DEEP EXCAVATIONS

A practical manual

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Thomas Telford

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Preface

The objective in preparing this book has been to assemble both practical rules and details for the economic and efficient execution of deep excavations. Although formal Codes of Practice may begin life with the same intention, it appears that they may also constrain innovation and ingenuity behind a set of rules and regulations which, as time passes, become regarded as the only basis of acceptable foundation engineering design. It is hoped that the collected data and experience that follow present examples of design and solutions to construction problems which dispel the conception of design by regulation.

Any presenter of practical experience has two difficulties: the first is the need to ensure that the contents of the text are up-to-date, the second to ensure that the data and examples have worldwide application and are not only directed to the Author's locality. The libraries of the Institution of Civil Engineers and the Institution of Structural Engineers have assisted in literature searches to ensure that references are not dated; the help of their staff is gratefully acknowledged. The Author has benefited from the publication, shortly before the completion of the text, of the British Standard Code of Practice on Earth Retaining Structures, BS 8002. The presentation of typical construction costs may also suffer the pitfalls of not being up-to-date at the time the book is read. The costs relate to those in the UK in 1994, and the reader is asked to show forbearance if both time and location make the data presented less than precise.

The Author is also aware that the solution to an excavation problem in one geographical location may not succeed elsewhere, where ground conditions, the geometry of the substructure to be built, or other critical circumstances, are not sufficiently similar to replicate that success. A knowledge of precedents, nevertheless, has been quoted (by Professor R.B. Peck) as an essential prerequisite to the successful practice of subsurface engineering and, together with familiarity with soil mechanics and a working knowledge of geology, these are the attributes needed for the efficient appraisal and solution of deep foundation problems. The Author therefore trusts that the balance of experience, geotechnical theory and basic geology is correct and will provide a long-term reference.

The Author gratefully acknowledges the permission of publishers and authors to reproduce design data and construction details from published work. In particular, the authors and publishers of the Construction Industry Research and Information Association Report on Cofferdams, the Hong Kong Government Departmental Report on Design Methods for Excavations, and many papers from the Proceedings of the Institution of Civil Engineers, London, are especially thanked.

The Author acknowledges the help and criticism of colleagues during the preparation of the book, in particular David Puller, and the patience and care of secretaries Jan Caddy and Carolyn Puller.

MJP
Sevenoaks, Kent

Conversion factors

Some of the cases and examples cited in this book used Imperial units. For the convenience of readers the following list of conversion factors is provided.

To convert:	multiply by:	To convert:	multiply by:
in to cm	2.54	cm to in	0.394
ft to m	0.36	m to ft	2.742
yd to m	0.914	m to yd	1.094
miles to km	1.609	km to miles	0.621
in^2 to cm^2	6.452	cm^2 to in^2	0.155
yd^2 to m^2	0.836	m^2 to yd^2	1.196
in^3 to cm^3	16.387	cm^3 to in^3	0.061
ft^3 to m^3	0.0283	m^3 to ft^3	35.315
lb to kg	0.454	kg to lb	2.205
kips to kg	453.952	kg to kips	0.002205
tons (UK) to metric tonnes	1.016	metric tonnes to tons (UK)	0.984
lbf to N	4.448	N to lbf	0.225
kgf to N	9.81	N to kgf	0.1019
lbf/in^2 to Pa	6.895	Pa to lbf/in^2	1.450
$kips/ft^2$ to kg/cm^2	0.488	kg/cm^2 to $kips/ft^2$	2.048
in^2/s to cm^2/s	6.452	cm^2/s to in^2/s	0.155

Deep excavations: a practical manual. Thomas Telford, London, 1996.

Contents

1 Introduction

The purpose of the book is to present, in a selective way, design and construction for deep excavations made for civil engineering purposes. Emphasis is placed on descriptions of work constructed, in some instances within the Author's personal responsibility, but for the most part as observed by the Author and as described and reported by others.

Both temporary construction and permanent work are described. The design of temporary construction to support soil and rock at the excavation periphery, and to exclude groundwater, frequently becomes the responsibility of the contractor, and there may be incentives to devise methods which economize in construction time and cost. The term 'temporary' may mislead; on major works, measures of peripheral soil support and groundwater exclusion may require sufficient strength and durability to last several years. The difficult task of the temporary works designer, to provide an adequate but time-related solution without waste, should not be underestimated. The final cost of temporary works may also depend upon the ease of their eventual removal. In some cases, their incorporation into the permanent works, as with slurry walls, can mitigate against the cost of soil support during the construction period.

Permanent construction is divided in the text, for the sake of convenience, into work in shafts and caissons, basements and cut-and-cover construction, with some obvious overlap with temporary works. Traditionally, the design of permanent work and responsibility for its adequacy lies with the consulting engineer. The exceptions are those projects designed by the owner's organization or which are the subject of a turnkey or design-and-construct arrangement where a contracting firm may assume professional responsibility for permanent construction. The designers of temporary and permanent works are therefore often not the same persons on a particular job, and in many instances they are not employed by the same organization.

The choice of deep excavation work to include in the book has been guided by a definition used in the CIRIA report[1] on trenching practice. This report covered trench excavations to a depth of 6 m. As an approximate division between shallow and deep excavations, this 6 m depth has been adopted by the Author; for the most part, this book features work greater than this depth. Mention should nevertheless be made of the high risk of failure in excavations less than 6 m deep and covered by the CIRIA report, which stated that a considerable number of deaths and injuries occur in trench collapses in the UK each year; the analysis of fatal accidents between 1973 and 1980 showed the cause of such accidents to be:

- unsupported excavation: 63% of cases
- working ahead of support: 20% of cases
- inadequate support: 14% of cases
- unstable slopes of open cut: 3% of cases

Further, it was shown that more than one-third of the fatal accidents analysed for this period occurred in made ground or where the soil had been disturbed by earthworks.

Safety and avoidance of damage

The design of soil support to deep excavations on land must ensure an adequate factor of safety against collapse in the short term, that is, during the construction period, and an adequate factor of safety against collapse for the design life of the permanent substructure. In addition, in both the short and long term the design of the works must be such as to contain deformation of the soil or rock adjacent to the excavation to limits which do not cause distress to existing structures or services. Standards of construction at each stage must be such that the work complies with the strength assumptions used in the design and is sufficiently durable to avoid deterioration and movement or collapse. Additionally, construction standards must be such to avoid loss of ground into the excavation which might cause an unacceptable risk of subsidence or collapse.

Similar criteria apply to deep excavations below river or sea beds. For the most part, deformation or subsidence will be less important unless existing works are nearby. The risk of scour effects to the sea or river bed, which are possible as a result of the new works themselves, may prove an additional hazard which could cause structural collapse, and must be guarded against. Variations in sea and river states in terms of tide, storm swells and wave conditions all require assessment in terms of risk to safety and to structure; given the likely consequences of collapse during construction or thereafter to foundation works at sea or on major rivers, strenuous care is required in the assessment of such factors.

The risk to construction personnel and users of the permanent structure must be defined separately to the risk of subsidence damage to property and services. In the former case, the awesome consequences of inadequate standards of design and construction in deep excavation works should be self-evident, and particularly so on the site of the works. Frequent reference is made to modes of failure throughout the text; the experience of previous failures is seldom reported, and even less so is this experience used.

Design and construction works for the support of deep excavations require investigations of the site topography, the subsoil and groundwater conditions, the states of sea and river water and the stability of sea and river beds, where applicable, the risk of seismic loading, the extent of superimposed loads, the state of existing structures and services, and the availability and quality of available structural materials. None of these matters are treated specifically, although without such information, of adequate quality, prior to design of both the temporary and permanent works, all reference to avoidance of risk to life and reduction of damage to property becomes meaningless.

Construction regulations

In many developed countries, the design and site works for deep excavations are subject to statutory regulations which are devised to maintain minimum standards of site construction safety.

In the UK the principal legislation is the Health and Safety at Work Act 1974[2] which specifies the responsibility of employers, suppliers and employees for all work, and is complementary to specific construction regulations which include:

(*a*) for any type of building work and most types of civil engineering and construction works:
  (i) The Construction (General Provisions) Regulations 1961[3]
  (ii) The Construction (Lifting Operations) Regulations 1961.[4]

(*b*) for work under compressed air: Work in Compressed Air Special Regulations 1958[5]

(*c*) for work above water: various dock regulations, which come under the Factories Acts, Merchant Shipping Act, Coast Protection Act, Port and Harbour Regulations, etc.

(*d*) for work with explosives: Construction Regulations Part VI — Explosives

1961;[6] Explosives Acts 1875 and 1923,[7] together with Orders in Council Nos 6, 6A, 6C, 12, 13, 16 and 28; Mines and Quarries (Explosives) Regulations 1959;[8] Conveyance of Explosives Bylaws SI No. 230.

An inspectorate, the Health and Safety Executive, is responsible for the implementation of the Health and Safety at Work Act and the Construction Regulations. These pieces of legislation place duties on both employer and employee: while the employer must provide safe access, a safe place of work and a safe system of work for employees, every employee must take reasonable care for the safety of others and cooperate with the employer. The employee must not intentionally or recklessly interfere with or misuse anything provided in pursuance of the requirements for health, safety and welfare. All cofferdams and caissons must be properly constructed, altered or dismantled under competent supervision and, wherever possible, by experienced operatives. Every cofferdam or caisson must be provided, so far as is reasonably practicable, with ladders or other means of escape in case of flooding. Inspections of cofferdams, caissons and trenches must be inspected when work is in progress and, in addition, they must be thoroughly examined and records made whenever explosive charges have been fired, whenever any damage has occurred, or, in any case, every seven days. Other regulations require the inspection of lifting plant and excavators, and the management is held responsible for the competence of designers, supervisors and operatives.

In 1992 an extension to the Health and Safety at Work Act required all employers in the UK, not only those in construction, to carry out assessments of risks to safety. A further widening of responsibility was made in 1995 when the Construction (Design and Management) Regulations 1994 placed a duty on designers to avoid risks to safety and health.

The above in no way gives a complete explanation of safety legislation applying to deep excavation sites in the UK but shows the change, particularly at site, made in 1961 and thereafter with the introduction of statutory regulations for site safety.

Permanent works, for building construction, such as basements, are the subject of building standard control legislation either nationally or by city in all developed countries.

Contractual responsibility: client, engineer and contractor

In addition to the responsibility spelt out in statutory regulations for works on site and the design of temporary works, the contracts between employer and contractor and client and consultant will define responsibility for the adequacy of both the temporary and permanent soil support. Contract conditions will vary between countries and from job to job.

In the UK, an earlier standard form of contract between contractor and client for civil engineering works, the ICE form (fifth edition), was commented on by Abrahamson,[9] who concluded that the responsibility for temporary works was complex and examined four issues:

(*a*) Responsibility of the contractor to the employer. The contractor's responsibility to the employer is clear by virtue of the clause wording: 'The contractor shall take full responsibility for the adequacy, stability and safety of all site operations and method of construction'. So, if any temporary works design by the contractor, or subcontractors (whether nominated or not), are inadequate, the deficiency must be remedied by the contractor, and if any damage to either permanent or temporary works is caused by the deficiency the contractor becomes liable to rectify this also. Temporary works designed by the engineer do not become the design responsibility of the contractor under this clause.

(*b*) Responsibility of the engineer to the client. The engineer has a plain duty to the client to ensure that the permanent works are not distressed by loads induced from the temporary works and that the temporary works are built in accordance with the design whether by the contractor or the engineer. In addition, the engineer carries a duty to the client to design the temporary works where it would not be satisfactory to allow the contractor to make the design.

(*c*) Responsibility (or perhaps lack of responsibility) of the employer, via the engineer, to the contractor. The engineer has no obligation to the contractor to detect or prevent faults in the temporary works. While the engineer has rights of control under the contract, the contractor cannot excuse bad workmanship in temporary works on the basis that the resident engineer made no objection.

(*d*) Responsibility of the employer, engineer and contractor to employees and members of the public. The contractor is liable to employees and other third parties if a duty of care is not discharged in designing or constructing temporary works, and the contractor is probably also liable for a defective design by the engineer when an experienced contractor would have known it to be defective. Abrahamson concluded that the engineer's liability to third parties as a result of temporary works failure was most difficult to define. Without doubt the contractor, under this particular form of contract, does hold much of the responsibility for the safe design and performance of temporary works, such as temporary soil support. In particular, he holds responsibility towards the client for the adequacy in design and construction of such temporary support works by subcontractors and even nominated subcontractors.

It is evident that works such as piling or diaphragm walls which at different stages serve functions both of temporary and permanent soil support require special reference in the conditions of contract for such work. The case law on such matters is probably sparse and the division of responsibility between contractor and engineer may be without legal precedent. In the UK it is not unusual for a subcontractor to provide the design of a specialist soil support system, say a diaphragm wall, to act both during construction and as part of the permanent structure. Presumably, under ICE conditions, the engineer's approval of the subcontractor's design for the permanent performance of the wall element would to some extent relieve the contractor's responsibility in that direction, whereas the performance of the same element during construction would remain solely the contractor's responsibility. It is interesting to note that severe distress of such a wall panel, should it occur during construction, say below formation level, would not necessarily become apparent at that time and may only be revealed by the non-performance of the permanent works. The assessment of fact and the legal position of works designed to act in temporary and permanent stages may prove to be complicated.

Causes of failure in deep excavations

The failure of a soil support system does not necessarily occur by structural collapse; other types of failure include excessive deformation of the soil and soil support structure, inadequate groundwater exclusion, and insufficient durability of the soil support structure resulting in failure over time.

In the Author's experience the causes of failure may be summarized as follows.

(*a*) Open excavations

(i) inadequate site investigation resulting in optimistic design assumptions of soil, rock strength and groundwater conditions
(ii) inadequate appreciation by the designer of susceptibility to settlement of adjacent structures and services
(iii) lack of appreciation by the designer and constructor of the effects of weathering and time on soil strength.

(*b*) Braced excavations

(i) inadequate site investigation resulting in optimistic design assumptions of soil and rock homogeneity, strength of soil and rock fabric, and groundwater conditions
(ii) inadequate quality of structural detailing
(iii) inadequate coordination between designer and constructor
(iv) lack of appreciation by the designer of the limitations of specialist techniques such as diaphragm walling and anchoring
(v) lack of appreciation by the designer of the influence of deflexions in soil support structure and retained soil deformations
(vi) changes in loading from natural conditions — groundwater, tidal states, waves, temperature — and lack of appreciation by the constructor of the possible consequences of these changes
(vii) changes in soil and rock conditions and the lack of appreciation by the constructor of the possible consequences
(viii) overloading of soil support structure by temporary plant loads
(ix) bad workmanship in site temporary works.

Sowers[10] stated that within his experience failures of anchored sheet pile walls and braced excavations seldom occur as the result of inadequacies of modern earth pressure theories. Instead, they are caused by the more obvious neglect of backfill loads, construction operations that produce excessive earth pressures, poorly designed support systems and inadequate allowances for deflexions, deterioration and corrosion, and poor design in construction details.

In the Author's experience, structural failure of braced and anchored walls has usually occurred within the strutting or anchorage, or by passive soil failure below formation level caused by inadequate sheeting penetration. In other instances, fewer in number, very bad standards have caused gaps within walls allowing cofferdam blows to occur with extensive loss of ground from behind the walls.

Deflexions caused by loads applied to soil support systems observed by the Author have generally been less than the tolerances allowed for in construction of the systems and have not resulted in extensive remedial works.

The extent of soil deformations around large excavations are referred to in published work more readily than records of structural failure; Peck[11] and Clough and Davidson[12] reviewed the likely range of horizontal and vertical movements. Clough and Schmidt,[13] in considering the design and performance of excavations in soft clay, used data from Peck,[11] D'Appolania[14] and Goldberg *et al.*[15] to show that settlements associated with excavations where basal stability is a problem exceed those where no such stability problem exists.

Records of soil deformations caused by particular excavations in London were referred to by Cole and Burland,[16] and more recently by Wood and Perrin.[17] Burland *et al.*[18] examined movements near excavations into London clay and stated that while the magnitude of ground movement will depend on methods of construction and day-to-day sequences of work made on site, it should be possible to make reasonable estimates of upper and lower limits of movement, especially when field measurements add to knowledge of the conditions. Calculated predictions of deformation using numerical methods rely on accurate assessment of soil deformation parameters; back analysis of field measurements from nearby excavations may provide these values.

Clough and Davidson[12] concluded that for a given depth of excavation the amount of ground movement depends on the properties of the retained soil and not on the stiffness of the temporary supporting wall, but Goldberg *et al.*[15] stated that wall stiffness is an important factor in such soil deformation. Defining wall stiffness by a parameter EI/h,[4] where EI is the flexural stiffness of the wall and h is the distance between supports, and plotting this against the stability number for excavation in clays H/c_u, boundary lines were given to show orders of expected lateral wall movement.

The use of temporary berms and other construction procedures to reduce movements were referred to by Clough and Schmidt[13] and will be treated in detail in the following chapters.

Subsidence may occur near excavations as a result of the installation process of walling, sheeting and anchorages in addition to deformations caused by loading. Deformations which occur during installation of unlined borings for piles and diaphragm walls and movements caused by pre-loading of ground anchors are rarely appreciated at the design stage.

White[19] referred to several cases where the declination of rock anchors used to support temporary sheeting caused settlement to occur as a result of the vertical load component overstressing rock below the tip of soldier piles.

Risk evaluation

Casagrande[20] emphasized that risks were inherent in any project, their existence should be recognized and, using steps representing a balance between economy and safety, these risks should be treated systematically. Casagrande defined 'calculated risk' in two parts:

(*a*) the use of imperfect knowledge, guided judgement and experience, to estimate the possible ranges for all pertinent quantities that enter into the solution of the problem

(*b*) decisions, on an appropriate margin of safety, or degree of risk, taking into consideration economic factors and the magnitude of losses that would result from failure.

Casagrande did not quantify risk. Later, Whitman[21] reported considerable advances in reliability and probabilistic theory, but stressed that use of such methods was no substitute for physical measurements and sound engineering interpretation. Concluding, Whitman said that the satisfactory evaluation of risk could be answered in two ways.

(*a*) If a relatively large probability of failure (0·05 or more) under design loading were tolerable, then this risk could be evaluated (by reliability theory) with sufficiency accuracy for decision-making purposes. This situation applies only when economic loss and not safety are of concern.

(*b*) If a very small probability of failure (say less than 0·001) under design loading conditions is required, the actual risk cannot be evaluated by analysis. However, conducting a formal evaluation of the probability of failure can help greatly in understanding the risk and what might best be done to reduce it.

The design of many deep excavation schemes must certainly lie within the second category where the acceptance of failure probability must be very low indeed by virtue of risk to life.

Whitman illustrated his paper with applications of reliability theory to examine systematic and random errors when evaluating risk in slope stability, factors of safety in risk analysis of liquefaction, and the use of system analysis techniques for quantifying risk on a project basis. Examples of risk evaluation were given for an industrial plant built on potentially liquefiable sands and for earth dam construction.

Höeg and Muraka[22] considered the conventional design of a simple gravity retaining wall and made a statistical analysis of the wall so designed. For given soil properties and backfill height, this design used factors of safety of 1·9, 3·7 and 1·6 against overturning, bearing failure and sliding, yet despite these apparently conservative values the statistical analysis showed that the corresponding failure probabilities were 1/10 000, 13/1000 and 3/1000. The probability of bearing failure is particularly high, and large differences were also indicated in failure probabilities between each failure. This example shows the ease with which conventional factors of safety are able to mislead.

Höeg and Muraka then redesigned the gravity wall using probabilistic methods, evaluating initial costs, construction costs, costs of failure and the probabilities of failure by overturning, bearing and sliding, to determine the expected total cost. The optimum design was the system with minimum expected total cost.

The principle of risk assessment is within the scope of this book, but probability is not (for this see Whitman[21] and Höeg and Muraka[22] and the references therein).

The primary intent of Höeg and Muraka was to provide a model for the probabilistic design, by similar methods, of more complicated structures such as braced and anchored walls, but despite their intrinsic logic there is little indication that such methods have gained acceptance by designers.

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2 The control of groundwater

The occurrence of groundwater on site directly influences construction methods, permanent works design and, thereby, construction time and cost; possibly, in the longer term, the durability of the structure and its maintenance costs will also be affected. This chapter considers only the first of these, construction methods of dewatering deep excavations. It addresses groundwater control in three ways: the problem presented to the constructor, the techniques available to him, and the calculation methods to assist in the design of groundwater control. The chapter draws heavily from the CIRIA report[1] and BS 8004.[2]

Groundwater problems

The sources of groundwater on a particular site may be three-fold: rainfall, run-off, or groundwater flow through pervious soils or rock from streams, rivers or the sea. Variations in soil and rock conditions, in particular permeability, horizontally and vertically, causes variations in groundwater flow both on the ground surface and below it. The degree to which groundwater is contained by relatively impermeable soil above or below a permeable stratum will in turn influence any excess or artesian pressure within stored groundwater.

Reducing the quantity of groundwater within subsoil adjacent to an excavation by a dewatering process such as pumping will in turn increase the strength of the soil as the groundwater pressure, or pore pressure, is reduced. Reduction in groundwater head therefore reduces the load, say on the bracing to a deep excavation, and provides a method of improving soil strength.

The effective stress is the difference between the applied, total stress and the pressure induced by loading to groundwater within the pores of saturated soil. As the soil is loaded, say from a building foundation, the increase in load is shared between the soil structure and the pore-water within the soil. The stress carried by the soil structure, known as the effective stress σ', is therefore equal to total stress σ less pore-water pressure (u). Since water possesses no strength, the soil reduces in volume as the water is displaced. The rate of this change in volume and change in pore pressure depends on the permeability of the soil fabric and physical drainage conditions. The dissipation of pore pressure and the volume changes stop when equilibrium is reached with external forces applied to the soil mass.

So, the shear strength of the soil τ' depends on the effective stress and at failure is

$$\tau' = c' + \sigma' \tan \phi' \qquad (1)$$

where c' is the cohesion of the soil in effective stress terms, and ϕ' is the angle of shearing resistance in effective stress terms. The total stress concept, therefore, is a stress state which applies only at one instant, whereas the fully drained equilibrium condition when effective stress is maximized occurs with time. This time is relatively short with a granular, permeable soil but longer with an impermeable, cohesive soil.

The reduction of groundwater within an excavation may be necessary for access

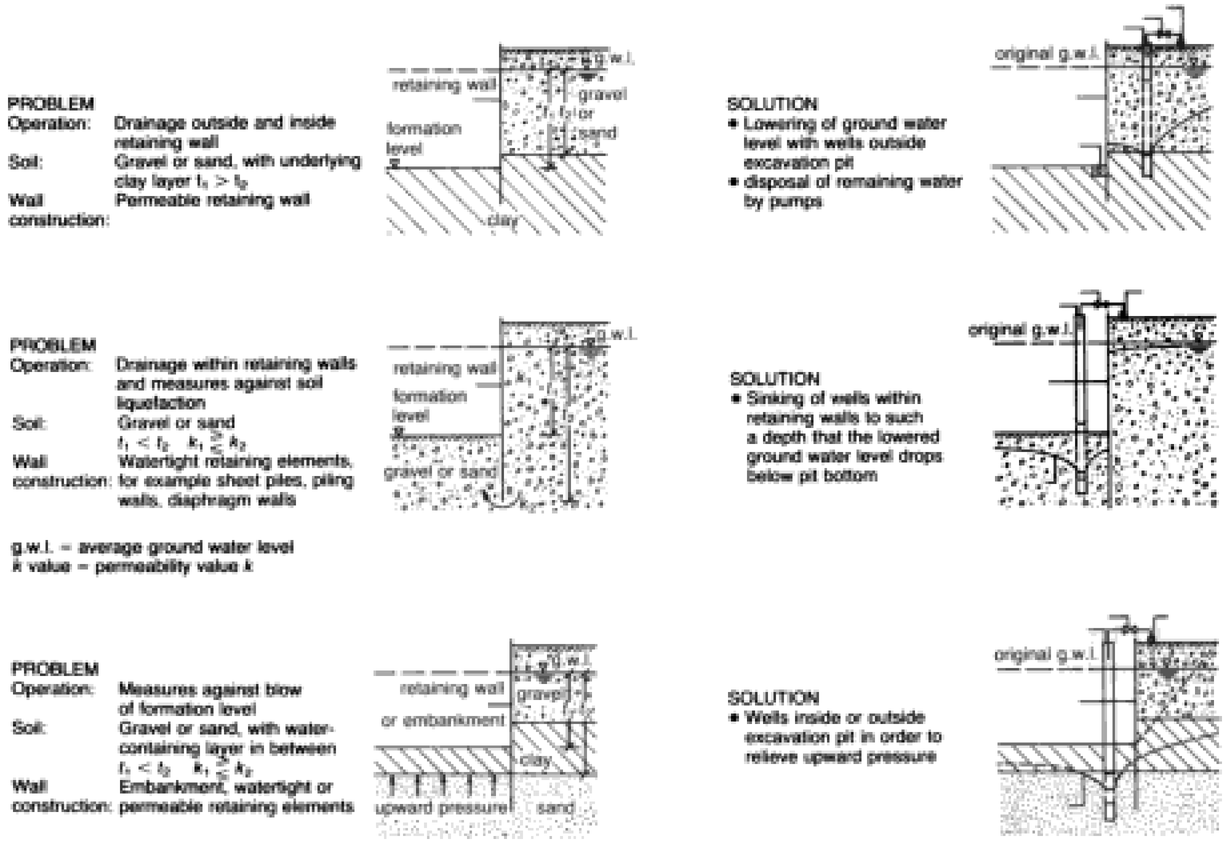


Fig. 2.1. Groundwater control applied to a deep sheeted excavation in various ground conditions

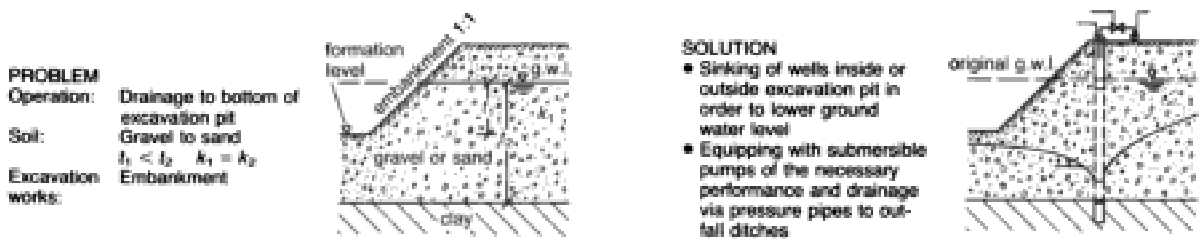


Fig. 2.2. Groundwater control applied to a battered excavation

by workers and machines. However, removal of groundwater from below the excavated level and externally to the excavation may be required to improve soil stability below and around the excavation itself. Examples of groundwater control applied to a deep sheeted excavation in various ground conditions are shown in Fig. 2.1. Reduction in groundwater levels and piezometric head allows progressive excavation in the dry, reduces pressure on the sheeting, prevents base uplift at formation level of the completed excavation, and allows the strength of the soil to increase progressively as effective stress conditions apply to a fully drained, dewatered soil condition. Fig. 2.2 shows a similar improvement to working conditions and soil strength in a battered, open excavation.

The quantity of groundwater that may be abstracted from individual sites may be restricted by local legislation. In Berlin, for example, a maximum discharge from any one site into watercourses has for some years been limited to 500 000 m^3.

Available methods of groundwater control

The four main methods used to exclude groundwater from deep excavations are:[1]

(*a*) Stopping surface water from entering the excavation by using, for example, cut-off ditches, low walls and embankments.
(*b*) Allowing water to flow into the excavation and subsequently pumping it from drainage grips, ditches or French drains.
(*c*) Pre-draining the soil by lowering the groundwater level ahead of the excavation, for example, by use of wellpoints or deep wells.
(*d*) Stopping the groundwater from entering the excavation by a cut-off wall within the soil, such as a cement–bentonite slurry wall.

Table 2.1 shows the wide range of available techniques, which fall broadly into these four categories. Selection of the most effective method at minimum cost will depend on a number of factors, such as the dimensions of the excavation (in particular its depth), the thickness and type of soil strata, the depth of the excavation relative to soil types, the magnitude of groundwater pressures in each stratum, the prevention of damage to nearby structures and services, and the length of time the excavation is to remain open.

Preliminary guidance on choosing the best method may be gained from Table 2.2, which shows the influence of the width and depth of the excavation. The range of application of dewatering techniques (related to permeability and drawdown) are shown in Fig. 2.3, and similarly the range of groundwater exclusion methods varying by soil particle size are shown in Fig. 2.4.

In very broad terms, open excavations are frequently dewatered using single- or multi-stage wellpointing systems and sheeted excavations for basements, or cut-and-cover construction often use sump pumping where the sheeting can be economically driven or excavated into an impermeable stratum to get a natural seal. Where a natural cut-off cannot be obtained, a horizontal grout plug can be injected to obtain a cut-off. Where artesian pressure heads need to be relieved in deep strata below excavations, this may be done by relief wells or deep pumped wells. For both open and sheeted excavations, deep wells with submersible pumps at the well screens are often used to obtain a drawdown which would not have been possible by wellpointing or sump pumping. The use of vertical cut-off walls to isolate construction areas from inundated surrounding areas of subsoil is becoming more popular as economical systems of bentonite–cement slurry walls become available.

A site investigation is necessary before choosing a dewatering method. This will accurately define the depths and type of strata, from which permeabilities may be estimated and groundwater levels assessed. The site investigation must disclose any tidal influences. Where groundwater is to be removed from the site, its method of disposal must be investigated and any risk of contaminants within the groundwater carefully evaluated.

The contractor is therefore faced with a choice of dewatering or groundwater exclusion methods which will each have advantages and disadvantages in terms of cost, overall efficiency, time and convenience. Some indication of the cost of each method can be seen from Tables 2.3–2.5, originally included in reference 1 but here augmented and corrected to 1995 prices.

In terms of overall efficiency, BS 8004[2] recommends the conditions that should be fulfilled when dewatering excavations:

(*a*) The lowered groundwater level should be kept under full control at all times, to avoid fluctuations which could affect the stability of the excavations (and presumably the continuity of construction work within it).
(*b*) The method adopted should be chosen so that the excavation remains stable

Table 2.1. Methods of groundwater control (CIRIA[1])

Method	Soils suitable for treatment	Uses	Advantages	Disadvantages
Group 1: surface water control				
1. Ditches 2. Training walls 3. Embankments	All soils if used in conjunction with polythene sheeting	Open excavations	Simple methods of diverting surface water	May be an obstruction to construction traffic
Group 2: temporary groundwater control				
Internal pumping				
4. Sump pumping	Clean gravels and coarse sands	Open, shallow excavations	Simple pumping from ground.	Fines easily removed Encourages instability of formation
5. Gravity drainage	Impermeable soils	Open excavation especially on sloping sites	Simple pumping equipment	
Groundwater lowering				
6. Wellpoint systems with suction pumps (including the machine-laid horizontal system)	Sandy gravels down to fine sands (with proper control can also be used in silty sands)	Open excavation including progressive trench excavations. Horizontal drain system particularly pertinent for pipe trench excavations outside urban areas	Quick and easy to install in suitable soils. Economical for short pumping periods of a few weeks	Difficult to install in open gravels or ground containing cobbles and boulders. Pumping must be continuous and noise of pump may be a problem in a built-up area. Suction lift is limited to about 4·0–5·5 m, depending on soils. If greater lowering is needed, multi-stage installation is necessary
7. Eductor system using high-pressure water to create vacuum as well as to lift the water	Silty sands and sandy silts	Deep excavations in space so confined that multi-stage wellpointing cannot be used. More appropriate to low permeability soils	No limitation on amount of drawdown. Raking holes are possible	Initial installation is fairly costly. Risk of flooding excavation if high-pressure water main is ruptured
8. Shallow bored wells with suction pumps	Sandy gravels to silty fine sands and water bearing rocks but particularly suitable for high permeability soils	More appropriate for installations to be pumped for several months or for use in silty soils where correct filtering is important	Generally costs less to run than a comparable wellpoint installation, so if pumping is required for several months costs should be compared. Correct filtering can be controlled better than with wellpoints to prevent removal of fines from silty soils	Initial installation is fairly costly. Pumping must be continuous and noise of pump may be a problem in a built-up area. Suction is limited to about 4·0–5·5 m, depending on soils. If greater lowering is needed, multi-stage installation is necessary
9. Deep-bored filter wells, i.e. those with submersible pumps (line-shaft pumps with motor mounted at well head used in some countries)	Gravels to silty fine sand and water-bearing rocks	Deep excavations in, through or above water-bearing formations	No limitation on amount of drawdown as there is for suction pumping. A well can be constructed to draw water from several layers throughout its depth. Vacuum can be applied to assist drainage of fine soils. Wells can be sited clear of working area. No noise problem if mains electricity supply is available	High installation cost

Table 2.1 continued

Method	Soils suitable for treatment	Uses	Advantages	Disadvantages
10. Electro-osmosis	Silts, silty clays and some peats	Open excavations in appropriate soils or to speed dissipation of water during construction	In appropriate soils can be used when no other water-lowering method is applicable	Installation and running costs are usually high
11. Drainage galleries	Any water-bearing strata underlain by low permeability strata suitable for tunnelling	Removal of large quantities of water for dam abutment, cut-offs, etc.	Very large quantities of water can be drained into gallery and disposed of by conventional large-scale pumps	Very expensive, galleries may need to be concreted and grouted later
12. Collector well	Clean sands and gravel	Dewatering deep confined aquifers	Minimizes number of pumping points	Only suitable for large excavations
Group 3: exclusion methods				
Temporary methods				
13. Ammonium/brine refrigeration	All types of saturated soils and rock	Formation of ice in the voids stops water flow	Imparts temporary mechanical strength to soils. Treatment effective from working surface outwards. Better for large applications of long duration	Treatment takes time to develop. Initial installation costs are high and refrigeration plant is expensive. Requires strict site control. Some ground heave
14. Liquid nitrogen refrigeration	As for 13.	As for 13.	As for 13, but better for small applications of short duration or where quick freezing is required	Liquid nitrogen is expensive. Requires strict site control. Some ground heave
15. Compressed air	All types of saturated soils and rock	Confined chambers such as tunnels, shafts and caissons	Gives stability to sides of chamber by limiting ingress of water. Reduces pumping to a minimum	High set-up costs; possible health hazards
16. Slurry trench cut-off with bentonite or native clay	Silts, sands, gravels and cobbles	Practically unrestricted. Extensive curtain walls round open excavation	A rapidly installed, cheaper form of diaphragm wall. Can be keyed into impermeable strata such as clays or soft shales	Must be adequately supported. Cost increases greatly with depth. Costly to attempt to key into hard or irregular bedrock surfaces. Upper limit in soils of permeability 5×10^{-3} m/s
17. Impervious soil barrier	Silts, sands, gravels and cobbles	As for 16.	Relatively cheap. Local materials may be used	Must be placed some distance from excavation. Restricted depth of installation
18. Sheet piling (can be permanent)	All types of soil (except boulder beds and where natural or unnatural obstructions exist — particularly timber baulks)	Practically unrestricted	Well understood method using readily available plant. Rapid installation. Steel can be incorporated in permanent works or recovered	Difficult to drive and maintain seal in boulders. Vibration and noise of driving may not be acceptable. Capital investment in piles can be high if re-usage is restricted. Seal may not be perfect

Table 2.1 continued

Method	Soils suitable for treatment	Uses	Advantages	Disadvantages
Permanent methods — diaphragms				
19. Diaphragm walls (structural concrete)	All soil types including those containing boulders (rotary percussion drilling suitable for penetrating rocks and boulders by reverse circulation using bentonite slurry)	Deep basements. Underground car parks. Underground pumping stations. Shafts. Dry docks, etc.	Can be designed to form part of a permanent foundation. Particularly efficient for circular excavations. Can be keyed into rock. Minimum vibration and noise. Treatment is permanent. Can be used in restricted space. Can be put down very close to existing foundation	High cost may make it uneconomical unless it can be incorporated into permanent structure. There is an upper limit to the density of steel reinforcement that can be accepted
20. Secant (interlocking) and contiguous bored piles	All soil types, but penetration through boulders may be difficult and costly	As for 19. Underpasses in stiff clay soils	Can be used on small and confined sites. Can be put down very close to existing foundations. Minimum noise and vibration. Treatment is permanent	Ensuring complete contact of all piles over their full length may be difficult in practice. Joints may be sealed by grouting externally. Efficiency of reinforcing steel not as high as for 19.
Permanent methods — grouted cut-offs				
21. Thin, grouted membrane	Silts and sands	As for 16.	As for 16.	The driving and extracting of the sheet pile element used to form the membrane limits the depth achievable and the type of soil. Also as for 16.
22. Jet grouting	All types of soil and weak rocks	Practically unrestricted	As for 16.	Expensive
23. Cementitious	Fissured and jointed rocks	Filling fissures to stop water flow (filler added for major voids)	Equipment is simple and can be used in confined spaces. Treatment is permanent	Treatment needs to be extensive to be effective
24. Clay/cementitious grouts	Sands and gravels	Filling voids to exclude water. To form relatively impermeable barriers (vertical or horizontal). Suitable for conditions where long-term flexibility is desirable, e.g. cores of dams	Equipment is simple and can be used in confined spaces. Treatment is permanent. Grout is introduced by means of a sleeved grout pipe which limits its spread. Can be sealed to an irregular or hard stratum	A comparatively thick barrier is needed to ensure continuity. At least 4 m of natural cover needed (or equivalent)
25. Silicates, Joosten, Guttman and other processes	Medium and coarse sands and gravels	As for 24, but non-flexible	Comparatively high mechanical strength. High degree of control of grout spread. Simple means of injection by lances. Indefinite life. Favoured for underpinning works below water level	Comparatively high cost of chemicals. Requires at least 2 m of natural cover or equivalent. Treatment can be incomplete in silty material or in presence of silt or clay lenses

Table 2.1 continued

Method	Soils suitable for treatment	Uses	Advantages	Disadvantages
26. Resin grouts	Silty fine sands	As for 24, but only some flexibility	Can be used in conjunction with clay/cementitious grouts for treating finer strata	High cost, so usually economical only on larger civil engineering works. Requires strict site control
Permanent methods — soil strengthening				
27. Electrochemical consolidation	Soft clays	Improved shear strength of soft clay without causing settlement	See 'Uses'	Installation and running costs are usually high

Table 2.2. Depth and width restrictions for excavations that use groundwater control methods (CIRIA[1])

	Depth limits	Width limits	Other limits
Groundwater control by pumping			
1. Sump pumping	Limits of excavation: Up to 8 m below pump installation level	Increasing width increases required sump and ditch capacity	Flatter slopes may be required for unsupported excavations in silts and fine sands
2. Single system wellpoints	Maximum limit of drawdown: 3–4 m in silty fine sands 5–6 m generally	Limited by soil cone of depression (R_o)	Space required for unsupported side slopes
3. Multi-stage wellpoints	Unlimited	Limited by soil cone of depression (R_o)	Requires increasingly larger land-take for side slopes
4. Horizontal wellpointing	Limits in installation below ground level: 4 m normally 6 m maximum	As for 2a.	Segmental installation lengths usually 100 m long Space required for a machine 13 m by 3 m
5. Eductor	Unlimited but for wellpoint type drawdown usually restricted to 25 m	As for 5.	As for 5.
6. Shallow wells	Limit of drawdown: 6–8 m below pump installation level	Not usually critical, but the wider the excavation the more wells are required Limited then by soil cone of depression (R_o)	
7. Deep bored wells	Unrestricted using submersible pumps	Not usually critical, but the wider the excavation the more wells are required	Extremely large excavation may require ancillary wells within the excavation
8. Electro-osmosis	Limits of excavation: 8 m below pump installation level	Not critical	Available power supply
9. Drainage galleries	Can be installed at any depth where access is available	Unlimited	May require large working space at installation level
10. Collector well	As for 7.	As for 7.	As for 7.

Table 2.2 continued

	Depth limits	Width limits	Other limits
Groundwater control by exclusion			
11. Freezing	Unlimited (cases recorded to >900 m below ground level). Depends on depth to which receiving holes can be drilled. Liquid nitrogen required for deeper projects	Not critical, excavation base can be frozen. However, because of economics usually confined to narrow excavation	Circular construction highly desirable for stability. Long time required for installation and freezing
12. Compressed air	10 m below water level without medical lock. 35 m with medical lock	Depends on depth below ground level	Must be used in an enclosed environment, as in tunnels and shafts
13. Slurry trenching	25 m below ground level or as restricted by reach of digging plant employed	None	
14. Impervious soil barrier	Usually 5 m or less	None, since cut-off achieved	Must be placed some distance from excavation. Space is required for construction
15. Sheet piling	Recommended maximum below ground level 26 m. Have been used to >30 m, but piles may not then be recoverable	None, providing adequate penetration achieved. Wide excavation may require ancillary central dewatering	Overhead space for driving required. When used as cofferdam, ratio of width to retained height >0·8. Noise problem
16. Diaphragm wall	Installation below ground level to 40 m normal. Up to 100 m can be achieved	None, but minimum diameter of a circular cut-off about 4·5 m	Space required for a stabilizing bund if wall is not tied
17. Secant (interlocking) and contiguous bored piles	Maximum depth of installation 30 m below ground level or to hard strata		Overhead space for boring required
18. Thin grouted membrane	Limits of installation below ground level: 15 m if driven (usual) 25 m if vibrated	None	
19. Jet grouting	Cannot be used through hard rock		
20. Grouting processes	Determined by depth to which receiving hole can be drilled and presence of strata which cannot be penetrated by chosen grout. 12 m below ground level for driven lance methods (e.g. Joosten). >250 m for tube-à-manchette methods in soft deposits	Unlimited, but more efficient in confined areas rather than as curtains	
21. Electrochemical consolidation	Not critical, but preferably <8 m	Not critical	Available power supply

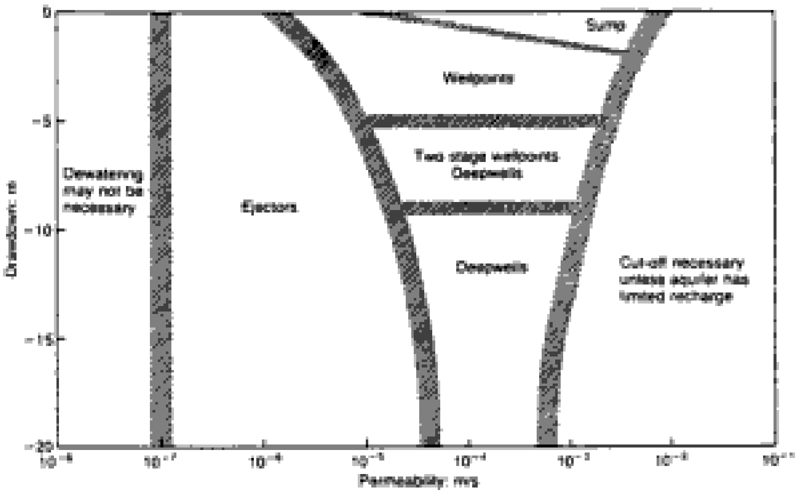


Fig. 2.3. Tentative range of application of dewatering techniques related to soil permeability and drawdown (Roberts, T.O. and Preene, M. Range of application of construction dewatering systems. Groundwater problems in urban areas. *Thomas Telford, London, 1994, 415–423)*

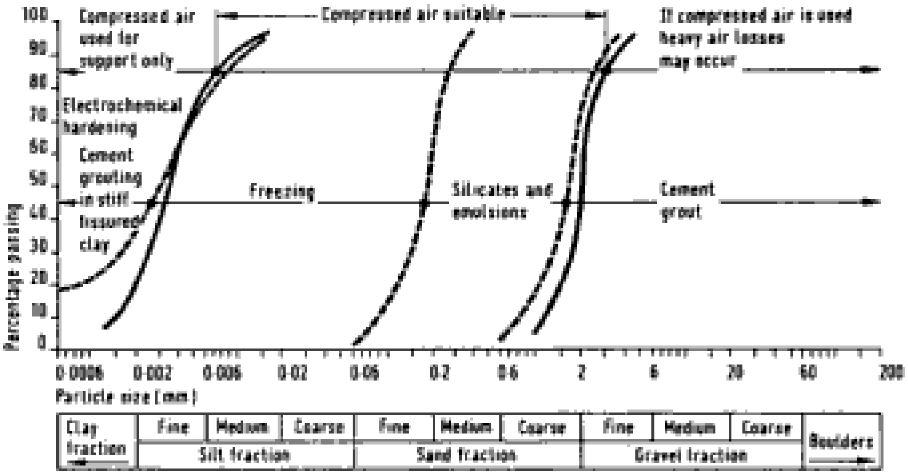


Fig. 2.4. Tentative ranges of soils for groundwater exclusion methods (CIRIA[1])

at all times, that is, slips do not occur in the sides of the excavation and excessive heaving of the base does not arise.

(*c*) When the aquifer to be drained consists of a fairly uniformly graded granular material, it can establish itself as a material filter to prevent loss of ground as a result of the pumping. (If this is not the case, filter material needs to be placed around the well screen to avoid the transport of fines, particularly in sandy silts and fine sands. The quantity of fines pumped should be checked by using a silt trap within the discharge pipework.)

(*d*) There should be an adequate margin of pumping capacity and standby power plant.

(*e*) Water removed by pumps should be discharged clear of the excavation areas, avoiding erosion, silting or contamination of existing drains or water courses.

(*f*) The pumping methods adopted for groundwater lowering should not lead to damage of existing structures (in particular, the risk of removal of fines by pumping and the long-term consolidation of soils, especially fine-grain soils, should be carefully evaluated; care in filter and well screen design will determine the extent of the transport of fines by pumping). The risk of settlement caused by removal of groundwater will be determined by the

Table 2.3. Approximate costs of groundwater control (1995 prices) (CIRIA[1])

Method	Relative cost approximation (1995 prices)
Sump pumping	Cheapest. Cost of excavating sumps and pump hire. Principal costs can be fuel consumed: for 150 mm pump, fuel costs approximately £30 per 24 h; pump hire: £120 per week
Wellpointing	Very competitive for reasonable length excavations to moderate depth over short period. Approximate costs for 100 m nominal length of wellpoint equipment with wellpoints at 1·5 m centres: mobilization, installation and demobilization — £2,500 (sum). Hire of equipment including two 150 mm pumps — £450–£500 per week. Cost of operating and maintaining system and fuel costs — £550–£600 per week. Disposable wellpoints cheaper for long-term projects
Shallow wells	More expensive than wellpoints for same depth; but competitive on confined sites with medium to high permeability
Eductors	Expensive, but can be the best engineering solution
Deep bored wells	Relatively costly depending on number installed, depth and strata. Usually only economic for large projects. Installation only costs approx. £250 per linear metre of well, with operating costs extra
Horizontal wellpoints	Expensive mobilization but can be competitive for the right job, e.g. pipelines. Installation only: approximately £10–£12 per linear metre installed plus hire of pump
Electro-osmosis	Very high energy costs
Electrochemical consolidation	Very high energy costs

Table 2.4. Relative costs of permanent methods of excluding groundwater (CIRIA[1])

Method	Relative cost
Impervious soil barrier	Inexpensive form of cut-off for shallow depths. May use local materials
Sheet piling	Cheapest form of cut-off in most soils, particularly granular, provided piles can be extracted. May be more expensive than diaphragms or contiguous piles in clay
Slurry trenching	May be competitive at moderate depth or where no space restriction exists. Costs increase quickly with increasing depth
Thin grouted membrane	Cheaper than diaphragm walls as no bentonite is wasted
Diaphragm wall	Very expensive, but may be cheaper than grouting. More cost effective if part of permanent structure
Contiguous/secant bored piles	Very expensive, but cheaper than diaphragm walls for depths to about 10 m in stiff clays only; cost reduced if part of permanent structure. Secant piles even more expensive

Table 2.5. Approximate costs of geotechnical processes for excluding groundwater (CIRIA[1])

Method	Relative cost (1995 prices)
Grouting:	(Payment is normally by quantity of materials injected)
Clay	Cheapest
Cement/fly ash	Cheap less than £20/m^3
Cement	Cheap
Cement/bentonite	£25/m^3 of stabilized soil
Bentonite gel	£30/m^3 of stabilized soil
Silicates	Joosten £25/m^3 Aluminates £40/m^3 Esters £45/m^3
Resins	£200–£300/m^3 of stabilized soil
Chemicals	Up to £300–£350/m^3 of stabilized soil
Compressed air	Very expensive. High initial set-up costs
Freezing	Extremely expensive. Usually regarded as a last resort. More expensive than electro-osmosis. Only economic at great depth

Note: installation methods and times for different grouting systems differ, therefore the above figures do not offer a strict comparison.

extent of drawdown and the consolidation characteristics of the soil, and both should be examined carefully, especially when nearby buildings are sensitive to settlement.

(*g*) Apart from economy in pumping, the water level should not be lowered further than necessary to keep the excavation clear of water at all times.

(*h*) The method applied should avoid excessive loss of ground by seepage from the sides of the excavation.

The methods available to implement these conditions are described in more detail below.

Surface drainage

It is prudent to avoid surface run-off entering the excavation by the use of a cut-off ditch or French drain (using a porous pipe with a granular surround). This applies particularly on sloping ground where the cut-off drain is placed on the side of the excavation with greatest elevation.

Gravity drainage

In relatively impermeable soils water can be conducted to sumps by open-jointed pipework surrounded by gravel with very little fall. Some advantageous drawdown will occur if the drainage trench excavations are lowered when groundwater levels are close to the main formation level.

In deep cofferdams with a clay formation, gravity drainage is frequently used to conduct groundwater to a sump sited outside the plan area of the permanent works. These temporary drains should be grouted on completion of the permanent works construction to avoid introducing a weak bearing area below the permanent works and a source of free water which could cause leakage through the permanent structure. Where possible, cofferdam widths should be increased to accommodate these drains outside the permanent works and to accommodate any tolerance required in the installation of the sheeting to the cofferdam.

Sump pumping

Sump pumping is the traditional method of removing groundwater from within a sheeted deep excavation. The sump, ideally formed within the sheeted enclosure outside the plan area of the permanent construction, is equipped with a suction head to a lift pump or a filter head to a submersible pump where the lift is more than 6 m or so below the pump. The suction or filter head can often be made with a perforated 45 gallon oil drum surrounded by a gravel filter medium. Frequently, a greater pump capacity is needed to pull the groundwater down in the sump, either prior to bulk excavation or during the course of excavation, than is required thereafter to maintain the depressed level of groundwater. It may be convenient to install the pumps in multiple units so that the pumping capacity can be reduced in the steady-state pumping condition.

Pumping from wellpoints or wells

Wellpoints and deep well systems installed prior to excavation and outside the excavation area can be used to cause groundwater to flow away from the excavation to improve stability to its side batters and base, and to allow construction works to proceed in the dry. Traditionally, wellpoints have been an easily installed system in loose to medium-dense and well-graded sandy gravels. With a drawdown limit for each wellpoint of 5 or 6 m from the level of the pump suction, either single- or multi-tier systems can be used in battered excavations. Multi-tier systems are installed progressively as drawdown is achieved at each level of wellpointing. The wellpoints consist of small diameter wellscreens approximately 50 mm in diameter, and are generally spaced at regular intervals around the perimeter of the excavation. When wellpoints are spaced relatively close to one another, the completed overlapping cones of depression of the installation are such that the depressed water level is brought below the final formation level (Fig. 2.5). The wellpoints are normally jetted in, although soils such as open gravels may lead to high-jetting water loss, and other soils such as compact sands and stiff clays and boulder clays may be difficult to penetrate by jetting, in which case pre-boring may be needed. The wellpoints, once installed, are connected to a header pipe, typically 150 mm in diameter, by swing-joint connections equipped with a gate valve to each wellpoint. The header pipe is joined to the suction side of a vacuum-assisted centrifugal pump with delivery to a convenient point of discharge. Fig. 2.6 gives indication of the soil grading in which wellpoints are economical.[1] Wellpoints are extracted after use by jetting out, although this may prove difficult if the dewatering system has been in place for a long time. Where soil contamination constitutes a serious risk of corrosion to the wellpoint screen, a disposal wellpoint with a plastic slotted tube and nylon filter fabric may prevent the screen from blocking during long-term pumping.

Typical wellpoint installations for large open excavations of increasing depth are shown in Fig. 2.7. After drawdown has been obtained, it may be possible to remove the highest tier of wellpoints for reuse at a lower level. It should be noted that multi-stage systems often impose large construction widths to accommodate the batters and the berm widths at each wellpoint tier. Heavy construction plant may damage header pipework and it is usual to install valves within the header pipework to allow speeding isolation and replacement of any damaged sections.

Wellpoints are spaced from 1 to 4 m apart, depending on soil conditions and drawdown requirements. Nomograms for spacing[1] wellpoint installations in clean, uniform sands and gravels and stratified clean sand and gravel are shown in Fig. 2.8. The spacing should be based on the most permeable of the strata, and it should be noted that the lower the permeability of the ground, the steeper the drawdown curve. Variations between horizontal and vertical soil permeability should be noted; in stratified sands the horizontal permeability (expressed in cm/s) may be up to one order less than the vertical soil permeability.

The design of wellpoint systems is considered in more detail by Powers,[3] who gives methods for estimating yield and drawdown for confined and unconfined aquifers for various well layouts.

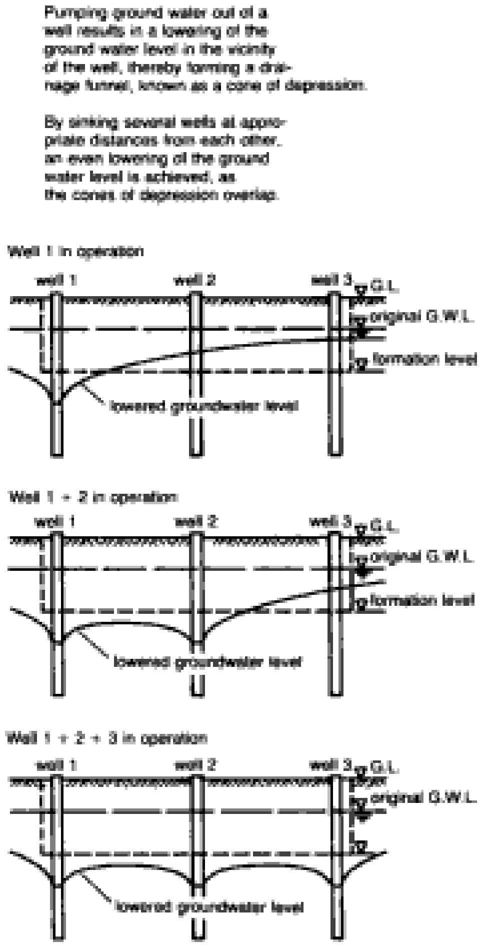


Fig. 2.5. Overlapping cones of depression in a wellpoint installation to produce a depressed water level below the formation level

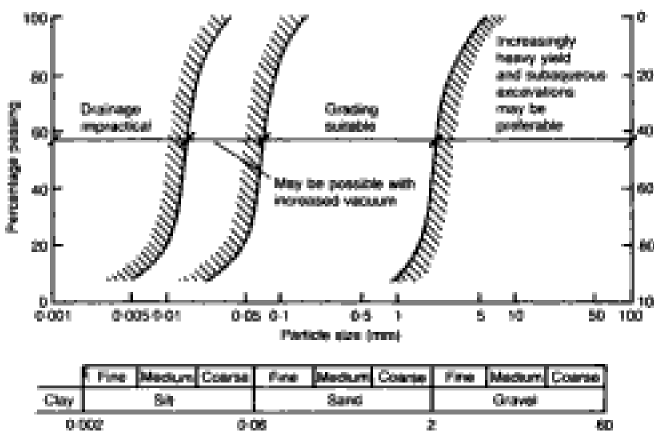


Fig. 2.6. Approximate soil grading limits for effective wellpoint dewatering (CIRIA[1])

Drawdown by wellpoints becomes progressively slower as soil grain size reduces, and becomes very slow in silts, silty sands and soils with a D_{10} grain size less than about 0·05 mm. Friction losses within pipework become critical in such soils and efficient drawdown is less likely. In these circumstances, vacuum wellpointing can be used to advantage. A bentonite clay seal is used to maintain a high vacuum

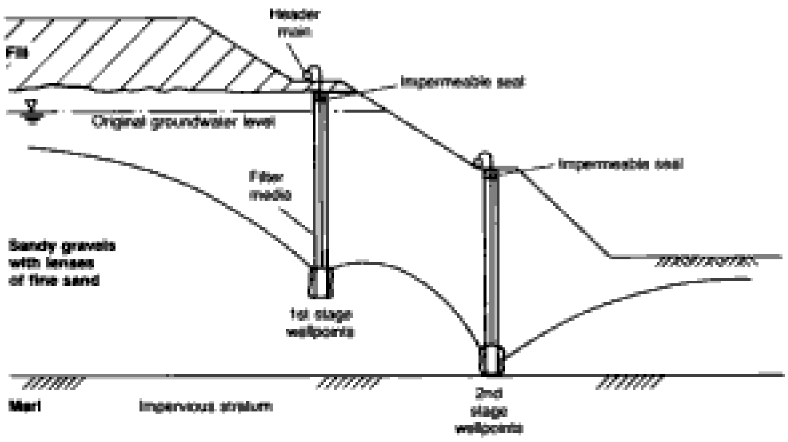


Fig. 2.7. Cross-section of a large open excavation showing typical wellpoint installation applied in two stages

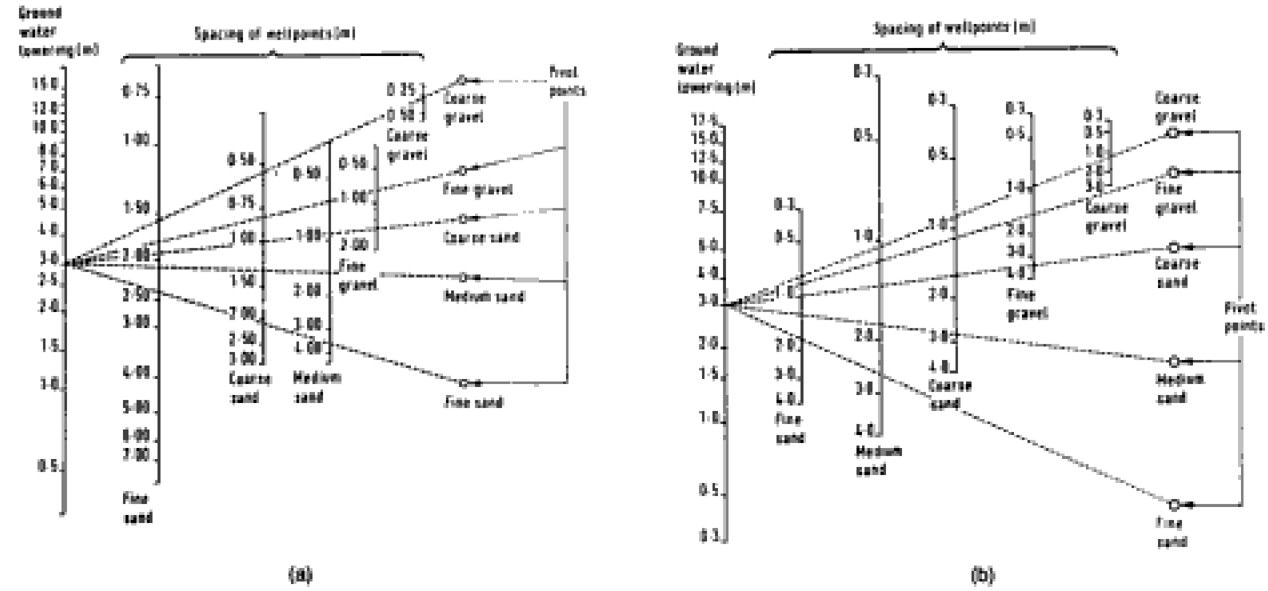


Fig. 2.8. Nomographs for wellpoint spacing: (a) uniform sands and gravels; (b) stratified sand and gravel (CIRIA[1])

at the well screen and although a conventional drawdown of groundwater is not achieved, continuous pumping and close spacing of the vacuum wellpoints prevent groundwater flow from the soil towards the excavation. A typical sealed vacuum wellpoint is shown in Fig. 2.9.

Wellpointing is often advantageous for dewatering trenchworks and can be applied to either one or both trench sides. The system can be installed progressively for long trenchworks by leapfrogging in increments of 100 to 120 m. Within this length successive operations, of wellpoint installation, operation, trench excavation, trench backfilling and wellpoint extraction, are carried out prior to moving forward to the next incremental length. Although systems installed to one trench side only allow favourable access for construction activities, the extent of drawdown over the trench width may not be sufficient other than in homogeneous permeable soils. The effective depth of trench dewatering is limited of course by the 5 to 6 m maximum depth of the wellpoint itself. Fig. 2.10 shows a typical installation for a single-sided wellpoint system.

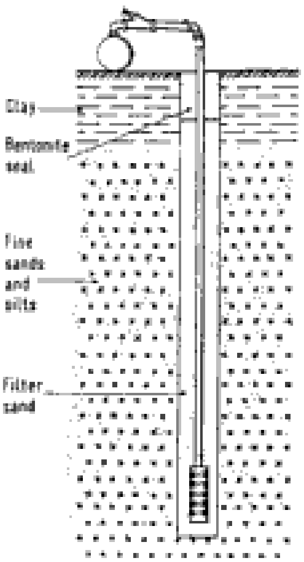


Fig. 2.9. Vertical cross-section of a sealed vacuum wellpoint

Some improvement to drawdown in a layered depth can be achieved by vertical sand drains to improve vertical permeability. The use of a granular soil surround to the wellpoint, known as 'sanding-in', achieves the same purpose. The use of a double-sided wellpoint system for trenchworks is shown in Fig. 2.11. This system provides a more effective drawdown and allows a greater trench width than the single-sided system for similar soil conditions. The improvement in drawdown reduces the risk of instability at the bottom of the trench. This also results from groundwater flow towards the wellpoints and away from the bottom of the trench.

Where fine-grained soils predominate, a vacuum can be applied to the wellpoints to improve system effectiveness, although the time taken to achieve full drawdown may prove to be longer than desired. Some reduction in efficiency may also occur with time as air is drawn into the system through ineffective seals and joints. The effective depth of drawdown may be increased in all soils by a multi-stage system, as shown previously in Fig. 2.7.

The use of a dewatering system outside a sheeted excavation such as trenchworks reduces the groundwater pressure from the sheeting and trench bracing, but means a drawdown of the water table from the area outside the excavation which is limited in extent only by the permeability of the soil itself and the volume of the aquifer. Similarly, the yield of groundwater to achieve this drawdown and the pump capacity needed is controlled by the permeability of the subsoil and the aquifer volume. Where the drawdown curve is flat, in relatively permeable strata, the removal of groundwater, sometimes unintended, from beneath existing structures and services a considerable distance from the excavation may cause settlement and subsidence damage.

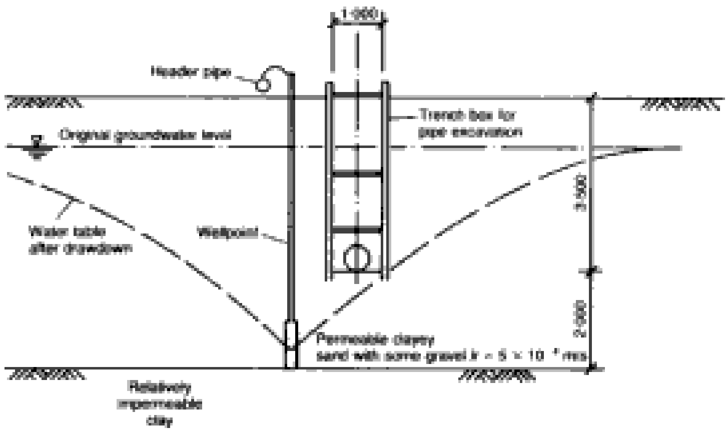


Fig. 2.10. Typical installation of a single-sided wellpoint system

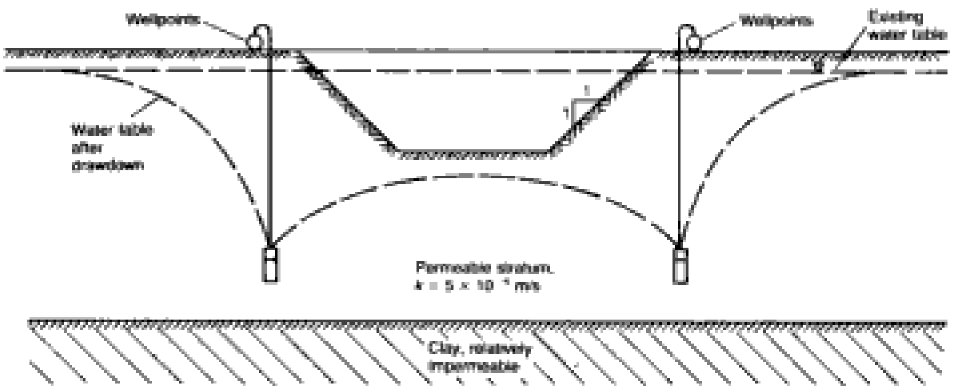


Fig. 2.11. Typical installation of a double-sided wellpoint system

In addition to vertical drainage, horizontal drains and wellpoints are also used, particularly for trenchworks. In layered soils the difference between horizontal and vertical permeability of the soil fabric may be as large as one order when permeability is measured in cm/s. It is therefore illogical to install dewatering means that rely on the vertical flow of groundwater to the wellpoint or well screen if the means are available to lay a horizontal drain at the required depth to achieve drawdown. Trenches for drains are dug by backhoe or, alternatively, specially-built machines for laying a perforated pipe within a gravel surround may be used. The maximum depth for the operation of these excavator/pipelayer machines is only of the order of 6 m, but they find application for shallow trenchworks in homogeneous sandy soils and have attractive installation outputs of up to 1000 lineal metres of drain per day.

Ejector systems, also known as eductor systems, may provide an alternative to both conventional and vacuum wellpointing, although their adoption probably requires a more reliable and detailed knowledge of subsoil and groundwater conditions. Overall, the plant and installation costs are likely to be less than those for deep wells, and although individual ejector efficiency is low, the operational depths are not limited in the same way as wellpointing and therefore multi-tier systems become unnecessary.

Both twin and single pipe ejectors are used; typical pipe layouts are shown in Fig. 2.12. In the twin-pipe ejector, high-pressure water through the supply pipe and the body of the ejector is fed through the tapered nozzle. A partial vacuum

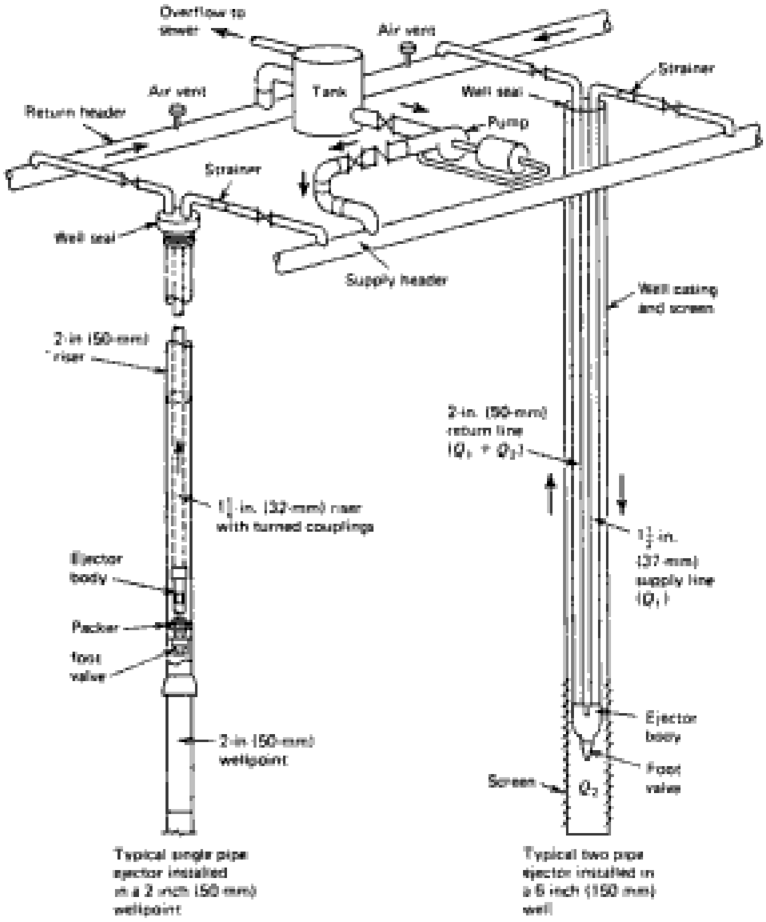


Fig. 2.12. Single- and twin-pipe wellpoint ejector systems (Powers[3])

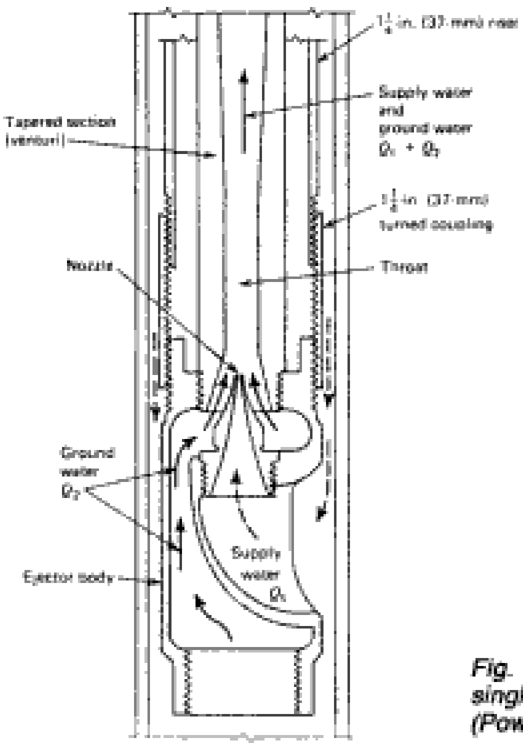


Fig. 2.13. Cross-section of single pipe ejector (Powers[3])

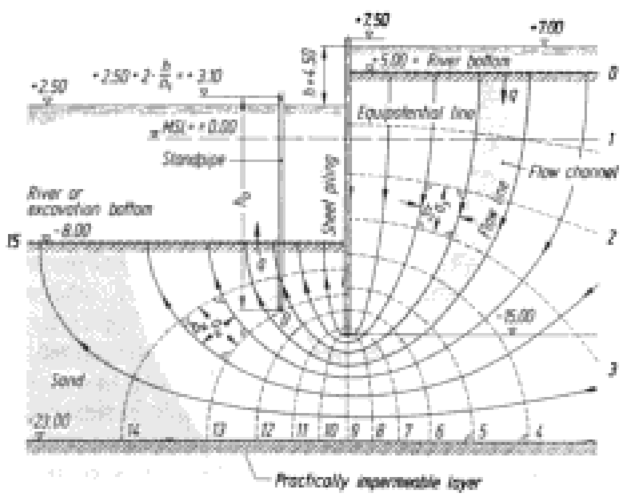
is caused in the suction chamber, causing groundwater to be sucked through the foot valve. Incoming supplies from the downward supply pipe and through the foot valve mix in the suction chamber and pass into the venturi where the velocity decreases and the pressure increase is sufficient to send the combined supplies through the second pipe to ground level. This twin-pipe system is usually installed in a casing and well screen, unlike the single-pipe system, which is shown in section detail in Fig. 2.13. The single-pipe ejector can be installed within a 50 mm dia. wellpoint and, like the twin-pipe model, is self-priming and will evacuate air from its own well screen. The single-pipe system relies on downward flow of water (this time in the annular space between the casing and the inner return pipe) to a suction chamber and then, with groundwater sucked in, the water passes into a venturi chamber where increased pressure causes the supply and the groundwater to flow upward to ground level for disposal.

Ejectors are best suited for removal of groundwater from finer-grained, low permeability soils and are used in deep excavations and shafts beyond the depths economically operated by multi-tier wellpointing. Their operation depends on submergence below groundwater level, otherwise cavitation occurs. Deep well systems are likely to be used in preference to ejectors unless this submergence condition is reasonably certain to be maintained. Case studies of the use of ejectors and vacuum wellpoints in fine-grained Eocene soils in the UK were described by Preene,[4] and methods of estimating steady-state flow rate discussed.

The most convenient subsoil conditions for excavation through inundated soils are where an impermeable stratum can be economically reached by sheeting driven, or a cut-off installed, from an upper level and penetrating below excavation level. Some penetration of the impermeable stratum will be needed to form a seal, and this may be achieved with difficulty in hard soils or rock strata. Grout injection may improve this seal. In these circumstances, groundwater flow below the sheeting or cut-off will be small, and providing there are no serious openings or 'windows' within the sheeting or cut-off below excavation level, there will be no risk of base failure and heave of the completed excavation at formation level. It is only necessary to pump out the volume of groundwater within the sheeting near formation level and to maintain the dewatered level. Where such a cut-off does not occur at convenient depth, a seal can be obtained with a horizontal cut-off by jet grouting or by intrusion grouting where soil permeability makes this feasible. In these circumstances, again, only limited dewatering is needed within the sheeting to keep the formation dry, providing the sheeting or cut-off is effective below formation level and leakage through the grout plug is small.

Where an impermeable stratum does not occur at convenient depth or the use of a horizontal grout curtain is not feasible, the flow of water through permeable strata occurs downwards, under the sheeting, and then upwards to formation level. As the penetration depth of the sheeting is increased, the flow path is increased and the quantity of exit water and its exit velocity are reduced proportionally. This ingress of groundwater upwards to formation level may have serious effects because of the induced seepage forces on the passive wedge of soil below excavation level supporting the external sheeting. Marsland[5] showed the relevance of this seepage force in model tests and, later, Soubra and Kestner[6] considered the effects of this seepage force on alternative passive failure mechanisms. The risk of instability below formation level within a sheeted excavation is described later in this chapter. In finer-grain soils where the differential head of groundwater between the outside and inside of the sheeting to the excavation is high, soil liquefaction can occur when the vertical effective stress within the soil reaches zero, a condition caused by seepage forces. This condition can be examined by considering the critical exit velocity with the use of a flow net, as shown in Fig. 2.14, and will give an early indication of risk. Fig. 2.15 shows the reduction in factor of safety against piping as the width of a trench in fine sand is progressively reduced. The possibility of base instability within excavations in soft clays should be examined, as described in chapter 7 on cofferdam design.

Where the ingress of groundwater below sheeting is such that seepage forces are excessive or the quantity of water is too great to remove by horizontal drains and sump pumping, the use of deep wells near the toe of the sheeting becomes

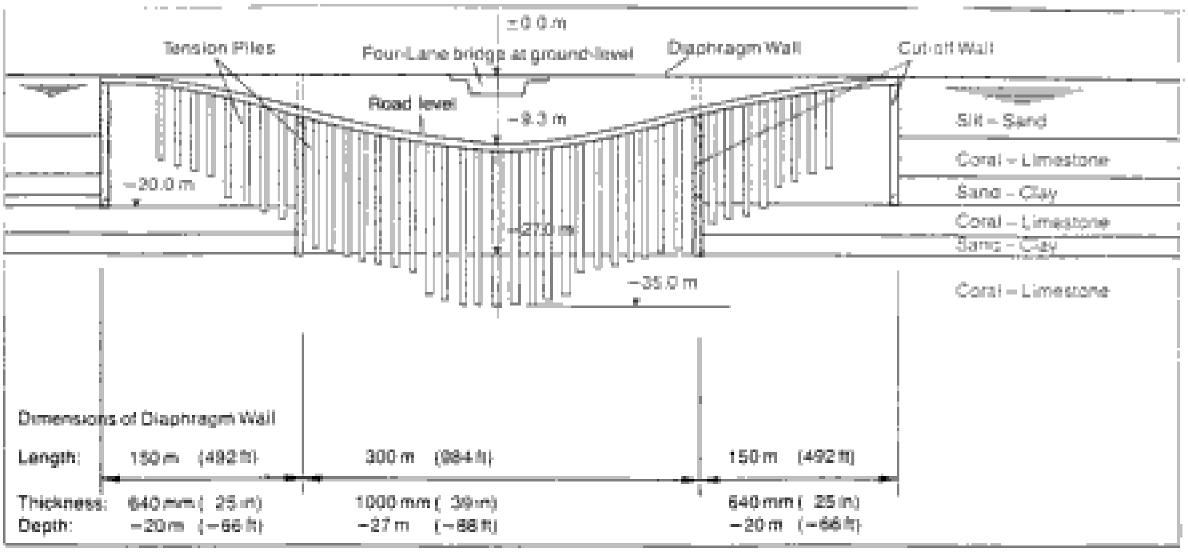


Exit gradient $= \dfrac{15}{14 \times 1.5} = 0.71$

Factor of safety against piping $= 1/0.71 = 1.4$

Fig. 2.14. Effect of seepage pressure on base stability within an excavation in sand: calculation of exit gradient and factor of safety against piping

The subsoil conditions are also shown in Fig. 2.18; 1 m thick loose sand and gravel fill overlaid the reclamation material consisting of very loose sand and a medium dense to dense clayey sand to a depth of 7 m. Below this reclamation a layer of soft coral limestone interbedded with dense coralline sand 7 m thick overlaid dense clayey sand. The existing groundwater level was generally 1 m deep over the whole of a fairly flat site. The underpass walls were designed to be built in structural diaphragm walls and the whole structure was anchored by tension piles. The structural walls, spanned by a reinforced concrete floor slab, resembled a floating structure anchored by the piles. The structural walls did not achieve either a temporary or permanent seal into impermeable soil or rock at depth. Cross diaphragms in self-hardening slurry works subdivided the excavation into three compartments, but none of these walls achieved a cut-off with depth. The three compartments were successively excavated and dewatered by a total of 29 deep



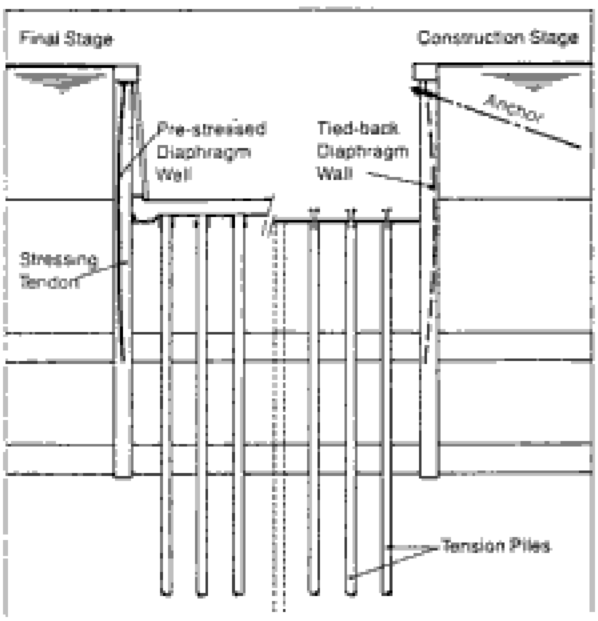
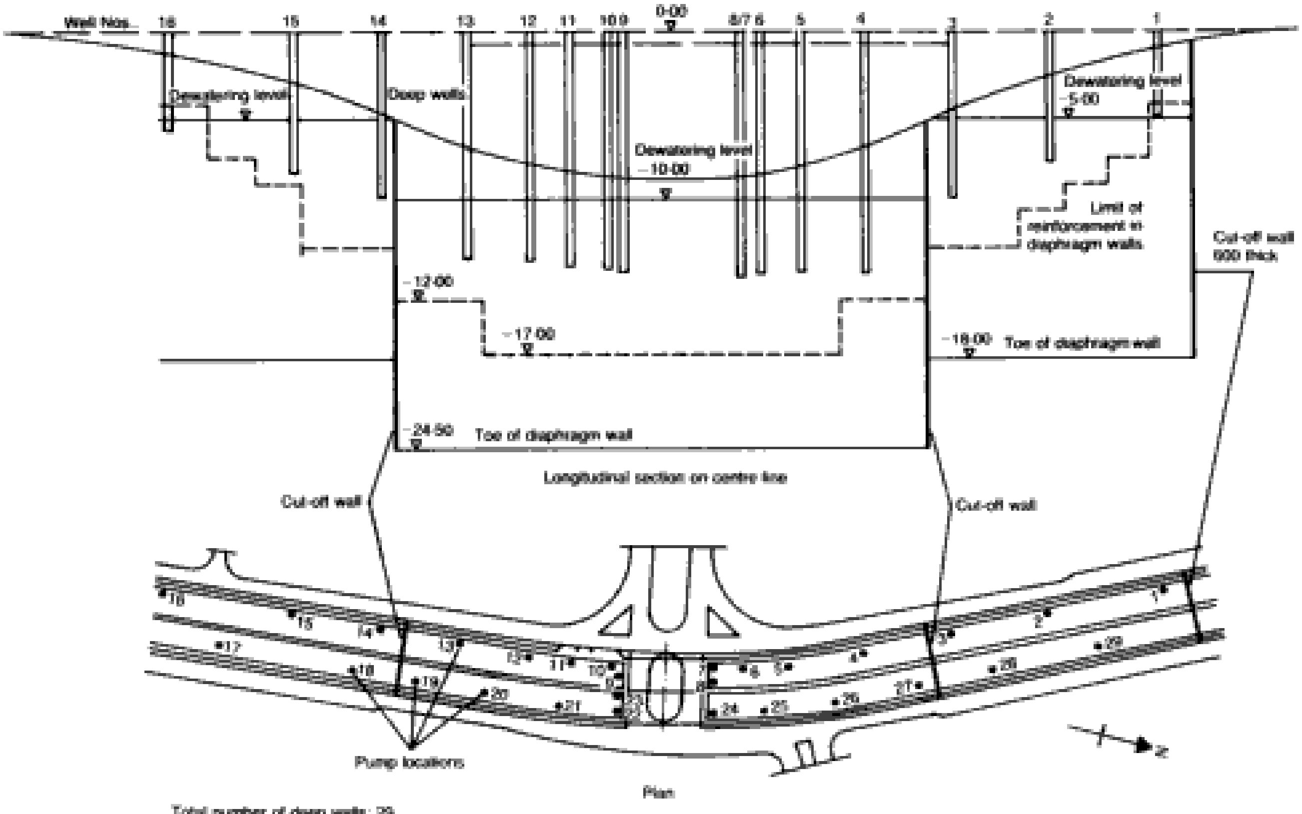


Fig. 2.18. Diwan Underpass, Jeddah, Saudi Arabia. Longitudinal and transverse section.

wells. The specification for the wells and the pumps is shown in Fig. 2.19. The cost of the four cross diaphragm slurry walls was significantly less than savings made from reducing the programme time; early commencement of excavation within the first compartment was possible prior to completion of the structural diaphragm walls further along the underpass. Compartment working also reduced drawdown of groundwater at the shallower excavation levels at each end of the underpass. The dewatering scheme was particularly successful, bearing in mind the limited opportunity for pumping prior to bulk excavation within a very short construction programme worked on a 24 hour, seven day basis. At the deepest section of the underpass, the deep wells reduced groundwater to a metre or so below formation level with two to three weeks of pumping prior to excavation.

In such conditions, the penetration depth of the external diaphragm walls is selected to meet the most severe of four design criteria: first, the minimum depth to avoid piping of loose sand below formation level; second, the minimum depth to give sufficient passive resistance to support the external walls prior to casting the floor slab to the underpass; third, the optimum depth to extend the drainage path to formation level for external groundwater; last, to site the well heads within the walls at the most economical depth. The optimum depth for the well heads



Total number of deep wells: 29
Well numbers: 1–6, 12–21, 25–29
Type of pumps: KSB 200–14/2, 5·9 kW, 380 V, 50 Hz, 13 A
Capacity of pumps: 15–20 l/s for 20 m head
Riser pipe: 100 mm
Well numbers: 7–11, 22–24
Type of pumps: Guinard 58", 11 kW, 380 V, 50 Hz, 24 A
Capacity of pumps: 20–25 l/s for 35 m head
Riser pipe: 100 mm

Fig. 2.19. Location of cut-offs and deep wells for the Diwan underpass, Jeddah, Saudi Arabia (courtesy of Bauer)

minimizes the number of wells required to achieve drawdown, at the same time reducing the quantities to be pumped in terms of available pump capacity and energy used. Wall penetration and the design of the dewatering scheme in sheeted deep excavations is referred to later in this chapter.

Use of relief wells

The use of vertical relief wells each consisting of a granular filter backfill placed within and surrounding a slotted well screen can be effective, without pumping, for transferring perched water tables to lower, partly-filled aquifers below deep excavations. Ward[8] also referred to the use of this simple procedure to relieve the artesian pressure which had caused uplift of the formation and damage to pipework in a trench excavation penetrating laminated sand and silty clay with entrapped artesian groundwater.

Groundwater control by cut-offs

Relatively impermeable walls or cut-offs can be effective in temporarily excluding groundwater from deep excavations, reducing the need for removal of groundwater by pumping. The use of cut-offs can therefore materially improve construction progress by dividing a basement or cut-and-cover excavation into compartments. This allows excavation to proceed below the original water table in construction areas where curtilage soil-retaining walls have been completed and achieving a cut-off into an impermeable stratum below the aquifer prior to the completion of the whole perimeter wall. Cut-offs can also be effective in restricting the flow of groundwater from aquifers exposed by battered, open excavations.

The cut-off may possess both flexural strength and relative impermeability, such as walls built from sheet piling, reinforced concrete piling and diaphragm walling, or the cut-off may have low permeability, relatively high flexibility and low flexural strength. Cut-offs in the latter category include unreinforced diaphragm walls in plastic concrete, thin diaphragm cut-offs made by grout injection, and unreinforced bentonite slurry and cement—bentonite slurry walls and membranes. These low strength cut-offs rely for their support above formation level on soil berms between the cut-off and the excavation, and find economic application in temporary works where only the property of relative impermeability is needed. Specialist methods such as installation of freeze soil barriers using ammonia/brine or liquid nitrogen find application in particular locations in fine-grained saturated soils which justify relatively high installation and running costs. Table 2.6 summarizes cut-off methods.[7]

The effectiveness of cut-offs constructed from grout curtains and slurry trenches was discussed by Ambraseys[9] and later by Brauns[10] and others. The effectiveness of cut-off walls using jointed methods such as sheet piling, concrete piling and concrete diaphragm walls was reviewed by Telling *et al.*,[11] referring to the use of the cut-off below dam structures. Analytical appraisal of cut-off efficiency, while important for permanent cut-offs below dams, is not of prime importance for temporary works in deep excavations, although the practical aspects of avoiding local windows by bad workmanship causing leakages in temporary cut-offs may prove to be vital.

Cut-off construction using slurry wall techniques

The development of slurry cut-off walls began in the mid-1940s when the US Corps of Engineers used the permanent cut-off construction below levees on the Mississippi river. Wide cut-offs excavated by dragline were put down under bentonite slurry, and the excavated soils were later backfilled by dozer. The slurry

Table 2.6. Summary of cut-off methods (Cashman[7])

Method	Soils suitable for treatment	Uses
1. Sheet piling	All types of soil (except boulder beds and rock)	Practically unrestricted
2. Diaphragm walls (structural concrete)	All soil types including those containing boulders (rotary percussion drilling suitable for penetrating rocks and boulders by reverse circulation using bentonite slurry)	Deep basements Underground car parks Underground pumping stations Shafts Dry docks
3. Slurry trench cut-off	Silts, sands, gravels and cobbles	Practically unrestricted Extensive curtain walls round open excavations
4. Thin grouted membrane	Silts and sands	As for 3.
5. Contiguous bored pile walls	All soil types but penetration through boulders may be difficult and costly	As for 2.
6. Cement grouts	Fissured and jointed rocks	Filling fissures to stop water flow (filler added for major voids)
Grouted cut-offs		
7. Clay/cement grouts	Sands and gravels	Filling voids to exclude water To form relatively impermeable barriers — vertical or horizontal Suitable for conditions where long-term flexibility is desirable, e.g. cores of dams
8. Silicates: Joosten, Guttman and other processes	Medium and coarse sand and gravels	As for 7. but non-flexible
9. Resin grouts	Silty fine sands	As for 7. but only some flexibility
Freezing		
10. Ammonium/brine refrigeration	All types of saturated soils and rocks	Formation of ice in the voids stops water flow
11. Liquid nitrogen refrigerant	As for 10.	As for 10.

stabilizes the trench during excavation and is left in place, with the backfilled soil used to form the cut-off. A filter cake slurry deposit forms on the trench sides during excavation and acts compositely with the backfilled soil and slurry to resist the flow of water across the trench. Many cut-offs of this type were subsequently made in the USA to depths of 30 m or more and widths varying up to 3 m for temporary and permanent control of seepage into excavations, as foundation and embankment cut-offs for water-retaining structures, and to prevent seepage of various pollutants from contaminated groundwater. D'Appolonia[12] described construction methods by dragline and backhoe and provided values of wall permeability as a function of the bentonite content of completed backfill and the permeability of soil–bentonite backfill related to fines content. Overall, however, the application of such wide, deep soil–bentonite slurry cut-offs has been replaced in deep excavation works by bentonite–cement slurry membranes which are installed by backhoe to relatively shallow depths up to 10 m, to greater depths using conventional diaphragm wall equipment, or a combination of both. These cut-offs are narrower than the earlier soil–bentonite walls and vary between 600 mm and 1 m. These bentonite–cement slurry cut-offs may also be backfilled with excavated soil, as described by Hetherington *et al.*,[13] where impermeability requirements are less rigorous and a cheaper membrane material is needed. This innovation was first used in 1975 at Alton Water Dam, Ipswich, UK, for the construction of a permanent seepage cut-off to an impounded area. Initially, the 600 mm wide trench was excavated by backhoe and then by conventional diaphragm wall equipment, a hydraulic grab mounted on kelly bar equipment. Conventional, temporary tubular stop ends were used at the end of each working day, but guide trench construction was limited to the use of timber baulks. An initial site trial used six panels with varying cement contents for the slurry. With a constant bentonite content of 4% by weight, the cement was varied in increments from a minimum of 70 kg/m^3 to a maximum of 150 kg/m^3: permeability test results varied from 1×10^{-8} m/s to 1×10^{-9} m/s at the lower and higher cement contents, respectively. The water bleed did not appear to vary with cement content; a final cement content of 125 kg/m^3 was used.

The properties of bentonite–cement slurry cut-off walls, known as self-hardening slurry walls, were described by Caron.[14] He examined the characteristics of the slurry in its two separate phases, initially as a means of trench support and subsequently as the hardened trench fill mixture. During the excavation phase the initial properties of the self-hardening slurry are similar to those of a bentonite slurry, but over time the cement content causes stiffening and as excavation proceeds the bentonite–cement slurry progressively combines with soil particles. Later, water bleed from the slurry and filtration through the sides of the trench cause further stiffening. The change in slurry stiffness and the increase in viscosity with time in the initial excavation phase were described by Caron for a range of cement types, retarder additives and soil addition to the slurry. Hardened slurry properties were also examined, but as a first approximation Caron concluded that slurry strength was governed by the cement/water ratio, viscosity by the bentonite/water ratio, and the setting time by the retarder/water ratio. More detailed analysis showed the relationships were a little more complicated, as shown in Table 2.7.

The principal characteristics to be addressed by the designer are cut-off strength, minimum strain at failure and permeability. For a typical self-hardening slurry for groundwater cut-off, these properties would be specified as follows:

- unconfined compressive strength: at 28 days, samples from panels, 90% of all results in the range 100–500 kN/m^2
- permeability: target value less than 1×10^{-9} m/s; test results from control samples from panel: all samples less than 1×10^{-8} m/s and at least 50% of all samples less than 1×10^{-9} m/s

Table 2.7. Hardened slurry properties

Characteristic	Principal factor	Secondary factors
Strength	Cement/water ratio	Bentonite/water, retarder/water ratios
Viscosity	Bentonite/water ratio	Cement/water, retarder/water ratios
Settling time	Retarder/water ratio	Cement/water, bentonite/water ratios

- consolidated drained triaxial testing: all results of samples tested from panels to demonstrate strain at failure in excess of 5%.

On a cut-off contract (Kielder Water Dam, Icos, 1976) the mix proportions to achieve a similar specification were: bentonite, 54 kg/m^3 of slurry; water, 942 kg/m^3 of slurry; Portland cement, 28·8 kg/m^3 of slurry; ground blast furnace slag, 67·2 kg/m^3 of slurry; retarder additive, nil. The parameters specified were:

- permeability: 500 h after placing under a hydraulic gradient of 450, the sample first saturated under a back pressure of 150 kN/m^2: 10^{-8} m/s
- deformation: minimum deformation under a deviator stress of 125 kN/m^2 at 2% strain per hour with cell pressure of 500 kN/m^2, 90 days after mixing: 5%.

The cut-off at Kielder was 19 m deep and 600 mm wide.

Overall, it is rare that the minimum compressive strength of the slurry proves difficult to achieve with economical cement contents. Caron proposed an empirical relationship (Fig. 2.20) using test results with bentonite from various sources, and concluded that a 28 day compressive strength $R = 10^4 \times$ (cement/water ratio)^2 kPa was dependable for cement/water ratios between 0·1 and 0·7. Caron commented that deviations from this relationship could be attributed to the nature of the bentonite, but considered that this contradicted the more reasoned assumption that the cement should have a more important influence on slurry strength than the colloidal agent that keeps it in suspension. Tornaghi,[15] in a review of self-hardening slurries using Italian cements, commented that the value of the multiplier to the square of the cement/water ratio could vary considerably, depending on the nature of the cement, from a value of 10^3 to more than 10^4 kPa for cement/water ratios between 0·15 and 1.

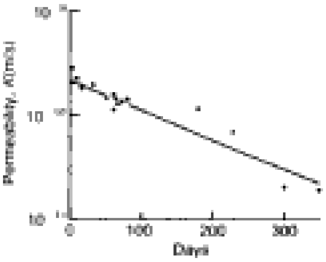


Fig. 2.20. Development over time of the permeability of a bentonite–cement slurry (Caron[14])

The specified values of minimum deformation at failure, commonly of the order of 5% axial strain, imply a plastic failure and preclude brittleness from the failure mechanism, presumably to accommodate lateral ground movement during the performance of the cut-off. In practice, this value is not difficult to achieve with normal bentonite and cement contents in drained tests, although as Tornaghi reminded us, the rate of strain in drained tests on cement–bentonite mixes is critical in its effect on test values of axial strain at failure.

The permeability of the cut-off material is generally of the order of 10^{-8} m/s after one month, according to Tornaghi, a somewhat higher figure than that shown by Caron (Fig. 2.20).

The effects of mixing methods on the properties of slurries were reported by Jefferis.[16] He concluded that poorly-mixed slurries never develop cut-off properties that are as good as efficiently-mixed slurries and, for cement–bentonite slurries in particular, good mixing reduces bleeding of the fluid material and the permeability of the set material, although some increase in strength and brittleness also occurs.

The design of cement–bentonite slurry mixes can be varied (as at Kielder) by the inclusion of cement-replacement materials such as pulverized fuel ash and ground blast furnace slag for economy. The use of ground clays other than bentonite can also reduce material cost. Apart from material costs, the largest cost variables are the provision and type of guide walls and the means of slurry trench excavation. The risk of 'windows' occurring within cut-offs excavated by backhoe should be noted as it can affect overall cut-off performance detrimentally.

Tornaghi described two examples of the use of cement–bentonite cut-offs to exclude groundwater in deep excavations. The first, installed by Rodio in 1970, was used to cut-off groundwater flow through fill, filty sands and sands and gravels in a deep excavation at a nuclear power complex at Caorso, near Milan. A plan and cross-section through the excavation is shown in Fig. 2.21. The cut-off is

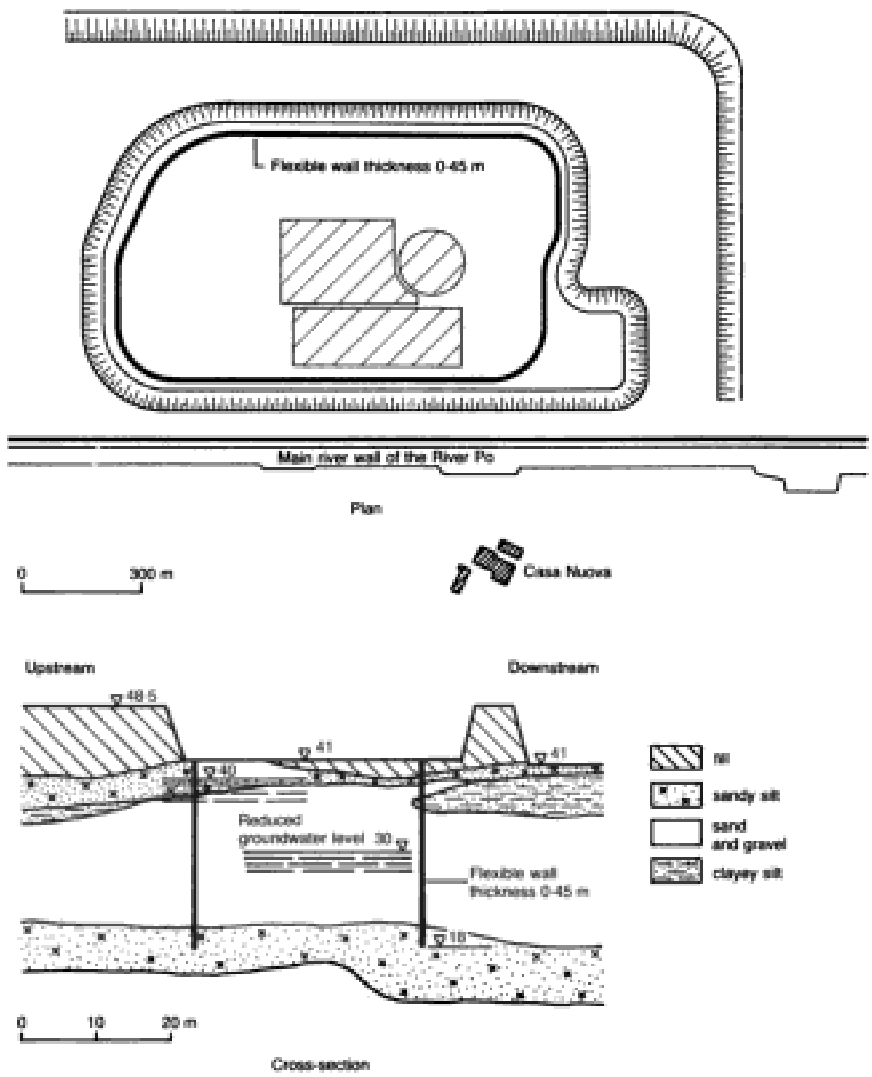


Fig. 2.21. Plan and cross-section of cut-off construction for deep excavation at Caorso, Italy (Tornaghi[15])

450 mm thick and has a mean depth of 23·5 m. The second example, installed in 1982 for a deep excavation at the nuclear power complex at Montalto di Castro, near Rome, is shown in plan and cross-section in Fig. 2.22. The cut-off, 800 mm wide, was taken through silty sands containing silty clay layers and through a conglomerate rock to obtain a cut-off into clay. The total depth of the wall was 34 m and the depth of cut-off into the clay was 2 m. The required drawdown of the groundwater was approximately 20 m which was maintained by ten deep wells shown in plan position in Fig. 2.22.

Cut-off construction using grout curtains installed by intrusion grouting or jet grouting

Where peripheral walls to a deep excavation are required to resist soil pressures and provide a barrier to the ingress of groundwater, structural walls are necessary.

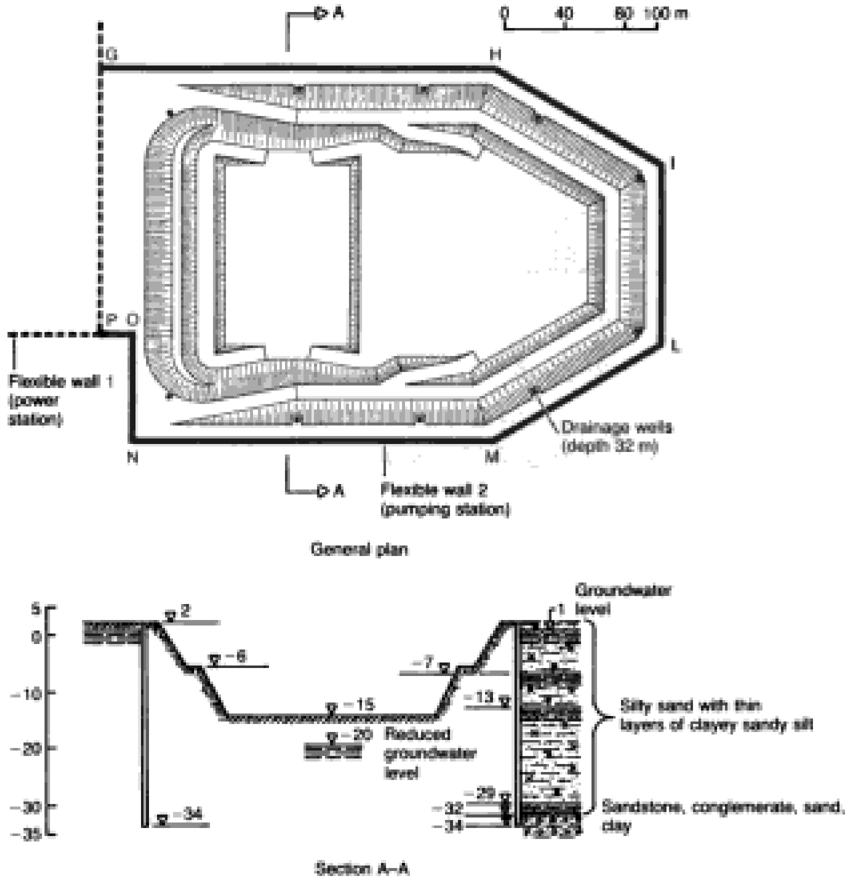


Fig. 2.22. Plan and cross-section of deep excavation with cut-off and ten deep wells at Montalto di Castro, Italy (Tornaghi[15])

Where such walls need to be extended in depth to prevent ingress of groundwater or where the waterproofing of structural wall elements requires improvement, it may be economical to use grout injections or jet grouting techniques.

The process of grout injection into soils is used both to strengthen the soil mass and to reduce its overall permeability. The acceptance of grout by the soil structure and the travel of the grout within the soil mass is governed by the permeability of the soil fabric. Where soil permeability is insufficient for grout take and grout pressure is adequate, the grout enters the soil mass as a grout body and does not permeate between soil grains. Such an effect, known as claquage or hydrofracture, is the basis of compaction and squeeze grouting. In contrast, jet grouting is used to destroy the soil structure deliberately and replace it with a soil/grout mixture. Jet grouting can be used in most soil types but is particularly effective in sands and cohesionless silts. Although very high jet pressures are used, hydrofracture pressures are not likely to be reached in jet grouting, the grouted void being formed by the erosive action of the jets and the high pore-water pressures set up at shallow depth in the soil near the jet.

The jet grouting process deserves more explanation. The process was originally introduced in Pakistan by the UK contractor Cementation, but was later developed by Japanese firms. In the early 1980s the process was imported to Europe from Japan.

The two systems most frequently used are mix-in-place and replacement grouting, as shown in Fig. 2.23. Both systems use water or air/water jets with diameters

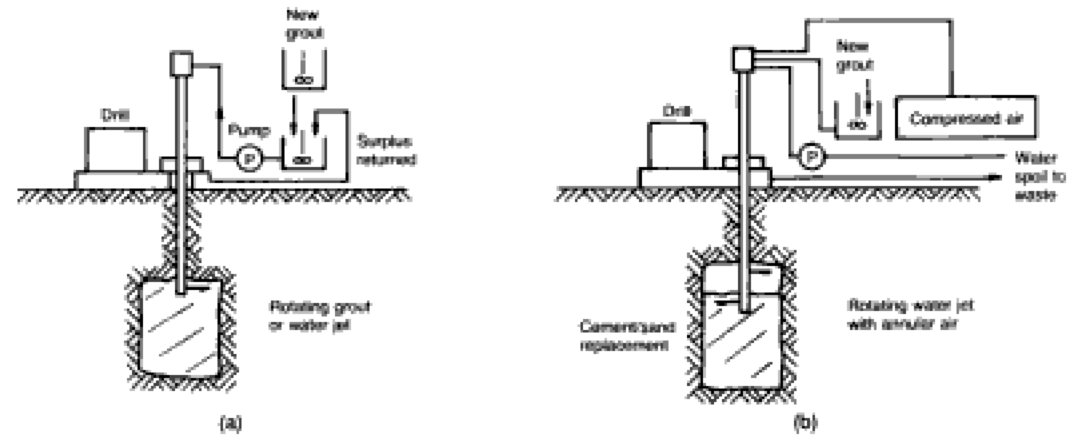


Fig. 2.23. Jet grouting systems: (a) mix in place; (b) replacement method

in the range 2 to 3 mm. The first is the simpler method in which a single grout jet at a pressure of 300 to 500 bar is rotated during extraction from a pre-drilled hole. Rotation speed is 10 to 20 rpm and the rate of extraction has to be low (about 100 to 150 mm/min). Since grout loss has to be low, mix-in-place is carried out in a tight hole. Eroded soil brought to the surface is recirculated with additional grout. Columns between 400 and 800 mm in diameter are formed in sands, more in coarse sands and less in cohesive soils.

The second method, replacement grouting (known as the Kajima–Keller process), uses a triple tube drill stem to grout from an open hole. The tube delivers a jet of water at very high pressure (400 to 500 bar) which breaks up the soil at the periphery of the bore. The cuttings are returned to the ground surface by this water by direct circulation. To improve the cutting action, a shroud of air (at low pressure, 5 to 7 bar) can be used to wrap the water cutting jet. This air jet improves soil cutting performance and the efficiency of soil cuttings returning to the ground surface. Grout is then pumped down the third tube at a pressure of 5 to 7 bar to enter below the eroding jet. The system, cutting and grouting simultaneously, is rotated and lifted simultaneously at about 5 rpm and 50 mm/min, respectively. Column and panel construction by the triple jet process is shown in Fig. 2.24.

The grout holes can be located within 300 mm of existing foundations so that the jet can undercut them by as much again and bores can be inclined below existing foundations. The process has been used to depths in excess of 40 m. A wide range of soils above and below groundwater level can be treated, from gravels to clays, although stones tend to remain at the base of the column in open gravels.

Typical dimensions and properties of the soil/cement grout mass are shown in Table 2.8.

Quality control may be applied on site by pre-contract trials of column construction. Site measurements are made of lift speed and rotation, depth, pressures and flow rates of grout, water and air. Grout mix quality tests and core sampling are used where appropriate, and the specific gravity of the grout waste slurry is measured. Both cement and cement/pulverized fuel ash grouts may be used, the latter weaker but costing less than the cement grout alternative.

Two examples are presented to show the use of jet grouting to improve grout water exclusion from deep excavations. Fig. 2.25 shows the use of a jet-grouted cut-off to allow a new underground railway tunnel construction in Rotterdam to pass under the existing Blaak station built on a wall in H-section steel profiles (the 'D' wall) which terminated in the middle of the Pleistocene bed of fine sands at level −24. To allow excavation under the station without considerably reducing the water level, the H pile wall was extended downwards by a cut-off in secant

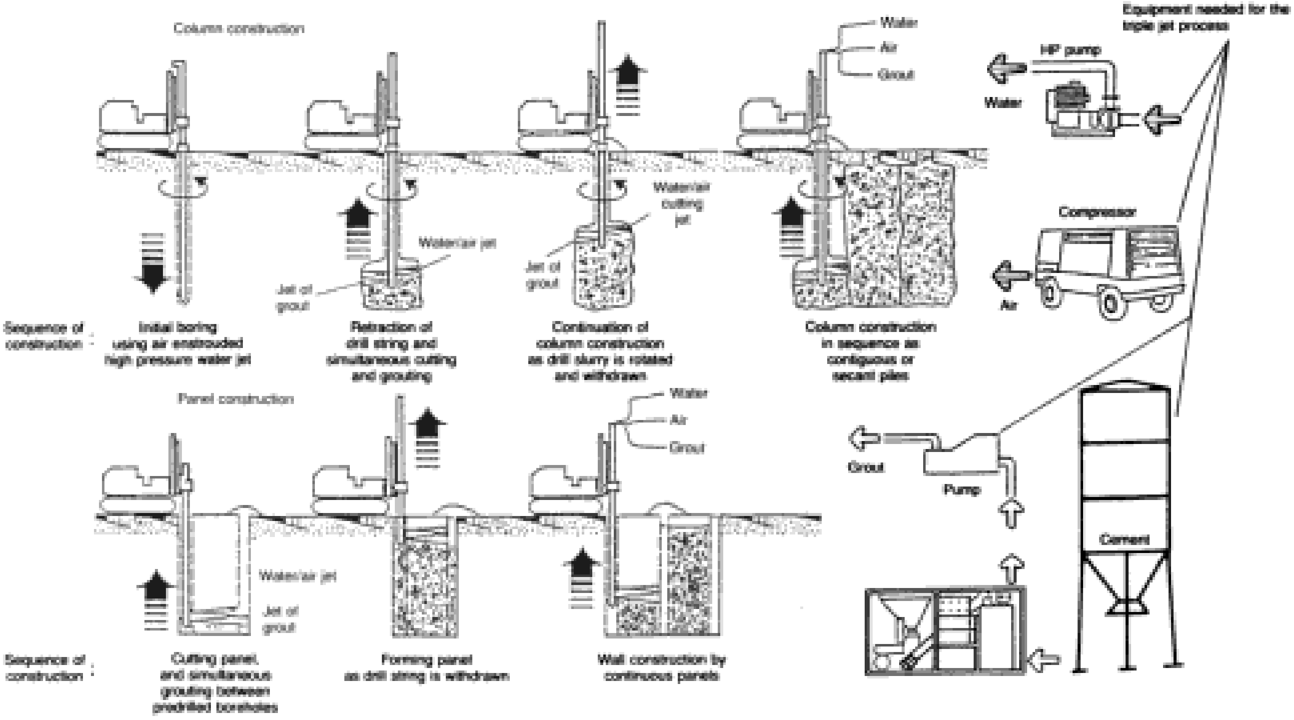


Fig. 2.24. Jet grouting: column and panel construction by the triple-jet process (courtesy of Bachy)

Table 2.8. Typical dimensions and properties of the soil/cement grout mass

	Granular soils	Cohesive soils
Diameter (m)	0·8 to 1·8	0·5 to 1·5
Unconfined compressive strength (N/mm^2)	1 to 10	0·5 to 5
Shear strength (kN/m^2)	500	250
Permeability (cm/s)	10^{-4} to 10^{-7}	10^{-4} to 10^{-7}

jet mix columns to a cut-off in clay at level −30 using the triple jet method. The grouting was made from a working platform at level −8 under Blaak station with a headroom of only 4·3 m. The groundwater was reduced temporarily to level −0·5 during the jet grouting works. To avoid risk of instability of the existing station during installation of the cut-off, the row of jet grouted columns was offset a small distance from the line of the H pile wall and the gap between the H piles and the jet grout secant wall was plugged with a double row of short jet grout columns.

The second example shown in Fig. 2.26 is the use of jet-grouted columns at the rear of hand-dug caissons for a peripheral basement wall to a commercial development in Hong Kong. The soil on the west side of the basement was retained by conventional diaphragm wall construction to depths greater than 20 m, while hand-dug caissons were used at shallower depths on the side where bedrock was at depths between 5 and 20 m. Prior to the installation of the caisson wall a cut-off was necessary to avoid groundwater drawdown outside the site and the consequent settlement to neighbouring structures. The cut-off was made by a grout

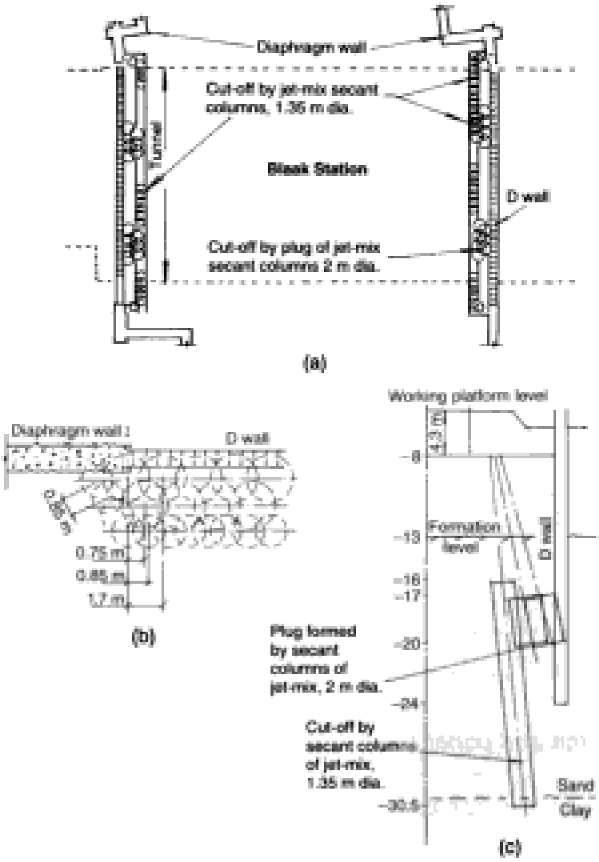


Fig. 2.25. Example of jet grouting for cut-off construction at the Blaak metro station, Rotterdam: (a) plan of cut-off walls; (b) jet mix column installation; (c) cross-section of deep and shallow jet mix walls (courtesy of Bachy)

curtain where soils were groutable using bentonite cement grout in the first phase followed by silicate gel injection using tubes á manchettes. Where the fine silty sands overlying bedrock were not easily grouted, the cut-off was made from secant jet mix columns. The columns, shown in Fig. 2.26, were generally 0·8 m in diameter and a pressure of 200 bar was used to install them with a cement grout proportion of 350 kg/m^3 of soil cement grout.

Horizontal grout curtains

Within a sheeted or walled deep excavation a horizontal grout curtain may be needed to reduce the vertical inflow of groundwater from permeable strata below formation level where the peripheral sheeting or walls do not achieve a cut-off into bedrock or a relatively impermeable stratum. Such a situation arises where a deep permeable stratum or layered permeable strata exist. The depth of a peripheral vertical cut-off wall may prove uneconomic in these soil conditions and the alternative of a box-like cut-off using a horizontal grout curtain to seal the base of the excavation enclosure may be attractive.

Two options are available. A shallow horizontal curtain may be formed by jet grouting or, where soil conditions allow, a deeper horizontal curtain may be formed by intrusion grouting, using either clay–cement or silicate grouts, depending on soil permeability. It is often economical to anchor shallow grout curtains with mini piles or deeper jet-grouted tensile anchors, which reduces the effective span of the grout curtains and resists the upward groundwater pressure without the assistance of the significant dead weight of the overburden pressure of the soil above it. The installation of jet-grouted columns improves the subsoil strength below formation

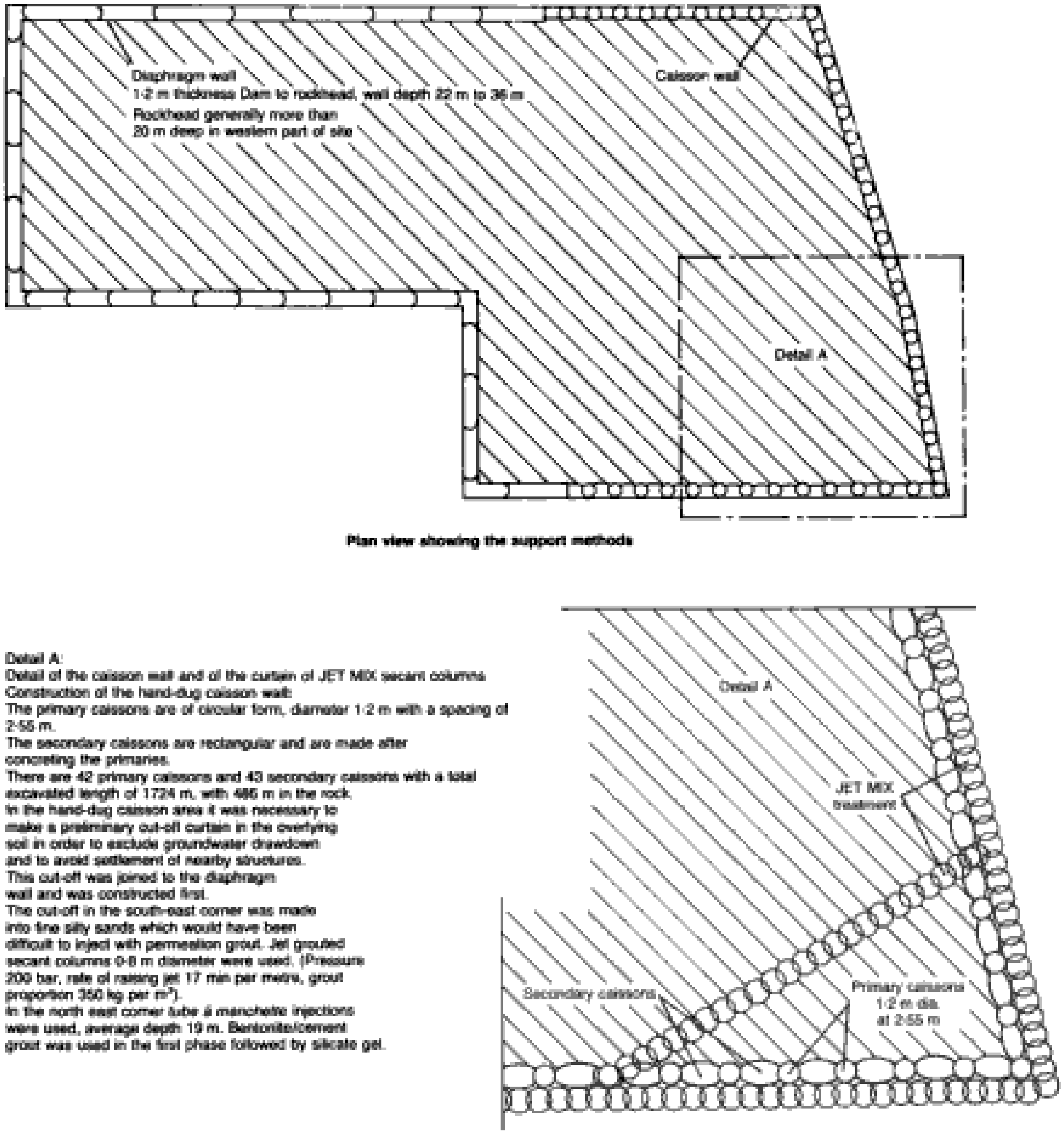


Fig. 2.26. Example of jet grouting for cut-off treatment at the rear of hand-dug caissons at Kowloon, Hong Kong (courtesy of Bachy)

level and increases the subsoil passive resistance, giving support to the peripheral walls. The shallow jet-grouted, anchored cut-off therefore has the prime objective, of excluding groundwater, but it can also materially assist in supporting the peripheral walls. Deeper silicate grout curtains, which in turn require deeper peripheral walls, may be formed more economically by intrusion grouting rather than jet grouting where soil permeability allows. These deeper cut-offs benefit from the dead weight of the retained soil above the grout curtain to resist the upward groundwater pressure and probably avoid the need for intermediate anchoring from jet-grout columns. Specified maximum leakage rates in the range 2–5 l/s per

1000 m^2 of plan area would be used in this type of construction in cohesionless soils. More traditional methods of underwater concreting may prove economical as horizontal cut-offs, particularly in long, narrow excavations. Additives are frequently used with underwater concrete to improve flowability, waterproofness and control setting time.

Design of dewatering systems

Design objectives

The objectives of the methods described to control groundwater in the previous sections, by exclusion (with surface drains or by cut-off walls) or by removal of groundwater (by drainage or by pumping), are to:

(*a*) lower the water table and intercept influencing seepage which would otherwise enter the excavation and interfere with the work
(*b*) improve the stability of slopes and prevent slippages
(*c*) prevent heave in the bottom of excavations
(*d*) reduce lateral pressures on temporary sheeting and bracing.

Design methods to remove groundwater by pumping, either by various wellpointing methods or deep wells, will be considered in the remainder of this chapter. It is assumed that the means of dewatering has been selected, the soil properties, including permeability, are known or can be estimated, and the depths of each stratum and the height of groundwater have been determined. The design will determine the number and spacing of wellpoints or wells to achieve the required drawdown and the required capacity of pumping to remove the groundwater yield at the well heads. The design yield, or quantity of water to be pumped, depends on various factors including the permeability of the soil fabric, the groundwater source (whether flow is radial or from a line source such as a river or shoreline), whether the wells penetrate fully or partly to the base of the aquifer, the plan shape of the excavation and the plan layout of wells.

The design methods make fundamental assumptions regarding soil and groundwater flow which are only partly fulfilled on site; therefore, a balance has to be struck between the results of such calculations and experience, preferably local experience, of the selected dewatering method. The assumptions made include:

(*a*) the aquifer extends horizontally with uniform thickness in all directions without encountering recharge or barrier boundaries
(*b*) the aquifer is isotropic, that is, the permeability is the same in all directions
(*c*) the aquifer releases water from storage instantly when the head is reduced
(*d*) the pumping well is frictionless and has a very small diameter
(*e*) under steady-state seepage conditions, Darcy's law applies and flow is laminar (Darcy's law states that for a given soil the velocity of flow is directly proportional to the hydraulic gradient and the soil permeability, $v = ki$, where v is velocity of flow, i is the hydraulic gradient and k is the soil permeability).

Standard well formulae largely remain the basis of design for typical dewatering systems as described here. Relationships for discharge into a slot from a line source or sources are used with empirical expressions for radius of influence to model flow quantities to wellpoints or wells for trenchworks. Formulae for radial flow to a single well are used to model flow to a whole excavation, which is square or rectangular in plan. The application of two-dimensional flow nets is convenient, although not necessarily accurate, to obtain estimates of quantities of groundwater flow into sheeted excavations and to verify the stability of the formation soils due to seepage pressures as a result of the dewatering. The design is therefore a process of approximation and does not replace either previous experience or common sense.

Powrie and Preene[17] compared flow rate/drawdown relationships computed by equivalent well and infinite slot techniques with finite element methods, and advised the range of validity of the equivalent well method for various excavation plan geometries. The same authors compared case records of 30 dewatering schemes in fine-grained soils[18] to assess the validity of analysis methods in steady-state conditions. They concluded that equivalent well/slot methods could be effectively used to estimate flow rates for dewatering systems in these soils providing the method of analysis is appropriate to the boundary conditions in the field. Close sources of recharge were found to increase flow ratio considerably, and it was recommended their effects be modelled by flow net techniques.

Both two-dimensional steady-state flow programmes and equivalent well formulae therefore provide means of estimating flow rates, each with advantages and disadvantages. The importance of accurate selection of a value for soil fabric permeability nevertheless remains common to both. Preene and Powrie[18] confirmed the widely-held practical opinion that the most reliable permeability estimates are obtained from field pumping tests with drawdown measured by piezometers. The method of analysis should be varied according to the aquifer boundary conditions, using the well formula where the recharge aquifer is not close to the well, and finite element analysis otherwise.

Where no pumping test is available and the soil has isotropic permeability and contains less than approximately 20% of silt and clay size particles, Hazen's rule should be used to estimate permeability from grading curves, providing representative particle size test results are available. (Hazen's rule is $k = CD_{10}^2$, where k is permeability (in m/s), D_{10} is the 10% particle size (in mm) and the constant C is usually taken in the range 0·01 to 0·0125). From tube samples the mean value of permeability from all curves should be used, but from bulk samples the minimum value is likely to be more reliable.

If the soil contains more than 20% of silt and clay particles, or less significant anisotropic permeability, in situ rising or falling head tests should be used in boreholes. Providing that an adequate number of tests are made, the maximum result should be used. The use of permeability results from laboratory testing is not recommended.

In the next section, well formulae are described as a series of cases depending upon the source of water, confinement of the aquifer and penetration of the well. Design methods are then proposed which use either well formulae or flow net methods to determine the quantity of water flowing towards the pumps, and hence the number, and size, of pumps required.

Well formulae

Mansur and Kaufman[19] estimated discharge and drawdown from various well configurations for the cases presented below.

Dewatering for trenchworks

Case 1. Partial penetration by a single row of wellpoints of an unconfined aquifer with gravity flow fed from a single line source (Fig. 2.27(*a*)). Application: narrow trench work, wellpoints to single side, unconfined aquifer, river or similar line source.

The total discharge Q from wellpoints is

$$Q = \left[\left(0{\cdot}73 + 0{\cdot}27\,\frac{(H - h_o)}{H}\right)\frac{kx}{2R_o}\,(H^2 - h_o^2)\right]\ (\mathrm{m^3/s}) \tag{2}$$

and the maximum residual head h_D downstream from the slot is

$$h_D = h_o\left[\frac{1{\cdot}48}{R_o}\,(H - h_o) + 1\right] \tag{3}$$

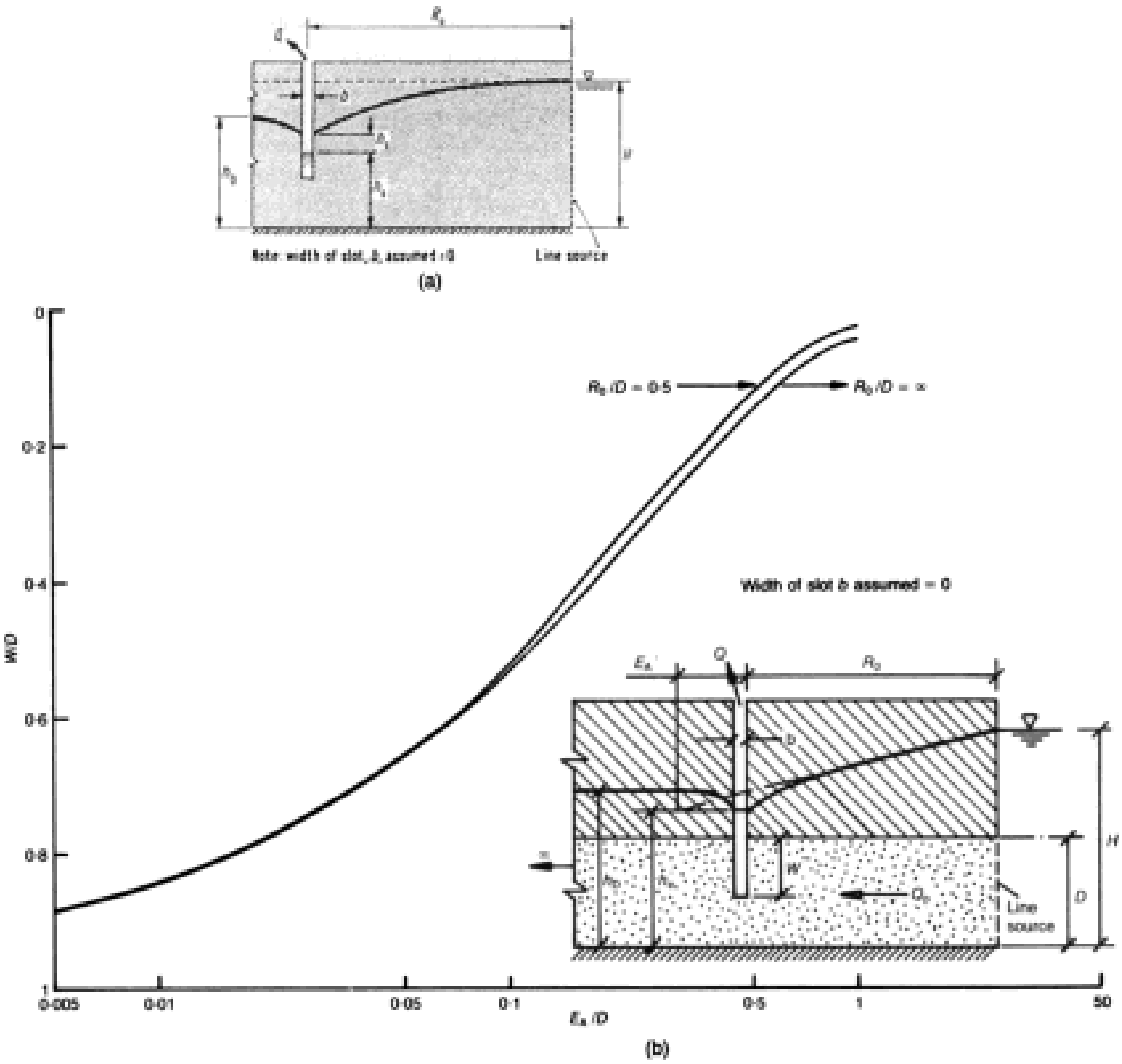


Fig. 2.27. Partial penetration by a single row of wellpoints from a single line source: (a) gravity conditions; (b) artesian conditions, plot of W/D against E_A/D *(CIRIA*[1]*)*

where x is the length of the trench (m), H is the height of the static water table (m), h_o is the height of the water table in wells, h_s is the difference in head between the outside and inside of the well (it is small, approximately $0{\cdot}001H$), k is the soil permeability (m/s), and R_o is the distance of the line source, taken as equal to the radius of influence R_o (m). Mansur and Kaufman stated that these expressions for Q and h_D are valid for the ratio of radius of influence to the height of the static water table equal to or greater than 3·0. This ratio covers most site conditions.

Case 2. For artesian conditions, partial penetration of a single row of wellpoints fed from a single line source. Application: narrow trench work, wellpoints to single side, artesian conditions, river or similar line source (Fig. 2.27(*b*)).

The total discharge is

$$Q = \left[\frac{kDx(H - h_s)}{R_o + E_A}\right] \text{ (m}^3\text{/s)} \qquad (4)$$

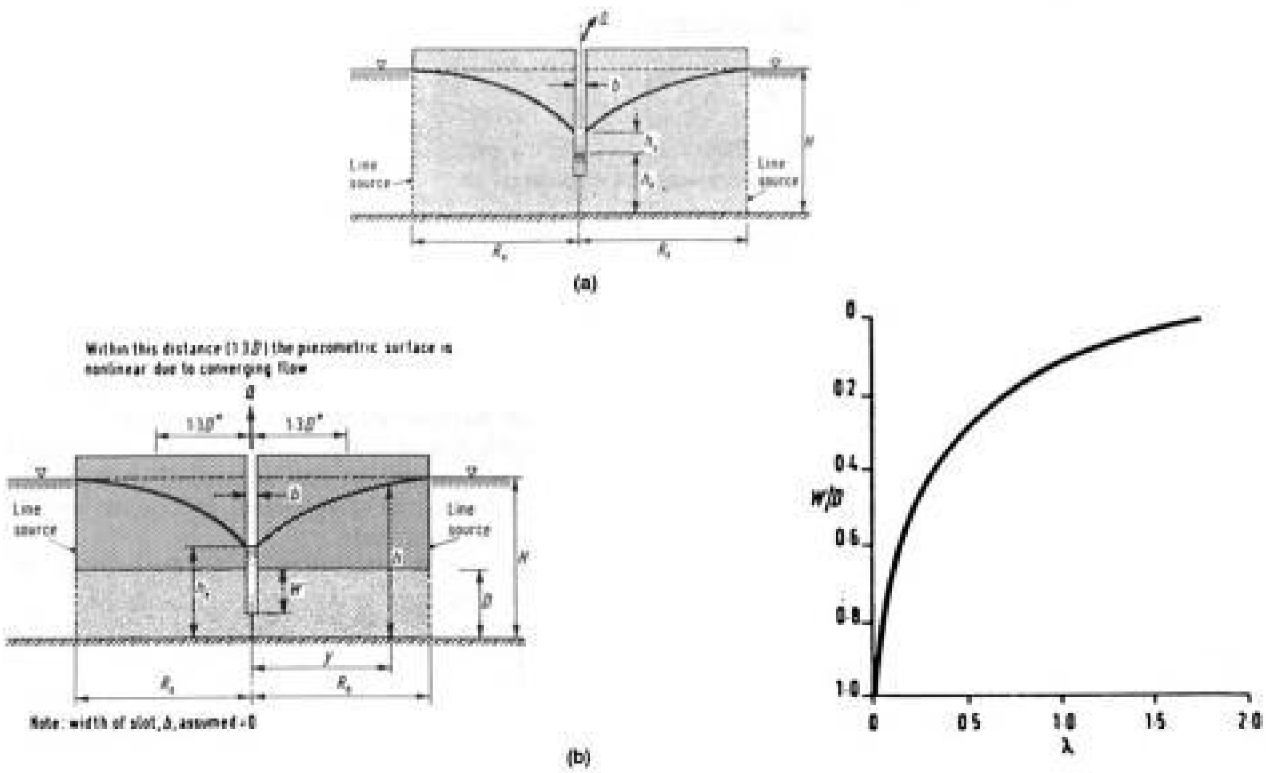


Fig. 2.28. Partial penetration by a single row of wellpoints midway between parallel line sources: (a) gravity conditions; (b) artesian conditions, plot of W/D against λ (CIRIA[1])

where E_A is an extra length factor which depends upon the ratio of slot penetration to the thickness of pervious stratum, and is obtained from Fig. 2.27(b); h_e is the head of water at the well above the base of the acquifer, and D is the thickness of the aquifer. The maximum residual head downstream from the slot is

$$h_D = \frac{E_A(H - h_e)}{R_0 + E_A} + h_e \tag{5}$$

Case 3. Partial penetration by a single row of wellpoints of an unconfined aquifer with gravity flow midway between parallel line sources (Fig. 2.28(a)). Application: narrow trench work, wellpoints to single side, unconfined aquifer, two line sources, say two rivers, a trench midway between them.

