only pure axial forces. This approach generally results in fairly substantial structural sections, especially under deep cover, and deformations are normally small, thus minimizing the degree of soil–structure interaction. In soil–structure systems incorporating rigid culverts, both the axial and flexural stiffnesses of the culvert will be dominant and far in excess of the stiffness of the surrounding soil mass.

12.1.2 Flexible structures

This category will contain structures with little resistance to bending, the structural integrity of the system being mainly dependent on the confining capability of the soil mass around the culvert. Flexible culverts will usually consist of thin-walled corrugated-steel structures. Handling and construction of culverts in this category could present a problem because of their inherent lack of stiffness. For this reason, manufacturers of corrugated-steel structures do not recommend flexibilities greater than those that give satisfactory handling performance. This rule allows construction to take place without large deflections or the necessity for any internal bracing or propping.

12.1.3 Intermediate-stiffness structures

Structures in the intermediate-stiffness category fall between the two extremes of rigid and flexible and are more difficult to define since they range from stiff but not rigid structures to fairly flexible structures that have satisfactory handling criteria. Towards the rigid limit of this category, it would be expected that culverts would still have similar bending and shear capacities as rigid types, but that deflections may now be of significant proportions resulting in a degree of soil–structure interaction. Towards the flexible limit of the category, culverts of intermediate stiffness will have relatively low flexular strength and will rely predominantly on the surrounding soil mass for confinement to provide an adequate structural performance. The design of these structures will be aimed at constructing an installation that will be able to maintain the original shape of the culvert with minimal deflections.

To some extent, categorizing of structures will also depend on the soil stiffness, since the complete installation relies both on the stiffness of the soil for support as well as the inherent strength of the structure.

12.2 Longitudinal settlement effects

In many situations, settlement along the length of a culvert will introduce a longitudinal effect, and it is necessary to ensure that the culvert can withstand the associated stresses and strains or to incorporate flexible construction joints to reduce the effects. Longitudinal effects may be introduced by a variation in the loading on the culvert, either dead or imposed loads, or by variable bedding or foundation soil support. Either of these factors can result in an irregular longitudinal deflection profile.

Probably the most common situation in which differential vertical settlement of culverts occurs in highway construction results from their installation beneath embankments. Large differential settlements occur beneath high embankments over soft alluvial soils, and large relative deflections are experienced along culverts.

A rational approach to the design of culverts where 3-dimensional effects are likely to be significant is to carry out a traditional settlement analysis for the proposed line of the culvert to estimate the magnitude of the differential movements that may be expected if the culvert was not present. This analysis will tend to indicate the worst differential settlement environment in which the culvert is likely to exist. It will also provide data that will aid the designer in choosing between a reinforced-concrete or a corrugated-steel structure. Computer programs such as those developed by Murray (1974) are capable of providing settlement profiles beneath embankments to assist designers.

In view of the varying response and distribution of stresses caused by longitudinal effects on flexible-corrugated and reinforced-concrete structures, the 3-dimensional behaviour of these two types of structure will be discussed separately.

12.2.1 Corrugated-steel culverts

Because of the flexibility of these structures and the corrugated nature of their plates, it is known that, when subjected to general differential movement along their length, they will deflect longitudinally to conform with the surrounding environment. This deflection will impose flexural stresses arising from the change in longitudinal curvature of the culvert. The magnitude of these stresses should be considered in conjunction with the combined in-plane hoop stresses calculated, assuming 2-dimensional behaviour. The capacity of the circumferential bolted connections should also be adequate to resist the bending moment induced in the culvert.

In addition to the longitudinal deflection of a culvert arising from differential foundation settlements, culverts installed under embankments will also experience longitudinal tensile strains caused by lateral spread of the foundation soils. The problem of lateral movement is particularly significant in soft alluvial or organic soils. As an embankment or foundation soil spreads laterally, frictional resistance on the culvert walls will transfer longitudinal tensile load to the structure, whose magnitude will increase from zero at the extremity of the embankment to a maximum at the centre.

Depending on the anticipated magnitude of lateral spread and the theoretical tensile load that can be transferred to the culvert by frictional resistance, there are two alternative solutions. First for cases where only small extensions are expected and where the total tensile load is manageable, the local capacity of the circumferential bolted joints should be designed to withstand the induced loading. Secondly, where the total lateral movement precludes the foregoing solution, the culvert can be constructed in discontinuous sections that are sufficiently overlapped such that relative slippage between adjacent sections may occur without endangering the continuity of support to the surrounding fill.

When considering the adoption of an overlap system to allow relative slip of adjacent culvert sections, it should be appreciated that the movement may not occur slowly or uniformly as the embankment spreads because of the corrugations in the steel plate. Instead, tensile forces will build up until sufficient energy is available to cause the overlapped sections to rise up over the crown of the corrugation at which point there may be a relatively sudden movement and release of tensile force. The magnitude of the force required to cause this movement will depend on the normal soil pressures around the pipe and the frictional resistance between sections. Even although these movement joints are incorporated in a structure, the capacity of the circumferential bolted seams of the individual sections should still be checked for the anticipated longitudinal tensile loadings.

12.2.2 Reinforced-concrete culverts

Unlike corrugated-steel structures, reinforced-concrete structures are very rigid longitudinally because of their box-type construction. Because of the stiff nature of these structures compared to the foundation media, the culvert will tend to act as a beam when subjected to differential longitudinal settlement, rather than deflect in sympathy with the foundation. In designing reinforced-concrete culverts, it is therefore generally necessary to choose between introducing regular movement joints to allow articulation of the structure or to resist the additional stresses arising from the beam action.

The introduction of a reinforced-concrete culvert through an embankment that is likely to experience differential settlement will significantly alter the loading pattern on the culvert from that predicted by 2-dimensional analysis. A thorough analysis of the interaction involved between the embankment, culvert and foundation soil would be possible only by utilizing a 3-dimensional finite-element foundation, involving considerable computer time and expense. It is therefore necessary to assess the likely effect of differential longitudinal settlement on the loading pattern without the benefit of detailed analysis.

Since a concrete culvert will generally possess sufficient stiffness and strength to prevent large longitudinal deflections, it will offer additional support to the embankment at the areas of maximum settlement and will tend to span over these areas and create a stress redistribution along the culvert. This will result in non-uniform base pressures beneath the culvert, with lower pressures where the embankment is being supported and higher pressures towards the ends of the culvert.

It should be appreciated that the interaction induced by the longitudinal differential movements will affect the vertical loading on the culvert. It is particularly important to appreciate that the pressures on the roof slab may be significantly higher than those experienced by the base at the midspan of the culvert. The reverse is true at the end.

The design of a reinforced-concrete culvert to withstand the additional loading arising from differential longitudinal settlement should be carried out in the following manner:

- identify section or sections of culvert likely to experience differential longitudinal settlement because of variation in applied and/or foundation support
- assess the increase in the culvert roof loading because of

the resistance to settlement by the beam effect and design the cross-section in this area for this higher loading
- design the areas of the culvert subjected to higher base loading accordingly
- finally, design the culvert to resist the force induced by the beam action, with sufficient reinforcement to cater for the longitudinal bending and shear forces.

Depending on circumstances, it may not be economic to design the structure to resist the additional loads that are attracted because of differential longitudinal settlement. In these instances it may be practical to introduce discontinuities in the culvert structure in the form of movement joints. If movement joints are introduced, the effect of lateral embankment spread causing tensile strains in section of the culvert should still be recognized, and their effects on the integrity of any movements joints should be assessed.

References

CIRIA (1978): *Design and construction of buried thin-walled pipes*, CIRIA report no. 78, July, London

Duncan, J. M. (1979): 'Behaviour and design of long-span metal culverts', *Proc. ASCE, Jnl. Geotech. Eng. Div.*, **105**, no. GT3, p. 399, March

Katona, M. G., Smith, J. M., Odello, R. S., & Allgood, J. R. (1976): *Cande – a modern approach for the structural design and analysis of buried culverts*, Federal Highway Administration, Washington DC, Final report no. FHWA-RD-77-5, Oct.

Kloppel, K., & Glock, D. (1980): 'The dimensioning of embedded flexible pipes of the type Armco multiplate based upon the load bearing capacity 1974', Summary and extracts from *Theoretische und experimentelle Untersuchunger zu den Tragiastprobleme biegeweichen in die Erde eingebetteter Rohre;* K. Kloppel & D. Glock, 1970

Marston, A., & Anderston, A. O. (1913): *The theory of loads on pipes in ditches and tests of cement and clay drain tile and sewer pipe*, Iowa Engineering Experiment Station Bulletin 31

Marston, A. (1930): *The theory of external loads on closed conduits in the light of the latest experiments*, Iowa Engineering Experiment Station, Bulletin 96, **28**, no. 38, Feb

Murray, R. T. (1974): *Two dimensional analysis of settlement by computer program*, TRRL report LR617

Spangler, M. G. (1933): *The supporting strength of rigid pipe culverts*, Iowa Engineering Experiment Station, Bulletin 112, **31**, no. 37, Feb.

Spangler, M. G. (1941): *The structural design of flexible pipe culverts*, Iowa Engineering Experiment Station, Bulletin 153, **40**, no. 30, Dec.

White, H. L., & Layer, F. R. (1960): 'The corrugated metal conduit as a compression ring', *Proc. Highway Research Board*, Washington DC, **39**, p. 389

13 Conclusions and recommendations

1 It is important to promote an awareness of the mechanism known as soil–structure interaction and to encourage research into the real behaviour of structures.

2 Techniques are now available for executing static and dynamic interactive analyses. In exceptional circumstances rigorous analyses may be required to determine load distributions within structural systems. Such analyses should be used with considerable caution and with an appreciation of the many idealizations made. Future analytical studies should focus on the need to provide relatively simple aids for routine design rather than the provision of novel but complex mathematical solutions.

3 Studies of the performances of building structures should be initiated as a routine procedure. Such studies in conjunction with the preparation of the records of development of cracks and other forms of damage would provide valuable quantitative information on the relationships between relative movement, damage, serviceability and function. Structural damage arising from foundation movements is only one aspect of the wider problem of serviceability. Frequently the problem of accommodating differential foundation movements, as with creep, shrinkage and structural deflections, may be solved by designing a building structure and, in particular, its cladding and partitions to accommodate movements rather then to resist them. Greater emphasis should be placed by engineers on methods of avoiding interactive effects and associated damage.

4 Interactive effects on bridges are experienced mainly at bridge abutments that retain embankments and with long-span bridges. Engineers should have an appreciation of global behaviour rather than placing emphasis on detailed analysis of separate elements of the bridge foundations.

5 Proper recognition must be given to the effects of ground stiffness on the performance of offshore structures, and analyses must use realistic values of stiffness to model real behaviour.

6 Storage tanks present a particular challenge to designers because of the problems of shell distortion caused by the large settlements that are permitted in order to effect economies.

7 It is important to recognize the interdependence of retaining walls and ground in that the ground not only generates loading but also adjusts and distributes earth pressures to accommodate small movements. Clearly the initial *in situ* stresses are an important aspect controlling the behaviour of retaining walls.

8 The relief of stress in ground caused by construction is particularly relevant to the design of tunnels. The redistribution of earth pressure because of structural deformations must be recognized in the design of buried structures.

9 The science and practice of the behaviour of structures in contact with ground can be advanced only through field measurements. In this respect there is a need for simple, reliable, and economic instrumentation.

APPENDIX Interactive analysis of building structures

A.1 General

This Appendix reviews the currently available techniques for the analysis of the total soil–structure system. In order to assist the engineer the more readily available computer codes that utilize these techniques are listed in Table A1. Many of the available packages are capable of dealing with 3-dimensional modelling, can allow a variety of non-linear response actions to be modelled, and can deal with dynamic as well as static response. Interactive analysis combining all these aspects is available only in a few packages, but in general, realistic numerical modelling of interaction problems is readily available. Brebbia (1981) reviews the different analysis capabilities of the different packages. Contributions to advances in numerical modelling techniques are contained in Wood (1980), Desai *et al* (1982) and Desai & Sargan (1984).

Most of the techniques relate directly to movements induced by the structures themselves but may be employed in the analyses of structures affected by indirect causes. It is possible to identify and classify structure and foundation structure in a number of ways, but the following scheme has the advantages for the present purposes:

1 *Structure*

1.1 *Classification by overall geometry of structure*

- (*a*) low buildings (say, height less than minimum width, or not exceeding two storeys)
- (*b*) high buildings of fairly uniform height
- (*c*) buildings consisting of a high tower on a low podium.

1.2 *Classification by structural system*

- (*a*) loadbearing walls (masonry and *in situ* concrete)
- (*b*) precast (large-panel) concrete walls and floors
- (*c*) rigid frames – column and beam, or column and slab
- (*d*) frames stiffened by infill panels
- (*e*) external frames stiffened by stiff cores (core–column structures)
- (*f*) articulated frames

2 *Foundation*

2.1 *Foundation structure*

- (*a*) pad or strip footings
- (*b*) raft foundations
- (*c*) piled foundations.

Clearly there are many combinations of structure and foundation structure, and not all of them are amenable to a reasonably detailed analysis. This Appendix can deal with the analyses of only a few of these combinations. The objectives of such analyses are the evaluation of the settlement pattern of the structure (both in the short- and long-term scales) and the consequent redistribution of loads and stress resultants within the structure. The more developed methods may attempt to model the changing structural stiffness, its effects as building progresses, and during the lifetime of the structure.

All of these analytical approaches require the use of a computer unless applied only to the simplest structures and are beset by difficulties in regard to deciding what are to be the mathematical models for both the soil and the structure. This Appendix gives guidance in both of these areas of uncertainty, and uses examples to indicate the sort of information that can be provided by full interactive analyses.

Table A1 Some computer programs usable for interactive analysis*

name	author	availability	type
LAWRAFTS	L. A. Wood	Service in Information and Analysis (SIA) Ltd. London Department of Civil and Structural Engineering, South Bank Polytechnic	finite-element raft or rafts on layered anisotropic boundary element soil model. Linear and non-linear soil response
LAWPILE	L. A. Wood	Department of Civil and Structural Engineering, South Bank Polytechnic	finite-element piles and pile cap in layered boundary element, soil model
P GROUP P GROUP 3	HECB	Various bureaux	boundary-element pile-group analysis
PIGLET	M. F. Randolph	University of Cambridge	influence-value pile-group analysis
RAFTISM	John Laing	SIA	sub-grade model to be used with iterative finite-element grillage
CRANLAY	Harrison *et al*	CSIRO, Australia	integral transform solutions for circle and strip boards on layered anisotropic elastic media
FOCALS	L. J. Wardle *et al*	CSIRO, Australia	finite-element raft layered anisotropic elastic media using integral transforms
PROFILE	Sharrock	Scott Wilson Kirkpatrick & Partners, Basingstoke	integral transforms influence value for vertical displacement of arbitrary loaded areas on layered anisotropic elastic media
SETT 2	J. A. Hooper	Ove Arup Partnership, London	finite-element raft on layered anisotropic elastic media

*The list of programs in Table A1 is not necessarily comprehensive. The inclusion of a program in the Table does not imply any guarantee or underwriting.

A.2 The structural model

A.2.1 Introduction

This subsection is concerned solely with the analytical model for the structure, although it should be recognized that the appropriateness of a particular model for a given structure will be influenced by the treatment adopted for the foundation and the soil. In general, it is the stiffness of the structure when spanning as a beam or slab across the foundation structure that the model is required to represent with tolerable accuracy rather than, for instance, its stiffness as a vertical cantilever, as is more commonly required in structural design. Clearly, this former stiffness will be closely related to the stiffnesses, in the same sense, of the floor systems, but the manner in which the stiffness of the individual floors should be aggregated will be highly dependent on the nature of their vertical interconnections and on the height of the building. In addition, the vertical interconnections will make a more direct contribution to the total stiffness when they take the form of walls or framing systems that are very stiff in their own planes. The typical structure is an extremely complex assembly of elements whose stiffness varies with time, especially during the construction process. In the present state of the art, considerable approximation and compromise may therefore be necessary in its modelling. Although comparison has been made between field measurement and analysis that clearly indicates the influence of superstructure stiffness (Wood & Perrin, 1984 & 1986), the very nature of design and construction suggests that in most cases a good deal of experience and judgment over what account is to be taken will be required.

