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Design and construction of deep basements including cut-and-cover structures

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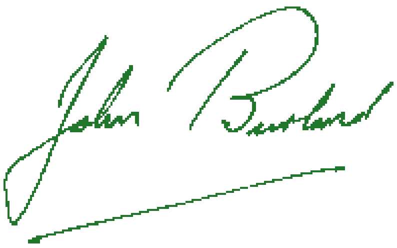
Foreword

How deep is a deep basement? It is not possible to give a precise definition to this question. For the purposes of this report, a deep basement is one for which the depth, structural arrangement and loads, ground conditions and groundwater conditions are such that careful consideration has to be given to the geotechnical aspects as well as to the structural aspects, including interactions between the two. This will normally be the case for a basement greater than about 5-6m deep but in some circumstances it may be less.

Therefore the design and construction of a deep basement is an exercise in ground-structure interaction. It requires all the traditional skills of the engineer including: reliance on observation and measurement; a deep understanding of both geotechnical and construction materials; an appreciation of the effects of groundwater and seepage; the development of appropriate conceptual and analytical models; and above all, judgement based on a knowledge of case histories and construction methods – well-winnowed experience.

The purpose of this report is to draw attention to the key aspects of the design and construction of deep basements and to provide some examples of case histories and construction methods. It is apparent that a wide range of disciplines is involved and it would have been unrealistic to treat each in any depth. Whilst this document tends to reference UK practice and legislature, much of the guidance is generic and applicable internationally. It is hoped that the references will be helpful in enabling those who wish to do so to go further. Perhaps the overall message must be that, in a subject as complex and wide-ranging as this, there can be no short cut to an in-depth understanding of the many specialist aspects. But an overall conceptual understanding can and must be developed. It is hoped that this report will assist in achieving this.

The report has taken a long time to prepare for which I am largely responsible. A large number of people have put an immense amount of effort into the report. In particular I am indebted to Brian Bell and Malcolm Puller who have so willingly pulled the report together and brought it to life. I am also very grateful to the staff of the Institution whose patience I have sorely tried.



John Burland
Task Group Chairman

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1 Design considerations

1.1 Introduction

Two basic considerations must be borne in mind in the design of a deep basement or some other cut-and-cover construction such as a tunnel. First, the designer has to take into account that an excavation cannot be made without causing movement of the surrounding ground (see Chapter 2) and, second, the designer has to consider how the basement will be built (see Chapter 4) taking into account all the implications arising from the construction method envisaged and the professional responsibilities involved. It is essential to determine acceptable maximum ground movements, and these will depend not only on engineering considerations but also on the economics of the particular circumstances. Construction methods must be fully considered and discussed at the design stage, since different construction methods may need totally different design approaches.

In addition to these two technical considerations is the imperative need for design and construction that results in the building or structure being finished within the specified time at the agreed price. No client is interested in an advanced technological solution to the subsurface construction that prolongs the construction period and increases final cost. Generally, simplicity and speed of design and construction are basic requirements.

1.2 Client's general requirements

In the case of basements, once the general uses have been established, whether it is to be garage, warehousing, departmental store, showroom, archive, etc., it is essential to ascertain the client's particular service requirements. It is also necessary to find out whether any use is likely to change. The client's requirements will largely dictate the internal arrangements and floor construction, and will also affect the sub-divisions necessary to reduce the risk of spread of fire or to isolate areas of high fire risk.

Once the basement storeys have been designed and detailed, even minor changes in layout or use can result in extensive redesign and redetailing, and therefore precise client requirements are needed. The client should be advised at concept stage to consider the implications that could arise if changes are necessary after the work is completed. Changes in layout and use could present serious problems in a completed building where the location of fire lifts, staircases, ventilation ducts, smoke outlets, etc., satisfactory for one particular use and layout, would be totally inadequate for another, and where the cost of adaptation could be prohibitive.

Facilities for fire fighting and the provision of adequate ventilation and smoke outlets can present special problems, particularly when the building does not adjoin public roads or an immediate open space accessible on at least three sides. The local building control and fire authorities should be consulted to ensure that acceptable arrangements would be achieved for securing adequate ventilation, the provision of fire-fighting lobby-approach staircases, fire-lifts, fire appliances including falling mains, the effective rapid removal of smoke and means of access for fire-brigade appliances.

Appendix B gives the special services requirements for deep basements.

1.3 The ground

The construction of a deep basement or cut-and-cover structure is a problem in soil-structure interaction. Whereas, for building foundations, the engineer can often bypass the problem by adopting rigid piled foundations, this is seldom possible with deep basements since the retaining structures and foundations are far too intimately linked to the surrounding ground. Before becoming enmeshed in detailed analysis and design, the engineer must have a thorough grasp of the following:

- The effective stress principle
- The ground profile beneath and around the proposed excavation
- The groundwater conditions
- A picture of the short- and long-term global movements.

1.3.1 The concept of effective stress

The cornerstone of modern soil mechanics is Terzaghi's principle of effective stress. If the mechanical behaviour of the ground beneath and around a deep excavation is to be understood, even in general terms, knowledge of the effective stress principle is essential.

An element of saturated soil consists of discrete solid particles of various sizes and shapes, which are in mechanical contact with each other and form a solid 'skeleton' or 'structure'. The voids within this 'structure' are filled with water. The strength of the solid particles is generally large and deformation or failure of an element of soil results mainly from slip at grain contact points rather than crushing of the grains themselves. It is important never to forget that soil is a particulate material and its behaviour results from this. Nevertheless, the grains of the soil are so small in comparison with the samples we take, or the structural elements we place in contact with it, that we treat it as a continuum in the same way that we treat concrete and steel as continua even though they are made up of discrete crystals.

The effective stress principle

The effective stress principle can best be introduced by first defining what is meant by effective stress and then stating the principle:

Any plane through an element of soil has acting on it a resultant normal stress σ and a shear stress τ. In addition, the water in the pores will be under a pressure, u, known as the porewater pressure. By definition, the effective normal pressure σ' acting across the plane is the difference between the resultant or total normal pressure and the porewater pressure. Thus:

$$\sigma' = \sigma - u$$

Since water cannot carry shear, a shear stress τ will always be an effective stress.

i.e. $\tau = \tau'$

An effective stress may be thought of as that part of the total stress that is transmitted through the soil skeleton. However, it is misleading to call it the 'intergranular' stress since this refers to the complex state of stress at grain contact points.

The effective stress principle states that all measurable effects of a change in stress (such as compression, distortion or a change in shearing resistance) are due exclusively to changes in effective stress. The validity of this principle for saturated soils has been demonstrated experimentally many times.

Soil properties

The one-dimensional compressibility of a soil, m_v, is expressed in terms of changes in vertical effective stress as follows:

$$\Delta V/V_o = m_v \Delta\sigma'_v$$

where $\Delta V/V_o$ is the volumetric strain, which, because the compression is one-dimensional, is equal to the vertical strain $\Delta\varepsilon_v$.

Similarly, the rigidity of a soil G' is expressed in terms of changes in shear stress as follows:

$$\Delta\gamma = \Delta\tau/G'$$

where $\Delta\gamma$ is the shear strain.

The strength of a soil in terms of effective stress is defined by Coulomb's equation:

$$\tau_f = c' + \sigma' \tan\varphi'$$

where τ_f is the shear strength, c' is the effective cohesion, and φ' is the effective angle of shearing resistance of the soil. Both of these parameters refer to the soil in its undisturbed state of stress and stress history and should be determined for the range of stresses applicable to the particular problem.

It can be seen that soil properties that relate to effective stresses are denoted by a prime (′), as are effective stresses.

The importance of effective stress

It is clear from the above that an increase in effective stress (σ') results in compression of the soil and an increase in strength. This increase in effective stress could result either from an increase in the total stress (σ) with the porewater pressure (u) remaining constant, or it could result from a reduction of the porewater pressure with the total stress remaining constant - the result is the same.

Similarly, a decrease in effective stress results in swelling of the soil and a reduction in strength. Again, this decrease in effective stress could result from either a decrease in the total stress with the porewater pressure remaining constant or it could result from an increase in the porewater pressure with the total stress remaining constant.

It is evident from the above that, 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. Porewater pressures in the ground can change because of seepage, groundwater-table fluctuations, increases of applied total stress (resulting in consolidation) and decreases of applied total stress (resulting in swelling). All these effects will change the effective stress and result in important, sometimes catastrophic, soil behaviour. Any process that results in a decrease in effective stress is potentially dangerous, since it results in swelling and reduction in strength.

Undrained strength

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

1.3.2 Ground profile

The ground profile can be established by careful visual and tactile examination and description of the vertical succession at a number of locations within and outside the proposed excavation. This can be done by examining continuous cores or, in some cases, sinking a trial shaft (see Figure 1.1 and Section 8.3). In most cases, the key design decisions can be made on the basis of this information. (The results from detailed laboratory and in-situ tests enter the picture when detailed analysis and design are carried out.) Conversely, lack of knowledge about the ground profile and groundwater conditions are the most important causes of failure and delay. Quite minor stratigraphic features can be significant. For example, for the underground car park at the Palace of Westminster, the depth of the retaining walls and bored pile foundations were determined by the finding that a layer of clay containing sand and silt partings (see Figure 1.2) was present just below the excavation level[1.1].

The code of practice for Site Investigation[1.2] gives helpful advice on the examination and description of soils. A simple and systematic method of soil description is given in reference 1.3. The engineer responsible for the design of a deep excavation should always examine a number of continuous cores in person, if necessary with the help of an experienced geotechnical engineer, so as to be completely familiar with the ground profile and what it looks like physically.

Not only should the completed profile of backfill over the basement or tunnel be considered but also the temporary. It is rare for backfill to be placed and compacted in the uniform layers so beloved by specifiers. Indeed, in one instance, the contractor needed to backfill 8m over his tunnel to restore site access at ground level across the severed site, in the form of a localised embankment. As a result, the base of the tunnel required many times more longitudinal reinforcement locally than would be required for a wished-in-place structure; some longitudinal steel was also needed in the tunnel roof.

Fig 1.1 Ground profile exploration being carried out for the foundations and deep basement of the YMCA, London in the mid-seventies. Present requirements for the descent of augered shafts are covered by BS 8008:1996.

Fig 1.2 Silt and sand partings

1.3.3 Groundwater conditions

An appreciation of groundwater conditions is as important to the design and construction of deep excavations as is knowledge of the ground conditions: indeed the two go hand in hand. Many serious problems result from an insufficient understanding of the groundwater regime and the effects of excavation. Great care is needed in interpreting groundwater conditions from borehole logs. It is essential that groundwater and its control get explicit treatment from the very beginning of the conceptual design[1.4,1.5]. The Site Investigation should include studies aimed at clarifying and quantifying the groundwater conditions and consideration should always be given to carrying out pumping trials, since an adequate evaluation of mass permeability is often difficult to achieve using conventional boreholes and testing. Chapter 3 describes the importance of developing a strategy for understanding the groundwater regime. It summarises the various methods of groundwater control and discusses the problem of changes in regional groundwater levels (see Section 3.4).

1.3.4 Global movements

During the development of the conceptual design and construction method, the engineer should attempt to sketch the likely pattern and form of short- and long-term ground movements. This helps to understand in a qualitative manner the factors controlling ground movements and the mechanisms of movement. The process is analogous to sketching the deflected shape of a structure as an aid to understanding 'how it works'.

When ground is excavated, horizontal stresses are removed from the sides and vertical stresses are removed from the base. It is important to distinguish between the effects of horizontal and vertical stress relief.

Figure 1.3 illustrates the movements that result from the relief of horizontal stress in front of an embedded cantilever retaining wall (a), and a propped cantilever wall (b). It can be seen that, although vertical movements will be about the same in both cases (and depend on the type of ground), horizontal displacements will differ appreciably, being much greater in the case of the simple cantilever.

For excavations in deep layers of clay, relief of vertical stress from the base of the excavation can give rise to important displacements, which are often overlooked. The mechanisms of behaviour are perhaps best explained by considering first the familiar case of a downward vertical stress on the surface of a clay soil as shown in Figure 1.4a.

In the short term, no drainage occurs, and distortion takes place at constant volume. The ground beneath the loaded area settles and outside it rises. The shape of the deformed profile of the ground surface depends critically on the depth of the clay

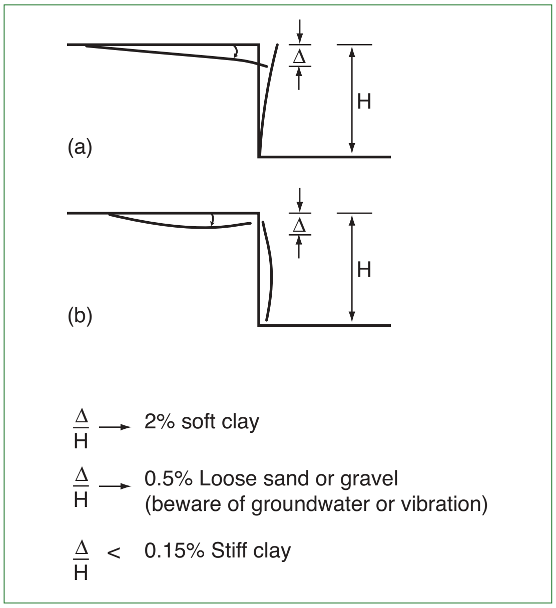

Fig 1.3 Wall movement due to horizontal stress relief

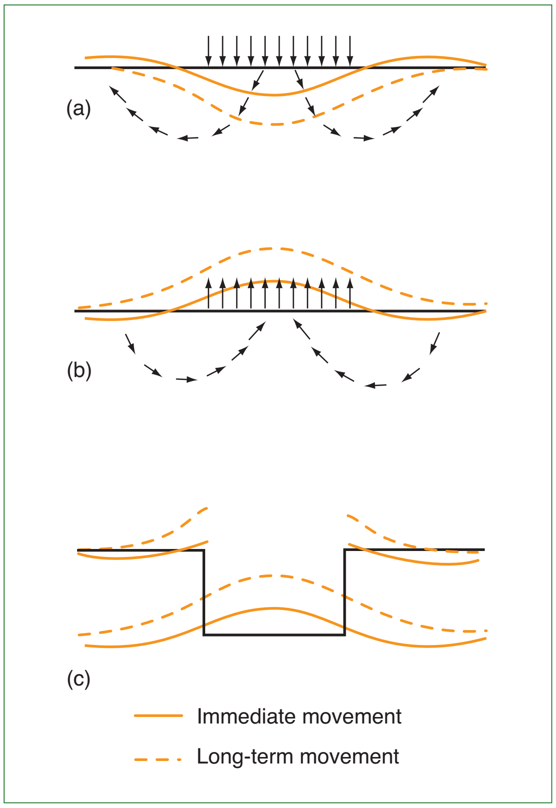

Fig 1.4 Vertical displacements

stratum and the variation of stiffness with depth. In the long term, drainage occurs, settlement continues to increase beneath the loaded area and spreads outside it as shown by the broken line.

In Figure 1.4b, instead of downward pressure, an upward stress is applied to the soil surface. The behaviour will be the reverse of case (a). In the short term, heave will occur under the stress-relieved area, with settlement outside it. In the long term, the ground will continue to heave under the stress-relieved area and this will spread to the surrounding ground. Case (b) is analogous to vertical stress relief at the base of an excavation as shown in Figure 1.4c. Even if the sides of the excavation are restrained from deflecting, settlements can occur in the surrounding ground in the short term, with uplift in the base. In the long term, the settlements of the ground around the excavation will tend to reduce and heave may even occur if there is net long-term relief of stress at the base of the excavation.

Vertical stress relief at the base of an excavation can also give rise to important horizontal ground movements. Following the same argument as before, Figure 1.5a shows the deep-seated outward displacements beneath the edges of a loaded area. If, instead, an upward stress is applied, deep-seated inward displacements occur, as shown in Figure 1.5b/1.5c. Hence, vertical stress relief at the base of an excavation in a deep clay layer will induce deep-seated inward displacements that cannot be significantly reduced even by installing successive levels of internal props as excavation proceeds.

Case records are the best guides to the magnitude of the ground movements and references to a number of these are given in Chapter 2. In recent years, considerable advances have been made in analytical methods of predicting ground movements around deep excavations and these are also discussed in Chapter 2. These methods are particularly powerful when they can be calibrated from field measurements around deep excavations in similar strata.

Progress in recent years in the design and construction of deep excavations in urban environments has been due largely to careful monitoring and analysis of movements beneath and around them. Published case records are an invaluable source of information and experience. Back-analysis can also be carried out to assist understanding.

Every opportunity should be taken to design and install a monitoring system; guidance is given in Appendix D. Monitoring is a valuable means of control when restrictions have to be imposed to limit possible damage and where it is important to check the validity of design assumptions. This is an important ingredient in the use of the Observational Method (see Appendix E).

The Observational Method can in turn be an important ingredient in the adoption of Value Engineered (Value Managed in USA) solutions to construction problems, where a holistic approach is taken. For Value Engineered solutions to be proposed requires a big carrot: contract conditions should be adopted which create a favourable climate for suggestions to be made, evaluated and embraced by all. No contractor will want to see his work content and associated profit reduced by an instructed change in method, from his well-intentioned initiative, resulting merely in reduced quantities.

1.4 Methods and types of construction

There is a variety of methods of excavating deep basements and cut-and-cover structures depending on a number of factors such as the required degree of control of lateral movements and settlements, the size and depth of excavation and the form of structure contained within the basement. Chapter 4 describes the five most common types of construction method

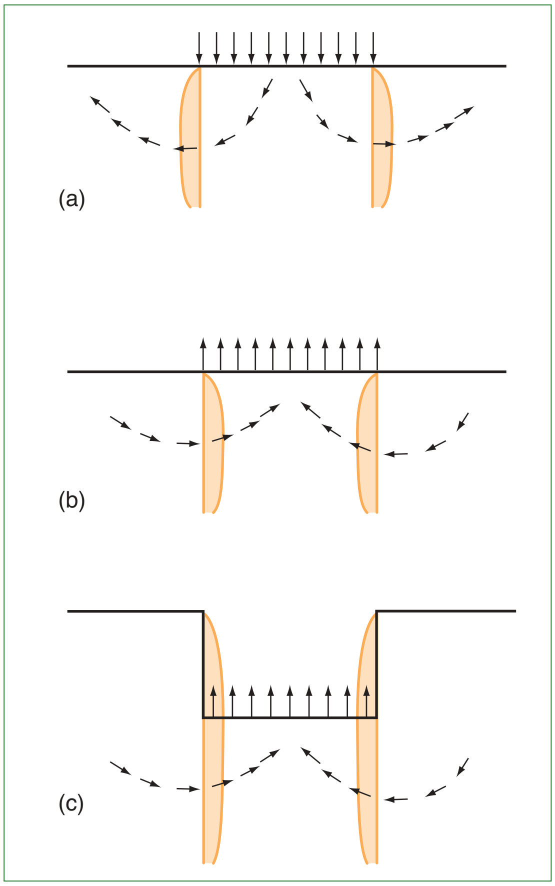

Fig 1.5 Lateral displacements

Fig 1.6 Airport Express Tunnel, Hong Kong
© Aoki Corporation

together with the range of types of retaining walls and support systems. Chapter 9 deals briefly with the important practical considerations of carrying out the excavation and removing the spoil from the site.

The timing of construction activities is also vital to the choice of construction method. Where, for example, an excavation is driven through reclamation (see Figure 1.6) special consideration such as hydraulic surcharge may be used to accelerate settlement ahead of construction. Similarly, recognition is needed of the possible differential settlements that can result from uneven backfilling above a cut-and-cover structure as in the case of the access embankment shown in Figure 1.6.

1.5 Retaining walls

Propping forces generated by the retaining walls must have a clear and simple route through the sub-structure, with any out-of-balance earth pressures resulting from sloping ground (say) catered for.

The retaining walls of a deep basement or cut-and-cover excavation interact with the surrounding ground in a much more complex manner than do simple traditional cantilever walls retaining fill. For a traditional retaining wall, it is usual to design for fully active earth pressures because only small movements are required for them to develop. However, for deep retained excavations, particularly where multi-propped in-situ retaining walls are used, this approach may seriously underestimate the earth pressures. In these circumstances, it is better to start with the initial in-situ 'at rest' earth pressures and assess how much they might be reduced by the installation method, the excavation and construction procedures and the rigidity of the supports together with many other factors. It is thus important to gain a clear understanding of the many factors controlling ground movements and earth pressures. These questions are discussed in Chapter 5 and techniques

Fig 1.7 Broomfield Bank WWTW, Dover: buried in hillside © AMEC

for assessing the in-situ ‘at rest’ lateral pressures are summarised in Chapter 8.

A retaining wall system required to balance unequal forces is exemplified by a waste water treatment works buried in a chalk hillside with 0.5m of soil on the roof (see Figures 1.7 and 1.8), where a clear understanding of the load path was essential.

1.6 Foundations

The design and analysis of foundations are discussed in Chapter 6. The foundations of deep excavations not only have to carry the vertical and horizontal loads imposed by the superstructure and retaining walls but also have to accommodate the short and long-term movements resulting from the excavation process. Of particular concern are the effects of swelling and the associated strength reduction caused by the relief of vertical stress and seepage into the excavation. In assessing the stability of the retaining walls, these effects must be taken into account, as they also must in the design of spread footings and rafts.

Piled foundations are often used to carry loads down below the swelling zone or as anchors to reduce heave. Such piles may be subjected to large tensile forces induced by the heave unless special relief measures are adopted. Their design requires an effective stress approach, since traditional empirical methods based on factoring the undrained strength are inappropriate.

Unless special relief measures are adopted, a key design decision is whether to use a ground-bearing slab at the base of the excavation or a suspended floor with a void beneath it to permit unrestrained heave. A ground-bearing slab must be designed to accommodate the heave and associated swelling pressures that develop beneath it, as well as any horizontal forces from propping the retaining walls. Provision must also be made for either resisting hydraulic pressures or dissipating them by a suitable under-drainage system.

A suspended floor must be adequately drained and vented so that gas cannot accumulate (see Chapter 7).

1.7 Ground gases

The question of the accumulation of gases generated within or passing through the ground is so important that a short chapter is devoted entirely to this topic (see Chapter 7).

1.8 Site Investigation

A discussion of Site Investigations for deep excavations has been deliberately left until Chapter 8 after the special design and construction requirements have been covered in the earlier chapters. It must be emphasised strongly that an experienced geotechnical engineer intimately associated with the design should always control the planning and execution of such Site Investigations. The importance of a detailed description of the ground profile and knowledge of the groundwater conditions has already been emphasised. The most relevant quantitative geotechnical parameters are the undrained strength, the drained strength parameters measured over the

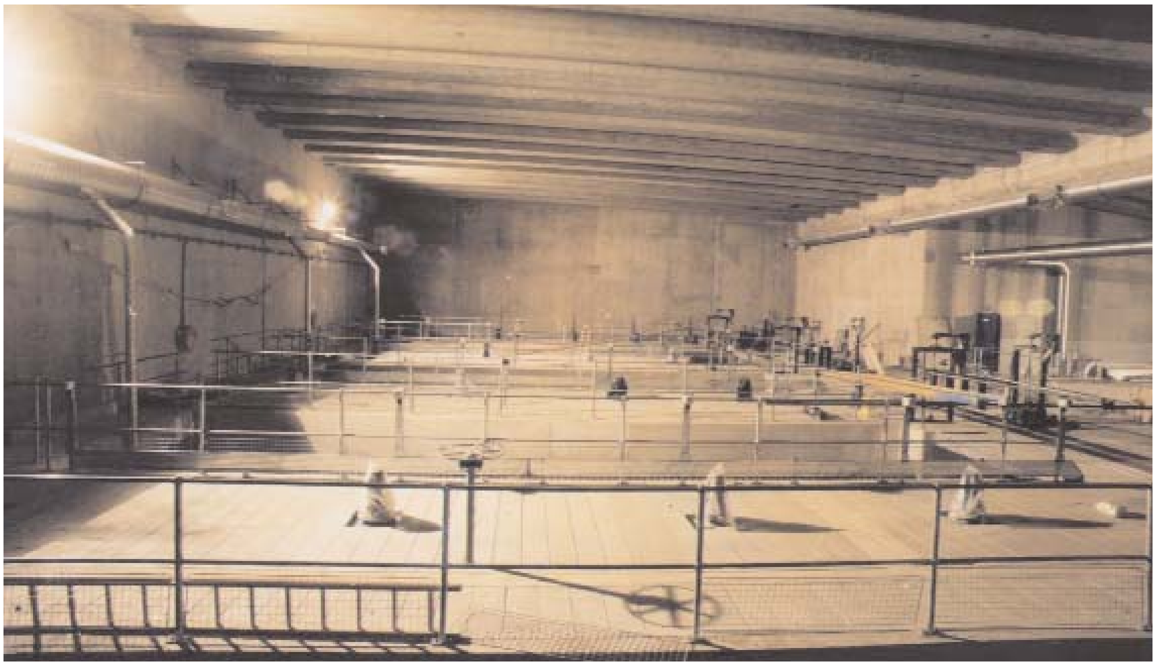

Fig 1.8 Roof carrying 500mm of soil and light vegetation © AMEC

appropriate stress range, the stiffness of the ground, particularly at small strains, the assessment of in-situ 'at rest' pressures and the determination of the in-situ permeability of key soil strata.

1.9 Structural analysis

Structural analysis is only a part of the design process. Undue emphasis may be attached to analysis because it appears scientific and provides numerical answers. However, achieving coherence in the design concept, particularly in foundation engineering, is an art dependent on engineering judgement based on experience[1.6]. Analysis must be viewed in the context of the many idealisations that have to be made: no analytical model or the input to it will ever fully represent reality.

Design must address the complexities of soil-structure interaction by providing for the range of possible actions and allowing for uncertainties, particularly in construction[1.7]. Indeed, the construction method, quality of workmanship, and the ground and groundwater conditions encountered all bear on the validity of the design assumptions.

It is important to get the basic concepts right in the early stages of design. The soundness of these concepts must then be reviewed as the detailed information of the site, ground conditions and construction methods become available. The methods of analysis may range from simple hand calculations to complex computer-based techniques using finite-element or finite-difference methods[1.8-1.10]. 'Wished-in-place' structural analysis is no longer defensible, most codes of practice make it clear that the designer of permanent works is required to take into account the temporary stages through which the permanent works pass. An example of what can be achieved to create an open structure, with good visual contact between its users at upper and lower levels, is given by Figure 1.9.

Computers are useful in undertaking comparative evaluation and in testing the sensitivity of the soil-structure interaction to variations in assumed soil parameters. For example, variations in the distribution of stiffness or strength with depth can significantly affect deformed shapes and bending moment distributions down an embedded wall. This pronounced sensitivity is not the norm in the analysis of engineering structures generally, where small changes in strength factors are often secondary or even insignificant. It highlights the importance of parametric studies, in conjunction with specialist advice in the choice of geotechnical parameters.

Fig 1.9 Boon Keng Station, Singapore: double storey height free of struts © Benaim

Caution should also be exercised in exploiting the analytical power of computers to refine designs[1.11-1.13]. Conditions of failure are notoriously difficult to model, but soil in a state of local, or confined, failure is the norm for soil-structure interaction problems. It is appropriate, therefore, that the design should be broadly verifiable by basic hand calculations using simplified assumptions. Terzaghi often said that any theory that was not simple was of little use in soil mechanics[1.6]. Golder[1.14] stated that a design relying for its success on a refined calculation was a bad design!

1.10 Protective measures

Because of the uncertainties in the precise magnitude of ground movements that are inherent in carrying out deep excavation work adjacent to existing structures, the need for protective measures must always be carefully considered. In Chapter 10 a variety of protective measures is described which includes measures that are internal to the excavation itself, underpinning measures and various ground treatment methods including the relatively new method of compensation grouting that was used successfully on the Jubilee Line Extension. Most of these measures require reliable monitoring for their successful implementation and this important topic is discussed in Appendix D.

1.11 Durability and waterproofing

Chapter 11 deals with the important and controversial subjects of workmanship, durability and waterproofing. The degree of acceptable waterproofing varies according to the proposed use of the basement or excavation. Major factors to be considered are normally appearance and finishes, materials to be stored, mechanical and electrical equipment present and, above all, possible long-term structural damage. For the purposes of this report, and following CIRIA Report 139[1.15], the requirements for waterproofing may be broadly placed into four performance grades; Basic Utility, Utility, Habitable and Special. The client must be made aware of the differences between these four grades, and the cost implications, and decide which grades are required at the various locations within the basement.

It is now normal practice to incorporate as large a proportion of the temporary support structures as possible into the permanent works. In practical terms, this means that advantage is taken to construct slender in-situ retaining walls to the boundary limits and support them at frequent intervals; there is often no connection between the vertical and the horizontal permanent structure at the boundary until much of the other structure has been completed; for example, a berm may be left to support the toe of the wall. The practicality of constructing a continuous and effective water-excluding membrane is thus severely impaired, and the logical design is one in which vapour transmission, and perhaps some minor seepage, is accepted and dealt with by way of ventilated cavity walls and underslab drainage.

Even where the retaining walls are constructed in open cut, applying external membranes is not necessarily the most appropriate approach to waterproofing. It is more important to ensure good-quality low-permeability reinforced concrete, with adequately controlled early-age thermal cracks that are allowed to seal autogenously. Much useful advice on how to achieve this is given in Chapter 11.

Parts of a basement given over to archives, etc. requiring an especially high degree of water exclusion (Special grade) can generally be successfully dealt with by encapsulating them within separate protection. Examples are given in Chapter 11 of standard details for achieving appropriate grades of waterproofing.

1.12 Safety

The industry record shows clearly that construction is a dangerous activity. In recent years, the whole atmosphere and attitude towards safety on construction sites have changed and there will be significant changes in the future. The Construction (Design and Management) Regulations 1994[1.16] imposes broad and wide-ranging duties on all parties with the general purpose of securing the health and safety of persons at work. Chapter 12 highlights those aspects of health and safety that relate particularly to deep excavations. Ultimately, safety depends on the will and determination at all levels of management to instil the attitude that safety is everybody's responsibility.

1.13 Legal and contractual issues

Chapter 13 deals with legal and contractual issues. Deep basements and cut-and-cover constructions present special problems, which may not be adequately covered in standard forms of contract and which should be addressed in the contract documents. Additional conditions may be advisable, for example, to cover such matters as waterproof construction, damage to the basement structure due to heave or ground movement, increased cost arising out of disused services, etc. When specialist contractors are employed, consideration must be given to a clear demarcation of design liability and allocation of other responsibilities between contractor, specialist contractor, the professional team and the client.

An important issue is responsibility for Site Investigation. In practice, the engineer should carry out such Site Investigations as are appropriate to enable adequate information to be obtained for the design of the structure, having regard also to the temporary works and methods of construction. The information should be in a report available to tendering contractors.

Another issue of particular importance is that of responsibility for temporary works. These range from straightforward items, such as falsework and shoring, to complex operations involving temporary use of permanent works where responsibility for design and use needs to be clearly defined.

1.14 Communications

Because of the interaction between design assumptions and the order and methods of construction, communications between the designer and the site are particularly important. Chapter 14 lists the key information that should be supplied by the engineer and the contractor both at the tender stage and during construction. One tool very effective in communicating the designer's and constructor's assumptions to each other is the Precedence Network, which lists the activities which must be completed before another activity may start: an example is given in Chapter 14.

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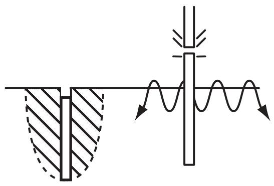
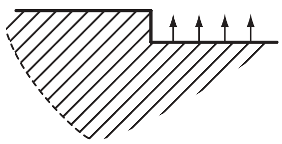
2 Ground movement

Movements are caused by various mechanisms:

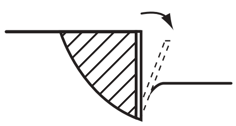
- Ground 'disturbance' during installation of in-situ walls such as that due to vibration, loss of ground or heave.



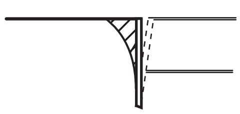
- Movement due to vertical loading or unloading of the ground below the basement.



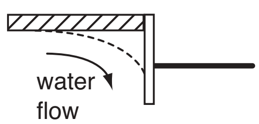
- Movement resulting from reduction of lateral pressure from the inner face of the retaining structure, due to bulk excavation or the installation of large bored piles within the excavation.



- Movement in the props supporting a wall (e.g. because of temperature changes, shrinkage or loss of support).



- Movement due to changes in groundwater conditions.



- Movement due to poor workmanship, such as over-excavation, loose props and badly installed walls.

Fig 2.1 Triggers of ground movement

2.1 Introduction

The ground will always move when a basement is constructed. This can cause damage to surrounding structures, roads and services depending on their sensitivity and the magnitude and distribution of the movement. Detailed observations of ground movement are often made[2.1-2.5]. However, the amount and extent of movement can be controlled by choice of method of construction and by maintaining an adequate standard of workmanship. The choice of method of construction depends on what might be affected by ground movement, e.g. buildings inside or outside the basement, services, tunnels, etc., and their tolerance to movement (see Figure 2.1)

The calculation of ground movements is not straightforward because of the complexity of the problem, and much experience is required to make any sensible use of complex analyses when they are warranted. Optimum use must therefore be made of precedent.

2.2 Causes of movement when excavating

2.2.1 General

Each soil (or rock) type has its own problems, and regional experience is of paramount importance. Methods of coping with these problems economically and safely have evolved and it is unwise to step outside the bounds of experience without full justification and careful observation of performance.

Some of the more frequently encountered soil conditions are listed in the following sections along with an outline of the associated problems.

2.2.2 Clays

When constructing a basement in a low permeability material (clay), movements will be time-dependent. Initially, the clay will respond in an undrained fashion with no volume change. The movements will be essentially the result of shear distortion and will be accompanied by a change in porewater pressure. As time goes by, the clay will begin to drain, causing a general volumetric expansion when the clay has been unloaded, or compression when it has been loaded. Eventually, when these porewater pressures have dissipated, the clay will have reached a fully drained state and movement will cease apart from possible small creep movements.

During drainage, the strength of the clay changes. This is because, in the case of expansion, water is drawn into the clay, softening it and reducing its strength. Thus, for example, in front of a wall in stiff clay following excavation, the clay will gradually expand and soften following the relief of the overburden pressure. The consequent loss of resistance may dominate the wall during this 'drainage' stage, especially with cantilever walls. The relative magnitudes of undrained and drained movements, and the rate at which the latter develop, depend on the nature of the clay and can be significantly affected by the presence of high-permeability layers within the soil. However, because of the time taken to construct a basement, it is inevitable that some drainage will occur. Assessing the importance of these issues is a matter of a detailed knowledge of the soil profile, experience and study of case histories.

Soft clays

Special problems arise when excavation takes place within soft clays, because the reduction of vertical pressure inside the excavation decreases the ability of the soil below the level of excavation to sustain the vertical pressure applied by the soil outside, i.e. an undrained bearing capacity failure can take place (see Figure 2.2a).

In soft clay, the depth to which excavation can proceed before such failure starts may be small, and so large ground movements may develop. This will generally start when the base stability number, $N = \gamma H/c_u$, exceeds around 3-4 (for detailed guidance see reference 2.6). Uncontrolled deformation is likely for N = 6 - 7 (see Figure 2.2b).

Since this movement occurs below the level of excavation, horizontal props alone cannot eliminate it. It has to be controlled by ensuring that:

- the wall itself prevents movement by being sufficiently stiff,
- the wall is adequately embedded below the deforming zone (by keying the wall into a stronger stratum) (see Figure 2.2c), or
- in-situ props are cast below excavation level using diaphragm walling techniques, jet grouting, or tunnel struts (see Section 4.5.1).

The effects of 'yield' of ground beneath an excavation are not restricted to excavation in soft clays. They can also occur in any soil if excavation is deep enough, or more commonly if base failure occurs because of uplift pressures from water in permeable strata beneath clay layers (see Section 2.2.4).

Driving sheet piles may cause large ground vibrations at considerable distances from the excavation (see Figure 2.2d).

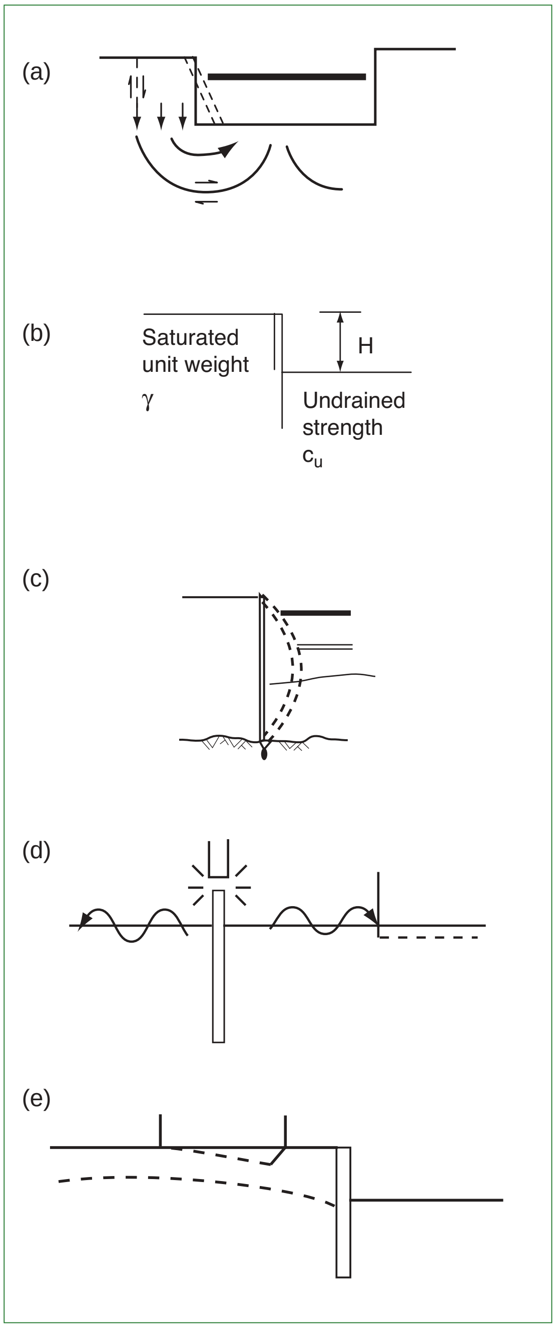


Fig 2.2 Movements in soft clay

Large surface settlements may occur outside the excavation owing to consolidation after excavation because of changes in groundwater conditions (see Figure 2.2e).

Stiff clays

Stiff clays are generally good materials in which, to work provided that the effects of drainage are limited by the application of support or loads. Particular problems related to this area include:

- *Movement of unsupported (cantilever) walls due to drainage of soil in front of the wall.* This can occur rapidly if the ground is not protected from

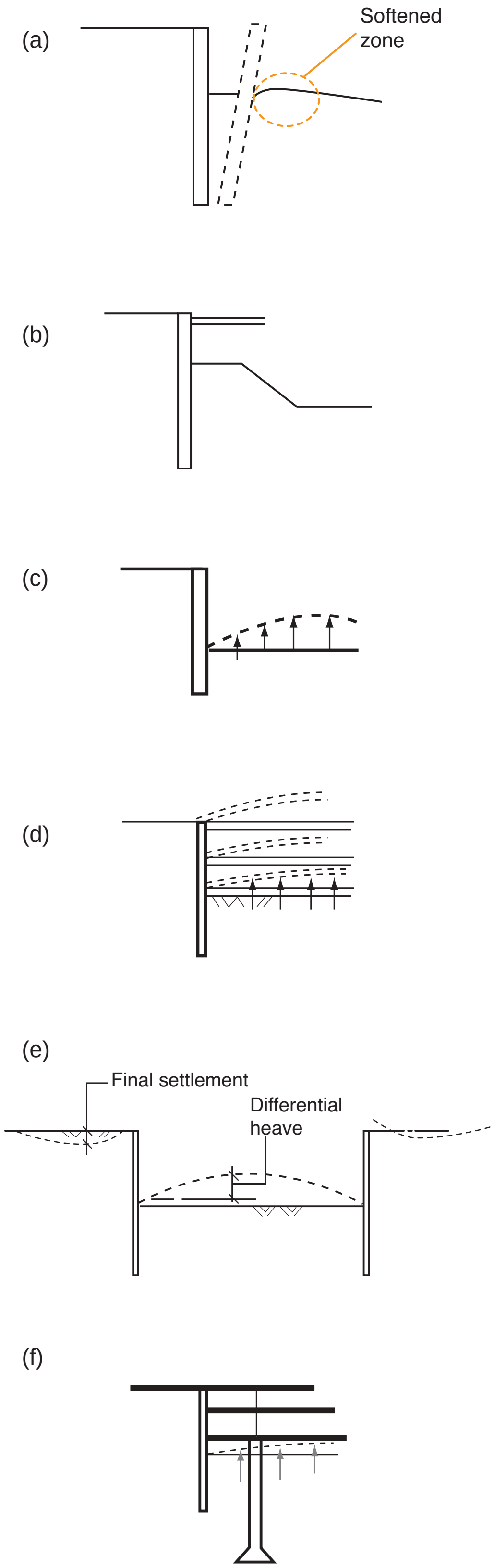


Fig 2.3 Movements in stiff clay

water ingress. Because of drainage, the resistance in front of the wall may drop significantly, increasing forward movement; this must be expected (see Figure 2.3a).

- *Movement of the toe of propped walls during construction.* In the right circumstances, the clay in front of the toe of a retaining wall can drain rapidly. To limit the effects of this, a sequence of construction must be chosen so that the toe area is not left exposed for long unless additional support is provided. One common method is to leave soil berms around the periphery of an excavation, to be later removed and replaced with permanent support (see Figure 2.3b).
- *Upward movement of the ground within the excavation.* If left unloaded, the clay under an excavation may expand causing structures supported on it to lift (see Figure 2.3c/2.3d).

Stiff clays may possess high locked-in lateral stresses. Thus, the process of excavation releases large stresses, building up large support loads. Adopting a ‘soft’ support system, e.g. flexible props and flexible walls, may reduce the loads and stresses in the structural elements with a consequent increase in movements outside the excavation.

Since stiff clays have sufficient bearing capacity to carry building loads, it is possible to found on rafts within basements. However, the long-term drainage of the clay may lead to large vertical ground movements if there is a significant net pressure increase or decrease on the soil surface (see Chapter 6). The sides of an excavation will, to an extent, be restrained by the ground outside and the retaining walls, and long-term movements can result in significant differential vertical movement across a basement and/or differential movement between the structure and the surrounding ground. This problem can be alleviated by isolating the structure from the soil, i.e. supporting it on piles with a heave gap (see Figure 2.3e/2.3f).

2.2.3 Granular soils

The process of basement construction in high-permeability soils, e.g. sands, will result in an almost instantaneous response to changes in loads and groundwater conditions, i.e. fully drained conditions develop very rapidly.

The problems associated with granular soils are principally concerned with the control of groundwater to avoid loss of ground due to high hydraulic gradients and movements during the installation of walls, and include:

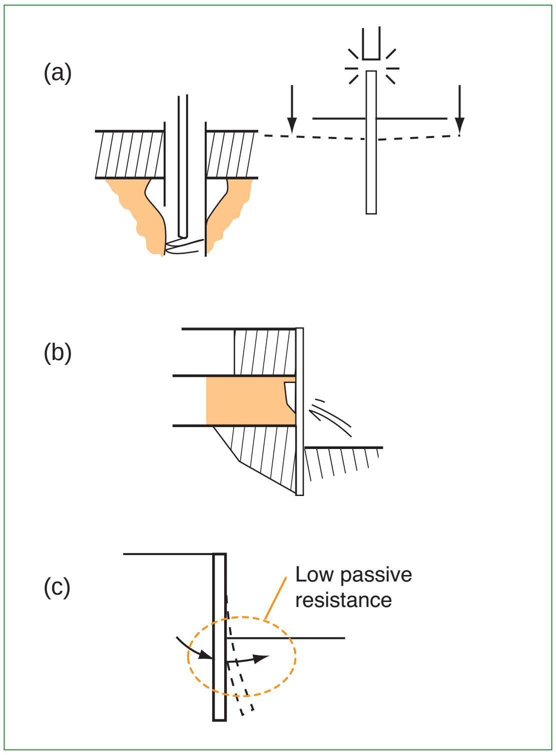


Fig 2.4 Failure in granular soil

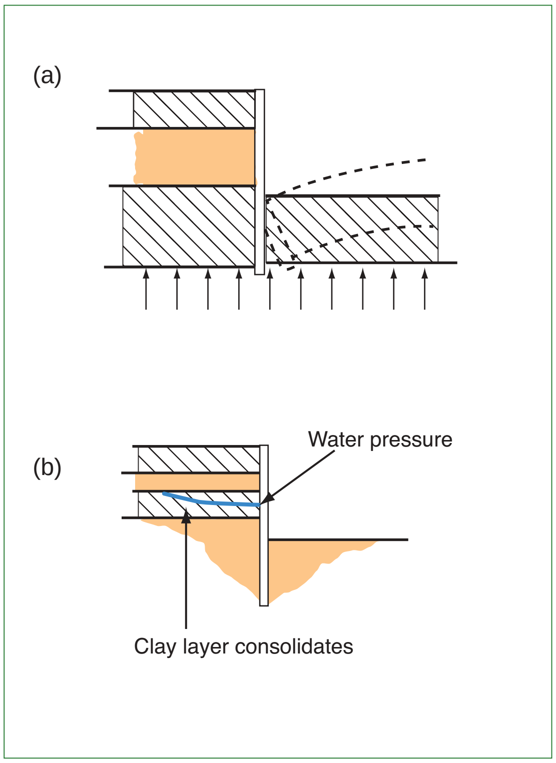


Fig 2.5 Failure in alluvial soil

- Settlement occurring around a retaining wall during its installation by the compaction of loose sands/silts owing to vibration or loss of ground during drilling[2.7] (see Figure 2.4a).
- Difficulties with preventing water seeping through a wall during excavation giving rise to a local lowering of water table outside the excavation and loss of fines through the wall, causing eventual settlement (see Figure 2.4b).
- Insufficient penetration of the wall or insufficient dewatering within the excavation leading to high hydraulic gradients, piping of the basement floor or large-scale heave. Seepage flows also reduce the passive pressure restraining the toe of the wall and if significant can cause forward movement of the toe or, in extreme cases, wall failure (see Figure 2.4c).
- Difficulties with preventing water seeping through the wall below the excavation level increasing the upward flow of water into an excavation, so reducing the resistance of the soil in front of the wall.
- Changes in water conditions in loose granular material causing large settlements, particularly if the rise or fall of water levels through them is significant.

2.2.4 Mixed alluvial soils (sands/silts/clays)

In free-draining materials layered with clays and silts, the problems described in the previous section on granular soils apply, but there may be other problems such as:

- Base failure occurring, owing to water pressure below impermeable layers, e.g. clay (see Figure 2.5a), unless this pressure is relieved.
- Difficulties in sealing the wall below excavation level allowing high water pressures to build up beneath clay layers within the basement, causing heave of the basement floor.
- Seepage of water through a wall above excavation level lowering the water table locally outside the excavation, causing soft layers to consolidate and loss of fines through the wall, leading to eventual settlement (see Figure 2.5b).
- Changes in long-term groundwater conditions outside the basement causing large and extensive settlements if a soft clay/silt layer is present.

Most of these conditions involving permeable strata are concerned with groundwater control. Predicting and controlling them are discussed in Chapter 3. Many case histories are presented in reference 2.8. Reliance on passive resistance in soft and loose soils can result in large movements.

2.2.5 Soft rocks

Rock excavations are often open and present different problems to other materials. Their behaviour is dominated by discontinuities, weathering (in some cases causing swelling[2.9]) and the effects of water, e.g. in soluble deposits.

2.2.6 Fill materials

Excavation through fill materials may present problems, particularly when they are variable and contain obstructions that may cause difficulties installing walls. The variability of fill makes it extremely difficult to predict likely ground movements with any accuracy.

The behaviour of backfill material behind a basement wall needs to be carefully considered if it is to contain services or support structures. Clay fills in general, and heavy clay in particular, can settle or heave appreciably depending on the history of the material and method of placement. Heavy clays compacted at low moisture content not only expand upwards but may exert considerable lateral pressures on basement walls[2.10].

Fills should not be relied upon to develop passive resistance.