The discharge is

$$Q = \left[\left(0{\cdot}73 + 0{\cdot}27\,\frac{(H - h_0)}{H}\right)\frac{kx}{R_0}\,(H^2 - h_0^2)\right] \tag{6}$$

Mansur and Kaufman reported that this expression, as in cases 1 and 2, is based on model studies by Chapman[20] for gravity flow from a line source to a single partially penetrating slot. The model test showed slight irregularities and the equation should therefore be regarded only as an estimate of flow required to provide given head reduction.

Case 4. For artesian conditions, partial penetration by a line of wellpoints midway between two parallel line sources (Fig. 2.28(b)). Application: narrow trench work, wellpoints to single side, artesian conditions, two line sources, say two rivers, a trench midway between them.

The total discharge is

$$Q = \frac{2kDx(H - h_o)}{R_o + \lambda D} \tag{7}$$

At distance y from the slot, when y exceeds $1{\cdot}3D$, the head h increases linearly as y increases and can be expressed as

$$h = h_e + (H - h_e)\left(\frac{y + \lambda D}{R_o + \lambda D}\right) \tag{8}$$

where λ is a factor dependent upon the ratio of slot penetration to aquifer thickness (see Fig. 2.28(*b*) in which W is the depth of the base of the well below the upper horizon of the aquifer).

Dewatering for a wide trench or narrow rectangular excavation

Case 5. Partial penetration by a double row of wellpoints of an unconfined aquifer with gravity flow midway between two parallel line sources (Fig. 2.29(*a*)). Application: wide trench works with double row of wellpoints, unconfined aquifer, two line sources, a trench midway between them.

Q is the total combined flow from both slots and is twice that for a single line source (see equation (2) in case 1). The head is

$$h_D = h_o\left[\frac{C_1 C_2}{L}(H - h_o) + 1\right] \tag{9}$$

C_1 and C_2 are obtained from Fig. 2.29(*a*).

Note that for large or square excavations, wellpoints will be required on all four sides of the excavation. A conservative approximation of the pumping capacity needed can be made by calculating the values of Q separately for opposite sides of the excavation.

Case 6. For artesian conditions, partial penetration by a double row of wellpoints midway between two parallel line sources (Fig. 2.29(*b*)). Application: trench works with double row of wellpoints, artesian conditions, two line sources, a trench midway between them.

Again, Q is the total combined flow from both slots and is twice that for single line sources (equation (4)). Values of E_A from Fig. 2.27(*b*) are as for case 2. The head h_D midway between the slots can be calculated as before from equation (5) (except where the slots are very close; in this case, a conservative estimate results from the calculation).

Dewatering for square or rectangular plan shape unsheeted excavations

Forchheimer[21] derived a formula from a system of perfect gravity flow wells of equal length and capacity. This work forms the basis of the design of dewatering systems based on radial flow to a number of wells.

Case 7. Full penetration by single well of unconfined aquifer with gravity flow fed by circular source. Application: square and rectangular plan shape excavations, unconfined aquifer (Fig. 2.30(*a*)).

From Darcy's law it can be shown that

$$Q = \frac{\pi k(H^2 - h_w^2)}{\log_e (R_o/r_w)} \tag{10}$$

and drawdown $(H - h)$ at a distance r from the well can be obtained from

$$(H^2 - h^2) = \frac{Q}{\pi k}\log_e (R_o/r) \tag{11}$$

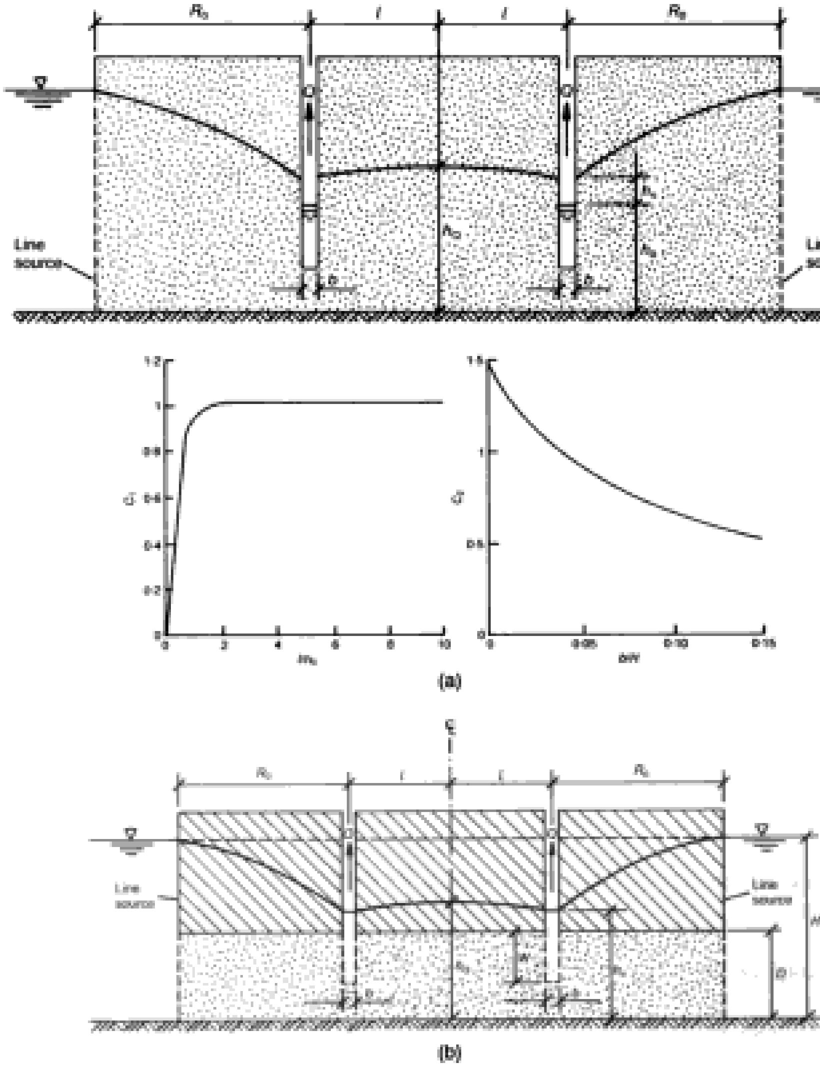


Fig. 2.29. Partial penetration by a double row of wellpoints midway between parallel line sources: (a) gravity conditions, plots of C_1 against l/h_s and C_2 against b/H; (b) artesian conditions (CIRIA[1])

Case 8. For artesian conditions, full penetration by single well fed by circular source (Fig. 2.30(*b*)). Application: square or rectangular plan shape excavations, artesian conditions.

$$Q = \frac{2\pi kD(H - h_w)}{\log_e (R_o/r_w)} \tag{12}$$

and drawdown $(H - h)$ at distance r from the well can be obtained from

$$H - h = \frac{Q}{2\pi kD} \log_e(R_o/r) \tag{13}$$

Case 9. Full penetration of circular arrangement of wells in an unconfined aquifer. Application: square or rectangular plan shape excavations, unconfined acquifer.

From Forchheimer's work it can be shown that with a circular arrangement of wells

$$Q = \frac{\pi k(H^2 - h_c^2)}{\log_e R_o - \log_e a} \tag{14}$$

where Q is the total flow to the circular well array, a is the radius of the wells

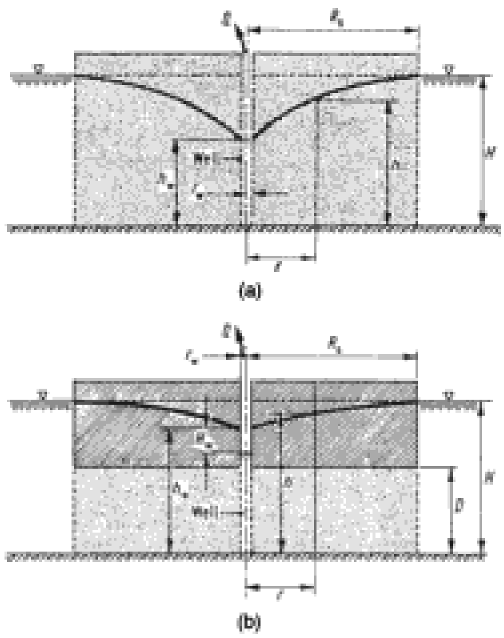


Fig. 2.30. Full penetration by a single well fed by a circular source: (a) gravity conditions; (b) artesian conditions (CIRIA[1])

from the centre of the circular well array, and h_0 is the height of water above the impermeable stratum at the centre of the circular well array.

Design methods

Formulae which estimate flow and drawdown from line sources to slots, and radial flow to a single or circular array of wells, and flow nets used to model two-dimensional flow conditions, are used in the remainder of this chapter to solve design problems for dewatering excavations of different sizes and construction. The design work of estimating pump resources, pump depths and locations should all be considered as much art as science, with adequate allowance for in situ conditions which may differ from those presumed in the calculations.

Dewatering of trench using progressive wellpoint system: design procedure

(*a*) Determine the geometry of the trench cross-section, the extent of drawdown required and the permeability of strata.

(*b*) Determine the extent of the dewatered length of the trench works. Assume the dewatered length will need to be twice the excavated length at any one stage.

(*c*) Determine the side slopes to the excavation and whether single- or double-sided wellpoint installation is required.

(*d*) Check the wellpoint will partially penetrate the aquifer.

(*e*) Consider the radius of influence R_0 for the required drawdown

$$R_0 = 1500(H - h_D)k^{1/2} \tag{15}$$

Check that $R_0/H \geq 3$.

(*f*) If a double line of wellpoints is required, check the operating level at the line of wellpoints from the relationships h_0/H and h_D/H for different R_0/H (Fig. 2.31). Check the required drawdown at the line of wellpoints from the header pipe level to the top of the wellpoint does not exceed the maximum wellpoint drawdown of 5 to 6 m.

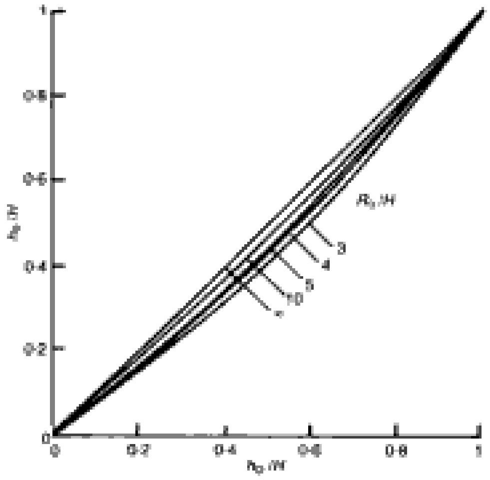


Fig. 2.31. Plot of h_o/H against h_D/H for various R_o/H (CIRIA[1])

(*g*) Check the value of h_D from the value of h_o chosen for wellpoint tip elevation from

$$h_D = h_o\left[\frac{C_1C_2}{R_o}(H - h_o) + 1\right] \text{ (see below)} \tag{16}$$

(*h*) Compute the total flow Q from

$$Q = \left[\left(0{\cdot}73 + 0{\cdot}27\,\frac{(H - h_o)}{H}\right)\frac{kx}{2R_o}(H^2 - h_o^2)\right] \text{ (m}^3\text{/s)} \tag{17}$$

for double-sided wellpointing (divide the total flow by two for the flow from a single row of wellpoints).

(*i*) Estimate spacing, and hence the number of wellpoints, from the nomograph for uniform clean sand and gravel or stratified clean sand or gravel (Fig. 2.8).

(*j*) Calculate flow per wellpoint and check the capacity of the wellpoint from Fig. 2.32.

(*k*) Check approximate size of header pipe required for flow calculated per side (Fig. 2.33).

(*l*) Check head losses in the system and calculate loss in the header pipe from

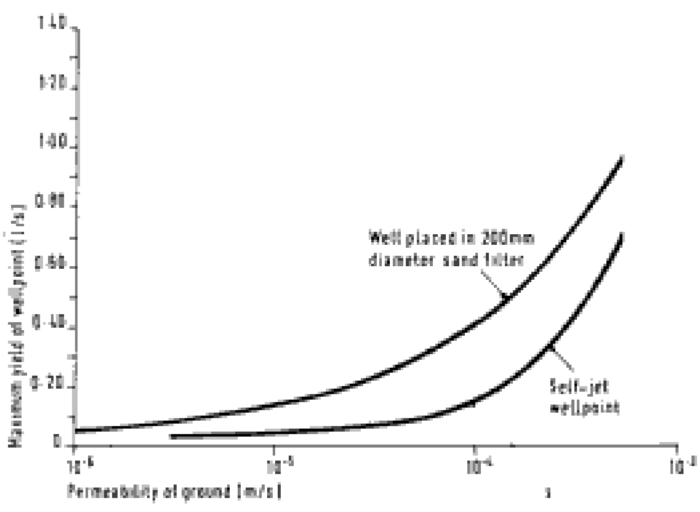


Fig. 2.32. Maximum yield of wellpoints as a function of soil permeability (CIRIA[1])

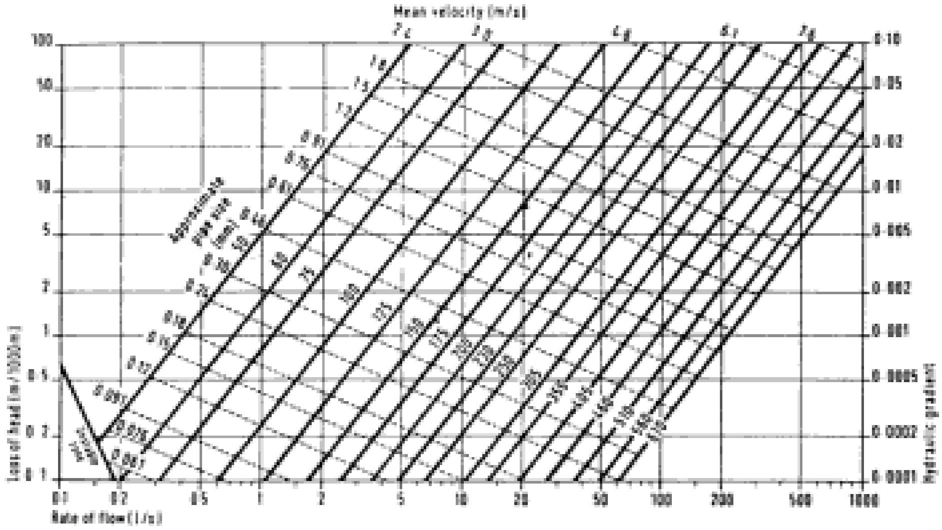


Fig. 2.33. Wellpoints: header pipe capacity and friction losses (CIRIA[1])

Table 2.9. Allow equivalent pipe length for fittings (valves, bends and tees). Calculate the total head loss.

(*m*) Determine the pump size from total head (required drawdown and head loss) and required capacity (allow at least 50% above calculated capacity per side to achieve initial drawdown). Refer to the pump manufacturer's data for vacuum-assisted pumps (see Fig. 2.34 for typical head capacity and priming time curves). Note that the estimated radius of influence R_o calculated in this method is the value when equilibrium drawdown has been established. Before this, during the initial drawdown period, the radius of influence will be reduced and the yield will be greater; both pump and pipework sizes must allow for this.

(*n*) For single- and multi-stage installations to relieve artesian pressure from a contained aquifer, calculate R_o using appropriate well formulae: h_D from equation (5) and Q is twice that for single line sources (equation (4)), and follow the design procedure for installation with the gravity flow method above.

Dewatering of square or rectangular plan shape excavation by single- or multi-level wellpoints

(*a*) Use well formulae for case 5 for opposite sides of the excavation, estimating the radius of influence, drawdown and total flow, as detailed in the section above. Estimate the spacing of wellpoints from the total flow on each side of the excavation using nomographs as before, and determine the header pipe dimensions, head loss and size of pump or pumps, as above.

(*b*) For multi-stage wellpointing, make successive calculations of drawdown, calculating the radius of influence, drawdown and total flow for each stage.

(*c*) For artesian conditions use well formulae for case 6, calculating in turn the radius of influence, drawdown, total flow, number of wellpoints, header pipe size, head losses and pump size.

Dewatering of deep square or rectangular plan shape excavation with battered side slopes by deep wells

The following method was described by Hausmann.[22]

Table 2.9. Friction losses in valves and fittings, expressed as a length of straight pipe (in m) (CIRIA[1])

	Type of fitting	Diameter (mm)								
1. Gate valve	Open	1·1	1·4	1·7	2·0	2·8	4·3	5·2	6·4	7·6
	¼ closed	6·1	7·9	10·1	12·2	18·3	24·4	30·5	41·2	48·8
	½ closed	30·5	39·6	51·8	59·5	91·5	122·0	152·0	213·0	244·0
	¾ closed	122·0	159·0	213·0	244·0	366·0	488·0	610·0	854·0	976·0
2. Standard tee	Flow in line	2·9	4·3	5·0	5·9	9·1	11·9	15·1	22·0	24·7
	Flow to/from branch	9·8	12·8	16·8	19·8	30·5	39·6	50·3	73·2	82·3
3. Standard 90° elbow		4·9	6·4	8·4	9·9	15·2	19·8	25·2	36·6	41·2
4. Medium sweep 90° elbow		4·3	5·5	6·7	7·9	12·2	15·9	21·3	28·0	32·0
5. Long sweep 90° elbow		3·2	4·3	5·3	6·1	9·1	12·2	15·2	21·3	24·4
6. Square (90°) elbow		9·8	12·8	16·8	19·8	30·5	39·6	50·3	73·2	82·3
7. 45° elbow		2·3	3·1	3·7	4·6	6·4	8·5	10·7	15·2	18·3
8. Sudden enlargement	d/D = ¼	4·9	6·4	8·4	9·9	15·2	19·8	25·2	36·6	41·2
	d/D = ½	3·2	4·3	5·3	6·1	9·1	12·2	15·2	21·3	24·4
	d/D = ¾	2·9	3·7	4·9	5·6	8·4	11·0	13·7	19·8	22·9
9. Sudden contraction	d/D = ¼	2·3	3·1	3·7	4·6	6·4	8·5	10·7	15·2	18·3
	d/D = ½	1·7	2·3	2·9	3·4	4·9	6·4	8·2	11·3	12·8
	d/D = ¾	1·1	1·4	1·7	2·0	2·8	4·3	5·2	6·4	7·6

(*a*) Make an initial estimate of the total quantity of groundwater to be pumped by replacing the actual excavation with a circular plan shape of approximately equal area. Use the well formula from case 9 (equation (14)). If the actual excavation is rectangular with length X and width Y, then

$$a = (XY/\pi)^{1/2} \tag{18}$$

Use the radius of influence R_o from

$$R_o = 3000(H - h_c)k^{1/2} \tag{19}$$

(*b*) Estimate the number of wells needed. For an individual well of radius r_w the discharge quantity is

$$Q_i = 2\pi r_w h_w k i_e \tag{20}$$

where h_w is the height of well screen, and i_e is the average entry gradient. According to empirical findings, i_e should not exceed $1/(15k^{1/2})$ to avoid turbulence and filter instability for spacings larger than 15 well diameters. Thus, the capacity of an individual well should be limited to

$$Q_i = 2\pi r_w h_w k^{1/2}/15 \tag{21}$$

and if h_o, the height of the top of the well screen above the bottom of the aquifer, is set equal to h_w, the height of the well screen, then

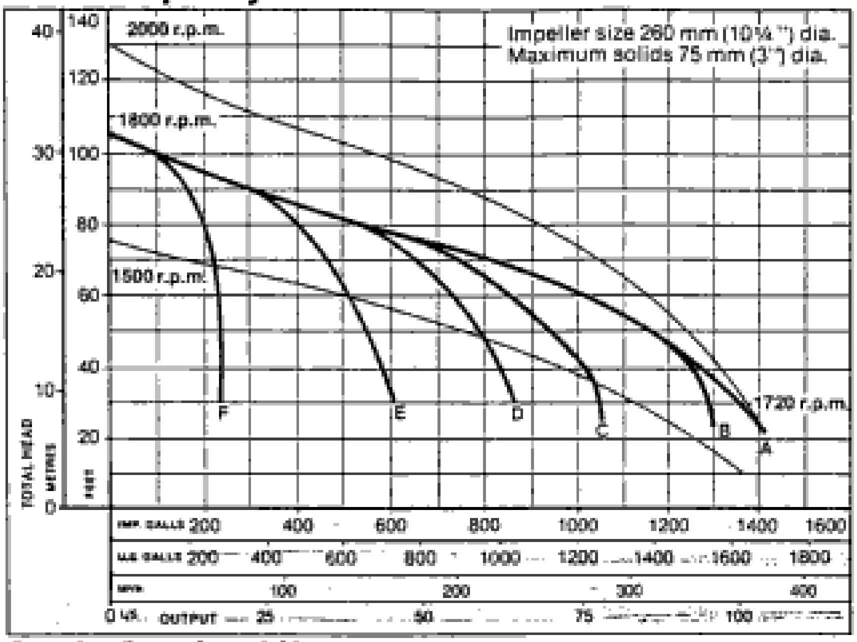
$$Q_i = 2\pi r_w h_o k^{1/2}/15 \tag{22}$$

If the number of wells is n, then $n = Q/Q_i$.

Sykes Super UV150/HP is an automatic priming, solids handling centrifugal pump with 150mm (6") suction and discharge connections. In standard form the pump, which incorporates a bearing housing, is coupled to a Lister HR3 air-cooled diesel engine and is mounted on a four-wheeled chassis with Ackerman steering connected to a towbar.

Suitable for all contractors duties where high performance is the main criterion, the Super UV/150/HP will pump water containing a high proportion of abrasive solids, crude sewage, thick slurries, gaseous sludges and trade effluents. It is ideally suited for wellpoint dewatering applications. Solids up to 75mm (3") diameter can be passed, the pump will prime and reprime automatically at suction lifts down to 9m (30ft) and will deal with intermittent flows under snore conditions.

Head/Capacity Curve



Static Suction Lift

A—1.5m (5ft) using 3.1m (10ft) 150mm (6") i/d hose
B—3.1m (10ft) using 6.1m (20ft) 150mm (6") i/d hose
C—4.6m (15ft) using 6.1m (20ft) 150mm (6") i/d hose
D—6.1m (20ft) using 9.1m (30ft) 150mm (6") i/d hose
E—7.6m (25ft) using 9.1m (30ft) 150mm (6") i/d hose
F—9.1m (30ft) using 12.2m (40ft) 150mm (6") i/d hose

Priming Time

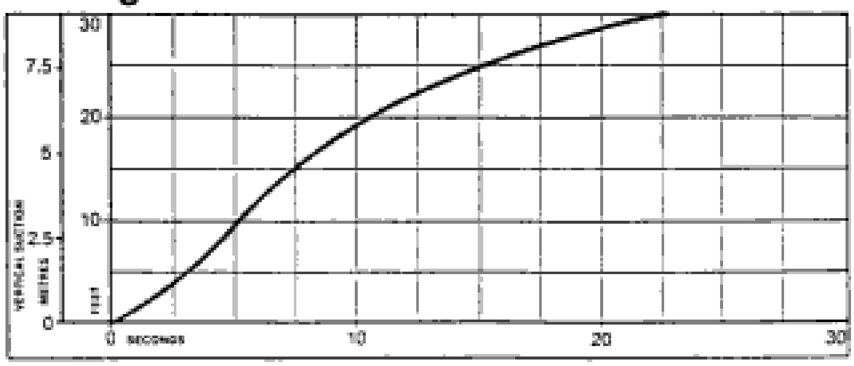


Fig. 2.34. Head/capacity and priming time curves for Sykes Univac pump, 150 mm suction and discharge.

(c) Check the original estimate of h_e using

$$Q_{total} = \frac{\pi k(h^2 - h_e^2)}{\log_e R_o - (1/n)\log_e(x_1, x_2, \ldots, x_n)} \tag{23}$$

where $x_1 \ldots x_n$ are the radial distances of the wells from the centre of the excavation. Solving for h_e results in a new, improved value for h_e. Using this value, a new R_o and Q are computed. Steps (a) to (c) are repeated until h_e assumed is sufficiently close to h_e calculated in step (c).

(d) Return to the original excavation. Distribute the n wells around the perimeter. Check the water level at critical points below the excavation (at the centre and at the corners) using equation (23). If the water level is too high, the overall pumping rate Q_{total} has to be increased. This in turn results in a reduced value for h_e and may result in an entry gradient i_e in excess of the maximum recommended of $1/(15k^{1/2})$. If so, increase the number of wells and repeat the calculation.

(e) Submersible, centrifugal pumps with one or more impellers driven by an electric motor are used for most deep well installations. The required pump capacity is

$$N = \frac{Qh\gamma_w}{\eta} \tag{24}$$

where γ_w is the density of water to be pumped, and η is the system efficiency. Taking into account friction loss in the delivery pipework, η is usually in the range 0·3 to 0·5. Assuming $\eta = 0\cdot3$ and $\gamma_w = 10$ kN/m^3,

$$N = \frac{Qh}{40}\ (\text{kW}) \tag{25}$$

where Q is in litres/s and h is in metres.

Dewatering of sheeted excavations where the sheeting does not achieve a seal within an impermeable stratum or horizontal grout cut-off

The design of dewatering systems for sheeted excavations is most conveniently undertaken with the use of two-dimensional flow nets, making provision for the length of the excavation in the calculation to take into account three-dimensional groundwater flow. Where the soil type or depth of excavation vary around the perimeter of the excavation, separate flow net computations can be made for each length and summated.

Flow net construction

The three-dimensional flow of water through a porous media can be represented by the La Place equation which when simplified into two dimensions can be modelled by flow net construction. The flow net consists of flow lines, or stream-lines, which represent an almost infinite number of groundwater flow paths and intersecting equipotential lines. The family of curves therefore consists of curvilinear squares. Flow nets can be constructed in four ways: by computer program, by graphical means, by electrical analogy or by physical model. The last alternative is both expensive and time-consuming and can in practice be deleted from the list. On the other hand, computer programs based on the finite element or finite difference methods provide the most convenient and fastest solution (standard spreadsheet programs can be used employing finite differences as described by Williams *et al.*[23]).

The graphical construction of flow nets was described by Taylor[24] and these methods still provide a practical means of determining the base stability of an excavation and the groundwater flow rate into the excavation. For the relatively complex geometry of a sheeted excavation, computer methods based on finite differences/elements may require considerable experience to calculate accurate flow rates, and the graphical method may be preferred.

The electrical analogy method was described by Montague and Thomas[25] and until the widespread introduction of personal computer programs was used in preference to physical modelling, especially for flow net construction for flow through dams and embankments. The basis of the analogy is that with electricity the current is proportional to the voltage drop, whereas with groundwater flow through soils seepage is proportional to head dissipated; in this analogy, conductivity corresponds to the permeability of the soil. This method no longer finds application and flow nets are now constructed either by hand, graphically, or by computer program.

A flow net may represent flow in plan or in vertical section. It may be convenient to summarize the rules which apply for the graphical solution:

(*a*) Flow lines and equipotential lines intersect at right angles and form curvilinear squares.

(*b*) Where the entire section cannot be divided conveniently into squares, a row of rectangles will remain and the ratio of the lengths of the sides of each rectangle will remain constant.

(*c*) A discharge face under atmospheric pressure is neither an equipotential nor a flow line; therefore, squares are incomplete and flow lines need not intersect such a boundary at right-angles.

(*d*) In gravity flow systems, equipotential lines intersect the phreatic surface at equal intervals of elevation, each interval being a constant fraction of the total net head.

(*e*) In sedimentary strata it is not unusual for the horizontal permeability to be greater than the vertical permeability. It is necessary to model this difference in seepage calculations by the graphical construction of flow nets with a deliberate scale distortion (this was described by Taylor[24]). The scale change to the geometry of the vertical section is made prior to drawing the flow net as follows. If the horizontal permeability is k_h and the vertical permeability is k_v, the transformed section of the natural horizontal scale is x and the distorted scale x_1 required to take into account the difference in vertical and horizontal permeability is

$$(k_v/k_h)^{1/2}/x \tag{26}$$

(*f*) To determine the quantity of seepage, the discharge q per unit width and the head h at any point can be determined by

$$q = kH_1\frac{N_f}{N_e} = k(H - h_e)\frac{N_f}{N_e} \tag{27}$$

and

$$h = \frac{n_e}{N_e}H_1 = \frac{n_e}{N_e}(H - h_e) \tag{28}$$

where k is the coefficient of permeability of the soil, H is the total head at entry, h_e is the head at the flow exit, H_1 is the overall net head $(H - h_e)$, N_f is the number of flow channels in the net, N_e is the total number of equipotential drops between the full head H and head h_e at the point of flow exit, and n_e is the number of equipotential drops from the exit to the point at which head h is desired.

A range of flow net constructions is shown in Fig. 2.35. Flow nets can also be used to model flow through strata having two or more different permeabilities; the solution to these composite sections was described by Cedergen.[26]

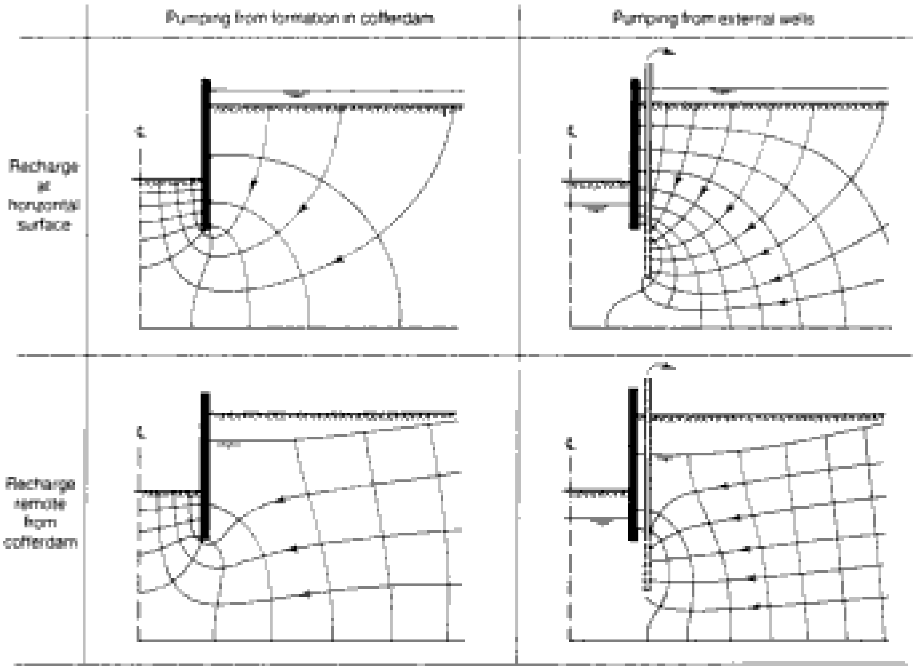


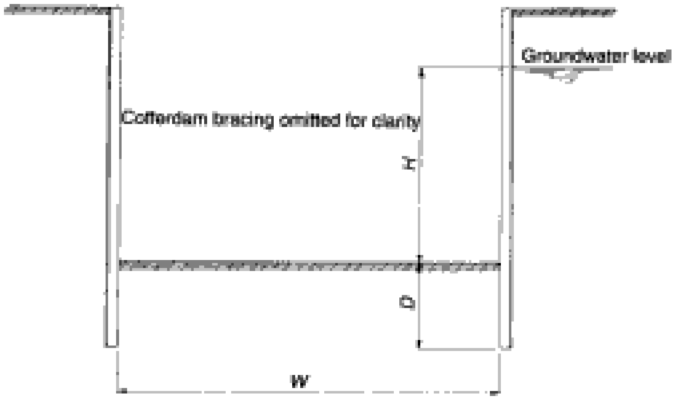
Fig. 2.35. Typical flow net construction

Stability of the base of dewatered sheeted excavations

The risk of failure at the base of a sheeted excavation due to seepage below the sheeting was referred to by Terzaghi and Peck[27] and McNamee,[28] and is summarized in BS 8004[2] and the German Recommendations of the Committee for Waterfront Structures.[29]

McNamee[28] referred to local failure by piping or boiling and a general failure of the soil below formation level by heave. BS 8004 concluded that to avoid reduction in passive resistance in a sheeted excavation below formation level in a cohesionless stratum, with pumping from sumps at formation level, the minimum penetration depths should be as shown in Fig. 2.36. The German Waterfront Code, however, provided more detailed guidance for a similar excavation. It referred to risk of base failure in a cohesionless stratum in two ways, as did McNamee. The following summarizes its recommendations, referring to the specific problem shown in Fig. 2.37. First, the risk of local failure by piping is regarded as a risk which cannot be accurately assessed by analytical methods because of the diversity of conditions, including variations in local soil conditions. Where these variations are not excessive, the Waterfront Code[29] concluded that the danger of failure increases in proportion to the overall head difference inside and outside the excavation, and increases in the presence of loose, fine-grained non-cohesive or weakly cohesive material in the subsoil, especially where loose sand lenses occur. The failure risk does not generally occur in strong cohesive soils. Such a failure is first indicated at the formation level by ground swelling and ejection of soil particles in the progressive failure manner shown in Fig. 2.38. An effective preventative measure at an early stage would be to place several thick layers of granular materials, as a graded filter, to retain the subsoil but avoiding excessive restriction of water flow. Only if such measures failed, or were applied too late, would it be necessary to flood the excavated space to equalize the water pressure.

Although the Waterfront Code[29] advised care and observation to counter risk of piping, that is, the local reduction of effective stress to produce quick conditions,



Minimum values for depth of cut-off for cohesionless soils where there is no significant lowering of the external water level

Width of cofferdam, W	Depth of cut-off, D
$2H$ or more	$0.4H$
$1H$	$0.5H$
$0.5H$	$0.7H$

H is the height from lowered water level to excavation level.

Fig. 2.36. Minimum values for cut-off depth for cofferdam sheeting in cohesionless soils (BS 8004[2])

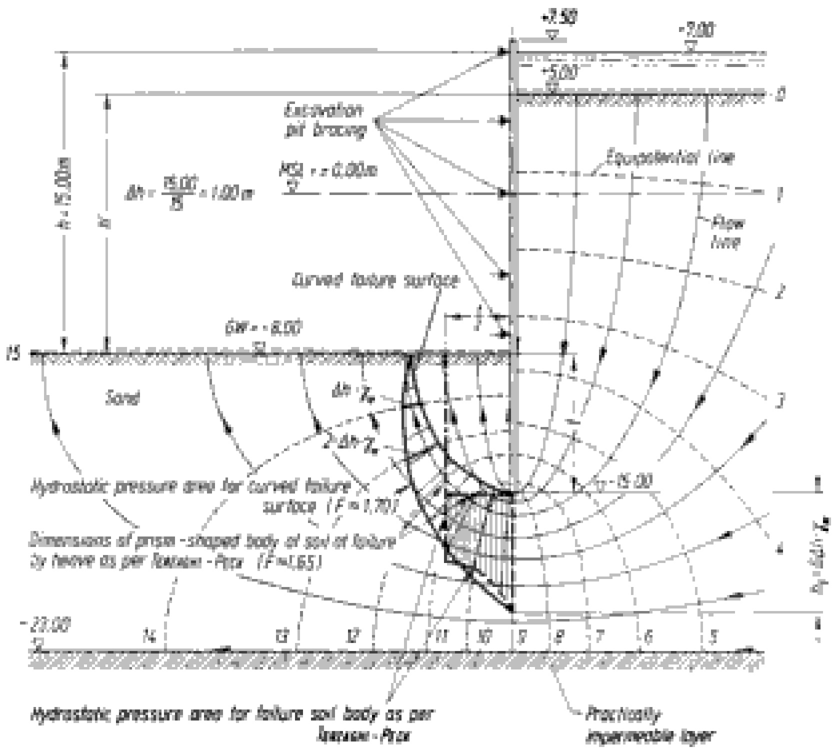


Fig. 2.37. Factors of safety against failure by heave at formation level calculated using the exit gradient from the flow net with a curved failure surface and a prism-shaped failure soil body (due to Terzaghi and Peck)[29]

it is prudent to check the value of hydraulic gradient at the exit point with the minimum seepage path length. The maximum exit gradient can be calculated by the use of a flow net, from

$$i_{exit} = (h/N_d)/a \tag{29}$$

where a is the length of the flow element at the formation level, and N_d is the

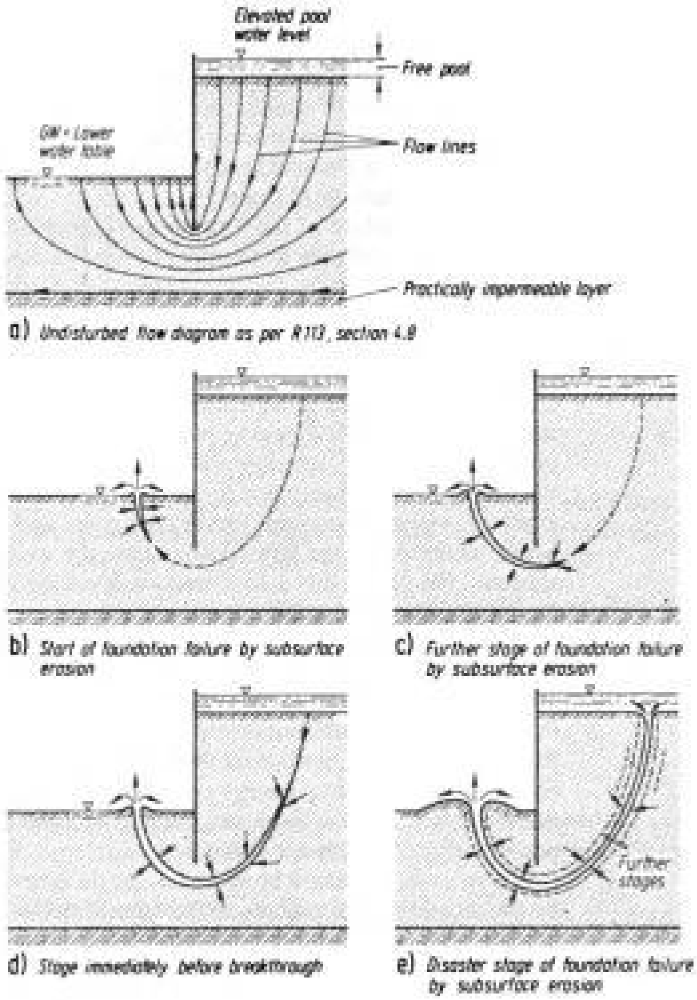


Fig. 2.38. Development of foundation failure by subsurface erosion[29]

number of equipotential drops. The factor of safety against piping is i_{cr}/i_{exit}, where i_{cr} is the critical hydraulic gradient. It can be shown that for zero effective stress

$$i_{cr} = \frac{G_s - 1}{e + 1} \tag{30}$$

where G_s is the specific gravity of the soil grains and e is the voids ratio of the soil. For most soils, i_{cr} lies in the range 0·9 to 1·1, with an average value of 1·0. A minimum factor of safety against piping, i_{cr}/i_{exit}, of 1·5 is advisable.

Reference 29 considered failure risk by heave at the base of the excavation and (see Fig. 2.37) recommended examination of the stability, in the vertical direction, of volumes of soil contained between the inside face of the sheeting and alternative failure planes. One trial failure plane would be that recommended by Terzaghi and Peck,[27] enclosing a rectangular block of soil of depth equal to the sheeting penetration and width equal to half the sheeting penetration. Other failure planes, including those with curved surfaces, would be examined. The vertical stability of the block of soil is assessed as the ratio of the weight of soil w_{cs} above the failure plane to the vertical force c_{fl} caused by the seepage water. As a factor of safety, a ratio of at least 1·5 is recommended.

The vertical component c_{fl} of the total seepage pressure may be conveniently calculated by flow net. In this method, the residual hydrostatic pressure acting normal to the failure surface is plotted at each intersection of the equipotential lines and the failure surface, the area of the plot representing the total seepage pressure acting on the trial failure surface. The pressure at each intersection point may be computed from $n\Delta h\gamma_w$, where n is the number of equipotentials at the intersection, Δh is the total head difference divided by the number of equipotentials, and γ_w is the density of water.

The risk of failure of the formation to sheeted excavations in soft clays which is not related specifically to seepage pressures is described in chapter 7 on cofferdam design.

The risk of formation heave or local failure by piping changes when deep wells are used to remove groundwater below formation level. When deep wells are used outside the sheeting, drawdown is increased and the quantity of water pumped also increases compared with drawdown and flow quantity when the wells are sited inside the sheeting. It should be noted, however, that where deep wells are installed outside the sheeting, seepage pressures tend to be reduced on the subsoil between the sheeters below formation level, whereas when the wells are sited between the sheeters, seepage pressures tend to increase, and the risk of base in stability and heave increases. In the latter case, where the well screen is installed between the sheeters, the closer this is to formation level the higher the seepage pressure and the instability risk, but the drawdown outside the sheeters is less and the quantity of water pumped is smaller. Where deep wells are used between sheeters which do not find a seal with the toe of the sheeters in an impermeable stratum, the optimum sheeter depth will be defined by four factors: the risk of instability of the soil below formation level by seepage pressure, the risk of drawdown outside the sheeters causing settlement damage to existing structures, the pump resources required (and the pump energy consumed) to obtain and maintain adequate drawdown below formation level, and last, the maximum values of strut or anchor loads, particularly at the lowest frame level.

Where a cut-off cannot be obtained, the designer frequently defines sheeter depth by the stability requirements of the subsoil below formation level and the lowest bracing loads. The dewatering scheme design is then applied to ensure the necessary drawdown below formation level to obtain dry working conditions with the minimum number of pumps. However, a more refined approach is advisable, with coordinated sheeting depth, formation stability, bracing design and required pumping capacity, with improved economy.

Development of drawdown with time

Dewatering systems are designed on the basis of a steady-state condition being achieved at the required drawdown. Invariably, construction programmes on site are tight and the period of pre-drainage to achieve this steady-state may become critical; indeed, extra pump capacity may be needed to avoid delay. Assessment of the period to achieve drawdown may therefore become important. Hausmann[22] referred to work by Theis,[30] later modified by Jacob,[31] which gives a solution. Theis provided a non-equilibrium well formula for gravity flow to a single well based on the following assumptions:

(*a*) the well penetrates a homogeneous, isotropic, horizontal aquifer overlying an impermeable stratum
(*b*) the Dupuit–Thiem approximation holds; the hydraulic gradient below any point of the drawdown curve is equal to the slope of the drawdown curve at this point
(*c*) the aquifer is not recharged
(*d*) water flows out of the pores of the soil simultaneously with the drawdown of the phreatic surface

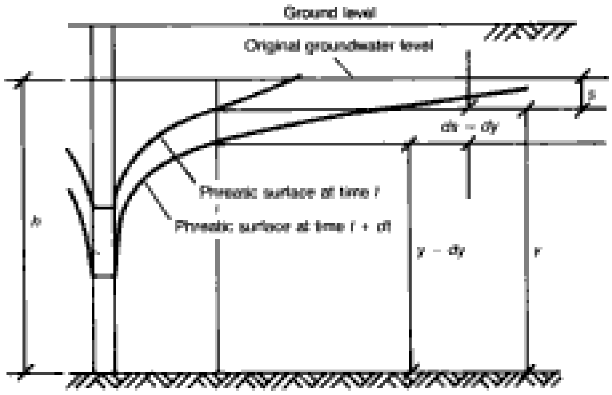


Fig. 2.39. Lowering of phreatic surface with time (Hausmann[22])

(*e*) the drawdown s is small relative to the aquifer thickness h so that $(h - s)$ is approximately equal to h. This is equivalent to assuming a non-artesian aquifer of thickness $m = h$.

Theis, as reported by Hausmann, equated the water flow through a cylindrical area around the well at a distance x to the volume of water removed from the soil beyond x due to lowering of the phreatic surface, as shown in Fig. 2.39. Theis showed that the drawdown is

$$s = \frac{Q}{4\pi km} W(u) \tag{31}$$

where Q is the quantity of flow, k is the permeability of the soil, m is the thickness of the aquifer, and $W(u)$ is the well function and is defined by

$$W(u) = -0{\cdot}5772 - \log_e u + u - \frac{u^2}{2 \times 2!} + \frac{u^3}{3 \times 3!} - \frac{u^4}{4 \times 4!} \pm \ldots \tag{32}$$

and

$$u = \frac{x^2 S}{4tkm} \tag{33}$$

S represents the storage coefficient of the aquifer and is the ratio of the volume of drainable water to total soil volume for an unconfined aquifer, and may be as low as 10^{-5} for confined aquifers which remain saturated during pumping; for unconfined aquifers, it ranges from 0·01 to 0·03. t is the pumping time. $W(u)$ is tabulated in hydrology textbooks (e.g. Driscoll[32]).

Hausmann showed, from Jacob's modification of the work by Theis, that if t is sufficiently large, u is small and for $u < 0{\cdot}05$ the well function may be approximated by

$$W(u) = -0{\cdot}577 - \log_e u = \log_e \frac{2{\cdot}25}{4u} = \log_e \frac{2{\cdot}25kmt}{x^2 S} \tag{34}$$

The drawdown as a function of time is then

$$s = \frac{Q}{4\pi km} \log_e \frac{2{\cdot}25kht}{x^2 S} \tag{35}$$

To apply Jacob's formula (for artesian flow) to unconfined flow, make $m = h$ and use the analogy $2ms$ (artesian) $= h^2 - y^2$ (unconfined), which holds for fully penetrating wells. Thus

$$s = \frac{h^2 - y^2}{2h} \tag{36}$$

and

$$\frac{h^2 - y^2}{2h} = \frac{Q}{4\pi km} \log_e \frac{2 \cdot 25kht}{x^2S} \tag{37}$$

Drawdown s can therefore be plotted against t and x, using the geometry of the section to obtain h and y and using soil property values for k and S.

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3 Open excavation: side slopes and soil retention

This chapter examines the problems of supporting side slopes or cut faces to the periphery of wide, deep excavations and to smaller sites where the soil batters can be accommodated in the available land area. For large cuts associated with heavy earth moving, the scale and size of these cuts means cross-bracing would be quite impractical, whereas for smaller sites the choice between battered or braced excavations is dependent on available space and cost considerations. The wide, open excavations considered may be for permanent use or as part of temporary works for construction. The only criteria for inclusion is that the work is related to civil engineering and not to an industrial process such as mining or quarrying.

Battered excavations

Temporary battered excavations are more economical in direct cost and construction time compared with retention schemes for soil support, and find application where the slopes can be accommodated within the site area and groundwater discharge on to the slopes is small or can be controlled. The decision to omit a support or retention system can only be made on the basis of adequate subsoil knowledge and appropriate stability analyses, however, and should not be made solely on the basis of a cost calculation and an assumed gradient to the batter.

The stability of a cut slope in granular soils can be determined simply from knowledge of the angle of shearing resistance for the soil or the soil layers. In normally consolidated cohesive soils, the stability can be assessed by a repetitive calculation of disturbing and restoring moments for trial potential failure surfaces, the restoring force being based on quick undrained shear strengths of the soil on the failure surface. Where possible, it is usual to avoid the effects of disturbance on the measurement of clay strengths by measuring in situ shear strengths with a vane apparatus rather than relying on laboratory test values.

In cut slopes within over-consolidated clays, it is necessary to relate the method of analysis to the time the cut is to remain exposed to take account of drainage and the equalization of excess pore pressure generated by the relief of vertical overburden pressure. While an effective stress analysis using drained shear strength parameters would apply to the fully drained condition for permanent works, the designer of temporary batters for construction periods of, say, six months has some difficulty in assessing short-term slope stability without knowing the rate of pore-water pressure dissipation. The choice of slope angle in these clays will depend on the consequences of slope failure. If safety is not endangered by soil slippage, the financial cost of repairs to the slope can be calculated and compared with the actual cost of reducing the slope angle to achieve greater stability.

Vaughan and Chandler[1] calculated soil failure strengths from short-term slope failures in various stiff, over-consolidated clays and concluded that average clay strengths on the failure surface lie in the range 50% to 100% of the undrained shear strength of 38 mm dia. samples. Short-term slope stability in over-consolidated clays in practice depends on the precautions taken to avoid groundwater discharging down the slope (say from permeable strata above the clay) and rainwater falling on to the slope. Harmful effects on stability of the swelling of such clays, due to relief of overburden pressure and the further softening and swelling as groundwater enters fissures, are good reason to adopt measures to protect the slope. Simple measures such as polythene sheet covers or sprayed concrete blinding can prove beneficial and cost-effective. Slope stability analysis methods were reviewed by Bromhead.[2]

Improving the stability of slopes

There are six methods for improving the stability of a cut slope:

(*a*) regrading the profile of the slope and, for example, weighting the toe of the slope locally with a soil berm to reduce the disturbing movement
(*b*) using tensioned ground or rock anchors to increase the effective stress on the potential failure surface, thereby improving soil strength
(*c*) intercepting potential failure surfaces with sheet piles or jet grouted columns installed from the face or the top of the slope
(*d*) increasing the effective vertical stress on the potential failure surfaces by reduction of pore-water pressure by drainage
(*e*) improving composite soil strength by regrading the slope and the inclusion of reinforcement to intercept the potential failure surface, using reinforced soil
(*f*) driving soil nails through potential failure surfaces.

These methods are well established and enable the engineer to minimize land take at the batters to the periphery of a wide excavation, and may produce a steeper slope with more economy by virtue of reduced excavation quantities. Methods (*a*)–(*d*) are described by Bromhead,[2] Mitchell[3] and Barley.[4] Methods (*e*) and (*f*), reinforced soil and soil nailing, deserve fuller explanation within the context of deep excavations.

In the UK the Department of Transport's manual for reinforced soil and soil nailing[5] defines these techniques as follows.

- Reinforced soil is the technique whereby fill material (frictional or cohesive) is compacted in successive layers on to horizontal placed sheets or strips of geosynthetic or metallic reinforcement
- Soil nailing is the technique whereby in situ ground (virgin soil or existing fill material) is reinforced by the insertion of tension-carrying soil nails. Soil nails may be of metallic or polymeric material, grouted into a predrilled hole or inserted using a displacement technique. They will normally be installed at a slight downward inclination to the horizontal.

In summary, reinforced soil is built-in fill from the base upwards, whereas soil nailing is used in cuts in virgin soils working from the ground surface downwards.

Reinforced soil

Although historically several soil reinforcement methods have been exploited for both civil and military use, the modern form of earth reinforcement was developed and introduced commercially by Henri Vidal in the 1960s. His concept was a composite material of frictional soil and a reinforcing strip which enabled the gravity forces on a wall or slope to be resisted by tensile forces generated in the strip and thence transferred by friction to the soil. The first major structure to be built by this method was a retaining wall near Menton, France, in 1968. By the end of the 1960s, walls were reinforced with galvanized steel strips placed horizontally in layers and connected to a shaped sheet metal member which formed the facing unit. In 1970 a cruciform-shaped precast reinforced concrete member was introduced to replace the sheet metal facing, and this form of wall facing is now widespread and has become the means of identifying Reinforced Earth walls. Over 15 000 structures have been built throughout the world by Vidal's original company, the Reinforced Earth Company Ltd, and many more have been built

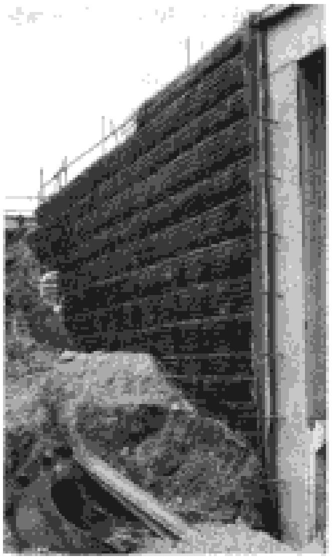


Fig. 3.1. Reinforced Earth construction to bridge abutment at trunk road improvement to A1(M) in Hatfield, Hertfordshire, UK (courtesy of Netlon)

by competitors using similar methods and, in some cases, alternative reinforcement materials. At the end of 1989 the Reinforced Earth Co. listed 9000 retaining wall structures and 2000 bridge abutments in service. Recently, research into the Reinforced Earth wall has centred on the durability of the galvanized steel strips, organic coatings for steel strips in aggressive environments and the effects of seismicity on Reinforced Earth walls. The development of polymer materials in either fabric or grid (geogrid form) in the 1970s allowed the introduction of alternative reinforcement materials in both walls and slopes. Use of these materials, which are visco-elastic, brought the need for further investigation of reinforcement properties, this time into short- and long-term creep in addition to durability. A typical Reinforced Earth wall using polymeric geogrid reinforcement is shown in Fig. 3.1.

In the UK, the basis of permanent works design for reinforcement of highway slopes is the Department of Transport's manual,[5] and in France it is Recommendations and Rules, LCPC, 1979.[6] A Code of Practice for design and construction of Reinforced Earth walls was published by the British Standards Institution in 1995 (see bibliography).

Four types of reinforcement have been used in current designs:

(*a*) a metal strip, typically a galvanized, high adherence steel strip (by Reinforced Earth Co.)
(*b*) polymer geogrids (by Netlon Ltd)
(*c*) polyester bands known as Parastrip (by Exxon)
(*d*) synthetic fibres mixed in place with soil, known as Texsol.

The strips are listed in order of use in the UK.

The reinforcement strips must have the following characteristics:

(*a*) a high resistance to tension, a failure mode which is not brittle and only a very low, limited creep
(*b*) a high friction coefficient with the backfill material
(*c*) low deformability under working loads (not exceeding a few percent)
(*d*) they must be flexible enough not to limit the deformability of the Reinforced Earth material and to make construction easy
(*e*) high durability
(*f*) they must be economical.

The cost of reinforcement in the Reinforced Earth wall is a relatively high proportion, between 20 and 30%, of the total plant labour and material costs, and is therefore a prime issue in terms of overall wall costs. Typical current total costs for Reinforced Earth walls are £100 to £210 per square metre of wall face.

Metal strip reinforcement

The metallic reinforcing strip currently recommended is galvanized mild steel, generally 5 mm thick and in standard widths of 40 or 60 mm. The surface of the strip is ribbed to improve soil-reinforcement friction. The strips are called high adherence strips. In permanent walls it is usual to allow a sacrificial thickness of the metal to be lost due to corrosion (from 0·5 to 2 mm depending on design life and exposure). It is also usual to place dummy strips through the wall face on aggressive sites in order that strips can be jacked out occasionally to monitor the rate of corrosion. The galvanized steel strip and precast concrete facing panel are competitive for vertically-faced walls of medium to large height, due to the high elastic modulus of mild steel and the low creep properties.

Figure 3.2 shows a typical cross-section of a Reinforced Earth structure using metal strip reinforcement and precast concrete panel facing. The economics in construction time and cost are determined by a simple and repetitious construction process. After placing the base course of the facing panels on a prepared foundation,

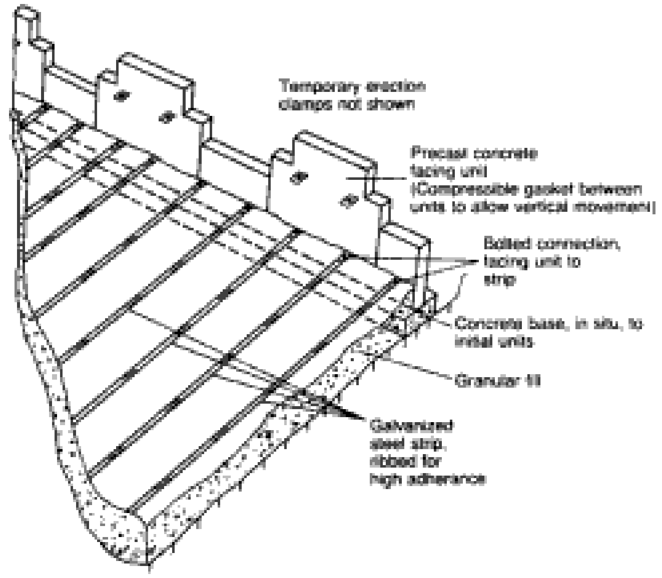


Fig. 3.2. Reinforced Earth wall during construction showing reinforcing strips and precast concrete facing units (courtesy of Reinforced Earth Ltd)

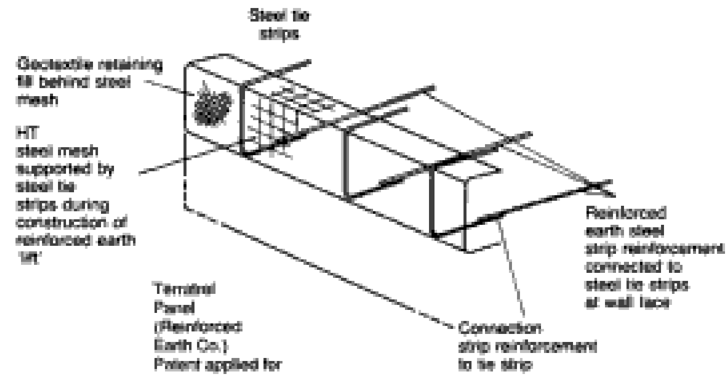


Fig. 3.3. Terratrel panel for use in steep slope construction (courtesy of Reinforced Earth Ltd)

each additional panel mechanically interlocks with the previous course. The reinforcing strip and backfill are placed, and compact backfill is then placed in successive layers. Varying shapes of facing panel are available to suit aesthetic requirements. More recently, the Reinforced Earth Co. has developed a design technique which has been used in many steep and near-vertical slopes. In particular, the method, using steel strip reinforcement and a soft facing of steel mesh backed with geotextile, suits the requirements of an economic, quick-to-build, soft-faced steep slope able to accept vegetative cover. This technique is called Terratrel.[7] The use of wrap-around geotextiles to steepen soil slopes has often been associated with relatively slow construction methods, with excessive amounts of plant and labour. Terratrel uses low-cost steel mesh facing panels in discrete heights between reinforcement levels, allowing a repetitive erection technique without the need for propping of framework. Fig. 3.3 shows a typical panel. The technique allows high adherence galvanized steel reinforcing strips to be fixed directly to the steel mesh facing unit, which is sufficiently rigid to support itself as a vertical cantilever, fixed in advance of earth filling. The geotextile used as a backing material behind the steel mesh facing retains the soil and protects from soil wash-out before vegetation can become established. Plant growth can be started by hydroseeding, or turf can be placed behind the mesh as construction proceeds to give an instant green effect. Steep slopes have been built using a coarse stone fill behind the mesh to give a deliberate gabion-type finish. The method is subject to patent application in the UK by the Reinforced Earth Co. In the UK, an 11 m high, 80° slope of 2000 m² face area has been built at Leeds, and lower slopes have been built at Nottingham (the A52 road improvement), the Elan aqueduct at Birmingham, and the A74 Bogbain bypass in Scotland.

Polymer geogrids

Tensar geogrids are high-strength polymer grids specifically made as tension-resistant inclusions in soils. The manufacturing process begins with an extended sheet of polyethylene or polypropylene which is punched with a regular pattern of circular holes. This sheet is then stretched under controlled heat conditions so that randomly long chain molecules are drawn into an aligned state. This process of stretching increases the tensile strength and stiffness of the polymer. The resulting geogrid structure of ribs and bars produces an effective means of transferring friction from soil to geogrid. The prime disadvantage of using a polymer grid is the low elastic modulus and high creep value compared with steel strip. The influence of long-term strength has been the subject of vigorous research,[8] and the influence of load capacity on the design life of the geogrid can be predicted by extrapolation from laboratory curves of strain against load with increasing time at various temperatures. Details of characteristic strength derived in this way are shown in Fig. 3.4. The overall effect of this long-term creep feature of the polymer is, however, to limit the use of geogrids to relatively modest vertical wall heights (typically 6 to 8 m) in order to remain competitive with metal strip Reinforced Earth walls. The prime advantages of geogrids are their resistance to attack from all aqueous solutions of acids, alkalis and salts encountered in soils, and high resistance to biological attack. The geogrids are made from polymers formulated to resist ultraviolet light degradation, and when buried or covered by vegetation their life is indefinite. Resistance to corrosion is therefore considerably greater than that of steel strip.

Polyester-reinforced thermoplastic webbing

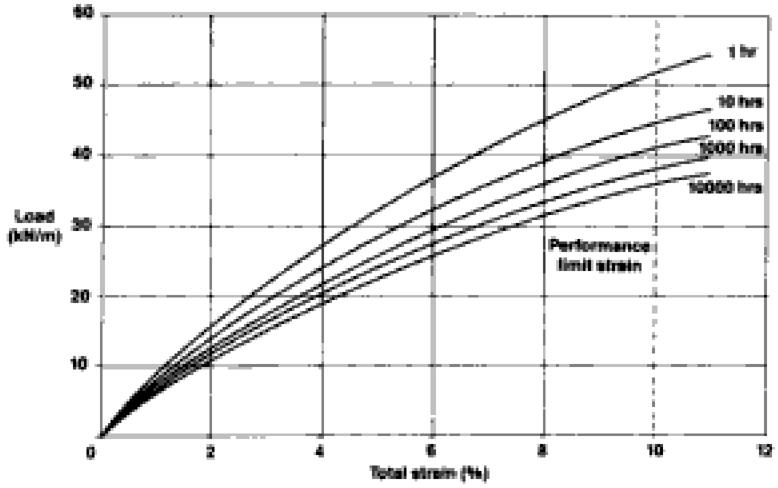
High-strength polyester-reinforced thermoplastic flat webbing also provides a tensile reinforcement material for Reinforced Earth walls, steepened slopes and mattresses for settlement control. The webbing, known as Paraweb, is manufactured in two forms, either bonded in strips to Terram melded geotextile or woven into mats.

The composite Parawab and Terram is supplied in rolls 4·5 m wide and up to 150 m long; Paraweb strip is typically 50 to 80 mm wide with centres 240 mm apart within the Terram. Using variable material properties, a range of ultimate tensile strengths are available within the range 9·5 to 125 tonnes per metre width.

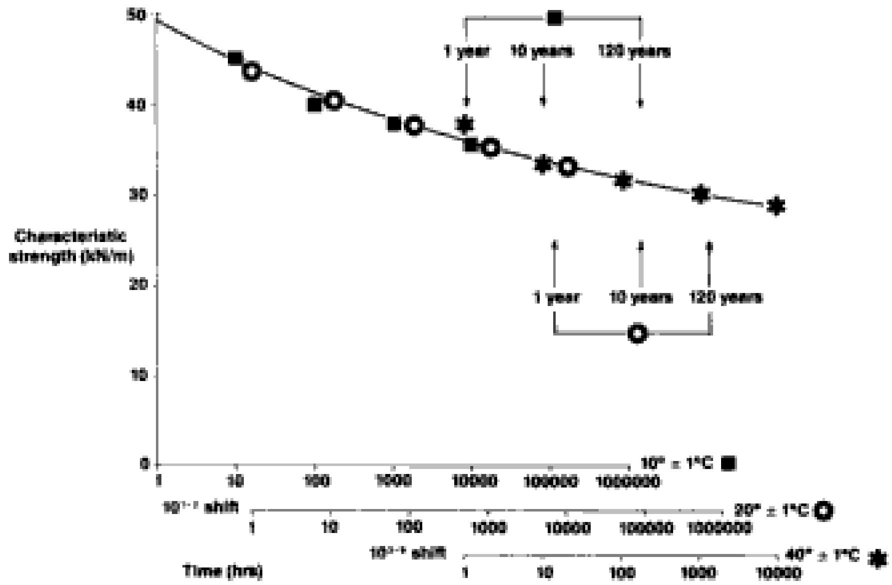
Polyester and polyaramid bands with a ribbed surface for improved soil-to-band adhesion are also produced by Exxon in widths of 90 mm and thicknesses of 6, 7 and 8 mm, with breaking loads per strip of 5, 10 and 15 tonnes, respectively. These strips are used as reinforcement in a similar manner to steel strips in Reinforced Earth walls with precast concrete or metal sheet facing panels. The plastic strips overcome the problems of durability associated with steel strips and the disadvantages of long-term creep associated with polyethylene geogrids. Meshes of these strips are also manufactured under the trade names Paragrid and Paralink.

Synthetic fibre mixed with soil in situ

The use of a fine diameter synthetic fibre mixed in place within the soil to form tensile reinforcement has been patented by the French Bridges and Roads Research Laboratory (LCPC). Contracts have been confined to France, Italy, Holland and Japan, but since 1985 approximately 50 retaining walls have been built using Texsol, with typical heights from 4 m to more than 20 m and front slopes between 45° and 80°. Texsol can only be used to enhance the slope angle to embankments in



Isochronous curves for Tensar SR80 at 10° ± 1°C



Characteristic strength curve for Tensar SR80. Time-temperature superposition

In-soil temperature	Characteristic Strength (f_k) of Tensar geogrids (kN/m)		
	SR55	SR80	SR110
10°C UK (5) and temperate climates	22.0	32.5	45.0
20°C Warm climates	20.5	30.5	42.0

Characteristic Strength of Tensar geogrids for design lives of up to 120 years

Fig. 3.4. Derivation of characteristic strength of Tensar Geogrid SR80 from short-term stress–strain tests (courtesy of Netlon)

granular fill materials. Multiple continuous threads are used in the ratio 0·1 to 0·2% by weight of natural soil. The growth of vegetation to walls built in this way is a natural process, although problems of erosion to the wall face may arise at times of heavy rainfall prior to the establishment of the plants.

Application of Reinforced Earth in excavation works

Retaining walls

Initial projects in the early 1970s for the Reinforced Earth Co. were a 23 m high wall on the Nice–Menton highway at Peyronnet, France, the walls to the coal and ore handling facility at Port Dunkirk, major retaining walls to highways in California, Quebec and Balboa, Spain, and an 11 km long wall built on the Reunion Island in the Indian Ocean.

After transportation works, industrial and commercial sectors are the next largest area for Reinforced Earth application. These include crushing and screening facilities, sloped wall bunkers for coal and ore storage, safety containment dykes for low petroleum gas (LPG) and crude oil tanks, civil and military projects providing protection against explosions, and hydraulic structures such as dam spillways, canal and river walls, coastal defence structures, marine walls and reservoirs.

The use of retaining walls in the UK has been curtailed by patents registered by Vidal in the late 1960s (for the process of Reinforced Earth) and in the early 1970s (for the facings). The Department of the Environment obtained a settlement with the Reinforced Earth Co. to build Reinforced Earth walls on public works in the mid-1980s and, more recently, the patent for the wall facing has expired. Prior to this, competitor companies (for instance, Soil Structures Ltd offering Websol, using Paraweb reinforcement and precast concrete facing units) were unable to offer schemes for vertical walls in the UK.

The use of Paraweb as a reinforcement material for vertical Reinforced Earth walls may sharply increase as a result of the expiry of the patent. Paraweb is highly resistant to corrosion attack and possesses long-term creep properties which are much improved on those of Tensar geogrids. The present UK market for Reinforced Earth vertical walls may therefore see the incursion of Paraweb (manufactured by Linear Composites Ltd and marketed by Exxon Ltd). The total area of all vertical Reinforced Earth walls constructed internationally is estimated to be of the order of 70 000 m^2 annually, with a capital value of approximately £140 million and a reinforcement cost of £2 million.

Steep slopes

The economic advantages of steepening soil slopes are self-evident both in terms of construction cost and land take. Increasingly, geotextile reinforcement is being used to reinforce fill slopes or regraded slopes in cut. These geotextile layers are placed successively within the slope as the filling is placed and compacted. As deformation within a steep slope is less critical, it is possible to use poorer grades of fill material within steep slopes than vertical wall structures.

For slopes between 45° and 90° a face support will generally be required; if geogrids or Paragrid are being used, the material can be wrapped around successive lifts of fill, thereby making rigid facings unnecessary. The use of Terratrel walls may prove cost-competitive in steep walls. Alternatively, soil reinforcement by Texsol synthetic fibres may be considered. For slopes with grades less than 45° it will not usually be necessary to use wraparound geogrid or Paragrid, and in such cases the use of a Tensar mat pinned to the slope may be necessary to protect seeded slopes from erosion before substantial growth.

Design of steep slopes and walls

The design of slopes and walls in reinforced soil in the UK is based on a method described for highway works in the Department of Transport's manual.[5] In France, the design is based on a different analysis described in the Ministère des Transports publication.[6] The UK method is known as the two-part wedge or tieback wedge, and the French method is known as the coherent gravity method.

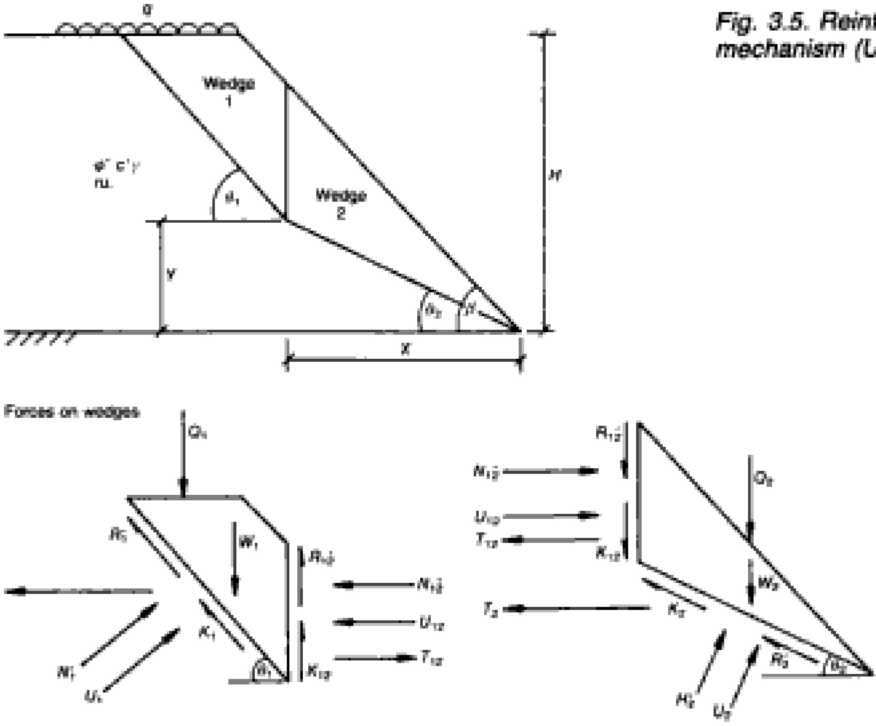


Fig. 3.5. Reinforced soil: two-part wedge mechanism (UK Department of Transport[6])

Notation

Q_1 and Q_2:	total surcharge on wedges 1 and 2
W_1 and W_2:	weight of wedges 1 and 2
T_1 and T_2:	sum of reinforcement force, wedges 1 and 2
R_1' and R_2':	tangential effective force, base of wedges 1 and 2
R_{12}':	tangential effective force inter-wedge boundary
N_1' and N_2':	normal effective force, base of wedges 1 and 2
N_{12}':	normal effective force, inter-wedge boundary
U_1 and U_2:	porewater force, acting on base of wedges 1 and 2
U_{12}:	porewater force acting on inter-wedge boundary
K_1 and K_2:	cohesion force, wedges 1 and 2
K_{12}:	cohesion force acting on inter-wedge boundary

The two-part wedge analysis is an oversimplified but conservative method, using limit state principles with partial factors. The ultimate limit state is defined as collapse conditions with fracture or pull-out of the reinforcement, and sliding or bearing failure of the base material. The serviceability limit state is defined as when slope or wall movements are excessive, affecting adjacent structures or services.

The Department of Transport manual prescribes that a partial factor of unity be applied to the self-weight of the disturbing mass of soil and any surcharge loads, and suggests that the resisting forces of soil and reinforcement strength are based on characteristic strengths factored by material partial safety factors. No further factor of safety is applied due to the conservatism of the assumed wedge mechanism.

The wedge geometry, shown in Fig. 3.5, is based on assumptions that the inter-wedge boundary is vertical and that the base of the lower wedge intersects the toe of the slope. The analysis resolves forces parallel and perpendicular to the lower surface of each wedge in turn and assumes limiting friction. A general formula derived in this way may be simplified with further conservatism by assuming that the angle of friction between the wedges is zero. The expression for the total quantity of reinforcement force then becomes

$$T_{tot} = T_1 + T_2 = \frac{[(W_1+Q_1)(\tan\theta_1 - \tan\phi_1') + (U_1\tan\phi_1' - K_1)/\cos\theta_1]}{(1+\tan\theta_1\tan\phi_1')}$$

$$+ \frac{(W_2+Q_2)(\tan\theta_2 - \lambda_s\tan\phi_2') + \lambda_s(U_2\tan\phi_2' - K_2)/\cos\theta_2]}{(1+\lambda_s\tan\theta_2\tan\phi_2')}$$

where T_1 is the sum of reinforcement forces acting on wedge 1
T_2 is the sum of reinforcement forces acting on wedge 2
W_1 is the weight of wedge 1

Q_1 is the total surcharge force on wedge 1
θ_1 is the base angle of wedge 1
ϕ_1' is the ϕ' design value acting on the base of wedge 1
U_1 is the pore-water force acting on the base of wedge 1
K_1 is the cohesion force acting on the base of wedge 1
W_2 is the weight of wedge 2
Q_2 is the total surcharge force acting on wedge 2
θ_2 is the base angle of wedge 2
λ_s is the base sliding factor
ϕ_2' is the ϕ' design value acting on the base of wedge 2
K_2 is the cohesion force acting on the base of wedge 2.

This expression can be further simplified[5] depending on whether the inter-wedge boundary lies to the left or right of the crest of the slope. The critical wedge geometry is found after a series of trials; a computer program can be used to solve the expression where a number of sections need to be analysed. The manual gives recommendations for the values of soil parameters, using residual values of cohesive soil strengths where displacements are likely to be large, and critical state values for granular soils.

The French coherent gravity method is based on four assumptions.

(*a*) Only cohesionless fill is used in the construction of the wall.
(*b*) The reinforced soil mass consists of an active zone and a resisting zone separated by a line of maximum tension in the reinforcement. The failure mode is similar to that of a mass of cohesionless soil supported by a rigid wall rotating about the top.
(*c*) The stress state within the reinforced mass varies from an at-rest condition at the top of the structure to an active stress at a critical depth.
(*d*) The grids interact and interlock with the surrounding fill to provide an effective resistance against pull-out.

In the coherent gravity method the vertical stresses in the calculations of external stability are generally based on the Meyerhof distribution, and calculations for internal stability give reinforcement anchorage lengths beyond the assumed line of maximum tension between active and restraining zones.[6]

There are, therefore, significant differences between the French and UK design methods, and structures designed according to the French coherent gravity method may not satisfy the UK Memorandum.

Soil nailing

Soil nailing is the strengthening of existing ground by the introduction of steel reinforcement into the exposed face. There are two particular applications: the retention of a vertical or near-vertical cut soil face, and the stabilization of a soil slope. The soil nailing process is typically a top-downwards operation, whereas Reinforced Earth construction is usually bottom-upwards.

The 'nails' are usually steel rods 20 to 40 mm in diameter or small steel angle sections typically 50 × 50 mm (the latter are used in particular in a patented method known as an Hurpinoise wall). The nails are grouted into pre-drilled holes or driven using a percussion drilling device or high-energy firing device. The nails are not prestressed and their density is relatively high (1 bar per 0·5 to 0·6 m^2). The nail length is dependent on slope geometry and soil conditions, but would typically be 0·4 to 0·5 times the vertical height of the cut soil face. The soil is progressively excavated in depths of about 1 to 1·5 m and the exposed soil face is protected by a layer of sprayed concrete (gunite or shotcrete) reinforced with high tensile

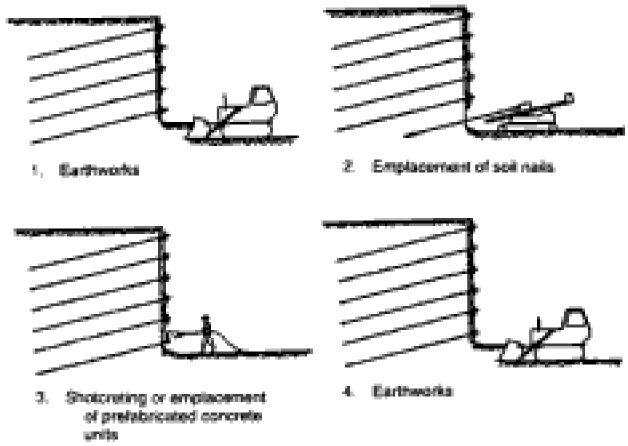


Fig. 3.6. Execution phases of soil nailing (Project Clouterre[10])

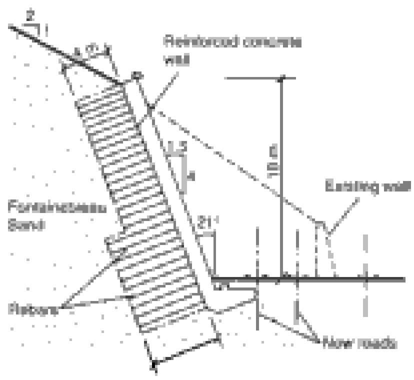


Fig. 3.7. First soil-nailed wall at Versailles, France in 1972/73 (Project Clouterre[10])

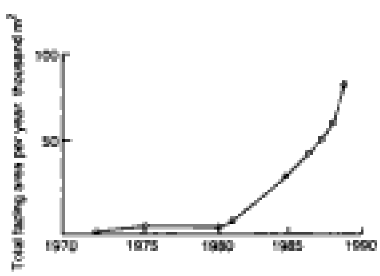


Fig. 3.8. Increase in the use of soil-nailing in France, 1972–89 (Schlosser et al.*[11])*

steel mesh. The nails are driven or drilled into the face before or after guniting, depending on risk of soil instability. The process is shown in Fig. 3.6.

The first soil nailed wall was completed in 1973 in Versailles, France, to retain a cut for a railway (Fig. 3.7). The technique is applied most economically in granular soils and relies on relatively dry ground conditions for practical installation. The growth of the method in Western Europe has centred on France. Fig. 3.8 shows the extent of the increasing use of soil nailing, which is explained by its low cost compared with traditional soil support systems.

In the mid-1980s, the use of soil nailing was advancing so quickly in France that the state of site and design work was exceeding theoretical knowledge and proven methods. In part, specialist geotechnical firms found that compared with general contractors there was insufficient expertise in installation and little sophistication in equipment to keep the market to themselves. The use of the process therefore expanded rapidly in the hands of general contractors. In Germany and the UK the method found less favour, possibly due to less favourable ground conditions.

Following the growth of this method, a four-year national research programme called Clouterre was commissioned by the French Ministry of Transport. Twenty-one firms and research centres collaborated under the technical direction of Professor F. Schlosser. The report incorporates the results of full-scale trials of nailed cuts and includes directives in both design and specification.[10]

Design methods

For design purposes, a general stability analysis is applicable with potential failure surfaces either falling outside the volume of reinforced soil, partly within it, or wholly outside the reinforced volume. Fig. 3.9 shows the location of the potential failure surfaces. The potential failure surface which intersects the reinforcement will cause bending and shear stresses within the reinforcement in addition to the tensile, pull-out forces. Fig. 3.10 shows the maximum distortion to a flexible nail at the failure surface. The failure surface is not the classical Coulomb wedge but the locus of maximum tensile forces in the nails. This potential failure surface is approximately parallel to the wall facing in the upper part of the retained height

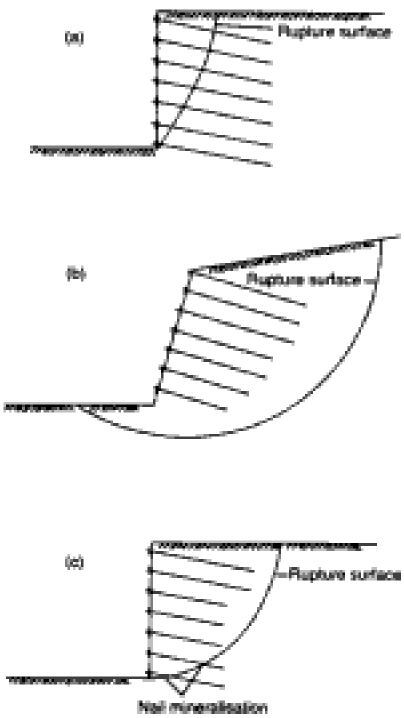


Fig. 3.9. Locations of potential failure surfaces in a nailed wall (Project Clouterre[10])

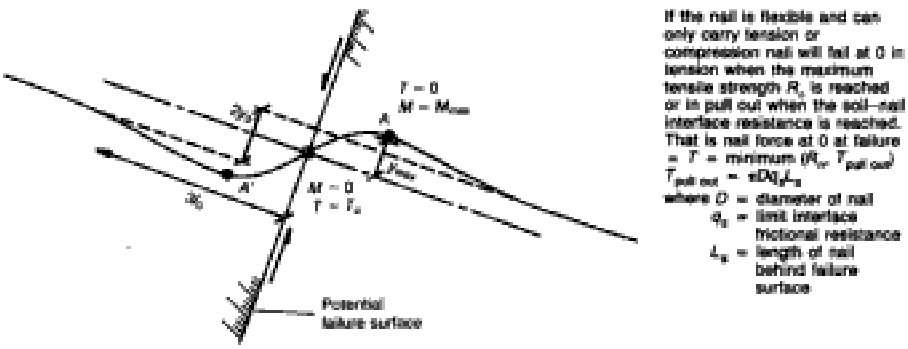


Fig. 3.10. Distortion of a flexible, long nail at the failure surface (Schlosser et al.[11])

and separates the soil mass into two zones: the active zone, where shear stresses between soil and nails are directed towards the facing; and the passive zone, where shear stresses are directed towards the inside of the soil mass beyond the potential failure surface.

The stiffness of the nails also modifies the classical Rankine earth pressure distribution against the wall facing. The soil pressures in the upper part are near K_0 earth pressure at rest, but reduce to less than K_a values lower down. For retaining walls, two principal methods have been applied, particularly in France:

(*a*) 'micro piles', where the nails are relatively long and have considerable tensile strength (200 to 500 kN), grouted in pre-drilled boreholes with relatively wide spacing (about one bar per 3 to 6 m^2)

(*b*) 'driven or fired nails', where the reinforcing nails are shorter and have less strength in tension (50 to 150 kN), driven or shot into the soil face with a closer spacing (one bar per 0·5 m^2).

Dimensions of soil nailed walls depend on the strength of the retained soil. An indication of this effect is shown in Figs 3.11 and 3.12. For the micro pile-type solution, the ratio of nail length to the height of vertical facing (*L/H*) decreases from about 1·7 to 0·5, and the cumulative length of nails from 4·5 to 1·5 m per m^2 when ϕ increases from 25° to 55°. For the driven or fired nails, the *L/H* ratio and the cumulative length of bars are nearly constant irrespective of the value of ϕ: *L/H* is approximately 0·5 and the cumulative nail length is from 11 to 12 m per m^2. The choice between micro pile and driven nails depends on cost, available equipment and the nature of the soil (in terms of ease of drilling or driving of the nails and pull-out resistance).

An important aspect of wall design is the development of adequate pull-out resistance to the nail. This resistance, which is the aggregate of friction between retained soil and the surface of the nails, varies according to installation method, grouting technique (with or without pressure) and soil type (cohesive or cohesionless); the optimum conditions are dense granular soils where dilation occurs during shearing. Where such dilatancy is restricted by the dense soil mass, high restraining frictional resistance is set up along the nail. These frictional resistances therefore vary from as high as 600 kPa for dense granular soils to less than 100 kPa for loose sands. For cohesive soils, the maximum soil/nail adhesion is less than that for granular soils and ranges from 50 to 100 kPa, reducing as the soil becomes

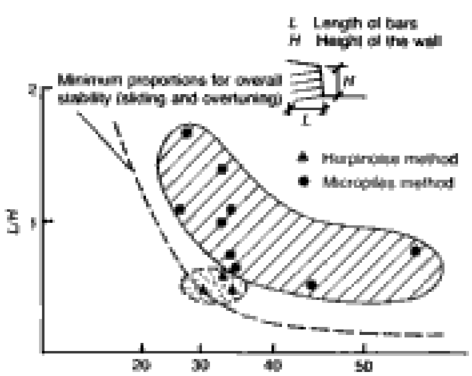


Fig. 3.11. Observed proportions of height of wall to length of nails as a function of retained soil strength (Project Clouterre[10])

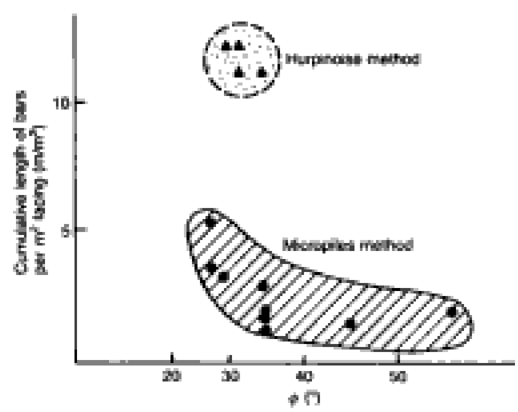


Fig. 3.12. Nailed walls: cumulative length of nails per square metre of wall face as a function of soil strength (Schlosser et al.[11])

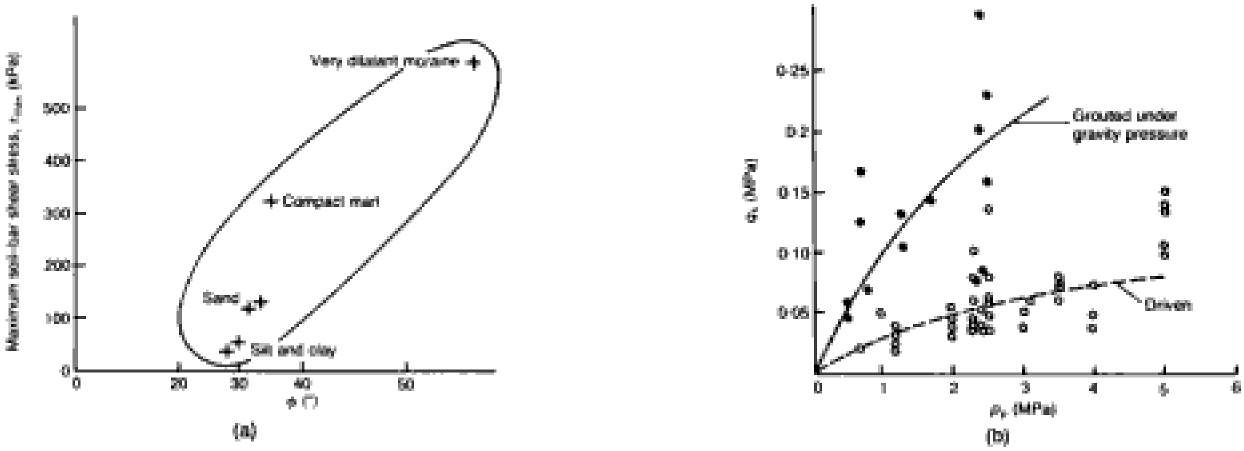


Fig. 3.13. Values of soil– nail surface friction as a function of soil strength: (a) expressed as angle of soil shearing resistance for various soils; (b) expressed as limit pressure from pressuremeter tests for grouted and driven nails (Schlosser et al.[11]*)*

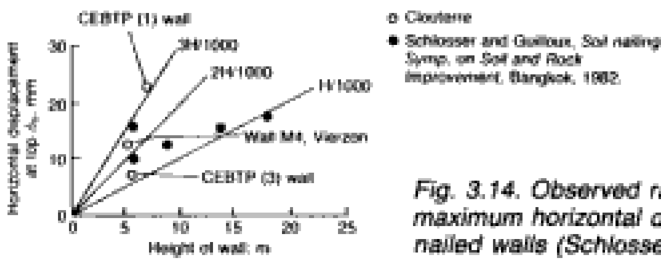


Fig. 3.14. Observed range of values of maximum horizontal displacement of nailed walls (Schlosser et al.[11]*)*

saturated. With the French preference for pressuremeter testing, a large number of pull-out tests were undertaken in the Clouterre research project and limit pressuremeter pressure and soil–nail frictional resistance were correlated (Fig. 3.13). In all cases where economical wall design is necessary, a knowledge of pull-out tests for nails is essential, and before introducing soil nailed walls into previously untried soil conditions there should be a short test programme of nail pull-outs.

Movements which occur in other reinforced soil structures (such as Reinforced Earth walls) similarly occur in nailed walls and are inherent in the method due to the relatively flexible structure produced by reinforcing soil. Displacements which occur during excavation and are largely complete thereafter vary between $H/1000$ and $H/3000$, where H is the wall height. Fig. 3.14 shows some recorded values.

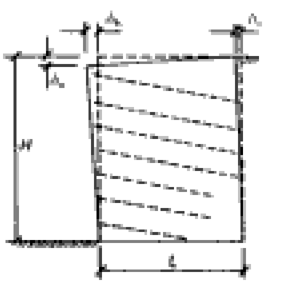


Fig. 3.15. Definitions of displacements δ_H, δ_V and δ_O (Schlosser et al.[11]*)*

Schlosser *et al.*[11] referred to three displacements, δ_H, δ_V, δ_0 (Fig. 3.15), and observed δ_h to be approximately the same as δ_V. The length λ is defined as that minimum distance from the back of the wall to the point where lateral movement δ_0 reduces to zero. The value of λ is a function of soil type (coefficient K), the inclination from the vertical of the wall η, and H, according to the empirical relationship

$$\lambda = K(1 - \tan \eta)H \tag{37}$$

The values of δ_H, δ_V and K, from the expression for λ, are summarized in Table 3.1.

Schlosser *et al.*[11] reported that corrosion should be allowed for in structures with an expected service life of more than 18 months. In France it is usual to allow a sacrificial thickness for corrosion of the nails. The total thickness is calculated so that, given the expected degree of corrosion, an adequate thickness of nail remains

Table 3.1. Displacements in nailed walls

Soil type	Weathered rocks, stiff soils	Sandy soils	Clayey soils
$\delta_H = \delta_V$	$H/1000$	$2H/1000$	$3H/1000$
K	0·8	1·25	1·5

Table 3.2. Sacrificial nail thickness allowed with respect to expected degrees of soil corrosion

Class	Soil character	Service life ≤ 18 months	Service life 1·5 to 30 years	Service life 30 to 100 years
IV	A little corrosive	0	2 mm	4 mm
III	Fairly corrosive	0	4 mm	8 mm
II	Corrosive	2 mm	8 mm	Plastic barrier
I	Strongly corrosive	Compulsory plastic barrier		Compulsory plastic barrier

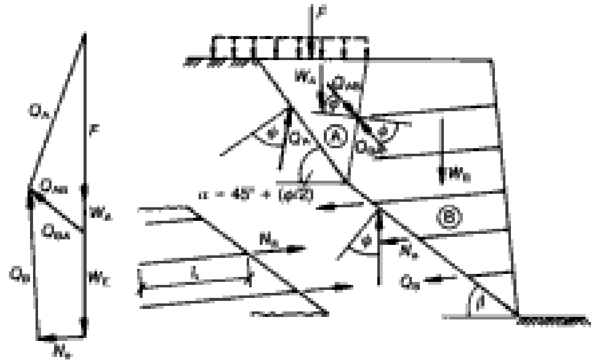
at the end of the service life. The Clouterre recommendations specify degrees of corrosion which include resistivity and soil moisture content, and these are included in Table 3.2 showing the recommended extra thicknesses of steel.

The modes of failure of a nailed wall were referred to earlier, and basically can be compared with a classical wedge failure but with modifications. In all cases the overall stability of the wall should be checked, that is, over a potential failure surface which passes to the rear of, and beneath, the nailed volume of soil to emerge at formation level in front of the wall. A conventional Bishop, $\phi' c'$ analysis should be used to check that this overall stability is adequate.

The stability of the nailed wall itself depends on the safety of the anchorage in a passive soil zone beyond the potential failure surface of the reinforced soil wedge. At the upper part of the wedge a volume of nailed soil exerts relatively high pressures towards the excavated face. The potential failure surface passes through the locus of points of maximum tension in each nail and is not the classical Coulomb surface. Nevertheless, although there is risk of over-simplification, it is recommended that the graphical analysis shown in Fig. 3.16 is used to define the maximum nail force N necessary to limit equilibrium. Trial failure surfaces for varying values of inclination β of the lower wedge are used to calculate the maximum tension in the nails N_u. An overall factor of safety against pull-out of the soil nails of 1·5 for permanent walls and 1·3 for temporary walls is regarded as adequate, where pull-out tests have been performed on a site of known, or similar, soil conditions. In addition to this stability analysis, the wall should be checked the risk of movement from published site measurements of walls in similar soil conditions. In particular, the proximity of existing structures, highways and services should be noted and estimates made of the lateral distance to the point of zero horizontal soil movement.

The design methods recommended by Clouterre, summarized by Schlosser,[11] comprise an extension of the classical limit equilibrium method (method of slices) to reinforced soils, and take into account the bending stiffness and shear resistance of the nails. It is the mode of failure of the stiff nail system (micro piles, drilled and grouted) which justifies this more sophisticated analysis. Stiffer nails can break in bending or shear at the potential failure surface; the soil below the nail can fail in bearing or the nail can pull out. Schlosser also recommended partial factors of safety to be used in an overall stability analysis.

The design of the facing depends on the tension in the nails at the facing T_0 and uniform soil pressure acting on the facing. T_0 can be calculated directly using recommended values of T_0/T_{max} given in the Clouterre report, where T_{max} is the maximum tension in the nail in service. Clouterre advises the following relationships:



Failure mechanism of nailed wall.

$\Sigma N_a = T_m \times l_i$

where T_m is the friction per unit length of nail and l_i is the nail length beyond the slip surface.

Factor of safety FS $\eta = \dfrac{\Sigma N_a}{N_e}$

where ΣN_a is the sum of maximum tensions arising in the soil nails at the crossing points with failure surface.

N_e is the nail force necessary to the limit equilibrium.

Fig. 3.16. Graphical analysis to determine the factor of safety of wall stability for nail length

$$
\begin{aligned}
T/T_{max} &= 0{\cdot}5 + \frac{(S - 0{\cdot}5)}{5} && \text{for } 1\text{ m} \leq S \leq 3\text{ m} \\
T_0/T_{max} &= 0{\cdot}6 && \text{for } S \leq 1\text{ m} \qquad (38) \\
T_0/T_{max} &= 1{\cdot}0 && \text{for } S \geq 3\text{ m}
\end{aligned}
$$

where $S = \max(S_V, S_H)$ in metres and T_{max} may be estimated as the ultimate nail pull-out force used in the design. S_V is nail vertical spacing, S_H is nail horizontal spacing.

Methods of installation and equipment

The stages in construction of the nailed wall are:

(*a*) excavation at the face of the wall in layers in 1 to 1·5 m deep increments defined by anchor spacing; trimming to face of wall, to line and verticality (or batter)
(*b*) application of gunite (shotcrete) to the exposed soil face
(*c*) gunite to be steel mesh-reinforced; installation of nails by drilling and grouting or driving; making good gunite perforation by nails (where conditions allow, it may be preferred to install nails prior to gunite application).

Excavation adjacent to the wall face would typically be made by backhoe with the assistance of a wheeled front-end loader to prepare a temporary platform at each successive excavation stage. Where the wall face has a batter (a slight batter of 5° to 10° is not unusual), more work is involved trimming to line and batter with some hand work. The application of gunite (shotcrete) is by hand-held hose from a small portable mixer/compressor unit. Water is applied at the nozzle. Typical gunite thicknesses are 100 to 150 mm applied in successive layers approximately 50 to 75 mm thick. Reinforcement mesh is pinned to the first layer of the hardening concrete. The method of inserting the nail into the soil face will have been decided prior to beginning the excavation. There are two alternatives:

(*a*) an unprotected metal rod or angle section driven from the soil face
(*b*) a steel rod section placed within a drilled hole and grouted at gravity, or higher, grout pressure.

Another method of nail installation has recently been introduced into the UK whereby steel bars up to 6 m long are 'shot' into the soil surface. It may be presumed that only in soft soils would such a penetration be achieved, and it may be concluded that the extent of penetration required for relatively deep excavations, more than 10 m deep, may be difficult to achieve.

Option (*b*) requires the installation of steel rod nails, typically 32 to 50 mm dia. bars within bores 100 to 150 mm in diameter. The bores, as for option (*a*), would be horizontal or with a small declination, and would be installed using rotary air or water flush. The rig required for drilling the relatively short bores would be a tracked hydraulic rig similar to that used for ground anchor installation. Depending on soil conditions, it would be necessary to case these relatively short bores, the casing being withdrawn as grout is introduced. Relatively low grout pressures would be used (less than 5 bar). Although the drilled and grouted nail solution uses traditional 'ground anchor' expertise, and the nails are spaced at wider centres than the driven nail option, the cost of drilled and grouted nails is higher than the ungrouted driven nail solution.

Soil retention: further wall constructions

Three further methods of soil retention using flexible wall construction deserve brief mention: ladder walls, gabions and crib walls.

Ladder walls

While soil nailing provides an integral soil and reinforcement mass, an earlier installation, the ladder wall, used a facing tied back into fill with rods transferring their load in friction part into the fill and part to an anchorage block forming a composite inclusion. Invented by Andre Coyne in 1926, the method was subsequently used for retaining walls and dam construction in France. Walls up to 20 m high were built using the method for river training walls, to spillway channels and embankment cofferdam structures. The method produced a flexible construction which allowed movements in the wall foundation (for example, due to karstic conditions or due to seismicity) without detriment to the wall itself. Fig. 3.17 shows a cross-section of a dam construction by ladder wall where the anchorage forms the protection to the downstream slope of the dam.[12]

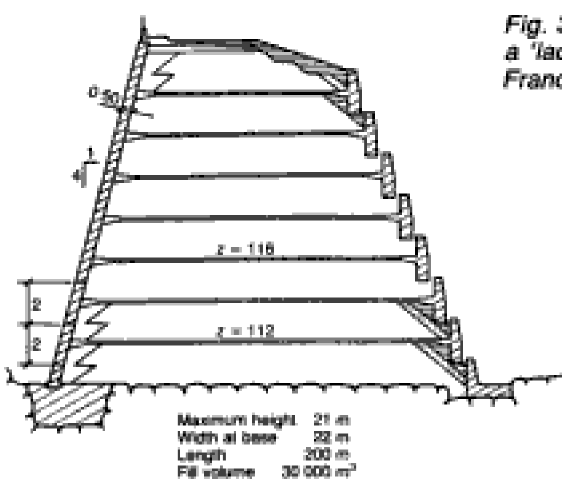


Fig. 3.17. Cross-section of a 'ladder wall' at Conqueyrac, France (Chabal et al.[12]*)*

Gabions

The use of gabions is a traditional method of retaining wall construction which has some advantages over masonry or reinforced concrete wall construction:

(*a*) gabion construction does not require skilled labour in its construction
(*b*) the permeability of the gabions allows both free drainage and soil retention
(*c*) the gabion wall is very flexible and allows considerable distortion before collapse
(*d*) large walls can be built relatively cheaply where the source of stone is local to the construction site.

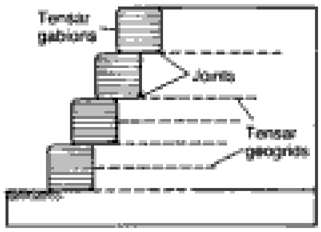


Fig. 3.18. Tailed gabion wall constructed with Tensar polymer grid cages and Tensar geogrid tails to form a Reinforced Earth structure (courtesy of Netlon)

The gabion wall can be constructed with a vertical, stepped or sloping front face and is designed as a conventional gravity wall against overturning, sliding and excessive base pressure. The steel gabion cage itself can be protected by galvanizing or plastic coating, or the whole cage may be made from polymer grid material. Typical, economical gabion wall heights range from 5 to 10 m depending on soil conditions. As an aid to stability it is usual to tilt the face of the wall at a batter of approximately 1 in 10. Overall economy may be improved by using a front gabion unit tied to a Reinforced Earth tendon, in steel or high-strength polymer, as shown in Fig. 3.18.

The Maccaferri company manufactures a patented, combined gabion box and reinforced soil system using either zinc- or PVC-coated wire mesh. The facing units, between 0·5 and 1·0 m thick, can be constructed as a single element and used in either a vertical or stepped face, as shown in Fig. 3.19(*a*) and (*b*). The wire mesh units can be filled with soil inside a geotextile and hydroseeded to produce a green face (Fig. 3.19(*c*)). An example of a stone-filled Maccaferri Terramesh wall to steepen an existing slope for a housing development site in Taiwan is shown in Fig. 3.19(*d*).

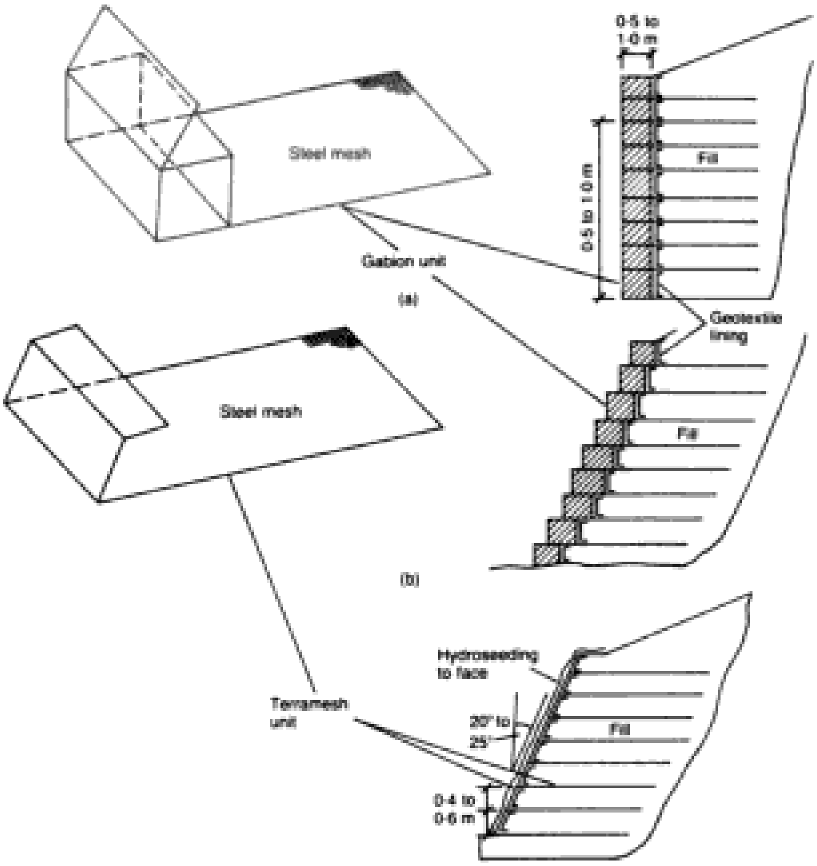
The cost of gabion construction obviously depends on the availability and cost of local stone, since the cost of the gabion cage is a small proportion of total cost. As an indication, for medium height walls where stone cost is limited to £10 per tonne delivered, the unit rate for gabion construction is approximately £40 per m^3, excluding the cost of backfill behind the gabion. For walls 8 m high, with stone costing £10 per tonne delivered, and an allowance of £60 per lineal metre for backfill, the unit cost of wall face area is approximately £110 per m^2 (at 1994 prices).

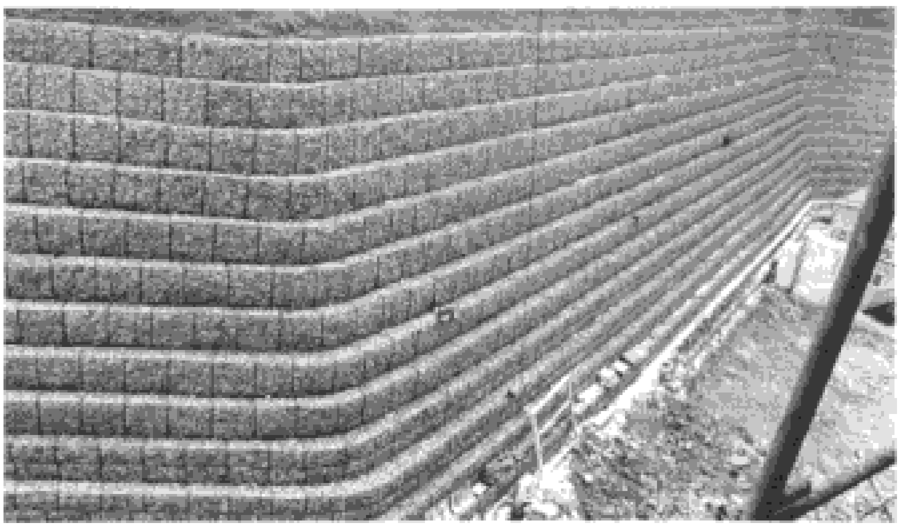
Crib walls

The use of crib walls, which were popular for road schemes in North America for some years, has been established in the UK in the last ten years. Manufacturers in the UK include Andacrib and Permacrib, and materials for the crib are either concrete or timber. Typical examples of crib wall construction using concrete and sheet steel are shown in Fig. 3.20.

The crib wall is a flexible construction and can withstand some settlement and movement without detriment. The crib wall is only applicable in dry subsoil conditions or where the rear of the crib can be drained to prevent discharge through the face. The crib itself is backfilled with granular material (old railway ballast is typical), but graded granular material is necessary behind the crib to avoid loss of fines into the crib. Typical wall heights which can be built by proprietary crib walls are between 3 and 9 m, depending on the number of units in wall thickness. Composite crib and anchor constructions have been successfully constructed in Europe using vertical reinforced concrete anchor beams between sections of crib wall.

Daley and Thomson[13] described practical design and construction relating to crib walls in Hong Kong, and included a description of a major crib wall constructed at Tsing Yi in the late 1960s. The wall, shown in cross-section in Fig. 3.21, was

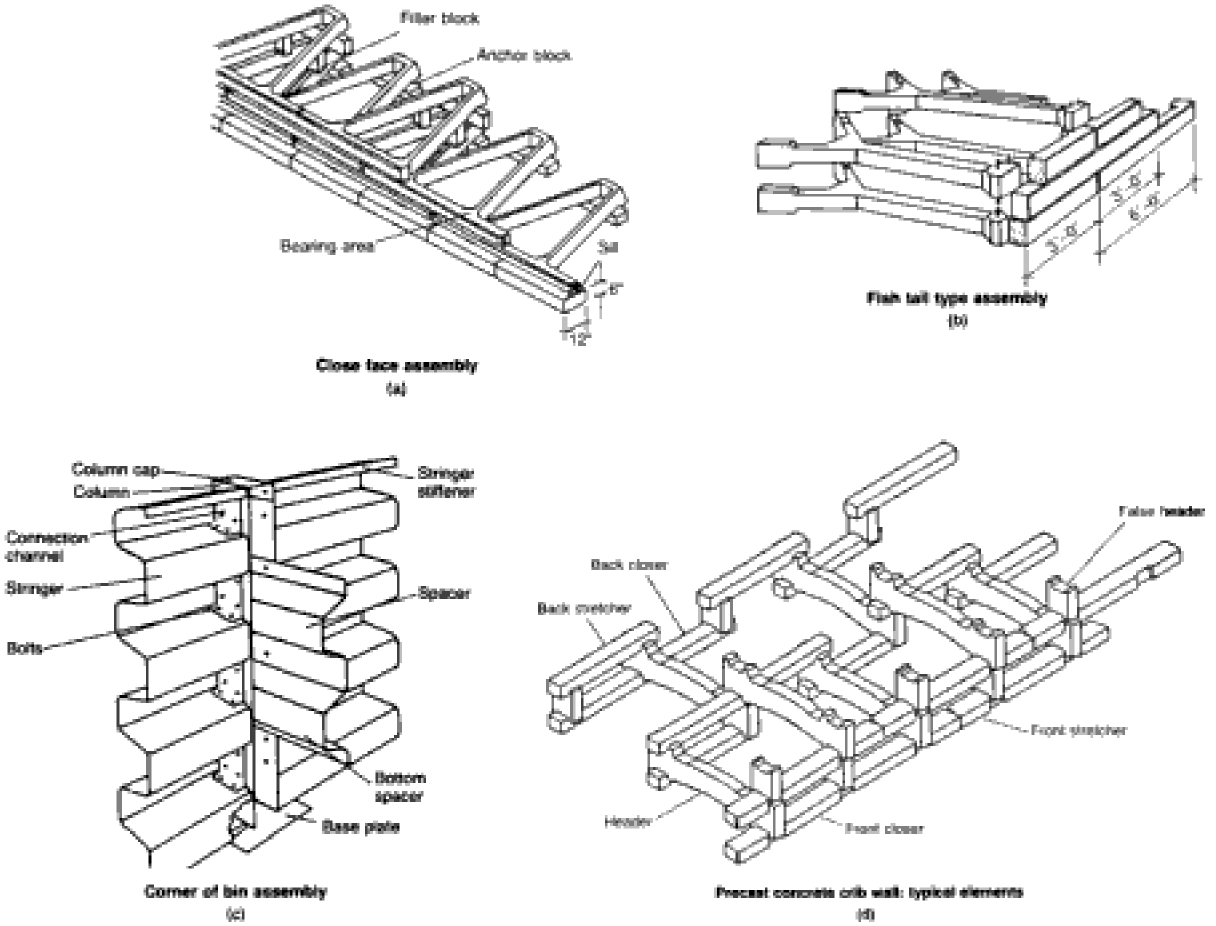




(d)

Fig. 3.19. Combined gabion box and reinforced soil system known as Terramesh: (a) vertical face; (b) stepped face; (c) with a hydroseeded face; (d) example of a stone-filled Terramesh wall in Taiwan (courtesy of Maccaferri)

used to stabilize an existing slope with slope angles varying between 50° and 55°. The crib wall was 75 m long and up to 11 m high. At its highest the wall is concave in plan, but is convex at one end where the fill height is lower. The considerable thickness of the wall is noteworthy.



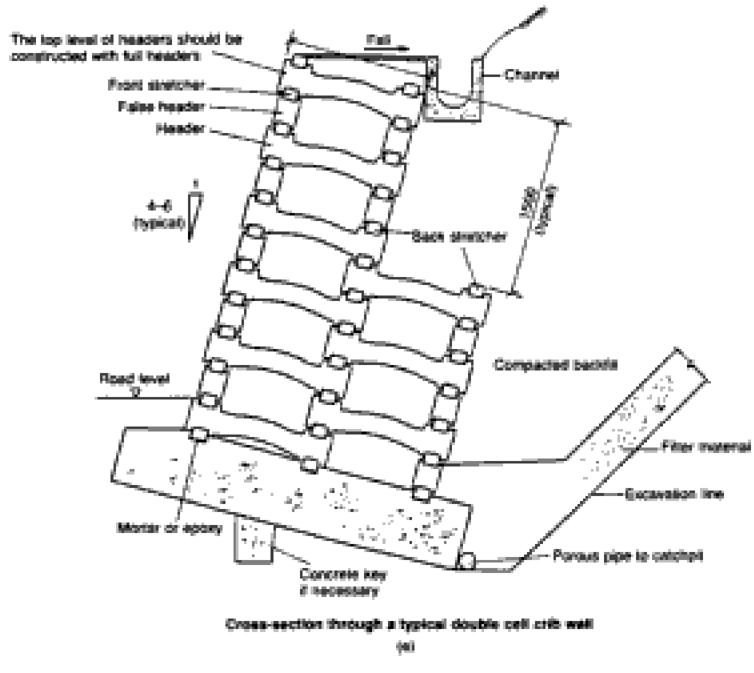


Fig. 3.20. Examples of crib wall construction in steel sheet and precast concrete: (a), (b) alternative close face and fish-tail type walls in precast concrete (courtesy of US Navy); (c) corner of a bin construction in pressed steel sheet; (d) typical construction of a precast concrete crib wall with an open face, double-cell width; (e) double-cell wall in cross-section (Daley and Thomson[13])

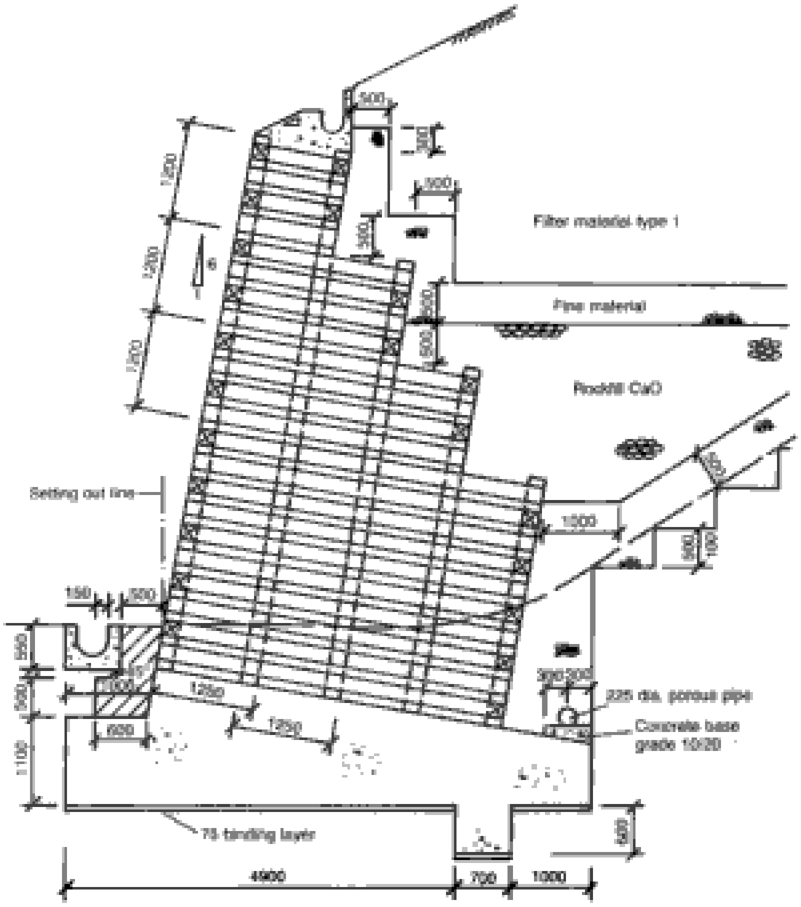


Fig. 3.21. Typical cross-section through four-cell wide crib wall in Tsing Yi, Hong Kong (Daley and Thomson[13])

Approximate current construction costs for supply and erection of a 9 m crib wall and a 6 m crib wall in reinforced concrete are £225 per m² and £185 per m² of wall area, respectively.

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4 Vertical soil support: wall construction

Options for sheeting and walling

While the intended use of a large underground excavation may well define its size, shape and location, and therefore the overall method of construction, the sheeting or walling techniques used at the periphery of these excavations may be common to a number of underground excavation types. Excavations for building basements, cofferdams for land-based industrial facilities, cofferdams for bridge-works, cut-and-cover structures and shaft construction frequently share methods for wall construction to support the peripheral subsoil and exclude groundwater. The sheeting or walling selected for a particular project may provide temporary soil support prior to the permanent substructure construction, or it may serve as temporary soil support before being incorporated into the works as the permanent means of soil retention.

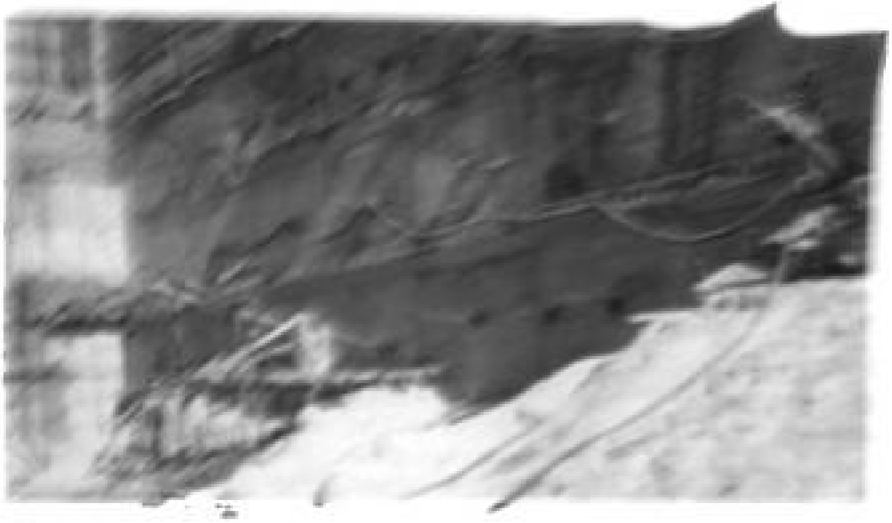
The type of peripheral sheeting used will therefore be influenced by the sub-structure construction method, and will vary geographically due to soil and groundwater conditions, proximity to the source of materials, and the skill of local contractors. The sheeting should be selected after an accurate comparison in terms of construction time and cost, although it rarely is. A typical list of available methods, which is not necessarily exhaustive for any one location, is:

- plate and anchor wall method
- Berlin wall method: vertical soldiers and horizontal laggings or reinforced concrete skin wall
- sheet piling: pitched and driven or pitched into slurry trench excavation
- contiguous bored piling
- secant piling, both hard–hard and hard–soft techniques
- soldier pile tremie concrete method (SPTC)
- diaphragm walls: reinforced concrete cast in situ
- diaphragm walls: reinforced concrete precast
- diaphragm walls: post-tensioned

Plate and anchor wall by underpinning

The use of a conventional underpinning method, casting a reinforced concrete wall in short lengths and depths along the curtilage of a basement and securing each individual wall section by ground anchor, is feasible in certain soil conditions. Fig. 4.1 shows a site in Madrid where dry, relatively dense sand subsoils allowed the excavation to proceed in short lengths without soil loss on the site boundary, each short length of underpinning being anchored before further excavation. It is not suggested that the underpinning replaces the permanent basement retaining wall.

The use of underpinning to retain soil to a basement excavation has been advanced by the practice of using ground anchors to stabilize a grouted mass or wall of subsoil. Fig. 4.2 shows a basement excavation for the Commerz Bank in Frankfurt-am-Main. The adjacent building was underpinned by cement and chemical grouting of the sandy gravel below its foundations; this block of grouted subsoil was then retained by soil anchors and steel walings below the building. The photograph



[illegible]

Fig. [illegible] Basement construction at Frankfurt am Main: anchored grouted wall underpinning and anchored Berlin wall [illegible]

also shows the use of the Berlin method to retain the subsoil to the site boundary to the street side. The single layer of anchors was used to support the wall of consolidated soil, acting as a propped gravity structure, whereas the sheeted wall was retained by a multi layer anchor system.

Figure 4.[illegible] shows the combined use of steel strutting and ground anchors to support [illegible] on a subway section in Nuremburg in Bavaria. Sodium silicate [illegible] consolidate the cohesionless quartz sands. The grouted wall [illegible] layers of ground anchors were used with three frames [illegible] The anchoring of jet-grouted soil masses is an alternative [illegible] anchor method.

Vertical soldiers and horizontal lagging, Berlin method

The use of vertical soldier piles with horizontal lagging is known as the Berlin method because it was originally used on works for the Berlin subway prior to the Second World War. The method owes much of its later popularity to three factors: the comparative cheapness of timber as a structural material; the economy of boring with power augers in certain soils; and the improved methods of anchoring that subsequently became available.

The method consists of boring holes on the wall line, typically at 2 to 3 m centres, placing vertical steel joist soldier piles within the holes and concreting the base of each joist below final formation level. Lean mix concrete is often used for this purpose. In suitable soils the steel soldier pile may be driven. As bulk excavation proceeds, horizontal lagging timbers or precast concrete units are wedged between the soldier piles. Steel section walings are placed to take the thrust from the soldier piles to ground anchors drilled at intervals along the length of the waling beam. Alternatively, each soldier pile is anchored and a waling is not needed.

In shallow excavations, the horizontal lagging timbers may be replaced by a reinforced concrete skin wall spanning the soldier piles and cast successively in short lifts as the excavation proceeds. The soldier piles may be cantilevered or propped by raking shores.

The use of the Berlin method is limited to relatively dry ground or dewatered soils which are capable of self-support as each lagging is secured. Where flow of groundwater is limited in granular soils, inclined wellpoints may be tucked between the horizontal timbering. The method is generally only used as temporary soil support and requires construction of the permanent wall by conventional methods. The very big advantage secured by the anchored Berlin wall is the extent of the clear working space to the permanent works which is unimpeded by rakers, shoring, or soil berms. Nevertheless, the method does occupy a considerable thickness of construction which remains around the site periphery without contributing to permanent soil support; more important, perhaps, the construction tolerances may be such that it may not be possible, in deep basements, to use the horizontal laggings as a back shutter to the permanent wall works. The method becomes more economical in some urban areas where the soldier piles can be placed temporarily outside the site boundary, behind the laggings, and extracted after use.

The use of the Berlin wall method may not be economical in terms of the construction width and the plan area occupied. The total width of the temporary and permanent wall becomes the sum of each of the following items:

(*a*) minimum width of the working space between rear of temporary wall and site boundary; applicable where the site boundary is, for instance, the flank wall of an adjoining property
(*b*) construction tolerance of the temporary wall: typically 1 : 100 verticality tolerance for augered piling
(*c*) width of the temporary wall
(*d*) working space for the permanent wall, where the Berlin wall is not used as a back shutter
(*e*) width of the permanent wall together with construction tolerance.

Fig. 4.3. Alternate layers of ground anchors and steel strutting used to support deep excavation between chemically-consolidated sand walls, Nuremburg (courtesy of Bauer)

Figure 4.4 shows the total width of the construction.

The method realizes the full consequences of poor workmanship in subsidence, excavation and instability risks. Peck[1] showed alternative arrangements for securing lagging to soldiers (Fig. 4.5). Poor contact between laggings and the excavated soil face may induce movement, and Peck recorded that although use of the method shown in Fig. 4.5(*b*) allows the soldier piles to be incorporated within the permanent wall reinforcement, loss of soil during installation has caused settlements three times those at a basement where the method shown in Fig. 4.5(*a*) was used in similar conditions. In the UK, the practice of allowing unsecured laggings to slip between the soldier pile flanges progressively as the bulk excavation

Fig. 4.6 Deep excavation through sandy silts and silty sands supported by anchored Berlin wall and dewatering by deep wells, Hamburg, Germany (courtesy of Bauer)

Fig. 4.7 Dual method: Berlin wall on [illegible] car park, [illegible] (courtesy of Bauer)

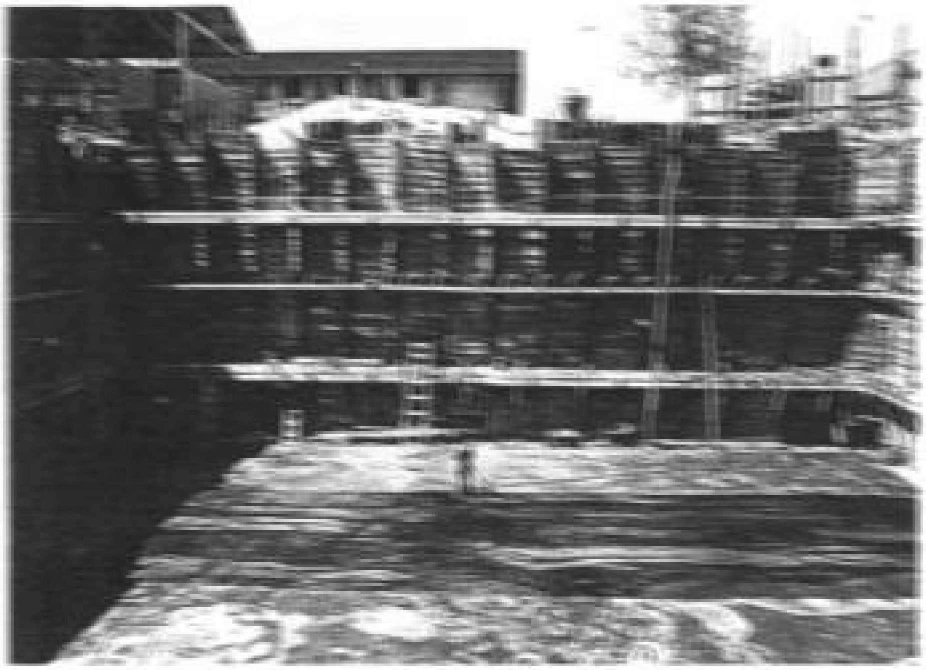


Fig. 4.8 Berlin wall method using steel soldier piles and horizontal timber laggings with wellpoint dewatering for a sewage [illegible] in Al-Khobar, Saudi Arabia (courtesy of Bauer)

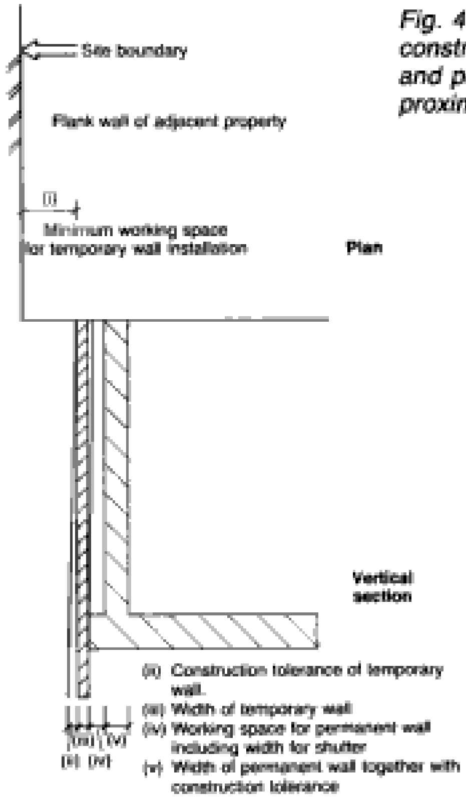


Fig. 4.4. Overall thickness of basement wall construction including means of temporary and permanent soil support, showing proximity to site boundary

continues is a particular example of poor workmanship with an inherent risk of soil movement and subsidence.

Figure 4.6 shows an anchored Berlin wall with good tolerances and carefully-wedged horizontal timber laggings. This excavation, at Raschplatz subway station in Hannover, was 24 m deep and the subsoil was silty sand varying to a sandy silt. Fig. 4.7 also shows the use of anchored soldier piles with horizontal laggings, but with dual methods of construction. The upper part of the walls had steel H-beams placed into pre-drilled holes, with concrete slabs spanning horizontally; the lower part was formed by bored piles by Benoto rig, with horizontal spanning concrete slabs. The maximum depth of the excavation was 28 m, retained by eight layers of anchors. Note the economy in ground anchors achieved by diagonal cross bracing between soldier piles near the basement corner.

Although the method is only feasible in relatively dry subsoils, if the soils allow economical soil anchoring and dewatering, the Berlin wall finds use in apparently uneconomical conditions. Fig. 4.8 shows an excavation for a sewage water pumphouse at Al Khobar, Saudi Arabia. The groundwater drawdown from 1 m below ground level to a depth of 11 m was by three-stage wellpointing and deep wells in a sandy subsoil.

Particular attention is drawn to the risk of failure of anchored Berlin walls by the vertical component of the anchor force transferring to the base of the soldier pile. Vertical settlements of the wall require consideration. The location of anchorages at re-entrant corners also requires attention where interaction may occur due to the proximity of anchors from adjoining walls at the re-entrant angle.

More recently, in the UK the use of vertical soldiers, raker support and horizontal spanning reinforced concrete skin walls have found favour for shallow basement excavations in London. Excavation by backacter for soldiers and the use of trench boxes followed by backfilling and re-excavation for successive skin wall pours

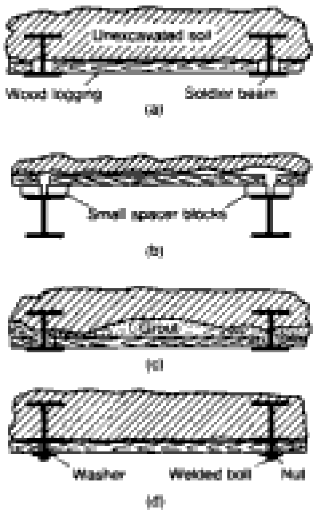


Fig. 4.5. Methods of transferring earth pressure from lagging timbers to soldier piles in Berlin walls: (a) lagging wedged against inside flanges of soldiers; (b) lagging set behind flanges of soldiers; (c) grout or mortar filling between lagging and soil; (d) contact sheeting secured to face of soldiers (Peck[1])

Fig. 4.6. Deep excavation through sandy silts and silty sands supported by anchored Berlin wall and dewatered by deep wells, Hannover subway (courtesy of Bauer)

Fig. 4.7. Dual method Berlin wall for multi-storey car park, Zurich (courtesy of Bauer)

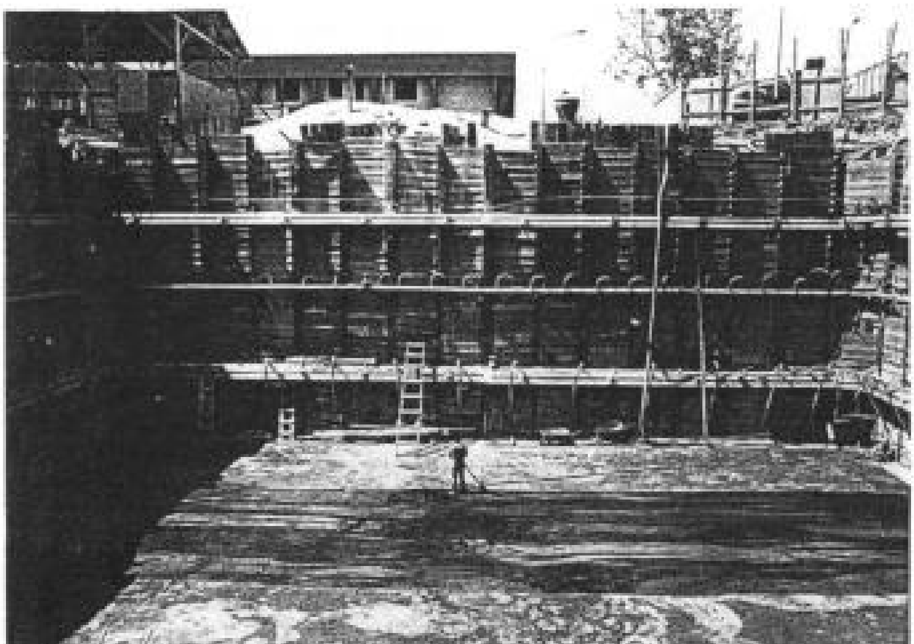


Fig. 4.8. Berlin wall method using steel soldier piles and horizontal timber laggings with wellpoint dewatering for a sewage pumphouse at Al-Khobar, Saudi Arabia (courtesy of Bauer)

have proved economical where groundwater levels are low and fill or gravel conditions are reasonably dense. In parts of Germany a particularly low price is obtained by the use of shotcrete between and over the vertical soldiers in lieu of horizontal laggings.

Sheet piling

The economical use of sheet piling in basement construction is influenced by five factors.

(*a*) The ability to withdraw the piling after use. Where city sites require maximum occupation of the site area by the basement, it is not unusual for pile extraction to be precluded by the lack of working space for extraction equipment, cranes and handling space. The difficulties of extracting piles at an urban basement site can therefore minimize their re-use despite the use of a back shutter or sheeting lining to separate the permanent wall construction from the sheet piling.

(*b*) Depth of the basement. On urban sites the transporting, handling, pitching and driving of long piles can become onerous tasks. The storage of long piles on a typical city site may become a serious logistical problem; perhaps the method would fall further into disfavour if the actual cost of pile storage, handling and re-handling, stacking and re-stacking were accurately known. A typical use of sheet piles for an urban basement construction is shown in Fig. 4.9. This basement extension to a telephone exchange in North London was built in 1969. Subsoil conditions consisted of London clay from ground level to a considerable depth below final formation level, although the upper 15 m section was weathered. Telecommunications equipment sensitive to vibration was nearby, so hydraulic pile installation equipment was specified. This equipment was ideally suited to the subsoil conditions, allowing pile installation to proceed without noise (apart from the power generator) and vibration. The use of ground anchors and temporary embankments allowed construction works to proceed largely unimpeded from strutting and raking shores. Unfortunately, the site working

Fig. 4.9. Anchored sheet piles in London clay for a telephone switching centre, London

space did not allow the extraction of any of the temporary sheet piles to the basement periphery.

(*c*) Soil conditions and the ease of pile installation. The section of sheet pile to be used will depend on the requirements of flexural strength and the strength to resist driving stresses. Design calculations should therefore be reviewed in the light of anticipated driving conditions. Costs calculated by Potts and Day[7] for sheet piling, diaphragm walls and secant piles designed by modern methods showed favourably for the sheet pile wall used in two underpasses and a basement project. Their comparisons may prove to be inaccurate due to underestimates of the section required for practicality of driving. Use of high-frequency vibratory equipment to install sheet piles in cohesionless soils and hydraulic pile hammers in clays allows minimal noise levels during pile installation, but increased vibration or reduced output result directly from any natural or man-made obstructions. Where obstructions seriously impede progress, pre-boring may be needed. When hard driving through obstructions occurs there is risk of torn clutches (perhaps occurring below final formation level) causing inadequate groundwater cut-off and severe reduction in pile flexural strength.