The main areas of approximation and compromise are as follows:

- *Linear or non-linear analysis*
In general, the problem is non-linear and time-dependent. In order to make the complete interaction problem manageable it will often be necessary to adopt approximate linear-elastic material properties for the structural members. Well established analytical techniques are then available, although the analysis of all but the simplest structures will still present a problem of considerable magnitude but not beyond the capabilities of powerful minicomputers such as the Prime 2250 and Vax 11/730, which are to be found in many firms.
In the present state of the art it is feasible to include non-linear and time-dependent behaviour provided that the appropriate constitutive relations can be defined. The degree of refinement that may be considered desirable or necessary in a given interactive analysis is then difficult to define. It may be severely limited by the complexity of the structure, but as a general rule the degree of sophistication adopted for the superstructure should be related to that adopted for the foundations and soil mass.

- *Plane or 3-dimensional analysis*
From the computational point of view, it is usually a considerable simplification to consider a structure as a series of independent plane frames or walls. This simplification may be admissible where the foundation is planned as a series of isolated or strip footings. But even here it must lead to some error if only because it ignores the stiffness in the transverse direction contributed by the floor systems. In general, both the structure and the foundation must be considered 3-dimensionally.

- *Analysis of tall buildings*
It is generally true that successive storeys above ground will make progressively smaller contributions to the total effective stiffness of the structure. The reduction in stiffness of vertical elements is only one of the reasons for this. A second is the tendency for inequalities in the stressing of walls and columns arising from foundation movements to disappear with height as a result of differential vertical strains. A third is simply the earlier construction of the lower storeys. It follows that it should, in principle, be possible to obtain a correct estimate of overall stiffness for any building by analysing fully a certain number of storeys at the foot and ignoring completely the stiffness of the storeys, provided that there are no large changes in planform or construction.
The classification given above in terms of overall geometry of structure is relevant to the choice of structural model chiefly in this connection. For low buildings and for the low podium in buildings of type 1.1(*c*) (see subsection A.1), it will usually be necessary to analyse completely the full height. For tall buildings a choice must be made between similarly analysing the full height and recognizing in the analysis the factors leading to reduced contributions by the upper storeys to the total effective stiffness and, on the other hand, analysing fully only the lower storeys and ignoring completely the stiffness of those above. Until more analyses have been made on this basis and compared with field observations, there is little guidance that can be given on the numbers of storeys to be considered in particular cases.

- *Allowance to be made for walls, partitions and floors in framed buildings*
In the design of framed buildings for vertical and side loads, the contributions of walls, partitions and floor slabs to overall stiffness are often recognized only tacitly in, for instance, assuming that there will be no sidesway when designing the columns. For analyses of soil–structure interaction, a more explicit recognition of these contributions is required. The contributions of the floor slabs should always be added to that of the beams. In addition, a realistic allowance should be made for the in-plane stiffnesses of walls and partitions, particularly where these are built as infill panels to the frame or as continuous shear walls.

In this connection it should also be remembered that it is a 'most probable' estimate of stiffness that is required and not, as in some other aspects of design, a minimum likely one. In practice it may sometimes be difficult to estimate this directly. Upper and lower bounds should then be estimated, and both should be considered in the interaction analysis. A lower bound to the overall stiffness of the structure will, for instance, give safe design loads for interior columns, while an upper bound will give safe design loads for exterior columns.

A.2.2 Framed structures

This clause deals chiefly with structural systems classified as type 1.2(*c*) in subsection A.1, but should read in conjunction with clause A.2.3 for infilled frames, type 1.2(*d*).

The behaviour of plane frames has been discussed by Grasshof (1957), Sommer (1965), Heil (1969) and Lee & Brown (1972). Meyerhof (1953) has, in addition, analysed an 'equivalent structure' that included the effects of the upper storeys, infill panels and external cladding, while Litton & Buston (1968) have provided some guidance on the way in which interactive effects diminish with height in a plane bare frame. At present though, it seems desirable to model fully the whole height of a structure where possible.

Reinforced-concrete frames present problems in the determination of section properties that do not arise with steel frames. They arise because of variations in concrete behaviour with age and loading history and because full details of reinforcement, for instance, may be unknown at the time of making the analysis. The effects of creep can be allowed for by the use of a low effective modulus of elasticity (1970). Uncertainty about reinforcement in beams and slabs may be circumvented by calculating on the basis of the gross (uncracked) concrete section ignoring all reinforcement. The dispersion of actual final stiffnesses about the values so calculated will probably be covered by assuming a coefficient of variation of 20%. Columns in multi-storey structures are unlikely to crack significantly because axial stresses tend to be large in comparison with bending stresses. Calculation on the basis of the gross concrete section alone would therefore almost invariably underestimate stiffnesses. It is suggested that reasonable estimates of section properties may be obtained here by assuming 2½% steel in the absence of better data. This reasoning does not apply, however, to columns in single-storey buildings. They should be treated in the same way as beams.

In reinforced-concrete framed structures that are cast *in situ*, beams are usually integral with floor slabs, so that the latter act, in part, as the flanges of wide T- or L-beams. In accordance with BS 8110 (1985) it is suggested that, in calculating beam stiffnesses, the following widths of slab should be considered to act as integral flanges:

T-beam: $2 \times 0.07 \times$ span
L-beam: $0.07 \times$ span

The resulting stiffnesses should be regarded as lower bounds. Upper bounds may be calculated by including also the stiffness of the remaining widths of slab to their centre-lines when bending about their own neutral axes.

Similar calculations should be made where reinforced-concrete slabs span between steel beams and are connected to these beams by shear connectors. Where there are no shear connectors, the effective stiffnesses will probably be bracketed by considering as one bound the lower bound for the composite system and as the other bound the simple sum of the beam stiffness and the stiffness of the half-widths of the adjacent slabs bending about their own neutral axes.

A.2.3 Infill panels in framed structures

General

There are many experimental data on the behaviour of brickwork (Polyakow, 1960; Sachanski, 1960; Mainstone & Weeks, 1970; Fiorato *et al*, 1970; and Mainstone, 1971), concrete, (Mainstone, 1971; Benjamin & Williams, 1960; Smith, 1966; Smith & Carter, 1969; and Simms, 1967), and steel (El-Dakhakhni & Daniels, 1973; and Nyberg, 1976). In addition, steel infill panels are amendable to analysis (Bryan, 1973; Miller, 1972; Oppenheim, 1973; Davies, 1976; and Constrado, 1977). A few tests have also been made on infill panels of concrete blockwork (notably by Simms, 1967). Virtually nothing has been done on infill panels of other materials, but it is not thought that any of these is likely to have both strength and stiffness sufficient to justify their consideration in an interaction analysis. In practice the designer will usually have to consider only infill panels of brickwork, blockwork, or concrete, and should bear in mind that the contributions these can make to the overall stiffness of the above-ground structure is dependent on the tightness of fit in the frame. In particular, where horizontal movement joints are provided at the top of infill panels (as is necessary with clay brickwork in reinforced concrete frames), the stiffening effects of such infill panels will be greatly reduced and should probably be ignored.

Although a horizontal in-plane racking load was applied to the specimens in nearly all the tests referred to, the data are equally applicable to the situation in which racking is caused by differential vertical movements of columns. With rare exceptions, early separations of the unloaded corners of the infill panels from the frames, usually coupled at slightly higher deflections with some internal cracking, led to the infill panels behaving essentially as diagonal struts. For design purposes it therefore seems preferably so to regard them, rather than as true shear panels with nominal shear moduli to compensate for the highly non-uniform and predominantly compressive internal stress patterns.

Where there are unreinforced openings in the infill panels, stiffnesses will be reduced. In the absence of adequate experimental data, it is suggested that where openings cut the diagonal that forms the axis of the effective strut and occupy more than one-sixth of the frame opening, the infill panels in question should be ignored. Where the openings are of lesser size or do not cut the diagonal, the infill panels should be rated at half the stiffness of similar full infill panels, although small openings in corners that the interaction analysis shows to be unloaded may be ignored completely.

For external cavity walls where only the internal leaf is an infill to the frame and the external leaf is continuous beyond it, the effective thickness should be taken as that of the internal leaf only.

Brickwork infill panels
Brickwork infill panels are highly non-homogeneous. Nominally identical infill panels will differ markedly from one another in the precise distribution of this non-homogeneity and may, in consequence, differ considerably in behaviour. This difference may be further aggravated by differences in the degree of fit between the infill panels and the bounding frame. In all cases, moreover, cracking and partial separations between the infill panel and the frame will soon develop as the latter distorts. Ultimate loads of the combined frame–infill system will be reached only after extensive cracking and at racking distortions well in excess of those likely to be relevant in soil–structure interaction analyses.

A typical load/deflection response is illustrated diagrammatically in Fig. A1. It is not only significantly non-linear, but also time-dependent and influenced initially by the construction sequence. These characteristics preclude accurate modelling for normal design purposes. Because an infilled frame may, over the relevant range of distortions, be an order of magnitude stiffer than the bare frame, a realistic (although approximate) treatment of the infill panels must nevertheless be regarded as essential to a meaningful interaction analysis.

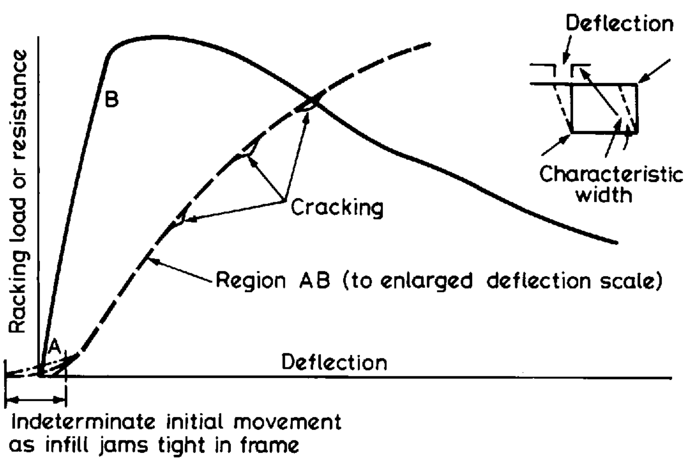


Fig. A1 Typical load/deflection curves

The equivalent diagonal strut, already referred to above, is suggested as the appropriate model (Mainstone, 1971; Smith & Carter, 1969; and Mainstone, 1974). The stiffness over the range AB in Fig. A1 may be estimated by replacing the brickwork with an equivalent diagonal strut of width equal to one-tenth of the characteristic width indicated on the Figure. In the absence of better data, the modulus of elasticity may be taken as $7kN/mm^2$ for high-strength brickwork, but should probably be reduced to, say, $5kN/mm^2$ for lower-strength brickwork. Any marked initial lack of fit will reduce the initial effective stiffness, and some recognition should be given to this if poor fits are expected. Irrespective of lack of fit, a coefficient of variation of at least 40% should be allowed for.

Blockwork and plain-concrete infill panels
Infill panels of plain concrete are more nearly homogeneous, and the only tests that have permitted a direct comparison with brickwork infill panels (Mainstone, 1971 & 1974) show less variability in the behaviour of the concrete panels. The general pattern of behaviour is, however, similar, and the equivalent diagonal strut is again the appropriate model. The infill panel should again be replaced by such a strut, but with a slightly reduced width of one-fifteenth of the characteristic width indicated in Fig. A1 and the normal modulus of elasticity appropriate to the concrete used. It is suggested that infill panels of concrete blockwork should be treated similarly in the absence of adequate experimental data.

Reinforced-concrete infill panels
Only in infill panels of reinforced concrete with adequate reinforcement tied into the bounding frame or continuous with the reinforcement of adjacent infill panels will a state approximating to uniform shear arise as a result of a racking deformation. In such cases, the shear stiffness of the infill panels may be calculated simply on the basis of the vertical cross-sectional area of the infill panel and a shear modulus equal to 40% of the normal elastic modulus.

Steel infill panels
Steel infill panels are not used at present, but their use as partitions to limit sidesway in multi-storey steel-framed buildings has been advocated. If they are used, guidance on their treatment can be found in Bryan & Davies (1976).

A.2.4 Loadbearing-wall structures (masonry and *in situ* concrete)
Loadbearing-wall structures tend to be stiffer than framed structures on account of the high in-plane stiffness of the walls and the need for a greater provision of walls of a given strength than in an infilled frame. Differential settlements thus tend to be small, although they may still lead to unwelcome cracking.

Walls may be of brickwork, blockwork or *in situ* concrete, and it is assumed that in buildings of more than, say, three storeys the floors will be continuous *in situ* reinforced-concrete slabs. Gravity loads will lead to predominantly compressive stresses in the walls, at least in the more important lower storeys. All three materials may thus, for this purpose, be treated similarly except that only concrete walls should be assumed to possess any stiffness in bending out of their own planes. This stiffness may be taken as that of the gross unreinforced-concrete section.

There are two main types of structure to be considered. In the crosswall structure (see Fig. A2a), all major walls span parallel to one another and to the longer axis of the building. In the other type (see Fig. A2b), walls span in at least two directions.

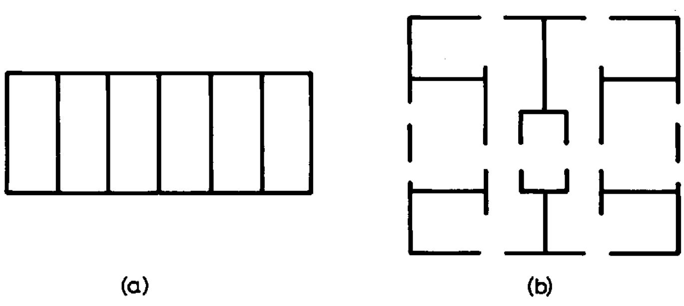


Fig A2 Loadbearing – wall structures
a crosswall construction
b walls spanning in two directions

Crosswall structures are mostly used up to only relatively low heights, so it will usually be essential to analyse the full height. On the other hand, it is probably permissible to analyse independently the overall stiffness across the building and along its length. For masonry walls, the latter may be taken as simply the aggregate stiffness of all the individual floor slabs each bending about its own neutral axis and for concrete walls that of a longitudinal frame in which the walls are considered as up-ended slabs with stiffnesses as suggested above. The stiffnesses across the building on the planes of the crosswalls should be calculated as described below. For the other type, a 3-dimensional analysis is essential, although it may be possible to limit this to the lower storeys of a tall building. In one retrospective analysis of such a building (Hooper, 1976), good agreement between calculated and observed differential settlements was obtained by making a rather arbitary analysis of only the first two storeys of a 22-storey building. Further guidance, similar to that given by Litton & Buston (1968) for bare frames, may be obtained from MacLeod & Hosny (1976).

For the analysis of the in-plane stiffness of the walls and, in particular, of walls connected by floor slabs across opening, a frame idealization is recommended. In this, the walls are treated as columns bending in their own planes and connected to the slabs at nodes situated at their edges. The finite in-plane widths of the walls are thus directly recognized (MacLeod, 1973, Fig. 1). By thus considering all walls and the interconnecting slabs, a 3-dimensional equivalent frame is obtained for structures of type shown in Fig. A2b. Out-of-plane bending of the walls is here neglected whatever the material since it is insignificant in contributing to overall stiffness as compared with in-plane bending. Alternatively, the so-called continuous-connection method may be used (Rosman, 1970; and Petersson, 1974), but the use of finite elements is not recommended because a large number of elements is required to achieve accuracy equal to that obtained with the frame method.

Non-linear analysis for such structures has not yet been widely developed. Some work on post-yield behaviour has been reported (Paulay, 1970 & 1971; and Nayar & Coull, 1976), but realistic non-linear interaction analysis will also have to include long-term creep effects. A way of allowing for creep in concrete has already been given in clause A.2.2. For creep in brickwork, guidance is given by Lenczner & Salahuddin (1976).

A.2.5 Large-panel structures

A large-panel structure behaves as a loadbearing-wall structure (see clause A.2.4) with the added difficulty of assessing the effect of the connection. No clear guidance as to when the effect of the connections should be included in an analytical model is available. For lateral load analysis of tall large-panel walls, the flexibility of the vertical connections does not have a significant effect (Bhatt, 1973; and MacLeod & Green, 1975). It is relatively easy to include the effect of the connections (MacLeod & Green, 1975; Pollner *et al*, 1975; and Burnett & Rejendra, 1972), but the relevant shear-slip relationships cannot yet be confidently predicted.