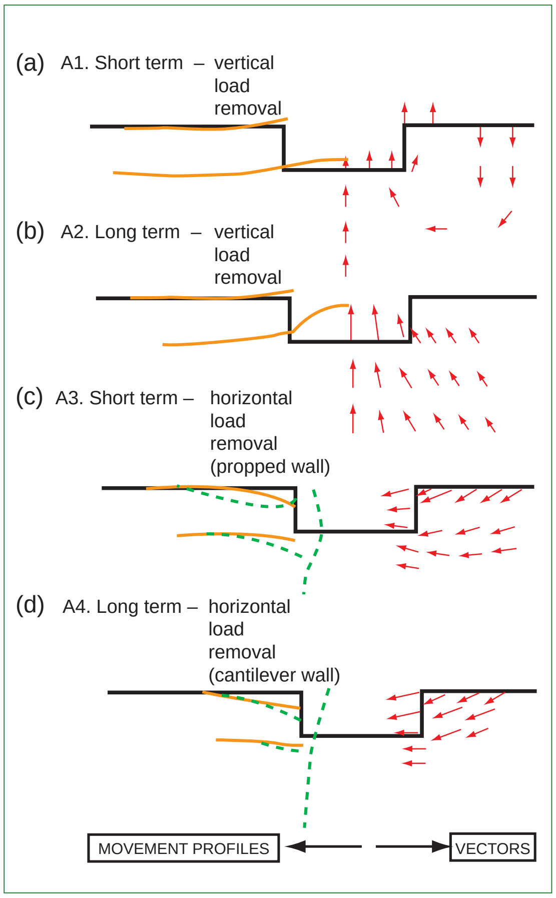


Fig 2.6 Movements around and beneath excavations in clay

2.3 Quantifying movements due to basement construction

2.3.1 General

The previous sections have drawn a general picture of the causes of ground movement and the significance of various factors under various conditions. In the following, the objective is to explain the basic response of ground to basement construction and indicate how movement can be estimated assuming that the support systems are installed without defects, and the walls and base of the excavation are designed with an adequate factor of safety.

Figure 2.6 shows the movements that typically occur around an excavation in clay. In Figures 2.6a and 2.6b, the effects of changes in vertical load alone are shown, assuming the walls are prevented from moving horizontally, whereas in Figures 2.6c and 2.6d the effects of stress changes at the horizontal boundaries are shown assuming no reduction of vertical stress on the excavation floor. The relative magnitudes and direction of movements are given at the levels of the base of the excavation and the ground surface. Undrained and drained movements are shown separately. These assume that the water level is initially at the base of the excavation.

For convenience, further discussion of movements is divided into what happens inside and what happens outside the basement.

2.3.2 Vertical movement within the basement area

Undrained movements are usually calculated by elastic theory. The calculation of drained movements is based on estimating the long-term changes in effective stress in the ground beneath the completed structure. To obtain the changes in effective stress, it is necessary to calculate not only the changes in stress induced by the completed structure but also the initial and final groundwater pressure pattern. Care should be taken to use the net stress changes in such a calculation; for instance, immediately beneath a raft the net stress is given by the contact stress after building completion minus the upward water pressure minus the initial effective stress at that level before excavation began.

In undertaking these calculations, estimates made using laboratory results should be viewed with extreme caution, as they are likely to over-predict the deformations[2.11].

Reference should be made to published field observations[2.12, 2.13]. Deciding whether to assume drained or undrained conditions is discussed in Section 5.2.9.

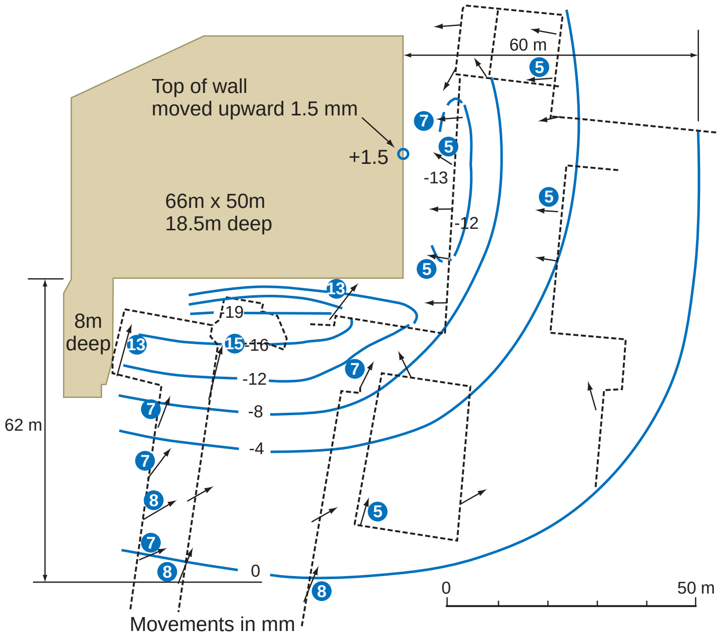


Fig 2.7 Observed vertical and horizontal movements around the Palace of Westminster car park

2.3.3 External movements

In recent years many case histories have been published of the movement of walls and ground outside basements. Peck[2.1] in his 1969 state-of-the-art report on deep excavations and tunnelling presented a comprehensive survey of movements around deep excavations constructed using conventional support methods. This work was updated by O'Rourke in 1981[2.14] and Clough and O'Rourke[2.15]. In 1979, Burland et al.[2.16] summarised the results of over ten

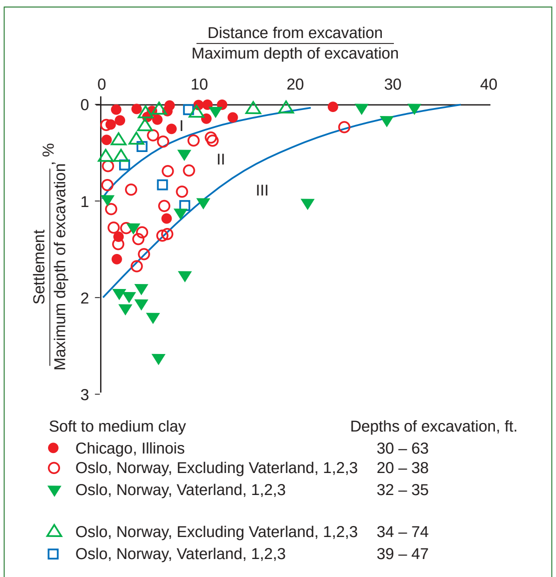


Fig 2.8 Summary of settlements adjacent to open cuts in various soils, as function of distance from edge of excavation (Peck [2.1])

years' research into the behaviour of ground around deep excavations in London Clay. The Norwegian Geotechnical Institute[2.17] has published excellent case histories of excavations in soft clay in the Oslo area, while Wong[2.18] summarises the results of observations around excavations in soft clay in Singapore. Figure 2.7 shows the measured horizontal and vertical ground movements around the underground car park at the Palace of Westminster[2.19].

These observations lead to the conclusions that ground movements depend upon the type of soil and the method of construction. Generally, observations are dominated by settlement measurements; Peck's conclusions are summarised in Figure 2.8. Observations in London show Peck's envelopes are generally conservative for well-supported excavations, settlements rarely exceed 0.15% of the excavation depth, but that movements often extend to three to four times the excavation depth behind the basement wall. Settlement around loose sands or gravels is generally around 0.5% of retained height, and most movement occurs within a distance equivalent to the excavation height, provided that groundwater is satisfactorily dealt with.

Horizontal movements are generally of similar magnitude and distribution to vertical movements, but may be much larger for open excavations in over-consolidated (stiff) clays. Figure 2.9, and its accompanying Tables 2.1 and 2.2, summarise the results of many observations of horizontal movements of retaining walls in UK stiff clays. Maximum horizontal movement at any location on the wall is plotted against maximum excavation depth for various support conditions; cantilever walls, anchored walls, propped or strutted walls, and those constructed by top-down methods. With some cantilever walls, a distinction is drawn between short-term (end of excavation) and long-term movements. The magnitude of these movements is affected by many factors including excavation geometry, the nature and extent of the overburden to the clay, the efficacy of support conditions and, to a lesser extent, wall stiffness. The mode of deformation of walls is similar for each type of wall. With this figure, it is possible to estimate the maximum expected horizontal movement of the ground surface by equating it to the maximum wall deflection. The maximum settlement will be of a similar magnitude. Both maxima will occur roughly at a distance behind the wall equal to the depth at which maximum wall displacement occurs.

Table 2.2[2.18] summarises measurements of horizontal wall movements made on excavations in soft clay.

Table 2.1 Case histories of deep excavations in London Clay

Type of wall	Key	Location	Wall	Comments	Reference
Cantilever	C1/C4	Sloane Hotel	DW	Gravel over London clay.	Crofts et al. (1977)[2.20]
	C2	Dunton Green	BP	Wall in Gault clay. Permanent props below excavation level installed after excavation.	Garrett & Barnes (1988)[2.21]
	C3	A329(M) Reading	DW	London clay over Woolwich and Reading beds.	St. John (1976)[2.22]
	C5	Debden	DW		Crofts et al. (1977)[2.20]
	C6	Bell Common	BP	Claygate beds over London clay. Cantilever wall, propped by roof slab with compressible joint.	Tedd et al. (1984)[2.23]
	C7	British Library	BP	(First stage) Gravel over London clay.	Raison (1985)[2.24]
	C8	Chestenham	BP	Lias clay. Cantilever with temporary berm.	Ford et al. (1991)[2.25]
Anchored	A1	Shepherds Bush	DW	Gravel over London clay. 2 levels of anchors.	Littlejohn & MacFarlane (1975)[2.26]
	A2	Neasden Underpass	DW	Gravel over London clay. 3 levels of anchors.	Sills et al. (1977)[2.27]
	A3	Guildhall	DW	Gravel over London clay. 3 levels of anchors.	Littlejohn & MacFarlane (1975)[2.26]
	A4	Vauxhall			Crofts et al. (1977)[2.20]
	A5	Victoria Street	DW	Gravel over London clay. 1 level of anchors.	Hodgson (1975)[2.28]
	A6	Bloomsbury	DW	Gravel over London clay. 1 level of anchors.	Tomlinson et al. (1975)[2.29]
Strutted/propped	S1	Chelsea	DW	Brickearth and gravel over London clay berm and inclined props.	Corbett et al. (1975)[2.30]
	S2	Britannic House	DW	Gravel over London clay. Berm and inclined props.	Burland & Cole (1972)[2.2]
	S3	Charing Cross Road	DW	Gravel over London clay. Temporary steel struts spanning site.	Wood & Perrin (1984)[2.31]
Top-down construction	TD1	YMCA, Tottenham Court Road	DW	Gravel over London clay. Over row of anchors. First slab at 10m depth.	St. John (1976)[2.22]
	TD2	New Palace Yard, Westminster	DW	Gravel over London clay.	Burland & Hancock (1977)[2.19]
	TD3	Aldersgate Street	DW	Gravel over London clay.	Fernie et al. (1991)[2.32]
	TD4	Queen Elizabeth II Conference Centre, Westminster	DW	Gravel over Gault clay.	Burland & Kalra (1986)[2.11]
	TD5	Lion Yard, Cambridge	DW	Gravel over London clay.	Lings et al. (1991)[2.33]
	TD6	Bloomsbury	DW	Gravel over London clay. Circular basement.	Tomlinson et al. (1975)[2.29]

Note:
DW = Diaphragm wall. BP = Bored pile wall.

Table 2.2 Horizontal movements of walls following excavations in soft clay.

Case	Newton Cross	Telecom Building	Vaterland 1 (NG1 1962a)			Vaterland 3 (NG1 1962b)		Rochor Complex	MOE Building			San Francisco (Mana et al.[2.34])	
L (m)	*26.0*	*42.6*	*100.0*	*100.0*	*100.0*	*100.0*	*100.0*	*76.0*	*110.0*	*110.0*	*110.0*	*81.0*	*81.0*
B (m)	*9.0*	*27.0*	*11.0*	*11.0*	*11.0*	*11.6*	*11.6*	*50.0*	*70.0*	*70.0*	*70.0*	*40.5*	*40.5*
H (m)	*2.0*	*4.0*	*4.0*	*6.0*	*8.0*	*7.0*	*9.1*	*6.3*	*3.7*	*5.1*	*6.9*	*4.6*	*10.6*
T (m)	*9.0*	*21.0*	*9.4*	*7.4*	*5.4*	*17.0*	*14.0*	*11.7*	*17.3*	*15.9*	*14.2*	*23.8*	*18.3*
D (m)	9.0	26.0	9.4	7.4	5.4	5.0	2.9	17.7	17.3	15.9	14.2	23.8	18.3
Unit Wt (kN/m^3)	17.0	15.0	16.5	19.5	19.5	17.0	17.0	16.0	15.0	15.0	15.0	17.6	17.6
q (kPa)	0.0	0.0	39.0	39.0	39.0	0.0	0.0	0.0	0.0	0.0	0.0	60.0	60.0
c_U (kPa)	10	25.6	30	30	30	30	30	23.6	15	15	15	58	66
E_U/c_U	150	200	150	150	150	200	200	200	200	200	200	250	250
Δmax (mm)	65	56-84	59-78	103-137	137-190	76	114-140	110-148	140	200	330	20–60	72-150
Δmax/H (%)	3.25	1.4-2.1	1.48-1.95	1.72-2.28	1.71-2.38	1.09	1.25-1.54	1.75-2.35	3.78	3.92	4.85	0.43-1.30	0.69-1.44
N_C	5.45	5.3	5.75	6.25	7.22	6.28	6.69	9.20	8.86	9.01	9.67	5.23	6.36
S_L	0.62	0.44	0.68	0.85	0.90	0.63	0.77	0.46	0.43	0.57	0.63	0.46	0.58
E_s (kPa)	845.07	3535.30	2364.86	1836.76	1662.70	3347.02	2763.62	3285.51	2104.91	1910.91	1530.39	9783.33	9808.14
d (m)	4.5	13.5	5.5	5.5	5.4	5.8	5.8	11.7	17.3	15.9	14.2	20.3	18.3
$0.35\gamma H / E_s$ (%)	3.16	2.00	1.59	2.04	2.22	1.03	1.25	2.06	4.31	4.61	4.80	1.28	1.25

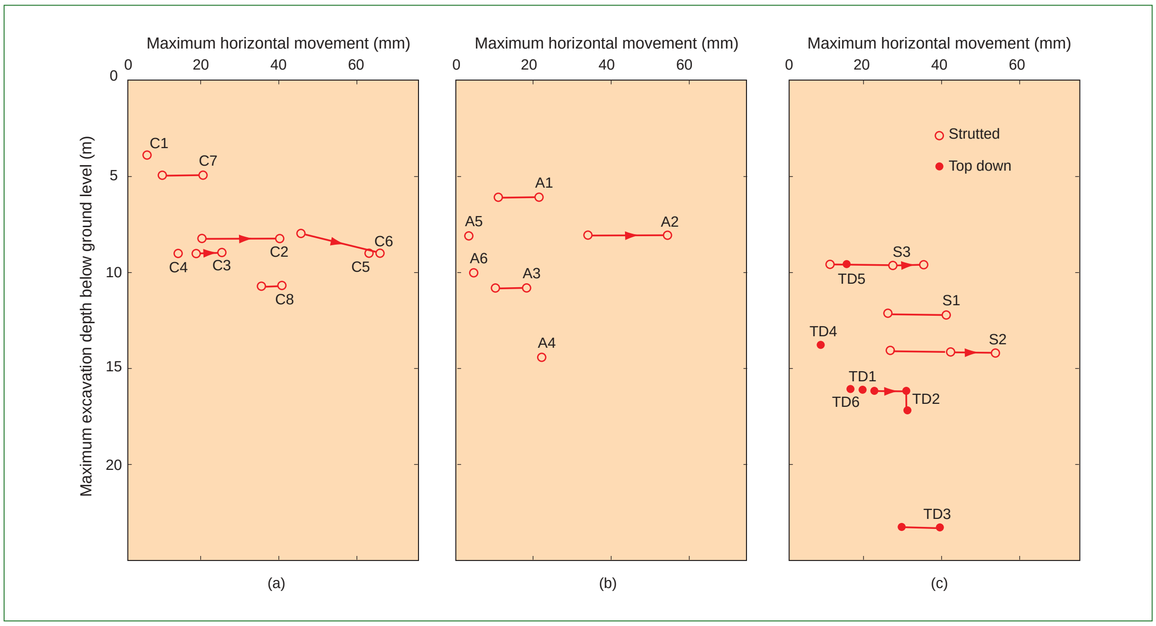


Fig 2.9 Horizontal movements of retaining walls in London clay (see Table 2.1)

Care must be taken when considering the use of anchored walls to ensure that the anchorage zone is well beyond the zone of movement affected by excavation. Reference 2.16 shows that, if this is not done, movements may be significantly greater than would otherwise be expected.

Installing in-situ walls causes additional settlement, often concentrated around the walls themselves. Figure 2.10, derived from reference 2.15, summarises settlements around diaphragm walls constructed in various deposits.

2.3.4 Effect of excavation geometry

For convenience, excavations are often modelled by a plane section. However, reality may be different. Basements may have regular or irregular shapes and the distribution of movements in plan may have as significant an effect as the distribution along a section. Figure 2.7, for instance, shows the measured distribution of movements around an underground car park, and it may be seen that movements, particularly horizontal movements, diminish towards the corners. If basements have corners facing into the excavation (re-entrant corners; see Figure 2.11), movements will be magnified in these areas unless additional measures are provided to prevent movement.

2.4 Effect of ground movement on surrounding structures and services

2.4.1 General

Ground movements are not of themselves undesirable. It is only when they may affect either the building being constructed or nearby buildings, services or tunnels that they need to be considered. Their tolerance to differential movement should be ascertained.

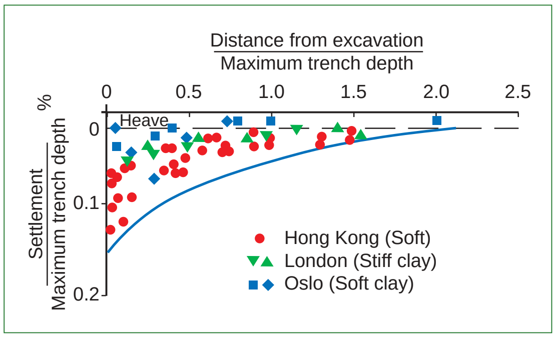


Fig 2.10 Observed settlements in different clays

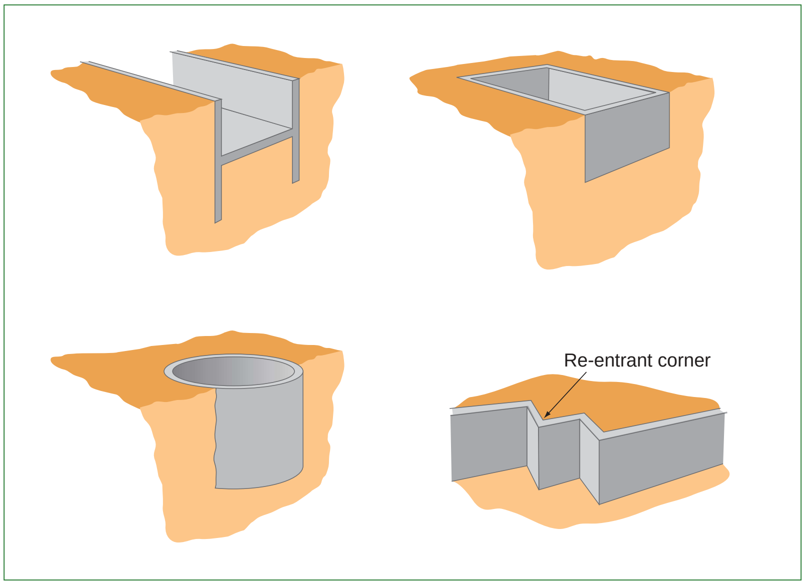


Fig 2.11 Examples of different geometries

2.4.2 Tolerance of buildings

Assessment of degree of building damage can be a highly subjective, and often emotive, matter. In the absence of objective guidelines, based on experience, extreme attitudes and unrealistic expectations towards building performance can develop. It is worth stressing that most buildings experience a certain amount of cracking, often unrelated to foundation movement, which can be dealt with during routine maintenance and decoration.

Clearly, if an assessment of risk of damage due to ground movements is to be made, the classification of damage is a key issue. In the UK, the development of an objective system of classifying damage has proved to be very beneficial in creating realistic attitudes towards building damage and also providing logical and objective criteria for designing for movement in buildings around deep basements and other excavations.

Three broad categories of damage can be considered that affect: (i) visual appearance or aesthetics, (ii) serviceability or function and (iii) stability. As foundation movements increase, damage to a building will progress successively from (i) through to (iii). It is only a short step from the above three broad categories of damage to the more detailed classification given in Table 2.3 developed by BRE[2.35] and adopted by the Institution of Structural Engineers in its report on *Soil-Structure Interaction*[2.36]. This defines six categories of damage, numbered 0 to 5 in increasing severity. Normally categories 0, 1 and 2 are related to aesthetic damage, 3 and 4 to serviceability damage and 5 to damage affecting stability. Detailed discussions of the background to and application of Table 2.3 can be found in references 2.35, 2.37 and 2.38.

Reference 2.37 and 2.39 clearly explain the mechanisms causing cracking, and the magnitudes of strains likely to cause damage in otherwise unstressed buildings. The findings are based on the concept of limiting tensile strain in typical building materials. Previous work[2.40] considered only angular distortions caused by differential settlement. It has been shown[2.41] that the categories of damage given in Table 2.3 can be broadly related to ranges of limiting tensile strain ε_{lim} as given in Table 2.4. This table is important as it provides the link between estimated building deformations and the possible severity of damage.

Outside a deep basement, horizontal extension of the ground as well as settlements may develop. The concepts of limiting tensile strain have been extended to include such horizontal ground strains. Methods of assessing the potential for damage due to ground movements are given in references 2.38, 2.41, and 2.42. Figure 2.12 is an interaction diagram[2.42] showing the relationship between the relative deflection Δ/L and the horizontal ground strain ε_h for a building of length to height ratio L/H = 1.

It is difficult to predict accurately the likely magnitude of cracks occurring in a building as a result of the movement of the ground beneath it. This is because the building itself modifies the distribution of movement, and may tend to move bodily. Also, if cracking does occur, it will do so at points of weakness, and movement will thereafter be concentrated at these points. Ultimately the decision as to what can be regarded as acceptable movement must depend on the condition, nature and value of the adjoining structures.

2.5 Concluding remarks on predicting ground movement

Precedent alone may often be sufficient to enable the designer to be satisfied that ground movements are unlikely to be large enough to be of concern. However, when the effects of movement could cause damage, it is advisable to attempt to calculate what the movements might be. The same applies when no precedent is available.

Predicting ground movements is complex and is discussed further in Chapter 5, as affected by wall displacements, and in Chapter 6 in relation to foundations and base slabs.

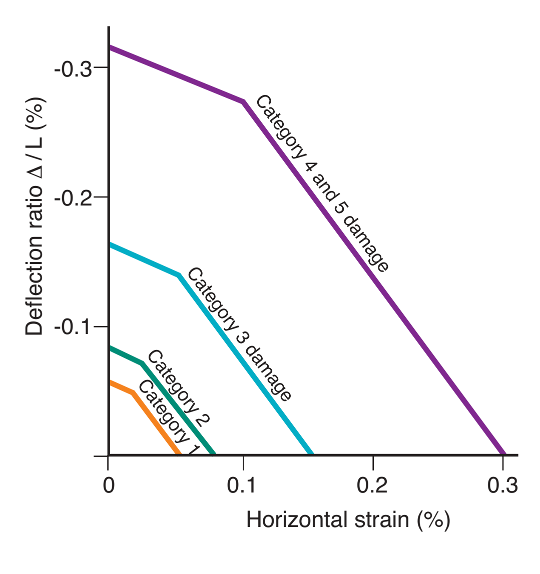


Fig 2.12 Relationship of damage category to deflection ratio and horizontal strain for hogging (L/H = 1)

Table 2.3 Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry

Category of damage	Normal degree of severity	Description of typical damage (Ease of repair is shown in colour) Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.
0	Negligible	Hairline cracks less than about 0.1mm
1	Very Slight	Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1mm.
2	Slight	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to 5mm.
3	Moderate	The cracks require some opening up and can be patched by a mason. 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. Typical crack widths are 5 to 15mm or several > 3mm.
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably†. Walls leaning† or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15 to 25mm but also depends on the number of cracks.
5	Very severe	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. Typical crack widths are greater than 25mm but depends on the number of cracks.

† Note: Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

Table 2.4 Relationship between category of damage and limiting tensile strain (ε_{lim}) (after Boscardin and Cording[2.41])

Category of damage	Normal degree of severity	Limiting tensile strain (ε_{lim}) (%)
0	Negligible	0 - 0.05
1	Very slight	0.05 - 0.075
2	Slight	0.075 - 0.15
3	Moderate*	0.15 - 0.3
4 to 5	Severe to very severe	> 0.3

*Note: Boscardin and Cording describe the damage corresponding to ε_{lim} in the range 0.15 - 0.3% as 'moderate to severe'. However, none of the cases quoted by them exhibits severe damage for this range of strains. There is therefore no evidence to suggest that tensile strains up to 0.3% will result in severe damage.

Realistic prediction of ground movements requires an understanding of the nature of the ground including:

- stratigraphy
- groundwater conditions
- permeabilities
- deformation properties
- initial stresses
- strengths

and of the nature of the construction within the ground including:

- surcharges to the ground
- sequence of excavation
- sequence of propping
- nature of the props, permanent and temporary
- flexibility and strength of the retaining wall
- method of supporting the internal structure
- mass of the internal structure
- construction programme.

Therefore, before time and money are invested in complex analysis, ground conditions must be fully understood and the principles of the construction method established.

2.6 Monitoring ground movements

Monitoring ground movements can be beneficial as a means of control when movement restrictions have been imposed to limit possible damage, and when there is some uncertainty about the validity of the design assumptions.

In either case, it is essential that adequate effort be put into the conception and management of a scheme, and resources are allocated to enable results to be reviewed continuously by someone who understands the underlying principles. An outstanding example of such a scheme is given in reference 2.43. A good monitoring scheme will be simple to operate, free from interference during construction, and flexible. Zero points must be established both in terms of initial readings before work starts, and stable references during the construction period. Methods of measurement are discussed in Appendix D.

Much has been learned in the past by carefully observing the effects of deep basement construction on surrounding ground and structures. Without such observations, it is unlikely that the construction of many of the larger basements now being built would have been considered an acceptable risk. Empirical evidence combined with the ability to replicate performance using both simple and complex models is the only sure method of being able to extend current design practice. The cost of monitoring is small, but the benefits both to the team developing a site and the industry in general can be significant. There are still many aspects of deep basement performance that are not fully understood and there is ample scope for collecting evidence to enable greater economies to be made and to reduce risks in future developments.

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3 Groundwater control

3.1 Introduction

Many of the serious problems that occur with deep basements result from an insufficient understanding of the groundwater regime and the effects basement construction has upon it. Too often, groundwater receives scant treatment during the Site Investigation and the problems of control are left to the contractor to sort out late in the day, with little real data to work from and a contract system too inflexible to facilitate the development of properly engineered solutions. Water in the ground is as important to basement design and construction as is soil or rock. It must receive explicit consideration from the beginning. A well-planned strategy for understanding the influence of the in-situ soils, rocks and basement construction on the groundwater regime in the short and long term should be established at the earliest stages. The value of developing a strategy in this way is described in more detail in reference 3.1. Examples of what can go wrong in projects when groundwater does not get explicit consideration are also given.

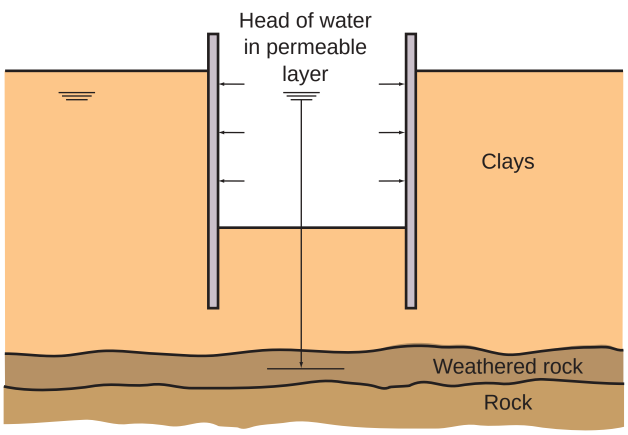

Fig 3.1 Water pressures likely to cause base failure

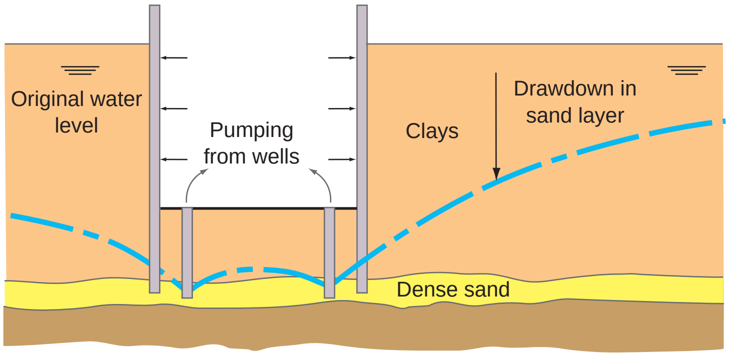

Fig 3.2 Drawdown extending beyond site

The first essential step towards developing successful groundwater control is a proper understanding of the geology, both regional and on site. Stratification can be important. Permeable layers, for example at or close to rockhead (see Figure 3.1), can give enormous problems if they remain undetected until late in a project. Water pressures in such layers below the base of excavations can be high in comparison to the reduced overburden pressure, causing the excavation formation to blow or rapidly soften. Conversely, if the water pressure in such layers is reduced through basement dewatering, the effects of drawdown may extend well beyond the confines of the site (see Figure 3.2). Where compressible strata such as soft clay or peat overlie such layers, they too will be subject to reduced water pressures, with a corresponding increase in effective stress and hence consolidation settlement. This settlement may well be detrimental to structures or services near the site.

If, on the other hand, the presence of permeable layers is appreciated early enough, they can generally be used beneficially in the groundwater control plan. The same is true of layers of low permeability.

Pumping trials should be carried out as part of the Site Investigation: indeed, their benefit in developing the strategic thinking is difficult to overstate (see Chapter 8). Not only is it possible to obtain more confident estimates of permeability and flow rates in this way, but with suitable monitoring the natural inhomogeneity of the ground begins to reveal itself and the effects of geological stratification on drawdown can be determined.

Even so, the pumping trial will provide only an understanding sufficient to develop a preliminary design of the groundwater control system. Successful groundwater control depends on good initial data and a planned approach to getting better data as the work proceeds. Further tests and trials will thus be necessary as the system is installed, the design being modified in the light of the feedback.

In the following sections, dewatering methods are briefly discussed. These are most commonly applied as temporary measures to facilitate basement construction below the water table. Permanent

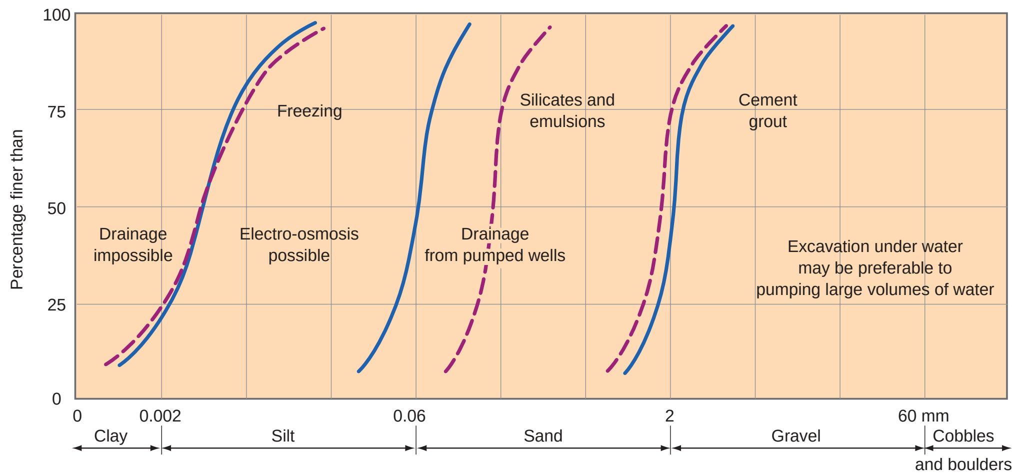

Fig 3.3 Approximate range of particle size for (i) various groundwater lowering processes (grading curves shown full) and (ii) various injection processes (grading curves shown broken). After Glossop and Skempton[3.2]

groundwater control is then considered, followed by a brief review of the problems associated with long-term changes in groundwater.

3.2 Methods of groundwater control

The control method will depend mainly on site conditions and on soil characteristics, especially its particle size distribution. Figure 3.3 shows the range of particle size for various groundwater control methods. Advantages and disadvantages of each are discussed in references 3.3-3.5.

3.2.1 Pumping from sumps

Pumping from open sumps is the most widely used of all methods of groundwater lowering, the cost of installing and maintaining the plant being low. The method is an alternative where wellpointing or bored wells cannot be used because of boulders or other such obstructions in the ground. However, it has the disadvantage that the groundwater flows into the excavation and, with a high head or steep slopes, the sides may collapse. There is also the risk in open or timbered excavations of instability of the base because of upward seepage towards the pumping sump.

The greatest depth to which the water table may be lowered by the open sump method is not much more than about 6m below the pump, depending on its type and mechanical efficiency. For greater depths, it is necessary to re-install the pump at a lower level, or to use a sinking pump or submersible deep-well pump suspended by chains and progressively lowered down a timbered shaft or perforated steel tube.

Further details can be found in references 3.3 and 3.4.

3.2.2 Wellpointing

The wellpointing system of groundwater lowering involves installing a number of filter wells outside the excavation. These are connected by vertical riser pipes to a header main at ground level, which is under vacuum from a pumping unit. The water flows by gravity to the filter well and is drawn by the vacuum to the header main and discharged through the pump.

The wellpointing system has the advantage that water is drawn away from the excavation face, thus stabilising the sides and permitting steep slopes. Indeed, slopes steeper than 1:1 are commonly used when wellpointing in moderately dense fine sands: in contrast, with open-sump pumping, where the water flows towards the excavations, the slopes must be cut back to 1:2 or 1:3 for stability. Therefore, wellpointing gives a considerable saving in total excavation and permits working in fairly confined spaces.

In the right ground conditions, installation is rapid, and the equipment reasonably simple and cheap. There is the added advantage that the water is filtered as it is removed from the ground and carries little or no soil particles with it. Thus, the danger of subsidence of the surrounding ground due to loss of ground is less than with open-sump pumping.

Wellpoints act most effectively in sands and in sandy gravel of moderate but not high permeability. The drawdown is slow in silty sands, but these can be effectively drained. There have been instances of silts and sandy silts being drained by the vacuum process, the upper part of the riser pipe being enclosed with clay to maintain a high vacuum surrounding the wellpoint by means of which the soil is slowly dewatered. Details of wellpoint installation, including

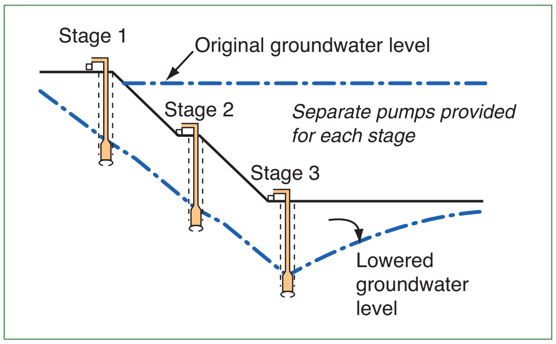

Fig 3.4a Three stage wellpoint installation

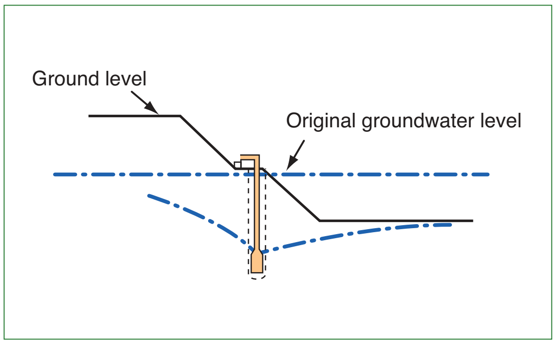

Fig 3.4b Reduction in ground level before installation of wellpoint

the design of filter materials, are described in references 3.3 and 3.4.

One disadvantage of the system is its limited suction lift. A lowering of 4.5-5.5m below pump level is generally regarded as a practical limit. If greater lowering is attempted, excessive air will be drawn into the system through joints in pipes, valves etc., with consequent loss in pumping efficiency. Ground consisting mainly of large gravel, stiff clay, or sand containing cobbles or boulders presents another disadvantage, as it is impossible to jet down the wellpoints and they have to be placed in boreholes or holes formed by a 'puncher', with consequent higher installation costs.

The limitation in the drawdown of water level to about 5m below pump level means that wellpoints for deeper excavations must be installed in two or more stages. A section through a three-stage installation is shown in Figure 3.4a. There is no limit to the depth of drawdown in this way, but the total excavation width at ground level becomes large. The upper stages can often be removed or reduced when the lower ones are working.

It is often possible to avoid two or more wellpoint stages by excavating down to water level before installing the pump and header. The procedure in its simplest form is shown in Figure 3.4b.

Impervious layers of silt or clay are often met in water-bearing sandy soils. Layers as thin as 3mm, if continuous, can be troublesome in a dewatering scheme, causing breaks in the drawdown curve. One possible cure is to jet holes on the side of the wellpoints away from the excavation and fill them with coarse sand. These sand columns provide a path for the water to seep down to the wellpoints more readily than towards the sides of the excavation, preventing weeping from the sides of the excavation.

Layers of highly permeable material can also cause difficulties, as the water will tend to bypass the wellpoints. The solution is to jet at close intervals in a row around the excavation to intermingle the various layers. Settling tanks should be used to check whether fines are being drawn off.

3.2.3 Disposal of pumped water

Where recharge is not used and water arising from pumping operations is discharged into water courses or public sewers, care must be taken to ensure it is not contaminated and does not exceed the limits placed by local or river authorities on total quantity or disposal rate. Control testing for contaminants and physical measurement of flow may prove necessary.

3.2.4 Pumping from deep wells

In permeable strata where the water table is deep or where it is necessary to lower the water levels below 5m, space and cost constraints may prevent multistage wellpointing. An alternative approach is to install a few deep wells, each with a filter well screen and a submersible pump. The well size and spacing should be based on pumping trials as described in references 3.3-3.6.

Pumping from deep wells can significantly lower groundwater levels as far as several hundred metres beyond the site boundary, either within the surface layer or in confined permeable strata at depth intercepted by the deep wells[3.1,3.7-3.9]. The effect of such drawdown must be assessed early in planning the work. Again, pumping trials are invaluable, with piezometers in appropriate strata at various distances from the site.

3.2.5 Pumping using cut-offs

Dewatering for basement construction by any method can have widespread and damaging effects for neighbouring structures, particularly those founded on near-surface compressible soils underlain by more permeable deposits. Alternatively, severe drawdown may adversely affect the ability of other groundwater users in the vicinity to pump for water supply. In such cases, there is a need to limit drawdown beyond the site boundaries by some form of cut-off, such as sheet piles, diaphragm walls and bored piled walls[3.9-3.12].

If there is a layer of impermeable material such as clay at a reasonable depth beneath an excavation, the possibility of toeing the sheet pile into this material should be considered, as pumping costs can be drastically reduced (see Figure 3.5). However, if there is only a thin layer of clay close to the foundation level, care should be taken to ensure that an artesian head sufficient to cause uplift does not arise. Although the groundwater may be virtually eliminated by this method, emergency pumping plant should always be available.

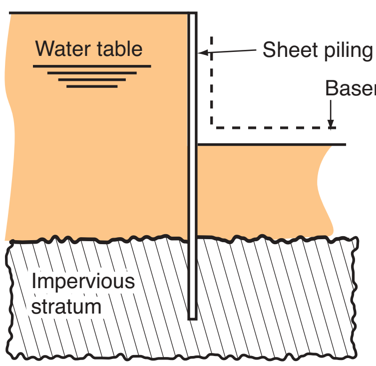

Fig 3.5 Sheet pile cut off wall

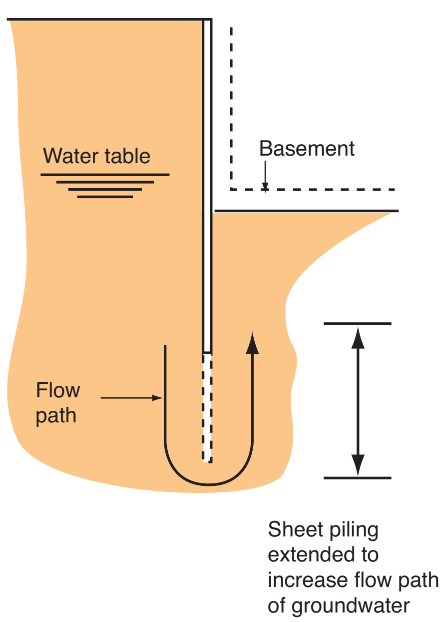

Fig 3.6 Sheet pile in permeable soil

Where the head of water is not excessive, cut-off walls in permeable soils can sometimes be taken deeper to increase the flow path and thus reduce the groundwater flow into an excavation. However, the economics of extra walling compared to reduced pumping costs needs to be considered (see Figure 3.6). Also, the effects of changes in earth pressure resulting from dewatering, and particularly its cessation, must be taken into account.

3.2.6 Relief wells

The problem of groundwater under high pressure in a pervious layer is more common beneath deep excavations than is generally realised and is a real hazard. The water will tend to force itself up into the excavation, softening the excavation base. If the pressure is high enough, it could lift it bodily. Sheet piles and other forms of cut-off wall frequently leak[3.13,3.14] and there is a danger that high water pressures outside may penetrate into strata beneath the excavation.

Pressure-relief wells are a simple, economical and extremely useful way of dealing with such problems. They can take the form of boreholes filled with gravel drilled into the base of the excavation. Each relief well connects into a layer of coarse gravel at formation level. Water flowing up the wells can escape through the gravel layer to a pumping sump. The flow, and therefore the pressure relief, must be maintained while casting the base slab and until the weight of the structure is sufficient to resist the water pressure.

3.2.7 Recharging

A cut-off barrier is often not in itself sufficient to prevent unacceptable drawdown outside the area of construction. Either the aquifer extends to considerable depth or difficulties are experienced in obtaining a satisfactory seal at the rockhead, for example. In such cases, recharging may be an effective answer.

The principle of this method is shown in Figure 3.7. Water drawn from the dewatering system within the excavation is passed back into the ground outside the cut-off barrier through a series of recharge wells, thus maintaining the water pressures in the ground outside the site. Methods of design for recharging and the precautions to be taken to prevent silting and chemical clogging are discussed in references 3.3, 3.7, 3.8 and 3.15-3.17. In recent years, recharging has emerged as an increasingly used and versatile tool, either in the main dewatering works or as an option depending upon the latter's effectiveness.

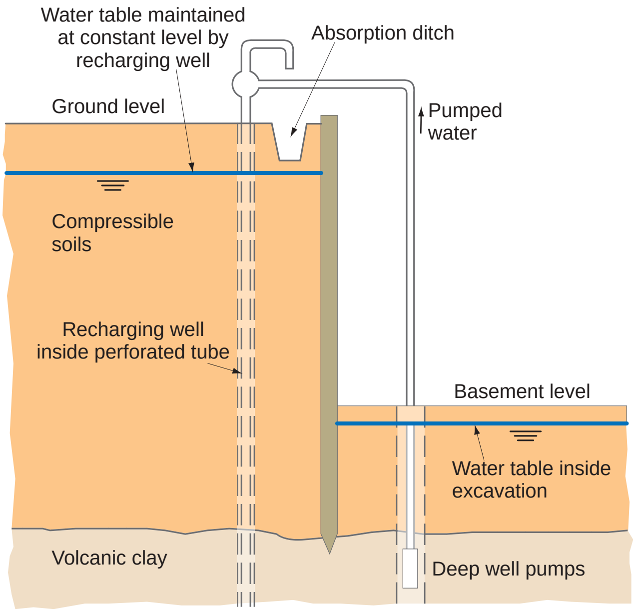

Fig 3.7 System of recharging wells

3.2.8 Grouting

In ground where the permeability is so high that wellpointing or bored wells need a high pumping capacity, or in water-bearing rock formations where wellpointing cannot be used and bored wells are costly, other means should be sought to control groundwater. One method is to inject fine suspensions or fluids into the pore spaces, fissures or cavities in the soil or rock to reduce their permeability. The type of injection material is governed by the particle-size distribution of the soil or the fineness of fissuring in rock strata. Figure 3.3 gives a guide to the suitability of various injection processes for soils. Another guide for the suitability of suspension grouts is given by the groutability ratio. Thus, for a soil to accept a suspension grout, the ratio of the D15 size of the soil to the D85 size of the grout must be greater than 11 to 25 for cement grouts and greater than 5 to 15 for clay grouts. (D15 and D85 refer to the diameter of the particles corresponding to 15% passing and 85% passing taken from the grading curve for the soil).

Grouting costs money. The aim should be to keep the volume of the injected material to a minimum. In this respect, chemical grouts have some advantages. Whereas they cost much more than cements or clays by weight, they can be considerably diluted and still work effectively to reduce the permeability of the ground. There are various admixtures for controlling the viscosity and gelling properties of suspensions and fluid chemical grouts, so limiting their spread in the ground and minimising the thickness of the impermeable barrier. Fluid grouts can be more effective than suspensions in reducing the permeability of the ground because all the pore spaces are filled, whereas suspensions may fill only the larger voids. Useful information on the principles and practices of grouting as a means of excluding groundwater are included in references 3.10 and 3.18. In coarse granular materials or rocks, the excavation is surrounded by a grout curtain consisting of two rows of primary injection holes at 3-6m centres in both directions, with secondary holes, and possible tertiary holes, between them (see Figure 3.8).

In coarse granular materials extending to considerable depths, it may be appropriate, though costly, to introduce a horizontal grouted cut-off at a suitable depth below the base of the excavation to prevent uplift and reduce water flow. Again, a series of injection phases will be needed to form an effective barrier[3.19-3.22].

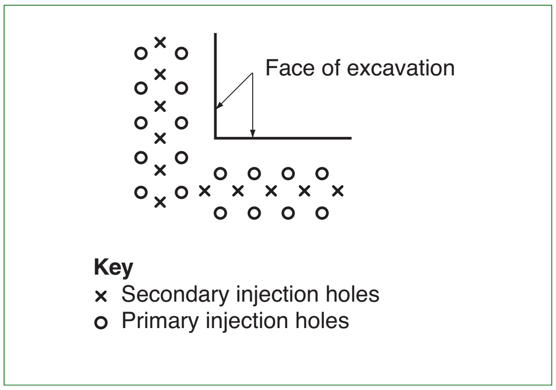

Fig 3.8 Layout plan of grout injection holes

3.2.9 Ground freezing

Because of its high cost, freezing the ground to prevent inflow of water into excavations is usually regarded as a last resort[3.23]. The system also has the drawback that it may take many months to drill the holes, install the plant and freeze the ground. Also, freezing certain types of ground causes severe heaving. Freezing has been used to control groundwater in tunnelling work and in deep-shaft excavations where the pressure of water is too high for compressed air to be used. It is unlikely to be attractive in deep-basement construction on grounds of cost, and in built-up areas because of problems with adjacent buildings.

3.3 Permanent groundwater control

In designing deep basements, two broad options can be considered: the first is to size and provide support to the floors and walls in such a way as to resist fully the water pressures in the ground; the second is to incorporate permanent drainage to reduce these pressures and so allow structural design which is less heavy. The final choice will depend on cost, serviceability, and the effect of drawdown on the area around the site.

Permanent drainage commonly takes the form of a drainage blanket beneath the basement floor slabs linked to relief wells that go down far enough to control high water pressures in layered strata below the basement. Permanent drainage behind retaining walls is easy to install for basements constructed in open cut. However, such drainage is difficult and seldom done for sheet piled walls and in-situ walls. Where water flow rates into the drainage system are large or where significant drawdown occurs beyond the site, a perimeter cut-off wall and/or grouted barrier may be needed, as for temporary dewatering.

Serviceability is an important consideration in permanent dewatering systems. The possibilities of silting, corrosion and clogging by chemical deposition need to be carefully considered[3.3,3.8,3.24]. The transport of fines can be controlled by appropriate filter design in wells and drainage blankets. An understanding of the groundwater chemistry at the site is important so that suitable corrosion protection measures can be incorporated. It may be necessary to design the elements of the drainage system to resist chemical attack or to make access possible so that affected parts can be replaced or cleaned.

Sustained and concentrated groundwater flows in the ground around a site induced by installing a permanent drainage system can progressively remove soluble materials, in turn causing settlement[3.25,3.26]. The chemistry and stratification of the ground around the site must therefore be carefully considered.

The continued operation of the system in the face of mechanical failure of the pumping equipment or power supply needs also to be addressed. Where appropriate, back-up pumps and power supply should be provided or contingency arrangements designed to allow water to flood parts of the basement through pressure-relief valves in the event of excessive pressure build-up beneath the slab.

A further possible hazard is methane in solution in the groundwater (see also Chapter 7). This may be introduced to the permanent drainage system, and gradually accumulate in sufficient concentration to be an explosive hazard. If there is a risk of such gases being present in the groundwater and in closed spaces in the basement, the system should be designed to vent them safely[3.27,3.28].