(*d*) Noise and vibration. The use of acoustic covers to pile frames has recently allowed sheet pile installation in built-up areas where it would not have been possible previously, but the vibration caused by such methods may not be permitted, particularly in cohesionless soils or sheet piling toed into soft rocks or dense soils. Existing buildings and facilities, especially medical facilities, may be sensitive to disturbance by vibration, and alternative methods such as diaphragm walling may be necessary in such circumstances. Environmental legislation may severely limit working hours in urban areas.

(*e*) Incorporation of the sheet pile wall into the permanent works. Where permanent groundwater levels are low, it may be possible to use the sheeters as part of the permanent wall, with suitable allowance for corrosion risk. Available sheet pile sections are shown in Fig. 4.10(a) and (b).

Combined walls

Where soil and groundwater conditions impose high loads on sheet piles, or where permanent construction requires the minimum number of bracing frames, it may be necessary to stiffen and strengthen sheet pile sections to withstand the higher bending moments induced in the sheeting. The methods available are:

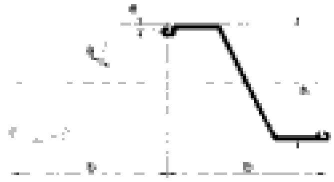
(*a*) The use of combined walls. Steel sheet pile box sections or tubular steel members are introduced at regular intervals along the length of the wall. The combined section is assumed to span vertically and the equivalent section modulus per linear metre of wall is calculated on the basis that the deflexions of the king piles and the intermediate sheet piles are similar. Combinations of sheet piles, both U and Z sections, with intermediate king piles are shown in Fig. 4.10c and d.

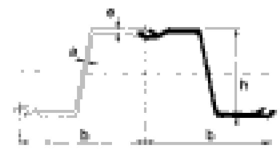
(*b*) The use of steel sheet pile sections reinforced with steel beams, such as the Peine pile, by Arbed, Fig. 4.10e.

(*c*) The use of jagged walls: sheet piles with crimped interlocks and joined by double interlock joint units, shown for U and Z pile sections in Fig. 4.11. For walls with anchoring or bracing support, stiffeners are required to the jagged wall units at support levels.

(*d*) The use of steel sheet piles, either singly or in doubles, at regular intervals with steel tubular piles, in a similar way to the use of box pile sections.

An example of the use of a combined sheet pile and tubular steel section wall is shown in Fig. 4.12. The cofferdam, built on the approach to the River Medway crossing at Rochester, UK, was also used to fabricate the submerged tube tunnel



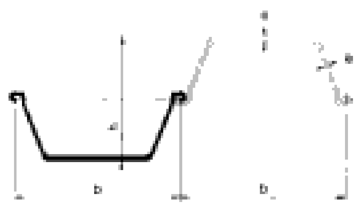


Section	Width	Height	Thickness		Sectional Area	Mass		Section modulus	Moment of inertia
	b mm	h mm	e mm	a mm	cm²/m	kg/m of single pile	kg/m² of wall	cm³/m	cm⁴/m
AZ 13	670	303	9,5	9,5	137	72,0	107	1 300	19 700
AZ 18	630	380	9,5	9,5	150	74,4	118	1 800	34 200
AZ 26	630	427	13,0	12,2	198	97,8	155	2 600	55 510
AZ 36	630	460	18,0	14,0	247	122,2	194	3 600	82 800

All AZ sections may be rolled up or down by 1.0 mm.

BZ 7	550	190	8,0	8,0	116	51,0	93	750	7 160
BZ 32 *	500	352	19,0	11,5	265	103,8	208	3 200	56 220
BZ 37 *	500	360	20,0	12,0	298	117,0	234	3 650	65 650
BZ 42	500	354	24,0	14,0	345	135,3	271	4 200	73 920

All BZ sections may be rolled up or down 0.5 mm.
* To be replaced by AZ 36 in 1994.



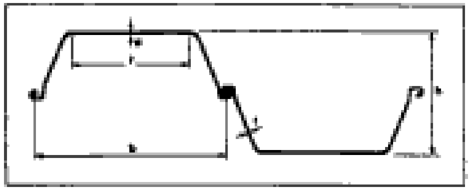
Section	Width	Height	Thickness		Sectional Area	Mass		Section modulus	Moment of inertia
	b mm	h mm	e mm	a mm	cm²/m	kg/m of single pile	kg/m² of wall	cm³/m	cm⁴/m
PU 6	600	226	7,5	6,4	96	45,3	75	600	6 780
PU 8	600	280	8,0	8,0	116	54,5	91	830	11 610
PU 12	600	360	9,8	9,0	140	65,9	110	1 200	21 600
PU 16	600	380	12,0	9,0	159	74,7	124	1 600	30 520
PU 20	600	430	12,4	10,0	180	84,3	141	2 000	43 000
PU 25	600	452	14,2	10,0	200	94,1	157	2 500	56 500
PU 32	600	452	19,5	11,0	243	114,6	191	3 200	72 290

L 2 S	500	340	12,3	9,0	177	69,7	139	1 600	27 200
L 3 S	500	400	14,1	10,0	201	78,9	158	2 000	40 010
L 4 S	500	440	15,5	10,0	219	86,2	172	2 500	55 010

JSP 2	400	200	10,5	-	153	48,0	120	874	8 740
JSP 3	400	250	13,0	-	191	60,0	150	1 340	16 800

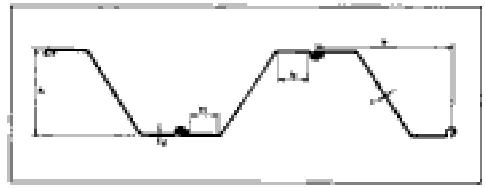
(a)

Fig. 4.10 (a) Properties of steel sheet piles as provided by Arbed —note that BZ sections have double grip interlocks whereas the AZ series have similar interlocks to Larssen U-shaped piles (courtesy Arbed)



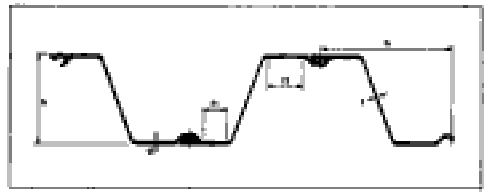
LX and Larssen Steel Sheet Piling

Section	b mm (nominal)	h mm (nominal)	d mm	t mm (nominal)	f Flat of Pan mm	Sectional Area cm²/m of wall	Mass kg per linear metre	Mass kg/m² of wall	Combined Moment of Inertia cm⁴/m	Section Modulus cm³/m
LX8	600	310	8.2	8.0	250	116	54.6	91.0	12861	830
LX12	600	310	9.7	8.2	386	136	63.9	106.4	18723	1208
LX16	600	380	10.5	9.0	365	157	74.1	123.5	31175	1641
LX20	600	430	12.5	9.0	330	177	83.2	138.6	43478	2022
LX25	600	450	15.6	9.2	330	200	94.0	156.7	56824	2525
LX32	600	450	21.5	9.8	328	242	113.9	189.8	72028	3201
6W	525	212	7.8	6.4	331	108	44.7	85.1	6459	610



FX Steel Sheet Piling

Section	b mm (nominal)	h mm (nominal)	d mm	t mm (nominal)	f1 mm (nominal)	f2 mm (nominal)	Sectional Area cm²/m of wall	Mass kg per linear metre	Mass kg/m² of wall	Combined Moment of Inertia cm⁴/m	Section Modulus cm³/m
FX13	675	300	9.5	9.5	127.7	154.5	137.8	73.1	108.3	19693	1313
FX18	675	380	9.5	9.5	135.4	160.3	147.6	78.2	115.9	34201	1800
FX26	675	430	13.2	12.2	127.1	146.6	194.9	103.3	153.0	55963	2603
FX36	675	460	18.0	14.0	148.8	158.3	244.8	129.7	192.2	82915	3605



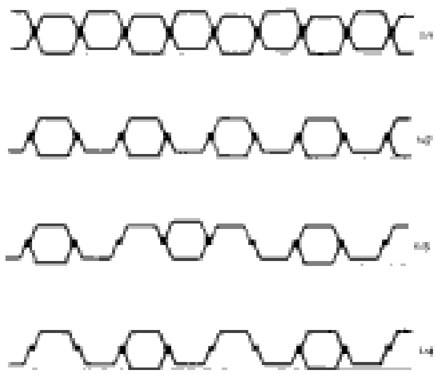
Frodingham Steel Sheet Piling

Section	b mm (nominal)	h mm (nominal)	d mm	t mm (nominal)	f1 mm (nominal)	f2 mm (nominal)	Sectional Area cm²/m of wall	Mass kg per linear metre	Mass kg/m² of wall	Combined Moment of Inertia cm⁴/m	Section Modulus cm³/m
1BXN	476	143	12.7	12.7	78	123	166.5	62.1	130.4	4919	688
1N	483	170	9.0	9.0	105	137	126.0	47.8	99.1	6048	713
2N	483	235	9.7	8.4	97	149	143.0	54.2	112.3	13513	1150
3N	483	283	11.7	8.9	89	145	175.0	66.2	137.1	23885	1688
3NA	483	305	9.7	9.5	96	146	165.0	62.5	129.6	25687	1690
4N	483	330	14.0	10.4	77	127	218.0	82.4	170.8	39831	2414
5	426	311	17.0	11.9	89	118	302.0	100.8	236.9	49262	3168

(b)

Fig. 4.10 (b) Properties of steel sheet piles produced by British Steel — note that FX sections have similar inter-locks to LX and Larssen 6W sections (courtesy British Steel)

Sheet piles walls with reinforcing box piles



Type of reinforcement

The reinforcement may be :

1. On the wall height :
– over the total height : reinforcing box piles
– partially : forming sheet piles with changing inertia by welding especially prepared shorter piles onto it.

2. In the wall length :
– in total : reinforcement 1/1
– partially : reinforcement 1/2, 1/3, 1/4.

Characteristics of some combinations

Section	1/1 Mass kg/m²	Section modulus cm³/m	1/2 Mass kg/m²	Section modulus cm³/m	1/3 Mass kg/m²	Section modulus cm³/m	1/4 Mass kg/m²	Section modulus cm³/m
PU 12	220	2 795	165	1 560	146	1 645	137	1 440
PU 16	249	3 695	187	2 065	166	2 195	156	1 920
PU 20	281	4 570	211	2 565	187	2 730	176	2 395
PU 25	314	5 645	235	3 145	209	3 395	196	2 980
PU 32	382	7 245	287	3 995	255	4 345	239	3 605
L 2 S	279	3 735	209	2 110	186	2 195	174	1 920
L 3 S	316	4 640	237	2 620	210	2 740	197	2 405
L 4 S	345	5 720	259	3 215	230	3 415	215	2 995

Combinations

Built up U-Box Piles - U-Sections

The use of triple intermediary piles is subject to a careful check of the soil conditions.

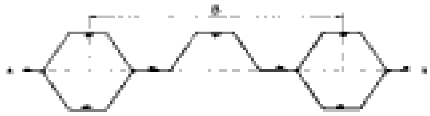
Section	Combination	Intermediate sheet pile	System width mm	Mass (kg/m²) Length of intermediates 100 %	70 %	Section modulus cm³/m	Moment of inertia cm⁴/m
L 3 S / 600 x 12	1/4	PU 12	2 300	**204**	**175**	**3 975**	208 100
PU 20 / 600 x 12	1/4	PU 12	2 400	**200**	**175**	**4 130**	221 900
L 3 S / 700 x 12	1/4	PU 12	2 300	**212**	**185**	**4 530**	259 600
PU 20 / 700 x 12	1/4	PU 12	2 400	**208**	**183**	**4 680**	275 100
L 4 S / 600 x 12	1/4	PU 12	2 300	**210**	**184**	**4 400**	239 100
PU 25 / 600 x 12	1/4	PU 16	2 400	**227**	**199**	**5 035**	301 500
L 4 S / 800 x 14	1/4	PU 16	2 300	**249**	**220**	**5 830**	375 000
PU 25 / 800 x 14	1/4	PU 16	2 400	**245**	**217**	**5 805**	376 500
PU 32 / 600 x 12	1/4	PU 16	2 400	**244**	**218**	**6 165**	369 900
PU 32 / 800 x 14	1/4	PU 16	2 400	**262**	**234**	**7 040**	457 500

Statical values of other combinations on request.

(c)

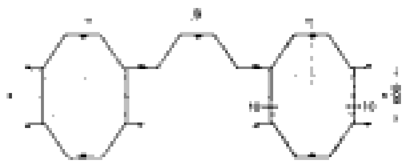
Fig. 4.10 (c) Combination piles: sheet pile properties of combination section walls with box piles using U-shaped sections (courtesy Arbed)

Combinations
AZ Box Piles - AZ Sheet Piles



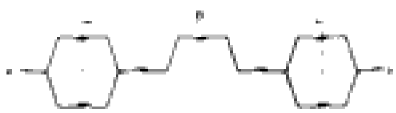
Section	Dimension B mm	Mass Length of intermediates 100% kg/m²	60% kg/m²	Moment of inertia cm⁴/m	Section modulus cm³/m
CAZ 13 / AZ 13	2 680	147	130	60 910	2 600
18 / AZ 13	2 600	156	134	95 600	2 910
18 / AZ 18	2 520	160	138	105 560	2 795
26 / AZ 13	2 600	168	146	151 240	3 530
26 / AZ 18	2 520	166	173	162 660	3 795
36 / AZ 13	2 600	221	188	217 030	4 700
36 / AZ 18	2 520	228	208	230 540	4 980

Combinations
Built up AZ Box Piles - AZ Sheet Piles



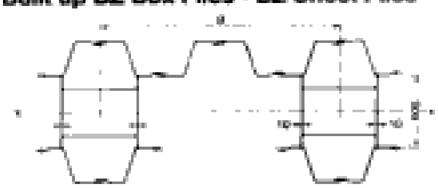
Section	Dimension B mm	Mass Length of intermediates 100% kg/m²	60% kg/m²	Moment of inertia cm⁴/m	Section modulus cm³/m
CAZP 13 / 800 × 10 / AZ 13	2 680	196	175	322 060	5 340
18 / 800 × 10 / AZ 13	2 600	208	184	457 330	5 865
18 / 800 × 10 / AZ 18	2 520	215	191	426 660	6 280
26 / 800 × 10 / AZ 13	2 600	240	220	583 150	8 000
26 / 800 × 10 / AZ 18	2 520	252	228	608 260	8 395
36 / 800 × 10 / AZ 13	2 600	289	257	777 050	10 235
36 / 800 × 10 / AZ 18	2 520	298	267	808 340	10 505

Combinations
BZ Box Piles - BZ Sheet Piles



Section	Dimension B mm	Mass Length of intermediates 100% kg/m²	60% kg/m²	Moment of inertia cm⁴/m	Section modulus cm³/m
CBZ 7 / BZ 7	2 200	125	106	21 380	1 115
32 / BZ 32	2 300	279	238	168 210	4 665
37 / BZ 37	2 300	313	266	186 890	5 280
42 / BZ 42	2 300	362	306	216 600	6 120

Combinations
Built up BZ Box Piles - BZ Sheet Piles



Section	Dimension B mm	Mass Length of intermediates 100% kg/m²	60% kg/m²	Moment of inertia cm⁴/m	Section modulus cm³/m
CBZP 7 / 600 × 10 / BZ 7	2 200	182	163	211 475	4 215
32 / 600 × 10 / BZ 32	2 300	366	317	702 140	10 770
37 / 600 × 10 / BZ 37	2 300	398	361	786 160	12 080
42 / 600 × 10 / BZ 42	2 300	403	366	912 040	13 060

(d)

Fig. 4.10 (d) Combination pile walls with box piles using Z-shaped sections (courtesy Arbed)

units for the crossing. The depth of the cofferdam at the river entrance was approximately 17 m, and it was necessary to provide only one cofferdam frame as high as possible to accommodate the height of the units passing under the frame at the time of flooding the cofferdam and floating the units to their final location in the river crossing. Due to the flexural strengths required from the sheet piles, a combined section of Larssen sheeters and 1·2 m dia. steel tubes was used, the pile interlock being continuously welded to the steel tube at its junction with the sheeter. The construction sequence for driving the wall through alluvial deposits into chalk required extremely small verticality and positional tolerances for the steel tubes. The tubes, driven by vibrators from either river craft or on land, were installed prior to the sheeters using either leaders or a mechanical template device. Considerable care was required to ensure that the intermediate sheeters, driven later, clutched for this whole length (22·5 m) between the 30 m long tubes.

Combination 22/13
Combination 24/11

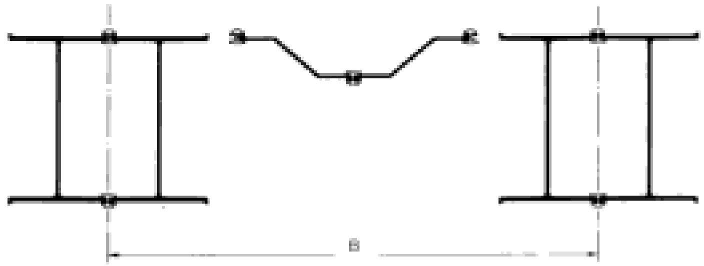


Figure shows HZ 775 A, ZH 9,5/12,0 Scale 1:30

Section	B cm	Properties per meter of wall [5]) Sectional area cm²/m	Moment of inertia cm⁴/m	**Section modulus cm³/m**	Mass of combination with intermediary sections ZH 9,5 $I_{ZH} = 60\,\%\,I_{HZ}$ kg/m²	ZH 9,5 $I_{ZH} = I_{HZ}$ kg/m²	ZH 12,0 $I_{ZH} = 60\,\%\,I_{HZ}$ kg/m²	ZH 12,0 $I_{ZH} = I_{HZ}$ kg/m²
HZ 575 A	206,5	306,7	145940 *159200*	**5075** ***5100***	211	240	218	251
HZ 575 B	206,5	324,5	160910 *174280*	**5560** ***5580***	226	255	232	265
HZ 575 C	206,5	348,3	177920 *191400*	**6105** ***6125***	244	273	250	284
HZ 575 D	206,5	376,0	197720 *214540*	**6735** ***6765***	265	295	271	306
HZ 775 A	206,5	362,3	308380 *334070*	**7960** ***8070***	255	284	261	294
HZ 775 B	206,5	380,1	336410 *361240*	**8610** ***8715***	269	298	276	309
HZ 775 C	206,5	419,6	376700 *408850*	**9620** ***9755***	299	329	305	340
HZ 775 D	206,5	437,5	404370 *438680*	**10275** ***10400***	312	343	319	354
HZ 975 A	206,5	400,2	516150 *558370*	**10590** ***10800***	285	314	291	324
HZ 975 B	206,5	418,0	558840 *601240*	**11415** ***11620***	299	328	306	339
HZ 975 C	206,5	465,2	630290 *683020*	**12825** ***13080***	335	365	341	376
HZ 975 D	206,5	483,1	673860 *726770*	**13655** ***13900***	348	379	354	389

Combination 26/11

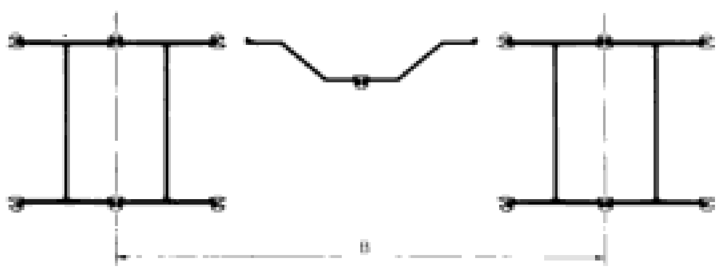


Figure shows HZ 775 A, ZH 9,5/12,0 Scale 1:30

Section	B cm	Properties per meter of wall [5]) Sectional area cm²/m	Moment of inertia cm⁴/m	**Section modulus cm³/m**	Mass of combination with intermediary sections ZH 9,5 $I_{ZH} = 60\,\%\,I_{HZ}$ kg/m²	ZH 9,5 $I_{ZH} = I_{HZ}$ kg/m²	ZH 12,0 $I_{ZH} = 60\,\%\,I_{HZ}$ kg/m²	ZH 12,0 $I_{ZH} = I_{HZ}$ kg/m²
HZ 575 A	206,5	326,5	177570	**6175**	233	256	239	266
HZ 575 B	206,5	344,3	192580	**6650**	247	270	254	281
HZ 575 C	206,5	368,1	209580	**7190**	266	289	272	299
HZ 575 D	206,5	400,7	237320	**8085**	292	315	298	325
HZ 775 A	206,5	382,1	366110	**9450**	276	299	283	310
HZ 775 B	206,5	399,9	393200	**10095**	291	314	297	324
HZ 775 C	206,5	444,3	448840	**11465**	326	349	332	359
HZ 775 D	206,5	462,1	476550	**12110**	339	362	346	373
HZ 975 A	206,5	419,9	608570	**12485**	306	329	313	340
HZ 975 B	206,5	437,7	651340	**13305**	321	344	327	354
HZ 975 C	206,5	489,9	745700	**15170**	362	385	368	395
HZ 975 D	206,5	507,8	789320	**15995**	375	398	382	409

Fig. 4.10 (e) Peine pile walls: combination pile walls with steel joists, Z-section sheet piles and connectors (courtesy Arbed)

Jagged Wall U

The arrangement into a jagged wall offers economic solutions where high section moduli are needed.

The sheet piles are delivered in doubles, threaded and crimped in the mill alternately to S and Z form.

If the OMEGA 18 section is delivered separately, it has to be threaded on the job site and fixed by welding to every double pile. In this case it is not taken into account for the calculation of the section modulus.

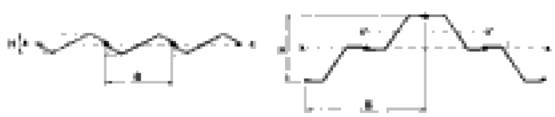
Omega sections threaded in the mill are welded in a way to allow for a respective contribution to the section modulus. See different columns in the table.

For walls with an anchoring or strut system, stiffeners have to be provided at the support levels.

Section	Corner section	Width B cm	Height H cm	Mass kg/m²	Section modulus without Omega cm³/m	With Omega cm³/m	Moment of inertia without Omega cm⁴/m	With Omega cm⁴/m
PU 6	OMEGA 18	92	80	118	2 840	3 460	113 000	138 800
PU 8	OMEGA 18	92	88	128	3 310	4 155	145 000	181 200
PU 12	OMEGA 18	92	88	163	4 285	5 130	188 500	225 400
PU 16	OMEGA 18	92	93	182	4 865	5 655	226 500	272 400
PU 20	OMEGA 18	93	97	202	5 575	6 440	270 650	312 600
PU 25	OMEGA 18	92	101	224	6 310	7 225	319 000	364 900
PU 32	OMEGA 18	92	101	268	7 765	8 665	392 000	437 700
L 2 S	OMEGA 18	76	82	202	4 405	5 260	179 750	213 700
L 3 S	OMEGA 18	76	83	225	5 225	6 070	216 600	251 000
L 4 S	OMEGA 18	76	86	244	5 690	6 695	254 700	289 500

Jagged Wall AZ

Threaded in a reverse position AZ sections may form arrangements at multiple steps for special applications.
The two step arrangement with crimped interlocks on the neutral axis gives an important increase of the moment of inertia. For projects with limited deflection this is an interesting solution.
For sealing screens the 0 step arrangement represents a most economical solution (reduced height, reliable thickness, low resistance...).



0 step arrangement

2 step arrangement

0 step

Section	Dimensions B mm	H mm	Steel section cm²/m	Mass kg/m²	Moment of inertia cm⁴/m	Section modulus cm³/m	Statical moment* cm³	Coating area** m²/m²
AZ 13	715	188	108	100	2 840	308	-	2.28
AZ 18	714	224	120	104	4 290	380	-	2.28
AZ 26	750	236	169	133	6 580	555	-	2.41
AZ 36	750	263	207	162	10 380	790	-	2.45

2 steps

Section	Dimensions B mm	H mm	Steel section cm²/m	Mass kg/m²	Moment of inertia cm⁴/m	Section modulus cm³/m	Statical moment* cm³	Coating area** m²/m²
AZ 13	1 040	562	137	107	47 650	1 520	1 190	2.45
AZ 18	1 260	715	150	118	76 880	2 140	1 595	2.79
AZ 26	1 266	807	185	155	126 010	3 540	2 765	2.82
AZ 36	1 260	873	247	194	186 150	4 310	3 219	2.93

* [illegible]

** [illegible]

The same arrangements are possible for the sections of the BZ-series.

Fig. 4.11. 'Jagged' pile combination sections. Note the use of an 'Omega' section to join sheet piles delivered to site in double units crimped together (courtesy of Arbed)


Fig. 4.12. Combined tubular steel and steel sheet pile wall, River Medway crossing, UK (courtesy of Tarmac)

Contiguous bored piling

The use of low-cost augers and, more particularly, continuous flight auger (CFA) rigs to drill successive unconnected piles provides an economical wall for both temporary and permanent use for excavations to medium depth where soil conditions are amenable. The contiguous wall can only be used where groundwater is not a hazard or where grouting or jet grouting can be used to remedy leakage between the piles. The risk of 'windows' between adjacent piles can be assessed by applying maximum verticality tolerances quoted for auger and CFA pile installation, typically 1 in 100 with depth, and allowing this maximum tolerance to adjacent piles in opposite directions. Where soil and groundwater conditions are favourable, bored piles can be installed as anchored piles with the space between them shotcreted to provide soil support as excavation proceeds.

The nominal diameters of CFA injected piles are 300, 450, 600 and 750 mm. Since grout or high slump concrete is introduced through the auger stem, CFA injected piles do not require top casings during the grouting or concreting process. The depth of wall constructed by this method is, however, limited by the length of reinforcement cage which can be introduced through the cement or grout. A commonly quoted maximum length of 10 to 12 m may be assumed in a grouted pile, although much greater lengths may be installed using either a vibrator to drive, and later extract, an H-pile section mandrell to which the reinforcing cage is attached. A permanent steel joist may also be used to reinforce the piles to lengths greater than 12m.

Inclined pile walls installed by mechanical auger should be mentioned. Sloped sheeting raked towards the centre of the basement may reduce strut loading. Schnabel[3] reported that where sheeting had sloped at an angle of about 10° from the vertical, the measured strut loads were consistently less than two-thirds of the computed strut loads for vertical sheeting in the same ground.

The principal advantages of contiguous pile walls are:

(*a*) low cost and speed of construction for temporary and permanent soil support where drilling conditions are conducive

(*b*) cleanliness and comparative quietness of the installation process; low level of vibration during pile installation

(*c*) for small excavation depths, the ability to minimize distance between the wall and existing walls

(*d*) the range of soil conditions in which CFA piles can be used is wide: granular soils are suitable with few exceptions; cohesive soils are suitable except where penetrations greater than 8 m into hard clay are required; intermediate $c-\phi$ soils are suitable; soft clays, where c_u is less than 10 kN/m^2, or weak organic soils, are unsuitable due to wall bulging; soft rocks, e.g. soft marls and chalk, are suitable (although relatively small penetrations into rock chalk and hard mudstones can cause problems); but hard rocks are not suitable.

The principal disadvantages of contiguous pile walls are:

(*a*) the risk of groundwater ingress between piles
(*b*) limitations of the depth of CFA piles: 12 m long cages are the practical limit without special installation measures, although steel joist sections can be used to reinforce deep piles
(*c*) low efficiency of circular cross-section to pile in bending
(*d*) where independent lining is not needed, the extent of additional works needed to form an acceptable surface to the wall
(*e*) the minimum distance between the wall and existing structure is large for deeper walls.

So CFA rigs, without the need for temporary casings, generally provide a cheap wall quickly in granular and clay soils and are not affected during installation by the presence of groundwater. But depths are limited to small to medium depth excavations. Auger rigs as recently developed in Germany can develop torque up to 30 T m and with this equipment walls can be built with casings to medium to large depths. Vertical tolerances, however, may limit depth, and layers of harder rock will slow progress and increase cost.

The development of mini-rotary rigs has permitted the construction of small (up to 300 mm) diameter contiguous pile walls within a minimum axial distance of 250 mm to an existing structure. A typical rig is shown in Fig. 4.13. These rigs are also suitable for pile installation in conditions of low headroom. Rotary drilling rigs equipped with rock roller core barrels can be used to excavate in hard rock with unconfined compressive strengths of 150 MN/m^2.

Secant piles

The principal disadvantages of contiguous pile walls — the gaps between piles and the resulting problems of lack of waterproofness — are effectively overcome by interlocking or secant piles.

The method consists of boring and concreting primary, or female, piles at centres slightly less than twice the nominal pile diameter. Secondary, or male, piles are then bored at mid-distance between the female piles, the boring equipment cutting a secant section from them. Male piles are bored through female piles before the concrete has achieved much of its strength; should this operation be delayed, wear on the cutting edge to the casing is likely to be much increased. Concrete quality control is also important in this respect and deviations in maximum strength may be as important as minimum strength deviation.

As for contiguous pile walls, the secant pile method suffers from the poor efficiency of the circular cross-section in bending. In some instances reinforcement may be bunched at opposite sides of the pile diameter to ensure maximum effectiveness, but the risk of displacement may preclude this in longer piles. It is usual to reinforce only the male piles, particularly because of the risk of cutting reinforcement in a displaced cage when the male pile is being bored. Deep walls (in London and elsewhere secant walls have been constructed to depths of 40 m) have utilized steel joist sections placed into the male pile bore as reinforcement, and either rectangular reinforcement cages or steel joist sections may be used in female piles in deep walls where such reinforcement is needed for flexural strength. Weaker grades of concrete can be used in female piles when wall flexural strength is not

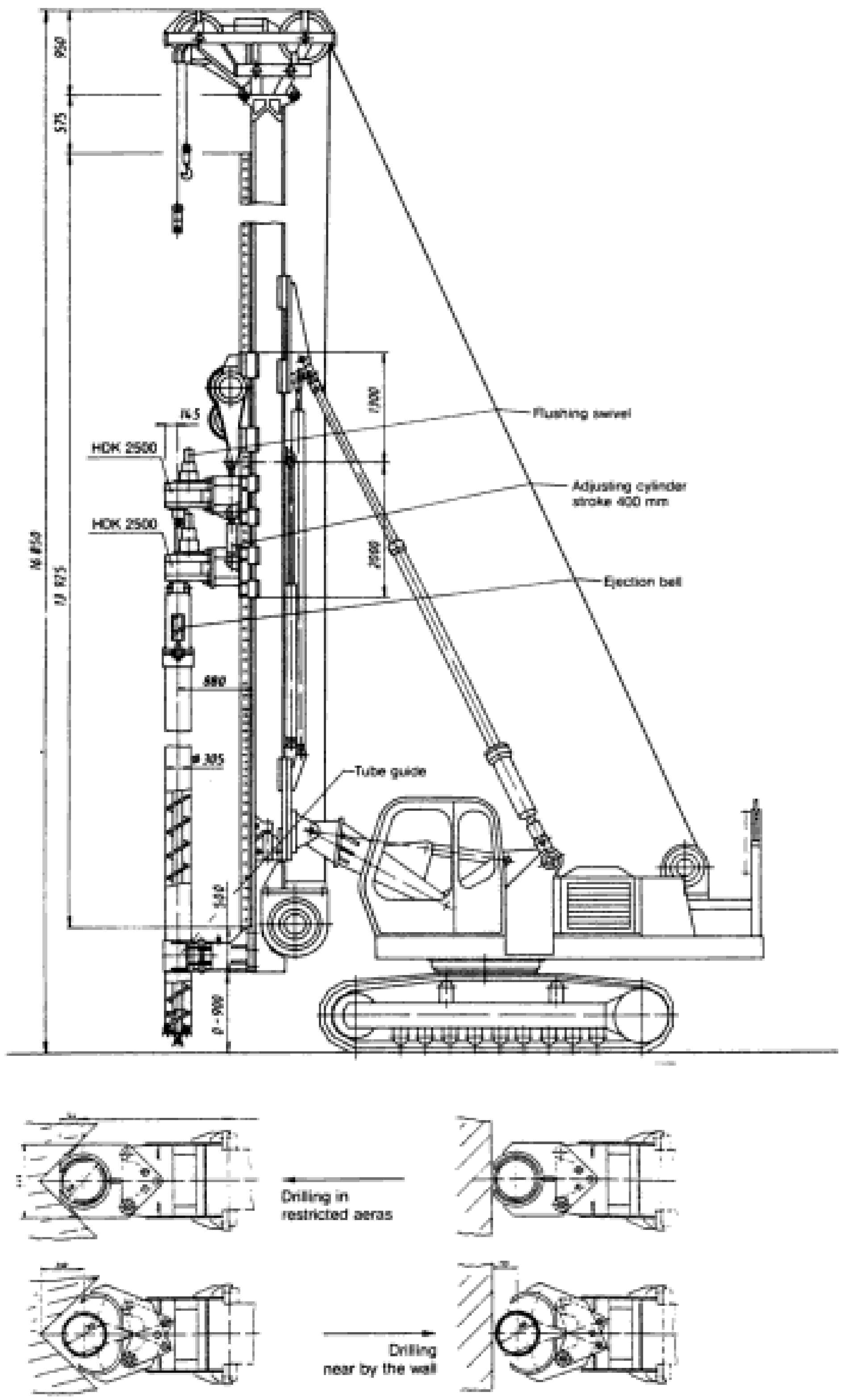
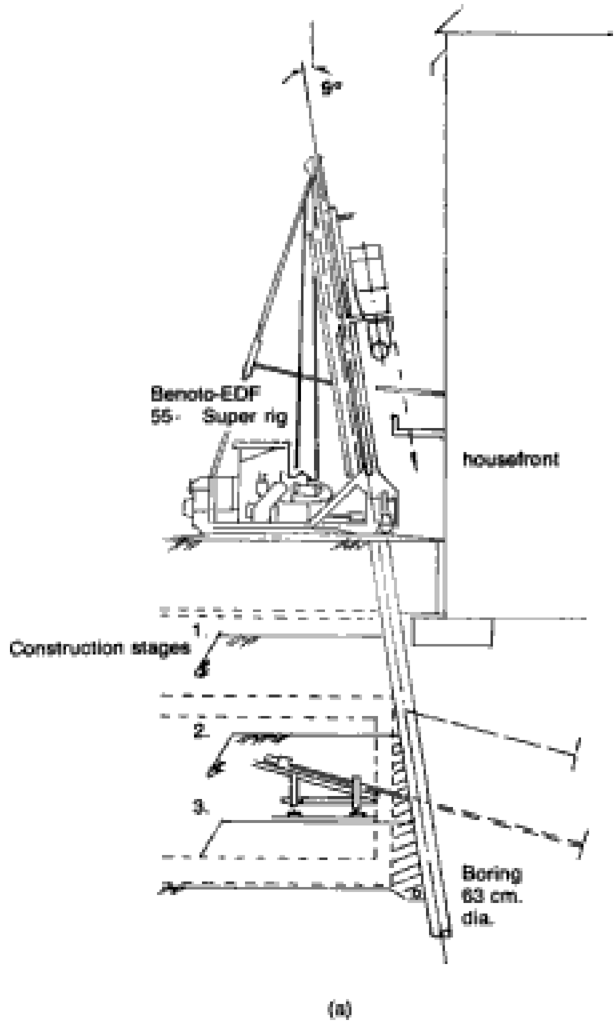


Fig. 4.13. Mini pile rig suitable for contiguous bored pile installation close to existing structures (courtesy of Ingersoll Rand)

critical. This form of construction, known as hard–soft secant piling, may suffer from lack of durability of the soft piles for permanent construction. Junctions with floor and raft slabs can be made with secant pile walls in a similar way to diaphragm walls using bend-out bars or bar connectors and polystyrene blockwork secured to the reinforcement cage.

Inclined walls in secant piling may also be installed at a small inclination to the vertical. In Europe, sloping walls raked away from the basement excavation to increase basement width have been used to dramatic effect. Fig. 4.14 shows a typical installation.

The spacing, centre-to-centre, of secant piles is a matter for judgement by the piling contractor. Fig. 4.15 shows support to an excavation for the Piccadilly line underground railway extension to Heathrow Airport, where 880 mm dia. piles through Thames gravel were spaced initially at 770 mm centres, but in severe groundwater conditions the pile spacing was reduced to 750 mm centres to ensure weatherproofness. Although an internal lining was intended to this wall, the uniformity of finish on the surface of the piles was sufficiently good to leave them exposed without lining in the running tunnels.



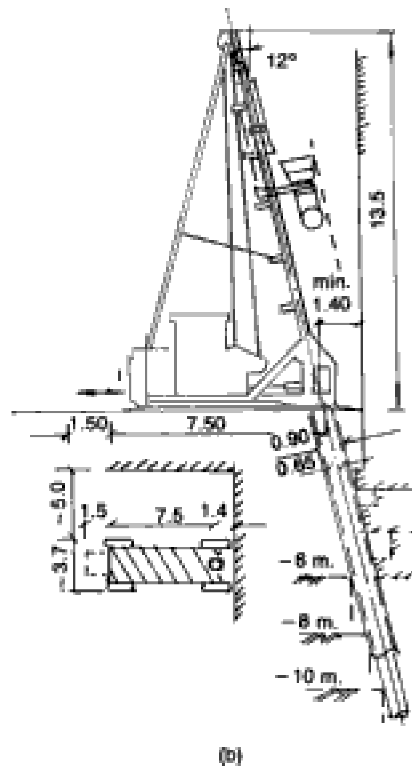


Fig. 4.14. Inclined secant pile walls: (a) construction sequence; (b) Benoto rig (working dimensions)

Fig. 4.15. Secant piles, Piccadilly Line underground railway, Heathrow Airport (courtesy of Lilley)

The application of secant piling prior to the introduction of high-powered rotary drilling machines by Bauer and other manufacturers was limited to relatively expensive walls bored by Benoto rigs. These rigs used double-walled, jointed casings oscillated into the ground with a semi-reciprocal motion as the bore was advanced by hammer grab excavation. Verticality tolerances were superior to those of rotary rigs then in use, in the range 1 : 300 to 1 : 400 with depth. The method using Benoto rigs (and similar Libore rigs) survives in some areas where it was originally popular, but the latest auger rigs using heavy jointed casings and powerful casing oscillators have proved more successful.

The latest rigs, with usable torques in the range 11 to 30 T m, offer a number of improvements:

(*a*) a range of diameters of cased secant piles from 600 to 1500 mm, with production rates increasing with reducing diameter (previously, Benoto piles were limited to 880, 1080 and 1180 mm diameters)
(*b*) difficult conditions including obstructions can be excavated without chiselling
(*c*) the introduction of CFA construction methods for hard–hard secant walls in the range 410 to 880 mm in diameter
(*d*) the use of CFA equipment to bore hard–soft walls where alternate, female, piles are cast using bentonite/cement or bentonite/cement/pulverized fuel ash concrete to form piles of lower strength which are unreinforced.

The secant method therefore provides a waterproof wall which can be built to a considerable depth, in the range 30 to 40 m, and can cope with most types of obstruction. The hard–soft version is limited to shallow excavation depths because of the reduced flexural strength of the wall and some doubt concerning the long-term durability and waterproofness of the soft piles.

Modifications to rotary equipment have minimized the working space required between the rear of the secant pile wall and the site boundary. For relatively shallow walls, up to 12 m deep, Bauer equipment, a modified BG11 rig, permits installation of 410 mm dia. piles with an axial spacing from an existing structure of 305 mm.

In 1988 at Staines, UK, Bachy–Bauer installed 410 mm dia. hard–hard walls using such a rig only 100 mm from an existing wall. The pile diameter was restricted and so, therefore, was the excavation depth.

Guide walls are needed for all forms of secant pile construction, and mention should be made of the cost and time liability this represents.

Closely spaced anchors, or struts, are used to support secant piling using steel waling members or in situ concrete bearing pads. Secant pile walls may also be used to carry vertical loads. Fig. 4.16 shows the method of use of secant piling with anchors installed below existing buildings.

The method has been used successfully in London for the construction of a deep basement to the British Library (Fig. 4.17). The basement extends to four floors, with the maximum depth 24 m below ground level. Underground railway tunnels are bridged by a concrete raft at the centre area of the site where the basement depth is limited to two floors. The maximum depth of the secant pile is 30 m. Due to the high sulphate content in the subsoil and the need to minimize the generated heat of hydration caused by high cement contents, the concrete mix to the piles incorporated 85% cement replacement with a ground granulated furnace slag and an aggregate of maximum size 40 mm. Although a diaphragm wall 1·2 m thick was originally intended for use at the site, the existence of large vertical service ducts at close centres adjacent to the wall caused high design values of shear at the junction of floor slab and wall. The large steel joist sections which reinforce the male piles of the secant wall ideally resist these shear forces.

Diaphragm walls

The first diaphragm walls were tested in 1948 and the first wall was built, using bentonite slurry as a means of support, by Icos in Italy in 1950. That contract was for a cut-off wall on a dam site. Structural walls followed, and in the late 1950s the construction of the Milan Metro allowed Icos to use diaphragm walls extensively in cut-and-cover construction; a method of top-down working known as the Milan method was developed. The first structural diaphragm wall in the UK, at Hyde Park Corner, London, in 1961, was also built by Icos using its then

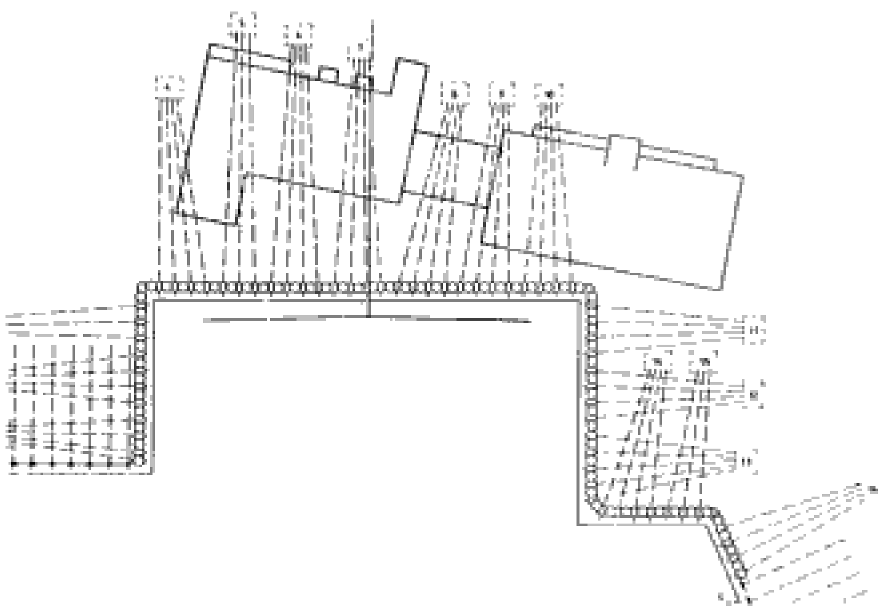


Fig. 4.16. Secant pile wall anchored to deadmen below an existing structure (courtesy of Bauer)

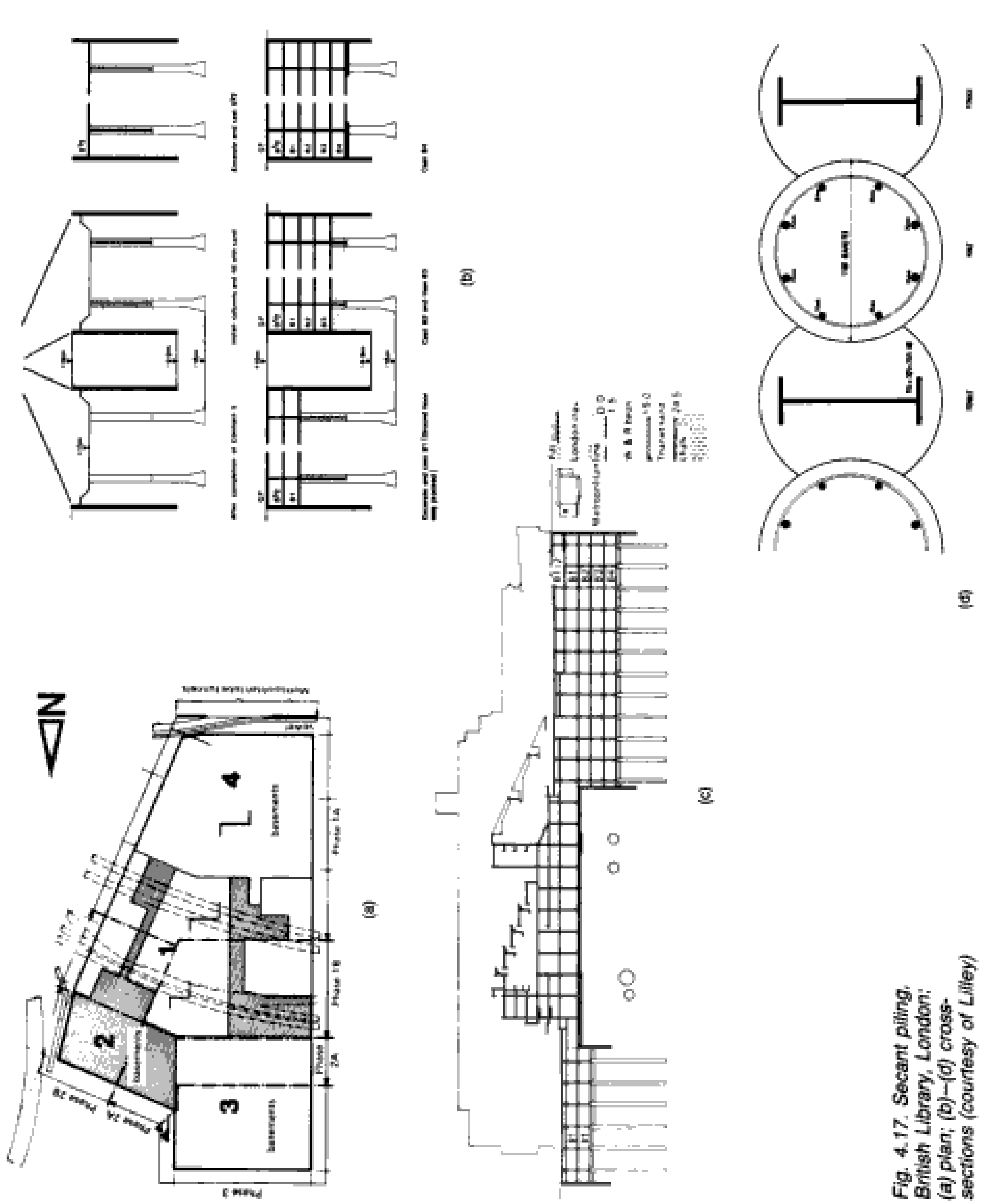


Fig. 4.17. Secant piling, British Library, London: (a) plan; (b)–(d) cross-sections (courtesy of Lilley)

patented process. Many contracts followed for Icos in Europe. These contracts, to build basements, underpasses, marine structures and metro works, were all excavated using rope grabs suspended from, first, tripod rigs and, later, from cranes.

The development of diaphragm walling was encouraged by competition among specialist international firms from the mid-1960s. These firms, including Icos in Europe and the USA, Soletanche and Bachy in France, Trevasani and Rodio in Italy, and Cementation in the UK, all developed excavation systems of their own. Two preferences quickly became evident: rope grabs suspended by rope either from tripod rigs or cranes, or grabs, generally hydraulic powered, mounted on kelly bars mounted on cranes. These two methods of excavation continued into the 1980s. The major problem which these methods failed to solve was the excavation of rock. In the 1970s, rotary percussive tools using reverse circulation were used to excavate, albeit slowly, into hard rock.

In the early 1970s the reverse circulation system was used in Japan for a new generation of rail-mounted rigs, manufactured by the Tone Drill Co. Although these rigs were adequate for excavation in granular soils (but less so in cohesive soils), they were the forerunners of modern reverse circulation rigs and used shaker screens and hydrocyclons to clean slurry in a similar way to the latest Hydrofraise and Trenchcutter rigs. Despite the introduction of these modern reverse circulation rigs, excavation by grab still has its place, on small sites and in certain soil conditions, but it is worth noting that kelly mounted rigs are rarely used and preferences are for heavy rope grabs suspended from cranes.

Incentives to develop excavation equipment (particularly in rock) have come mainly from the need to improve output at the lowest cost; without exception, the operation of panel excavation still remains the critical operation in diaphragm wall contracts despite the complexity of all other operations. Minimization of machine downtime is a vital factor in overall excavation production.

In addition to excavation output, increasing wall depths and the requirements of wall verticality tolerance have placed demands on the development of excavation equipment. To a large extent the requirements of depth and tolerance, together with the particular soil or rock type, define the optimum excavation equipment: rope grab or Fraise. Modern Fraise machines are capable of cutting through rock with a compressive strength of the order of 2000 bar, and of excavation in soils to depths in excess of 100 m. Trenchcutter machines equipped with cutters with small roller bits on their outer edge were developed for similar tasks. Verticality tolerances of the order of 0·3% can be made to depths of 30 m. The manufacturers of these rigs are currently Soletanche, Bauer, Casagrande and Tone.

Developments in mechanical excavation equipment have radically changed the scale of diaphragm wall contracts that can be undertaken. The Seaforth Dock contract (145 500 m^2 of wall) and Redcar Ore Terminal (64 680 m^2, depth 45 m), both excavated by rope grabs, were among the largest projects of their time (1968–72). The Medinah car park project in Saudi Arabia, completed in two years with two Hydrofraise working rigs and five rope grabs, has a total wall area in excess of 320 000 m^2 and to a maximum depth of 55 m.

Accompanying these changes in excavation method has been the development of techniques on site:

(*a*) temporary stop end fabrications which allow water bar installation in panel joints and do not require extraction during, or immediately after, concreting
(*b*) fabricated permanent stop ends which allow the transfer of shear and tensile forces through the panel joints
(*c*) re-usable precast concrete guide walls
(*d*) improved slurry cleaning equipment
(*e*) polymers for artificial slurries
(*f*) improved cage handling methods

(*g*) smaller reverse circulation rigs for urban sites (City Cutters)
(*h*) improved cutters such as rock roller bits on Trenchcutter rigs.

While site operations were improving, diaphragm wall designers were also introducing innovations:

(*a*) structurally-efficient plan shapes of diaphragm walls
(*b*) post-tensioned diaphragm walls
(*c*) precast concrete diaphragm walls
(*d*) combined slurry cut-off walls, permanent precast concrete diaphragm walls and temporary soil retaining walls.

A review of recent developments in the construction of structural diaphragm walls by Puller and Puller[4] is referred to in greater detail in chapter 8.

Advantages of diaphragm wall construction

The diaphragm wall is generally efficient in cost and construction time where it is used for both permanent and temporary subsoil retention for walls of medium, and greater, depth

An early review of the economics of basement construction[5] emphasized that cost comparisons between types of basement construction could only be made after a complete analysis of the unit costs of temporary and permanent walls, the costs of shoring or anchoring and the costs of internal lining walls and basic finishes. The diaphragm wall method compares well in a complete analysis of this type. Additional data on cost comparisons are given at the end of this chapter.

Typical diaphragm wall areas undertaken in the UK vary from less than 1000 m^2 to more than 6000 m^2. A medium-size contract would have an area between 3000 and 4000 m^2. A typical site in the London area is shown in Fig. 4.18. The site was occupied by storage buildings and a shopping area which were demolished in two stages to form the extension to a department store with a double-storey basement. The first basement floor can be seen under construction in Fig. 4.18, which shows the high degree of site utilization despite the need to retain an existing facade to the upper site boundary and the presence of a flying shore to support it. The 800 mm thick diaphragm wall used one level of anchors. The ground conditions were typical of the area, sandy gravel overlying London clay, which in turn overlaid a dense fine sand. The diaphragm walls supported vertical columns to the building periphery.

Elsewhere, in major city centres, large schemes for office and retail use occupy progressively larger and deeper floor space as land values escalate. In Hong Kong in 1994 the LDC H-6 project in Queens Road Central consisted of a perimeter diaphragm wall of 18 000 m^2, 45 m deep and 1200 mm thick, together with 46 barrettes with a total area of 8500 m^2, to the same depth and thickness. The soil conditions generally consisted of a 15 m depth of fill and alluvium containing cobbles and boulders underlain to a depth of 60 m by completely decomposed granite with SPT values from 50 to more than 240. Within the upper fill and boulder layers, a hydraulic grab was used for pre-excavation to an average depth of 12 m. Within this depth, where no boulders greater than 700 mm were met, an average 7 to 10 m^2/h could be achieved, but this was reduced to 30 to 50% of this output where longer boulders were broken by chisel. The pre-excavated trench, backfilled with lean mix concrete, was then excavated using Bauer BG 30 Trenchcutters. The average penetration rate achieved on primary panels was:

- excavation in lean mix concrete: 3 min per metre, 65 m^3/h;
- excavation in completely decomposed granite, SPT less than 50: 5 min per metre, 45 m^3/h;

Fig. 4.18. Basement construction: diaphragm wall anchored below existing property and highway for a department store, Croydon, UK (courtesy of Lilley)

- SPT 50 to 100: 7 min per metre, 30 m^3/h;
- SPT 100 to 300: 15 min per metre, 13 m^3/h.

These rates, achieved only during the period of machine operation, do not include breakdown, waiting and working time. The production rate for secondary panels was approximately half that of the primary panels.

Diaphragm walls allow effective transfer of vertical load from the building superstructure to subsoil below basement level

Concern that the presence of a filter cake from the slurry between panel concrete and subsoil could seriously reduce transfer of load in surface friction/adhesion and end bearing made UK designers reluctant to use diaphragm walls as load-bearing units in the early years after their introduction. Although published results of comparative load tests of diaphragm panels are still relatively scarce, these results may be usefully augmented by consideration of load tests on piles cast under slurry, a summary of which was given by Fleming and Sliwinski.[6] Xanthakos[7] reviewed the available test data and concluded that while, in 1979, there was no evidence that load transfer characteristics of slurry panels differ materially from those in dry construction, design should be conservative, especially on sites where panel load tests were not justified on cost grounds.

The practical outcome of this situation, therefore, is that diaphragm walls are commonly used for the transfer of vertical load. Indeed, diaphragm wall units are now widely adopted as barrettes to transfer load and moment from structure to subsoil. An early paper by Corbett *et al.*[8] gave details of a typical load-bearing basement wall at Kensington Town Hall in London. The three-storey basement car park was on average 13 m deep and approximately 140 m × 65 m in plan. The building was situated in a residential area and lack of noise and vibration was regarded as very important. The cross-section through the structure, north to south,

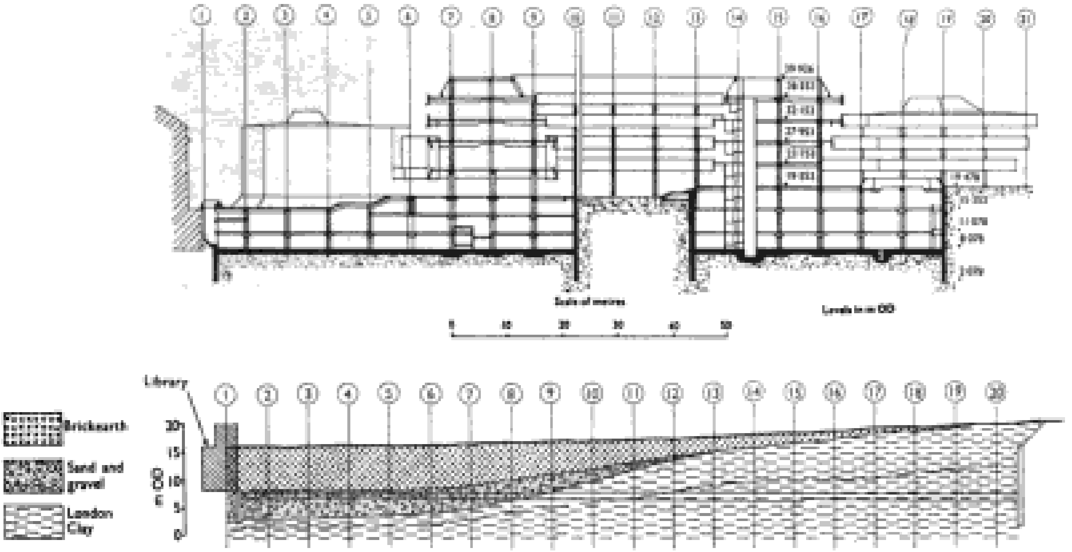


Fig. 4.19. Basement construction: diaphragm wall propped from completed raft construction. Section through substructure and geological section, Kensington Town Hall, London (Corbett et al.[8])

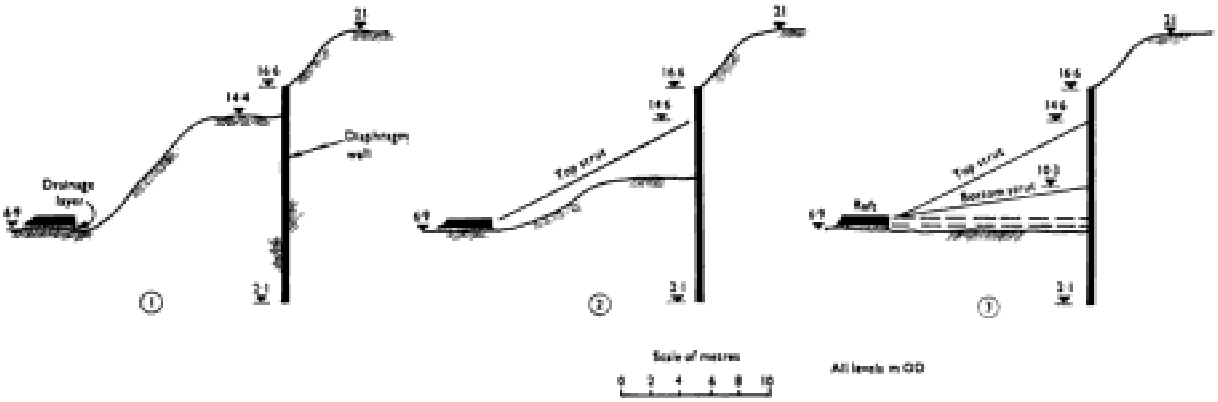


Fig. 4.20. Basement construction: excavation sequence for north diaphragm wall by Rendex steel struts from completed centre area of raft, Kensington Town Hall, London (Corbett et al.[8])

and the geological section are shown in Fig. 4.19. The method of propping the diaphragm wall with two levels of Rendex struts from the completed central raft is shown in Fig. 4.20. The peripheral superstructure loads are carried by the diaphragm walls and a vertical load test carried out on site showed that, compared with other large diameter bored pile designs, the performance of the test panels was not adversely affected by the bentonite. Fig. 4.21 shows the relatively small extent of horizontal movement of the north wall that occurred during basement excavation.

Diaphragm wall construction causes minimum noise and vibration disturbance

The extent of noise and vibration in diaphragm wall installations is limited to that associated with normal civil engineering plant, cranes, generators, pumps, compressors, power packs etc. for excavation in soils. Obstructions or rock strata may lead to chiselling and some vibration where conventional grab excavation

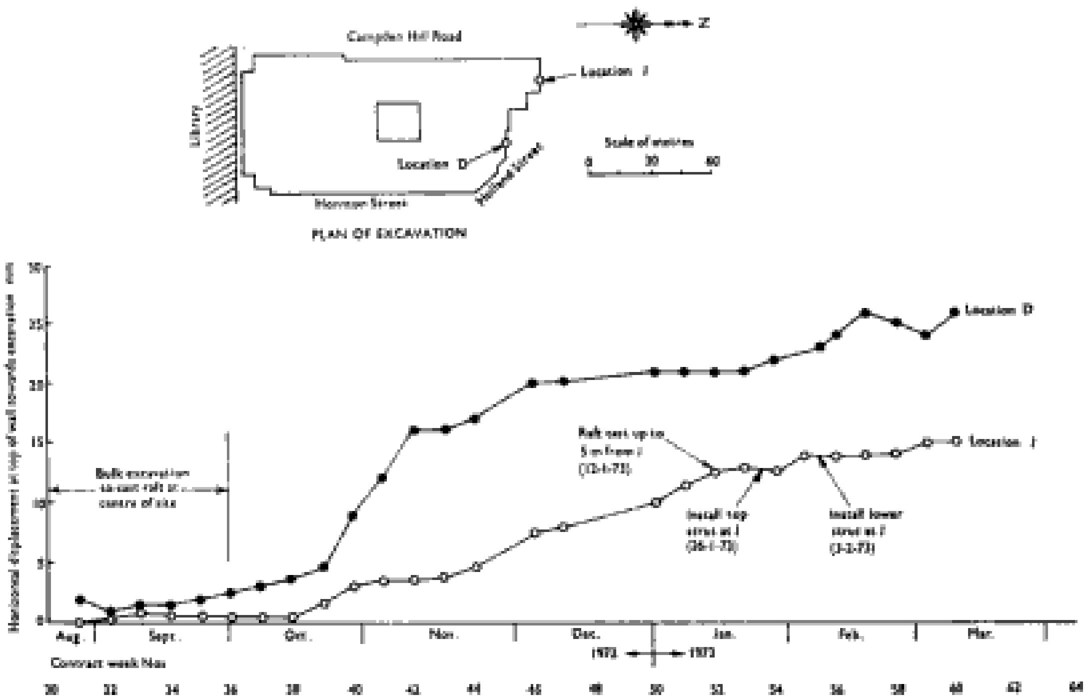


Fig. 4.21. Extent of maximum horizontal movement of north diaphragm wall, Kensington Town Hall, London (Corbett et al.[6])

is used, although the use of modern reverse circulation rigs, Hydrofraise by Soletanche and Rockcutter by Bauer, would overcome this. Where such vibrations do occur, soil from the side of the slurry trench may collapse, leading to additional panel overbreak. The extent of chiselling and vibration using conventional grab excavation can be reduced by the successive use of auger coring buckets along the length of the panel excavation.

It should be remembered that even where soil conditions allow relatively quiet plant operation the diaphragm wall process necessitates extraction of temporary stop ends from panels other than secondary panels, and this operation is only possible some hours after the completion of concreting. Unless expendable reinforced concrete stop ends are used (a practice which may bring other problems) some noise from cranage is inevitable, with out-of-hours stop end extraction.

Quality control of diaphragm wall construction

Considerable skill and care is required on the part of the specialist contractor installing diaphragm walls. Five items within the Author's experience warrant particular attention.

Continuous construction

The construction technique is continuous, from commencement of panel excavation to installation of stop end formers and reinforcement cage, concreting and withdrawal of panel stop ends. Any major disruption, whether caused by ineptitude or other circumstances, can cause loss of ground or poor standards of wall construction which may be difficult and expensive to rectify.

It is vital to maintain punctual completion of the construction cycle of excavation, panel preparation for concreting, reinforcement installation, concreting and panel finishing. Factors which can disrupt this operation sequence must be removed. Critical matters include the optimum panel size to ensure compatibility with excavation and concrete outputs; permissible daily working hours allowed in the contract to facilitate daily completion of important stages in the cycle, such as concreting and stop end removal; and high standards of site supervision to ensure availability of resources of plant, labour and material.

Quality

The quality of the work (which is only disclosed when bulk excavation of the site is undertaken) is directly dependent on the subsoil strata since the soil forms a temporary shutter against which the in situ concrete is cast.

In terms of wall finish, the smoothness of the diaphragm wall concrete will improve with reducing soil particule size. One would expect to obtain a better surface finish for a diaphragm wall excavated in clay compared with one in sandy gravel. Obstructions in the diaphragm wall excavation, whether man-made or natural, may not only prove difficult to remove from the panel excavation, but their removal may cause some dislodgement of the soil on the panel face, the resulting void subsequently being filled with concrete. Where this void is exposed later during site bulk excavation, some remedial work may be needed to remove the excess concrete.

Overbreak on the panel face maybe caused, therefore, by removal of obstructions, but more likely by collapses of the panel face caused by instability. Panel stability, reviewed by Xanthakos[7] and Puller,[9] was originally studied by several authors including Nash and Jones,[10] Muller-Kirchenbauer,[11] Piaskowski and Kowalewski,[12] and Fernandez.[13] Within the Author's practical experience, panel stability is much dependent on the ability of the soil around the panel effectively to arch during all stages of the panel excavation. In loose sands, sandy gravel and open gravels large volumes of overbreak frequently occur below guide trench level. Typical overbreak removal operations cause difficulties of access and construction delay. The cost of removal is frequently three or four times that of the original concrete as placed. A differential of at least 1·5 mm height between mud level within the panel excavation and groundwater level within the subsoil adjacent to the panel is necessary to avoid panel instability. Particular attention is needed where groundwater flows underground or through a closing panel. The cause of serious panel instability is frequently the neglect of maintaining such a differential height, and in isolated cases the installation of a wellpoint dewatering system has been necessary during the diaphragm wall works to improve the differential height and so increase panel stability.

Where conventional grab excavation is used there is risk of loss of the panel face, particularly in granular soils, from below guide trench level, by the pumping action during raising and lowering of the excavation grab within the bentonite slurry. Instances of major panel collapse caused by panel instability in very weak clays were reported by Mastikian[14] but fortunately are rare. Some wall face collapses have also been observed in deep panels in highly fissurred over-consolidated clays, such as London clay, where fissuring has much reduced the intact soil strength.

A repetitive construction problem can occur in diaphragm wall operations where the temporary tubular stop-end former does not fit in the subsoil at the end of a panel being concreted and allows wet concrete to pass around the stop-end former. The resulting annular ring of concrete is shown in Fig. 4.22. This obstruction to the succeeding panel excavation may be removed by a single grab bite excavation immediately following the removal of the stop-end former, taking care to avoid damage to the green concrete within the panel. Failure to remove this annular ring of concrete at an early stage may result in much chisel work later or, worse, the construction of a poor joint formed with part of the annular ring still intact.

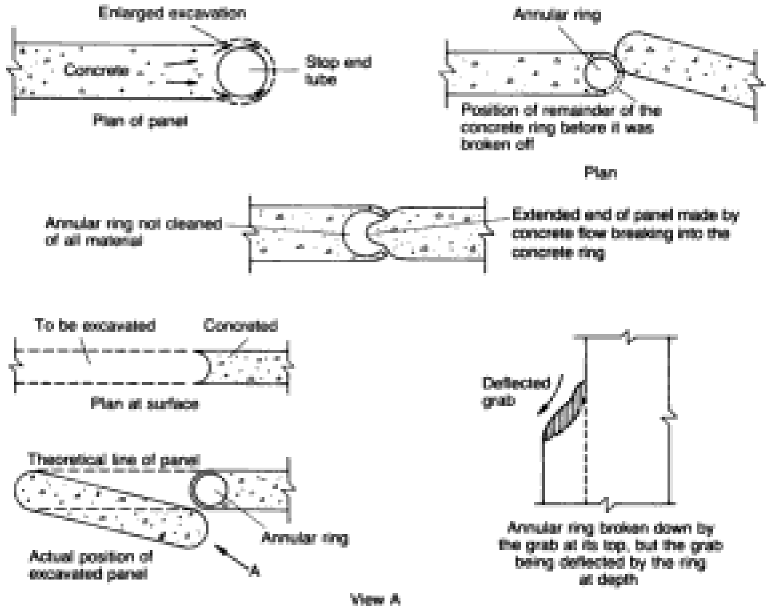


Fig. 4.22. Risk of faulty diaphragm wall construction caused by an annular ring of concrete bypassing the stop-end: (a) cause of annular ring of concrete; (b) faulty construction (misalignment of adjacent panels); (c) faulty construction causing vertical inclusion at panel joint; and (d) faulty construction (misaligned panel at joint below ground)

Inclusions

The combined use of slurry made from mud and high-graded concrete within the same wall panel construction requires considerable skill and care to ensure homogeneous finished concrete without deleterious slurry inclusions in the panel.

Bad slurry inclusions, particularly at panel joints, can cause unacceptable standards of structural competence or waterproofness of the wall and must be avoided. Cambefort[15] showed examples of poor quality diaphragm walls caused by inadequate mud quality and other reasons. Fig. 4.23 shows a void caused by a large mud inclusion. The cause of such inclusions may not necessarily lie with poor quality slurry; slow, erratic concrete pours with non-cohesive and low slump concrete may also be a contributory cause. The risk of slurry inclusions within wall panels is increased when tremie spacing is too large in long panels and at tee or corner panels.

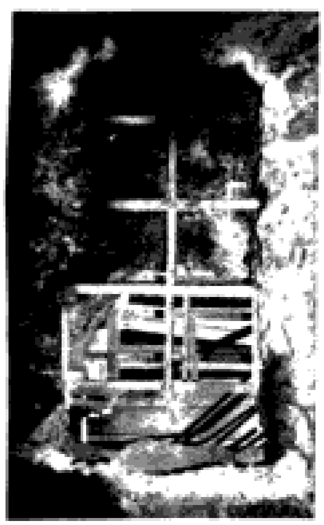


Fig. 4.23. Large mud inclusion within a diaphragm wall panel

Tolerances

There is considerable risk of wall construction outside specified tolerances. Tolerances for verticality, concrete protrusions on the wall face, positions of box outs for recesses and locations of reinforcement are detailed in model specifications for diaphragm wall construction; in the UK the model specification published by the Federation of Piling Specialists[16] lists the recommended tolerances.

(*a*) Guide trench: line of finished face nearest to excavation: ±15 mm in 3 m; vertical to within 1:200; minimum clear distance between the faces of the guide walls shall be wall thickness plus 25 mm, and the maximum distance shall be wall thickness plus 50 mm.

(*b*) Diaphragm wall: exposed wall face and panel ends shall be vertical within a 1:80 tolerance. In addition to verticality tolerance, a further 100 mm shall be allowed for protrusions resulting from irregularities in the ground as excavated.

(*c*) Box outs: vertical and horizontal tolerances of ±75 mm and a horizontal tolerance, at right-angles to the wall, of ±75 mm.

(*d*) Reinforcement: horizontal tolerance of cage head at top guide wall, measured along the trench, ±75 mm; vertical tolerance of cage head

±50 mm; lateral tolerance of reinforcement in the direction across the width of the wall 50 mm.

(*e*) Concrete tolerance: where the final trimmed level of the diaphragm wall is up to 1·0 m below the top of the guide wall, the casting tolerance will be 600 mm above the trim level; for each additional 1 m depth of final trim level, or part thereof, allow an additional 150 mm level tolerance.

Similar tolerances are listed in later model specifications, for example the Institution of Civil Engineers' Specification For Embedded Retaining Walls and the draft European Specification for Diaphragm Walls.[18]

Waterproofness

The waterproofness of a diaphragm wall structure depends on standards of design and construction. Design methods to achieve high standards include the provision of an integral water bar system between the diaphragm wall panel joints and the construction joints between the wall and the basement slab and within the basement slab itself. Waterproofness of basement construction is discussed more fully in chapter 8. In general, the following factors affecting waterproofness should be noted:

(*a*) soil and groundwater conditions: the greatest risk occurs with high groundwater in loose granular soils and very weak silts, silty clays and highly fissured over-consolidated clays

(*b*) compliance with good standards of diaphragm wall excavation, slurry control and concreting: avoid very long periods between excavation and concreting

(*c*) wall thickness: avoid very thin walls less than 600 mm

(*d*) plan shape of excavation: there is some evidence that leakage at panel joints may tend to appear near return angles of wall (joints of panels at the centre of a long length of straight wall may also tend to open during bulk excavation and can also cause leakage)

(*e*) depth to formation level: the greater the basement depth the greater the leakage risk, primarily due to increasing hydraulic head with depth

(*f*) panel length: shorter panels mean more panel joints but may reduce the risk of inclusions

(*g*) means of diaphragm wall excavation: improved verticality tolerance and lack of vibration with cutter excavation should generate better wall quality than grab excavation and in turn, waterproofness standards should be improved

(*h*) extent of reinforcement density in diaphragm wall: high reinforcement density may reduce in situ concrete quality and, in turn, reduce waterproofness

(*i*) top-downwards construction: this is likely to minimize wall movements and improve waterproofness, particularly at panel joints.

Precast diaphragm walls

The disadvantages of using the subsoil face as the shutter for concrete placed within the diaphragm wall panel are obvious. Two French firms, Bachy and Soletanche, have developed methods of using precast concrete wall panels to overcome these drawbacks. The technique was first introduced in 1970 but has tended to be less popular in recent years. There are no known jobs in the UK using the precast system.

Colas de Francs[19] claimed the following distinct advantages for the precast prefabricated diaphragm over other types:

(*a*) general appearance: no cutting back is required and the finished surface is agreeably clean

Fig. 4.24. Precast diaphragm wall construction: a high standard of wall finish is obtained using the Prefasil system, Schipol, the Netherlands (courtesy of Bachy)

(*b*) the shape of the diaphragm can be tailored to form an integral part of the final structure, satisfying technical and economic considerations
(*c*) improved concrete quality and accuracy in placing reinforcement gives considerable savings on materials; prefabricated diaphragms are generally 30% thinner than conventional diaphragm walls
(*d*) the prefabricated diaphragm can be built and installed in the ground to finer tolerances, and openings can be positioned more accurately
(*e*) watertightness at the joints and in the wall itself is better than with conventional diaphragms.

The improved surface finishes can be seen in Fig. 4.24, an example of the Bachy system known as Prefasil walling. Whether the advantages of finish, improved tolerances and reduction in wall thickness offset the cost penalty of these methods can only be appraised on a job-by-job basis.

Panel widths and lengths are limited to the capacity of mobile lifting equipment suitable for operation on construction sites, and in practical terms this is of the order of 30 tonnes. The panels are usually cast on site; some additional site area may be needed to do this although less space is needed to store reinforcement since prefabrication of reinforcement cages is not necessary with precasting. Fig. 4.25 shows a panel being lifted.

Fig. 4.25. Lifting a precast diaphragm panel at a nuclear power station at Nogent-sur-Seine, France (courtesy of Soletanche)

The principal differences between Soletanche's Panosol and Bachy's Prefasil method lies in the mechanical connection between panels and the timing of the use of cementation slurry to surround the precast unit and support the subsoil. In the Panosol method, the panel is dug under a cementations slurry containing retarding and regulating additives, the slurry being removed without difficulty from the smooth face of the wall during bulk excavation of the site. Slurry between the rear face of the precast panel and the subsoil remains permanently as an inert filler material between wall unit and subsoil, the final strength of the set slurry being designed to exceed neighbouring soil strength. In the Prefasil technique the slurry is not designed for the dual purpose of a digging slurry and a medium or long-term soil support; the panel is dug under a conventional bentonite mud which is later displaced by the introduction of a cementatious grout just before the precast unit is positioned in the panel excavation. Bachy claims that its process gives flexibility in site operations, allows a wide range of grout strengths to be used, is particularly convenient when large vertical loads are being carried by the panels, and the process also avoids risk of contamination of the grout by soil during the

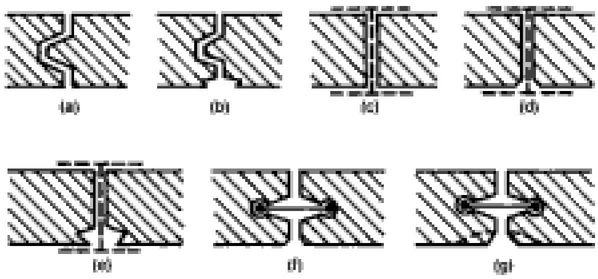


Fig. 4.26. Horizontal cross-section jointing detail for Panasol diaphragm wall panels: types (a) and (b) are standard joints; types (c)–(e) use temporary metal guides; types (f) and (g) use preformed water bars (courtesy of Soletanche)

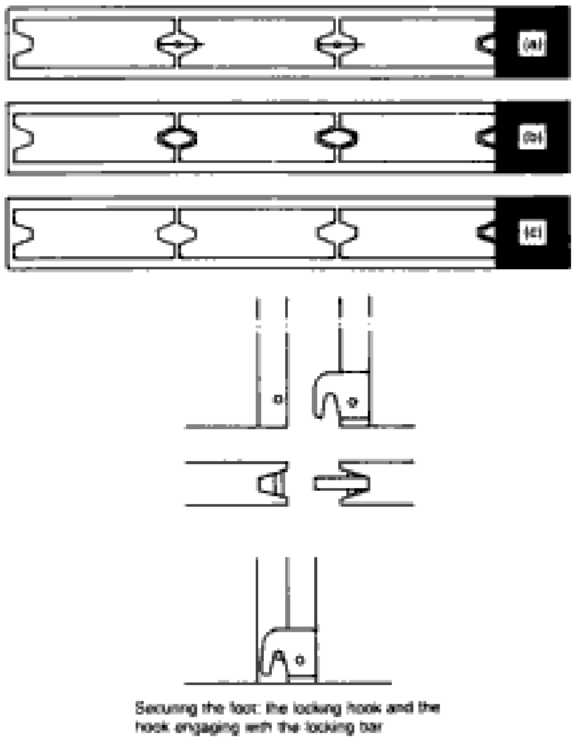


Fig. 4.27. Jointing details for Prefasil diaphragm wall panels — mechanical locking hook joint at the foot of the panel and types of joint in horizontal section: joint type (a) uses a preformed water bar, type (b) uses a reinforcing key, and type (c) uses grout only (courtesy of Bachy)

excavation process. The methods of panel connection for the Soletanche system are shown in Fig. 4.26, and Fig. 4.27 shows the technique used by Bachy.

Precast diaphragm panels may be designed to span vertically between ground anchor levels, taking care to avoid movement caused by prestressing forces to the anchors compressing slurry between wall and subsoil. Alternatively, soldier panels, themselves anchored, may support panels spanning horizontally between them. In these cases the soldier wall units may be extended in depth to improved bearing strata to support vertical load. Precast diaphragms may also be used in a composite construction of structural wall and self-hardening slurry cut-off, as shown in Fig. 4.28.

Post-tensioned diaphragm walls

Another innovation pioneered by Icos in the late 1960s replaced vertical reinforcement within the diaphragm wall panel by a prestressing tendon, the reinforcing steel in the cage being used only to lift and position the tendon. The use of post-tensioning therefore reduces steel reinforcement costs to a minimum,

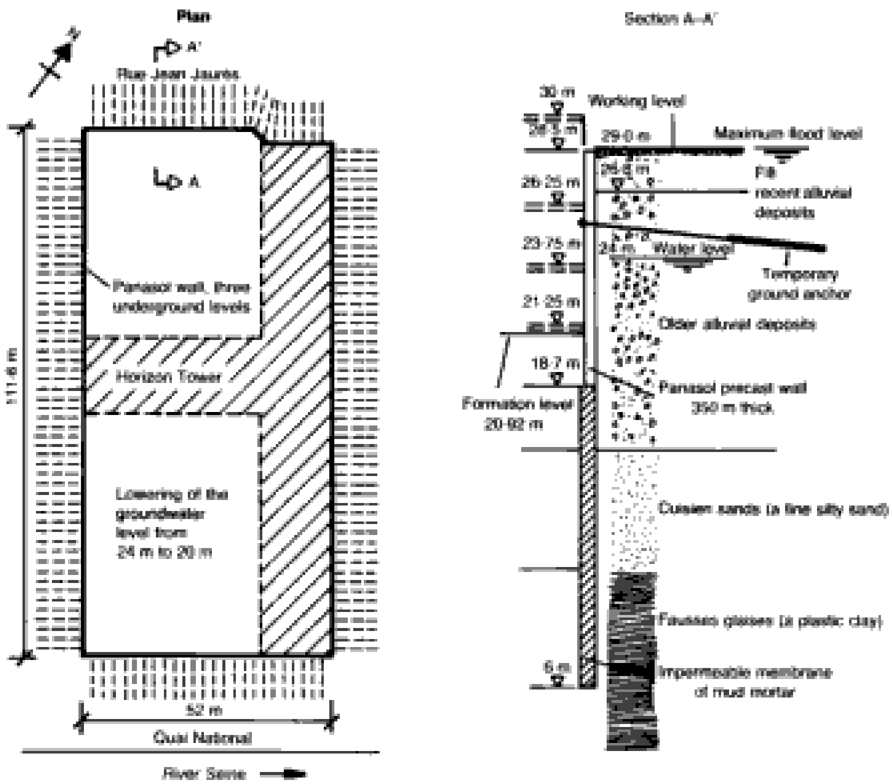


Fig. 4.28. *Composite wall construction using precast wall and slurry cut-off, Horizon Tower at Puteaux, Paris (courtesy of Soletanche)*

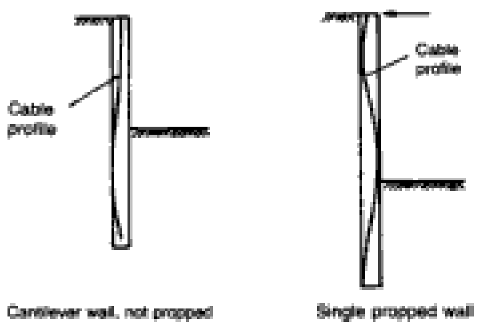


Fig. 4.29. *Post-tensioned cantilever and single-propped diaphragm walls showing cable profile and bending moment diagrams for the wall section (Gysi et al.[20])*

although its relatively small use in the UK may indicate relatively high cost of tendons and stressing operations.

The feature of the method is that stressing of the panel is undertaken before bulk excavation is carried out and while the wall panel is fully embedded. Tendon forces and eccentricities are calculated on loading on the final structure with no tension across the concrete section, the panel movement during stressing being minimized by the surrounding soil, the soil restraint varying between full passive pressure and earth pressure at rest. The cable profiles for a cantilever wall and a propped diaphragm wall are shown in Fig. 4.29. Fig. 4.30, taken from Gysi *et al.*[20] shows earth pressure, bending moments and stresses in a typical wall unit before removal of the surrounding soil, the moment applied to the wall section by the prestressing force being effectively reduced by the soil stiffners preventing tension developing across the section due to prestress.

Figure 4.31 shows examples given by Gysi *et al.*[20] The considerable heights of cantilever wall shown, between 7 and 9·6 m, and the relative slenderness of

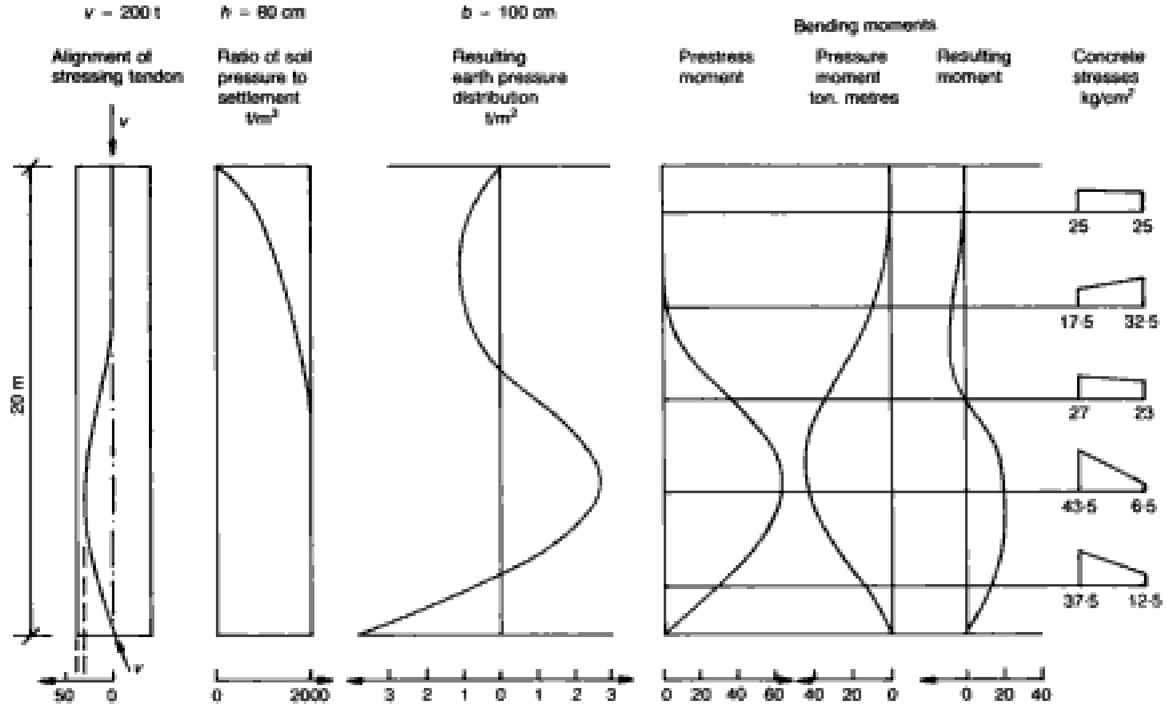
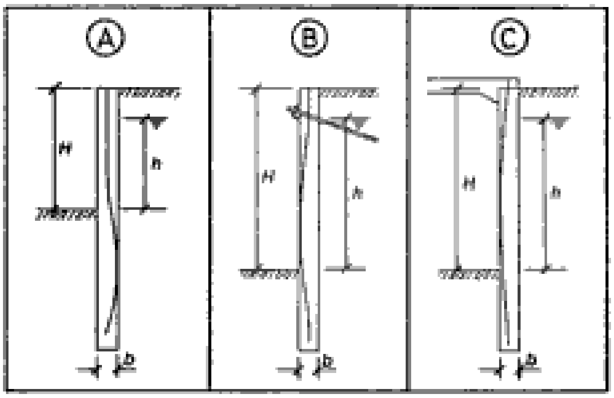


Fig. 4.30. Earth pressure, bending moments and stresses in a post-tensioned diaphragm wall before excavation in front of the panel (Gysi et al.[20])



Site	*Type*	*H:m*	*h:m*	*b:m*	*Max prestress: (N/mm²)*
Centrale PTT Bellinzona	A	7·0	2·7	60	3·6
Admiral SA, Paradiso	A	7·6	5·10	80	3·5
German Embassy, London	A	9·6	5·70	90	3·1
ETA-Werke Grenhen	B	13·2	6·2	80	3·7
Propr. Fabriane, Lugano	C	11·6	4·6	60	3·6
Centrale TT, Moralto	C	15·0	—	80	2·1

Fig. 4.31. Examples of post-tensioned walls (Gysi et al.[20])

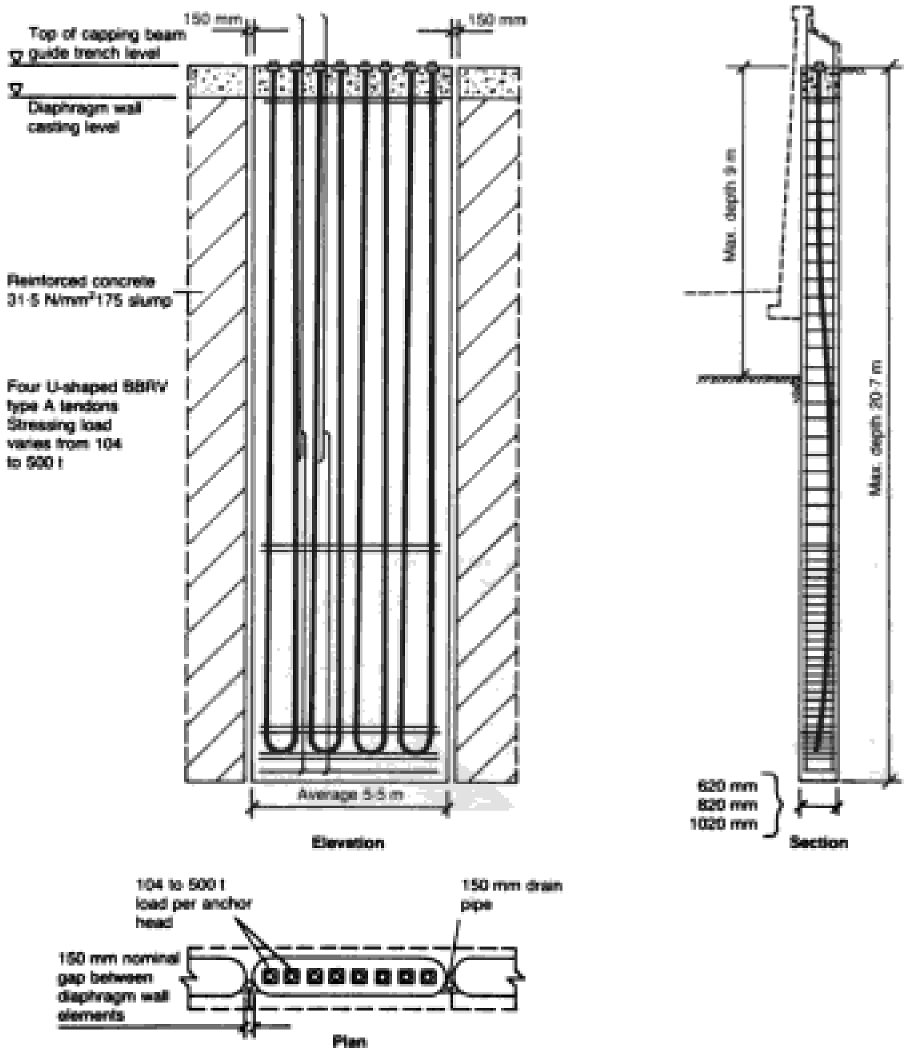


Fig. 4.32. Typical diaphragm wall reinforcement cage with curved tendons, Irlam O' Heights near Manchester, UK (Fuchsberger and Gysi[21])

the wall section would appear conducive to excessive soil deformations behind the wall but none is reported. It is possible that the buttressing effect in plan of return walls at the corners of the excavation may have been taken into account in the wall design. In practice, the length of each basement wall section between return angles considerably influences the amount of horizontal movement after bulk excavation at the critical mid-point between corners of any basement.

It should be mentioned that the hoisting and lowering of diaphragm wall cages may cause displacement of steel and, in the case of prestressed walls, displacement of the tendon. Checking the tendon position, and correcting if necessary, assumes considerable importance when the tendon force and actual eccentricity are viewed together.

A typical tendon/reinforcement cage detail is shown in Fig. 4.32 after Fuchsberger and Gysi.[21] Although designs have been prepared to use large diameter high-tensile steel bars of the Lee–McCall type with anchorages cast in to the base of the wall panel, this method has not been used to date, to the Author's knowledge. While the dependence on sound anchorage to each bar is obvious using this method, the compressive stresses and yielding induced in concrete within the anchorage zone of looped tendons at stressing should not be overlooked.

Soldier pile tremie concrete method

There have been some variations on the Berlin method of soil support; by using a mass concrete wall placed under bentonite slurry to replace horizontal lagging timbers, the SPTC method has found favour in the USA. The reinforced concrete walls are of limited span and are dependent on subsoils of low overbreak risk. The method was reported by Peck.[1]

Mesh-reinforced gunite has been used successfully to replace the mass concrete walling used in the SPTC method. Figs 4.33 and 4.34 show basement excavations

Fig. 4.33. Composite use of contiguous bored pile walls and gunited steel soldier beams retained by ground anchors, Stuttgart (courtesy of Bauer)

Fig. 4.34. Underpinned walls (left and centre) and walls retained by gunite overmesh between anchored steel soldiers, Linz (courtesy of Bauer)

in Stuttgart and Linz, respectively, where gunite over mesh has retained subsoil between anchored vertical steel beams. The absence of groundwater is a pre-requisite for the successful use of gunite over mesh.

Construction economics

Previously in this chapter the distinction was drawn between excavation methods and the alternatives of peripheral sheeting construction. The permutations of each are considerable. The choice of excavation method may well be constrained by factors other than cost, such as environmental matters like noise, vibration and site traffic, short completion times or the proximity of adjacent structures and services, etc. The choice of peripheral sheeting may, however, be made with greater freedom. Some constraints other than cost will no doubt still apply (noise and vibration levels vary from one sheeting installation to another, for instance), but within the choice of peripheral sheeting method lies considerable scope for economy and careful comparisons should be made method by method on each excavation project.

Before this, however, a number of conclusions can be drawn regarding the practical application of the peripheral wall or sheeting methods.

(*a*) High-torque CFA methods are limited to 35 m depth in medium diameters, less in larger diameters.
(*b*) Low-torque CFA methods are limited to 18 m depths and pile deviations may prove limiting.
(*c*) In ground without obstructions or rock, CFA methods generate the least noise and vibration.
(*d*) In particular cases (e.g. loose sand and gravel over a very stiff clay), CFA methods can cause over-excavation and lead to settlement of adjacent structures. Generally, however, CFA methods are suitable for working close to existing structures.
(*e*) For temporary works, the hard–soft secant wall constitutes an efficient wall solution. For permanent works, however, durability and strength considerations may limit its application.
(*f*) Cased secant methods provide a high degree of risk-free excavation in difficult soil or rock conditions or in granular soils near existing foundations.
(*g*) The watertightness of hard–hard secant piling can reach that of a well-built diaphragm wall. With increasing depth, the risk of gaps between secant piles increases, with resulting lack of watertightness.
(*h*) For walls greater than 25 m deep, cased secants with high-torque rigs and diaphragm walls are the only methods available. At greater depths, 40 to 45 m, diaphragm walls are the only suitable method.
(*i*) Box outs are more easily accommodated in diaphragm walls than in secant piles.
(*j*) Construction speed and job size affects wall method selection. Where progress is unimpeded by rock or other obstructions production rates vary in the range from 20 m^2 per day for a classical secant rig, 50 to 60 m^2 for a high-torque cased secant rig, and 70 to 80 m^2 for a CFA secant rig. Diaphragm wall grab units produce 60 to 100 m^2 per day, while hydraulic cutters can produce in excess of 200 m^2 per day (all 12 h day shifts). It is possible to use several rigs on larger sites if a slower technique is chosen.

A comparison by Sherwood *et al.*[22] of relative costs of various walling techniques in different ground conditions and site circumstances is shown in Table 4.1. Bearing in mind that market fluctuations can cause periodic changes in secant pile and diaphragm wall prices, the conclusions that may be drawn from this table are that diaphragms and hard–soft piles are comparable in cost in reasonable soils, sands and fills, where obstructions are few, but high-torque rigs using hard–hard

Table 4.1. Relative costs of various walling techniques in different ground conditions and site circumstances (Sherwood et al.[22])

Ground conditions and site circumstances	Wall thickness (mm)	CFA hard/soft secant low-torque rigs	CFA hard/hard secant high-torque rigs	Classical cased oscillator hard/hard secant	Cased hard/hard secant high-torque rigs	Grab diaphragm wall	Hydraulic cutter diaphragm wall
Sands and fine-grained	<650	1·0	1·4	—	1·6	1·05	1·1
fills.	650–800	1·0	1·2	—	1·35	1·0	1·1
No obstructions.	850–1000	—	1·25	1·4	1·15	1·0	1·1
Open site	1050–1200	—	—	1·05	1·15	1·0	1·05
	1200–1500	—	—	—	—	1·0	1·1
Clays and fine-grained	<650	1·0	1·3	—	1·5	1·0	—
fills.	650–800	1·0	1·15	—	1·3	1·0	—
No obstructions.	850–1000	—	1·25	1·4	1·15	1·0	—
Open site	1050–1200	—	—	1·1	1·15	1·0	—
	1200–1500	—	—	—	—	1·0	—
Sands and fills containing	<650	—	1·0	—	1·2	1·05	1·1
some wood, bricks, etc.	650–800	—	1·0	—	1·1	1·05	1·1
and brickwork.	850–1000	—	—	1·15	1·0	1·0	1·1
Open site	1050–1200	—	—	1·0	1·0	1·0	1·05
	1200–1500	—	—	—	—	1·0	1·1
Clays and fills containing	<650	—	1·0	—	1·25	1·05	—
some wood, bricks, etc.	650–800	—	1·0	—	1·15	1·05	—
and brickwork.	850–1000	—	—	1·2	1·0	1·0	—
Open site	1050–1200	—	—	1·0	1·0	1·0	—
	1200–1500	—	—	—	—	1·0	—
Sands, clays and fine-	<650	1·0	1·4	—	1·6	1·25	—
grained fills.	650–800	1·0	1·2	—	1·35	1·2	—
No obstructions.	850–1000	—	1·05	1·2	1·0	1·0	—
Small congested site	1050–1200	—	—	1·0	1·0	1·05	—
	1200–1500	—	—	—	—	1·0	—
Sands, clays and fills	<650	—	1·0	—	1·2	1·25	—
containing some wood,	650–800	—	1·0	—	1·1	1·25	—
bricks, etc. and brickwork.	850–1000	—	—	1·5	1·0	1·2	—
Small or congested site	1050–1200	—	—	1·0	1·0	1·2	—
	1200–1500	—	—	—	—	1·0	—
Sands, clays and fills containing heavy obstructions including mass concrete, some steel etc. Open or small congested site	All thicknesses	—	—	—	1·0	—	—
All sandy formations with substantial rock layers. Open site	All thicknesses	—	—	—	1·25	—	1·0
All clayey formations with substantial rock layers	All thicknesses	—	—	—	1·0	—	—
All formations with substantial rock layers. Small or congested site	All thicknesses	—	—	—	1·0	—	—

secant methods are necessary where there are heavy obstructions or substantial rock thicknesses. Hydraulic cutters for diaphragm walls are favoured cost-wise for sandy formations with substantial rock layers on large open sites.

Reference should also be made to mobilization costs for walling equipment. A low-torque CFA rig can be mobilized for less than £10 000, while a high-torque cased secant machine can cost £70 000 to £100 000 to establish on site. Diaphragm wall mobilization costs range from £50 000 for a single rope grab unit with equipment, to more than £100 000 for a single Hydrofraise unit with equipment. Minimum economical job sizes are therefore probably of the order of 1500 to 2000 m^2 for grab excavation and 5000 m^2 for Hydrofraise or Trenchcutter work. A further discussion of the costs of walling basement construction is included in chapter 8.

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5 Design of vertical soil support

This chapter addresses the key design items of earth (and water) pressure on a vertical wall and the analysis of the wall to withstand these pressures. Cantilevered, single propped (or anchored) and multi-propped walls will be considered for both temporary and permanent works.

Earth pressures

Limiting horizontal pressure

Mohr's circle of stress can be used to illustrate graphically the limiting horizontal soil pressures which can be generated in active or passive states at either side of a retaining wall. The pressures will be considered in terms of effective stress, ignoring, for the time being, the effects of wall friction or wall adhesion.

The problem of assessing earth pressures can be solved using the Coulomb equation for shear strength $\tau' = c' + \sigma' \tan \phi'$, where c' represents effective soil cohesion, σ' represents effective vertical stress, and ϕ' represents the angle of shearing resistance in terms of effective stress. Figure 5.11 shows the introduction of effective stress parameters, the total vertical stresses and water pressures (and hence effective vertical stresses) to calculate active and passive earth pressure according to whether horizontal stress is less than or greater than vertical stress. Mohr's circles are seen to touch the failure envelope to give limiting values in the extremes of minimum and maximum horizontal stresses.

Using active and passive earth pressures coefficients,

$$\sigma'_H = K_a \sigma'_v \tag{39}$$

and

$$\sigma'_H = K_p \sigma'_v \tag{40}$$

where

$$K_a = \frac{1 - \sin \phi'}{1 + \sin \phi'} \tag{41}$$

and

$$K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} \tag{42}$$

Table 5.1 Estimation of ϕ' for cohesive soils (CIRIA[1])

Plasticity index	ϕ'
15	30°
20	28°
25	27°
30	25°
40	22°
50	20°
80	15°

The values of ϕ' and c' can be obtained for clay samples from drained triaxial tests or undrained tests with pore-water pressure measurement. The effects of stress level, rate of strain for testing, the degree of weathering and of sample swelling before the test, will all influence the test result. When test results are not available for ϕ', the relationship in Table 5.1 may be used for conservative values. The design parameters ϕ' and c' will cause considerable variation in values of limiting soil pressure, and while it is not unusual to assume c',=0, the effect of this assumption is to reduce to low values the limiting passive pressures immediately below dredge or formation level. In turn, this tends to produce high wall or sheeting moments and increased strut or anchor loads near this level.

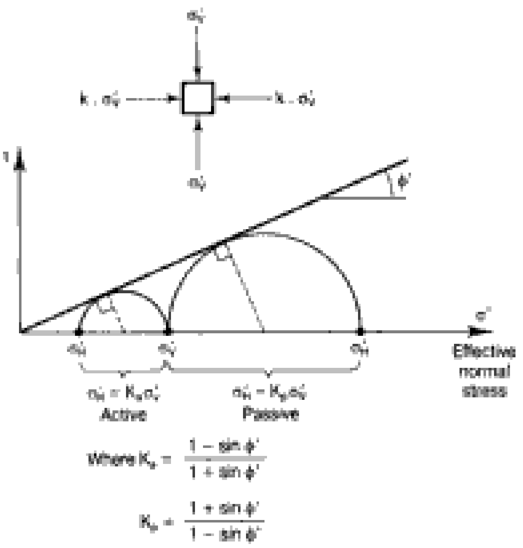


Fig. 5.1. Mohr's circle values of limit pressures and values of K_a and K_p

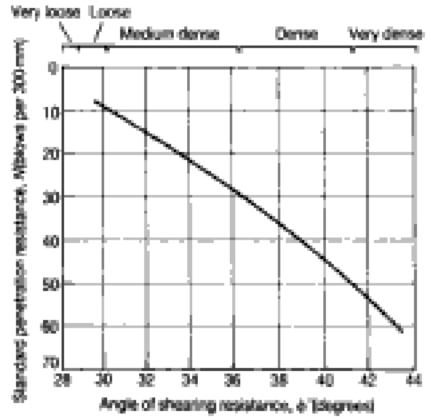


Fig. 5.2. Relationship between standard penetration resistance and φ' (CIRIA[1])

Table 5.2 Estimation of φ' for weak rocks (BS 8002[2])

Stratum	φ'
Chalk	35°
Clayey marl	28°
Sandy marl	33°
Weak sandstone	42°
Weak siltstone	35°
Weak mudstone	28°

Note 1. the presence of a preferred orientation of joints, bedding or cleavage in a direction near that of a possible failure plane may require a reduction in the above values, especially if the discontinuities are filled with weaker materials
Note 2. chalk is defined here as unweathered medium-to-hard, rubbly or blocky chalk

The values of φ' in cohesionless soils should be based on in situ tests where possible. The relationship between standard penetration resistance and φ' is shown in Fig. 5.2. Table 5.2 gives approximate values for the effective angle of shearing resistance of soft rocks considered, somewhat conservatively, as mass of granular fragments (from reference 2).

Horizontal earth pressure: at rest

In an undisturbed soil mass the horizontal earth pressure, the at rest pressure is $K_o\sigma_v'$, where K_o is the ceofficient of earth pressure at rest and σ_v' is the effective vertical stress. For normally-consolidated soils K_o is approximately equal to the empirical expression

$$K_{ONC} = \frac{\sigma_H'}{\sigma_v'} = 1 - \sin\phi' \qquad (43)$$

For normally-consolidated clay K_{ONC} lies in the range 0·55 to 0·65. For over-consolidated clays the removal of considerable overburden pressures within geological time leaves horizontal stress as the major principal stress. The reduction in vertical stress is shown at any depth by the overconsolidation ratio (OCR), which is the ratio of the maximum vertical effective stress ever experienced by the soil to the current vertical effective stress at that depth. The value of σ_v' can be obtained from laboratory consolidation tests and the general relationship between K_o and the OCR is given in Fig. 5.3 after Brooker and Ireland.[3] The maximum

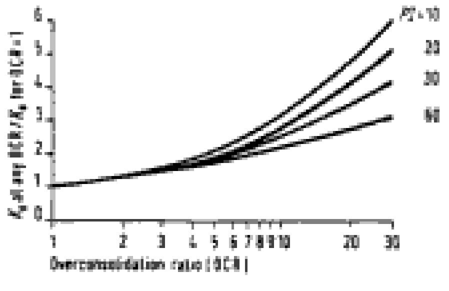


Fig. 5.3. Relationship between coefficient of earth pressure at rest K_0 and overconsolidation ratio (Brooker and Ireland[3])

value of K_o may lie in the range 2 to 3 for some clays, reducing with depth since OCR reduces with depth.

Horizontal earth pressure based on long-term, drained, effective stress values

The limiting horizontal active and passive earth pressures acting on the wall at any depth z are given, respectively, by

$$p_a' = K_a\sigma_v' = K_a(\gamma z+q-u) \tag{44}$$

and

$$p_p' = K_p\sigma_v' = K_p(\gamma z+q-u) \tag{45}$$

and for a soil with a cohesion intercept

$$p_a' = K_a(\gamma z+q-u)-2c'(k_a)^{1/2} \tag{46}$$

and

$$p_p' = K_p(\gamma z+q-u)+2c'(K_p)^{1/2} \tag{47}$$

where γ is the bulk soil density (saturated density if below water level), q is the surcharge on ground surface, and u is the pore-water pressure. These values, known as Rankine values, are subject to change when the effects of wall friction and wall adhesion are considered. Walls are not made of perfectly smooth material and wall friction δ and adhesion c_w vary soil stresses proportionately.

The limiting effective active and passive pressures acting horizontally at a depth z are:

$$p_a' = K_a(\gamma z+q-u)-K_{ac}c' \tag{48}$$

and

$$p_p' = K_p(\gamma z+q-u)+K_{pc}c' \tag{49}$$

where

$$K_{ac} = 2\left(K_a(1 + \frac{c_w}{c'})\right)^{1/2} \tag{50}$$

and

$$K_{pc} = 2\left(K_p(1 + \frac{c_w}{c'})\right)^{1/2} \tag{51}$$

c' is the effective shear strength.

The pore-water pressure is added to the effective horizontal earth pressure to give the sum of earth and water pressure: $p_a=p_a'+u$ and $p_p=p_p'+u$.

The earth pressure coefficients K_a and K_p have been calculated for various curved failure surfaces; those frequently used are due to Caquot and Kerisel[4] and are based on a logarithmic spiral surface as shown in Figs. 5.4 and 5.5. The coefficients derived form these figures have been corrected to give horizontal pressures although the actual pressures are inclined at the angle of wall friction δ to the horizontal. Value of K_{ac} and K_{pc} can be calculated from the expressions above.

Horizontal earth pressure based on short-term undrained stress values

In fine-grained soils such as fine silts and clays, the relatively low permeability only allows slow changes in moisture content, pore pressure and overall volume. The instant that load is applied, pore pressure increases and the shear strength

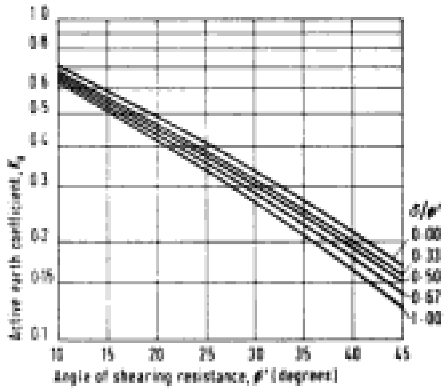


Fig. 5.4. Coefficient of active earth pressure (horizontal component) for a horizontal retained surface (Caquot and Kerisel[4])

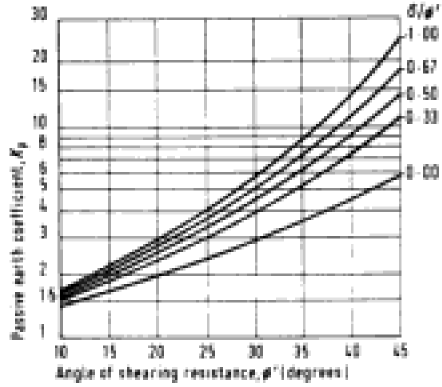


Fig. 5.5. Coefficient of passive earth pressure (horizontal component) for a horizontal retained surface (Caquot and Kerisel[4])

of the soil, which depends on the locked-in effective stress, is only able to increase slowly as pore pressures reduce. This immediate shear strength, the undrained value c_u, applies before any volume change and pore pressures alter or, expressed more exactly, the value of c_u can only be used where a new system of boundary stresses is imposed but prior to any dissipation of the new excess pore water pressures. The undrained shear strength c_u is only correctly used immediately load is applied and it is strictly illogical to use it as soon as pore pressures change. As effective stresses change with pore pressure, shear strength also changes. In clay soils, where the retaining wall structure deforms and attempts to move away from the retained soil bulk, negative pore pressures are generated in the retained soil as excavation proceeds in front of the wall. In highly fissured or laminated clays the reduction in negative pore pressure may proceed relatively quickly and the original value of c_u quickly becomes inapplicable. The use of c_u in such clays can therefore become over-optimistic in retaining wall analysis when the effective stress (on which soil strength depends) reduces as negative pore pressure relaxes.

In soft and very soft homogeneous clays which do not benefit from drainage paths due to fissuring and jointing, low overall permeability of the soil fabric may prevent more efficient change in pore pressure, and the use of c_u, although strictly applicable at the instant of stress change prior to pore pressure change, can be more satisfactorily relied upon. Indeed, for retaining walls founded in soft clays it is prudent to obtain both drained and undrained soil strengths and undertake analysis of both total and effective stress conditions. Temporary works design in these soils, in which the wall structure may have a limited design life, say, of six months, may therefore use total stress methods with some confidence, and the design parameter of undrained shear strength can be conveniently and accurately obtained from in situ shear tests, such as vane tests. Where such short-term conditions do not apply, either by the greater permanence of the temporary wall or due to its role in the permanent structure, analysis of drained effective stress becomes necessary. The choice between the two methods, undrained or drained, depends strictly on the rate of change of pore pressure which, in turn, depends on the permeability of the structure, although it must be restated that the undrained analysis is only strictly applicable immediately before any dissipation commences. In practice, especially for small schemes, sufficient undrained shear measurements may be more readily available than test results from drained samples. However, the availability of test results should not excuse the application of an illogical method of analysis. The short-term undrained method is therefore presented here with the recommendation that its use should be restricted to soft clay conditions for temporary works of short duration. Schemes of longer duration should be checked by both drained and undrained analyses.

In terms of undrained conditions the limiting horizontal active and passive earth pressures acting on the wall at any depth z are given by

$$p_a = K_a\sigma_v - K_{ac}c_u = K_a(\gamma z + q) - K_{ac}c_u \tag{52}$$

and

$$p_p = K_p\sigma_v + K_{pc}c_u = K_p(\sigma_v + q) + K_{pc}c_u \tag{53}$$

The pressure coefficients are

$$K_{ac} = 2\left(1 + \frac{c_w}{c_u}\right)^{1/2} \tag{54}$$

and

$$K_{pc} = 2\left(1 + \frac{c_w}{c_u}\right)^{1/2} \tag{55}$$

Tension cracks

The active pressure near ground level will be a minimum for the retaining wall and, in cohesive soils, surface tensions within the clay can cause cracks which can fill with groundwater or rain water. In this case, the water pressure within the potential crack should be allowed as a pressure on the back of the wall. Wall adhesion will not apply over the depth of the crack, which is $(2c_u - q)/\gamma$.

Minimum values of calculated active pressure

In the UK the prolonged use, from 1951 until 1994, of the Civil Engineering Code of Practice No. 2[3] for earth retaining structures indicated both the sound principles on which it was based and the difficulties which have been experienced in agreeing revisions to it. Although the Code was based on calculation of earth pressure in terms of total stress and undrained shear values, it prudently recommended the use of minimum active pressures for design irrespective of those calculated from test values of c_u using the expression $p_{an} = K_a\sigma_v - K_{ac}c_u$.
The Code stated that for cohesive soils the methods of calculation were not final. In the design of structures to retain cohesive soils, a suitable addition should be made to the calculated active pressure of the soil in order to provide for uncertainties. Where, as may be the case with stiffer clays, the total pressure on the wall as calculated according to the Code was small, the value of the total pressure to be assumed for the purposes of design should not be less than that found by assuming the horiztonal pressure at any depth to be that due to a fluid with a density of 30 lb/ft^3 (5 kN/m^3). This safety net for minimum pressure as the active side of the wall was most prudent but did not find mention in later Codes.

Softening of clays

The process of clays softening is accelerated over time by tension cracks near the ground surface, by the joints and fissures which frequently occur in stiff, over-consolidated clays, and by the laminations of silt or sandy silt partings which can occur in all clays. These routes for easier passage of water not only increase the permeability of the soil fabric but can form failure surfaces themselves of softened planes of clay dividing stiffer, unsoftened blocks of clay bounded by softened, lubricated surfaces. The changes in pore-water pressure as these pressures equalize with hydrostatic groundwater levels are therefore also associated with a loss of clay strength due to softening along surfaces such as joints, fissures, laminations and tensile zones. For this reason it is often prudent to assume a complete loss of cohesive strength in terms of effective stress as the clay softens with time along preferred drainage surfaces.

Wall friction and wall adhesion

The effect of assumptions regarding wall friction in granular soils and adhesion in clay soils on calculated values of active and passive pressure is considerable, and while it is self-evident that wall surfaces are not smooth, design values of friction and adhesion should be chosen carefully. The mobilization of wall friction, to reduce earth pressures on the active side and increase earth pressure on the passive side, necessitates the downwards movement of a soil wedge on the active side and the upwards movement of a similar soil wedge on the opposite, passive side, as shown in Fig. 5.6. Important changes to these relative soil and wall movements are caused by the downwards vertical wall movement due to vertical loads on the wall and the vertical load components of inclined pre-loaded ground anchors, both of which tend to mobilize passive wall friction/adhesion but tend to reduce active friction/adhesion.

Where neither vertical load nor the effects of ground anchorages apply, values of wall friction originally recommended by the Code appear to be realistic. Table 5.3 shows values of active wall friction/adhesion. For passive walls, use 50% of the active values.

For temporary works using sheet piling for temporary cofferdams, the CIRIA report[1] recommends the values shown in Table 5.4. The relative movement between wall and soil necessary to generate wall friction or adhesion may not always occur; where the toe of the wall is founded on hard rock, wall friction/adhesion values quoted in Table 5.4 should be reduced by 50% for dense granular materials and stiff overconsolidated clay. For overlying loose granular soil, where the wall is founded on rock, wall friction/adhesion should be ignored. For anchored walls where there is a tendency for the wall to move upwards relative to the soil, zero wall friction should be allowed.

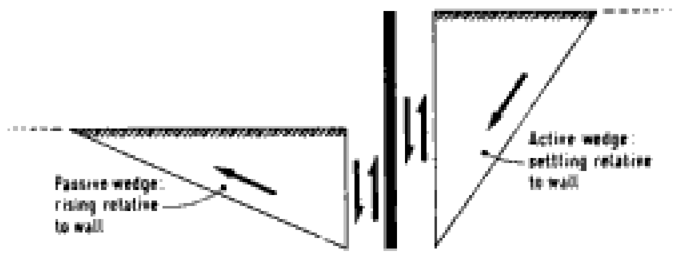


Fig. 5.6. Wall friction acting on active and passive soil wedges (Padfield and Mair[6])

Table 5.3 Values of active wall friction angle and adhesion

Construction	Wall friction angle	Wall adhesion
Concrete, brick	20°	Where c_u is less than 50 kN/m^2 use $c_w = c_u$
Steel piling, tar/bitumen coated	30°	Where walling or sheeting does not penetrate any appreciable depth use $c_w = 0$
Uncoated sheet piles	15°	Where $c_u > 50$ kN/m^2 use $c_w = 50$ kN/m^2

Table 5.4 Active and passive values of wall

	Wall friction angle, δ	Wall adhesion
Active	$0{\cdot}67\ \phi$	$0{\cdot}5\ c'$ or $0{\cdot}5\ c_u \ngtr 50$ kN/m^2
Passive	$0{\cdot}5\ \phi'$	$0{\cdot}5\ c'$ or $0{\cdot}5\ c_u \ngtr 25$ kN/m^2

Neither wall adhesion nor wall friction should be allowed where the proximity of machinery, railways or vehicular traffic causes vibrations which could be transmitted through the subsoil to the wall surface.

With cast in situ reinforced concrete diaphragm walls and reinforced concrete piled walls cast under bentonite slurry, values of wall friction/adhesion for active and passive pressure can be assumed to be similar to those for concrete walls given in Table 5.3.

Magnitude of movement needed to mobilize limit pressures

The variation in horizontal pressure, either active or passive, generated by movement of the wall away from or towards the soil mass, is shown in Fig. 5.7. The curve, first derived by Terzaghi and Peck,[17] was based on tests in dense sand. Two conclusions are evident:

- the passive coefficient is much higher than the active coefficient
- the deformation needed to fully mobilize the limit passive pressure is much greater than that required to fully mobilize the limit active pressure.

Padfield and Mair[6] illustrated the relative deformation needed to fully mobilize K_a and K_p by means of the stress path followed by a soil element behind the wall in both normally- and over-consolidated clay conditions. On the active side there is no change in vertical stress due to excavation, while on the passive side the effective stress path in drained loading may take varying directions depending on the relative importance of the relief of overburden to excavation and the horizontal pressure exerted on the wall.

The stress change needed to fail a normally-consolidated clay in active pressure is much less than that required to fail the same clay in passive pressure. The stress path for over-consolidated clay shows the reverse to be true, however. Terzaghi and Peck's[7] plot of wall movement against mobilized pressure for dense sand may not therefore, accurately show the strains necessary to generate limit active and passive pressures in both normally- and over-consolidated clays.

The publication of BS 8002[2] introduced the concept of limit state design, with greater emphasis on wall conditions at the serviceability limit state than collapse conditions at the ultimate limit state. The basis of this method is the reduction of soil peak shear strength values on both the active and passive sides of the wall, to values which would be representative of mobilized shear strengths with the permitted wall movement. BS 8002[2] specified that where wall displacements are required to be less than 0·5% of the wall height, the representative undrained shear strength should be divided by a mobilization factor M not less than 1·5. For designs using effective stress values, the value of M was specified to be 1·2 to reduce horizontal wall movements to 0·5% of wall height. For effective stress design, the calculation of wall equilibrium and the structural components of the wall was specified to be based on the lesser of:

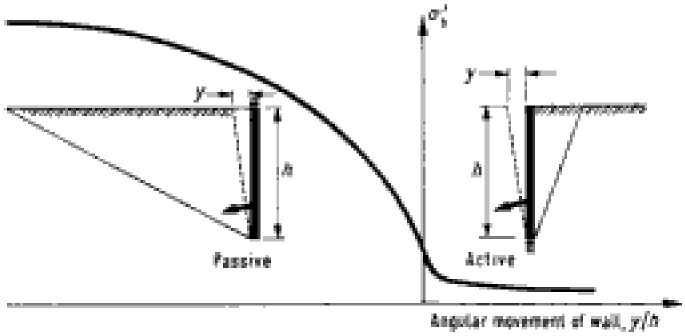


Fig. 5.7. Relationship between strain required to mobilize active and passive pressure for medium dense sand (Padfield and Mair[6])

(*a*) the representative peak strength of the soil divided by a factor $M = 1 \cdot 2$,

$$\text{designation } \phi' = \frac{\text{representative } \tan \phi'_{\max}}{M}$$

$$\text{design } c' = \frac{\text{representative } c'}{M}$$

(*b*) the representative critical-state strength of the soil.

Increased earth pressures on the active side and reduced earth pressures on the passive side at the serviceability limit state due to the use of the reduced peak shear strength values are sufficient, the code states, to allow structural design of the wall components normally to be made without the application of partial load factors to bending moments and internal forces derived from the earth pressures.

This principle of design using soil strengths which can be mobilized at working conditions in the serviceability limit state is logically extended in BS 8002 to consideration of wall friction and wall adhesion. It recommends that the design value of friction or adhesion at the interface with the wall should be the lesser of either the values obtained from tests or 75% of the design shear strength to be mobilized in the soil itself, that is, using

$$\text{design } \tan \delta = 0 \cdot 75 \times \text{design } \tan \phi'$$

$$\text{design } c_w = 0 \cdot 75 \times \text{design } c_u$$

and since for the soil mass $M = 1 \cdot 2$,

$$\text{design } \tan \phi' = \frac{\text{representative } \tan \phi'}{1 \cdot 2}$$

this is equivalent to

$$\frac{\text{design } \delta}{\text{representative } \phi'} = \frac{2}{3}$$

and, similarly, in total stress analysis

$$\frac{\text{design } c_w}{\text{representative } c_u} = 0 \cdot 5 \quad \text{after taking } M = 1 \cdot 5$$

The overall effect of reducing peak shear strengths to shear strengths which are mobilized in working conditions by using such M values as are specified in the code may not be necessary when walls are designed in stiff over-consolidated clays, where relatively small movements are necesary to mobilize limiting active and passive pressures. On the other hand, the recommendation that bending moments and internal forces derived from earth pressures calculated from mobilized soil strengths (peak strengths reduced by the mobilization factor M) without the application of a further partial safety factor on earth pressures be allowed in designs based on ultimate structural strengths may not accurately reflect variations in load which occur in practice along the length of a wall.

BS 8002 also specified that in checking the wall stability and soil deformation all walls should be designed for a minimum design surcharge loading of 10 kN/m^2 and a minimum depth of additional unplanned excavation in front of the wall. This additional design depth was specified to be not less than 0·5 m or 10% of the total height retained for cantilever walls or of the height retained below the lowest support level for propped or anchored walls. It should be noted that the code is primarily applicable to walls up to about 8 m high, and no differentiation is made between temporary and permanent walls. The imposition of such rules for all walls irrespective of design life would appear to be over-stringent.

Wall flexibility

The stiffer the wall, the greater the earth pressure it attracts and, conversely, the greater flexure of the wall the less pressure (and moment) induced in it by the soil it retains. This phenomenon, originally demonstrated in model tests on an anchored wall by Rowe[8] will be referred to in greater detail later in this chapter.

Consider, however, the difference in deformed shape of a relatively stiff anchored wall and that of an anchored flexible wall, as shown in Fig. 5.8. The flexible wall distorts outwards by a considerable amount at mid-span relative to the stiff wall, causing a reduction in pressure and an effective reduction in vertical span of the wall due to a rise in elevation of the centre of passive support below excavation level. Anchor loads and pressure below excavation level therefore both increase, but bending movement in the wall reduces due to both earth pressure reduction at mid-span and the reduction in the span itself. Due to the movement of the flexible wall away from the soil at mid-span, the soil tends to arch vertically and, providing neither the anchor nor the passive support below formation level yield once mobilized, the abutments to the arch sustain more load and relieve soil pressure from mid-span.

Surcharge loading: sloping ground

The effect of a sloping ground surface can be taken into account using curves due to Caquot and Kerisel[4] reproduced in Fig. 5.9, giving values of K_a and K_p for vertical walls, inclined backfill and wall friction varying from $\delta=0$ to $\delta=\phi'$. The effect of layered strata behind the wall should be taken into account in calculations by using modified coefficients for each layer from the curves. The values presented assume that the sloping ground extends a sufficient distance from the wall to exceed the lateral dimension of the potential critical slip plane of the failure wedge (approximately at an angle of $45°+\phi/2$ to the horizontal from the toe of the wall for the active case, and $45°-\phi/2$ for the passive case).

The CIRIA report[1] gives a simple adjustment to the pressure diagrams (Fig. 5.10(a) and (b)) to show the effect of berms, on the active and passive sides of the wall in cohesionless soil. The report notes that the risk for sliding failure should be checked at the base of the berm, especially where the berm soil has a higher angle of shearing resistance than the soil beneath it. Where a gravel berm overlies a stiff clay, the report suggests it may be too optimistic to use the undrained clay strength in checking sliding.

The NAVFAC design manual[9] gives a graphical method for the calculation of passive soil resistance due to an earth berm, as shown in Fig. 10(c).

Surcharge loading: point loads and line loads

The CIRIA report[1] reproduced a simple method due to Krey for use in granular soils. The horizontal pressures caused by concentrated point and line loads as estimated by Krey are shown in Fig. 5.11 and should be added to the earth pressure diagram.

An alternative method of computing the effects of point and line loads by Terzaghi was reported in the NAVFAC design manual[9] and was commented upon in the

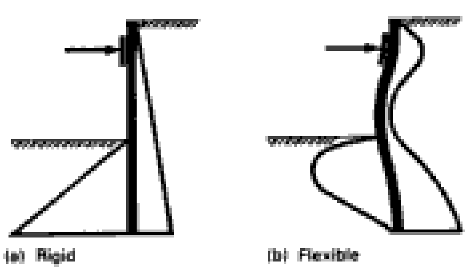


Fig. 5.8. Influence of wall flexibility on pressure distribution: (a) rigid wall; (b) flexible wall (Padfield and Mair[6])

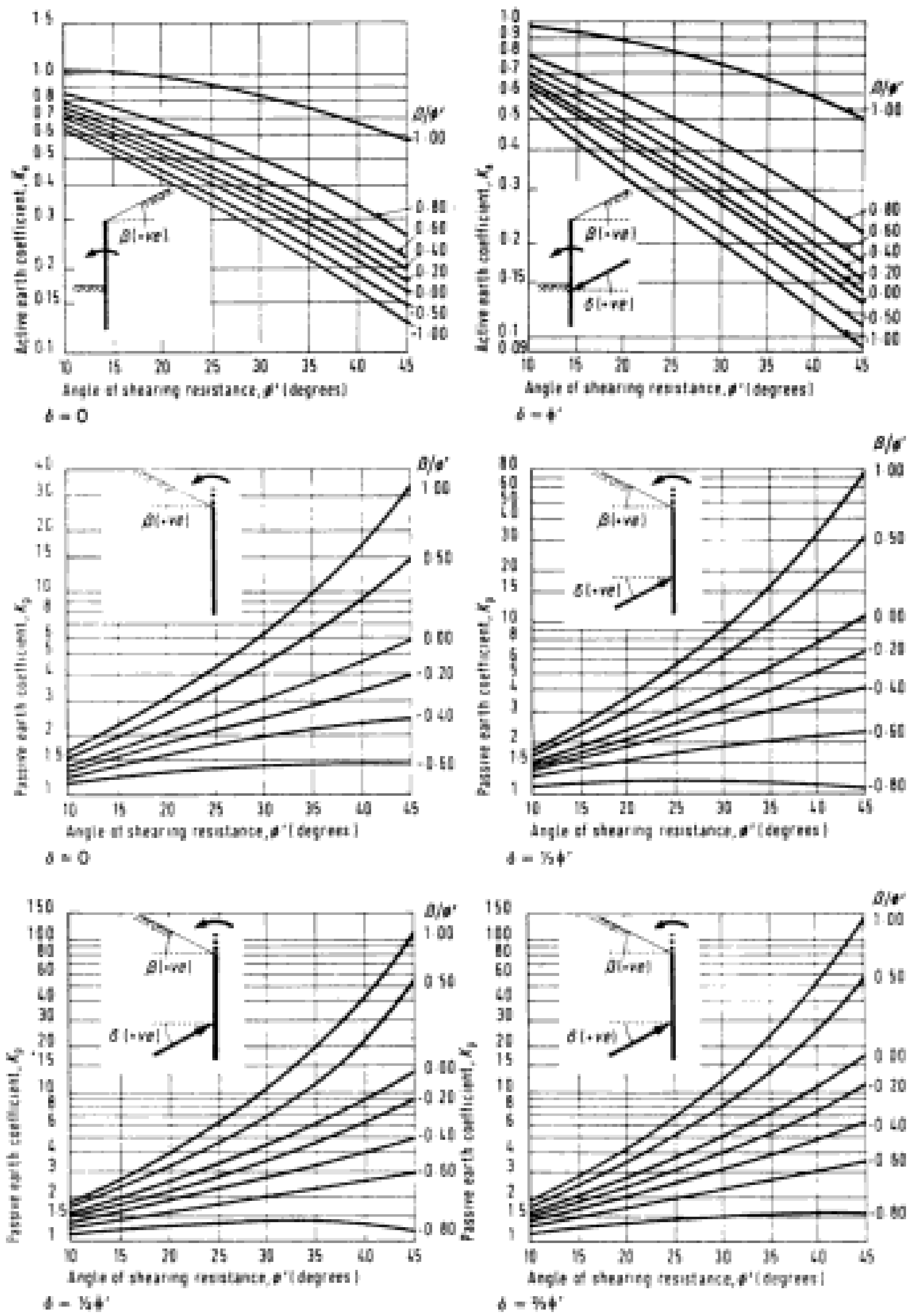
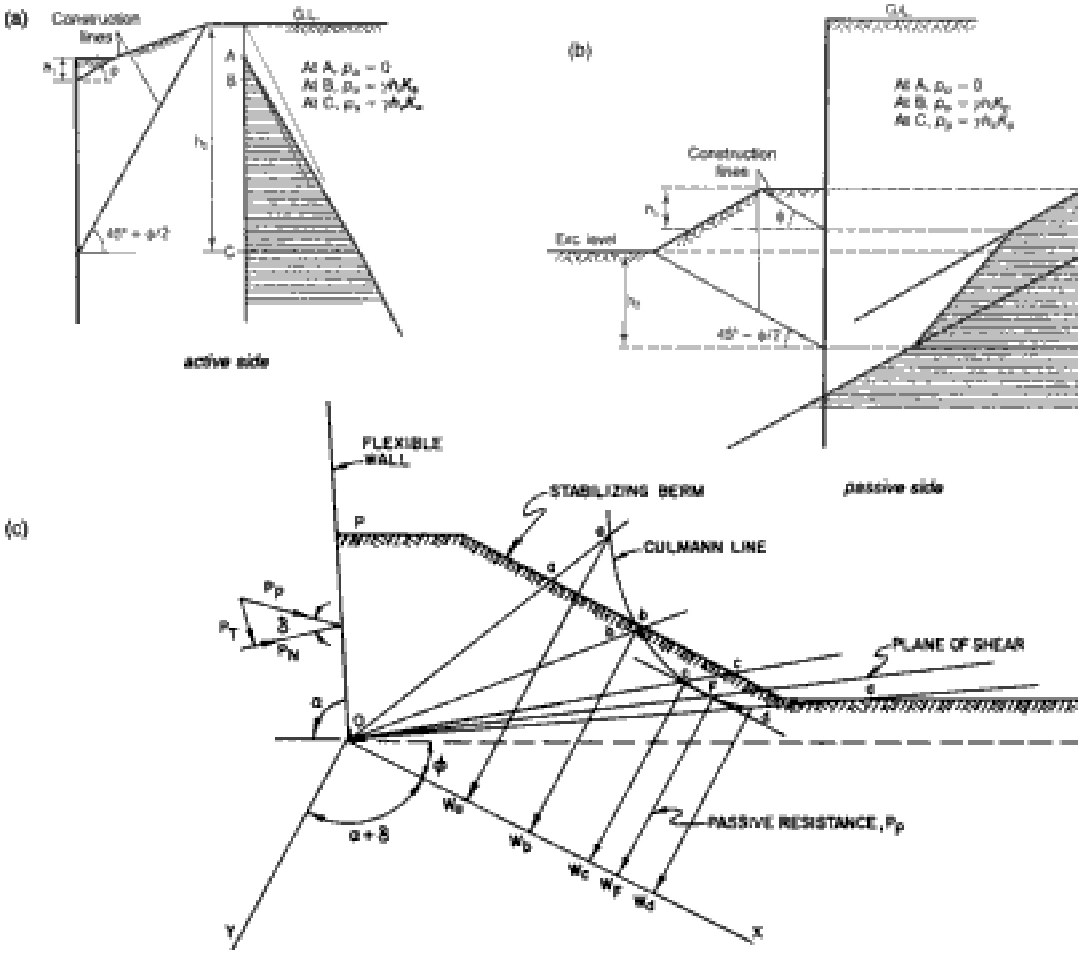


Fig. 5.9. Coefficients of earth pressure (horizontal component) for vertical walls with inclined backfill for varying wall friction δ: (a) active; (b) passive (Caquot and Kerisel[4])

CIRIA report.[1] According to the CIRIA report the method, a modified Boussinesq distribution taking into account field and model measurements, possibly underpredicts the pressure and tends to place the centre of pressure influence too low down the wall. Some caution should be applied, therefore, in the use of the method where the applied loads are high in relation to the weight of soil retained. It should be noted, however, that the method presumes an unyielding rigid wall and the lateral pressures are already approximately double the values obtained from elastic equations. Design charts from NAVFAC[7] for line and point loads are reproduced in Fig. 5.12.



1. Draw berm to scale.

2. Layout OX from point O at angle δ below horizontal.

3. Layout OY from point O at angle $(\alpha+\delta)$ below OX.

4. Assume failure surfaces originating at point O and passing through points a, b, c, etc.

5. Compute the weight of each failure wedge.

6. Layout the weight of each failure wedge along OX to a convenient scale.

7. Draw a line parallel to OY for each failure wedge from its weight plotted on OX to its failure plane (extrapolated where necessary).

8. Connect the intersecting points from 7 above with a smooth curve - this is the Culmann Curve. Draw a tangent to this curve which is also parallel to OX.

9. Through the tangent point F, draw a line parallel to OY to intersect OX at W_F. Distance FW_F is the value of P_p in the weight scale.