A.2.6 External frames stiffened by stiff cores (core–column structures)

Where tall buildings derive their lateral stiffness largely from a stiff reinforced-concrete core, it should be sufficient to assume that this core is completely rigid and to model the surrounding structure as a single slab whose stiffness is the aggregate stiffness of all the individual floor slabs, with some allowance for the reduced contributions of slabs in the upper storeys of tall buildings.

This simple model will be seriously defective only in the case of very tall buildings where significant further lateral stiffness is provided by the peripheral columns and beams acting as an outer stiff tube. In these cases it will be necessary to model also these outer columns and beams by an equivalent peripheral beam. Similar modelling may also be desirable in the case of lower buildings in which the external frame is stiffened considerably by infill panels and cladding. To decide whether it is desirable, the bending stiffness of a typical storey considered as a horizontal beam should be estimated in accordance with the recommendations of clause A.2.3 and compared with the typical slab stiffness.

A.3 Soil model

The three principal ways of modelling the soil are to assume that it can be treated as:

(*a*) a set of linear unconnected springs
(*b*) a half-space continuum
 (i) elastic theory used for both stresses and strains
 (ii) elastic theory used for stresses only
(*c*) a layered continuum

It must be stated that model (*a*) (commonly known as the Winkler approach) cannot be recommended for the analysis of rafts and continuous footings. Although having the apparent advantage of being easily included in standard computer programs for structural analysis, it is a poor physical model. In particular, the results of an analysis based on the use of this model may be excessively sensitive to the pattern of applied load as illustrated by Wood *et al* (1980) in Fig. A3 and, in addition, is incapable of taking into account interactions between non-structurally connected areas.

In model (*b*)(i) the stress and strain distributions in the soil are assumed to be those corresponding to a half-space (Cheung & Zienkiewicz, 1965). In practice, this often has severe limitations because it does not take account of soil layering or the variation of modulus with depth within a given layer. However, a useful extension of the model is to assume a half-space stress distribution and then calculate the strains, and hence the settlements, using the various deformation moduli of the soil as a model (*b*)(ii) (Wood, 1978).

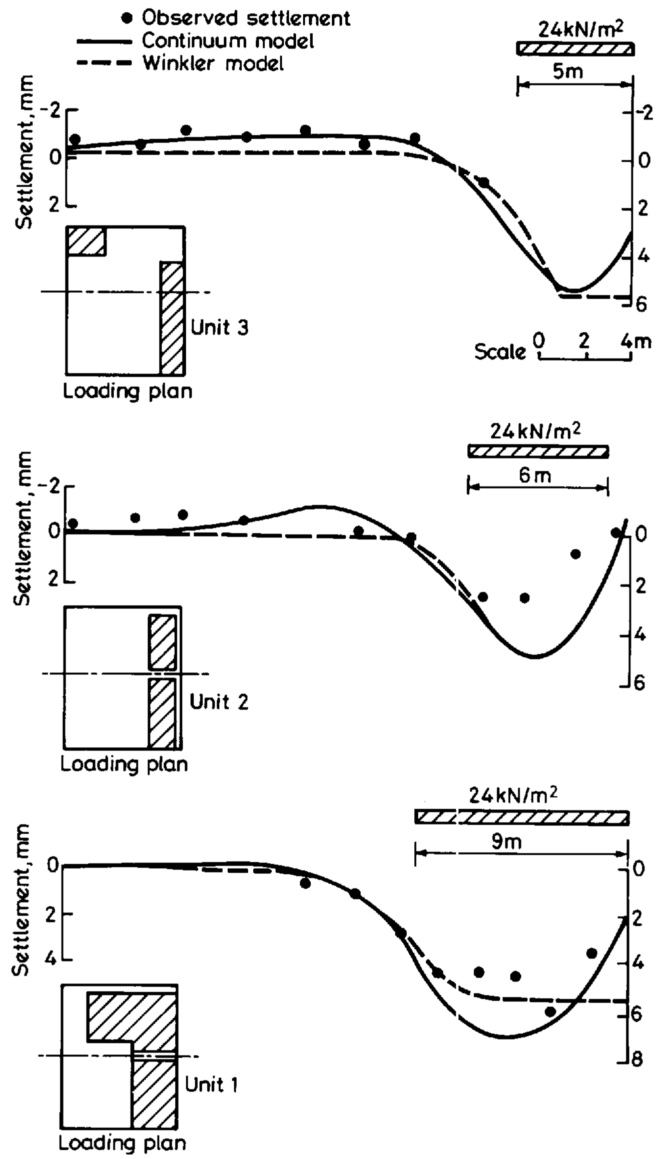


Fig. A3 Computer and observed deflections of a ground bearing warehouse slab
Based on Wood *et al.* (1980)

In model (c) the 'exact' stresses and strains in a layered soil mass are calculated. Even for the linear elastic case there are few analytical solutions available. However, solutions to a wide range of problems can be obtained using numerical methods (Fraser & Wardle, 1979).

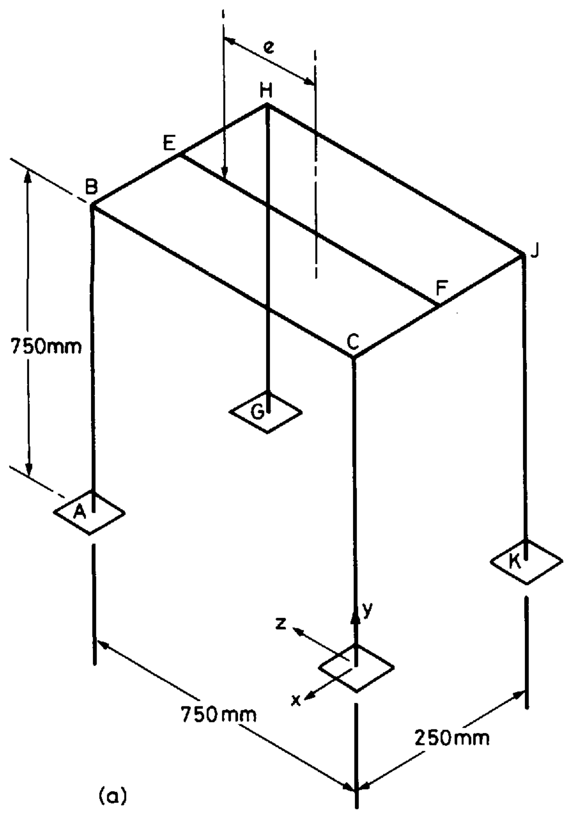
It may be noted that, using model (b) or (c), surface settlements and horizontal displacements outside the foundation plan area can be obtained readily. These data are useful in assessing the effect of construction ground movements on existing buildings.

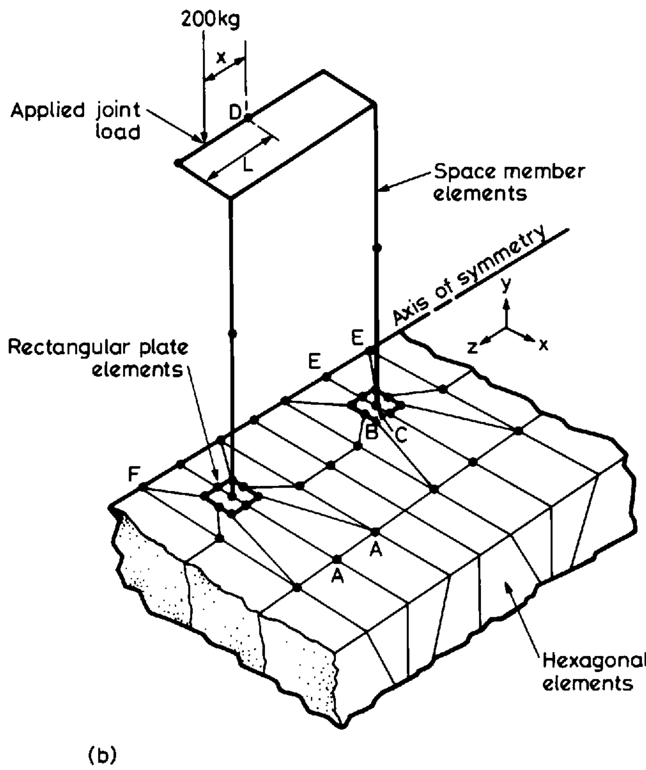
The principal stress/strain relationships that can be used for the soil are:

(i) linear elastic
(ii) non-linear elastic
(iii) elastoplastic
(iv) viscoelastic
(v) consolidating
(vi) critical state.

In most practical cases, the magnitude of applied loading is relatively low, and only models (i) (ii) and (iii) need be considered for the soil. Even then, the analysis of all but the simplest soil–structure interaction problems requires the use of a substantial computational effort.

A summary of the principal analytical methods is given in subsection A.4.





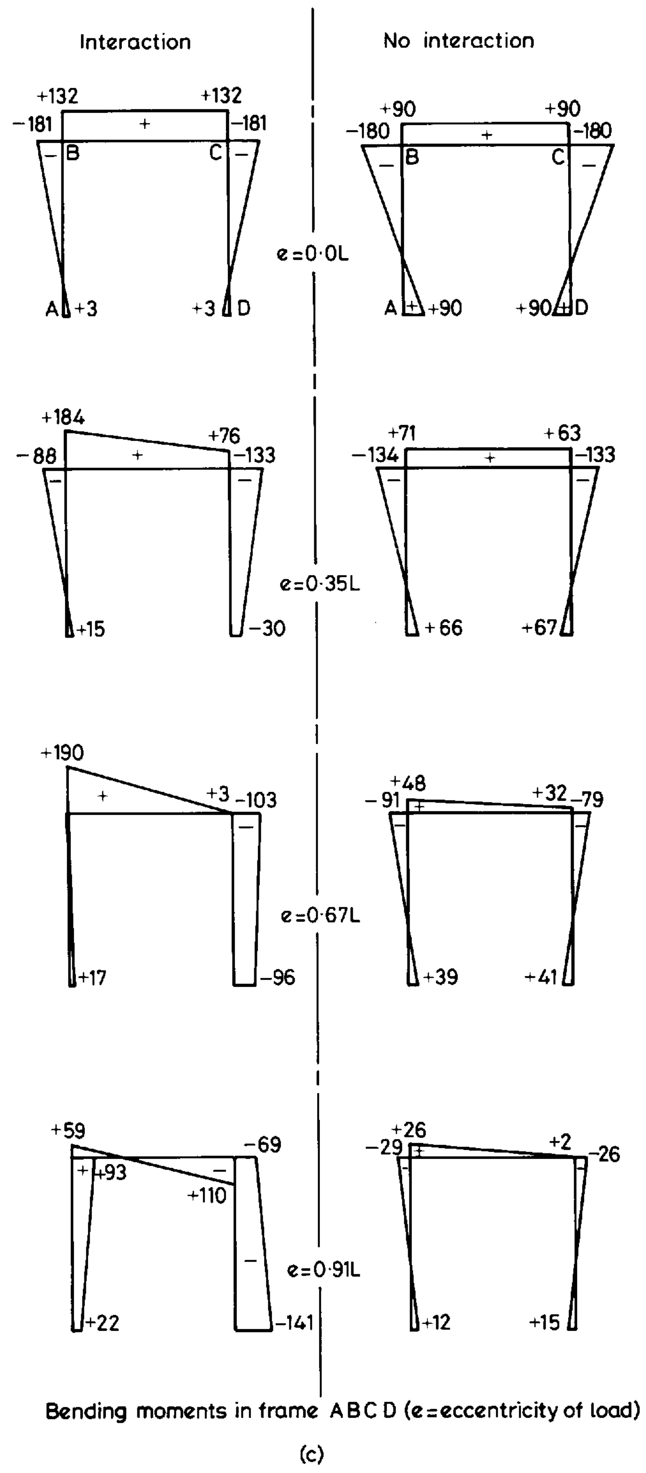


Fig. A4 Results for a simple symmetrical 3-d frame illustrating the change in bending moment when interaction is taken into account

Based on Majid & Cunnell (1976)

A.4 Analysis

A.4.1 Introduction

General

For all but the simplest of structures, it is necessary to form interdependent but separate mathematical models of the major components. It is only in this manner that the size of the problem may be reduced to manageable proportions. The three basic components may be regarded as:

- the structure
- the foundation structure
- the soil

although further subdivision of these may be necessary for particular situations.

Mathematical models of these three components may then be formulated relating the forces and moments acting at their common interface with the corresponding displacements and rotations. These unknowns having been evaluated, the stress resultants arising from the combined action may in turn be determined from analysis of the basic components separately.

In order to solve the equations governing the behaviour at the interface two basic techniques – iterative and non-iterative – have been employed.

Iterative methods

For the structure it is possible to obtain a relationship between the net applied forces and displacements at the interface between soil and structure, employing standard methods of analysis. Similarly, standard soil-mechanics procedures may be used to relate the surface settlements to the as yet unknown ground reactions.

These two sets of relations are evaluated as influence coefficients, and iterative methods are used to establish common displacement profiles and ground reactions at the interface. Techniques of this type have been used by Chamecki (1956), Zbirohowski-Koscia & Gunasekera (1970) and Larnach (1970). This approach has the advantage that it is readily understandable and, in the case of simple structures, may be undertaken as a hand calculation. However, it is relatively inefficient in terms of computational effort and has to a large extent been superseded by non-iterative methods.

Non-iterative methods

Within the development in recent years of structural-analysis techniques using finite elements and the corresponding increase in the power and availability of computers, more direct methods of solution have been developed in which the common displacement profile and ground reactions at the common interface are determined in a single discrete step. These may be divided into those employing normal finite-element models for both structure and soil, and those utilizing some form of boundary element (or boundary integral technique) in order to model the soil coupled to a finite-element model of the structure. The former is perhaps best suited to 2-dimensional plane strain or axisymmetric representation, whereas the latter lends itself to 3-dimensional problems. Both techniques may take into account loss of contact between soil and foundation and incorporate the effects of non-linear soil behaviour.

A.4.2 Pad and strip footings

King & Chandrasekaran (1975) illustrated the importance of giving consideration to the interaction between the superstructure, foundation and soil with reference to a 2-dimensional frame supported on strip footings. 3-dimensional analyses can also be carried out (Majid & Cunnell, 1976), and although computing costs are relatively high, these analyses are useful for setting limits on the validity of results obtained from more approximate methods.

The results of such analysis illustrating the effect of settlement on the bending moments in a simple symmetrical space frame subjected to eccentric loading are shown in Fig. A4. The compressible foundation refers to a 900mm thick bed of sand. A hyperbolic stress/strain relationship was used to represent the sand, and the incremental method was used for the non-linear analysis. Measured displacements of the model space frame agreed well with computer values.

The preceding examples utilized finite-element models for both the structure and the soil. Wood & Larnach (1972 & 1975) and Fraser & Wardle (1976), while retaining a finite-element model for the structure, extended the simple half-space boundary soil model proposed by Cheung & Zienkiewicz (1965) to include heterogeneous elastic continua of finite depth.

The application of this method to the prediction of settlements and bending moments has been undertaken by Wood *et al* (1977) for a multibay two-storey reinforced-concrete framed office block originally described by Webb (1975). The building has been the subject of a settlement survey, and a comparison between computed and measured settlements, together with the general arrangement of two bays of the structure, are shown in Fig. A5. Complete estimates were made of the secondary bending moments incurred because of the settlement problem. Considerable judgment had to be exercised in order to establish suitable structural and soil models because, as in most case histories, all relevant data were not available.