A similar problem can occur where de-oxygenated air in the ground is transmitted to the surface via the drainage system. Cavities in a basement structure subject to human access and which connect with the drainage system should therefore be adequately ventilated.

Designing permanent drainage for deep basements is often not straightforward and specialist advice should be sought.

3.4 Changes in groundwater regime

In designing a basement and the necessary groundwater control measures, assumptions have to be made about ambient water levels and pressures. The natural variation in these levels should also be considered explicitly and limiting values chosen. Groundwater conditions found during the Site Investigation or later may not necessarily provide appropriate design values. Water levels may change through seasonal effects, variation in nearby river levels or tidal cycles. The likelihood of flooding or the effects of local drawdown due, for example, to the construction of similar basements nearby should be considered carefully.

The risk of an increase in groundwater level at one side of a basement construction due to groundwater flow must be considered in both the temporary and the permanent condition. During construction, an impermeable wall around the excavation may increase groundwater in permeable soils where flow is predominantly across the site and where the plan length of the excavation is substantial. The risk of this dam effect in increasing groundwater height may be shown by piezometer readings before construction. Monitoring piezometers after installing the wall will indicate any increase and remedial pumping measures to reduce excessive pressure may be needed. There may be a similar long-term risk that will require remedial drainage measures.

In many cities of the world, the water regime is changing with time. Pumping deep aquifers on a regional scale may be gradually lowering of water levels in the ground, e.g. Helsinki, Mexico City and Venice[3.1]. In other cities, such as London, Birmingham and Tokyo, groundwater levels are now rising because of a reduction in pumping from extraction wells[3.27,3.28]. Such a rise could significantly affect the design of deep basements and their foundations. The design implications are considered in Chapter 6. Basement designs should take account of possible future groundwater levels in aquifers below their sites. Sometimes it will be appropriate to install permanent groundwater control systems to deal with the possibility that water pressures will increase in future (see, for example, Figure 6.5). Usually, where the regional water regime is changing significantly, the problem is being studied and designers should consult local reports and briefing documents.

Where a difference in water head, however temporary, can occur across the basement/station, the stiffness of the structure and its enclosing soil should be considered, load paths identified, and the strength of components on the load paths checked. Where in-plane membrane stresses are used to transfer these forces, the effect of perforations for escalators, tunnel eyes, etc., should be considered.

In London, a consortium of Thames Water, London Underground Ltd and the Environment Agency has produced a strategy for dealing with the problem of rising groundwater. The team, under the acronym of GARDIT (General Aquifer Research, Development and Investigation Team), has published a proposal[3.29]. This indicates that groundwater levels have been rising at the rate of 2m per year but that selective pumping can reverse this trend. Government has adopted the proposals, so engineers should carefully monitor the effectiveness of these proposals when embarking on the design of a basement. There is no evidence of similar schemes elsewhere in the UK.

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4 Methods and type of construction

4.1 Introduction

This report addresses both deep basement and cut-and-cover construction. The techniques share some design and construction matters but the depths and spans of cut-and-cover works often require consideration of large design moments and buoyancy forces. Cut-and-cover construction for metro construction, station boxes and entrance sections to road tunnels is often below trafficked streets. Existing services there, sometimes uncharted, can seriously increase construction time and cost. Horizontal alignment of road and metro tunnels built by cut-and-cover may also be adversely constrained by the alignment of existing roads and work sites.

Deep excavations for basements and cut-and-cover structures require secure earth and groundwater retention in the temporary, construction and permanent support phases. There are several wall construction techniques for each of these requirements but application depends initially on the method of substructure construction, of which there are five categories:

- open excavation where the face of the excavation is unsupported
- open excavation where the face of the excavation is supported by nailing or similar techniques to allow steepened batters in the temporary state
- bottom-up excavation where the excavation face is temporarily supported laterally as excavation proceeds
- top-down excavation where the permanent works, walls, floors and roof are used to give lateral support at the periphery of the excavation in both temporary and permanent states
- semi-top-down construction with
  - ❍ minimal temporary works (e.g. plunge columns only)
  - ❍ maximum opening sizes in the permanent works for ease of excavation.

4.2 Methods of construction

4.2.1 Construction in open excavation

This is applicable where the site has room to accommodate soil batters, unlikely on most urban sites. Assessing the slope stability of these batters will turn on an evaluation of soil strengths and groundwater conditions and a risk assessment of the consequences of slope failure. A dewatering system may be needed to depress the groundwater levels locally for the construction period; some settlement of the surrounding ground may result. Figure 4.1 shows a slope designed for medium-term stability with protection applied to the slope surface to reduce the adverse effects of weathering.

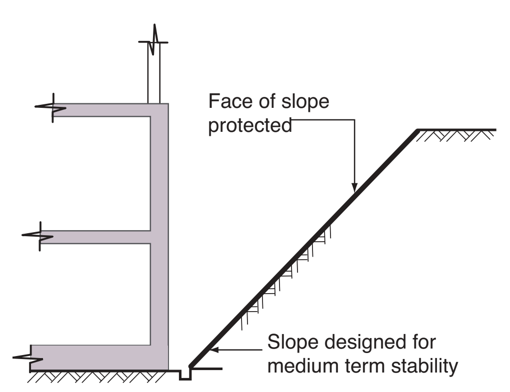

Fig 4.1 Construction in open excavation

4.2.2 Construction within soil slopes of increased inclination

A reduced plan area for that required for open plan excavation may lead to the adoption of toe walls using crib walls, gabion walls or nailed slopes. Nails installed by drilling and grouting or pneumatically using steel or carbon fibre nails may appear costly but may prove to be a feasible solution when comparison is made with methods that provide lateral support to a vertical excavation.

4.2.3 Bottom-up excavation

Bottom-up excavation is the traditional alternative to open excavation and some examples are given in Figure 4.2. Removal of as much spoil as possible by quick and economic methods by direct access is achieved in excavations of moderate depth although deeper excavations may be constrained by access ramp dimensions. The elements of support for bottom-up construction are:

- temporary and permanent retaining walls
- temporary support to retaining walls. This can include soil berms, horizontal props and rakers or soil and rock anchors.

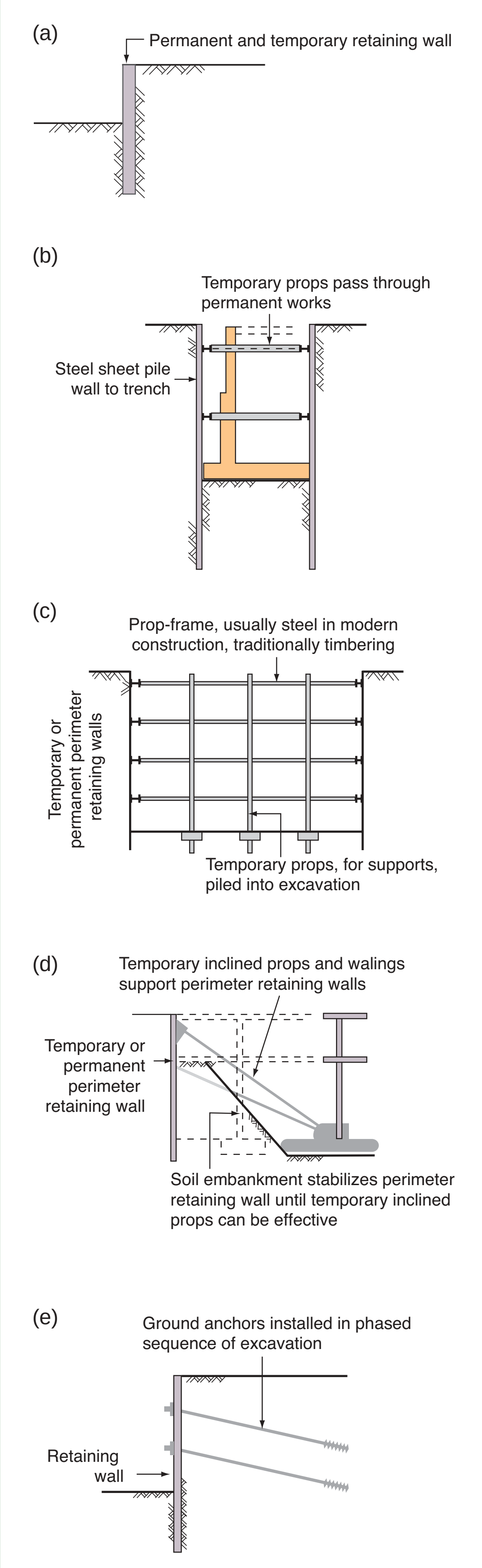

Fig 4.2 Methods of bottom up construction

4.2.4 Top-down construction

Top-down construction (see Figure 4.3) uses the permanent walls and floors progressively to maintain retention of the surrounding soil and groundwater. Its principal advantage is the reduction in the extent of temporary works and the prospect of simultaneous substructure and superstructure construction. Control of lateral movement and settlement is better than with bottom-up construction. Where heave of the

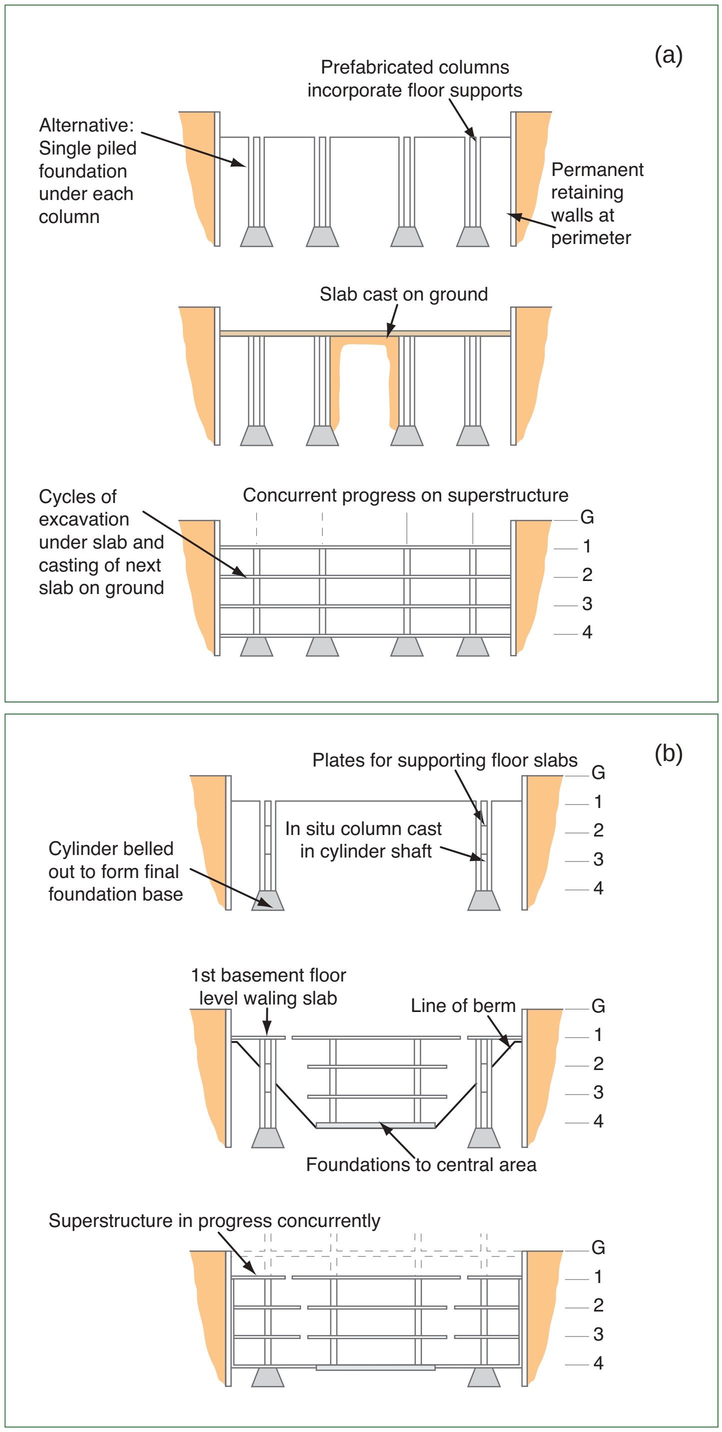

Fig 4.3 Top-down construction

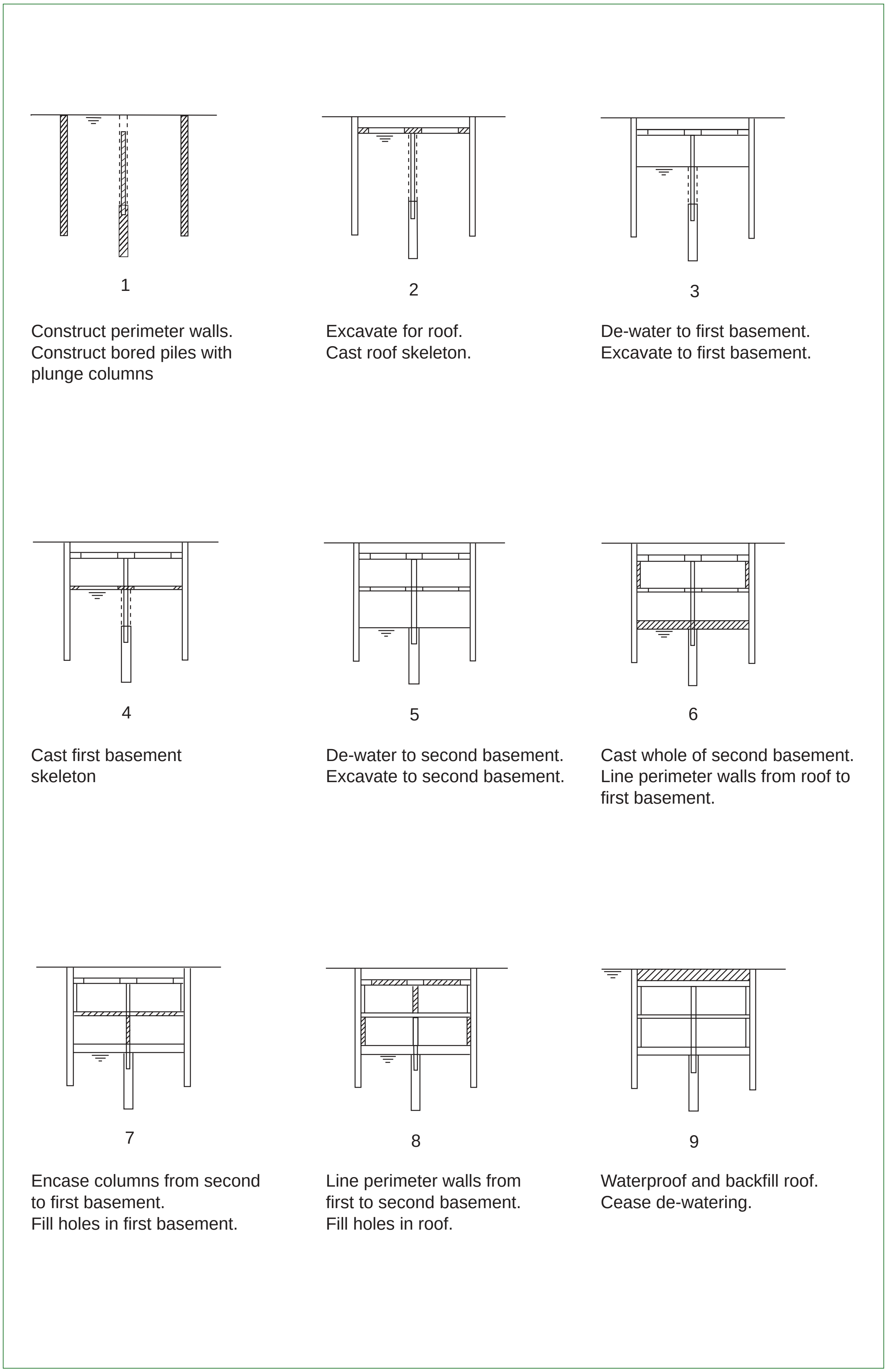

Fig 4.4 Staged construction sequence, semi top-down

excavated ground surface can be expected, the lowest basement floor may need to be isolated in the short term from upward pressure from the soil below it. It is particularly applied to basements and cut-and-cover construction of greater depth; time and cost may be saved. The apparent advantage of simultaneous excavation and superstructure activities may be diminished by lack of site space and access for these operations. Some savings may result from the reduced need for propping to soffit shutters for floor slabs cast before excavation beneath them.

The sequence of top-down requires structural column installation as 'plunge columns', with the column foundation below final excavation level.

The development of the construction sequence was originally shown by Zinn[4.1], although the earliest deepest examples were those in Paris[4.2]. In later years, the method was developed in Hong Kong and now virtually all deep basement works there are constructed by this method.

4.2.5 Semi-top-down construction

This method of construction uses the ground-floor slab or substructure roof as a working platform with large openings, the slab being designed to act as a frame to provide lateral support to the external walls. Excavation takes place beneath this floor and the arisings are removed through the large openings. This technique, with only a skeletal structure for the working platform and one intermediate floor, was used for a 25m-deep excavation for station construction on the Singapore MRT[4.3] (see Figure 4.4). After initial construction of the roof, its skeletal plan shape provided a working platform for excavation down to concourse level using backhoes and long-arm excavators. The concourse slab was then built, again with large openings to allow excavation beneath, at the same time supporting the sidewalls to the box. Figure 4.5 (a-c) show semi-top-down construction for cut-and-cover construction on Singapore MRT works.

4.2.6 Bottom-up and top-down methods

Combinations of bottom-up and top-down methods of basement construction can have advantages. For example, the central core may be constructed as if for bottom-up, leaving the perimeter walls behind soil berms or temporarily braced. Perimeter top-down construction is then propped from the central floors. This mix of methods was used for the basement of the Main tower in Frankfurt[4.4]. The five-storey basement, in weak Frankfurt Clay below the 198m-high tower, was founded on a piled raft. This part of

Fig 4.5a Semi top-down construction, Singapore M.R.T, C705 Boon Keng Station
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Fig 4.5b Semi top-down construction, Singapore M.R.T, Illustrating Plunge Columns
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Fig 4.5c Semi top-down construction, Singapore M.R.T, C705 Potong Pasir Station
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the structure was constructed top-down within a secant pile wall to exclude a high groundwater level. The top-down construction was preceded, however, by the bottom-up excavation of a smaller pit in the centre of the foundation area in which the heavy reinforced concrete core of the tower was erected (see Figure 4.6). Within the waterproof outer pile wall, the initial pit was propped by four frames of steel bracing. Once the concrete core had been constructed up to ground level and the entire first basement level completed, the remainder of the basement between the outer pile wall and the initial pit was excavated by the top-down method. In this way, superstructure core construction was well advanced in parallel with basement construction.

An early application of a beneficial mix of bottom-up and top-down methods within the same basement area was in 1962-63 for a basement up to 18m deep from road level at Britannic House in London. Figures 4.7-4.10 show the sequence of construction[4.5]. An external diaphragm wall box was initially supported by a soil berm while the central raft and then the core and tower columns were constructed bottom-up. Once the strutting floor was finished, top-down excavation was carried out below the floor and the structure of the lower floors and raft was completed.

4.2.7 Flying shores

Flying shores may be needed to support party walls of adjoining properties across a narrow site. Using top-down construction, the new basement floors may serve the same purpose, supporting loads from existing footings at the site boundary. In such circumstances, property owners and their advisers must be aware of their responsibilities under the Party Wall Acts[4.6].

4.2.8 Observational Method

The Observational Method (see Appendix E) provides economical construction as long as the initial design can be modified during construction. The method, first described in detail in 1969[4.7], is based on the most probable design. However, if monitoring shows performance exceeding predicted behaviour, contingency plans are triggered. The response time to implement contingency arrangements must be appropriate to avoid risk. The method and successful applications are described in references 4.8-4.10.

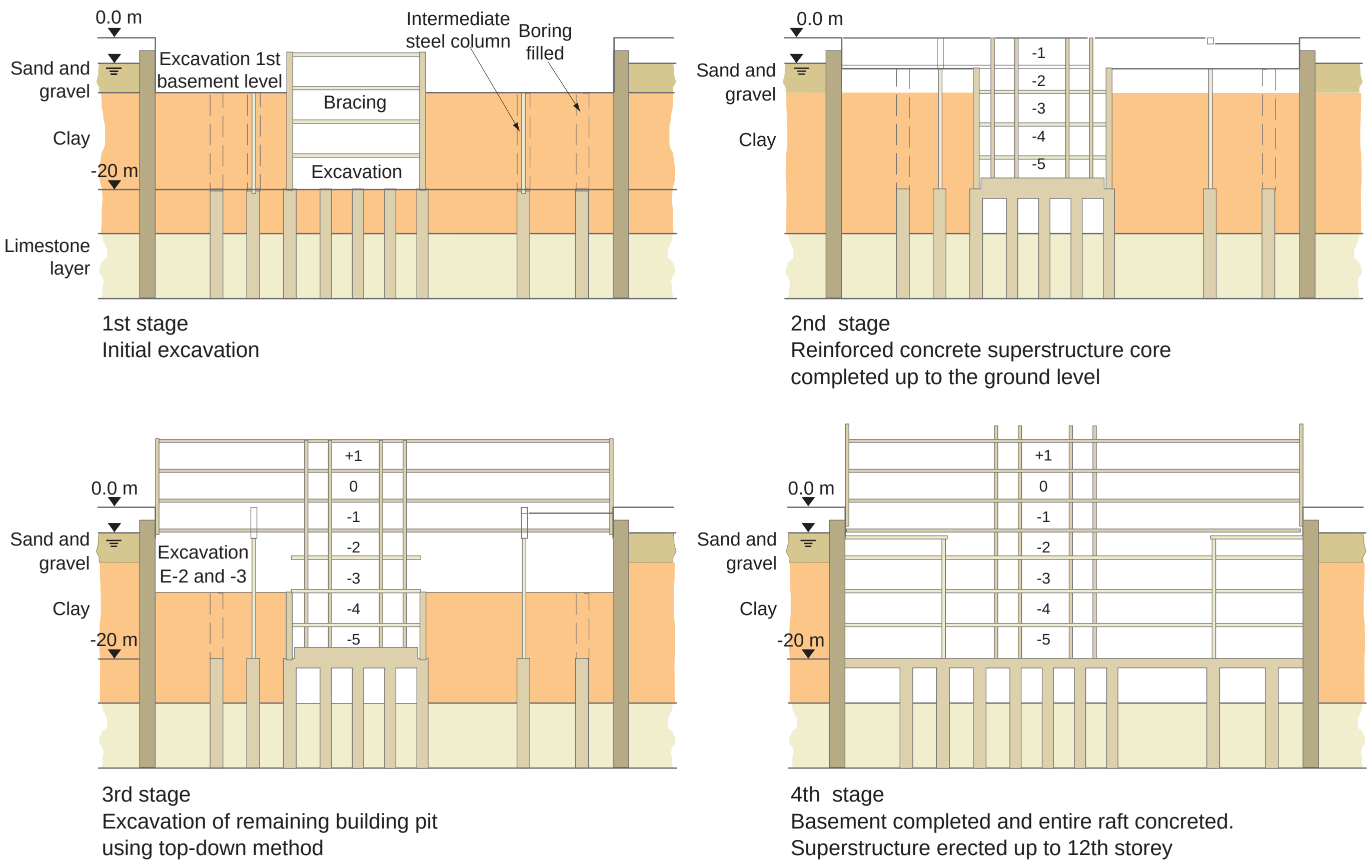

Fig 4.6 Modified top-down method for the Main Tower: construction process

4.3 Types of wall

There is a range of wall types to fulfil either temporary or both temporary and permanent soil support. Their availability varies geographically according to market demand, predominating subsoil conditions and specialist local labour resources. Table 4.1 summarises wall types with details of depths, installation verticality tolerances, advantages and limitations. The use of gravity walls such as crib and gabion walls has not been included, although both may find application to stabilise or support slopes to allow open excavation for permanent substructure construction. Later use of such walls has included soil reinforcement with metal strips and geogrid mesh.

The wall types shown in plan in Figure 4.11 are described in the following subsections, with the exception of king post walls.

4.3.1 Sheet piles

The economic choice of sheet piles for basement and cut-and-cover construction depends primarily on soil conditions, depth of excavation and any restrictions on noise and vibration. Typical pile hammers are shown in Figures 4.12 and 4.13. Recent changes to available sections by steel producers have increased the flexural strength of sheet piles, and developments in pile installation methods (using hydraulic clamps and ram equipment) have reduced installation noise and vibration compared with conventional driven operations. These changes, together with improved methods of sealing pile clutches, have led to the

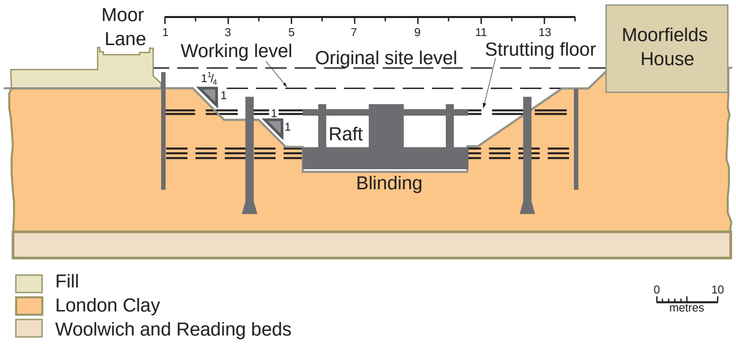

Fig 4.7 Britannic House, London: Method of construction

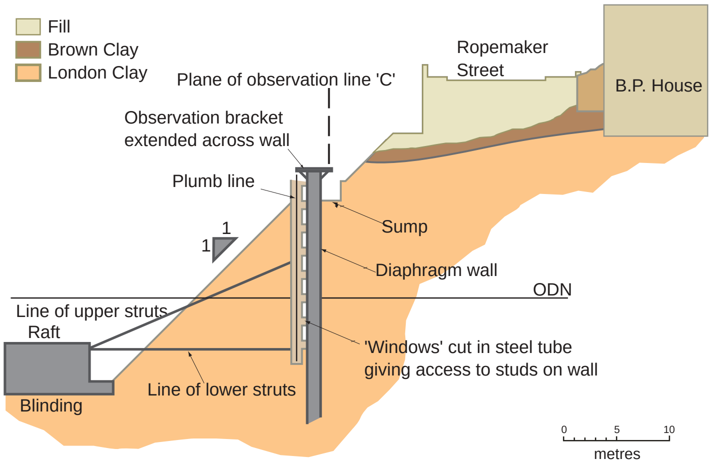

Fig 4.8 Britannic House, London: North diaphragm wall after bulk excavation and before excavation for struts

Fig 4.9 Britannic House, London: North diaphragm wall after bulk excavation and before excavation for struts

Fig 4.10 Britannic House, London: Showing top struts positioned and preloaded

Table 4.1 Wall types for temporary and permanent soil support in basement construction

Wall construction	Temporary/permanent support	Typical wall depth	Typical retained height	Usual installation tolerance: verticality	Advantages/disadvantages	Remarks
King post wall: steel UC soldiers and timber or r.c. (or p.c./p-s.c. + grouting) skin wall/lagging	Usually only temporary support	King posts typically 6 to 20m	3.5m as cantilever 12 to 15m anchored	1:100	Generally only used where groundwater is below formationlevel. Not feasible in soft and loose soils.	(Also known as post-and-lagging or Berlinoise.) Where good construction tolerances apply the wall surface may, be used as a permanent back shutter to an r.c. wall.
Steel sheet piling	Temporary or permanent support (e.g. in car park basements).	Typically 10 to 15m. Max pile length ~30m.	8 to 12m as single propped wall	1:75	Vibration and noise can be overcome in some soils by use of hydraulic press equipment. Risk of declutching by obstructions.	Re-use of sheet piles will often determine cost viability of temporary sheet piling.
R.C. Piles Contiguous piles	Temporary and permanent support (where r.c. facing wall is used).	12 to 20m	6 to 15m, propped or anchored	1:100	Cheapest form of r.c. piles when installed by cfa equipment. Not a water resistant wall.	Can be used with jet grouting to provide permanent water and soil exclusion.
R.C. Piles Hard/soft secant	Temporary and permanent support, see note regarding durability.	12 to 20m propped or anchored	6 to 15m,	1:125	The use of a weak concrete mix to allow economical excavation of secant by male piles may also have durability disadvantages long term.	May only be considered water resistant in the short term.
R.C. Piles Hard/firm secant	As for hard/soft secant				The use of a stronger mix for female piles than that used for hard/soft secants may improve water resistance and durability long term.	
R.C. Piles Hard/hard secant	Temporary and permanent support, usually permanent.	15 to 30m	10 to 20m, propped or anchored	1:125 to 1:200	Depth limited by vertical tolerances which influence depth of cut secant joint, and their water resistance. Avoids the use of slurry.	Female pile may be reinforced with UB section, male by UB or circular rebar cage. Shear plates may be welded to UBs before insertion for floor connections.
Diaphragm walls Installed by grab	Permanent (if temporary, then left in place)	15 to 30m	12 to 25m, propped or anchored	1:125	Heavy installation plant and increasing difficulties in disposal of slurry pose disadvantages.	Solution to deep walls in variable soil conditions with water retention. Difficulties may arise with excavation of obstructions, natural or otherwise. The wall surface may serve as the final finished surface for some applications.
Diaphragm walls Installed by cutter	Permanent (if temporary, then left in place)	15 to 50m	12 to 35m, propped or anchored	1:400	Improved installation tolerances but minimum job size influenced by large mobilisation and demobilisation costs.	

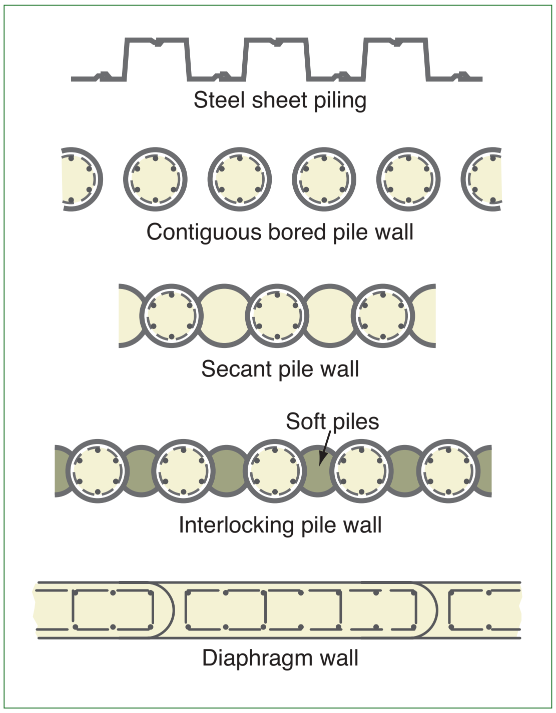

Fig 4.11 Examples of permanent walls

Fig 4.12 Sheet pile installation by high frequency vibrator

greater use of sheet piles for both temporary and permanent basement walls with high standards of water resistance even in water-bearing ground[4.8].

The use of sheet piles together with structural steel sections (known in Continental Europe as 'Peine piles') or with tubular steel sections ('Combi walls') produces walls of considerable flexural strength and finds particular application in cut-and-cover works where a reduction in lateral bracing is particularly advantageous.

Fig 4.13 Sheet pile installation by hydraulic hammer © BSP International Foundations Limited

4.3.2 King post walls

Walls for temporary soil support during construction using soldiers, or king posts, of steel sections with horizontal timbers spanning between them (or reinforced concrete skin walls spanning between king posts) are used extensively in non-water-bearing ground. The king posts may cantilever for shallow excavations or may be propped with rakers, bracing or ground anchors for deeper excavations. The wall is often used as a permanent back shutter to the permanent reinforced concrete basement wall. Figure 4.14 shows an anchored king post wall for basement construction in Medinah, Saudi Arabia.

4.3.3 Contiguous bored pile walls

Closely spaced bored in-situ concrete piles, installed by auger or continuous flight auger (cfa rig), provide an economical wall for excavations of moderate depth in subsoils that are easily drilled and where free groundwater is limited. A typical cfa rig is shown in Figure 4.15. The advent of powerful rotary machines (with maximum torque at the rotary table of the order of 50 tonne-m) has promoted the use of this low-cost piled wall with minimum installation noise and

Fig 4.14 King post wall construction, Medinah, Saudi Arabia

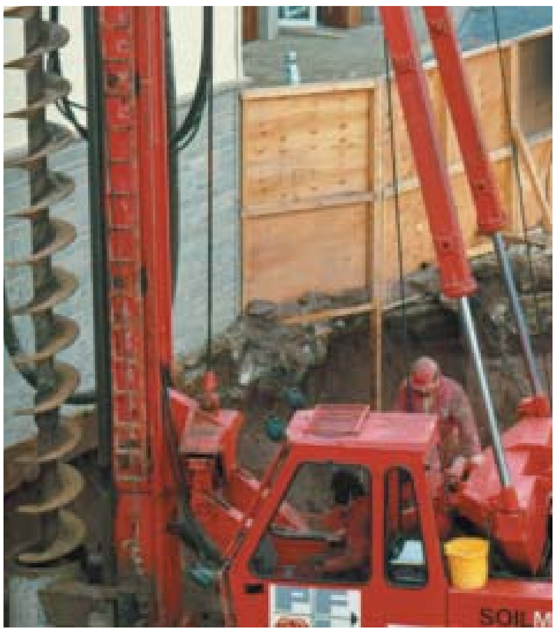

Fig 4.15 Continuous flight auger (cfa) rig

Fig 4.16 Hard-soft secant wall construction, Carlton Gardens, London

vibration. Installation tolerances and minimum distance to existing walls for plant operation need to be noted. Where groundwater is likely to flow or seep into the gaps between piles, it may be necessary to plug them with in-situ concrete or jet grouting behind the piles. Contiguous bored piling must be lined or faced with a reinforced concrete wall if there is risk of water ingress or loss of loose soil through the gaps between piles. Independent blockwork walls with a drained cavity may also be used. In determining available floor area or width within the substructure, the additional thickness of facing or blockwork walls must be allowed along with wall installation tolerances.

4.3.4 Secant pile walls

Secant pile walls are formed by installing augered and cased or cfa piles on a hit-and-miss basis at pile centres slightly less than pile diameter. The initial (female) piles may be concreted with normal mix concrete (hard-hard secant wall) or with a weaker grade concrete allowing the male piles to cut the secant area into the female pile cross-section with less effort (hard-soft secant wall). A typical hard-soft secant wall is shown in Figure 4.16. A compromise in the reduction of the strength in the weaker pile is also sometimes used (hard-firm secant wall).

When cfa rigs are used to install the secant piles, the reinforcement cage is pushed into position through the wet concrete or the cage is vibrated to a lower level using a vibrator and steel mandrel. The use of cfa piles in secant walls is therefore restricted to depths of 12-20m. Installing reinforcement can become more difficult if the pile concrete stiffens as free water drains from the mix into the surrounding ground.

Secant pile walls are preferred in granular water-bearing soils, where contiguous piles are unlikely to be satisfactory. Constructing guide walls for secant pile installation involves additional time and expense.

Hard-soft secant pile walls, installed by cfa rigs, provide a most competitive solution for both temporary and permanent soil retention in water-bearing free-draining soils. In such conditions, however, the risk that the concrete will become less durable and waterproof must be assessed; where necessary, a lining wall should be installed to counter such risk.

4.3.5 Diaphragm walls

The use of slurry-supported trench operations filled with tremied concrete to provide a wall for both temporary and permanent soil retention, as introduced by the ICOS Company in the 1960s, has developed during the past 20 years with important improvements in excavation and slurry-cleaning equipment. In particular, the use of cutter-mill excavation equipment based on the reverse circulation of soil cuttings and slurry has allowed the construction of deep walls (structural walls up to 60m and more) with exacting standards of vertical tolerance (between 1:200 and 1:400). Figure 4.17 shows grab excavation equipment and Figure 4.18 a modern cutter rig developed for working in low headroom. A conventional cutter is shown in Figure 4.19 and a specially developed mini-cutter for constricted urban sites in Figure 4.20.

Using heavy steel reinforcement to withstand high flexural wall moments can delay the placing of reinforcement (see Figure 4.21) and make it difficult to ensure homogeneous in-situ concrete.

Early developments in diaphragm wall design included the use of precast post-tensioned wall elements and post-tensioned in-situ walls. Neither of these innovations has found favour in the UK, although the improved surface finish of precast elements and the reduction of reinforcement quantities in post-tensioned walls can prove advantageous. In some countries, these methods can be subject to patent restrictions.

Any prestressing is undertaken before the soil in front of the wall is excavated and while the wall is fully embedded on both sides. Tendon forces and eccentricities are determined using the final loading of the structure and the retained soil with no tension developed across the wall cross-section. The soil restraint during prestressing is calculated by assuming full passive pressure and earth pressure at rest. For examples of prestressed walls and a description of the method, see references 4.12 and 4.13.

4.4 Selection of wall type

In practice, diaphragm walls have tended to find use in basements and cut-and-cover structures of larger plan area and greater depth and especially where groundwater exclusion applies. For modest depths and basements of up to two storeys, bored piled walls are likely to prove more economic, especially where soil conditions allow efficient drilling with limited overbreak.

Comparisons of alternative wall construction options should take account of the total construction cost, including the cost of facing walls, together with the long-term financial effect of loss of finished plan area and width. References 4.13-4.15 give relative cost data.

4.5 Types of support system

Various methods can be used to restrain the peripheral walls while the permanent structure is being built. The

Fig 4.17 Grab excavation equipment for diaphragm walls

Fig 4.18 A cutter rig working in low headroom

© Benaim

Fig 4.19 Reverse circulation cutter for diaphragm walls

Fig 4.20 Cutters of mini-rig for restricted urban sites © Bauer Spezialtiefbau

Fig 4.21 Diaphragm wall cage during insertion into a slurry filled trench

design of such supports and the wall itself needs to address ultimate limit state (i.e. collapse conditions) and serviceability limit state (i.e. deformation of the wall and settlement and displacement of the subsoil surrounding the structure and below it).

In the serviceability state, settlements of existing services and structures close to the new works may require attention. The acceptable deformation of structures in terms of horizontal strain and angular deformation is discussed in references 4.16 and 4.17. The maximum allowable deformation may be specified at serviceability limit state, in which case care must be taken to ensure accuracy as unnecessary cost consequences may result.

The construction constraints in selecting a method of construction, wall type and support system are summarised below. Further discussion on geotechnical aspects can be found in Chapter 5.

4.5.1 Temporary restraint

Cantilever walls

The satisfactory use of cantilever walls to excavations deeper than 3.5m will depend on the subsoil/ groundwater conditions, the imposed live loads and the permissible wall deformation and ground settlement. Existing services and structures may be at risk because of unacceptable soil movement. The wall stiffness is critical, even with what seems an adequate depth of penetration into soil of apparently high stiffness. Assuming that wall collapse is safeguarded by adequate wall penetration, it is possible that cantilever walls will still deform excessively.

Soil berms

The usefulness of temporary soil berms to provide lateral support to walls during substructure construction is well established. The lateral support provided by the berm is transferred to temporary props or rakers before the complete removal of the berm and the casting of the base slab or raft. Lateral wall displacement during this process may be improved somewhat by excavating the berm in short lengths or by a hit-and-miss sequence. An analysis of the efficiency of berms in restricting wall moments and movement is given in reference 4.18, which concludes that, while the horizontal resistance of berms has a beneficial influence on general wall behaviour, it depends on berm size. For berms of small volume (i.e. shorter than 2.5m), the dimensions of the berm are the prime influence on wall deflections and moments. On the other hand, for larger berms it is the volume of the berm that has the greatest influence.

Rakers and struts

Rakers may be used as the sole temporary support to walls of modest excavation depth. Reaction may be gained at the base of the raker from the raft or basement floor. Outer walls may require re-propping from the completed floor after the removal of temporary rakers and before construction of the permanent floors above.

For deep excavations for basements and cut-and-cover construction, multi-bracing with strutting right across an excavation, or multi-layers of anchors provides support to avoid collapse or excessive settlement. A typical heavy bracing system for cut-and-cover construction is shown in Figure 4.22. Where adjoining settlements are predicted to be excessive, preloading props can sometimes be used to reduce them. In all cases, the standard of workmanship of the temporary works will define the extent of lateral deformation of the wall and the resulting surface settlements. Replacement props may be needed to support outer walls as temporary props are removed to accommodate the new permanent structure.

Ground anchors

Ground anchors are installed at the perimeter retaining wall level by level as the internal excavation progresses. If the adjacent land belongs to others, wayleave will be required. The main benefit from anchoring is the unobstructed working space for permanent works construction. Post-tensioned anchors also help to reduce wall deformation and in turn restrict settlements. Load capacity may relax during the period of permanent works construction and monitoring and re-stressing may be necessary. Anchor costs may detract from the advantages gained by an unobstructed excavation. The high safety factors required in the UK for temporary anchors have detracted from their use for this purpose.

With anchors, walings must be provided with adequate restraint against rotation and walls checked for vertical displacement as a result of the vertical component of the anchor force.

Reduction of settlement

Measures to reduce soil deformation and the resulting settlement to nearby structures include top-down construction, walls and temporary strutting of increased stiffness, post-tensioned ground anchors and preloading of temporary strutting. These measures, however, may not adequately reduce settlements due to soil deformation below final excavation level of deep basement works. To achieve this, it may be feasible to prop external walls ahead of the main excavation by diaphragm walls or tunnelling. Each of these methods may carry substantial cost penalties, which can only be accepted if predicted settlement risk is considered excessive. The tunnel-prop method has been used on two sites in London, the first at Barbican Arts Centre[4.19] and more recently at the excavation for Westminster Underground Station[4.20].

In the latter scheme, shown in cross-section in Figure 4.23, both low-level diaphragm crosswalls and tunnel struts were considered as ways of reducing deformation and settlement risk to nearby structures (including the Big Ben clock tower). Low-level tunnel struts were chosen to avoid the perceived

Fig 4.22 Construction proceeding below heavy temporary steel tubular struts
© Balfour Beatty AMEC

risk of poor contact between the outer box diaphragm wall and the diaphragm crosswall together with the difficulty of installing jacking equipment under 40m head of bentonite slurry. Three 1770mm-diameter hand-dug tunnel struts using precast concrete segments and reinforced concrete filling were used with hammerhead walings 1800mm deep. Access was via 3m-diameter lined pile shafts. Within each strut, a 2440mm-diameter jacking chamber was installed with a jack capacity of 38 MN and a stroke of up to 50mm.

4.5.2 Permanent restraint

In the permanent condition, outer walls will be propped at successive floor levels or at the substructure roof. Only low compressive stresses will usually be induced in the floors or roof although, where moment fixity occurs at their junction, moments will be distributed in proportion to their relative structural stiffnesses and the resulting stresses must be allowed for. Where there are large openings in the floors close to their junction with the wall, shear stresses must be checked and reinforcement added if necessary.

Care must be taken to ensure that, after permanent works construction, any space resulting from the extraction of temporary walls is adequately backfilled to avoid excessive settlement of adjacent ground.

In top-down construction, basement floors are supported by plunge columns as excavation proceeds.. These columns are then used as permanent columns to support the superstructure, sometimes with strengthening steelwork or reinforcement. Tolerances for top-down piles and plunge columns quoted in the CIRIA report for embedded walls[4.15] are shown in Table 4.2.

4.5.3 Effect of installation of sheet piles and soil retention walls

Any ground settlement due to the installation of sheet piles, bored piles and diaphragm walls caused by vibration, soil deformation or loss of ground is in addition to settlement caused by the main excavation. The effects of vibration from sheet pile installation and recommended minimum distances from existing structures and services are summarised in reference 4.11.

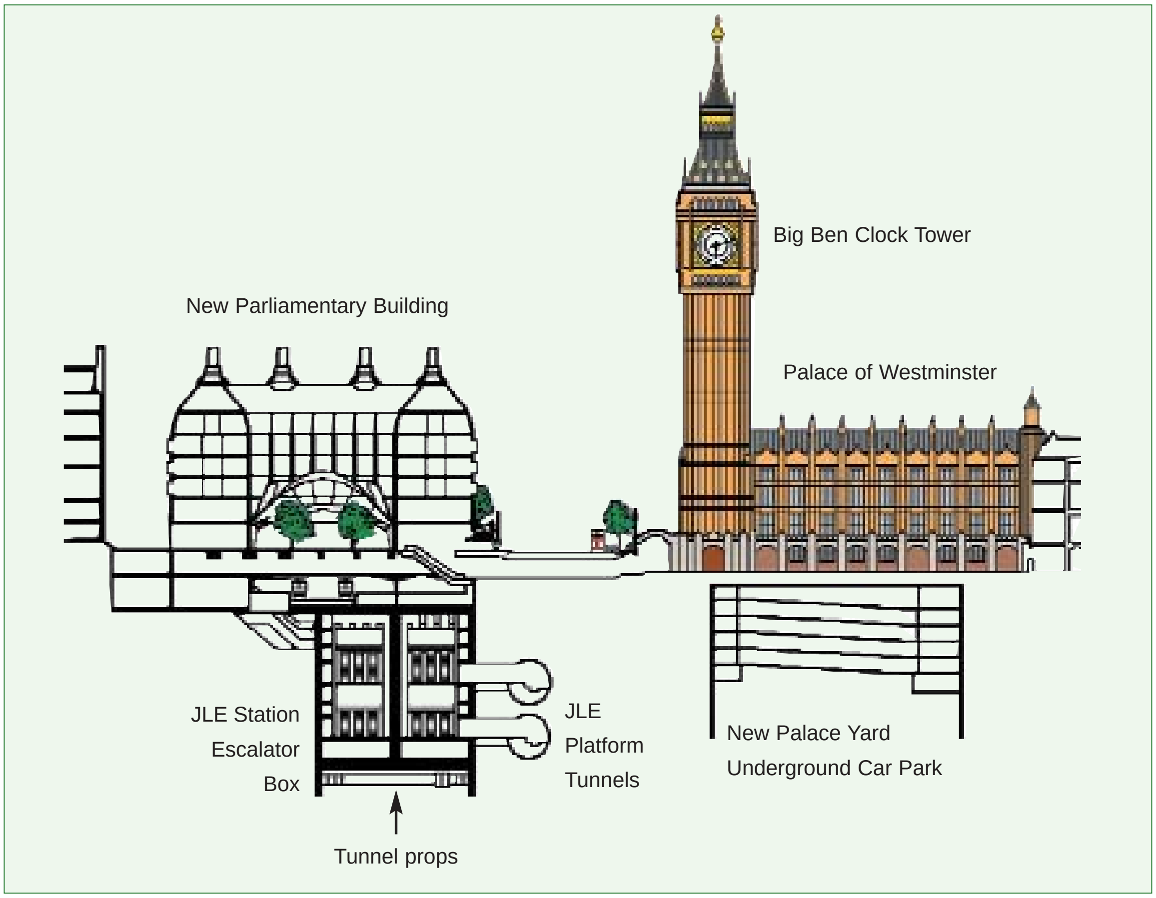

Fig 4.23 Cross section through Westminster Underground Station, London

© Mr David Harris, Geotechnical Consulting Group

Table 4.2 Tolerances for top down piles and plunged columns

Vertical support details	Setting out tolerance at ground level	Verticality tolerance
Cast-in-place pile	±75 mm	1 in 75
Cast-in-place pile with casing and a high degree of control	±25 mm	1 in 150
Plunge column	±75 mm	1 in 75
Plunge column with a high degree of control	±25 mm average ±10 mm optimum	1 in 250 average 1 in 400 optimum

4.5.4 Groundwater

Where excavation is made below the water table and peripheral walls do not achieve a cut-off into an impermeable stratum, dewatering may prove necessary within the excavation or from below it. The reduction in groundwater level may in turn cause local settlement through loss of fines or by consolidation over a wide area. In some instances, the effects of drawdown and the resulting settlement can be mitigated by recharge of groundwater outside the perimeter walls. The methods of dewatering and the calculation methods to predict drawdown are reviewed in references 4.13, 4.21 and 4.22 and in CIRIA report *Control of groundwater for temporary works*[4.23]. Groundwater control is discussed in more detail in Chapter 3.

Reference 4.24 gives an example of dewatering and recharge of a deep basement in Hong Kong. The five storeys were installed top-down through completely decomposed granite and below a groundwater table only 1.5m below ground. To reduce settlements due to dewatering, the basement box of diaphragm walls was constructed to rockhead 47-62m below ground level and curtain grouting was carried out to a depth of some 10m beneath the wall. Short-panel diaphragm wall construction, 4m long, was used to reduce settlements due to panel installation. To make the diaphragm wall box watertight, water bars were installed in the wall panels to a depth of 30m, with milled joints below this depth. A multiple well-pumping test carried out after completion of the diaphragm wall achieved drawdown in the box of 24-30m below ground level with only an average of 0.5m drawdown in standpipes outside the box. Recharge wells outside the box were not used during the pumping test but some recharge was subsequently used during basement excavation.