10. Normal component of the passive resistance, $P_N = P_p \cos \delta$.

11. To compute pressure distribution on the wall, assume a triangular distribution.

Fig. 5.10. (a), (b) Pressure diagrams showing the effect of sloping ground on the active and passive sides of the wall (CIRIA[1]); (c) Culmann method for determining passive resistance of earth berm (US Navy[9])

Let Q_C = concentrated load in kN
Q_L = line load in kN/m run of wall

Equivalent line load = $\dfrac{Q_C}{2A+L}$ kN/m run of wall over wall length of $(2A+L)$ metres

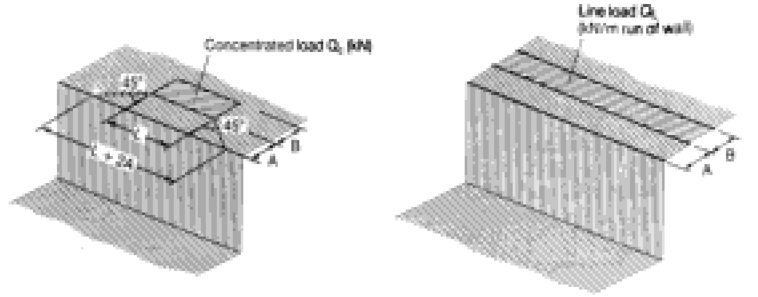


Fig. 5.11. Effect on surcharge loading of point and line loads (CIRIA[1])

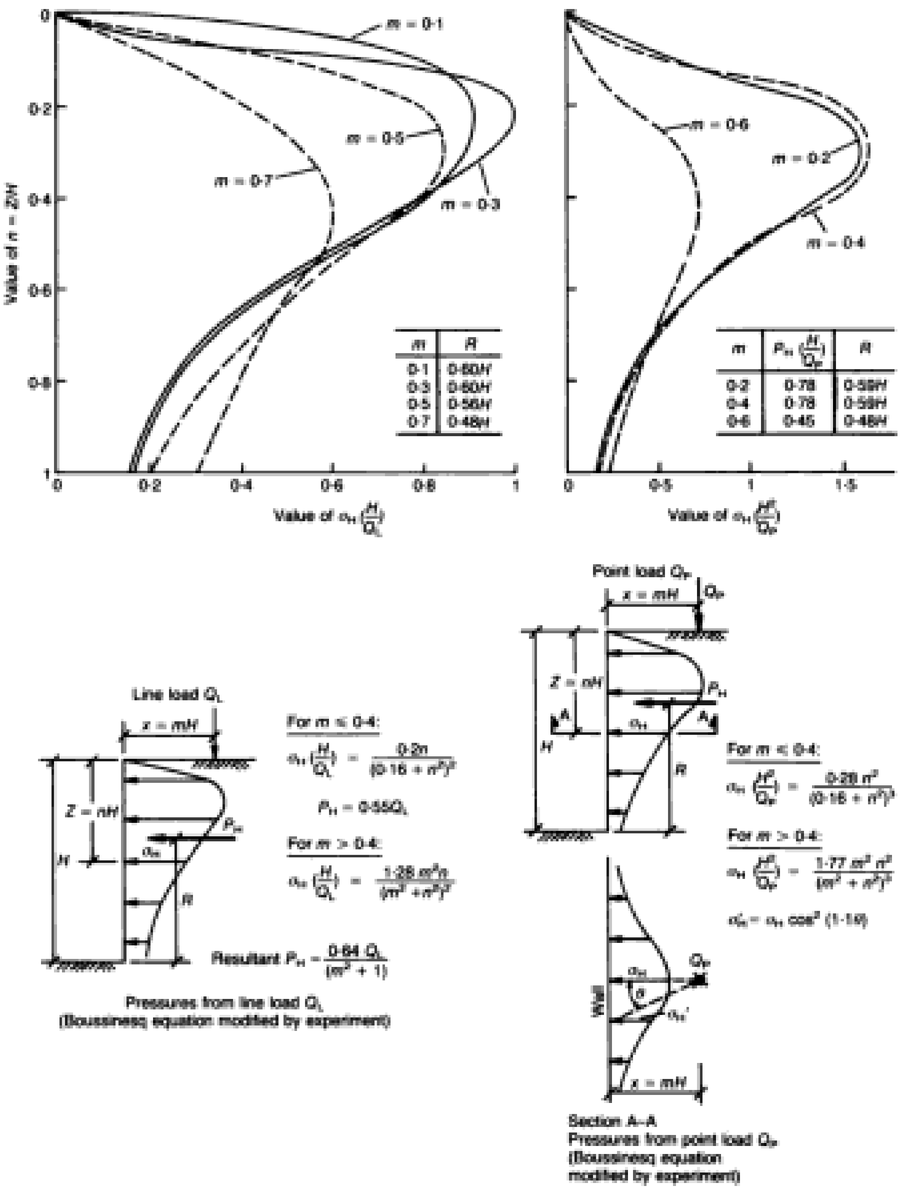


Fig. 5.12. Horizontal pressures on a rigid wall from line (left) and point loads (right) (US Navy[9])

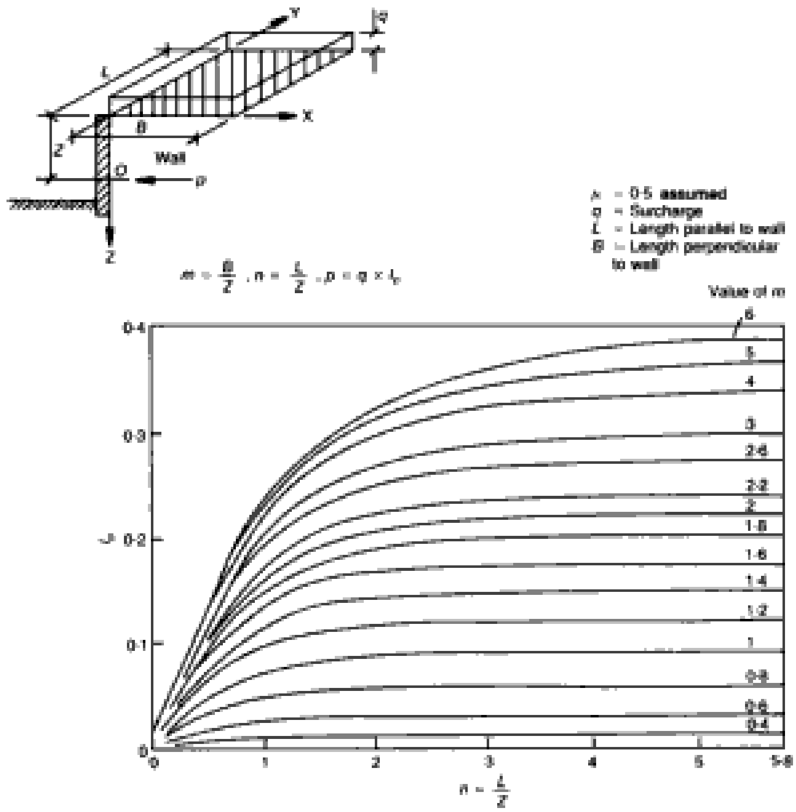


Fig. 5.13. Lateral pressure on an unyielding wall due to uniform surface load (US Navy[9])

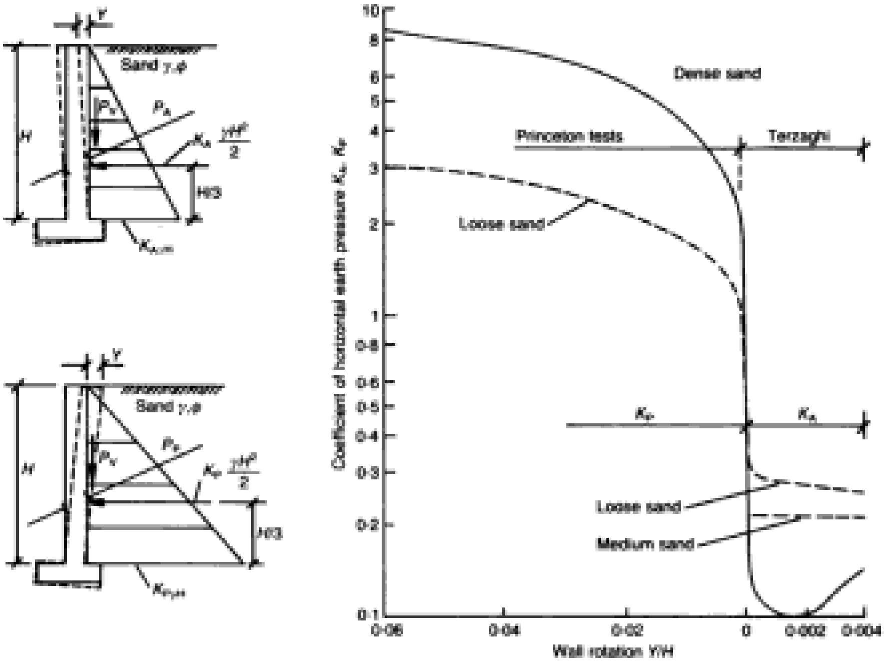
Surcharge loading: uniform rectangular surface load

The NAVFAC design guide also provides the means to estimate the lateral pressure on an unyielding wall due to a uniform rectangular load on the surface at the rear of the wall, after Goldberg *et al.*[10] (Fig. 5.13). The guide comments that any yield of the wall during application of the load will tend to reduce the pressures calculated from the values given by Goldberg *et al.*

Wall movement

The effect of wall movement on lateral earth pressures should be restated because of its vital effect on both active and passive pressures. Earth pressures at rest are reduced to limiting active values by movement of the wall away from the soil; much larger movements are needed to increase the at rest pressure to limiting passive values. The relationship between wall rotation and the coefficient of earth pressure for sands is reproduced in Fig. 5.14. The curve can be used to assess the movement needed to obtain limit pressures in sands of varying compaction state. The NAVFAC guide[9] also suggests that the effects of wall translation on limit pressures can be assessed by assuming the effect of translation is equivalent to the movement at the top of the wall due to rotation, as shown in Fig. 5.14.

The extent of wall movement relative to the retained soil therefore determines that proportion of the limit pressure which applies. This movement may be derived from elastic deflexion of the wall or sheeting itself, the compression which occurs in strutting or raking shores (or the curtailment of such movement by pre-loaded supports or anchors), the movement needed to mobilize passive resistance of a soil at an intermediate stage of excavation in front of the wall, or the rotation or translation of a rigid wall.



Magnitudes of wall rotation to reach failure

Soil type and condition	Rotation Y/H*	
	Active	Passive
Dense cohesionless	0·0005	0·002
Loose cohesionless	0·002	0·006
Stiff cohesive	0·01	0·02
Soft cohesive	0·02	0·04

Y = horizontal displacement
* H = height of the wall

Fig. 5.14. Effect of wall rotation on active and passive pressure in loose and dense sand (US Navy[9])

With the exception of the methods specified in BS 8002,[2] the above means of calculating earth pressures have led to designs based on limit pressures where these do not necessarily apply if the extent of proposed wall movements do not justify them. Soil pressures based on the stiffness of the wall, its support and the soil itself are evaluated more realistically by soil/structure analyses based either on the Winkler spring theory or by finite element methods. These methods are referred to again later in this chapter.

Temporary works

Padfield and Mair[6] conveniently listed the dissimilarities between temporary works and permanent works designs as follows:

In temporary works . . .

(*a*) In clays, negative pore-water pressures are present in the soil. These dissipate with time but while they act they cause the soil to have a greater shear strength than it has in the long term.
(*b*) Support conditions are often different in the temporary works before base slabs, floor slabs or other permanent supports are constructed. Temporary works often rely more on the soil for their support than do permanent works.
(*c*) Loading conditions are often different. Applied surcharges during temporary works construction are often not present in the long term.
(*d*) Ground movements are often a matter of concern during construction, but there is not usually time for long-term movements to develop.
(*e*) A greater proportion of soil strength may be mobilized during temporary

works depending on constraints operating during construction. However, the consequences of excessive ground movements or instability can be as severe as for permanent works.

(*f*) Higher stresses may be permitted in the design of the wall in temporary works.

The report concludes that temporary works may be designed either: by assuming full equilibrium (full drained) pore-water pressures (i.e. long-term strength), the expected loads in the temporary phase and a very low safety factor; or by taking the higher short-term strength and a higher safety factor. (The choice is essentially one of modifying the parameters used and the safety factor adopted in passing from temporary to permanent works phases.)

Mixed total and effective stress design

Since the original ground surface and the soil below it on the active side of the wall are usually more susceptible to softening than the soil below final formation level, a method of calculating earth pressures using effective stress values on the active side and undrained values on the passive side has gained some credibility. The benefit, in terms of calculated earth pressures, is obtained principally from passive pressures based on the limit pressure expression $p_p = \sigma_v + 2c$, that part of the expression relying on cohesion applying at formation level on the passive side. Padfield and Mair[6] recommended that the risk of clay softening on the passive side below formation level can be estimated by reducing the undrained strength/depth profile by 20 to 30% (using 0·7 to 0·8 of the unsoftened design line for c_u) and ignoring the clay strength in the upper 1 m thickness of clay at the surface of the excavation. Overall, the loss of safety for the wall as built will depend on several design matters, which include the choice of parameters, the choice of total or effective design method and the factors of safety that are applied to each component of the wall, the bracing or anchor that supports it, and the extent of embedment of the wall in the soil below formation level. If more optimistic methods are chosen for wall analyses (by using total or mixed total and effective stress methods) these should reflect more stringent factors of safety in the derivation of wall member dimensions. Conversely, if effective stress methods are used throughout, say for temporary works design of limited design life, the final factors of safety in the dimensioning of wall and strutting may be chosen more optimistically. Recommended factors of safety will be reviewed later in this chapter.

Design water pressures

The design of walls to support soil will include consideration of the support of water pressures where there is groundwater above the final excavation level in front of the wall. Where support walls are designed for excavations to cofferdams and shafts below river or sea bed,[3] the pressures from river or sea water levels are evident to the designer and would comprise the major part of pressures on the wall. Where earth support walls are required for excavations on land, the designer has to evaluate the highest ground water levels during the design life of the structure and incorporate these with earth pressures into design pressures for the wall. In excavations on land where groundwater is high, the water pressure will constitute a large proportion of the horizontal force on the active side of the wall, but is a lesser proportion than the earth pressure on the passive side. If water levels rise, the total force on the active side of the wall increases, while if water pressure rise below formation level on the passive side, in most cases total passive resistance from water and soil reduces.

Where vertical soil support walls on land penetrate granular soils with a high water table but obtain a cut-off at depth, the water pressure on each side of the wall is calculated without complication (Fig. 5.15). The hydrostatic pressures on each side of the wall are calculated from $u = \gamma_w x$, where γ_w is the density of water

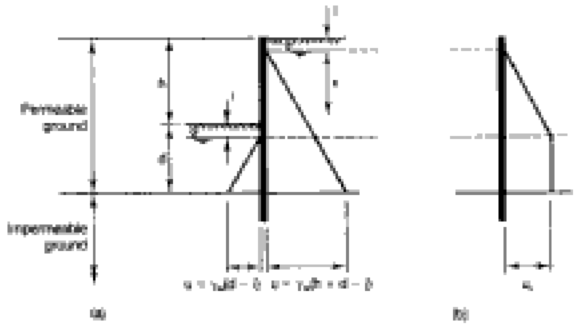


Fig. 5.15. Hydrostatic water pressures on walls in permeable ground with effective cut-off: (a) gross water pressures; (b) net water pressures (CIRIA[7])

(in fresh water it is $9{\cdot}81\,kN/m^3$, in salt water it is $10{\cdot}00\,kN/m^3$), and x is the depth below the groundwater table.

Additional pressures due to heavy rain may need to be considered in defining groundwater levels, and where surfaces are unpaved, full water pressures should be taken in tension cracks in cohesive soils. In cohesionless soils or where drainage is introduced at the rear of the wall, an increase in soil density to its fully saturated value should be allowed for. If the retained cohesionless soil has low permeability but drainage holes through the wall are adequate to discharge the water flow through the retained soil, reference 5 recommended that the effect of the flow of water towards the wall and the increased total active thrust on the wall should be taken into account by calculating total active thrust on a vertical wall

$$P_a = K_a \gamma_b \frac{H^2}{2} + 0{\cdot}5\gamma_w \frac{H^2}{2} \tag{56}$$

where γ_b is the submerged density of the soil, and H is the vertical height of the earth retained.

Reference 5 added that where drainage through the wall was not adequate to avoid an increase in head of groundwater in the retained soil during storm conditions, this increase should be allowed for, and if in extreme conditions, the head can reach the top of the wall, this should also be allowed for.

Other conditions which may affect static groundwater levels considered in designs include the effect of waves on retaining walls for marine structures and river and sea cofferdam walls, and the dynamic forces of waves breaking on beaches which can be transmitted to retaining walls for deep escavations below beach level. Some guidance on these matters may be obtained from the work of Sainflou[11] and Minikin.[12]

Where walls are situated in tidal conditions the pore pressure will vary with the tide; the effect of cycles of loading and any tidal lag may require special consideration. Guidance is available in reference 13.

Where the vertical wall for soil retention penetrates cohesionless soil but does not secure a cut-off in an impermeable stratum at the toe of the wall, the effect of water seepage from the retained soil on the active side of the wall to the passive side should be taken into account. Over time a steady seepage state will develop. In design, the effect on pressure in these conditions may be conveniently assessed by use of flow nets, the construction of which were explained in chapter 2. A flow net illustrating these conditions is shown in Fig. 5.16. The total head at any point is

$$H = H_1 - (H - H_2)\frac{d}{n_d} \tag{57}$$

where H_1 and H_2 are the heads of groundwater on either side of the wall, d is the number of flow net equipotential drops to the point considered, and H_d is the total number of drops. Also, at any point the total head is

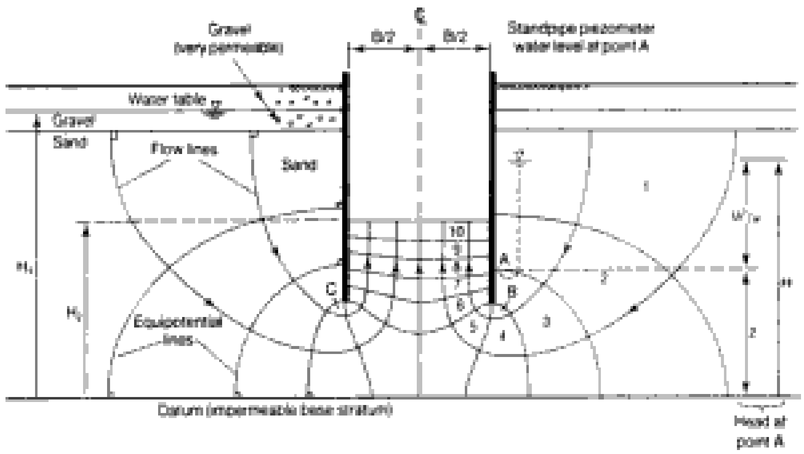


Fig. 5.16. Flow net: flow into base of twin-wall cofferdam in sand (CIRIA[1])

$$H = \frac{u}{\gamma_w} + z \tag{58}$$

where u is the pore-water pressure, and z is the height of point considered above datum. Since H can be calculated by flow net and γ_w and z are known, u can be calculated. The flow net is also used to check the factor of safety against piping (see chapter 2).

Padfield and Mair[6] described a simple short-cut for calculating pore-water pressure at any point by assuming that the head difference between each side of the wall, active and passive, is dissipated uniformly along the flow path; referring to Fig. 5.17, head difference = $(h+i-j)$ and flow path length is $(2d+h-i-j)$.

Therefore, the head at any point x below the water table on the active side is

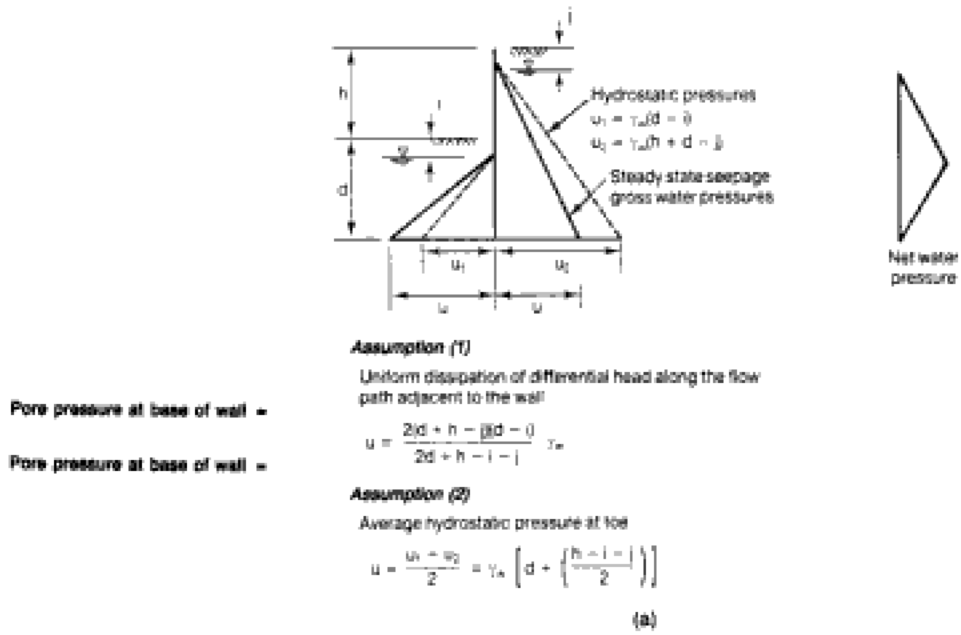
$$H_x = \frac{x}{2d+h-i-j}(h+i-j) \tag{59}$$

and since $u = H\gamma_w - z$, pore-water pressure at toe of wall is

$$u_{toe} = \frac{2(d+h-j)(d-i)}{2d+h-i-j}\gamma_w \tag{60}$$

The CIRIA report[1] refers to another, even simpler, approximation in which the water pressure at the toe of the wall is taken as the average at that depth for active and passive sides of the wall. The report concludes that all uses of steady-state flow are best checked by flow net as a first approximation; the reduction of excess head linearly along the length of flow path at the wall can lead to over-optimistic calculated pore pressures in the narrow flow paths that occur betwen the retaining walls of a narrow cofferdam. In wider cofferdams (where width is greater than four times the differential water head) and single retaining walls where the seepage pressure is free to dissipate horizontally through the passive soils, the first approximation is satsifactory.

The reduction in pore pressure from one side of the wall to the other in the steady-state condition can be assessed by such methods of calculation. More importantly, however, is the effect of preferential drainage paths which can be caused by sand or silt partings in laminated groundwater flow below a vertical wall. Where the pore-water pressure does not reduce gradually because of a horizontal drainage route below the toe of the wall, an excessive pore-water pressure occurs on the passive side, resulting in high net pressures and forces on the wall. Fig. 5.18 illustrates this risk.



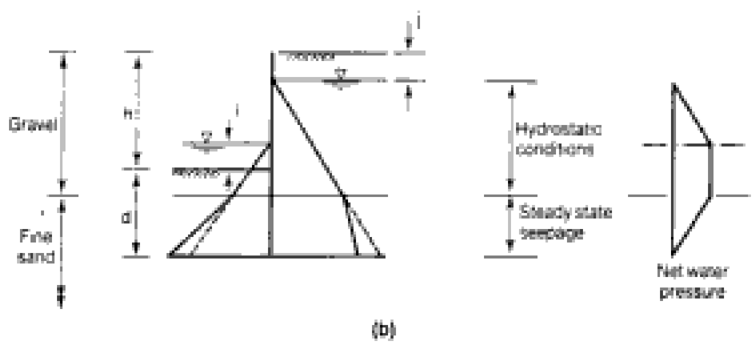


Fig. 5.17. Water pressures — simplifying assumptions for steady-state conditions: (a) uniform ground; (b) layered ground (Padfield and Mair[6])

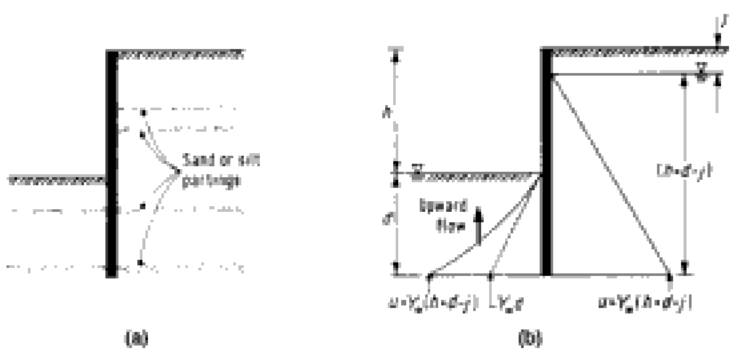


Fig. 5.18. Excessive pore-water pressure on passive side due to underflow: (a) soil cross-section showing pervious inter-layers; (b) water pressure diagram (Padfield and Mair[6])

Design of cantilever walls and single-prop walls

The design of cantilever and single prop walls requires that the serviceability limit state is considered in terms of wall deformation and soil deformation at the rear of the wall. The ultimate limit state is as follows.

(*a*) Overall instability: the provision of sufficient embedment depth to prevent overturning of the wall. For a propped wall the minimum penetration is just sufficient to avoid forward movement of the wall without preventing rotation of the wall at its toe. Such a condition is known as the free earth support condition. As wall penetration is increased beyond this minimum, fixity of the wall is progressively increased until fully fixity is obtained and the toes if unable to rotate when earth and water loading is applied to the wall. This state is known as the fixed earth condition. For the cantilever wall without prop support, fully fixity must be obtained to prevent rotation and collapse of the wall.

(*b*) Foundation failure: in cohesive soils, the wall penetration depth must be sufficient to prevent basal failure after excavation to formation level in front of the wall.

(*c*) Hydraulic failure: in cohesionless soils, the penetration of the wall must be sufficient to avoid piping in front of the wall after excavation to formation level.

Overall stability

The overall stability of both cantilever and single prop walls is often assessed using limit equilibrium methods in which a factor of safety is applied to an analysis of failure conditions to ensure their avoidance. Such factors of safety can be applied in a number of ways and have attracted comment from Potts and Burland,[14] Symons[15] and Padfield and Mair.[6] The various factor methods are as follows.

(*a*) Factor on embedment. A factor of safety is applied to embedment depth. The method is described in references 16 and 17.

(*b*) Factor on moments of gross pressure. This method applies as a factor of safety to moments of gross pressure on the passive side only. Water pressure is not factored. The method is described in references 5 and 9.

(*c*) Factors on moments of net pressure. The net pressure diagram is calculated and the factor of safety is defined as the ratio of moments of the net passive and active forces about the toe of the wall.

(*d*) Factors on net passive resistance. Potts and Burland[14] developed this method which defines the factor of safety as the ratio of the moment of the net available passive resistance to the moment activated by the retained material including water and surcharge. When using effective stress methods with $c' = 0$ and where there is no surcharge on the passive side, the active pressure diagram is modified so that no active pressure increase occurs below formation level and any such increase as calculated is deducted from the passive pressure diagram. Where total stress or effective stress methods are used, reference should be made to Potts and Burland.[14]

(*e*) Factor on shear strength on both active and passive sides. Soil shear strengths are reduced by dividing c' and $\tan\phi'$ by the factor of safety, and the active and passive pressure diagrams are calculated using these reduced values. The reduced values approximate to mobilized values. Bending moments and prop loads derived from the calculation can be used for wall design if they are treated as ultimate limit state values. This method is recommended in BS 8002.[12]

(*e*) Factor on shear strength of passive side only. The passive resistance is factored but no factor is applied to the active side.

Comparative studies show that there is no relationship between factors of safety applied to the methods listed. There is little doubt that the preferred method is really defined by personal use and experience, but some useful observations were made in reference 18 as follows:

(*a*) The factor on embedment is empirical and should be checked by applying a second method.

(*b*) The method of factoring moments on gross pressure may give excessive penetration at low angles of shearing resistance (ϕ' less than 20°), so use varying factors for different ranges of ϕ'.

(*c*) The factoring of net passive pressure moments tends to give high penetration values.

(*d*) The factors on modified net passive resistance as recommended by Burland and Potts[14] appear to give consistent results in a reasonable range of soils and wall dimensions.

Factors of safety recommended by Padfield and Mair[6] for use in stiff clays with methods for factoring embedment, moments of gross pressure, net passive resistance and shear strength on both active and passive sides are reproduced in Table 5.5. Two approaches are used: approach A is based on moderately conservative parameters, and approach B uses worst credible soil parameters, geometry and loading in the design. While applying to stiff clays, the factors of safety listed can also be taken as indicative values in granular soils.

Table 5.5 Factors of safety for methods of analysing embedment (Padfield and Mair[6])

Method		Design approach A: recommended range for moderately conservative parameters (c', ϕ', or c_u)		Design approach B: recommended minimum values for worst credible parameters (c' = o, ϕ')		Comments
		Temporary works	Permanent works	Temporary works	Permanent works	
Factor on embedment, F_d	Effective stress	1·1 to 1·2 (usually 1·2)	1·2 to 1·6 (usually 1·5)	Not recommended	1·2	This method is empirical. It should always be checked against one of the other methods.
	Total stress*	2·0	—		—	
Strength factor method, F_s	Effective stress	1·1 to 1·2 (usually 1·2 except for ϕ' > 30° when lower value may be used)	1·2 to 1·5 (usually 1·5 except for ϕ' > 30° when lower value may be used)	1·0	1·2	The mobilized angle of wall friction δ_m and wall adhesion c_{wm} should also be reduced
	Total stress*	1·5	—	—	—	
Factor on moments: CP2 method, F_p	Effective stress	1·2 to 1·5	1·5 to 2·0	1·0	1·2 to 1·5	These recommended F_p values vary with ϕ' to be generally consistent with usual values of F_s and F_r
	$\phi' \geq 30°$	1·5	2·0	1·0	1·5	
	ϕ' = 20 to 30°	1·2 to 1·5	1·5 to 2·0	1·0	1·2 to 1·5	
	$\phi \leq 20°$	1·2	1·5	1·0	1·2	
	Total stress*	2·0	—	—	—	
Factor on moments: Burland-Potts method, F_r	Effective stress	1·3 to 1·5 (usually 1·5)	1·5 to 2·0 (usually 2·0)	1·0	1·5	Not yet tested for cantilevers. A relatively new method with which little design experience has been obtained
	Total stress*	2·0	—	—	—	

* Speculative, treat with caution

Note 1. In any situation where significant uncertainty exists, whether design approach A or B is adopted, a sensitivity study is always recommended, so that an appreciation of the importance of various parameters can be gained

Note 2. Only a few of the factors of safety recommended are based on extensive practice experience, and even this experience is recent. At present, there is no well-documented evidence of the long-term performance of walls constructed in stiff clays, particularly in relation to serviceability and movements. However, the factors recommended are based on the present framework of current knowledge and good practice

Note 3. Of the four factors of safety recommended, only F_p depends on the value of ϕ'

Fixed earth support: cantilevered wall

Certain simplifying assumptions are made due to the relative complexity of the calculation. Figure 5.19 shows an idealized pressure diagram and typical shear, bending moment and wall deflection diagrams. The simplified procedure is as follows:

- the pressures at the toe of the wall are replaced by a resultant force F_3 acting at C
- forces F_1 and F_2 act through the centres of pressure of their respective areas
- depth BC is found by assuming a level for C and calculating the moments for the forces F_1 and F_2 about level C; repeat until moments balance
- increase the depth BC by 20% to give the design penetration BD
- calculate the maximum B_M at the point of zero shear.

This method can be used for layered soils; net water pressures should be included in the active pressure diagram where groundwater occurs. The factor of safety can be applied by using limit pressures and increasing the depth of embedment. The bending moment distribution at limiting equilibrium is assumed to correspond to working conditions and a factor of safety is applied separately to ultimate material stresses to provide permissible stress values. Alternatively, the earth pressure distribution may be obtained by applying a factor of safety to the limit pressures or, similarly, a mobilization factor to peak shear strengths as specified by BS 8002.[7] The bending moment and shear force distribution is then calculated by considering moment and force equilibrium, and the wall section is designed using values from these distributions and the ultimate design strength of materials (design strength at the ultimate limit state is derived from material characteristic strength reduced by a partial factor of safety).

Bending moment and shear force in the cantilever wall may also be analysed using Winkler spring, finite element or boundary element methods. These methods produce a more realistic soil interaction solution and provide values of lateral wall deformation.

A simple analysis of a cantilever wall in uniform soil conditions using limit pressures, originally by Lee,[19] is shown in Fig. 5.20. This algebraic solution is included for historic purposes rather than practical use and shows the relative complexity of the solution if neither simplifying assumptions or modern methods of calculation are used.

Worked examples of cantilever wall analysis were compared by Padfield and Mair[6].

The importance of wall flexibility related to wall deformation and depth of embedment was originally shown by Rowe.[8] The earth distribution was shown by model tests to vary according to the flexibility of the wall:

$$\text{Wall flexibility} = EI\,\frac{d^4y}{dx^4} \tag{61}$$

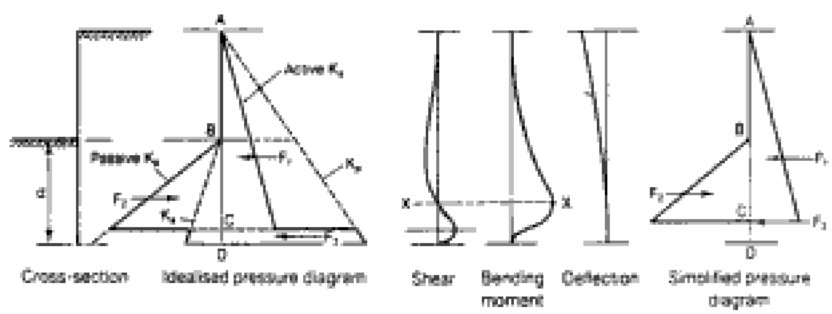
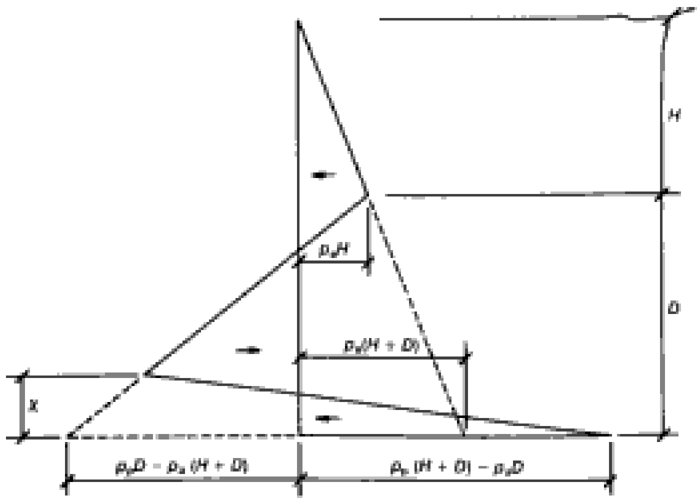


Fig. 5.19. Analysis of cantilevered wall (CIRIA[1])



Using limit pressures,

equating horizontal forms and multiplying by two:

$$p_a(H + D)^2 - p_pD^2 + (p_p - p_a)(H + 2D)X = 0$$

which gives

$$X = \frac{p_pD^2 - p_a(H + D)^2}{(p_p - p_a)(H + 2D)} \qquad (1)$$

equating moments of these values about the toe of the piles and multiplying by three:

$$p_a(H + D)^3 - p_pD^3 + (p_p - p_a)(H + 2D)X^2 = 0 \qquad (2)$$

To simplify, use a factor f such that $D = fH$. Substituting for D in (1)

$$X = \frac{p_pf^2H^2 - p_a(H + fH)^2}{(p_p - p_a)(H + 2fH)}$$

Substituting for D and X in (2)

$$p_a(H + fH)^3 - p_pf^3H^3 +$$
$$(p_p - p_a)(H + 2fH)\left\{\frac{p_pf^2H^2 - p_a(H + fH)^2}{(p_p - p_a)(H + 2fH)}\right\}^2 = 0$$

Simplifying

$$p_a(H + fH)^3 - p_pf^3H^3 + \left\{\frac{p_pf^2H^2 - p_a(H + fH)^2}{(p_p - p_a)(H + 2fH)}\right\}^2 = 0$$

Extending

$$p_aH^3(1 + 3f + 3f^2 + f^3) - p_pH^3f^3 +$$
$$\left\{\frac{p_pH^2f^2 - p_aH^2(1 + 2f + f^2)}{(p_p - p_a)H(1 + 2f)}\right\}^2 = 0$$

Multiplying across and dividing by H^4

$$p_a(p_p - p_a)(1 + 5f + 9f^2 + 7f^3 + 2f^4) - p_p(p_p - p_a)(f^3 + 2f^4)$$
$$+ p_p^2f^4 - 2p_pp_a(f^2 + 2f^3 + f^4) +$$
$$p_a^2(1 + 4f + 6f^2 + 4f^3 + f^4) = 0$$

Substituting the values of p_a and p_p gives an expression in f^4, from which f is obtained and therefore D.

Fig. 5.20. Algebraic expression for embedment of a cantilever wall using limit pressures in a cohesionless soil (Lee[19])

where E is Young's modulus of the wall material, I is the section modulus of the wall transverse cross-section per unit length, and y and x are horizontal and vertical coordinates of the vertical cross-section of the wall. The reduction in depth to full fixity caused by increasing wall flexibility produced reduced bending moments in the wall section and, according to Rowe, produced savings of up to 20% by weight of the wall compared with design methods based simply on limit pressures. The method was not widely adopted for cantilever pile design but served as an early indicator of the importance of wall and soil interaction.

Free earth support: single propped walls

The earth pressure, shear and bending moment distribution and wall deflexion diagrams which apply to a typical single propped (or anchored) wall are shown in Fig. 5.21. The method of calcualtion uses limit pressures or pressures calculated on peak shear strengths reduced by a mobilization factor as defined in BS 8002, and is as follows:

(*a*) draw earth pressure diagrams using limiting or mobilized pressures for soil pressure in cohesionless or cohesive homogeneous or layered soils. Include net water pressure, modified to include the effect of steady flow beneath the wall and equalization of pore pressures either side of the wall at the toe. Use tension cracks filled with water where applicable in clays and apply minimum fluid pressures when necessary
(*b*) calculate F_1 and F_2 to act through the centre of their respective pressure areas
(*c*) calculate the penetration BD by assuming a trial depth of penetration D and taking moments of forces F_1 and F_2 about the tie rod level. Repeat with trial values of penetration D until these moments balance
(*d*) the prop load T can then be found by balancing forces $T = F_1 - F_2$
(*e*) calculate the bending moment in the wall section at the level of zero shear.

Fixed earth support: single propped walls

The analysis of the fixed earth, single propped (or anchored) wall is made less complicated by use of work due to Blum[20] in the 1930s. He approximated the

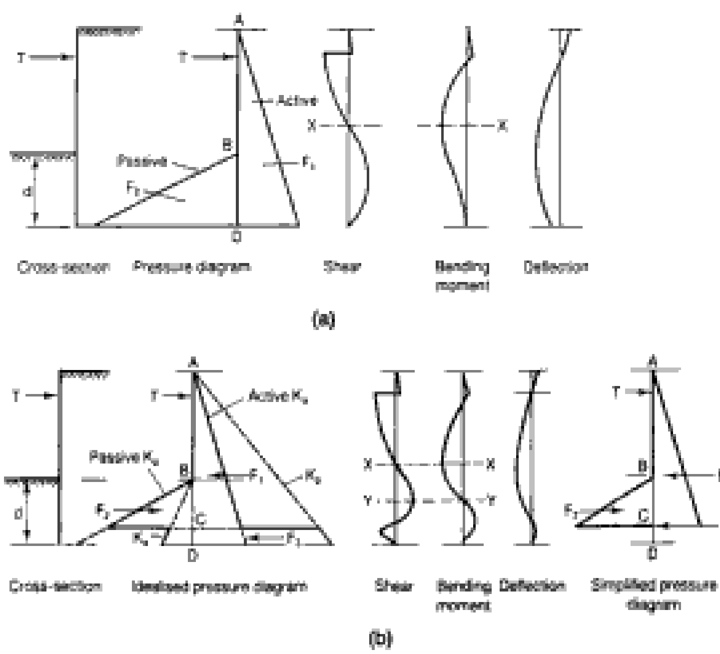


Fig. 5.21. Single propped or anchored wall with: (a) free earth support; (b) fixed earth support (CIRIA[1])

Table 5.6 Zero moment depths to ensure fixity

Angle of shearing resistance ϕ	Depth to zero moment as a proportion of depth from ground surface to formation level H
20°	$0{\cdot}25H$
25°	$0{\cdot}15H$
30°	$0{\cdot}08H$
35°	$0{\cdot}035H$
40°	$-0{\cdot}007H$

bending moment in a single anchored wall with fixed earth support by reducing it to zero at the point of zero net earth pressure below formation level. Blum gave approximate depths to this point for relatively uniform soils where the wall is sufficiently deep to ensure fixity; these depths are given in Table 5.6. This support may be solved graphically[16] or by calculation as follows:

(*a*) Draw earth pressure diagrams using limiting or mobilized pressures for soil pressure in cohesionless, cohesive or layered soils. Include net water pressure modified to include the effect of steady flow beneath the wall and equalization of pore pressures on either side of the wall at the toe. Use tension cracks filled with water where applicable in clays. Apply minimum fluid pressures ($5\ kN/m^3$) where necessary.
(*b*) Draw the net pressure diagram from ground level to the first point of zero pressure below formation level.
(*c*) Calculate active forces F_1 and F_2 from the gross pressure diagram.
(*d*) Take moments of forces F_1 and F_2 and tie rod force T about the first point of zero pressure from the net pressure diagram and equate to zero.
(*e*) Calculate value of force T from this expression.
(*f*) Assume depth of point C point of assumed application of passive force F_3 and recalculate F_1 and F_2 to this depth. Take moments of T, F_1 and F_2 about this level and repeat with change in depth to point C until the moments balance. As with the cantilever wall calculation, the calculated depth to point C is increased by 20% to give design penetration depth.
(*g*) Draw the shear force diagram and calculate the maximum bending moment in the wall at the point of zero shear force.

This design procedure, as for free earth support design, is readily made by computer program and where soil deformation prediction is required can be the subject of finite element or finite difference computations.

Anchored bulkheads

The description of a propped wall includes within this generic phrase various constructions such as sheet piling, diaphragm walls and reinforced concrete pile walls braced by struts or raking shores, or sheeters and walls of similar construction with tie rods or stressed ground anchors. One of these types of constructions, sheet piling to form a wall or bulkhead tied back by steel rods to an anchor block, known as a dead man, or to anchor piles, was the subject of a historical keynote paper by Terzaghi.[21] This method of construction, which is widespread on waterfronts where it is used to achieve a quayline with reclamation behind it at minimum cost, does not strictly lie within the subject of deep excavations, but no technical discussion on braced sheeting would be complete without reference to the paper and the controversial discussion which followed it. The principal purpose of the paper was to identify and rectify errors in accepted bulkhead design at that time on the basis of tests and observations by Terzaghi and model tests by Rowe.[8,22,23] The paper reached these principal conclusions.

(*a*) The identification of the type of soils and fills and their in situ properties of uniformity, relative density and strength are vital matters which are frequently overlooked in anchored bulkhead design.

(*b*) The distribution of earth pressure on the bulkhead is unlikely to conform to the Coulomb distribution, because of the extent of deformation of the soil structure. This deformation depends on soil and wall stiffness.

(*c*) If maximum bending moments are calculated on walls in sand assumed to extend to sufficient depth to achieve full fixity irrespective of wall flexibility and sand relative density, errors are likely and these are on the unsafe side.

(*d*) For sheet piles driven into clay, the assumption of full fixity at depth will probably not apply as time elapses and no reduction in the calculated maximum moment in the all should be allowed due to wall section flexibility to compensate for this loss.

(*e*) Anchor tension depends on several factors other than the properties of the backfill material and the flexibility of the wall or sheeting. Therefore, the anchor pull should be computed on the assumption of free earth support. Anchor pull may be greater than that calculated using Coulomb's theory, and may increase due to repetition of loading and unloading from heavy surcharge. An unequal yield of adjacent anchorage produces variations in tie rod pull. Given these risks, more conservative stresses should be used in anchor design than are applied in sheet pile bulkhead design.

These conclusions broadly still apply, although alternative methods of analysis have been developed in which soil, wall and anchor stiffness can be modelled and deformation and induced stress in all three can be calculated. Terzaghi concluded: 'Because of the great variety of subsoil conditions which may be encountered, the subject (anchored bulkheads) does, and always will, leave a wide margin for judgment — and also for misjudgment'.

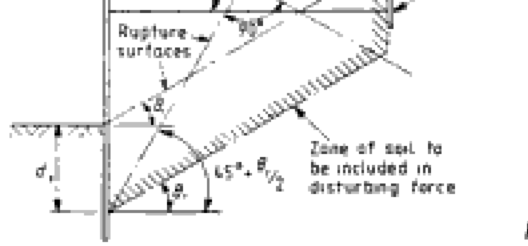
Anchorage location

Anchorages, dead men or injected tendons must be located behind potential failure surfaces at the rear of the wall. Fig. 5.22 shows the recommended geometry for analysis of dead men locations: from BS 6349.[13]

Foundation failure

The risk of base failure to an excavation by upward heave applies particularly in very soft and soft clays and silty clays, typically, quick estaurine deposits. The failure is analogous to a bearing capacity failure of foundation, only in reverse; the failure is a shear failure in the soil below formation level, but caused by relief of load (the relief of overburden) and not by the application of load as occurs in a conventional foundation bearing failure.

The methods of Terzaghi[24] and Bjerrum and Eide[25] can be applied to calculate the factor of safety against base failure; these are shown in Fig. 5.23. Terzaghi's



For free earth support d_x is the depth of embedment of sheet piles.
For fixed earth support d_x is ⅔ depth of embedment.

Fig. 5.22. Location of deadman anchorage in granular retained fill (BS 6349[13])

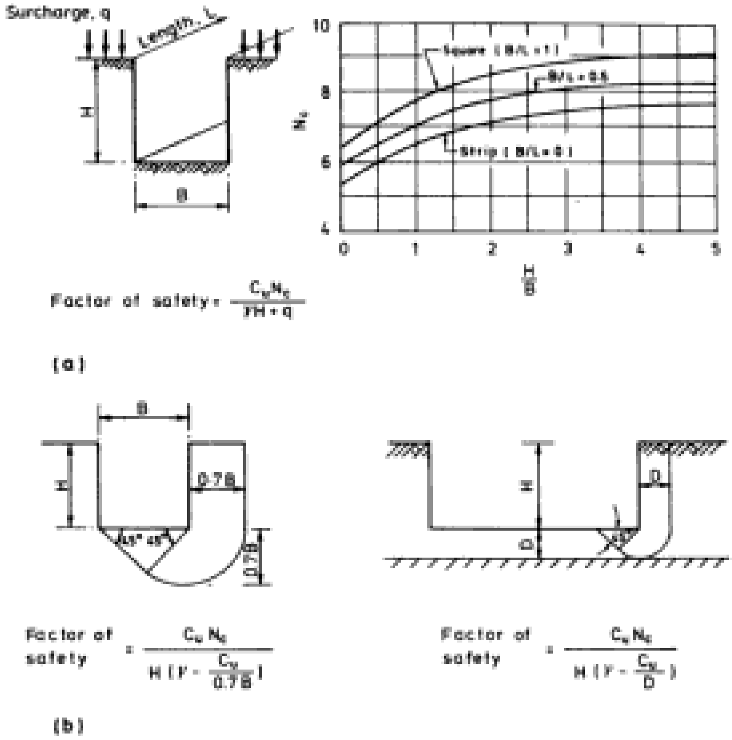


Fig. 5.23. Calculation of factors of safety against basal heave in cohesive soils: (a) deep excavations with H/B > 1; *(b) for shallow or wide excavations with* H/B < 1 *(GCO publication[18])*

method is primarily applicable to shallow or wide excavations, while the method of Bjerrum and Eide is suitable for deep and narrow excavations with no nearby underlying stiff clay to inhibit failure. Both methods neglect the effect of wall penetration below formation level and therefore results may prove to be conservative, especially where stiffer clays exist with depth. A third method[9] for predicting the basal safety factor where stiff clays exist at depth, is shown in Fig. 5.24.

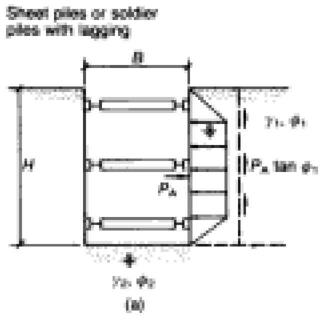
The factor of safety against basal failure is generally required to be not less than 1·5. If uncorrected values of in situ vane tests are used, the actual factor of safety may be close to 1·0 according to Aas.[26] (A vane correction described by Bjerrum[27] is necessary to obtain more reliable values of safety factor.)

With a factor of safety, based on corrected vane results, which is less than 1·5, substantial soil deformation is likely. If such soil movement is not acceptable, a factor of safety not less than 2·0 is recommended. Increase in movement occurs as the basal factor of safety decreases, and increases rapidly as a factor of safety of 1·0 is approached. Although basal heave is rare within excavations in cohesionless soils, a basal heave analysis is included in Fig. 5.24(a) for completeness together with basal heave analysis in clay as described in NAVFAC[9] in Fig. 5.24(b) and (c). Fig. 5.25 shows the values of bearing capacity factors for use in these analyses. Field and finite element analysis predictions of the correlation between movement and basal failure factor of safety are shown in Fig. 5.26.

Cantilever and single prop walls, particularly on sloping sites in soft clays and loose granular soils, should always be checked against risk of deep-seated circular slip failure.

Hydraulic failure

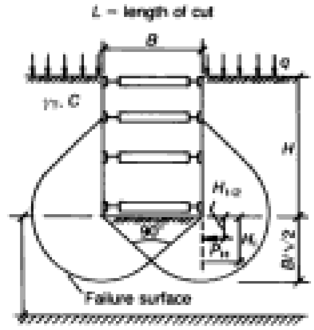
The risk of piping failure to the base of an excavation in cohesionless soils was described in chapter 2. Design charts for penetration of cut-off walls to prevent hydraulic failure in sand and stratified soil are reproduced in Figs 5.27 and 5.28.



Stability is independent of H and B, but varies with γ, ϕ and seepage condition.

Safety factor $F_s = 2N_{\gamma_2} \left(\frac{\gamma_2}{\gamma_1}\right) K_a \tan \phi_1$

where N_{γ_2} is the bearing capacity factor.
If groundwater is at a depth of B or more below base of cut γ_1 and γ_2 are taken as moist unit weights.
If groundwater is static at base of cut γ_1 is the moist weight and γ_2 the submerged weight.
If seepage is moving upward to base of cut γ_2 = (saturated unit weight) − (uplift pressure).



If sheeting terminates at base of cut the

safety factor, $F_s = \dfrac{N_C\, C}{\gamma_T H + q}$

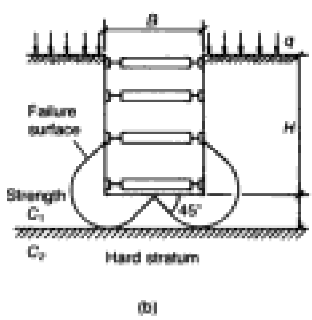
N_C = bearing capacity factor, (fig 5.28.) which depends on dimensions of the excavation: B, L and H (use $H = Z$).
C = undrained shear strength of clay in failure zone beneath and surrounding base of cut.
q = surface surcharge.

If safety factor is less than 1·5, sheeting must be carried below base of cut to insure stability.

Force on buried length:

If $H_1 > \dfrac{2}{3}\dfrac{B}{\sqrt{2}}$, $P_H = 0{\cdot}7\,(\gamma_T HB - 1{\cdot}4\,CH - \pi\, CB)$

If $H_1 < \dfrac{2}{3}\dfrac{B}{\sqrt{2}}$, $P_H = 1{\cdot}5\,H_1\left(\gamma_T H - \dfrac{1{\cdot}4\,CH}{B} - \pi\, C\right)$

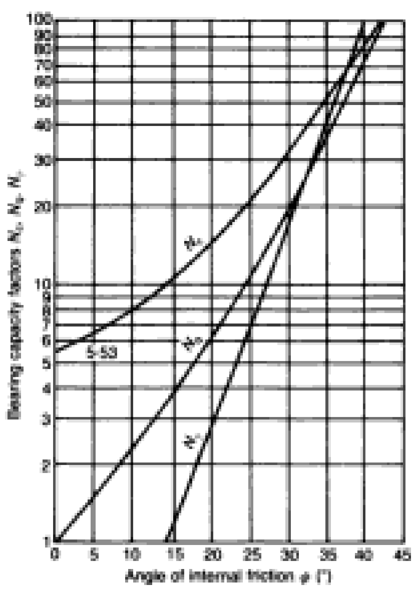


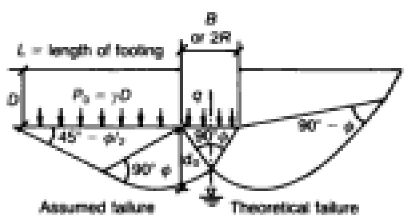
continuous excavation, $F_S = N_{CD}\,\dfrac{C_1}{\gamma_T H + q}$

Rectangular excavation, $P_S = N_{CR}\,\dfrac{C_1}{\gamma_T H + q}$

N_{CD} and N_{CR} = bearing capacity factors (fig 5.29) which depend on the dimensions of the excavation: B, L and H, (use $H = Z$)

Fig. 5.24. Calculation of factors of safety against basal heave in: (a) cohesionless soil; (b) cuts in clay of considerable depth; (c) cuts in clay limited by hard stratum (US Navy[9])





Assumed conditions

1. $D \leq B$
2. Soil is uniform to depth $d_b > B$
3. Water level lower than d_b below base of footing
4. Vertical load concentric
5. Fraction and adhesion on vertical sides of footing are neglected
6. Foundation soil with properties C, ϕ, γ

Fig. 5.25. Plot of bearing capacity factors against angle of shearing resistance (US Navy[9])

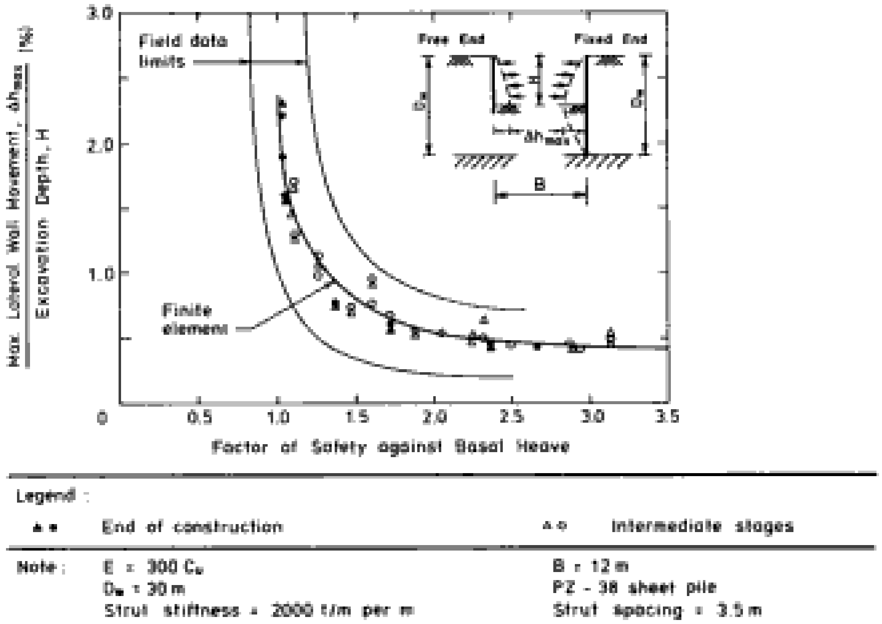


Fig. 5.26. Analytical relationship between maximum lateral wall movement and factor of safety against basal heave from field data, free end and fixed end walls, various sites (note is for finite element analysis data)

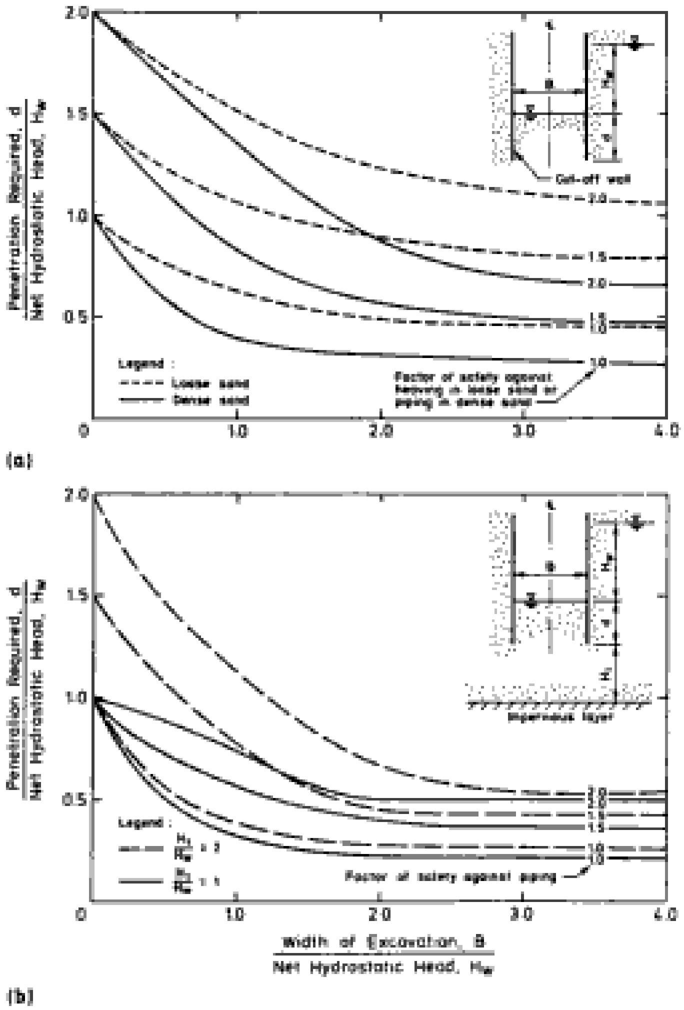
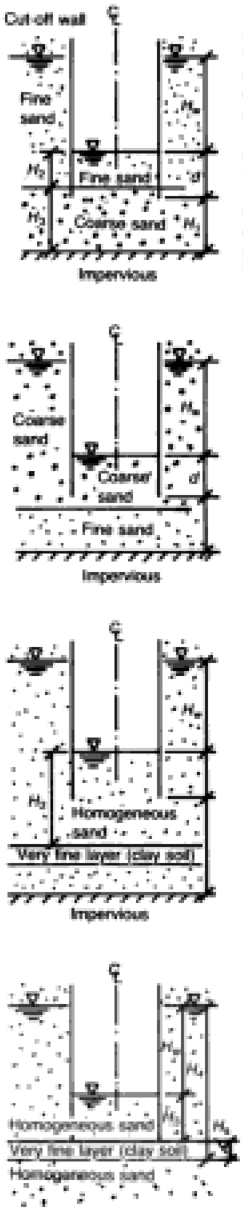


Fig. 5.27. Penetration of cut-off wall to prevent hydraulic failure in homogeneous sand: (a) in sands of infinite depth; (b) in dense sand of limited depth (US Navy[9])



(a) Coarse sand underlying fine sand
Presence of coarse layer makes flow in the fine material more nearly vertical and generally increases seepage gradients in the fine material compared to the homogeneous cross-sections of Fig 5.27.

If top of coarse layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Fig. 5.27 (a) for infinite depth apply.

If top of coarse layer is below toe of cut-off wall at a depth less than width of excavation, then uplift pressures are greater than for the homogeneous cross-sections. If permeability of coarse layer is more than ten times that of fine layer, failure head H_w = thickness of fine layer (H_2).

(b) Fine sand underlying coarse sand
Presence of fine layer constricts flow beneath cut off wall and generally decreases seepage gradients in the coarse layer.

If top of fine layer lies below toe of cut-off wall, safety factors are intermediate between those derived from Fig. 5.27 for the case of an impermeable boundary at (i) the top of fine layer, and (ii) the bottom of the fine layer assuming coarse sand above the impermeable boundary throughout.

If top of fine layer lies above toe of cut-off wall, safety factors of Fig. 5.27 are somewhat conservative for penetration required.

(c) Very fine layer in homogeneous sand
If top of very fine layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Fig. 5.27 assuming impermeable boundary at top of fine layer apply.

If top of very fine layer is below toe of cut-off wall at a depth less than width of excavation, pressure relief is required so that unbalanced head below fine layer does not exceed height of soil above base of layer.

To avoid bottom heave when toe of cut-off wall is in or through the very fine layer ($\gamma_s H_3 + \gamma_c H_4$) should be greater than $\gamma_w H_5$

γ_s = saturated unit weight of the sand

γ_c = saturated unit weight of the clay

γ_w = unit weight of water

If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above fine layer and pressure relief is required as in the preceding case.

Fig. 5.28. Penetration of cut-off wall to prevent hydraulic failure in stratified soil: (a) coarse sand underlying fine sand; (b) fine sand underlying coarse sand; (c) very fine layer in homogeneous sand (US Navy[8])

Wall flexibility

Rowe's work[8,22,23] requires special attention. Following a series of model tests at Manchester, initially on sands of varying relative density, Rowe was able to show that interaction between soil and wall was different for steel sheet piles and reinforced concrete sheet piles because of the greater flexibility of the steel sheet pile. This greater flexibility causes a redistribution of earth pressure which differs considerably from the Coulomb distribution, as shown in Fig. 5.29. The flexure of the wall causes reduction in pressure at mid-height and causes the resultant passive force to rise with an increase in fixity for the flexible pile. These changes reduce the design bending moment for a flexible pile, although too often such reductions are not applied to ensure the pile does not crumple during driving.

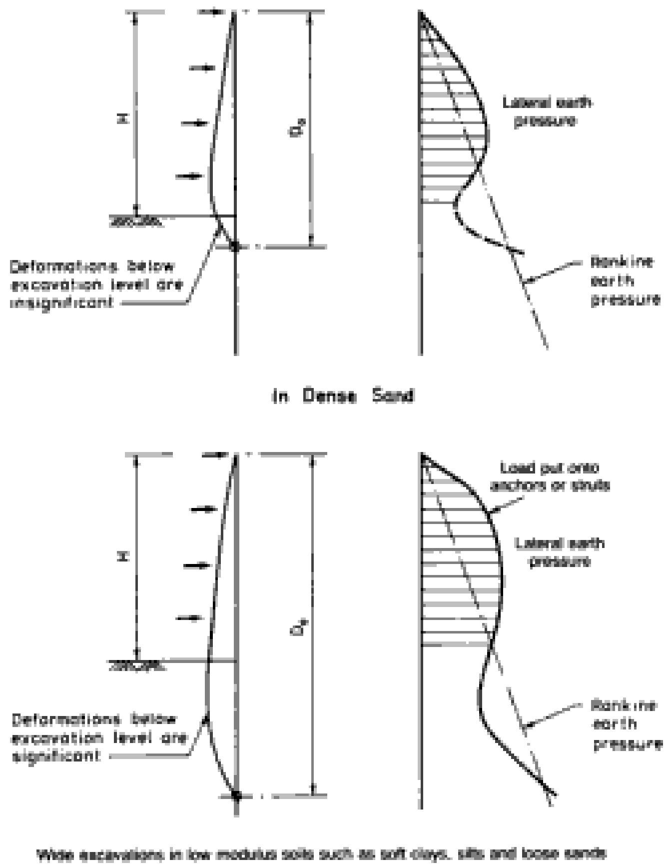


Fig. 5.29. Deflection of a sheet pile and redistribution of active earth pressure (Bjerrum et al. *(see bibliography))*

Materials and stresses

For cantilever and single propped walls, the component parts are designed from the net earth pressure diagram at the ultimate limit state using unfactored soil pressures and wall geometry obtained from the overall stability analysis (using one of the methods described above). The wall section, tie rods and walings are designed using ultimate limit state conditions with calculated loads and moments factored by a partial factor of not less than 1·5 to obtain design values. Alternatively, and as defined in BS 8002,[2] earth pressures may be calculated for the serviceability limit state using a mobilization factor on peak soil strengths. In this case the code recommends that application of a further partial factor of safety on calculated values of loads and moments is not necessary. Crack widths are calculated for serviceability limit state in reinforced concrete walls. Characteristic and design strengths of construction materials for cantilever and single prop sheet pile walls are given in Table 5.7.

Table 5.7 Characteristic strength of steel sheet piles

Grade of steel		Characteristic strength	Working stress
BS 4360	BS EN 10 025	BS 5950 (N/mm^2)	BS 449 (N/mm^2)
43A	FE 430A	265	180
50A	FE 510A	345	230

Note: The 'A' subgrades (which are not impact tested) appear only in Annex D of the UK edition of the European Standard. Other editions use the 'B' suffixes.

Multi-prop walls

The descriptions of design methods for cantilever and single propped (or anchored) walls referred to computations using limit pressures and the application of factors of safety. Only the methods due to BS 8002[2] have introduced design using earth pressures at the serviceability limit state. Using these methods the bending moment in the walling can be estimated relatively quickly by hand calculation (adopting Blum's methods[20] for cantilever and fixed earth support walls) or even more conveniently using finite element, finite difference or Winkler Spring analytical methods. Soil deformation behind the wall may be predicted, if needed by the finite element or finite difference programs. Design requirements and analysis methods for multi-prop walls are a different matter, however. The method of construction for these walls is usually sequential, installing the sheeting or walling and excavation in stages followed by installation of the prop or anchor at each installation stage. The sheeting or walls will, in all likelihood, penetrate the ground below the final excavation level. the extent of wall deformation in this sequence of operations is restricted, although the passive resistance of soil below excavation level at each stage is mobilized to support the wall prior to installation of the bracing or the anchor at that level. Despite the frequent support for the wall, therefore, horizontal deformation of the wall occurs at each passive soil zone prior to installation of the prop or anchor at that level. The wall distorts inwards to mobilize this passive resistance, the wall movement occurring below each stage of excavation. Pore pressure dissipation may occur in cohesive soils during the period needed for strut or anchor installation at successive levels.

The extent of wall movement also depends on the stiffness of the propr or anchor once installed at each level. Where the soil is relatively stiff, say dense sands or gravels, the extent of forward movement of the sheeting at each excavation stage to mobilize soil passive pressure will be relatively small, and active earth pressures on the wall will considerably exceed Rankine values above dredge level; redistribution of earth pressure is only likely to occur between the lowest strut and formation level. In wide excavations in soils such as soft clays or loose sands where stiffness is low, the successive deformations below excavation level at each propping factoring level are considerable. Load is redistributed between the struts, and the sum of the strut loads considerably exceeds the Rankine values.

Where prestressed ground anchors are used at each excavation stage the earth pressure on the wall is determined by the prestress levels and subsequent relaxation, and by relative wall and soil stiffnesses. Design methods which have been used for many years have been based on calculations of anchored walls using a Coulomb distribution assuming no prestress applied. This non-prestress value of anchorage at each excavation stage is subsequently used as the actual value of prestress applied to the tendon. This empirical method successfully restricts soil movement but in turn inhibits the Coulomb earth pressure distribution, on which the calculation is based, from developing, the actual pressures on the retained side of the wall being higher (and nearer K_o or K_p values) than those calculated. More recent methods using Winkler spring and finite element programs allow an assumed anchor prestress load to be introduced to the analysis, from which the actual earth pressures are calculated on the basis of the soil movement permitted by wall and soil stiffness and the extent of the anchor prestress.

The alternative limit states for multi-prop walls are similar to those for cantilever and single-prop walls:

(*a*) overall stability: risk of strut failure, bending stress failure in sheeting or passive failure of soil below stage excavation level or final formation level
(*b*) foundation heave: in soft clays, risk of failure by unloading; bearing capacity failure
(*c*) hydraulic failure: piping in cohesionless soils with high external groundwater table.

The serviceability limit states are:

(*a*) deformation of sheeting: the acceptable limits of sheeting deformation will depend on the purpose of the excavation and whether the works are temporary or permanent, or a combination of both. Where walls or sheeting is temporary the deformation must not exceed that which would occupy space required for the permanent works nor cause difficulties with sheeting removal if this is intended. For permanent works, deformation of the wall or sheeting must neither impair the durability of the substructure nor cause visual offence

(*b*) soil movement behind wall or sheeting; vertical settlements: the extent of settlement behind the support for the excavation must not exceed the permitted settlement of existing structures, highways or services, unless the consequences of this can be estimated accurately and are acceptable

(*c*) cracking in reinforced concrete walls: at the serviceability state, cracking will occur on the tension face of reinforced concrete walls due to application of load, in particular earth and water pressures and surcharge loading, and also on each face of the wall due to early thermal cracking of the concrete. For building substructures the provisions of BS 8110[28] will apply to crack control in walls or, where more rigorous waterproofing is needed, BS 8007[29] may be specified. For highway structures in the UK, design flexural and tension cracks complying with the BS 5400[30] are specified. Design crack widths of 0·25 mm, complying with 'severe' conditions are usual although the pressure of saline or sea water may reduce this value to 0·15 mm. Additional longitudal steel may therefore prove necessary in diaphragm walls to control crack widths caused by loading, although the application of rules to minimize vertical crack widths due to thermal shrinkage of concrete in panels of limited individual length may be neither satisfactory nor necessary. Such rules for reinforced concrete works are referred to in the UK Department of Transport's Standard BD28/87.[31]

The available methods of design for multi-propped walls are:

(*a*) empirical methods: largely based on strut load envelopes recommended by Peck[32] for three categories of soil: sands, soft to medium clays and stiff clays (Reference 5 included an amendment to the original Terzaghi and Peck distribution, which was apparently for use in sands)

(*b*) computer methods based on Winkler spring theory: beam-spring approaches

(*c*) full soil–structure interaction analysis by finite element, boundary element or finite difference methods.

Empirical method based on strut load envelopes

The most widely used design methods for multi-braced walls are those due originally to Terzaghi and Peck,[17], whether for temporary works (including piled and diaphragm walls permanently anchored or braced by floor construction, as in top-downwards construction) or permanent works. The strut load envelopes due to Terzaghi and Peck are shown in Fig. 5.30. Note that these diagrams are not intended to represent actual earth pressure or its distribution with depth but load envelopes from which strut loads can be evaluated. Clay is assumed to be undrained and only total stresses are considered. Sands are assumed to be drained (through the sheeting) with zero pore pressure. Where drainage is precluded behind a non-permeable wall, hydrostatically-distributed water pressure should be added to strut loads. Sheeting or walling should be designed using the Coulomb earth pressure distribution with hydrostatic water pressure except where drainage occurs through sheeting. Where inclined ground anchors are used, and especially where anchors are prestressed, great care must be taken to avoid downward penetration of walling, sheeting or soldiers due to the vertical component of the anchor load. The factor

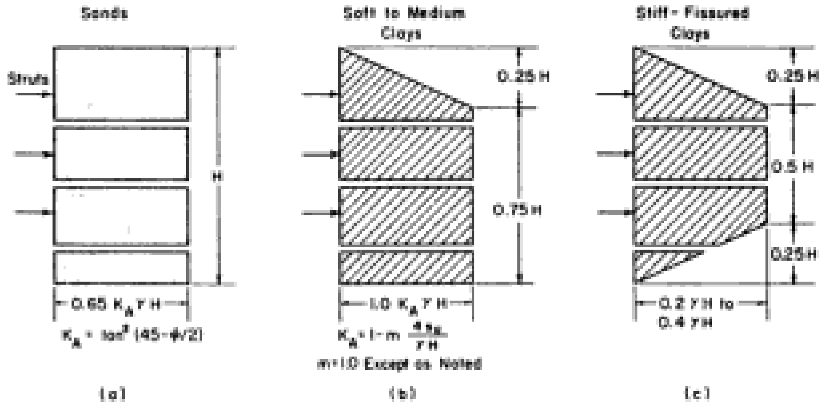


Fig. 5.30. Apparent pressure diagrams for computing strut loads in braced cuts (Terzaghi and Peck[7])

of safety against a bearing capacity failure due to this component should always be checked. Wall friction should not be used in this check.

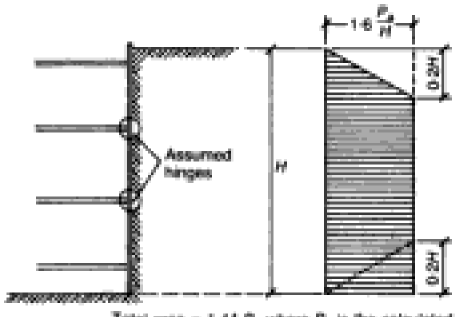
Reviewing the use of such strut envelope diagrams, Peck[32] pointed out that in 1969 the empirical procedures for estimating strut loads in soft-to-medium clays were less satisfactory than the method for excavations in sand. In terms of empirical methods, however, no modified or improved envelopes have replaced these procedures in the 25 years since, and Peck's method is still applicable. Peck did point out, however, that the behaviour of the soil and the bracing system depends on the stability number

$$N_c = \frac{\gamma D}{c_u} \tag{62}$$

where c_u represents the clay beside and beneath the cut to the depth that would be involved if a general failure were to occur due to the excavation. Peck added that when the depth of excavation corresponds to values of N_c greater than 6 or 7, extensive plastic zones have developed, at least to the bottom of the cut, and the assumption of a state of plastic equilibrium is valid. The movements are essentially plastic and the settlements may be large.

Reference 5 modified the Terzaghi strut load envelope diagram for strutted excavations in dense sand. This diagram (unlike those reviewed by Peck) contains in-built factors of safety; the total pressure represented by the trapezium is 28% in excess of the calculated thrust to allow for inequality between strut loads (Fig. 5.31).

The empirical strut load envelopes reviewed by Peck (Fig. 5.30) do not include the effects of the toe of the sheeting or walling extending below the final formation level, and yet from the point-of-view of reduction of strut loads in the lower struts



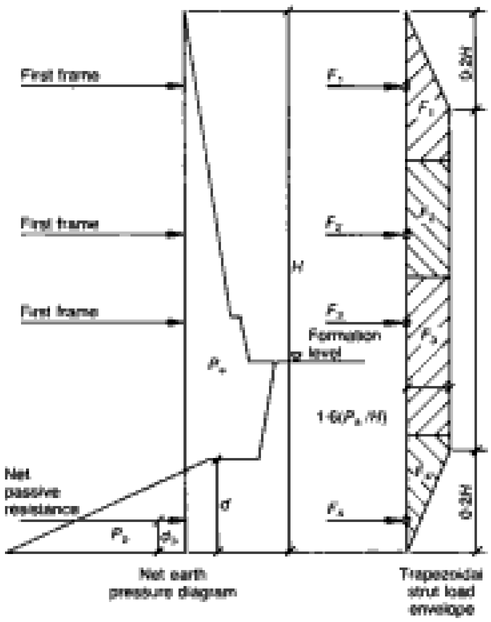
Total area = 1·44 P_a where P_a is the calculated total earth thrust with zero wall friction

Fig. 5.31. Modified strut load envelope (from reference 5)

(or anchors) and the improvement to ingress of groundwater at formation level, an extension of sheeting vertically below formation level is desirable and often achievable. There are two design methods which allow this penetration of walling and sheeting to be taken into account in strut load calculation.

The first method assumes that the passive reaction below formation level resists active pressure below that level together with a portion of the load from the strut load envelope between the lowest strut level and formation level. This method applies to strutted excavations in uniform soil conditions of reasonable strength such as medium-dense to dense granular soils and stiff clays In less competent soils (loose sands and gravels and soft clays), passive resistance against the sheeting below formation level may be less effective, and in such cases Goldberg *et al.*[16] advised that the sheeting should penetrate to such depth to avoid piping failure (as discussed in chapter 2) and the sheeting should be designed as a cantilever from the lowest strut level.

The second empirical method, the source of which is not known, has been used since the mid-1950s and has been adequately confirmed. The procedure is shown in Fig. 5.32. An additional 'strut' is assumed to act on the strut load envelope to represent the passive resistance acting on the sheeting below formation level. the level of this 'strut' is determined from the net pressure diagram, as shown in Fig. 5.32. The 'strut' load calculated from the strut load envelope is then



1. Construct net earth pressure diagram, using limit pressures P_a and P_p
2. Calculate value of P_a, total active pressure
3. Calculate trapezoidal strut load envelope ordinate 1·6 (P/H)
4. Calculate strut envelope forces F_1, F_2, F_3, F_4 from shaded areas of strut load envelope (F_4 represents net passive resistance below dredge level and acts at centre of pressure, net passive diagram)
5. Calculate factor of safety, mobilized passive resistance = $\dfrac{\text{Net passive resistance } P_p}{\text{Calculated mobilized passive resistance } F_4}$

Fig. 5.32. Construction of trapezoidal strut load envelope for braced excavation to take into account passive resistance below formation level

compared with the available passive resistance using the limiting pressure from the net pressure diagram for the selected penetration depth. The bending moment in the sheeting is calculated from the net pressure diagram.

The strut loads calculated for each successive construction stage are summarized and the highest value at each strut level is used for strut and waling design purposes. Similarly, maximum moment and shear values are calculated for the sheeting or walling for each construction stage from the net pressure diagram and the critical values used for sheeting design.

Potts[33] reviewed analysis methods for earth retaining structures and concluded that all simple methods of analysis (to include stress field and limit analysis using upper and lower bounds of plasticity in addition to equilibrium methods such as the Coulomb wedge analysis) were flawed. These approaches assumed the soil to be everywhere at failure and did not accurately represent working load conditions, being unable to distinguish between excavation and backfilling or to evaluate in situ stress conditions. Only empirical parameters take account of wall flexibility, and the difference between strut and anchor performance is not defined in these methods. The effect of surcharges is only taken into account in an approximate way and, additionally, no information is provided on either wall or soil deformations. Nevertheless, Potts concluded that such methods remain the mainstay of most design work and, where the empiricism has been verified by field observations, their application may not be inappropriate, although in more complex soil–structure problems such observations are unlikely.

Beam spring approaches

These approaches, popular since the advent of powerful personal computers, make use of one of two approximations to represent soil stiffness. The soil is either assumed to be modelled by a set of unconnected vertical and horizontal springs (Borin[34]) or a set of linear elastic interaction factors (Papin *et al.*[35]), and props or anchors are modelled as simple springs.

The initial, at rest, values on each side of the wall are allowed to reach equilibrium in a series of iterations using numerical methods such as finite elements or finite differences, until earth pressures lie between at rest values and active values on the retained side of the wall, and between at rest and passive values on the excavated side. The deformed shape of the wall is calculated but no prediction can be provided regarding soil deformation. The effect of temporary soil berms is seldom modelled accurately, if at all, in these programs and where soil stiffness is inputed in terms of subgrade reaction values there is often a lack of user confidence in the selection of accurate values. Programs which are based on the use of elastic interaction factors often have difficulty in accurately applying wall friction/adhesion.

Numerical analysis of soil–structure interaction

Full analysis of soil and structure stiffness and their interaction using realistic soil constitutive models and accurate boundary conditions has been possible for some years but is still only applied to complex solutions requiring skilled analysts. The principal advantages of such an approach include the ability to model wall and soil deformation and stress in a realistic sequence of operations that follow actual construction stages. These analyses show both immediate deformation and time-dependent changes related to pore pressure equilization. Pre-judged failure modes are not needed because these are all revealed by the analysis. Use of low-strain values of soil stiffness are essential in such approaches. As computer technology progresses, the application of these methods to three-dimensional problems as well as to more routine two-dimensional problems may become more universal.

Factors of safety: multi-propped wall design

The attention given to comparative methods of applying factors of safety to the overall stability of cantilever and single prop (or anchored) walls has not extended to multi-prop walls. The design should always consider hydraulic failure by piping from the base of the excavation and the risk of failure of the base of the excavation in soft clays. Overall, stability consideration in a multi-propped wall will rely on three matters: the risk of circular slip instability below the whole excavation in clay conditions on sloping sites, and the risk of inadequate prop capacity and wall penetration below final formation level.

The wall penetration depth should always be regarded as a minimum when calculated by methods described in this chapter. Adequate embedment depths will depend on the risk of hydraulic failure as well as passive resistance to the sheeting and relief of load from the lowest frame of bracing. In all multi-propped (or anchored) walls the principal risks of failure in soils other than very soft clays will stem from the adequacy of embedment depth of the wall or sheeting and the sufficiency of strutting or anchoring. To counter these risks in temporary works design requires care and experience. Where limit state design methods and ultimate limit state conditions are used it is recommended that a minimum partial factor of safety of 1·4 should be applied to wall and sheeting moments and a value of 2·0 to strut or anchor loads, and unfactored soil parameters should be used to prepare the earth pressure or strut load envelope diagrams.

In reinforced concrete and prestressed concrete multi-propped walls, crack widths are calculated and checked for the serviceability limit state and reinforcement provided where appropriate.

Soldier pile walls: Berlin walls

The use of soldier piles, often supported by tiers of ground anchors with horizontal timbers (or laggings) or in situ concrete placed conveniently or sprayed as shotcrete, provides an economical form of soil support, particularly in dry subsoils. Care should be taken, however, in assessing the passive resistance below final formation level of such a wall system. The passive resistance of soil at the front of the solider piles within their embedment depth can only be mobilized by movement of the solider pile towards the excavation. Soldier pile walls are relatively flexible constructions, and in relatively dense granular soils and stiff clays this can reasonably be assumed. In these circumstances it is also reasonable to allow an effective width of soil acting to provide passive resistance in excess of the net width of the steel or reinforced concrete pile acting as the soldier. Where soldier spacing centre-to-centre is greater than four times the net width of the soldier below formation level it is reasonable to assume an effective width of soil to provide passive resistance equal to twice the soldier width below formation level.

It is essential to check the bearing capacity at the base of such soldiers where inclined anchors are used. The vertical components of all anchors should be summated as they are prestressed together with the soldier pile self-weight comprising the vertical load at the soldier base.

Fig. 5.33 shows a soldier pile wall constructed by Bilfinger and Berger. Part of the wall is built in a patented form known as a 'Heidelberg' wall where the walings and soldiers can be extracted for reuse.

Construction details

A selection of construction details from the Author's files is given in Figs 5.34 to 5.36. The wall types shown include anchored sheet pile walls, Berlin walls using timber laggings, cofferdam details and multi-prop walls using secant piles and diaphragm walls.

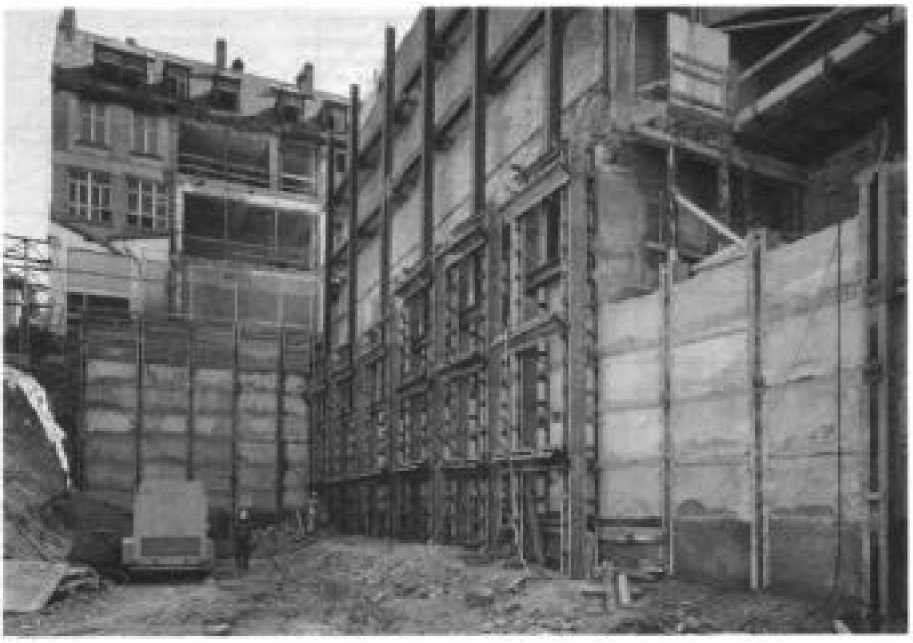


Fig. 5.33. 'Heidelberg' soldier pile and lagging wall allowing extraction of steelwork wallings and soldiers (patented by Bilfinger and Berger)

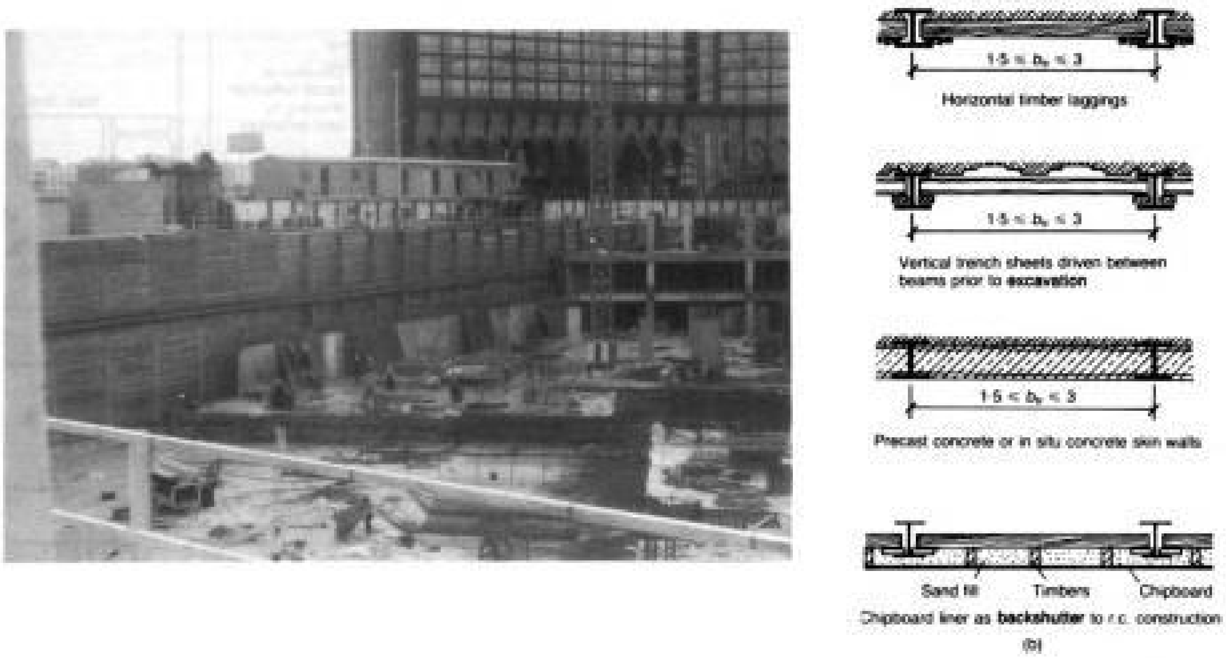


Fig. 5.34. (a) Three-storey basement construction, anchored Berlin Wall, excavation through loose sands and gravels with local dewatering; (b) derivatives of Berlin walls (dimensions in m)

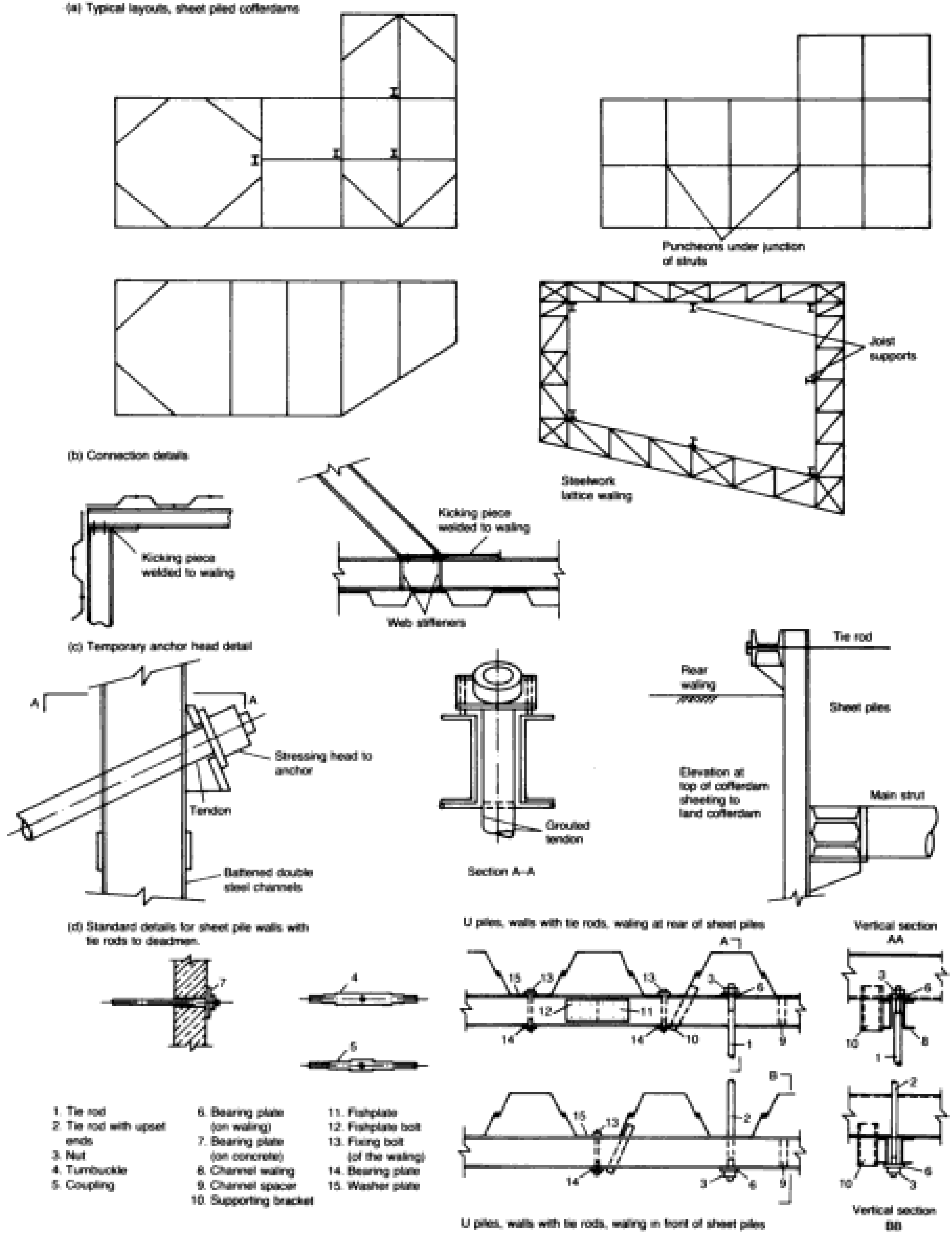


Fig. 5.35. Cofferdam standard construction details: (a) typical layouts, sheet piled cofferdams; (b) connection details; (c) temporary anchor head detail; (d) standard details walls with the rods to deaden

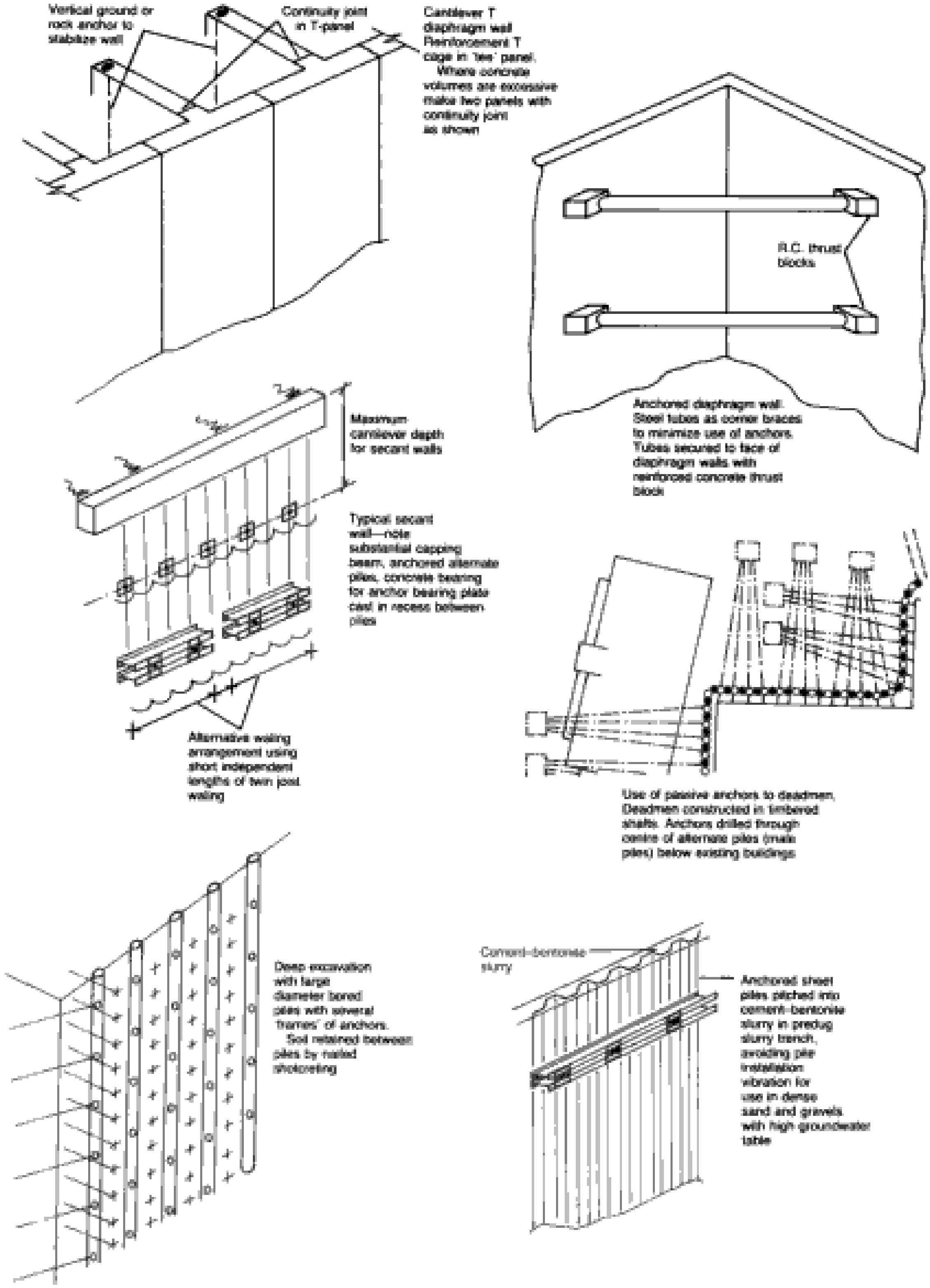


Fig. 5.36. Schematic arrangements for anchored and braced piled and diaphragm walls

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6 Cofferdam construction

Design and construction responsibilities

Packshaw[1] defined a cofferdam as a temporary structure built to exclude earth and water from a construction area and thus permit the work inside to be carried out in the dry. Nevertheless, he conceded that 'in the dry' should not be taken too literally; seepage upwards through the formation and possibly leakage through the sheeting of the cofferdam walls may bring considerable water quantities into the working space. Packshaw said that to fulfil the purpose of the cofferdam:

(*a*) The walls of the dam and any internal bracing must withstand the loads and stresses imposed on them.
(*b*) The amount of water entering the dam must be controlled by reasonable pumping and not interfere with the permanent construction inside it.
(*c*) It must be possible to excavate down to the required level without causing the ground to boil, heave or flow into the dam in an uncontrolled manner.
(*d*) The walls must not deflect inwards so much as to interfere with the permanent construction inside the dam.
(*e*) The cofferdam must have overall stability against unbalanced earth pressure or ground movements such as circular slips.

The responsibility for the safety of cofferdam structures is frequently defined by statutory regulations. In the UK the Construction Regulations stipulate obligations on both designers and contractors. By the former, excavations where soil is supported for a height greater than 1·2 m are required to be: timbered or supported by good construction free from patent defect, of adequate strength, properly maintained and secured. In particular, where there is risk of flooding in these works, means of escape must be provided. All excavation support deeper than 1·2 m is required to be regularly inspected by a competent person.

In terms of the share of responsibilitiy for cofferdam design and construction, other than health and safety laws, this depends on the nature of the contracts between Employer, Engineer and Contractor. In practice, both design and construction are largely carried out by the Contractor, with whom these responsibilities mainly lie, although not completely so. In the UK for instance, a well used contract form, the ICE Conditions of Contract[3] requires the Engineer's approval to temporary works design, but such approval does not relieve the Contractor from responsibility under contract. It should also be remembered, however, that Engineer and Contractor have duties of care under common law in the UK and damages may be applied to both parties should, for instance, the Engineer be aware of deficiencies in design or construction which result in failure and harm to life or property.

With increasing use of diaphragm wall construction in cofferdam work in both temporary and permanent works phases, it is frequently necessary for Contractor and Engineer to accept responsibility for design in each phase, the Contractor for the temporary works and the Engineer thereafter for the permanent works. Where stressed induced during the temporary works period cause later distress there is every opportunity for dispute between the parties.

Successful cofferdam work incurs minimum cost because of its temporary nature. The design is frequently made two or three times: at feasibility stage by the Engineer, at bid stage by the Contractor, and at construction stage by the successful Contractor. The Contractor's design engineer is finally set the task of designing a stable cofferdam which complies with the permanent works requirements at minimum cost in terms of construction, maintenance and final removal. The cofferdam must be safe, and appear so to those who work in it. Normal design criteria for permanent works may not prove economical for temporary construction and factors of safety and soil design parameters may require much care in selection. In particular, the choice of working stresses for construction materials which have been used before and suffered some deterioration will require judgement. Although cofferdam works are frequently considered as short-term structures, the period of their use may range from months to years depending on the scale of the contract and any extensions to the contract period. The requirement for structural durability with time therefore depends on assessment of risk and the consequences of failure to 'life and limb' and construction damage. Only the latter may be influenced by insurance cover to temporary and permanent work.

The quality of design data from site investigation should also be referred to. Packshaw[1] stated some basic rules:

- The site investigation should be carried deep enough to determine soil and water pressures, especially artesian pressures. A CIRIA report[4] suggested an investigative depth to at least 1·5–2 times the excavation depth, possibly more in weak soils.
- The site investigation should extend around the cofferdam site to a distance equal to its depth to formation level.

As a prerequisite to the investigation, accurate records of existing levels, site history, seasonal variation in groundwater levels, tides and flood frequency are essential. Unless these data are available the cofferdam designer and constructor cannot fulfil their responsibilities irrespective of those defined by statute and contract.

Type of cofferdam

Cofferdams may be divided into three categories:

(*a*) sheeted types, usually with external shores, anchors or internal bracing
(*b*) double skin types, where sheeting is used to form cells, circular in plan shape, or with parallel walls, each containing fill material; the strength of these structures depends on the composite action of fill, sheeting and the underlying soil support
(*c*) gravity and crib types, where structures made from mass concrete, often precast, or from soil and rock fill resist, by their own weight, disturbing forces of river and sea water flow and groundwater pressure.

Design ingenuity has developed many variations on these types as cofferdam locations, sizes and soil and water depths have demanded. The choice of cofferdam type is wide, and analysis of construction expense and time will frequently be needed for a particular site before a choice can be made. The head of water to be retained, soil properties above and below formation level and depth to rockhead are often critical matters influencing the choice of method. In some instances the principal choice remains between cofferdam and caisson construction, especially where excavation depths are considerable in poor ground. Improvements to construction methods and mechanical equipment have tended to increase use of cofferdam construction to greater depths in recent years. Typically, in the 1960s and 1970s the economical depth of cofferdams would have been in the range 15 to 20 m of water, depending on subsoil type. In the 1990s, however, the economical limit of cofferdam construction frequently extends 30 to 40 m or more, and caisson work continues to decline in popularity.

Sheeted cofferdams

Where space allows, battered excavations may be economical, particularly in clay subsoil or where dewatering is feasible. Where space is confined or groundwater is less easily dealt with, the use of a temporary wall, cantilevered or supported, becomes necessary at the curtilage of the permanent works.

The use of cantilevered sheeting, in steel sheet piling, bored piles for diaphragm walling, will only serve excavations of limited depth, although this depth may be effectively increased by either a short batter at the top of the wall or a temporary berm of soil against the wall at formation level. This berm is removed in short lengths as construction of the permanent wall proceeds. Cantilevered walls are particularly economical where relatively dry cohesionless soils overlie clays of reasonable strength at moderate depth. Such conditions frequently occur in London, with the top surface of London clay occurring at depths of 5 to 7 m. Although the upper horizon of the London clay may be weakened by weathering, a rule of thumb is often used to make the length of pile embedment equal to the depth from ground level to formation level. Excessive horizontal movement at the head of cantilevered walls can be reduced by use of in situ capping beams at the head of diaphragm walls or bored piles. In other instances, the buttressing effect of return angles in plan in the sheeting may limit excessive deformation to allowable values or restrict the plan length, requiring temporary propping to mid-lengths between return angles in the wall.

Where depth to formation level within a large excavation is such to preclude support from cantilever sheeting because of unacceptably high sheeting moments and deflexions, the sheeting is temporarily supported by raking shores, spanning where possible from partially-completed permanent work or, where space allows and easement is gained, by ground anchors. Fig. 6.1 shows a sheet piled excavation supported by raking shores, the temporary berm being removed as the rakers are secured. The relatively high expense of the double support operation, which is uneconomic for small-scale excavation operations, should be noted.

Circular cofferdams

The extent of internal bracing within a sheeted excavation can be reduced by the adoption of a cofferdam that is circular in plan. The plan shape of the permanent

Fig. 6.1. Sheet piled excavation supported by raking shores with temporary berm support being removed by tracked excavator, London (courtesy of AMEC)

works needs either to fit accurately within the circular cofferdam to to be reappraised to do so.

The advantages gained by circular structures rest in the comparative economy of structural rings built to resist hoop compression. However, the CIRIA report[4] restated that circular walings used to support sheet piles in circular plan forms require relatively small distortion to reach critical instability, and advised the empirical rule $d \geq D/35$, where d is the difference between outer and inner radii of the ring beam, and D is the diameter of the cofferdam to the inner face of the piles.

Stiffness is therefore the essential criteria for circular walings. For this reason reinforced concrete is frequently preferred for circular waling construction. Although steel walings are used more frequently for cofferdams over water, the empirical proportions of $d \geq D/35$ must be observed and care taken to restrain the inner flange of the waling beam to prevent buckling. The CIRIA report gave values for safe waling loads for reinforced concrete waling sections suitable for cofferdams between 5 and 35 m. Although reinforced concrete waling sections cast against sheet pile, bored pile or diaphragm wall sheeting have the practical advantage of avoiding the use of timber packings between waling and sheeting, they also have disadvantages in the curing time needed before they contribute to cofferdam strength, delaying continuous excavation and possibly impeding extraction of sheet piles for re-use.

In relatively shallow excavations it may prove economical to anchor a square sheet piled enclosure to an outer circumscribing reinforced concrete ring waling, as shown in Fig. 6.2. Deeper excavation will require successive internal ring walings with sheeting of circular plan form. Fig. 6.3 shows a land cofferdam with an upper waling cast externally to the sheeting and a lower waling cast inside.

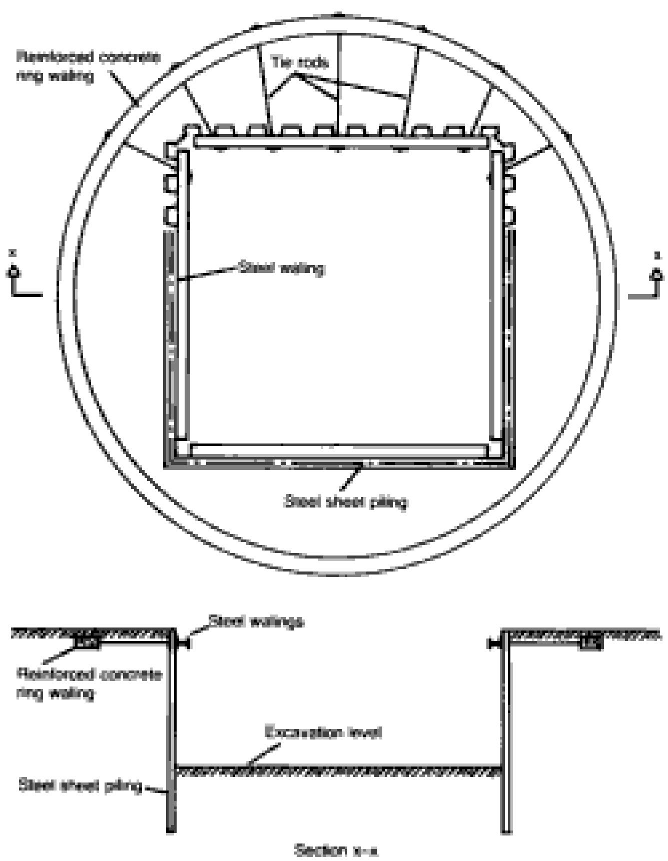


Fig. 6.2. Square sheet piled cofferdam anchored to outer circumscribing reinforced concrete ring waling (Packshaw[1])

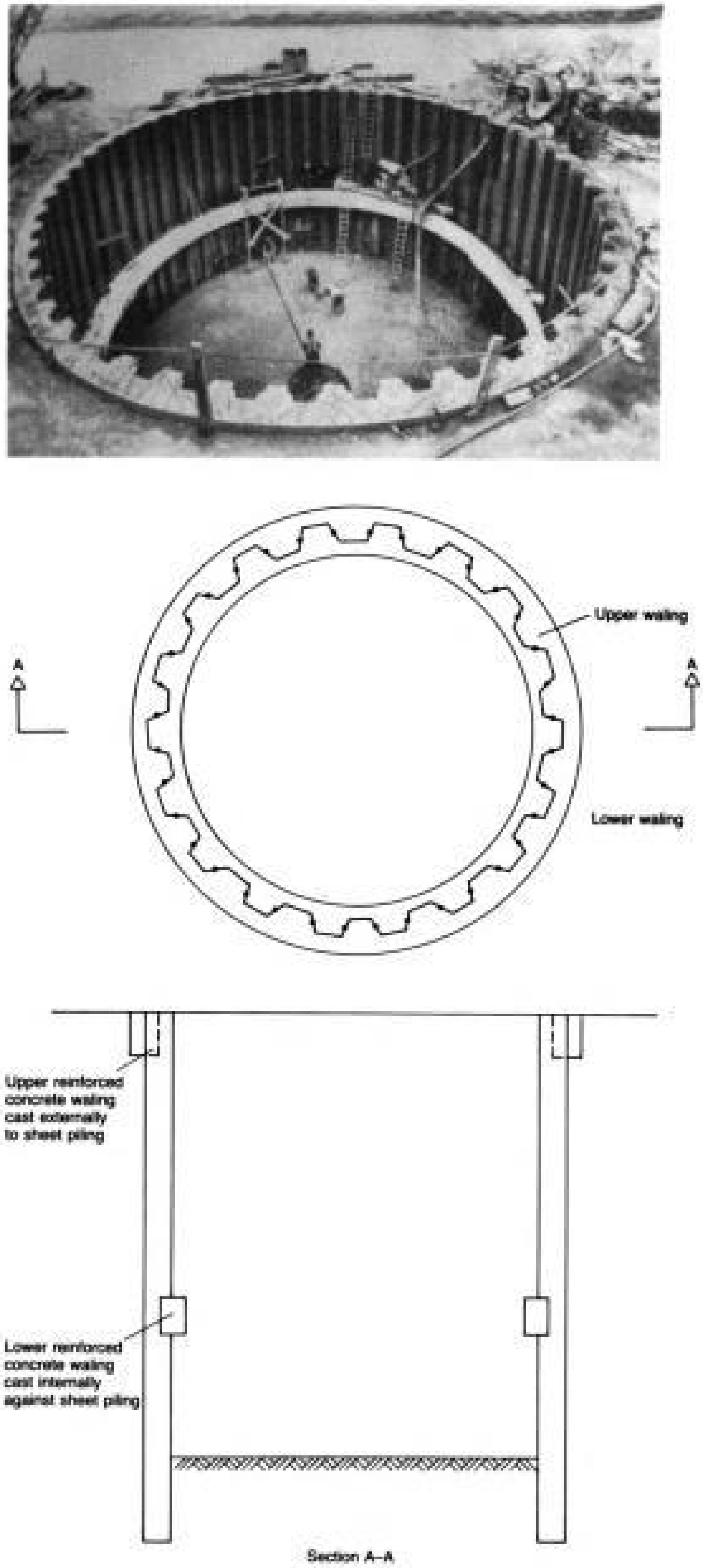


Fig. 6.3. Circular cofferdam with reinforced concrete ring walings, outer waling uppermost, inner waling above formation level

Packshaw[1] stated that the practical limit of a circular cofferdam was of the order of 45 to 60 m in diameter using sheet piling. Developments in piling equipment and the increasing international use of diaphragm walling, often incorporated into the permanent structure, have extended the use of contiguous and secant piles and diaphragm walling into cofferdams of circular plan shape. An example of a large pumping station at the Isle of Grain, UK, is shown in Fig. 6.4. The diaphragm walls, designed to span vertically between circular walings, form a circle of 75 m diameter. Fig. 6.5 shows a vertical section through a deep circular cofferdam built for a pumping station at Weston-Super-Mare, UK, in difficult ground conditions with a high water table. Frodingham No. 4 sheet piles 23 m long were supported by five ring walings in a cofferdam 30 m in diameter. Circular cofferdams on land require uniformity of soil and water pressure around the peripheral sheeting. Ground levels should therefore be sensibly level, soil strata relatively level and groundwater pressure constant around the cofferdam.

In a small number of diaphragm wall examples the limited overall diameter of the circular structure and tight control of panel verticality tolerance have allowed the hoop compression to be transferred efficiently from one panel to the next, avoiding ring walings completely. Due to the absence of bending stresses in these wall panels only limited quantities of reinforcement are required when hoop compression can be mobilized in this way.

Some variations on the circular plan shape can be economically used. Icos[5] documented the successful use of elliptical plan shapes for diaphragm wall cofferdams at Palermo, Legano and at the Quero hydroelectric plant, all in Italy. Double circular plan shape diaphragm walls in the form of a figure-of-eight were used by Icos in Cagliari and at Beckton in London. Soil conditions and the general arrangement of the walls at Cagliari are shown in Fig. 6.6.

On the Forth Road Bridge in Scotland cofferdams for the south main river pier were built in a figure-of-eight, two circular plan forms being connected by cross bracing. Fig. 6.7 shows the bracing frame arrangement and the construction sequence, including the sinking of caissons from the floor of the cofferdams.

Circular cofferdams, in pairs, were built for the east and west piers of the first Severn Bridge, UK. The marl into which the cofferdam sheeters were founded was too hard for pile driving. The toes of the sheet piles were therefore bolted to a concrete anchor ring which had been cast on to the rock surface between tides. In turn, the anchor ring was tied to the marl by dowel rods. The rock was excavated

Fig. 6.4. Circular diaphragm wall with internal reinforced concrete waling, pumping station, Isle of Grain, UK (courtesy of Laing)

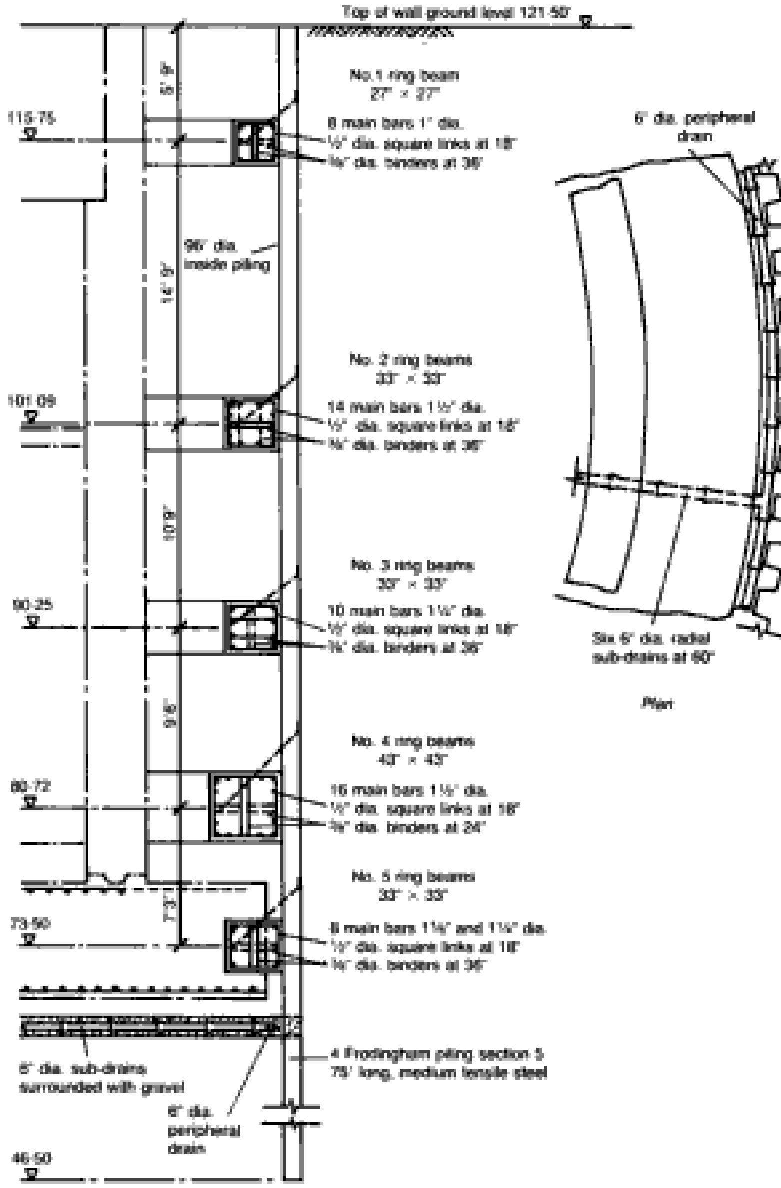


Fig. 6.5. Vertical cross-section of a circular cofferdam showing reinforced concrete ring waling construction, pumping station, Weston-Super-Mare, UK (Packshaw[1])

by mechanical loader after blasting with small charges, and a precast concrete segmental lining was grouted into place. The excavation and lining construction were carried out tidally. Excavation continued through the underlying mudstones and when a hard layer was met the cofferdam area was cleaned and covered with blinding concrete. Both ends of the cutwater pier were built to the high tide level within the cofferdams, the sheeting and bracing removed, and the central portion of the pier completed between tides. Fig. 6.8 shows the cofferdams and the construction sequence.

Braced cofferdams

The use of cofferdam plan shapes other than circular or elliptical requires structural support to the sheeting by internal bracing for all but shallow excavations. Before considering construction of braced cofferdams over water and land, the most frequent causes of failure are considered:[4]

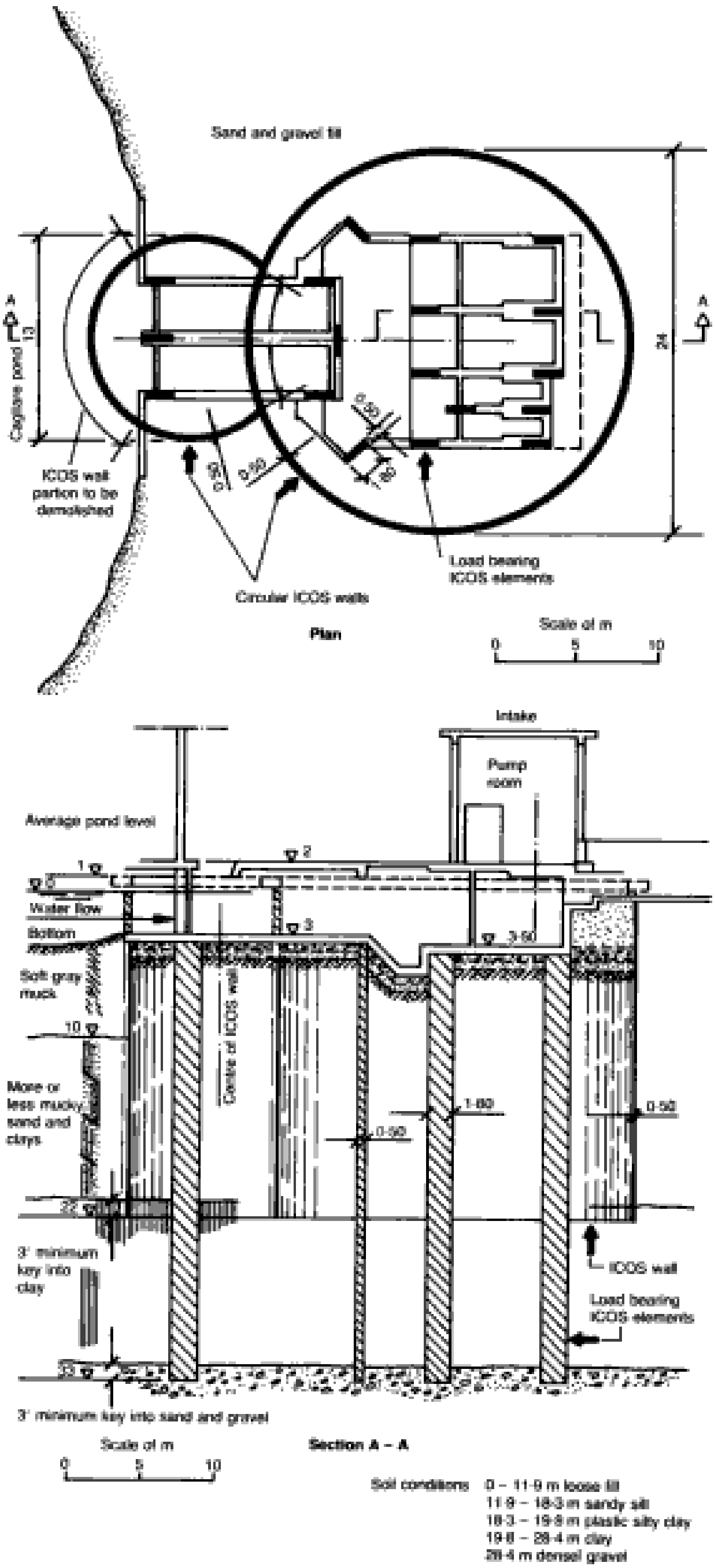


Fig. 6.6. Figure-of-eight planshape diaphragm wall water intake, Cagliari, Italy (courtesy of Icos)

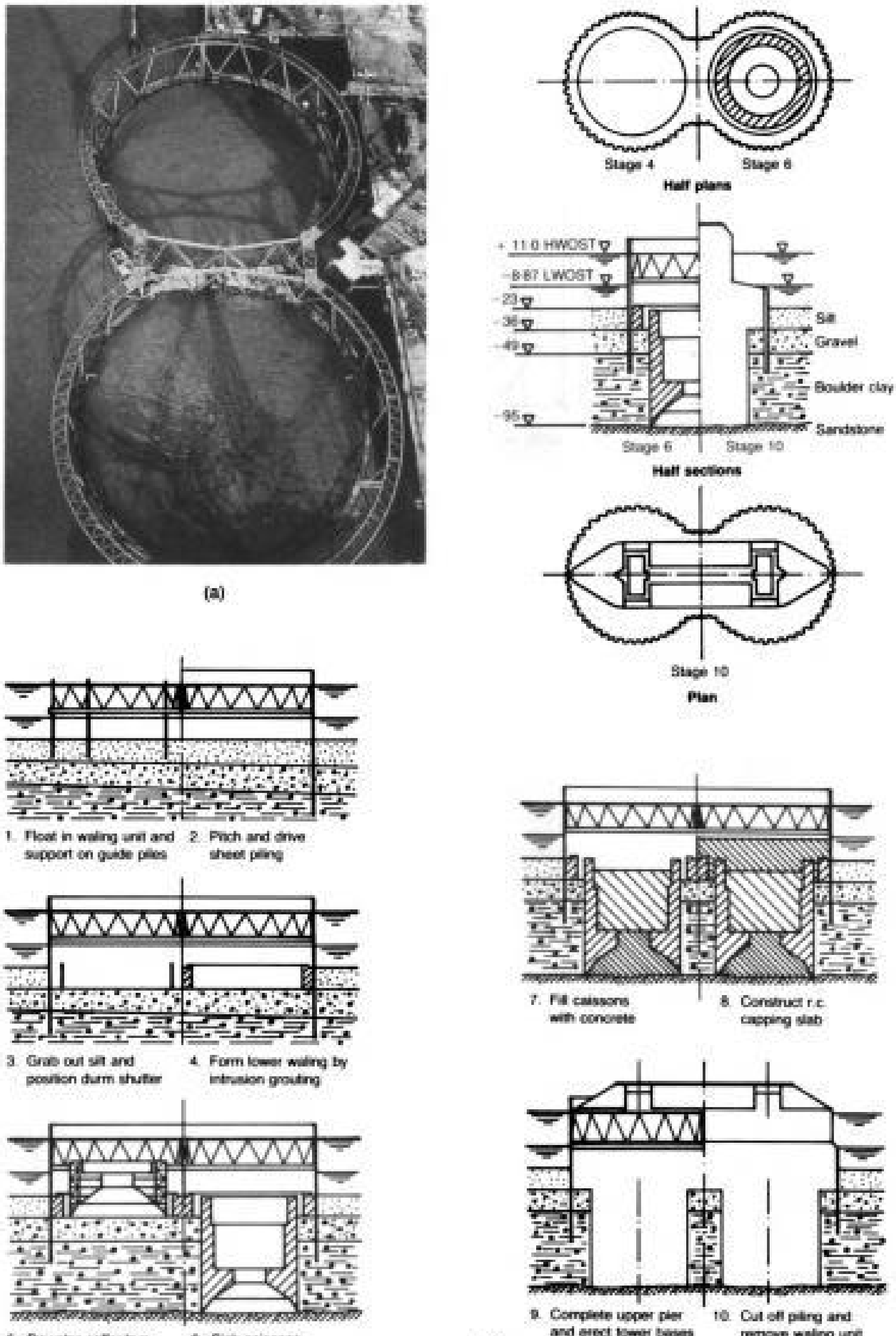
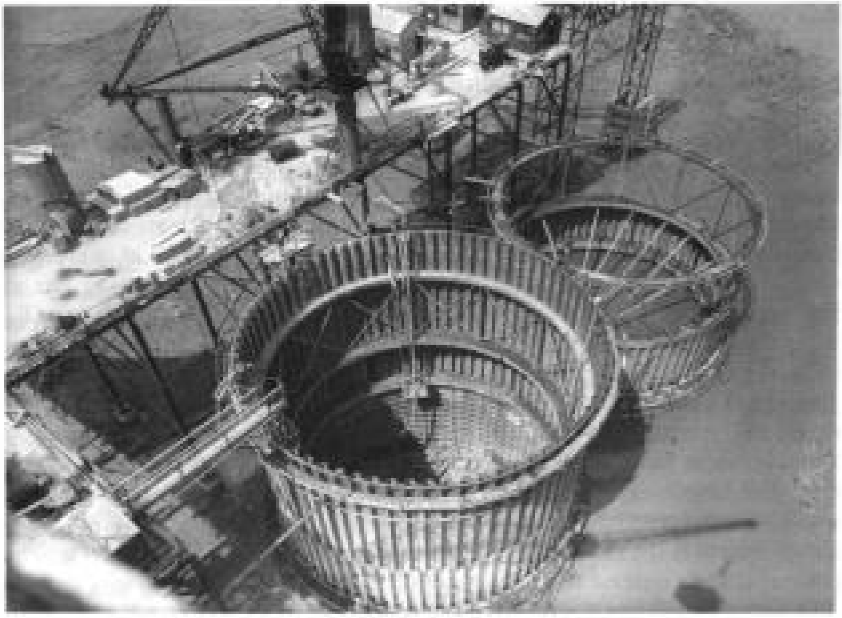
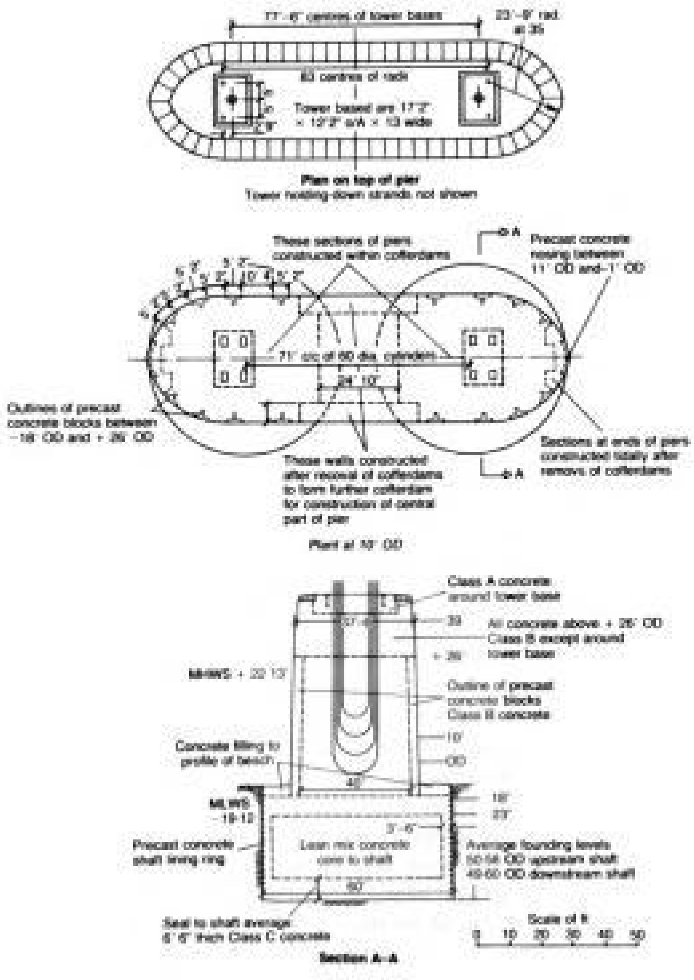


Fig. 6.7. Forth Road Bridge, south pier construction: (a) bracing frame floated into position prior to pitching sheet piles (courtesy of AMEC); (b) construction sequence (Anderson (see bibliography))



(a)



(b)

Fig. 6.8. Severn Bridge, west pier construction: (a) twin circular cofferdams with segmental ring construction below sheet pile walls (courtesy of AMEC); (b) construction details (Gowring and Hardie (see bibliography))

(*a*) failure to make allowance for variation of water levels due to seasonal effect, constriction of flow by the cofferdam itself, and overtopping by high tide or flood conditions
(*b*) failure to provide and use balancing and/or flooding valves
(*c*) variations between the ground conditions assumed for design and those revealed by the excavation
(*d*) failure to control the excavation levels, at all stage of construction, to those indicated in the design
(*e*) failure to provide adequate support to frames and lateral restraint to compression flanges of waling beams
(*f*) use of frames to support plant loads, such as pumps and generators, in excess of design allowances
(*g*) frame members damaged by impact from skips and grabs and inadequately repaired
(*h*) unauthorized strut removal or substitution
(*i*) uncontrolled water ingress through separated interlocks into the passive zone below formation level
(*j*) ill-fitting struts carrying unintentional eccentric loads.

Overall failure is more likely to occur as a result of inadequate strutting or passive soil failure due to inadequate sheeting embedment rather than flexural failure of the sheeting itself. Packshaw[1] examined the reasons for cofferdam failure by soil type and concluded that the cause of failure is almost invariably below formation level; either the soil inside the cofferdam is unable to resist external forces or it undergoes a change in its properties as a result of these forces. Base failure takes several forms:

(*a*) in permeable ground, if a 'blow' occurs because of inadequate cut-off or excessive pumping from sumps it is likely that the bottom of the piling will be forced inwards due to lack of passive resistance of the soil below formation level. As excessive load is transferred to the lower frames, progressive collapse follows to the whole structure.
(*b*) in highly permeable soil, the upward flow of water will reduce the passive resistance of soil below formation level. This may be sufficient to overload the lowest frame even if quick conditions do not develop at the formation.
(*c*) variations in loading, say tidal variations, not only test the stiffness and security of the bracing but, by repeated movement, may reduce passive resistance of the subsoil below formation. The lowest frame must be designed to allow for any transfer of load and should be located as near formation level as possible.
(*d*) in soft clays excavated below a critical depth the piling may deflect inwards.
(*e*) uplift due to water pressure may cause base failure. Clay below formation level with artesian water in sand lenses or within a cohesionless substrata are typical risk conditions.

The cause of failure of a major cofferdam on the River Thames did not fall into these categories and deserve mention. The cofferdam, for a river wall improvement, had five frames with one face towards the river and was 15 m deep below high water, 15 m wide and approximately 80 m long. All strutting and walings were of steel section, and standards of workmanship and maintenance were generally above average. One particular strut on the bottom frame was, however, badly fitted; the cut end of the strut was not square to its axis and without shims the eccentric load placed on this strut was sufficient to cause failure at the highest tide after full excavation. The strut bowed badly and, without knowledgeable supervision during a weekend, no remedial action was taken. The next high tide caused progressive failure to the passive soil resistance below formation level and also to each of the four frames in turn above the bottom frame. Bad workmanship, particularly on struts, should be added to the above list.

Braced cofferdams over water

In the case of braced cofferdams over water the sheeting is invariably steel piling, and the bracing, of walings, struts, king piles and puncheons, is nowadays usually in structural steel. Only where large quantities of fill material are economically available to build soil islands can diaphragm walling and secant piling be considered for use as cofferdam walls.

The minimum strength of sheet piles for braced cofferdams is the greater strength required either in flexure to resist soil and water pressure or to resist driving stresses. Sheet piles are manufactured in various profiles: the flexural strength of Z-type sheet piling with interlocks at the flanges can be assumed to be the strength of the completely clutched section; those sections clutched on the centre-line (U profile) depend on frictional resistance to shear within the interlocks to develop the full cross-section strength in flexure. Williams and Little[6] considered this matter in depth. BS 8004[7] recommends that where piling is cantilevered substantially above the first frame and passes through very soft clay, or pile cut-off is limited by shallow bedrock, the flexural strength of the whole clutched section should be obtained by welding the clutches. To summarize the advantages and disadvantages of each section, Z-profiles tend to have more closely fitting interlocks which are watertight for marine use, and deflexions tend to be less when used to cantilever. Z profiles, however, reduce in modulus if they are allowed to rotate during driving. In approximate terms, a rotation of 5° results in a 15% modulus reduction. U profiles, on the other hand, have a greater single section modulus and are less prone to deviate in penetration of dense soils and can be re-used more often. The rotation permitted by the interlocks is greater for the U profile than the Z profile (9° compared with 3°), so the U section is better when driving sheeting to a tight radius.

The sheet piles are pitched and driven in panels. Both U or Z section sheet piles should be driven in pairs wherever possible. The toes may need to be strengthened for very hard driving, although this may make extraction difficult at a later stage. Controlled blasting of a narrow trench in the rock may assist where sheet piles need to be driven into strong bedrock.

After pitching in pairs the piles are usually driven in echelon order (1-3-5-2-4-6). In difficult conditions the piles may be driven in two stages, first to part depth and then to full depth with a larger hammer. Piles may also be driven in panels by pitching and driving the first pair to part, but firm, penetration, then pitching the remainder of the panel in pairs. The last pair is fully driven and then the remaining pairs are fully driven successively back to and including the first pair. Jetting and pre-boring may sometimes be used to overcome difficult drilling; jetting should be replaced by conventional driving in the final metre of penetration to avoid loose subsoil at the pile toe.

The initial panel in a cofferdam should begin with the pile pair adjacent to a corner pile, the last panel concluding with the corner pile, all the piles being pitched and interlocked in this last panel before any are driven. For small cofferdams all the piles should be pitched before driving is begun. Cofferdams in water may be built by using the top frame, supported by temporary piles as a template. This template should be built to the design cofferdam dimensions, whereas lower frames should be made to reduced dimensions to allow for sheet pile verticality tolerances. The setting-out line of the top frame should therefore allow:

- total verticality and positional tolerance of sheet piles
- a width dimension to allow cofferdam drainage at formation level
- adequate working space and accommodation for shuttering, where used.

The choice of driving equipment is large and depends on the size and weight of the sheet piles to be driven, embedment length, subsoil conditions, and constraints on noise and vibration. In the UK, legal power is given to local municipalities under the Control of Pollution Act 1974 to impose limits on site noise where this adversely affects the quality of life. BS 5228[8] gives guidance on vibration and noise control due to piling. It is worth remembering that while noise gives rise to complaint, vibration gives rise to structural damage. A summary of pile hammer types and their use is given in Table 6.1. As a general rule, it is best to be on the heavy side with hammer capacity: light hammers tend to spread the top of the pile without achieving increased penetration. Impact hammers (air, diesel, hydraulic, drop hammer) are the most versatile tools, although vibrating hammers are highly efficient in low to medium density granular soils. Hydraulic thrust pile drivers, relying on reaction through a hydraulic clamp on to neighbouring piles already partly driven, are useful in stiff cohesive soils.

The depth of embedment of the sheeting is dependent on the need to effect a cut-off in an impermeable strata, to reduce risk of boiling and the quantity of pumping in cohesionless soils, to alleviate risk of base heave where hard strata underlie soft clay and silts, and to generally reduce sheeting movements by mobilizing passive resistance of subsoil below formation level. The height of sheeting driven for tidal works will normally be based on the highest tide levels estimated during the period of the works, with allowance for flood risk. The freeboard will take into account wave action caused by wind and, in navigable rivers, will allow for the wake caused by passing vessels.