A.4.3 Raft foundations

Whereas in the case of pad and strip footings the major interactive effects manifest themselves in the stress resultants induced within the superstructure frame, the primary consideration for a raft foundation is the effect on the raft itself. Again, superstructure stiffness will have a marked influence on the behaviour of the raft and should not be ignored, although quantitative assessment of all but the simplest of wall systems connected to the raft may prove difficult. However, often the raft is itself a major contributor to the overall stiffness of the building. Since the raft is in intimate contact with the supporting soil, the interactive effects are, perhaps, most marked in consideration of its own behaviour. Indeed, in the design of a raft foundation, it is totally unrealistic to ignore deformations and rely on moments and shears obtained from analysis of the conventional flat-slab model. Conversely, it is equally unrealistic to compute deformations without consideration of structural stiffness and then to design on the basis of the corresponding stress resultants. A rational design approach must be based on the results of an interactive analysis. Although numerical methods are in existence that facilitate the analysis of rafts of arbitrary shape, much of the earlier analytical work relates to the single case of a circular raft.

Circular rafts

The relatively simple case of a uniformly loaded circular raft of constant thickness founded on an elastic continuum is particularly useful in making preliminary assessments of possible interactive effects.

The major governing parameter is K, the relative stiffness of the structure and soil. The importance of this concept of relative stiffness must not be understated. For example, two identical structures one supported on rock and the other on an alluvial mud will behave in totally different ways categorized by their different relative stiffness: the former being a flexible response and the latter a rigid one. For a plane circular raft resting on a thick homogeneous elastic layer the definition of K is reasonably straight forward and may be expressed as:

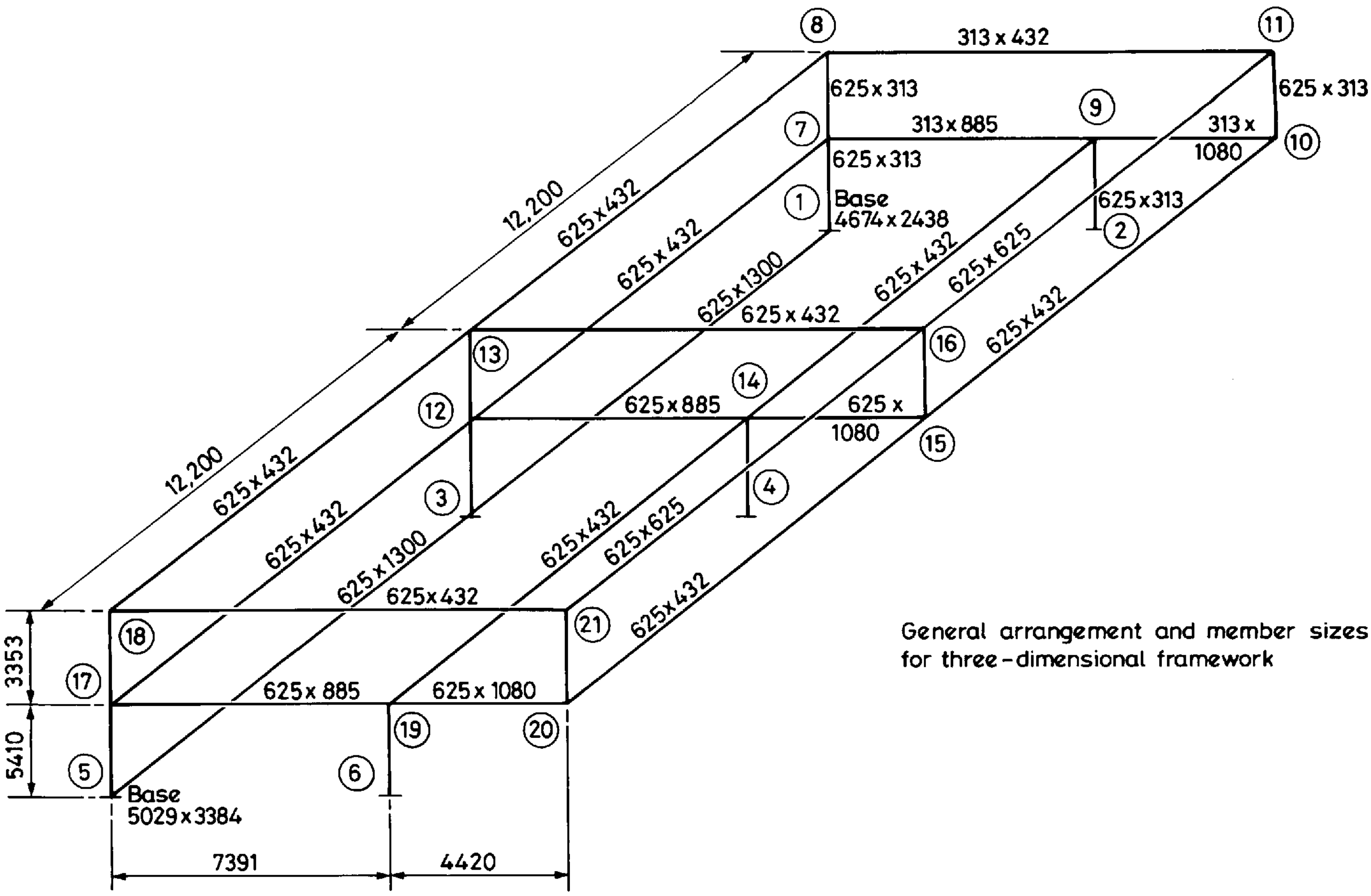
$$K = \frac{E_c\,(1 - s^2)}{E_s\,(1 - c^2)} \cdot \frac{t^3}{R^3}$$

where E_c and E_s are the elastic moduli of the raft and soil material, respectively, c and s are the corresponding Poisson's ratios. The raft is of constant thickness, t, and radius, R.

Examples of the moment/curvature relations for a uniformly loaded circular raft resting on the surface of a deep homogeneous isotropic stratum are shown in Fig. A6, where q denotes the applied load intensity. Thus uniformly loaded rafts with $K<0.1$ may be termed flexible and those with $K>10$ considered as being stiff. However, in the zone $0.1<K<10$ the moment/curvature relations change quite rapidly.

The curves shown in Fig. A6 relate to both frictionless (Brown, 1969) and fully adhesive contact (Hooper, 1974) between the raft and soil. It should be noted that for ν_s = ½, the adhesive behaviour coincides with that for frictionless contact. Furthermore, in practice is it likely that slippage will occur if the surface tractions cannot be sustained by the soil. Therefore in the majority of cases, the frictionless contact model is likely to prove behaviourally more correct.

The highest moments are generally given by the case of



Column base	Column base foods			Computed settlements			Measured settlements (June 1973)	Settlement ratios	
	NI* kN	WI+ kN	Change %	NI* mm	WI+ mm	Change %	mm	Computed WI* / measured	Computed WI+ / measured
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(j)	(k)
1	1156	1232	6.6	209	214	2.6	160	1.31	1.34
2	1235	1248	1.1	248	248	0	190	1.31	1.31
3	1248	913	−26.8	267	218	−18.4	168	1.59	1.30
4	1343	1185	−11.8	279	251	−10.0	210	1.33	1.20
5	838	1176	40.3	153	188	22.8	165	0.93	1.14
6	893	959	7.4	190	204	7.4	160	1.19	1.28
							Mean	1.28	1.26
							σ	0.196	0.069
							variance	0.153	0.055

NI* = no interaction
WI+ = with interaction

A/30

Fig. A5 3-dimensional frame analysis
Based on Wood *et al* (1977)

frictionless contact, and these should be used for design purposes. The case of completely adhesive contact usually represents a lower bound to the problem. In practice, interfacial slip may occur (Hooper, 1976) in which case the computed results will be approximately midway between those given by the two bounding solutions. It may be assumed that these conclusions also apply to non-circular rafts.

Most soils exhibit some degree of heterogeneity and anisotropy, and in many situations the compressible soil layer is of finite thickness. The effect of linear heterogeneity on the value of maximum bending moment and the settlement of a rigid raft has been considered by Brown (1974) and subsequently by Wood (1977). The results are shown in Fig. A7, where K, the relative stiffness, is defined in terms of E_s (0), the elastic modulus at the soil surface, and the degree of linear heterogeneity is quantified by the ratio of Young's moduli at the surface, E_s (0) and at a depth equal to the radius, E_s (R). Settlement results produced by Carrier & Christian (1973) and Boswell & Scott (1975) are also shown for comparison.

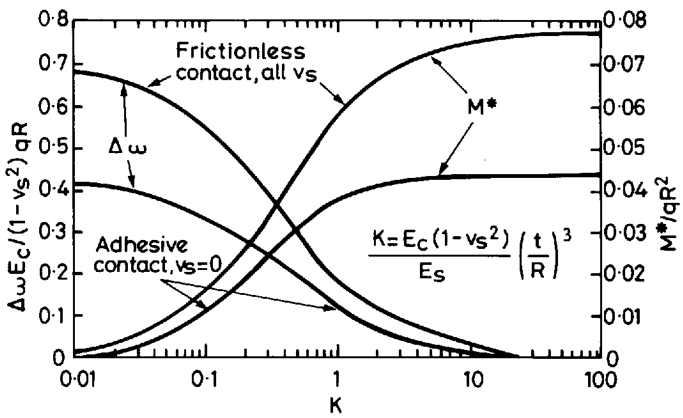


Fig. A6 Maximum computed values of differential settlement w and bending moment M for uniformily loaded circular raft on deep elastic layer*

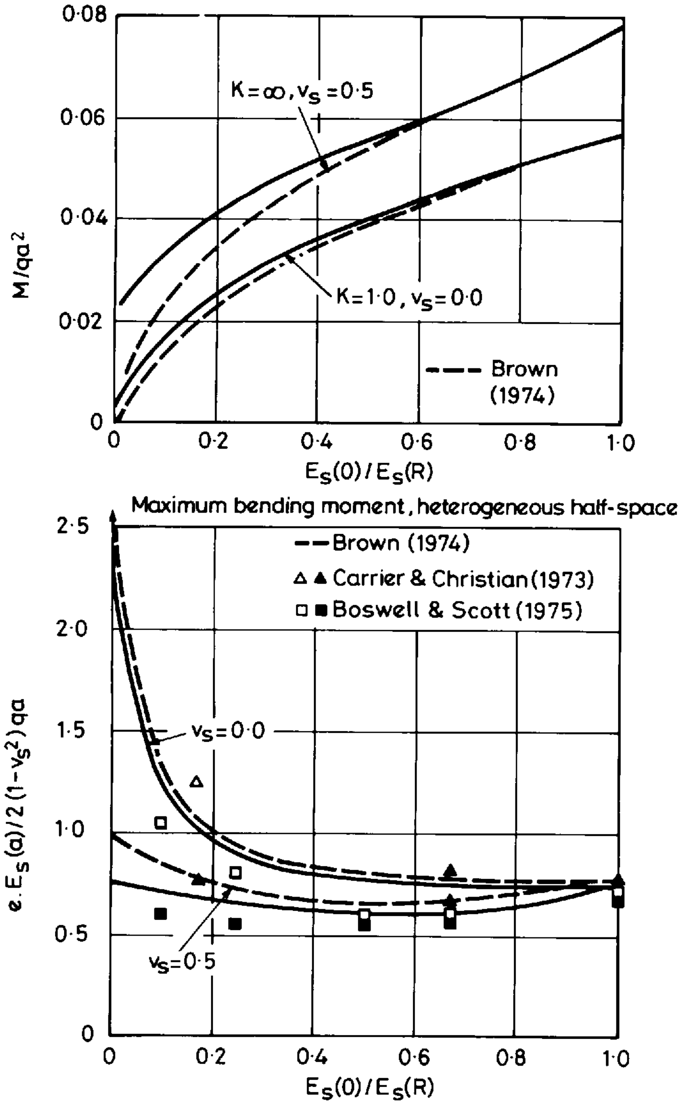


Fig. A7 Rigid raft settlement, heterogeneous half-space

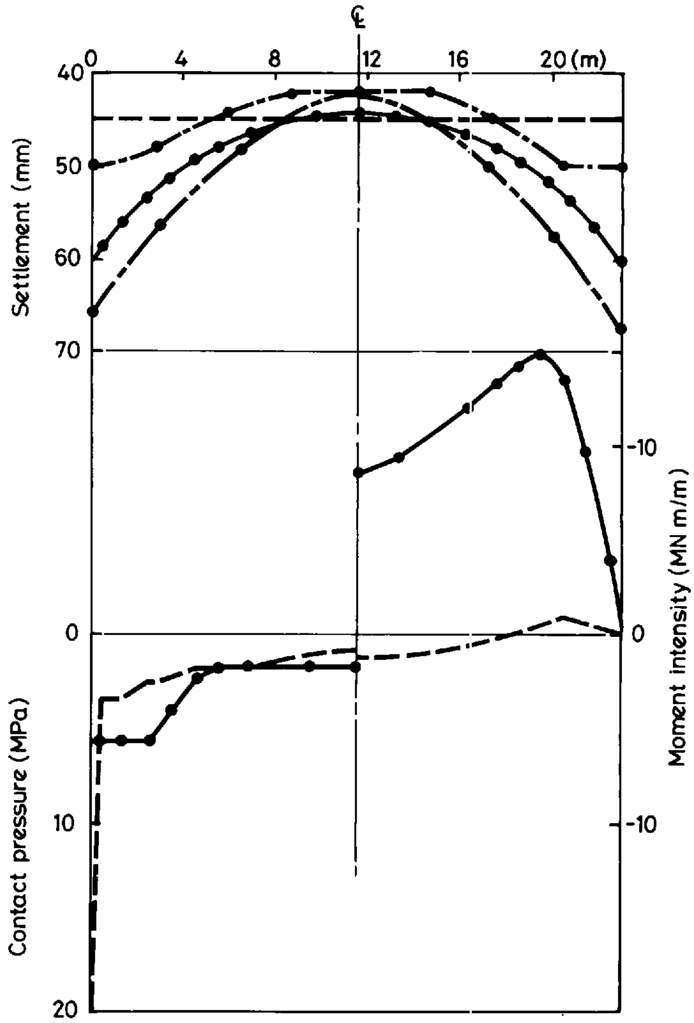


Fig A8 Settlement, moment and contact pressure profiles along diameter of silo

● measured along; two diameters
– – – elastic —— non linear
Based on

Borowicka (1936), Smith (1970) and Zbirohowski-Koscia & Gunasekera (1970) have also considered the behaviour of circular rafts. A feature of the performance of the raft is the pronounced saddle-shaped interfacial contact pressure distribution predicted by an elastic analysis, giving rise to an infinite pressure under the edge of a uniformly loaded raft. Reference to the shear-strength characteristics of the soil would usually suggest that the edge pressure is not sustainable and local yield of the soil would occur leading to a more uniform contact pressure distribution and a corresponding reduction in maximum bending moment. However, Wood (1978) has considered such behaviour in relation to a circular silo founded on chalk. The silo is carrying a uniformly distributed and a concentrated edge load. Some of the results are shown in Fig. A8, where it is noticeable that the elastic analysis predicted almost uniform settlement, with correspondingly low induced movements, whereas when local yield of the chalk was taken into account, the silo developed a hogging deformation with a corresponding increase in maximum moment.

Hooper & West (1983) have also given detailed consideration to the behaviour of circular rafts on yielding soil and have proposed the use of a redistribution algorithm rather than the iterative procedure used by Cheung & Nag (1968).

Non-circular rafts

The unlimited variation in geometry and loading is such that few generally applicably theoretical studies have been made. Those made by Gorbunov-Possadov & Serebrjanyi (1961), Chan & Cheung (1974), and Fraser & Wardle (1976) relate to uniformly loaded square or rectangular rafts. These results, although of considerably academic interest, are of limited use in practice, where the usual requirements are for a method of analysis that can deal with rafts of any plan shape and flexibility subjected to any distribution of applied load.

The finite-element method first described by Cheung & Zienkiewicz (1965) is ideally suited for this purpose. The plan shape of the raft is subdivided into a number of rectangular finite elements, with vertical loads applied at the nodes. Some types of structure founded on the raft can also be incorporated into the analysis. In the case of a plain raft, the stiffness matrices of the raft and the soil continuum are added, and the results given in terms of settlement, contact pressure and bending moment.

The method has been extended by Wood & Larnach (1975 a & b) to include the effects of soil layering, based on the assumption that for a given surface load the stress distribution within the soil is the same as that in a homogeneous half-space. This approximation is analagous to the Steinbrenner (1934) model adopted in numerous settlement calculations and has been shown by Wood (1977 & 1978) to provide results comparable with those of more rigorous work referred to above and to be most satisfactory for design (Wood & Perrin, 1985; and Hooper, 1984). Hooper & Wood (1976) extended the soil model to include the special case of transverse isotropy.