In deep permeable subsoil, groundwater flow into the soil floor of the basement box may cause instability and deep wells may be precluded because of excessive drawdown outside the box. In such circumstances, there are two possible counter measures. In the first, a horizontal grout blanket is injected within the soil below formation level before basement excavation; tension anchors or piles may be needed to hold this blanket down. The alternative is a concrete plug placed underwater by tremie[4.13,4.25,4.26].

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5 Design and analysis of retaining walls

5.1 Introduction

Design of retaining walls has traditionally been carried out using simplified analyses or empirical approaches. Methods have been developed for free-standing gravity walls, embedded cantilever walls (fixed earth support) or embedded walls with a single prop (free earth support). These are described in BS 8002[5.1] and CIRIA Report C580[5.2]. Because of their statically indeterminate nature, multiple propped walls have often been dealt with using empirical approaches[5.3,5.4].

Suitable factors of safety have been applied to cater for uncertainties about soil properties, to allow for the often-approximate nature of the calculation model and to ensure that retaining wall displacements are acceptable. Development of these factors has been based on experience, often as a result of trial and error.

The introduction of inexpensive but sophisticated computer hardware and software has led to considerable advances in the analysis and design of retaining walls. Much effort has gone into modelling the behaviour of walls in service and investigating the mechanisms of soil-structure interaction[5.5]. Although these newer methods have not replaced the traditional approaches for most routine projects, the ability to predict service loads and wall movements with some confidence has been revolutionary. In particular, they have advanced the understanding of wall behaviour and have enabled the major influences and key areas affecting the design of walls to be identified.

The following section on stability summarises how retaining walls and excavations may become unstable and move considerably. Wall displacements and forces, including earth pressures, bending moments and shear forces are discussed and a number of computer analysis methods are presented.

5.2 Stability considerations

5.2.1 Limiting earth pressures

The soil pressures to be resisted by an earth retaining structure very much depend on the magnitude of strains permissible in the ground. The pressures of the ground at active and passive failure define the lower and upper limits of these forces and related strains. The lower (active failure) or upper (passive failure) limits are reached when the soil is allowed respectively to extend or compress laterally to permit full mobilisation of the soil's shear strength. These two extremes are usually expressed by the coefficients of active and passive earth pressure, K_a and K_p respectively. These coefficients give the ratio between lateral and vertical effective pressures at active and passive failure. They are calculated from the soil strength, the angle of wall friction and the geometry of the wall and the soil surface[5.6].

5.2.2 Water pressures and the effects of seepage

The forces exerted by groundwater on a retaining wall are often greater than those from the soil. Careful consideration should therefore be given to variation of water levels and pressures on each side of the wall, both during construction and for the permanent structure.

Even more significant can be the effects of seepage of water around the base of the wall and into the basement area. This will tend to reduce water pressures below hydrostatic on the outside of the wall and increase water pressure above hydrostatic on the excavation side. The higher pressures inside will result in lower vertical effective stresses and in turn result in lower passive earth pressures. Thus, it is vital that seepage effects are properly accounted for in assessing stability and wall performance.

5.2.3 Gravity walls

The stability of gravity retaining walls is illustrated in Figure 5.1. Active pressures are assessed and applied to the retaining wall and, if appropriate, passive pressures are assumed in front of the wall. Water pressures are added commensurate with the drainage and seepage regime around the wall. The resulting force *R* is then calculated and stability is achieved if *R* can be resisted by the soil beneath the toe. The ability of the soil to resist the force is calculated using conventional bearing capacity considerations.

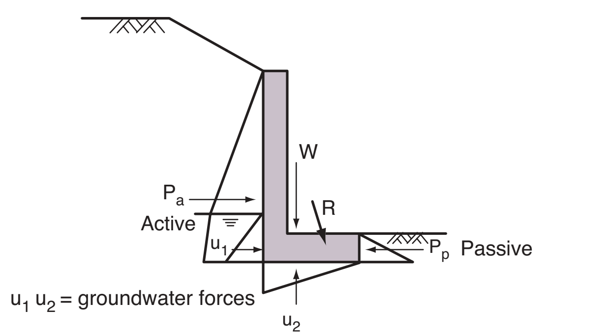


Fig 5.1 Typical gravity retaining wall

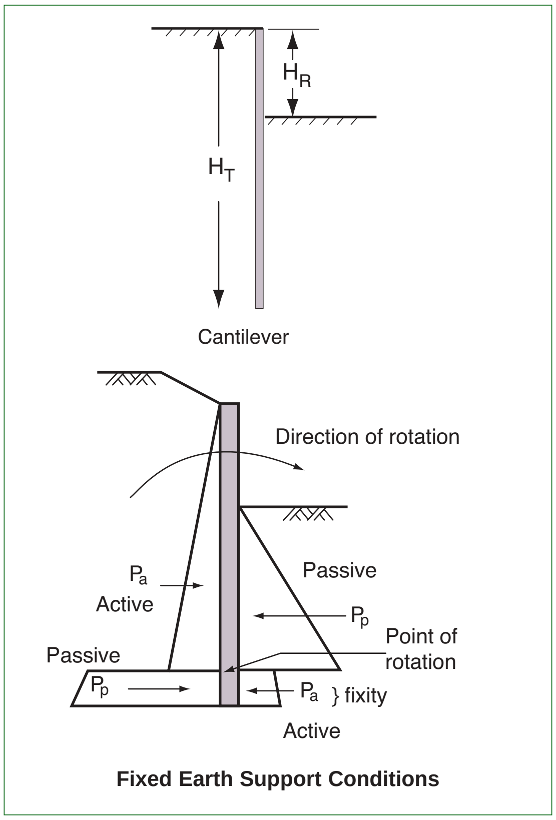


Fig 5.2 Cantilever wall stability

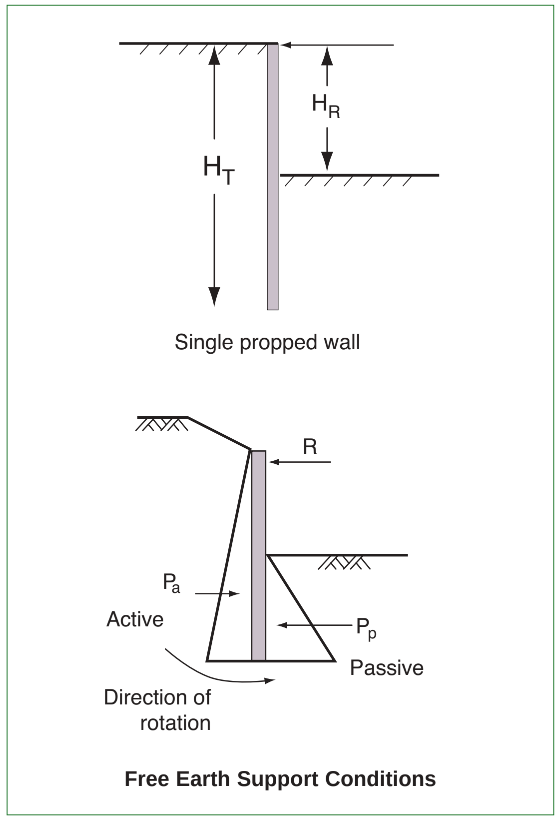


Fig 5.3 Stability of singly-propped walls

5.2.4 Cantilever walls

The stability of cantilever walls is illustrated in Figure 5.2. The mode of failure of the wall is by rotation about a point near the toe and the resulting active and passive pressures are shown in the Figure. This is a statically determinate system and, for any given active and passive pressure limits, there is only one depth of wall where a solution can be found[5.2].

5.2.5 Singly-propped walls

The stability of singly-propped walls is similar to cantilever walls and is illustrated in Figure 5.3. Here the failure is by rotation of wall about the prop level. Again, this is a statically determinate problem[5.2].

5.2.6 Multi-propped walls

Instability of the wall can arise in the cases of cantilever and singly-propped walls. Overall instability is unlikely to arise in the cases of multi-propped/anchored walls because of the redundant nature of the structure. However, local instability may arise as the result of local overstressing and the formation of a hinge where, for example, the multi-propped/anchored wall terminates in clay and where a void is left at the bottom of the excavation (see Figure 5.4). The amount by which the toe of a wall extends below excavation level may be due to a temporary works stability requirement or to limit seepage; see Chapter 3. As the clay softens, movement will occur towards the excavation, with soil moving into the void. The toe of the wall will be pushed in this direction and, if sufficient strength is not provided, the toe could be overstressed. Although this is unlikely to result in general instability, it is highly undesirable as it could allow water ingress and is almost certain to promote movement in the soil at the sides of the excavation. This could have detrimental effects on the foundations of any adjacent structures or on nearby services. There are no generally accepted methods for analysing such failure by way of hand calculations.

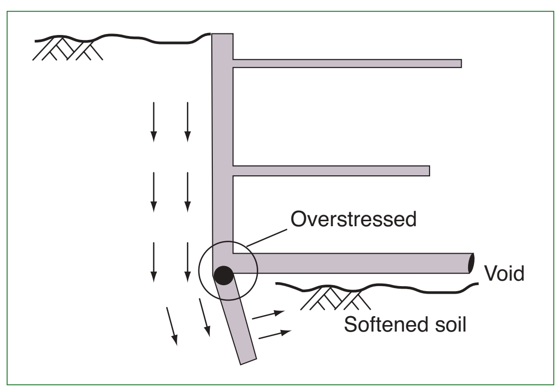


Fig 5.4 Multi-propped wall stability

Some of the approaches that are available to overcome such a problem are listed below;

- Increase the strength of the toe of the wall especially where it connects to the base slab. This may be expensive (see Figure 5.5a).
- Do away with the void under the base slab. This will result in a build-up of pressure on the base slab, which must be accounted for in the slab design (see Figure 5.5b).
- Increase the vertical effective stress in the soil immediately in front of the toe of the wall. This can be achieved by installing pin piles[5.7] or by using a partial soil-bearing base slab (see Figures 5.5c and 5.5d). Extend walls deeper into stronger soil if such soil is present (see Figure 5.5e).

5.2.7 Circular basements

In some circumstances, the circular basement plan can provide an economical solution, obtaining the benefit of a circular structure with induced hoop compressive stresses by radial ground and groundwater pressures. Primarily, the plan geometry of the required substructure must be such that it fits within the circular plan without excessive waste of space. In addition, however, uniform hoop compression will occur only where ground levels are flat and ground and groundwater conditions are uniform around the plan shape.

Reinforced concrete piles and diaphragm walling are both used in circular basements. If used, each pile is designed to span vertically between circular walings or internal lining walls. A large circular basement (or cofferdam) using secant piles was used for a large circular excavation at Heathrow airport, London[5.8]. A circular piled cofferdam, 60m in diameter and 30m deep, was installed through disturbed ground following a tunnel collapse. An internal continuous reinforced concrete lining, cast progressively with excavation, supported the piles, with the lining acting in hoop compression.

When diaphragm walls are used in circular plan shapes, a segmental plan results from the use of straight panels. In some instances, these walls are designed to span vertically between circular walings while elsewhere the walings are dispensed with and the walls themselves allowed to act in hoop compression. One of the largest circular diaphragm walls to date was built for the basement of the new world library, the Bibliotheca in Alexandria, Egypt[5.9], a cross-section through which is shown in Figure 5.6. The diaphragm wall was 160m in diameter and 35m deep. It was designed to resist earth and groundwater pressures and seismic forces. Continuity of reinforcement was provided through vertical joints in the diaphragm wall to ensure development of hoop stresses.

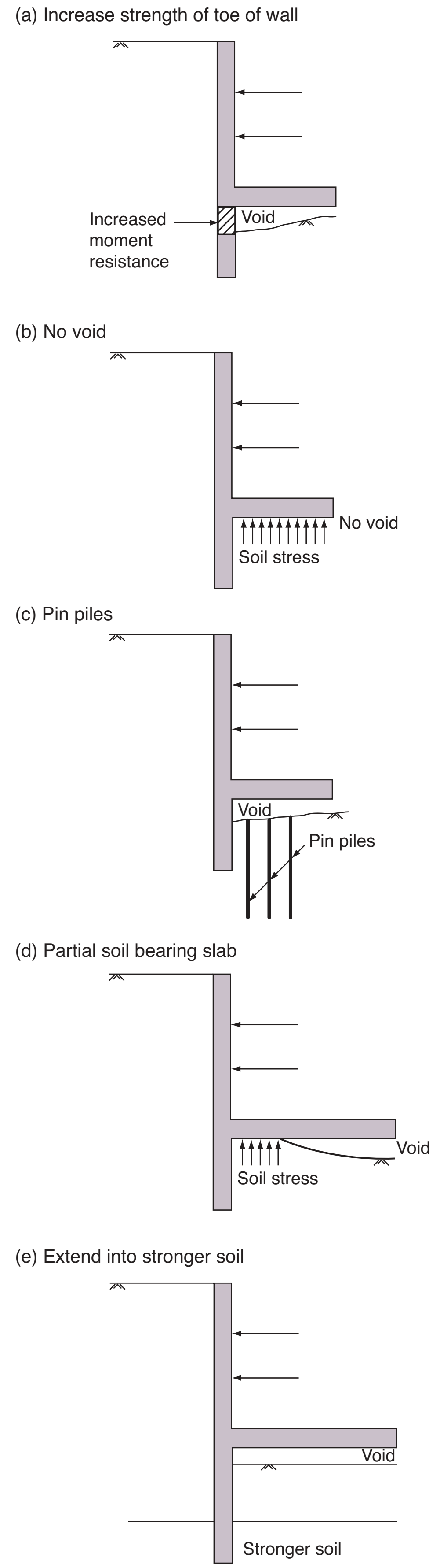


Fig 5.5 Methods for dealing with potential toe instability

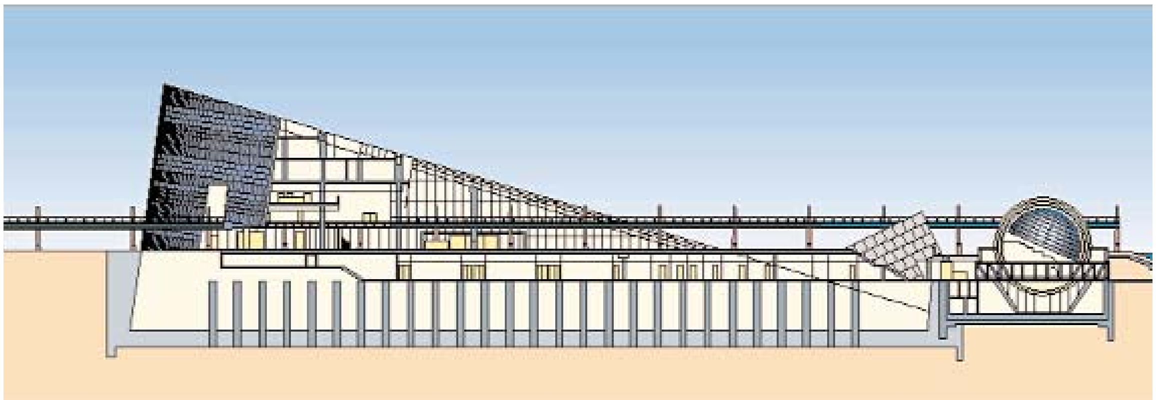


Fig 5.6 Cross section through the basement of the Bibliotheca in Alexandria, Egypt[5.9] © Hamza Associates

5.2.8 Factors of safety

At present, there are several ways in which the factor of safety can be applied for wall stability. Some engineers adopt partial factors, others factor soil strength or embedment depth or use a lumped factor applied to some combination of the active and passive earth pressures[5.10-5.12]. It is important to note that, to arrive at the same wall design, each approach requires a different numerical factor of safety. Care must therefore be taken to ensure that any factor of safety is consistent with the approach adopted. The only consistent approach for all types of wall is the use of a factor on soil strength. This is the approach adopted by CIRIA Report C580[5.2] which also offers guidance on the appropriate choice of strength parameters. The factored soil strength, or 'allowable mobilised strength', can be used consistently in calculations of both bearing capacity and wall stability. Factors of safety can be increased to a magnitude sufficient to limit movements to an acceptable level, the values being based on experience. This method should be used only when displacements are not a critical consideration.

As noted in Chapter 2, determining soil and wall movements is difficult and is likely to remain only approximate until further numerical analyses are calibrated against field experience. Consequently, the recommended factors of safety used for stability analyses are often large enough to limit movements to an acceptable level, the values being based on experience.

5.2.9 Temporary works design

During the early stages of construction, temporary props/anchors may be used to support a wall. Later, these may be replaced by permanent support in the form of floor slabs that are an integral part of the finished structure. Any basement may therefore be subjected to two different sets of loading and support conditions, namely that occurring during construction (temporary works) and that in the finished state (permanent works). As conditions may be very different in the two situations, both must be considered carefully.

Soil behaviour can be time-dependent. This is particularly true for clay soils, which have different characteristics under short-term (undrained) and long-term (drained) loading. It is not a problem for sands and gravels, which usually behave in a drained manner except under dynamic loading. Depending on the past stress history of the clay, whether normally or over-consolidated, and the nature of the construction, the long-term strength may be higher or lower than the short-term. For typical deep basement construction, the soil is likely to be weaker in the long term. While it is possible to estimate long- and short-term soil strength, it is much more difficult to predict the length of time for the strength to reduce from one to the other.

This does not greatly affect permanent works design, which is usually based on long-term conditions, but can give rise to problems for temporary works. In such cases, the likely reduction in soil strength while the temporary works exist should be estimated. This will depend on the soil and groundwater conditions and in particular on any fissures and silt or sand partings in the clay. This can be a sensitive issue because any underestimate of the strength reduction could lead to an unsafe situation, while an overestimate may lead to expensive (and unnecessary) temporary support requirements.

It is recommended that, unless there are good reasons to the contrary, analyses should be undertaken to show that the factor of safety using effective strength calculations based on long-term conditions is greater than one. This is particularly relevant if circumstances might lead to a temporary stage of excavation being delayed beyond the anticipated period. Further guidance on temporary works design is given in CIRIA Report C580[5.2].

5.2.10 Base heave failure

Two types of instability of an excavation base can be identified. The first arises as a result of excess porewater pressures in underlying soil layers. For example, if a thin layer of clay overlies sand or gravel with a sufficiently high porewater pressure, the clay can be forced into the excavation base. This may happen because the retaining wall does not provide an effective cut-off to high water pressures outside the site. In sands, on the other hand, retaining walls may not be deep enough to reduce the water pressures arising because of seepage around the base of a wall to acceptable levels, and a piping failure may result. This type of instability is discussed in Chapters 2 and 3.

The second type of instability arises if the soil at the base is not strong enough to support the stresses imposed by the soil adjacent to the excavation. In this case, soil is again forced into the base of the excavation, causing large movements in the adjacent soil. This second type of instability is discussed below.

The process of digging an excavation can be compared with that of loading a foundation; in the latter, if too much load is applied, the soil will become over-stressed and fail. Likewise, with an excavation, if too much soil is removed, failure may occur as soil flows in to the base of the excavation (see Figure 5.7). This is known as base heave failure. As with a loaded foundation, the controlling factor will be the soil strength and, because the excavation is rapid, it is the short-term soil strength that is usually relevant. The most widely used method of calculating the possibility of such instability is similar to that of determining the bearing capacity of foundations[5.13].

Base heave failure is particularly relevant to excavations in soft clay. Often, it may only be possible to excavate a few metres of soil before base heave failure. Some of the approaches used to extend the excavation depth in such soils are:

- Extend the side supports deeper below excavation level. This only works if the supports penetrate more competent soils at a greater depth (see Figure 5.8a).
- Dig the excavation as a series of smaller excavations with a reduced plan area. The rationale behind this is again based on the foundation analogy, in that reducing the aspect ratio of the plan area is likely to increase the bearing capacity factors (see Figure 5.8b).
- Excavate under water or bentonite mud. This involves flooding the excavation and is often undesirable (see Figure 5.8c).
- Increase soil strength before excavation by freezing, grouting or in-situ mixing (see Figure 5.8d).
- Reduce effective excavation depth by removing soil adjacent to main excavation. This is usually only resorted to in an emergency (see Figure 5.8e).

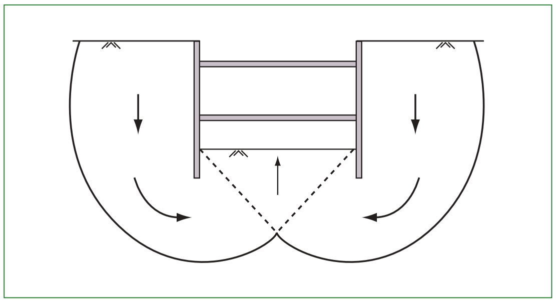


Fig 5.7 Base heave failure

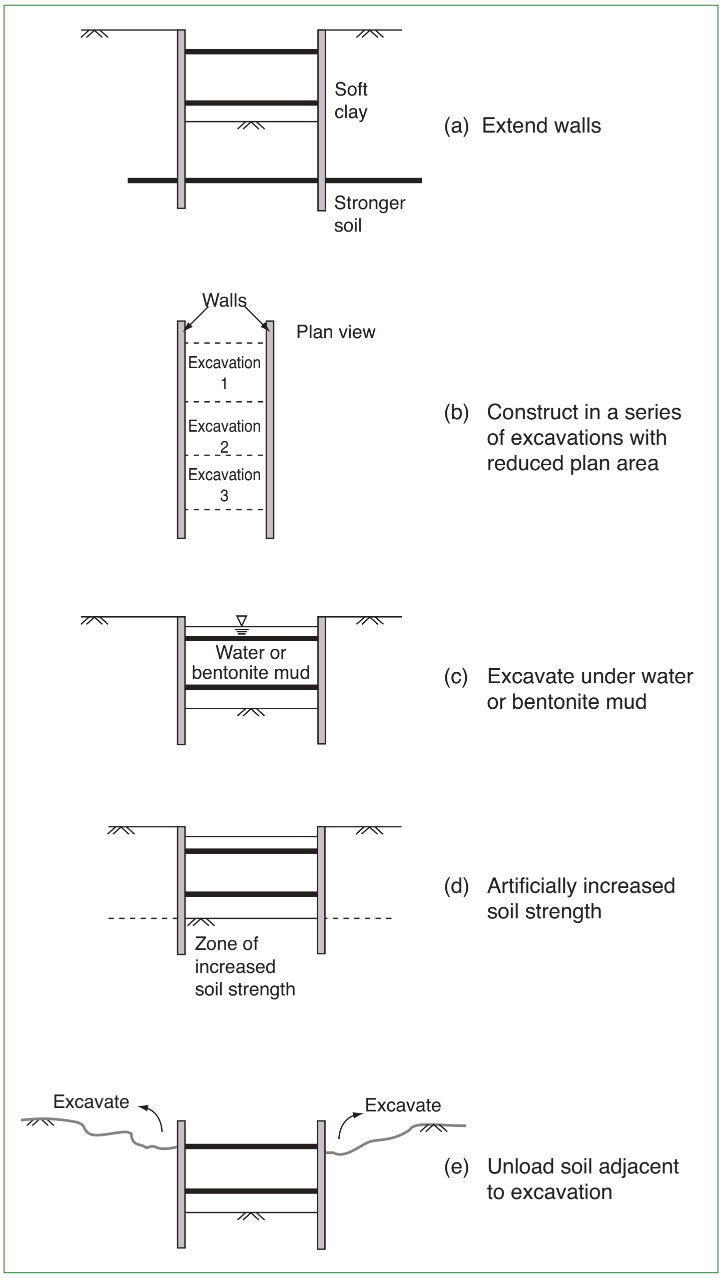


Fig 5.8 Methods of extending excavation depth in soft clay

Some of these approaches require specialist expertise and can be expensive. It is therefore important that the potential for base heave failure be identified early in the design to enable an appropriate selection of support and construction sequence. If such problems are not identified until after the design has been finalised or, even worse, after construction has begun, remedial measures could be at best expensive and in some cases impossible without a complete redesign. In this respect, sufficient attention must be given to temporary works early in the design process.

For stiff clays, the soil strength is usually much greater and increases with depth. Base heave failure is therefore not usually a problem. In the same context, there are also few problems with excavations in sand.

5.3 Earth pressures

In designing a retaining structure to support the sides of an excavation, the magnitude and distribution of the stresses and movements likely to be induced in the structural components must be estimated. Both the temporary and permanent works stages of construction should be considered. These end pressures will depend on the initial in-situ soil stresses, the wall construction method and perhaps its stiffness, and the number and stiffness of supports.

5.3.1 Backfilled walls

With gravity retaining walls, if soil is backfilled behind the wall, the compaction process will induce both transient and residual horizontal pressures on the wall. The amount of these pressures depends on the type of fill, state of compaction and flexibility of the wall and its supports. Simplified methods have been published for assessing earth pressures during and on completion of backfilling for stiff walls[5.14-5.15] and more flexible walls[5.16].

5.3.2 Initial in-situ earth pressures and the coefficient of earth pressure at rest K_o

In its initial natural state, before any construction, the horizontal effective stresses in the ground will be somewhere between those associated with active failure and those associated with passive failure. The coefficient of earth pressure at rest, K_o, is defined as the ratio between the horizontal and vertical effective stresses in this initial condition.

The magnitude and variation with depth of the initial 'at rest' horizontal effective stresses depend on the loading history of the soil. Figure 5.9 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 K_o has a constant value K_{onc} given by the expression $K_{onc} = 1 - \sin\varphi'$ derived from the work of Jaky[5.17]. During erosion of the overlying deposits, unloading takes place and the effective stresses follow the unloading path A to B, shown in Figure 5.9. It is evident that, as unloading takes place along AB, K_o increases towards the passive earth pressure coefficient K_p.

Estimates of K_o in such deposits can be obtained from a knowledge of the over-consolidation ratio (OCR) using the expression $K_o = K_{onc}\,(OCR)^{\sin\varphi'}$. This equation[5.18] is not applicable if the deposit has been subsequently reloaded (e.g. deposition of surface gravels, as in many parts of London), as the effective stresses then follow the path BC and tend towards the initial loading path.

From the above, it is clear that, for a given soil deposit, K_o can vary from location to location depending on the stress history at each. Its value must lie between K_a and K_p and its relative position between these limits will govern the amount of movement required to mobilise either (see Figure 5.10). For example, in a normally consolidated soil, K_o is only slightly larger than

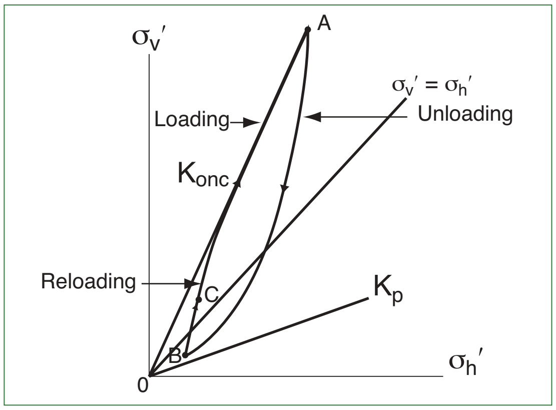


Fig 5.9 Effect of stress history on over-consolidated clays

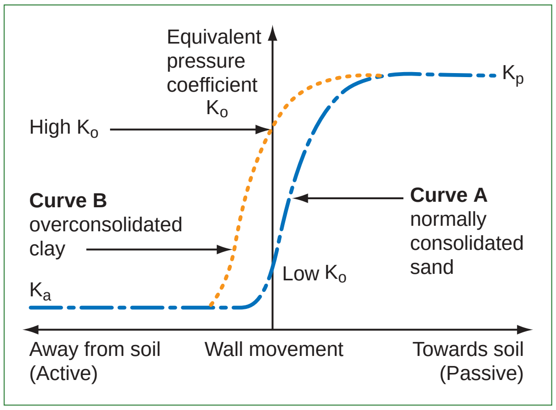


Fig 5.10 Relationship between earth pressure coefficient, K, and wall movement

K_a. Little horizontal movement will therefore be necessary to mobilise active earth pressure conditions, whereas significant movements will be needed to mobilise passive conditions. For the same soil in a heavily over-consolidated condition, K_o is much larger, being only slightly less than K_p. Large movements will then be required to mobilise active earth pressure conditions and much smaller movements to mobilise passive conditions.

5.3.3 Effects of wall and prop stiffness

For embedded walls, the stiffness of a support system can have a significant effect on the magnitudes of the pressures to be resisted and the ground movements. Figure 5.11 illustrates the effects of changing the wall stiffness on the earth pressure and movements for a singly-propped wall where the prop is infinitely stiff. In this case, as the wall stiffness reduces, movements increase and redistribute the earth pressures. This redistribution, which reduces earth pressure behind the central portion of the wall and increases it at the top of the wall behind the prop, is a direct result of the increasing wall movement. The earth pressure redistribution in turn leads to a substantial reduction in wall bending moments but at the expense of increased movements.

The effects of the wall and prop stiffnesses on bending moments and movements depend very much on the propping and excavation sequences. In a typical multi-propping wall, it is found that once the wall is stiff enough the soil will tend to move by a similar amount regardless of how stiff the wall itself becomes. In this instance, stiffening the wall tends to increase the bending moments rather than reduce movements.

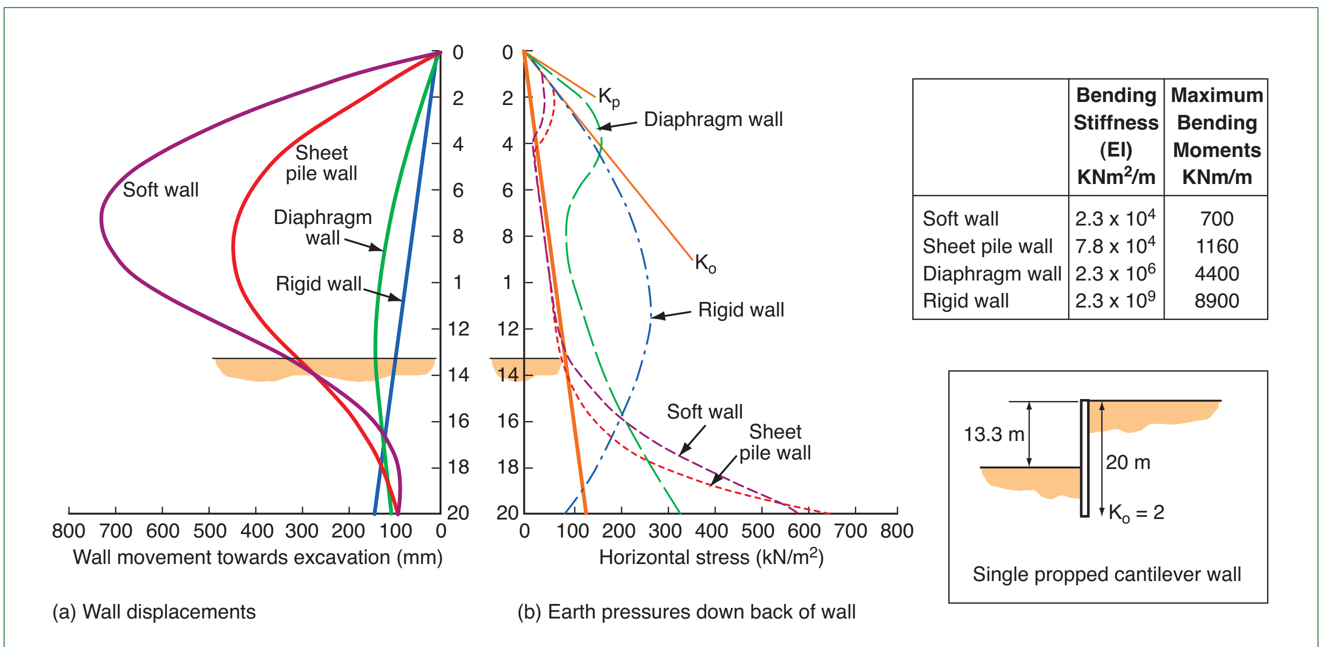
5.3.4 Design earth pressures

The simplest approach is to make use of the wall stability calculations to obtain values of maximum thrusts, shear forces and bending moments. For single propped embedded cantilever walls, the calculated bending moment is often reduced using a bending moment reduction factor[5.12,5.19]. This factor varies with wall stiffness and is based on observations from laboratory model tests[5.20]. For stiff walls, such as concrete diaphragm or secant pile walls, installed in stiff clay (high K_o), such simple methods may severely underestimate the likely structural stresses[5.21,5.22]. Empirically derived soil pressure distributions are available for multi-propped situations; these can be used to estimate structural loads[5.3,5.23].

Computer programs for estimating earth pressures are discussed in Section 5.5.

5.4 Design of wall members

For situations where temporary works enable traditional construction techniques to be adopted, only the long-term or permanent conditions need be checked. However, for many basements the wall structural member is also used to provide temporary support to the excavation during construction. Often, loading conditions between the temporary and permanent situations differ and both cases must be considered.



	Bending Stiffness (EI) KNm²/m	Maximum Bending Moments KNm/m
Soft wall	2.3 x 10⁴	700
Sheet pile wall	7.8 x 10⁴	1160
Diaphragm wall	2.3 x 10⁶	4400
Rigid wall	2.3 x 10⁹	8900

Fig 5.11 The results of numerical analysis illustrating the influence of wall stiffness on displacement and earth pressure

5.4.1 Applied wall forces

Design wall forces are derived from the following:

- Soil and groundwater pressures acting behind and in front of the wall together with surcharge pressures due to adjacent buildings or roads.
- Reactions from the support systems, both temporary and permanent. These forces may also give rise within the wall member to axial compressions or tensions due to inclined anchors or struts. CIRIA Report C517[5.4] provides design guidance for temporary props based on extensive field measurements of prop loads for flexible and stiff walls and for a wide range of ground conditions.
- Abnormal loadings, particularly from higher groundwater levels caused by flooding or water-filled tension cracks, or construction surcharges. These will often be brief and can be considered as total stress loads. For longer-term conditions, it is only necessary to consider average or ambient loading conditions.
- Building loads such as floor slabs and columns. These forces may be eccentric to the wall and generate bending moments.

5.4.2 Bending moments and shear forces

BS 8110[5.24] and BS 5400[5.25] require partial factors to be applied to working loads of 1.4 and 1.5 respectively to obtain ULS values. BS 8110 states that this factor can be reduced if the loads are derived from an elastic analysis. If soil strengths are factored in order to derive the loads[5.26], in accordance with the recommendations of CIRIA 580[5.2] and BS 8002[5.1], the requirements in the structural codes are not appropriate, since the loads have already been factored. This somewhat confusing situation is discussed more fully in Section A8.2.7 of CIRIA Report 580[5.2].

BS 8002[5.1] suggests that soil structure interaction calculations, modelling the SLS, can also be used to estimate the structural loads, and implies that these loads are not factored to provide a ULS value. CIRIA 580[5.2] however, recommends that the SLS values are multiplied by 1.35. It also recommends that both SLS and limit equilibrium calculations are carried out, and that the value used for the ULS structural calculation is the greater of the two values:

- the values derived using the factored soil strengths
- the SLS values multiplied by 1.35.

5.4.3 Wall movements and cracking

When subjected to the complex loading from soil, groundwater and structure, the wall structural member will, to a greater or lesser extent, deform. As discussed in Section 5.5.3, the wall stiffness often has little influence on the total deformations, which are governed primarily by soil conditions, the method and sequence of construction and the wall support system. This is discussed in Chapter 2. Little can be done to prevent the wall member from cracking. However, as it is often the primary defence against groundwater ingress, consideration must be given to controlling this cracking. Guidance is given in BS 8007[5.27]. Long-term durability also depends on the severity of cracking. These aspects are considered in detail in Chapter 11.

5.5 Computer programs for designing retaining walls

Any analysis involves simplifications and idealisations. An appropriate analysis for a particular problem is one that adequately models the dominant effects without being over complex. One of the dangers of computer programs is that they are easy to use without the user necessarily having an understanding of the principles and idealisations on which they are based. In the following sections, computer programs for the analysis of retaining walls are described briefly. It is important that, before using any of them, the engineer should understand the principles on which they are based and their limitations. Often quite simple programs are adequate for analysing bending moments and shear forces in a wall. Such programs are likely to be completely inadequate for modelling ground movements around the retaining wall.

5.5.1 Limit equilibrium programs

These programs are based on the simplest form of analysis. A limiting condition is assumed and equilibrium applied to obtain a solution. Programs are available to analyse gravity walls, embedded cantilever and singly-propped walls. Active and passive soil conditions are usually assumed and various types of factor of safety can be introduced. For multi-propped situations, empirically derived soil pressure distributions are sometimes employed. These programs are best used to obtain basic wall dimensions such as wall embedment. Although they can estimate structural loads under working conditions, they are only approximate and not reliable for the complex situations usually found in deep basements; their use for this purpose is therefore not recommended. Such programs do not account for soil-structure interaction and cannot estimate wall and/or soil movements.

When considering the stability of gravity retaining walls or anchored embedded walls, it may be necessary to calculate slope stability. Several programs for such analysis are available. Generally, they allow for both circular and non-circular slip surfaces and use factored, or mobilised, soil strength. Again, they are based on limit equilibrium assumptions.

It must be emphasised that, where possible, all of the above calculation methods should be calibrated against case histories. With all methods, many assumptions are required for the input parameters and even finite-element analyses cannot be relied upon to give sensible results unless some calibration is done. For the simpler methods, calibration becomes more important because they are more limited than the finite-element method in their ability to extrapolate from a situation where the results are known to other situations.

5.5.2 Beam-on-spring model

For embedded walls, a more realistic estimate is often needed, and the calculation should take soil-structure interaction into account. The simplest of these represent the wall as a structural member usually employing a finite-difference or finite-element approximation, with the soil as a series of unconnected springs. The construction sequence is simulated by adding and subtracting loads from the wall. Both structural stresses and wall movements are calculated. While such programs represent a significant improvement over the simpler limit equilibrium approaches, they still have severe limitations. For example:

- It is difficult to select appropriate spring stiffnesses to represent the soil.
- By representing the soil by a set of independent springs, it is difficult to reproduce the observed stress redistribution arising from wall flexibility.
- They generally do not allow for the influence of the release in vertical stress caused by the process of excavation. Deep-seated movements arising from this process are not included in the analyses.
- It is difficult to include the effects of any soil berms.
- Only the wall movements are calculated, making it difficult to estimate the movements of adjacent structures.

5.5.3 Boundary element programs

In these programs, the soil to each side of the wall is represented by a boundary element, as in reference 5.28. These programs overcome most of the difficulties listed above apart from the estimation of the movements of adjacent structures. They also involve many assumptions and simplifying idealisations and, while they can give a good understanding of how the overall system behaves and which parameters are likely to control the designs, they may not give realistic displacement predictions.

5.5.4 Full numerical analysis

These programs are usually based on the finite-element method and, while it is in principle possible to analyse the complete three-dimensional construction process from temporary to permanent works, current limitations on computing resources usually restrict analyses to two-dimensional plane strain or axi-symmetric sections. With such an approach, it is possible to simulate the construction process and include all significant structural members. Stresses, strains and movements both in the soil and the structure can be predicted. The effects on adjacent structures such as tunnels, sewers and buildings can also be assessed. However, the method is more expensive than the simpler approaches and requires detailed information on soil properties, etc. Over the past decade, full finite-element analyses have become more widely used, especially for some larger deep-basement projects in London[5.22, 5.29-5.31].

A recommended compromise approach for design purposes is to carry out a limited number of full numerical analyses in combination with simpler calculations. The full analyses are used to calibrate the beam-on-spring approach, which is then used to assess the effects of design modifications. Once the design is finalised, it may be necessary to carry out a few additional full numerical analyses.

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6 Foundations

6.1 Introduction

BS 8004[6.1] gives guidance on the design and construction of foundations. Eurocode 7 (EC7)[6.2], currently in draft, contains a more formalised approach to foundations, including those in basements. In this Chapter, emphasis is placed on matters of special significance to foundations in deep basements.

6.2 Loads

The foundations in a deep basement will frequently be used to carry the temporary loads during excavation and construction as well as the permanent loads after completion. The loads during construction may be different in type from the permanent loads and will sometimes dictate foundation design. Therefore, foundations in deep basements should be designed to accommodate the various situations that might occur during construction and use of the structure and must not be considered in isolation from the construction method.

The loads applied to the foundations of deep basements will often have horizontal as well as vertical components. Vertical components are generally derived from the weight of the structure within and above the basement, together with the contents. Horizontal components of load usually result from the earth pressures applied through the retaining walls. These are sometimes temporary, occurring only during construction (see Figure 6.1), but may be permanent, especially on sloping sites where earth pressures are not balanced across opposite sides of the basement (see Figure 6.2).

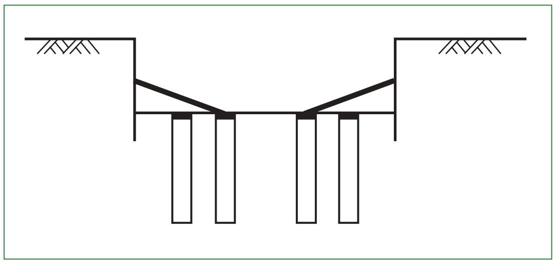

Fig 6.1 Temporary horizontal loads from props

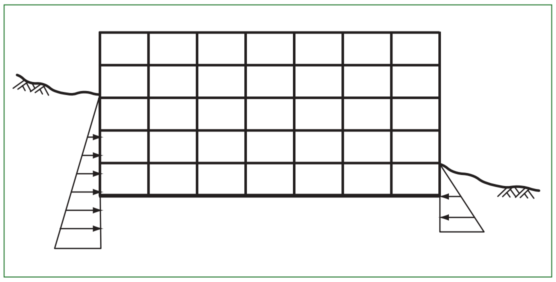

Fig 6.2 Permanent horizontal loads from earth pressure on sloping site

Loading due to water pressures can be critical and is discussed in more detail in the next section. Such pressures must be balanced by the weight of the structure, and critical conditions may occur during construction before the full weight of the structure has been developed.

6.3 Water pressures

A proper understanding of water pressures and possible changes around and beneath a basement is crucial to its design. Changes of water pressure in the soil affect its state of stress and hence its strength on which the safety and stability of the foundations depend. Swelling or consolidation may also be caused, leading to movement of the foundations.

In clay soils, removing overburden during excavation often temporally reduces the water pressure beneath a basement. Over a period of time, which depends on the bulk permeability, water may be drawn into the clay and water pressures will rise again.

Upward flow of water into the basement may significantly reduce the effective stresses. Three possible situations are shown in Figure 6.3. Soils containing permeable layers require special care, as lateral seepage of water beneath the excavation may cause uplift or disturbance to the overlying less-permeable material.

Water pressures in the ground may have been lowered by exploitation of an aquifer for water supply. In some cities, use of aquifers has been much reduced in recent years and water pressures are therefore recovering. A report prepared by CIRIA on the situation in central London[6.3] lists other major cities where this phenomenon has been noted.

To reduce water pressures, a drainage blanket may be constructed beneath a basement. Where there is a possibility of unacceptably high water pressures at depth, it is sometimes appropriate to incorporate relief wells in the design. These may be either a construction expedient or may have a permanent function, in which case provision for maintenance should be made. It is important to check that wells

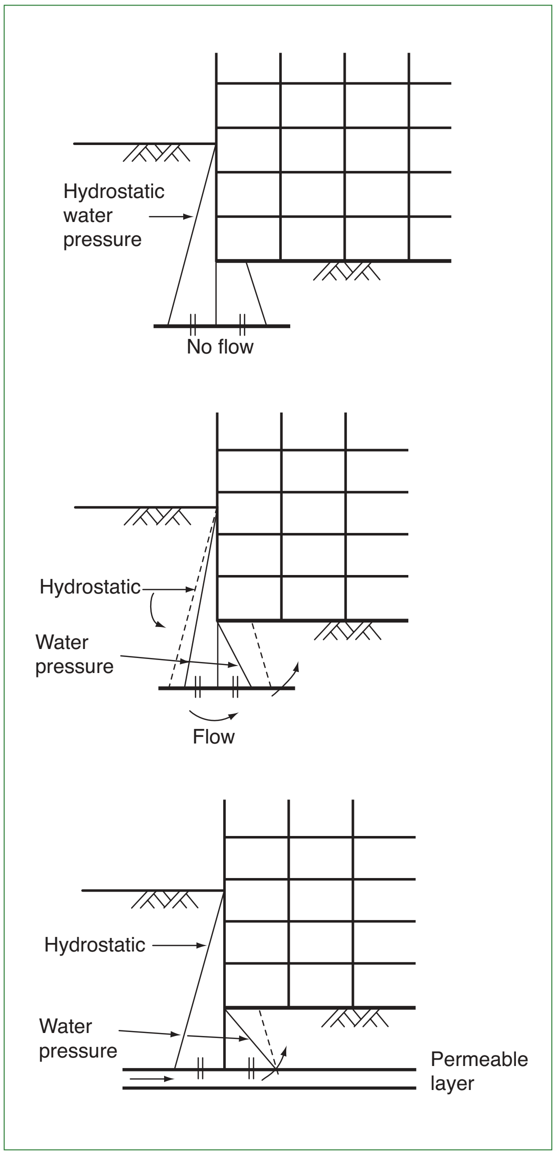

Fig 6.3 Examples of reduction in effective stress due to water flow

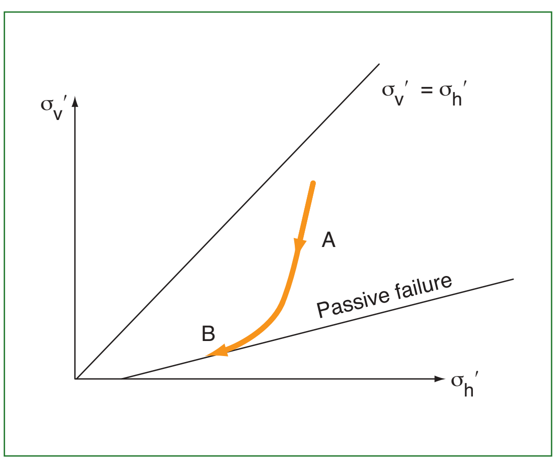

Fig 6.4 Effective stress path for a soil element as progressive excavation takes place

constructed for the benefit of one situation do not create a problem elsewhere. Design and maintenance requirements for wells are considered in Chapter 3. There is a danger of gas (methane or deoxidised air) from relief wells in basements and the more general problems of gas are discussed in Chapter 7.

6.3.1 Buoyancy and flooding

The basement must not float: the factored downward forces must exceed the factored buoyancy forces. Specifications from experienced underground railway clients such as Mass Transit Railway Corporations in Hong Kong, Singapore and London include clauses formulated on the lines shown in Table 6.1 (with specified minimum material densities tabulated appropriate to the site's location).

These authorities also, prudently, require the threshold to their underground basements (stations) to be not less than 1.0m above local ground level, to guard against flooding of their entire network from one source of water ingress: no apertures below this level are permitted.

6.3.2 Water pressure on foundations

In addition to countering buoyancy, foundations should be designed for the most onerous of suitable load cases, 'normalised' by dividing by an appropriate allowable overstress factor, see for example Table 6.2.

In this example, normal groundwater level is assumed to be 2m below finished ground level, and construction loading (load case 5) includes the (semi-top-down open) roof slab with 10kPa construction surcharge. Tension in the foundations may arise from some load cases.

6.4 The influence of excavation on strength and bearing capacity

The ability of a soil to sustain loads without unacceptable displacements depends on its effective strength parameters c′ and φ′ and the effective stresses acting on it. The ground beneath a basement may be subject to some important changes of effective stress, which will affect the performance of both shallow and deep foundations (see Section 1.3.1).

The construction of a basement usually involves removing overburden pressure, some of which may subsequently be replaced by the weight of the structure. If the effective stress in the ground is reduced, its strength will also decrease, but this effect may be delayed in clay soils. The immediate effect of removing overburden pressure is to reduce the vertical total stress and, as drainage occurs, this will eventually reduce vertical effective stress.

Table 6.1 Example partial factors of safety for buoyancy calculation

	Downward forces *D*		Upward forces *U*
Condition	Partial factor of safety on weights (γ_m)	Partial factor of safety on friction (γ_m) (i.e. on sides, piles, anchors)	Partial factor of safety either on water density or on displacement (γ_f)
During construction	Steel 1.00 Concrete 1.03	2.0	1.01
In service	Steel 1.03 Concrete 1.05	3.0	1.05
Extreme event (flooding to 1m above ground level)	Steel 1.01 Concrete 1.04	2.5	1.03
Criterion (for each condition)	$\Sigma(D/\gamma_m) > \Sigma(U^*\gamma_f)$		

Note: The 2% difference between the densities of fresh and sea water should be noted. If the centre of (factored) buoyancy does not reasonably closely correspond in plan to the centre of (factored) gravity, the eccentricity should be accounted for.