In exposed conditions on the sea shore it is at times economical to use half-tide cofferdams, with no works being carried out at peak tides, the cofferdam being pumped out after the tide. As with any cofferdam over water, adequate sluice valves to allow discharge of water are essential when overtopping is considered. In addition, internal bracing must be sufficiently rigid to allow some reverse head and avoid

Table 6.1 Types of pile driving equipment (CIRIA[4])

Type of driver	Soil compatibility	Noise output	Quiet versions	Vibration output	Extract	Rate of penetration	Pairs/single driving
Double-acting air hammer	All soils except stiff clay	High	No	Low	Yes	Medium/low	Both
Single-acting diesel	All soils	High	No	Medium/low	No	Medium	Pairs
Double-acting diesel	All soils	Medium/low	No	Medium/low	No	Medium/low	Pairs
Cable operated drop	All soils	Medium/low	Yes	Medium/low	No	Medium/low	Pairs
Hydraulic drop	All soils	Medium/low	Yes	Medium/low	No	Medium	Both
Vibrator	Granular and soft clays	Medium	N/A	Low	Yes	High	Both
High-speed vibrator	Granular and soft clays	Medium/low	N/A	Low	Yes	High	Both
Hydraulic thrust	Granular with jetting and clays	Low	N/A	Nil	Yes	Slow/Medium	Single/Panel

loosening of strutting. Tie rods and external walings are sometimes installed above the top frame, not to resist reverse water heads after overtopping but to give some means of tightening the cofferdam after dewatering.

The struts of all cofferdams are made deliberately short to allow wedging or paging in timber wedges, perferably in elm, between sheet piles and the rear face of the walings. This shortening depends on frame depth and the driving accuracy of the sheet piles, which directly depends on quality of workmanship and subsoil conditions. The practice of using short lengths of reinforced concrete to pack the gap between strut ends and the face of walings is inadvisable since this cube of concrete can be dislodged by accidental blows from excavating plant and time is needed for the concrete to cure before the strutting becomes effective. A well built and maintained cofferdam will therefore have each pile pan timber paged against the back of the waling, these wedges being driven tight at each tide, the end-plated struts remaining square to the face of the waling.

The struts, typically from tubular steel, Rendex box piles, universal column or battened twin joist sections, are designed at spacings to comply with the overall geometry of the cofferdam. In addition, strut centres must allow lower frame members to be threaded through them and allow excavating grabs and excavation plant to pass vertically without striking the frames. The number of frames will depend on the imposed loading, the strength of the sheeting and the frame material available. The method of excavation within the dam will be controlled directly by the vertical height between frames; efficient excavation equipment such as tracked loading shovels can only be used with reasonable headroom. Frame levels will also condition lift heights of concrete pours for permanent works within the cofferdam. This factor applies particularly where the permanent works are built against the sheeting and where projecting vertical reinforcement lies below walings. Bar couplers may be necessary in these circumstances to reduce the difficulties of excessive reinforcement splice lengths.

The cofferdam frames are supported by steel brackets welded to the sheeting. The use of hanging rods to suspend frames beneath the top frame, which is supported on brackets, is inadvisable; all frames should be supported on an adequate number of brackets welded to the sheeting.

The static calculations for the cofferdam structure will take into account soil and water pressures, but loads due to construction plant such as pumps being placed on the frames, the dynamic effect of waves, accidental collision from service vessels and blows caused by excavation grabs and equipment on struts, must also be considered or avoidance measures taken during the construction phase.

The effects of scour on cofferdams in fast flowing rivers will be referred to later. Measures to prevent scour during the construction period include river bed protection by rock fill, precast concrete blocks and tetrapods, rock-filled gabions and grouted mattresses. Routine observation of the depth of scour is essential, particularly where there is risk of under-scour to river bed protection.

In cofferdams built in waters with a high tidal range the sequence of load application and reduction causes movement within the sheeting which is unavoidable and can cause heavy leakage of water into the dam through the pile clutches. Traditional, practical methods of reducing this problem include lead-wool caulking, asbestos-string caulking and, more crudely, the spreading of fly-ash or sawdust on to the water outside the cofferdam as the tide ebbs. As the dam is pumped out quickly the fine material is taken into the clutch and helps to seal the leak. More recently, durable elastic sealants have been applied to the interlock at site or as a complete filler to the interlock prior to pile despatch from the works.

The construction phase where excavation has been made to a level just below the level at which a frame is to be built was mentioned earlier. The risk factor at the lowest frame level depends on the time taken to build the frame in position, to prop the sheeting and make secure. All materials will then be ready to drop into position and weld and page up tightly as the last excavation is taken away. Where conditions are suspect, caution may dictate that this should be done at low tide where possible and also, of course, in short lengths, taking only sufficient excavation to fix one length of waling. Where high bending moments in the sheeting are unavoidable it may be necessary to fix frames, usually after the first frame, below water. Frames may be slung and fixed by divers prior to pumping out the dam.

River cofferdams: construction examples

The river cofferdam examples that follow illustrate some of the constructional principles to follow, and the dangers and faults to avoid.

(*a*) The Thames Barrier cofferdams were described by Grice and Hepplewhite.[9] The flood defence barrier was built by the municipal authority, the Greater London Council, to prevent flooding of some 45 square miles of London during critical tides and winds.

The geological section across the Thames at the barrier site is shown in Fig. 6.9, and a general view during construction is shown in Fig. 6.10. Eleven deep river cofferdams were built between 1974 and 1980 to house the two abutments and nine piers. Cofferdams 4 to 8 are among the largest river cofferdams built in the UK to date. The seven southern structures were founded on the chalk and the remaining four northern structures were built in the Thanet sand, a very dense grey-green silty fine sand. On the south side, the upper surface of the chalk is about 10 m below datum and dips gently northwards. On the north bank, the chalk is about 25 m below datum and is overlain with the Thanet sand. The chalk, which is fissured and jointed with flints, is classified as upper chalk and is generally unweathered where covered with sand but is moderately to severely weathered elsewhere.

As shipping access was needed through the works, the south side of the river was closed first for construction of piers 6 to 9 and the abutment; construction work then moved to the north-side cofferdams.

The southern cofferdams were built from Larssen No. 6 grade 50B sheet piles 34 m long, fully driven in pairs by piling hammers ranging from the Delmag D44 to the Delmag D62. Final driving sets varied from 400 to 1200 blows per metre. The construction sequence was to install the top frame at level +2·1 OD and then excavate underwater to −18·00 just below bottom frame level. The excavation was completed to −23·75 OD, the formation prepared and the 5 m thick concrete plug placed by tremie. Intermediate frames were installed in cofferdams 6 to 8 after pre-assembly

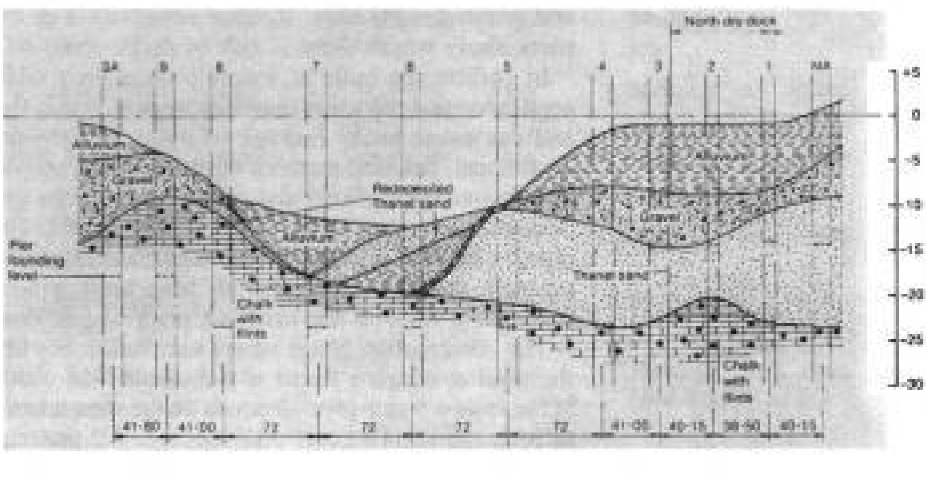


Fig. 6.9. Thames Barrier: geological section across the site (Grice and Hepplewhite[9])

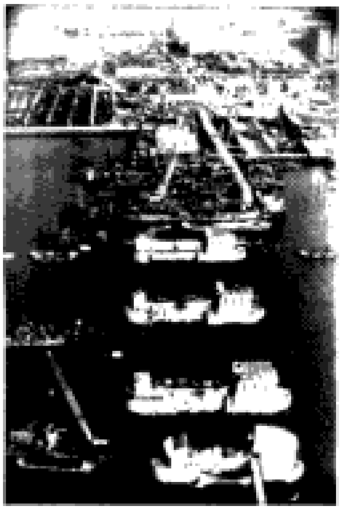


Fig. 6.10. Thames Barrier: general view during cofferdam construction (Grice and Hepplewhite[9])

above low water in a position under the top frame. The dams were then dewatered. Access for the south-side cofferdams is shown in Fig. 6.12(a) and (b). The part-plan and cross-section of the pier 6 cofferdam is shown in Fig. 6.11. The walings were fabricated from 914 × 419 mild-steel universal beams and the struts from beams or tubes. To minimize obstruction to permanent work construction, no vertical bracing was used. The variable gap between the sheet piling and the back of the walings was packed by divers using a grout bag secured on a mild-steel mesh framework hung from the waling. The bag was then grouted with an expanding grout from a surface mixer. A short length of steel beam was used where the gap at the rear of the waling was excessively large. To facilitate removal of the bag at a later stage, a 25 mm hole was left to allow the placing of a small explosive charge.

The construction of all southern cofferdams was retarded by excavation difficulties in the chalk. The alluvium overlying the chalk was removed by rope grabs from cranes, and then kelly-mounted augers and hydraulic grabs were used on the chalk itself. The chalk shoulders left beneath the wide walings were removed with some extra difficulty by kelly-mounted grabs, by chisels and hydrojets. Blasting was also used, with charges of Polar ammon 80% gelignite limited to 250 g of charge per cubic metre of chalk blown. The charges were placed at least 0·5 m from any chalk/water or chalk/pile interface, with separate detonation by Cordtex fuse. After completion the engineers responsible suggested that a cofferdam slightly larger in plan size may have been more economical and would have reduced the difficulty of removing the chalk from under the walings despite the additional excavation and temporary works.

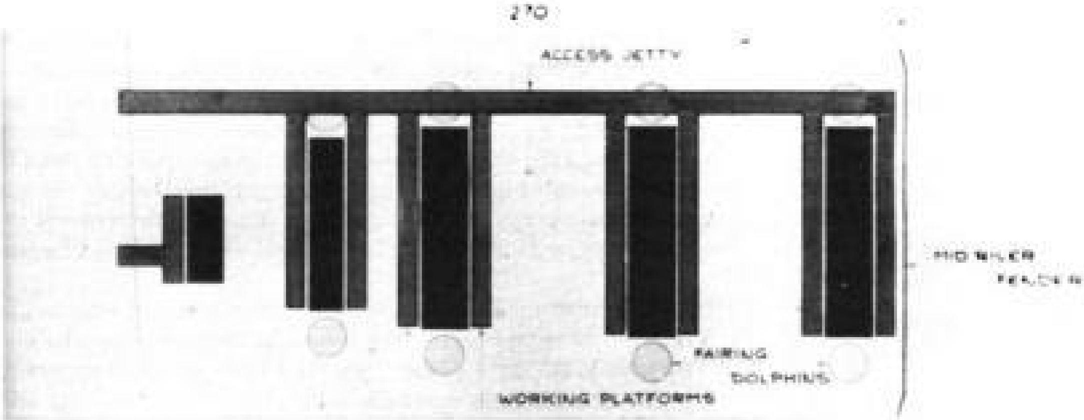
After excavation of the chalk, air lifts were used to sweep and clean the chalk surface to final level, the whole finished surface being inspected and probed by Mackintosh probes from a diving bell.

With the exception of the southern abutment, the concrete plug weight was insufficient to resist the upward pressure and an extensive pressure relief system was needed under each base to achieve a factor of safety against uplift of at least 1·3. Typically the system consisted of 20 tubes, each 865 mm in diameter, temporarily supported from the bottom cofferdam frame. After concreting the plug, and prior to dewatering the cofferdam, the tubes were bored out by auger to 10 m below formation level and filled with gravel. Piezometers and standpipes were installed and monitored to check actual uplift pressure.

The cofferdams were pumped in stages with progressive checks on the effectiveness of the pressure relief wells and the integrity of each successive bracing frame. Sluices in the sheeting allowed for reflooding if this was deemed necessary. Considerable leakage through the clutches in all dams was only partly sealed with sawdust and fly-ash. Some split clutches had to be plated over, and water ingress was sufficient to warrant hanging heavy plastic sheets from the walings to reduce nuisance to operatives in the cofferdam (Fig. 6.11(e)).

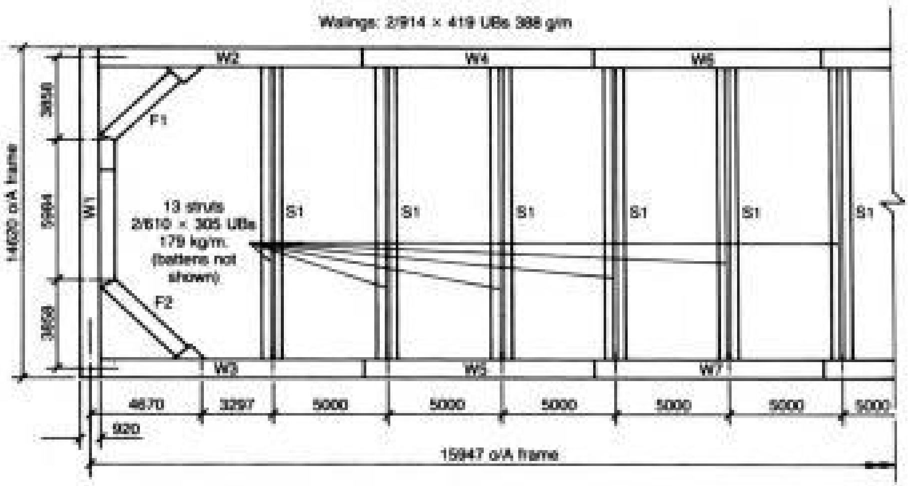
Additional works were required to the pier 7 cofferdam. One of the fault lines in the chalk crossed pier 7 diagonally, and several split clutches were discovered during diving inspections. After the water in the dam had been pumped out, high readings were obtained for uplift pressures on the bottom of the plug and a fine crack was found across the top of the tremied concrete. The dam was reflooded and a reverse filter installed on the outside of the sheeting, with a grouted cut-off on the far side of the filter to prevent further ingress of water from outside the cofferdam. The solution was successful.

The task of placing the concrete plug by tremie to all cofferdams on the



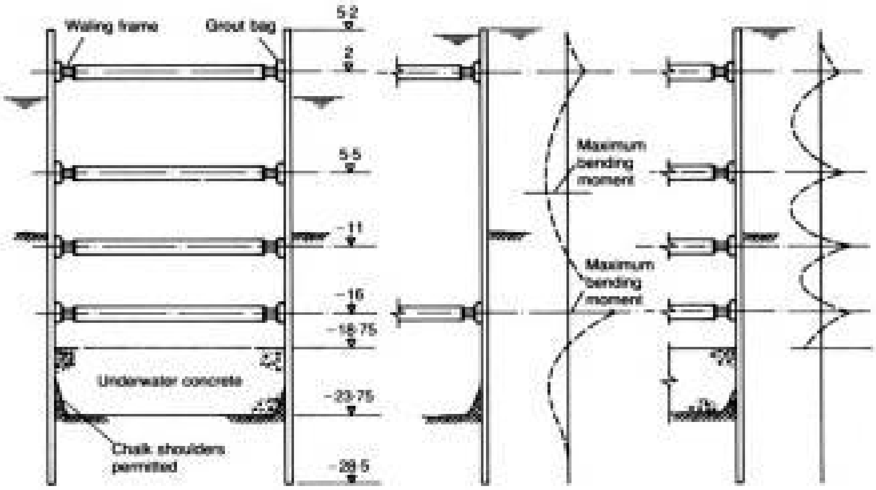
(a)

(b)



(c)

Fig. 6.11. Thames Barrier: (a), (b) river access to southern cofferdams (courtesy of Tarmac); (c) general arrangement of frame at level −5.50, pier 6 cofferdam, plan and cross-section;



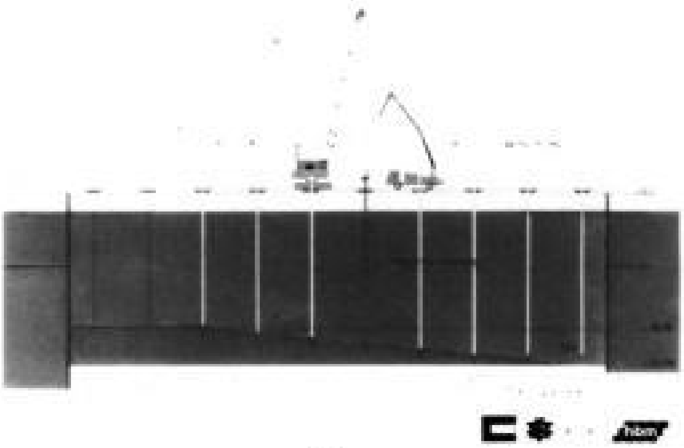
(d)

(e)

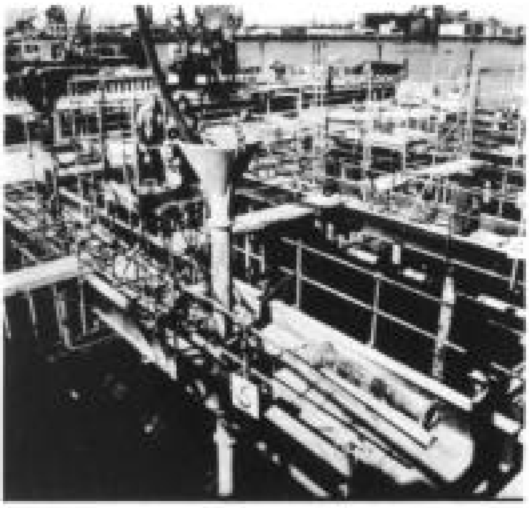
Fig. 6.11 (cont'd) (d) bending moments in sheet piles after excavation and dewatering (Grice and Hepplewhite[9]); (e) view of south-side cofferdam showing heavy leakage (courtesy of Tarmac)

scheme was a major operation. Early trials had established a high slump concrete mix incorporating pulverized fuel ash as half of the cementations content to reduce the heat of hydration. In each cofferdam pour of 8000 m^3 the site-mixed concrete was poured continuously through tremies handled by tracked cranes. The tremies, 300 mm in diameter, were arranged on a 7·5 m grid as shown in Fig. 6.12. The ends of all tremies were submerged in the concrete throughout the pour. Concrete was delivered to the tremies by 3 m^3 Trucrete vehicles to 125 mm mobile concrete pumps. The pour was brought up in layers of 2 m maximum thickness, the concrete being built up at one end of the pour and advanced forward on the chalk subsurface to scour any silt remaining to the far end of the dam. The major south-side pours, working night and day, took five days to complete.

The north bank cofferdams required extensive sheet piling through the Thanet sand. Experience at pier 2 showed a large number of split clutches in the Larssen No. 6 sheet piles. Pre-boring was used to alleviate the hard driving for the sheet piles on cofferdam 1. For the remaining sheeting at



(a)



(b)

Fig. 6.12. Thames Barrier: (a) longitudinal section through cofferdam showing arrangements for tremie concreting; (b) view of tremie works in progress

piers 3 to 4, Peine piles made from grade 50 steel, but with clutches of higher grade, were driven into pre-augered secant bores which has been filled with a low strength pulverized fuel ash–cement-bentonite grout. The clutches of each pier were stitch-welded 10% to prevent slippage during driving the 38.4 m long piles. Each pair weighed 21 tonnes and the toe of each pair was sculptured with toe plates to reduce skin friction during driving.

A part-plan and cross-section through the pier 4 cofferdam shows the Peine piles (Fig. 6.13). Their clutches proved very efficient and their double skins gave an open box which permitted filling or grouting as an additional sealing measure if needed. The increased strength of the Peine piles allowed the use of only one frame composed of walings from twin 914 × 419 universal beams in high-yield steel and struts from twin beams of the same section.

The maximum load on the walings was 150 tonnes per metre run, with a maximum axial compressive strut load of 1200 tonnes from the walls. With the risk of cofferdam collapse due to dislodgement of members of the single frame, higher factors of safety were used in the frame design.

Figure 6.14 shows the detail of a rocker bearing which was used to

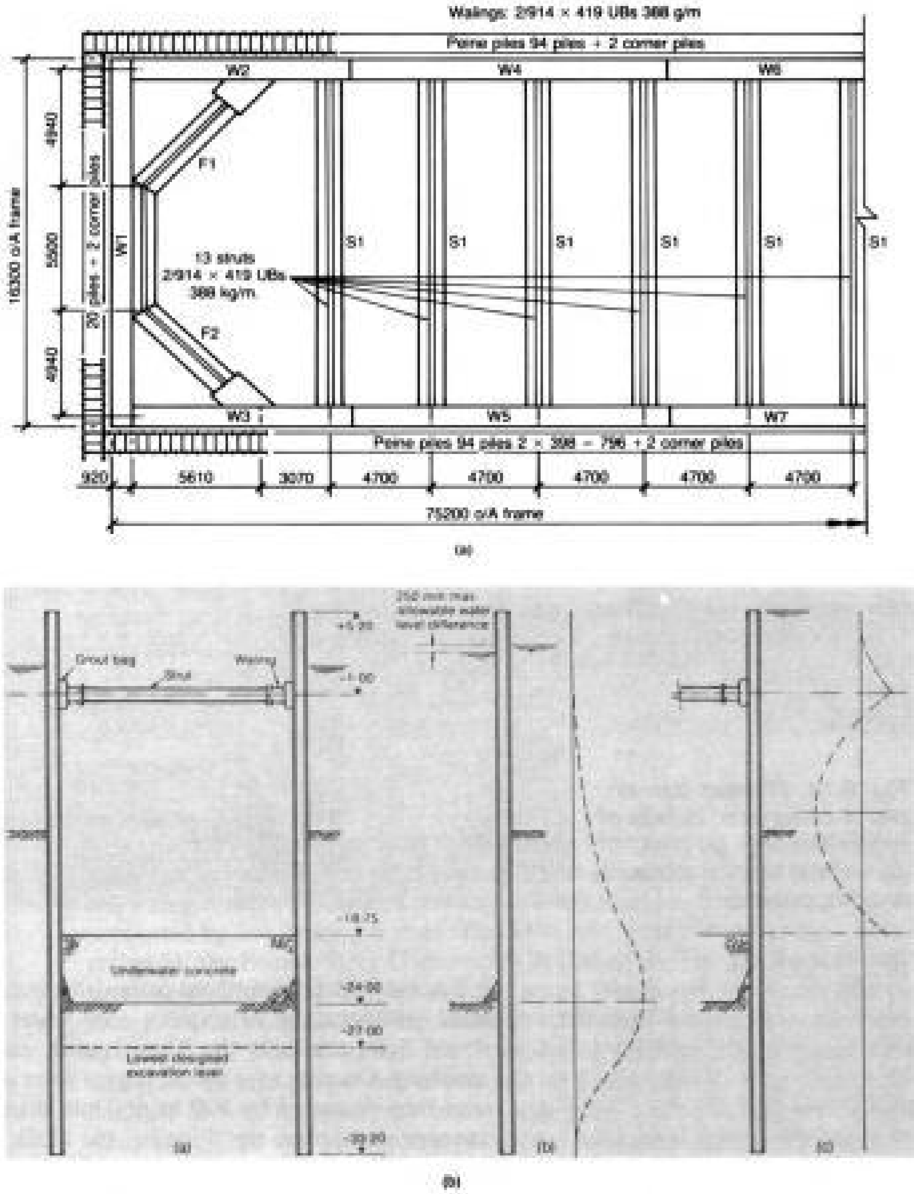


Fig. 6.13. Thames Barrier: (a) general arrangement of frame to pier 4 cofferdam, part plan and cross-section; (b) bending moments in sheet piles after excavation and dewatering (Grice and Hepplewhite[9])

distribute the applied loading from the sheet piling equally to each beam of the twin waling member, the rocker bearing accommodating the deflected form of the sheet piles. A secondary waling with header beams set into the pile heads of 305 × 305 column section was used to smooth out pile deformation during re-strutting of the sheet piles against the permanent works as the main strutting was removed.

For the 9005 section Peine piling, driven using D62 hammers on two Menck MR 60 rigs working 16 hours per day, the period of driving was nine weeks for 235 pairs. The average final blow counts were 600 blows per metre.

With the exception of cofferdam 5, the cofferdams formed from Peine piles were closed with pre-welded Larssen clutches. The seal was then enhanced by grouting within the blister. This precaution was adopted because of the rigidity of the Peine section and the double clutching incorporated in it.

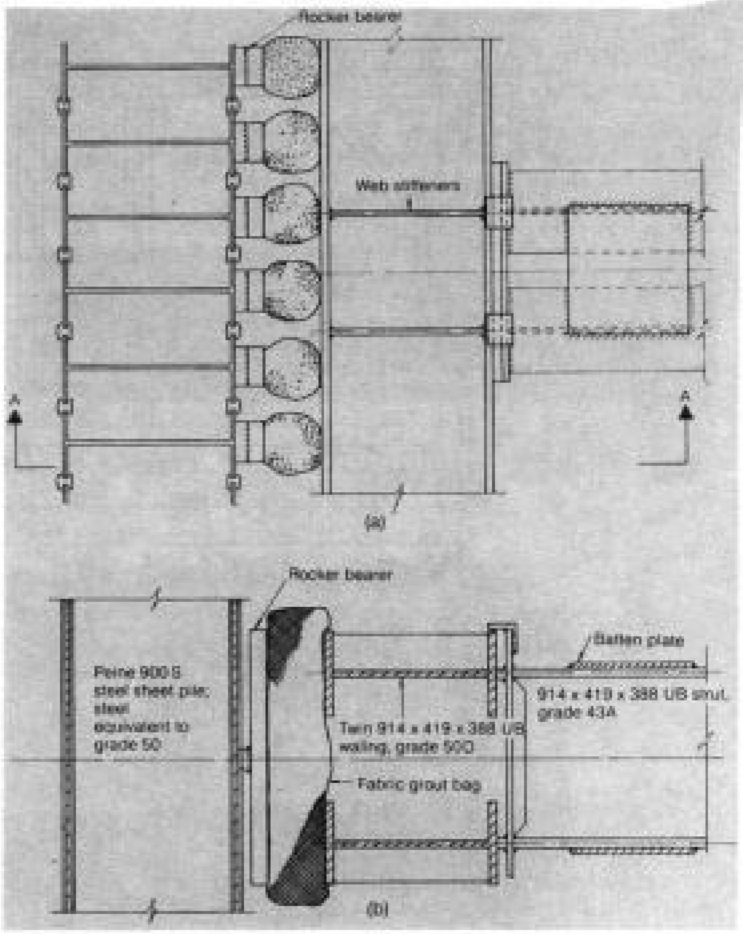


Fig. 6.14. Thames Barrier: pier 4 cofferdam, details of rocker bearing: (a) plan; (b) vertical section (Grice and Hepplewhite[9])

Excavation of the northern cofferdams was made easier by the single frame used to support the Peine piles. Pier 3 was founded on Thanet sand, piers 4 and 5 on chalk. In the Thanet sand, excavation was by augering and airlifting. Once into the chalk, augers were used to break up the chalk which was then removed by 500 to 660 mm diameter airlifts aided by a heavy reverse circulation rig. Finally, the chalk surface was cleaned by airlift sweeps. Fig. 6.15 shows sections of the north cofferdam and details of the construction pressure relief system.

(*b*) River Hull Barrier. Flood barriers built across other English rivers in the late 1970s also serve to illustrate construction techniques in river cofferdams. The tidal surge barrier built across the River Hull and completed in 1979 required deep cofferdams for both the monoliths on which the towers were founded and the cills. The works were described by Fleming *et al.*[10] Fig. 6.16 shows the cofferdams during construction.

The feature of the works (unlike the Thames Barrier) was that the length of the piles was large in relation to the plan area of the cofferdam. The foundation solution for the barrier towers was to found them on mass concrete monoliths within dense glacial sand and gravel approximately 20 m below the river bed level. To avoid disturbance to the subsoil below formation, the contract specified that excavation for the monoliths should be underwater and that the water level in the cofferdam should be maintained at least 0·5 m above the tidal level outside. Mass concrete to a depth of at least 14 m had

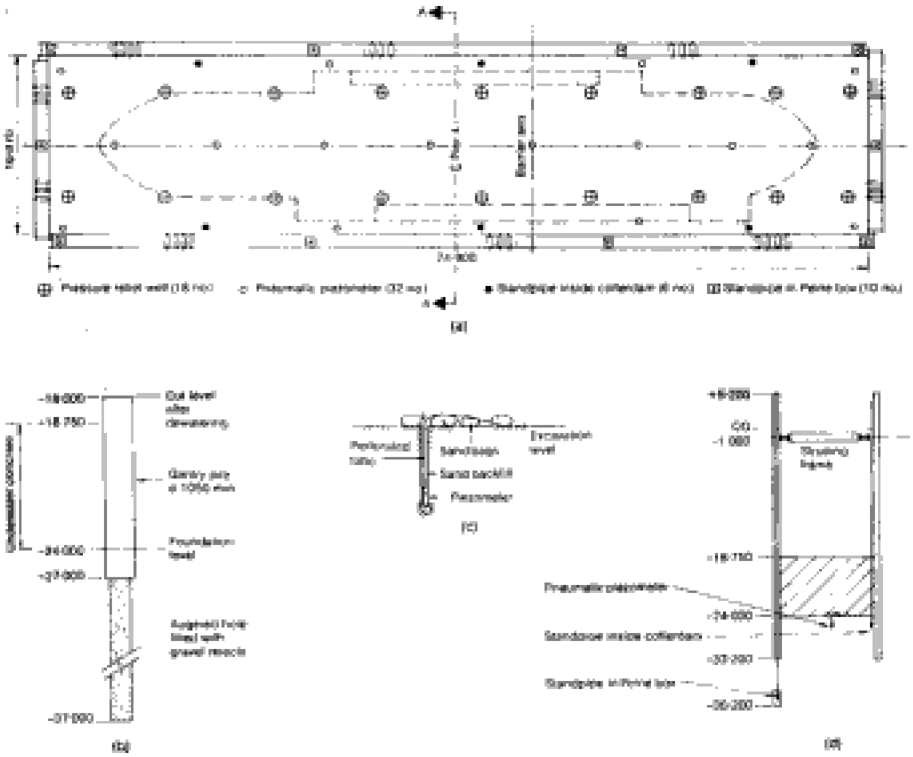


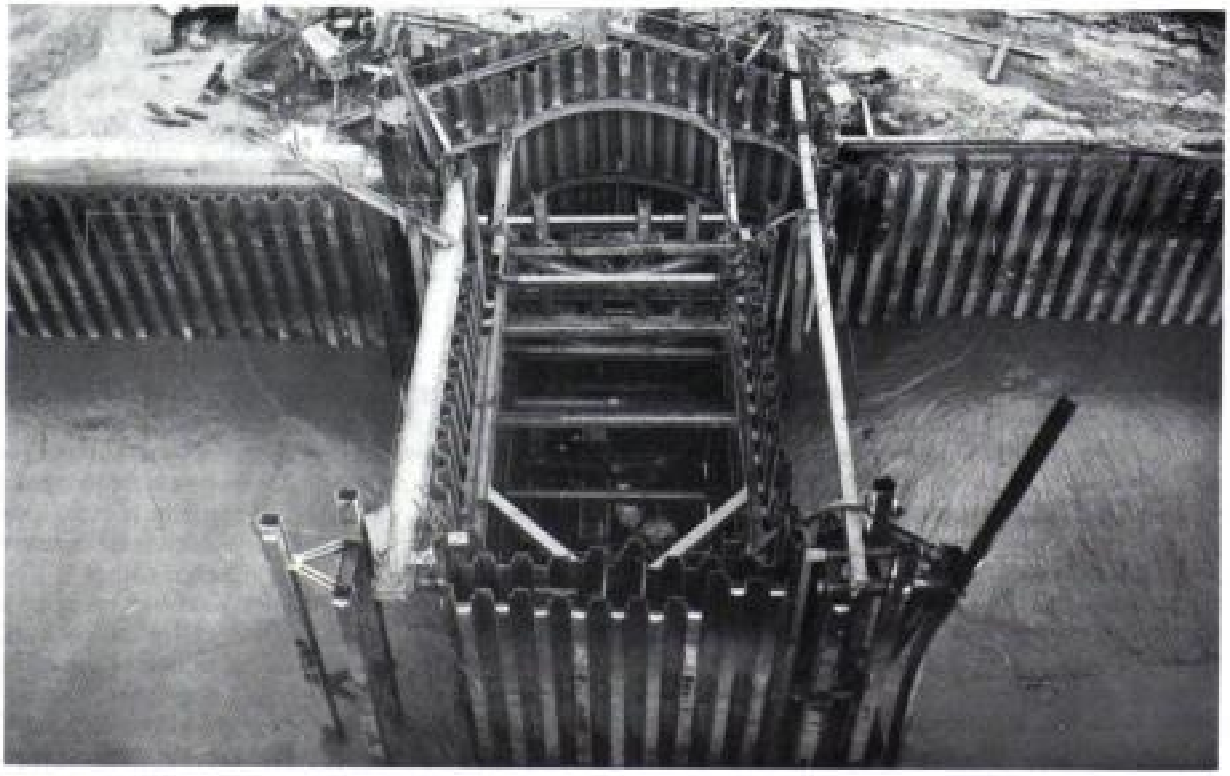
Fig. 6.15. Thames Barrier, north cofferdam construction: (a) plan of cofferdam showing relief well positions; (b) cross-section of cofferdam at A-A; (c) vertical section through relief well; (d) sequence of restrutting to allow removal of original frame to accommodate pier construction (Grice and Hepplewhite[9])

to be placed by tremie to form the monolith before piping was guaranteed not to occur. A cofferdam sheet piled in two stages was originally considered, but a single-stage cofferdam was constructed with Larssen No. 6 sheeters supported by five bracing frames. The 34 m long sheet piles were the longest rolled by the British Steel Corporation at that time. The top three frames were to be fixed above water and the lower two below water, by divers. Due to the small radius of the monolith cofferdam walls it was necessary to reduce the friction in the interlocks during pitching and driving. This was done by crimping every in-pan pile so that interlocks were in line. In addition it was necessary to ensure that piles were pitched and driven vertically. This was ensured by providing substantial temporary supports at close centres, by instrument checks for verticality and by temporary welding of each in-pan pile to the supports. Driving was carried out in small stages with the toes of the sheeters all at approximately the same depth below ground level. Suspended drop hammers of ram weight 5 tonnes and 7·5 tonnes were used to begin driving, which was completed by a Delmag D46. Excavation was by rope grab, and the two lower frames were suspended from frame 3 before the dam was flooded. The packing from the two lower frames to the sheeters was of pieces of universal column, measured individually by the divers and fitted with a single bolted clamp.

Airlifting was used for the final excavation. During the bottoming-up of the west monolith cofferdam a blow of approximately 80 m^3 of loose sand occurred through a split in the intermittent welding of one of the corner piles. The split, about 50 mm wide, started about 2.5 m above formation level. With the risk of further loss of ground if the sand had been removed, it was left in place and later consolidated by grouting after the tremie concrete had been placed. During the final excavation work for the east monolith

Fig. 6.16. River Hull tidal surge barrier: (a) pitching 34 m long Larssen no. 6 piles to monolith cofferdam; (b) completed cill cofferdam; (c) (facing page) sheeting and bracing to monolith cofferdam (courtesy of Dawson)

(a)



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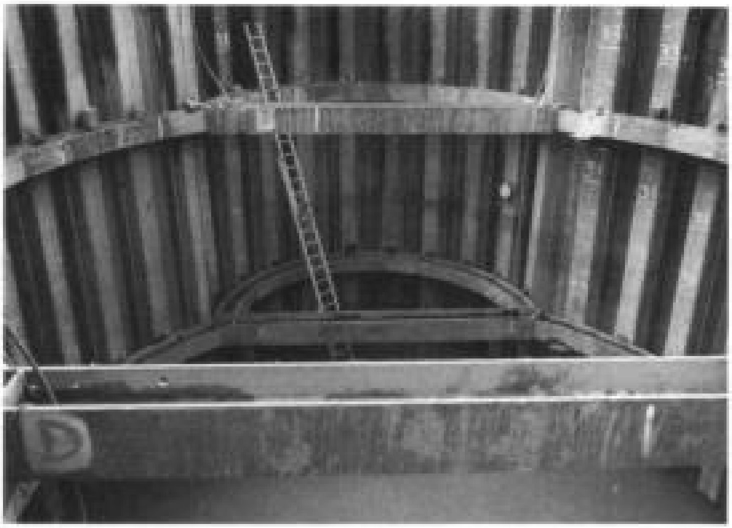


Fig. 6.16 (cont'd)

(c)

cofferdam a digging grab impact caused the lower three frames to drop to the bottom of the excavation. The lowest frame was refixed in an intermediate position and there was no inward movement of the sheet piles. Due to the head of water maintained inside the cofferdam, a differential of about 1 m at all times, the net loading in the frames was very small and the two dislodged frames were not refixed.

The monolith was concreted underwater by tremie to a depth of about 16 m. A total of 1550 m^3 of ready-mix concrete, retarded for 7 h, was placed through four tremies in 34 h continuous pours for each monolith.

(*c*) The Fobbing Horse Barrier. Fig. 6.17 shows an aerial view of works on the Fobbing Horse Barrier in Essex, part of the Thames Flood Defence Works completed in the early 1980s. The bracing details of each cofferdam are shown in Fig. 6.18. The south-west pier is strutted by diagonal steel tube bracing, while the north-east pier uses the alternative method of cross strutting with Rendex box pile sections. Both steel tubes and Rendex sections are efficient strut members but suffer the disadvantage of low flexural strength compared with universal beam and column section.

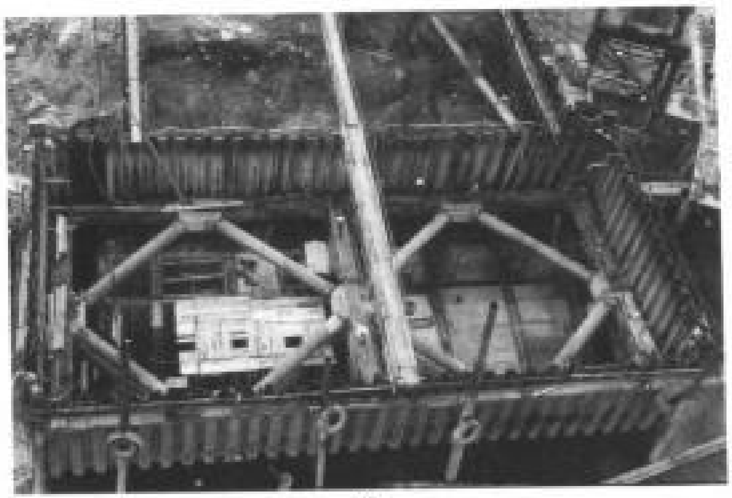
(*d*) Forth Road Bridge. Successive stages of cofferdam works to a suspension bridge tower construction are shown in Fig. 6.19. The corner braces used to provide horizontal support to each of the three frames are augmented by one cross strut to each frame on the longer side of the cofferdam. Note the increased structural stiffness provided by relatively light diagnonal steel bracing between each of the frames on the longer side, and between the puncheon and upper and lower frames across the cofferdam.

(*e*) Kingsferry Bridge. The layout of cross bracing and diagonal bracing to river cofferdams for a bridge construction at the Isle of Sheppey in Kent is shown in Fig. 6.20. Diagonal braces are used, it will be noted, to give maximum space for the passage of excavation grabs and equipment and to minimize expense in cross strutting.

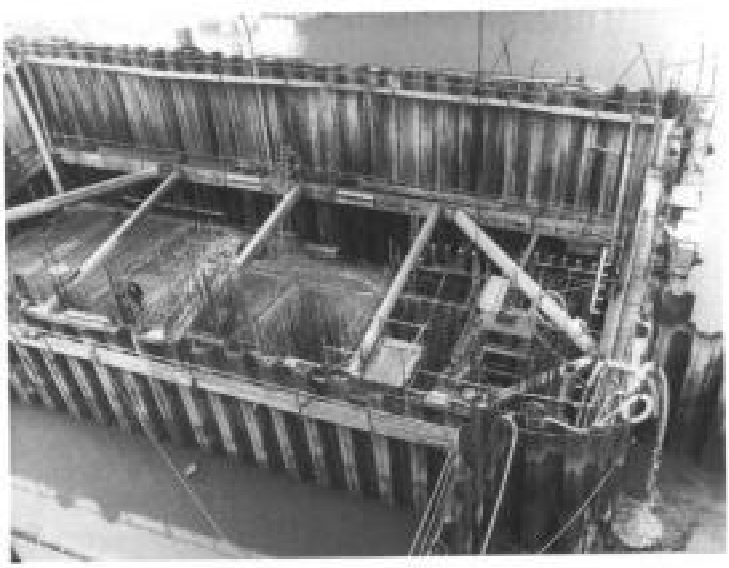
Braced cofferdams on land

Cofferdams constructed on land and braced for support include excavation for generating plant, pumping stations and underground facilities for various industrial purposes. The problems which arise with such cofferdams and their method of solution are sometimes common with other deep excavations on land for building

Fig. 6.17. View of Fobbing Horse Barrier job site (courtesy of AMEC)

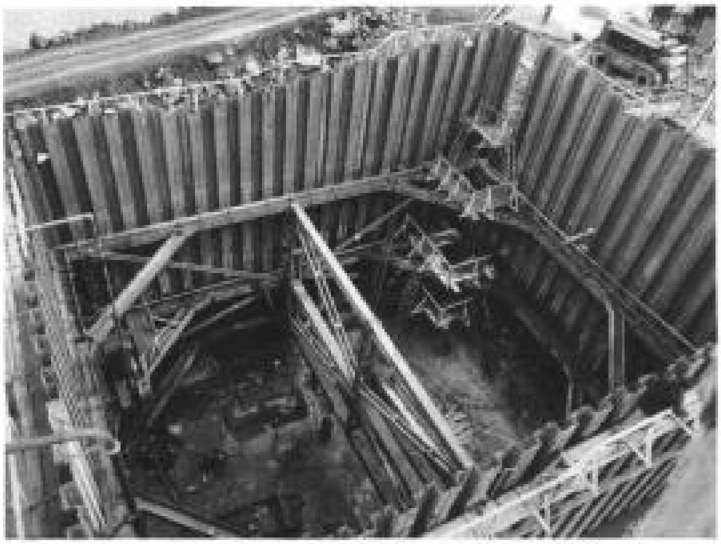


(a)



(b)

Fig. 6.18. Fobbing Horse Barrier: (a) south-west pier cofferdam; (b) north-east pier cofferdam (courtesy of AMEC)



(a)

(b)

Fig. 6.19. Forth Road Bridge: (a) south-side tower cofferdam construction (note diagonal bracing); (b) tower base construction within cofferdam (courtesy of AMEC)

basements and cut-and-cover construction for transportation systems. Nevertheless, because building basements and tunnels built in trench possess some special features they are considered separately in chapters 8 and 9. The purpose of this section is to describe constructional features of braced land cofferdams, typically those used in pumping stations and similar industrial subsurface structures.

The five main differences between constructional features of land and marine braced cofferdams are:

(*a*) Land cofferdams require much less work to provide temporary access for plant, labour and materials than many jobs over water.
(*b*) While the choice of sheeting for schemes over water is often limited to steel sheet piling, a wider range of sheeting and walling methods are available for work on land. The location, depth of excavation, depth to the groundwater table, subsoil conditions and depth to bedrock, if present, will all help to determine the most economical sheeting system. In addition to steel sheet piles, the choice for work on land includes diaphragm walling by in situ

reinforced and post-tensioned concrete and precast methods, continguous piles, secant pile walling using hard–hard and hard–soft methods, and temporary soil support from Berlin type walls with soldier piles and horizontal lagging timbers.

(*c*) The means of bracing land cofferdams includes ground and rock anchors in addition to internal bracing from steel struts and walings.

(*d*) the proximity of existing structures to the excavation requires greater emphasis on problems of soil deformation around the excavation periphery for cofferdams built on land.

(*e*) The loads applied to land cofferdams are not likely to include tidally varying groundwater but allowance must be made for superimposed loading from other structures, and from traffic and site plant around the excavation periphery. In addition, in less temperate lands, the effect of freezing soils on soil pressures and bracing loads must be considered.

Choice of sheeting

Whereas methods such as the Berlin wall provide temporary soil support to allow construction of the permanent wall, secant or contiguous piling and diaphragm walling allow the temporary walling to be incorporated into the permanent wall structure. The saving in construction time and cost had led to the increased use of these latter systems in land cofferdams.

In other instances, the experience of specialist contractors with diaphragm wall construction has allowed the introduction of composite sheeting methods. Although only used for temporary support, sheet piles pitched rather than driven into slurry trench excavations have proved attractive cost-wise and avoid the environmental problems of noise and vibration to adjoining structures. It is often necessary to place tremie concrete below formation level to provide sufficient passive resistance to sheet piles pitched in this way.

An example of Berlin walling is shown in Fig. 6.21. The method, for a pumping station in Jubail, Saudi Arabia, found favour because of a low groundwater table, an essential prerequisite for Berlin walls unless dewatering is to be employed, and possibly due to lack of competitiveness by specialist diaphragm walling firms for a relatively small job which would be distant from plant and resources.

(a)

Fig. 6.20(a)

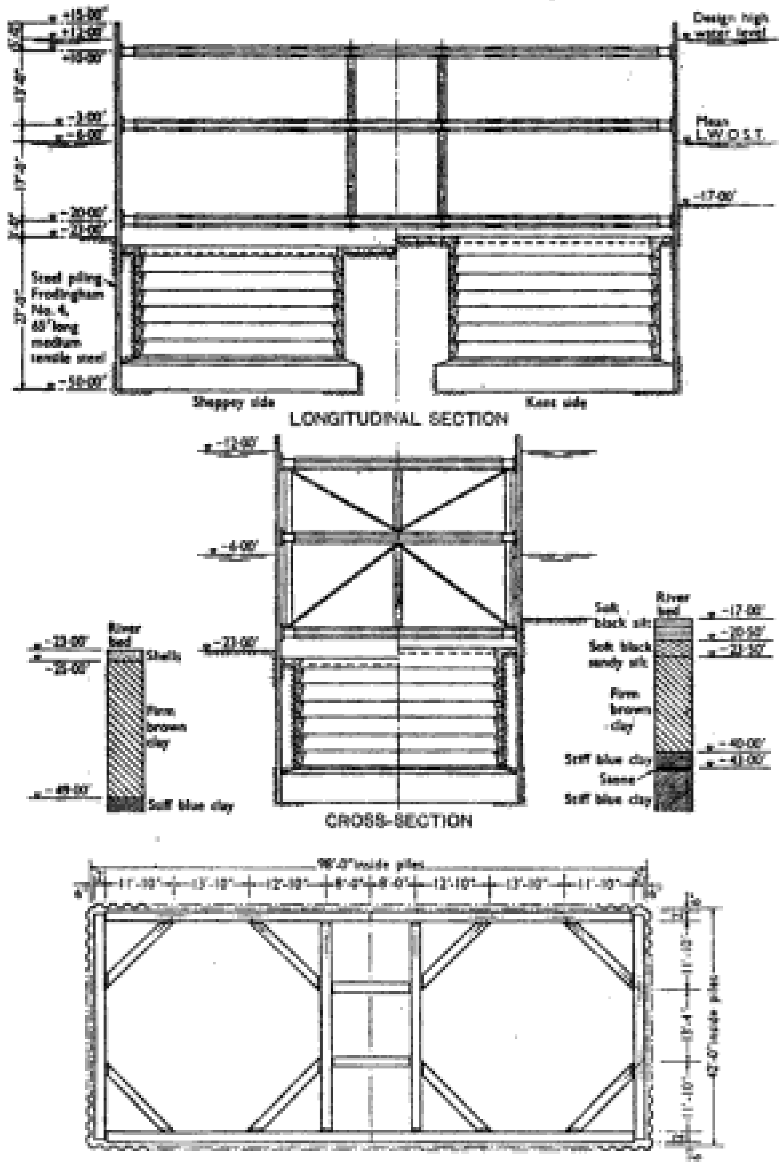


FIG. 4b.—PLAN OF COFFERDAM FOR MAIN PIERS, KINGSFERRY BRIDGE
All frames are similar, and all struts and walings are 24 × 7·5-in. R.S.J.'s in pairs

1. Frames at levels + 10·00 ft and − 3·00 ft to be fixed between tides before closing dam.
2. Dam to be closed at low water and pumping started at once to avoid reversed pressure.
3. Remove all silt, mud, and soft material from bottom: then level and place mass concrete blanket over exposed clay not less than 12 in. thick, leaving working space to construct the first in-situ concrete ring.
4. Excavate for piers in 3 ft 0 in. to 4 ft 0 in. cuts and support sides with in-situ mass concrete rings.
5. Undercut the base to be excavated and concreted in radial segments.
6. Bottom frame may be struck at any time after concrete blanket near river bed leve has hardened.
7. Walings from bottom frame slung below second frame and subsequently used as a jury frame strutted from new construction before striking the second frame.
8. Second frame walings lifted and used similarly as a jury frame below the top frame.

(b)

Fig. 6.20. Kingsferry Bridge, Isle of Sheppey; (a) (facing page) view of site (courtesy of AMEC); (b) cross-sections through river pier cofferdam and construction sequence (Packshaw[1])

Fig. 6.21. Berlin wall method using anchors to scale pit building, depth to formation level 14 m, groundwater table at a depth of 3 m, steel plant, Jubail, Saudi Arabia (courtesy of Bauer)

Figure 6.22 shows the use of sheet piling to the substructure of an ash pit at West Thurrock Generating Station, UK.[11] The works, in difficult ground conditions, were sited adjacent to a reinforced concrete chimney already partly built. The ash pit, founded on the gravel some 16 m below ground level, was built in cofferdam in preference to sinking a caisson because of the risk of undermining the foundations to the chimney if a soil blow were to develop under the cutting edge. The soft marsh clay was comparatively impermeable and had a shear strength of about 30 kN/m^2. The underlying gravel contained an artesian head of groundwater which almost reached ground level and rested in turn on fissured chalk which also contained groundwater. The excavation would have been very difficult in the dry in the cofferdam and it was decided to excavate to third frame level in the dry and take out the remaining 6·4 m depth underwater prior to concreting a plug between levels −8·54 and 13·72. Pressure relief wells were also installed to relieve the artesian head in the gravel. These wells consisted of 670 mm dia. bores filled with gravel on a 4·6 m grid, and were installed before the excavation started.

After excavation down to just below second frame level, with top and second frame fixed, sand and silt began to flow from the relief wells which had previously discharged satisfactorily. The cofferdam was immediately flooded and six deep wells with submersible pumps were installed outside the cofferdam and 6·5 m below the clay. The cofferdam was pumped out and the third frame fixed in position. The cofferdam was then reflooded to level −1·82 and the excavation completed underwater with the artesian head relieved by the deep wells. The cofferdam was made rigid by welding sheet piles, walings and struts. By taking these measures, diagonal bracing was not needed. The concrete plug was formed by grouting a gravel layer 5·18 m thick underwater using the Intrusion Prepakt method, and after completion the cofferdam was pumped dry. A 380 mm layer of blinding concrete and a 1·2 m reinforced concrete floor slab, both anchored into the anti-flotation plug, completed the ash pit bottom.

This cofferdam was built just before the introduction of diaphragm walling and secant piling into the UK: in the soil and groundwater conditions that existed at the West Thurrock site it is doubtful whether these alternative methods would have shown any advantage over sheet piling had they been available. However, it is likely that jet grouting methods would have been used in lieu of the Intrusion Prepakt system.

Measures to reduce settlement around land cofferdams deserve discussion. The extent of such deformation in a stable cofferdam depends on soil bracing and sheeting stiffness and the vertical spacing of bracing frames. Diaphragm wall and reinforced concrete pile sections are stiffer than conventional sheet pile sections and their use is likely to minimize soil deformation adjacent to the excavation. Further precautions may prove necessary, however, and these include installation of frames or other support at minimum vertical centres, pre-loading of struts, use of prestressed soil and rock anchors, use of jet grouting and pin piling to artifically stiffen soil and increase passive resistance to the sheeting below formation level.

The use of flat jacks to stress the frames of a land cofferdam to reduce soil movement and the resulting subsidence was reported by Colingridge and Tuckwell.[12] The excavation, in London, was for a new sub-surface station for the Post Office. It was 61 m long, 29 m wide and 22·3 m below ground level. At the time it was one of the largest excavations made in Central London.

The contract specified earth pressure at rest in the temporary works design. The ground conditions, typical of London, were made ground and Thames gravel overlying London clay. The specified values of K_0 were 0·75 in the clay and 0·5 in the fill and gravel. To exclude groundwater in the gravel and provide continuous support for all soil above the London clay, a peripheral sheet pile wall was specified

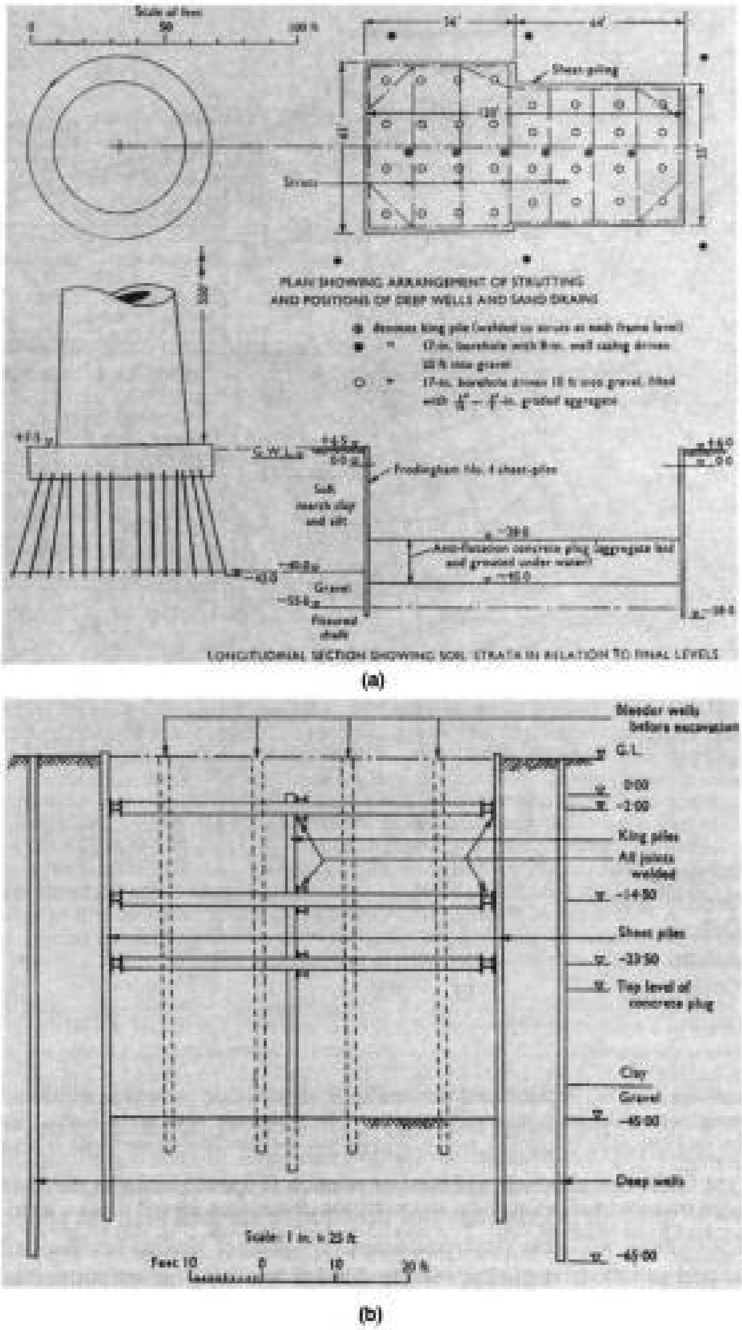
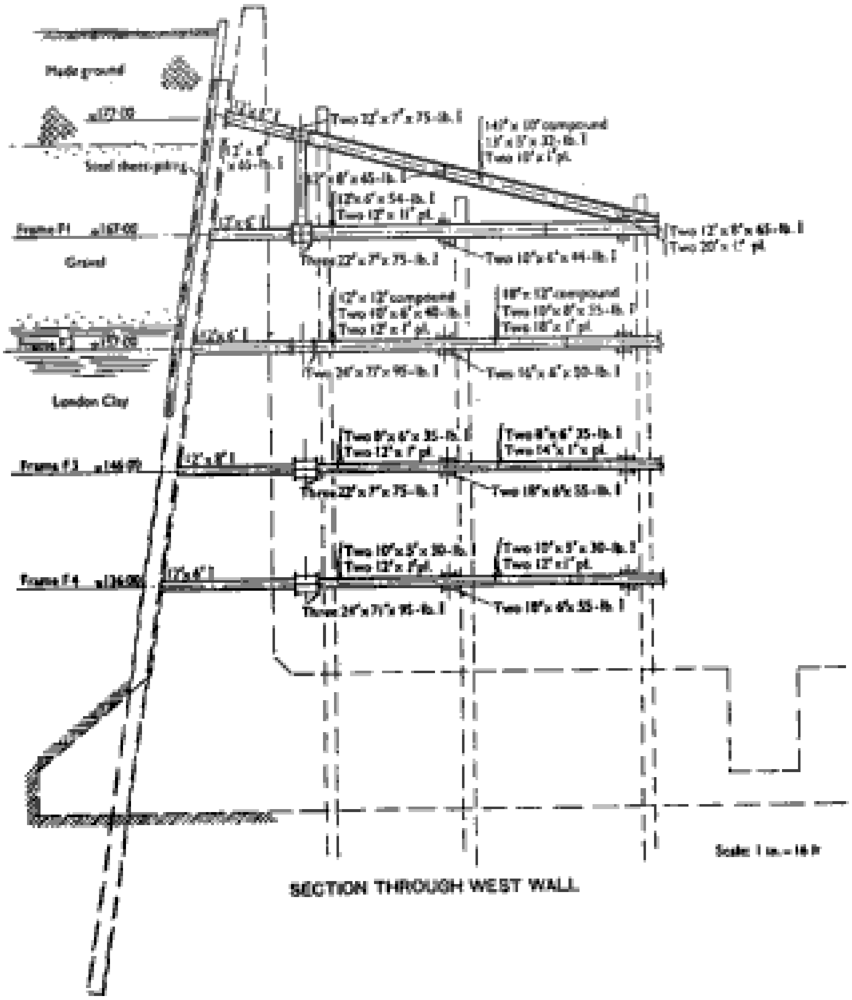


Fig. 6.22. West Thurrock Generating Station: (a) plan and longitudinal section of ash pit disposal cofferdam showing deep wells, sand drains and anti-flotation plug; (b) transverse section of cofferdam (Wakeling and Hamilton[11])

to be driven from below basement level of existing buildings to a penetration of 2·5 m into the clay.

The method of excavation support is illustrated in Fig. 6.23. Steel H soldier piles 25 m long were driven to the batter of the outer face of the permanent wall at 1·6 m centres, achieving penetration 4·6 m below formation level of the permanent structure. An intermediate, 0·23 m thick reinforced concrete wall spanned the soldier piles. Four horizontal frames and a top raking frame were used to support the H sections, needle joists being provided between the soldier



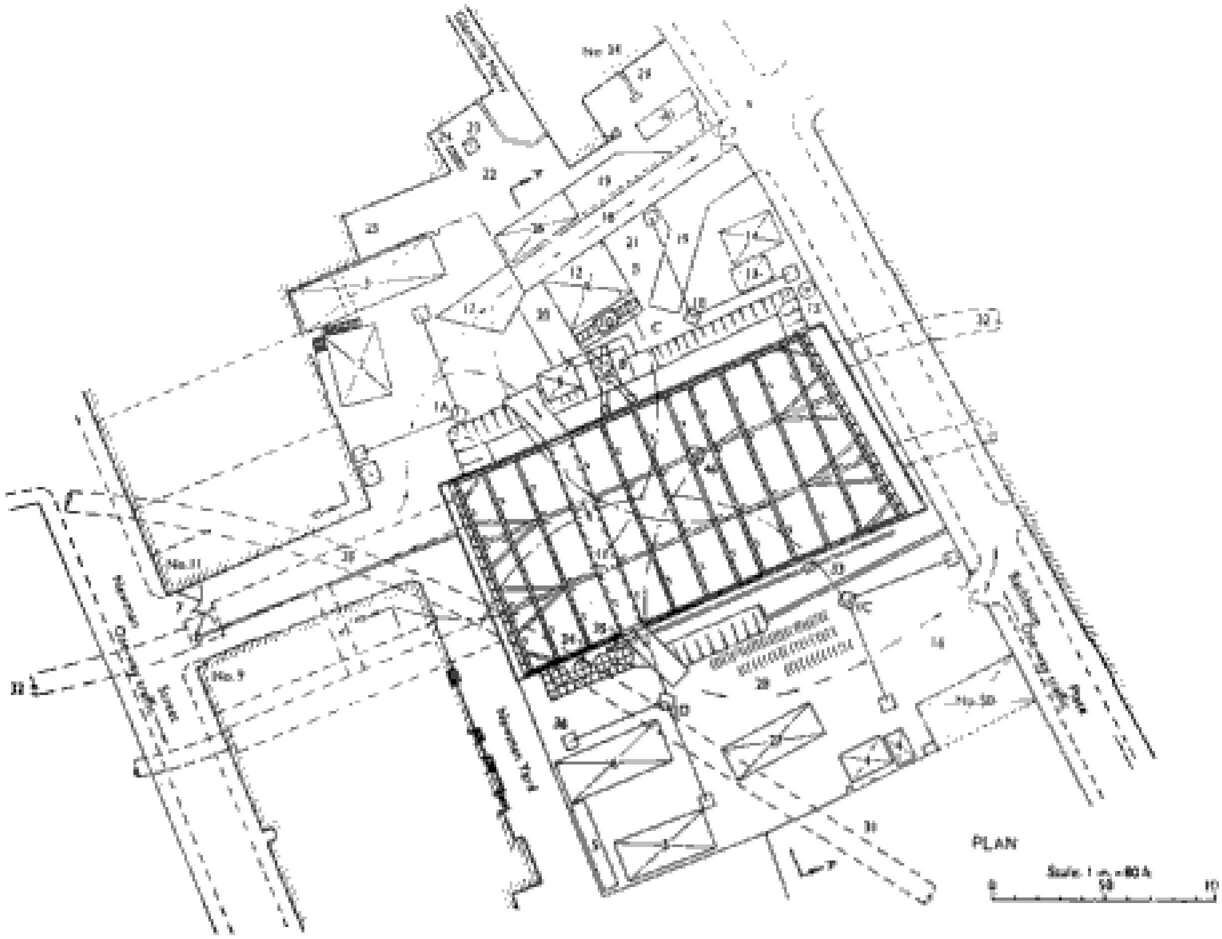
(a)

Fig. 6.23. (a) Section and (b) plan of cofferdam and framing, Post Office underground station, London (Collingridge and Tuckwell[12])

piles and the walings. Provision was made for jacking each needle joist. As each frame level was reached during bulk excavation, the steelwork was assembled and stressed to a load calculated to retain the subsoil in a state of rest. The aim of needle joists between the main struts (Fig. 6.24) was to allow a more uniform distribution of pressure to the soil than the application of load to the sheeting at the strut positions. Freyssinet hydraulic flat jacks, with small travel and high capacity, were linked by hydraulic connection and pressure was applied simultaneously in four operations per frame. One operation used 16 jacks at one end to stress the frame longitudinally, and three operations using 12 jacks on each side were used to stress the frame transversely. The two jack sizes used were 27 cm diameter rated at 64 tonnes and 35 cm rated at 114 tonnes. The total jack load was applied in increments of 25%, and a complete jacking operation was completed in half a day. Allowance for temperature stresses avoided overstressing in hot weather, jack load being reduced by 10% for every 5·5 °C above 21 °C at the time of stressing.

Collingridge and Tuckwell[12] concluded that pre-loading of steel frames with calculated jack loads proved a satisfactory method of preventing ground movements, the uniform transfer of strut load to the soil being very important.

The use of flat jacks to pre-load frames in an attempt to avoid settlement damage



(b)

has reduced to some extent since the development of ground anchors, which serve the same purpose. Nevertheless, easement arrangements for anchor installation below neighbouring land and highways frequently experience problems, and it is in these particular sites that pre-loading methods still find application.

The use of pre-tensioned anchors through vertical sheeting or walling to reduce movements caused by large excavations is only effective where the anchorage zone lies outside the soil movement zone associated with the excavation. Long-term soil movement due to consolidation within the anchorage zone limits the application of soil anchors for permanent anchorage to clay soils. Tomlinson[13] has reported a summary of observations of maximum horizontal movement of soil support for excavations in normally consolidated, overconsolidated clays and gravels. These results are summarised in Table 6.2. The prediction of vertical deformation adjacent to excavations is reviewed in chapter 11.

An example of pumping station construction using diaphragm walls of varied plan shape is shown in Fig. 6.25. The site, at Redcar, UK, was close to an existing quay wall built in diaphragm wall construction. The groundwater level was high and the subsoil conditions were fine sands overlain by filling composed of broken blast furnace slag. Groundwater was lowered by deep wells in the fine sand located within the cofferdam, and despite large overbreak to the diaphragm walling within

Table 6.2 Observed values of maximum horizontal deflexion of sheeting to excavations on $/\delta h_d$ (Tomlinson[13])

Soil type	*Wall type*	*Number in sample*	*Maximum horizontal deflexion/excavation depth %*		*Range of excavation depth: m*
			Range	*Average*	
Soft to firm normally consolidated clays	Anchored diaphragm wall	3			
	Strutted diaphragm wall/secant pile wall	4	0·08–0·58	0·30	9 to 24
	Strutted sheet pile, soldier pile and concrete infill	5			
Stiff to hard over consolidated clays	Anchored diaphragm wall	2			
	Strutted diaphragm wall/secant pile wall	6	0·06–0·30	0·16	10 to 30
	Strutted sheet pile, soldier pile and concrete infill	1			
Sands and gravels	Anchored diaphragm wall	2			
	Strutted diaphragm wall, secant pile wall	5	0·04–0·46	0·19	7 to 20
	Strutted sheet pile, soldier pile and concrete infill	1			
	Anchored sheet pile, soldier pile with concrete infill or timbered	4			

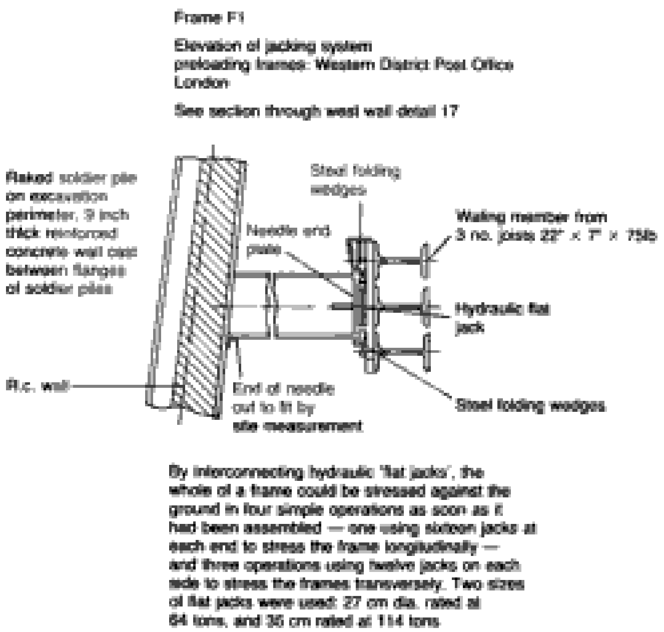


Fig. 6.24. System for pre-loading cofferdam frames, Post Office underground station (Collingridge and Tuckwell[12])

the fill material, the works were completed successfully. Fig. 6.26 shows the individual panel components of the construction; the action of the heavily reinforced capping beams in limiting movement by any out-of-balance forces on panel units should be noted.

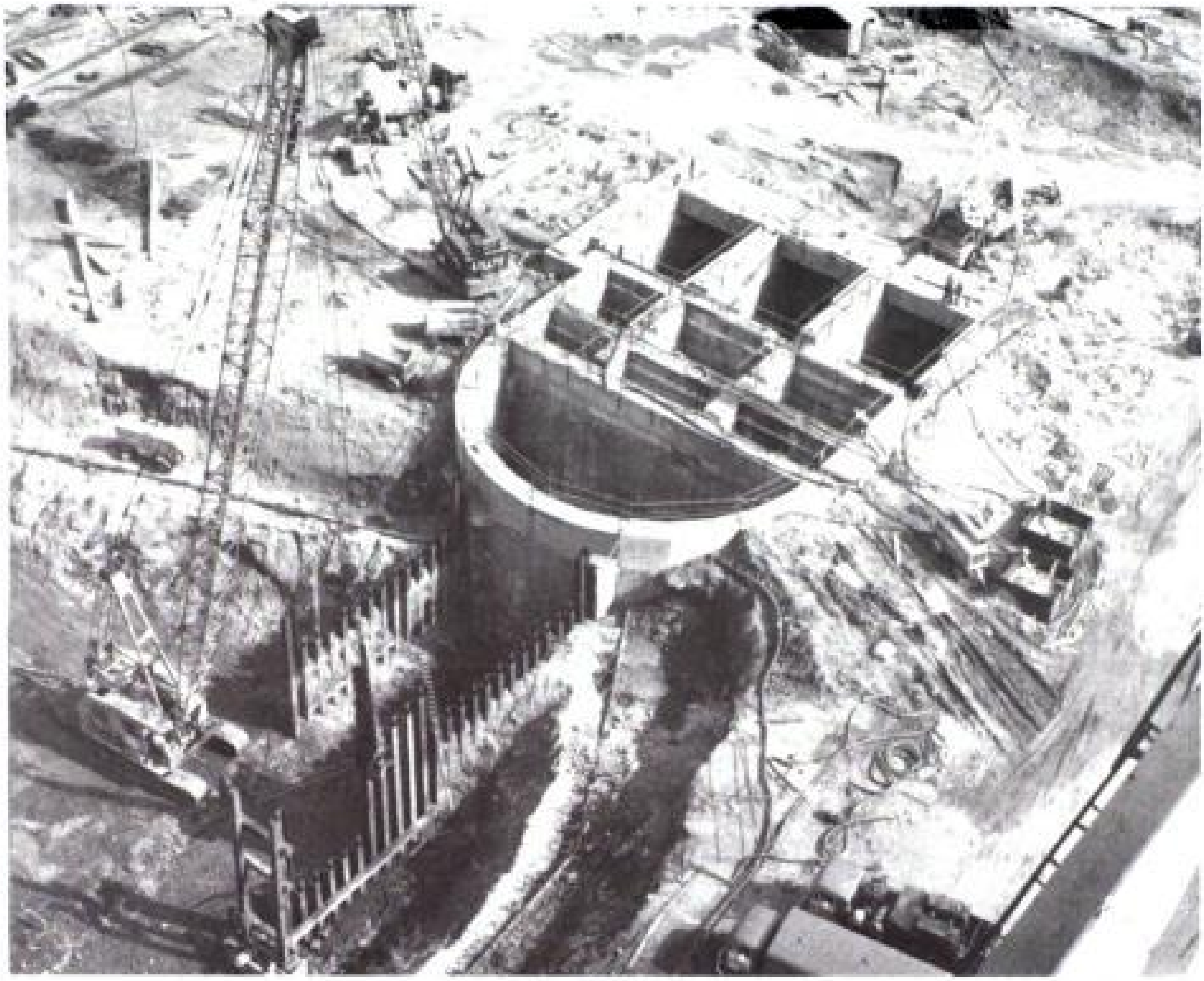


Fig. 6.25. View of construction of diaphragm wall to pump house, steelworks, Redcar (courtesy of Lilley)

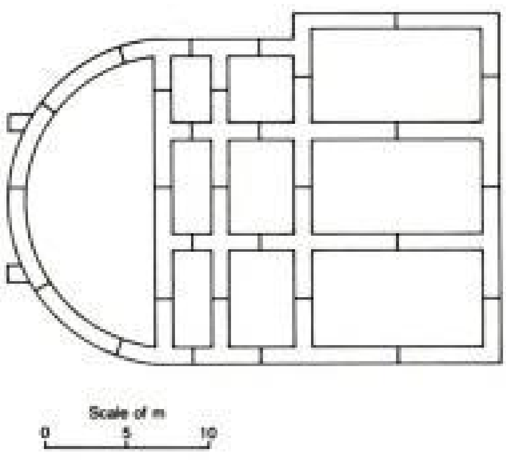


Fig. 6.26. Redcar pumping station: panel layout to 1 m thick diaphragm walls spanning vertically between base slab and capping beam without bracing

Double-skin cofferdams

Double-skin cofferdams built from steel sheet piling and enclosing soil or rock fill divide into two categories.

(*a*) Double-wall cofferdams. These consist of two parallel lines of sheet piling tied by steel rods at one or more levels between external walings and a fill material, such as sand, gravel, hardcore or broken rock, between the sheet piles. The filling requires adequate drainage which may be maintained by internal sluices or deep wells below the fill. Berms may be used to reduce lateral movements in the pile – fill – soil structure and increase stability. If bedrock occurs at shallow depth and the sheeters cannot be driven into it, this type of cofferdam may not prove adequate to resist seepage nor will it be economical because of high sheeting moments.

(*b*) Cellular cofferdams. These are enclosures, often circular, made from straight web steel sheet piling to contain a filing of sand, gravel and broken rock. The straight piles, which have a high interlock strength, resist pressure from the filling and contained water by circumferential tension in the piling. Adequate drainage of the fill is needed to reduce pressure on the sheeting and to avoid reduction of the shear strength of the filling. The cellular cofferdams are econominal for greater water depths, large retained heights, long structures, and where bracing and anchoring is not possible. The additional requirement in sheet pile area is compensated for in the length of piling and pile section and the absence of walings and anchors. The fill quantities may outweigh advantages in pile tonnage, however. An example of the typical use of a cellular cofferdam, to exclude river water from a large excavation, is shown in Fig. 6.27. The works, for a new lock at Kwaasdmechelen on the Canal Albert in Belgium, were completed in the early 1970s.

Double-wall cofferdams

Examination of the possible modes of failure of double-wall cofferdams serves to illustrate design and construction precautions.

(*a*) Sheet pile flexure; tie rod breakage; passive soil failure at foot of sheet piles.
To avoid overstress in bending or excessive flexural deformation of the sheeters, the pressure from the contained fill and groundwater should be

Fig. 6.27. Cellular cofferdam with berm to cut-off canal water from a large lock excavation, Canal Albert, Belgium (courtesy of Arbed)

as low as possible. In particular, it is essential that the fill should be drained adequately at all times during the cofferdam life. The drainage system may consist of weep holes or sluices in the inner line of sheet piles, possibly connected to an internal filter layer of gravel within the filling. In addition, deep wells through the filing may be used; these wells, with vertical filters extending upwards into the filling, also serve to reduce the exit gradient of water seepage under the dam. Efficient maintenance of the drainage system is vital, and new weep holes must be cut where migration of fine soil within the fill causes blockage.

The extent of pile penetration into the subsoil or rock beneath the dam and the location of the lowest tie rods determine the passive forces mobilized in front of the sheeting. The lowest ties should be installed as low as possible, at low tide level in tidal rivers. Pile penetration will depend on subsoil conditions. Penetration into rock may not be possible; if this can only be achieved by heavy driving, split clutches may occur which in turn could lead to seepage problems and loss of ground. To alleviate problems of piling into rock it may be possible to excavate a trench into rockhead at low tide followed by concreting the pile toe into the trench. In such conditions, construction costs increase and the use of double-wall cofferdams may be uneconomical.

An example of a cofferdam founded on rock at a power station site in India was given by Ellis.[14] The cofferdam, shown in section in Fig. 6.28, was built to allow a dry, unrestricted area for the construction of a colling water pumphouse at Chola in Southern India. Although a feature of the work was the small penetration of the piles into bedrock, it should be noted that the second row of ties were at a relatively low elevation. Note also the plan shape of the dam, which was such to substantially buttress the structure

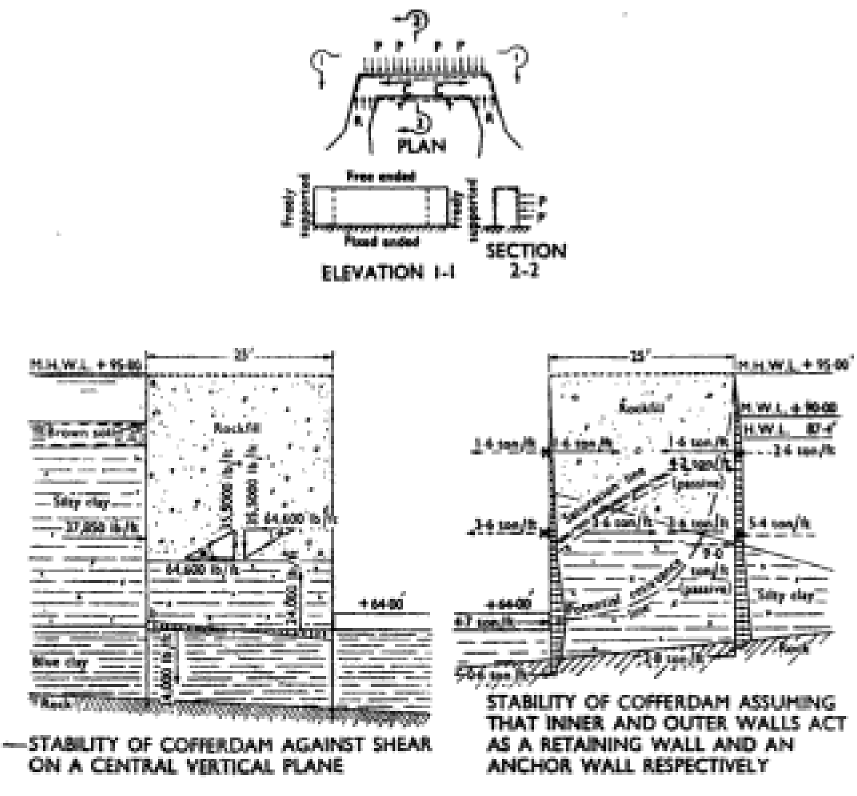


Fig. 6.28. Chola cofferdam, India: (a) plan and assumed support conditions; (b) basis of design, stability considered by shear induced on a vertical plane and assuming inner and outer walls act as a retaining wall and anchor wall (Ellis[14])

at each end, reducing deformation within the dam and stresses in sheet piles and ties.

It is sometimes worth incorporating anchor cells within long lengths of double-wall cofferdams. The cells contribute to construction sequencing but, most importantly, will confine damage to an isolated length of dam should any failure or deformation occur, easing remedial work. A typical arrangement is shown in Fig. 6.29. It may be noted that the cell acts as a strongpoint in the cofferdam and reduces deformation and stresses in piles and soils.

(*b*) Sliding. The total horizontal thrust from river water and ground pressure on the outside of the dam will be resisted by passive resistance of soil below formation level inside the dam and shearing resistance mobilized under the cofferdam due to the weight of fill on cohesionless subsoil or due to the cohesive strength of clay subsoil. The condition is unlikely to dictate cofferdam geometry if pile embedment is sufficient to counter all other modes of failure.

(*c*) the risk of the rectangular cross-section of the cofferdam adopting a lozenge shape will only be reduced by the following measures:

(i) cofferdam fill of adequate uniform quality as placed to ensure resistance to shearing forces mobilized within the filling and resisting tilt
(ii) adequate drainage within filling, to ensure maximum shear strength of fill at all times
(iii) adequate penetration of internal and external lines of sheet piles to mobilize reaction to shearing resistance between fill and face of sheet piles.

The height-to-width ratio necessary to avoid tilt may be low, of the order of 1·0. In addition, substantial soil or rock berms may be necessary to avoid unacceptable deformation at the top of the dam. Fig. 6.30 shows the cross-section of the double-wall cofferdam used during construction of the outer entrance to Gallion's Lock on the Thames in London. It is interesting that despite the extent of the berms and the penetration of the sheeters into the chalk, horizontal deformation of the cofferdam crest at high water reached 0·36 m. Packshaw[1] suggested that the deflexion of the cofferdam may have been aggravated by two factors: the hard-core filling, deposited through water, may have been rather loose, and the rather compressible chalk on which it was built may have contributed to the movement by consolidation of the chalk. The row of deep wells into the chalk should be noted, reducing the piezometric head in the filling and the exit gradient of seepage water under the dam.

Chalk was also used as filling to a double-wall cofferdam during dry construction at Immingham, UK.[15] The cross-section of the dam is shown in Fig. 6.31. There was concern at design stage of the extent of maximum

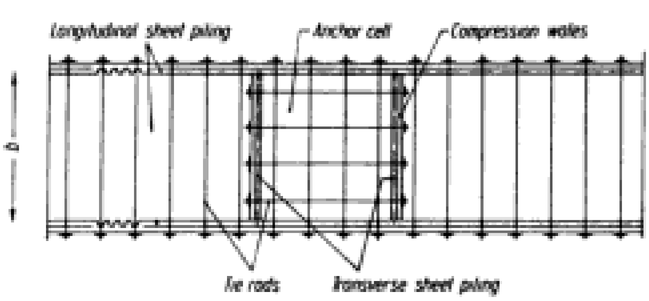
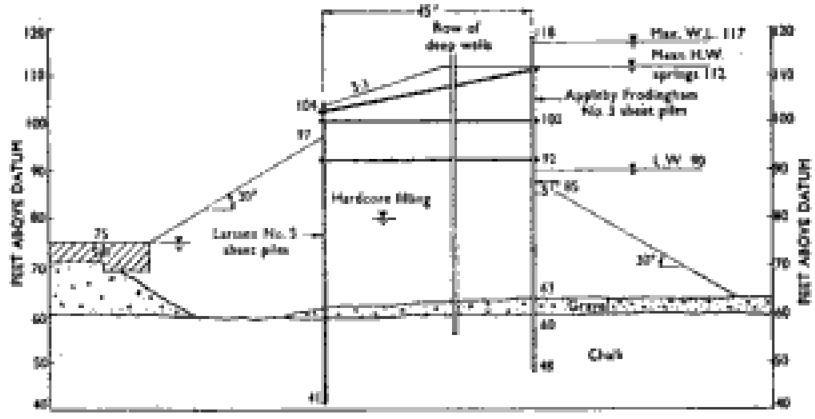


Fig. 6.29. Typical detail in plan of strongpoint in a double-walled cofferdam (German Waterfront Code[18])



DOUBLE-WALL COFFERDAM. GALLIONS LOCK OUTER ENTRANCE

Fig. 6.30. Gallions Lock, double-wall cofferdam at outer entrance showing dimensions of berms, piling and filling

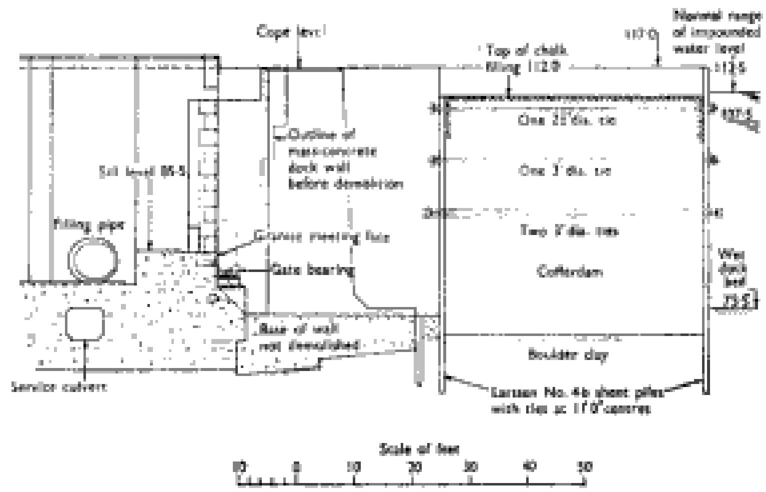


Fig. 6.31. Cross-section of the entrance cofferdam, Immingham dry dock

friction which could be mobilized between the chalk fill and the sheet pile face, so tests were made in a large shear box, with a rusted sample of steel in the lower part of the shear box and chalk in varying compaction states in the upper half. A design value for angle of wall friction of 46° was used as a result of the tests.

A cofferdam built across the entrance to St Katherine's Lock in London during replacement of the lock cill had a width-to-height ratio of 0·81. The cross-section and plan of the cofferdam are shown in Fig. 6.32. The dam, founded on Thames gravel, was drained by sluices in the inner line of piling and by two deep wells with submersible pumps in the gravel. The use of puddle clay seals to achieve watertightness against the existing jambs of the lock entrance is a rather outdated method, but proved successful. The seals provided some support to the dam which had very limited length, but deflexion at the top of the piling under high water reached 0·3 m.

(*d*) Piping. Like any other cofferdam retaining water or groundwater, double-walled cofferdams founded on granular subsoil are at risk from piping failure by flow beneath the structure into the dewatered area. To avoid piping, the emergent hydraulic gradient can be reduced by adequate embedment of the sheet pile walls; stabilizing berms also increase the length of the drainage path to reduce this gradient.

(*e*) Bearing capacity failure. In very soft clays at the founding level of the

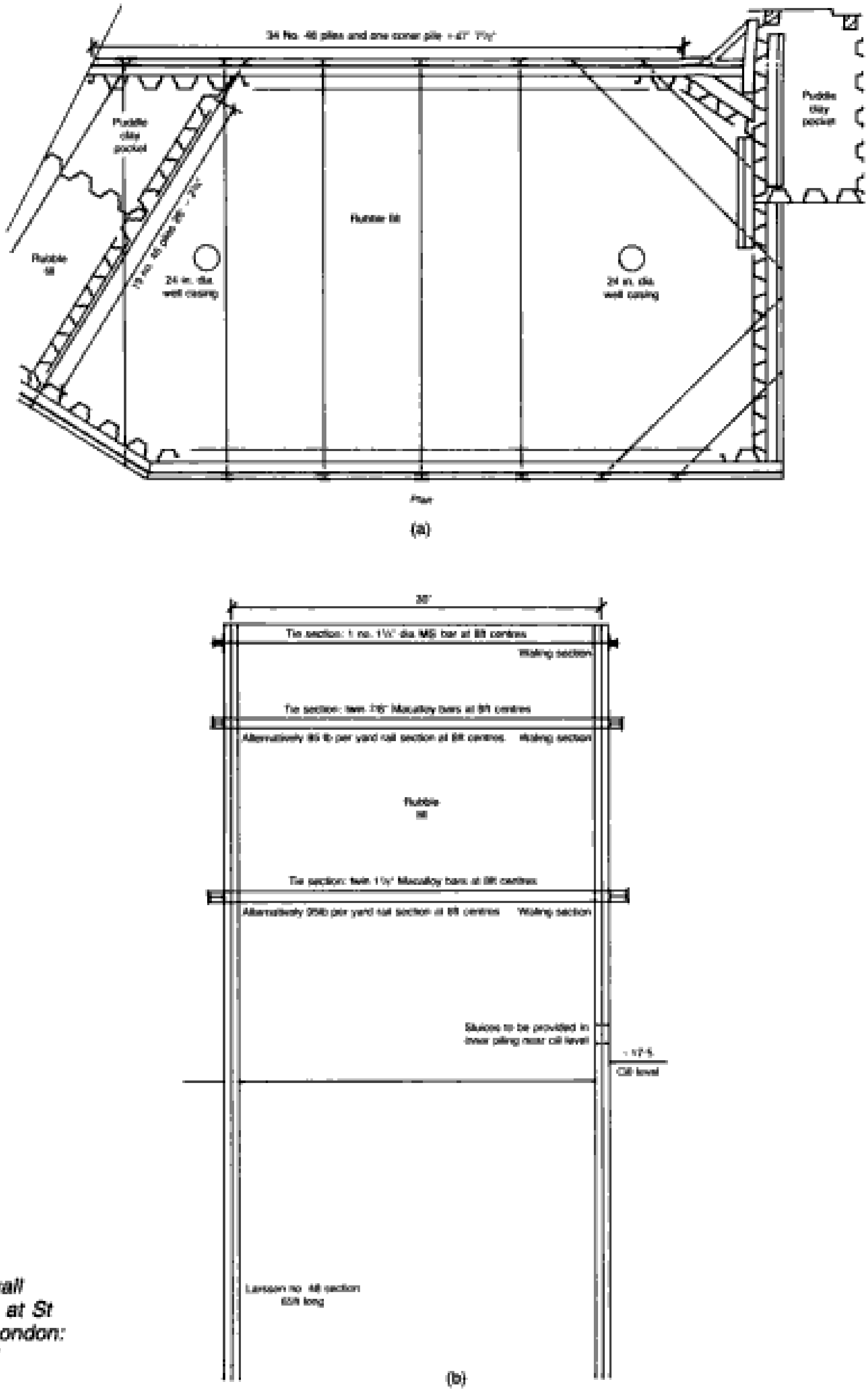


Fig. 6.32. Double wall entrance cofferdam at St Katherine's Lock, London: (a) plan; (b) vertical cross-section

cofferdam, the risk of bearing failure may be such to require removal of the clay between the piling to a level where better ground occurs.

(*f*) River bed scour. The risk of scour to the outer sheet piles in swiftly flowing rivers, or where flow is constricted, must be examined in the same way as for any other river cofferdam.

The construction of three double-wall cofferdams at Alton, Illinois, was described by White and Prentis.[16] At the site, 25 miles north of St Louis, the Mississippi River is about half a mile wide and almost 10 m deep, with a current of just over 2 m/s. Model tests were used to design streamlining lead-in piling to the cofferdams to reduce scour. The streamlining fin to cofferdam 1 is shown in Fig. 6.33. The 18·3 m long steel sheeting piles were pitched and driven from a barge to a timber framework suspended from wooden piles previously driven. Added protection was obtained by guying the streamlining fin to the cofferdam and by dumping rip-rap at its upstream end. The works were successful in preventing scouring and erosion of the cofferdams without causing harm; sand and silt were deposited along the entire length of the river leg. The cross-section of the double-wall cofferdam is shown in Fig. 6.34.

Cellular cofferdams

Cellular cofferdams are constructed of flat sheet pile sections with high interlock tensile strength. They offer the advantage that they can be designed as stable gravity structures even when embedment is difficult where rock occurs at founding level or subsoil prvents pile penetration.

In plan, cellular cofferdams are typically circular, diaphragm or clover leaf types. Circular types may also be joined together. These four plan forms are shown in Fig. 6.35.

Circular cells have the advantage that they can be individually built and filled; the smaller linking areas are built later. The width of circular structures increases in proportion to their height, and as the diameter increases the interlock tension also increases. When the diameter becomes excessive and the interlock tension exceeds permissible limits, flat cells may be used. Individual flat cells are not stable on their own, and unless special measures are taken, cofferdams of this type must

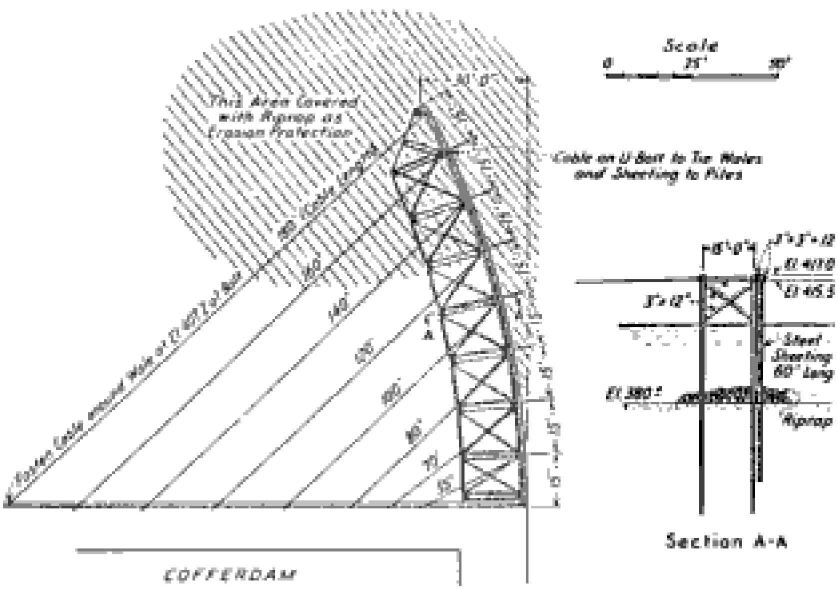


Fig. 6.33. Detail of streamlining fin, dam 26, Mississippi River, Alton, Illinois (White and Prentis[16])

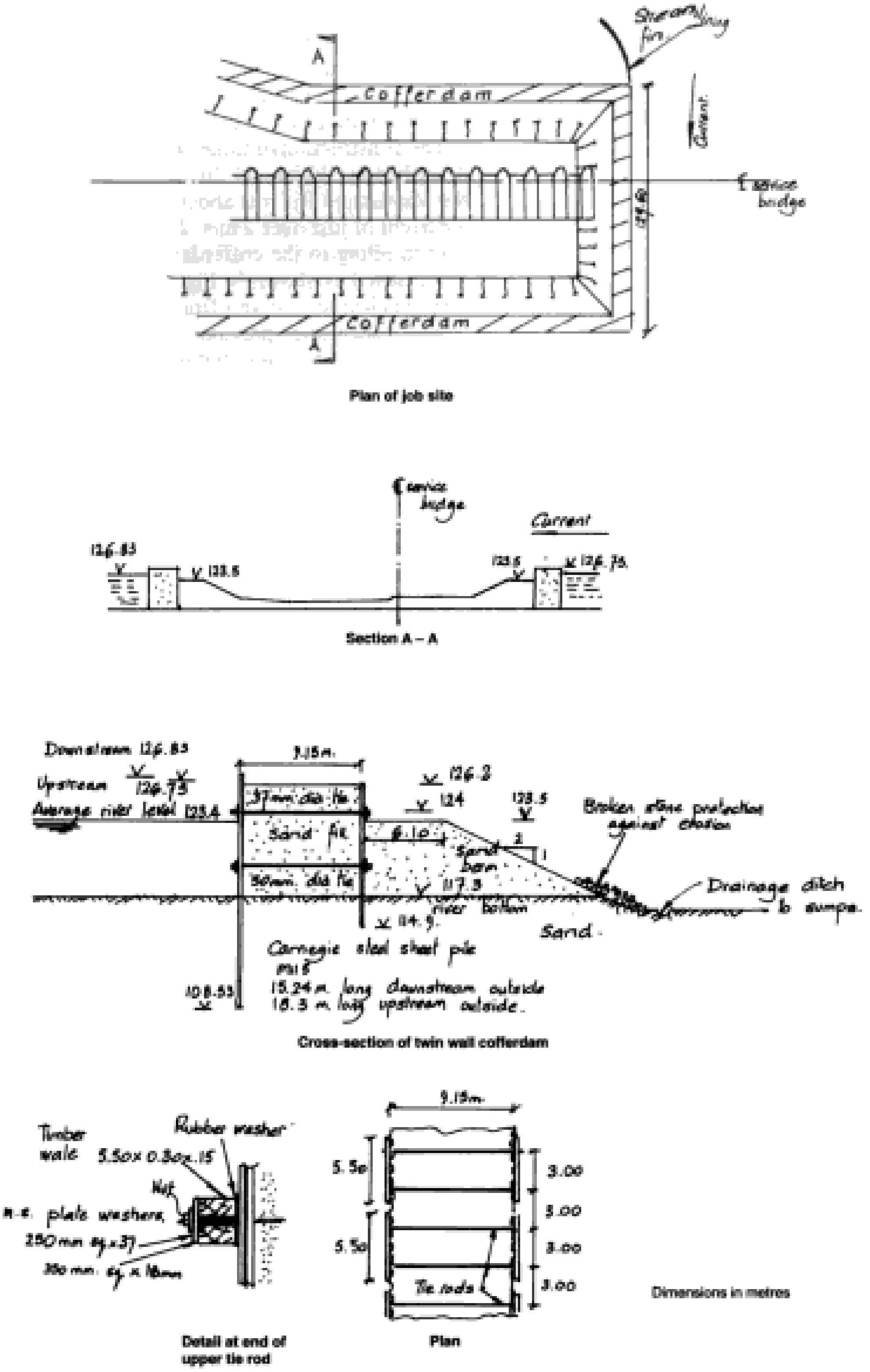


Fig. 6.34. Double wall cofferdam on the Mississippi River, Alton, Illinois: plan and cross-section (White and Prentis[16])

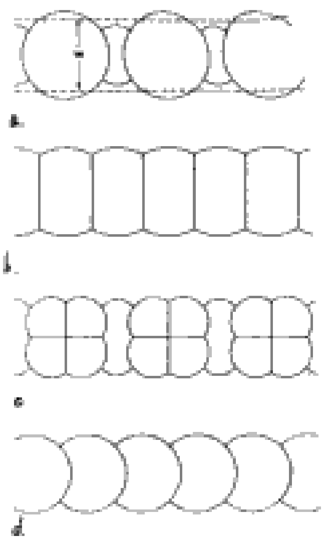


Fig. 6.35. Types of cellular cofferdam plan forms: (a) circular cells connected by arcs; (b) semi-circular cells with straight diaphragm cross walls; (c) clover leaf cells from four circular arcs of sheeting fixed on two transverse walls perpendicular to each other and connected by small arcs; (d) circular arcs joined to each other

be filled in stages. The use of intermediate strongpoints is desirable in long lengths of cellular cofferdams built with flat cells, particularly when there is risk of rupture or storm damage. Unless isolated strongpoints are provided, failure in an individual cell can lead to progressive collapse of several neighbouring cells.

Heights and loadings being equal, flat cell cofferdams require greater weight of steel per lineal metre than circular cofferdams. Comparisons based on total pile tonnages can mislead, however, as the total cost of cofferdam construction greatly depends on the cost of fill as placed. While flat cell construction may require larger pile tonnages than circular cells, flat cells are probably easier and quicker to build and use fewer expensive special connecting piles. The flat cell type also has the advantage that the effective width may be increased without increasing tension in the interlocks.

The earliest cellular cofferdam was built in the early 1900s at Black Rock Harbour, Buffalo, NY. Each of the 27 cells was just over 9 m square in plan, with a similar unsupported height. As all the walls were straight, large deformations were expected, but in one cell an excessive bulge of more than 1 m occurred between cross walls, although inward movement of the cofferdam crest did not exceed 25 mm. In 1915–16, the Black Rock Harbour cofferdam was repeated in a dam at Troy, NY. This time the straight outer walls were replaced with curved walls between the cross walls to form the flat cell type cofferdam.

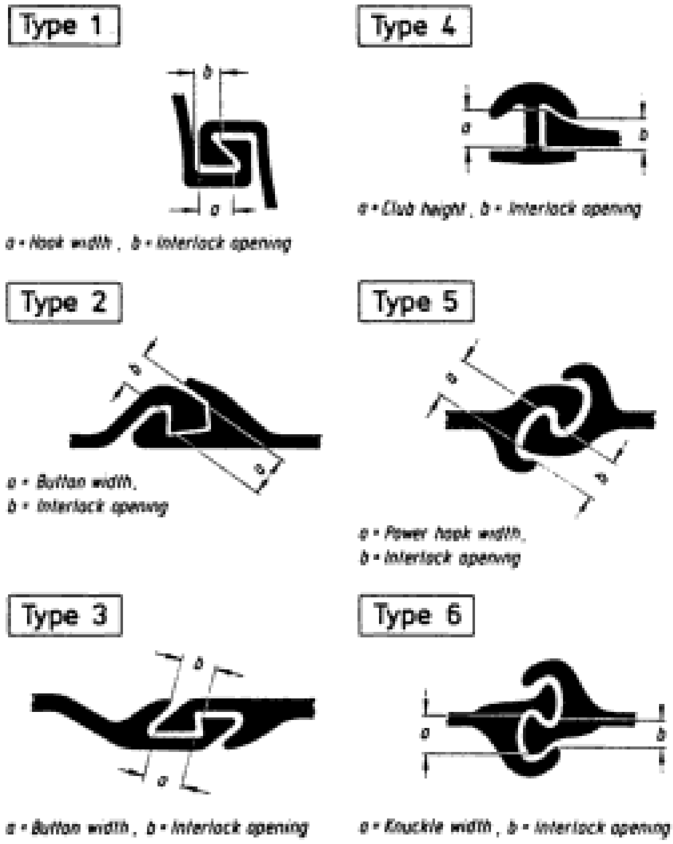
In 1910, a cellular cofferdam with 20 circular cells was used to raise the battleship Maine from the bottom of Havana Harbour in Cuba. This cofferdam, the first to use circular cells, was built on soft silts and mud overlying medium soft clay; the cell filling was clay. Inward deflections began to occur during pumping out of the cells and it was necessary to build a berm on the inside of the structure. During final stages of dewatering it became necessary to strut the cofferdam from the ship's hull. The cofferdam was, nevertheless, a success and led to the increased use of the cellular cofferdam and steel sheet piling.

Terzaghi[17] stated that the design of cellular cofferdams based on a foundation other than rock requires more judgement than the design of a double-wall cofferdam with a broad inner berm on a similar base. It may also be said that the construction requires more judgement and experience.

Cellular dams may fail in the same way as double wall types: by sliding or tilting, by failure of the base, by piping failure or by scour. In addition, they may fail by breakage of the interlocks and bursting of the cells. This latter cause, the vulnerability of the interlocks, is the major failure risk, and many of the failures by bursting which have occurred at the time of filling the cells (or immediately afterwards) have been blamed on driving out of lock. Considerable care and diligence must be applied in handling piles and in pitching and driving them to avoid ruptured interlocks.

Before any installation begins, pile dimensional tolerance, pile straightness, quality of fabrication of special piles, cleanliness of interlocks and any possible flaws in steel quality must be thoroughly checked. Careful inspection of welding quality in the fabrication of special piles is essential. These inspections should be made on second-hand piles in particular, but tolerance and quality inspection is always necessary, even on new deliveries. A surplus number of piles must be ordered as a contingency to allow for reject piles if the progress of the works is not to suffer.

One of the most important checks is the gauging of all interlocks. Typical permissible interlock tolerances are summarized in Fig. 6.36.[18] The need for interlocks as delivered to comply strictly with tolerances is self-evident when the large proportion of the mass of the pile section concentrated at the interlock is examined in cross-section. Fig. 6.37a shows sections of flat web sheet piles manufactured by Arbed with high-strength interlocks. The strict dimensional tolerance necessary to achieve three-point interlock contact is obvious. Fig. 6.37b shows alternative single and triple point interlocks previously available.

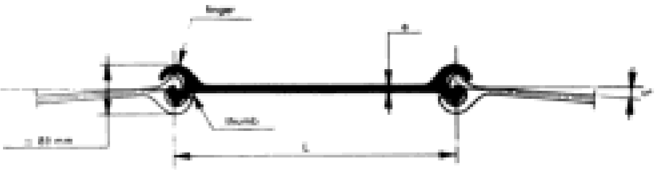


Type	Design dimensions (acc. to section drawings)	Tolerances of design dimensions		
		Designation	plus:mm	minus: mm
1	Hook width a	Δa	2.5	2.5
	Interlock opening b	Δb	2	2
2	Button width a	Δa	1	3
	Interlock opening b	Δb	3	1
3	Button height a	Δa	1.5–2.5*	3
	Interlock opening b	Δb	4	0.5
4	Club height a	Δa	1	3
	Interlock opening b	Δb	2	1
5	Power hook width a	Δa	1.5	3.5
	Interlock opening b	Δb	3	1.5
6	Knuckle width a	Δa	2	3
	Interlock width b	Δb	3	2

* Depending on the section

Fig. 6.36. Typical permitted interlock tolerances (German Waterfront Code[18])

The extent of pre-excavation before cofferdam construction begins depends largely on depth of site overburden. The objective of prior excavation is tow-fold: to remove shallow obstructions and to minimize the penetration of the driven pile. Where shallow overburden exists, a site strip 1 m deep will be sufficient to remove tree stumps and boulders. If the overburden is deep it may be advisable to take out 6 to 7 m to reduce driving depth. Where bedrock outcrops it is usual to place 2 m of free-draining fill to provide a toe for the piles during pitching and driving. If



Section	Nominal width L mm	Web thickness e mm	Deviation angle α°	Perimeter of a single pile cm	Steel section of a single pile cm²	Mass per m of a single pile kg/m	**Mass per m² of wall kg/m²**	Section modulus of a single pile cm³	Moment of inertia of a single pile cm⁴	Coating area* m²/m
AS500-12,0	500	12,0	6	138	92,1	72,3	**145**	47	180	1,14
AS500-12,5	500	12,5	6	138	94,8	74,4	**149**	47	180	1,14
AS500-12,7	500	12,7	6	138	95,8	75,2	**150**	47	180	1,14

Note: All the sections interlock together

* Excludes inside of interlocks.

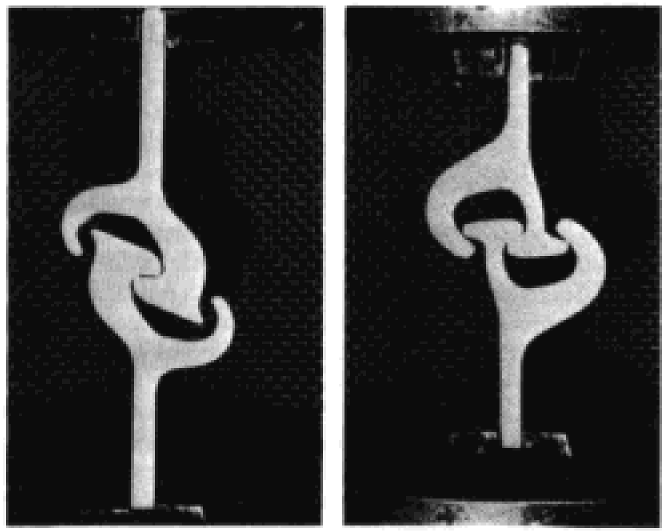
(a)

PBP one point contact

Strength of interlock
250 to 350 t/m
14,000 to 19,600 lb/in

ABP three point contact

Strength of interlock
350 to 500 t/m
19,600 to 28,000 lb/in



(b)

Fig. 6.27. Flat pile sections (a) modern sections with high strength interlocks, and (b) alternative sections previously available showing single point (left) and three-point contact (right) at interlock (courtesy of Arbed)

overburden consists of very soft silts or clays or soils containing many cobbles and boulders, it must be removed.

A template must be used to set the sheet piles to a cellular cofferdam. The templates are usually made with two rings and supported by at least four bearing piles. The template is usually made 200 mm smaller than the net driving line to allow the piles to rotate slightly to adjust to correct arc during pitching. Fig. 6.38 shows piles pitched to a template.

To pitch or set the sheet piles to the template it is usual to use the four special junction piles as 'key piles'. With large cells, and in strong current conditions, additional key piles may be needed to stiffen the sheeting during pitching and subsequent driving. These key piles consist of a straight web pile with a steel joist section welded to the inside of the web. The sheet piles are pitched working away from the key piles, blocking off alternate piles from the template. Guy lines to

Fig. 6.38. Cellular cofferdam construction, flat sheet piles pitched and driven to a template (courtesy of Arbed)

the tops of evenly-spaced piles control verticality, especially in windy conditions and strong currents. Sheeting cannot be accurately pitched in flows faster than 1·2 m/s unless current deflector bulkheads are used, cantilevered from the adjacent completed cell. Closures during pitching should be made mid-way between the key piles; sheeting is then picked up, several piles together, and 'shaken out', especially near the closures. The purpose of shaking out is to ensure that the interlocks run freely and allow some rotation of individual piles to give a smooth arc against the template.

Sheeters should be driven in pairs, ideally with a hammer of energy 1200 to 2000 m kg. Larger hammers should only be used carefully to avoid split interlocks, especially on long piles. Piles are best driven in increments, the maximum increment of one pile compared with its neighbour not exceeding 2 m. Jetting methods can be used to good effect in sands but should be used simultaneously both inside and outside the cell to maintain verticality. Vibratory hammers are efficient in granular soils.

Where struts have to be spliced because of their length, this is done by driving the bottom section to full penetration and burning a staggered splice line, the stagger being 1·5 to 2·0 m between adjacent sheeters. Driving flat web sheet piles longer than 15 m in one piece is difficult. Where piles are being driven to achieve a cut-off into a sloping rockhead, final penetration should be made on single piles to reduce the risk of 'windows' under the piles.

After all piles have been driven and the template removed, filling may begin: cell filling should be granular, free-draining material with a reasonable proportion of fines (say, a maximum of 15% passing a 100 sieve and a maximum of 5% passing the 200 size). In large cells, small boulders up to 300 mm would be acceptable. The filling, which in circular cells should be made from the centre of the cell to avoid deformations, may be placed hydraulically, from grabs, by conveyor or by end dumping from trucks. Flat cells are best filled by grab or dragline, and to avoid distortion the differential fill height between adjacent cells should not exceed 1·5 m.

If cellular cofferdams are likely to be over-topped by high tides or flood, the filling must be protected on top. A concrete cap 200 to 300 mm thick is often provided, also protecting the fill from occasional high waves.

It is essential to provide sufficient flood gates to allow drainage of the enclosed area in the event of flooding. After filling and during dewatering, weep holes should be burnt through the inner sheeting to allow efficient drainage of the fill: 25 mm diameter holes at 2 m centres in every fifth web regularly rodded will suffice. The rate of pumping out should be regulated by the rate of draining the cell fill; in large dams a maximum rate of 1·5 m per 24 hours should apply.

Cell deformation will have occurred during cell filling operations, the cells barrelling at a distance of two-thirds of the cell height from the crest as the slack in the interlocks is taken up. As the cofferdam is pumped dry there is further settlement of the cell fill, and with further flooding and dewatering the total settlement in high cells may reach 150 mm. Horizontal deformation of up to 1% of the unsupported height can be expected at the cofferdam crest after dewatering.

Where a number of circular cells on a curved plan form span wide openings, the end cells transfer horizontal thrust to an existing wall or natural abutment. Figure 6.39 shows the plan and section of the circular-cell cofferdam used on the Bangor–Brewer Bridge, Maine, USA. Horizontal bracing was used to strut the tops of four key cells in the structure.

Gravity type cofferdams

The earliest, and simplest form of cofferdam over water was the earth or rock dam, built to isolate a construction area which can then be pumped dry for works and bridge foundations. The design of such cofferdams must take into account the slope stability of the embankment, allowing for seepage forces, the need for

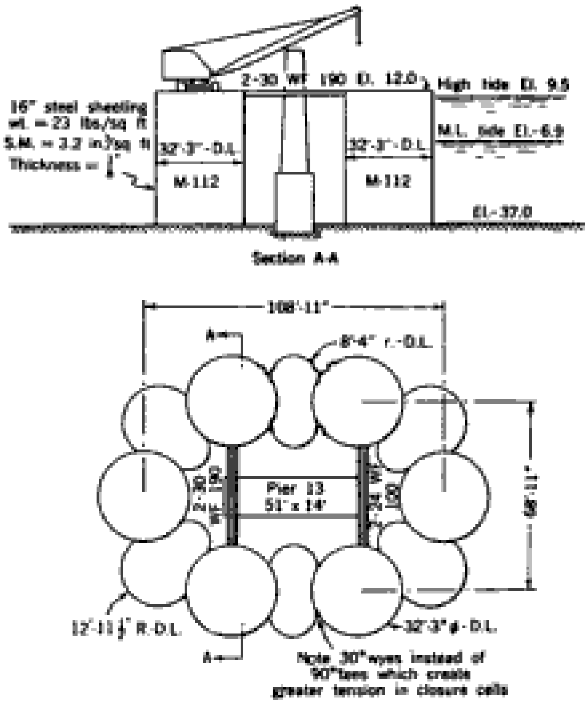


Fig. 6.39. Circular cell cofferdam complex with horizontal bracing, Bangor–Brewer Bridge, Maine, USA (White[19])

scour protection to the river bed and the embankment, and sufficient crest height against over-topping by waves or floods.

The economics of embankment cofferdams obviously depend on the availability of soil and rock for filling and slope protection. In addition, however, pumping costs to keep the enclosure dewatered will depend on the type of soil fill and the permeability of the soil under the embankment. If pumping costs are excessive it may prove economical to install a cut-off through the embankment and into the underlying soil or rock to reduce seepage. Relatively shallow cut-offs may be constructed from sheet piling; deeper walls may be built economically using the slurry trench technique.

During the 1930s several large sand embankment cofferdams were built across the River Nile to allow construction of barrages for irrigation purposes. A major scheme was described by Lee[20] for remodelling works on the Assuit Barrage. The works involved new masonry, sluice gates, lift bridges and improved apron slabs to the original barrage at a site where the Nile is over 800 m wide. Fig. 6.40 shows typical sections through the upstream and downstream embankments, known as sudds. The cut-offs were made from Larssen No. 2 sheet piles driven through the pumped sand banks. The sand, impregnated with silt on the upstream face outside the piling, was graded to a batter of 1:3; on the downstream side, where the sand was coarser and cleaner, the gradients were 1:7 or steeper. Dewatering was carried out slowly to allow the sudds to drain gradually, after which there was little seepage through the sudds. The work was carried out in four low water seasons in successive years, working progressively across the Nile. At the end of each season, the sudds were slowly rewatered and the sheet piles extracted. The greater part of the sand fill to the sudds was removed by the scour of the high river waters.

At sites where timber is cheap, timber cribs can be used to form a gravity cofferdam. The ribs, sometimes shaped to the river bed profile and with pockets up to 3·5 m square, are launched and floated into position and filled with rock and finer material to reduce permeability.

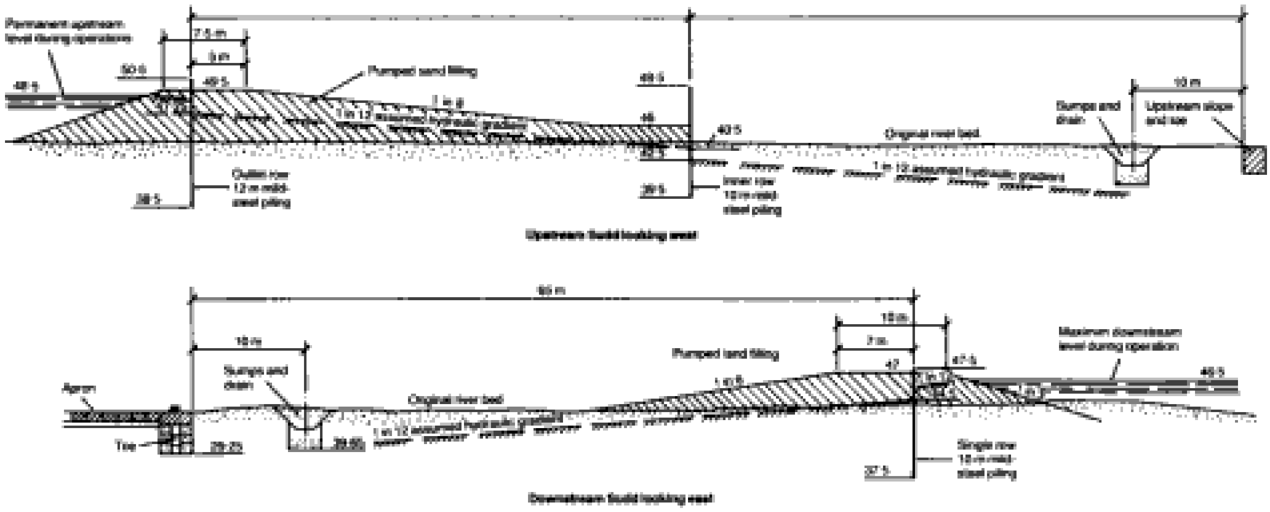


Fig. 6.40. Typical sections, upstream and downstream of sudds for reconstruction of Assuit Barrage, River Nile (Lee[20])

Concrete blocks can also be used to form a gravity dam structure to exclude river water from lock and dock constructions. Gabions filled with rock and smaller material to reduce leakage can be used for the same purpose.

One of the most dramatic sequences of work occurs on river or marine construction during closure of a cofferdam to block all flow. This situation occurs typically where a barrage or dam is to be constructed within a cofferdam across a flowing river. Often the dam is built in several stages and as the river flow becomes progressively restricted scour of the river bed and of the exposed extremes of the cofferdam structure increases rapidly. In the final closure work, time is absolutely critical. White[19] reported some of the measures used in such closures.

(*a*) On rivers with sandy and silty beds, large dredgers have been used to replace material scoured away.
(*b*) On swifter flowing rivers with gravelly bottoms, rocks have been dumped from trestle bridges to keep the trestle anchored and stop stream flow. This done, sand and clay were placed against the upstream side of the rock fill to improve watertightness.
(*c*) Where the depth of the river has been so great as to prohibit access trestles, cableways have been used to drop heavy rocks or concrete blocks into the river.

The second-stage closure of the cofferdam for the Chief Joseph Dam on the Columbia River was only achieved by retention of large rocks, each weighing from 10 to 30 tonnes, by individual cables held in place by a large cable spanning upstream of the 15 m closure gap. River flow at the time of closure exceeded 7 m/s.

Mackintosh[21] referred to other closure works for dams on the Columbia River. At the McNary Dam the original 800 m river width had been progressively narrowed to a gap just over 70 m wide between tow steel cells. The river level had to be raised 5 m in order to pass through the spillway blocks, and this caused a maximum water velocity of more than 9 m/s. The closure was made over a period of 37 days by dropping a total of 2088 concrete tetrahedra, each weighing 12 tonnes, into the gap from a cableway.

Mackintosh also gave details of the closure of the Dalles Dam, where dumped rockfill, incorporated into a permanent embankment, was used to close a 150 m wide channel. At the deepest point the river was 55 m below low water and in the first stage the gap was reduced to just over 16 m by dumping rock from barges.

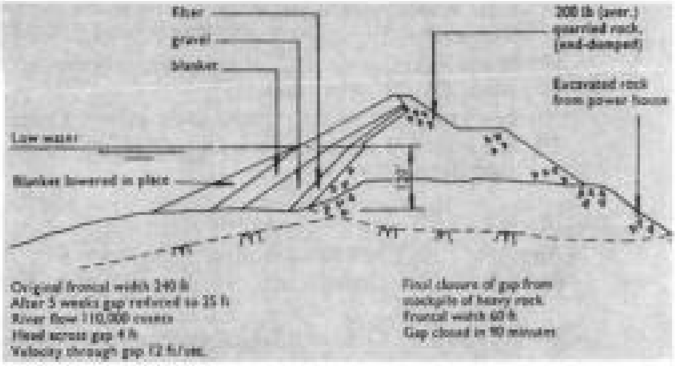


Fig. 6.41. Cross-section of Dalles Dam, Columbia River, USA (Mackintosh[21])

To begin the final closure operation, end dumping of rockfill was carried out on a 70 m width, working outwards from the right-hand river bank until the channel was only 7·6 m wide. This last operation took five weeks in a river flow of 3120 cumecs and a velocity through the final channel of 3·7 m/s. Rocks ranged from 100 to 259 kg. Rock filling was followed by end dumping smaller rock and filter and then a sand blanket was placed, partly from barges, to form the section shown in Fig. 6.41. The final closure operation concentrating all resources on a stockpile of heavy rock, was achieved on a width of 18 m in a period of only 1·5 hours.

A spectacular closure on the River Saguency in Canada should also be mentioned. A vast, 28 m high concrete monolith was topped by blasting into the river. The base of this mass was pre-shaped to conform with river bed soundings and weighed 11 000 tonnes. The river flow was more than 9 m/s, but the monolith was accurately placed and successful in its purpose.

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7 Cofferdam design

Design of braced sheeted cofferdams

In plan the design size must allow for installation tolerances of sheeting or walling and sufficient space to accommodate supporting members such as walings and anchor heads to the sheeting or walling. The design of member sizes will primarily address ultimate limit state conditions, but the serviceability limit state of deformation must be applied to ensure that deformations of the cofferdam members, particularly the sheeting, are not excessive and thus impair construction of the permanent works to the required dimensions and thicknesses.

The design life of the cofferdam will inevitably influence the type of materials used and their durability, but temporary works of very short duration must not be built with inferior materials which are unable to support the construction loads and the earth and water pressures imposed upon them. The sufficiency of second-hand construction materials for cofferdam works is particularly important in this regard.

The cost of the cofferdam must be minimized irrespective of the cost allowance previously made at tender or job negotiation stage. This cost, however, must take into account the efficiency (and cost effectiveness) of construction works within the cofferdam. For example, cost-effectiveness may be optimized by spending more on an anchored cofferdam than a braced alternative, obtaining permanent works construction unimpaired by bracing and the need to re-prop bracing. The construction cost of the cofferdam must include the costs of its removal; extraction and re-use of the materials elsewhere will be influenced by the design. The incorporation of the temporary works into the permanent works should always be considered even if in contractual arrangements it is not particularly conducive to do so. The obvious example of the application of such a strategy is the use of structural diaphragm walls to provide both temporary soil support during construction and thereafter to form the permanent substructure walls. These arrangements rarely cause the practical difficulties and disputes between owners, engineers and contractors that lawyers predict.

Site investigation

The frequent inadequacy of site investigations for heavy civil engineering works has been detailed elsewhere,[1] but this shortfall of data for permanent works design often follows the discovery by the temporary works designer of an even greater shortfall in the availability of soils, groundwater and tidal information. The site investigation, conducted to a national code such as BS 5930[2] or DIN 4020,[3] requires an initial definition of scope to include number, depth and type of borings and in situ tests, and a laboratory testing programme which satisfies the needs of both temporary and permanent works designers. A CIRIA report on the design and construction of sheet piled cofferdams[4] recommends the following checklist for information required from the investigation.

- location of boreholes with respect to the cofferdam
- date of boring and ground level at boring based on same datum reference as construction drawings

- diameter of boring, drill rig used and whether water was added to the boring
- depth and thickness of all soil strata to at least two times the proposed excavation depth, possibly more in weak soils
- bulk unit weights for each soil type
- standard penetration tests and other relevant in situ tests, such as cone penetrometer tests
- grading curves, Atterberg limits for clay, undrained and, where appropriate, drained shear tests
- vane test results for soft clays, with sensitivity values
- groundwater strike, rate of ingress, standing levels, any permeability test data, casing depth at all stages of boring
- position and detail of piezometers, type and readings
- tidal variations and lag
- site geology
- previous site history
- report interpretation.

The design life of the cofferdam works will also influence the designer's choice of soil strength parameters, based on undrained or drained effective stress values for clay. The prime matters affecting the period of full drainage and effective stress analysis are the drainage path length to permeable strata and the permeability of the consolidating stratum and the permeable drainage strata. Where, for example, an otherwise relatively impermeable clay structure in the active or passive zones is interlayered with permeable sandy beds or laminations, the rate of drainage will increase and pore pressures will return to hydrostatic pressures more rapidly, possibly within the design life of the cofferdam works. Application of total stress, undrained analysis is therefore valid only for a very short period after the application of load to the cofferdam sheeting and support, but the period for full dissipation of excess pore pressure may vary from days to months. In these circumstances the designer's best option will probably lie in the use of undrained analysis in homogeneous clays, particularly soft clays, using effective stress parameters as a check. The use of effective stress analyses without a total stress analysis would then be reserved for areas of good soil drainage; laminated soils or strata of shallow depth, and analyses made at the end of a long construction period of or for a later permanent works phase.

Due to the relative cost of drained triaxial tests and their duration it is likely that the number of available test results for a particular clay stratum will be limited and the designer may have little confidence in their statistical average, especially for soils where previous results are not available. This is less likely to be the case for undrained triaxial test results. In this situation the designer may prefer to keep to undrained total stress analyses, risking the use of inappropriate earth and water pressures for the design of cofferdam sheeting sections and supports.

Design parameters for soil

Earth pressure calculations, as detailed in chapter 5, rely on parameters for soil density and soil strength in terms of cohesion and the angle of shearing resistance ϕ. Preliminary designs may be made using empirical values for a known soil, and some assistance is given in selecting these in reference 5. Calculation values are advised in Table 7.1 The angle of shearing resistance in effective stress can be estimated from the clay plasticity index, which was shown in Table 5.1. Note that clays with laminations or seams of silts or sands show lower plasticity values than clay alone, and care must be exercised in tests in such clays. If in doubt, the higher plasticity of those given by testing and from Table 5.1 should be used.

For preliminary calculations, ϕ' for sands and gravels can be estimated from values of standard penetration tests, as shown in Fig. 5.2. Similarly, values of

Table 7.1 Preliminary design soil parameters (reference 5)

Type of soil	Density: Above water, calγ (kN/m^3)	Density: Submerged, calγ' (kN/m^3)	Final strength: Angle of internal friction, calϕ' (degree)	Final strength: Cohesion calc' (kN/m^2)	Initial strength* cohesion of undrained soil, calc_u (kN/m^2)	Modulus of volume change, calE_s (MN/m^2)
Non-cohesive soils						
Sand, loose, round	18	10	30	—	—	20–50
Sand, loose, angular	18	10	32·5	—	—	40–80
Sand, medium dense, round	19	11	32·5	—	—	50–100
Sand, medium dense, angular	19	11	35	—	—	80–150
Gravel without sand	16	10	37·5	—	—	100–200
Coarse gravel, sharp edged	18	11	40	—	—	150–300
Sand, dense, angular	19	11	37·5	—	—	150–250
Cohesive soils	(Empirical values for undisturbed samples from the North German Area)					
Clay, semi-firm	19	9	25	25	50–100	5–10
clay, difficult to knead stiff	18	8	20	20	25–50	2·5–5
Clay, easy to knead, soft	17	7	17·5	10	10–25	1–2·5
Boulder clay, solid	22	12	30	25	200–700	30–100
Loam, semi-firm	21	11	27·5	10	50–100	5–20
Loam, soft	10	9	27·5	—	10–25	4–8
Silt	18	8	27·5	—	10–50	3–10
Soft, org. slightly clayey sea silt	17	7	20	10	10–25	2–5
Soft, very org. strongly clayey sea silt	14	4	15	15	10–20	0·5–3
Peat	11	1	15	5	—	0·4–1
Peat under moderate initial loading	13	3	15	10	—	0·8–2

cal ϕ' = calculation value of the angle of internal friction in cohesive and non-cohesive soils
cal c' = calculation value of the cohesion, corresponding to cal ϕ'
cal c_u = calculation value of the shear strength from undrained tests in saturated cohesive soils
* Appertaining angle of internal friction is to be assumed at cal ϕ_u'

undrained cohesive strength c_u can be estimated from standard penetration tests on over-consolidated clays from the empirical expression $c_u = nf_s$ where n is the test value and f_s is a coefficient which varies with the plasticity index.[7] Values of c_u as a function of effective overburden pressure P_n (for normally-consolidated clays) were given by Skempton:[8]

$$c_u/P_n = 0{\cdot}11 + 0{\cdot}0037 \times \text{plasticity index} \tag{63}$$

Kenney[9] gave preliminary values for the relationship between ϕ' and plasticity index for normally-consolidated clays based on observations of more than 60 soils (Fig. 7.1).

The value of ϕ' for weak rocks may be estimated for preliminary design purposes from a rock description given in BS 8002[10] (Table 5.2). These indicative values are considered conservative, being based on granular fragments rather than intact rock, taking into account closely-jointed rock with a very low value of rock quality designation (RQD).

Design of soil support and structural members to the cofferdam

The use of limit equilibrium methods to design cantilever and propped walls

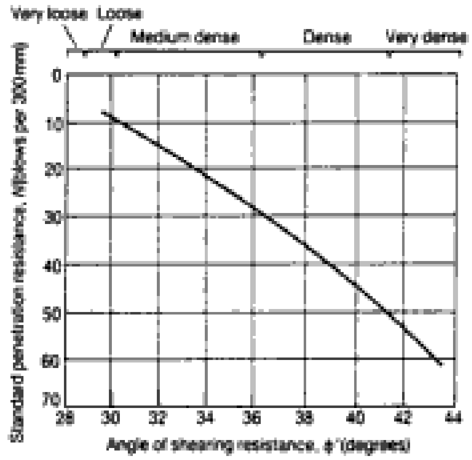


Fig. 7.1. Relationship between plasticity index and angle of shearing resistance (Kenney[9])

for overall stability was described in chapter 5. A factor of safety is applied by increasing embedment, decreasing soil strength or factoring moments of pressure diagrams, to achieve walls of sufficient depth to avoid overturning. The earth pressures on which these analyses have been based are Coulomb values of limit pressures, that is, they are maximum passive pressures and minimum active pressures which are generated by wall movement at failure.

These limit pressures occur, therefore, at the ultimate limit state of wall collapse by overturning. The applied factor of safety, which increases wall embedment, takes into account that in the serviceability condition, when wall/soil deformation is less, the soil passive resistance will be less than that generated in the ultimate limit state and the soil active pressure will be greater than at the failure. Recommended factors of safety for cantilever and propped wall with free earth support are:[4]

(*a*) Effective stress analysis
- (i) Factor on strength
  - $\phi' \leq 30°$ $\quad F_s = 1\cdot2$
  - $\phi' > 30°$ $\quad F_s = 1\cdot2{-}1\cdot1$
- (ii) Factor on moments
  - Gross pressures (CP2)
  - $\phi' \leq 20°$ $\quad F_p = 1\cdot2$
  - $\phi' = 20{-}30°$ $\quad F_p = 1\cdot2-1\cdot5$
  - $\phi' \geq 30°$ $\quad F_p = 1\cdot5$
  - Net pressures (British Steel)
  - ϕ' all values $\quad (F_{np}) = 2\cdot0$
  - Burland-Potts
  - ϕ' all values $\quad I_r) = I\cdot5$

(*b*) Total stress analysis
- Factor on strength $\quad (F_s) = 1\cdot5$
- Gross pressures $\quad (F_p) = 2\cdot0$
- Net pressures $\quad$ Not recommended
- Burland-Potts $\quad (F_r) = 2\cdot0$

For propped walls with fixed earth support the penetration will always be greater than that required for free earth support, and will give an adequate factor of safety against rotation about the prop. If a simplified method is used for the calculation of penetration, 20% extra penetration is used not as a factor of safety but the additional depth required to correct for the simplification used in the method.

In geotechnical analysis of overall stability of cantilever and propped walls it is customary to apply best estimated values of applied loads, unit weights of soils and water pressure in the calculation of limit pressures. Although contrary to the application of limit state design in its entirety, it is not recommended that load factors should be applied to the various loads on the retaining structure to produce factored bending moments for the wall. As previously stated, unfactored loads with application of increased embedment, factored soil strength or factored moments of the earth diagram should be used according to choice.

The wall, dimensioned according to one of these methods, with an applied factor of safety, will be checked for sliding, basal failure and hydraulic failure, the factor of safety for each being calculated using limit pressures where earth pressure calculation is required.

Next is the calculation of the sizes of structural members. Two methods of design are available for the design of the sheeting and its support by walings, struts and anchors: these are the permissible stress method and the limit state method.

In the permissible stress method, best estimates of load are used. Limit pressure diagrams (or strut load envelopes based on the total pressure from limit earth pressure diagrams) are used to calculate bending moments and shears in the wall/sheeting and in the walings, and thrusts or tensions within the struts or anchors. Permissible stresses based on ultimate stresses reduced by a factor of safety are then calculated from the moments, shears, thrusts and tensions to define the size of these structural members. Permissible stress design is therefore based on limit pressures which do not occur in the serviceability state and, further, the ultimate limit state failure conditions related to these limit pressures, which are based on the monolithic failure of earth masses, may be unrelated to the ultimate limit state conditions (and therefore the earth pressures at these conditions) of collapse of the structural components.

The limit state method uses conventional methods of design for reinforced concrete, steelwork and timber structural members in which the application of factored loads produces factored bending moments, shears, thrusts and tensions, which are then used with the ultimate or characteristic strength of the material to define its size. This method is relied upon in the current draft of Eurocode 7 and although the logic of this method is flawed when using limit pressures, it appears to be less flawed than using permissible stress design with such pressures. For this reason, and because of the trend towards greater use in structural design of limit state design, its application is adopted here. It is recommended, however, that instead of applying load factors to the soil, water and surcharge loads on the wall, a factored bending moment is more appropriate, being less complicated to analyse and simplifying the geometry of the bending moment diagram. Recommendations regarding this factored bending moment (and shears, thrusts and tensions) are as follows.

(*a*) apply a factor of safety of 1·4 to factor the moments, shears, thrusts and tensions as calculated from the wall analysed by: increasing embedment, factoring moments on the gross pressure diagram, factoring moments of net pressure, or factoring moments of net passive resistance (after Burland and Potts[11]).

(*b*) use moments, shear, thrusts and tensions as calculated from the wall analysed by reducing shear strength of the soil on the active and passive sides or the passive side only; treat the values obtained directly as ultimate limit state values.