A more precise extension of the same method to a transversely isotropic layered system has been described by Wardle & Fraser (1974) in which the analysis is based on the exact stress distribution within the layered anisotropic continuum. Subsequently Fraser & Wardle (1976) have produced a series of influence charts from which the performance of uniformly loaded rectangular rafts may be determined. All of these models have to be handled using numerical techniques. The inaccuracy brought about by the use of a numerical model is demonstrated in Fig. A9 for a square raft. The raft is subdivided into n square finite elements. As n is increased $1/\sqrt{n}$ tends to zero and the solution tends to the exact value as shown in Fig. A9. But note the steep slope of the two curves and the practical limit on the value of $1/\sqrt{n}$.

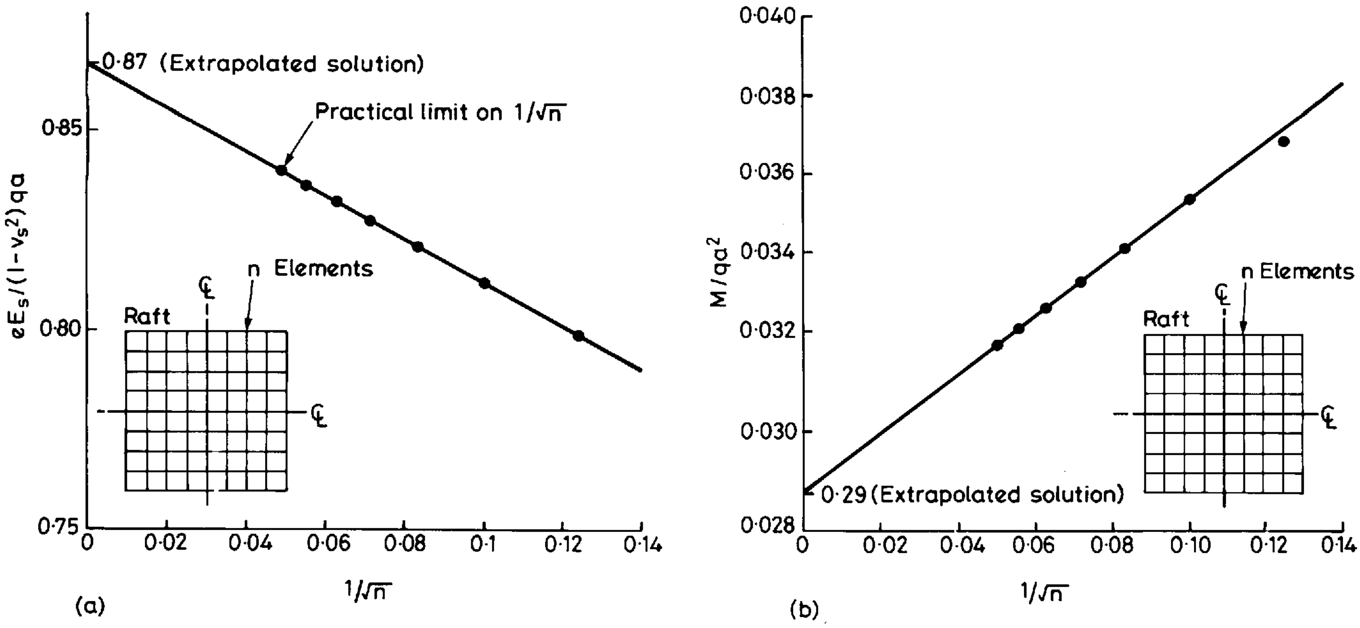


Fig. A9 Convergence of solutions for rigid raft
a settlement
b maximum bending moment
Based on Fraser & Wardle (1976)

Loss of contact and local yielding of the soil may be incorporated into all the foregoing numerical models. Results presented by Wood & Buttling (1984) for a complex of grain silos and associated buildings founded on soft clay illustrate the prediction power of these techniques in taking account of interaction between independent structures. In addition the use of a time-marching consolidation model (Wood, 1980; and Wood & Larnach, 1977) is also shown in Fig. A10d.

A further extension of the method for analysing multistorey multibay framed rafted structures on heterogeneous soil foundations has been described by King & Chandrasekaran (1974). They represented the soil by 3-dimensional finite elements and evaluated its response as a foundation-support stiffness matrix. Complete interactive behaviour was then evaluated using a foundation-structure approach where a preliminary analysis of the structure, under the action of the loads applied to it, yields the boundary stiffness matrix and 'fixed-end' load vector at the column–raft junctions. The raft is then analysed under the action of these fixed-end loads and any loads applied directly to it while subject to the external constraints provided by the boundary stiffness of the structure and the stiffness of the foundation support. The structure is then reanalysed under the action of the loads applied to it, and the boundary displacements are computed from the raft analysis.

Hain & Lee (1974) also advocate the foundation–structure approach to this type of problem. It provides scope for the refinement of the model used for the structure, e.g. to allow for the influence of cladding or for the simplification of the model used for the soil. The alternative use of the half-space stress/layered continuum strain-type of model for analysing a rafted frame has also been considered by King & Chandrasekaran (1977).

Hooper (1984) and Hooper & Philiastides (1986) present some interesting design case studies of raft foundations. Unfortunately, comparison between measured performance and computer prediction is not available in any of these studies. The need for this validation of the numerical procedures is now in general much overdue.

A.4.4 Piled foundations

The analysis of piled foundations presents a considerably more difficult problem than plain raft foundations, and there appears to be no 'exact' analytical method currently available. Interesting results for symmetrical pile groups embedded in an isotropic elastic half-space have been given by Poulos (1968), Butterfield & Banerjee (1971 a & b), Davis & Poulos (1972) and Hongladaromp *et al* (1973), and results for uniformly loaded piled strip footings have been given by Brown & Wiesner (1975). However, these results are of limited practical use as they do not include such basic parameters as soil layering and non-uniform applied loading. Banerjee & Davis (1978) have given consideration to the effect of soil heterogeneity on the performance of single piles. Randolph (1981 & 1983) has also developed approximate techniques for the assessment of the behaviour of long slender flexible piles embedded in a linearly heterogeneous soil deposit. Methods to take account of local yield of the soil around the pile have been suggested by Poulos (1975) and Wood (1979).

Where the piled foundation is approximately axisymmetric, the finite-element method of analysis may be used (e.g. Hooper, 1973; and Naylor & Hooper, 1975). However, in this model the piles are represented as concrete annuli; such an approximation may become less appropriate as the value of Poisson's ratio for the soil increases. Ottaviani (1975) has used 3-dimensional finite elements to study the interaction of a pile group, but the economics of such a model prevent it from becoming a valid design tool.

For non-circular foundations, it may be necessary to carry out some form of approximate analysis, such as that described by Hooper & Wood (1977) for an asymmetric piled foundation. In this approach the familiar notion of simplifying the raft/pile system to an 'equivalent submerged raft' has been adopted. Where no allowance is made for the skin friction acting on the pile block, the results suggest that the 'equivalent' raft be located near the top of the piles. However, if the working skin friction developed on the pile block is taken into account (Wood, 1978) then a depth equal to two-thirds of the pile length may be adopted. Some typical results are shown in Fig. A11. Smith *et al* (1970) assessed the influence of pile stiffness on the foundation stresses of multistorey shear-wall structures using finite-element methods of analysis. Allowances were made for the effect of the piled foundations by representing each pile as a horizontal and vertical spring. The spring stiffnesses are determined from pile loading tests, and the proposed form of analysis may be reasonable since a linear relationship exists between load and pile deflection up to normal working loads on piles.

In many instances, especially for large prestigious buildings, a piled foundation design is used in order to limit settlements to acceptably small values. When this option is chosen, it is not unusual for the pile layout to be designed to carry the total load of the structure. Simons (1976) and Burland *et al* (1977) suggested that this need not be so. Padfield & Sharrock (1983) have used the

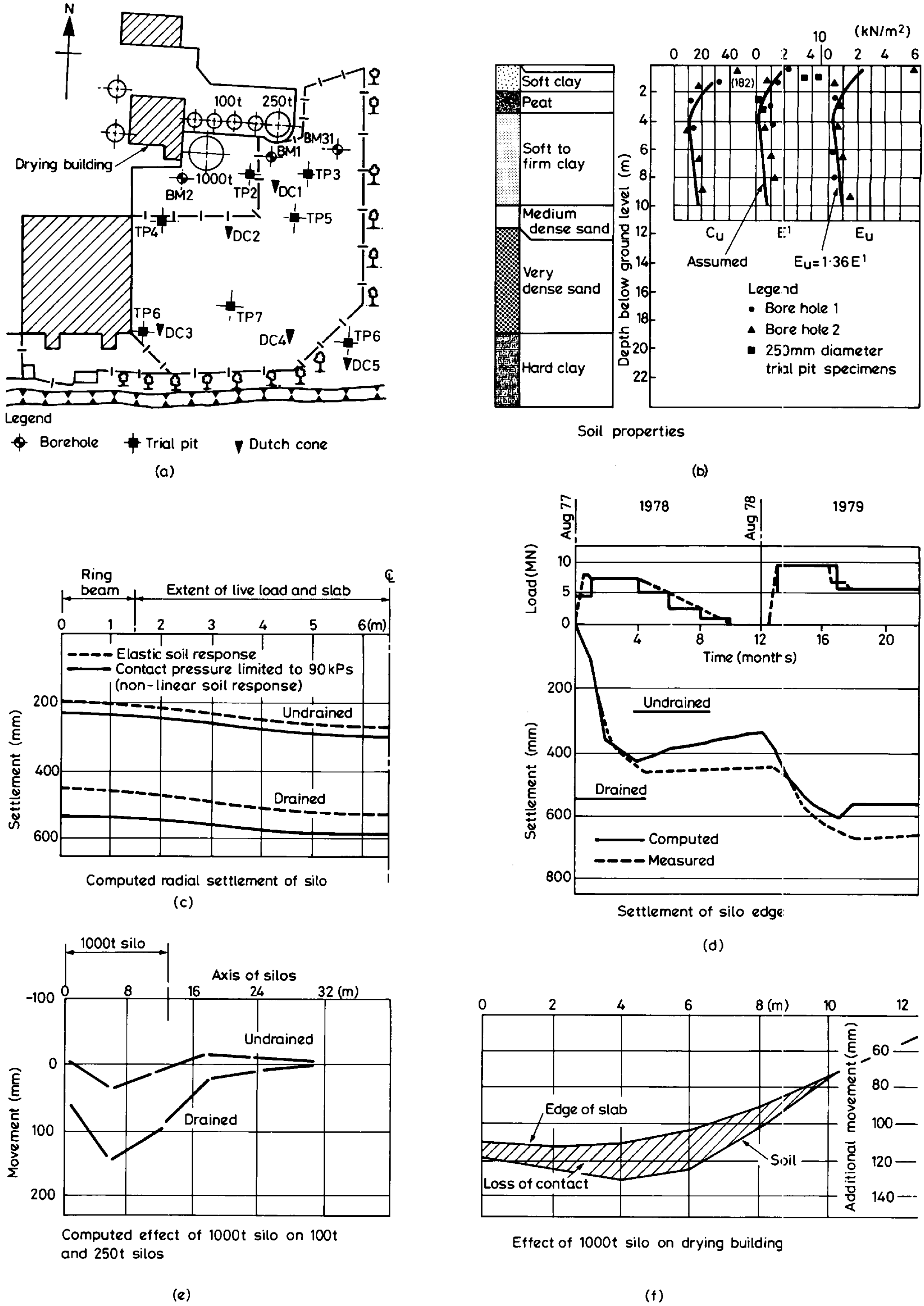

N
Drying building
100t
250t
1000t
BM31
BM1
BM2
TP2
TP3
DC1
TP4
DC2
TP5
TP6
DC3
TP7
DC4
TP6
DC5
Legend
Borehole
Trial pit
Dutch cone
(a)
Soft clay
Peat
Soft to firm clay
Medium dense sand
Very dense sand
Hard clay
Depth below ground level (m)
(kN/m2)
(182)
Cu
E1
Eu
Assumed
Eu=1·36E1
Legend
Bore hole 1
Bore hole 2
250mm diameter trial pit specimens
Soil properties
(b)
Ring beam
Extent of live load and slab
Elastic soil response
Contact pressure limited to 90kPs (non-linear soil response)
Settlement (mm)
Undrained
Drained
Computed radial settlement of silo
(c)
Aug 77
1978
Aug 78
1979
Load (MN)
Time (months)
Settlement (mm)
Undrained
Drained
Computed
Measured
Settlement of silo edge
(d)
1000t silo
Axis of silos
Movement (mm)
Undrained
Drained
Computed effect of 1000t silo on 100t and 250t silos
(e)
Additional movement (mm)
Edge of slab
Loss of contact
Soil
Effect of 1000t silo on drying building
(f)

Fig. A10

case study reported by Cooke *et al* (1981) of a 16-storey block of flats on London clay to demonstrate the feasibility of settlement-reducing piles. These are piles designed to a factor of safety of one and located in plan position in order to reduce differential settlement, and hence bending moments, to a minimum. They show, in a redesign exercise, that instead of the 351 piles actually used as few as 40 settlement-reducing piles located towards the centre of the building would be sufficient to reduce differential settlements to a tolerable level. Cooke (1986) has produced a most valuable review of piled-raft behaviour.

A.5 Dynamic response of soil–structure systems

A.5.1 Dynamic behaviour

General

In general, the combined dynamic characteristics of a soil–structure system govern the nature of its response to any given dynamic excitation. Dynamic amplifications and/or attenuations, and the amounts of damping (material or radiation) that occur in different parts of the system are of fundamental importance. If the nature of the subsoil significantly affects either the nature of the excitation or the behaviour of the soil–structure system, then due allowance for the soil should be made in the design. Amplification or attenuation of displacements or forces may be associated not only with a given total system, but also with components of a system, and in order to achieve a satisfactory design an analysis may be necessary of the degree of amplification or attenuation likely to occur individually in the soil, the structure or in parts thereof.