Table 6.2 Example allowable overstress factors for a 20m deep basement

Load case	Design condition	Allowable overstress factor
1	Flood water at normal groundwater level +4m	1.0
2	Groundwater at normal groundwater level -1m	1.0
3	Groundwater at normal groundwater level -6m	1.25
4	Groundwater below underside of lowest base slab	1.4
5	During construction	1.0

The resulting changes in horizontal effective stress are more complicated and are illustrated in Figure 6.4 which shows the effective stress path followed by an element of soil near the toe of an embedded retaining wall as excavation takes place under drained conditions. Initially, as overburden is removed, the vertical effective stress reduces more rapidly than the horizontal effective stress and the effective stress path approaches the passive failure line. Once the passive stress limit is reached, further reductions in vertical effective stress result in much larger reduction in horizontal effective stress as the effective stress path moves along the passive failure line towards the origin. During this stage there may be a significant reduction in strength. Recent research on stiff clays has indicated that, even in this situation, the clay may retain more strength than was previously expected[6.4]. There is, however, some evidence that this effect may be diminished if fluctuating loads or water pressures disturb the ground.

In the presence of high water pressures, the effective stresses beneath an excavation may reduce significantly. In the CIRIA study of central London[6.3], for example, it is suggested that around the base of piles in the situation shown in Figure 6.5 the effective stresses could become negligibly small. If it occurred, it would lead to an almost total loss of bearing capacity. But this is an extreme case and there is evidence that a substantial reduction in overburden pressure (e.g. 50%) may lead to quite small changes in the ability of the ground to carry load from piles[6.5-6.6].

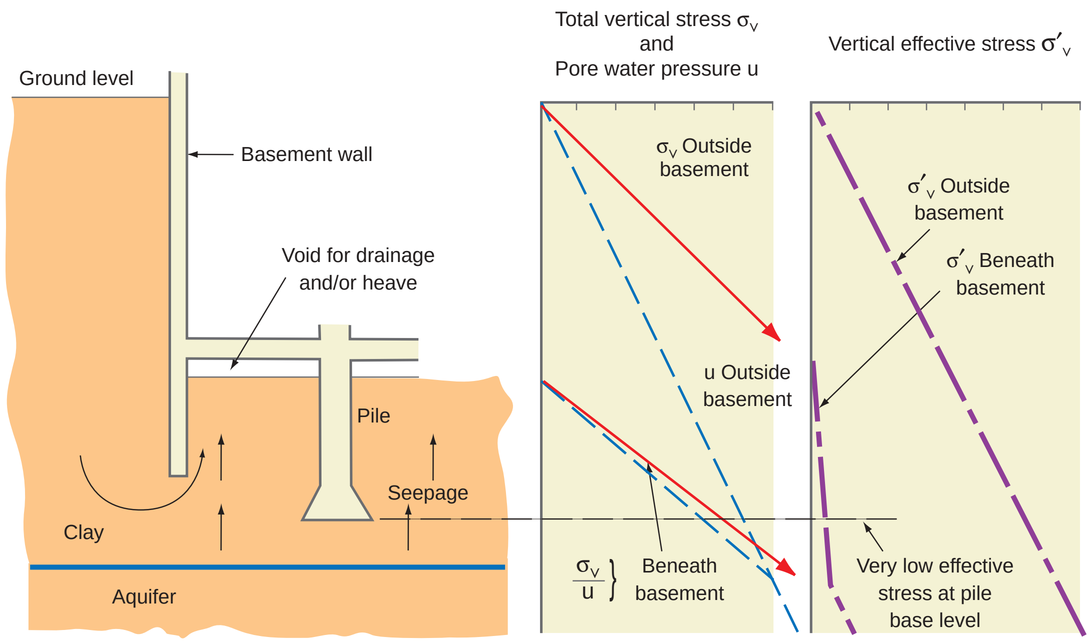

Fig 6.5 Influence of a rise of groundwater level on the vertical effective stresses beneath a basement

6.5 Ground movements

Chapter 2 contains a detailed description of the ground movements that can take place beneath and around basement excavations. Foundations should be designed to accommodate the following vertical movements:

- Immediate heave caused by undrained distortion of the clay, and possibly some swelling, as the basement is excavated. This may cause extension of piles constructed before excavation, but probably occurs before the construction of spread footings.
- Immediate settlement caused by the gross weight of the structure during construction and the net weight after dewatering ceases.
- A combination of the heave caused by excavation and settlement caused by loading, which takes place gradually as water enters or leaves the clay.

Generally, spread foundations only experience the last two of the above.

In addition to the effects of swelling, foundations may be affected by horizontal and vertical movements caused by shear distortion of the soils. These are particularly significant near the perimeter of excavations where they are caused by the difference in overburden pressure within and outside the excavation. Placing additional loads next to the excavation may add to this effect.

In soft clays, severe distortions may take place beneath the excavation during construction if the depth of excavation exceeds about $4c_u/\gamma$ where c_u is the undrained shear strength and γ is the unit weight of the soil (see Section 2.2.2). In sands and stiffer clays, this is unlikely to be a problem as long as there is an adequate margin of safety against instability in the long term, when the clay beneath the excavation may become softer due to unloading. Nevertheless, even in stiff clays, movement of foundations and distortion of piles may merit careful analysis during design.

6.6 Spread foundations

The design of all spread foundations should take account of the strength of the ground, and hence the bearing capacity of the foundation, and likely ground movements. As pointed out above, it is particularly important in basements to appreciate that the strength of the ground may decrease when overburden is removed. Moreover, ground movements may include heave caused by swelling. In clay soils, these may happen some time after the foundations and structure have been constructed. When calculating the long-term movements of a spread foundation it is important to work in terms of the initial and final effective stress distribution with depth beneath the foundation. Thus consolidation or swelling result from net increases or net decreases in effective stress respectively. The tolerance of the structure to the combined effects of heave and settlement occurring at different times must be considered. In granular soils, upward heave movement is, for all practical purposes, concurrent with excavation.

The design of rafts requires reinforcement to be provided firstly to distribute locally applied loads and secondly to cater for bending due to curvature caused by heave or settlement. However, adding reinforcement does not, on its own, stiffen the raft sufficiently to reduce curvatures caused by heave or settlement. The main purpose of such reinforcement is to control cracking. The structure as a whole may have sufficient global stiffness to reduce curvature of the raft.

6.7 Bearing capacity of piles

Because of the stress changes that occur beneath basements in stiff clays, pile capacity should normally be assessed using effective stress methods, such as in reference 6.7. Alternatively traditional methods may be used by reducing the undrained strength to allow for the softening effect of the removal of overburden stress. It has become common in some circumstances to grout pile bases to improve their load-deformation performance by prestressing the base and reversing the shaft friction. This effect has been amply demonstrated by loading tests. However, for a well-constructed pile, this procedure has little, if any, effect on the pile's ultimate bearing capacity. If grouting is carried out before excavation on piles constructed from a higher initial surface level, and excavation then causes upward movement of the piles, the prestress effect can be reduced or lost.

In testing piles beneath a deep basement, it is often convenient to carry out the test from the level at which they were installed. It is essential to allow for the fact that removing overburden stress will reduce the measured capacity of the pile. It has been shown that artificial raising of the groundwater level near the test pile may, where the soils are sufficiently permeable, be used to simulate the effects of excavation[6.8].

6.8 Piled rafts and piles in tension

It is often advantageous to use a piled raft at the base of an excavation. Frequently, the raft alone would have adequate bearing capacity but the piles are needed to reduce settlement[6.9-6.11]. In these cases, straight-shafted piles may be designed to mobilise all their shaft resistance, since their primary purpose is to enhance the settlement characteristics of the raft.

When a pile-enhanced raft is to be used in this way, the distribution of load between the piles and raft is complex and must be analysed by a suitably qualified geotechnical engineer. The complexity is increased when there is a prospect of long-term changes in the groundwater regime. Clearly, the sum of the downward forces at any stage has to be equal to the sum of the upward forces, but this check may not be simple and is sometimes misapplied. At certain stages during construction, piles may be required to act in tension and must be reinforced accordingly.

If heave forces have not been analysed in detail, a conservative assumption would be that the upward pressure on the base of a raft connected to piles and resting on stiff clay would correspond to the overburden pressure removed. To obtain a more realistic, though less cautious, estimate of heave pressures the following method is sometimes used:

- Estimate the total heave that would occur in the effective long term if there were no raft.
- Estimate the proportion of this heave that will be prevented by connecting the raft to the piles; this might typically be 50-80% in a plastic clay.
- Assume that the long-term heave pressures will be the same proportion of the total overburden pressure removed by excavation.

Swelling of a clay soil before such a slab is cast will reduce heave pressures.

An alternative construction method to a pile-enhanced raft is a suspended ground floor slab with a void beneath it deeper than the anticipated heave. This will remove a significant proportion of the uplift force from piles used for structural support, but accumulated gases must be vented and water pumped away (see Chapter 7).

Even with a suspended basement slab, piles may still be subjected to tensile forces because of differential soil movement along the length of the pile. If a pile passes through a clay layer which is partially unloaded by the excavation, the soil at the top of the pile moves upwards; if more than by about 1% of pile diameter, shaft friction is fully mobilised, putting the pile into tension.

Typical stages in the life of a pile are illustrated in Figure 6.6, for top-down construction, disregarding the added complication of changes in the water regime. Shaft adhesion is shown as having a limit which is uniform with depth, for simplicity.

After the pile has been installed (Figure 6.6, stage (a)), it has little residual axial stress (unless it has been heavily driven) and this condition is often ignored. At the end of excavation, stage (b), some proportion of the total heave has occurred, maximum upward displacement being near the surface of the excavation, reducing with depth. Most of the adhesion has been mobilised: at the top, the soil has moved upwards relative to the pile, and at the bottom downwards. Maximum tension occurs near the middle.

When construction has concluded, stage (c), the top of the pile has moved downwards owing to the applied load: i.e. the soil at the top has moved upwards relative to the pile. The effect of the applied load has been to change the distribution of adhesion as shown in (c) such that more 'upward' adhesion has been mobilised than before in (b) to resist the downward load. The maximum tension in the pile has been reduced but some has remained near the bottom of the pile.

Finally, in the long term, stage (d), the balance of the heave has occurred, lifting the base slab and in turn the top of the pile. Now, for the first time, the soil has moved downwards relative to the pile. There is some residual 'downward' adhesion at the bottom of the pile. The force distribution is shown in (d) with tension at the top and bottom and a small compression near the middle.

There is an interesting conclusion. From the adhesion diagram (d), it can be seen that there is surplus 'downward' adhesion in the central section of the pile available to be mobilised to resist uplift owing to water pressure, provided the pile has been adequately reinforced.

The purpose of reinforcing piles in this state is generally to prevent large cracks developing, which might later lead to differential movements within the finished structure. However, many structures have been built with basements around 3-5m deep without any special pile reinforcement to prevent heave cracking of concrete, and there are no reported cases of detrimental structural behaviour.

If the weight of the structure exceeds the weight of excavated ground, it may be possible to deal with this type of cracking by post-construction grouting. However, where the weight is less than the weight of excavated ground, pile reinforcement is probably unavoidable to prevent what would otherwise be significant cracks.

Sometimes, the use of reinforced piles to resist tension may lead to uplift of the pile base. This will be of concern if the base is required to transmit load and it may be necessary either to ensure that the piles are long enough to avoid this or to provide for base grouting.

6.9 The use of piles to strengthen soils in front of a retaining wall

The reduction of vertical effective stress due to excavation leads to swelling and softening of clay soils and this, in turn, can a reduce passive resistance at the base of an embedded retaining wall.

Small diameter 'pin' or nailing piles have recently been used in a deep basement in front of the retaining walls to restrict heave and retain passive resistance[6.12-13] (see Figure 5.5c) An alternative is to load the soil near the toe vertically by a suitably designed ground-bearing slab held down by piling. It should be borne in mind that piles placed immediately in front of a retaining wall within an excavation may be subject to lateral displacements from the retaining wall, and this could lead to cracking of the piles. Where movements are expected to be significant, the piles should be suitably reinforced.

6.10 Vertical bearing capacity of piled walls

Where piles are placed as contiguous, interlocking, or secant pile walls or where reinforced concrete diaphragm retaining walls are used, they may be required to carry vertical loads as well as bending moments caused by retained earth pressures. In either case, normal practice is to postulate that load is transferred only to the soil below the excavation level. The wall, however constructed, is treated as a deep strip footing with side friction using the appropriate bearing capacity factors for this case.

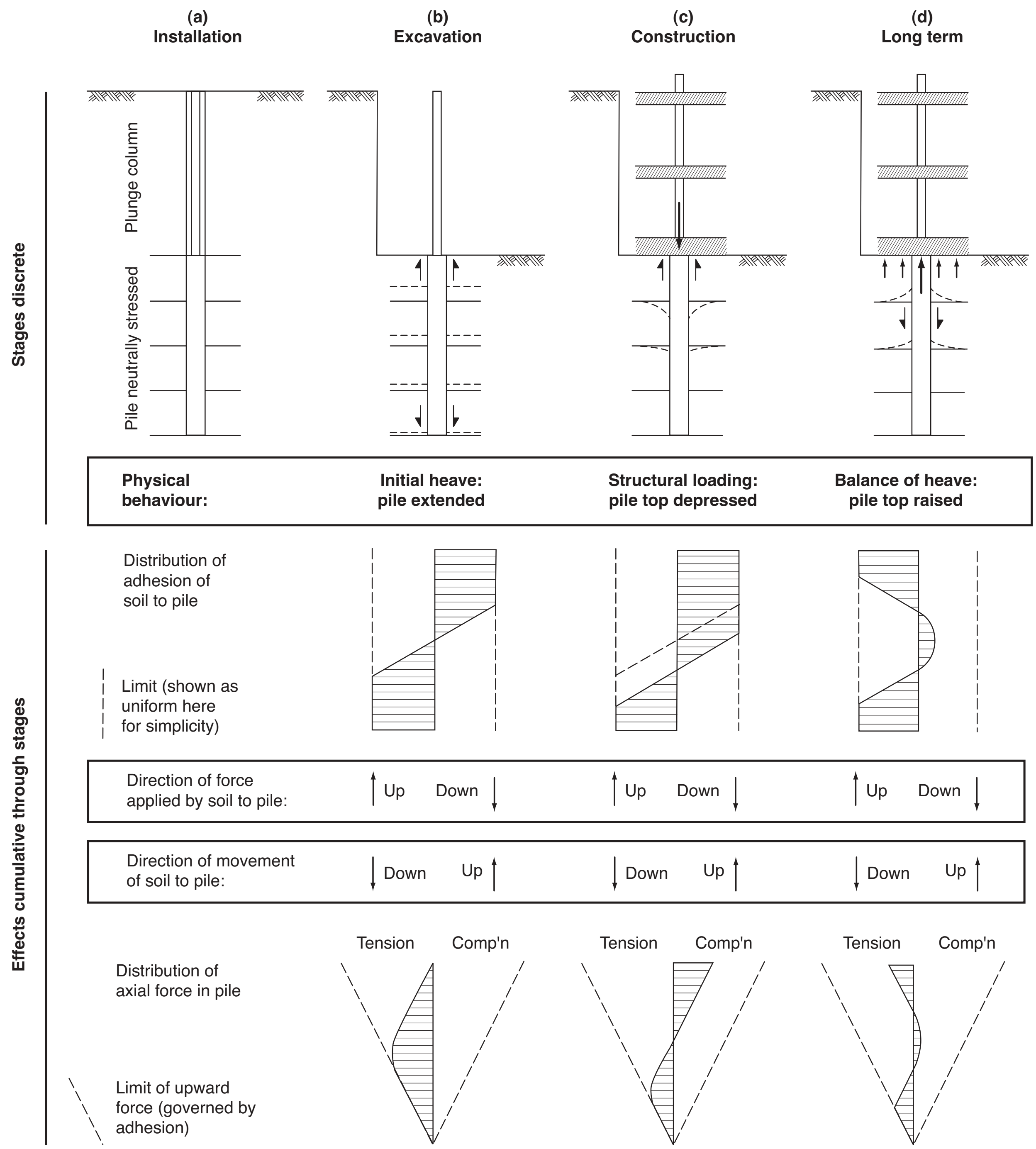

Fig 6.6 Derivation of axial force in pile at various stages

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7.1 Introduction

There is increasing awareness of the need to identify and neutralise or ventilate toxic gases and asphyxiants in underground structures. Examples of such gases include the following:

- Methane (CH_4) - a colourless, odourless, flammable gas. In sufficiently high concentrations, it can be explosive as occurred at Abbeystead[7.1,7.2], Figure 7.1, where, as the result of such an explosion, 16 people were killed and 28 injured. It may be produced during the biological degeneration of organic material from landfill sites. Sometimes known as firedamp or marsh gas, it may occur in coal-bearing strata and in solution in groundwater.
- Radon (Rn) - a naturally occurring, colourless, odourless, almost inert but radioactive gas. It is most commonly found in Devon, Cornwall, parts of Somerset, Northamptonshire, Derbyshire and Wales but may be present in other areas of similar granite-bearing geological history.
- Hydrogen sulfide (H_2S) - a water-soluble, pungent-smelling substance found, for example, in North Sea gas. It may be transported in groundwater.
- Hydrogen cyanide (HCN) - a colourless gas with the characteristic odour of almonds, is formed by the action of acids on metal cyanides. Deadly poisonous, it is used in the production of acrylate plastics and may therefore be found on old factory sites.
- Carbon dioxide (CO_2) - a colourless, odourless gas soluble in water. It is used as part of the refrigeration process. It can come out of solution in groundwater and collect in sumps, etc.
- Carbon monoxide (CO) - a water-soluble, colourless and odourless gas. It is flammable and highly toxic. It is formed by the incomplete combustion of carbon and is the principal gaseous product from appliances used for space and water heating in industrial and domestic installations.

Fig 7.1 The Abbeystead explosion © Lancashire Evening Post Limited

7.2 Precautions

It is essential that any Site Investigation (see also Chapter 8) for a proposed basement includes information on toxic gases. In particular, the possibility of the importation of gases from adjoining areas must be investigated.

Where possible, the source of a toxic gas should be removed. This may be feasible where discrete pockets of organic material are found within the site boundary and can be removed. Dealing with gases that have their origins on adjacent sites may require patient negotiation with adjoining owners. Agreements reached must be carefully documented within properly authorised legal documents.

Where it is not possible to remove the source of a gas, it will be necessary to introduce adequate controls to monitor, collect and maintain concentrations within safe levels. The risk of carbon monoxide leaking from gas mains into a basement and associated voids must be considered and appropriate levels of ventilation maintained. For some deep basements, it may be necessary to provide mechanically assisted ventilation with sufficient redundancy to cover emergency loss of power.

Voids under suspended slabs need to be ventilated to disperse accumulated gases.

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(Building Report Note No: 28 Ref BC/BSR/4/17)
Note: This note concerns the explosion of town gas in a large unventilated cavity alongside a shopping centre. The gas had emanated from a fractured 10cm gas main and filtered into the cavity through the soil of an embankment. There were 21 deaths and many more were injured.

The Building Regulations 2000. Statutory Instrument 2000 No. 2531. The Stationery Office, UK, 2000.
Note: Requirements C1 and C2 state the following.

C1. Preparation of site.
The ground to be covered by the building shall be reasonably free from vegetable matter.

C2. Dangerous and offensive substances.
Precautions shall be taken to avoid danger to health and safety caused by substances found on or in the ground covered by the building.

8 Site Investigation

8.1 Introduction

It will be evident from preceding chapters that the successful design and construction of deep basements depends on the careful consideration of the interaction of soil, groundwater and structure in a number of different, though often interrelated, situations. Site Investigation, therefore, should be designed to elicit a thorough understanding of:

- the ground conditions at the site before construction, including the effects of previous use
- the way in which these ground conditions will influence the design and progress of the works
- the way in which the construction of the basement will influence, both in the short and long term, ground conditions around the site and the neighbouring structures built on and in that ground.

In the following, it is assumed that the reader is familiar with good practice in the design and implementation of Site Investigation as described in such references as BS 5930: *Code of practice for site investigations*[8.1], and BS 1377: *British Standard Methods of tests for soils for civil engineering purposes*[8.2]. Comment is limited to those aspects of Site Investigation that are particularly important to basement work and where some emphasis and explanation may therefore be helpful.

It should be emphasised that a geotechnical engineer experienced in such matters from the design team should always control the planning and execution of such Site Investigations.

8.2 Desk Study

The Desk Study provides an essential opportunity to gather together, collate and assess as much information as possible on the site and its environs before any fieldwork is undertaken. This is particularly useful for deep basements where many of the engineering and planning issues are complex. The information gleaned from the Desk Study will enable the principles and options for design to be evaluated at the earliest stage in the project. This in turn will allow proper direction to be given to the more detailed surveys and investigations that will follow.

The design of a deep excavation depends on the composition of the materials around and beneath the site, their stratification, permeability, strength and stiffness, and the stress condition existing in them. These are all a function of the geological history of the site and its environs. A sound understanding of the geology is therefore of fundamental importance. Information from the Desk Study will begin to build this three-dimensional picture of the geology and enable future Site Investigation to be designed in such a way that those elements of the ground critical to the construction of the basement are properly investigated and described. For example, the presence of layers of soil at depth with different permeabilities and water pressures may significantly affect basement construction (see Chapters 2, 3, 4 and 5). Evidence of such layering may well emerge from the Desk Study and enable the disposition of boreholes and sampling to be well defined.

There have been problems where attention has been concentrated on the site itself to the exclusion of surrounding areas. From preceding chapters, it will be clear that the influence of basement works extends well beyond the boundary walls, in the form of changes in ground stresses, deformations and groundwater effects, often to distances many tens of metres away. The Desk Study provides the opportunity to assess the basement in the context of these surrounding areas, their topography, geology and hydrology.

It will be necessary to determine the historical land use of the area, the remains of which may influence design and construction, along with the state of neighbouring buildings, their foundations, basements and services and the presence and condition of any tunnels for sewers, underground railways, etc. The sensitivity of these adjacent structures to the likely changes can be assessed and limits set for the control of movements around the site. There are many cases in which the design and construction of the basement have been dictated by the need to minimise these changes and this often has a significant bearing on the proposed Site Investigation. Estimates of pressure exerted on the site by previous structures will assist later assessments of settlement and/or heave.

In many urban locations, the site may be archaeologically important. Again, early assessment of the archaeological potential during the Desk Study will give maximum time to agree appropriate action for examination and recording of finds and/or conservation, so that later costly delays can be avoided (see also Appendix A).

The frequency and magnitude of data on variations in groundwater levels associated with flooding, seasonal and tidal effects should be carefully considered. Information on longer-term changes in regional water levels, due for example to the reduction or cessation of water extraction, should be identified[8.3].

The need for and extent of any chemical investigation of the site should be evaluated during the Desk Study. In particular, the past uses of the site and its environs should be studied to identify whether significant chemical contamination of the soil could have occurred[8.4]. The presence of organic soils, significant thicknesses of fill, methane-generating layers, or any other material of a potentially deleterious nature should also be identified for further investigation.

Existing Site Investigation data will be highly relevant, as will experience from other local basement construction, the methods used and problems encountered.

8.3 Physical investigation of the site

With a broad view of the likely ground conditions and constraints on construction having been obtained from the Desk Study, the range of options for design can be identified. The Site Investigation can then be designed to confirm and develop the understanding of the stratigraphy and groundwater conditions, and to test and sample the materials to provide parameters for design. With important works or difficult ground conditions or when little information is available from the Desk Study, it may be desirable to carry out a preliminary Site Investigation to ascertain general ground conditions. This would be followed by the main investigation with full testing and sampling.

There are many boring, probing, in-situ testing and sampling techniques available[8.1,8.5] and the choice will depend on the scale and difficulty of the basement proposed and the ground conditions expected. The aim will always be to obtain quality information. In basement work, quite minor stratigraphic variation can be significant, particularly in assessing groundwater effects and their control. A complete picture of the vertical succession by continuous sampling is thus desirable. This can be obtained in many cases by rotary coring, or profiling with a piezocone. Alternatively, it may be valuable to sink a shaft to make a detailed visual inspection of the vertical succession in-situ and to obtain large samples for special laboratory tests. Such a shaft also permits direct inspection of the location and amount of seepage from various strata.

In any Site Investigation, an experienced geotechnical specialist from the design team should be present on site to direct the work and make day-to-day decisions on its scope.

8.4 Groundwater investigation

In many Site Investigations, too little emphasis is placed on assessing the groundwater regime and much useful information is overlooked. An understanding of the groundwater and its interaction with the soil and rock is of paramount importance in most basement work (see Chapter 3). Water levels and rates of ingress should be measured during drilling and related to the sequence of strata. Sufficient standpipes and piezometers designed with a response time appropriate to the ground conditions should be installed to define variations in water head both vertically and horizontally in and around the site.

Measurements of water levels in standpipes and piezometers should not be restricted to the period when drilling is under way. Long-term monitoring should be carried out, as necessary, to detect seasonal variations, tidal effects and to correlate with flood conditions.

Groundwater flows across the site should be identified, since basement construction in the long term may impede them, disrupting or modifying otherwise established water levels.

In-situ permeability tests may be carried out in boreholes, preferably with water being drawn out of the ground rather than fed into it, so that fissures, joints and interstices of the soil are flushed of drilling debris rather than clogged by them.

With permeable ground and where water control measures are to be designed, full-scale pumping tests will be invaluable. The scope will depend on local conditions[8.6-8.8].

8.5 Parameters for design

8.5.1 Classification tests

Tests for moisture content, plastic and liquid limit and particle size distribution should be carried out to help in the correct description of the materials encountered[8.2,8.9]. Many broad correlations exist between the results of classification tests and other engineering parameters such as strength. Knowledge of classification tests gives a useful check on the veracity of test results for these other parameters.

8.5.2 Strength parameters

Undrained conditions

The short-term shear strength of clays, before drainage has had time to occur, can be measured directly in the field using vane testing equipment and in the laboratory, commonly in the triaxial test on 100mm-diameter specimens. It can also be usefully estimated using correlations with the results of Cone Penetration Tests[8.10] or Standard Penetration Tests[8.11,8.12] or with plate-bearing tests[8.13]. Some care is required in applying the results of any of these laboratory or field tests, since each is influenced to a greater or lesser extent by the soil fabric, homogeneity of the sample, in-situ conditions, and disturbance during sampling[8.14].

Drained conditions

Undrained strength is a function of the in-situ effective stresses operating at the time of testing. During excavation, overburden and lateral support are removed, and so in-situ stresses will change significantly during the construction of a deep basement. It will therefore be necessary to assess strength in terms of drained or effective stress parameters, so that retaining walls and foundations at the base of the excavation can be designed to perform satisfactorily within the newly imposed stress regime.

In sands and gravels, correlations exist between the angle of shearing resistance φ' and the results of Standard Penetration Tests[8.15,8.16] and Cone Penetration Tests[8.10]. As such materials are generally difficult to sample without disturbance, laboratory strength tests in the triaxial test or shear box can usually only be carried out on reconstituted specimens.

For clayey materials effective stress parameters can be obtained most easily from undrained triaxial tests with pore water pressure measurement[8.9].

Estimating and measuring soil strength in terms of effective stress parameters are often not straightforward. The advice of an experienced geotechnical engineer should therefore be sought.

8.5.3 Stiffness parameters

The stiffness of soil, often expressed as a modulus of elasticity, varies significantly with strain level, the magnitude of stiffness reducing as strain increases. For example, in stiff over-consolidated clay, the stiffness in undrained loading at 1% strain may be only 20% of the stiffness at 0.1% strain[8.17,8.18]. Thus, it is important in assessing stiffness to understand the level of strain likely to be mobilised. In retaining wall design where movements are to be limited, the appropriate level of stiffness may be much higher than beneath the spread foundations supporting the building. Anisotropy in the ground also needs to be considered carefully. Stiffness in horizontal loading of over-consolidated clay, for example, may be double or treble the stiffness in vertical loading.

Each of these considerations will affect the choice of the most relevant type of test to measure stiffness and the choice of appropriate direction of loading.

Estimating stiffness

Stiffness of soils is also influenced by stress history. This makes direct measurement of stiffness in the laboratory quite problematical. The processes of boring and sampling themselves change stresses, while sample disturbance may destroy the fabric of the specimen and the stress pre-conditioning inherent in the in-situ material resulting from its depositional history.

Consolidation tests in the oedometer, for example, commonly used to measure stiffness of stiff clays in drained loading and/or unloading, generally give stiffnesses that are low compared with full-scale performance. On the other hand, oedometer tests on soft normally consolidated soils can give reasonably reliable data on stiffness under loading and unloading.

Plate-bearing tests are often used to measure stiffness of soils in-situ[8.17]. Tests can be carried out in shafts at different depths and oriented to give vertical or horizontal loading. However, they also suffer from the problem of disturbance caused by excavation and difficulties of bedding the plate. It is thus difficult to measure small strains accurately and to reproduce the higher stiffnesses observed in full-scale structures.

Generally, it will not be appropriate to attempt to measure stiffness directly either in the laboratory or in the field.

The most reliable way of estimating stiffness for basement structures is by back-analysing displacement records for case histories of structures in comparable ground conditions, where the ground has been subjected to similar types of stress change. Many case histories have been published (see Chapter 2).

Variation in soil conditions between case history and the site of the proposed basement can be accounted for by using well-established relationships between stiffness and other parameters such as strength. In clays, it has been shown that, for a particular type of clay, the ratio of stiffness to undrained shear strength is the same at the same level of strain. This relationship has enabled stiffness to be estimated in clays using Cone Penetration Tests and Standard Penetration Tests, since these effectively measure the undrained strength of the ground, which can then be related to stiffness using the established empirical relationships.

In sands and gravels, the stiffness in vertical loading back-figured from monitored structures founded on these materials has been correlated with Standard Penetration Test 'N' values[8.16] and Cone Penetration Test data[8.10]. It has been shown that the ratio of stiffness to 'N' values is approximately constant for a given strain level[8.16]. In over-consolidated granular material, stiffness decreases rapidly with strain level, as in clays. For normally consolidated granular material, the stiffness is at least one-half that for over-consolidated materials at the same strain and the variation with strain level is much less.

There are fewer case histories of full-scale structures in sands and gravels subjected to horizontal loading or unloading. However, evidence suggests that horizontal stiffness is usually likely to be higher than vertical stiffness.

Recent advances in measuring stiffness

Many recent advances have been made in the design and use of devices for measuring low levels of strain accurately during laboratory testing of triaxial specimens. With such tests following the undrained loading of stiff clays, it has been possible to reproduce in the laboratory the high levels of stiffness at low strain levels observed in full-scale structures, and the pattern of decreasing stiffness with increasing strains[8.17].

Self-boring pressuremeter[8.19] equipment is being developed capable of measuring horizontal stiffness in-situ in a variety of soils, both clays and sands, with minimal disturbance[8.20]. This is a promising development and may become more widespread as experience is gained in its interpretation.

8.5.4 In-situ stresses

In basement design, knowledge of the in-situ stresses before construction is essential, particularly in determining the performance of retaining walls. The vertical total stress can be estimated with reasonable confidence from laboratory measurements of bulk density. From this and data on the groundwater pressure, the vertical effective stress can be obtained.

Horizontal stresses, however, are the large unknown, but must be estimated for basement wall design under working conditions and in predicting ground movement.

Geological history of the site gives the first clues to likely in-situ stresses. Section 5.3 describes the relationship between the overconsolidation ratio and the coefficient of earth pressure at rest K_0.

Field tests

Various Site Investigation devices are now available for estimating lateral stresses. Lateral stresses are particularly difficult to measure, since intervention into the ground inevitably alters them. The self-boring pressuremeter[8.19] allows a measuring device to be inserted with a minimum of disturbance and probably provides the best direct measurement of in-situ lateral stresses in both clays and sands.

Other devices such as the push-in pressure cell and the Marchetti Dilatometer[8.21] provide a measure of the contact stresses on the side of a spade-cell after it has been pushed into the ground. Of course, the process of inserting the device modifies the local stresses, but these can be related to the original in-situ stresses by empirical relationships. Another approach for clays is to estimate lateral stresses using the piezocone[8.22].

All these methods have uncertainties and work continues to improve lateral stress measurement and estimation.

Laboratory tests

In-situ horizontal stress in stiff clays can be estimated in the laboratory by measuring the capillary pressure of samples[8.23,8.24]. More recently, a 'filter paper test' has been developed[8.25] and shown to give consistent results.

8.6 Chemical testing

As with any Site Investigation, it will be important to carry out routine tests to check on the aggressive qualities of the ground and groundwater that may have a deleterious effect on construction materials[8.2,8.3,8.26-8.28]. If dewatering is contemplated, either as a temporary expedient or as a permanent measure, a more detailed analysis of the groundwater chemistry and biochemistry will be needed so that wells, drainage blankets and other parts of the permanent drainage system are not subjected to corrosion, precipitation of minerals or growth of bacteria (see also Chapter 7).

Where the Desk Study indicates the possibility of methane beneath or near the site, or where appreciable deposits of organic matter are found on the site or nearby, gas concentrations should be measured during borehole drilling. After drilling, gas standpipes are required to take gas samples and to measure flow rates. Guidance on field sampling for methane is included in reference[8.29]. Methane is soluble in water and can be transferred to the site over some distance from a natural or man-made source in the vicinity. Groundwater should be tested for dissolved gas[8.29].

The extent of chemical contamination of soil and groundwater is assessed by laboratory analysis of samples collected under strict sampling protocols (see Chapter 12). The degree of contamination may be assessed by DEFRA guidelines[8.30] and published criteria[8.31].

Radon gas, which occurs naturally in some granite rocks, can diffuse to the ground surface. It can be a health hazard if allowed to accumulate. The National Radiological Protection Board (NRPB) has published a report giving results of a nationwide survey of dwellings[8.32]. Tests for radon emission can be carried out as described in reference 8.33.

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9 Excavation

9.1 Introduction

It is vital for the engineer to have an appreciation of the construction methods to be adopted, since these matters have an important effect upon the design and a profound influence on the cost of the work. If it is proposed to use, for example, top-down construction, it should be fully understood that excavation by this method is much slower, as excavating machines have to be smaller to gain access below the slab. Clearly, there are technical advantages in top-down construction, since it improves support to the excavation at all stages and no special temporary works are involved. It is a method often favoured on sites where good support must be maintained to adjacent buildings founded at a much higher level. The penalty of slower construction is mitigated when the superstructure is constructed while the basement is excavated and basement floor constructed, but this can only be done where site space and access allow. Storage of construction materials may be particularly difficult on small sites for top-down construction.

It is considerations such as these, including a knowledge of the methods of construction and the practical limitations of diaphragm walls, contiguous piling, secant piling, etc., together with the effects of noise and vibration associated with particular techniques, that can significantly affect the designer's thinking when preparing detailed proposals.

9.2 Methods of excavation

The method adopted in any particular case depends upon many factors, including:

- type of ground to be excavated, i.e. whether cohesive, non-cohesive, rock, etc.
- accessibility of the site
- whether the site will be congested with other plant or temporary works when equipment is working
- knowledge of the detailed design of the external retaining walls and foundations to plan the excavation procedure and work sequence
- method of disposal of the soil
- availability of plant
- overall construction programme, to decide speed at which work needs to be done
- knowledge of previous use of site to assess the possibility of encountering obstructions from old foundations
- proximity of existing buildings
- groundwater and necessity for dewatering, pumping, etc.
- possibility of contaminated ground
- restrictions, due to off-site access and traffic, on the delivery of materials and removal of excavated soil.

9.3 Considerations affecting the use of plant in deep excavations

A checklist of the main items to be considered is set out below:

- time allowed for excavation
- nature of ground, hard or soft rock, fine or coarse-grained, presence of boulders or obstructions
- cohesive or loose soil
- wet or dry conditions
- abrasiveness of soil to be excavated
- noise and vibration
- depth of excavation
- site location
- restrictions on working hours
- method and sequence of construction
- plan size of excavation
- type and number of excavating machines
- method of raising, loading, transporting and disposing or reuse of excavated material
- means of support to the excavation.

The basement excavation must fit efficiently into the construction plan, allowing phased permanent construction to follow the excavation in a logical and economical sequence.

9.4 Unrestricted sites

For sites where deep excavation is possible in open cut, it is necessary to determine the maximum safe gradient of the batters to the excavation, taking into account the construction period, the consequences of any slippage, likely weather, soil and groundwater conditions, and available slope protection. The position, gradient and size dimensions of muck-away ramps and any temporary on-site muck storage require early consideration.

9.5 Restricted sites

9.5.1 General

Because of high costs of urban land, deep basement construction is often carried out in areas already congested and confined. The excavation method and programme will depend on the general factors previously mentioned and on the selected walling system and its support for temporary or temporary/permanent soil support. Some alternatives are described in the following sections.

9.5.2 Diaphragm walls

Constructing the permanent wall before excavation (i.e. using a diaphragm wall or contiguous or secant pile system), with the wall temporarily tied back by a system of ground anchors, will give maximum scope for the excavation equipment. It allows the plant to work without interruption within the basement area with no impedance from temporary shoring. However, ground anchors should be used with caution in view of their possible effects on adjacent buildings, and the deflections of the retaining walls. Approval from surrounding owners or highway authorities will be needed if anchors cross the site boundary.

Before a diaphragm wall or secant pile wall is constructed, reinforced concrete guide walls must be built. A guide trench can be a significant construction in its own right and needs to be removed, at least on one side of the wall, before excavation can proceed. Where the wall is to be strutted from the central permanent construction, and provided the proportions of the excavation allow, an earth berm may be left against the wall. This berm is removed only after bracing has been positioned, removal usually being done by small excavators able to work within the confines of temporary shores. Where the permanent wall is strutted from a central dumpling or permanent raft, construction is necessarily slow. The temporary bracing may need to be replaced by further temporary re-strutting before the permanent basement walls and floors are constructed.

9.6 Obstructions

Most urban developments are in areas where there have been previous buildings, and obstructions are nearly always met in excavation and piling. It is important to make reasonable allowance for these possibilities in pricing and programming the work. If not too large, old brick foundations can be grabbed out by machines, but pneumatic breakers or hydraulic bursters may often be needed. It is a good idea to establish the location and nature of any obstructions by probing ahead of any necessary piling.

9.7 General removal of spoil from site

On open sites, the cost of spoil removal is usually low, although disposal may be as expensive as on a congested site. The spoil is excavated, placed into dump trucks and usually taken off site immediately.

Fig 9.1 Changi Airport: excavation by long-reach/dipper excavator © Benaim

On congested sites, spoil removal is expensive since it may have to be handled twice or three times. If ramp access is possible, the excavating machine can load trucks. Otherwise spoil has to be grabbed from a spoil heap and loaded into trucks at ground level. Where the site is very congested, it is common to transfer the spoil into a receiving container, which then discharges through bottom-opening flaps into trucks at ground level: long-armed dipper dredgers may be used, as at Changi (see Figure 9.1). Conveyor belt systems may also be used to move suitable spoil.

If spoil has been contaminated with bentonite or is wet, it should be left to drain before being taken from site. Bentonite slurry should be removed in sealed containers such as sludge tankers. Specially registered tips will be needed for spoil disposal. Roads should be kept clear of mud, with wheel-cleaning units for trucks leaving the site. It is usually obligatory to have mechanical sweepers keeping the roads clean in the immediate vicinity while the excavation work is in progress. Disposal of soil contaminated by bentonite or other impurities is an important consideration in planning and costing excavation work. The locations of tips that will accept such material are likely to influence both construction time and cost.

9.8 Piling within basements

When a deep basement is supported on piles, the piling work has to be phased into the construction programme. Although piling may be left until after excavation, this would entail providing access for piling equipment and removal of spoil from borings. For deep basements, it is common to delay excavation and to pile from the original ground level, leaving a length of empty bore above pile heads. Empty bores must be made safe by adequate barriers and warning signs, or by backfilling.

10.1 Introduction

Table 2.3 (Chapter 2) categorises potential damage to adjacent, or even distant, property affected by the excavation and de-watering for a new basement. Recently, and increasingly so, damage more serious than the 'slight' category is unacceptable to building and utility owners, and specific potential victims may demand 'very slight' or even 'negligible' categories, particularly for buildings of historic or economic importance or for essential services.

There are several steps that the owner of the new basement may take to avoid contention and distress. These protective measures may be considered in three categories, examined in detail later in this chapter:

- Internal structural measures, which include all actions taken within the new basement during its construction to reduce the ground movements generated at source.
- External structural measures, which reduce the impact of ground movements by increasing the capacity of the adjacent structure to resist, modify or accommodate those movements.
- Ground treatment measures, which include all methods of reducing or modifying the ground movements generated by constructing the basement, by improving or changing the engineering response of the ground.

Frequently, these measures are used in combination and the effects of the combination must always be assessed.

While prevention is normally better than cure, unnecessary prevention can be obtrusive and is undesirable. Observational techniques can be employed to advantage, where ground movement can be monitored and action taken if pre-established trigger levels are reached (see Appendix E).

A comprehensive treatment of effective protective measures employed on the Jubilee Line Extension is given in reference 10.1.

10.2 Internal structural measure: strutting and sequence

The stiffness of the retaining walls has a great impact on ground movement outside the new basement. Walls in cantilever deflect a great deal more than walls strutted at several levels, and are to be avoided in sensitive urban areas. The superior strength of steel walls often means that they are less stiff than concrete walls, and it may be more important to design them for an acceptable stiffness than for an acceptable stress.

The stiffness of the struts also has an impact. Again, steel struts, often chosen for their lightness and ease of recovery, may need to be designed for stiffness rather than for strength. Preloading them to a level that matches their expected maximum load will minimise wall movement at strut levels. If the preload at higher levels would be too great before the excavation, it may need to be adjusted during excavation.

The stiffest strutting is that offered by the basement slabs, which is one of the reasons for the popularity of top-down construction. Casting the complete, permanent, slabs on plunge columns requires strong columns and possibly foundations whose design may be governed by the temporary rather than the permanent condition. In such cases, semi-top-down construction is being recognised as a practicable option[10.2]. Here, the perimeter of each slab is cast to act as a waling (say 2-3m wide), with enough of the remainder of the slab to act as struts in both directions, with starter bars or couplers for later infilling. The reduced weight of this grillage gives considerable economy in the plunge columns and their foundations. Indeed, their design may be dictated by the flexural requirements of the slab-to-wall connection. The waling strip will cantilever from the walls via starter bars or couplers, using the concept of 'shear friction'[10.3, 10.4].

With excavations that are extensive in plan area (making the cost of strutting across the whole site excessive) the berms may be left in place, the structural slab cast and raking struts from the wall to temporary corbels on top of the slab left in place during excavation. Once top-down excavation is complete, the slabs are completed bottom-up and the raking struts removed.

Where exceptionally stiff walls are required, it may be necessary to activate strutting before excavation or before de-watering. If this is before excavation, some form of deep level strutting can be achieved by jet-grouted struts, by plain concrete diaphragm wall panels, by secant piling in 'blind bores', or by tunnelled struts. If before de-watering, the base slab may be tremied in, using special cohesive mixes for underwater placing around reinforcement cages positioned by diver. Alternatively, permeation grouting (see Section 10.8) with microfine cement grouts may be used beforehand to create a stiffened raft of granular soil at the base of the excavation.

The sequence is important. For example, horizontal movement will be significantly reduced if a level of temporary strutting is used at ground level before excavating to the next level of strutting: once this level of strutting has been activated, the temporary strutting may be removed for re-use lower down. Minimising the extent of the de-watering within the excavation is also important but it is essential to avoid excessive softening of the soil offering passive pressure resistance at the toe of the perimeter wall and to prevent the possibility of base heave.

Construction within a stiff cofferdam will help reduce ground movement, but extracting the temporary wall may require grouting of the void thus created. When designing the struts for a stiff cofferdam in overconsolidated clays the risk of increasing strut loads with time should be considered due to re-establishment of at-rest pressures during the expected life of the wall.

In all cases, the soil-structure interaction should be considered and, if appropriate, two-dimensional finite-element analysis can be used to establish the likely magnitude of ground movement outside the new basement.

The extreme case of building a new basement under an existing building and through its piles merits special consideration. The act of excavating for diaphragm walls near existing friction piles will relax the grip of the ground on the piles and may lead to excessive settlement or even collapse. It will be prudent to carry out trials on a loaded dummy pile outside the building. Such a trial was carried out at Changi Airport MRT Station, where a settlement of only 3mm was found[10.5], confirming the design predictions.

10.3 External structural measure: underpinning

Underpinning systems have developed from the requirements of supporting and strengthening structures built in former times, and, with the modern trend of incorporating deep basements in buildings, a retention system is often an essential element of the underpinning solution. These systems seek to limit movements by introducing support to existing buildings whose materials are in various states of stress, and to control safely the interaction between existing and new works.

Modern society tries to preserve the historical character of towns and cities, and the construction of deep basements has made engineers aware of the problems of maintaining the equilibrium of old buildings and avoiding damage to brittle facades and walls. Incomplete knowledge of the history and condition of old buildings, combined with their foundations and ground support, requires considerable judgement, risk assessment and practical measures when designing underpinning and retention systems. Competence requires an understanding of materials science, combined with an awareness of the distributions of load unique to a particular structure and its ground conditions.

In 1882, Stock[10.6] wrote a classical treatise on underpinning and retention to educate his younger architect contemporaries. Knowledge of the subject, according to Stock, was acquired only by a wearisome search of the little information kept in different libraries, with the additional difficulty that two of the best authorities on the subject wrote in a language other than English. As the situation today is quite different, with comprehensive published guidance (see references), no attempt will be made here to present advice already covered adequately in other modern publications.

10.3.1 General advice

The design and construction of deep basements should recognise the information provided by adequate site and ground investigations, and the potential for disturbance of existing buildings should be carefully assessed.

It should be appreciated that structural strengthening measures may be an essential requirement in conjunction with underpinning to avoid movements that could damage adjacent property. Discussions must be held with owners of adjacent properties to resolve such difficult matters having legal significance: in England and Wales Party Wall Surveyors have unique responsibilities and powers in law in this respect. The condition of adjacent property must be carefully investigated and all features recorded, including photographic records clearly displaying the condition of the buildings before underpinning.

Ground anchors for retention systems beneath adjacent property require special legal and contractual arrangements with the owners involved, and are unlikely to be allowed to remain in place on completion. Where deep excavations and retention systems are constructed alongside roads, and ground anchors are being contemplated, consultations should be held with the highway authority.

Three main categories of structure and classes of underpinning have been identified[10.7].

Categories of structure

- Ancient: greater than 150 years since completion
- Recent: 50-150 years since completion
- Modern: less than 50 years since completion

Classes of underpinning

- Conversion works (not considered here)
- Protection works
- Remedial works (not considered here)

Knowledge of building construction, as practised in these three eras, is of great assistance to those preparing designs for each of the classes of underpinning.

Generally, constructing deep basements will involve protection works. The protection of existing service pipes, sewers, optical fibre telecommunication ducts, high-voltage lines, etc., is also an important consideration (see for example the temporary support of the essential manhole in Figure 10.1).

Successful underpinning requires knowledge of the state of balance of a building and its foundations and of the ground conditions. Paths of primary and secondary load transfer need to be fully investigated within any structure to be underpinned, as do the probable concentrations of stress in the building while in its passive condition.

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

Transfer of load from a structure to its underpinning components needs to be carefully executed. As well as the need to restrict movements, 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. Figure 10.2 illustrates how the load on a pile was transferred by beams strapped to its sides and jacked before the pile was amputated.

If only parts of a foundation are to be underpinned, the engineer should be satisfied that any movements between those parts being underpinned and the remainder will be acceptable. Properly designed and executed partial underpinning has been shown to be successful.

10.3.2 Shallow underpinning

Some forms of underpinning involve a sequence of partial excavations for installing 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 the structure remains safe. The structure and existing foundations should be able to arch safely over partial

Fig 10.1 Potong Pasir Crossover, Singapore: support of existing manhole

Fig 10.2 Changi Airport Station: pile underpinned by transfer beams and amputated

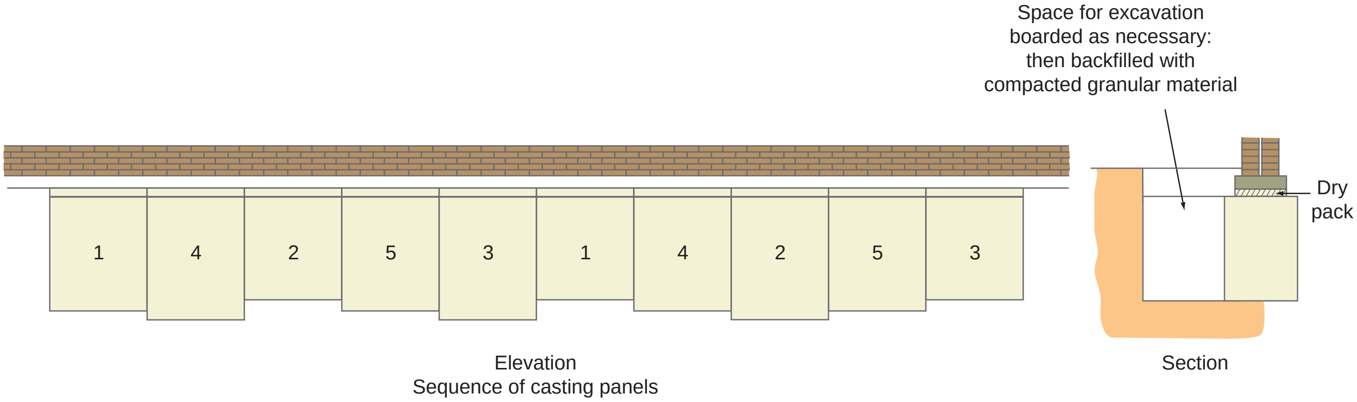

Fig 10.3 Hit-and-miss sequence of shallow underpinning

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, additional work should be done to strengthen the weak base materials before underpinning begins. A suitable 'hit-and-miss' sequence for traditional shallow depth underpinning is illustrated in Figure 10.3, ensuring no excavation is carried out next to green concrete.