Serviceability limit state conditions of wall deformation and cracking and of soil deformation, heave and settlement, cannot be examined by the preceding limit pressure methods. Analysis using Winkler spring theory, as discussed in chapter 5, produces a deflected wall shape with springs to model soil and strut stiffness and applied loads to model anchor tensions. A typical computer program, Lawall,[12] begins with limit pressures and with successive iterations allows the wall to deform in order that active and passive pressures and struts or anchor forces balance, allowing that portion of the limit pressures on each side of the wall to balance. If a full interactive numerical soil–structure analysis were carried out the deformed soil structure would be computed in addition to the deformed shape of the wall and the stresses through the soil structure. The stresses in the active and passive zones of the wall are no longer related to limit pressures on the wall but relate to a serviceability condition. A factor of the order of 1·4 should be applied to wall moments and shears, and up to 2·0 to strut forces and anchor tensions to approximate ultimate limit state conditions, although it will be appreciated that changes in soil deformation at failure would, in turn, alter applied pressure to the wall nearer to limit values, possibly reaching yield values over at least part of the wall.

Material stresses

A summary of the properties of materials used in cofferdam construction was given in Table 5.7.

Overall stability

As has been mentioned, the overall instability of a cofferdam construction should always be checked. Such risk occurs in sloping ground or, typically, in a riverfront cofferdam where the rear wall of the cofferdam supports retained ground and the front face supports tidal river conditions. Where the difference in height is small it may be possible to transfer load from the higher to the lower side by keeping the top frame as low as possible. Where the differential height is greater it maybe expedient to rake the top frame from one side to the other or, alternatively, anchor the sheeting at the higher side (Fig. 7.2).

Where cohesive soils extend to considerable depths on sloping sites, check that a deep-seated potential slip surface is not a failure risk. Where such a failure risk exists stability can be increased by driving the sheeting to greater depth to intercept the potential failure surface; jet-grouted columns may be installed below cofferdam formation level before excavation to achieve the same objective.

Bottom failure by piping and basal heave

Repeating earlier advice, the risk of hydraulic failure by piping should be checked for narrow cofferdams that do not achieve a cut-off in cohesionless soils with a high external water table. Risk of basal failure in cofferdams in soft clay should also be checked.

Aggressive site conditions: marine and river cofferdams

The extreme exposures to which cofferdams in both river and sea waters are subjected vary according to geographical location and the size and depth of the cofferdam. Matters to be assessed include the effect of wave forces on the face of the structure, over-topping of the cofferdam walls, scour, protection from vessel impact, and the impact and pressure of ice on the face of the cofferdam.

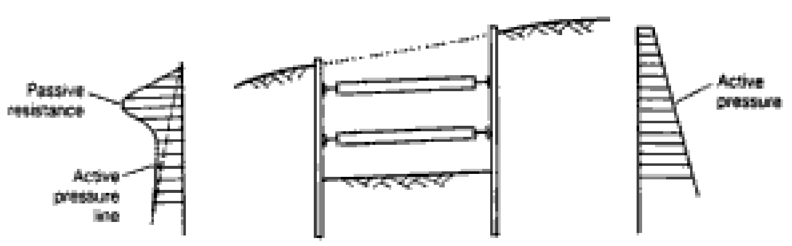


Fig. 7.2. Cofferdam construction in sloping ground (CIRIA[4])

The effects of wave action on the face of the cofferdam are discussed in BS 6349[13] and in reference 5. These loads on cofferdams are due to waves in deep water, where waves are reflected, and those in shallower water where waves break on the structure or some distance from it. Sainflou[14] and Minikin[15] determined the forces due to reflected and breaking waves, respectively, referring, however, to waves on backfilled waterfront structures with standing water within the backfill. These methods should be used cautiously when determining forces on sheeted structures. The very high loads that can be imposed on structures due to breaking waves, impact pressures of 10 000 kN/m^2 or more, identify the critical nature of this loading, especially since there is neither a reliable method of calculation nor an empirical solution for determining such loads. BS 6349 refers to breaking wave forces calculated by Minikin's method as high as 18 times those calculated for non-breaking waves. Measurements from physical models may assist where the scale of the cofferdam works justify such a study.

Waves caused by ship movement may also require consideration. In restricted waters the action of a headwater wave caused by water displacement in front of a vessel may require assessment in determining the maximum head of water on the face of a cofferdam structure; conversely, water drop occurring to the stern of a vessel may also cause variation in loading along the length of a long cofferdam. These matters, together with the action of bow and stern waves, are discussed in reference 5.

Reference should be made to the effect of pressure transmitted through coarse graded beach deposits to the walls of cofferdams sited between high and low water, and to the dynamic effect on the flow of water below wave-facing cofferdam walls where a cut-off is not obtained by the toe of the walls. In neither case is there guidance from published work and only caution can be advised in the selection of factors of safety on strut or anchor design and on the risk of hydraulic failure at the base of the cofferdam excavation.

The risk of over-topping of the cofferdam sheeting depends on the freeboard allowed to assessments of maximum tide heights, surge and wave heights from natural and man-made causes. It is essential that where there is any risk of the cofferdam flooding from over-topping, the cofferdam sheeting should be tied to prevent it bursting outwards under the action of a water-filled cofferdam. Adequate sluices with safe locations for operating controls are essential to avoid the risk of collapse of a water-filled cofferdam on an outgoing tide.

Scour protection will be needed where cofferdams are sited in fast flowing rivers or tidal conditions. Large cofferdams for bridge piers in fast flowing rivers may benefit from tests using physical models to determine the extent of scour and any increased risk due to obstruction of the flow by adjacent cofferdams. The upstream and downstream ends of a cofferdam may be shaped to reduce scour by providing cutwaters. Where protection is necessary to avoid erosion of the river or sea bed, rock or concrete blocks may be suitable. Grout mattresses, weighted by rockfill, may be adequate for river cofferdams. Where the scour risk is lower it may be sufficient to design the cofferdam sheeting embedment and strutting to ensure that collapse does not follow scour action.

In busy navigable waterways it will be necessary to protect the cofferdam by fendering or strongpoints built into the cofferdam sheeting to avoid damage by vessels. Barge traffic, in particular, appears to cause high collision risk. River authorities frequently regulate the extent of fendering required for river cofferdam works and specify the signage needed.

General layout of the cofferdam

The plan shape of the cofferdam will conform approximately to the plan shape of the permanent works to be built within it. In general terms, the cost of cofferdam construction of constant depth is directly proportional to its plan area. Any wastage of space in the choice of cofferdam plan shape therefore does not affect cost. The design of circular cofferdams is reviewed later, but in terms of relative economy only square-shaped plan structures or those with circular plan shape storage wells, such as pumping stations, can be accommodated in circular cofferdams cost-effectively.

For rectilinear cofferdam plan shapes the most economical arrangement uses maximum straight runs of piling with the minimum of return angles. The length of strutting across the cofferdam will define the need to support struts by puncheons and prop them laterally to reduce effective lengths of strutting in terms of buckling. Maximum strut length in steelwork is likely to be of the order of 40 m, and distances in excess of this may need to rely on raking shores from a completed central raft section or support from ground or rock anchors. Diagonal corner braces should always be used in frames unless this hinders the permanent construction. Some typical arrangements of framing in plan are shown in Fig. 7.3. The vertical location

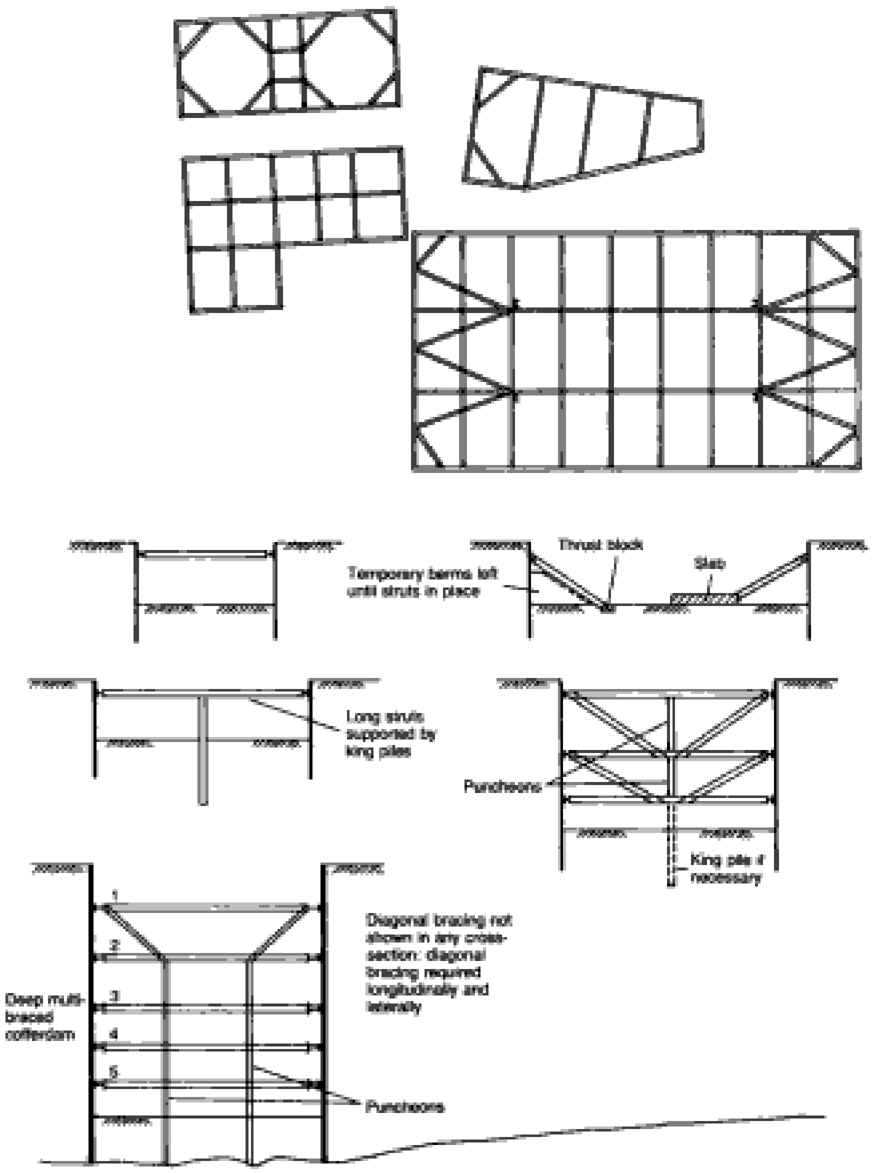


Fig. 7.3. Typical cofferdam framing in plan and in vertical cross-section (CIRIA[4])

of frames must be such that the maximum bending stress induced in the sheeting between frames is approximately constant. Overstressing of the sheeting must be avoided, particularly below the lowest frame, which should be located as low as possible to avoid risk of passive failure in the soil. The location of the penultimate frame is also important: the vertical height between final formation level and the penultimate frame should not be excessive in order that sheeting stresses and passive soil stresses are not exceeded immediately prior to placing the bottom frame.

Where soil strengths are low, near or below formation level, and earth and water pressure loading on the sheeting is relatively high, it may be necessary to avoid excessive sheeting stresses and risk of passive failure of the soil below formation level by placing the lowest frame underwater by flooding the cofferdam and using divers. The frame levels should generally be chosen to allow concrete lifts to be poured economically. Care may be needed to avoid starter or splice steel reinforcement from one pour being obstructed by the waling of a cofferdam frame. This applies particularly to the lowest frame impeding vertical starter steel in the kicker from the base slab. It may be possible to locate this front and rear starter steel to the wall construction on either side of the temporary waling, or couplers may be needed to extend the steel using the shortened height of the starter steel.

It is essential that load is transferred efficiently from soil to sheeting to waling to strutting, without any doubt as to the direction of the transfer. The principal reasons for failure of sheeted cofferdams are poor workmanship in connections causing insecure transfer of load, inadequate strut sections, inadequate embedment of the sheeting and overload due to inadequate allowance for surcharge loading. These modes of failure should never be forgotten.

Use and design of ground anchors

The economical use of anchors in land cofferdams depends on the strength of the subsoils in which the anchor is to be founded. Dense sands and gravels would be preferred to cohesive subsoils at a similar depth.

The opportunity to use anchors will depend on the ownership of land at the periphery of the cofferdams and on permission to found anchors in neighbouring land. The presence of existing substructures or basements may obstruct anchor installation. Where internal angles occur in the line of sheeting, anchors from adjacent walls have to be carefully located to avoid anchors obstructing each other (Fig. 7.4).

Earth pressures acting on an anchor installation in a temporary cofferdam depend not only on soil strengths but on wall and soil stiffnesses, anchor spacing, anchor yield, the prestress locked into the anchors as installed and loss of prestress with time. The anchored wall may be designed with active and passive earth pressure diagrams using limiting pressures, as described in chapter 5 for multi-propped walls. The loads in the anchors can be obtained from the empirical trapezoidal strut load envelope diagrams. Alternatively, methods based on Winkler spring analysis may be used with personal computer programs in which earth pressures are computed from trial anchor loads input to produce deformed wall profiles, the anchor loads being varied in successive runs to produce acceptable maximum wall deflexions.

Temporary anchors used in cofferdam construction have stressing tendons to transfer load from the fixed anchorage on the face of the cofferdam wall to a fixed anchorage within the subsoil outside the potential failure wedge at the rear of the wall. Fig. 7.5 shows typical details and geometry. In cohesionless soils the fixed anchorage is formed by pressure-grouted techniques, while in cohesive soils, tremie methods of pouring grout may also be used. BS 8081[16] provides a comprehensive review of the design and construction of both temporary and permanent anchors in soil and rock (it also has an extensive list of references). It depicts five anchor types.

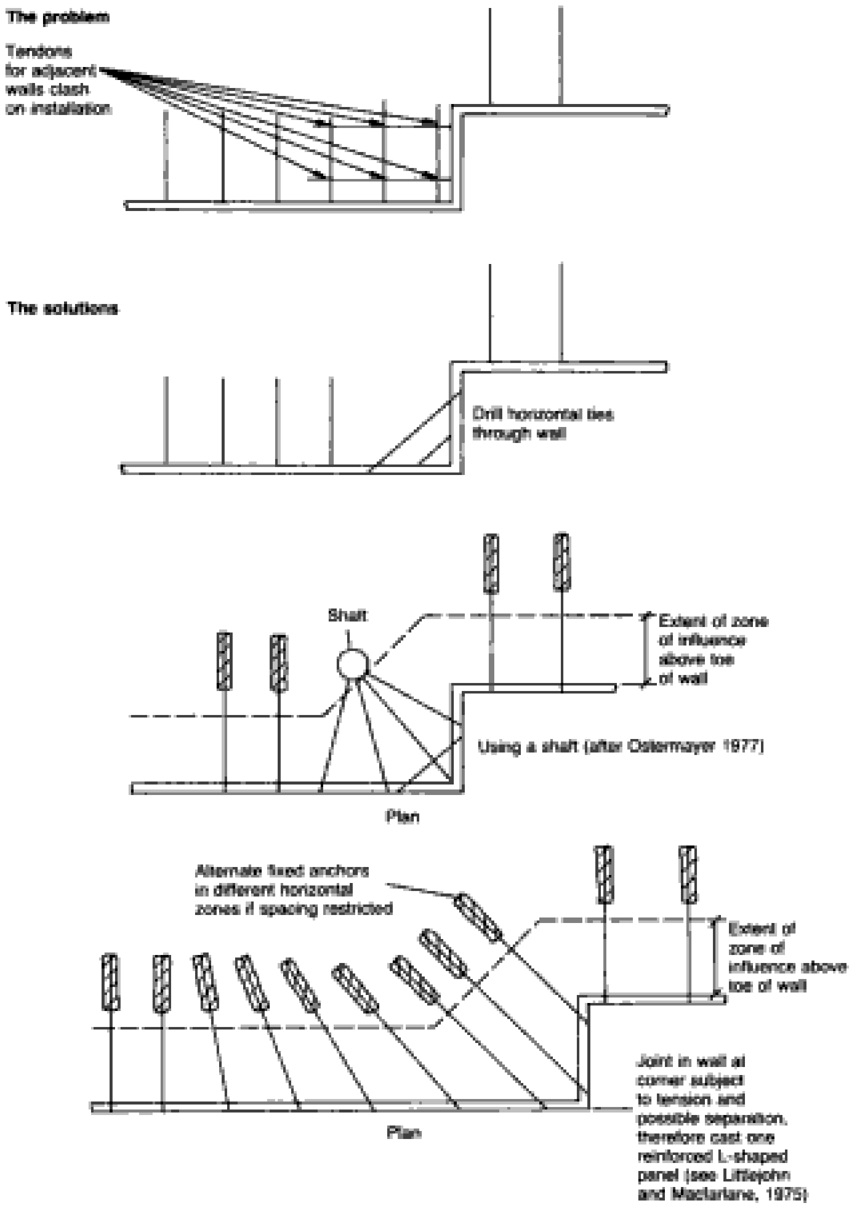
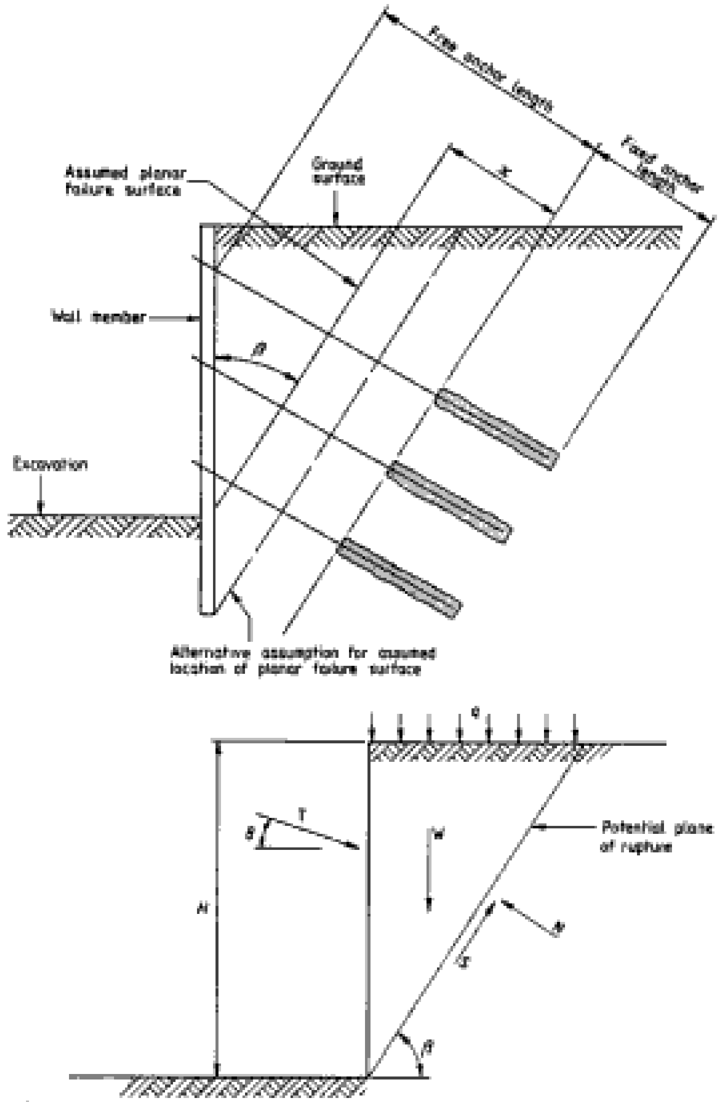


Fig. 7.4. Arrangement of ground anchors at re-entrant angles to retaining walls in plan (BS 8081[16])

(*a*) Type A: anchorages consisting of tremie, packer or cartridge-grouted straight shaft boreholes temporarily lined or unlined depending on hole stability.
(*b*) Type B: anchorages consisting of low pressure (typically injection pressures less than 1000 kN/m^2) grouted boreholes with an increased diameter within the fixed length.
(*c*) Type C: anchorages consisting of boreholes grouted to high pressure (typically injection pressures more than 2000 kN/m^2). The lengths of these anchors are enlarged by hydrofracturing of the ground to give a grout fissure system beyond the minimal borehole diameter. Post-grouting using tubes à manchette is frequently used with only a small quantity of secondary grout being required.
(*d*) Type D: tremie-grouted boreholes consisting of a series of bells or under-reams mechanically formed within the fixed length of the anchor.
(*e*) Type E: other types of anchor formed by jet grouting or similar techniques.



where

q is the surcharge;

W is the weight of sliding wedge $= 0.5\gamma H^2 \cot\beta$;

γ is the density of wedge;

H is the depth of excavation;

T is the anchorage force;

θ is the angle of inclination of anchorage force;

β is the angle of inclination of potential plane of rupture;

ϕ' is the effective angle of shearing resistance of retained ground;

c' is the effective cohesion of retained ground.

In practice T is expressed in terms of β, and β is then varied and plotted against values of T. The value of β, corresponding to the maximum value of T, defines the critical plane of rupture.

The key relationships are

$$T = \frac{(q + \gamma H/2)\, H \cos\beta\, (S_f - \cot\beta \tan\phi') - c'H/\sin\beta}{\sin(\theta + \beta) \tan\phi' + S_f \cos(\theta + \beta)}$$

and

$$S_f = \frac{c'H/\sin\beta + [(q + \gamma H/2)\, H \cos\beta \cot\beta + T \sin(\theta + \beta)] \tan\phi'}{(q + \gamma H/2)\, H \cos\beta - T \cos(\theta + \beta)}$$

where

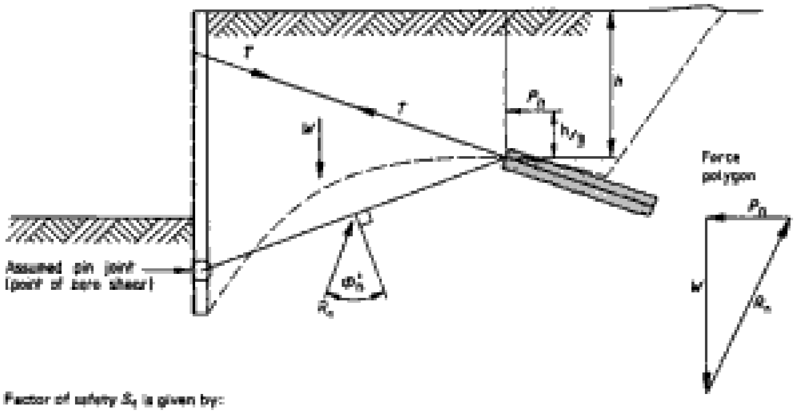
S_f is the required factor of safety (typically 1.5);

N is the normal force on wedge $= (q + \gamma H/2) H \cos\beta \cot\beta + T \sin(\theta + \beta)$;

S is the shear resistance of the retained ground $= c'H/\sin\beta + N \tan\phi'$;

T should not exceed the working load of the anchorage support per unit width.

Fig. 7.5. Ground anchors: wedge method of analysis (BS 8081[16])



Factor of safety S_f is given by:

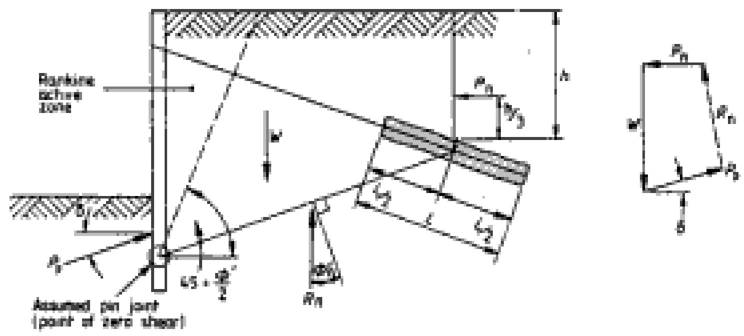
$$S_f = \frac{\tan \phi'}{\tan \phi'_n} \geq 1.5$$

where

ϕ'_n is nominal angle of shearing resistance (in degrees).

NOTE. If ϕ'_n has been correctly assumed, the weight W and the forces R_n, and P_n are in equilibrium. If this is not the case ϕ'_n has to be altered.

(a) Modified by Locher (1969) and Littlejohn (1970 and 1977)



Factor of safety S_f is given by:

$$S_f = \frac{\tan \phi'}{\tan \phi'_n} \geq 1.2$$

where

ϕ'_n is the nominal angle of shearing resistance of the soil (in degrees).

(b) Modified by Ostermayer (1977)

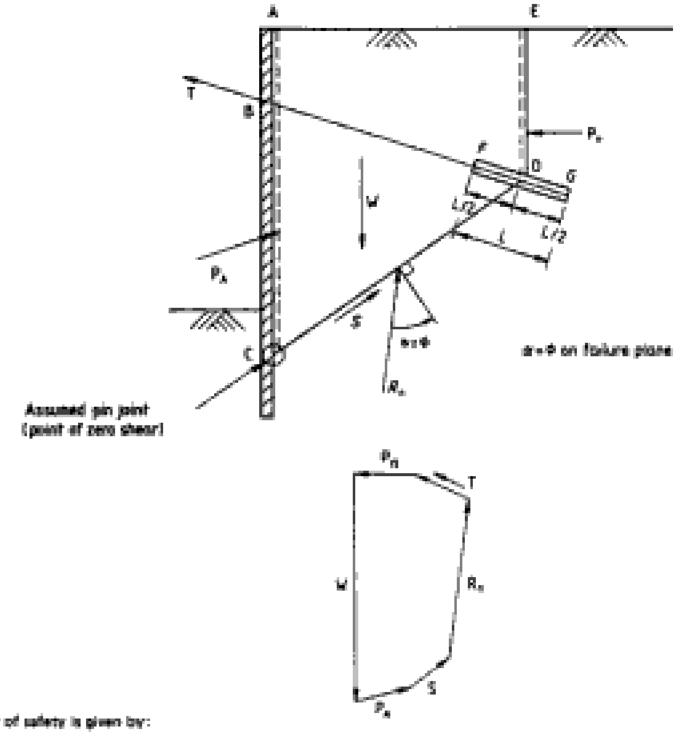
Fig. 7.6. Ground anchors: sliding block method of overall stability analysis (BS 8081[16]): (a) as modified by Locher and Littlejohn; (b) as modified by Ostermayer

BS 8081 refers to four items which have to be addressed in the design of these anchors: overall stability, depth of embedment, fixed anchor dimensions, and group effect. To check overall stability in cohesionless soils, several analysis methods are detailed in BS 8081. The simplest method is based on a planar failure surface. The failure wedge may be assumed as based on Coulomb wedge with an angle to the of the wall of $\beta = 45° + \phi/2$ (Fig. 7.5). A further method known as the sliding block method is shown in Fig. 7.6 with a subsequent variation shown as Fig. 7.7. The fixed anchor dimensions based on safety factor methods as detailed below comply with the recommendations of BS 8081. Suitable safety factors for temporary and permanent anchors are reproduced from BS 8081 in Table 7.2.

From BS 8081, for fixed anchor design in rock, for type A anchorages, the ultimate load carrying capacity is estimated from

$$T_f = \pi D L \tau_{ult} \tag{64}$$

where τ_{ult} is the ultimate bond or skin friction at the rock/grout interface (kN/m²),



Factor of safety is given by:

$$S_f = \frac{\tan \phi'}{\tan \phi'_m} \geqslant 1.3$$

for non-critical applications.

$$S_f \geqslant 1.5$$

for critical applications

(after Kranz (1953) and Ranke and Ostermayer (1968)).

W is the weight of soil mass within the failure surface.

P_n is the design force acting on the surface DE. A driving force due to water must be considered when below the water table. While P_n has been drawn horizontally, it could have been an inclined force.

R_n is the frictional component of soil resistance. This force is applied at an angle, $\alpha = \phi$ (full obliquity) to the normal base of the soil mass. It should be noted that α cannot be greater than the internal friction angle of the soil. Mobilized shear resistance acting along the plane is $(R_n \cos\phi) \tan\phi$.

S is the component of soil resistance due to cohesive soil strength. (Generally ignored).

P_A is the active earth force between point A and point C. Point C is the point of zero shear.

T is the anchorage force.

(c) (after Cheney 1984).

Fig. 7.7. Ground anchors: revised sliding block method of overall stability analysis (BS 8081[16])

D is the diameter of the fixed anchor (m), and L is the length of the fixed anchor (m). Bond values recommended for design are shown in Table 7.3.

For fixed anchor design in cohesionless soils for type B anchorages

$$T_f = Ln \tan \phi' \tag{65}$$

where ϕ' is the effective angle of shearing resistance (degrees) and n is a factor to allow for drilling technique, depth of overburden, fixed anchor diameter and grout pressure. Littlejohn[18] suggested that n ranges from 400 to 600 kN/m for coarse sands and gravels, and from 130 to 165 kN/m in fine-to-medium sands. These values were measured in borehole anchor diameters of approximately 0·1 m. Where the design diameter is larger, n should be increased in linear proportion. Alternatively,

$$T_f = A\sigma_v' \pi DL \tan \phi' + B\gamma h \frac{\pi}{4} (D^2 - d^2) \text{ (side shear + end shear)} \tag{66}$$

Table 7.2 Minimum safety factors recommended for design of individual anchorages (BS 8081[16])

Anchorage category	Minimum safety factor			Proof load factor
	Tendon	Ground/grout interface	Grout/tendon or grout/encapsulation interface	
Temporary anchorages where a service life is less than six months and failure would have no serious consequences and would not endanger public safety, e.g. short-term pile test loading using anchorages as a reaction system.	1·40	2·0	2·0	1·10
Temporary anchorages with a service life of say up to two years where, although the consequences of failure are quite serious, there is no danger to public safety without adequate warning, e.g. retaining wall tieback	1·60	2·5*	2·5*	1·25
Permanent anchorages and temporary anchorages where corrosion risk is high and/or the consequences of failure are serious, e.g. main cables of a suspension bridge or as a reaction for lifting heavy structural members.	2·00	3·0†	3·0*	1·50

* Minimum value of 2·0 may be used if full-scale field tests are available.
† May need to be raised to 4·0 to limit ground creep.
Note 1. In current practice the safety factor of an anchorage is the ratio of the ultimate load to design load. the table defines minimum safety factors at all the major component interfaces of an anchorage system.
Note 2. Minimum safety factors for the ground/grouts interface generally lie between 2·5 and 4·0. However, it is permissible to vary these, should full-scale field tests (trial anchorage tests) provide sufficient additional information to permit a reduction.
Note 3. The safety factors applied to the ground/grout interface are invariably higher compared with the tendon values, the additional magnitude representing a margin of uncertainty.

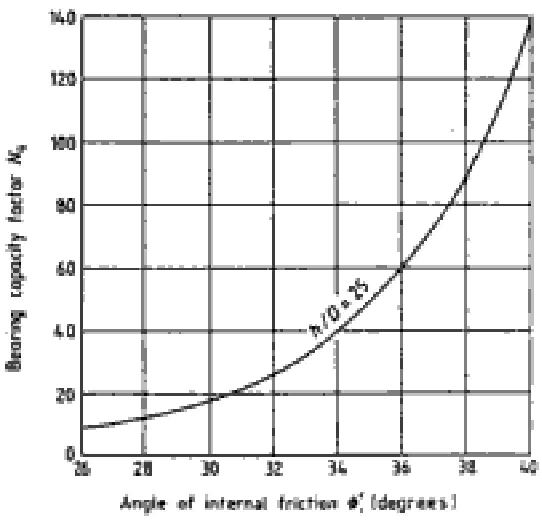


Fig. 7.8. Berezantzev's curve of bearing capacity factor N_q against angle of shearing resistance

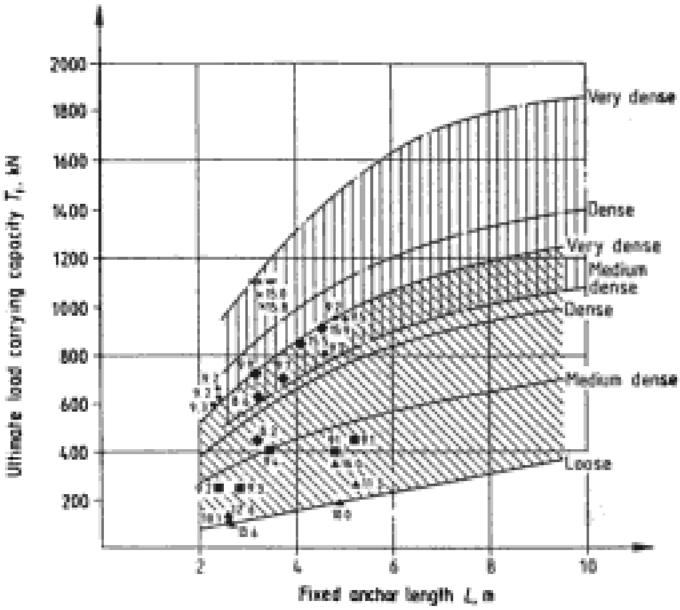
where A is the ratio of contact pressure at the fixed anchor/soil interface to the average effective overburden pressure (it has a value between 1 and 2, depending on construction technique), γ is the unit weight of soil overburden (use submerged weight below water table((kN/m^3), h is the depth of overburden to the top of the fixed anchor (m), σ_v' is the average effective overburden pressure adjacent to the fixed anchor (kN/m^2), B is the bearing capacity factor and is equivalent to $N_q/1 \cdot 4$ (refer to Berezantev's curve, Fig. 7.8), d is the diameter of the grout shaft above the fixed anchor (m) and D is the diameter of the fixed anchor.

Table 7.3 Rock/grout bond values which have been employed in practice (Littlejohn and Bruce[17])

Rock type	Working bond (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	Factor of safety Measured	Factor of safety Design	Source
Igneous						
Basalt	1·93		6·37		3·3	Britain—Parker (1958)
Basalt	1·10	3·60				UDA—Eberhardt and Veltrop (1965)
Tuff	0·80					France—Cambefort (1966)
Basalt	0·63	0·72				Britain—Cementation (1962)
Granite	1·56	1·72				Britain—Cementation (1962)
Dolerite	1·56	1·72				Britain—Cementation (1962)
Very fissured felsite	1·56	1·72				Britain—Cementation (1962)
Very hard dolerite	1·56	1·72				Britain—Cementation (1962)
Hard granite	1·56	1·72				Britain—Cementation (1962)
Basalt and tuff	1·56	1·72				Britain—Cementation (1962)
Granodiorite	1·09					Britain—Cementation (1962)
Shattered basalt		1·01				USA—Saliman and Schaefer (1968)
Decomposed granite		1·24				USA-Saliman and Schaefer (1968)
Flow breccia		0·93				USA-Saliman and Schaefer (1968)
Mylontised prophyrite	0·32—0·57					Switzerland—Descoeudres (1969)
Fractured diorite	0·95					Switzerland—Descoeudres (1969)
Granite	0·63	0·81				Canada—Barron *et al.* (1971)
Metamorphic						
Schist	0·31					Switzerland—Birkenmaier (1953)
All types	1·20					Finland—Maijala (1966)
Weathered fractured quartzite	1·56	1·72		1·1		Britain—Cementation (1962)
Blue schist	1·52	1·67		1·1		Britain—Cementation (1962)
Weak meta sediments	1·10	1·23		1·1		Britain—Cementation (1962)
Slate	0·43					Britain—Cementation (1962)
Slate/meta greywacke	1·57	1·73		1·1		Britain—Cementation (1962)
Granite gneiss	0·36—0·69					Sweden—Broms (1968)
Folded quartzite	0·51					Australia—Rawlings (1968)
Weathered meta tuff		0·29				USA—Saliman and Schaefer (1968)
Greywacke	0·34					Germany—Heitfeld and Schaurte (1969)
Quartzite	0·93—1·20	1·02—1·32		1·1		Britain—Gosschalk and Taylor (1970)
Microgneiss	0·95					Italy—Mantovani (1970)
Seridite schist	0·05					Italy—Berardi (1972)
Quartzite/schist	0·10					Italy—Berardi (1972)
Argillaceous and calcareous schist	0·63					Italy—Berardi (1972)
Slate	0·95	1·24		1·3		Switzerland—Moschler and Matt (1972)
Highly metasediments	0·83	1·08		1·3		USA—Buro (1972)
Slate and greywacke	1·08	1·40		1·3		Germany—Anon (1972)
Various metasediments			1·57			Germany—Abraham and Prozig (1973)
Micashist/biotite gneiss	0·53	0·80		1·5		USA—Nicholson Anchorage Co. Ltd (1973)
Slate	0·60	0·90	1·80	1·5	3·0	Britain—Littlejohn and Truman-Davies (1974)
Sound Micaschist	1·74	2·16				USA—Feld and White (1974)
Micaschist	0·52—0·74			1·24		USA—Feld and White (1974)
Very-poor gneiss and mud band	0·07					USA—Feld and White (1974)
Carbonate sediments						
Loamy limestone		0·63				Italy—Berardi (1960)
Fissured limestone and intercalations	1·08	1·19		1·1		Britain—Cementation Co. Ltd (1962)
Limestone	0·65					Switzerland—Muller (1966)
Poor limestone	0·32					France—Hennequin and Cambefort (1966)
Massive limestone	0·39—0·78					France—Hennequin and Cambefort (1966)
Karstic limestone	0·54					France—Hennequin and Cambefort (1966)

Table 7.3 (continued)

Rock type	Working bond (N/mm^2)	Test bond (N/mm^2)	Ultimate bond (N/mm^2)	Factor of safety Measured	Factor of safety Design	Source
Tertiary limestone	1·00		2·83		2·8	Switzerland—Losinger and Co. Ltd (1966)
Limestone			4·55–4·80			Switzerland—Ruttner (1966)
Marly limestone	0·03–0·07 (average) 0·21–0·36 (measured)					Italy-Berardi (1967)
Limestone			0·27			USA—Saliman and Schaefer (1968)
Limestone	0·28					Italy—Berardi (1969)
Dolomitic limestone			1·80			Canada—Brown (1970)
Marly limestone	0·39–0.94					Italy—Berardi (1972)
Limestone	0·26					Italy—Berardi (1972)
Limestone/puddingstone	0·44					France—Soletanche Co Ltd (1968)
Limestone	1·18	1·42		1·2		USA—Buro (1972)
Chalk			0·70			Britain—Associated Tunnelling Co. Ltd (1973)
Dolomite		1·66				Canada—Golder Brawner (1973)
Dolomitic siltstone	0·43					USA—White (1973)
Limestone and marly bands	0·37	0·55		1·5		Italy—Mongilardi (1972)
Arenaceous sediments						
Sandstone	1·44	1·58		1·1		Britain—Morris and Garrett (1956)
Hard sandstone	1·42	1·56		1·1		Britain—Cementation Co. Ltd (1962)
Bunter sandstone	0·95	0·98		1·03		Britain—Cementation Co. Ltd (1962)
Sandstone	0·76	0·84		1·1		Britain—Cementation Co. Ltd (1962)
Sandstone	0·74					Czechoslovakia—Hobst (1968)
Sandstone	0·31	0·40	1·73	1·29	5·6	USA—Drossel (1970)
Sandstone	0·80					USA—Thompson (1970)
Poor sandstone	0·40					Germany—Brunner (1970)
Good sandstone	1·14					Germany—Brunner (1970)
Sandstone and Breccia	0·38					France—Soletanche (1968)
Sandstone		0·95				Australia—Williams *et al.* (1972)
Bunter sandstone	0·60	1·20		2·0		Britain—Littlejohn (1973)
Sandstone	1·17					Australia—McLeod and Hoadley (1974)
Argillaceous sediments						
Shale	0·62					Canand—Juergens (1965)
Marl	0·10	0·28		2·8		Italy—Berardi (1987)
Shale	0·30		0·63		2·1	Canada—Hanna and Seaton (1967)
Very weathered shale			0·39			USA—Saliman and Schaefer (1968)
Shale	0·13–0·24					USA—Koziakin (1970)
Grey siltstone	0·62					Britain—Universal Anchorage Co. Ltd (1972)
Clay marl	0·14–0·24	0·21–0·36		1·5		Germany—Schwarz (1972)
Shale	0·62					Canada—McRosite *et al.* (1972)
Argillite	0·82					Canada—Golder Brawner (1973)
Mudstone	0·63	0·88		1·4		Australia—McLeod and Hoadley (1974)
Miscellaneous						
Bedded sandstone and shale	0·20–0·50					Italy—Beomonte (1961)
Porous, sound goassamer	1·57	1·72		1·1		Britain—Cementation Co. Ltd (1962)
Shale and sandstone	0·07	0·10		1·5		USA—Reti (1964)
Soft rocks	0·75					Sweden—Nordin (1966)
Sandstone and shale	1·82					Poland—Bujak *et al.* (1967)
Siltstone and mudstone	1·65					Australia—Maddox *et al.* (1967)
Fractured rock (75% shale)						
Poor			0·24			USA—Saliman and Schaefer (1968)
Average			0·35			USA—Saliman and Schaefer (1968)
Good			0·75			USA—Saliman and Schaefer (1968)
Limestone and clay breccia	0·20–0·23					Italy—Berardi (1972)

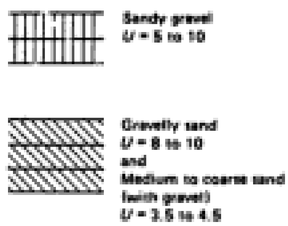


NOTE 1. Field evidence is limited to a fixed anchor range of 2 m to 5 m approximately.

NOTE 2. The relationships between soil density and standard penetration test (SPT) values are not in accordance with BS 5930.

	Type of soil	Density	Standard penetration test (SPT) N (bl/0.3 m)
●	Gravelly sand	Very dense	120
●	U = 1.6/0.16	Dense	60
■		Medium dense	43
▲		Loose	11
×	Sandy gravel U = 15/0.3	Very dense	>130

where
U Uniformity coefficient (D_{60}/D_{10})
D_{60} Maximum particle size of the smallest 60 %
D_{10} Maximum particle size of the smallest 10 %



Diameter of grouted bodies
D = 0.1 m to 0.15 m

Fig. 7.9. Ultimate load capacity as a function of fixed length in sandy gravels and gravelly sand (Ostermayer and Scheele[19])

For type C anchorages, in cohesionless soils, design methods have relied heavily on field tests in a range of soils rather than load capacity expressions. Fig. 7.9 shows load carrying capacity in sandy gravels and gravelly sands for increasing anchor length. More recently Sherwood and Harris[20] proposed an empirical design method for regroutable anchors which uses their experience over 15 years to establish limit shaft friction values in clays and silts and in sands and gravels over the groutable length of the proprietary TMD anchor from Bachy. They argued that while BS 8081 recognizes that pressure-grouted anchorages are superior to other types in terms of load capacity, BS 8081 does not differentiate between anchors installed with uniform injection pressure and those with repetitive and selective high-pressure injection. Sherwood and Harris concluded that factors of safety recommended by BS 8081 may be appropriate to less sophisticated ground anchors but are much too conservative for anchor systems which produce more uniform anchorage performance as a result of their installation procedure. They reasoned that preliminary anchor tests should be used as a means of refining design and reducing the factor of safety.

For type D anchorages, BS 8081 does not suggest a design method for cohesionless soils due to lack of published data but warns that if shaft enlargements are required to take all the anchor load, the shear across the interface between the nominal diameter shaft and the enlargements will require examination.

For fixed anchor design in cohesive soils, for type A anchorages, load carrying capacity may be estimated from

$$T_f = \pi D L \alpha c_u \qquad (67)$$

where c_u is the undrained shear strength (kN/m^2) and α is the adhesion factor (in the range 0·28 to 0·36 for stiff clays and 0·48 to 0·6 for stiff to very stiff marls).

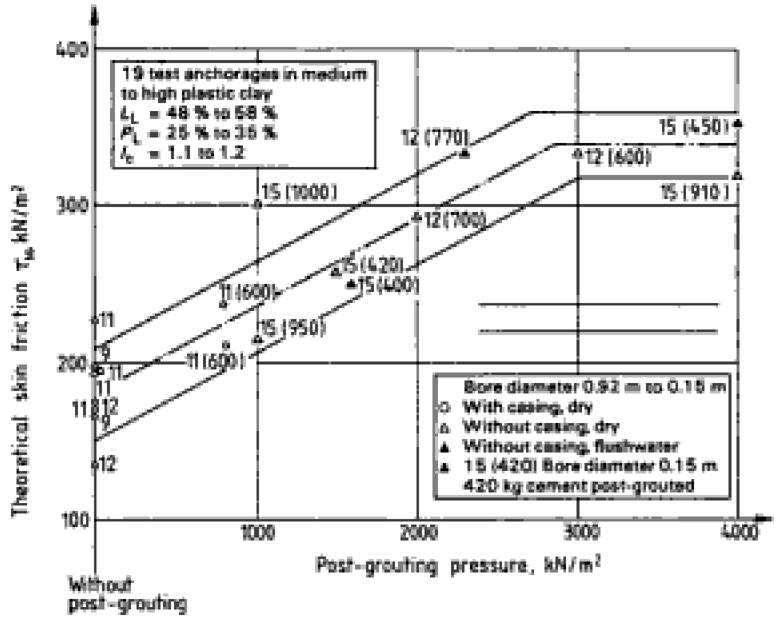


Fig. 7.10. Influence of post-grouting pressure on skin friction in a cohesive soil (Ostermayer[21])

NOTE. The theoretical skin friction is calculated from the borehole diameter and designed fixed anchor length.

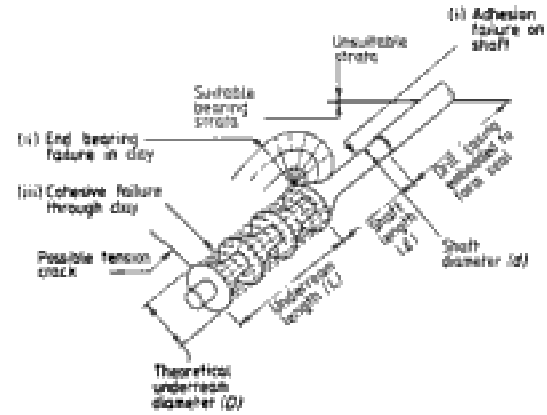


Fig. 7.11. Diagram of multi-underream anchorage at ultimate capacity (BS 8081[16])

For type C anchorages in cohesive soils, BS 8081 reproduces the work by Ostermayer[21] as a design guide for borehole diameters between 0·08 m and 0·16 m. The effect of post-grouting pressure on skin friction is shown in Fig. 7.10.

For type D anchorages in cohesive soils, the ultimate load capacity of multi-under-reamed anchorages is given by

$$T_f = \pi D L c_u + \frac{\pi}{4}(D^2 - d^2) N_c c_{ub} + \pi d l c_a \tag{68}$$

(side shear + end bearing + shaft resistance)

where D is the diameter of under-ream (Fig. 7.11) and N_c is the bearing capacity factor (a value of 9·0 is often used), c_{ub} is the undrained shear strength in the clay at the top end of the fixed length, and c_a is the shaft adhesion (in the range 0·3 to 0·35c_u) (kN/m²). BS 8081 comments that under-reaming is best suited to clays with an undrained cohesion c_u greater than 90 kN/m². Poor under-reams are likely if this value reduces to 60 to 70 kN/m², and under-reaming becomes almost impossible if c_u is less than $50\,kN/m^2$. Under-reaming is difficult in low plasticity soils where the plasticity index is less than 20.

BS 8081 suggests that to limit interaction between fixed anchors the spacing between anchors, centre to centre, should not be less than four times the enlarged fixed anchor diameter; in practice a minimum spacing of 1·5 to 2·0 m is usual. The tolerance of borehole enlargement should be considered when anchor spacing is decided, particularly with long anchors.

High grout pressures should be avoided for anchors founded at shallow depth where subsoil movement due to these pressures would be detrimental to existing structures or services.

The maximum values of grout/tendon adhesion recommended by BS 8081 for cement grout with a minimum compressive strength of $30\,N/mm^2$ before stressing are

- for clean plain wire or bar: $1{\cdot}0\,N/mm^2$
- for clean crimped wire: $1{\cdot}5\,N/mm^2$
- for clean strand or deformed bar: $2{\cdot}0\,N/mm^2$
- for locally noded strands: $3{\cdot}0\,N/mm^2$

Minimum recommended tendon bond lengths for cement or resin grouted anchorages are 3·0 m where the tendon is homed and bonded in situ, and 2·0 m for tendons bonded under factory-controlled conditions.

For prestressed anchors the tendons consist of steel bar, strand or wire, either singly or in groups. Prestressing steel is covered in sections 2 (for non-alloy steel wire) and 3 (non-alloy 7-wire strand) of BS 5896[22] and in BS 4486[23] (low alloy steel bar). Typical sizes and specified characteristic strengths from BS 8081[16] are reproduced in Table 7.4. The load-extension characteristics of these and alternative materials for tendons are shown in Fig. 7.12. It is usual to proof-load temporary works anchorages to a load equal to 1·25 times the required unfactored working load and then lock-off the anchor at 1·1 times the unfactored working load.

Design of the bearing plate beneath the stressing head requires care. A typical arrangement is shown in Fig. 7.13. The bearing is a thick steel plate, often stiffened to span between wedge-shaped plates which transfer the load from the stressing head through the bearing plate to the steel waling or soldier members. Where the bearing head is bedded on to the concrete wall surface (as for piled or diaphragm

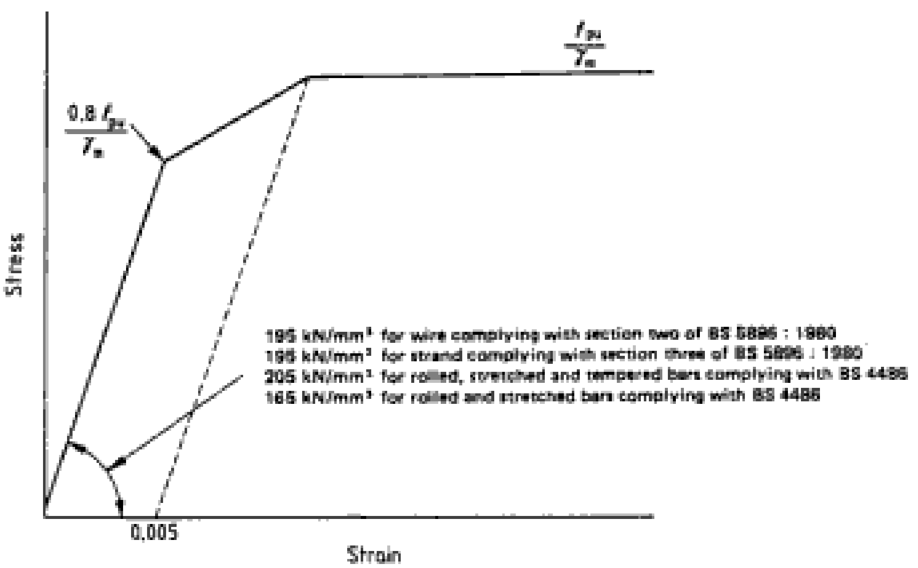


Fig. 7.12. Short-term design stress–strain curves for normal and low relaxation wire and bars used in anchors (BS 8081[16])

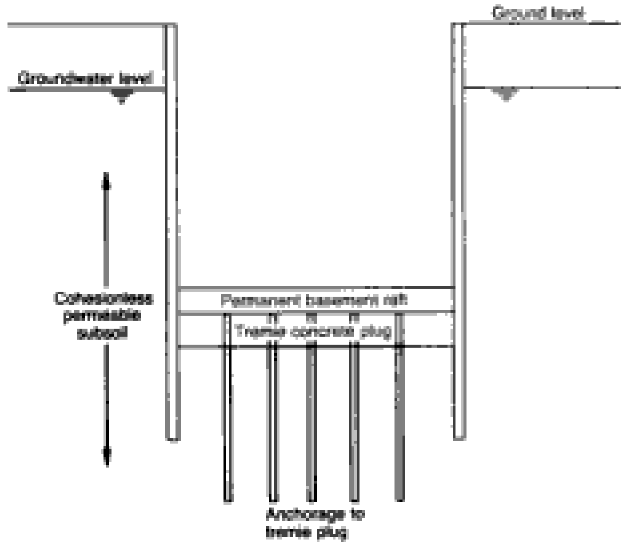
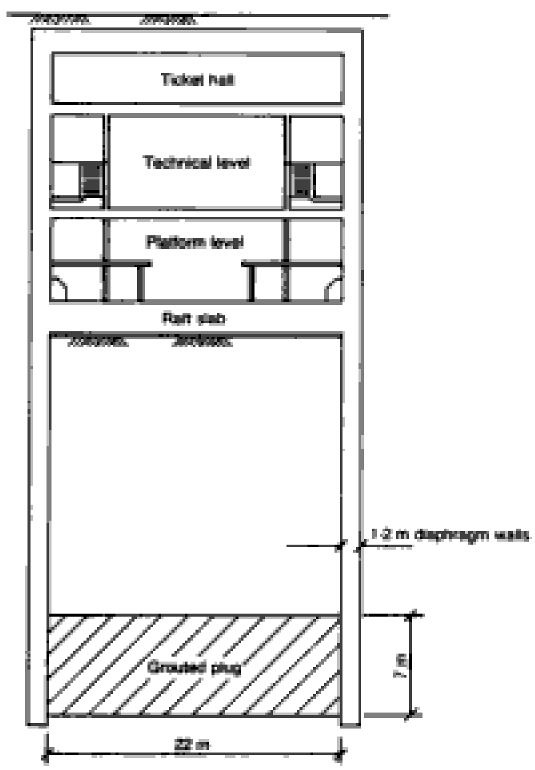


Fig. 7.14. Cross-section of cofferdam showing anchored plug



Grouted plug by two-stage injection by *tube à manchette* with two-stage treatment using cement/bentonite and sodium silicate.
Subsoil conditions consist of shallow fill deposits underlain by approx. 6 m. of stiff clay underlain by extensive sand deposit to depth below diaphragm wall construction.
Groundwater generally 3 to 4 m. below ground surface: perched water table in fill and confined within aquifer below clay stratum.

Fig. 7.15. Grouted soil plug to metro station excavation, Cairo Metro, 1994 (from Tunnels and Tunnelling*)*

Table 7.5 Guide for the selection of pile size to suit driving conditions in granular soils (CIRIA[4])

Dominant SPT (N value)	Minimum wall modulus (cm^3/m)		Remarks
	BSEN 10 025 Grade 430A, BS 4360 Grade 43A	BSEN 10 025 Grade 510A, BS 4360 Grade 50A	
0–10	450		Grade FE 510A for lengths greater than 10 m
11–20		450	
21–25	850		
26–30		850	Lengths greater than 15 m not advisable
31–35	1300		Penetration of such a stratum greater than 5 m not advisable*
36–40		1300	Penetration of such a stratum greater than 8 m not advisable
41–45	2300		
46–50		2300	
51–60	3000		
61–70		3000	Some declutching may occur
71–80	4200		Some declutching may occur with pile lengths greater than 15 m
81–140		4200	Increased risk of declutching. Some piles may refuse.

* If the stratum is of greater thickness use a larger section of pile

cofferdam is made to the full depth, reducing the lower strut loads and possibly even reducing the number of cofferdam frames. Typical details are shown in Fig. 7.15.

Sheet piling: selection of section

Where steel sheet piling is used for the outer walls of a land or marine cofferdam, the choice of section depends both on the flexural strength needed to resist earth and water pressures and the strength needed to resist driving stresses. Although these latter stresses may be reduced dramatically by jetting through granular soils, this provision may not necessarily be available. The CIRIA report[4] provides some assistance to the designer in assessing the pile section needed for driveability in both cohesionless and cohesive soils. Table 7.5 from the report is based on experience of sheet piles, approximately 500 mm wide, driven in panels. The two criteria for sufficiency of the pile section during driving or vibration is that the head of the pile should not be damaged unreasonably and the toe of the pile should not be damaged and become declutched. Resistance to pile penetration in granular soils is primarily at the toe and the effect of skin friction on a moving pile is not severe. Pile penetration is therefore a function of the relative density of the soil at the pile toe. For cohesive soils, resistance to pile installation is primarily due to soil adhesion to the pile face and little resistance occurs at the pile toe. The resistance to penetration in clay is therefore a function of clay strength c_u and the length of pile within the clay. Damage to the pile toe is unlikely but resistance to pile buckling is necessary when the driving resistance, in terms of clay adhesion, is overcome by a hammer of sufficient capacity. Table 7.6 provides guidance for pile driving in cohesive soils and appeared in the *Specification for Steel Sheet Piling* published by the Federation of Piling Specialists.

It should be noted that piles driven into clay and installed by a pile driver which uses hydraulic rams should be set out in a plan arrangement which takes account of the requirements of the pile driver. Two types of pile driver are common: the first drives piles in panels of six to eight piles in a straight line, while the other drives one pile at a time, relying on reactions from a previously driven pile.

Table 7.6 Guide to selection of pile size to suit driving conditions in cohesive soils (CIRIA[4])

Clay description	Minimum wall modulus (cm^3/m)		Maximum length (m)
	BSEN 10 025 Grade 430A, BS 4360 Grade 43A	BSEN 10 025 Grade 510A, BS 4360 Grade 50A	
Soft to firm	450	400	6
Firm	600–700	450–600	9
Firm to stiff	700–1500	600–1300	14
Stiff	1600–2500	1300–2000	16
Very stiff	2500–3000	2000–2500	18
Hard (c_u>200)	Not recommended	4200–5000	20

Note: The ability of piles to penetrate any type of ground is also a function of attention to good pile driving practice and this table assumes that this will be the case

Design of the bracing

Walings

Where walls or sheeters span vertically, walings are needed to transfer loads from the sheeting to the struts, which provide the gracing, or the anchors, which retain the sheeting. The walings need not be continuous as, for example, in hammer head struts used against diaphragm wall panels where separate waling reinforcement may be included within the panel reinforcement to the wall. Alternatively, with anchored diaphragm walls it is common to incorporate waling steel to the full panel width without external walings. Where secant pile walls are used in cofferdams, walings may not be necessary where every pile or alternate piles are anchored. Common waling arrangements are shown in Fig. 7.16.

Where steel walings are used it may be convenient to use steel beams in pairs in order to provide adequate width on which to seat the bracing struts. It is often convenient to weld end plates to each length of waling to connect them together. It is vital that where rakers or sloping struts are used the tendency for the waling to turn on its support must be resisted. Fig. 7.17 shows a typical detail. Where anchors are used with walings, the spacing between steel beams must be sufficient to accommodate the tendon between them.

Where steel sheet piling is used it is usual to make the walings continuous over two supports. Unless the piling can be driven to good tolerances in vertical and horizontal alignment it is prudent to allow walings to extend mid-span between struts without connecting one to the other. Where tolerances are likely to be well maintained, it is advantageous to connect the ends of walings behind the incoming strut (Fig. 7.18). Where walings are continuous over two spans and joined behind struts the design moment is $WL/10$, but where they cantilever to half span the design moment becomes $WL/8$. The steel waling is designed with a load factor of 1·4 in accordance with BS 5950,[28] or where reinforced concrete walings are designed in accordance with BS 8110[29] a similar load factor of at least 1·4 should be used.

Where steel sheet piles are braced by steel walings any irregular alignment of the steel piles is rectified by hardwood wedges or pages. Where the alignment is particulary poor, concrete infilling can be used between the waling and the sheeters. If diagonal struts transfer longitudinal thrust into the waling, the waling must be designed to take both this thrust and bending stresses due to the span between struts. It may be necessary to weld steel angles to the back of the walings prior to erection in order that shear keys can be formed by concreting the leg of the angle into the pan of the sheet pile. This will be required if the available length of waling is short and therefore the frictional resistance between waling and sheet pile is insufficient to transfer the thrust (Fig. 7.19).

Where heavily loaded struts or highly loaded anchors bear on steel walings it

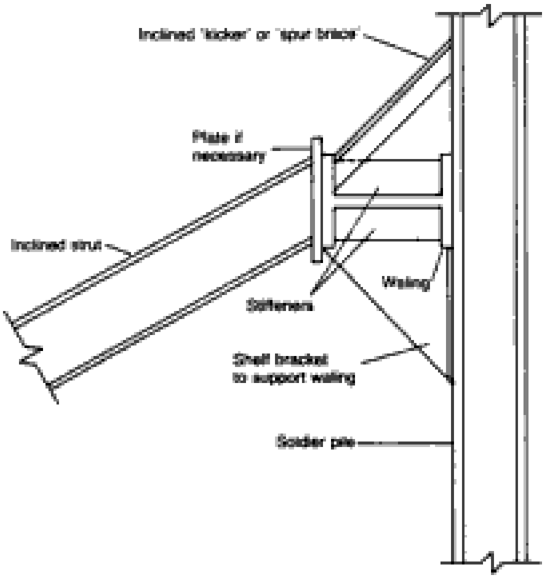


Fig. 7.17. Typical steelwork detail at junction of rakes and waling with bracing to prevent rotation of waling

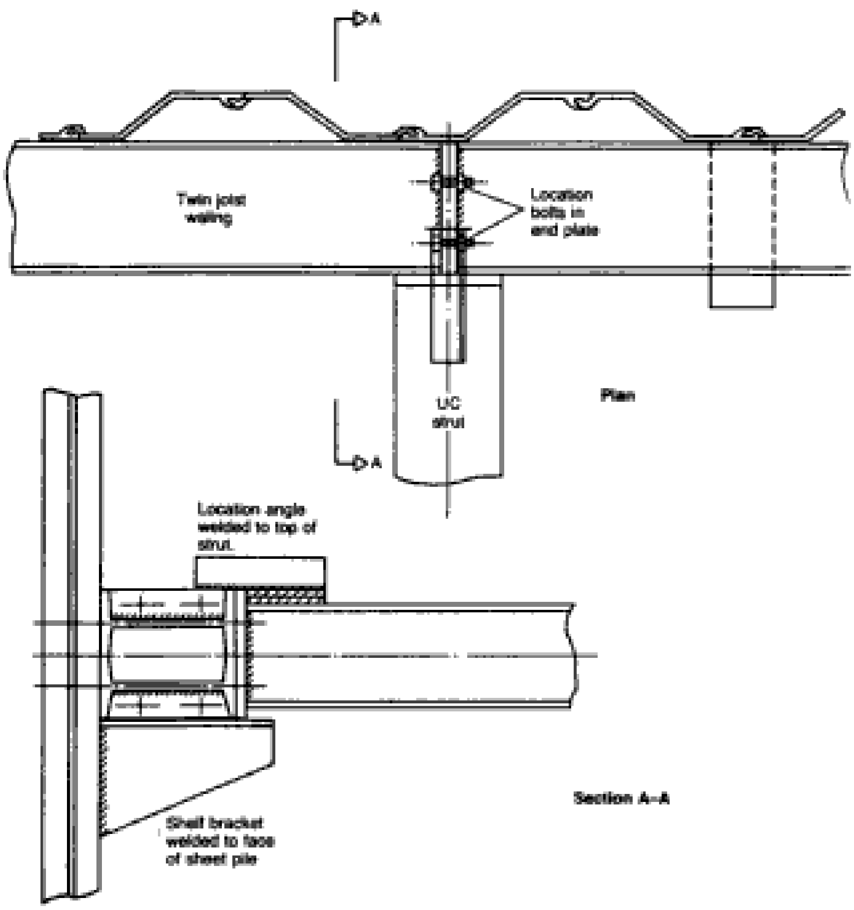


Fig. 7.18. Detail of typical waling connection behind strut in light cofferdam steelwork

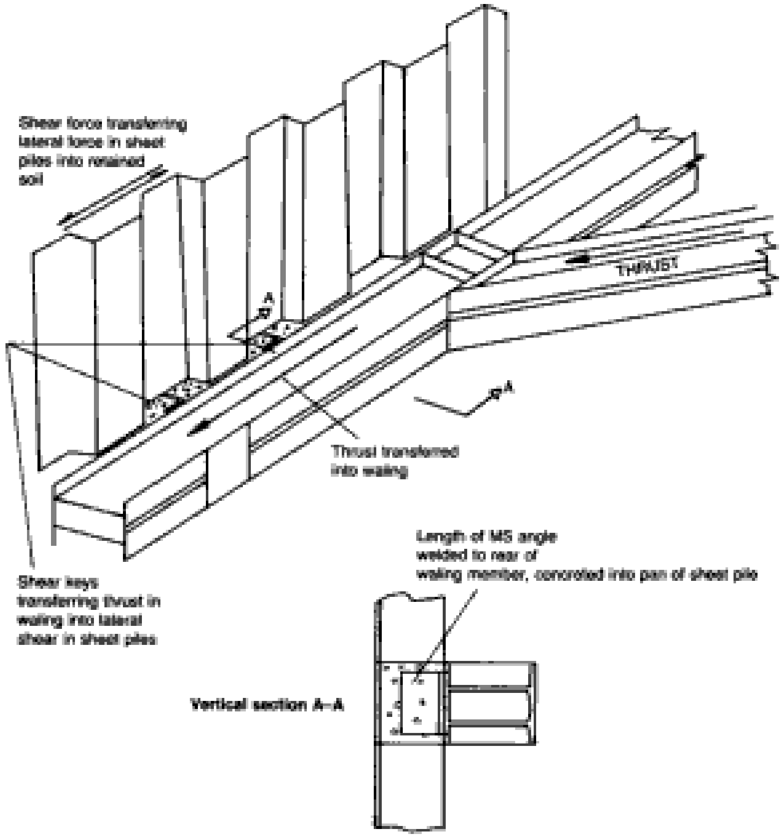


Fig. 7.19. Sheet piled cofferdam construction: shear keys at rear of waling transfer thrust from diagonal strut into short waling length and through sheeters into soil at rear of piles

Passive anchors

The design of anchored walls is described in chapter 5. Tie rods from the sheeting wall are anchored to dead men, an anchor wall or an A-frame of driven piles. Anchor walls are designed on the basis that net available passive resistance is equal to passive pressure less active pressure. No allowance should be made for surcharge in front of an anchor wall in this calculation, and wall friction should be ignored because of the risk of vertical movement of the wall to the detriment of this friction. Tie rods, based on a factor of safety of 1·5 to 2·0, are designed using the following working stresses:

- mild steel (BS 4360[30] grade 43A): 111 N/mm^2
- high-yield steel (BS 4360[29] grade 50B or 50C): 140 N/mm^2.

Tie rods may be housed in pipework to avoid the effects of fill settlement and can be wrapped in Denso tape to reduce corrosion.

Struts

The most likely collapse mechanism of a braced cofferdam is the buckling of its strutting, beginning with the lowest frame and continuing progressively to the highest frame. The collapse of the lowest frame may be associated with inadequate penetration of the sheeting and passive failure below formation level; it may occur during extreme loading, such as high water for a river cofferdam or high waves in storm conditions for a cofferdam in open water. It may also be associated with poor workmanship in bracing or piling, or both. Collapse due to failure of walings or flexural failure of the sheeting itself are much less likely. The extra care that

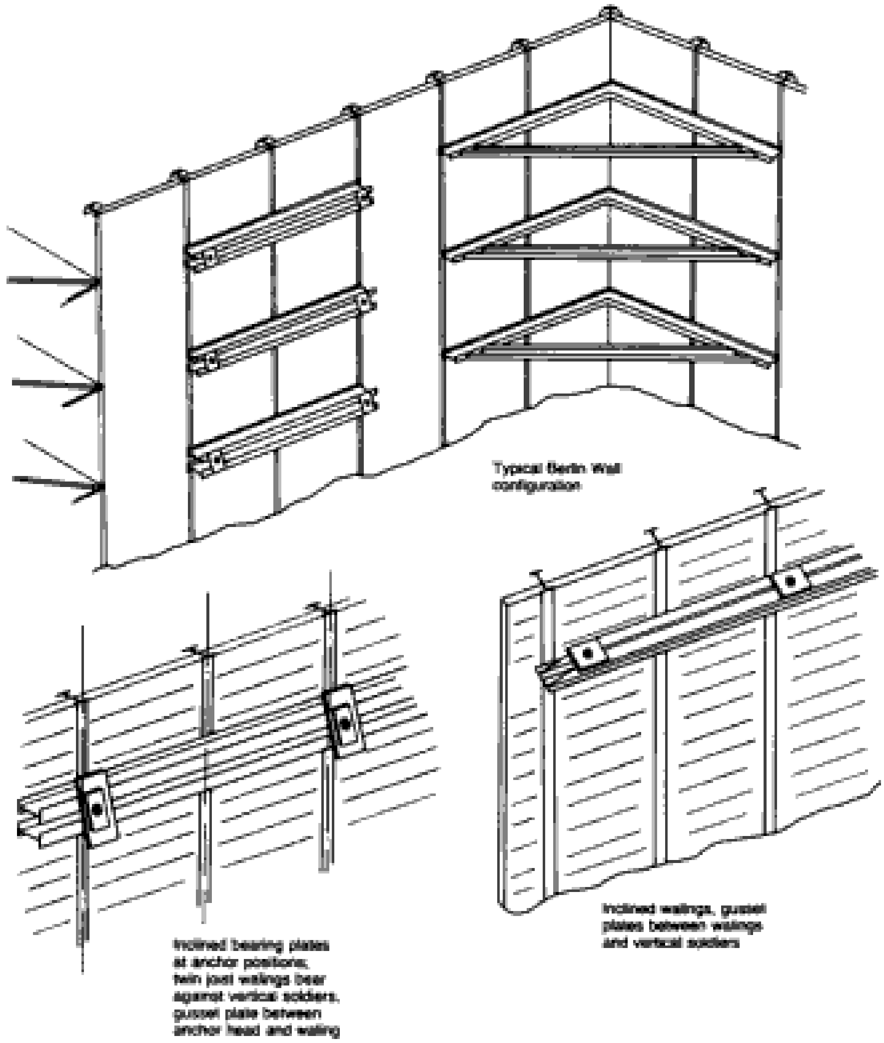


Fig. 7.21. Waling details with inclined anchors, shown with typical Berlin wall

It is essential to support struts adequately at the walings at each end of the strut. Steel location angles may be useful welded to the strut end plates prior to bolting or welding into position. It is vital that the struts are square to the walings in plan and the end plates bear uniformly on the walings to avoid eccentric loading. (It is, nevertheless, worth checking the effect of eccentricity of the thrust in the strut by, say, 10% of the strut width or depth in each direction.) The effect of materials and plant loads placed on the strut should be added to the self-weight of the strut in considering the combined effect of compression load and bending.

Struts are designed to BS 5950[28] with a load factor of at least 1·4 and, where loading or workmanship are uncertain, as high as 2·0. The effective length of the strut is assumed to be equal to its actual length unless braced laterally or vertically at mid-span (Fig. 7.24). Temperature effects should be considered where frames are exposed to sunlight in tropical countries, or to extremes of cold weather. The effect of sunlight on long steel struts can be reduced by painting them white.

It is essential that cofferdams are stiffened along width and length by diagonal bracing to avoid risk of collapse into a lozenge shape. Typical bracing examples steelwork are shown in Fig. 7.25.

The vertical spacing of struts depends on both the strut capacity and the flexural strength of the sheeters or walling. In practical terms, the minimum frame spacing

Fig. 7.22. Tubular steel struts used to brace cofferdam at Dartford Creek, UK (courtesy of AMEC)

Fig. 7.23. Battened steel beams used as struts and braces on River Thames cofferdam, London

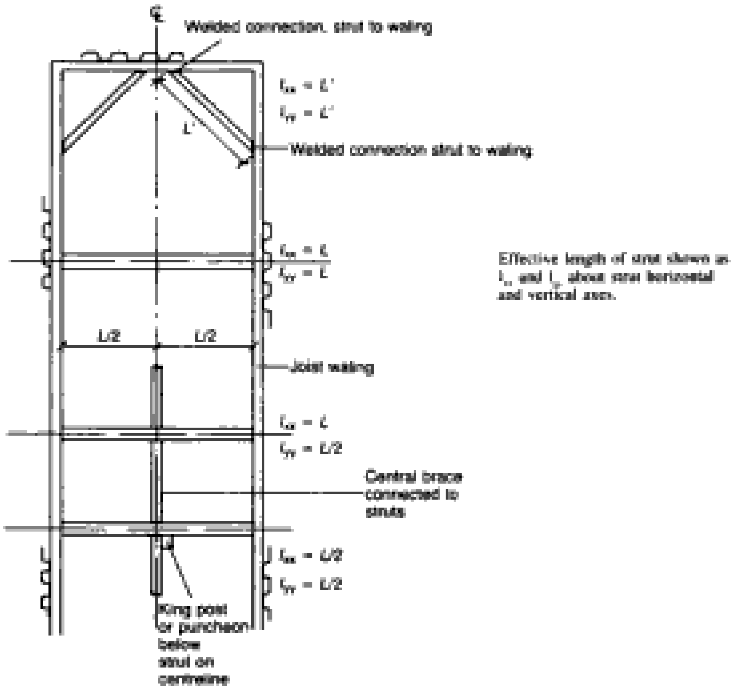


Fig. 7.24. Plan of cofferdam construction showing effective lengths of struts used in design

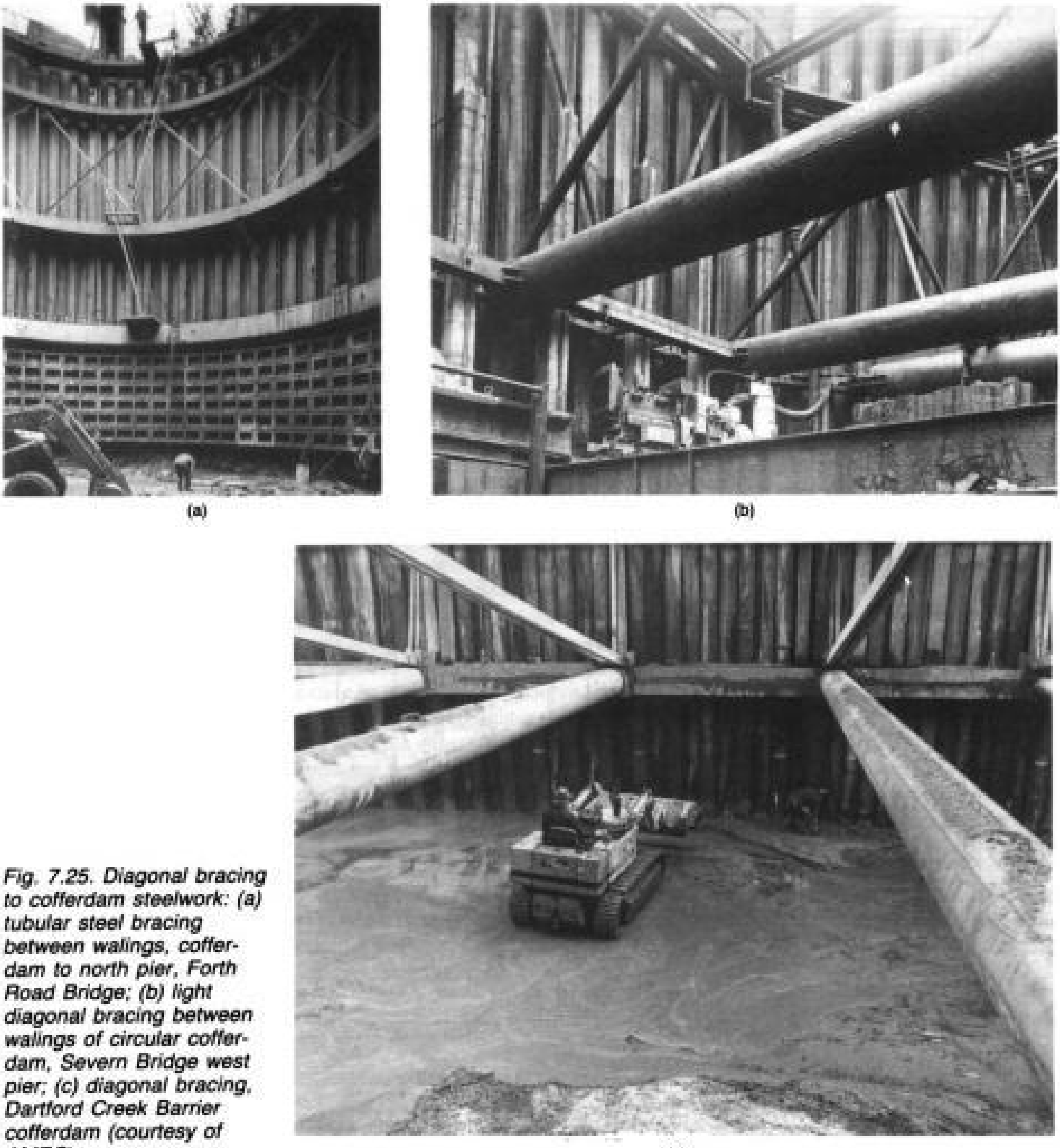
needs to be sufficient to allow mechanical excavation plant to pass under the frame prior to placing the next lowest frame and to allow sufficient excavation of the cofferdam. The placing of the lowest frame in a multi-frame cofferdam is the period of greatest risk to the bracing. At this stage the excavation is usually close to final formation level. It is likely that the next highest frame will be highly loaded and the factor of safety against passive failure will be at its lowest for all stages of the excavation. The bending stress and deformation of the sheeting will be at their highest values, and although two-dimensional analysis will be unable to show any benefit, it will be a practical advantage to carry out excavations for the lowest frame in short lengths where stresses in the soil, bracing and sheeters are excessively high at this stage. It is essential that all bracing components are fabricated ready for installation in this bottom frame, to avoid the cofferdam remaining unpropped at the lowest level for any lengthy period, especially at high tides, where these apply. It should be noted that the use of anchors on the bottom frame does not allow the sheeters to be secured speedily at this critical lowest level since the anchor grout requires a minimum time to achieve sufficient strength in order to apply prestress and thence to secure the tendons.

Where reinforced concrete diaphragm walls are used to retain soil at the curtilage of a cofferdam it is possible to use the strength of the wall panel, designed as a plate, with support from passive soil resistance at the bottom of the panel and point supports from hammer head struts at the vertical joints between panels. This arrangement obviates the need for either external or internal walings, but high shear stresses may occur within the panel near the stub end of the strut and require shear reinforcement. The use of reinforced concrete struts for this arrangement suffers from the comparatively high self-weight of the units, the difficulty of altering the reinforced concrete section for use elsewhere, and the high cost of disposal.

Walling details

Items that need to be addressed in detailing the wall reinforcement include:

- lifting and bracing steel
- method of joining sections of cages



(a) (b) (c)

Fig. 7.25. Diagonal bracing to cofferdam steelwork: (a) tubular steel bracing between walings, cofferdam to north pier, Forth Road Bridge; (b) light diagonal bracing between walings of circular cofferdam, Severn Bridge west pier; (c) diagonal bracing, Dartford Creek Barrier cofferdam (courtesy of AMEC)

- method of lifting
- lateral spacing between reinforcement cages of adjacent panels
- type of box-out for junction with floor slabs
- details of starter steel for floor slabs
- use of couplers
- detail of access through panel for ground anchors
- inclusion of reinforcement for walings
- minimum spacing of reinforcement steel and provision of cover
- consideration of water bars in vertical panel joints.

Table 7.7 Safe load (in kN/m) in reinforced concrete waling to circular cofferdams[34]

D, diameter of cofferdam (m)	Size of waling $d \times b$ (mm): 450×300, six 20 mm dia. bars	600×400, ten 20 mm dia. bars	750×500, ten 25 mm dia. bars	900×600, fourteen 25 mm dia. bars	1050×700, twelve 32 mm dia. bars
5	280	500			
10	140	250	390		
15	90	165	260	375	
20		125	195	280	380
25			155	225	305
30				185	255
35					215

Based on: (i) permissible compressive stress in concrete not to exceed 5·2 N/mm²
(ii) waling load (in kN/m) = $1{\cdot}5EI/10^3R^3$; E = Young's modulus, for concrete, E = 13 800 N/m, I = moment of inertia about xy axis (cm^4, R = cofferdam radius (m)
(iii) depth of waling d to be not less that $D/35$
(iv) need to check tension in waling beam if sheet piles distort under load and concentrate load on top and bottom of waling beam

$d \geq D/35$, where d is the depth of the ring beam (m) and D is the diameter of the inner face of the cofferdam sheeters (m). Table 7.7 gives safe loads for reinforced concrete walings of specific size and reinforcement for cofferdams of varying diameter.[34] Care is needed to avoid uneven loading of the waling either at the top or lowest level of the waling due to non-verticality of sheeters. Any uneven distribution of applied load over the depth of the waling could induce torsional stress within it.

The peripheral walls to circular land cofferdams may be built in contiguous or secant piling or diaphragm walling. Such walls are particularly economical where they are used as temporary support during construction and as permanent walls to the final structure. Secant or contiguous piles, spanning between ring walings in reinforced concrete, are limited to approximately 30 m in depth, depending on ground conditions. Maintaining the secant connection between adjacent piles may prove difficult in some ground conditions and would certainly require the use of large piling plant and casing oscillators.

Diaphragm walling has advantages and disadvantages over bored pile walls. In depth, for instance, with modern reverse circulation trench cutters diaphragm walling can be installed in a range of soils and soft rocks to depths in excess of 50 m. In shafts of small diameter, the length-to-breadth ratio of each wall segmental panel may be such that the wall is kept in hoop compression by earth and groundwater pressures and little or no bending occurs within the wall panel. In these circumstances, the wall requires minimal reinforcement; in fact the concrete strength may be adequate to withstand the hoop compression. In practice, it is best to consider the risk of at least one panel joint failing to transfer load to its neighbour because of poor panel verticality. Sufficient vertical steel should be allowed for the introduction of an emergency waling.

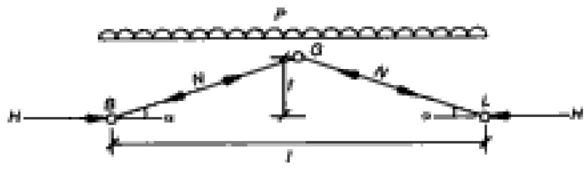
Where the circular cofferdam diameter is larger, however, the deviation angle between the centre-line of each diaphragm wall panel is reduced and horizontal bending occurs within the panel in addition to hoop compression. In this case it is generally more economical to span the wall vertically and introduce temporary walings. In top-downwards construction, permanent floor slabs provide both temporary and permanent support to allow the wall to span vertically between them. Fig. 7.27 shows a method of stress analysis for diaphragm walls without walings designed to span circumferentially.

1. Design of circular diaphragm walls can be made by considering a unit depth of wall of low adjacent segmental panels assuming the circular panel joints to constitute structural pinned joints, the two panels forming a three-pinned arch. Hoop compression or thrust within the arch and the bending moments within the panel are calculated as follows:

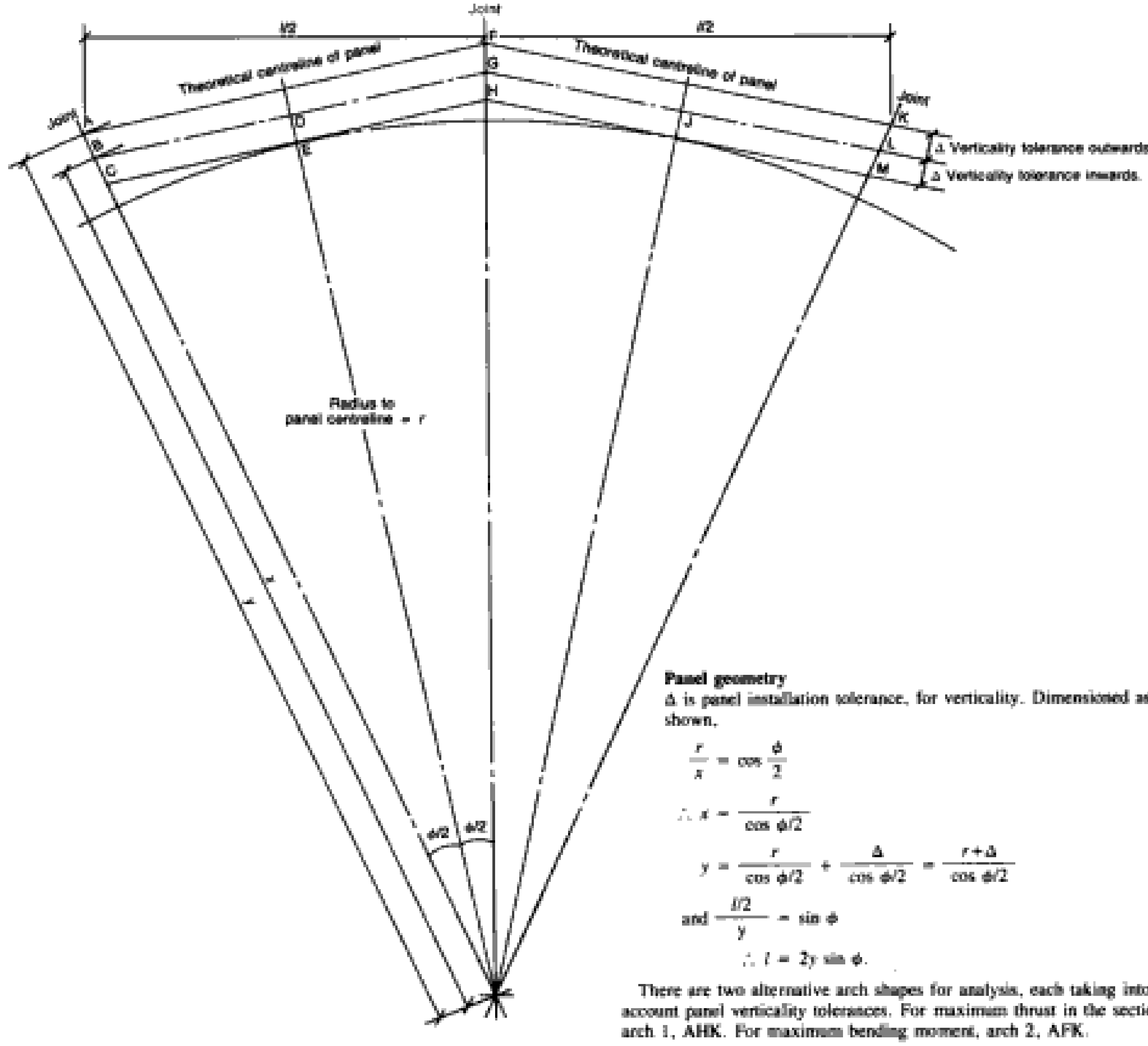
   For a three-pinned arch with uniformly distributed load p. Horizontal force at pinned support

   $$H = \frac{pl^2}{8f} \text{ and axial thrust } N = \frac{H}{\cos \alpha}$$

   Bending moment in panel = (Free span moment) − (H × rise of arch).
2. Consideration should be given, however, to the effect of panel verticality tolerances. If the maximum permitted verticality tolerance is Δ the true shape of the arch may be displaced within the limits AEK to CHM.



(a)



Panel geometry

Δ is panel installation tolerance, for verticality. Dimensioned as shown,

$$\frac{r}{x} = \cos\frac{\phi}{2}$$

$$\therefore x = \frac{r}{\cos \phi/2}$$

$$y = \frac{r}{\cos \phi/2} + \frac{\Delta}{\cos \phi/2} = \frac{r+\Delta}{\cos \phi/2}$$

$$\text{and } \frac{l/2}{y} = \sin \phi$$

$$\therefore l = 2y \sin \phi.$$

There are two alternative arch shapes for analysis, each taking into account panel verticality tolerances. For maximum thrust in the section, arch 1, AHK. For maximum bending moment, arch 2, AFK.

(b)

Fig. 7.27. Circular diaphragm wall design: calculation of hoop compression and bending stress from a three-pinned arch

Care is needed when using temporary circular stop ends to ensure circular walls of limited diameter remain properly connected at the joints. Flat-section temporary stop ends are difficult to use reliably and Trenchcutter and Hydrofraise type diaphragm wall equipment, which are able to cut concrete in the previous panel to form the joint, are often preferred. Practical difficulties may also be caused by instability of the last panel where the construction is in water-bearing ground. Pore water pressure tends to rise within such an enclosure and temporary wellpointing may be necessary to avoid a differential head of groundwater between the inside and outside of the closure panel.

Sheet pile walls across dock entrances

There are several means of building a cofferdam across the entrance to lock or dock construction to exclude external river or sea water in order to escavate the floor of the dock or replace existing cills and gates. For new construction, cellular cofferdams or twin, parallel-walled cofferdams are frequently used. To exclude water from an existing dock, raking struts to the dock floor or walings from the sides of the dock, with diagonal struts to divide the waling span, are useful options. For an existing dock entrance it is often feasible to span walings across the whole entrance width by driving sheet piles to a circular arc in plan, braced by arch walings of steel or reinforced concrete, using the existing walls at the entrance as 'abutments' to the arch walings.

Estimating costs of temporary cofferdams

Many design curves have been published for cantilever and braced sheet pile cofferdam construction. Those due to Packshaw (Fig. 7.28) show the section modulus of the piling and the number of bracing frames of normal construction in average soil conditions; bending moments induced in cantilever cofferdams for various heights and conditions; and the section modulii, waling loads and maximum penetration depths for one- and two-frame cofferdams in cohesionless soils for various sheeting and braching strengths. The greatest contribution of these graphs may be to prevent serious errors in the estimation of sheeting and bracing requirements during cost estimating by the engineer or contractor.

Double-wall cofferdams

Double wall cofferdams are gravity structures consisting of twin parallel lines of sheeters driven below dredge level, tied together at one or more levels by steel ties and filled with selected material, preferably cohesionless soil. The width-to-height ratio of the structure is at least 0·8, and it is usual to place a berm of granular soil on the inside face to extend the drainage path of water passing beneath the cofferdam in order to avoid piping near the inside line of sheeters. The stability of a double-wall cofferdam depends on the strength of the sheeters and the ties, on the shear strength of the fill material and the soil at foundation level. This type of cofferdam is not suitable where strong bedrock occurs at shallow depth below formation level as it is necessary for the sheeters to penetrate sufficiently to avoid passive failure in front of them. It may be necessary to reduce the level of water in the fill material between the piles to increase the effective shear strength of the fill and reduce the pressure on the sheeters. Submersible pumps may be needed for this drawdown. In any case, it is essential that sluices are provided on the inner line of piles to reduce the level of the phreatic surface as much as possible. Where the length of the cofferdam exceeds its width by four or five times it is usual to drive a cross wall of sheeters to connect the inside and outside line of sheeters. This assists craneage access for construction and reduces the consequences of failure by forming compartments of restricted size.

the effect of water levels caused by passing vessels and the risk of collision by river traffic, must all be considered when assessing this horizontal loading. A concrete slab cover should be provided on the top of the cofferdam where waves can overtop the outside sheeting and increase the water level in the filling.

The possible modes of failure were reviewed in chapter 6. Each of these failure modes is addressed below for the design of double-wall cofferdams.

(*a*) Tie rod design and water pressure. Where the sheeters do not achieve penetration below the minimum required for free earth support, due to earth and water pressures within the retained fill, the structure will act as a gravity structure when loaded with the outside head of water. In this situation the sheet piles should be designed using at-rest pressures for the fill (using the coefficient for each pressure at rest k_0 for the filling) because deformation of the sheeters is restricted by the ties and it is necessary to restrict movement at the head of the cofferdam. Walls should be designed for the most severe assumptions of internal water pressure. Where, for instance, hydraulic filling is used to place sand backfill between the sheeters, the design water level within the cofferdam should be considered *vis-à-vis* the rate of drainage possible from sluices of flap valves on the inside face. In the worst situation, the water level within the filling may reach the level of the top of the sheeters if the rate of pumping the fill is high and the rate of drainage is low.

(*b*) Sliding. Resistance to sliding is provided by the passive resistance of the soil on the inside face of the cofferdam, the shear strength of the sheeters and the frictional resistance beneath the material filling the cofferdam. Referring to a typical cross-section (Fig. 7.29)

$$\text{Resistance to sliding} = P_p + S_1 + S_2 + S_{soil} \tag{71}$$

and

$$\text{Factor of safety against sliding failure} = \frac{P_p + S_1 + S_2 + S_{soil}}{P_A} \tag{72}$$

(*c*) Overall stability of the cofferdam structure. To establish the efficiency of the fill material between the sheeters in resisting any tendency for the top of the cofferdam to fail by moving horizontally, a stability analysis may be made using a force polygon as suggested by Bishop[35] in the discussion to Packshaw's paper. This analysis was first used for a double-wall cofferdam at the entrance to Gallions Lock in London. The analysis expresses the factor of safety against shear failure between blocks of filling (as shown in Fig. 7.30) as the ratio between the tangents of the angle of shearing resistance for the fill and the mobilized angle of shear on the vertical face of the blocks. Friction between the filing and the internal face of the

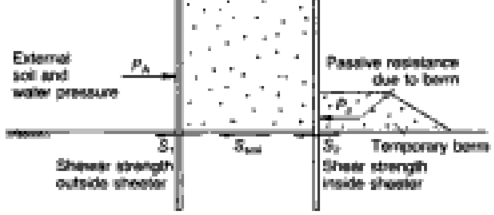


Fig. 7.29. Sliding and resisting forces in double-wall cofferdam, vertical cross-section

sheeters is also taken into account, and the factor of safety against slippage between sheeters and fill is expressed as the ratio of the tangents of the angle of wall friction and the mobilized wall friction as shown on the force polygon. Bishop pointed out that the tie-rod pull did not appear as a term in the analysis because its value is small and it was safe to omit it in examining the factor of safety against shear between the vertical elements. There is no other reason, however, for excluding this term, the value from the analysis of the sheet pile walls being included in the force polygon. Factors of safety of the order of 1·5 would be regarded as satisfactory for this stability analysis.

An alternative method of checking the overall stability of the double-wall cofferdam is described in reference 5. Fig. 7.31 shows the cross-section of a shallow cofferdam founded on soil with free earth support to the outside sheeters. The failure plane at the base of the wall is approximated by a logarithmic spiral centred at point 0. The factor of safety against overturning is then the ratio of overturning and resisting moments about the centre of the most unfavourable failure surface. The minimum factor of safety must be at least 1·5. If this is not obtained in the trial, the required stability may be obtained by increasing the width, improving the quality fo the fill or deeper driving of the sheeters, or a combination of all three. Where

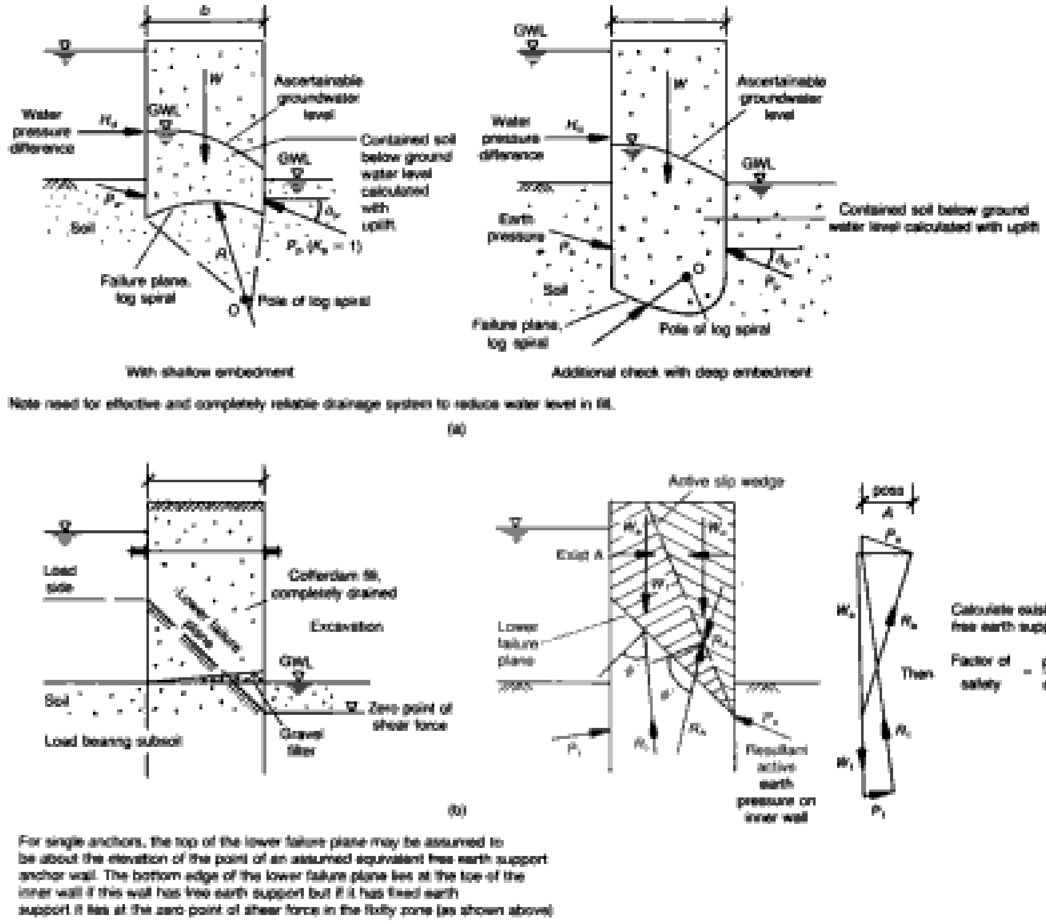


Fig. 7.31. Calculation of double-wall cofferdam stability: (a) cofferdam embedded in load-bearing soil, stability analysis; (b) investigation of the anchorage of the inner line of sheet piles. (German Waterfront Code[5])

(*g*) Scour on the outside of the cofferdam. Where river currents cause risk of scour, the outer face of the cofferdam must be protected to avoid instability of the outer line of piles. This protection may consist of rock or precast concrete blocks placed against the outer face, or grouted mattresses laid on the river bed.

Lateral deformities at the head of a double-wall cofferdam are frequently greater than similar gravity structures when loaded with river water at a high elevation on the outside face. These deformations are associated with mobilization of the shear strength of the cofferdam filling and the friction between the filling and the inside surface of the sheeters. The deflection of the top of the riverside sheeting at the Gallions Lock cofferdam was 350 mm after it had been subjected to the full water load; the height between high water and the inside cill was 12·8 m. A similar displacement was measured at the head of the St. Katherine's Lock cofferdam described in chapter 6. The use of strongpoints, as was shown in Fig. 6.29, reduces such horizontal movements.

Cellular cofferdams

The construction of cellular cofferdams was described in chapter 6. Their use extends to piers, dolphins and breakwater structures, but cellular cofferdams also provide economical soil retention temporarily and permanently in deep, wide excavations and exclusion of river and sea water from deep excavations for lock, gravity dock and similar massive, large plan area structures. These gravity structures, depending on the weight of the retained fill and the tensile strength of the sheeters that retain it, are often used where pile driving conditions preclude deeper sheet piling for braced cofferdam construction and where internal bracing or sheeting is unacceptable or impractical. The most popular plan shapes are circular cells with one or two connecting arcs, and diaphragm cells with outer arcs and straight cross walls. Circular cells consist of independent self-supporting structures, the diameters of which are a function of the interlock strength of the sheet pile section; the greater the support height of soil or water, the larger the cell diameter and, in turn, the greater the pile interlock tension. These factors encouraged the development of a straight web pile with three-point contact at the interlock to give increased interlock tensile strength compared with previous single-point contact interlocks.

Before design can begin the necessary data must be collected. For a cofferdam within a river these would consist of:

- tidal data and rate of flow; and prediction of both during the design life of the cofferdam
- scour behaviour
- river bed profiles
- soil profile and test data to define strength, permeability and consolidation properties
- borrow areas for suitable filling, and spoil disposal areas
- water quality for cofferdams with a long design life
- previous site use and obstructions
- collision risk from river traffic
- river regulations; permission needed for cofferdam construction.

For maritime works, wind and wave data would be needed, including wave height and period, tide dates and storm risk; also ship collision assessment.

The design of cellular cofferdams was reviewed by Dismuke[37] and summarized in BS 6349.[13] Design of a cellular cofferdam should address the following modes of failure (shown in Fig. 7.33):

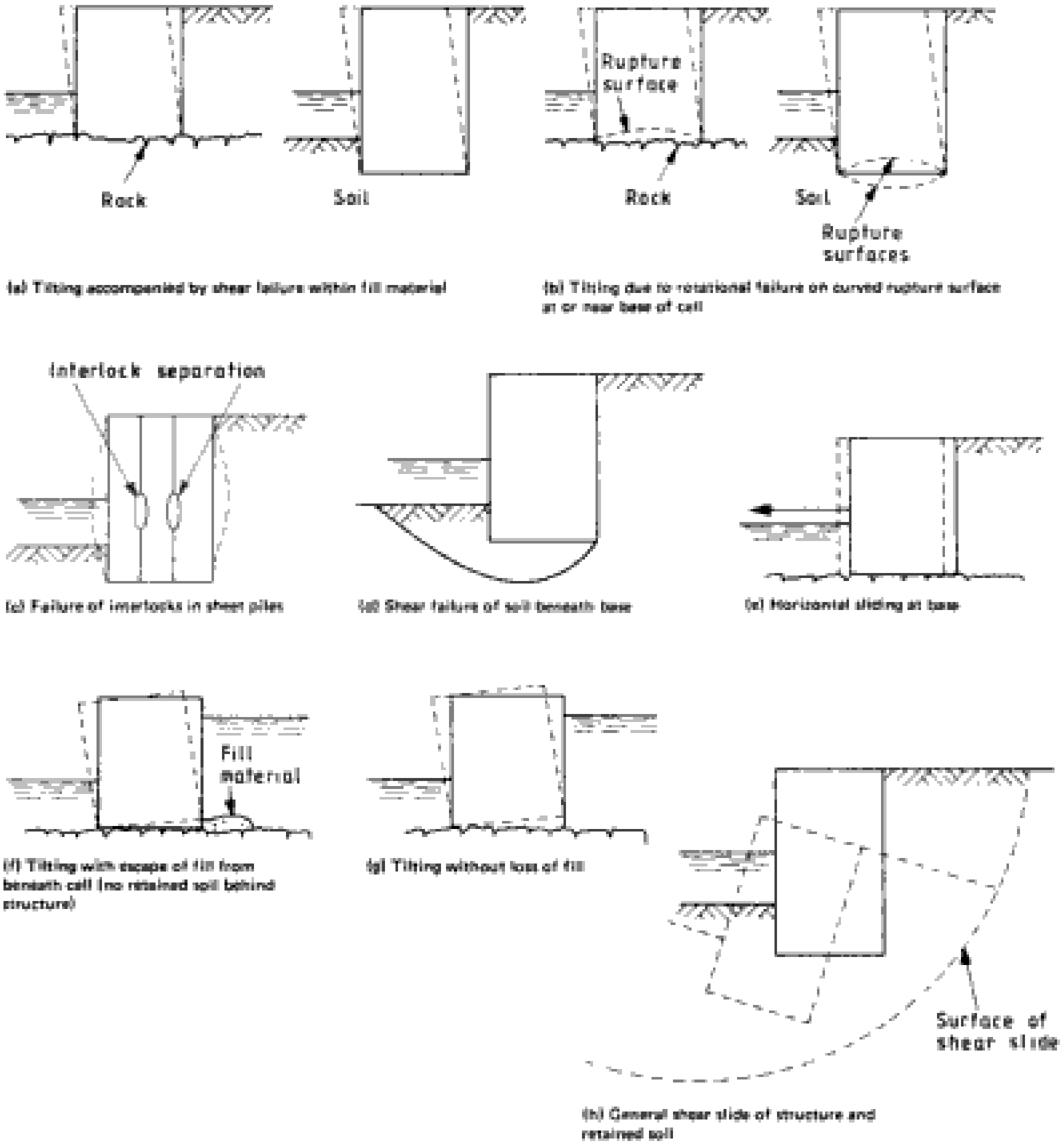


Fig. 7.33. Modes of failure of cellular cofferdams (BS 6349[13])

- excessive tilting or rotational failure on a curved rupture surface at or near the base of the cell: internal stability
- interlock and connection failure
- stability of base and sliding
- loss of cell fill due to piling rise
- overturning.

(*a*) Internal stability. Three methods of analysis of cell stability were compared by Dismuke:

(i) The vertical shear method developed in the late 1930s and 1940s, proposed by Terzaghi[38] and later developed by US Tennessee Valley Authority engineers.[39]

(ii) The horizontal shear method developed by Cummins.[40] This method was introduced because of inconsistencies in the vertical shear method.

The method assumes that horizontal shear planes develop within the cell fill and implies that fill on the unloaded side of the cell could be reduced without affecting stability. This conclusion is not practically sound and should not be used in design to reduce fill levels within cells.

(iii) Methods due to Brinch-Hansen[41] and described in detail by Ovesen.[36] Two variations, known as the equilibrium and extreme methods, led from observations that a circular rupture surface occurred at the base of a model double-wall cofferdam which was loaded to failure: the extreme method is recommended in reference 5 and is the basis of design for cell diameter or width described in the following part of this chapter.

Before describing the extreme method due to Brinch-Hansen it is necessary to refer to the method of changing a cellular cofferdam plan shape to a rectilinear shape to reduce computation. Fig. 7.34 shows show this can be done for cofferdams using circular, diaphragm and cloverleaf cells.

The extreme method assumes that the cell is filled with granular material and is founded on a rock or granular soil base. To simplify the rather complicated calculations of the internal forces on the rupture line at the base of a cofferdam founded on rock, the kinematically-true circular rupture line is substituted by a logarithmic spiral satisfying the polar equation $r = r_o \exp(\alpha \tan \phi)$. Such a spiral has a characteristic that its radius vector at any point makes an angle ϕ with the corresponding normal. In cohesionless soil, with an angle of shearing resistance ϕ, the resultant of all internal forces within the spiral will thus be directed towards the rotation point of the spiral. The method is similar, therefore, to that described for twin wall cofferdams. Thus, this calculation for cofferdams on rock involves the following steps (Fig. 7.35(a)):

(i) Generate log spiral locus line ($r = r_o e^{\alpha \tan \phi}$) for angle of shearing resistance ϕ with spiral through the feet of the walls.
(ii) Compute external and gravity forces (P, γBH, W_p, W_f) and reactions (S_b, S_r).
(iii) take moments about the rotation point of the spiral and find the factor of safety

$$f = \frac{M_{\text{stabilize}}}{M_{\text{disturb}}} \tag{73}$$

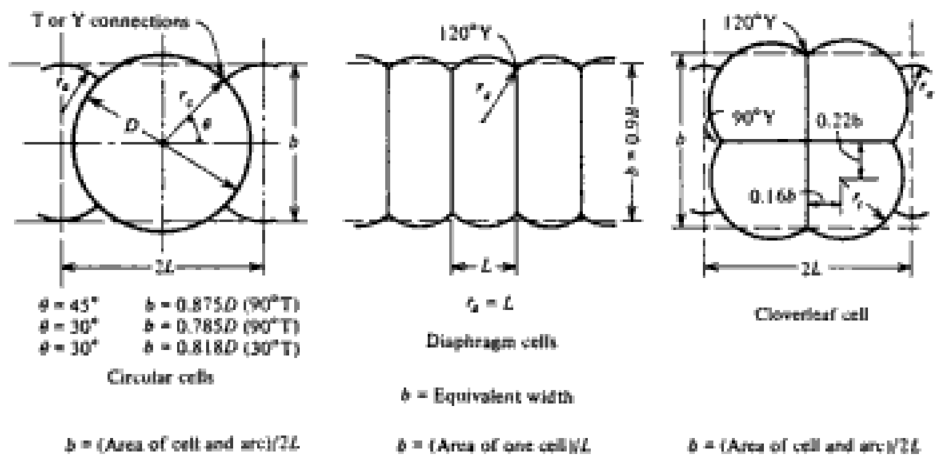


Fig. 7.34. Equivalent rectilinear width and design geometry for cellular cofferdams of circular, diaphragm and cloverleaf cells (Dismuke[37])

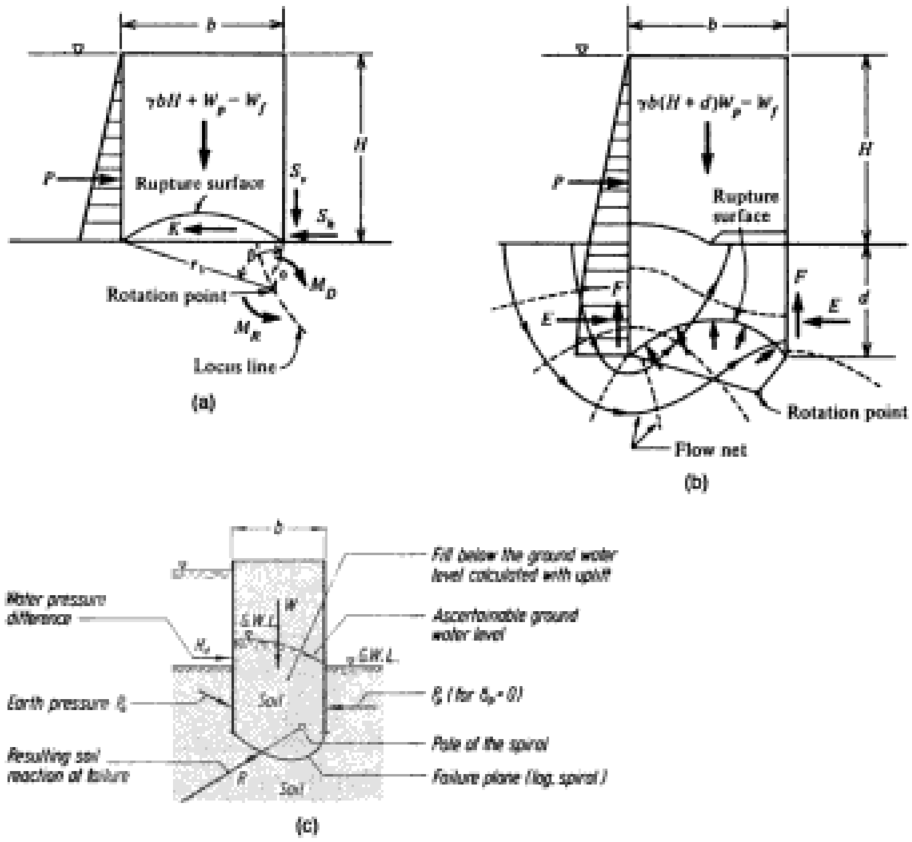


Fig. 7.35. Stability analyses for cellular cofferdams by the extreme method: (a) founded on rock (W_p = weight of piling. W_f = weight of fill below rupture surface); (b) founded on rock overlain by soil; (c) founded on soil, additional check for deep embedment with concave failure surface (GrW = groundwater level)

(iv) Find the critical moment, that is the moment for which f is a minimum, by changing the position of the spiral. If minimum f is greater than unity the cofferdam is stable, although a minimum value of 1·5 is required for design acceptance.

If the cofferdam is founded on rock which is overlain by soil (Fig. 7.35(b)) or if the cofferdam is founded in soil, the disturbing forces are increased by the active earth pressure on the outside of the cofferdam and reduced by the passive earth pressure on the inside. Since deformation will be small it is usual to limit passive pressure to the at rest value $K_o = 1$ for sheeting with shallow embedment, and to calculate K_p setting wall friction equal to zero for sheeting with deep embedment.

Where sheeting is driven deep to provide stability, a check for concave failure planes is necessary in the same way as for twin walled cofferdams (Fig. 7.35(c)). The spiral is then located so that its centre of rotation does not lie beyond the line of action of passive force P_p with the angle of wall friction equal to zero.

(*b*) Interlock and connection forces. The cell hoop force outside the connecting arcs and the hoop stress may be calculated from

$$t_a = p \times r_a \tag{74}$$

and

$$t_{c1} = p \times r_c \tag{75}$$

where t_a is the hoop or interlock force for connecting arcs, t_{c1} is the hoop or interlock force for cells outside arcs, and p is the lateral unit pressure (taken as earth pressure at rest, $K_o = 1 - \sin \phi$, at the base of the excavation).

Dismuke[37] pointed out that the greatest interlock force, located just inside the arc connection, is frequently overlooked. The cell hoop force at the arc connection is

$$t_{c2} = pL \sec \theta \qquad (76)$$

where t_{c2} is the circular cell hoop or interlock force between arcs, L is half the centre-to-centre distance between cells, and θ is the angle between the centre-lines of the cells and a line from centre of a cell to the point on cell periphery where the arc connects.

The relative hoop forces, at any level in the cell, are shown for circular and diaphragm cells in Figs 7.36 and 7.37, respectively.

Arc connections between circular cells and diaphragm cells are the most highly stressed part of the sheeting and are the principal point of failure risk. The connections are usually made through T and Y junction piles. Dismuke indicated the theoretical direction of loads acting on T and Y

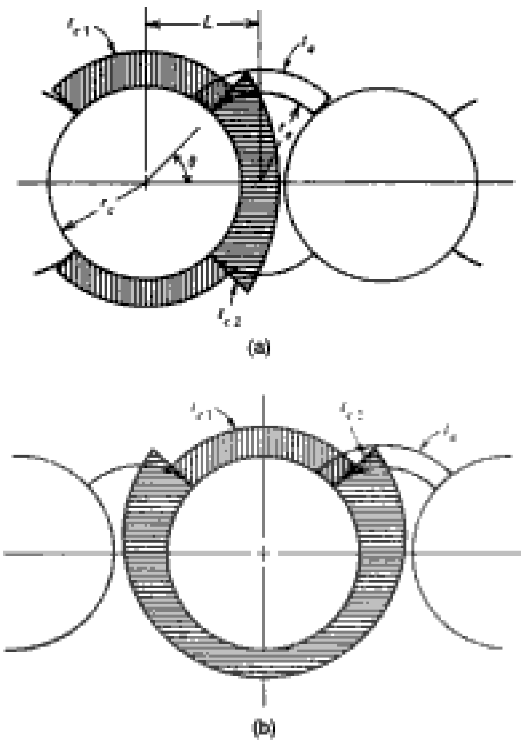


Fig. 7.36. Calculation of hoop forces in circular cofferdam cells: (a) circular cell cofferdam; (b) circular cell bulkhead (Dismuke[37])

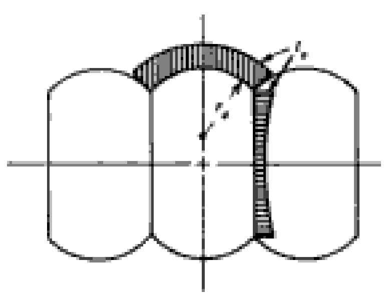


Fig. 7.37. Calculation of hoop faces in diaphragm cells (Dismuke[37])

slippage of the sheet piles, loss of interlock friction normally maintained by tension at the interlock could occur.

Figure 7.39 gives expressions for assessing the bearing capacity of the soil below the cofferdam, treating the cell as a rigid body with uniform base pressure. Fig. 7.39 also gives the method for calculating the factor of safety of internal instability due to settlement where the base soil is a compressible clay. The factor of safety against sliding is

$$f = \frac{W \tan \phi' + P_p}{P} \tag{77}$$

where W is the weight of fill and piling, ϕ' is the angle of shearing resistance of cell fillings, P_p is the passive resistance at the inner line of the piling, and P is the lateral force from soil and water on the outer line of the piling.

A value of 1·25 to 1·3 would be considered satisfactory for temporary works. It is not usual for sliding to be a critical mode of failure for cells, except for those founded at shallow depth on rock.

(*d*) Piling pull-out. There is risk to sheet piles on the outside of cells of pull-out as a result of overturning moment due to lateral loading by soil and water. If this occurs, cell fill may be lost and the cell would fail if the quantity were large. For cells founded on rock, the factor of safety against piling rise is

$$f = \frac{b(P_w + P_a)f_{sp} + P_p \dfrac{H_b}{3}}{P_w \dfrac{H}{3} + \dfrac{P_a H_s}{3}} \tag{78}$$

where b is equivalent cell width, f_{sp} is the coefficient of friction between the cell filling and sheet piling, H_b is the berm height, P_w is the lateral force due to external water pressure, P_a is the lateral force due to external active soil pressure, P_p is the lateral force due to passive resistance due to the berm, H is the height of the water head above formation level on the outside face, and H_s is the overburden height on the outside face. Values between 1·25 and 1·3 would be acceptable for temporary works.

For cells founded pile pull-out is the factor of safety against sands and clay

$$f = \frac{\text{Resistance to pull-out per unit length of cofferdam}}{\text{Pull-out force per unit length of cofferdam}} = \frac{C_p}{F_p} \tag{79}$$

The pull-out force F_p is (Fig. 7.40)

$$F_p = \frac{P_w H + P_a d - P_p H_B}{3b\left(1 + \dfrac{b}{4L}\right)} \tag{80}$$

where d is the pile embedment below formation level, and L is the cell module (see Fig. 7.34).

The resistance to pull-out C_p for cells on sand bases is

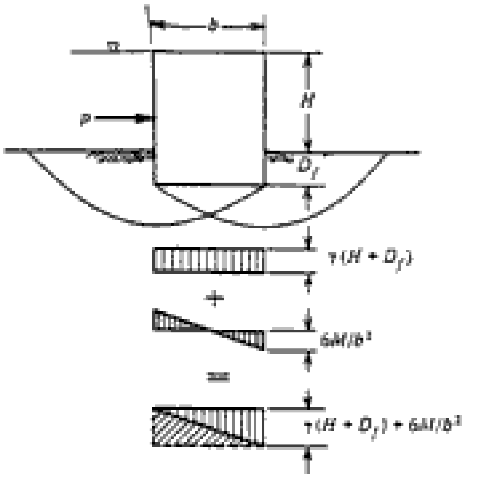
$$C_p = \tfrac{1}{2} k_a \gamma d^2 \tan \delta \; 2\pi r_c \tag{81}$$

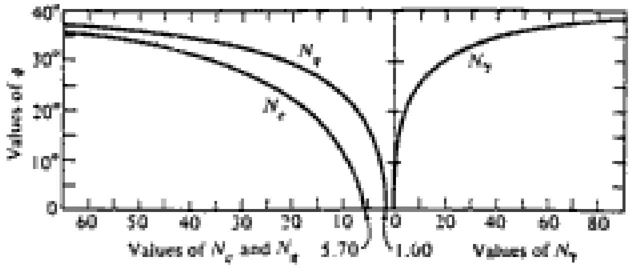
On clay bases

$$C_p = c_a \, 2\pi r d \tag{82}$$

where c_a is the adhesion of clay to sheet piles. The factor of safety should be less than 1·25 for temporary works.

(*e*) Overturning. If the cell is assumed to rotate about its toe, in a similar way





Bearing Capacity Factors (Terzaghi and Peck, 1967)

Ultimate Bearing Capacity

$q_f = 1/2\, b\gamma N\gamma + CN_c + \gamma D_f N_q$

for strip loaded area

$q_f = 0.6\, \gamma b N\gamma + 1.3\, N_c + \gamma D_f N_q$

for circular load areas

Note: When $D_f = 0$, the critical condition exists.

where

γ = Unit weight of soil around cell (pcf)
b = Equivalent cell width (ft)
N_γ = Bearing capacity factor
C = Cohesion (psf)
N_c = Cohesive factor
D_f = Ground surface to toe of cell (ft)
N_q = Surcharge factor

$$F.S. = \frac{q_f}{\gamma(H + D_f) + \dfrac{6M}{b^2}} \quad \begin{matrix} \geq 2.0 \text{ for sand} \\ \geq 2.5 \text{ for clay} \end{matrix}$$

Internal instability due to settlement of compressible base (for soft and medium clays, q_u = 400 psf to 1000 psf)

$$F.S. = \frac{(P_p - P_s)\,(D/2)\, f_{ss}\,(b/L)\left(\dfrac{L + .25b}{L + .5b}\right)}{M} \quad \begin{matrix} \geq 1.25 \text{ (temporary)} \\ \geq 1.5 \text{ (permanent)} \end{matrix}$$

where

P_s = Inboard pressure
$_p$ = Passive pressure of berm and/or overburden on inside of cofferdam
f_{ss} = Coefficient of friction steel on steel
M = Overturning moment
D = Diameter
b = Equivalent width

Fig. 7.39. Stability of base soils below a cellular cofferdam (Dismuke[37])

to a gravity wall, the resultant moment due to cell weight and lateral forces is restricted to the middle one-third of the cell width. The factor of safety against overturning is

$$f = \frac{M_{\text{restoring}}}{M_{\text{disturbing}}} = \frac{(Wb)/2}{(PH)/3} \tag{83}$$

Design of gravity cofferdams

Precast concrete blockwork can provide weight stability for cofferdam structures and is economical in circumstances where the temporary works has a long design life and where alternative means of construction may be unavailable. Blockwork construction is traditional for quay walls and both horizontal and inclined bedding units have been used extensively. Similar construction methods can be used to

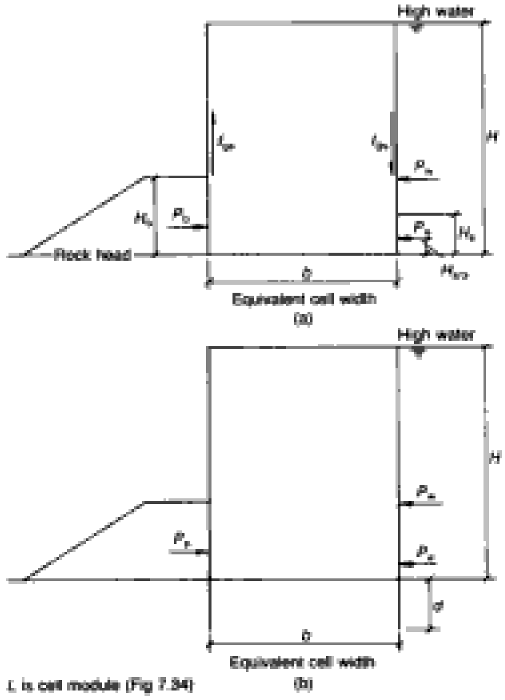


Fig. 7.40. Piling rise and pile pull-out calculations for circular cofferdams: (a) founded on rock; (b) founded in sand and clay

exclude water from a deep excavation in a river bed. Blockwork cofferdams find particular application at the entrances to docks and locks where excavation and maintenance work is required to the floor of the existing facility. Their use is, of course, limited to sites where adequate load bearing soil is present at reasonably shallow depth, or where the existing soils can be improved in situ at low cost by such methods as vibro-compaction or dynamic consolidation.

The dimensions and weight of the blocks will depend on the head of river water to be excluded from the excavation and the plant, labour and material resources available at the site. Access to the site of the cofferdam and working areas may favour or preclude the use and transport of heavy blockwork.

The blocks themselves should be as durable and watertight as possible with a strong dense concrete mix. They may be cuboid or wedge shaped but in all cases should include a key to ensure interlock between adjacent blocks.

The prepared soil bed on which the initial course of blocks is laid should be at least 0·5 m thick, and consist of crushed graded rock accurately levelled with the assistance of divers. In fine-grained subsoil it will be necessary to provide a soil or geotextile filter between the subsoil and the bedding to the units. To improve the stability of the blockwork against overturning and sliding it will frequently be necessary to construct a berm on the inward side of a blockwork wall built to exclude river water. At the same time this reduces the risk of hydraulic failure at the inside of the cofferdam by lengthening the drainage path of water flowing under the blockwork. A filter blanket would be incorporated at the base of the berm.

The design of a blockwork cofferdam, like other traditional gravity structures, consists of checks on sliding, overturning and bearing values, and restricting the resultant moment due to lateral forces and the weight of the blockwork to within the middle-third of the base width to avoid tension on the lowest course of blocks. These checks are made, course by course, from the top of the cofferdam to the underside of the initial course of blocks.

The head of water to be supported by the blockwork will be equal to the high tide level at a spring tide plus an allowance for wave height. An allowance for

8 Basement construction and design

Engineering and excavation

A number of factors control the relative difficulty of basement construction. Very often these factors cannot be changed by the designer, and include the location of the building, the proximity of existing buildings and services, previous site use, and the proposed use of the basement together with soil and groundwater conditions. The basement structure will be designed to overcome these constraints to transfer the loads from the superstructure to the subsoil. The method of basement construction and the type of peripheral basement wall will be selected to support soils and groundwater at the curtilage of the basement as economically as possible. The permitted soil deformation around the basement construction has to be assessed and complied with.

The process was itemized by Lambe[1], as shown in Table 8.1. The penultimate step, the splitting of responsibility for the stability of the works and the movements caused by the works, should perhaps be considered earlier in some projects. This division of responsibility, often between engineer and contractor, is not easily defined, especially where wall construction processes such as diaphragm walling and secant piling provide temporary and, subsequently, permanent soil support from the same wall element.

Increasingly clients and architects are demanding larger and deeper basements. This chapter reviews the development of construction methods and the range of basement walling methods available, and describes the design problems that arise.

Construction methods for soil support

Seldom does the location of a basement allow open battered excavations. Particularly on urban sites, space is limited and insufficient to accommodate the cut slopes of battered excavations; land is expensive and basement constructions inevitably occupy as much of the site as possible. The use of open excavations was reviewed in chapter 3, although mention will be made here of the need to review soil strength parameters critically for temporary cut slopes.

In certain soils, over-consolidated clays such as London clay for example, the soil strength characteristics are time-dependent, so the period for which the excavation is to be kept open must be carefully assessed. Where space allows the use of battered slopes, the cost penalty of a slope failure should be weighed against the cost of a full soil retention system using temporary walling. It is possible that a compromise solution, using soil nailing or similar ground improvement methods incorporating Reinforced Earth, may be economically attractive where some horizontal working space is available at the rear of the basement construction but is not sufficient to accommodate a full battered slope. Where some space exists behind the permanent basement wall the choice of method will be determined in permeable soils or granular soils by the extent of groundwater flow and the feasibility and cost of controlling groundwater during basement construction.

An example of a battered basement excavation with a slurry trench cut-off to control groundwater inflow and cut slopes designed on a cost against risk basis was given by Wakeling.[2] The excavation was 130 m × 80 m in plan, to a depth of 5·8 m in soft clay and gravel, extending into stiff fissured silty London clay

Table 8.1 Engineering on excavation: a checklist (Lambe[1])

Step	Activity	Considerations
1	Explore and test subsoil	
2	Select dimensions of excavation	Structure size and grade requirements, depth to good soil, depth to floor requirements; stability requirements
3	Survey adjacent structures and utilities	Size, type, age, location, condition
4	Establish permissible movements	
5	Select bracing, if needed, and construction scheme	Local experience, cost, time available, depth of wall, type of wall, type and spacing of braces, dewater excavation sequence, prestress
6	Predict movements caused by excavation and dewatering	
7	Compare predicted with permissive movements	
8	Alter bracing and construction scheme if needed	
9	Instrument — monitor construction and alter bracing and construction as needed	

to a maximum depth of 14·5 m. A bentonite slurry cut-off wall into the London clay contained groundwater in the upper gravels. Groundwater flow from the gravel and the underlying silty sands was controlled by gravel-filled counterfort drains dug down the slope during bulk excavation. The excavation was battered with side slopes of 1:1 with an intermediate berm at the top of the London clay. Plan and cross-section of the excavation are shown in Fig. 8.1. The method was successful and demonstrates the use of soil parameters based on partially-drained soil conditions. Wakeling reported that three slips occurred, all shallow-seated, in the batters within the London clay, reaching their greatest depth between two and five months after excavation. From back analysis on these slopes and using shallow-seated slides reported by Skempton and La Rochelle,[3] Wakeling reached a tentative conclusion: for excavations in stiff fissured clays, short-term shallow-seated slips are likely to occur when the computed failure strength exceeds the measured undrained shear strength in the clay by approximately 20%. The point of interest in this example is the cost-effectiveness of a relatively steep slope batter of 1:1 with some risk of minor failure accepted during a relatively short construction time. The use of fully drained parameters in the slope analysis would have led to flatter slopes, albeit with less risk of slippage.

Where compromise solutions to peripheral soil support are required, that is where some space exists at the rear of the permanent basement structure but is insufficient to accommodate a battered slope, crib walls and anchored crib walls can be used. A further solution is the use of soil nailing. These methods were discussed in general terms in chapter 4, but the relevance of soil nailing to basement construction deserves discussion here.

The soil nailing method developed from the use of fully-bonded rock bolts for tunnel support in the 1950s and 1960s. Using the same principles of ground support its use progressed from weaker rocks, such as marls and weak sandstones, into cemented sands, strong clays and, later, to a wider range of granular soils and middle-strength cohesive soils. The range of soils in which nailing can be used is relatively wide (weak clays and loose silts are probably precluded, and the presence of groundwater limits its application in any soil). Although finding widespread application for general soil support in basement schemes in France, Austria, Germany and North America, its application in the UK has been slow

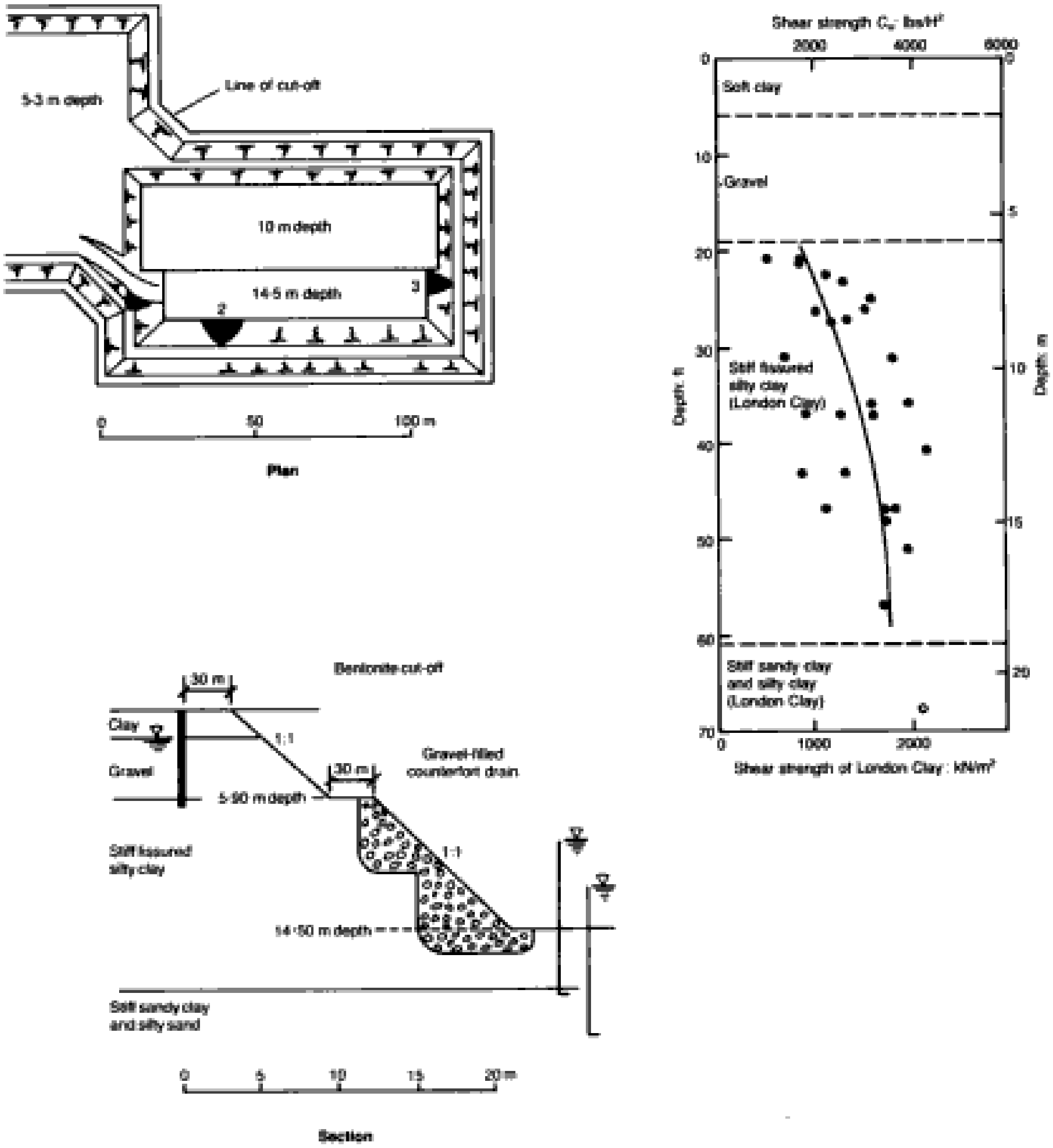


Fig. 8.1. Plan and cross-section of a battered deep excavation into London clay (Wakeling[2])

and only a few jobs have been completed. This patchy response is probably due to market conditions and is not a technical matter.