Amplification effects in the soil are particularly important when the vibrational energy travels through the soil to the structure, such as occurs in earthquakes. Although the overall intensity of vibration attenuates with distance from the energy source, some

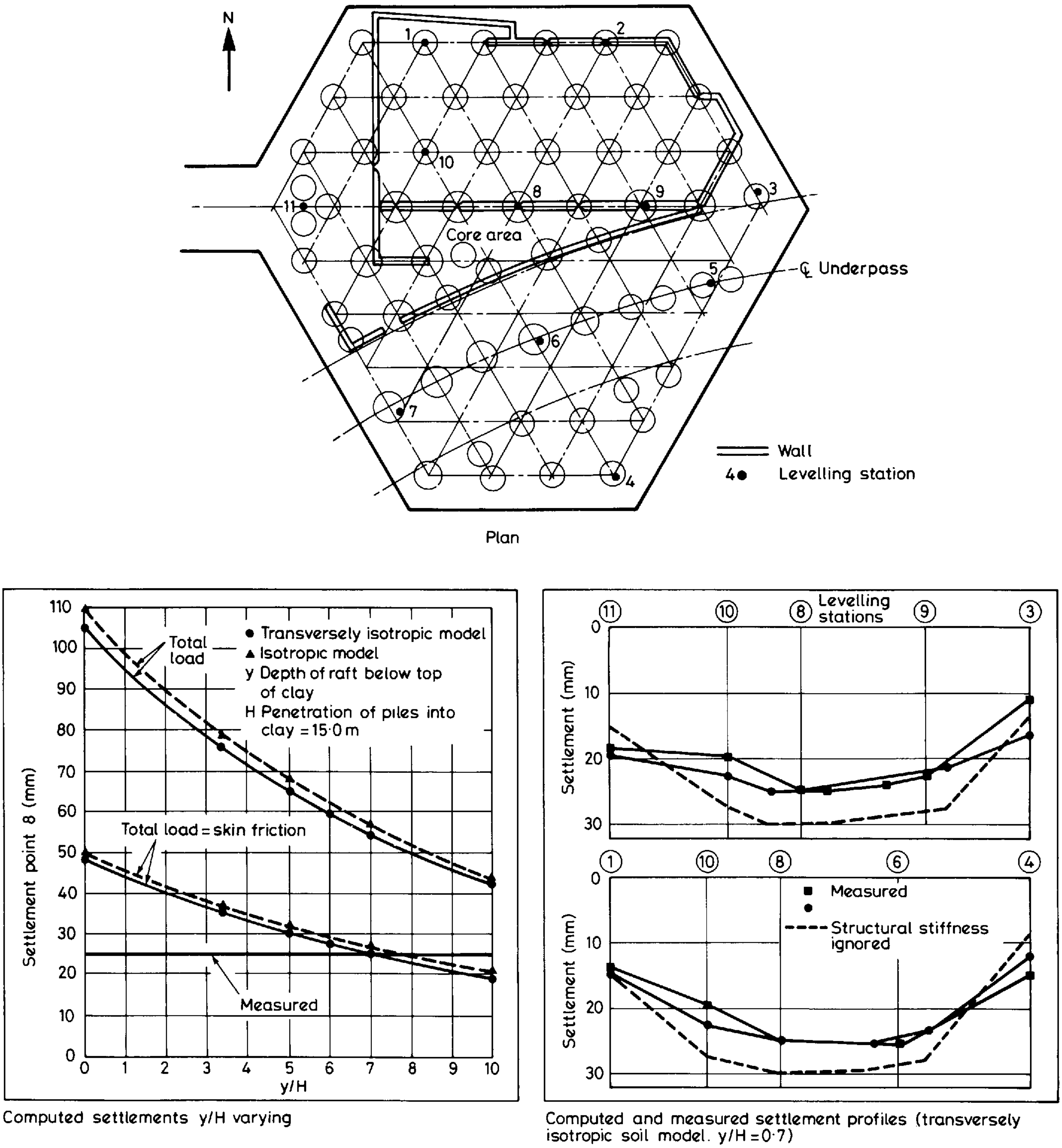


Fig. A11 Piled foundation
Based on Ward (1978)

parts of the frequency content may be amplified during wave propagation depending on the dynamic characteristics of the soil along the propagation path, particularly the natural periods of vibration and reflection. This phenomenon is widely discussed in earthquake literature and has been reviewed by Dowrick (1977). The dynamic characteristics of soil are briefly discussed below in terms of soil stiffness, material damping and radiation damping.

Soil stiffness

In situations where the soil is very stiff, the dynamic response of the structure will be effectively the same as for the rigid-base case. When the soil is not so stiff, the response of the structure may be significantly affected by the dynamic characteristics of the soil.

The mode shapes and periods of the soil–structure system are influenced by the stiffness of the soil part of the system. For example, the fundamental period of an offshore concrete gravity platform sited on 'firm' overconsolidated clay was 5.9s, and was only 2.95s with a rigid base (Watt *et al*, 1976). For buildings, the effect of soil stiffness on the fundamental period may be estimated from eqn. A7 as discussed in subsection A.5.5.

Soil stiffness is normally expressed in terms of shear stiffness G, which for small strains may be taken as the mean of the stress/strain curve. At large strains, the stress/strain curve becomes markedly non-linear, and the consequent dependence of G on the level of shear strain is shown for sands and clays in Fig. A12 (Seed & Idriss, 1970; and Seed *et al*, 1984). It will therefore be necessary to know the level of shear strain in the system for the dynamic excitation under consideration. For many cases the strain will be very small, but for earthquakes the shear strain may range from about 0.001% in small events to greater than 0.1% for large ground motions. Whitman (1976) suggests that for earthquake design purposes a value of ⅔G measured at the maximum strain developed may be used. Alternatively, an approximate value of G can be calculated from the relationship:

$$G = \frac{E}{2\,(1+\nu)} \;..... \text{(A1)}$$

where E is Young's modulus and ν is Poisson's ratio. If specific values of E or G are not available an indication of typical ranges of E-values for different soils is given in Table A2.

Table A2 Typical modulus of elasticity values for soils and rocks (Dowrick, 1977)

soil type	E N/mm^2	E/c_u
soft clay	up to 15	300
firm, stiff clay	10 to 50	300
very stiff, hard clay	25 to 200	300
silty sand	7 to 70	
loose sand	15 to 50	
dense sand	50 to 120	
dense sand and gravel	90 to 200	
sandstone	up to 50 000	400
chalk	5 000 to 20 000	2 000
limestone	25 000 to 100 000	600
basalt	15 000 to 100 000	600

Note that the values of E vary greatly for each soil type, depending on the chemical and physical condition of the soil in question. Hence the above wide ranges of E-value provide only vague guidance prior to test results being available. The ratio E/c_u may be helpful, if the undrained shear strength c_u is known, although the value of this ratio also varies for a given soil type.

Material damping

Material damping may be thought of as a measure of the loss of vibrational energy resulting primarily from hysteretic soil behaviour. Damping increases with the level of shear strain, as illustrated in Fig. A13 (Seed & Idriss, 1969; Seed *et al*, 1984) where a definition of damping ratio is given as a fraction of critical damping.

Such data that exist on soil damping are limited to the results of tests on small samples or from theoretical studies. Some average values for sands and clays are indicated in Fig. A14. Further information may be obtained from Seed & Idriss (1970). Shannon & Wilson (1972) and Seed *et al* (1984).

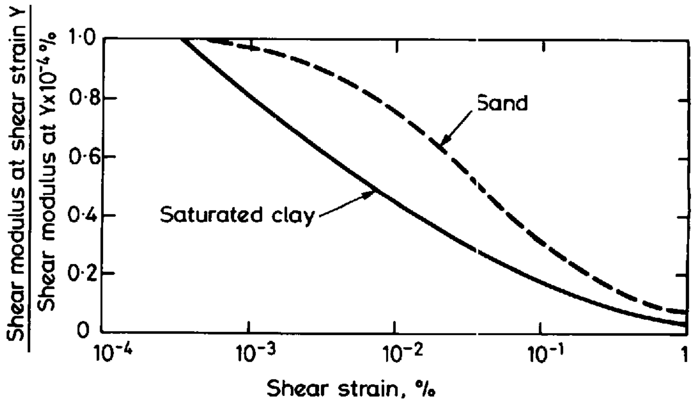


Fig. A12 Average relationship of shear modulus to shear strain for sands and saturated clays
Based on Seed & Idriss (1970) and Seed *et al.* (1984)

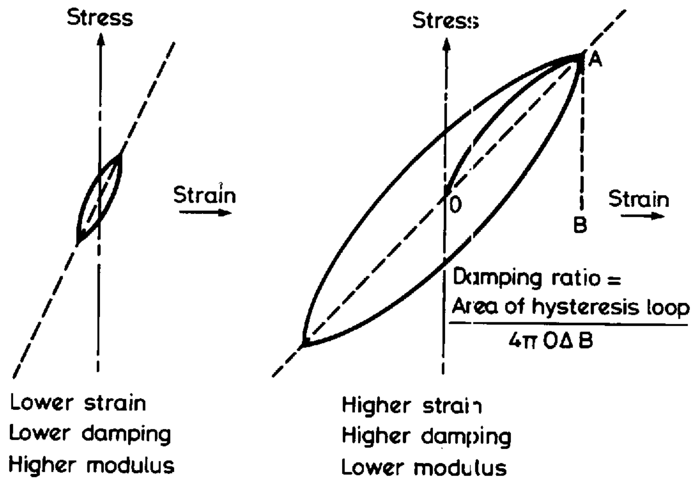


Fig. A13 Illustration defining the effect of shear strain on damping and shear modulus of soils
Based on Seed & Idriss (1969)

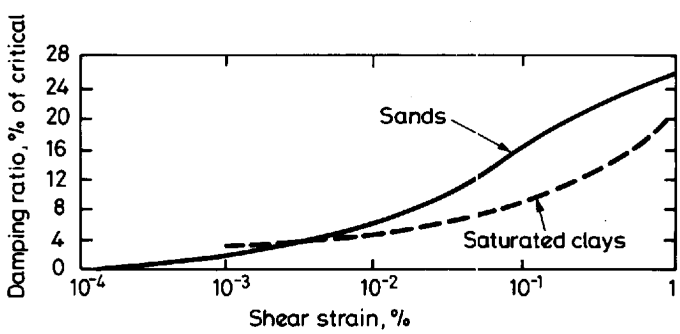


Fig. A14 Average relationship of internal damping to shear strain for sands and saturated clays
Based on Seed & Idriss (1970) and Seed *et al.* (1984)

Radiation damping

Radiation damping is completely different from and additional to material damping, being a measure of the energy loss from the structure through radiation of waves away from the footing. The amount of radiation damping that occurs depends on how much energy is converted into waves radiating from the footing and how much of these waves is trapped in the near fields by reflective boundaries in the soil. An upper bound on the amount of radiation damping is obtained from the elastic half-space theory, which does not incorporate reflective boundaries, and values for circular footings for machines as calculated by Whitman & Richart (1967) are shown in Fig. A15. As with material damping above, the radiation damping in Fig. A15 is expressed as a fraction of critical damping so that it may be conveniently treated as equivalent viscous damping.

In Fig. A15, m is the mass of the foundation block plus machinery, I is the mass moment of inertia of the foundation block plus machinery, R is the radius (or equivalent radius) of the soil contact area at the foundation base, p is the mass density of the soil and μ is Poisson's ratio for the soil. For rectangular bases of plan dimension $B \times L$, the equivalent radii are given by the following:

for translation $R = (BL/m)^{1/2}$
for rocking $R = (BL^3/3m)^{1/4}$
for twisting $R = BL(B^2 + L^2)/6m^{1/4}$

According to the results shown on Fig. A15, radiation damping may be quite large for horizontal and vertical translations (> 10% of critical), while for rocking or twisting it is quite small (about 2% of critical) for normal values of the mass ratios.

As mentioned above, radiation damping decreases when a harder layer underlies a softer surface–soil layer. Hadjian & Luco (1977) demonstrated this numerically, showing that the damping was less for a thin layer supported on harder rock than for a half-space having properties of the top layer.

A.5.2 Dynamic analysis

A wide range of physical conditions exist in which dynamic soil–structure interaction could be considered, as there are many possible combinations of the various types of excitations, structure and soil that are of engineering interest.

Many analytical tools are available, and these tools can be combined in a number of ways to treat various problems in different degrees of detail or sophistication. No universal method exists. For some problems no satisfactory analytical tool may be available, and empirical methods have to be relied on, such as those discussed for settlements arising from explosions and pile driving in subsection A.5. This review does not pretend to be thoroughgoing, but the elements that are common to a wide range of problems will be highlighted, and various analytical tools and techniques will be discussed.

In formulating any dynamic response analysis problem, as shown explicitly in the basic equation of motion.

$$mu + cu + ku = F(t) \quad \text{(A2)}$$

there are four main elements to consider, i.e. mass (m), damping (c), stiffness (k) and excitation ($F(t)$). The way each of these elements is handled varies from problem to problem, but various types of damping and non-linearity and any type of excitation can be treated at least in principle.

Details of the analytical techniques vary according to the nature of the excitation. These can be divided into two broad categories:

(*a*) those cases where the excitation is applied directly to the structure (e.g. wind, waves, machinery)
(*b*) situations where the excitation is applied to the structure through the soil (e.g. earthquakes, explosions and vibrations arising from pile driving, traffic and various other machines).

Early work on soil–structure interaction was related largely to machine foundations, and notable works to be referred to are those of Barkan (1962), Whitman & Richart (1967), Richart *et al* (1970). Guidance on the design and analysis of machine foundations is also given by CP 2012: Part 1 (1974). This work on machine foundations may well be directly applicable to other dynamic loadings in category (*a*) above, e.g. wave loading of offshore platforms.

More recent work has been inspired by the earthquake hazard (particularly to nuclear reactors), and useful references include the *Proceedings of the 8th world conference on earthquake engineering* (1984), a report by an *ad hoc* Group to the ASCE (1976) and Dowrick (1977). A general review of dynamic soil–structure interaction has been published by Wolf (1985). Although the analytical techniques for earthquakes might be more immediately applicable to other dynamic excitations in category (*b*) above, they are suitable for general application. For example, computer programs exist that are capable of dealing with both wave and earthquake loadings (DAFT, 1976), and techniques inspired by the earthquake frequency-dependent approach have been applied at manual calculation level to machine foundation problems, such as through the use of dynamic load factors presented by Danay (1977). An interesting combination of machine foundations and earthquake response has been described by Tajimi *et al* (1977).

Another way in which the nature of the excitation affects the

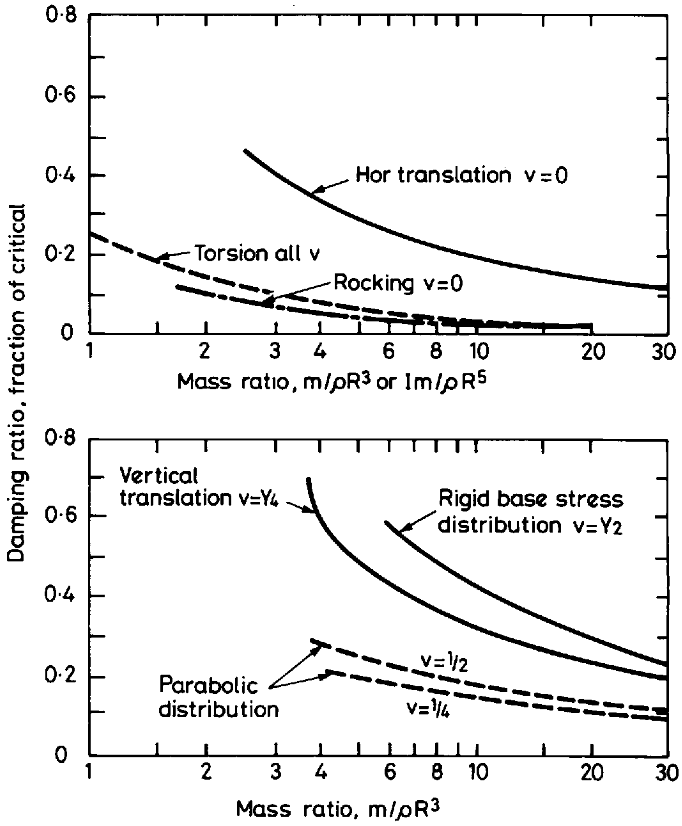


Fig. A15 Values of equivalent damping ratio for radiation damping of machines derived from the theory of circular footings on elastic half-space
Based on Whitman & Richart (1967)

detail of the analytical tools relates to the degree of certainty with which the excitation can be defined. In this sense, the choice of analytical method will lie between a deterministic and probabilistic approach. For ill-defined or random loadings, the latter method of describing the excitation may be preferable, and this technique is widely employed for earthquakes, waves and wind. The probabilistic approach suffers from the limitation that it employs the principle of superposition and hence is not applicable to full non-linear analysis. It also fails to give a clear mechanistic description of the behaviour of a system and therefore should not generally be employed without comparison with a deterministic analysis (Bell *et al,* 1976).

A.5.3 Soil models for dynamic analysis

In modelling the soil the choice lies between 'springs and dashpots' and finite elements (see Fig. A16). Springs and dashpots provide the simpler and cheaper approach, involving less degrees of freedom. The engineer will have to decide on the nature of the springs most appropriate to the problem in hand. He will need to decide how the stiffnesses of the springs will be determined, whether they will be linear or non-linear, and whether the springs and dashpots will be frequency-dependent or frequency-independent.

Static spring stiffnesses are obtained in a convenient form from the elastic half-space theory (Table A3). Their accuracy depends on selecting a suitably equivalent value of shear modulus G to

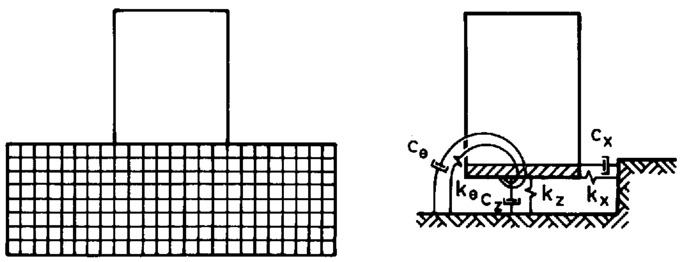


Fig. A16 Soil modelled with finite elements or springs and dashpots represent the actual subsoil as distinct from the idealized half-space. This equivalent value must allow for the change in stiffness with depth and with strain level (see Fig. A12). Where pure half-space values are not considered adequate, the static spring stiffness may be determined either from a layered half-space approach (Luco, 1976), or from static finite-element analyses (Kausel & Roesset, 1975). An indication of the difference in stiffness and radiation damping between the half-space and various layered conditions is given by Hadjian Luco (1977).