Special care should be taken when constructing traditional underpinning segments at 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 construction of a support beam will normally have to be done before underpinning can be constructed.

Shoring generally is used to provide temporary support to structures while the underpinning works are being executed. The complex 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 temporary shoring and acceptance of all structural loads by the underpinning.

To predict the performance of underpinning structures, the engineer needs to consider the state of the supporting soil and the effect that the underpinning technique will have. If the performance of the underpinning operation must be precisely controlled, the whole construction sequence and timing need to be specified comprehensively. Before a building is underpinned it will have consolidated the ground directly beneath its footings. Underpinning

Table 10.1 Classification of piles used for underpinning works

Ground removal	
Types of pile	Methods of construction or installation
Micro: $d<75$	Rotary Rotary percussive grouted hollow-tube reinforcement
Mini: $75<d<300$	Flight auger Rotary Rotary percussive with grouted rebar
Small: $300<d<600$	Flight auger Percussive (clay cutter and bailer) with concreted rebar
Ground displacement	
Types of pile	Methods of construction or installation
Micro: $d<75$	Driven or pushed tubular steel sections, open and closed ended
Mini: $75<d<300$	Driven or pushed tubular steel and concrete sections Driven or pushed steel sections (H-piles)
Small: $300<d<600$	Driven or pushed tubular steel sections, open ended

will remove some of this consolidated ground and apply load to less consolidated ground. 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 expensive will be the investigation.

10.3.3 Deep underpinning

There is generally no need for temporary support when piling methods of underpinning are employed. Conventional piles for underpinning are formed in-situ in bored holes, driven by vibratory hammers, or jacked. Bored cast-in-situ piles are the most common form and often have a minimum diameter of 300mm. Piles that are jacked or vibrated may be smaller and need not be circular. Table 10.1 presents a simple classification of piles for underpinning works.

Small-diameter piles may be installed close to or beneath the loads to be supported, and slim mini-piles are frequently drilled through existing foundations or the bases of thick walls (see Figure 10.4). Although the bearing capacities of individual mini-piles are small, the piles have proved effective in difficult situations and increasingly cost-effective as improved drilling and driving equipment have been developed.

10.4 External structural measure

10.4.1 Strengthening

Where the adjacent structure will have insufficient tensile capacity to cope with the expected ground movement, it may be strengthened by passive or active means. Passive means would include strapping of foundations; drilling, tying and grouting of foundations, masonry or lintels; and tying of walls at floor level(s). Active means would include drilling or strapping of masonry or concrete for subsequent post-tensioning to create a prestressed whole before any ground movement.

Although tensile horizontal ground strains generate tensile stresses in ground-level masonry, which the masonry at that level may not tolerate without cracking, more visibly destructive are the strains at high level in masonry generated by hogging

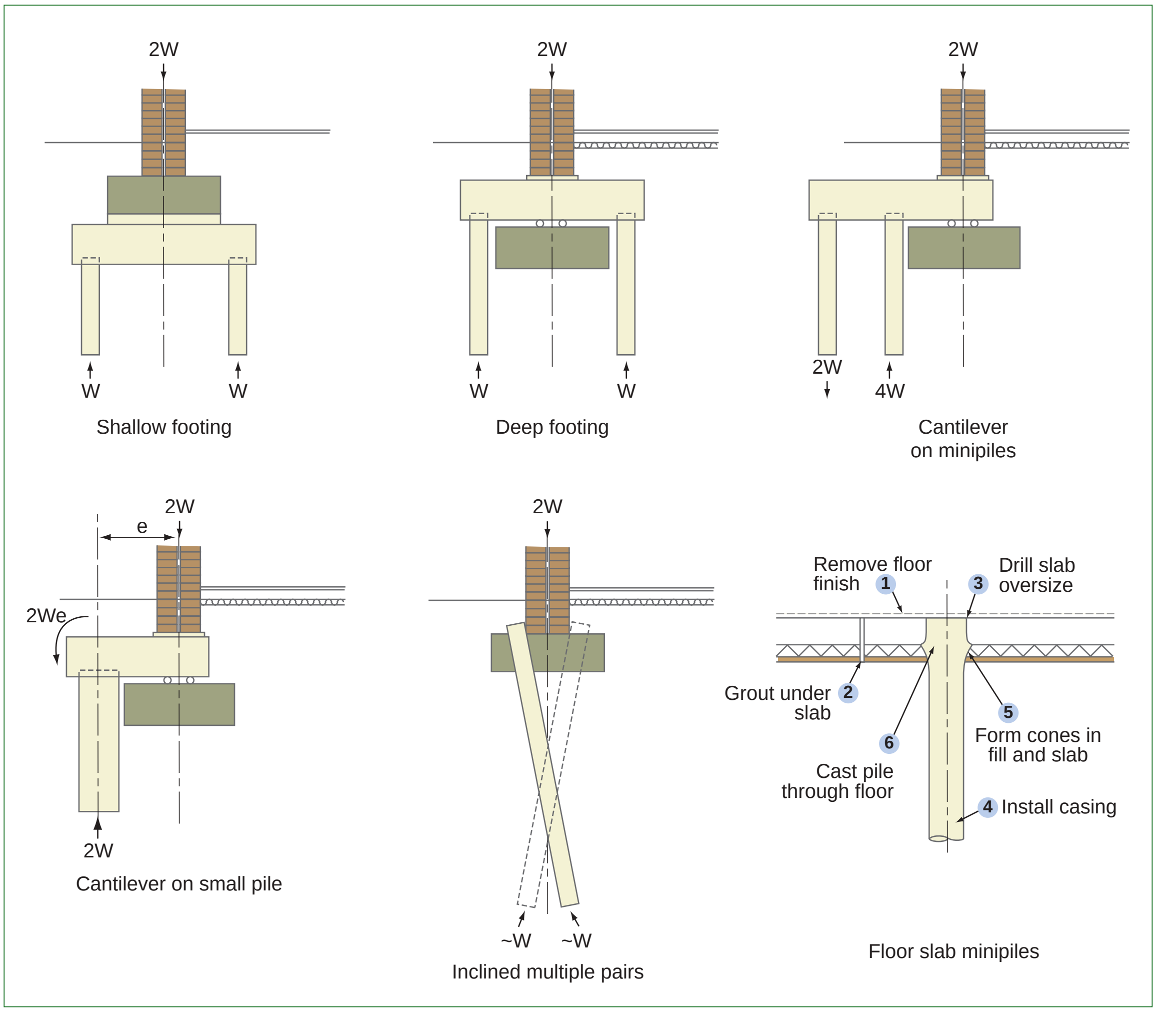

Fig 10.4 Domestic underpinning

Fig 10.5 Changi Airport Station: underpinning transfer beams being inserted © Benaim

curvature in the supporting ground. Strapping or tying at high level is less easy to apply and to conceal; a compromise solution is frequently required. Access for the insertion of large steel beams needs consideration (see Figure 10.5).

Tensile reinforcement should be chosen with care. Non-ferrous metals or composite fibres may be appropriate for certain exposure conditions, allowances being made for creep of the medium being tied and relaxation of the tying medium.

10.4.2 De-sensitisation

The sensitivity of an adjacent structure to ground movement may be reduced by: increasing the bearing of beams on their shelves; strengthening the connections between structural elements and/or between these and their finishes; slackening bolts or making saw cuts to allow articulation; temporarily removing sensitive finishes; or installing temporary supports. Vertical saw cuts in the facades of terraced houses should be used only with extreme care.

10.4.3 Load transfer

Jacks may be introduced between components of the building to accommodate expected movement, adjusted as necessary during the construction of the basement. Jacks must be fitted with locking rings or alternative load paths to relieve continual pressure on the hydraulic seals. Check valves must also be fitted.

10.5 Ground treatment

10.5.1 Compensation grouting

This is a relatively new technique that involves the injection of grout into the ground beneath the foundations of the structure to be protected in order to compensate for the foundation settlements and stress reductions caused by the excavation. The basic principle is illustrated in Figure 10.6. The volumes and timing of grout injection are based on detailed observations of performance with the aim of controlling the development of settlement and

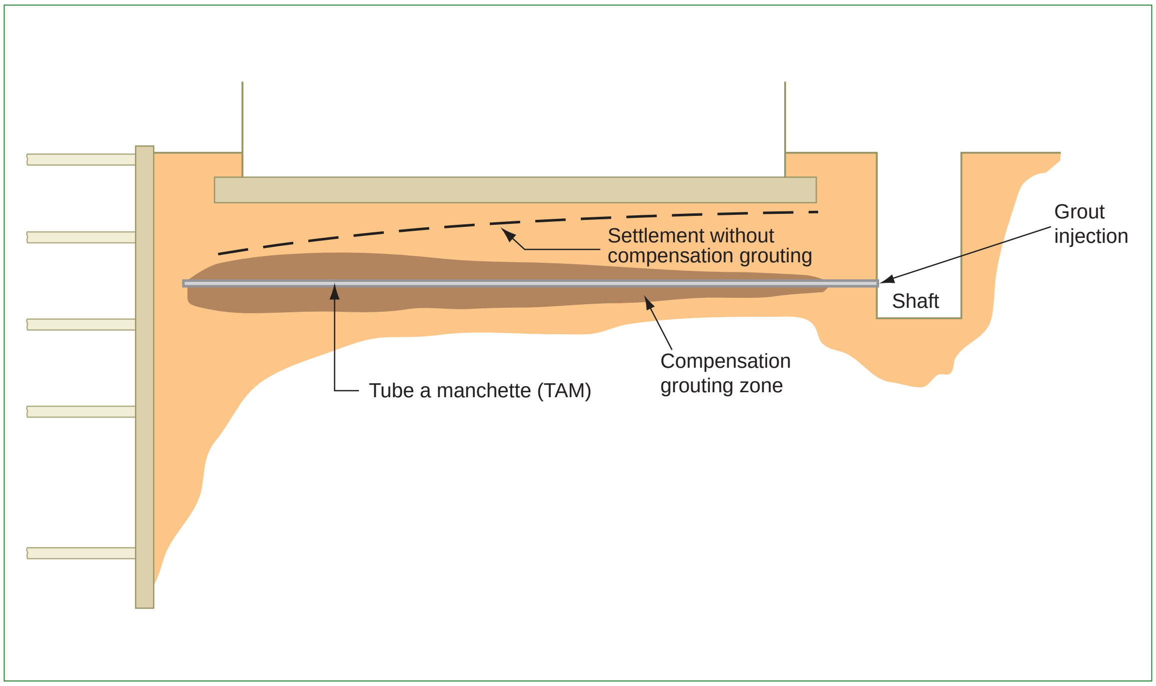

Fig 10.6 Compensation grouting

associated distortion of the structure. Therefore appropriate precise and reliable monitoring, together with rapid processing, interpretation and dissemination of the observations, forms an essential part of the whole operation.

Grouting has to be carried out in stages that match those of the excavation and de-watering, and thus form a series of small amplitude settlement and 'jacking' cycles (without the use of mechanical jack components). The aim is to minimise the amplitude of these cycles of settlement and jacking. It is not in the spirit of compensation grouting to carry out the 'jacking' in one operation after all the settlement has occurred.

Usually there are three stages to the grouting operations:

- Before excavation. Generally termed 'pre-treatment' or 'conditioning'. This is a preparatory phase to make the ground 'tight' following the installation of the grouting tubes (*tubes à manchette* or TAMs; see Section 10.5.2) and to ensure rapid response is obtained when it is required. Sometimes this stage results in some net uplift, or 'heave', of the building.
- During excavation. This phase is truly 'compensation' or 'concurrent' grouting, where injections are made contemporaneously with excavation such that movements are minimised and kept within pre-defined limits. Management of the concurrent grouting operations requires rapid decisions on proposed locations and volumes of injection, which may be modified on a shift-to-shift basis depending on the interpretation of performance. In other words, the planning of successive cycles of injection is informed and refined by the information gleaned from the earlier cycles.
- Between or after excavation. This stage is essentially grout jacking aimed at reversing settlements that have already occurred, to manage slow continuing settlement or to pre-lift in anticipation of settlement from further excavation. This phase is referred to variously as 'observational' or 'corrective' grouting.

The properties of the ground determine the design of the grout mix which, in turn, affects the shape and extent of the grout intrusion. For stiff clays, relatively fluid grouts are used which can spread laterally some distance along thin (1-2mm) horizontal fractures that are formed by the grouting process - hence the term 'fracture grouting'. In granular soils a more viscous grout is often used to form bulbs of grout by compacting the soil, termed 'compaction grouting'. Special grouting techniques and grout mixes have been developed for granular soils that permit repetitive grouting with more extensive intrusion from the grout pipe.

Compensation grouting was used very successfully for protecting a number of buildings along the route of the Jubilee Line Extension[10.1], for example the clock tower of Big Ben, see Figure 10.7. It is however an expensive undertaking. When adopting the Observational Method, compensation grouting can be used as a contingency action that may be adopted if the monitoring reveals behaviour outside the established acceptable limits (see Sections 1.3.4, 2.6 and Appendix E).

10.5.2 Ground improvement

The strength or stiffness of the ground affecting a sensitive building may be improved by several methods, including grouting, particularly permeation grouting, and drainage. These are used more as a way to improve the stability of the excavation, by controlling the flow of water in it, rather than as a protective measure. They may also reduce the magnitude of ground movement.

In granular materials, permeation grouting comprises the injection of grouts (often silicates) to fill the voids between the particles to create a soil mass whose permeability is significantly reduced.

Fig 10.7 Drilling rig lowered into shaft for installation of TAMs beneath the Big Ben Clock Tower © AMEC

In the case of London's Terrace gravels the natural permeability is frequently in the range of 10^{-2} to 10^{-3}m/s, which reduces to about $5x10^{-7}$m/s when effectively treated. Its strength and stiffness are also significantly increased.

Permeation grouting is most commonly carried out by controlled injections of known volumes of grout at specific points along the length of a pre-installed tube. The tube is fitted with ports at intervals which are covered externally by rubber sleeves. Any set of ports can be isolated by means of inflatable packers and grout is then injected with both the pressure and volume controlled, the grout is unable to re-enter as the rubber sleeve acts as a one-way valve. The system is known as 'tubes à manchette' or TAMs. Successive ports may be used and one of the great advantages of the system is that repeat grouting can be carried out through the same ports.

The homogeneously stiff raft thus created allows compensation grouting (see Section 10.7) to be used to best advantage.

10.5.3 Structural strengthening

Structural strengthening of the ground is defined as the introduction of structural elements that are neither part of the new basement nor attached to the structure to be protected. One example might be a curtain or cut-off wall between the two, the theory being that this wall will settle less than the ground and so reduce the settlement on the side remote from the excavation. If such a measure was being considered it would be important to ensure that the movements induced during its construction did not negate any beneficial effects its presence might bring during excavation.

10.5.4 Groundwater control

For effective excavation through granular materials, control of groundwater is essential, either by continual de-watering from sumps or well points or by grouting and a single de-watering. Successful control significantly reduces the magnitude of ground movements associated with excavation.

Where the toe of the retaining wall surrounding the basement terminates in granular material, drawdown of water inside the excavation will lower the water table for a considerable distance around (perhaps up to 2km away) and existing structures over a large area may become potential victims of settlement. To reduce the intellectual work associated with assessing the risk to all these buildings and to reduce the consequential physical work that might be needed, it may be prudent to consider: using re-charge wells; or, to avoid re-circulating a large volume of water, to extend the walls to a less permeable layer below; or to use permeation grouting between the toe of the wall and the less permeable layer below.

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11 Materials, workmanship, durability and water-resisting construction

11.1 Introduction

Durability and water resistance depend on:

- design and detailing
- materials
- construction
- workmanship
- environment.

These factors should not be considered in isolation. Material specification and selection and the relationship of workmanship to site conditions and buildability should be considered in the context of design and construction (see references 11.1 (clauses 2.1.1-2.1.4) and 11.2). Simplicity, ease and safety of construction will enhance the finished quality.

The basis should be good-quality reinforced concrete, with adequately controlled early-age thermal cracks that heal autogenously. A major conflict may arise if external membranes are also specified. Such impermeable membranes prevent autogenous healing (now known to be a mechanical sealing process) and encourage drying shrinkage cracks. Consequently, what may be intended as an additional line of defence can become the only one. Water stops may also contribute to the conflict, although BS 8007[11.3] at least is clear that water stops are unnecessary. While the Standard is specific about the objective of water-resisting construction, it is not clear enough as regards the inconsistency of using external impermeable membranes.

Specifying such combined systems may affect workmanship, adding to complexity and difficulty in application, particularly under heavy civil engineering conditions typical of cut-and-cover construction. Painting the earth-retaining face in such conditions with bitumastic paint may appear a cheap expedient, but may form only a short-life membrane, effective only long enough to inhibit autogenous healing. This removes the focus from the primary objective, namely good-quality reinforced concrete.

11.2 Reinforced concrete design for foundation engineering structures

The design of reinforced concrete structures is undertaken in accordance with relevant standards. Structural standards concentrate on ultimate and serviceability states of the permanent structure. Such codes tend to be directed at above-ground structures whereas those for foundations concentrate on geotechnical matters and refer structural design back to structural standards. This situation might be satisfactory were it not for significant differences between design and construction of above-ground structures and those pertaining to foundation engineering.

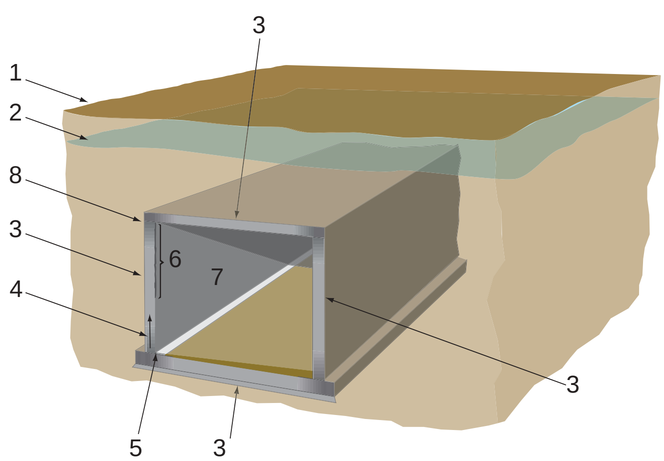
Apart from the predominant need in the latter to integrate structural design with soil-structure interaction and construction methods, there are particular differences as regards exposure conditions, durability, water resistance, crack control and movement joints. For instance, emphasis on concrete grade (compressive strength) in relation to durability requirements, based on generalised exposure conditions, is inadequate. Concrete permeability and crack control are more important than compressive strength but are less rigorously assessed or checked. Compliance with specified compressive strengths is much easier to achieve in practice, but does not provide an intrinsic measure of durability. Moreover, increasing cement content (for higher concrete grades) to improve durability can be counter-productive. Higher cement contents can make concrete more permeable and may increase the risk of alkali-aggregate reaction and early-age thermal cracking by raising hydration temperatures. For cast-in-place piles, concrete that is too strong also makes trimming back undesirably arduous.

A more appropriate approach to durability for buried structures is achieved with plasticisers, providing the necessary workability with low water/cement ratios, and cements blended with ground granulated blast furnace slag or pulverised-fuel ash.

Robust simplicity is the key. Movement joints, for example, should be avoided. The global effects of temperature and long-term drying shrinkage in buried structures are usually not critical[11.4-11.6], their primary effects tending to be differential through thick reinforced concrete sections. Shrinkage cracks will be exacerbated by the use of external impermeable membranes.

Total movements are thus minimal and the associated strains adequately controlled by longitudinal reinforcement initially required for early-age thermal crack control. Movement joints are always potential weak points and vulnerable to seepage and durability problems, particularly with large shear forces, as these will significantly concentrate stresses and complicate associated structural detailing. Also, structural standards do not adequately cover the design of struts that have lateral restraint and/or axial loads, both of which are deflection-dependent. Neither is shear in circular sections comprehensively addressed, a matter relevant to bored pile design and particularly to piled walls.

Other aspects of shortfalls in structural standards are discussed below, particularly with regard to the implications of chlorides on the durability of basements. More case histories, supported by comprehensive information and high-quality data, also need to be generated. The industry as a whole is notably poor in this regard. For example, considering the volume of reinforced concrete placed annually, water-retaining/excluding codes have been based on only one case history. Most empirical data has been derived from laboratory studies. It is worth considering the more frequent application of research to individual projects since, apart from creating a positive connection between design and construction, it calls for the production of reliable data of high quality. Current standards therefore present immediate problems to the designer in producing safe, appropriate and economic designs for foundation engineering structures. There is a need either for the existing codes to be made more specific and comprehensive or for a separate foundation engineering code to be drafted.



1 Ground level
2 Water table
3 Permeation diffusion and wick action
4 Wick action
5 Wick action and wetting and drying effects (concentration of salt etc. in drainage water due to evaporation)
6 Wetting and drying effects
7 50% relative humidity 20°C
8 Seepage through joint or crack

Fig 11.1 Schematic representation of various chloride, sulphate and alkali ingress mechanisms operating on a buried box type structure

11.3 Durability

Underground structures must be durable, since there is limited facility for inspection, maintenance and repair. However, exposure conditions with uncontaminated groundwater are moderate and typically much less severe than those to which surface structures are subjected. An important exception to this is buried box structures exposed to significant external water pressure in the presence of chlorides or sulfates, and so groundwater composition must always be checked. This is discussed in more detail below.

Exposure conditions at entrances to basements or, in cut-and-cover tunnels, adjacent to portals with permanent open-cut sections, may also create more critical conditions through higher temperature variation and freeze-thaw cycles. Buried structures generally, however, with a metre or more of ground cover do not usually suffer significantly from the deleterious effects of freeze-thaw action and wide variations of temperature in the permanent condition as would apply to a structure above ground. However, significant exceptions are possible and the overall environment needs careful appraisal.

The basic approach to durability is to relate concrete grade (compressive strength), cement content (and, in the case of sulfates, cement type or cement replacement), water/cement ratio, crack width, and depth of cover to the reinforcement, to the conditions of exposure. Reference 11.2 gives a detailed review of the durability of reinforced concrete structures, emphasising the importance of the four 'Cs': Constituents of the mix; Cover; Compaction and Curing. Notwithstanding the above considerations, the presence of aggressive agents such as chlorides or sulfates should always be treated with caution. Potential concentration levels of such chemicals, and the measures necessary to resist their effects, must be considered in each structure.

Buried box structures subject to external water pressure merit particular consideration. Permeation and diffusion conditions involved here with the wet outside/dry inside situation can lead to transmission, and eventually to a harmful concentration, of chlorides or sulfates within the structure. Without appropriate protection, reinforcement may corrode and concrete spall, well within the design life of the structure, even with low levels of such chemicals in the groundwater.

Wick action due to continual evaporation (encouraged by ventilation and air-conditioning) from the internal surface, the supply of oxygen to the structure and the electrical continuity of reinforcement[11.7] tend to exacerbate the situation (see Figure 11.1)[11.8].

Not only is chloride ingress faster, but corrosion, once initiated, will be more rapid owing to the 'hollow leg' phenomenon, the drier interior acting as a cathode, with potentially severe localised pitting corrosion developing at the outer face from macro-cell action under low oxygen conditions. For such exposure conditions with chlorides or sulfates, low concrete permeability, crack control, and, most particularly, water resistance assume special importance to achieve durability. Extra protection measures may also be required: robust and reliable impermeable membranes (such as welded steel sheeting, as used in immersed tube tunnels), cathodic protection, epoxy-coated reinforcement and stainless-steel reinforcement are relevant considerations, the last three relating specifically to protection against chlorides.

11.4 'Waterproofing'

Water resistance can be a controversial subject, not least because interpretations differ on the meaning of the terms 'waterproof' or 'watertight'. The preferred description is 'water-resisting', as used in the title of CIRIA Report 139[11.9], Section 1.4.3 of which gives useful definitions of these terms. This 190-page document is summarised by CIRIA Report 140[11.10].

In practice, 'watertightness' cannot be ensured and in general is not necessary. It is essential to assess the requirements for water resistance in the context of grades of performance[11.9,11.11], and the client must decide the required degree of moisture and water resistance, so that costing and design can take these into account.

Fig 11.2 Central Expressway II: phased passage under River Singapore in cofferdam © Benaim

Water management is the key. Water should be prevented, as far as possible, from entering the basement by the use of water-resisting construction. However, the designer should recognise that this limit state of water resistance is sure to be tested to the full in the life of the structure (see Figure 11.2), and should manage the inevitable entry of water. Likely points of entry should be determined, and the water collected and dealt with in a maintainable manner.

The basis of water-resisting reinforced concrete is good-quality design and construction, paying particular attention to the four 'Cs' (see Section 11.3 above and reference 11.2), to achieve dense, low-permeability concrete with well-controlled cracks. The combination of internal and external restraint to early-age thermal contraction, inherent with in-situ construction of thick reinforced concrete sections, tends to make cracking inevitable (see Section 11.5).

CIRIA Report 139[11.9] classifies water-resisting methods as being one (A, B or C), or a combination of two (C+A or C+B), of three types:

- A Structure requiring the protection of an impervious membrane (i.e. tanked)
- B Structure without a membrane (i.e. integral)
- C Drained cavity (for use with Type A or Type B structure or alone)

The combinations of Types are illustrated in Table 11.1. (An erratum in early versions of CIRIA Report 139 is reported in later versions: the labels for the three Figures illustrating Type C construction in Figure 1.1 of that report should read C1, C3 and C2 from top to bottom.)

The requirements for water resistance are placed by BS 8102[11.11] into four performance grades for the internal environment:

1. Basic Utility
2. Better Utility
3. Habitable
4. Special

Grade 1 is satisfied by the normal provisions of BS 8110[11.1] (see Table 11.2).

The Special grade is used where a vapour-controlled environment is required, as is necessary for sensitive equipment or archive storage. Ventilated cavity construction, with a vapour-proof internal membrane that can be inspected and maintained, is then appropriate. Failure to provide such a vapour barrier can lead to considerable client dissatisfaction.

Table 11.1 Combinations of water-resisting methods

Water-resisting methods (form of protection)	Type A	Neither A nor B	Type B
Type A or B or C alone	A: With membrane	C: Drained cavity alone (avoid)	B: Integral: without membrane
Type C with A or B	CA: Drained cavity with membrane		CB: Drained cavity: without membrane

Table 11.2 Provisions for Performance Grades

From Table 1 of BS 8102: 1990[11.11]				Abbreviated commentary given by CIRIA Report 39[11.10]
Grade	Basement usage	Performance level*	Form of protection	
Grade 1 Basic utility	Car-parking; plant rooms (excluding. electrical equipment); workshops	Some seepage and damp patches tolerable	Type B with RC design to BS 8110[11.1].	Visible water and BS 8110 crack width may not be acceptable. May not meet Building Regulations for workshops. Beware chemicals in groundwater.
Grade 2 Better utility	Workshops and plant rooms requiring drier environment; retail storage	No water penetration but moisture vapour tolerable	Type A or Type B with RC design to BS 8007[11.3].	Membranes in multiple layers with well lapped joints. Requires no serious defects and higher grade of supervision. Beware chemicals in groundwater.
Grade 3 Habitable	Ventilated residential and working, incl. offices, restaurants, leisure centres	Dry environment	Type A or Type B with RC design to BS 8007, plus Type C with wall and floor cavities and DPM.	As Grade 2. In highly permeable ground, multi-element systems (possibly including active precautions, and/or permanent and maintainable under-drainage) probably necessary.
Grade 4 Special	Archives and stores requiring controlled environment	Totally dry environment	Type A or Type B with RC design to BS 8007 and a vapour-proof membrane, plus Type C with ventilated wall cavity and vapour barrier to inner skin and floor cavity with DPM.	As Grade 3.

* See CIRIA Report 139[11.9] for limits on environmental parameters

All grades should be based on water-resisting reinforced concrete construction. The Basic Utility grade would generally be relevant to car parks and tunnels. The other grades relate, for example, to office space, the housing of sensitive equipment and underground stations. For Basic Utility grades, normal reinforced concrete is generally adequate, subject to appropriate design, detailing and construction. There should be no need, in the absence of aggressive chemicals such as chlorides or sulfates, to provide additional protective measures such as impermeable membranes.

It should be borne in mind that water tables are rising in some major cities and that measures exceeding those needed for present conditions (see Section 3.4) may be appropriate. Specifications for transportation tunnels, for example, typically permit a limited amount of controlled seepage. Undue seepage should not be tolerated, and seepage should not occur in good-quality construction.

Seepage may arise through early-age thermal cracks or through cracks induced by flexure in the longitudinal direction of low-height structures such as single-storey tunnels, such cracks tending to pass right through the section. The design should limit the widths of such cracks to 0.2mm[11.12,11.13]. At this width or less, cracks tend to heal autogenously[11.13,11.14]. This self-sealing process is usually effective within a few months of the start of seepage (usually well before the commissioning of new works) and can be encouraged by the controlled application of fresh water to the external concrete surface after the initial curing period.

For the Habitable grade, seepage or damp patches are unacceptable. Preventing the visible penetration of water is frequently achieved by the same reinforced concrete design as for the Basic Utility grade (namely Type B), but with internal drained cavity walls (Type C) provided. The use of external impermeable membranes may also be considered but with the important caveat that it potentially conflicts with the performance of water-resisting concrete. It is important to note also that, apart from the significant cost of membrane systems, their success depends on the highest quality of workmanship and materials.

Particular attention must be paid to simplicity in detailing, avoiding complex geometry, and to good site supervision. The quality of the concrete surface, especially with bonded membranes such as bitumen sheeting, is critical and is not easy to achieve in the arduous civil engineering conditions of cut-and-cover construction. There is usually significant pressure on the construction programme to backfill completed sections as soon as possible, often while the concrete is still in its early hydration phase. Such conditions are hardly conducive to the proper application of most bonded membrane systems. Leaks caused by defects in external membranes are practically impossible to locate and repair, since the water invariably enters the structure internally through cracks or other vulnerable points, such as any movement joints, at some distance from the external defect.

Potential of leakage for diaphragm wall construction in basements is frequently greatest in panel joints and joints between the lowest basement slab and the diaphragm wall. An example of the leakage on a vertical panel joint in a diaphragm wall is shown in Figure 11.3, and it should be noted that this particular leakage path can only be addressed by remedial grouting.

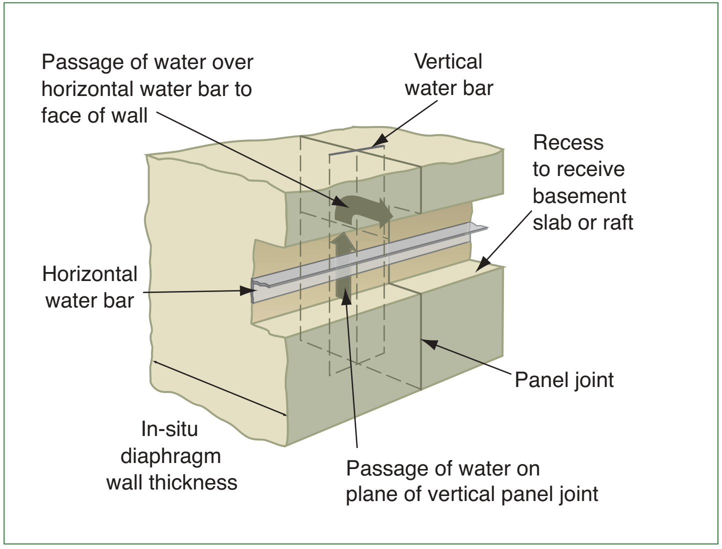
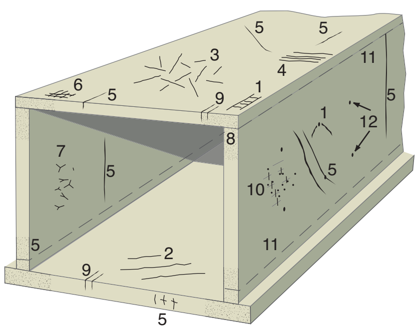


Fig 11.3 Passage of water on a vertical panel joint in a diaphragm wall

A further critical consideration is that many such latent defects are dormant and tend to manifest themselves only after the structure is commissioned. Consequently, the cost of the disruptive remedial works can be onerous. A contributory factor to the incidence of leaks from such latent defects is that applying the membrane early may prevent or retard autogenous healing of the cracked concrete. This process involves the transport of debris and fragments of soluble calcium hydroxide, $Ca(OH)_2$, within the crack, forming the insoluble compound calcium carbonate, $CaCO_3$, on contact with the carbon dioxide, CO_2, in the air. Calcium hydroxide is more freely available in young concrete and, moreover, long-term cracks in mature concrete will have suffered some degree of carbonisation from the air inside the structure. Such carbonisation of the crack surfaces will potentially inhibit the self-sealing process.

In short, there is much to be said for the simplicity of straightforward, robust, water-resisting concrete construction that, in the absence of an external impermeable membrane, can be fully tested and proved before handover. Such high-quality reinforced concrete should form the basis in cut-and-cover construction, and this approach must be followed with diaphragm or piled walls where the use of external 'waterproofing' is not possible.

Fig 11.4 Boon Keng Station, Singapore: couplers in diaphragm walls for crack control



1 Plastic settlement (over reinforcement and shutter ties)
2 Plastic shrinkage (diagonal)
3 Plastic shrinkage (random)
4 Plastic shrinkage (over reinforcement)
5 Early age thermal contraction (thick sections)
6 Crazing
7 Alkali aggregate reation
8 Shear
9 Tension bending
10 Thermal shock
11 Kicker
12 Shutter tie holes

Fig 11.5 Examples of intrinsic cracks in a reinforced concrete box structure

CIRIA Report 139[11.9] provides a useful assessment of the supplementary measures related to drained cavities and external 'waterproof' membranes. Attention to detailing and geometry is necessary to minimise potentially vulnerable points, such as stress concentrations or reinforcement congestion that may lead to poor concrete compaction. Wherever possible, water should be directed to flow away from or past the structure. Top slabs, for example, should be detailed with falls, preferably not less than 1 in 50, and walls that dam flow down a hillside should be provided with a maintainable bypass for the water.

Finally, the presence of aggressive agents in the groundwater always needs careful consideration. Water stops do not guarantee watertight construction and manufacturers generally decline to be responsible for fixing the material on contracts.

11.5 Structural details

Emphasis in structural detailing, as in overall layout, should be placed on simplicity, buildability and durability. The positioning of construction joints and associated reinforcement details may be critical and can clash with construction methods. Using couplers for reinforcement bars can eliminate congestion and ease construction operations. For example, reinforcement couplers at the tops of walls for wall/top slab continuity facilitate the use of travelling shutters for both the wall and top slab pours by eliminating the horizontal projection of starter bars beyond the walls (see Figure 11.4).

Care is needed where large diameter bars cross the reinforcement cage of diaphragm walls horizontally, to be fitted with couplers at the inside surface: the upward passage of bentonite during concreting can leave prisms of bentonite under the bars which then offer a leakage path. Also, even where the vertical joint between diaphragm wall panels is equipped with water stops, the inner half of the joint offers a leakage path into the structure past the base and roof slabs, and injection grouting should be used here (see Figure 11.3).

11.5.1 Crack control

It is a fundamental fact that reinforced concrete cracks. It is inevitable in its hardened state when tensile strains arise from imposed loads. Cracks also occur in setting and hardened states as a result of settlement, thermal, and shrinkage effects. Various types of crack and their causes are categorised in Figure 11.5. This list does not include micro-cracking, those cracks arising from bad practice (such as using calcium chloride as an accelerator), or those resulting from inadequacies in constituents (e.g. alkali-aggregate reaction). It is important to control potential cracking from all causes and many of them can be eliminated by good design and construction. Further details on the causes and repair of cracking can be found in reference 11.4.

The importance of a robust approach, with fully controlled crack widths, has already been emphasised. Cracking from long-term drying shrinkage is often raised as a concern but generally its effect is secondary. This particularly applies to thick sections and conditions associated with buried structures. Creep and the fact that reinforced concrete in contact with the ground is unlikely fully to dry out mitigate the effects of long-term drying shrinkage. It has been demonstrated[11.15] that early-age thermal strains are far greater than those of long-term shrinkage and are the prime reason for cracking in retaining walls and similar reinforced concrete structures. Two principal causes of cracking, thermal effects and flexure, are considered in more detail below.

11.5.2 Thermal effects

Thermal effects result from two main factors: seasonal temperature variations and early-age thermal contraction. For buried structures, the latter is dominant. For example, in long railway tunnels under normal operating conditions, the annual temperature variation is low and would not typically exceed 10C. Below a depth of two metres, the seasonal variation of ground temperature is generally only a few degrees Celsius, although it may be greater in the ground adjacent to the structure. With adequate reinforcement to control early-age thermal contraction, these effects should not be critical.

Temperature variations tend to result in wider temperature differences through the thick reinforced concrete sections, rather than in variations of mean temperature, so the need to accommodate any overall movements is usually negligible. Temperature variations at entrances are higher and should be evaluated separately.

Early-age thermal effects are critical. Hydration temperatures generated in thick sections are high and the contraction is restrained both internally (by the core) and externally (by connecting sections cast previously)[11.12], or by direct soil frictional resistance. The design approach for reinforcement to control resultant cracking must take into account significant differences between thin and thick sections, and the associated influence of the maximum and minimum cracking mechanisms[11.15]. The maximum crack width for early-age thermal effects should be limited to 0.2mm[11.3]. Control of early-age thermal cracks is critical because:

- they pass right through the structural section, thus allowing seepage and groundwater to reach the reinforcement at both faces
- they tend to run parallel to, rather than across, main flexural reinforcement. Site surveys have shown that cracks across the main reinforcement are unlikely to create significant corrosion problems but those parallel to reinforcement often contribute to corrosion. The presence of chlorides always needs particular consideration and the incidence of any cracking can then have a critical influence on corrosion and durability. Control of thermal cracking is enhanced if the distribution (or longitudinal) reinforcement is outside the main flexural steel. This usually not only simplifies steel fixing, but the main reinforcement will have extra physical cover.

Designers may also consider the lead taken by the designers of immersed tube tunnels, who have controlled early thermal cracking by passing cooling water through pipes embedded in the hydrating concrete or, more recently, in the steel shutters.

11.5.3 Flexural cracking

Flexural strains from imposed loads usually dominate in the transverse direction of tunnels. Deflections in the longitudinal direction are generally not critical and the quantity and spacing of longitudinal reinforcement is then governed by the requirements for early-age crack control. Flexural cracks taper towards the compressive zone and do not pass through the section. Their influence on durability is also not generally as critical as early-age thermal cracks. Consequently, limiting flexural crack widths at the ‘design’ surface of the concrete (at a distance equal to the nominal cover beyond the outermost bars) to the same maximum of 0.2mm may not be necessary. BS 8110[11.1] sets this limit at 0.3mm, for example. For a given bar cover and spacing, reinforcement area is inversely proportional to crack width, so changing the maximum width from 0.3mm to 0.2mm increases the reinforcement required by around 50%. The cost is therefore significant and the benefits of reducing the flexural crack widths in this range are questionable. Table 3.7 of CIRIA Report 139[11.9] provides further advice on this alternative basis to BS 8102[11.11].

Another major consideration is potential steel congestion arising from stringent limits on crack widths. Poor compaction of concrete and the plane of weakness associated with layers of closely spaced reinforcement bars may lead to serious durability problems.

11.6 Design guidelines

It is appropriate to draw attention to some important principles of design that need to be considered.

1. The client, on advice from his designer, should decide which of the four grades of water-resisting construction he requires for his basement. This decision should involve consideration of any reduction of floor space, where relevant, and the full costs of construction[11.9].
2. Whichever grade is used, consideration should be given to designing an external drainage system, if practicable, so that any external water pressures are reduced (see Section 11.8).
3. A 100% fully 'watertight', 'waterproof' or 'vapour-proof' basement is unlikely to be achieved in practice. The designer has to consider the practical construction methods available for any basement. The details should take into account the proximity of any adjacent structures or services, the type of concrete structure envisaged and that concrete is to be placed and compacted to form a dense, homogeneous and durable mass for water-resisting construction.
4. Trade names are not used in this document. It is for the designer/specifier to agree which materials are to be used to cover generic terms, such as, 'silt-resistant membrane', 'drainage tiles', 'damp-proof membrane', 'bituminous membrane', 'chemical colloidal grout', 'hydrophilic strip', 'reinjectable grout tube' and 'water stop'.
5. The use of an external tanking system will not necessarily produce a satisfactory 'Special' or 'Habitable' grade of basement construction, and may be counter-productive.
6. The cheapest form of construction, the 'Basic Utility' grade, cannot be expected to produce a basement that will meet, at a lower cost, any client's expectations for 'Special' or 'Habitable' grades.
7. There are proprietary systems for water-resisting construction. The designer/specifier should check claims made for them before deciding whether they are practicable and whether they should be included in the contract specification.
8. No services should be installed within any drainage cavity.
9. After construction of a 'Utility' type basement, it is not practicable to alter the design to full water-resisting construction. The effect on future users should be made known to the client.

The design process is iterative: a useful flowchart is given in Figure 3.1 of reference 11.9.

11.7 Maintenance

To ensure that groundwater is excluded, regular maintenance is essential. The design parameters used for the design life of the basement should be given to all relevant parties. A maintenance schedule should be prepared, addressing such items as the clogging of drainage systems by dust.

Under-slab and cavity wall drainage systems must be kept free of all obstructions, including the build-up of calcium carbonate. Access traps in walls and slabs must be easily accessible for maintenance.

Sump pumps, their standby pumps and automatic pressure switches, and water overflow indicators need to be checked regularly to ensure they are working. It will sometimes be prudent to provide monitoring systems directly linked to the Building Management Control panel. After a period of dry weather, extra checks should be carried out to ensure the system is operational.

11.8 Basement grades, types and details

For a satisfactory outcome to a basement project, close collaboration between the following is essential:

- client
- design team
- contractor and specialist subcontractors
- approval authorities (local, fire, etc.)

Clients must specify precisely the use for which the basement is intended. Table 11.2 lists basement grades and typical generic solution types to match these.

For dwellings, useful solutions and details are given in the *Basements for Dwellings Approved Document*[11.16]; Section 2: Site Preparation and Resistance to Moisture. Further details are given in *Basement Waterproofing; Design Guide*[11.17] and *Site Guide*[11.18].

Further information on construction methods and sequences for certain situations and comprehensive examples of details and construction methods can be obtained from CIRIA Report 139[11.9] and BS 8102[11.11]. Table 11.3, the precursor to many of these details, is reproduced for completeness, but reference should also be made to the above.

11.9 Steel construction

11.9.1 Materials and sections

Steel construction generally comprises steel sheet piling for earth and water retention and steel structural sections for any supporting structure of walings and struts. Such materials are available in two basic qualities, mild steel and high-yield steel. The standard designation for these is BS EN 10025[11.19], Grade S275 and Grade S355 respectively for structural sections, and for sheet piling is BS EN 10248-1[11.20], Grade S270GP and Grade S355GP respectively. Copper-bearing steel is also available. Structural sections are rolled to the shapes, sizes and tolerances given in BS 4[11.21], and sheet pile sections to those given in BS EN 10248-2[11.22]. See also Chapter 4.

A working bending stress of 65% of the minimum yield stress is used for permanent structures made from steel sheet piling, with an increase of about 12% for temporary structures or of up to 25% for temporary phases suffered by permanent structures.

11.9.2 Durability

Given a plentiful supply both of water and oxygen to its surface, steel will corrode. A reduction in the availability of either to a particular surface of the steel will correspondingly reduce the rate of corrosion. In most underground structures, the face of sheet piles in contact with the soil will be subject generally to low rates of corrosion because there is little or no oxygen in the soil, especially below the water table. Research and inspection of redundant structures has shown that corrosion rates are too small to be measured reliably. Information published by the manufacturers of such piles suggests a maximum total corrosion allowance (for the sum of the two faces) of 0.15mm/year for design purposes. Accelerated (low water) corrosion (ALWC) under certain bacteriological conditions, leading to premature localised failure, has been reported in recent years.

The effects of corrosion are disregarded in temporary works. Where the piles form part of permanent works, the exposed inner face requires more detailed consideration depending on the type of finish. The surface may be completely protected either by paint or by an in-situ skin of concrete. Alternatively, the steel may be left unprotected with a decorative skin of brickwork or precast concrete panels mounted in front of it and with a small air gap (usually 100mm) between the steel and the facing. In the latter, the sheet pile interlocks may be left unsealed against the ingress of groundwater and any seepage allowed to run down the face of the piles to a drainage gully at the base. Where such seepage is likely to persist for the life of the structure, additional allowance for corrosion must be made.

11.9.3 Water-resisting construction in steel sheet-piled structures[11.23]

Interlocks in steel sheet pile walls should not be relied upon to be completely watertight. Where complete watertightness is essential, the interlocks can be sealed, at least as far down as the excavation level for the basement, by welding or by caulking. If interlocks are not artificially sealed, fine soil particles within the soil mass will often be carried in by water seepage, and will tend to lodge and eventually form an effective seal. In soils with few fines, seepage may persist indefinitely.

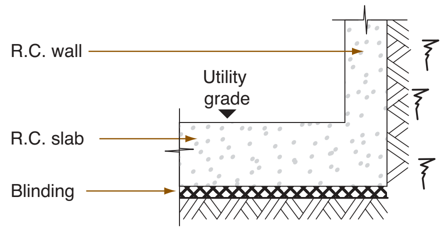
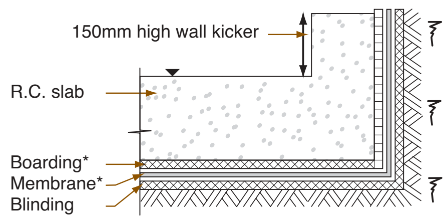
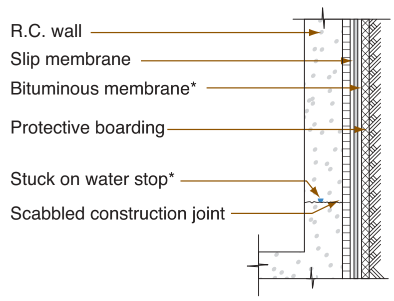
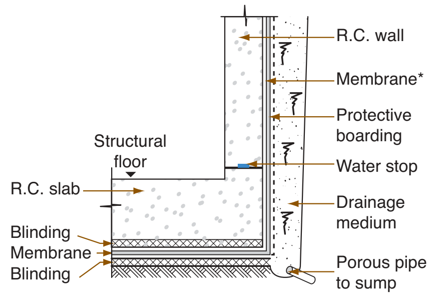
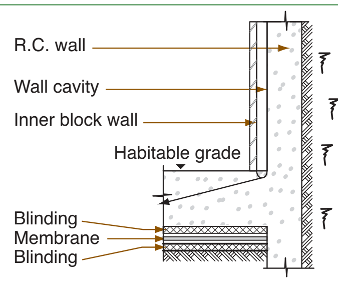
Gaining in popularity are steel sheet piles whose clutches are fitted with a containment device filled with a bituminous or other compound, pressurised by the pitching and driving process to form a seal. A bonus is that their water-resistance extends below excavation level, improving the passive pressure available in the soil at excavation level during construction.

Special provision is required to prevent seepage between contact faces of sheet piles and the edges of the bottom slab. If used, compressible water-resisting membranes are best located where contact pressure is greatest, usually towards the upper surface of the slab.

Water resisting construction in a sheet piled basement can be achieved by the inclusion of a suitable membrane between the face of the permanent sheet piles and the rear of the reinforced concrete basement wall as shown in Figure 11.6.