Soil nailing is in fact a reinforced earth technique and uses short tendons, driven or inserted into short bores in the excavated soil face, to improve the shear strength of subsoils. The exposed is retained and protected by a gunite layer reinforced with a wire mesh. The technique is described by Gassler and Gudehus[4] and Banyai,[5] but a complete description of its development — the soil–tendon interaction, design, construction and specification — is given in the report of the French Clouterre project.[6]

Typical cross-sections of soil-nailed excavations were reported by Barley[7] and

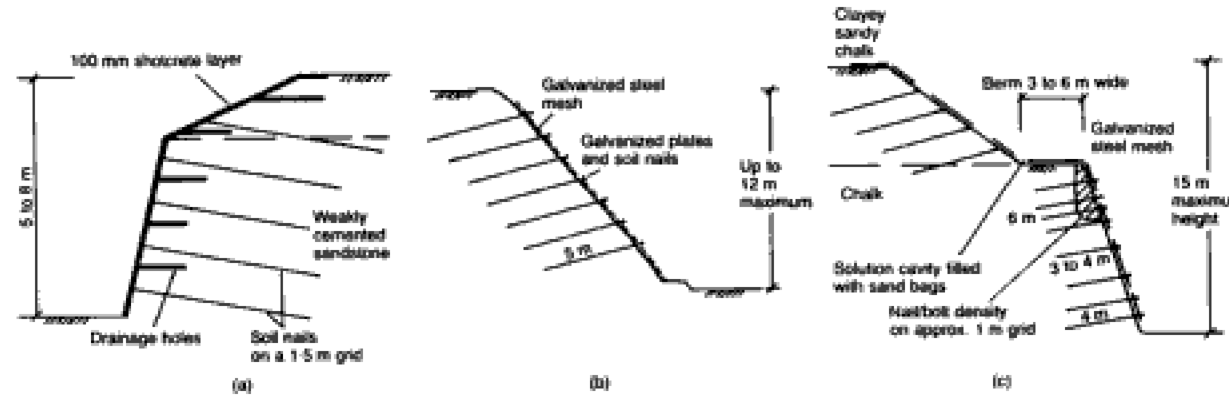


Fig. 8.2. Cross-sections of typical nailed slopes in the UK: (a) temporary soil-nailed slope; (b) soil-nailed slope; (c) rock-bolted and soil-nailed slopes (Barley[7])

are shown in Fig. 8.2. A soil-nailed excavation support in Pocking, Bavaria, is shown in Fig. 8.3.

The application of soil nailing to retain excavated slopes at the periphery of basements is referred to here as temporary support, but soil-nailed slopes can also be used for permanent works. Tendon durability and protection was discussed by Barley[7] and in the Clouterre report.[6] The use of alternative reinforcing materials such as glass-reinforced plastics may lead to an increase in permanently retained soil-nailed excavations.

The extent of working space at the rear of the permanent retaining wall is likely to reduce the nearer the basement site is to a city centre. In the remainder of this chapter, it is assumed that such space is limited. Soil support systems which incorporate both temporary and permanent support are likely to prove most efficient in minimizing the total width of soil support wall. Alternatively, the construction of sloping sheeting can lead to economics in construction cost where limited working space is available. Schnabel[8] reported that where sheeting sloped at an angle of about 10° from the vertical, the measured sheet loads were consistently less than two-thirds of the computed sheet loads for vertical sheeting in the same soil. A further study[9] of sloped sheeting supported by ground anchors presented model tests results in sand, confirming Schnabel's recommendations for soil pressures on an inclined anchored wall. It was noted from these model tests that inclined walls require a considerable base width if they are not to suffer a bearing capacity failure.

Underpinning in short lengths may prove necessary to avoid settlement of adjacent structures during basement construction. In dry soil conditions, where the water table lies beneath basement formation level, it may be sufficient, and expedient, to rely on the underpinning to provide horizontal soil support during basement construction in addition to its main purpose of vertical load transfer to depths below the new basement construction. Unless the underpinning is braced or propped from the excavation side its depth will be restricted in either concrete or grouted soil because of horizontal soil pressure at the rear of the underpinning.

Where ground conditions allow successive excavation in the dry an anchored reinforced concrete plate can be used to provide a continuous reinforcement wall at the periphery of the new basement. These ground conditions may be obtained by grouting in certain soil conditions, given legal consents. Fig 8.4 shows the plate method being used in a base extension in Zurich. Each cast in situ element was retained by ground anchors, excavated alternately at each level. The subsoil was a cohesive silty sandy gravel.

Fig. 8.3. Soil-nailed slope at Pocking, Bavaria (courtesy of Bauer)

Fig. 8.4. Plate method being used for a basement extension to the Technical University, Zurich (courtesy of Bauer)

The effectiveness of lateral support to a deep excavation thus turns on six factors: neighbours' rights, neighbouring construction, subsoil and groundwater conditions, neighbouring services, and the proposed construction depth and optimization of site area to give the best financial return.

In complying with these factors the majority of urban basements sites will not allow battered open excavations due to space limitations. Vertical peripheral soil support is therefore required, temporarily during construction and as a permanent retaining wall. During construction the simplest form for either sheeting or walling is to cantilever without propping. In typical basement excavations in London the maximum height of cantilever is generally of the order of 5·5 m from formation level. The extent of soil movement during and after bulk excavation, and the presence of delicate services or important highways at the rear of the wall, mitigate against the use of high cantilevers. Temporary berms at the front of the cantilevered wall reduce soil movements effectively but are unpopular because of the need to remove the berm successively in short lengths and small volumes. Although propped cantilevers provide more security against excessive wall movement, the cost penalties of providing this support and the obstruction to bulk excavation often proves unacceptable. In contrast, the economical use of peripheral steel sheet pile propped by steel raker tubes from a completed central raft construction with a temporary edge berm is shown in Fig. 6.1. A cost comparison on that particular site showed little difference between propped steel sheet piling with in situ permanent retaining walls and propped diaphragm wall construction.

Figure 8.5 shows why the use of temporary soil berms to reduce cantilever wall movement is unpopular with contractors. The basement, in West London, was large in plan area but limited in depth to 5·625 m from existing ground level. Maximum horizontal wall deformation was specified as 25 mm and ground conditions were medium dense sands and gravels overlying London clay. Two schemes were prepared by a specialist diaphragm contractor, the first with a temporary soil berm and minimum wall depth, the second with a free-standing wall of greater depth. Although the analysis showed a signficiant beneficial effect on wall deformation of providing a relatively small berm, the main contractor preferred the deeper wall without the berm, shown in Fig. 8.5(c). The cost of

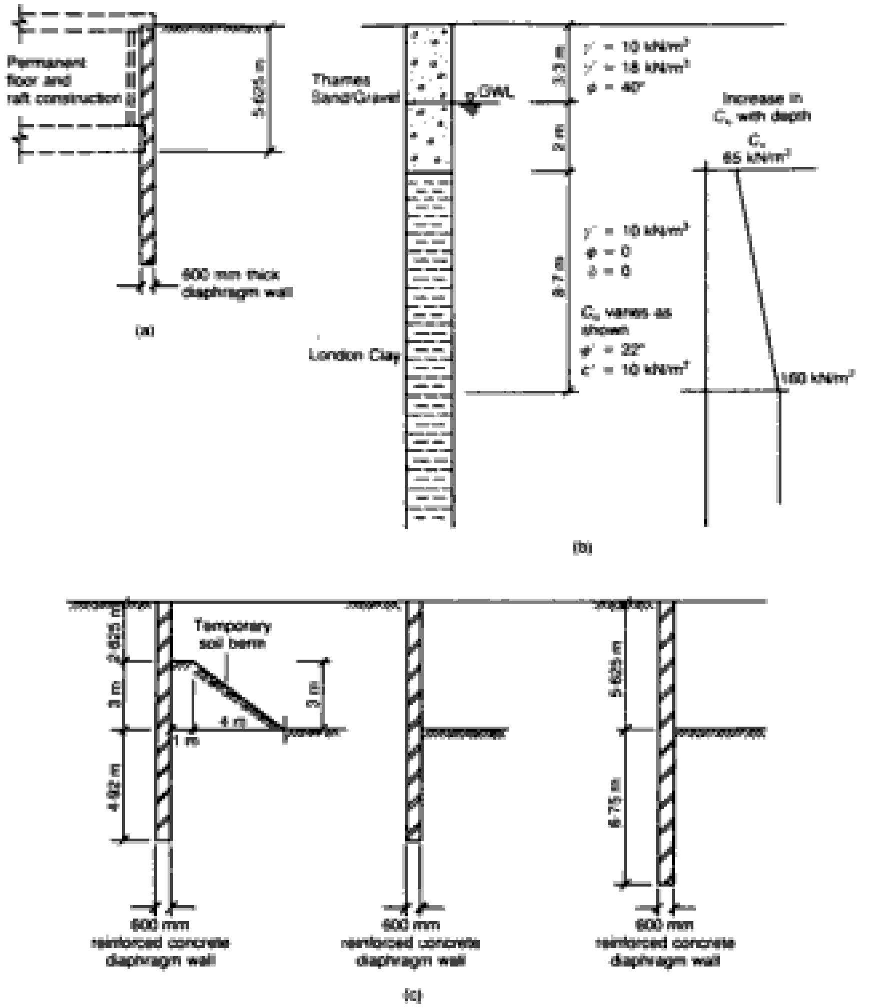


Fig. 8.5. Cantilever wall, West London: (a) wall cross-section; (b) soil profile; (c) calculated maximum deformation with and without soil berm

the additional walling was significant but avoided later excavation of small soil volumes and impediment to the base raft construction programme. During construction the maximum deformation of the cantilever wall without the berm was 19 mm.

Using finite element methods and assuming linear elastic perfectly plastic soil material (with the effects of enhanced stiffness at small strains), Potts *et al.*[10] reached a number of conclusions on the effectiveness of berms: for berms between 2·5 and 5·0 m high it is the volume of the berm, not its specific geometry, that dictates soil movements adjacent to the excavation, wall deformation and bending moments; as the height of the berm reduced below 2·5 m, berms of equal volume, but varying geometry result in different wall deflections and moments — deflections increase and the berm becomes less efficient.

To avoid the obstruction of temporary berms and rakers during construction, soil anchors may be used to comparatively shallow depth in some basements. This solution depends on the suitability of soil conditions for anchoring and the legal and practical implications of founding anchors outside the curtilage of the site.

Progressive development of construction methods for deep basements

As basement excavations increase in depth, excavation methods have become more complex, leading to top-downwards techniques which allow simultaneous

the completed wall; all frame levels, reinforcement splicing levels, concrete lift heights and restrutting levels were interdependent.

Peripheral walls propped by floor and raft

In the late 1950s the trench method began to be replaced by more efficient basement excavation methods. The changes exploited large diameter bored piles which were introduced into the UK at that time together with the simple innovation of using the horizontal strength of floors and raft sections as deep beams to span the length of breadth of the excavation. The Fu Centre, Hong Kong (Fig. 8.7) was built with a pile wall, but the base of the cantilever reinforced concrete wall was designed to span horizontally and resist all horizontal earth pressure, allowing the removal of temporary support without inducing purely cantilever moments into the wall.

In the tower block basement of the Hilton Hotel, London (Fig. 8.8) two waling beams, each forming part of a structural floor, were used to temporarily support the outer contiguous bored pile wall. The walings were designed as portal frames and the upper waling was supported at the bored pile wall and by bored piles inside the wall.

In the third phase of the development of this method, the Royal Garden Hotel site (Fig. 8.9) used diagonal struts, also supported on piles, to reduce the 72 m span of the longest side of the basement. Later, peripheral diaphragm walls propped by successive floors, designed to span as a horizontal frame, were used at Gardiner's Corner, London (Fig. 8.10).

Top-downwards construction

The fourth and fifth phases of the development of the basement excavation method are illustrated by a car park at Leicester Square and the Winter Gardens Theatre, both in London (Figs 8.11 and 8.12).

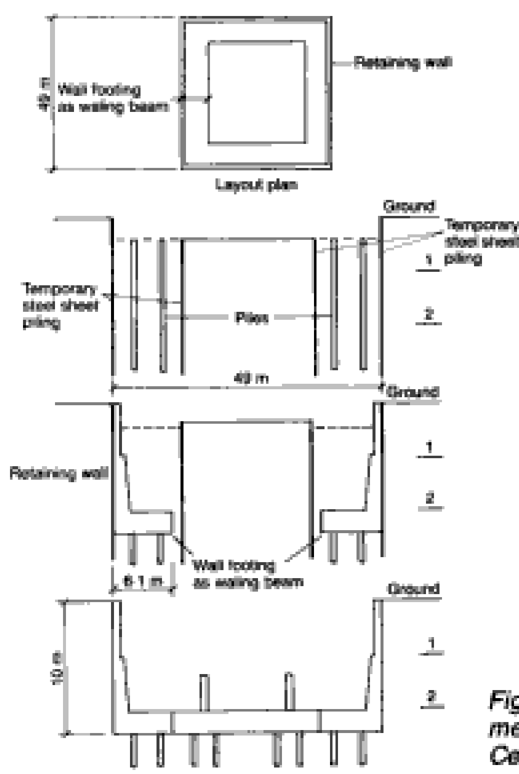


Fig. 8.7. Stages of basement construction, Fu Centre, Hong Kong (Zinn[12])

Fig. 8.10. Top-downwards basement construction with floors used as horizontal frame, Gardiners Corner, London (courtesy of Cementation)

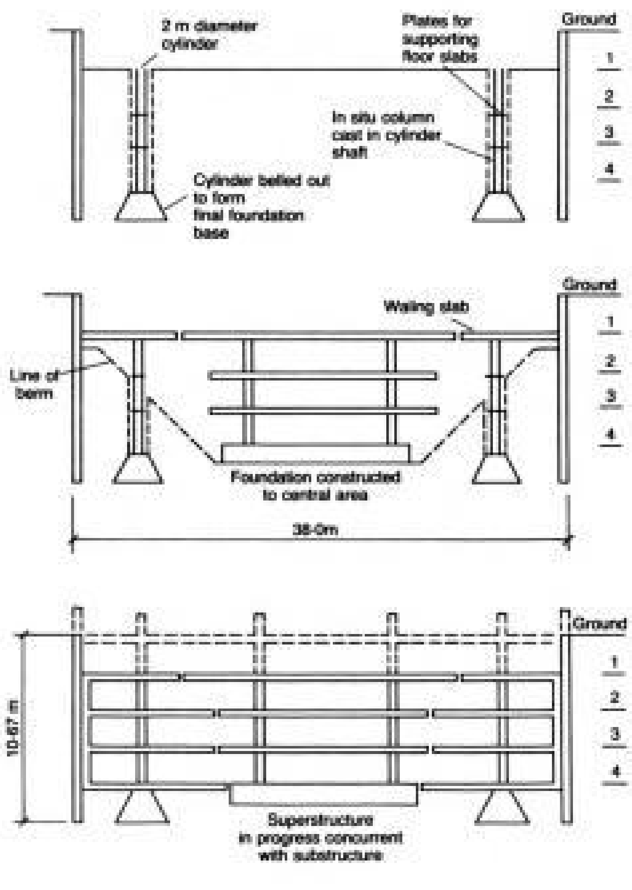


Fig. 8.11. Stages of car park basement construction, Leicester Square, London (Zinn[12])

use before the completion of the substructure. In 1972 the House of Commons underground car park in London was built with peripheral diaphragm walls with temporary support from floor slabs cast successively with continuing excavation. The basement, shown in section in Fig. 8.13, reached a maximum depth of 18·5 m with diaphragm wall 30 m deep. The prime concerns at design stage were to minimize soil movements due to the bulk excavation and limit the effect on nearby historic buildings. The risk of soil heave was more acute since there was no superstructure above the basement. The solution, to build a relatively stiff wall (1 m thick diaphragm) propped at relatively small centres (storey heights) with relatively stiff propping from in situ reinforced concrete floors was, therefore, designed to reduce the risk of settlement of existing structures rather than to reduce construction time. A detailed description was given by Burland and Hancock[15]; details of soil movements, which caused only minor cracking and movement to the adjacent buildings, were given by Burland *et al.*[16]

A more recent deep basement construction in London was described by Marchard.[17,18] This basement, constructed by top-downwards techniques to a depth of 23·9 m from ground level to the lowest basement formation level, was built for car parking below an eight storey office block superstructure. The basement is one of the deepest in London. Two details are worthy of note: firstly, precast concrete stop ends were used in the 1 m thick diaphragm wall construction and, although generally successful, Marchand commented:

> some of the joints between the precast stop ends and in situ concrete leaked and this was dealt with by grout injection. The sealant used has been specially developed for the mining industry and is pumped in as a fluid which changes to a flexible mass of matted rubber particles. This material can then flex without cracking. In a few places at low level clay had adhered to the stop end, leaving a strip of clay up to 70 mm wide between adjacent panels. This was raked out to a depth of 150 mm and made good in order to provide a waterproof joint.

Waterproofness of diaphragm wall basements is discussed later in this chapter. Where basement walls built by pile or diaphragm wall techniques remain unlined, the longevity of remedial measures to ensure acceptable waterproofness remains a matter of concern.

The second noteworthy innovation was the use of five rows of pin piles installed in front of the basement wall to stiffen the London clay and prevent softening with time. With increasing groundwater levels the construction would otherwise have required a substantial ground slab to prop the wall and prevent passive failure of the wall. The stiffened soil approach allowed the wall toe to be raised 5 m from the original design, a significant reduction in a very deep wall, originally almost 38 m deep from ground level.

In the 1960s top-downwards construction techniques required the setting of steel columns as part of the superstructure support within pile heads or pile caps at basement formation level. This operation required personnel to trim pile and set precast caps or make bases in situ for the column installation at final basement level. On this contract, a scheme for setting the steel columns directly into the wet pile concrete was investigated but rejected because of the risk of inaccurate placing of the columns. Liners, 21 m long, were used to gain access to each pile head. The operation was costly and time-consuming and was frequently underestimated in terms of construction time.

More recently a patented device has been perfected to enable the steel column to be placed very accurately both in verticality and position within an unlined box supported by bentonite slurry. The development of this jig therefore permits the use of slurry-support and allows the steel column to be placed in the wet concrete of the recently concreted pile.

The top-downwards method has obvious advantages in terms of soil movement

and completion time, but important disadvantages include the additional cost of excavation and removal of soil from beneath floor slabs in cramped conditions compared with conventional open excavation methods. Also, there is the congestion caused on site by superstructure and substructure contractors working within the same programme period.

Outside Europe the top-downwards method appears to have been adopted more recently. American practice is described by Fletcher *et al.*[19] The excavation of a large four-storey deep basement for the Milwaukee Centre, close to historic structures and within 3·5 m of the Milwaukee River, demanded a cut-off and control of groundwater, minimum soil movement and early completion. A major bracing or raker system was judged to be too cumbersome and costly and a temporary freeze wall system was dismissed because a permanent ground water cut-off would have been necessary. The top-downwards method with a deep diaphragm wall as a cut-off was adopted and proved successful.

Scope for innovation remains; in 1993 a contract for an opera house in Paris used the technique for a 28 m deep basement with anchored support in lieu of lateral support from basement floors as excavation proceeded. Large barrette sections were constructed for superstructure support. A typical cross-section of the basement is shown in Fig. 8.14. Due to the planned construction, after completion of the basement, of a Metro running tunnel on one side of the basement and in close proximity to it, anchor tendons constructed from glass fibre were adopted to avoid obstruction to the Metro tunnelling machine. The brittle, low shear strength of the glass tendons ensured that they would be easily removed by the tunnelling machine.

Peripheral sheeting or walling

The system adopted to sheet or wall the periphery of the excavation will be influenced by the choice of basement construction method, the suitability of ground and groundwater conditions, the need to build close to site boundaries and minimize wall thicknesses and, not least, by the local availability of materials and specialist plant and equipment. Peripheral sheeting methods available for sites in the UK are, typically,

- anchored underpinning: reinforced concrete plates and grouted soil
- king post or Berlin wall: vertical soldiers and horizontal laggings or reinforced concrete skin wall
- sheet piling
- contiguous bored piling
- secant piling
- soldier pile tremie concrete method (SPTC)
- diaphragm walls: reinforced concrete cast in situ
- diaphragm walls: precast reinforced concrete
- diaphragm walls: post-tensioned.

The general features of each method were reviewed in chapter 4, but their particular application to basement works is reviewed below.

Anchored underpinning

Where the total excavation depth of basement work is typically in the range 8 to 12 m and ground conditions are dry and capable of supporting a face 1·5 to 2 m deep and of similar length, the anchored plate method provides an economical temporary wall support if permission to install anchors outside the curtilage of the site is forthcoming. In conditions where soils lack the strength to stand unsupported to these modest depths, pre-grouting may prove worthwhile in granular soils. Where foundation loads from adjacent structures are such to necessitate transfer of load below the proposed excavation depth, pre-injection of the subsoil

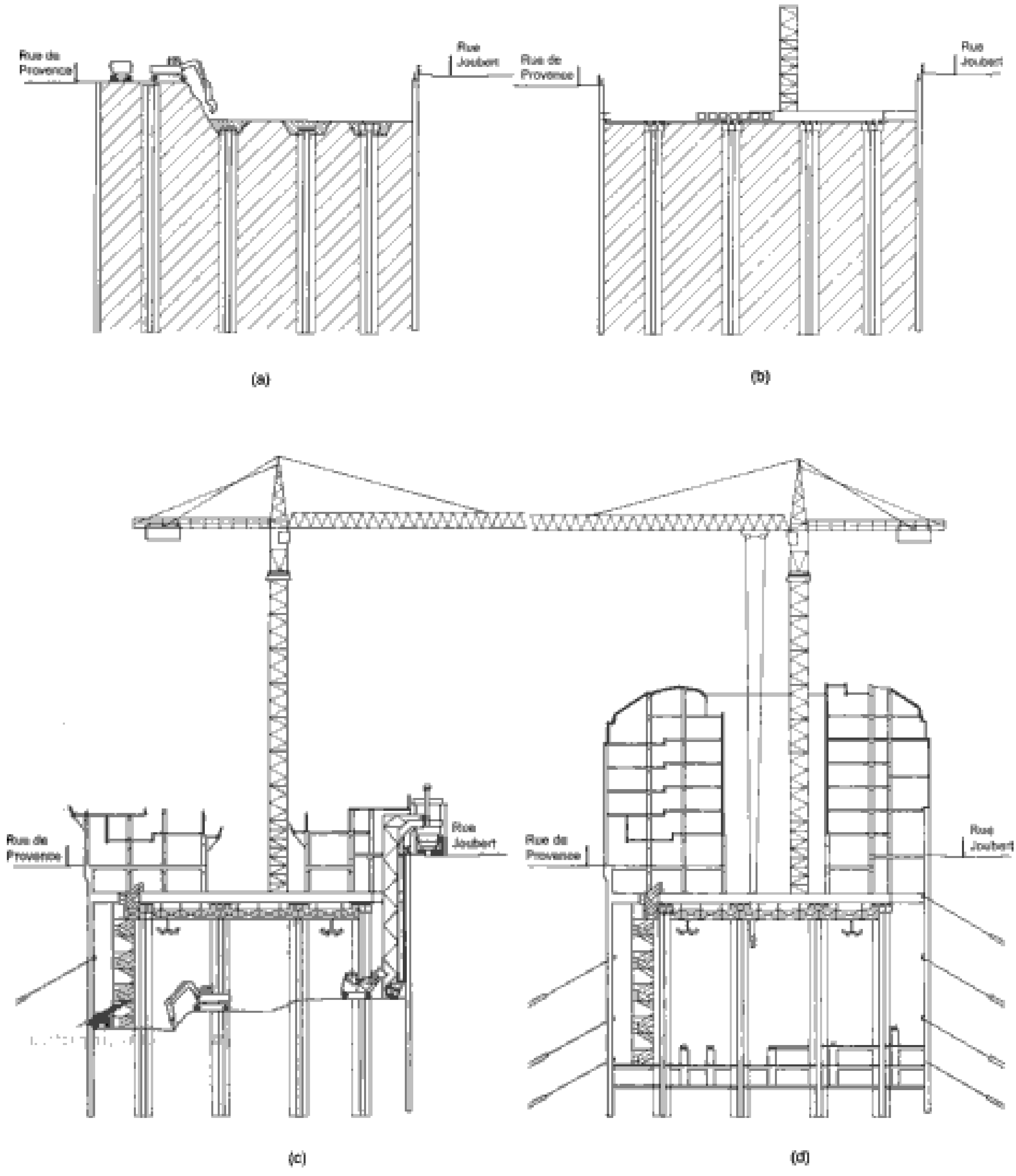


Fig. 8.14. Paris basement construction sequence, Provence Opera, Paris: (a) barrette sections installed; (b) ground floor construction; (c) superstructure construction and excavation below ground floor slab; (d) superstructure and substructure;

below the existing foundations, with anchorage to avoid lateral movement of the grouted soil mass, may prove an economical alternative to conventional mass concrete underpinning.

King post wall: Berlin wall

The king post or soldier pile and horizontal timbered wall, previously widely used in North America, has become increasingly popular for basement construction in Europe in recent years. For use in shallow excavations, the king posts may be cantilevered or propped by raking shores or anchored in successive layers as bulk excavation proceeds in deeper basement works. The king posts may be double joist or channel units battered together to allow the anchor to conveniently pass between. Fig. 8.15 shows soldier pile walls supported by anchors constructed in basement works in Saudi Arabia to depths exceeding 20 m. Subsoil conditions were layered washdown silts and silty sands.

The method requires moderately dry ground conditions with soil of sufficient strength to maintain a vertical face prior to support from the horizontal lagging being placed. King post centres vary from 1·5 to 3·5 m, depending on soil strength, depth of excavation and surcharge loads. A popular innovation is the use of in situ reinforced concrete skin walls cast against the exposed soil face, with thicknesses between 150 and 200 mm. The walls, which span horizontally between king posts, are cast in lifts between 1 and 1·5 m high, depending on the ability of the soil to stand without support.

The king post excavation may be bored by auger rig or, where headroom is limited, tripod rigs may be necessary. The toe of the king post is usually concreted to basement formation level. It is economical to use the face of the timber laggings or the face of the skin wall as a back shutter to the permanent basement wall, but allowance must be made for tolerances in the king post wall construction.

Due to the width of the king post wall and the permanent wall construction it may be necessary to drill the king post bores close to the site boundary. Where an existing structure is close to this boundary the minimum distance between king post bore and site boundary will be determined by the minimum overhang of the auger rig from the rear face of the pile bore. Table 8.2 shows the minimum dimension between an existing structure and the outer face of the new wall for various wall types.

It is usual for king post wall construction to be used only as temporary soil support. An exception was described by Mair.[20] In Marylebone, London, a king post wall

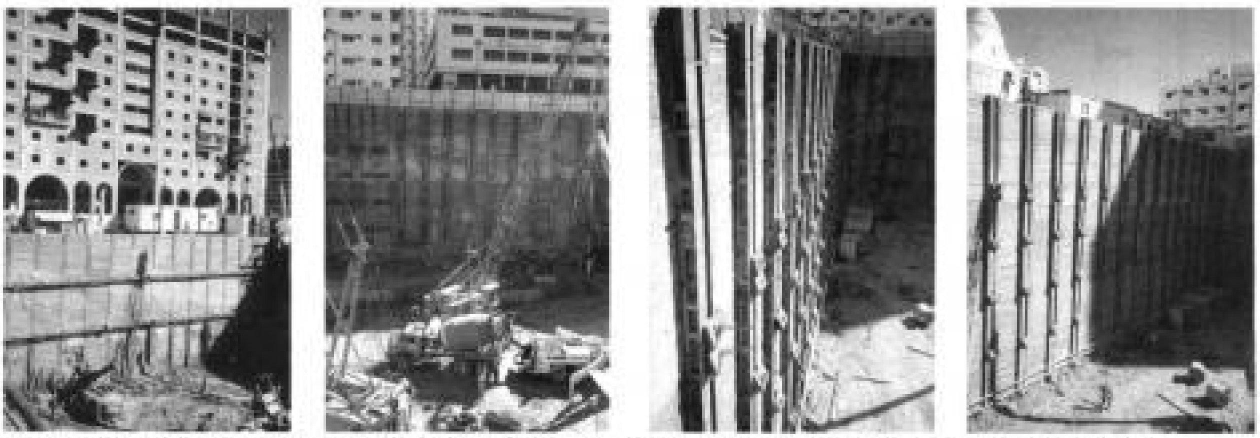


Fig. 8.15. Anchored soldier pile walls used in dry layered sand and clayey silt soil for deep basement construction. Note the unimpaired access for plant and site operations, Medinah, Saudi Arabia (courtesy of NCF)

Table 8.2 Minimum distances between soil support system and site boundary for various types of installation plant

Support system	Installation plant	Distance (mm)*
Underpinning	Conventional bulk excavation plant: e.g. hydraulic excavator with hydraulic grab	Nil
Steel sheet piling	Crane and piling hammer	500 mm, rear of sheeters to face of boundary wall
Contiguous bored pile wall	Bored pile:	
	tripod equipment typical 600 diameter pile	150 mm
	Large diameter rig:	
	Hughes CEZ 300 typical 740 pile	385 mm
	Hughes CEZ 450 typical 750 pile	385 mm
	Hughes KCA 100/130 typical 900 pile	450 mm
Contiguous bored pile wall and hard–soft secant wall	CFA rig:	
	Soilmec CM 45 typical 750 pile	350 mm
	Soilmec CM 48E typical 750 pile	350 mm
	Rotary rig CFA:	
	Bauer BG 11 typical 500 pile	400 mm
	Bauer BG 14 typical 600 pile	300 mm
	Bauer BG 26 typical 600 pile	150 mm
	Bauer BG 30 typical 750 pile	100 mm
Hard–hard secant wall	Bauer BG 7 FOW method 254, 273, 305, 343, and 406 mm diameter	Nil
Diaphragm wall	Rope suspended grab	200 mm
	City Cutter	150 mm
	Hydrofraise	150 mm
Berlin walls: soldier piles and horizontal lagging	Rotary piling equipment 600 diameter bore	400 mm
	Manual excavation: hydraulic excavator and trench box	200 mm up to 6 m depth

*Minimum distance between outer face of support system and site boundary. Distances quoted are those at ground level; consideration must be given to verticality tolerance of support system.

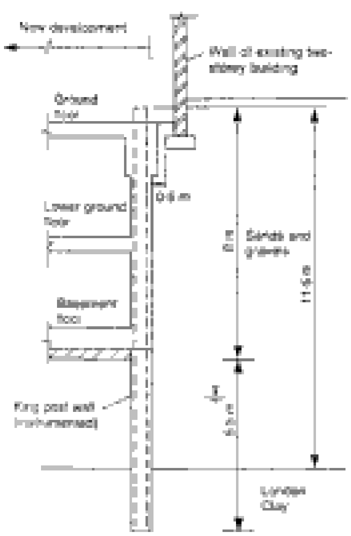


Fig. 8.16. Vertical cross-section through part of a substructure basement, Marylebone, London (Mair[20])

was used as the permanent peripheral retaining wall in a top-downwards type two-storey basement construction. A cross-section of the construction is shown in Fig. 8.16. King post wall construction was feasible because the whole depth of the basement, to 8 m, was accommodated within dry sands and gravels 11·5 m deep, below which was London clay. The water table in the sands and gravels was below the basement formation level, at a depth of 9·5 m. The king post centres were 1·5 to 1·8 m, and mass concrete infill was placed in 1 m lifts as excavation proceeded. As usual in top-downwards construction the king post wall was successively propped by ground, lower ground and basement floors.

Sheet piling

The use of sheet piling for temporary soil support to basement construction in urban areas is declining as environmental controls on noise and vibration progressively strengthen. Only where sheet piles can be installed by hydraulic means, generally in cohesive soils, can the effects of these controls on noise and vibration be avoided, although the problem of noise alone is effectively reduced by using acoustic covers on conventional drop hammer equipment.

Where noise and vibration are critical, as in most city centres, sheet piles can be installed by the joint use of slurry trench and sheet piling methods. The sheet piles are pitched into slurry trenches filled with cementatious self-hardening slurry and the toes of the piles concreted in by tremie pipe up to basement formation level. The technique, although uneconomic at first sight, is environmentally-friendly. The sheet pile section can be selected on the basis of flexural stress without consideration of driving stresses, and considerable accuracy can be achieved in pitching the sheeters into the slurry trench. The sheeters can obtain support from ground anchors with conventional steel walings or from bracing or raking shores.

Contiguous bored piling

In Europe, bored pile walls, with the piles drilled and cast adjacent to each other, have traditionally been constructed by tripod rigs but, as equipment has developed, these have been replaced by powered auger machines. These contiguous pile walls were particularly economical before slurry support techniques were developed where soil conditions were predominantly clay. In these jobs only a short length of top casing was necessary, separating the piles by approximately 50 mm. The piled wall depth was limited to some extent by the verticality tolerance that could be obtained by the augers, typically 1 % with depth. In the UK many basement walls were constructed in this way in the 1960s to the 1980s. The walls were anchored temporarily using steel walings or were braced with strutting or rakers. Grouting was used in permeable soils where groundwater entered in the gap between piles. In some instances the intended use of the basement allowed the bored pile wall constructed in this way to remain unlined, while for high-grade basements the piled walls were lined with reinforced cocnrete or an independent, non-load-bearing blockwork wall.

The advent of the continuous flight auger (CFA) piling rigs in the early 1980s, with their ability to operate without casings (even without a top casing), their high output and, for smaller low-torque machines, their ease of transport and erection on site produced economies which allowed them to replace conventional augers in most soil conditions. CFA piles for wall construction are typically 300, 450, 600 and 750 mm in diameter and are drilled to depths of the order of 20 m.

Hydraulic auger cleaners have been introduced to avoid contamination of new concrete with soil, and other innovations include a projecting tremie pipe from the base of the auger to pump concrete to a lower level than the core of soil progressively lifted by the auger. As a means of quality control, CFA rigs are now equipped with electronic sensors with visual cab displays to monitor and record auger depth, rate of concrete flow, torque, penetration and withdrawal rates and concrete pressure.

Shallow to medium depth basement walls in dry conditions are therefore economical when constructed in this way, the contiguous pile wall providing both temporary and permanent soil support. Each pile is reinforced and where piles exceed 10 to 12 m in depth, cages can be placed to the base of the pile. A mandrel vibrated into the wet concrete pulls the cage downwards. Where there is groundwater ingress between the piles, grout injection or jet grouting may be necessary.

CFA rigs operate in a wide range of soil and soft rock conditions, but hard rocks, rock chalk and strong mudstones cause obstruction and make the rigs uneconomic. Minimum distances for rig operation from existing wall boundaries are shown for a range of CFA and rotary rigs in Table 8.2. Some stated dimensions may be reduced by modifying the standard equipment. Bauer, in particular, has introduced the purpose-made S.O.B. rig to operate with reduced minimum distance.

Secant piles

Improvements in rotary rig and equipment design have, as with contiguous pile walls, changed construction methods for secant piles in recent years. Until the

Support for piled basement walls by ground and rock anchors

Secant pile walls may be supported by strutting and walings, rakers with walings or ground anchors. Where used, anchors may be taken axially through the piles or through the contact face between the piles. It may be sufficient to use anchors at alternate male piles or every fourth pile, depending on the extent of lateral load, anchor capacity, and the available shear resistance mobilized on the contact face between adjacent piles.

The decision to use ground anchors as a temporary wall support will be based on practicality, cost and installation time, which are all influenced by:

- depth of the basement
- groundwater conditions during anchor installation and, thereafter, during basement construction
- subsoil conditions and their suitability for accommodating anchors of adequate capacity economically
- maximum permissible soil and basement wall movements and the plan shape of the basement; the susceptibility of adjacent existing structures to soil movement caused by the basement excavation
- the basement construction programme
- the aggressiveness, if any, of groundwater
- the location of existing services
- the location of neighbouring substructures and/or basements
- legal permissions to accommodate anchorages outside the curtilage of the construction site
- the risk of obstruction of future works within the construction site by the presence of anchors

This list, in no order of priority, may not be exhaustive on any particular site, but indicates those items requiring earliest consideration. Some items are self-explanatory. Subsoil conditions will indicate likely anchor capacities, compact granular soils generally being preferred to cohesive soils in the fixed zone of each tier of anchors. Subsoil and groundwater conditions will dictate drilling costs for anchor installation, and the aggresiveness of groundwater and the period of use of the anchors will dictate the need for corrosion protection. The location of adjacent substructures and services will determine the practicality of installing anchors at the required elevations, and the plan shape of the basement may determine any difficulties caused by obstructing the drilling of anchors from an adjacent re-entrant basement wall. Above all, legal permissions and licences must be available from owners of adjacent land or highway authorities to allow anchor installation outside the site area. Where anchors are likely to obstruct future construction, a removable-type anchor may be necessary.

Wall movement is likely to be reduced by the use of anchors which are stressed after installation, particularly when fixed length anchors are founded in competent medium-dense or dense granular soil. When the anchors are founded in stiff cohesive soils only short-term benefit may be gained.

The programme implications of anchor installation also require examination and depend on the timing of bulk excavation following anchor installation. The sequence of drilling, tendon installation, grouting, grout strengthening and stressing for each bank of anchors has to be phased within the overall excavation programme. Comparison with an overall programme using alternative forms of wall support may be worthwhile.

The design and construction of ground anchors, described in chapter 7, is explained in detail in BS 8081,[21] which contains an extensive bibliography on ground and rock anchors. Littlejohn and Bruce[22] reviewed the state-of-the-art in rock anchoring and Barley[23] updated this, in particular giving observed bond stress values of both straight shafted and under-reamed anchors in chalk, mudstones, siltstones, shales, marls and sandstones.

Soldier pile tremie concrete method

The North American practice, popular in the 1960s, of modifying the king post wall method by excavating a panel by grab under bentonite slurry between the king posts and filling the panel excavation with unreinforced concrete by tremie, has rarely been used in Europe. In Germany, however, a modification of this method, using mesh-reinforced gunite sprayed between and over the king posts successively as built excavation proceeds, has gained acceptance. Although the construction provides only temporary soil support it is particulary economical in dry granular soils which can be excavated to a vertical face for 2 to 3 m without collapse in shallow to medium depth basements. The overall thicknesses of temporary and permanent walls and working space at the site boundary are usually not excessive.

Structural diaphragm walls

It may be argued that the most significant advance in basement construction has been the introduction of the structural diaphragm wall. The principal advantages of this form of construction, introduced into Europe by Icos in the 1950s and 1960s, were: the dual use of the wall to provide both temporary and permanent soil support; the efficiency in bending of the rectangular wall section compared with the circular pile cross-section used previously; the reduction in noise and vibration during installation compared with percussive drilling of sheet piling; the ease of installation of propping, strutting and anchoring against the wall face; the ease of applying finishes to the flat wall face; and the ability of the walls to transfer vertical loads.

The dual support provided by the diaphragm wall at construction stage and then during the basement life was often sufficiently economical to justify it on financial savings alone compared with other walling methods. Advantages such as the minimum thickness of construction required by the diaphragm wall for both temporary and permanent soil support were a bonus.

The principal disadvantages of diaphragm walling are the risk of loss or spillage of bentonite slurry, the relatively high cost of cleaning and the disposal of the slurry, the site space needed for large reinforcement cages and the large cranes needed to handle them. Above all, the need for continuity in the construction process from excavation through concreting to removal of temporary stop end formers, is a disadvantage of the method.

Icos[24] gave details of a wide range of basement constructions. These basements were generally of moderate depth, perhaps two or three basement storeys. The diaphragm walls were all excavated by cable grab mounted on tripods on rails. The wall depth attainable was considerable, however, up to 28 m in one example in Paris. The panels used were straight, ell- and tee-shaped and corrugated in plan. Fig. 8.18 shows the diaphragm wall options for basement excavations recommended by Icos.

As patent protection on the Icos wall waned, specialist firms in Europe introduced alternative excavation equipment, although in later years Icos persevered with rope grabs mounted on heavy tracked cranes. In Europe, kelly bar mounted hydraulic grabs became popular in the 1970s and the early 1980s. These grabs, which were capable of excavating wall widths between 500 and 1500 mm, were mounted on single or telescopic kelly bars to maximum depths of approxiamtely 25 m. Panels were dug in a series of grab 'bites' which were typically each 2·8 m long.

Excavation in medium-strong to strong rocks for diaphragm wall works was difficult for all specialist firms at this time. The usual method from the 1950s to the early 1980s was to use a drop chisel progressively along the panel under the slurry and alternatively grab the arisings and chisel again. Progress was slow and

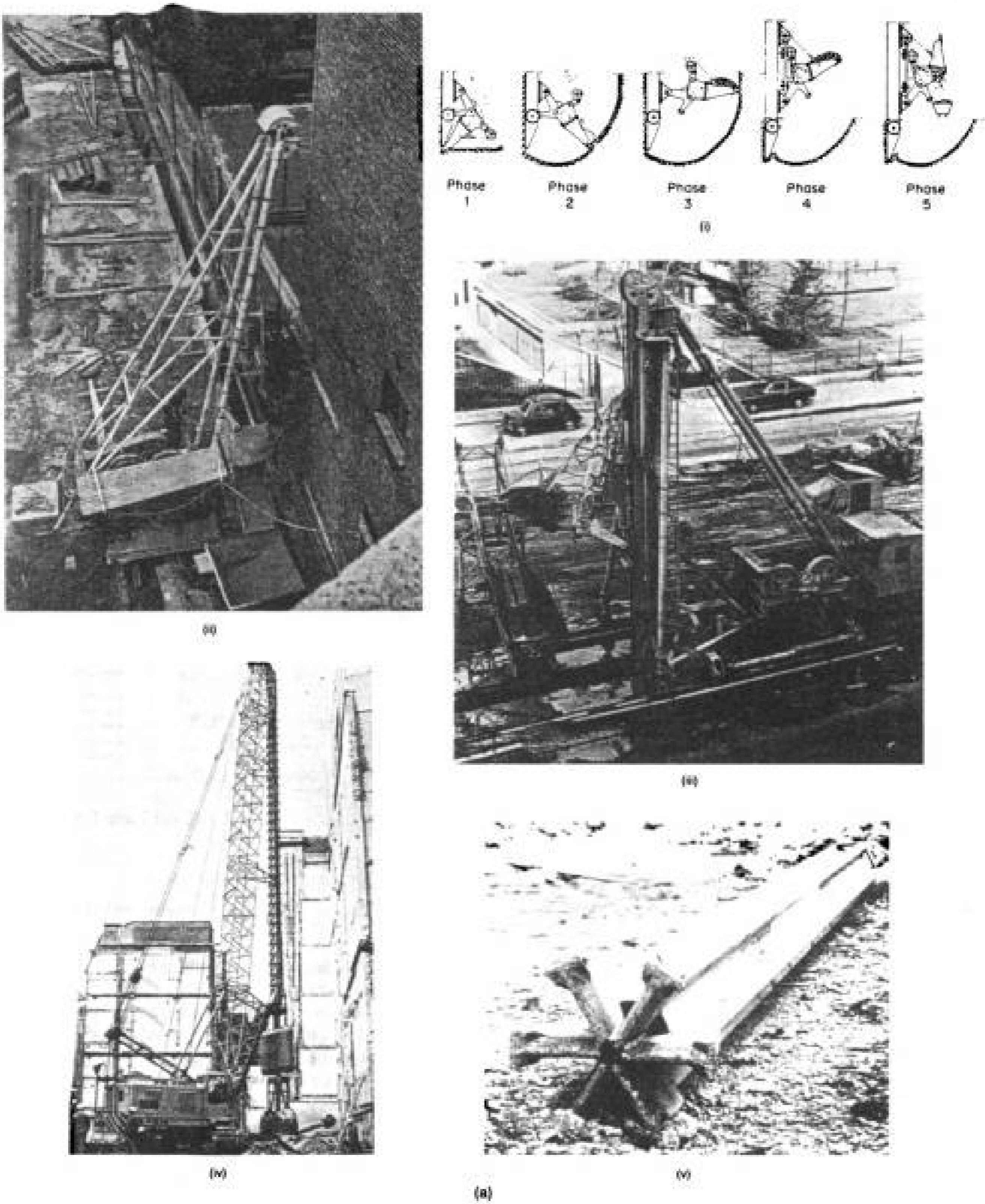
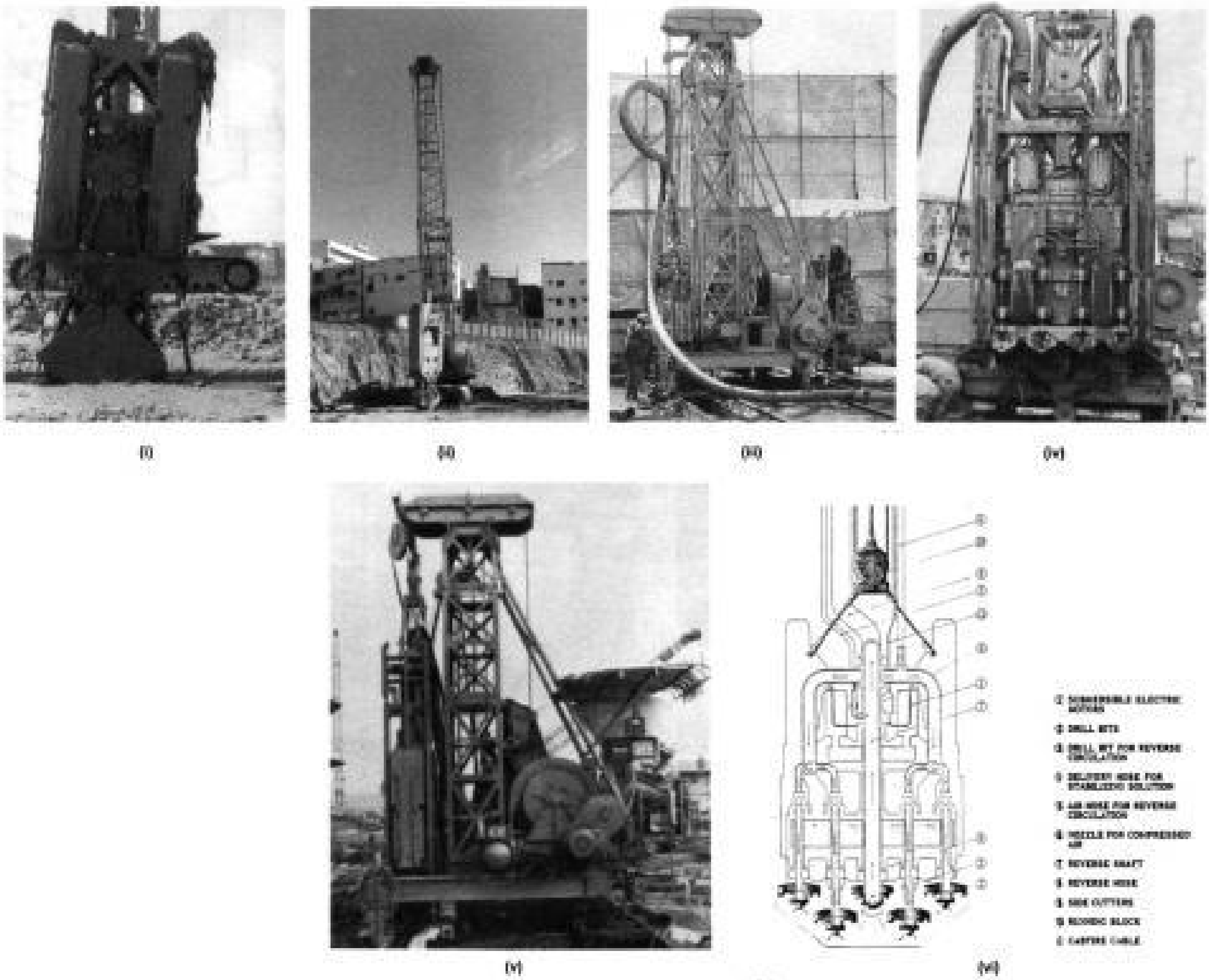


Fig. 8.19. Development of diaphram wall excavation rigs 1950s– 1960s: (a)(i) Icos tripod rig; (ii) action of Else bucket scraper; (iii) excavation with bucket scraper; (iv) hydraulic grab, kelly mounted; (v) rock chisel;



(b)

Fig. 8.19. (b)(i), (ii) crane mounted grabs; mechanism of submersible motor drill; (iii), (iv), (v) Tone Long Wall Drill; (vi) Tone Long Wall Drill: vertical section through cutter

Practical design and construction matters

A number of items require consideration in the pre-planning and design of diaphragm wall works.

(*a*) Panel size: the panel length will vary from a minimum of one grab bite and 6 to 7 m to a multiple of grab bites preferably made up of a number of whole bites with smaller widths between them. Grab bites vary between 2·3 and 2·8 m. The panel length will include two stop ends for the initial (primary) panels, or one stop end for mixed panels dug next to a completed panel. Secondary panels are those dug between two previously concreted panels. The length of the panels must first be assessed on the basis of panel stability (DIN standard 4126[25] gives methods of assessing panel stability for varying subsoils and surcharge loadings). It is necessary to limit panel length, and hence panel volume, to ensure that concrete outputs are sufficient to fill the panel within a reasonable period taking maximum daily working hours into account. Panels can often be dimensioned to ensure excavation of one panel each day (say a 12·5 m deep panel 4·5 m long dug in stiff

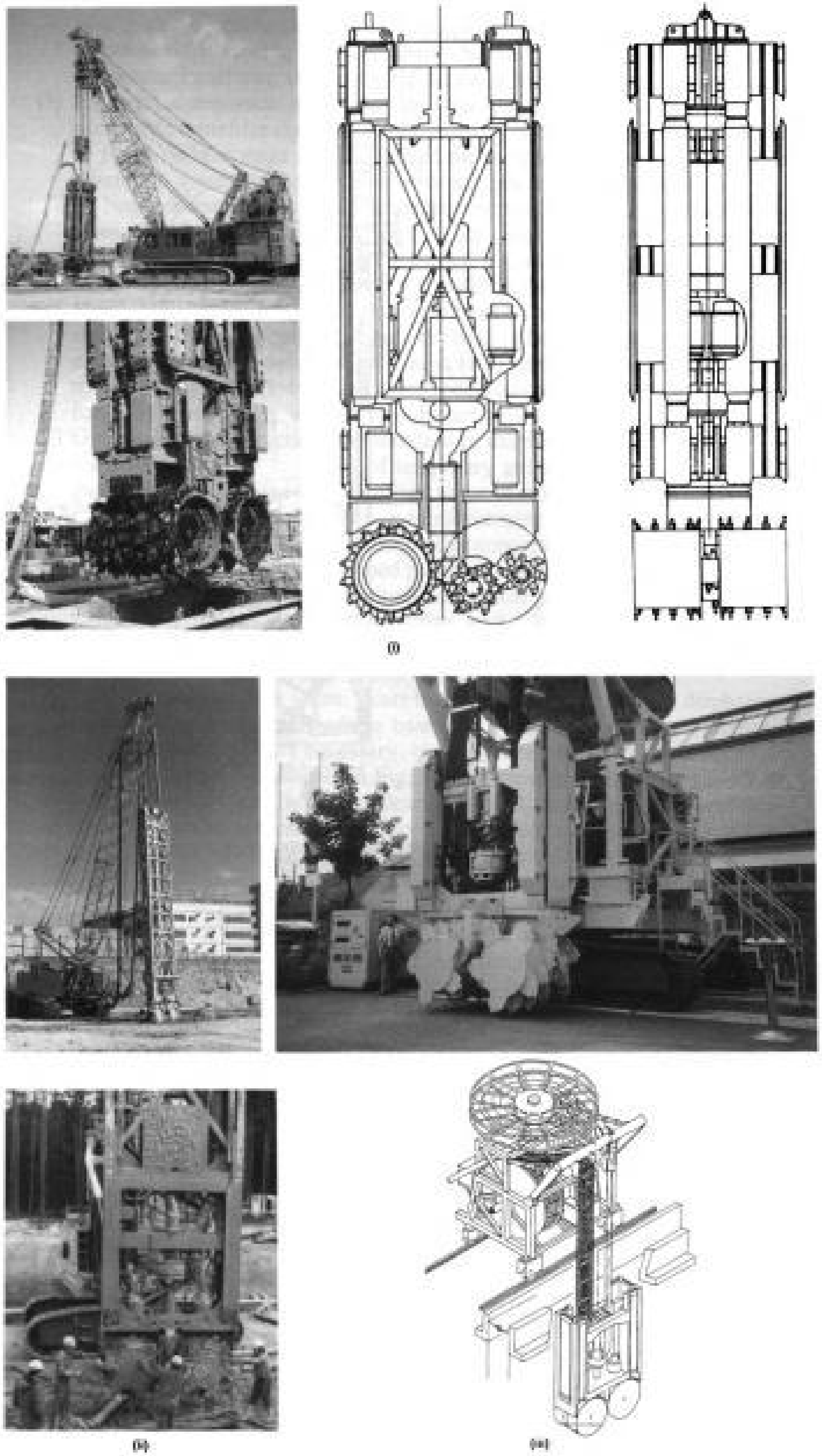


Fig. 8.19 (cont'd) (c)(i) Tone Electro-Mill Drill; (ii) Bauer BC 30 Trench Cutter rig; (iii) Bauer MBC 30 Trench Cutter rig

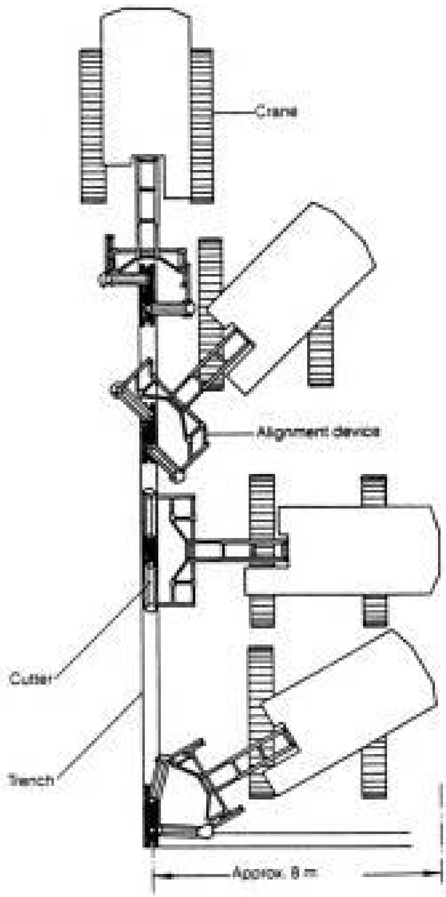


Fig. 8.20. Bauer City Cutter rig: use of alignment device on small base machine to improve maneouvrability in limited space

Fig. 8.21. Bauer MBC30 Trench Cutter rig

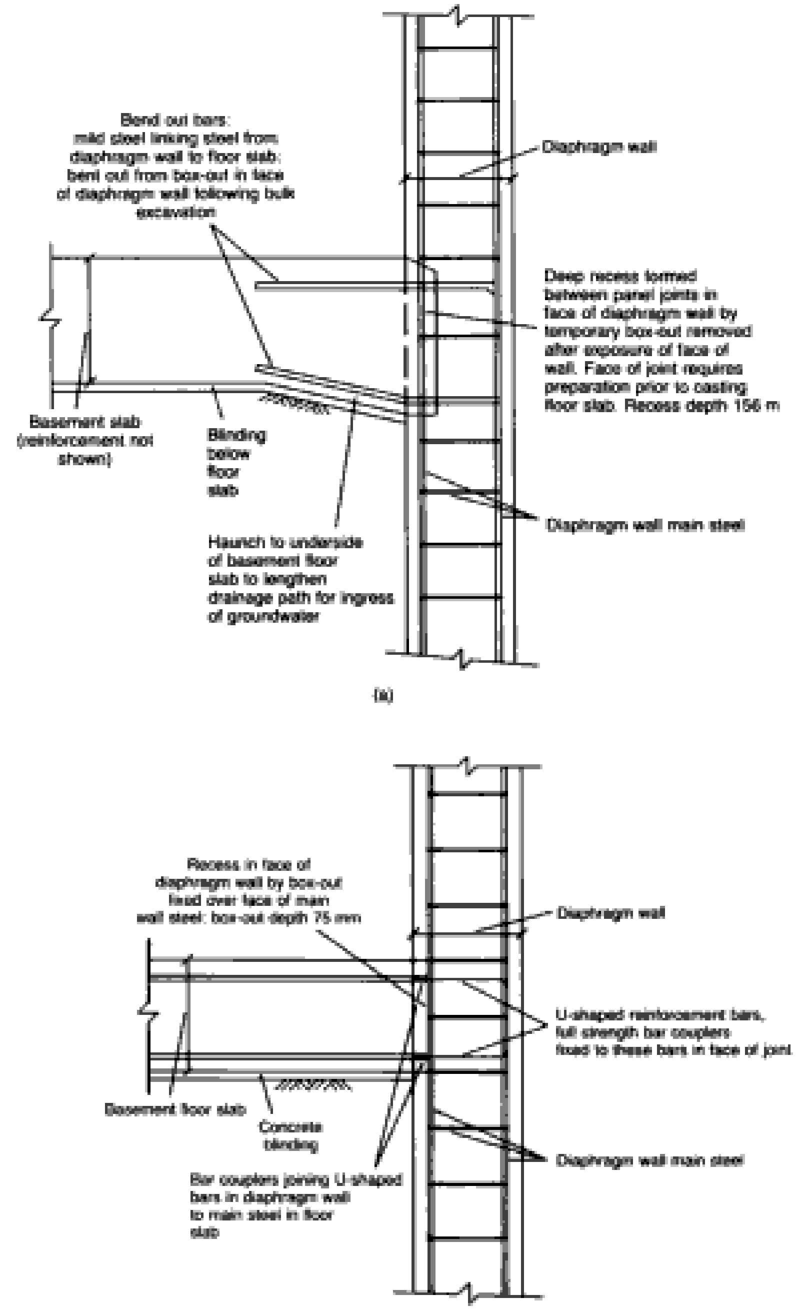


Fig. 8.22. Vertical cross-sections of typical horizontal joints between diaphragm wall and basement floor slab: (a) to achieve resistance to groundwater ingress and transmission of vertical shear; (b) moment connection by couplers

Dimensions of box-out in face of diaphragm wall are dependent upon construction tolerances for location of box-out. If these tolerances are ignored when box-out dimensions are selected, substantial waste may be needed in cutting new box-out shape to house floor slab after exposure of face of diaphragm wall

$80\,m^3$ per hour. In deep, large, T-shaped panels it may be necessary to use three tremies to maintain a uniform upper surface to the concrete as the pour continues, but generally two tremies are sufficient.

(*i*) Concreting rate: in the Author's view there is considerable risk of poor panel concrete when concreting rates drop below 15 to $20\,m^3$ per hour per tremie. Even when concrete mix quality varies from the optimum, a high

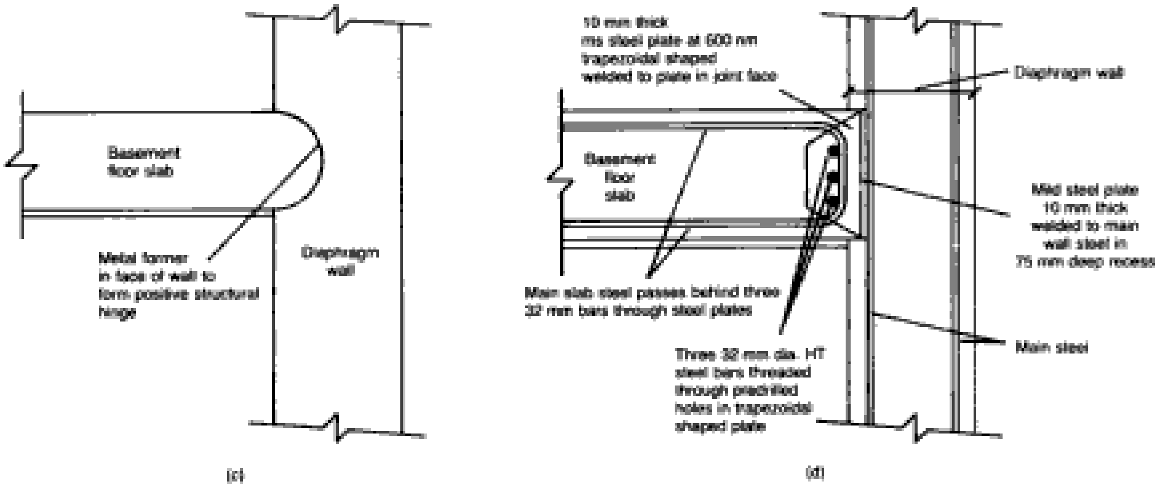


Fig. 8.22. (cont'd) (c) variation for a structural hinge; (d) variation for structural continuity

rate of concreting may be sufficient to displace the slurry, scour the surface of the reinforcement bars and flow between the reinforcement and around the box outs. Where the density of reinforcement bar is high, and especially with multi-layers of bars, a high concreting rate and a very workable cohesive concrete are essential.

(*j*) Slurry re-use: the earliest diaphragm wall jobs by Icos did not have the equipment to clean bentonite slurry after excavation and concreting. The slurry was used once and, after storage for much of the pour volume, was carted from site in road tankers or, on the earliest jobs, pumped into public sewers. With the advent of reverse circulation rigs which used large quantities of slurry to excavate each panel, cleaning technology from the oil industry was used to design and build shaker screens and hydrocyclones to clean the slurry. Compact, transportable units were made which could be brought to site and quickly mobilized for use. Nowadays, slurry is cleaned and re-used as a matter of routine on virtually all diaphragm wall works where either grab or reverse circulation rigs are used.

(*k*) Stop ends and extraction: Initially, tubular steel stop ends were used to form semi-circular joints between diaphragm wall panels. This practice, popular with Icos, the originator, and other European firms, gradually changed as rectangular formers gained acceptance. Both types of stop end were extracted as the concrete at the bottom of the panel started to set and gain strength. In very large pours, it was necessary to use high dosages of retarder additive within the concrete to ensure that the set was delayed. Even so, extraction of the stop ends often began before the concrete pour had been completed. With the advent of reverse circulation rigs with vertically mounted cutter wheels, it became possible to cut the vertical surface at the end of a concreted panel during excavation of the adjacent panel. This cutting back of the concrete surface avoided the use of temporary formers, and the new concrete in the second panel could be poured against a true vertical surface on the first panel. This procedure has since become less favoured because of the risk of heavy, calcium-contaminated slurry remaining near the cut surface, which could contaminate the concrete within the second panel, near the vertical joint. This slurry-contaminated concrete often proved to be porous and led to leakage of groundwater into the basement after excavation. The CWS-type stop-end former developed by Bachy incorporates single, twin

or triple water bars cast into the vertical joint and is released from the vertical surface of the concrete pour after the concrete has hardened during excavation of the second panel. The depth of panel in which this joint can be used is currently limited to 20 to 25 m. Fig. 8.24 shows the CWS joint details. Types of vertical panel joints are shown in Fig. 8.25.

Recent developments in diaphragm wall works

Since the conception by Veder of reinforced concrete walls cast into trenches dug under bentonite slurry, and development of the process by Icos and others, there have been many improvements in both application and mechanical plant. Puller and Puller[26] reviewed developments prior to 1992, including

(*a*) the use of polymeric slurries for excavation support
(*b*) the development of structurally-efficient diaphragm wall plan shapes based on improved joint efficiency between panels
(*c*) the use of post-tensioned diaphragm walls, either precast or cast in situ

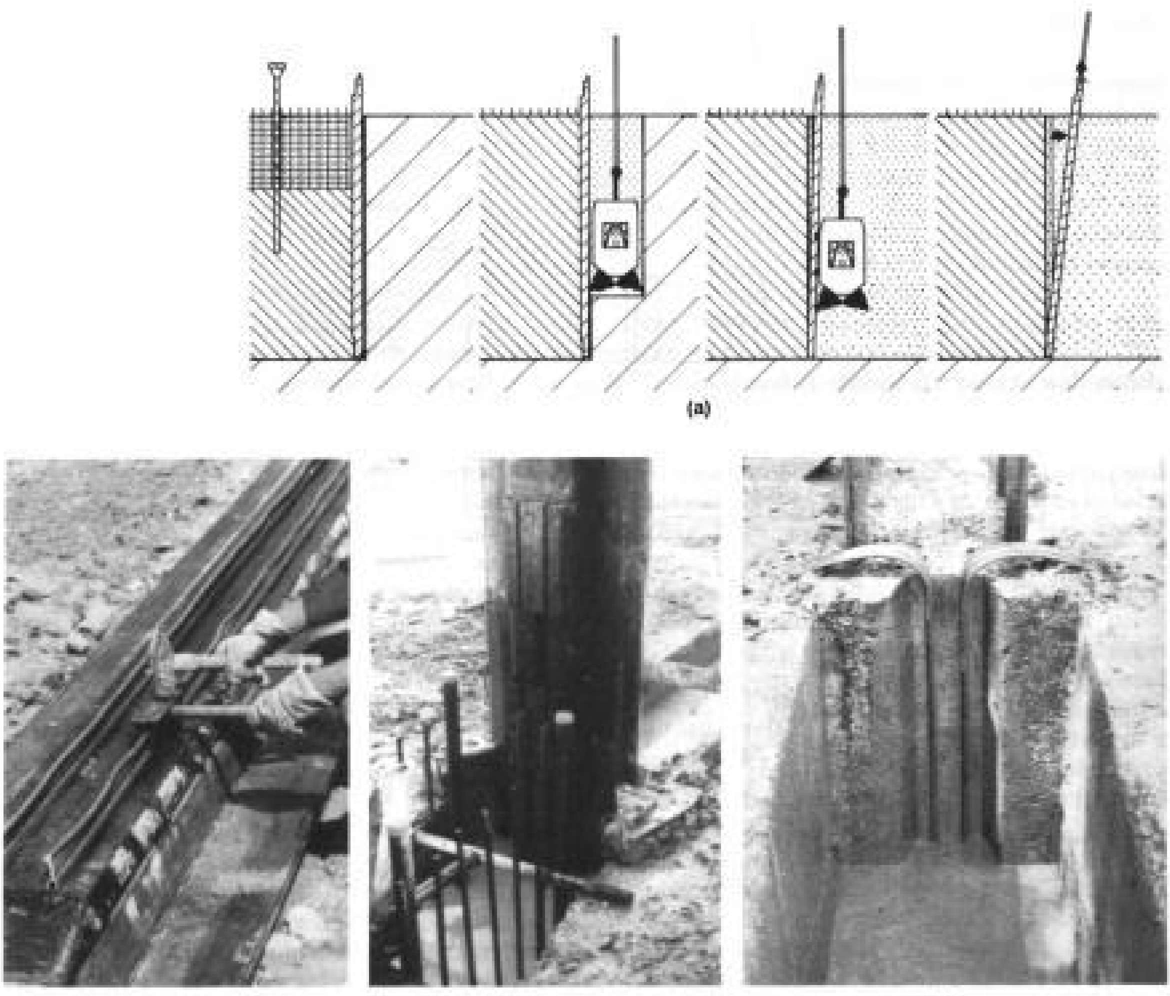


Fig. 8.24. CWS joint detail: (a) pulling out stop-end sideways after excavation of the adjacent panel; (b) stop-end blades installed in the CWS joint; (c) CWS joint before concreting; (d) CWS joint with water-stop installed (courtesy of Bachy)

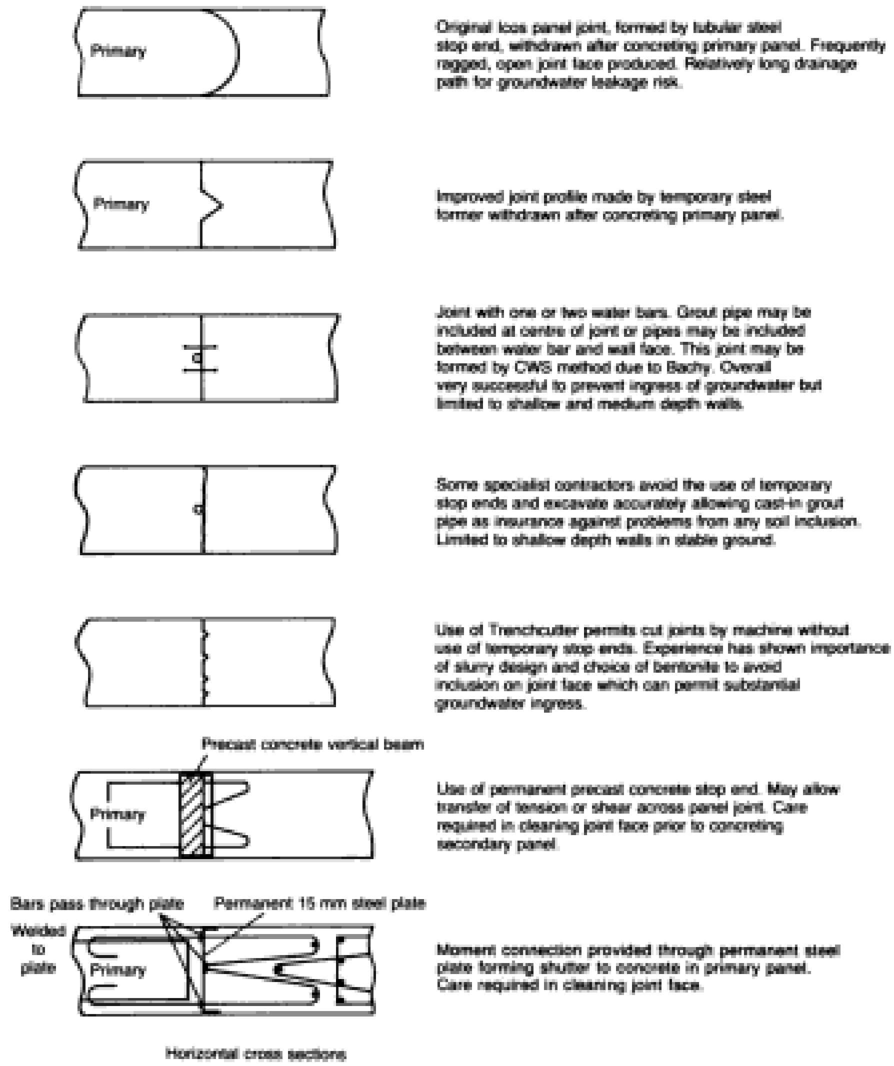


Fig. 8.25. Types of vertical panel joint used in diaphragm walling, shown in horizontal section

(*d*) the use of precast concrete diaphragm walls
(*e*) the use of reverse circulation excavation equipment such as the Hydrofraise and Trenchcutter rigs
(*f*) the development of excavation equipment for work on congested sites.

Items (*a*) and (*b*) are reviewed in more detail below; the remainder are referred to elsewhere in this chapter and in chapters 4 and 9.

While the original use of bentonite slurry to support diaphragm wall excavations has generally persisted for excavations made by grab, the larger quantities of slurry required for circulation purposes with Hydrofraise and Trenchcutter equipment have brought innovation to slurry design. Using experience from the soil drilling industry, polymeric slurries and mixes of polymeric and bentonite slurries have been used successfully on larger diaphragm wall jobs where the economies of scale have proved beneficial.

Polymeric slurry behaves as a pseudo-plastic fluid and, unlike bentonite slurry, acts in trench support without forming a filter cake. Within the polymeric slurry

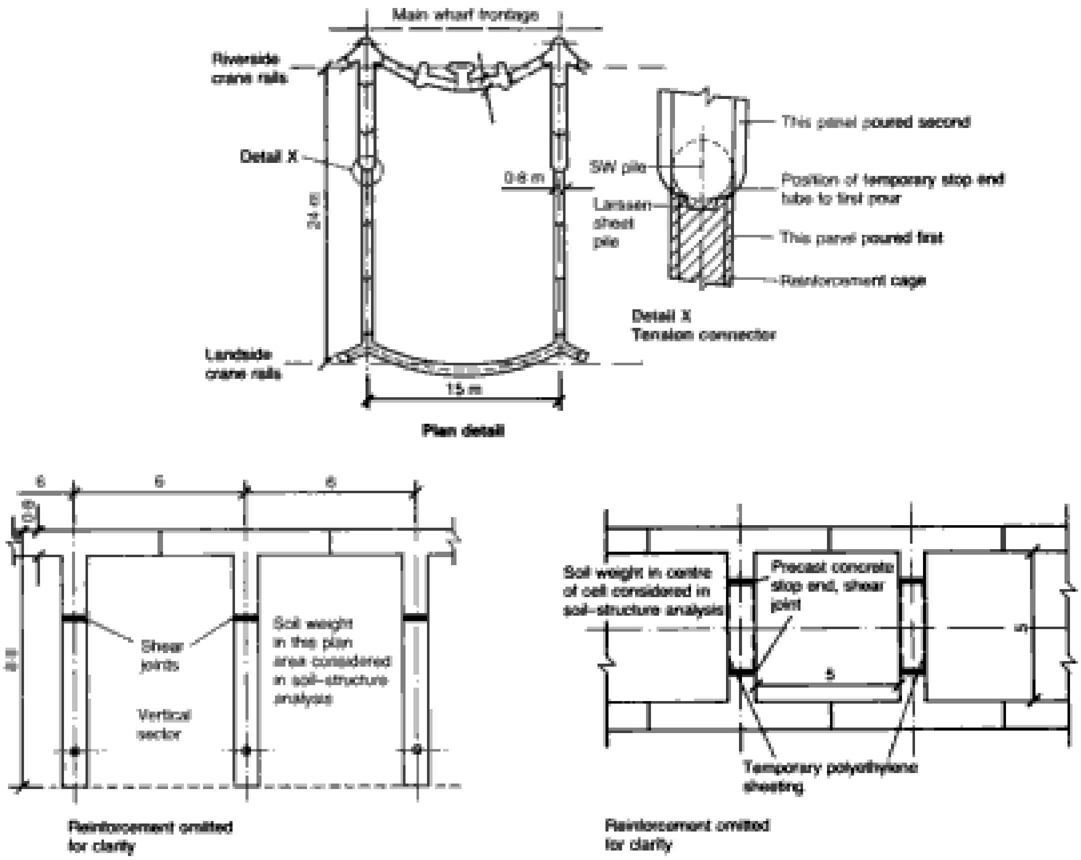


Fig. 8.26. Development of continuity between diaphragm wall panels to achieve improved, structurally efficient wall plan forms (Puller and Puller[26])

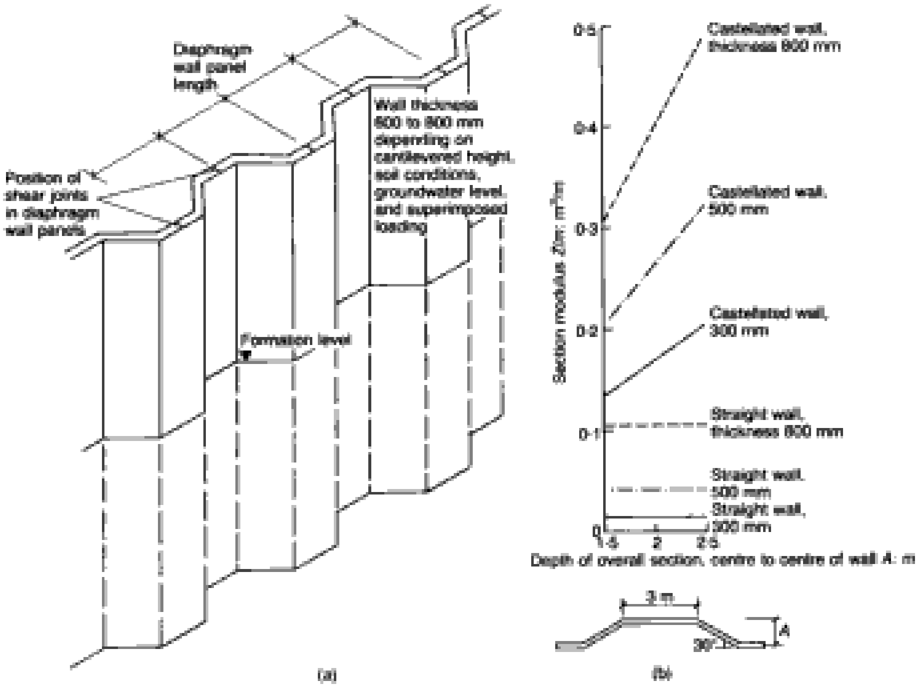


Fig. 8.27. Diaphragm wall built to castellated plan shape: (a) typical layout; (b) plot of section modulus of castellated section against overall depth of section showing improvement to flexural strength compared with a straight wall

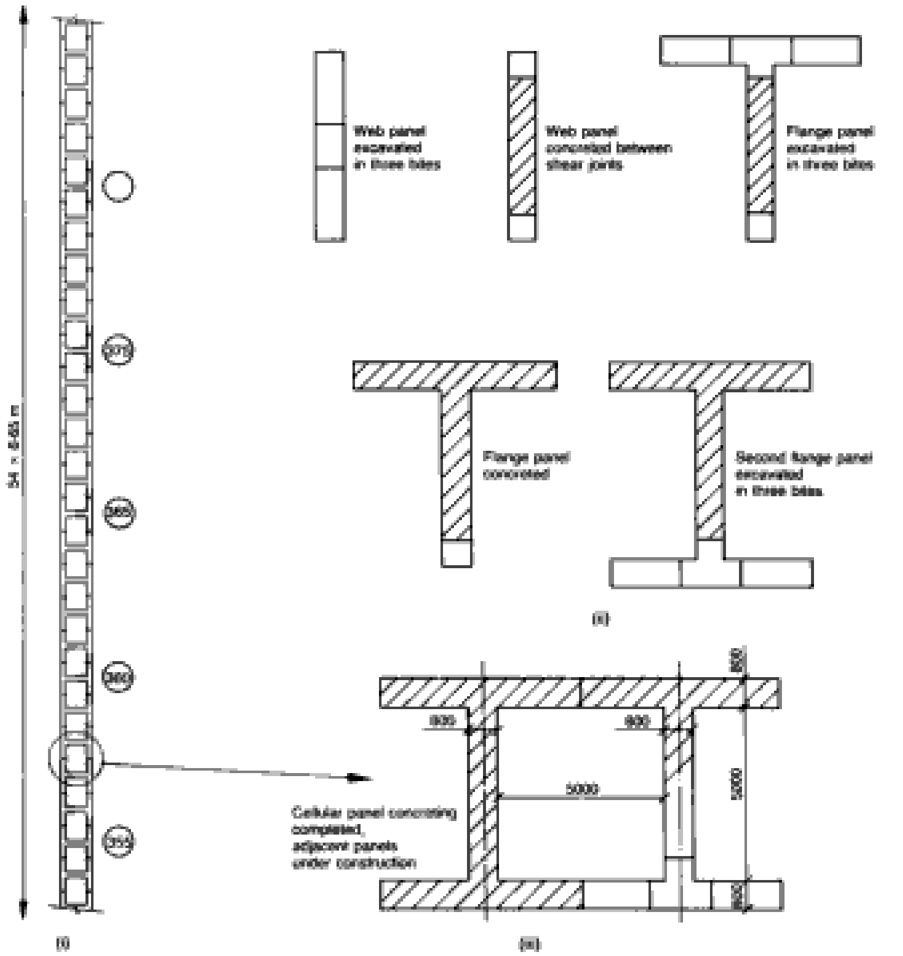
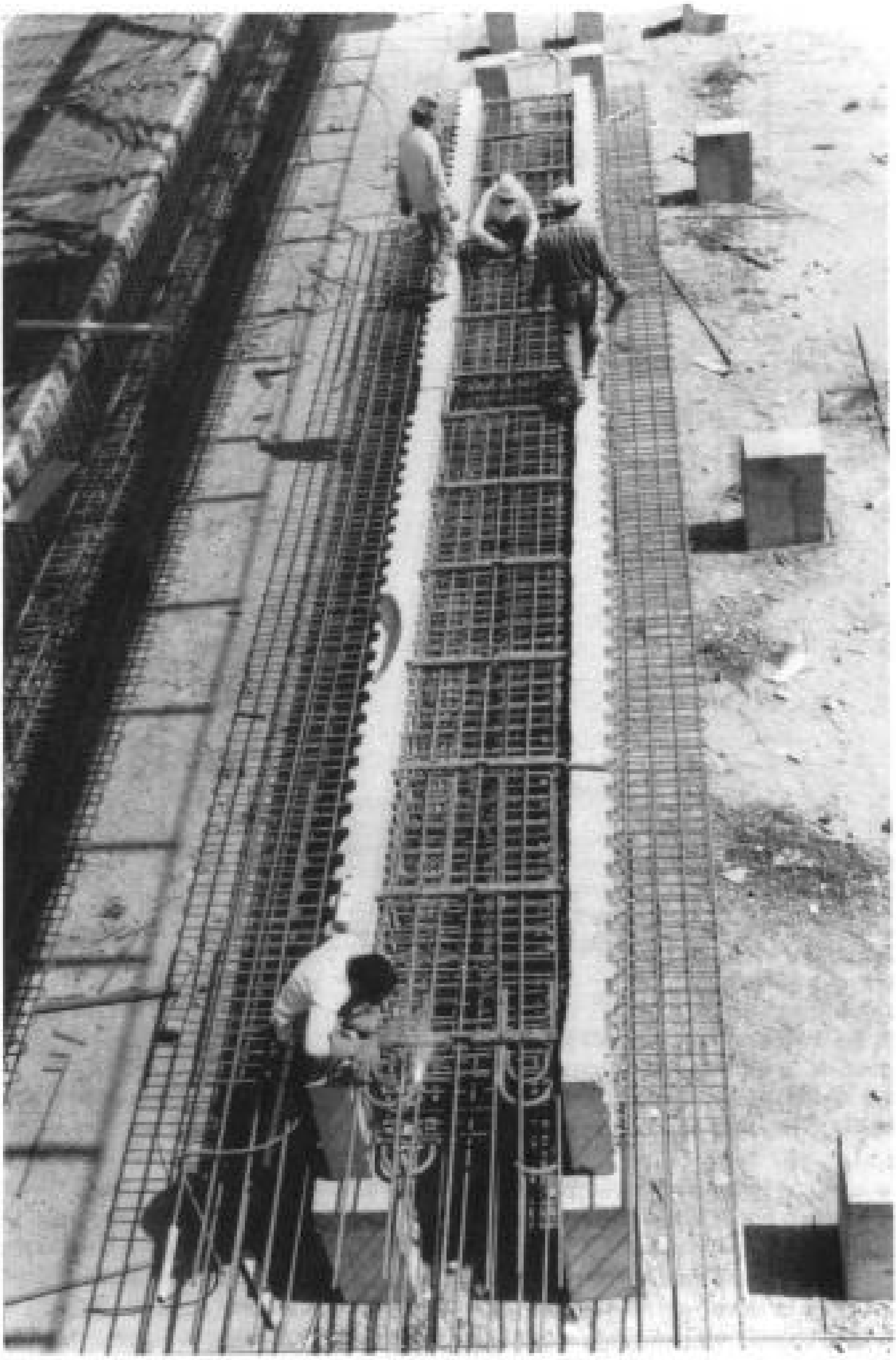


Fig. 8.29. Medinah car park: cellular wall construction: (i) plan of cellular wall construction; (ii) construction procedure for single wall unit; and wall unit construction adjacent to completed unit

excavated by Trenchcutter rigs until excavation depths increased below 20 m or so. Three Trenchcutter rigs and up to five rope grabs were used on a 24 hour, 6 day week basis. Concreted panel production averaged more than 4000 m^2 per week over much of the two year wall construction period and reached 7000 m^2 per week over several weeks at peak production.

The Medinah cellular wall was designed using a two-dimensional analysis of the soil–wall structure taking into account non-linear elastic–plastic soil conditions. Soil movement and stress levels in the surrounding soils were predicted for the modelled excavation stages by finite element analysis, taking into account the dissipation of negative pore water pressure with time for varying depths of rockhead and groundwater. The analysis methods were those described by Jardine *et al.*[30] The structure deformation results from the two-dimensional analysis were then used with a three-dimensional structural program to predict stress levels and design reinforcement within the cellular wall. This work was incomplete because the analysis did not include the deformations within the soil and the resulting changes in soil stress levels caused by excavation of the diaphragm wall panel itself. It had been realized prior to the Medinah design that accurate prediction of soil movement and stress adjacent to a completed diaphragm wall basement excavation had to include installation effects of the diaphragm wall panel during panel excavation and concreting. The effects of concreting on in situ soil stress had been shown[31] in measurements which concluded that induced stress caused by the



(a)

Fig. 8.31. Medinah car park: (a) fabrication of web panel with permanent precast concrete stop-ends to cellular wall;

shear stiffness of a three-dimensional structure in plane strain and the over-simplification of ignoring panel installation movement and stress in the subsoil. The maximum predicted horizontal soil movement of 27·5 mm after bulk excavation for the deepest cantilever walls was not reached, the total observed maximum horizontal deformation for both panel and bulk excavation for the cellular wall being less than 20 mm, allowing for dissipation of pore pressure with time.

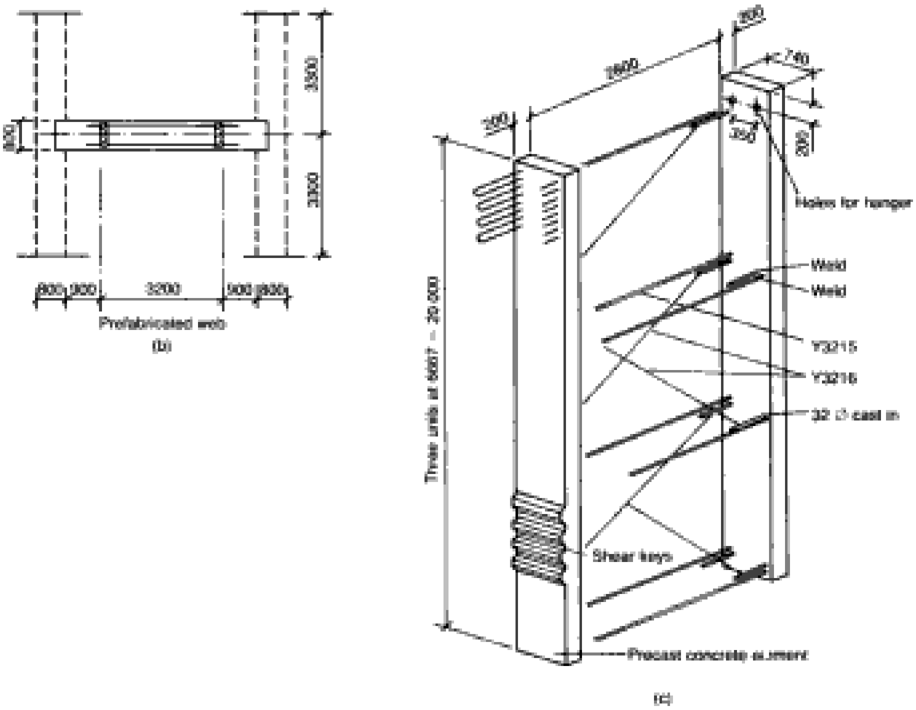


Fig. 8.31. (cont'd) (b) key plan of one element; (c) isometric of web panel

Composite walls and grouting techniques

Mention should be made of the use of diaphragm walls, either precast or in situ concrete construction, as part of a soil retention and groundwater control protection system incorporating non-structural slurry wall cut-offs and horizontal grout plugs over the plan area of the basement. Fig. 8.32 shows examples of composite precast diaphragm walls incorporating temporary Berlin walls cast in the upper section of one wall. The use of jet grouting and intrusion grouting to form a horizontal grout plug to control the inflow of groundwater to excavations was discussed in chapter 2.

Waterproofness of structural diaphragm walls

The difference between expectation and the actual performance of diaphragm walls regarding waterproofness has caused disappointment and dispute since the earliest structural walls in the 1950s and 1960s. Then, structural walls generally required the minimum of surface treatment to produce a dry face. Many of the earliest diaphragm wall basements in Paris and London were used for car parking and were either left without finishes or, at most, with an applied sand-cement render. Where leaks occurred these were sealed by application of surface chemical renderings such as Vandex or Xypex to make a crystalline waterproof coating to the wall. These walls, generaly 600 or 800 mm thick, were excavated by rope grab or hydraulic grab (rope or kelly mounted), and temporary tubular steel stop ends were used throughout. Only in basement construction where storage of perishable goods was planned, in shopping areas or office facilities, was a separate lining wall constructed.

In London, the earliest diaphragm walls were built by Icos from 1961 onwards. By 1974 a sufficient number of contracts had been completed to hold a keynote

> these joints are usually watertight but minor seepage through leaking joints can be dealt with by grouting or may even be tolerable in certain classes of structure.

Generally the attitude was optimistic, and risk of leakage was only considered after it had happened. The earliest model specification in the UK[36] did not refer to waterproofness and typical paragraphs in tender letters by specialist contractors stated

> the diaphragm walls will be constructed so as to be substantially watertight on initial exposure (free from running leaks but not damp proof) and we only accept responsibility for repairing leaks, within the exposed height, caused by faulty workmanship and/or materials. It should be noted that possible ingress of water into the excavation from below formation level is not prevented by the diaphragm walls.

Overall, specialist firms were relatively optimistic about the likely occurrence of leaks on jobs throughout the UK in a variety of soil conditions and for a range of basement uses. The risk of overbreak, panel collapse, loss of bentonite, displaced box outs, inadequate excavation rates, etc. were all critical tender risk assessments for the specialist contractor, and wall waterproofness and the cost of associated remedial works were not considered as important as they are today. In the UK in the 1960s and 1970s, specialist firms were usually awarded diaphragm wall contracts on the basis of design and construct after technical discussion with a consulting engineer or architect. The contractual risk for waterproofness (apart from damp patches) generally remained firmly with the specialist contractor. By the end of the 1970s most major consultancy firms were designing and specifying diaphragm wall schemes themselves and the contracts were let on a construct-only basis. At this stage, the overall use of the underground structure was clear to the designer, who could incorporate measures such as non-load-bearing blockwork walls and drainage channels to hide any persistent ingress of groundwater.

The use of basement lining walls, cut-and-cover works and underpasses has continued in the UK and Germany. In some instances (Lyons Metro 1981, Eastbourne Pumping Station 1993) an in situ reinforced concrete lining has been specified by the engineer or owner to be capable of withstanding the full groundwater pressure acting on the wall.

Increased use of blockwork lining walls and cast in situ linings was prompted in the UK in 1978 by the publication of a report by CIRIA.[37] This report, while not directed exclusively to the design of diaphragm walls, was used by diaphragm wall designers and specifiers. The report defined three grades of basement waterproofness.

(*a*) Utility grade: some seepage and damp patches can be tolerated provided water is drained away. This grade may be suitable for accommodating most mechanical and electrical plant and for car parking.
(*b*) Habitable grade: no visible penetration of water is permitted, but condensation may be allowed to form in exceptional circumstances. This is generally suitable for human occupation in offices, workshops and residential accommodation. There are no precise regulations specifying the conditions in rooms for human habitation. Neither the Building Regulations nor the London bye-laws have requirements other than for the prevent of the ingress of moisture which may damage the floor or walls.
(*c*) Special grade: no penetration of water or water vapour is permitted. This is usually only required for archives and stores which impose the most stringent requirements because of the susceptibility of the contents to deterioration in humid conditions. Condensation can be disastrous in these circumstances and measures for ventilating, heating or full air conditioning of the basement become essential. It should be emphasised that one cannot

rely on ventilation to remove humidity caused by water penetration, and the design must aim for the highest possible degree of water exclusion.

The report concluded that there were three design solutions available:

(*a*) Watertight concrete construction, relying on the watertightness of the concrete construction itself. It would provide a utility grade environment and, if there were no permanent head of water, it would provide habitable grade.
(*b*) Drained cavity construction: a watertight concrete construction with the added precaution of an inner leaf wall and (sometimes) cavity floor. All cavities had to be drained.
(*c*) An external waterproof membrane: a watertight concrete construction with the added precaution of an external waterproof membrane.

Until recently option (*c*) was not generally available for diaphragm walls but three contracts completed by Bauer in Singapore in 1993–94 successfully used a continuous outer plastic liner to completely encapsulate a diaphragm wall at reasonable cost.

These requirements have now become standard design principles for basement diaphragm wall work in the UK. Most basements are designed with a drained cavity construction and only those used for car parking have the option of exposed unclad diaphragm wall surfaces. Building owners are not prepared to allow unlined diaphragm walls (say for a basement for storage use) where there is reasonable chance of a change of use during the life of the basement. Current practice is therefore to make a drained cavity with a permanent pump provided at a sump at the lowest level. To make this provision, the design volume of the basement is reduced by the volume of the drainage cavity, the volume of the blockwork lining and the volume occupied by the verticality tolerance of the diaphragm walls (say for grabs 1:80 or 1:100), irrespective of whether this tolerance is used by the wall or not. It is widely acknowledged that this solution is uneconomic and often leads to a reduction in car spaces in basements where lining is used.

The use of lining walls does not automatically produce dry construction because faulty drainage cavities and drainage between basement floors often lead, over time, to damp blemishes on the exposed face of the lining because of leaks in the hidden diaphragm wall.

The current situation in the UK is this: the specialist contractor generally contracts to leave the exposed diaphragm wall free from running leaks (but not damp patches) and probably has half of the full retention money held against this for, say, twelve months from the end of the main contractor's contract. When leaks arise during this maintenance period the specialist contractor seals them by grouting and hopes that a final inspection at the end of the maintenance period will be the end of leakage responsibility.

The use of non-load-bearing lining walls in basements is similar in both Germany and the USA, although a slightly more optimistic view regarding the waterproofing efficiency of diaphragm walls may remain in France. The waterproofness of structural diaphragm walls has been reviewed in some detail.[38]

In the Author's experience, basement wall leakages occur at any of five locations:

- in the panel itself
- at vertical panel joints
- at horizontal bottom slab/wall joints
- at the top horizontal joint between panel and capping beam
- below formation level

(*a*) In the panel. Leaks and damp areas in the panel are caused by soil or slurry inclusions, random cracking perhaps due to shrinkage, or poor quality concrete. Xanthakos[39] concluded:

(*d*) Top horizontal joint between panel and capping beam. It is not unusual to find leakages in the horizontal joint at the top of a diaphragm wall with a capping beam or other in situ reinforced concrete where a high groundwater occurs or where rainwater is allowed to pond in porous backfill to the capping beam/guide wall excavations. Such leakages can occur even when the top of the diaphragm wall is adequately cut down to remove porous concrete and the surface is correctly prepared. Although such work is essential, the provision of a Hydrotite water bar strip in the horizontal joint on the earth face would overcome the risk of water leakage.

(*e*) Below formation level. This risk is often ignored, although the financial consequences of wall leakage or even loss of ground where 'blow' symptoms occur are likely to be very significant. The standard tender clause used by some diaphragm wall specialists in the past has stated:

> we only accept responsibility for repairing leaks, within the exposed height, caused by faulty workmanship and/or materials.

The implication must be that the specialist contractor would not be responsible for faulty workmanship and/or materials below formation level in the area of highest risk. The clause was rarely queried by main contractors or their clients and presumably relieved the specialist contractor of considerable risk.

The obvious preventative measures for leakages all refer to standards of workmanship and quality control during diaphragm wall construction. The importance of design decisions, however, regarding wall thickness, permanent stop end construction and provision of water bars in panel joints, should not be ignored when risk of leakage below formation level is assessed.

Up to the 1990s the general level of acceptance of wall waterproofness in the UK was based on the criterion that damp patches on the exposed wall surface would be accepted but running leaks would not, and grout or surface treatment would be accepted as a remedy. The most widely used model specification for diaphragm walls[35] does not refer to waterproofness, and the omission of any reference to waterproofing standards is even less understandable in either the DIN standard 4126[23] or the British Department of Transport's Specification for Highway Works.[40]

The UK Institution of Civil Engineers' draft specification for cast in place diaphragm walls[41] does make specific reference to waterproofness, however:

> The complete retaining wall shall be considered to have acceptable water retaining properties if, overall, the leakage of water per square metre of wall does not exceed the volume stated in the Particular Contract Specification or any other specific requirement. The Contractor shall be responsible for the repair of any joint or panel where, on exposure of the wall, visible running leaks are found or where damp patches longer than 1·0 m occur. Any leak which results in water emanating from the surface of the retaining wall shall be sealed.

This proposed clause places responsibility on the contractor to repair any leaks that are visible (and therefore above formation level), any leak on the surface of the wall (including below the formation level) and any damp patch longer than 1 m. The clause adds a maximum quantity of leakage per square metre, presumably applied to the whole wall area. The requirements are less than clear and no mention is made of the means of repair. The words 'water retaining properties' have been preferred to the term 'waterproofness'.

In the Author's opinion the specification clause currently used in the Middle East has many virtues and probably also summarizes usage in the UK. The clause is:

> Waterproofness: the diaphragm wall shall be watertight. Remedial measures shall be carried out as directed by the Engineer in areas that do not comply

> with this requirement. A panel will be considered watertight if within any area 1 m × 1 m the total damp surface area does not exceed 10% of that area. Any panel exhibiting wet surfaces will not be considered watertight. The Diaphragm Wall Contractor shall be responsible for the repair of any joint where, on full exposure of the wall, visible wall leaks are found.

This clause therefore does not define the location of any leakage but defines the size of damp areas, and refuses to accept running leaks above formation level. No overall acceptable leakage into the basement is quantified.

It is the Author's opinion that specification clauses for waterproofness of diaphragm wall works in general use are inadequate because they do not specify acceptance to adequate standards, they do not specify the method of repair and they do not approach the subject of waterproofness with the considerable importance and detail that it deserves. Several matters are ignored in the latest specifications. As an example, the building owner or tenant's interests are not served by the lack of consideration for leakages occurring after the contractual maintenance period by, say, rising groundwater, the failure of repairs to previous leaks, or movements between panels or basement floor joints due to long-term soil movement such as heave. Current specifications do not relieve the specialist contractor for leakage due to panel movements caused by application of superstructure loading or anchor stressing, matters that are frequently outside his control. The water bar system in the diaphragm wall should be specified by the designer. The wall system should be connected efficiently to the water bar system in the base slab, and should also be specified by the basement designer.

Overall, early optimism among specialist contractors in Italy, France and the UK has now been replaced by a realism that acknowledges that concrete tremied into a bentonite slurry in a series of panels will not automatically produce a dry basement. In Europe, internal lining walls are currently used to avoid the effect of the groundwater ingress. Unfortunately, these internal walls cover up a continuing risk to the building owner and mask the occurrence of further leaks or deterioration in the repairs to the original leaks.

Overall stability: design for uplift

Where hydrostatic groundwater pressures, during construction and within the design life of the structure, are at a higher elevation than the underside of the lowest basement floor level, it is necessary to examine the overall stability of the basement. Failure due to the lack of buoyant self-weight of the basement, and insufficiency of frictional forces to avoid upward displacement of the basement substructure, are fortunately infrequent, but not unknown. Vertical anchoring of the basement raft to rock strata or strong soils below the basement becomes necessary where hydrostatic uplift is severe and such strata exist at economical depth. Drainage of a granular blanket or porous no-fines concrete with permanent pumping may prove necessary where anchoring is not feasible. The rise of groundwater within the design life of the structure should be carefully assessed after consideration of the dead weight of the structure at successive stages of superstructure construction, including final completion; a factor of safety of the order of 1·4 should be obtained for the sum of downward dead weight, total vertical downward anchoring force and frictional resistance to the basement walls compared with upward hydrostatic force on the basement underside. Where diaphragm walls are used for basement construction, the frictional resistance in cohesionless material, or the wall adhesion due to clay strata, should be calculated in the same manner as frictional resistance to bored piles in similar soils, restricting the diaphragm wall surface used in the calculation to the inner and outer surface of the wall below formation level.

A broad cost comparison is reproduced in Fig. 8.36. This comparison includes completed wall construction for two- and three-storey basements in two particular soil and groundwater conditions. It would be unwise to draw conclusions from this small sample but broadly it showed that, in the UK, contiguous bored pile walls with an internal lining are comparatively inexpensive in good piling ground; secant piling is an economic choice in less conducive soil conditions and for deeper basements; where sheet piling is left in place, an expensive wall results; anchored diaphragm walling is competitive in deeper basements in good soil conditions.

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9 Cut-and-cover construction

Choice of wall system

As the name suggests, cut-and-cover construction consists of tunnel construction by deep excavation in trench, construction of the permanent tunnel structure, and subsequent backfill and reinstatement of the ground surface. The method is economical in comparatively shallow tunnel works and is typically applied in urban highway schemes and for urban metro stations and running tunnel construction.

Historically the method was used as an alternative to tunnel construction for underground railway and river-crossing highway schemes in European cities in the second part of the nineteenth century, particularly in London. Prior to the Second World War, metro construction in European cities such as Berlin and London exploited cut-and-cover construction and furthered construction techniques such as the Berlin method of soil support. Its use provided an alternative to boring for underground tunnels within a range of depths, typically 8 to 10 m, which were limited by cost and the availability of excavation plant and structural materal. Excavation plant and craneage was largely steam driven, and structural materials were usually timber or steel sheet piling.

The reconstruction of European cities in the 1950s, and the improvements to public transport facilities with progressive urbanization in the 1960s and 1970s, allowed the introduction of improved methods of tunnelling, including cut-and-cover techniques. In particular, improvements to excavation and drilling equipment, the availability of high-quality steel sections and reinforcement and the introduction of ready-mixed concrete transformed construction methods. A range of walling methods became available and alternative methods of installation were developed. Reinforced concrete piles were now installed by powerful rotary auger, steel sheet piling was driven by diesel hammer, vibrator or by hydraulic equipment, and new methods of walling such as diaphragm walling and methods of support such as ground and rock anchoring were introduced by innovative contractors and specialists.

This chapter describes cut-and-cover methods with reference to their application during the last 30 years on job sites in Europe and elsewhere.

While the choice between tunnel or surface construction may be clearly determined by the availability and value of land and the depth of the proposed permanent construction, the choice between bored tunnel and cut-and-cover construction methods may sometimes be less clearly defined. In other instances, however, the prevalent groundwater conditions, availability of construction site areas or the proximity of existing structures and their foundations, may pre-determine the use of either bored tunnels or cut-and-cover construction.

Although the construction methods of cut-and-cover work may appear to be more direct and free from the risks of bored tunnel construction, greater risk of subsidence due to shallow works and the disruption of traffic and services due to large-scale trench works may make cut-and-cover work less attractive. Megaw and Bartlett[1] listed the disadvantages of cut-and-cover in busy urban areas:

(*a*) Lengthy occupation of street sites with noise disturbances and disruption to access. This can be mitigated by mining excavation methods below a roof slab constructed at an early stage on the permanent tunnel walls. Roof slab construction allows speedy reinstatement of highways and surface works.

In special circumstances tunnelled headings may be used to build the permanent walls with the minimum of surface activity.

(*b*) In soft clays and silts, excavation in trench may be limited to maintain stability and reduce heave. Short-length working will be necessary and will increase construction and occupation time.

(*c*) Constraints on alignments by following existing streets may be undesirable, especially where small radius curves are introduced into metro construction. In some city area, basements which encroach beyond building lines may worsen the situation.

(*d*) Progress and cost of cut-and-cover schemes can be badly affected by diversion works to existing services, especially those inaccurately recorded or uncharted and disclosed during trenchworks. These works often require a break in the sequence of trench wall construction to divert the service and then construct the trench wall across the previous alignment of the service.

(*e*) Ground movement and subsidence of existing structures and services has to be avoided. Methods to reduce subsoil heave, loss of ground, and changes of groundwater level and flow entail cost and construction time penalties. The use of prestressed ground anchors, pre-jacking of struts, grouting works and groundwater recharge may all be necessary, particularly where sensitive or old buildings are nearby, and all have cost and time implications for cut-and-cover work.

The construction costs of cut-and-cover works increase significantly with depth, but the effect of construction depth on the cost of bored tunnel works is often much less. The choice of horizontal and vertical alignment for large-scale works such as metro construction additionally involves comparing the capital cost of alternative alignments using varying lengths of bored tunnel and cut-and-cover with the projected energy running costs of trains on those alternative alignments.

Four methods are available for cut-and-cover wall construction:

(*a*) Temporary support from braced or anchored steel sheet piling followed by permanent reinforced concrete wall construction.

(*b*) The Berlin wall method of temporary support using soldier piles and horizontal laggings, or sprayed concrete skin walls with bracing or anchoring followed by permanent reinforced concrete wall construction.

(*c*) Temporary concrete walls in contiguous, secant reinforced concrete piles or in situ diaphragm wall construction followed by permanent reinforced concrete construction.

(*d*) Temporary and permanent wall construction from walls in reinforced concrete secant piles, or cast in situ or precast diaphragm wall construction.

Either the top-downwards or conventional bottom-upwards construction method can be applied to cut-and-cover work, the wall of top-downwards construction usually being built by the methods in (*d*).

The choice of walling method depends on geology, depth of excavation and the presence of buildings or roads near the excavation. A review by Hulme *et al.*[2] of cut-and-cover walling methods for a large new transportation system showed the choices for each station or section of running tunnel on the Singapore MRT. Table 9.1 summarizes the walling methods used for the underground stations on the system. A similar comparison of cut-and-cover station walls on the initial Hong Kong MRT system was presented by McIntosh *et al.*[3] (Table 9.2).

Wall construction

Sheet pile walls

The traditional use of sheet piles in temporary soil support for cut-and-cover construction has been reduced by environmental pressures to avoid noise and

Table 9.1 Singapore Metro: construction methods for cut-and-over stations (Hulme et al.[2])

Station	Maximum depth of excavation (m)	Typical soil sequence*	Retaining system used	Special measures
Braddell	14·9	1F, G4	0·6 and 0·8 m diaphragm walls	
Toa Payoh	13·5	4F, 4K, G	Sheet piles	
Novena	14·7	½F, 14·2K, G	Sheet piles	
Newton	14·3	3F, 13K, G	0·8 m diaphragm walls	Jet grouting
Orchard	21	½F, G	Nailed slopes	
Somerset	16·2	2F, 8K, G	0·6 m diaphragm walls or sheet piles	
Dhoby Ghaut	16·1	1F, 10K, S	Sheet piles	
City Hall	22·3	3F, 2K, S3	King piles and shotcrete lagging	
Tanjong Pagar	17·9	½F, S	Slopes, anchors	
Outram	13·9	2F, 3K, S	8 m deep sheet piles over king piles and timber laggings	
Tiong Bahru	14·1	1F, S	King piles and shotcrete lagging	
Bugis	18·3	1F, 34K, O	1·2/1·0 m diaphragm walls	Lime piles
Lavender	16·5	3F, 20K, O	1·0 m/diaphragm walls	
Marina Bay	16·4	12F, 24K, O	Composite H pile/sheet pile	Underwater excavation

* F = fill
G = granite }
S3 = Jurong } including weathered rock
K = Kallang
S = boulder bed
O = old alluvium
Example: 3F, 13K, G = 3 m of fill overlying 13 m of Kallang deposit overlying granite (in this case completely weathered granite)

vibration due to pile driving in favour of the use of top-downwards techniques which favour walling methods that use both temporary and permanent soil retention. Nevertheless, reference to recent schemes such as the Singapore Metro and London's Limehouse Link shows that sheet piles can be effective in less urbanized areas, where excavation depths are limited to the order of 15 to 16 m and where soil conditions allow economical pile driving. Economical applications include river crossings and areas where groundwater is high, sites which are some distance from existing structures where the lack of flexural stiffness in the sheet pile section allows soil movements which would be inhibited by other, stiffer walling systems, and sites where there is sufficient working space to allow craneage to withdraw the sheeters after their temporary use. Sheet piles for deep excavations are less attractive in city centres where artificial or natural ground obstructions to driving may be found.

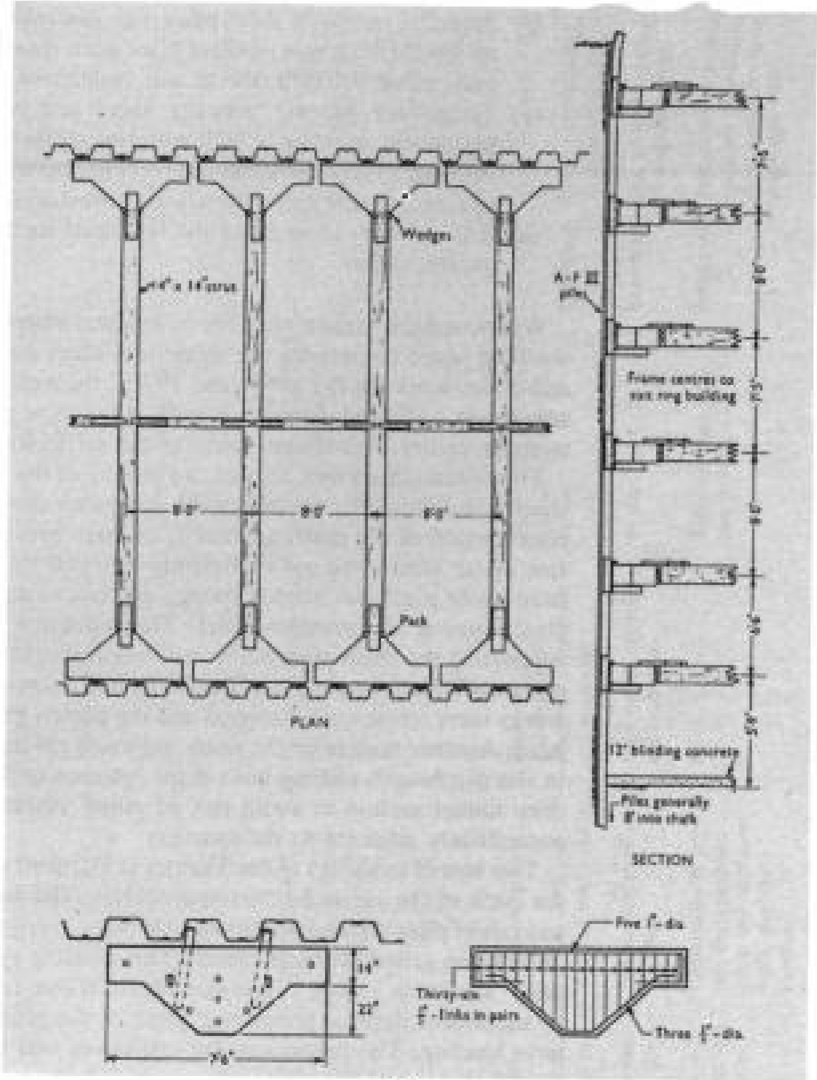
To summarize, the disadvantages of using sheet piles in the continuous walls of cut-and-cover works are:

(*a*) Noise and vibration during installation.
(*b*) Support is provided only during construction and permanent works are required for tunnel construction.
(*c*) Obstructions reduce driving efficiency and increase risk of damage due to vibration.
(*d*) Adequate allowance must be made for installation tolerances. The initial piling line must make allowance for verticality tolerance to ensure adequate width between sheet pile walls to accommodate the permanent works.
(*e*) Ingress of groundwater through pile clutches and split clutches may cause delay or even local failure.

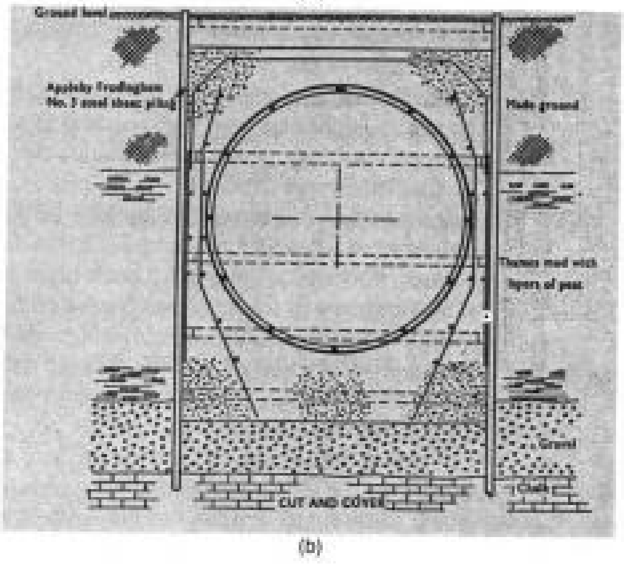
Table 9.2 Hong Kong Metro stations — adopted construction methods for cut-and-cover works (McIntosh et al.[3])

Station	Depth of excavation (m)	Cover to roof slab (m)	Depth to rock from surface (m)	Engineer's assumed method	Temporary works	Proximity to buildings	Walls	Constructor sequence	Special measures
Choi Hung	20	0–3	*	Temporary Berlin wall with preboring or diaphragm walls	Permanent walls	One end only	Hand-dug interlocking caissons	Top down	Skeletal roof of cross-beams with precast T beam infills
Diamond Hill	22	3	†	Steel I sections king piles and intermediate sheet piles	Permanent walls	No	Hand-dug caissons for steel piles and concrete jack arches	Top down	Walls are to be removable for future widening
Wong Tai Sin	24 max.	3·5–6·5	*	Diaphragm walls	Permanent walls	Medium height housing blocks	Diaphragm walls	Top down	Roof was clear spanning during excavation with concourse suspended from it
Lok Fu	27	2	0–30	Bored tunnel	Berlin wall of steel piles and concrete lagging — ground anchors	High rise housing block	In situ	Bottom up	Dewatering by ground treatment and wells
Kowloon Tong	18	2	†	Diaphragm walls	Part permanent walls — part Berlin type-ground anchors	No	Part diaphragm wall, part in situ	Bottom up	—
Shek Kip Mei	18–24	1·5–6·5	0–30	Open cut in rock, sheet piling with grouted anchors in soil	Berlin wall, part strutted, part ground anchors	High rise housing blocks and schools	In situ	Bottom up	Short length of station platforms in bored rock tunnel
Prince Edward	28	2	16–30	Diaphragm walls	Permanent walls	High rise commercial and residential	Benoto type secant piles and hand-dug caissons	Top down	Extensive grouting was used, plus dewatering and limited recharging
Argyle	25	3–5	†	Diaphragm walls	Permanent walls	High rise commercial and residential	Benoto type secant piles	Top down	Columns extended to underlying rock and vertically anchored; some areas of slab also anchored; grouting to walls; use of recharge wells
Waterloo	28	2	0–27	Part open cut, part diaphragm walls	Permanent walls	High rise commercial and residential	Benoto type secant piles to rock, then in situ	Top down	Underpinning to walls
Jordan	18–23	0–4.5	4–20	Diaphragm walls and rock anchors, in situ underpinning	PIP pile walls and 7 levels of steel strutting	High rise commercial and residential	In situ	Bottom up	Half of station anchored to underlying rock
Tsim Sha Tsui	17–21	3·5–7·5	9·5–13·5	Diaphragm walls and rock anchors, in situ underpinning	PIP pile walls and steel strutting	High rise commercial and residential	In situ	Bottom up	Part of station anchored to underlying rock
Admiralty	25	0–3	20	Diaphragm walls on rock; rock anchors and in situ underpinning	Combination of open cut, anchored sheet piling and permanent walls, also slurry trenches	No	Part diaphragm wall, part in situ	Bottom up	Part of station anchored vertically to underlying rock; underpinning to diaphragm wall
Chater/ Pedder	28	3	33‡	Diaphragm walls	Permanent walls with struts	High rise commercial and hotels, low rise historic buildings	Diaphram walls	Top down	Special measures to construct walls and groundwater recharging

* Not known. † Not known, large boulders. ‡ But rock level not proven in some sections.



(a)



(b)

Fig. 9.1. First Dartford Tunnel, cut-and-cover construction: (a) details of bracing frames; (b) cross-section (Kell (see bibliography))

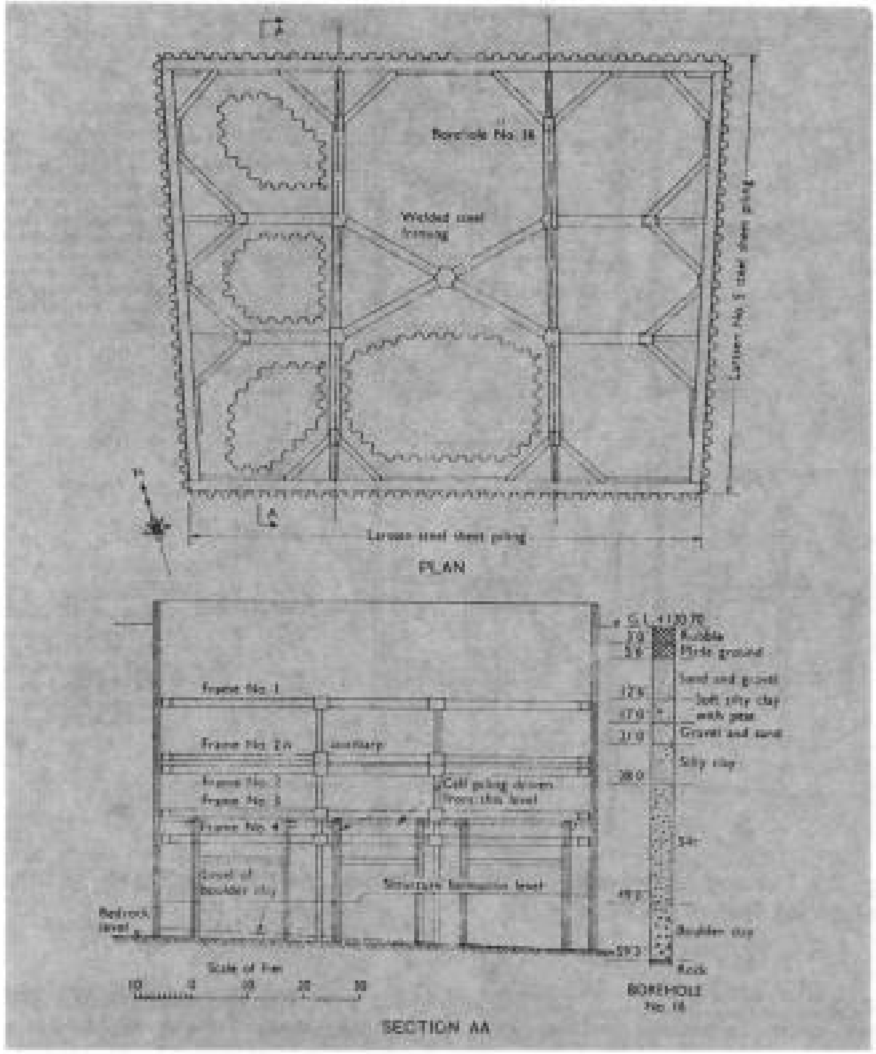


Fig. 9.2. Clyde Tunnel: plan and vertical cross-section of completed partial cofferdam (Haxton and Whyte[6])

second frame inserted and pre-loaded. Erection of the third frame was completed in the dry, and the fourth and final frame was built in short trenches where boulder clay did not exist above bedrock, the silt being excavated and replaced within sheet piled cells driven between the upper cofferdam frames by mass concrete. Excavation was completed to formation level over the remainder of the cofferdam (Fig. 9.2).

Anchored or braced king post walls

Although vertical soldier piles or king posts with horizontal poling boards spanning between them had been used most effectively in the sandy subsoils of Berlin in the 1930s, it was the development of powerful mechanical augers, anchoring methods and methods for spraying concrete which promoted its post-ware use. Unrestricted, wide, anchored excavations were now possible, and metro schemes, particularly in Germany, adopted the method for temporary soil support. The method is most economical where groundwater is absent or can be reduced by dewatering. Fig. 9.3 shows excavation below bracing with reduction of groundwater by pumping. Bigey *et al.*[7] referred to the 'methode Hambourgeoise'. The construction method for this adaptation of the original Berlin method is shown

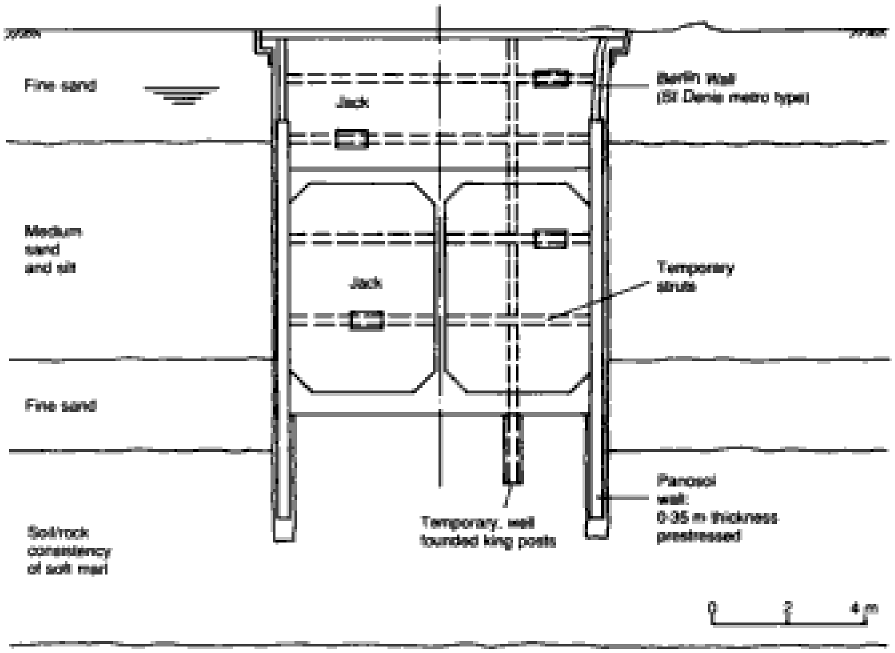


Fig. 9.6. Composite Berlin wall with prestressed precast diaphragm wall, Fukuoka Metro, Japan (courtesy of Soletanche)

between lateral ground pressure from the walls and the outward horizontal arch thrust.

Excavation was initially made to arch springing level between temporary anchored king post walls (Fig. 9.7). From this level, 2·5 m dia. contiguous piles were augered into bedrock. A dumpling between the contiguous bored piles could not be removed until the arch thrust had been developed from the backfill load over the arch. Successive stages of dumpling excavation and filling were carefully sequenced. Two hundred piles were installed at a peak rate of four piles per day.

Secant pile walls

Secant pile construction, alternate male and female piles interlocked to form a wall, provides an efficient and economical walling system to moderate and greater depths in a wide range of soil and groundwater conditions. A permanent wall is constructed to allow soil support during construction. The principal advantages

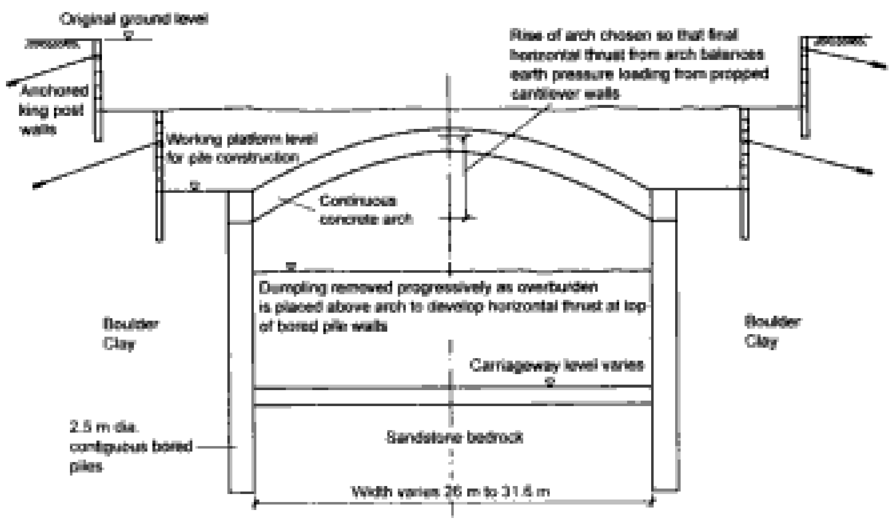


Fig. 9.7. Vertical cross-section of cut-and-cover structure, Liverpool approach to Mersey Kingsway Tunnel; contiguous bored pipe walls propped by concrete arch (after Megaw and Brown[8])

of the system are:

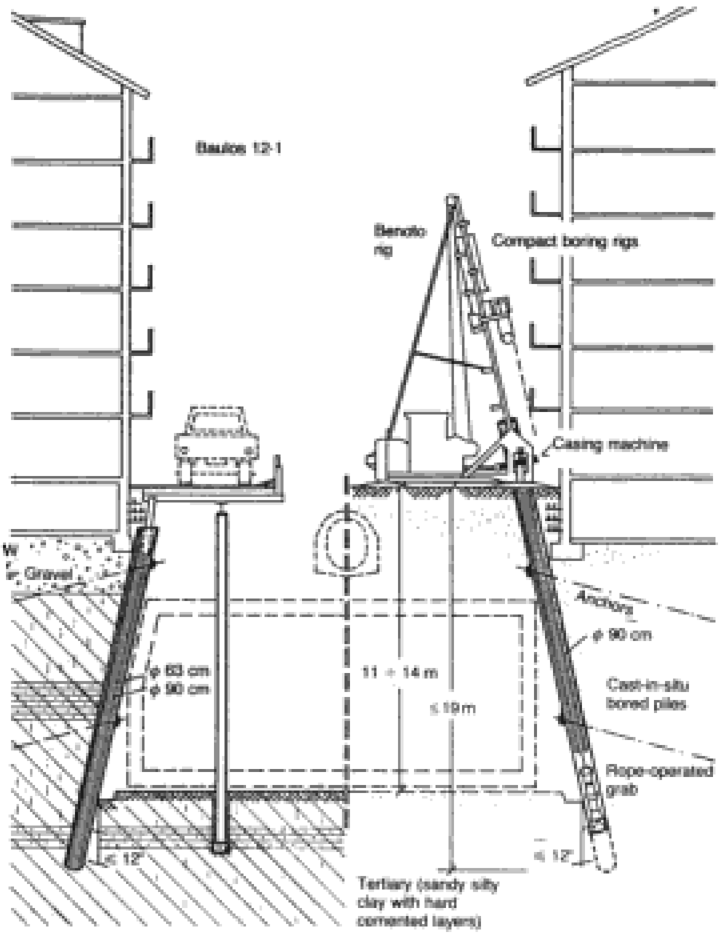
(*a*) Permanent structural walls are constructed in one operation ahead of excavation.
(*b*) The walls are substantially watertight.
(*c*) Excavation methods using heavy-duty rotary augers/buckets, temporary casings and casing oscillators or hammer grabs are highly efficient in hard soil and rock conditions. Excess heads of water or slurry within the temporary pile casing can be used to overcome onerous groundwater conditions.
(*d*) Good verticality tolerances can be achieved with twin-walled temporary casing, modern Benoto rigs augers and casing oscillators. Tolerances of the order of 1 in 200 to 1 in 300 may be expected, depending on soil conditions. Little overbreak may be expected and the pile finish is uniform.
(*e*) Pile installation is comparatively noise and vibration free although some vibration is inevitable if hammer grabbing is resorted to through dense granular or rock strata.
(*f*) Loss of ground during excavation is generally small. In soft silts and clays or where sand with a high piezometric head is penetrated, the temporary casing affords continuous lateral support and the stability of the base of the bore within the casing may be continuously retained by a head of water or slurry within the casing to ground level.
(*g*) Vertical loading of secant walls is viable because of the reliability of good soil density below the concreted base of the pile.
(*h*) Temporary gaps may conveniently be left in the secant pile wall to allow service access. Piles are temporarily filled with sand after boring at these locations and are concreted later.

A successful cut-and-cover construction in secant piling was completed in 1975 near London on the Piccadilly Line underground railway extension[9] At planning stage, cut-and-over construction was chosen for the 4 km length in preference to bored tunnelling because:

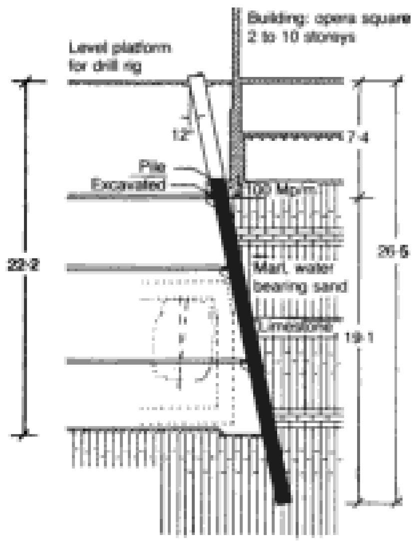
(*a*) a much greater length of railway reconstruction could have been necessary to obtain correct vertical alignment for the deeper bored tunnel option compared with the length required for cut-and-cover construction
(*b*) the bored tunnel, necesarily deeper than cut-and-cover, would have proved less convenient in operation, with more lifts and escalators
(*c*) specialist bored tunnel construction labour was not readily available
(*d*) cost assessments for the schemes showed the bored tunnel to be 10% more expensive (the cost assessments were later confirmed by contract prices.)

The method chosen by the Engineer at tender stage used in situ diaphragm walls to support the soil during construction and permanently. Excavation would have been made in two stages, initially to roof soffit level and then by mining beneath the completed roof slab to formation level. Sheet piling with normal reinforced concrete permanent structure was intended and finally constructed for cut and cover sections some distance from residential property. The diaphragm walls proposed by the Engineer were, however, substituted by secant pile walls at construction stage.

The change was made for three reasons. First, the secant pile method is a dry excavation process and the change avoided bentonite slurry spillage from excavation arisings during haulage through residential neighbourhoods. Second, due to planning requirements for the diaphragm process, some lengths of wall were required in advance of the main work and this would have required additional mud station set-ups. Third, the number of existing disused services and ducts which penetrated the walls would have led to difficult obstructions for diaphragm wall excavation with large slurry losses, whereas secant pile excavation using Benoto rigs did not suffer in this way.

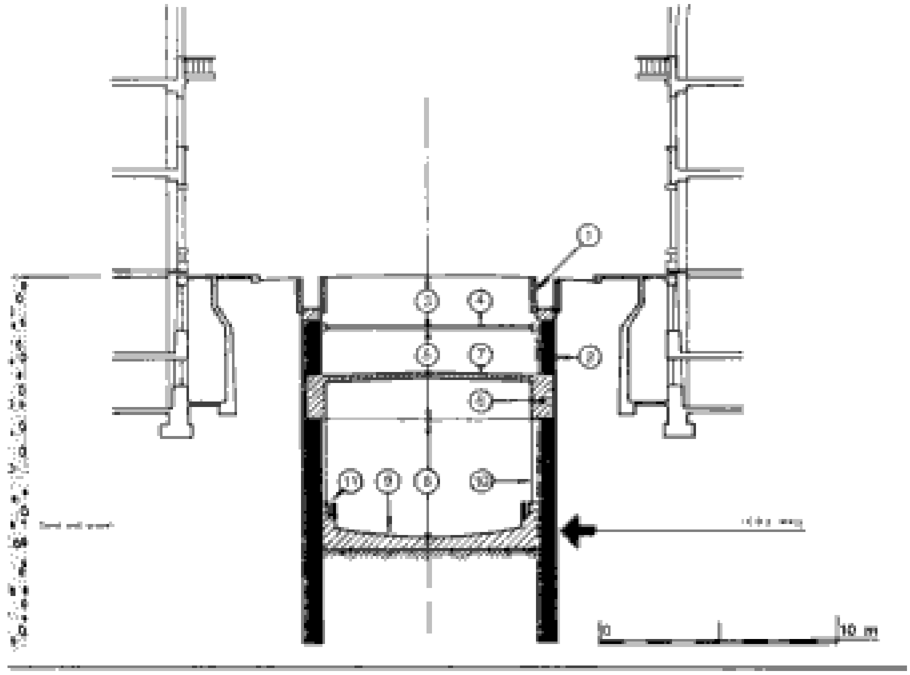


(a)



(b)

Fig. 9.10. Inclined secant pile construction for metro construction: (a) cross-section with temporary roadway, pile construction by Benoto rig, Munich metro; (b) inclined walls, Frankfurt metro



I - Excavation for the guide walls ①.

II - Building of the I.C.O.S. diaphragm ② forming the permanent side walls of the tunnel.

III - Partial excavation of the soil between the walls ③, placing of the temporary bracing ④, continuation of the excavation ⑤ in order to cut the keys ⑥ for the roof of the tunnel ⑦.

IV - Excavation under cover of the tunnel ⑧: the I.C.O.S. wall protects the excavation.

V - Construction of the invert ⑨, of the columns, withstanding uplift pressures on the invert, ⑩ and of the utility pipes ⑪.

Fig. 9.12. Construction sequence used by Icos in original cut-and-cover works for Milan Metro (courtesy of Icos)

the disturbance to highway and traffic by early reinstatement of the carriageway above the permanent cut-and-cover roof as excavation and invert construction proceeded beneath it. This method now familiar as top-downwards construction in both basement and cut-and-cover construction, became the basis of metro construction by Icos in many cities worldwide and, over time, by their competitors. Innovations were introduced by Icos[13] and later by others in Milan, on number 1 and 2 lines, structural steel column elements were lowered into barrettes, sections of Icos wall below tunnel invert level, to be used as structural support for reinforced concrete mezzanine floor and roofworks to the tunnel. Details of this construction are shown in Fig. 9.13.

Icos also introduced castellated sections of diaphragm walling on metro construction in Milan (Fig. 9.14). The section, of greater breadth than the straight wall, provides considerably enhanced strength in bending. More recently, panel joints which can transmit vertical shear and tension from one panel to its neighbour have allowed this efficient plan shape to be fully exploited with a continuous wall section.