Soil is a highly non-linear material (see Fig. A13). Techniques exist for full non-linear dynamic analysis, but in view of the expense of such analyses, and the lack of agreement on suitable 2- and 3-dimensional non-linear models for soil behaviour, full non-linear analyses remain essentially a research tool. Hence for practical design purposes, it is common practice to use an equivalent linear model. The cost problem is particularly pertinent for finite-element analyses. For important projects a non-linear formulation of the soil properties may become justified, e.g. non-linear soil springs have been used in predesign studies for large offshore concrete oil platforms.

Table A3 Equivalent lumped parameters for analysis of circular foundations (Whitman, 1976)

	vertical	swaying	rocking	twisting
spring constant	$\frac{4GR}{1-v}$	$\frac{8GR^3}{2-v}$	$\frac{8GR^3}{3(1-v)}$	$\frac{16GR^3}{3}$
mass ratio B	$\frac{M(1-v)}{4pR^3}$	$\frac{M(2-v)}{8pR^3}$	$\frac{3I(1-v)}{8pR^5}$	$\frac{I_t}{pR^5}$
damping ratio	$\frac{0.425}{\sqrt{B}}$	$\frac{0.29}{\sqrt{B}}$	$\frac{0.15}{(1+B)\sqrt{B}}$	$\frac{0.50}{1+2B}$
effective mass	$\frac{0.27M}{B}$	$\frac{0.095M}{B}$	$\frac{0.24I}{B}$	$\frac{0.25I_t}{B}$

G = shear modulus of soil
R = radius of foundation
M = mass of foundation
I = moment of inertia about horizontal axis
I_t = moment of inertia about vertical axis
v = Poisson's ratio
B = mass ratio

The frequency-dependent expression for the dynamic stiffness (impedance) k^* of a footing is of the form

$$k^*(w) = k(w) + iwc(w) \qquad \text{(A3)}$$

in which the real part of the expression may be considered as a stiffness and the imaginary part represents the radiation damping. (The zero-frequency value of the stiffness $K(w = o)$ is given in Table A3.) This type of formulation has been widely discussed in the literature, ranging from the original work of Bycroft (1956) to the work on elastic half-space by Luco & Westmann (1971) and Veletsos & Wei (1971) or the equivalent finite-element work of Vaish & Chopra (1974). These elastic formulations do not include material damping (a dashpot must be added to the system analytical model), but variation of the elastic approach directly incorporates linear hysteretic damping by replacing the term w in eqn. A4 by a term of the form ci and $c(w)$. This viscoelastic approach has been widely examined, for example by Veletsos & Verbic (1973) and Kausel & Roesset (1975).

As the frequency-dependent formulation is solved in the frequency domain, it is not appropriate either to modal analysis or to a full non-linear formulation, which requires a time-domain analysis. However, a reasonable approximation to the frequency-dependent springs may usually be achieved with equivalent frequency-independent springs, although the approximations may not equally good at all points in the system (Watt *et al*, 1976). For horizontal and vertical translation, the radiation damping ratios vary little with frequency, but for rotational motions the ratios vary considerably, and it is necessary to select a value of rotational damping coefficient $c(w)$ corresponding to the most important value of w for the system response. The value of the circular frequency chosen may perhaps be taken as that corresponding to the predominant period of the soil–structure system. This period may or may not be fundamental period, as indicated in Fig. A17.

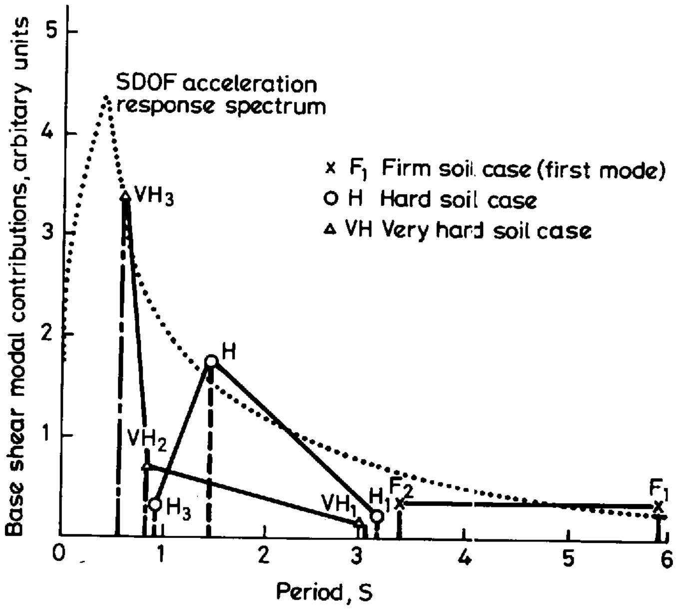


Fig. A17 Base shear contributions of the significant modes for a concrete gravity oil platform with 3-dimensional foundation stiffness
Based on DAFT (1976)

Deeply embedded foundations at present give rise to a greater degree of uncertainty as to the validity of analytical techniques than for structures founded at or near the surface. The layered half-space concept permits analysis of embedded foundation situations, and various studies have been made, including those of Bielak (1975), Luco *et al* (1975), Novak & Beredugo (1972) and Lin (1984).

The use of finite elements for modelling the foundations of a soil–structure system is the most comprehensive (if most expensive) method available. Like the half-space model it permits radiation damping and 3-dimensionality, but has the major advantage of easily allowing changes of soil stiffness both vertically and horizontally to be explicitly formulated. Embedment of footings is also readily dealt with. Although a full 3-dimensional model is generally too expensive, three dimensions should be simulated. This can be achieved by an equivalent 2-dimensional model, or for structures with cylindrical symmetry, an analysis in cylindrical coordinates may be used (Kausel, 1974).

In order to simulate radiation of energy through the boundaries of the element model three main methods are available:

- elementary boundaries that do not absorb energy and rely on the distance to the boundary to minimize the effect of reflected waves
- viscous boundaries that attempt to absorb the radiating waves, modelling the far field by a series of dashpots and springs, as used by Lysmer & Kuhlmeyer (1969). The accuracy of this method is not very good for thin surface layers or for horizontal excitation, although an improved version has been developed by Ang & Newmark (1971)
- consistent boundaries, which are the best absorptive boundaries at present available, reproducing the far field in a way consistent with the finite-element expansion used to model the core region. This method was developed by Lysmer & Waas (1972) and generalized by Kausel (1974). The latter method among other things allows the lateral boundary to be placed directly at the side of the foundation, with a considerably reduction in the number of degrees of freedom.

It should be noted that in recent work using the layered half-space technique, Hadjian & Luco (1977) point out that what is considered proper finite-element modelling for rocking motions may be completely inadequate for vertical or even horizontal motions.

Finally, it should be mentioned that piled foundations may be modelled as special cases of the above methods. Guidance may be obtained from Penzien (1975), Novak (1977), Margason & Holloway (1977) and the Applied Technology Council (1982).

A.5.4 Models for dynamic analysis

The structural modelling for dynamic soil–structure analysis poses no technical problems peculiar to the soil–structure concept, and normal modelling procedures for dynamic analysis apply, such as outlined by Clough & Penzien (1975). Problems of balance in the total-system modelling do arise, however, depending on the main purpose of a particular analysis. The number of degrees should be sufficient to express the mass and stiffness distributions adequately, as well as give response data relevant to the design of sufficient part of the system.

A.5.5 Seismic soil–structure interaction

In many cases it would not be desirable or economically feasible to carry out a full dynamic analysis of a soil–structure system along the lines noted above. It is thus convenient that the Applied Technology Council (1982) has given an approximate method of estimating soil–structure interaction effects on buildings. This method is based largely on work by Veletsos & Wei (1971), Veletsos & Verbric (1973) and Veletsos (1977), and is appropriate for use within the equivalent-static code approach to analysing earthquake forces in buildings. The resultant horizontal earthquake force, $\tilde{V}$, acting on a building may be found by deducting the soil–structure interaction $\triangle V$ from the horizontal force V for a fixed-base structure, i.e.

$$\tilde{V} = V - \triangle V \quad \text{(A4)}$$

where

$$\triangle V = \left[C_s - \tilde{C}_s \left(\frac{0.05}{\tilde{\beta}}\right)^{0.4}\right] \overline{W} \quad \text{(A5)}$$

where C_s and $\tilde{C}_s$ are the seismic design coefficients for the fixed base and the flexibility supported structure, respectivly, and $\overline{W}$ is the effective gravity load of the building, $\tilde{\beta}$ is the fraction of critical damping for a soil–structure system, and is found from

$$\tilde{\beta} = \beta_o + \frac{0.05}{(\tilde{T}/T^3)} \quad \text{(A6)}$$

where β_o is a foundation damping factor that allows for both material and radiation damping and is found from Fig. A18.

T and $\tilde{T}$ are the fundamental periods of vibration for the fixed base and the flexibily supported structure, respectively.

The Applied Technology Council (1982) gives simple formulae for estimating T of the form

$$\tilde{T} = T\sqrt{\left[1 + \frac{k}{k_x}\left(1 + \frac{k_x h^2}{k_1 k_\theta}\right)\right]} \quad \text{(A7)}$$

where k is the stiffness of the building when fixed at the base, given by

$$k = 4\pi^2 \frac{\overline{W}}{gT^2} \quad \text{(A8)}$$

where h is the effective height of the building (equals 0.7 times the height of a multistorey building).

k_y and k_θ are lateral and rocking stiffness, respectively, of the foundation, i.e. the spring constants given in Table A2 above.

The Applied Technology Council (1982) gives guidance on further foundation variables such as embedment, multiple footings and piles.

Using the above method, the wide variation in the degree of soil–structure interaction is readily seen as a function of $\triangle V/V$. For example, if $\tilde{T}/T = 1.1$, the reduction in seismic response $\triangle V/V$ is 15% or 4% if the ratio of effective height to radius, h/r, is 1.0 or 5.0, respectively.

It should be noted that the above method is appropriate for soils without strongly reflective layering at relatively shallow depths. Where such layering exists radiation damping effects are reduced, and eqn. A5 will overestimate the benefits of soil–structure interaction, and hence the method will be unconservative.

References

Ang, A. H., & Newmark, N. M. (1971): *Development of a transmitting boundary for numerical wave motion calculations,* Report to Defence Atomic Support Agency (US), Washington DC

Applied Technology Council (1982): *Tentative provisions for the development of seismic regulations for buildings,* ATC 3–06, National Bureau of Standards, NBS SP-510, USA

ASCE (1976): *Analyses for soil–structure interaction effects for nuclear power plants,* Report by the *ad hoc* Group on Soil–Structure Interaction, Nuclear Structures and Materials Committee, Structural Division, Final draft, April

Barkan, D. D. (1962): *Dynamics of bases and foundations,* McGraw-Hill

Bell, K., Hansteen, O. E., Larsen, P. K., & Smith, E. K. (1976): 'Analysis of wave–structure–soil system—case study of gravity platform', *Proc. conf. on behaviour of offshore structures,* Norwegian Institute of Technology, p. 846

Benjamin, J. R., & Williams, H. A. (1960): 'Reinforced-concrete shear wall assemblies', *Jnl. Struct. Div.,* ASCE. August, p. 1

Bhatt, P. (1973): 'Influence of vertical joints on the behaviour of precast shear walls', *Building science,* **8,** Pergamon Press, p. 221

Bielak, J. (1975): 'Dynamic behaviour of structures with embedded foundations', *Earthquake engineering structural dynamics,* **3,** no. 3, p. 259

Borowicka, H. (1936): 'Influence of rigidity of a circular foundation slab on the distribution of pressures over the contact surface', *Proc. 1st int. conf. SM&FE,* Harvard, **2,** p. 144

Boswell, L. F., & Scott, C. R. (1975): 'A flexible circular plate on a heterogeneous elastic half-space: influence coefficients for contact stress and settlement', *Geotechnique,* **25,** 3, p. 604

Brebbia, C. A. (1981): 'Boundary element method in plasticity', App. Math. Modelling, **5,** no. 4, Aug., p. 275

Brown, P. T. (1969): 'Numerical analysis of uniformly loaded circular rafts on elastic layers of finite depth', *Geotechnique,* **19,** no. 2, p. 301

Brown, P. T. (1974): 'Influence of soil inhomogeneity on raft behaviour', *Soils and foundations, Jap. Soc. SM&FE,* **14,** no. 1, p. 61

Brown, P. T., & Wiesner, T. J. (1975): 'The behaviour of uniformly loaded piled strip footings', *Soils and foundations, Jap. Soc. SM&FE,* **15,** no. 4, p. 13

Bryan, E. R. (1973): *The stressed skin design of steel buildings,* Constrado monograph, Crosby Lockwood Staples, London

Bryan, E. R., & Davies, J. M. (1976): *Diaphragm action in multi-storey buildings,* University of Salford, Department of Civil Engineering, Report no. 75-70, January

Burnett, E. F. P., & Rejendra, R. C. S. (1972): 'Influence of joints in panelized structural systems', *Jnl. ASCE,* **98,** ST9, p. 1943

Butterfield, R., & Banerjee, P. K. (1971a): 'Elastic analysis of compressible piles and pile groups', *Geotechnique,* **21,** no. 1, p. 43

Butterfield, R., & Banerjee, P. K. (1971b): 'Problem of pile group–pile cap interaction', *Geotechnique,* **21,** no. 2, p. 135

Bycroft, G. N. (1956): 'Forced vibrations of a rigid circular plate on a semi-infinite elastic space and on an elastic stratum', *Phil. trans., Royal Society,* London, Series A., **248,** p. 327

Chamecki, S. (1956): Structural rigidity in calculating settlements, *Jnl. Soil Mech. Found. Div., ASCE,* **82,** SM1, p. 1

Chan, H. C., & Cheung, Y. K. (1974): 'Contact pressure of rigid footings on elastic foundations', *Civ. Eng., ASCE,* no. 813, p. 51

Cheung, Y. K., & Zienkiewicz, O. C. (1965): 'Plates and tanks on elastic foundations—an application of finite element method', *Int. jnl. solids struct.,* **1,** no. 4, p. 451

Clough, R. W., & Penzien, J. (1975): *Dynamics of structures,* McGraw-Hill, New York

Constrado (1977): *European recommendations for stressed skin design,* ECCS-XVII-77-1E, no. 19, London

Cooke, R. W. (1986): 'Piled raft foundations on stiff clays – a contribution to design philosophy', *Geotechnique,* **36,** no. 2, June, p. 169

Cooke, R. W. *et al* (1981): 'Some observations of the foundation loading and settlement of a multi-storey building on a piled raft foundation in London clay', *Proc. ICE,* Pt. 1, **70,** Aug., p. 433

CP 2012 (1974): *Foundations for machinery,* Part 1: *Foundations for reciprocating machines,* British Standards Institution, London

DAFT (1976): *DAFT—Dynamic analysis using Fourier transforms for soil–structure systems onshore and offshore—theoretical formulation,* Ove Arup & Partners, London

Danay, A. (1977): 'Vibrations of rigid foundations', *Arup Jnl.,* Ove Arup Partnership, London

Davies, J. M. (1976): 'Calculation of steel diaphragm behaviour', *Jnl. Struct. Div., ASCE,* **102,** no. ST7, July, p. 1411

Davis, E. H., & Poulos, H. G. (1972): 'The analysis of pile raft systems', *Aust. Geomech. Jnl.,* **G2,** no. 1, p. 21

Desai, C. S. (1982): 'Mechanics of three-dimensional soil–structure interaction', *ASCE Jnl., Eng. Mech. Div.,* **108,** no. EM5, Oct., p. 731