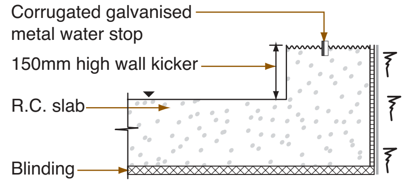
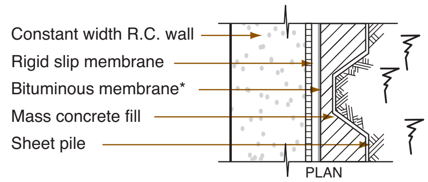
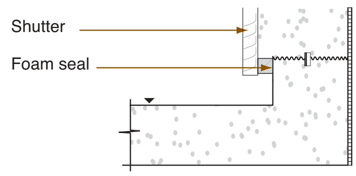
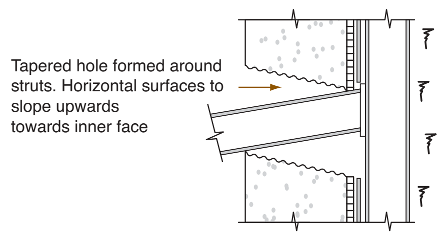
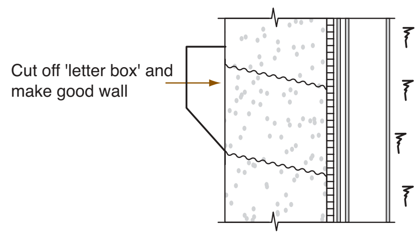
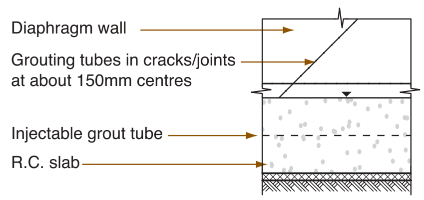
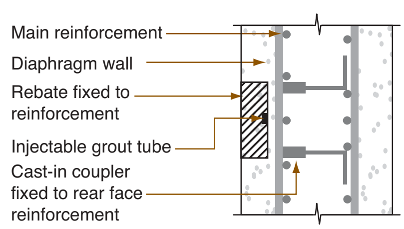
Fig 11.6 Water resisting membrane construction on a sheet pile wall for a deep basement

Table 11.3 A selection of grades and details (Pages 104 – 107)

Grade	Client requirements	Method	Sketch
Grade 1: Basic Utility	Basic car park. Water can exist on walls and floors.	Raft slab and reinforced concrete wall or faced up diaphragm or piled retaining wall.	
Grade 2: Better Utility	Water can exist on walls and floors.	Excavate to formation level. Lay 50mm C20 blinding. Trowel on bituminous membrane.*Lay protective boarding or slip membrane.	
	Slabs	Shutter, reinforce and concrete slab plus 150mm high wall kicker. Scabble construction joint. Place strip water stop on centre of wall.*	
	Walls	Trowel damp proof membrane on existing wall and fix protecting board*, or shutter, reinforce and concrete wall.	
Grade 2: Better Utility	External space available for installing drainage system and external membrane.	Install drainage. Place strip water stop.* Shutter, reinforce and concrete wall. Strike formwork. Trowel dampproof membrane on wall.* Fix protective board or slip membrane. Place gravel drainage layer.	
Grade 3: Habitable	For offices and non critical zones where some condensation can be permitted in exceptional circumstances.	Drained wall cavity. Traditional raft slab.	

*Note: When required by local or other authority.

Table 11.3 A selection of grades and details (cont)

Grade	Client requirements	Method	Sketch
Grades 3 & 4: Special and habitable	Traditional perimeter reinforced concrete walls. Water not to reach interior of finished basement wall.	150mm high wall kicker to be cast monolithic with raft. Puddle in* corrugated galvanised metal water stop prior to initial set of concrete. Scabble construction joints. Wall section to be constant width. Trowel on bituminous membrane.* Rigid slip membrane to be fixed to external wall. Wall cast in lengths up to 6m long. Scabble and clean construction joints.	  
		Foam grout seal on all formwork construction joints. Around struts and shores form a tapered hole. After permanent support slabs cast remove struts and cast infills using letterbox shutter (but see preferred detail page 106)	
		Cut off letterbox. After structure loaded, check walls for leaks. Grout seal with chemical colloidal grout. Grouting tubes at about 1500mm centres.	
	Alternative diaphragm wall type perimeter walls.	Rebates formed and reinforcement couplers cast in with diaphragm wall panels at floor levels.	
		Cut out box-out formers. Remove cap and infill of grease from coupler. Torque in threaded reinforcing bars for slab construction.	

*Note: When required by local or other authority.

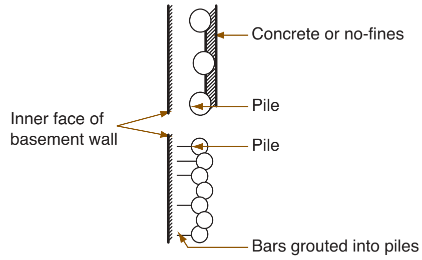
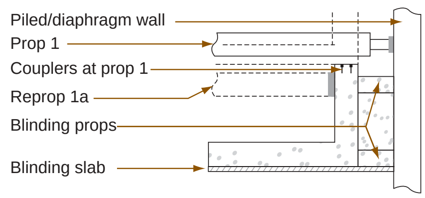
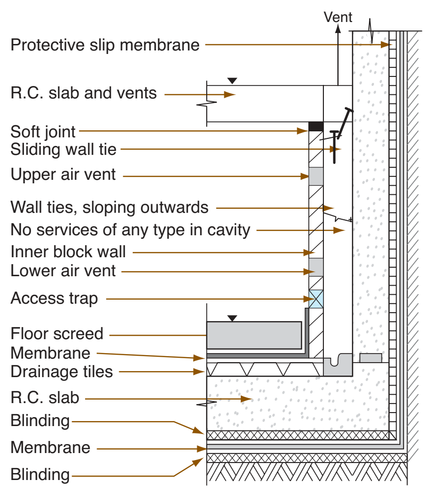
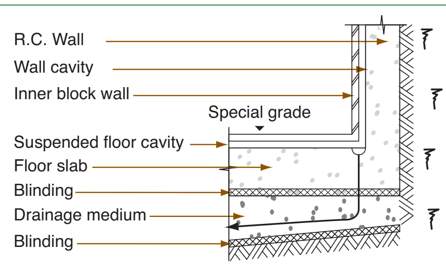
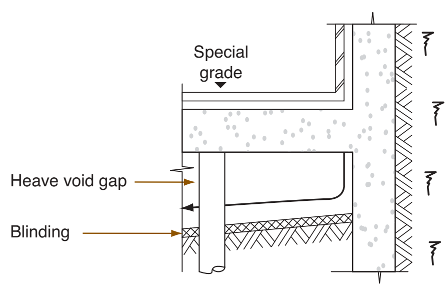
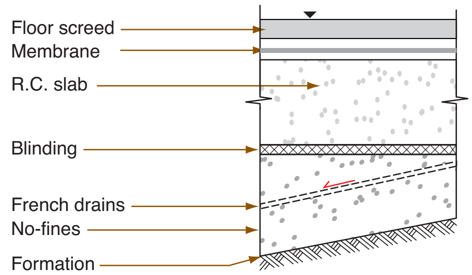
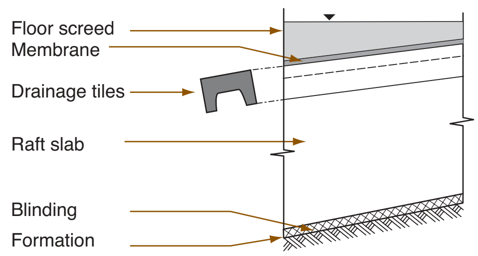
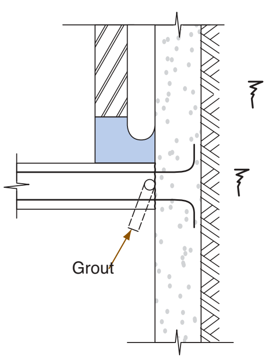
Table 11.3 A selection of grades and details (cont)			
Grade	**Client requirements**	**Method**	**Sketch**
Grades 3 & 4: Special and habitable (continuation)	Pile type perimeter wall.	For non secant pile types, cut out between piles and back fill with concrete or no-fines . concrete For floor slabs drill and grout-in reinforcing bars. Drill and grout bars into piles for facing wall (if required).	
		Preferred re-propping detail to avoid discontinuities at letter boxes.	
	Inner leaf cavity wall construction.	Construct concrete upstand to inner leaf wall. Cast drain to fall towards gullies. Treat inside face of concrete. Grout wall/slab joint. Install access trap in wall above gulley. Install air vents in inner leaf of wall at low and high levels. Any wall ties to fall towards outer face. Soft joint at top of wall. No services of any type to be installed in drainage cavity.	
Grade 4: Special	For offices or plant rooms requiring walls and floors to be completely dry, and no penetration of water vapour through the walls.	Drained cavity and under floor drainage. Drainage to run to a separate drainage system.	
	As above but heave not to affect services installed below slab.	Suspended slab over heave gap.	

Table 11.3 A selection of grades and details (cont)			
Grade	**Client requirements**	**Method**	**Sketch**
Grade 4: Special	Basement slab. Water not to reach finished floor.	Excavate to formation level. Lay 'Herring Bone' French drains to special drainage system. Wrap pipes in silt resistant membrane. Lay no fines concrete or 40mm all-in aggregate plus fibre filter. Shutter, reinforce and concrete raft slab. Lay damp proof membrane. Lay floor screed to protect membrane.	
Grade 4: Special (alternative)	Basement Slab. Water not to reach finished floor.	Excavate to formation level. Lay blinding. Shutter, reinforce and concrete raft slab. Lay drainage tiles, to falls, gullies installed. Lay damp proof membrane. Lay floor screed.	
Grade 4: Special	Suspended floor slab. Water not to bypass cavity construction.	Prepare reinforced concrete or diaphragm or piled wall and fix slab continuity reinforcement. On wall at middle of slab fix a continuous run of injectable grout tube. Grout points below (or above) slab soffit. Cast suspended floor slab. Cast concrete upstand and drain. Pump in chemical colloidal grout.	

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12.1 General

Safety is an important consideration on every site and there are legal obligations to be met. Safety is not optional. It is all too easy to have regard to the safety of individual operations and to forget that, on any site where there may be congestion of moving plant and equipment and a high level of noise, a person may easily become temporarily disoriented or diverted long enough to cause a serious accident. The accident record of the construction industry indicates clearly that it is a hazardous activity. Members of the public may also be at risk from construction operations.

Mention should be made of the high risk of accidents with excavations less than 6m deep. Of those accidents involving fatalities or major injuries from 1996 to 2001 for which details are available, analysis shows the following causes:

- unsupported excavation: 54%
- working ahead of support: 12%
- inadequate support: 16%
- unstable slopes of open cut: 6%
- other causes, principally unsafe machine operation: 12%.

Advice on safety in shallow excavations is provided in HSG 185 - *Health & Safety in Excavations*[12.1].

12.2 Legal

12.2.1 Health and Safety at Work, etc. Act 1974

In recent years, the issue of health and safety has assumed greater importance. In addition to common law liabilities arising from breach of duties owed in tort, health and safety legislation is part of the criminal law. The Health and Safety at Work, etc. Act 1974 (HSWA)[12.2] imposes broad and wide-ranging duties on all parties with the general purpose of securing the health and safety of persons at work. Within the legal framework of the general duties, there are many Regulations, often accompanied by Approved Codes of Practice (ACoPs) that take effect under the Act. The most important of these Regulations are described in this chapter. Inspectors have wide powers of investigation and enforcement, including the serving of improvement and prohibition notices, and the Act provides penalties for breach of its provisions including fines and imprisonment.

12.2.2 Management of Health and Safety at Work Regulations 1999

The Management of Health and Safety at Work Regulations 1999 (MHSW)[12.3] place duties on employers and the self-employed towards persons affected by the work. The main duties placed on employers are the assessment and reduction of risk, the need to establish emergency procedures, health surveillance, and information and training for employees. The duties in respect of risk assessment and reduction (Regulations 3 and 4) are considered particularly important.

12.2.3 Construction (Design and Management) Regulations 1994

These Regulations were the means by which the requirements of the Temporary and Mobile Construction Sites Directive were implemented in the United Kingdom. The main effect of the Construction (Design and Management) Regulations 1994 (CDM)[12.4] is to place duties on designers of 'structures' to assess the implications of their design on the health and safety of all persons affected by the building and maintenance of the 'structure' when in use. The definition of a 'structure' is wide-ranging. This is a somewhat radical departure from previous construction safety legislation, where most responsibility for safety in construction was placed on contractors.

CDM requires the client for a project to appoint a planning supervisor, whose main role is to co-ordinate the health and safety aspects of project design and initial planning. This is achieved by ensuring that:

- designers comply with their duties under the Regulations, particularly in the reduction and control of risk (Regulation 13)
- there is co-operation between designers for different parts of a project (Regulation 14(b))
- there has been Notification of the project to HSE (Form 10; Regulation 7)
- the Health and Safety Plan and File are prepared in accordance with the requirements of the Regulations.

The Health and Safety Plan (Regulation 15) should include:

- a general description of, and programme for, the project
- all information on the significant residual risks to the health and safety of those affected by the construction work
- details of the arrangements made by the principal contractor for the co-ordination and management of health and safety during the construction phase.

Once construction is complete, the client should retain the Health and Safety File (Regulation 14(d). It should contain information on the 'structure' relevant to the health and safety of those maintaining, repairing or renovating the structure.

12.2.4 Construction (Health, Safety and Welfare) Regulations 1996

The Construction (Health, Safety and Welfare) Regulations 1996 (CHSW)[12.5] consolidate the requirements of the Construction (General Provisions) Regulations 1961, the Construction (Working Places) Regulations 1966 and the Construction (Health and Welfare) Regulations 1966 into a single set of Regulations. They were drafted in a 'goal-setting' style, incorporating into British legislation the requirements of Annex IV of the Temporary and Mobile Construction Sites Directive (relating to workplace conditions, etc.). CHSW has introduced a number of new requirements for emergency lighting, traffic management, fire precautions and emergency procedures, means of escape in an emergency and welfare and site environment provisions. Essentially, however, the requirements are similar to the revoked legislation. Guidance related to these Regulations is provided in HSG 150 – *Health and Safety in Construction*[12.6].

12.2.5 Confined Spaces Regulations 1997

Some deep basements or parts of deep basements may be confined spaces within the meaning of the term in these regulations, e.g. basements constructed by any top-down procedure. The regulations, accompanying ACoP and guidance are contained in HSE publication *Safe work in confined spaces* L101[12.7].

12.3 Hazards

Most accidents are caused by falls or falling objects. In addition, when dealing with deep basement construction, the two main types of possible hazard are those affecting persons and property on the site; and those affecting adjacent property and its occupants.

12.3.1 Underground and overhead services

The presence of gas mains, a potential source of methane if damaged or fractured, electric cables, water mains and other services that may be damaged or cause injury must be investigated before work begins. Reference should be made to HSG 47 *Avoiding Danger from Underground Services*[12.8] and GS 6 *Avoidance of Danger from Overhead Electric Power Lines*[12.9].

12.3.2 Excavation stability

The stability of earth slopes and retention systems is a matter for sound engineering analysis and judgment, considering the forces involved not only in straightforward site conditions but also in reasonably foreseeable exceptional circumstances.

All excavations including open pits, boreholes and pile excavations should be clearly marked and fenced off.

12.3.3 Exceptional circumstances

Exceptional circumstances can easily arise, for example, through:

- the formation of spoil heaps, use of plant or stacking of materials, adjacent to a retention system which impose an extra surcharge
- build-up of water behind systems, where this has not been taken into account in the design
- unexpected vibration applied near the retained face
- failure of dewatering equipment.

It has also to be remembered that soil strength deteriorates rapidly, and a slope that is stable when excavated may not remain so. It should be noted that impervious membranes, such as pvc sheets, to cover exposed slopes may only offer a limited degree of protection against deterioration in both dry and wet weather.

12.3.4 Backfill materials

On sites where cranes have to move around, any previous excavations, perhaps to remove old foundations or other obstructions, must be backfilled with properly compacted and suitable materials. Many accidents have occurred through inadequate backfilling, sometimes concealed by a hardcore running surface. Even where sites have not had local excavations at the time of a contract, it is important to note that such excavations may previously have taken place. Operatives should be instructed to draw attention to any apparent soft areas when the site is first stripped.

12.3.5 Diaphragm walls

With diaphragm walls, the possibility of sudden loss of bentonite slurry should be considered, and all site staff should understand emergency arrangements before work begins.

12.3.6 Strutting and shoring

Where, because of adjacent excavation, there is risk to any building, there is a legal duty to take all practicable steps to prevent instability due to construction work. Such steps may include strutting, shoring or adequate underpinning.

12.3.7 Ramps and site transport

With deep basements, ramps, which are often trafficked by heavy earth-moving vehicles, present an additional risk (see Figure 12.1). Not only must they be of suitable material and of adequate width and spread, but the slope of the surface and its condition should be maintained so that vehicles cannot skid out of control in inclement weather. Other site equipment should be kept away from the ramp and vehicle wheels should be kept free of mud. The stability of the ramp and the consequences of brake failure should be considered during the design.

For further guidance on site transport reference should be made to HSG 144 *The Safe Use of Vehicles on Construction Sites*[12.10] and HSG 151 *Protecting the Public – Your Next Move*[12.11].

12.3.8 Settlement

An excavation may lead to the settlement of any adjacent ground, buried services or structure. Whether this is significant depends on the ground conditions, nature of the service or structure, and the sensitivity of adjacent structures. Movements should be monitored (see Appendix E) throughout the work by reference to a datum that will remain unaffected, and prearranged remedial works should be put in hand immediately the recorded movements exceed the limits agreed before the start of the work. These recommendations, which are essential when the specification sets limits for movements, should be regarded as good practice in all cases.

12.3.9 Piling

BS 8008: *Safety precautions in the construction of large diameter boreholes for piling and other purposes*[12.12] deals with the safety precautions that should be taken in the construction of boreholes for personnel access for inspection or working purposes. This standard deals with safety requirements for the equipment to be used and the gas hazards that might be met in deep boreholes. Its recommendations should be followed. Entry into the borehole should be avoided if reasonably practicable by making use of remote measuring equipment.

12.3.10 Erection and support of steel reinforcement

Reinforcement panels, particularly those containing a lot of reinforcement, can attract high wind loads. Therefore, a separate stability analysis is necessary.

Full-scale tests have demonstrated that lattice arrangements involving several layers of reinforcement connected with wire ties cannot be assumed to act compositely. Any analysis of the stability of such lattices should restrict the combined moment of inertia to the sum of the inertia for the individual bars.

12.3.11 Methane, oxygen deficiency and other atmospheric hazards

In recent years the dangers of methane gas accumulation in, and transfer to, poorly ventilated areas have become all too apparent, even in circumstances where this gas might not reasonably have been expected. Careful attention to the possible presence of combustible or noxious gases or vapours should be given, especially in under-floor voids and similar poorly vented areas (see Chapter 7). Oxygen deficiency can occur in any poorly ventilated confined space. Excavations adjacent to live sewers, or in ground containing rotting vegetation are at particular risk (see Section 12.2.5).

12.3.12 Fencing, lighting, etc.

The usual legal requirements for fencing, lighting and guardrails merit particular attention.

12.4 Electricity

Inevitably, many sites will require electrical installations for power and lighting, and it is necessary to recognise, for example, the potential hazards of overhead power lines and buried live cables (see Section 12.3.1).

Fig 12.1 Example of an access ramp

Initial reference should be made to HSG 141 *Electrical Safety on Construction Sites*[12.13]. This deals with temporary installations, portable apparatus, overhead and underground power lines, permanent installations, demolition, and safe working practices, along with references to other sources of advice.

12.5 Noise and vibration

Excessive noise and vibration are recognised as both publicly unacceptable and a hazard to health. Some hammers for driving sheet piling make excessive noise, as do rock breakers, compressors and similar equipment. There is no appreciable risk of damage to the human ear for noise up to 85 decibels. However, the noise close to a rock drill can exceed 110 decibels, for example, and hearing will be impaired by prolonged exposure. Hearing protection ('ear defenders') can reduce the noise level by about 30 decibels. It should be borne in mind that in confined spaces such as basements sound can be reflected from walls, causing even higher noise levels than would be the case on sites above ground. Vibration is most likely to be a hazard in the use of hand-held power tools, where exposure should be checked against acceptable limits; refer to HSG 88 *Hand-arm vibration*[12.14].

To limit disturbance to the public, it is often necessary to restrict working hours on noisy urban sites. This is typically 7am-7pm on weekdays, 7am-noon on Saturdays and all day Sundays.

There are now available several ways of reducing the noise formerly associated with conventional sheet-piling and driven-piling operations, although these may not be suitable for all ground conditions. These include hydraulic and vibratory pile-driving equipment, acoustic enclosures around hammers, and alternative wall types such as bored continuous pile walls, diaphragm walls and timbered faces. Quieter compressors have also been designed, and the more widespread use of certain types of hydraulically operated equipment is reducing noise on sites.

Under the *Control of Pollution Act* 1974[12.15], a Local Authority may serve a notice restricting any noise on a construction site amounting to a nuisance. It is possible for the employer or his representative to make an application under the Act for consent from a Local Authority before appointing a contractor. Reference should be made to BS 5228: 1992-1997 *Noise control on construction and open sites*[12.16].

12.6 Contaminated ground

A variety of short and long term health problems can arise from contact with contaminated ground. Potentially dangerous substances need to be identified, the related risks assessed and safe systems of work put in place in accordance with the *Control of Substances Hazardous to Health Regulations* (COSHH)[12.17].

Guidance on the range of potential contaminants and the means of controlling the related risks is provided in HSG 66 – *Protection of workers and the general public during development of contaminated land*[12.18].

12.7 Failures leading to injury or death

Failures in basement construction are poorly documented. Such high-consequence low-frequency events affect both the workforce and the public but are seldom aired. It is also probably the case in general that engineers, realising the inherent risks, have employed greater safety margins than are common in other structural situations. Often, it is the problems associated with isolated failures such as collapse of trenches and isolated sections of timbering, rather than more widespread failures, that cause serious accidents. However, there are instances of failure involving basement excavations and retention systems; in an extreme case, a five-storey building collapsed.

12.8 Supervision of work on site

Anyone undertaking construction work should have had suitable training, technical knowledge or experience, 'competence', and be under such supervision to be able to work safely and without risk to health. As appropriate to the nature of the activity, each employer must identify hazards, assess the risk and put in hand such control measures as are necessary at any time, following the 'hierarchy of risk control'. The designer's intentions for both permanent and temporary works with regard to the method and work sequence should be clearly communicated and understood.

12.9 Practical difficulties in construction on site

There are many constructional problems with deep basements, and each site has its own particular difficulties. The following list, which is not exhaustive, indicates where problems commonly arise (references in parentheses are to related legislation).

- lack of space to accommodate plant (see Figure 12.2), and the choice of auxiliary equipment best suited to the work [CHSW 5]
- too many operations running concurrently
- concurrently carrying out incompatible operations

- encroachment of storage and accommodation areas into areas needed for working [CHSW 5, 22 and 26]
- congestion of access to site, because of inadequate access or traffic conditions during the normal working day [CHSW 15, 17 and 19]
- effects of delay in one critical operation, causing other delays in concurrent or following items
- deterioration of soil in the basement of an excavation because of concentrated site traffic, leading to movement difficulties for plant, and making foundation problems worse
- restriction of working hours because of noise from plant, or for other reasons
- need to amend job specification during the work, leading to need for programme reorganisation
- extreme inclement weather [CHSW 24]
- misuse of plant [CHSW 27]
- stacking and storage of steel on overhead framing [CHSW 6 and 8]
- handrails for overhead access ways [CHSW 6]
- ventilation in top-down construction/dust extraction [CHSW 23]
- access and emergency access (mainly top-down construction) [CHSW 20]
- tower cranes and swinging loads ['LOLER' Lifting Operations and Lifting Equipment Regulations[12.19]]
- projecting steel from reinforced concrete [CHSW 5(2)]
- removal of temporary works – e.g. props and struts. Importance of installing props where required in working sequence [CHSW 9] – and the stability of excavations [CHSW 12]
- lighting [CHSW 25]
- electrical equipment and voltages
- confined space working

Note: CHSW stands for *Construction (Health, Safety and Welfare) Regulations 1996*[12.5].

12.10 Risk assessment

Risk assessments are now being used extensively in the UK as a means of documenting, defining and managing risk as a response to the CDM Regulations. (The phrase 'risk analysis' is normally reserved for formal Statistical Analysis of risk using mathematical modelling, and is outside the scope of this book.) A recent publication[12.20] has addressed managing geotechnical risk primarily as a means of improving production. Much of the desired improvement would also improve the health and safety risks on construction sites, particularly those involving excavation.

Fig 12.2 Example of a congested site © Paul Y. Foundation Limited

12.10.1 Production of a typical risk assessment

Approach

An analysis has been carried out for a cut-and-cover station. The Likelihood of a hazard occurring is estimated, as is the Severity of its effects. The two estimates are then compared in a matrix (the Risk Interaction Matrix, RIM), giving a resulting score (Risk Assessment Code, RAC): the higher, the less acceptable.

If the RAC (score) is higher than a pre-set target, Mitigating Measures are recommended. On the assumption that these Measures are effective, the Hazards are re-assessed. The resulting RAC, 'after', should be lower than before.

If this new RAC is not sufficiently low, the matter should be referred to the Client, who is in a position uniquely to decide whether the Hazard can be tolerated, with the consequent precautionary measures; or whether the Hazard should be designed out, acknowledging that this may result in additional costs.

This approach is used by many Mass Rapid Transit organisations.

Likelihood

Likelihood is rated thus (descriptions and definitions vary slightly between authorities):

Category	Description	Definition
A	Frequent	Occurs at least monthly
B	Probable	Occurs every few years
C	Occasional	Expected to occur several times in design life-time
D	Remote	Unlikely to occur in design life-time
E	Improbable	Extremely unlikely to occur in design life-time

Severity

Severity is rated thus (actual descriptions and definitions vary between authorities):

Category	Description	Definition
1	Catastrophic	Multiple fatalities per event
2	Major	Fatality or multiple severe injury/occupational illness per event
3	Minor	Single severe injury/occupational illness per event
4	Negligible	Minor injury, at most

Risk Interaction Matrix (RIM)

The two factors above are combined in the RIM below, to generate a Risk Assessment Code (RAC), which has a number between 1 and 20:

Severity (S)	Likelihood (L)				
	A Frequent	B Probable	C Occasional	D Remote	E Improbable
1 - Catastrophic	20	17	15	11	6
2 - Major	19	16	13	10	5
3 - Minor	18	14	12	7	4
4 - Negligible	9	8	3	2	1

It will be seen that hazards in the ‘Negligible’ Severity class and in the ‘Improbable’ Likelihood class attract low RACs.

Acceptability of Risk

The RACs indicate the importance attached to the combination of risks occurring, and gauge their acceptability.

The RAC is sensitive to the risk ratings chosen, particularly to the Likelihood (L). The actual values vary considerably between authorities:

RAC	Acceptability	Level at which to be agreed
20 - 15	Unacceptable	Client
14 - 10	Undesirable	Project Manager
9 - 4	Tolerable with a review	Designer for this component
≤ 3	Acceptable	Assessor

Risk Assessment

Hazards significant after mitigating measures have been taken will need to be referred upwards.

The mitigating measures will require mention (usually through the drawings) during the course of Contract Design (and before Construction) in the Project Health and Safety Plan, and after construction in the Health and Safety File.

Hazard Area	Hazard and Effect	Before Measure		
		L	S	R
1	**CONSTRUCTION**			
1.1 Diaphragm walls/piles	1.1.1 Leaking bentonite causing pollution	A	4	9
	1.1.2 Inadequate arching in soil causing local collapse of soil face	B	4	8
	1.1.3 Excess water pressure causing local collapse of soil face	C	4	3
	1.1.4 Persons falling into open trench/bore and drowning	B	2	16
1.2 Suspended slabs	1.2.1 Understrength falsework causing excessive deflection/collapse	D	2	10
	1.2.2 Premature stripping of falsework causing excessive deflection/collapse	D	3	7
1.3 Enclosed spaces	1.3.1 Accumulating noxious gases causing asphyxia	B	2	16
1.4 Lift shafts	1.4.1 Danger to operatives from falling items	C	2	13
1.5 Sprayed membranes or adhesives	1.5.1 Solvent evaporation causing asphyxia	B	4	8
2	**OPERATION**			
2.1 Suspended concourse slabs	2.1.1 Collapse of slab caused by (Civil Defence) loss of column below	E	1	6
2.2 Enclosed spaces	2.2.1 See 1.3.1 above.			
3	**MAINTENANCE**			
3.1 Enclosed spaces	3.1.1 See 1.3.1 above.			
3.2 Lift shafts	3.2.1 See 1.4.1 above.			
4	**DEMOLITION**			
4.1 No pre-stressed items:	No unusual hazards identified			

The following abbreviations have been used:

L	Likelihood
S	Severity
R	Risk Assessment Code (RAC)
D-wall	Diaphragm wall

Mitigating Measure	After Measure			Residual Risk
	L	S	R	
Seal drainage paths. Divert water courses. De-sand, restore pH, re-use as frequently as possible. Dispose of spent bentonite at licensed on-shore tip only, transporting in sealed containers.	D	4	2	Drains or water courses contaminated. Licensed tip contains pollutants.
Conduct trials to establish optimum length of wall panel.	D	4	2	
Maintain excess head of bentonite in trench	D	4	2	
Cover trench/bore with steel mesh except during excavation and insertion of rebar cage	E	2	5	
Falsework to be independently checked and permit to load issued. Back-propping of slab below (where applicable) to be checked.	E	3	4	
Designer to notify contractor of minimum strength requirement before removal of props.	E	3	4	
Identify and ventilate enclosed spaces. Train operatives. Supply breathing apparatus, harnesses and winch.	E	3	4	Rescue by trained staff only.
Provide lifting beams over, regularly tested. Provide staging.	D	3	7	
Do not use in enclosed spaces. Ventilate. Train operatives.	E	4	1	Rescue by trained staff only.
During construction, ensure continuity of (un-lapped) rebar from concourse to (Civil Defence) roof over	E	4	1	Tensile strain cracking in column, but no collapse

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13 Legal and contractual issues

13.1 Forms of contract and procedures

No recognised standard form of contract in the UK contains special conditions for the construction aspects of deep basements. Such work has in the past generally been governed by either the JCT[13.1] or ICE[13.2] forms of contract, although Government departments have long had their own special standard conditions suited to particular forms of building, e.g. GC/Works/1(1998) and (1999) Edition 3[13.3].

Traditional forms of contract are now supplemented by a variety of contracts providing for different methods of procuring construction, such as design and build contracts and construction management contracts. In some cases, standard terms of contract have been published for these different procurement methods, such as the JCT Standard form of Building Contract with Contractors' Design, or the Designed Portion Supplement and the ICE Design & Construct Conditions of Contract[13.4], often amended or supplemented by special conditions. The Engineering & Construction Contract[13.5], one of the New Engineering Contract suite of contracts, is becoming a little more common. This contract came out of the drive towards partnering and risk sharing which sought a different approach from the established forms. The contract comprises a set of core conditions with optional clauses designed to suit different circumstances and puts an emphasis on early collaborative management of changes and delays.

The prime objective of design build contracts is to place responsibility on the contractor. The JCT and ICE forms of design and build contracts are broadly based on the underlying traditional forms but adjusted to meet the contractor's different responsibilities and roles in the design and build context. An important consideration where design responsibility is imposed on the contractor is whether his responsibility should be equivalent to that of a professional designer, i.e. the obligation to exercise reasonable skill and care, as reflected in the JCT and ICE forms. Alternatively, the contractor may be required to assume an absolute obligation of fitness for purpose in respect of the design, which implies liability for defective design, irrespective of negligence. The practical implication of a fitness for purpose obligation in respect of design is that it is unlikely to be insurable either by the contractor or by a professional designer appointed as sub-consultant. Another key issue in design and build contracts is whether the contractor takes responsibility for the Employer's Requirements, which is increasingly sought by the employer who wants to transfer the whole of the design risk.

A common hybrid of design and build incorporates novation. The engineer and other consultants are appointed by the employer to work up the design to tender stage and these appointments are then novated to the successful design and build contractor, for whom they complete the design. In this way, the employer has more control over the early development of the design at the same time as ultimately having a single point of responsibility in the design and build contractor. For different reasons, this method of procurement usually finds less favour with contractor and consultant.

The Construction Management form of procurement, although less common than it used to be, is still preferred by some more experienced employers who are able to manage the higher degree of risk. With this form of procurement, specialist works contractors are appointed directly by the employer and managed by the construction manager. Work on the earliest packages can begin on site at a more advanced stage than under traditional procurement, which assumes completion of the design as a whole and fixing of cost before the construction contract is let. Under Construction Management, detailed design and procurement of the later packages can be carried out while the earlier packages are proceeding on site. This may offer some advantage for basement construction where an advanced contract can be let on terms and conditions drafted specifically for that particular section of the work. Thus, basement design work can be completed and the contract let to a specialist contractor experienced in basement construction before the start of the detailed design of the superstructure.

However, if different contractors are appointed under separate contracts, a clear demarcation is required between the contracts. Particular attention should be given to matters at the interface, such as handover dates, site facilities, condition of the site, accuracy of setting out and the status of temporary works. In addition, consideration should be given to the rights of the preceding contractors to gain access to the works to rectify defects and carry out maintenance works once later contractors are on site. Delays caused by one contractor to following contractors also need consideration, as the normal mechanism of providing for liquidated damages will not be appropriate.

Where the basement constitutes the major part of the work and there is little building work above ground, it may be appropriate to adopt the ICE form of contract. The ICE form best caters for unexpected ground conditions and adverse physical conditions and, in addition, has clearly defined responsibilities for temporary works. It is fair to say, however, that the ICE form allowing the contractor additional time and money for unexpected ground conditions is normally the first clause to be struck out by the employer, as it is not consistent with the current trend to transfer all risk.

13.2 Problems specific to basement contracts

The specialist works involved in the construction of a basement usually relate to the formation of the hole, which may involve embedded retaining walls and the support of those walls, if required, during the excavation. Once the hole is formed, the internal construction is very similar to the construction of the floors above ground. However, as ground conditions vary widely, each basement construction is unique. Different construction methods may be selected for the particular ground conditions and the standard form of contract may not necessarily be drafted sufficiently to cover all problems. At the pre-contract stage, consideration needs to be given to specific site problems and whether they require special treatment in the contract conditions. If the design and construction of the basement is done in advance of the superstructure, it is important that the column positions and the column loading for the superstructure have been resolved in order that adequate supports can be provided.

Additional conditions may be advisable, for example, to address the issue of what degree of waterproof construction is expected and permissible both during construction and after completion and what should be done if water ingress exceeds pre-determined levels. In view of the findings of CIRIA Report 69[13.6] (see also Chapter 3), this aspect is significant in those areas affected by rising groundwater levels.

There are specific issues with basement construction from a health and safety perspective which must be addressed at an early stage. For example, an embedded retaining wall acting in cantilever has many benefits including a reduction of the risks associated with either installing and removing heavy steel temporary props, or operating machinery in an enclosed environment beneath permanent slabs cast in a top down manner. The CDM Regulations[13.7] and the requirement to carry out risk assessments and to prepare a Health & Safety Plan will be of great importance.

Other special conditions might be included to define the duties and responsibilities of the parties in the event of :

- collapse arising from heave or ground movement
- support for adjoining owners and failure of the support systems
- Site Investigations and allocation of delay and increased costs arising out of the discovery of abandoned services or obstruction by services.
- the maintenance of existing groundwater levels outside the site.

If these matters are considered at pre-contract stage, the parties will be able to take steps to see that their respective risks are adequately managed.

13.3 Specialist contractors

Various items of work in basement construction involve specialist plant and/or personnel. These include embedded retaining walls, pile foundations, lowering groundwater, and geotechnical processes. Generally, specialist firms carry out such work.

It is common practice for the engineer to be involved in selecting a short list of specialists for specific tasks instead of unrestricted competitive tender. Although commercial pressures over which the engineer has no control may militate against this process, it allows specialists to be chosen on the grounds of expertise, experience and skill instead of lowest price. If required, competitive tenders may be invited following the pre-selection process. These procedures can be readily accommodated within most forms of contract.

When specialist contractors are employed, it is essential that consideration be given to a clear demarcation of the scope of design, whether this is for one element of the basement such as the retaining wall, or the whole basement design. The split of design responsibility between the specialist contractor, any main contractor, the professional team and the client must also be considered. Finally, it is important for there to be a single body to oversee and manage the integration of various elements to ensure the compatibility of the design and construction as well as compatibility between the design elements.

13.4 Responsibility for Site Investigation

Those who contract to carry out building or other work on or under land are under an obligation to satisfy themselves of the nature and characteristics of the land both on the surface and in the strata below. However, the opportunity for investigation by tendering contractors may in practice be limited and the only real investigations will have been carried out by the engineer, whose permanent design may also be dictated by consideration of the most desirable methods of construction.

In practice, the engineer should carry out such Site Investigations (see also Chapter 8) as are appropriate to enable adequate information to be obtained for structural and geotechnical design, having regard also to the temporary works and methods of construction. All the results of the Site Investigation should be clearly recorded in a report made available to tendering contractors along with other relevant information. Such information is often not made part of the contract documentation in order not to dilute the contractor's responsibility. Even so, an express disclaimer of responsibility by the employer for the information provided to contractors, and upon which they rely, may not in itself be effective in allowing the client to disclaim responsibility for the information it supplied.

Whether or not any duty to the contractor arises will depend on the language of the contract, specifications and other contract documents, the relevant correspondence between the parties and the knowledge, conduct and intention of the parties. A critical matter in the enquiry is whether the employer assumed responsibility for assembling and giving accurate and full information and whether the contractor relied on the employer to assemble and transmit such information.

It should be noted that, under the 6th and 7th Edition ICE Conditions of Contract[13.2], the employer is deemed to have made all relevant ground investigation data available to the contractor before submitting the tender and the contractor is deemed to have based his tender on the information made available to him as well as that based on his own inspection and examination. This provision is very frequently deleted by the employer.

13.5 Delineation of temporary and permanent works

Temporary works are those works, or parts of works, that are necessary for the construction, completion or maintenance of the permanent works but which are not necessary for the safety or proper function of the permanent works after completion. There is no legal definition of temporary works. The standard conditions of contract do not generally make reference to temporary works or make any distinction between permanent and temporary works. The exception is the ICE conditions, which define temporary works as 'all temporary works of every kind required in or about the construction and completion of the Works'. The 'Works' here are defined as 'the Permanent Works' together with the 'Temporary Works', and 'Permanent Works' means 'the Permanent Works to be constructed and completed in accordance with the Contract'.

One obvious difficulty with this definition arises from the inter-relationship of temporary works with permanent works. In excavations for deep basements, there is often a temporary condition for the permanent works, e.g. an unpropped retaining wall in its temporary condition. The design of the permanent works is dependent on the construction sequence, because of the stresses in the retaining wall locked in during the temporary condition.

Temporary works range from the pure type of temporary works like falsework, temporary propping and shoring at one end of the range to a temporary condition for permanent works at the other. The definition of temporary works also includes plant such as scaffolding or a tower crane base. In practice, the complexity of temporary works also varies widely. There are on the one hand the conventional varieties of temporary works that would be well within the expertise of a competent contractor, e.g. the design and installation of falsework. On the other hand, there are less commonplace temporary works that involve difficult technical features and would not necessarily be familiar to a competent contractor. Temporary works required in deep basement excavation can often fall into this category, e.g. propping for deep basement excavation involving a complicated method and construction sequence.

13.6 Responsibility for temporary works

The extent to which temporary works will be designed by either the contractor or the designer will in practice depend on the circumstances of each project and the complexity of the temporary works. Temporary works may be specifically designed and detailed by the structural designer and will in this way be the responsibility of the designer in the same way as the permanent structure. More commonly, the choice, design and construction of temporary works will be carried out by the contractor, in which case the responsibility for such work forms part of the contractor's general obligations under the contract.

The engineer's responsibility for temporary works derives from the terms of his engagement that in some cases may require a particular level of involvement.

Apart from the terms of his appointment, the engineer's specification, which often requires the contractor to submit details of temporary works to him for review or approval, may operate to extend the responsibilities that the engineer has under his terms of engagement, especially when the terms are unclear. The engineer will have statutory obligations, e.g. to comply with the CDM Regulations[13.7]. The engineer may also have obligations which arise, quite apart from any contractual duties he owes, from the duty of care he owes in tort, particularly in connection with his site activities. This duty can be said to amount to a duty to take reasonable care not to cause injury to health or safety or possibly damage to other property and may not necessarily be restricted to matters falling within the engineer's design. There have in recent years been significant legal developments restricting the circumstances in which a duty of care in tort arises and the scope of any such duty, and there are still many uncertainties. In the absence of a special relationship of reliance, however, it seems clear that any duty of care in tort is concerned only with protecting against personal injury or damage to other property and does not embrace any other kind of losses.

In practice, the engineer should have regard at the beginning of the project to the requirements for temporary works and should form a reasoned view as to which can safely be left to the contractor and which require greater involvement from the engineer, taking into account the degree of complexity of the temporary works, the inter-relationship with the permanent design and the consequences of failure, e.g. risk to adjoining properties. The engineer should also have regard to any relevant obligations he has undertaken in his terms of engagement.

Consistent with this approach, the requirements for temporary works need to be sufficiently specified in the contract documents. The engineer will need to identify site constraints and conditions, construction and project requirements. Standards and criteria for acceptance should be explicit. Where conventional temporary works are concerned, such standards and criteria can be simply stated in the specification. However, with unconventional or complicated temporary works, for example, temporary support to basement walls, such standards and criteria, together with particular restraints or requirements, will need to be specified in much greater detail, any special areas of concern being spelt out. It may be advisable for the contract documents to contain at least one method of construction, which, in the opinion of the engineer, would prove satisfactory. This should include a description of all special precautions necessary. Where such a specific method is described, enough information should be included in the contract documents to enable tenderers to prepare their schemes for alternative methods. Where access to the design calculations is needed to establish an alternative, the relevant information should be supplied and any special precautions explained, including any limitations that might be imposed.

13.7 Use of permanent works to support temporary works

It is sometimes proposed that permanent works be used to support temporary works, and often the temporary works impose forces on the permanent works. Therefore, all stages of construction from the erection of the temporary works to the completion of the permanent structure must be investigated so that the effect of one structure on the other can be fully assessed. A full exchange between the engineer and the contractor of complete and detailed information of all aspects relevant to the design and construction of the works is important in these circumstances. In particular, the engineer may need to clarify any limitations on the ability of the permanent works to provide an acceptable temporary works solution.

13.8 Adjacent structures

Certain legal principles relating to rights of support can be set out. However, before they can be applied to a specific problem, detailed consideration of the circumstances would be necessary. Every owner of land has a natural right of support, i.e. the right to prevent such use of the neighbouring land as will withdraw the support which the neighbouring land affords to his land. Where the act of a landowner results in subsidence of the neighbouring land alone, an action for infringement of the natural right of support can be brought, irrespective of negligence.

There is no natural right of support of a building as opposed to right of support of land in its natural state, but often the right of support of a building is acquired by grant. If that support is removed, the building owner will have a right of action for withdrawal of that support. If the building has no right of support, the building owner cannot in theory bring an action for withdrawal of that support, although to what extent this principle would be likely to be upheld in modern decisions of the courts is unclear.

There is no natural right of support from water and water flowing freely through undefined channels beneath the ground. A person can appropriate water that is flowing in undefined channels, although this may well be subject to qualification by a duty of care on the person abstracting water to see that the property of his neighbour is not damaged. If, for example, excavations result in changes in level of groundwater that cause damage to a neighbour's property, a claim in nuisance or negligence is likely to succeed. However, there is no general right to abstraction of water from any source of supply except in pursuance of a licence granted by the Environment Agency. This is subject to some exceptions within the provisions of the Water Resources Act 1991[13.8], for example, a right to abstract small quantities of water for drainage purposes.

In practice, the engineer must give consideration early in the planning to the question of movements in the ground adjacent to an excavation and the adjacent buildings likely to be affected by the works. In addition, services and utilities may also be affected.

In England and Wales, the Party Wall, etc. Act 1996[13.9] requires the owner of a proposed basement to give full information of his construction plans to all the adjoining owners and work cannot start until all their consents have been obtained. The employer is responsible for complying with this statutory requirement. Even where there is no statutory requirement because the proposed excavation falls outside England and Wales, this procedure is recommended.

The engineer ought to advise his employer of the likely movements in the ground as a result of the works and to recommend appropriate measures to minimise such movements. He ought also to recommend to his employer that all existing structures that might be affected by the basement construction should be surveyed with the owner's representative present. In such a survey, existing structural faults are recorded and agreed (dated and agreed photographs are valuable evidence of existing defects) and, if necessary, devices are fixed to measure any movement occurring during construction work, as well as precision levelling and the other forms of monitoring described in Appendix D. The surveys are invaluable in safeguarding the client's position should claims for damage to the adjoining properties be made subsequently.

13.9 Statutory requirements

There are many statutory requirements that may be of relevance to a contract for the construction of a deep basement. The list of the more important requirements is set out in Appendix C. Often, compliance with statutory requirements is an express contractual obligation, particularly for the contractor. Non-compliance may therefore amount to a breach of contract but it may also expose the contractor or the engineer to the risk of prosecution or an action for breach of statutory duty. Even if compliance with statutory requirements is not an express contractual provision, non-compliance may be prima facie evidence of negligence.

A number of statutory regulations are concerned with safety, which is now assuming much greater importance in a legal context as has been described in Chapter 12.

References

13.1 The Joint Contracts Tribunal. *Standard Form of Contract*. JCT, 2001.

13.2 The Institution of Civil Engineers Association of Consulting Engineers and Federation of Civil Engineering Contractors. *ICE Conditions of Contract*. 6th Edition. Thomas Telford, 1991.

13.3 Wood R D. *Contractors' Claims Under the GC/Works/1. Edition 2: Form of Contract.* BSRIA, 1988.

13.4 The Institution of Civil Engineers, Association of Consulting Engineers and Civil Engineering Contractors Association. *ICE Design and Construct Conditions of Contract*. 2nd Edition. Thomas Telford, 2001.

13.5 The Institution of Civil Engineers. *Engineering Construction Contract*. 2nd Edition. Thomas Telford, 1995.

13.6 Simpson B, Blower T, Craig R N and Wilkinson W B. *The engineering implications of rising groundwater levels in the deep aquifer beneath London*. CIRIA Special Publication 69. London, CIRIA, 1989.

13.7 *The Construction (Design and Management) Regulations 1994*. Statutory Instrument 1994 No. 3140. The Stationery Office, UK, 1994.

13.8 *The Water Resources Act 1991*: Elizabeth II. Chapter 57. The Stationery Office, UK, 1996.

13.9 *The Party Wall etc. Act 1996:* Elizabeth II. Chapter 40. The Stationery Office, UK, 1996.

14 Communications

14.1 Importance of communications

Communications between the designer and the contractor should be two-way and continuous. The designer should inform the contractor of his assumptions and design principles. The contractor should explain to the designer the methods he intends to use and give details of the design of any temporary works that such methods require.

The general question of communications between the designer and site is the subject of an Institution Report[14.1]. However, because of the interaction between design assumptions and the order and methods of construction, such communications are particularly important in deep basement construction for a number of reasons:

- during construction the structure or parts of it may have to behave in an entirely different manner and carry very different loading from that when completed
- temporary works designed wholly or partly by the contractor may play a critical role in the stability of the project and of the surrounding buildings
- part of the design may be provided by a subcontractor or specialist for items such as ground anchors, piling or diaphragm walls.

Many difficulties can be avoided by involving the contractor in the design, but the designer has to make assumptions about the order and methods of construction. At the same time the designer should try to give the contractor as much freedom as possible. The result can be that it is not always clear which aspects have been fully investigated and which remain to be resolved after a contract has been let.

The engineer should formulate at least one satisfactory method of executing the works, and it is recommended that this method be given to the tenderer as a statement when the inquiry is issued. At the same time, any limitations on the use of the permanent works in providing an acceptable temporary works solution should also be stated. Systems that will not be permitted, e.g. ground anchors founded in land owned by others, should also be noted. Conversely, a method statement on the execution of the works should be a tender requirement and, during the pre- and post-tender discussion with potential contractors, consultation should take place enabling a detailed appreciation by them of all the relevant factors and acquainting the engineer with the contractor's proposals.

14.2 Information supplied by the engineer

Information supplied by the engineer at the time of issuing an inquiry should include:

- any standards, criteria, conditions and constraints to be observed at each stage of the construction
- ground investigation and other relevant site data, including information on previous use of the site, existing services, adjacent foundations, underground workings and tunnels, mining workings, tidal flows and river levels
- basic data necessary for designing temporary works
- performance requirements in relation to the use of structures, and hence special features of design such as watertight construction
- description of assumed stages of construction with indication of how stability is maintained throughout each stage
- method statement showing the sequence of each operation: particular reference should be made to questions of flotation, temporary supports for retaining walls and precautions against damage to adjoining buildings
- restrictions on the way permanent works may be used in conjunction with temporary works designed by the contractor
- details of design and/or construction to be carried out by any specialist subcontractors
- drawings and specifications
- major programme dates
- data on work to be carried out by nominated subcontractors, including programme attendance and form of contract
- information about adjacent structures that may be affected by the contractor's operations
- details of special insurance requirements in relation to adjacent property.

14.3 Information submitted by the contractor

Information to be submitted by the contractor with his tender should include:

- general outline of proposed method of undertaking the works:
  - excavation, including main plant to be employed
  - spoil removal
  - sheet piling including method of installation equipment
  - bored piling/bored pile walls/driven piles
  - diaphragm walling.