Desai, C. S., & Sargand, S. (1984): 'Hybrid FE procedure for soil–structure interaction', *Jnl. Geotech. Eng.,* **110,** no. 4, April, p. 473

Dowrick, D. J. (1977): *Earthquake resistant design—a manual for engineers and architects,* Wiley, London

El-Dakhakhni, W., & Daniels, J. H. (1973): *Frame-floor-wall system interaction in buildings,* Fritz Engineering Lab. Report no. 376–2, Lehigh Univ., April

Fiorato, A. E., Sozen, M. A., & Gamble, W. L. (1970): *An investigation of the interaction of reinforced concrete frames with masonry filler walls,* Civil Engineering Studies Structural Research Series, no. 370, Univ. of Illinois

Gorbunov-Possadov, M. I., & Serebrjanyi, R. V. (1961): 'Design of structures on elastic foundations', *Proc. 5th int. conf. SM&FE,* Paris, **1,** p. 643

Grasshof, H. (1957): 'Influence of flexural rigidity of superstructure on the distribution of contact pressure and bending moments of an elastic combined footing', *Proc. 4th int. conf. SM&FE,* London, **1,** p. 300

Hadjian, A. H., & Luco, J. E. (1977): 'On the importance of layering on the impedance functions', *Proc. 6th world conf. on earthquake engineering,* New Delhi, p. 4–205

Hain, S. J., & Lee, I. K. (1974): 'Rational analysis of raft foundations', *Jnl. Geotech. Eng. Div., ASCE,* **100,** no. GT7, p. 843

Heil, H. (1969): 'Studies on the structural rigidity of reinforced concrete building frames on clay', *Proc. 7th int. conf. SM&FE,* Mexico, **2,** p. 115

Hongladaromp, T., Chen, N-J., & Lee, S-L. (1973): 'Load distribution in rectangular footings on piles', *Geotech. eng.,* **4,** no. 2, p. 77

Hooper, J. A. (1973): 'Observations on the behaviour of a piled-raft foundation on London clay', *Proc. ICE,* **55,** Pt 2, December, p. 855

Hooper, J. A. (1984): 'Raft analysis and design – some practical examples', *Struct. Eng.,* **62A,** no. 8, Aug., p. 233

Hooper, J. A., & Philiastides, A. (1986): 'Design of a flexible raft foundation of irregular shape', *Proc. ICE,* **80,** Pt 1, Aug., p. 1013

Hooper, J. A., & West, D. J. (1983): 'Structural analysis of a circular raft on yielding soil', *Proc. ICE,* **75,** Pt 2, June, p. 205

Hooper, J. A., & Wood, L. A. (1977): 'Comparative behaviour of raft and piled foundations', *Proc. 9th int. conf. SM&FE,* Tokyo

Hooper, J. A. (1976): 'Parabolic adhesive loading of a flexible raft foundation', *Geotechnique,* **26,** no. 3, p. 511

Hooper, J. A. (1974): 'Analysis of a circular raft in adhesive contact with a thick elastic layer', *Geotechnique,* **24,** no. 4, p. 561

Hooper, J. A. (1976): 'The effect of flexural cracking on differential raft settlements', *Proc. ICE,* **61,** Pt 2, p. 567

International Conference of Building Officials (US) (19xx): *Uniform building code*

Kausel, E., & Roesset, J. M. (1975): 'Dynamic stiffness of circular foundations', *Jnl. Eng. Mech. Div., ASCE,* **101,** no. EM6

Kausel, E. (1974): *Forced vibrations of circular foundations on layered media,* MIT research report R74-11, Massachusetts Institute of Technology

King, G. J. W., & Chandrasekaran, V. S. (1977): 'Interactive analysis using a simplified soil model', *Proc. int. symp. on soil–structure interaction,* Roorkee, India

King, G. J. W., & Chandrasekaran, V. S. (1975): 'An assessment of the effects on interaction between a structure and its foundation', *Conf. settlement of structures,* Cambridge, Pentech Press, London, p. 368

King, G. J. W., & Chandrasekaran, V. S. (1974): 'Interactive analysis of a rafted multistorey space frame resting on an inhomogeneous clay stratum', *Proc. conf. finite element methods in engineering,* Univ. NSW., p. 493

Lanarch, W. J. (1970): 'Computation of settlements in building frames', *Civ. eng. & pub. wks. rev.,* **65,** p. 1040

Lee, I. K., & Brown, P. T. (1972): 'Structure–foundation interaction analysis', *Jnl. Struct. Div.,* ASCE, **98,** no. ST11, p. 2413

Lenczner, D., & Salahuddin, J. (1976): 'Moisture, creep and masonry movements in masonry floors and walls', *Proc. of 1st Canadian masonry symposium,* Calgary, 7–10 June

Lin, A. N. (1984): 'Measured effect of foundation embedment on response', *Proc. 8th world conf. on earthquake. eng.,* San Francisco, **V,** p. 841

Litton, E., & Buston, J. M. (1968): 'The effect of differential settlement on a large, rigid, steel-framed, multi-storey building', *Structural Engineer,* **46,** no. 11, November, p. 353

Luco, J. E. (1976): 'Vibrations of a rigid disc on a layered viscoelastic medium', *Nuclear engineering & design,* **36,** no. 3, p. 325

Luco, J. E., Wong, H. L., & Trifunac, M. D. (1975): 'A note on the dynamic response of rigid embedded foundations', *Earthquake engineering & structural dynamics,* **4,** no. 2, p. 119

Luco, J. E., & Westmann, R. A. (1971): 'Dynamic response of circular footings', *Jnl. Engineering Mechanics Division, ASCE,* **97,** no. EM5, p. 1381

Lysmer, J., & Waas, G. (1972): 'Shear waves in plane infinite structures', *Jnl. Eng. Mech. Div., ASCE,* **98,** no. EM1, p. 85

Lysmer, J., & Kuhlemeyer, R. L. (1969): 'Finite element model for infinite media', *Jnl. Eng. Mech. Div., ASCE,* **95,** no. EM4, p. 859

MacLeod, I. A. (1973): 'Analysis of shear wall buildings by the frame method', *Proc. ICE,* **55,** September, p. 593

MacLeod, I. A., & Hosney, H. M. (1976): 'The distribution of vertical load in shear wall buildings', *Structural Engineer,* **54,** no. 2, February, p. 67

MacLeod, I. A., & Green, D. R. (1975): 'Three dimensional analysis of shear wall buildings', *Bull. IASS,* N60

Mainstone, R. J. (1974): *Supplementary note on the stiffnesses and strengths of infilled frames,* CP 13/74, Building Research Establishment, Garston

Mainstone, R. J. (1971): 'On the stiffness of infilled frames', *Proc. ICE,* Supplement (iv), 7360S, p. 57 (reissued as BRS CP 2/72)

Mainstone, R. J., & Weeks, G. A. (1970): 'The influence of a bounding frame on the racking stiffnesses and strengths of brick walls', *Proc. 2nd int. brick conf.,* p. 165 (reissued as BRS CP 3/72)

Majid, K. I., & Cunnell, M. D. (1976): 'A theoretical and experimental investigation into soil–structure interaction', *Geotechnique,* **26,** no. 2, p. 331

Margason, E., & Holloway, D. M. (1977): 'Pile bending during earthquakes', *Proc. 6th world conf. on earthquake engineering,* New Delhi, p. 4–233

Meyerhof, G G. (1953): 'Some recent foundation research and its application to design', *Structural Engineer,* **31,** June, p. 151

Miller, C. J. (1972): *Analysis of multi-storey frames with light* *gauge steel panel infils,* Dept. Struct. Eng., Report no. 349, Cornell Univ., August

Nayar, K. K., & Coull, A. (1976): 'Elastoplastic analysis of coupled shear walls', *Jnl. Struct. Div., ASCE,* **102,** no. ST9, proc. paper 12401, September, p. 1845

Naylor, D. J., & Hooper, J. A. (1975): 'An effective stress finite element analysis to predict the short and long-term behaviour at a piled raft foundation on London clay', *Conf. settlement of structures,* Cambridge, Pentech Press, London, p. 394

Novak, M., & Beredugo, Y. O. (1972): 'Vertical vibration of embedded footings', *Jnl. Soil Mech. & Found. Div., ASCE,* **98,** no. SM12, p. 1291

Novak, M. (1977): 'Soil–pile interactions'. *Proc. 6th world conf. on earthquake engineering,* New Delhi. p. 4–97

Nyberg, G. (1976): *Diaphragm action of assembled C-shaped panels,* Swedish Council for Building Research, Document D9

Oppenheim, I. J. (1973): 'Control of lateral deflection in planar frames using structural partitions, *Proc. ICE,* **55,** Pt 2, June, p. 435

Padfield, C. J., & Sharrock, M. J. (1983): *Settlement of structures on clay soils,* CIRIA special publication 27/PSA civil engineering technical guide 38, London

Paulay, T. (1971): 'Coupling beams of reinforced concrete shear walls', *Jnl. Struct. Div., ASCE,* **97,** no. ST3, proc. paper 7984, March, p. 843

Paulay, T. (1970): 'Elasto-plastic analysis of coupled shear walls', *ACI Jnl., Proc.,* **67.** no. 11, November, p. 915

Penzien, J. (1975): 'Seismic analysis of platform structure–foundation systems', paper no. 2352. *Proc. offshore technology conf.,* Houston, Texas, **111,** p. 153

Petersson, H. (1974): *Analysis of load bearing walls in multistorey buildings,* Chalmers University of Technology, Gothenburg

Pollner, E., Tso, W. K., & Heidebrecht, A. C. (1975): 'Analysis of shear walls in large-panel construction', *Canad. Jnl. of Civil Engineering,* **2,** no. 3, p. 357

Polyakov, S. V. (1960): *On interaction between masonry filler walls and enclosing frame when loaded in the plane of the wall,* Translations in earthquake engineeering, Earthquake Engineering Research Institute, San Francisco

Poulos, H. G. (1968): 'Analysis of the settlement of pile groups', *Geotechnique.* **18,** no. 4, p. 449

Proc. 8th world conf. on earthquake engineering (1984): 'Soil–structure interaction', **V,** Part 5.2, San Francisco

Randolph, M. F. (1981): 'Piles subject to torsion', *ASCE Jnl. Geotech. Eng. Div.,* **107,** no. 8, Aug., p. 1095

Randolph, M. F. (1983): 'Settlement considerations in the design of axially loaded piles', *Ground Eng.,* **16,** no. 4, May, p. 28

Richart, F. E., Hall, J. R., & Woods, R. D. (1970): *Vibrations of soils and foundations,* Prentice-Hall

Rosman, R. (1970): 'Analysis of spatial concrete shear wall systems', paper 7266S, *Proc. ICE,* Supplement (vi), p. 131

Sachanski, A. (1960): 'Analysis of the earthquake resistance of frame building taking into consideration the carrying capacity of the filling masonry', *Proc. 2nd world conf. on earthquake engineering,* Tokyo, **3,** p. 2127

Seed, H. B., *et al* (1984): *Moduli and damping factors for dynamic analyses of cohesionless soils,* Report no. EERC 84–14, Earthquake Engineering Research Center, University of California, Berkeley

Seed, H. B., & Idriss, I. M. (1970): *Soil moduli and damping factors for dynamic response analyses,* Report no. EERC 70–10, Earthquake Engineering Research Center, University of California, Berkeley

Seed, H. B., & Idriss, I. M. (1969): 'Influence of soil conditions on ground motions during earthquakes', *Jnl. Soil Mech. and Found. Div., ASCE,* **95,** no. SMI, p. 99

Shannon & Wilson (1972): *Soil behaviour under earthquake loading conditions,* State of the art evaluation for US Atomic Energy Commission, Agbabian-Jacobsien Associates, Seattle & Los Angeles

Simms, L. G. (1967): 'The behaviour of no-fines concrete panels as the infill in reinforced concrete frames', *Civ. eng. & pub. wks. rev.,* November, p. 1245

Smith, B. S., & Carter, C. (1969): 'A method of analysis for infilled frames', *Proc. ICE,* **44,** September, p. 31

Smith, B. S. (1966): 'Behaviour of square infilled frames', *Jnl. Struct. Div.,* ASCE. January, p. 381

Smith, R. C., Thorburn, S., & Tinch, W. R. (1970): 'The influence of pile stiffness on the foundation stresses of a multistorey shear wall', *Building science,* **6,** p. 21

Sommer, H. (1965): 'A method for the calculation of settlements, contact pressure, and bending moments in a foundation including the influence of the flexural rigidity of the superstructure', *Proc. 6th int. conf. SM&FE,* Montreal, **2,** p. 197

Steinbrenner, W. (1934): 'Tafeln zur Setzungsberechnung', *Die Strasse,* **1,** p. 121

Tajimi, H., Minowa, C., & Shimomura, Y. (1977): 'Dynamic response of a large scale shaking table foundation and its surrounding ground', *Proc. 6th world conf. on earthquake engineering,* New Dehlhi, p. 4–61

Vaish, A. K., & Chopra, A. K. (1974): 'Earthquake finite element analysis of structure–foundations systems', *Jnl. Engineering Mechanics Division, ASCE,* **100,** no. EM6, p. 1101

Veletsos, A. S. (1977): *Dynamics of structure–foundation systems,* Structural and geotechnical mechanics, Volume honouring N. M. Newmark (W. J. Hall, Ed), Prentice-Hall, Englewood Cliffs, New Jersey

Veletsos, A. S., & Wei, Y. T. (1971): 'Lateral and rocking vibrations of footings', *Jnl. Soil Mechanics & Foundations Division, ASCE,***97,** no. SM9, p. 1127

Veletsos, A. S., & Verbric, B. (1973): 'Vibration of viscoelastic foundations', *Earthquake engineering & structural dynamics,* **2,** p. 87

Watt, B. J., Boaz, I. B., & Dowrick, D. J. (1976): 'Response of concrete gravity platforms to earthquake excitations', *Proc. offshore technology conf.,* Houston, Texas, paper no. 2673

Webb, D. L. (1975): 'Observed settlement and cracking of a reinforced concrete structure founded on clay', *Prof. conf. on settlement struct.,* Cambridge, Pentech Press, London, p. 443

Whitman, R. V. (1976): 'Soil–platform interaction', *Proc. conference on behaviour of offshore structures,* Norwegian Institute of Technology, p. 817

Whitman, R. V. (1970): 'Soil–structure interaction', and 'Evaluation of soil properties for site evaluation and dynamic analysis of nuclear plants', *Seismic design for nuclear power plants,* ed. R. J. Hansen, MIT Press, p. 245

Whitman, R. V., & Richart, F. E. (1967): 'Design procedures for dynamically loaded foundations', *Jnl. Soil Mech. & Found. Div., ASCE,* **93,** no. SM6, p. 169

Wolf, J. P. (1985): *Dynamic soil–structure interaction,* Prentice-Hall, Englewood Cliffs, New Jersey

Wood, L. A. (1978): 'RAFTS: a program for the analysis of soil–structure interaction', *Adv. Eng. Soft.,* **1,** no. 1, p. 11

Wood, L. A. (1980): 'Analysis of piles and walls subject to lateral forces', *Ground Eng.,* **13,** no. 1, Jan., p. 28

Wood, L. A., & Buttling, S. (1984): 'Interactive analysis and observations of a grain silo complex', *Symp. on soil–structure interaction,* IStructE, London, p. 21

Wood, L. A., Larnach, W. J., & Woodman, N. J. (1977): 'Observed and computed settlements of two buildings', *Proc. symp. soil–structure interaction,* Roorkee, India

Wood, L. A., & Perrin, A. J. (1984): 'Predicted and observed behaviour of a raft foundation in London clay', *Symp. on soil–structure interaction,* IStructE, London, p. 25

Wood, L. A., & Perrin, A. J. (1985): 'The performanceof a deep foundation in London clay', *Proc. 11th int. conf. SM&FE,* **4,** p. 2277

Wood, R. H. (1958): 'The stability of tall buildings', *Proc. ICE,* **11,** September, p. 69

Zbirohowski-Koscia, K. F., & Gunasekera, D. A. (1970): 'Foundation settlement and ground reaction calculations using a digital computer', *Civ. eng. pub. wks. rev.,* **65,** p. 152

Index