Fig 14.2 Boon Keng Station, Singapore: stages of semi-top-down excavation and construction

- ground anchors
- strutting
- monitoring movements on temporary work and steps to be taken if these become excessive
- dewatering, including standby plant and extent of automatic switchover, monitoring of extraction flows and fines removal
- proposed use of permanent works as part of temporary works or to support them
- basic assumptions made in designing temporary works
- precautions to be taken to protect the environment, e.g. to avoid nuisance, dust, noise, etc.
- main items of plant to be deployed
- details of specialist subcontractors
- outline programme, including duration and critical items of design related to the sequence of the work, and where appropriate, rate of expenditure diagram.

It is on the basis of such an exchange of data that meaningful post-tender discussion can take place, enabling each side to appreciate the intentions of the other.

As mentioned in Chapter 1, one tool very effective in communicating the designer's and constructor's assumptions to each other is the Precedence Network. An example is given in Figure 14.1, illustrating the critical stages of a station built by the semi-top-down method (see Figure 14.2); here the interactions in timing between significant permanent and temporary works are set out formally, in this case in an Activity-on-Arrow chart. The example is for semi-top-down construction, which is highly interactive and provides limitless scope for misunderstanding in the absence of a formal document. Mere diagrams or statements of construction sequence are inadequate: the temporal interaction is essential, with permitted partial completion indicated where appropriate. Once understanding and agreement have been reached, the contractor has only to add durations to the activities and leads/lags for the network to be incorporated bodily into his construction programme. The Network also provides useful input to the Project File as required by CDM Regulations[14.2].

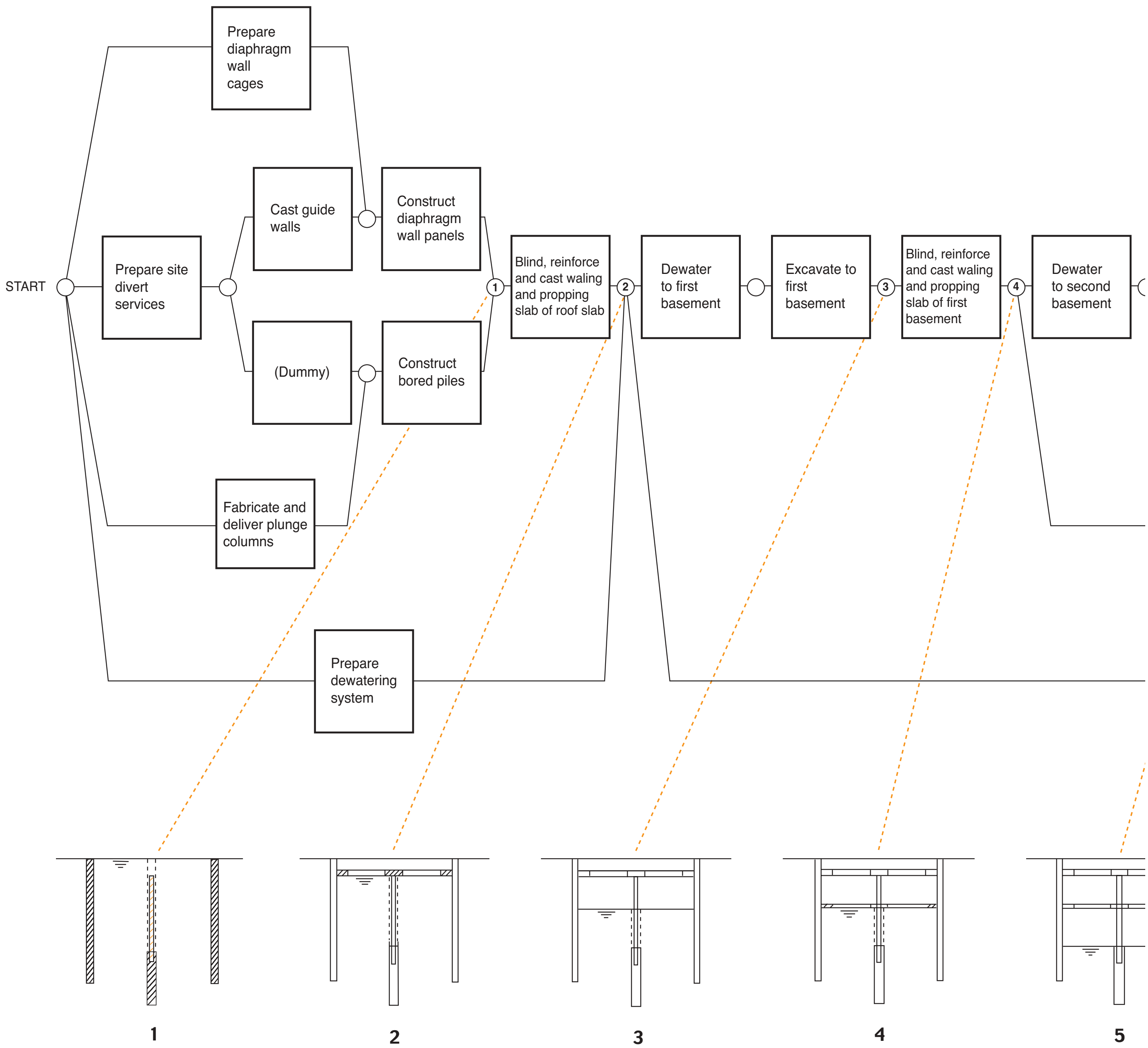
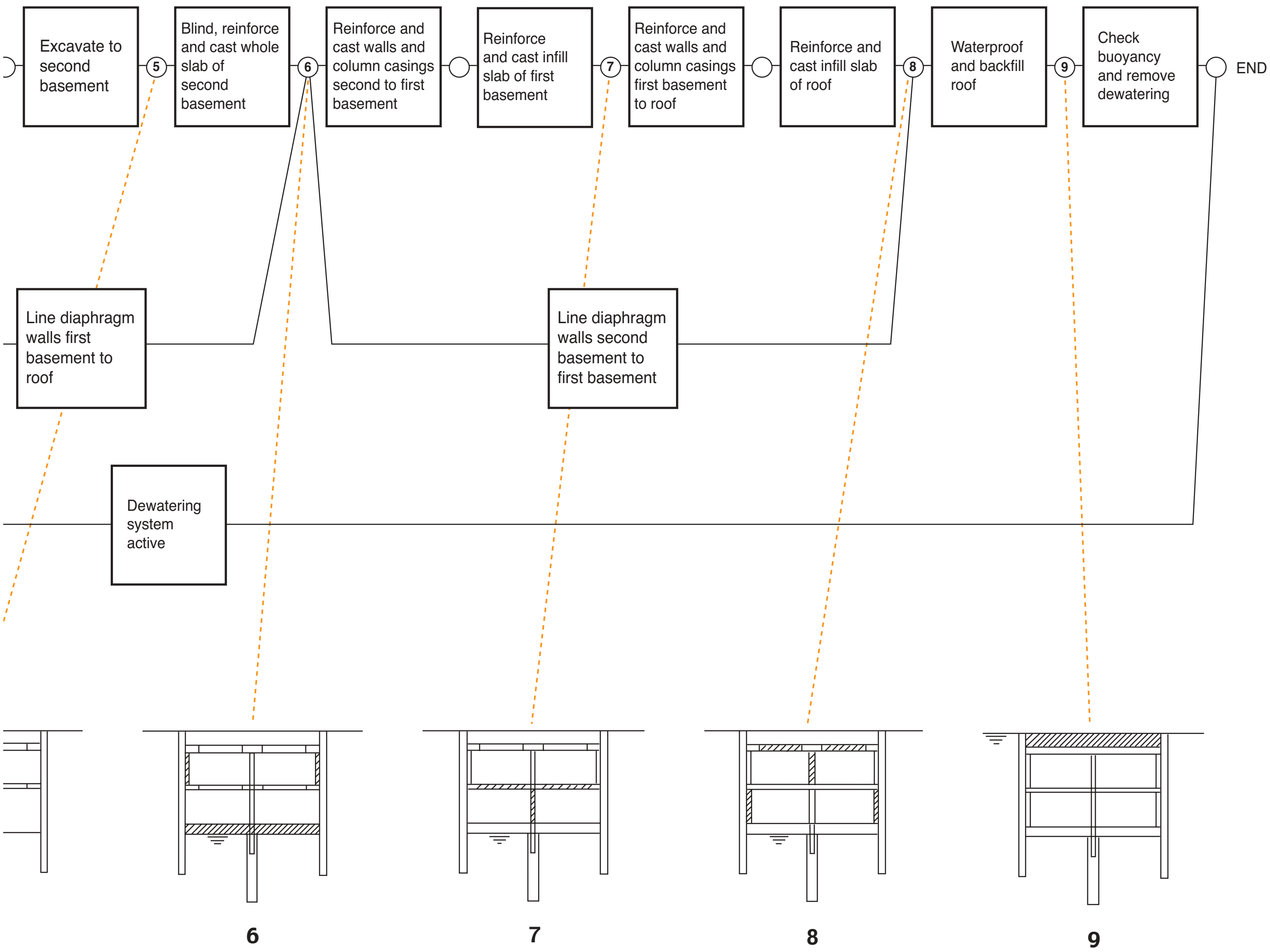


Fig 14.1 Example of Precedence Network (Activity-on-Arrow), for semi-top-down construction



Excavate to second basement
5
Blind, reinforce and cast whole slab of second basement
6
Reinforce and cast walls and column casings second to first basement
Reinforce and cast infill slab of first basement
7
Reinforce and cast walls and column casings first basement to roof
Reinforce and cast infill slab of roof
8
Waterproof and backfill roof
9
Check buoyancy and remove dewatering
END
Line diaphragm walls first basement to roof
Line diaphragm walls second basement to first basement
Dewatering system active
6
7
8
9

14.4 Post-contract and construction stages

When the contractor has been appointed and the method of construction has been agreed, it is necessary for:

- the designer to recheck the stage-by-stage stability of the construction, including checking the validity of assumptions made by the specialist subcontractors in any design they have prepared
- the designer to make sure that all local authority approvals have been obtained
- the designer or the contractor to design the necessary temporary works
- the contractor to submit for the agreement of all the parties a detailed programme for executing the works for the agreement of all parties.

The finally agreed scheme should be presented so that those who will be executing it on site will understand it. Continuous discussions must take place between the engineer and the contractor reviewing progressively the original assumptions as the works are executed.

For a deep basement carried out under a Civil Engineering form of contract, the Engineer instructs the contractor. However, contracts for buildings that include deep basements are usually carried out under JCT *Conditions of contract*[14.3] in which the Engineer officially has no standing. This means that he has no right to give instructions directly to the contractor and has to deal with each item of work via his architect. It is thus important the architect issues his Architect's Instruction as soon as he receives the recommendations from the Engineer.

14.5 Quality requirements

Any particular requirements for quality control should be stated in the contract specification. If normal documentation to BS EN ISO 9002:1994[14.4] for Quality Assurance is a client requirement, the degree of documentation required to be included within the project's Quality Plan should be stated for control and pricing purposes.

If the designer is working within the Quality Assurance requirements of BS EN ISO 9001:1994[14.5], this should be stated.

It is important for the designer to communicate full design criteria to all parties involved in the construction of the basements and to monitor the quality of the work throughout the construction.

References

14.1 Institution of Structural Engineers. *Communication of Structural Design*. Southampton, Hobbs, 1975.

14.2 *The Construction (Design and Management) Regulations 1994*. Statutory Instrument 1994 No. 3140. The Stationery Office, UK, 1994.

14.3 The Joint Contracts Tribunal. *Standard Form of Contract*. JCT, 2001.

14.4 British Standards Institution. *BS EN ISO 9002:1994. Quality systems. Model for quality assurance in production, installation and servicing*. London, BSI, 1994.

14.5 British Standards Institution. *BS EN ISO 9001:1994. Quality systems. Model for quality assurance in design, development, production, installation and servicing*. London, BSI, 1994.

Appendix A Archaeological implications

Excavations for basement construction may have important archaeological implications. Where significant remains are considered to exist, archaeological units supported by local societies and local authorities usually seek opportunities for investigation or excavations. Site owners, developers, consulting engineers and contractors should co-operate with the archaeological bodies from the earliest days of the design of the development so that opportunities for archaeological work are provided within the programme.

Economic construction of deep basements inevitably entails large-scale removal of material, which generally results in fill being mechanically removed, in the process destroying any archaeological record it may contain. Furthermore, because in an urban environment the boundaries of deep basements typically follow long-established property lines, constructing diaphragm walls and guide walls along these lines will consequently tend to destroy the stratigraphy relating to the complex social and political history between properties and effectively isolate the archaeology of the main block of material.

In addition, on sites with high water levels, with permanent basement dewatering introduced to facilitate the construction and reduce water pressures, the hydrological regime of the sub-strata, in which any material archaeological remains may be preserved, will change significantly. Dewatering can affect several times the area of the site, and the most drastic result can be the rapid decay of organic remains such as leather, wood and fabrics, which generally tend to be rarer types of find. Stone, pottery and other stable artefacts are normally not affected by dewatering. Removing water can also affect the distribution of vegetation, perhaps having a detrimental effect on the surface protection of sites of archaeological importance.

Only a small proportion of known sites of archaeological importance enjoys legal protection as scheduled ancient monuments. For most known sites, and for sites which have not been investigated but which are believed to contain archaeological deposits, the town planning system, together with provisions under the Ancient Monuments and Archaeological Areas Act 1979[A.1], remains the only means of protection or of ensuring that archaeological evidence is recorded when development takes place.

Part II of the 1979 Act provides for the designation by the Department of the Environment of Areas of Archaeological Importance (AAIs). Designation confers powers for carrying out investigation before a site is developed in the historical centres of (so far) five cities; Canterbury, Chester, Exeter, Hereford and York. AAI provisions are neither conservation nor preservation measures. Designation under the Act serves only to introduce the possibility of mandatory delay of development to allow archaeological work to proceed. In AAIs, developers are required to give six weeks' notice to the Planning Authority of any proposals to disturb the ground and the investigating authority nominated by the Secretary of State has power to enter and excavate. The Act does not provide funding as of right but represents an indication to developers of government concern for archaeological heritage.

Other relevant statutory provisions are found within Town and Country Planning legislation[A.2] and include measures for protecting listed buildings and for designating Conservation Areas. These measures are not directly applicable to the archaeological resource, but are relevant in that they help conserve parts of the environment, thus preserving any archaeological deposits within those sites. Listed buildings may also be scheduled ancient monuments.

Leaving aside statutory provisions, before any detailed Site Investigation is carried out, the available archaeological records of the area should be examined, and any such historical topographical research should include looking into the potential of the site for archaeological investigation. Guidance is available from local archaeological societies, professional archaeological units, county archaeologist and university academics. If the site is likely to have archaeological value, detailed examination of the fill from boreholes and trial pits will present an invaluable opportunity to examine the history and estimate the potential for archaeological excavation. The archaeological fieldwork, Site Investigation and thorough excavation should be carried out using the skills of the local archaeological unit, which is the body most familiar with local soil conditions and resources. The archaeologist may also provide relevant information during any Site Investigation.

Surface geology and the existence of man-made works, which may give rise to engineering problems, can often be determined by archaeological investigation. Equally, an archaeological interpretation of trial pits and boreholes can prove valuable in the geotechnical study.

It is strongly recommended that the archaeological history of a site and its excavation potential be determined as early as possible. This could be done at the same time as the Desk Study or Site Investigation. In many urban centres where the archaeological resource is easily predictable, it is becoming common practice for the local archaeological unit to become a legitimate member of the development team, much like the many other specialist consultancies in the project. Experience shows that, given co-operation between the developer and the archaeological body from the earliest days of the development, it is possible to integrate archaeological activities into the scheme with minimal risk of delay.

References

A.1 Ancient Monuments and Archaeological Areas Act 1979: Elizabeth II. Chapter 46. The Stationery Office, UK, 1979.

A.2 Town and Country Planning Act 1990: Elizabeth II. Chapter 8. The Stationery Office, UK, 1990.

Appendix B Special services: requirements for deep basements

B.1 Introduction

Smoke outlets, ventilation systems, fire-fighting and fire-resistance requirements can have a major influence on design details.

B.2 Requirements for fire-fighting ventilation and smoke outlets

In accordance with BS 5588-5[B.1], pressurised fire-fighting stairway shafts — to permit easy access for firemen — must be provided for all basements 9m or more below ground level.

Smoke outlets from all storeys below ground level along the street frontages or adjacent to external walls must be easily accessible to the fire brigade. They should:

- be at high level in the area they serve
- be as numerous and as large as possible
- aggregate not less than 2% of the floor area they serve
- be arranged so that a through draught can be created.

A higher standard may be required where warranted by the nature of the occupational use. Separate smoke outlets will normally be required from accommodation such as boiler rooms, oil-filled transformers, and other areas of special risk.

Any smoke-outlet shafts extending into or through other storeys should be enclosed by construction with the same standard of fire resistance as that required for the storey serviced or through which it passes, whichever is the greater. Where shafts from different parts adjoin, they should be similarly fire-separated from each other.

It should be appreciated that, for multi-storey basements, the perforations required through the uppermost basement floor and perimeter walls of that storey will be very large. This is because they must be able to accommodate smoke outlets from the upper basement storey itself, as well as perforations required for general ventilation ducting.

B.3 Plant rooms

Boiler rooms, generator rooms, oil stores, electrical switchgear, transformer chambers, etc. will all need to be within separate fire compartments; some will require individual fire-extinguishing systems. Their location may be (1) determined to minimise fire risk, means of escape, smoke venting, or (2) dictated by public utility companies who, along with Building Control authorities, should be consulted early. Internal communication from a fire-fighting stairway will require a ventilated lobby (0.4m^2 permanent ventilation) between such staircases and transformer chambers, boiler rooms or other areas of a higher fire risk.

References

B.1 British Standards Institution. BS 5588-5: *Fire precautions in the design, construction and use of buildings. Part 5: Code of practice for firefighting stairs and lifts*. BSI, 1991.

Appendix C Statutory requirements (see also Chapter 12)

The following Acts of Parliament and Regulations have a direct influence on the construction of deep excavations in England and Wales. There is different legislation in Scotland and Northern Ireland with generally the same intent.

- The Construction (Design and Management) Regulations 1994[C.1]
- The Building Regulations 2002 (in conjunction with the Approved Documents)[C.2]
- The Building Act 1984[C.3]
- The Health and Safety at Work etc. Act 1974[C.4]
- The Control of Pollution Act 1974[C.5]
- The Fire Precautions Act 1971[C.6]
- The Factories Act 1961[C.7]
- The Petroleum (Consolidation) Act 1928[C.8].

The Building Act 1984 is a consolidation Act bringing together all the relevant legislation concerning building from many former Acts now repealed and contains no new legislation: derivations are given on pages 123-130 of the Act. Inner London is also now included in the National Building Control System with just the retention of certain extra fire-safety powers under the London Building Act (1930-39)[C.9]. The London Building (Constructional) Bye-laws are entirely repealed.

Part VI of the London Building Act (Amendment) Act 1939 relating to rights of adjoining owners has now been repealed but incorporated within the Party Wall etc. Act 1996[C.10] applicable throughout England and Wales.

Health and safety aspects are now paramount on all construction sites. The Health and Safety at Work etc. Act 1974[C.4] is an enabling Act under which both statutory regulations made under powers predating the Act and new statutory regulations all take effect. The following are the principal statutory instruments:

- The Construction (General Provisions) Regulations 1961[C.11]. Special attention is drawn to Part IV Excavations, Shafts and Tunnels and Part VII Dangerous or Unhealthy Atmospheres
- The Lifting Operations and Lifting Equipment Regulations 1998[C.12]
- The Construction (Working Places) Regulations 1966[C.13].

Important new regulations effected under the Health and Safety at Work etc. Act[C.4] are:

- Reporting of Injuries, Diseases and Dangerous Occurrences Regulations 1995[C.14]
- Control of Substances Hazardous to Health Regulations 1988[C.15]
- Electricity at Work Regulations 1989[C.16]
- Noise at Work Regulations 1989[C.17].

It is recommended that early contact be made with the relevant building control authority and local authority both to inform them of the proposed development and to find out whether there are local conditions that may affect the design stage, e.g. ground conditions, buried services, underground railways, post office tunnels, local bye-laws, limitations on environmental noise or working hours, etc.

References

C.1 *The Construction (Design and Management) Regulations 1994*. Statutory Instrument 1994 No. 3140. The Stationery Office, UK, 1994.

C.2 *The Building Regulations 2000*. Statutory Instrument 2000 No. 2531. The Stationery Office, UK, 2000.

C.3 *The Building Act 1984*: Elizabeth II. Chapter 55. The Stationery Office, UK, 1984.

C.4 *The Health and Safety at Work Act 1974*: Elizabeth II. Chapter 37. The Stationery Office, UK, 1974.

C.5 *The Control of Pollution Act 1974*: Elizabeth II. Chapter 40. The Stationery Office, UK, 1974.

C.6 *The Fire Precautions Act 1971*: Elizabeth II. Chapter 40. The Stationery Office, UK, 1971.

C.7 *The Factories Act 1961*: Elizabeth II. Chapter 34. The Stationery Office, 1961.

C.8 *The Petroleum (Consolidation) Act 1928*: Elizabeth II. Chapter 32. The Stationery Office, UK, 1928.

C.9 *The London Building Acts 1930-1939*: Elizabeth II. Chapter 47. The Stationery Office, UK, 1930-1939.

C.10 *The Party Wall etc. Act 1996:* Elizabeth II. Chapter 40. The Stationery Office, UK, 1996

C.11 *The Construction (General Provisions) Regulations 196*1. Statutory Instrument 1961 No. 1580. The Stationery Office, UK, 1961.

C.12 *The Lifting Operations and Lifting Equipment Regulations 1998.* Statutory Instrument 1998 No.2307. The Stationery Office, UK, 1998.

C.13 *The Construction (Working Places) Regulations 1966*. Statutory Instrument 1966 No. 94. The Stationery Office, UK, 1966.

C.14 *The Reporting of Injuries, Diseases and Dangerous Occurrences Regulations 1995*. Statutory Instrument 1995 No. 3163. The Stationery Office, UK, 1995.

C.15 *The Control of Substances Hazardous to Health Regulations 2002*. Statutory Instrument 2002 No. 2677. The Stationery Office, UK, 2002.

C.16 *The Electricity at Work Regulations 1989*. Statutory Instrument 1989 No. 635. The Stationery Office, UK, 1989.

C.17 *The Noise at Work Regulations 1989*. Statutory Instrument 1989 No. 1790. The Stationery Office, UK, 1989.

D.1 Introduction

Monitoring of ground and building movements is essential if the Observational Method, with its many cost and safety advantages, is to be used. Moreover, monitoring provides a most valuable check on the design assumptions, the construction methodology and its quality. If there is any risk of damage to adjacent buildings it is advisable to monitor their movements to give early warning of possible damage. Such measurements can be invaluable in identifying the source of the problem and developing corrective measures. Monitoring data can also be used to provide factual quantitative evidence in the event of an adjacent building owner claiming that excavation of the deep basement has caused damage, a very common occurrence. The importance of case histories in developing the state of the art is emphasised in Sections 1.3 and 2.6 and the cost of monitoring can often be justified to the client by referring to the above benefits.

The design, commissioning, installation, measurement, processing and interpretation of field monitoring requires a great deal of experience and expertise. It is most desirable to seek the advice of someone who has such experience, preferably who does not have a commercial interest in the sale and installation of measuring equipment. It is vital that there should be continuity of experienced staff from design and installation right through to interpretation of the results. Careful thought must be given to the location and number of instruments. Appropriate location of the instruments requires an understanding of the likely mechanisms of behaviour and numerical analysis can assist in this. It is also important to bear in mind that too many instruments can overload staff with the result that there is insufficient time to properly process and digest the results. Measurements of movement should always be referred to stable reference points far enough away from the site to be outside its zone of influence. It is important to have as long a period for initial zero readings as possible so that the correct and stable functioning of the instruments can be established and their precision established for that site.

Reference D.1 gives comprehensive guidance on the wide range of monitoring instruments available and their use. Reference D.2 deals with the measurement of ground displacements around deep excavations. The most commonly used instruments for the monitoring of deep basements are the precise level, the total station, inclinometers and electrolevels. The use of these instruments has been described in detail in reference D.3 and only the key aspects will be described here.

D.2 Precise level

Precise levelling involves the measurement of the elevation of each measuring point to sub-millimetre accuracy relative to a datum point. It is common now to use a digital precise levelling instrument in conjunction with an invar bar coded staff. A key component of the monitoring is to use reproducible measuring points. The Building Research Establishment (BRE) socket and levelling plug have been specially designed so that the plug, which is removable, can be screwed into the socket for each survey to an accuracy of better than 0.1mm (see Figure D1). This figure also shows a smaller version of the BRE socket which makes installation easier and less obtrusive. Some alternative, inferior designs are available that rely on screwing the plug as tightly as possible into the socket. It has been shown that these lead to a much larger scatter of results than those obtained using BRE levelling points.

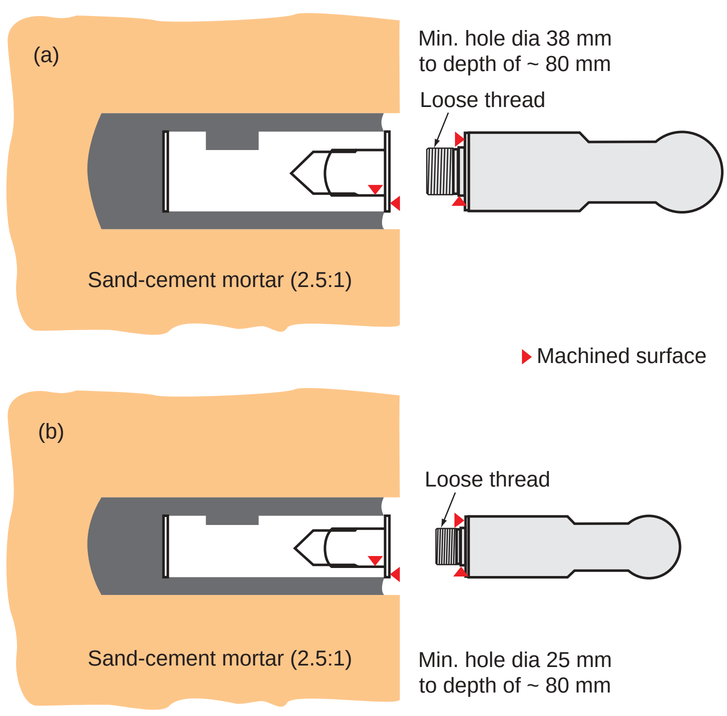


Fig D.1 Details of the BRE sockets and plug used in precise levelling (a) original size, (b) miniature version

Reference D.4 gives details of best practice for carrying out precise levelling. Approximately the same tripod locations should be used for each successive survey using the same monitoring points for change points. Distances to both intermediate and (particularly) change points should generally be less than about 20m. Backsights and foresights should be of similar distances as far as is practicable to minimise any collimation errors. The survey should always be closed either by returning to the initial datum or to a separate benchmark. Under favourable conditions, the closing error should be within 0.3mm. A larger error might have to be accepted under adverse surveying conditions. Accuracies of vertical displacement of about ±0.2mm can be obtained if best practice is followed.

D.3 Total station

Displacements in three dimensions can be obtained using a total station which consists of an automatic theodolite combined with an electro-distomat (EDM) system of distance measurement. A high precision total station can measure angles and distances to a resolution of 0.1 seconds of arc and 0.1mm respectively. The targets consist of retro-reflective prisms.

Reference D.3 summarises the procedures that were adopted for the measurement of building movements for the Jubilee Line Extension. Reference targets were mounted on adjacent buildings outside the zone of influence. The total station locations were carefully selected to maximise the number of targets that could be seen from each. It is important that each target is seen from at least two stations to supply redundant observations as this considerably increases confidence in the measurements. Also, when the angle of the instrument to the target becomes too oblique it is often not possible to measure distance. It is then essential that angle measurements are made from two stations to such targets.

A careful analysis of measurements made at the Jubilee Line Extension indicate that it is possible to obtain accuracies of displacement of about ±0.5mm vertically and ±1.0mm horizontally.

Recently, computer automated total stations have been used with great success allowing regular automatic monitoring of a large number of targets.

D.4 Inclinometers

An inclinometer is used to measure changes of inclination, usually at various depths down a borehole. Knowing the depths at which these changes have been measured it is a straight-forward matter to integrate the results so as to obtain horizontal displacements at the various depths. The traditional inclinometer is housed in a 'torpedo' which can be lowered down an inclinometer tube which has been located in the borehole and securely grouted. The torpedo is fitted with spring-loaded guide-wheels that locate in grooves length-wise along the inner surface of the inclinometer tube. Usually the intervals in depth between successive measurements coincide with the length of the torpedo.

Inclinometers can be used to measure horizontal movements at various depths down a retaining wall or in the ground behind the retaining wall. In the latter case the measurements are greatly enhanced if the inclinometer is combined with a borehole extensometer for measuring relative vertical displacements down the borehole.

The horizontal movements deduced from an inclinometer installation are only relative movements. In order to obtain absolute movements the displacements at the top or the bottom must be assumed or measured. Sometimes it is assumed that the bottom of the tube is stationary, perhaps by embedding it in rock or strong ground. In many published cases it is apparent that the assumption of zero displacement at the bottom is incorrect. It is good practice to measure the displacement of the top of the tube whenever it is practicable to do so. This ensures that the absolute horizontal movements can be obtained and also provides a valuable check on the accuracy of the inclinometer measurements.

D.5 Electrolevels

Electrolevels are small glass vials that contain an electrolytic fluid and three electrodes, which are partially immersed in the fluid as shown in Figure D2. The instruments are energised with a small electric current and the voltage of the arrangement is measured and converted to a digital reading. When the electrolevel is tilted, the length of immersion of

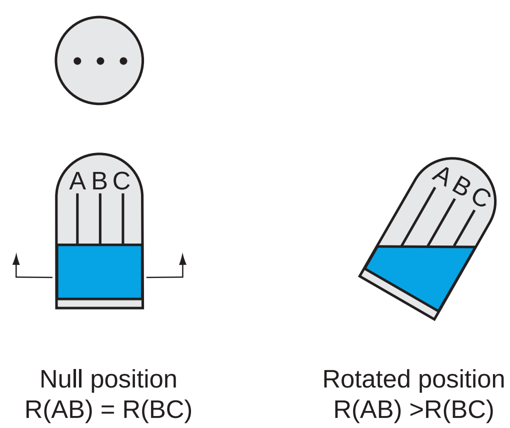


Fig D.2 Schematic diagram of an electrolevel each electrode in the fluid changes. This causes a change in the resistance of the circuit and a change in the measured voltage for a constant current through the circuit. Prior to installation the voltage change is calibrated against tilt over a two to three degree range of rotation in temperature controlled laboratory conditions. The calibration curve is nearly linear over a well defined range of tilt around the null position but at larger tilts it becomes highly non-linear.

An electrolevel can be used to measure rotations at a discrete point or, if mounted on a bar, between two points. The rotations can then be integrated from one end of a string of bars to provide a profile of displacements. Thus, when mounted on a string of horizontal beams they can be used to determine the profile of vertical displacements along a line. Similarly, when mounted on a string of vertical bars (perhaps down a borehole) they can be used to measure the profile of horizontal displacements at various depths. The main advantage of such systems is that they are able to provide real-time measurements during excavation, compensation grouting or other construction operations. If installed and used under suitable conditions, they can provide an accurate means of determining displacements and there are many examples of successful applications.

In practice, these devices can be highly temperature-sensitive and, unless adequately insulated or temperature compensated, thermal effects can completely mask the movements. They are also prone to long-term drift and, if used in this way, independent means of occasionally checking the measurements should be provided – perhaps by means of precise geodetic measurements.

D.6 General

Many other types of instrument may find application in the monitoring of deep basements including piezometers, borehole extensometers, precise taping, load cells and crack monitoring devices. These are described in the references to this Appendix.

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Appendix E The Observational Method (OM)

E.1 Introduction

It has always been a natural part of the process of civil engineering construction to make visual checks on uncertainties in the ground and on structural performance. This Appendix provides an historical perspective of the background to the Observational Method and describes recent developments and examples of its application.

In the late 1940s, an integrated process for predicting, monitoring, reviewing, and modifying designs evolved with the development of modern soil mechanics theories by Karl Terzaghi and Ralph B. Peck.

They stated: 'Design on the basis of the most unfavourable assumptions is inevitably uneconomical but no other procedure provides the designer in advance of construction with the assurance that the soil-supported structure will not develop unanticipated defects. However, if the project permits modifications of the design during construction, important savings can be made by designing on the basis of the most probable rather than the most unfavourable possibilities. The gaps in the available information are filled by observations during construction, and the design is modified in accordance with the findings'[E.1].

In his 1969 Rankine Lecture[E.2], Peck referred to this process as the 'Observational Method', emphasising that it had specific objectives to deliver cost and/or time savings while maintaining an acceptable level of safety. The OM indeed has formidable potential: to provide benchmarking data; to improve value/economy; to increase safety; to reduce design uncertainties; to strengthen links between designers and constructor to clarify construction control/management; and to motivate the project team.

A strong revival of interest in the potential of OM in the UK during the late 1980s and early 1990s through applications on projects such as the Channel Tunnel and Limehouse Link, led to the publication of the Ninth Geotechnique Symposium in print in December 1994[E.3]. An international symposium followed in London at the Institution of Civil Engineers in January 1995, and the original eleven papers, together with the report of the meeting and a summary section which included suggested ways forward and Peck's 1969 Rankine lecture, were published in a book by Thomas Telford in 1996[E.4]. The interest in the OM was not only driven by the need for more economical use of resources but also because tighter health and safety regulations requiring project participants to assess risk. This momentum continued through further applications on projects and research and development of its potential in current practice. A state of the art report was published by CIRIA in 1999[E.5]. This provides a broad range of possible applications for the OM and sets out robust procedures for its implementation compatible with current design codes and Health and Safety Regulations[E.6].

E.2 The traditional predefined design method and the Observational Method

The intention of the traditional approach is to produce a single robust design that is fully developed before start of construction and that has no special monitoring needs to prove its validity. Terzaghi noted that there was an understandable tendency towards over-conservatism to avoid the risks inherent in designing for average conditions. Instrumentation monitoring is sometimes used but passively, to confirm that design predictions are not exceeded. There is no primary intention to vary the design during construction. The CIRIA report[E.5] refers to this as the 'predefined design method'.

The OM, on the other hand, requires designers to consider the range of foreseeable conditions and to implement a design that, while still robust, more closely reflects the expected conditions. Designs are developed for this range and construction modification strategies planned before work starts on any element. Planning is important to ensure that modifications can be implemented quickly enough to avoid failure conditions developing. Monitoring is essential and is used to provide data for ongoing review of actual performance during construction. The monitoring results are compared with pre-assigned alert and trigger criteria, and planned modifications (if appropriate) or emergency plans (if required) can be introduced.

Peck[E.2] lists the following eight ingredients for a complete application of the method:

- explore sufficiently to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail
- assess the most probable conditions and most unfavourable conceivable deviations from these conditions; geology often plays a major role in this assessment

Table E.1. Comparison between the predefined design method and the Observational Method

The predefined design method	The Observational Method
Normally one set of soil parameters: e.g. moderately conservative or characteristic values (BS 8002[E.6]), but may do parametric study.	The range of foreseeable soil parameters is considered: e.g. most probable and most unfavourable.
One design and one set of predictions based on limited construction method considerations.	Two or more design and construction methods are sufficiently developed to include predictions for trigger criteria.
A construction method option may be outlined sufficiently for the design to be progressed. The contractor subsequently develops this in his method statement.	A flexible construction method statement is developed that can incorporate design changes and modification strategies: often developed jointly by the contractor and the designer.
Monitoring is limited to checking that predictions are not exceeded.	Comprehensive and robust monitoring, regularly reviewed, as the basis for management and design decisions.
Predictions are unlikely to be exceeded. Therefore construction programme is not constrained by monitoring results. If predictions are exceeded, unforeseen conditions have developed and the work may need to stop while problems are resolved.	The design, construction method and construction programme may be changed, depending on the review of monitoring results.
	Management of construction, monitoring, interpretation and modification plan or emergency plan implementation are required.
	The monitoring system must be sensitive enough to allow early discovery of a rapidly deteriorating condition. The modification plan must be rapidly implemented to ensure that the limiting trigger criteria are not exceeded.
Emergency plans are needed to control failure.	Emergency plans must be introduced in accordance with the Construction (Health, Safety and Welfare) Regulations 1996[E.7]. Extending the OM trigger criterion beyond the serviceability limit state, to ensure that failure does not cause injuries, can do this.
The OM may be initiated at this stage in its 'best way out' format.	It may be that the best way out of OM can be introduced to overcome unforeseen ground conditions.

- establish the design, based on a working hypothesis of behaviour anticipated under the most probable conditions
- select the quantities to be observed as construction proceeds, and calculate their anticipated values on the basis of the working hypothesis
- calculate the same quantities under the most unfavourable conditions compatible with the available data on the subsurface conditions
- select in advance a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis
- measure quantities to be observed and evaluate actual conditions
- modify design to suit actual conditions.

If there is little uncertainty about the ground and /or soil-structure interaction, there should be no need to follow the OM, and there would be no active monitoring or planned modifications. But, if there is significant uncertainly, the predefined design method could lead to a possibly unsafe or maybe unnecessarily expensive solution. The OM uses feedback from the monitoring results in a formal planned approach to provide an acceptably safe and more economic solution.

E.3 Recent developments in using the Observational Method

After Peck's 1969 Rankine Lecture, the OM gained worldwide recognition and was used in a wide range of ground engineering operations. However, it has not been referred to in British design codes, although the final draft of EC7[E.8] recognises it as a design method and states the requirements for using it. Similar requirements have been adopted in the Hong Kong *Guide to retaining wall design*[E.9]. One objective of the CIRIA Report[E.5] is to clarify OM concepts and to provide a clear framework. It provided the following definition of the method: 'the Observational Method in ground engineering is a continuous, managed, integrated, process of design, construction control, monitoring and review with enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety'.

The OM can be adopted from the outset, or later if benefits are identified. However, it should not be used where there is insufficient time to implement fully and complete safely the planned modification or emergency plans. Possible modes of failure must be carefully assessed and controlled, particularly those of a sudden or brittle nature, or those that could lead to progressive collapse. Safety is essential and a high degree of certainty in project performance and schedule is generally required. The OM overcomes the limitations of conventional design by evaluating feedback from actual conditions[E.10]. This improves risk management which can be further enhanced by use of the progressive modification approach (see following).

E.4 Implementation

The process of implementation (see Figure E.1) emphasises national and corporate policies, e.g. health and safety regulations, quality assurance, conditions of contract, and design codes. Good corporate and project team organisation are also essential.

Design and planning are concerned with gathering data, design, interpreting data, assessing risk, and allocating resources to achieve objectives and decide priorities. Design cases should cover all likely scenarios, and design modifications should be planned so that they can be introduced in time to stop risk increasing unacceptably. The construction control plan, monitoring plan, and monitoring specifications should be developed to set out agreed procedures and frequency for monitoring and reporting results.

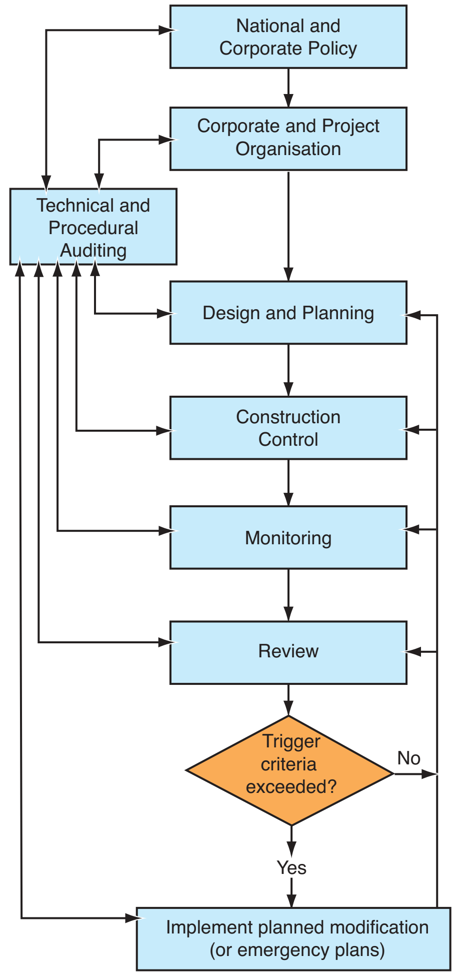

Fig E.1 Implementing the OM

Competent people should review instrumentation records and construction progress information. The planned contingency modifications will be implemented if the trigger criteria have been exceeded.

E.5 Progressive modification approach

The CIRIA report[E.5] identifies the recent development of progressive modification as the preferred approach where the design and construction team has limited experience of the OM or where incremental construction is proposed. In fact a step-by-step approach inherently offers many benefits including enhanced feedback and an improved potential to identify trends in performance. As noted, the OM facilitates design changes during construction and establishes a framework for risk management. It is not surprising that proposing changes tends to create concerns regarding safety and certainty. However, it is unfortunate that the method may be inappropriately associated with uncomfortably low safety margins coupled with the potential cost and delay of contingency measures. Progressive modification permits technical or contractual constraints to be addressed by accommodating the concerns of all parties involved in the project[E.10]. Such constraints have discouraged wider and more frequent application of the OM. Starting with a design based on estimations of the most probable conditions may not be acceptable. The associated level of risk perceived by some parties to the contract may be too high. Concerns may arise from lack of case history data or confidence in the quality of information and proposed parameters. Without an alternative strategy, use of the OM may not be approved.

With progressive modification, it is the overall performance that is progressively measured and evaluated including soil/structure interaction, construction methods, communication and teamwork. The objective is to demonstrate the basis for introducing design changes sequentially during construction that create cost or time savings, or to avoid unnecessary contingencies. The latter particularly applies to 'best-way-out' cases where phased construction allows feedback and re-evaluation of predictions for each subsequent phase.

This requires additional design work, monitoring and supervision but this should be absorbed in the overall benefits. The basis of the progressive modification approach is to:

- commence construction with a design providing an acceptable level of risk to all parties
- maintain or decrease this level of risk
- progress construction in clearly defined phases
- implement appropriate changes progressively and demonstrate acceptable performance through observational feedback.

Most potential for savings relates to temporary works or construction method and sequence. There may also be substantial savings in permanent works, for example through avoiding inappropriate protective works or providing the basis for innovation in future construction[E.10]. Some management considerations are shown in Figure E.2.

E.6 Risk, contractual aspects and value engineering

The OM is essentially a risk management system. Yet concerns about increased risk are usually among the first to be expressed when introduction of the OM is proposed. However, experience shows that proper implementation can lead to increased safety. This may be achieved, for example, by:

- avoiding inappropriate contingencies
- eliminating heavy and constricting temporary works and creating freer working space
- focusing awareness on the importance of teamwork, good communication, clear procedures, control during construction, and the need for planned contingency measures.

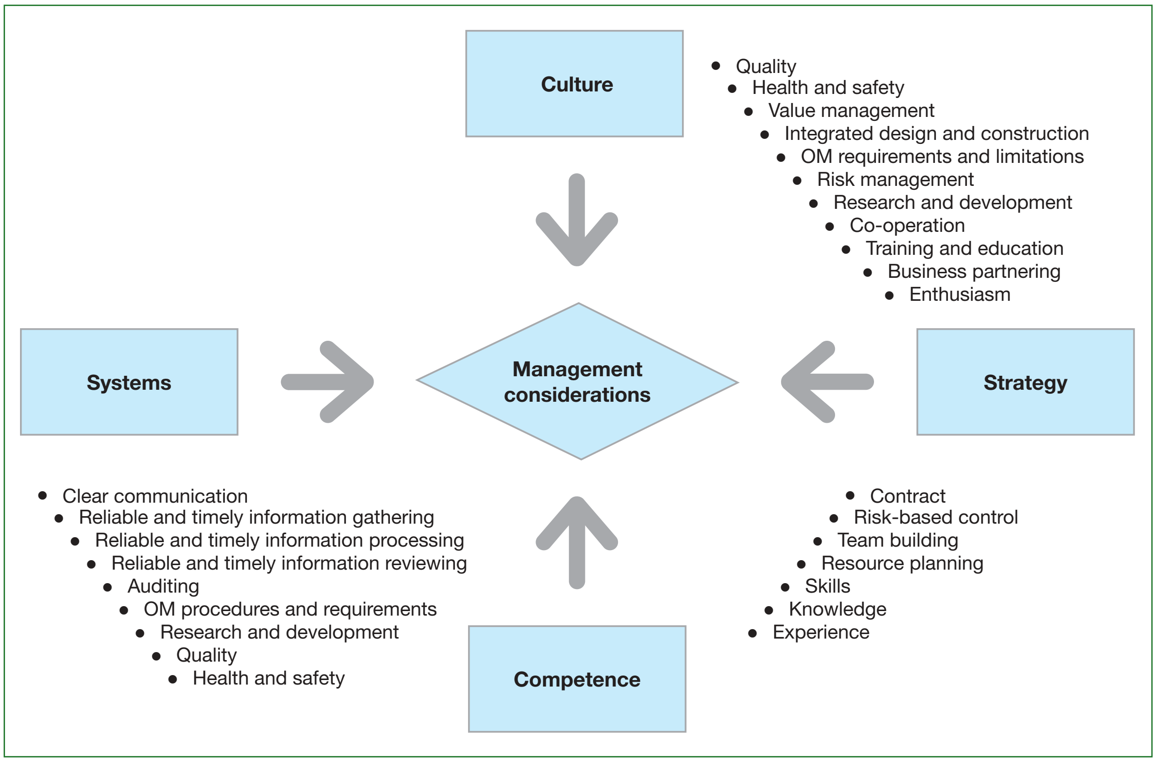

Fig E.2 Management considerations

The strong compatibility between the OM and value engineering was demonstrated at Limehouse Link[E.3] (see Figure E.3). Both techniques are directed at creating savings in cost or time[E.11]. They also demand an enhanced relation of design to construction and require similar contractual conditions. The inclusion of a value engineering clause in a construction contract can facilitate the introduction of the OM. The Heathrow Express cofferdam was another application where the two techniques were combined[E.10]. The New Engineering Contract[E.12] (NEC), adopted for this project, facilitates change and, with the single team culture, made the conditions very conducive to application of the OM. Published in 1995, this form of contract seeks to establish a fair balance of risk between the parties.

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Fig E.3 Results of value engineering: omission of struts by using the OM, temporary roof support and lighter slabs © Benaim

Glossary

Aquifer: A water bearing stratum that is highly permeable.

Artesian head: When the water pressure, or 'head', in a particular geological stratum is greater than the theoretical hydrostatic pressure from the near surface groundwater table, the stratum is said to be under artesian head or pressure.

Autogenous healing: This process involves the transport of debris and fragments of soluble calcium hydroxide, $Ca(OH)_2$, within a crack in concrete, forming the insoluble compound calcium carbonate, $CaCO_3$, on contact with carbon dioxide, CO_2, in air.

Crib wall: A retaining wall constructed from a timber, steel or concrete framework filled with boulder sized stones or rocks.

Cut-off wall: A wall installed in the ground for the purpose of preventing flow of water through it thereby causing the water to flow down and around it.

Drawdown: Reduction of groundwater pressure at some distance from a point where drainage or pumping is taking place.

Flying shores: Props or struts used to support free standing walls.

Gabion wall: A retaining wall constructed from a number of wire mesh boxes with internal diaphragms filled with boulder sized stones or rocks.

Heave: Upward movements of the ground.

Heave gap: A space left beneath the bottom slab of an excavation to allow the underlying soil to move upwards freely.

King post: Vertical steel section in temporary retaining wall to support horizontal laggings and transfer lateral soil thrust to ground anchors or struts spanning the excavation.

Loss of ground: Removal of soil due to the flow of water causing erosion.

Observational Method: See Appendix E for a detailed description of the method.

Over-consolidation ratio (OCR): The ratio between the maximum previous vertical effective stress and the present one in a soil stratum. Its determination usually requires a detailed knowledge of the geological history of the site. It is often estimated from the results of laboratory oedometer tests (one-dimensional).

Plunge column: Vertical steel section intended to carry the weight of basement floors during excavation, installed by being vibrated into the wet concrete of a pile and surrounded in granular material to stabilise it during excavation.

Rakers: Inclined props or struts used to support a retaining wall.

Soil berm: A narrow bank of soil located at the bottom of a retaining wall, to surcharge the soil in front of the wall.

Tremie (pipe): A steel jointed tube used to pour wet concrete or grout through water or slurry.

Tubes à manchette (TAMs): A system of grout tubes with sleeves and packers to allow stage grouting and regrouting at chosen depths.

Walings: Horizontal beams on the face of a retaining wall used to transfer lateral soil and water pressure to rakers, struts or ground anchor supports. (Also known as walers).

Wellpointing: A system of connected wells or bore holes in the ground from which water is being extracted by pumping.

INDEX

Index Terms	**Links**			