In the UK, diaphragm walls were introduced in 1962 for use on cut-and-cover construction for a road underpass at Hyde Park Corner in London.[14] Due to the proximity of a nearby hospital, the engineer had decided that driven sheet piling could not be used because of installation noise, and vibration so contiguous bored reinforced concrete piles were specified. Icos diaphragm walls were introduced

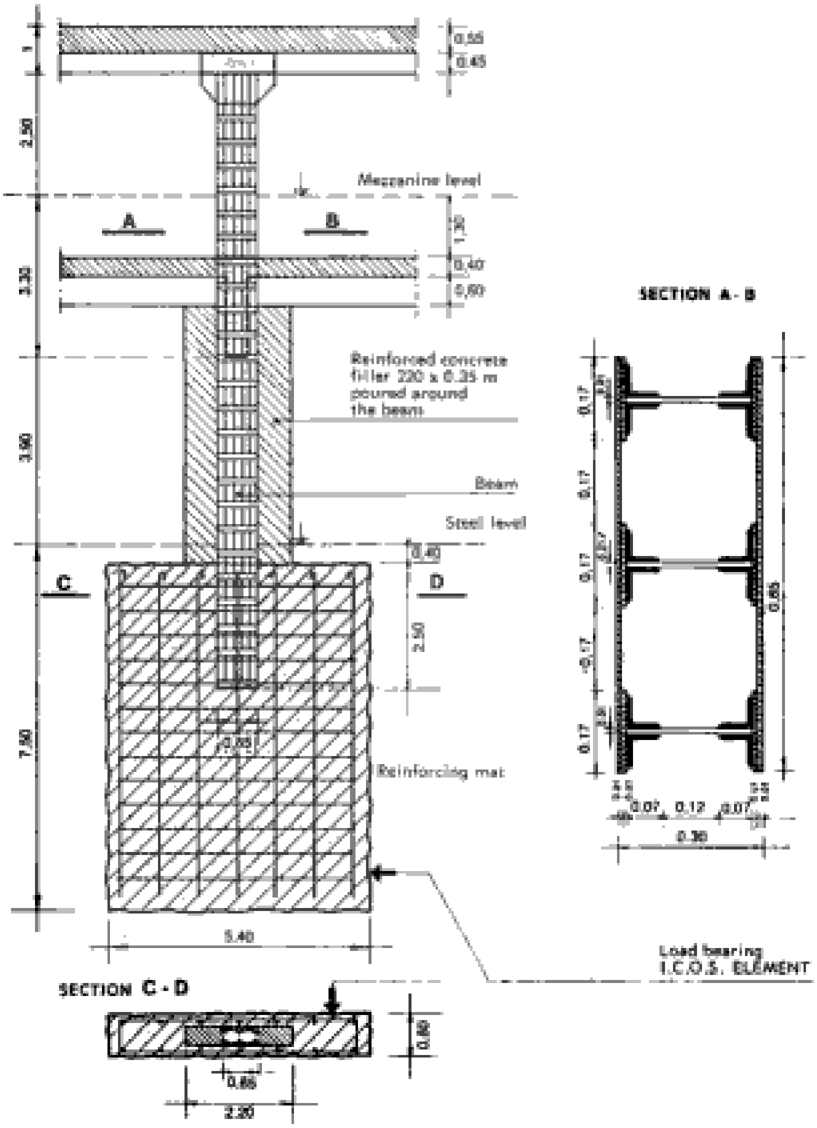


Fig. 9.13. Milan Metro: cross-section of fabricated steel columns cast into reinforced concrete base using diaphragm wall techniques for vertical load-bearing units (courtesy of Icos)

by the main contractor but neither the contiguous piles of the original scheme nor the alternative of diaphragm walls were considered as part of the permanent subsoil support. This diaphragm wall scheme was also successfully used to underpin the existing hospital walls where the underpass diaphragm wall was built less than 1 m from the main hospital walls and more than 8 m below it. These measures later became standard practice for such locations. The use of short panels, increased wall reinforcement, pre-loaded struts and reduced open lengths of main excavation limited horizontal and vertical soil movements and wall movements to less than 3 mm.

The reluctance of designers outside Europe to use diaphragm walling as a means of both temporary and permanent soil support persisted in the 1960s and 1970s. The cut-and-cover for the Calcutta Metro[15] used diaphragm wall only to resist buoyancy under permanent load conditions. A factor of safety of 1·5 was used against flotation with full soil cohesive strength being allowed in calculating wall adhesion to the clay subsoil.

In the UK, diaphragm walls were similarly used to resist buoyancy but with

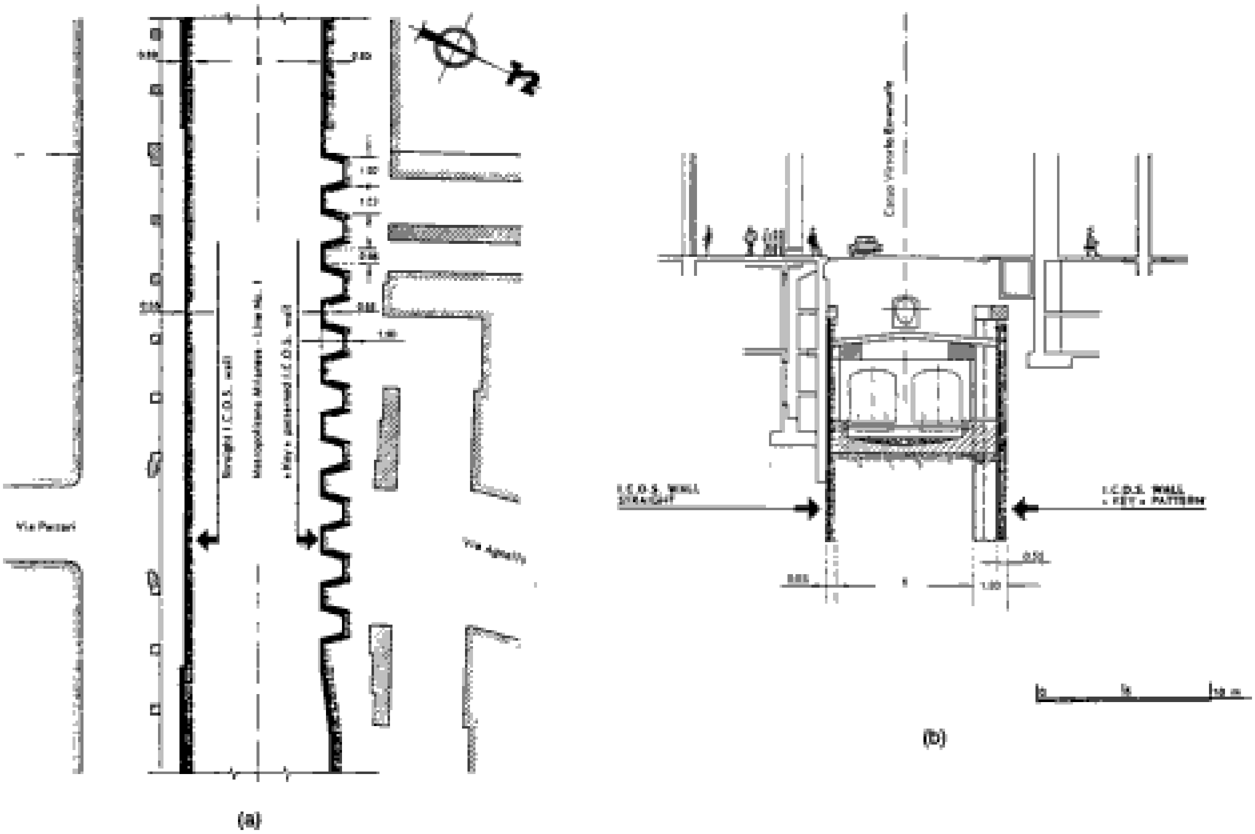


Fig. 9.14. Cut-and-cover construction using castellated plan shape diaphragm walls on one side: (a) plan; (b) cross-section (courtesy of Icos)

no assumed contribution to flexural strength of the rectangular cut-and-cover box structure housed between the walls. This reluctance to use the flexural strength of the diaphragm wall after construction was evident in the second tunnel crossing of the Thames at Dartford in 1972. By this time, diaphragm wall construction had gained wide acceptance in the UK and the walls at Dartford were extended to depths in excess of 30 m to minimize the length of the driven tunnel. Nevertheless, the flexural strength of the diaphragm walls was ignored for the permanent works.

The 800 mm and 1 m diaphragm walls at Dartford were excavated by kelly mounted hydraulic grabs through soft alluvial silty clays and dense gravels into hard chalk. Five frames of bracing were necessary to reduce flexural stresses in the wall as bulk excavation proceeded to the deepest sections at the junction with the bored tunnel, almost 30 m from ground level. These high flexural stresses were correctly anticipated by the wall designers who appreciated the relatively large wall movements which would be necessary to mobilize relatively small passive resistance in the soft clays at formation level and immediately below it. Following the innovation used by the contractor for the first Dartford Tunnel, concrete, hammer-headed struts were used throughout to brace sheet pile walls in the cut-and-cover length, thus avoiding the need for separate walings.

Shortly after the Dartford Tunnel cut-and-cover works had been constructed by diaphragm walling, the station at Heathrow for the Piccadilly Line extension was built in cut-and-cover box, the diaphragm walls acting as both temporary and permanent soil-retaining walls. Jobling and Lyons[16] said that cut-and-cover construction was chosen in preference to bored tunnelling for three reasons:

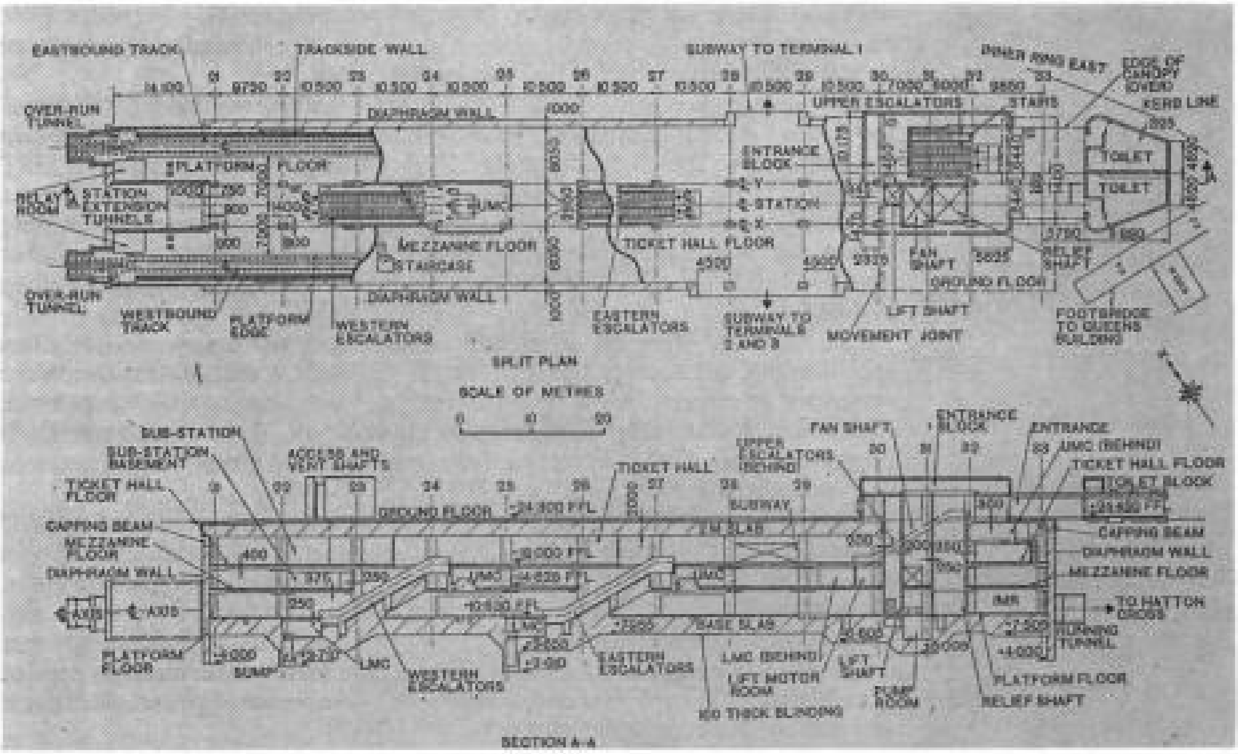


Fig. 9.15. Heathrow Central station, plan and vertical section (Jobling and Lyons[16])

(*a*) the use of station tunnels with space between escalator access tunnels would have used more plan area than the cut-and-cover box and left insufficient space for further station development for surface railways.
(*b*) It was considered very costly to provide foundations for proposed building development over driven tunnels.
(*c*) Subsoil strata, flood plain gravels overlying London clay, favoured box construction in diaphragm walling.

The box shown in Fig. 9.15 was typically metro station size, 131·5 m long and 22 m wide with a depth to formation level of 17 m to keep the tunnel drive below the flood plain gravels and within the London clay. The 1 m thick diaphragm walls, propped by three frames, were designed using earth pressures based on a value of $K_a = 0{\cdot}25$ for the gravel and $K_0 = 0{\cdot}75$ for the clay and a design groundwater level of 2 m below ground level. The base to the box, between 1·9 and 2·575 m thick, was designed as a beam on an elastic foundation using a modulus of elasticity of 107 000 kN/m^2 for London clay, and a modulus of subgrade reaction of 9055 kN/m^2. The design factor of safety against flotation of the box was 1·2 on completion, but a rubble drain beneath the base raft adjacent to the diaphragm walls restricted groundwater pressure on the raft during construction. The raft was not designed to resist hydraulic forces until loaded by the main internal columns.

The walls were temporarily braced by the frames of Rendex no. 6 struts at floor level and by 300 by 300 mm timber struts at platform level. A maximum deflexion of only 5 mm at the top of the diaphragm walls had been specified and pre-loading of the top frame and successive shimming of the second frame was necessary to achieve this. Strut loads were monitored; this showed that middle frame loads exceeded design values prior to the lower frame being placed. This lower frame consisted of timber struts spanning from the central raft section to the walls which

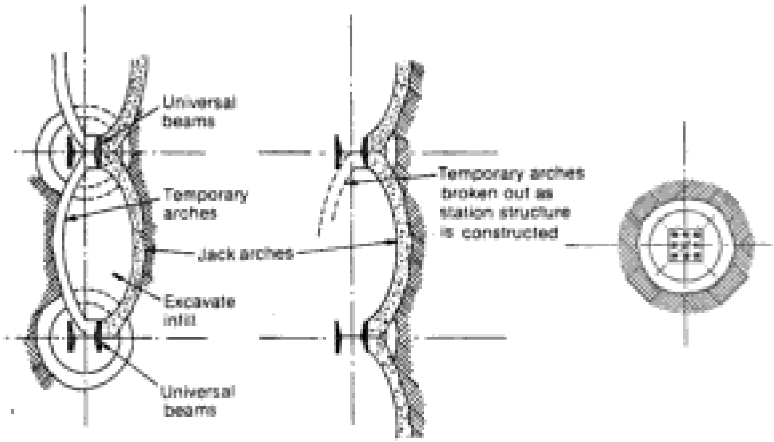


Fig. 9.17. Diamond Hill station, Hong Kong Metro: jack arches between soldier piles installed in hand-dug caissons (McIntosh et al.[3]*)*

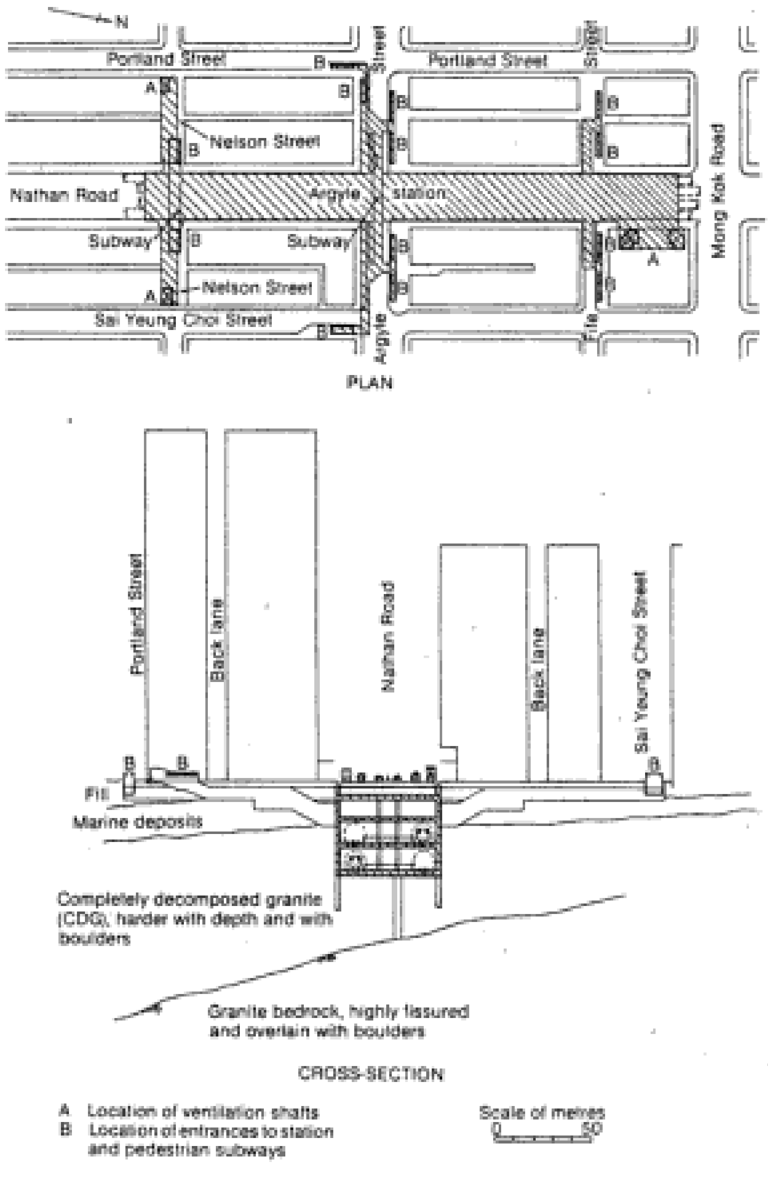


Fig. 9.18. Argyle station, Hong Kong Metro: plan and vertical section showing soil profile and location of existing buildings (McIntosh et al.[3]*)*

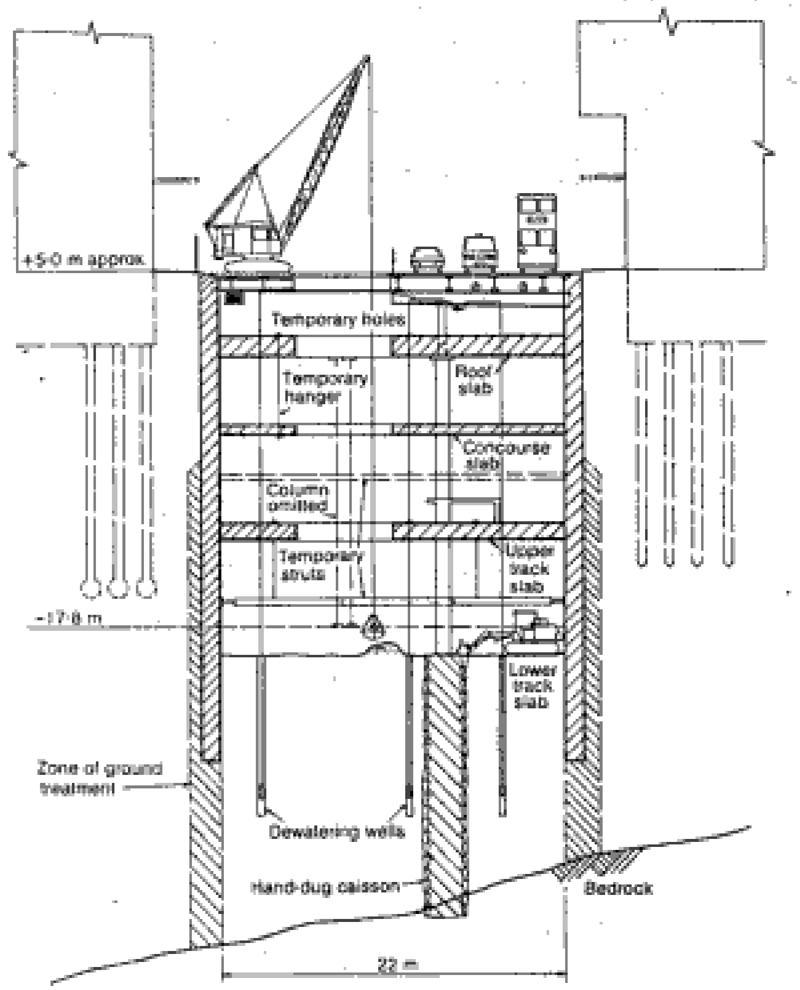


Fig. 9.19. Argyle station, Hong Kong Metro: typical cross-section showing top-down construction for lower track slab using secant pile walls and ground treatment to secure cut-off to bedrock (McIntosh et al.[3]*)*

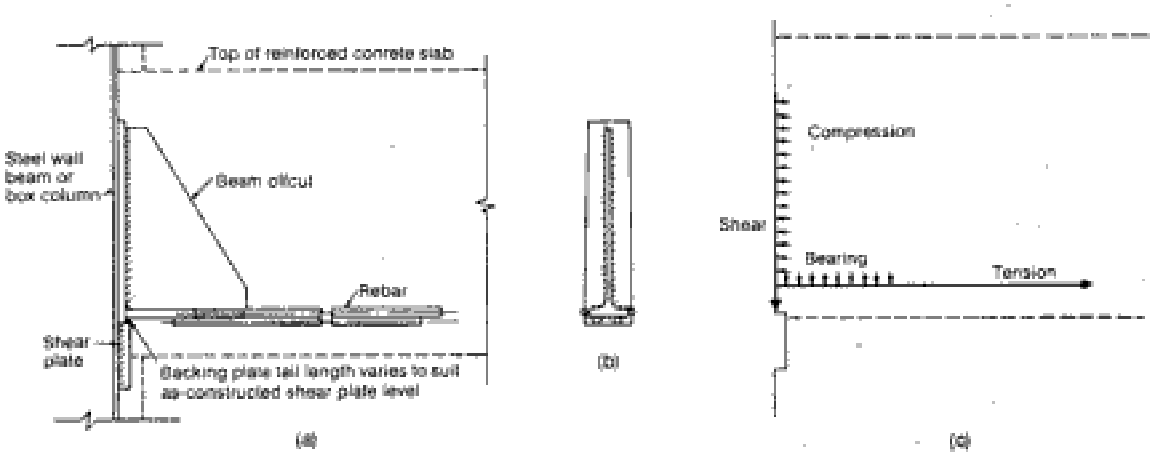


Fig. 9.20. Argyle station, Hong Kong Metro: detail of shear shoe and plate to transfer high loads from slabs to secant pile walls: (a) side view; (b) end view; (c) force transfer (McIntosh et al.[3]*)*

favoured the top-down construction method. Due to difficult subsoil conditions with large granite boulders and the need to penetrate bedrock, secant piles installed by Benoto rigs were chosen by the contractor. The pile, 1·2 m in diameter and bored at 1 m centres, were reinforced in both male and female piles by 914 × 305 universal beam sections. Fig. 9.18 shows a plan and cross-section of the station and Fig. 9.19 shows a typical cross-section during excavation for the lower track. To reduce settlements due to dewatering and subsidence of adjacent buildings, a bentonite cement and silicate grout curtain was made below the toe of the structural wall to form a cut-off to the box from the high water table where this could not be achieved by the Benoto rigs. This grout curtain produced excellent results, restricting the abstraction rate from the whole box to less than 0·0045 m^3/s under a differential head of more than 20 m. Design of the walls during construction and in the permanent case used active and at-rest earth pressures with plastic methods and limit state checks. Station columns, heavily loaded, in some cases up to 15 000 kN, comprised 1000 × 800 mm steel boxes in 50B steel, 20 m long. Fig. 9.20 shows a shear shoe and plate arrangement used to transfer high loads from slabs to walls and columns.

At Tsim Sha Tsui station, shown in Fig. 9.21, the bottom-up construction sequence with PIP piles was used by the contractor in preference to the original, pre-bid, top-down method using diaphragm walls, with the following advantages:[3]

(*a*) Where rock existed above formation level, difficult underpinning work was avoided.
(*b*) Less noise and vibration was caused by PIP piling.
(*c*) The PIP wall was narrower than the diaphragm wall.
(*d*) The PIP wall provided drier conditions in which to build the permanent structural box.

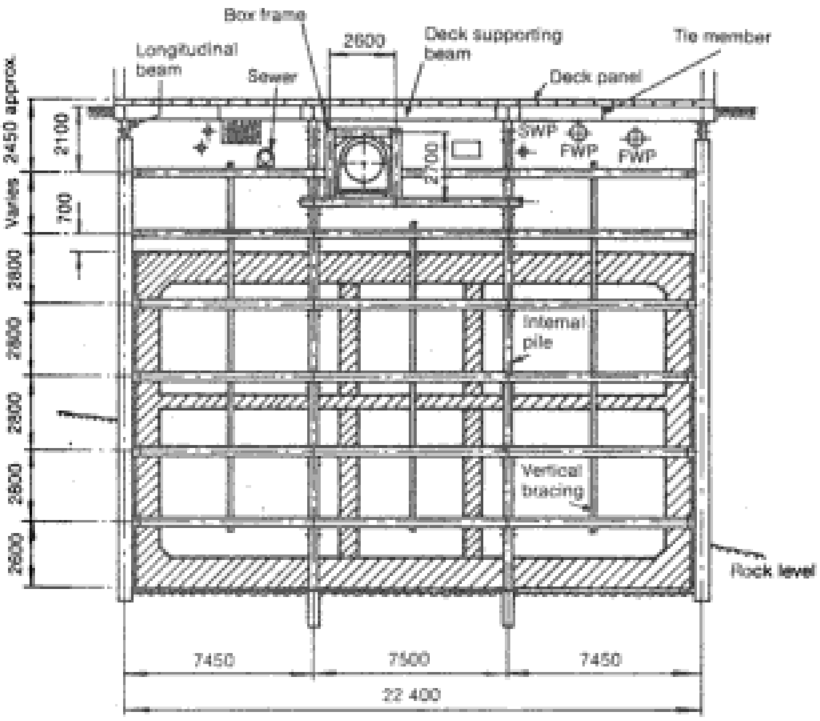


Fig. 9.21. Tsim Sha Tsui station, Hong Kong Metro: typical cross-section of cut-and-cover station constructed by bottom-upwards method showing temporary deck support (McIntosh et al.[3])

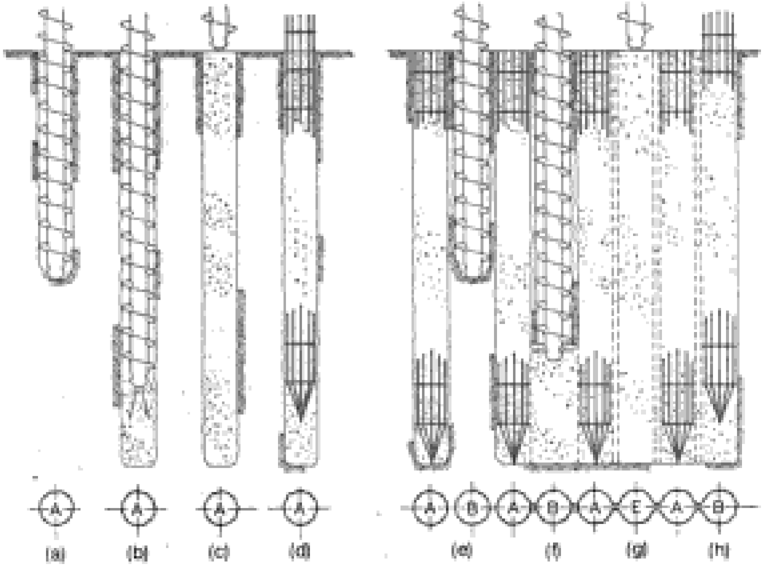


Fig. 9.23. Tsim Sha Tsui station, PIP construction sequence: (a) augering; (b) withdrawing auger and injecting mortar; (c) completing injection; (d) inserting reinforcement cage or steel column section; (e) augering; (f) withdrawing auger then injecting and jetting mortar; (g) completing injection and jetting; (h) inserting reinforcement cage or steel column section (McIntosh et al.[3]*)*

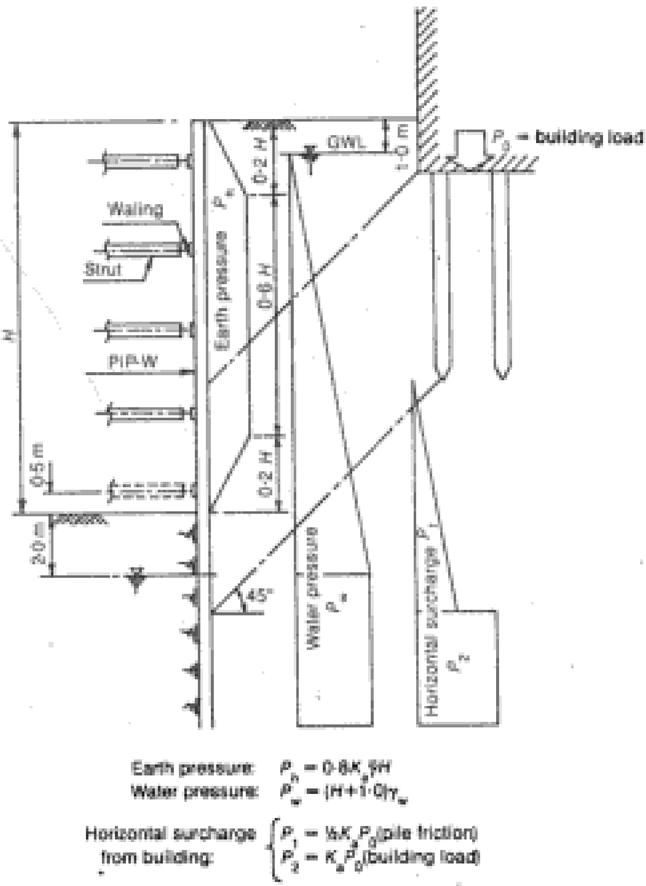


Fig. 9.24. Tsim Sha Tsui station: earth pressure, water pressure and horizontal surcharge loading diagrams (McIntosh et al.[3]*)*

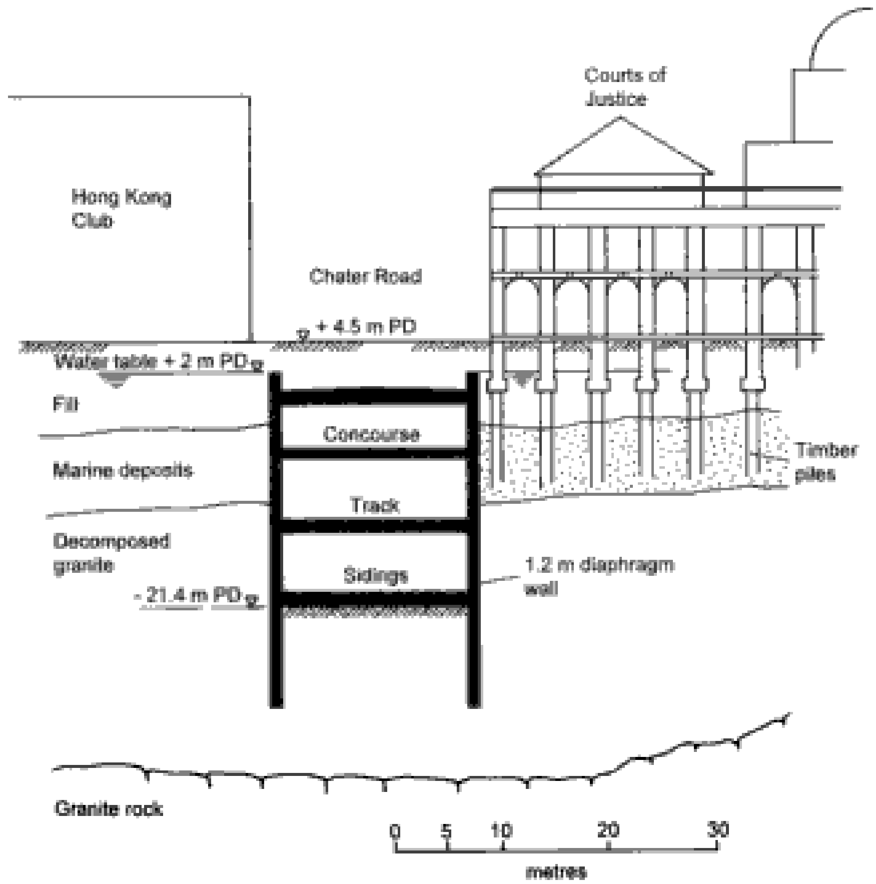


Fig. 9.25. Cross-section of Chater station, Hong Kong, showing soil profile and location of existing structures (Davies and Henkel[17])

cross-section of the temporary support and the permanent works in Fig. 9.21 shows the six frames of pre-loaded H steel strutting at centres of 2·1 m.

Considerable settlements to existing buildings resulted from the initial Hong Kong MRT construction. Settlement was primarily due to dewatering, diaphragm wall panel installation and bulk excavation. The soil conditions may be particular to Hong Kong although the cause of the settlement may apply elsewhere in soils with similar properties.

Davies and Henkel[17] referred to the construction of Chater station and settlements of the existing Courts of Justice building. A section of the construction and soil profile is shown in Fig. 9.25. The permeability of marine deposits was of the order of 10^{-7} m/s compared with 10^{-5} m/s for the underlying decomposed granite. Wide variations in drawdown were expected due to local variations in the geological profile, and preliminary studies showed unfavourable dewatering settlements could result. Pumping tests had shown that for each 1 m of drawdown a settlement of 4 mm would result. A system of groundwater recharge was used, however, both at the Courts and elsewhere, with beneficial results.

Settlements due to diaphragm wall panel excavation had unpredicted and serious consequences. Fig. 9.26 shows the extent of movements of three points, D, E and F, spaced 6 m, 15 m and 24 m, respectively, from the diaphragm wall. The progressive settlement of the points, even as diaphragm wall installation was completed well away from the vicinity of the points, is clear. This is a most unusual phenomenon. Measurements of soil movement due to diaphragm wall installation before and since in London,[18] Seville[19] and Singapore, in widely different soils, show similar, very small soil movements of a few millimetres due to panel

The repetitive nature of cut-and-cover construction, in which wall panels are progressively cast, bulk excavation made and bracing frames inserted, allows any observed production or technical improvement in walling or strutting to be introduced at an early stage as the work proceeds. The principles of observational soil mechanics as described by Peck[21] are particularly relevant to cut-and-cover construction. An example of the successful use of this technique is the Limehouse Link highway tunnel in East London which was built in the early 1990s. The original design of the top-downwards construction required temporary 1350 mm dia. steel props between diaphragm walls on each side of the cut-and-cover box below roof level. The props were lifted into place using hoists supported from the soffit of the roof slab. Excavation then continued to formation level below the line of struts. This excavation was slow and costly due to the presence of the struts.

The observational method was applied progressively in a number of stages. Initially, props were destressed and removed one at a time as wall movements were measured. Since wall displacements were small, a new section with 'soft' props was installed with a small gap allowed at the end of the strut prior to load take-up. Since movement again proved to be very small as excavation was taken below the props to formation level, the mid-height props were omitted and excavation was made to full depth prior to the installation of a strut at blinding level. The monitored wall movements were still very small and this allowed the omission of the centre struts completely. Eventually the blinding struts were also omitted. Contingency struts were always kept available, but were not needed. The trigger level for maximum wall movement was defined as 70 mm but the maximum recorded movement was 11 mm, and generally readings were less than 7 mm. Considerable savings resulted from avoiding the use of these heavy props.

Precast diaphragm walls

Precast concrete panels were introduced into diaphragm wall works in France by the firms Bachy and Soletanche during the early 1970s. Each company obtained patents for its particular technique. The innovation found early application in cut-and-cover construction and was used in Paris for underpass and metro construction, and in both Lille and Lyons for metro construction. The technique has not been used in the UK, and appears to have found less application in recent years in France.

In the Far East, in Hong Kong and Thailand, thick, heavily reinforced in situ diaphragm walls are preferred; in Japan there are only a few examples of precast walls; and there are no known precast walls in the USA. This lack of acceptance of a potentially attractive innovation is probably due to a unit cost disadvantage between in situ and precast walls. The introduction of the Hydrofraise machine and its use by the largest diaphragm wall contractors may also have detracted from the popularity of precast walls; the reverse circulation process cannot be economically applied when grout is used as the stabilizing fluid during excavation.

Two principal systems have been developed: the Panosol system by Soletanche and the Prefasil system by Bachy. Each method uses a cement–bentonite slurry to envelop the precast concrete units. After panel excavation is complete, however, the Bachy system replaces the bentonite slurry used during the excavation phase. The Soletanche system uses cement–bentonite slurry throughout, with any mud change between excavation and the final phase of placing the precast unit.

The principal feature of the precast diaphragm wall is the absence of any surface finishing subsequent to its exposure after bulk excavation. On bulk excavation the cement–bentonite slurry strips away from the inside surface of the wall to reveal the precast concrete surface, to true alignment. Fig. 9.28 shows the junction between in situ and precast diaphragm wall construction on the Cairo Metro.

The use of prefabricated diaphragm walls with the Panosol method for metro cut-and-cover construction was described by Namy and Fenoux.[22] They noted two fundamental disadvantages of in situ reinforced concrete diaphragm walls:

Fig. 9.28. Cairo Metro: junction between precast and in situ diaphragm walls

(*a*) The surface finish and quality of excavation of the wall depends on subsoil conditions.
(*b*) The waterproofness of the concrete and the joints may be inadequate.

The development of precast wall methods offers several advantages compared with in situ diaphragm wall construction:

(*a*) Site nuisance is reduced by more rapid execution. The sequence of panel excavation is simplified by successive panel excavation, whereas in situ diaphragms frequently use primary, secondary and intermediate panel excavation sequences to allow hardening of concrete. Remedial works in breaking down walls to level or to profile are largely unnecessary.
(*b*) Site concreting operations and stop end extraction are avoided.
(*c*) In the permanent phase, constructional thicknesses are reduced by the improved concrete qualities brought by precasting (a 400 mm precast wall can be equivalent to a 600 mm in situ wall panel). By incorporating water bars into the precast panels, good wall finishes and better waterproofness are possible.

The use of precast wall panels enables prefabricated units to be made up with soldier beams for temporary soil retention above the precast wall. An example of prefabricated, precast diaphragm construction using the Panosol system is shown in Fig. 9.29 in cross-section. The works, an extension of the Paris Metro in the heart of St Denis, extend 500 m along a confined route, 12 m wide wall-to-wall, bordered by old, delicate buildings. The congested working site is shown in Fig. 9.30, with heavy plant, six rigs within 200 m, precast panels ready for placing and muck-away lorries.

The subsoils consisted of fill, gypsiferous marls and clayey greensands overlying St Ouen limestone. The marls acted as an upper acquifier close to street level, and the lower aquifer of limestone has its peizometric head near the top of the greensand.

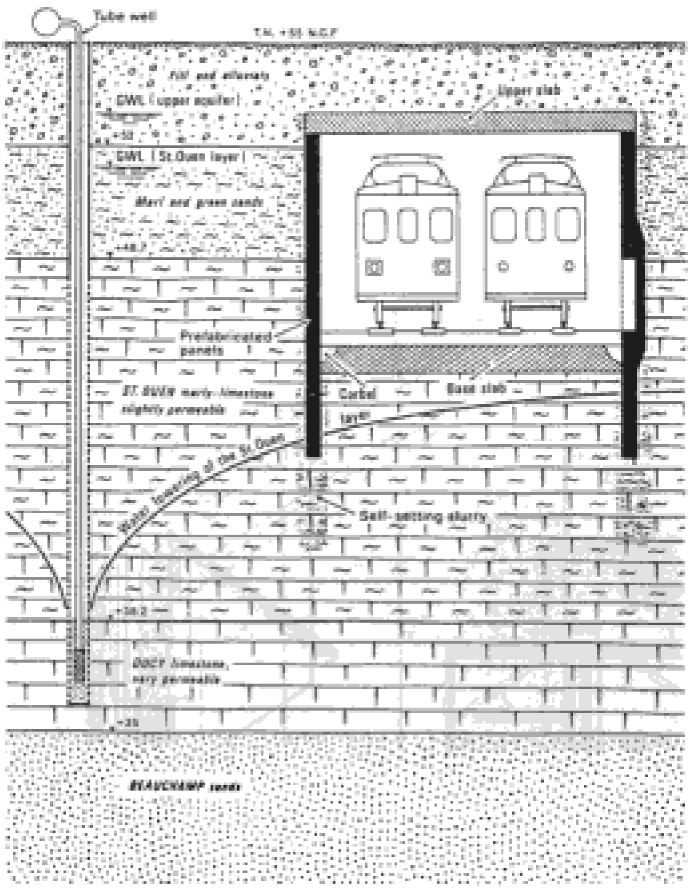


Fig. 9.31. Cross-section of precast diaphragm wall construction for the rail link to Charles de Gaulle Airport, Paris (courtesy of Soletanche)

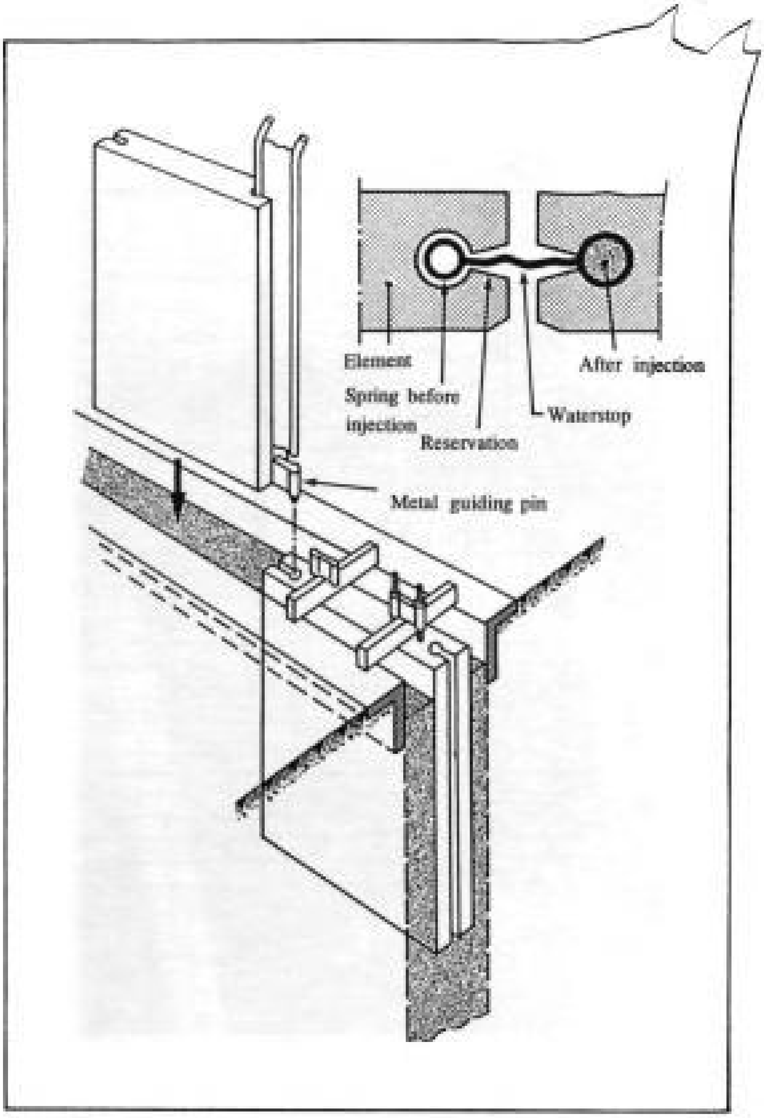
wide by 0·45 m thick. Extra-thick panels were provided at areas of high loading and at tunnel refuge holes. The site output averaged 30 m of precast panel placed daily, the side walls of the 300 m cut-and-cover section being constructed in less than five months.

New metro construction in Lyons and Lille during the late 1970s provided an opportunity for the use of prefabricated Panosol walls at two stations, Saxe-Gambetta and Gare de Lille, and on sections of running tunnel in cut-and-over construction.

Saxe-Gambetta station (Fig. 9.33) was built at the junction of two lines in Rhone alluvium 25 m thick underlain by a relatively impermeable sand. The groundwater table was at a depth of only 3·5 m and the alluvium was very permeable ($k < 10^{-3}$ m/s). The station incorporated a Panosol wall, to support the soil temporarily and provide a cut-off into the sand substratum, together with an in situ reinforced concrete tunnel section. A sandwich-type waterproof membrance was applied to the inside face of the Panosol wall.

The precast wall was designed to support all loads — soil and groundwater pressure and surcharge loads — during construction. In the permanent condition, the load was divided between soil load on the Panosol wall and water pressure on the reinforced concrete tunnel structure. In the temporary condition the permeable alluvium was impreganted with the cement–bentonite slurry used in the wall excavation, with a resulting reduction in short-term soil pressures and deformation.

The works were built in open trench (Fig. 9.33(b)) with the exception of one section beneath Gambetta Road where traffic could not be diverted (Fig. 9.33(c)). In this latter section, intermediate supports were needed from barette panels. The construction sequence for the works in the open was:



(a)

(b)

Fig. 9.32. Rail link to Charles de Gaulle Airport, Paris: (a) view of assembly of precast units in slurry trench; (b) view of completed cut-and-cover tunnel (courtesy of Soletanche)

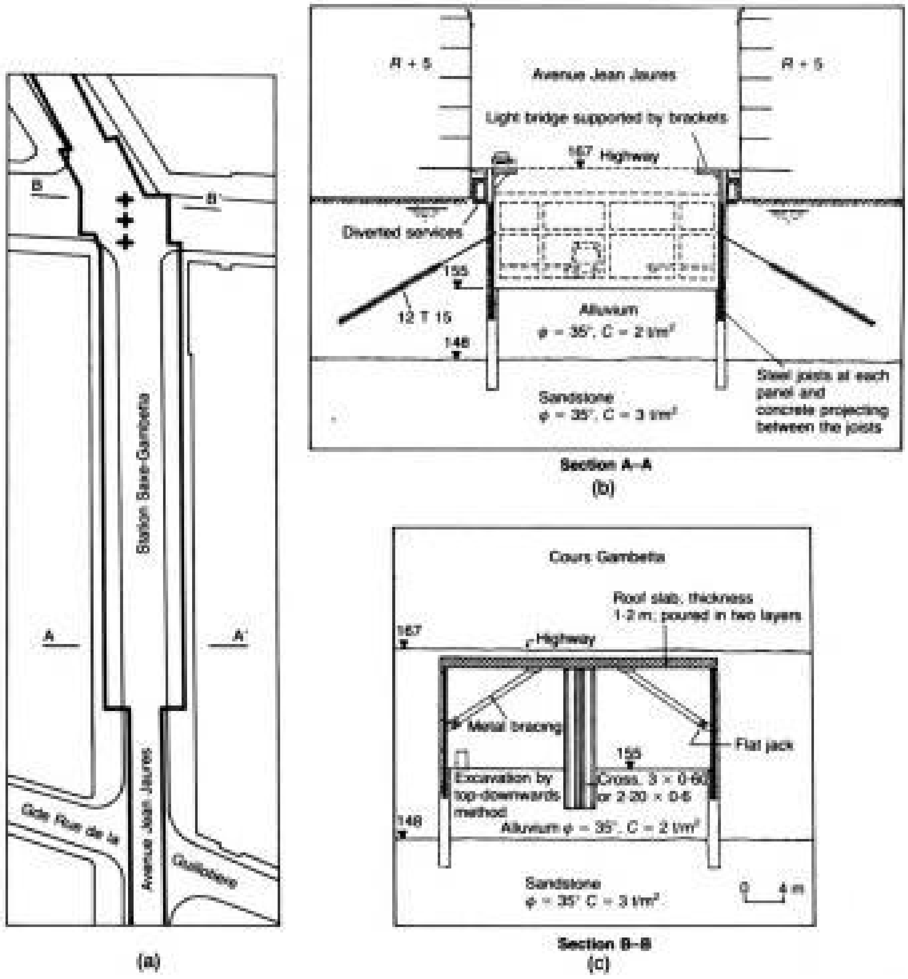


Fig. 9.33. Lyons Metro, Saxe-Gambetta station: (a) plan of site area; (b) section A-A, works in open trench; (c) section B-B, at junction, constructed by top-downwards method (courtesy of Soletanche)

Fig. 9.34. Lyons Metro, Saxe-Gambetta station: completed station excavation showing precast diaphragm walls (courtesy of Soletanche)

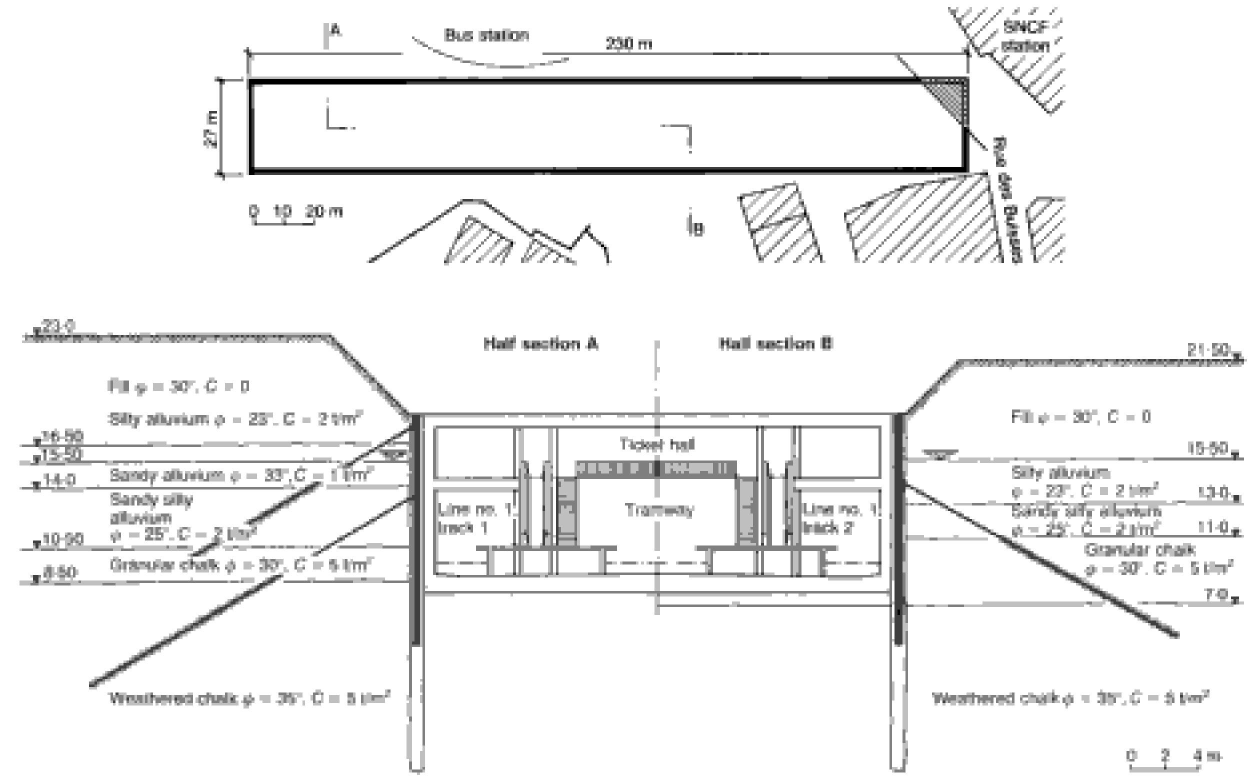


Fig. 9.35. Gare de Lille metro station: (a) plan; (b) transverse sections (courtesy of Soletanche)

(*a*) relocate existing utilities
(*b*) install precast diaphragm wall from street level (economy in the use of materials and panel weight was achieved by making a cut-off at the level of the relocated utilities, 3·30; an H beam was set in the top of each panel to allow cantilever support for a temporary roadway for light vehicles)
(*c*) excavate to groundwater level
(*d*) concrete between H beams beneath temporary roadway to support utilities
(*e*) install temporary ground anchors
(*f*) complete excavation and station construction (Fig. 9.34).

Gare de Lille station, which was planned to connect with a future line, was built in a larger box 230 m long, 27 m wide and between 14 and 16 m deep. The ground conditions consisted of fill and alluvium overlying chalk with a groundwater table at the base of the surface fill material and just below permanent roof level of the tunnel section. As at Lyon Saxe-Gambetta station, the same combination of Panosol precast wall and in situ reinforced concrete tunnel section with a sandwiched waterproof membrane was used. The plan and cross-section of the works is shown in Fig. 9.35. The rate of flow of groundwater into the completed excavation was limited to about 50 m^3 per hour, demonstrating the effectiveness of the cut-off.

Precast units in both reinforced and prestressed concrete were used in sections of the cut-and-cover for the running tunnels of the Lyons Metro, depending on depth to formation. Typical sections are shown in Fig. 9.36, illustrating the use of a grouted base within the walls below formation level and at a depth to balance the groundwater pressure within the alluvium. In some areas, to obtain cut-off within the underlying sandstone it proved more economical to extend the depth of the self-hardening slurry wall where the formation level was deeper.

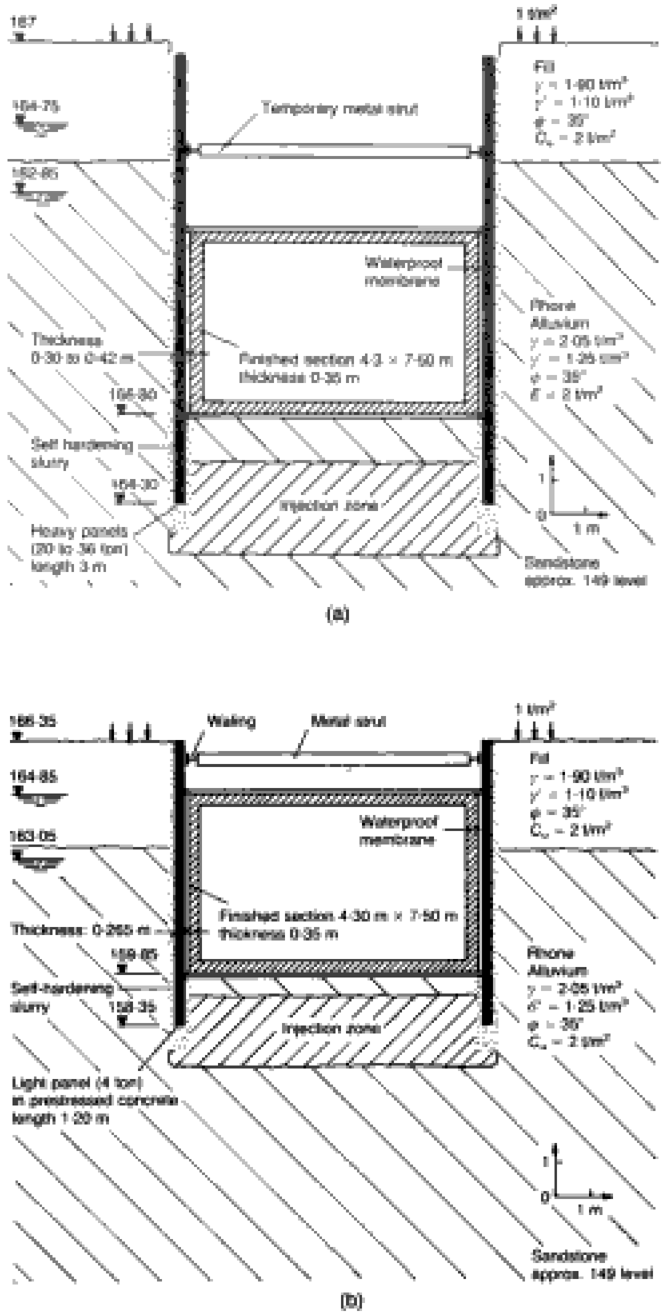


Fig. 9.36. Lyons Metro, typical sections of running tunnel: (a) deep section constructed with precast reinforced concrete units; (b) shallow section using precast prestressed concrete units (courtesy of Soletanche)

Ingress of the self-hardening slurry, containing between 150 and 250 kg of slag and cement per cubic metre, into the alluvium at the sides of the excavation was high, estimated at between 1 and 1·5 m^3 per square metre of wall area. The assumed short-term strength properties of the alluvium allowed for this loss and a value of 20 kN/m^2 was used for cohesion in the design of wall and strutting in the temporary condition. As the waterproofness of the permanent structure was achieved with the sandwiched impermeable membrane, the panels were made of rectangular section with no special jointing devices, temporary waterproofing being

Fig. 9.37. Lyons Metro: two views of exposed precast Panosol panels in running tunnel (courtesy of Soletanche)

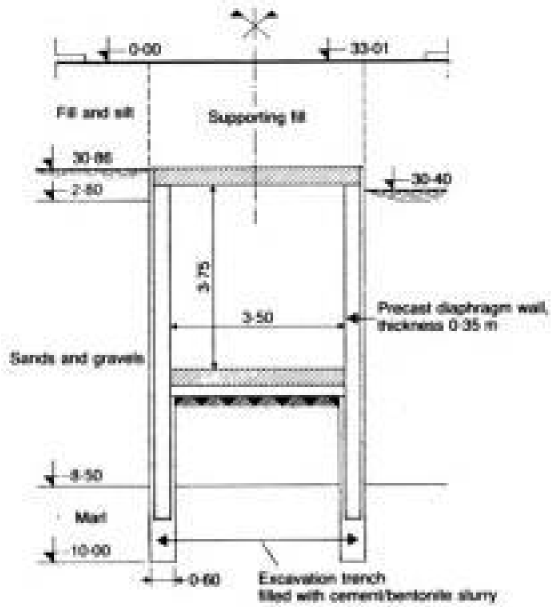


Fig. 9.38. Vitny culvert: cross-section of completed works (courtesy of Bachy)

obtained from the self-hardening slurry. Figure 9.37 shows illustrations of the Panosol wall construction applied to the Lyons Metro.

The versatility of precast diaphragm walls in cut-and-cover construction is demonstrated Figs 9.38 to 9.40. Bachy's Prefasil method was used to build a culvert at Vitny. Fig. 9.38 shows a cross-section of the completed works, which were constructed to high standards of finish, alignment and waterproofness. The culvert was located in a narrow commercial street in the centre of the town. The finished culvert, of internal rectangular section, is 3·75 m high and 3·5 m wide with approximately 3 m depth of cover from existing carriageway levels, the roof being just below groundwater level. The sequence of construction was:

(*a*) Construct a 350 mm thick precast diaphragm wall, within an excavated slurry trench 600 mm wide. The top of the precast wall was carefully levelled to the soffit level of the roof slab, and the cementatious slurry within the trench above this level was reinforced with steel mesh.

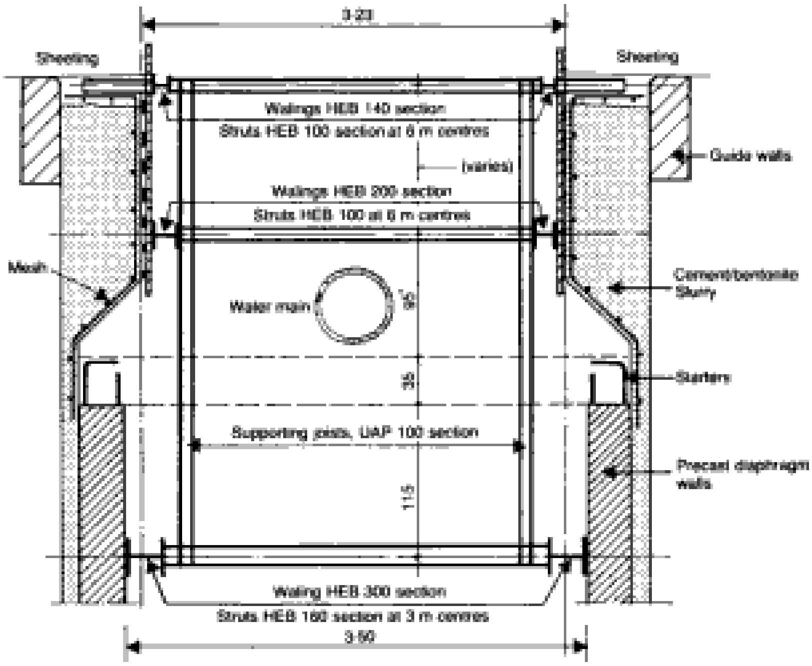


Fig. 9.39. Vitny culvert: cross-section showing temporary soil support above permanent walls (courtesy of Bachy)

(*b*) Excavate between the walls in a strutted excavation, the upper 3 m of exposed cementatious slurry being protected by sheeting behind vertical runners. The average depth of excavation to the underside of the culvert base slab was 7·3 m from ground level, the precast walls and the slurry securing a cut-off into the marl.
(*c*) Cast in situ reinforced concrete floor and roof slabs.
(*d*) Complete waterproofing of joints.
(*e*) Backfill over roof slab and reinstate carriageway.

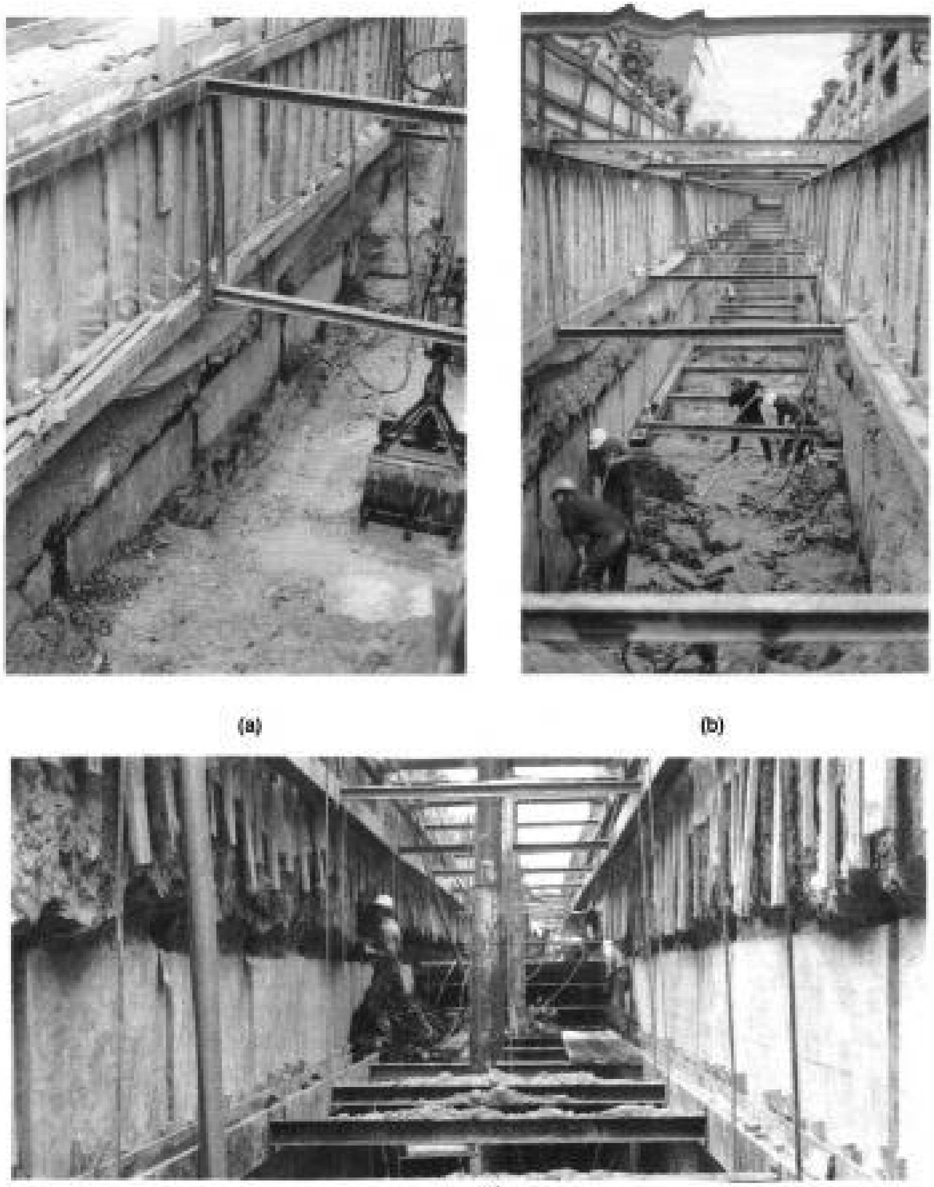
The culvert constructed under this contract was 700 m long, used 524 precast concrete wall panels weighing approximately 15 tonnes each. An output of 7·5 lineal metres of culvert structure was achieved per day.

Bachy's patented continuous water bar system was used in this work. A perspective view is shown in Fig. 9.41. Vertical sections of PVC water bar are cast into a recess in the face of the prefabricated panel. These are subsequently thermally welded to a third section of water bar in the horizontal plane after exposure of the wall following the main excavation. The horizontal water bar is cast into the in situ floor slab of the culvert. The vertical recess between the panels is finally filled with mortar reinforced with steel mesh.

Overall stability: design for uplift

Cut-and-cover works are frequently constructed in water-bearing soils and in such circumstances it is necessary to consider the risk of failure of the structural box by uplift pressures both during construction and during the design life of the structure. The total downward self-weight of the structure together with frictional resistance to uplift of the walls of the structure and vertical force due to anchors or tension piles is required to exceed upward hydrostatic forces by an acceptable factor of safety at each stage.

In particular, tidal conditions, should they exist, should be considered pessimistically over the design life with allowance for inaccuracy in predicted levels. It would be usual to consider the restoring force in this factor of safety to be based on



(a) (b) (c)

Fig. 9.40. Vitny culvert, successive stages in construction: (a) excavation by grab; (b) final stages of excavation and strutting; (c) culvert construction (courtesy of Bachy)

(*a*) dead weight of structural elements based on nominal dimensions
(*b*) height of fill above the roof of the cut-and-cover to final finished levels in permanent condition only
(*c*) frictional resistance due to piled walls or diaphragm walls based on the inner and outer surface of the walls below the underside of the base slab
(*d*) total resistance from anchors or tension piles based on the ultimate capacity of anchors or piles divided by 2·0, using conservative values of soil or rock parameters, unless the results of pull-out tests are available.

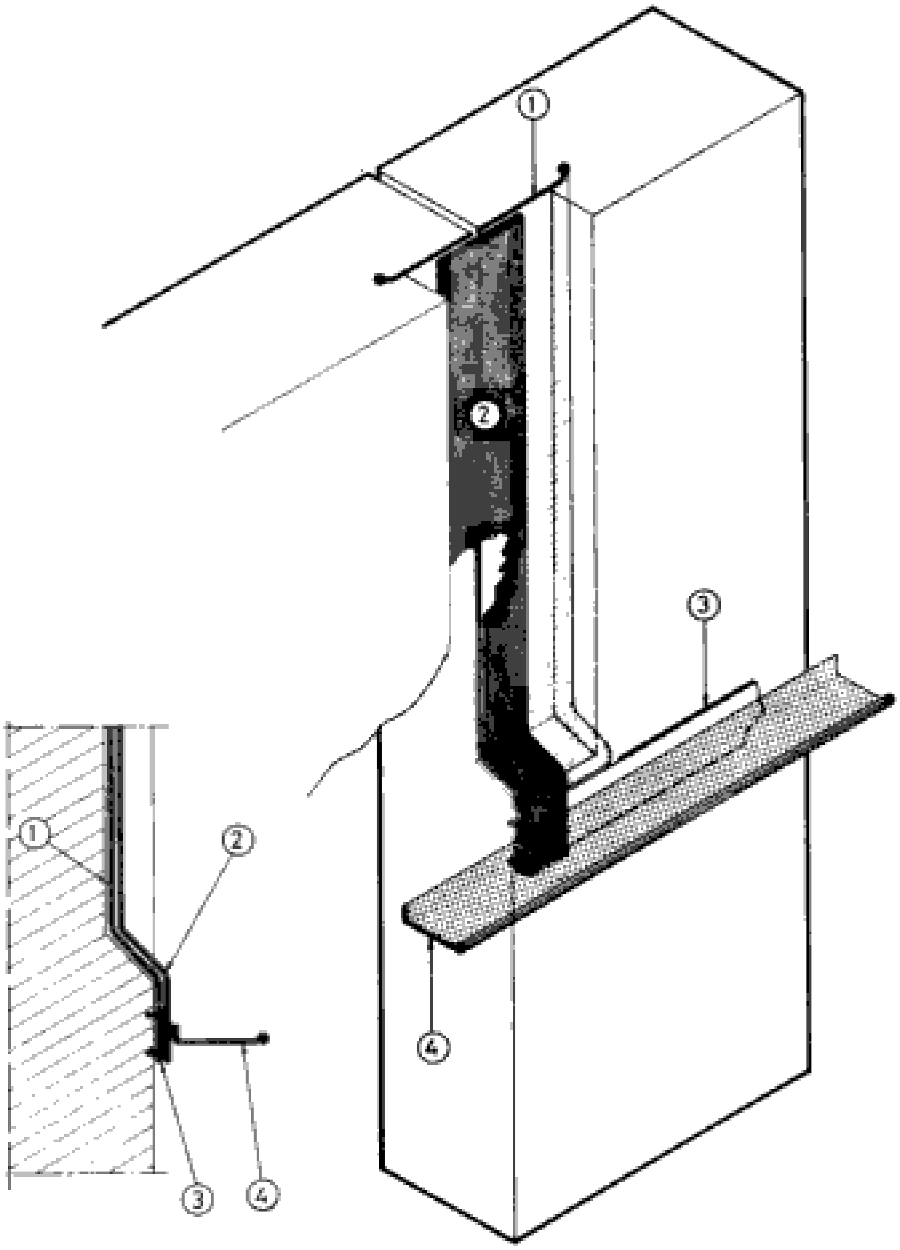


Fig. 9.41. Vitny culvert: water bar system with continuity between vertical and horizontal water bars (courtesy of Bachy)

Usually an overall factor of safety of at least 1·1 on dead weight of the structure and fill over is required; a minimum value of 1·4 is required when the effects of friction and resistance due to anchors or tension piles are included.

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10 Shafts and caissons: construction and design

Shafts for civil engineering purposes

Shaft construction deserves to be considered separately from both conventional cofferdam and caisson works. Often of small diameter, shafts may be rectangular in plan, or they can obtain maximum benefit from the arching action of soil around the shaft by conforming to a circular or ellipsoid shape. Inclined or vertical shafts may be built for permanent or temporary works. In specialist works such as tunnel construction, a shaft may serve a dual purpose; first, as a means of access to the tunnel drive during construction, and, at a later stage, as a means of ventilation to the completed tunnel.

A comparison of shaft and tunnel construction work shows the particular difficulties that can arise in shaft construction:

(*a*) groundwater can accumulate on the shaft working face
(*b*) works and materials have to be transported from the shaft face to ground surface
(*c*) excavated spoil has to be removed as a deadweight from the shaft face
(*d*) shaft linings have to be installed progressively downwards.

Although it is beyond the scope of this chapter, a study of installation techniques for deep shafts for mining works can assist in design and construction for shallower shafts for civil engineering purposes. Jones[1] reviewed current methods of shaft sinking and raise boring for mining work. Mechanical mole tunnelling machines were then in use for vertical shaft construction and possessed the twin virtues of increased safety and efficiency. Jones described the purpose of comprehensive investigation prior to sinking to determine the safest and most economical excavation method and best utilization of freezing, grouting and lining techniques. The practice of collaring to support the shaft near the ground surface using sheet piling, diaphragm walling or caisson reconstruction was also described, together with details of drilling jumbos, mucking out, and hoisting and lining methods. Excavation methods using raise boring and raise climbing machines and conventional long hole blasting were also described.

Shaft sinking and raising practice in several countries was reviewed at the 1959 Symposium on Shaft Sinking and Tunnelling[2] recording techniques in use at that time.

Permanent shafts

In works associated with tunnelling for underground railways and road tunnels, permanent shafts are used for lifts, escalators, staircase access and for ventilation purposes. In sewage disposal schemes, shafts find use in pumping station construction and drop shafts. Some of the deepest shafts are in hydro-electric schemes and pumped storage works.

The vertical connection of tunnel drives to penetrate the sea bed involves shaft work of a special nature. These works are typically necessary for cooling water intakes to power stations and for sewage outfalls. In some cases, offshore jack-up rigs have been used; the procedure is basically to install a bulkhead within a sea bed excavation, to raise a shaft from the tunnel to the underside of the bulkhead and then remove the bulkhead by blasting.

At Wylfa power station[3] a cooling intake was constructed on an exposed coastline through 11 m of water at low tide with a tidal range of 6 m. The headworks to the shaft were constructed from a pit blasted on the sea bed; within the pit were installed a cylindrical shell and bulkhead, concreted into the rock face. The lower section of the 1·2 m dia. shaft was then excavated upwards from the tunnel to meet the bulkhead. This pilot shaft was enlarged to 4 m diameter before removal of the bulkhead.

The Dublin outfall sewer at Howarth Head[4] was raised in sound rock from the end of the tunnel works to just below the sea bed to avoid headworks in expected high seas. After flooding the shaft and tunnel so constructed, the remaining length of shaft was removed by underwater blasting.

The use of concreted shafts and drilled shafts to house steel columns to withstand loads transformed from a superstructure overlaps with the subject of piling. This is inevitable as the size and power of mechanical piling equipment increases. Drilling using rock roller bits, large drag bits, mechanical augers and large diameter down-the-hole hammers, and the support of soil and rock by differential waterhead, drilling slurry or temporary casing, while defined in North America as 'caisson construction', is beyond the scope of this book.

Temporary shafts

Megaw and Bartlett[5] described the use of temporary shafts for tunnel works and stated the principal requirements of all-purpose working shafts for tunnels:

(*a*) They must be available form the earliest stage of construction until tunnel completion.

(*b*) A shaft 4 to 6 m in diameter is typically needed to accommodate hoisting equipment and provide access for workers. Note that in pipe jacking works the shaft diameter may depend on dimensions of precast pipe sections and access clearances; with shield-driven tunnels the shaft dimensions will depend on the size required for hoisting shield components and, unless a separate shield chamber is used, the space needed at the bottom of the shaft to allow shield fabrication. In addition, the shaft diameter, of at least 1·5 times the tunnel drive diameter, must be sufficient to allow break-out from the shaft bottom.

Megaw and Bartlett referred to the spacing of shafts in tunnel works: earlier, the progress of tunnel works dug by hand was improved by a large number of working shafts. In 1838, eighteen shafts, were used to drive the Kilsey Tunnel which was only 2·2 km long. Later railway works needed a shaft spacing of about 1 km for steam clearance purposes. The location of shafts remains a compromise between optimum tunnel alignment and the availability of adequate space at ground level. This problem was accentuated in earlier times when shield-driven tunnels could only be made in a series of straights.

Sinking methods

The sinking techniques adopted depend on the shaft use, its diameter and depth, soil and rock conditions, groundwater state, the proximity of other structures and their sensitivity to settlement.

Hand excavated shafts

Although hand excavation would appear to be expensive where soil is sufficiently stable to stand unsupported for small heights, the conditions are relatively dry and moderate labour rates prevail, the method can find economical application: in Hong Kong, for instance, 'family caissons' are taken down through residual soils where there is risk of granite boulders which impair excavation by mechanical plant. These hand-excavated shafts are lined with in situ unreinforced concrete, typically 75 mm thick, in small height lifts.

A traditional method of pier construction for foundation support was known as the 'Chicago method' since its introduction on the Chicago Stock Exchange in 1894. Soil support was obtained from vertical poling boards set on the pier periphery and held by steel rings. Hand excavation proceeded in depth increments of 1 to 2 m, depending on conditions, and a further set of boards and rings were placed. At full shaft depth a hand-belling operation was carried out to increase the shaft diameter if soil conditions allowed.

Hand excavation may also be more economical in other circumstances; in weak rocks, for instance, where belling operations may prove difficult for mechanical augers. At Hartlepool Nuclear Power Station, 17 piers, each 2·3 m in diameter, were used as support to the reactor. The boring was taken through 5 m of soft fill, clay and 30 m of glacial fill by rotary auger using bentonite slurry for soil support. Weak bunter sandstone at this depth was then hand-excavated a further 4 m from beneath a casing set into the rockhead and back-grouted throughout its height. The base was belled out by hand to 3·9 m diameter to reduce bearing pressures on the sandstone to 2·9 MN/m^2.

Mechanical excavation

Open shafts have traditionally been excavated by mechanical grabs suspended from cranes or derricks, where space allows, assisted by mechanical loading shovels at excavation level. In rock, shallow shafts are drilled by hand-held rock drills; in deeper works, a shaft jumbo is used with boom-mounted drills mounted on a folding frame. It is usual to pull 1 to 2 m on each round. Beyond a depth of 30 m or so it is usual to construct a temporary head frame and use muck skips or kibbles to muck out the shaft.

The distinction between piling and shaft construction may only depend on size where the shaft is to be backfilled to form a load-bearing member; mechanical augers are used together with casing, either temporarily or permanently as needed. Bentonite slurry may again be used as a method of soil support; in the reverse circulation process, slurry is used for both soil retention and as a means of transporting the excavated soil cuttings. A temporary top casing is used in these instances to avoid soil disturbance at ground level, to maximize the head of slurry and avoid contamination of concrete during placing of soil fall-ins. Shafts formed in this way are typically 2 to 3 m in diameter and up to 70 to 80 m deep, but much larger diameter and deeper shafts for civil engineering works have been successfully completed using purpose-made reverse circulation equipment.

Soil and rock support

The periphery of the shaft may require no lining in dry, sound rock, but otherwise the following methods are available for soil or rock support:

- in soils: timbering
  steel sheet piling
  precast concrete, cast-iron and pressed, welded steel and segmental linings
  diaphragm walling
  secant piling
  in situ concrete lining
- in rock: shotcreting
  rock dowels and rock bolting

(*a*) Timbering: in soft ground, vertical poling boards are driven ahead of excavation and secured by timber walings, strutting and diagonal corner braces. Construction depths are limited w[illegible] timbering, and the works are labour-intensive by modern standards.

(*b*) Sheet piling: deeper shafts may be sheet piled, the piles being secured by walings in timber, steel or reinforced concrete. Ground anchors may be used where space outside the shaft allows and space within is confined. Sheet piling may be impeded and pile clutches broken in hard driving caused by boulders or penetration into rockhead. Grouting may be necesary in such cases to avoid ingress of groundwater into the shaft.

(*c*) Segmental linings: where space and ground conditions allow, the first ring is built a small depth below ground level supported, if the shaft is large, by an external collar to avoid differential settlement around the ring. In dry, sound soil, rings are built successively downwards in an underpinning operation for each ring. After completion of the ring the annular space behind the excavated soil face and the outer segment face is grouted, thus avoiding later loss of ground and subsidence of the ground surface. Where soil conditions are less favourable it may not be possible to complete a whole ring without temporary support for each segment from the central dumpling. In tunnel works segments are in cast-iron or precast concrete. More recently, solid reinforced concrete segments, known as one-pass shaft linings, have become available and avoid further in situ lining works. These segments use stressed loop cross-joint connectors (details in Fig. 10.1) and allow the introduction of precast corbels for structural support for landings by bolting to the main lining (Fig. 10.2). Examples of conventional concrete segment and one pass linings are shown in Figs 10.3 and 10.4. Steel liner plates will themselves be adequate in small diameter shafts, but in larger diameter shafts a curved steel joist is set for every two or three courses of liner.

(*d*) Diaphragm walls and secant piling: the use of these methods to form the walls of shafts becomes more attractive when either can be incorporated into permanent work. In both cases the maximum depth of such shafts frequently depends on the verticality tolerances which can be guaranteed for the installation method. Both diaphragm walls and secand piling depend for groundwater exclusion upon the soundness of joints between panels or piles, and the efficiency of the mechanical interlock between the adjacent units depends on verticality accuracy at installation. In diaphragm wall shafts where a segmental plan shape can be used, structural integrity is necesary at each panel joint to transfer hoop compression between panels and retain structural stability. Depths of the order of 30 to 40 m are not unusual for diaphragm wall shafts, and the increasing use of reverse circulation cutter rigs, such as the Hydrofraise and Trenchcutter rigs, bring improved verticality tolerance to the works. Figure 10.5 shows the use of polygonal plan shaped shafts in Hong Kong for deep access shafts through decomposed granite, constructed by Bachy/Soletanche.

The construction of deep shafts using grab excavation methods can be made practicable by installing rotary bored piles at the junction of each diaphragm wall panel. The verticality tolerance of pile installation using special equipment is of the order of 1 in 300 or better. The piles are installed before the diaphragm wall panels and act as a guide to the panel grab, which is shaped to cut soil from the curved pile face. Icos used this procedure of interlocked circular units and rectangular panels to build a 4·6 m dia. shaft through fine sands to the very considerable depth of 72 m for mineworks at Speckholzerheide, Holland, in 1954. Improved excavation tolerances achievable with reverse circulation equipment such as the Hydrofraise and Tenchcutter rigs allow deep shaft construction without the need for bored piles at the junction of each panel. Shafts in excess of 50 m deep can be constructed with this type of equipment, the junction being achieved by

Fig. 10.2. Example of single-pass shaft lining with corbels (courtesy of Charcon)

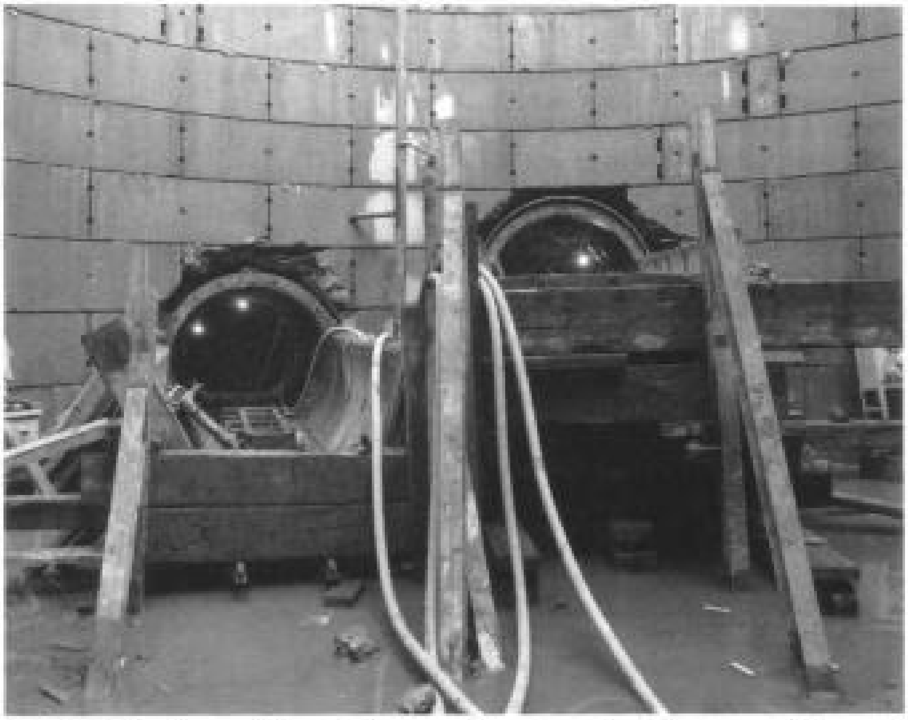


Fig. 10.4. Single-pass lining to line tunnel break-out shaft (courtesy of Charcon)

Fig. 10.3. Conventional bolted precast concrete segment lining (courtesy of Charcon)

Fig. 10.5. Polygonal plan shape shaft construction using diaphragm walls, Eastern Harbour Crossing, Hong Kong (courtesy of Bachy/Soletanche)

by secant piling methods using heavy rotary equipment, powerful casing oscillations and temporary, rigid twin-wall casings. The depth of excavation is limited by the accuracy of setting and dropping the casing to maintain the depth of the secant cut with the male secondary piles. A rotary rig with casing oscillator working immediately adjacent to an existing structure is shown in Fig. 10.6.

(*e*) In situ concrete lining: in rock excavation where the rock will stand temporarily without support, in dry conditions, an in situ lining of mesh-reinforced concrete can be used. Excavation is by drilling and blasting in successive increments of 2 m or more, with concrete pours 6 and 8 m high. Slip-forming methods can be applied in deep shafts. The use of in situ lining overcomes any difficulty in varying amounts of overbreak that occur when using segmental linings. Thermal insulation is necessary when concrete is placed against frozen soil or rock.

(*f*) Shotcreting: in shafts through rock which is relatively stable and dry, sprayed concrete, or shotcrete, may be used to stabilize the face. Applied in layers typically 50 to 60 mm thick, light mesh reinforcement may be pinned to the rock face.[6,7]

(*g*) Rock dowels and bolts: drilled radially from the shaft, dowels and bolts are used as rock reinforcement, and mesh secured to the face of the dowel or bolt may be used to secure the rock face. CIRIA report 101[8] describes rock reinforcement in underground excavations. Rock dowels may be cement or resin grouted or may consist of a hollow, high-strength steel tube with a longitudinal slit which compresses radially when driven into a drillhole exerting continuous outward pressure. Another type of dowl uses high-pressure water to expand a steel tube within a drilled hole. Rock bolts are

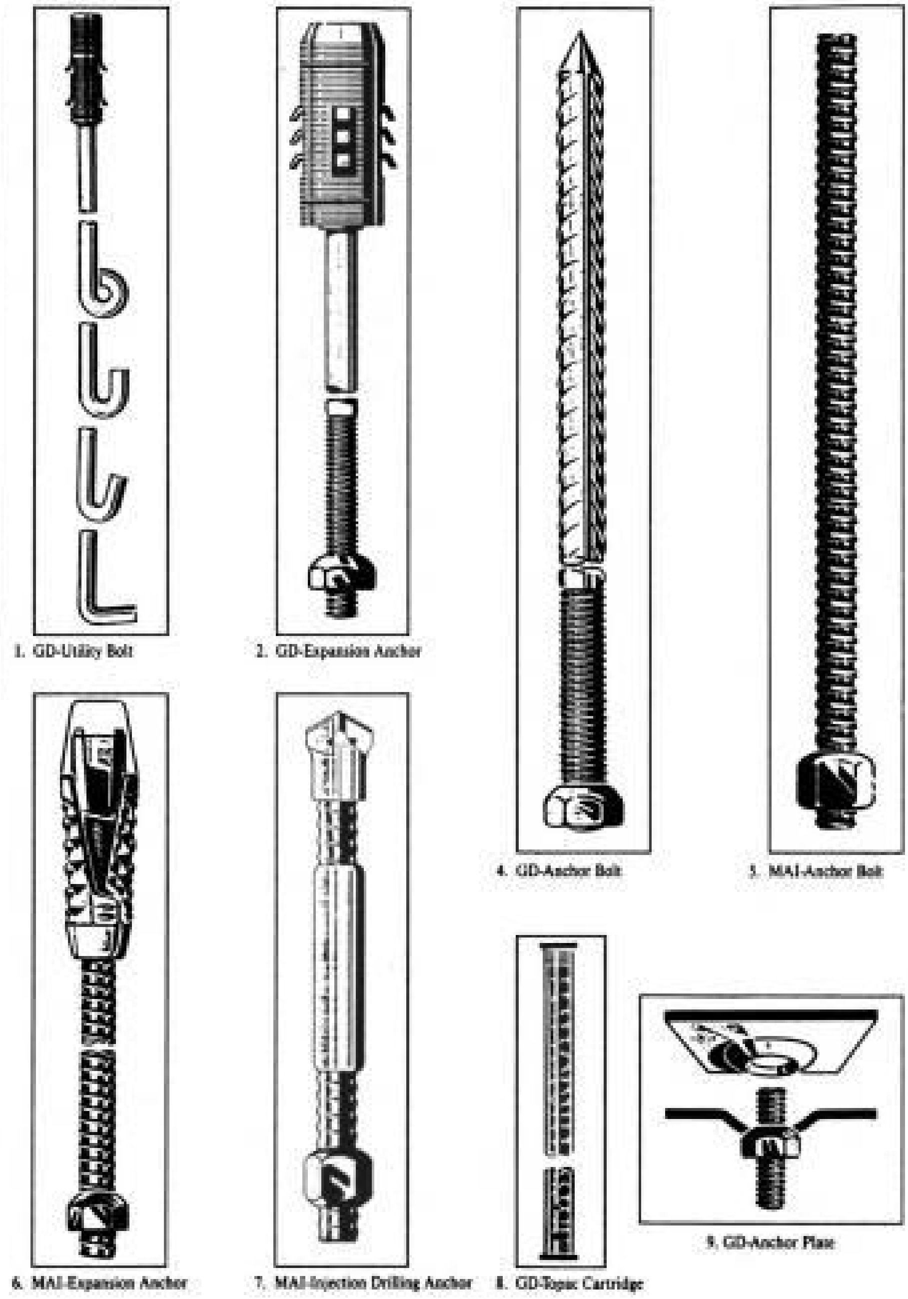


Fig. 10.7. Types of proprietary rock bolts and anchors used in shaft construction (courtesy of MAI Systems)

Where cast-iron segments are used, kentledge may be necessary above the air deck to alleviate the low tensile strength of the cast-iron; grouting behind the tubbing reduces risk of circumferential tension within the iron.

Caissons

The maximum practical depth of excavation within cofferdams is of the order of 25 m. To reach founding levels at greater depths previous generations of engineers have used caissons, either open well or pneumatic types. Historically, well caissons have been used for bridge foundations in India, Burma and Egypt, using masonry or brick for the caisson walls. Prior to Victorian times these wells were sunk by

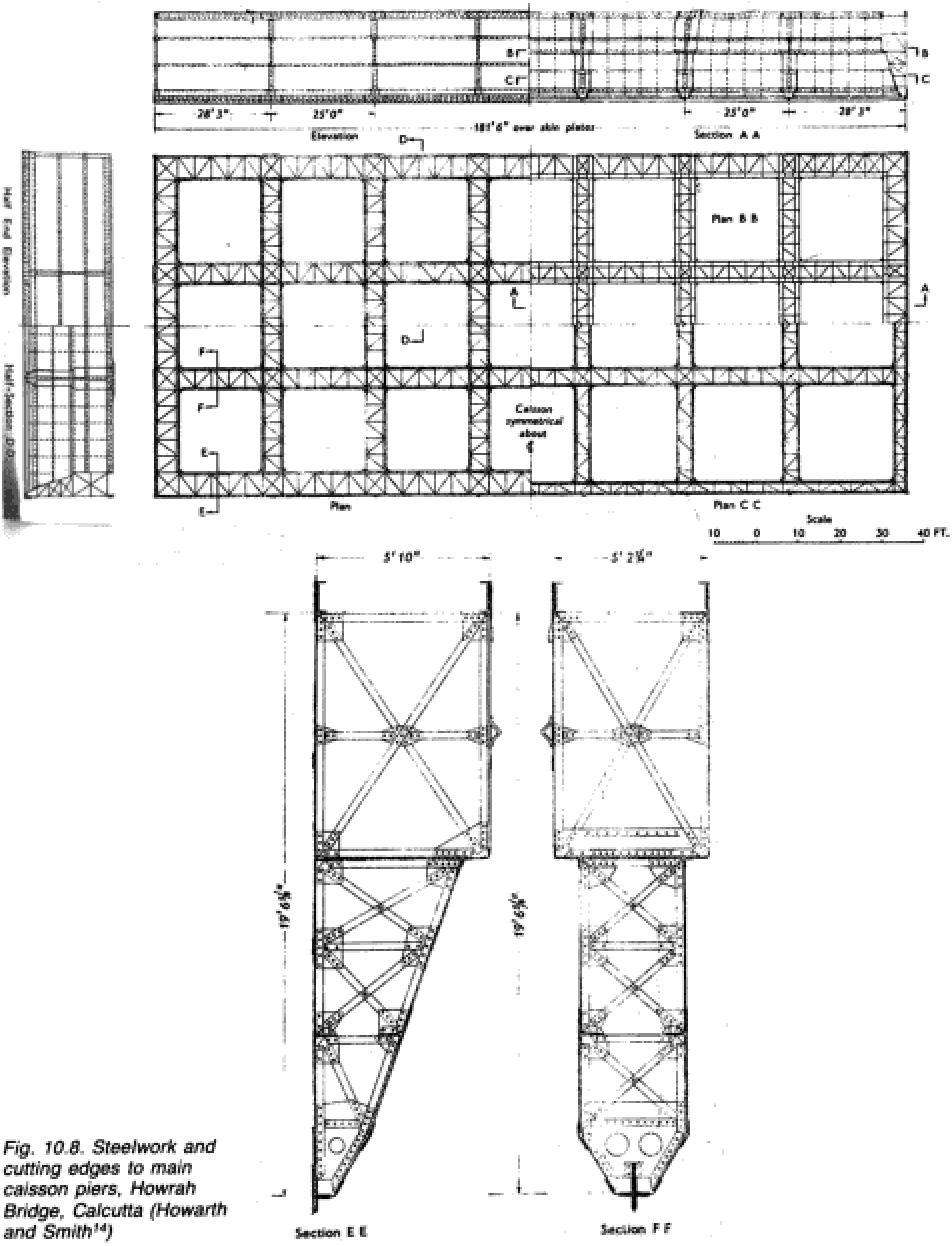


Fig. 10.8. Steelwork and cutting edges to main caisson piers, Howrah Bridge, Calcutta (Howarth and Smith[14])

(*d*) Avoid sudden movements of material. Dredge so as to remove mud from the centre, maintaining bearing on the cutting edges.
(*e*) Do not dredge below the exterior cutting edges until the caisson is founded. If the caisson will not settle satisfactorily, use jets on the side to reduce skin friction.
(*f*) In a large, rectangular caisson, sinking may be accomplished by rocking slightly about the short axis, dredging first from one end and then the other. This must never be done about the other axis.
(*g*) In unstable soils, avoid any action which might tend to make the soil under the cutting edge quick, particularly the use of blasting and pumping down the dredging wells. Jetting on the sides of the caisson, and in the dredge wells if mud plugs tend to form, is usually safe and desirable.

Tomlinson[16] pointed out that caisson proportions, while usually defined in terms of plan shape by the superstructure, are often a compromise. On the one hand, the walls are required to be thick to provide maximum weight for sinking, and on the other, they should be thin to allow the grab to work near the cutting edge. Lightness of weight is desirable to allow a shallow draught during towage from casting yard to mid-river site but this, in turn, is to the detriment of the caisson rigidity, which is essential during early stages of sinking when the caisson may be unevenly supported.

Tomlinson pointed out that the size and layout of the dredging wells depend mainly on the soil type. Dense sands and firm-to-stiff clays require a minimum number of cross walls and a minimum outer wall thickness consistent with the weight requirements for sinking and rigidity against distortion. The cross walls need not extend to cutting edge depth. In sands and soft silts, on the other hand, grabbing below cutting edge level causes soil to move towards the centre of the shaft where the excavation is kept low. Water and air/water jets may be used for excavation and lubrication purposes. The use of bentonite slurry lubrication for caisson sinking will be referred to later.

Traditional deep wells, each with one shaft, were specified on two bridges built across the River Ravi in Pakistan as part of the Indus Basin Settlement Scheme in the 1960s.[17] One of the new bridges replaced a railway bridge where the existing foundations were not deep enough for scour protection with the planned river discharge of 4300 m^3/s and modification was not possible.

The wells, 36·6 m below river bed level, were sunk by a method developed in France in 1927 by Cacot. The method reduced side friction and minimized the need for kentledge. Fig. 10.10 shows the well dimensions for the railway bridge and the hemispherical shape of the wide well base and the position of airlift pumps, or 'emulsifiers', used to excavate the soil on the well periphery. The reinforced concrete cutting shoe was cast on a form made from rendered dry-stacked bricks. Reinforced brick steining in 3 to 1 cement mortar was built over the shoe to a height that allowed the initial sinking to be made by crane and clamshell. The following stage of brick steining was then completed and the excavation was undertaken with central air lift. Sinking rates varied from small fractions of a metre to more than 3·5 m per day.

Positional tolerances could only be achieved by sinking at a rake or, more effectively when caught early, by dredging outside the well and surcharging one side of it. In three instances correction could only be made at considerable depth by divers using waterjets cutting below the cutting edge on one side. After completion of sinking, the spherical shoe was plugged with intrusion-grouted aggregate and the well was filled with sand. Savage and Carpenter[17] commented that while the technique was an improvement when kentledge was not available, they considered that well techniques would fall into disuse and be replaced by large diameter piles as the cost of masonry work increased.

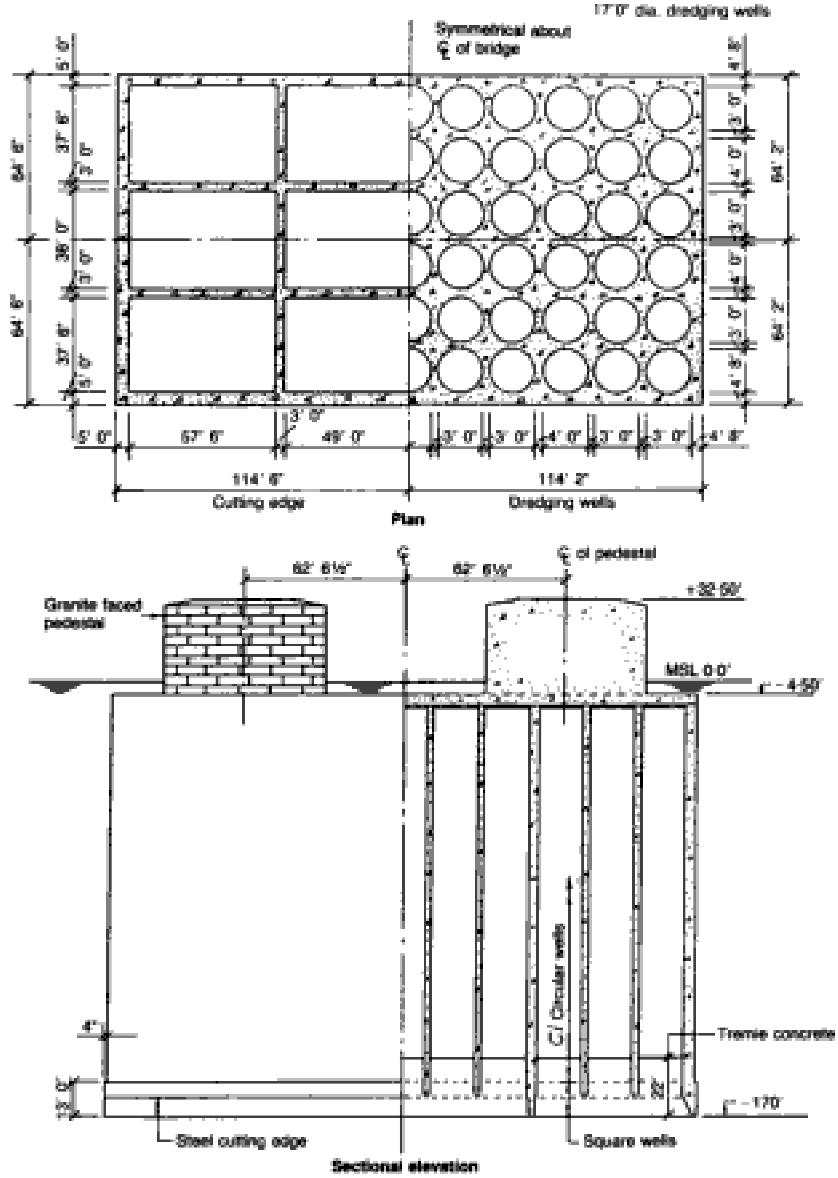


Fig. 10.13. New York City Narrows Bridge, Brooklyn pier: caisson plan and sectional elevation (Yang[19])

into stiff clay, tended to make the caisson sag and the caisson profile was constantly checked to avoid overstressing the structure. The average rate of sinking was 60 cm per day for the first two sinking stages, the rate depending only on the rate of sand excavation from the cells. The actual volume of excavation barely exceeded the theoretical volume during the early stages. Dredging efficiency was reduced at lower depths into the clay, due in part to the depth effect itself and also because of loss of soil from grabs. At the lowest depths, the daily rate of sinking reduced to 36 cm per day and the actual excavation volume exceeded the theoretical volume by some 27%.

The soil at the cutting edge became more silty and less able to resist excessive pore-water pressure. A differential head between 1 and 2·4 m was maintained between cell water level and mean tide level but several cave-ins occurred. The daily sinking record and the mass diagram during sinking of the Brooklyn caisson are shown in Fig. 10.14.

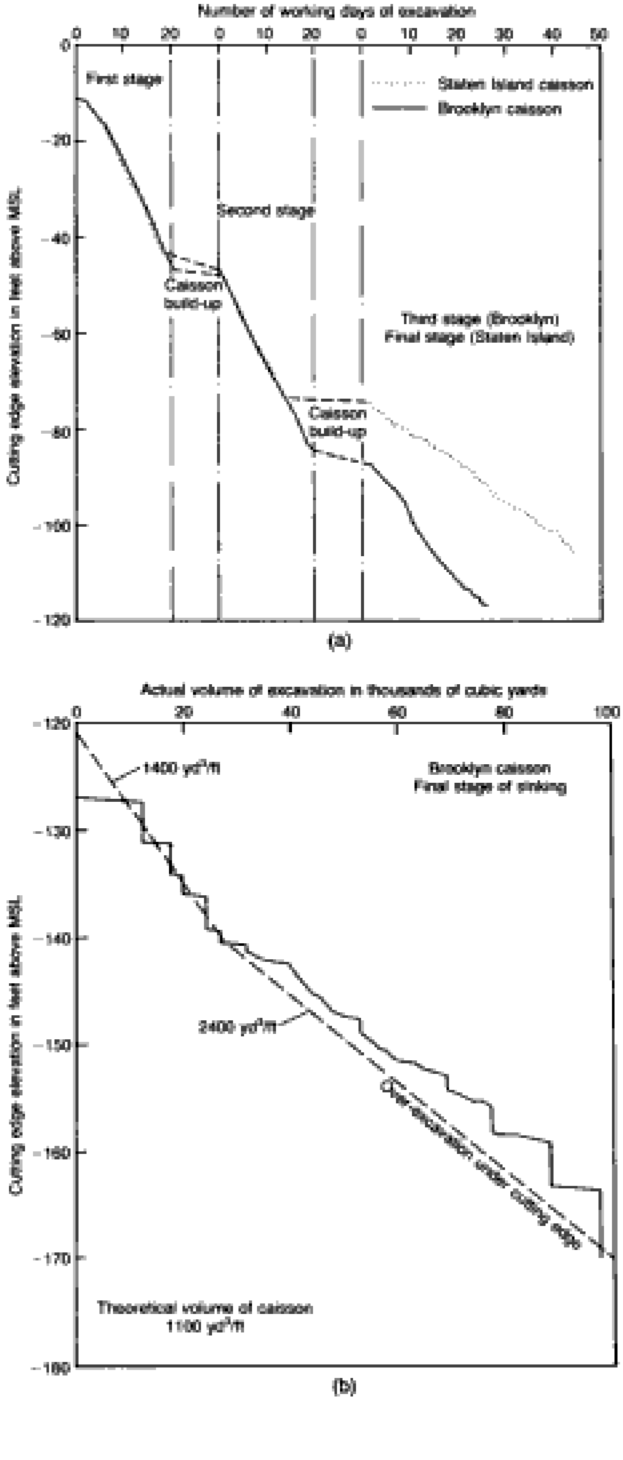


Fig. 10.14. New York City Narrows Bridge: (a) caisson daily sinking record; (b) mass diagram of sinking caisson (Yang[19])

In the 1970s the Barton tower of the Humber suspension bridge was also founded on caissons built on a temporary sand island. the figure-of-eight cofferdam containing the sand fill was subject to considerable scour and some 12 000 tonnes of chalk was needed to remedy this hazard, with much of the cofferdam piling requiring extension and redriving.

The two 24 m diameter caissons consisted of two concentric reinforced concrete

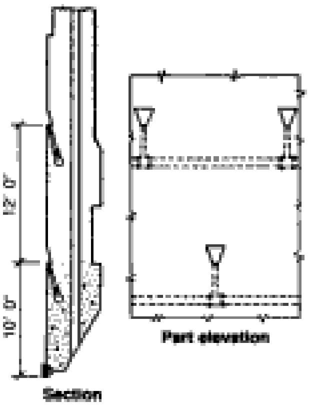


Fig. 10.16. Baton Rouge Bridge: two-tier jetting arrangement (Blaine[21])

Mitchell[20] referred to claims for reduction of up to 40% friction with the use of bentonite slurry. This slurry is injected from closely-spaced nozzles connected to a header tube cast into the caisson walls, and the injections are regular and in sufficient quantity to keep the annular space next to the caisson full of slurry. Blaine,[21] referring to the Baton Rouge Bridge, had concluded that the use of water jets from similar nozzles within the caisson wall were, as designed, little value in clay and very dangerous in sands, causing run-ins. At Baton Rouge the two-tier jetting arrangement (Fig. 10.16) was unsuccessful. Fine sand had offered the greatest resistance in caisson sinking, and this situation is common elsewhere. Blaine also concluded that, in sand, blasting was a method of desperation which seldom did any good and caused run-ins.

It should be noted that the obstruction to river flow by sand island construction may well cause excessive bed erosion. Tomlinson[16] noted river bed scour at sand islands formed within steel shells at the Baton Rouge Bridge. The scour, 12 m deep despite the use of woven board anti-scour mattresses, was caused by constriction of the 730 m river width to 97 m wide waterways between the sand islands. The fast flowing Mississippi River removed the whole sand filling to one 37 m diameter island in two to three minutes and caused severe tilt to the partly-sunk caisson.

Blaine[21] stated that irrespective of the detailing of erosion protection at Baton Rouge Bridge, the essential lesson was that the sand island method of caisson sinking is potentially a major erosion hazard. The damage to the caisson referred to by Tomlinson occupied the contractors's entire organization for three months following the accident, in providing further scour protection and plumbing and deepening the displaced caisson.

The south pier to the Forth Road Bridge, built in the early 1960s near the famous railway bridge, was founded on caissons sunk from the bottom of a river cofferdam. The Forth Bridge foundation works were described by Anderson.[22] The south pier foundation was originally designed as two rectangular caissons, floated into position and sunk under compressed air through the boulder clay. The contractor, John Howard and Co. Ltd, produced an alternative design using two circular caissons, which were intended to be driven through the clay as open caissons in the dry. The Contractor was confident that the boulder clay would allow sheet piling for the cofferdam to be driven, and also that it would form an adequate seal for caisson excavation in the dry. The construction sequence is shown in Fig. 10.17 with a view of the access to the south pier caisson prior to sinking shown in Fig. 10.18. The site of the pier, a quarter of a mile from the shore, had a steel and timber access jetty which ended with staging enclosing the pier site on three sides.

The cofferdam for the pier, described previously, included a lower ring waling formed successfully underwater by intrusion grouting. After dewatering the cofferdam, the caisson was erected on a 100 mm concrete binding. The caisson curb consisted of a heavy steel cutting edge mounted on skin plating. The caisson walls extended to a height of 8·5 m in reinforced concrete before sinking. During the erection work the caissons were supported around their internal periphery by concrete blocks at 1·5 m centres, and the space between the blocks was then filled to form a reinforced concrete ring beam of wedge section. After sinking through the gravel, the intermediate sections were removed by blasting and, as the caisson reached the boulder clay, the original concrete stooling was blasted away to allow the caisson to sink on the cutting edge alone.

Although air ducts and locks were ready, the method succeeded in the dry, in 'free air', as the caisson continued downwards at an average rate of 30 cm each day. Mechanical excavation on the caisson floor achieved an average of just less than 100 m^3 per day in the solid measure. The downstream caisson was kept 4·5 m lower than its neighbour to ensure that any heavy inflow below the cutting

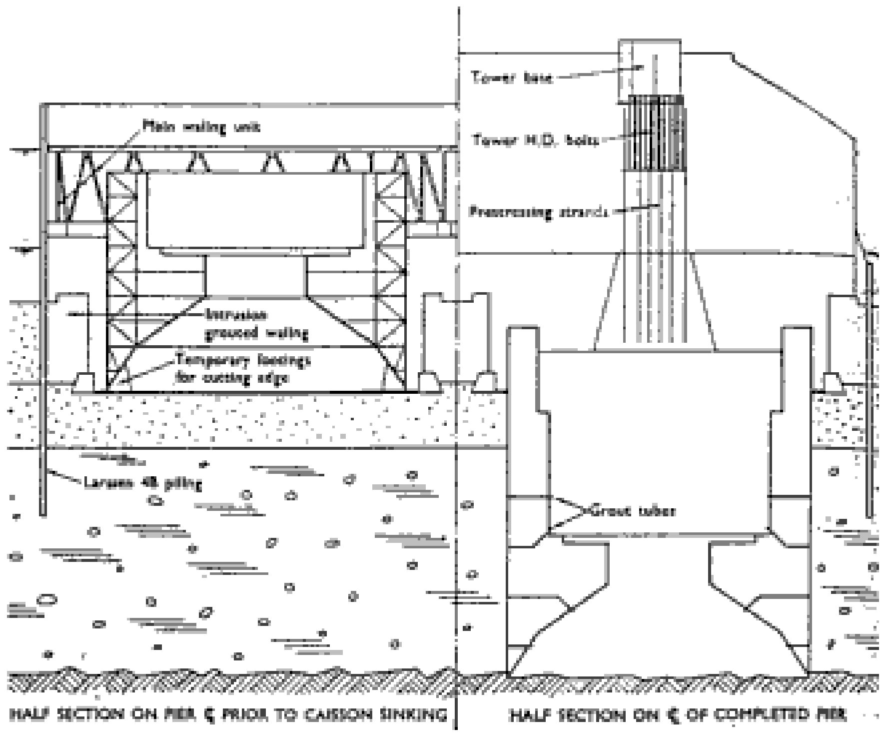


Fig. 10.19. Forth Road Bridge, south pier construction: half-sections showing caissons during sinking and completed pier (Anderson[22])

edge did not flood both caissons. There was some seepage but bentonite grouting remedied the situation and, additionally, reduced skin friction during caisson sinking.

The vertical walls extended to a total height of 22·5 m above the cutting edge and the caisson was founded on bedrock. Fig. 10.19 shows sections of the caisson and the finished pier. At the end of sinking both ciassons were vertical and less than 150 mm out of position.

The construction of caissons on the Lower Zambezi Bridge was described by Howarth.[23] On this contract both open and pneumatic caissons were used. It was estimated that 16 of the main wells could be sunk from sand banks, and from the exposed river bed in the dry season. It was thought that sinking of the remaining wells would likely be through varying depths of water and it was decided to use floating caisson sinking sets.

The arrangement adopted for the open wells to piers 28 to 32 in the 1932 dry season is shown in Fig. 10.20. This technique minimized the obstruction to river flow and, in turn, the extent of river bed scour. Wire ropes from the craft carrying the caisson curb and its attendant cranes were secured to heavy concrete or cast-iron anchors. Two 10 tonne sinkers were laid out upstream and four 5 tonne sinkers were used as breast moorings. The breast mooring lines from the port-side craft were moored to starboard sinkers and vice-versa. One 5 tonne sinker was laid out downstream.

Caisson buoyancy can be improved by the use of false bottoms to the wells, the temporary bottoms only being removed after the caisson has sunk a small depth into the river bed so that the caisson weight is then supported by outer friction and its cutting edge. In the 'floatation caisson' method devised and patented by Daniel E. Moran for the west bay caissons on the San Francisco–Oakland Bay Bridge in the early 1930s, compressed air was used to regulate the amount of draught and improve control of the caisson during sinking. The technique used a domed-end well as shown in Fig. 10.21. Fig. 10.22 shows the caisson for pier W4 at its final depth. The caisson, described by Little,[9] was 28 by 60 m in plan and the

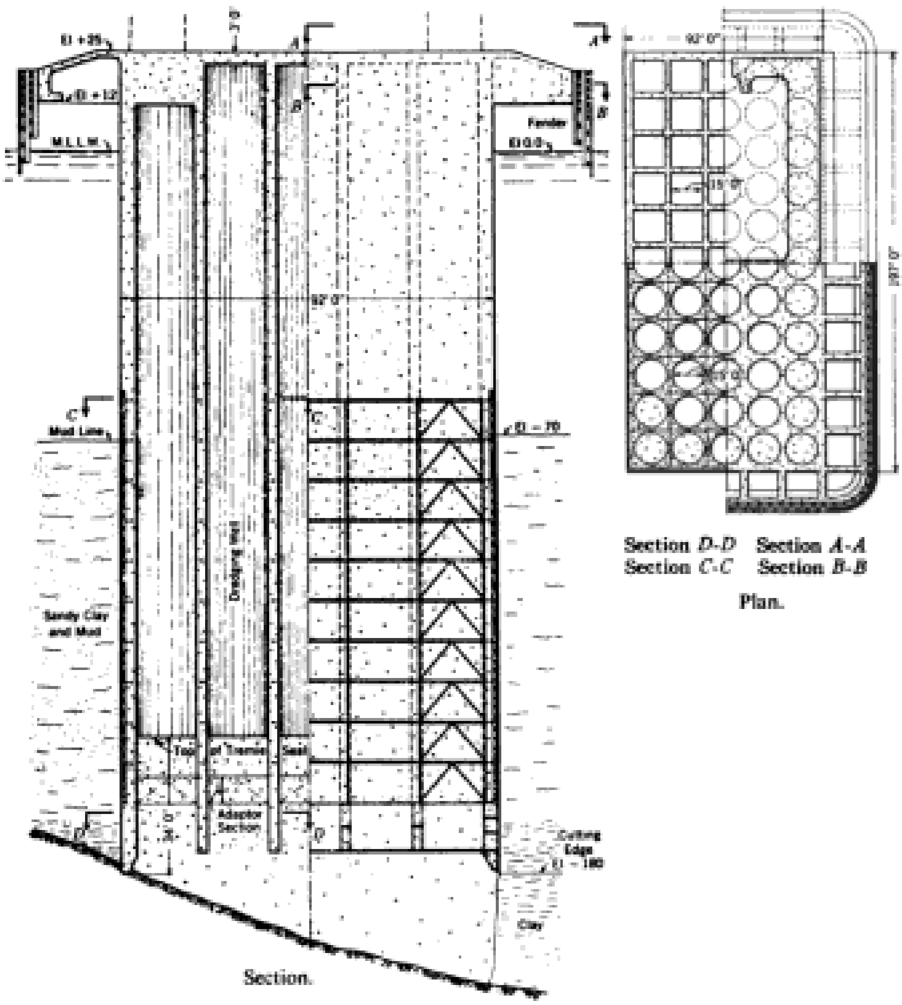


Fig. 10.22. San Francisco–Oakland Bay Bridge: plan and cross-section of caisson for pier W4 at full depth (Little[9])

procedures for founding caissons on sloping basalt at considerable depth below the river bed. At pier 4, this rockhead slope was defined by no less than 31 borings and an even greater number of jet probings. A sloping rockhead was catered for by constructing the caisson's cutting edge to the same gradient (Fig. 10.23). This created problems in constructing and floating the caisson which were overcome by the use of the air domes. By varying the air pressure in the wells the caisson list was corrected after launching and during sinking.

The open caisson therefore allows permanent pier construction to be taken down through sands, gravels and clays to adequate founding soils or rock at very considerable depths. These depths are often necessary because of very large projections of scour risk in fast flowing rivers, but are not without limit. Mitchell[20] referred to caissons being sunk to the exceptional depth of 105 m below the Jamuna River in Bangladesh in the early 1980s. In relatively small caissons, in the dry, excavation may proceed efficiently, but in the majority of cases excavation is only possible underwater and obstructions at shallow depth, such as tree trunks, may only be removed by a diver using small charges; obstructions at greater depths, such as boulders below the cutting edge, may produce unavoidable delays in underwater excavation. Inflow of material beneath the cutting edge may occur due to artesian pressure in groundwater, causing subsidence outside the caisson and loss of bentonite seal placed to lubricate the outer skin. As the caisson is

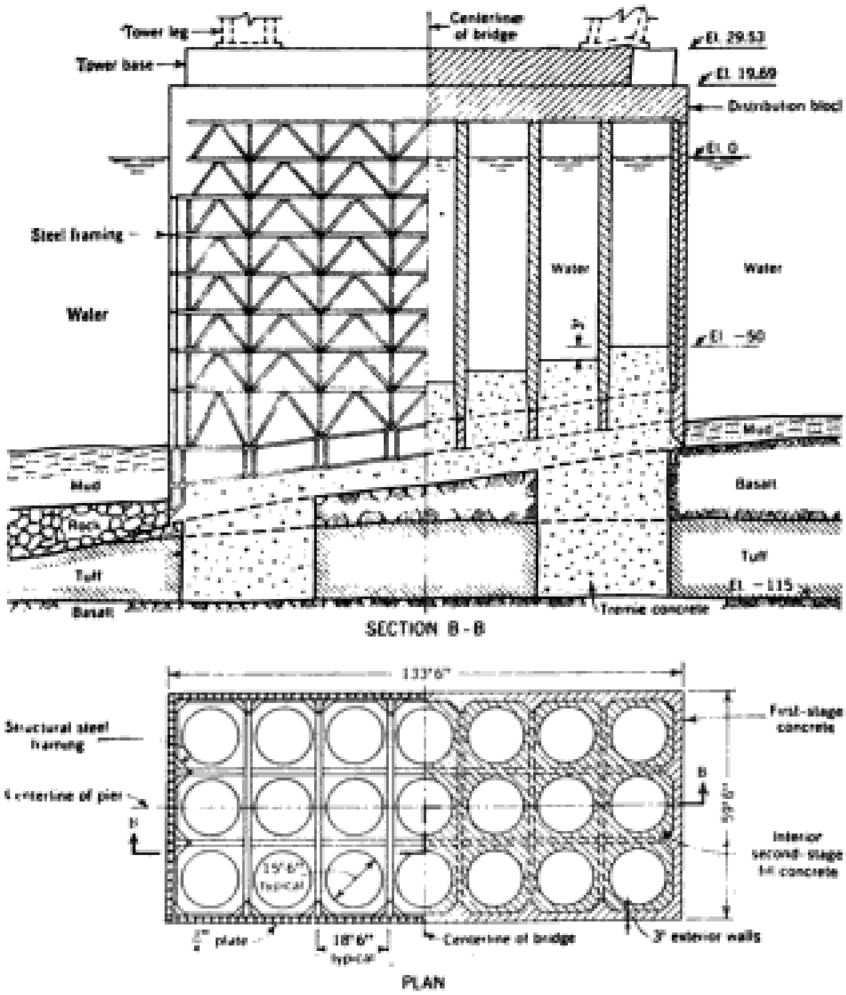


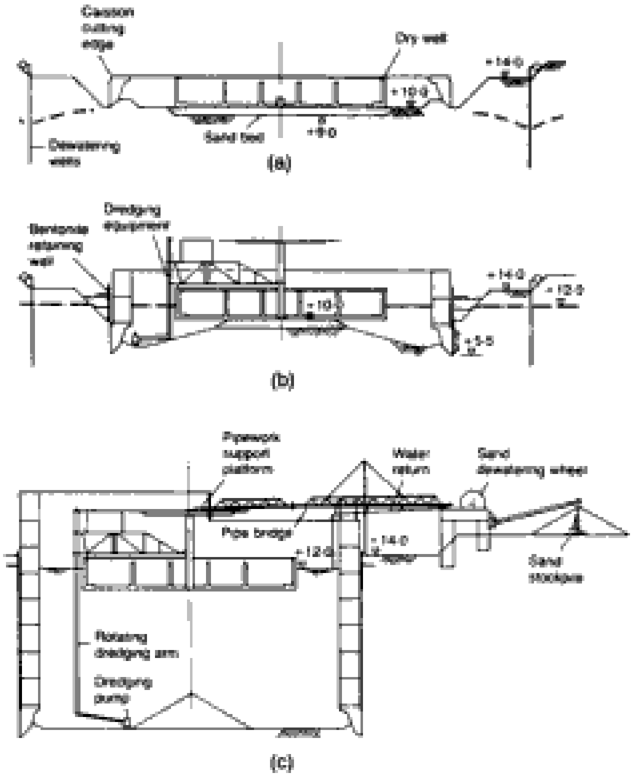
Fig. 10.23. Tagus River Bridge, tower pier 4: plan and cross-section showing sloping rockhead and shaped cutting edge (Riggs[24])

founded, sloping bedrock may cause difficulties in achieving uniform bearing for the pier. Unless the caisson is at depths where compressed air working can be used, underwater excavation and diving work may prolong construction.

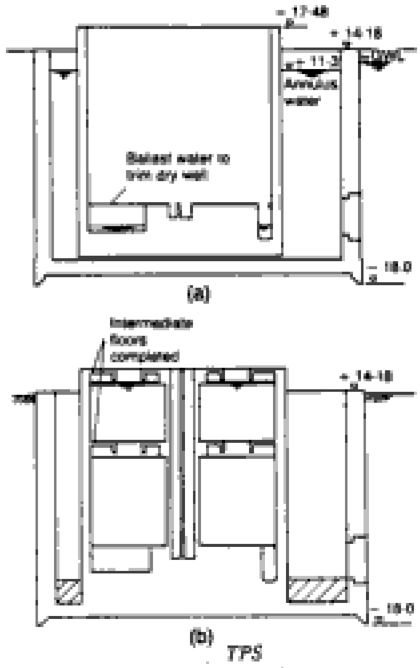
Hang-ups in caisson sinking sometimes occur unexpectedly and early estimates of skin friction may prove optimistic. Water jetting, air and water jetting, bentonite grouting, and even small charges fired in the contained water, may be tried to get the caisson moving. The use of extra kentledge is the final resort. It is highly desirable, therefore, that all caissons should be designed to allow extreme weighting by extra kentledge should difficult hang-ups occur.

Difficulties in sinking a large caisson on the Cairo waste water scheme in 1986 were described by Grimes *et al.*[25] The caisson, for a large pumping station at Ameria, had an external diameter of 45 m and was 37 m deep. The ground conditions were 8 m deep silty clay and made ground from ground level overlying dense to very dense sands and gravels together with fine thin hard grey silty clay layers at −10 and −13 below datum. Although the pre-contract design alternatives included the use of a diaphragm wall with a frozen soil base, the two lowest-bidding contractors both opted for a caisson solution.

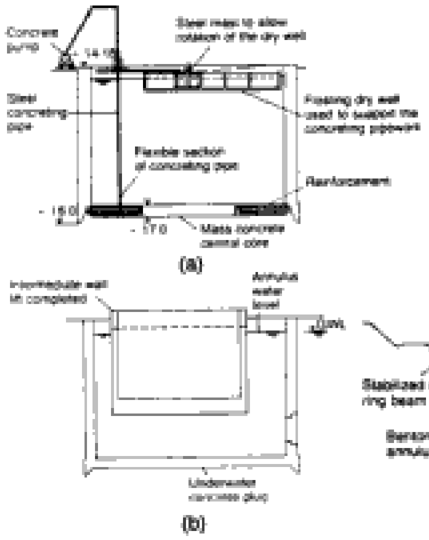
After sinking the caisson an an open well in the dry at shallow depth, wet caisson sinking continued below groundwater level. Dredging equipment operated from a floating platform which was used subsequently as the permanent base to the dry well of the pumping station. Fig. 10.24 shows the sequence of operations prior



TPS construction: (a) stage 1—construction of caisson wall with dry well base inside; (b) stage 2—wet excavaton by dredging from floating dry well; (c) stage 3—wet excavation and caisson wall pours carried out simultaneously



TPS construction: (a) stage 6—dry well landed on bearings and grouted under base; (b) stage 7—annulus is dewatered, ballast water added inside the dry well, permanent floor and radial walls added, then dry well dewatered



TPS construction: (a) stage 4—underwater concreting of base plug; (b) stage 5—construction of dry well walls

Diagram of second-stage excavating equipment for TPS

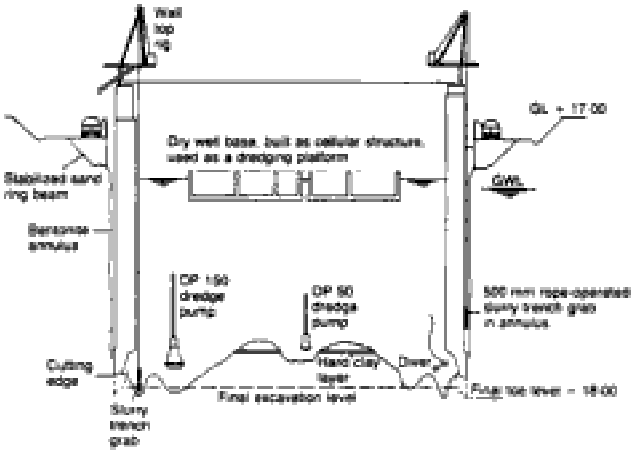


Fig. 10.24. Ameria pumping station, Cairo waste water project: sequence of construction stages 1 to 7 and second-stage excavating equipment (Grimes et al.[25])

to casting an underwater plug to the caisson on which the dry well base could sit. Bentonite was used to assist penetration of the caisson to a total depth of 28 m, but ground loss failures caused considerable difficulty. It became necessary to excavate by grab an annular space outside the caisson walls which was filled with bentonite slurry to achieve final levels.

After a study of the history of open caissons, Gerwick[26] concluded:

(*a*) most cases of initial tipping are caused by attempts to rush the work
(*b*) most cases of serious tipping occur when the steps taken to correct the minor initial tipping are too radical
(*c*) false-bottom caissons are the lowest in cost but are the most prone to tip
(*d*) dome caissons, such as used on the San Francisco — Oakland Bay Bridge, are very expensive and in practice do not give assurance against tipping
(*e*) false-bottom caissons which permit removal of the false bottom are of major value in preventing tipping
(*f*) the double-wall caisson, as used on the Mackinac Straits Bridge, is expensive but is unlikely to tip seriously because of its low centre of gravity and high centre of buoyancy
(*g*) any caisson may act, at times, as a false-bottom type due to soil plugs forming in the wells.

Open caissons in building construction

The use of caissons is not limited to bridge foundations; the technique has found application as a construction method for basements to buildings and for substructures to industrial structures such as pumphouses. In some instance, simultaneous superstructure erection has assisted caisson sinking. Since the 1960s, when slurry trench techniques provided an alternative method of basement construction in constricted urban areas, the use of caissons for building foundations has proved progressively less attractive.

The use of the method in Tokyo in 1950 to construct a four-storey basement of very large plan size is shown in Fig. 10.25. Three basement storeys for the Nikkatsu building were constructed above ground. Temporary timber bearing plates beneath internal columns and outer walls were used to control movement as the caisson was sunk. The triangular-shaped caisson had a plan area of about 0·4 hectare and weighed approximately 25 000 tonnes.

After the caisson was founded at its final level, excavation was made in gravel to cutting edge level and a raft constructed to form a fourth basement.[27] It was claimed that the cost of temporary diagonal bracing to stiffen the caisson structure during sinking was more than offset by the elimination of large quantities of shoring, strutting and sheet piling. It was noted that footways adjacent to the outside caisson wall subsided uniformly 150 to 175 mm; at a distance of 6 m, street subsidence was about 12 mm.

In the early 1970s a method of caisson construction due to Fehlmann and Lorenz was introduced into the UK. The patented method was based on a caisson curb of special design to allow the ciasson to sink under its own weight, the soil adjacent to the cutting edge progressively failing under shear. Bentonite slurry was used as a lubricant. A number of contracts were completed at that time, and 200 contracts had been completed in Europe by 1972.

A 45 m dia. open caisson was sunk using this method through stiff silty clays for a 3-storey basement below a 30-storey tower block at Whickham, County Durham. The cross-section and plan of the caisson are shown in Fig. 10.26. The caisson was divided into 16 sections by radial walls, and a floor in the form of a helix was cast prior to sinking to provide horizontal stiffness and act as a permanent access to car parking spaces in the finished basement.[28]

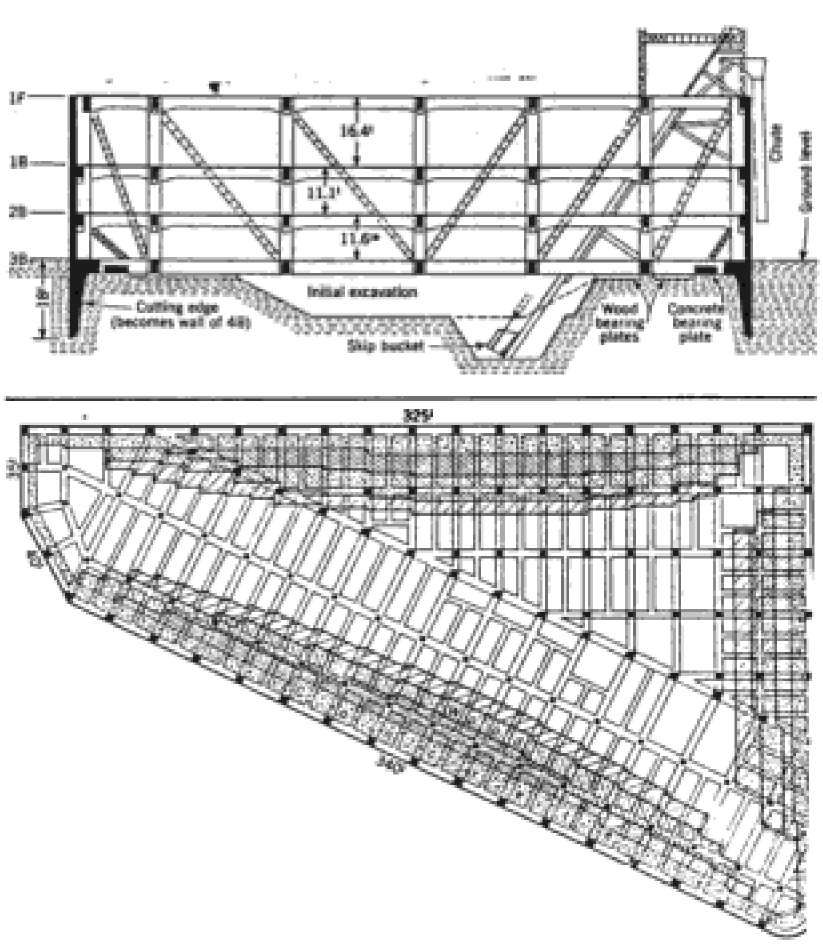


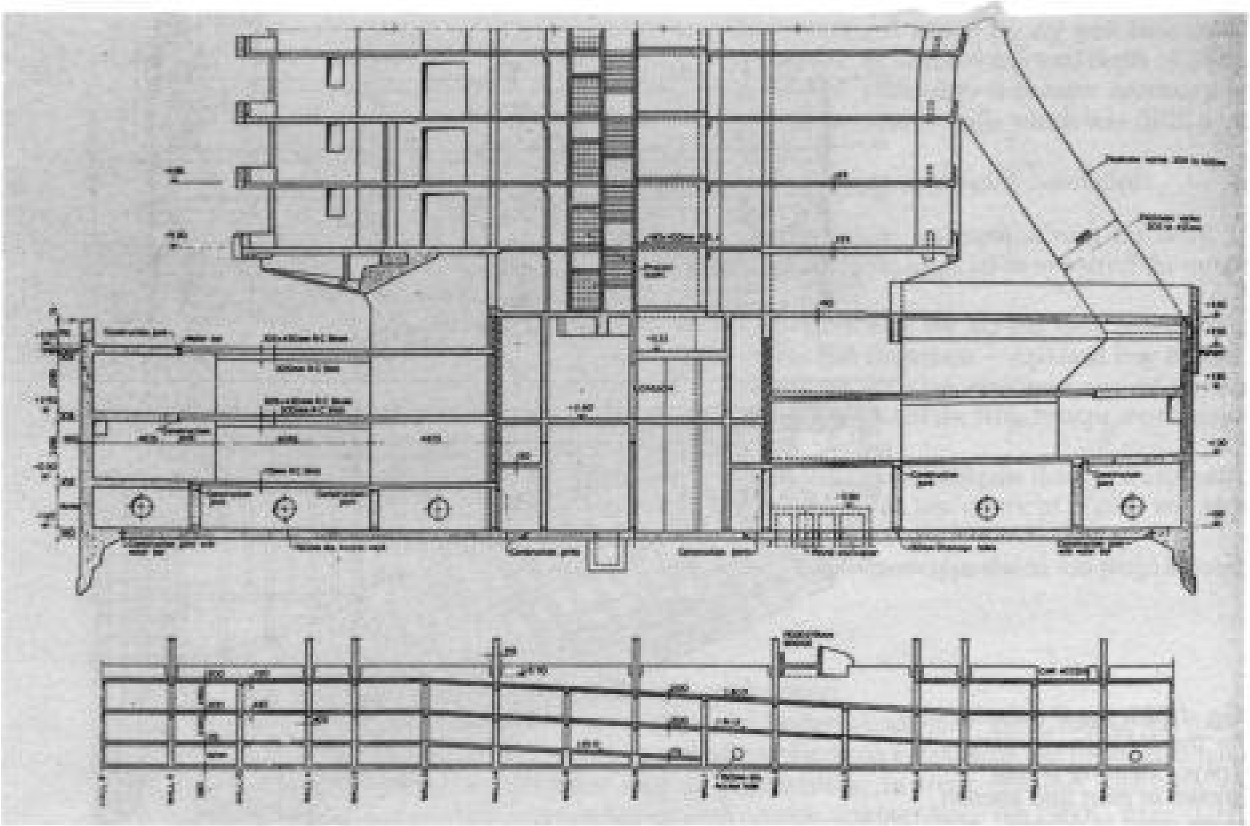
Fig. 10.25. Land caisson for the Nikkatsu building, Tokyo. Bearing plates shown in plan and section were used to control verticality and plan position of the caisson during sinking (Mason[27])

At Worthing the same method was used to sink a 26·5 m dia. caisson, 17 m deep, for use as a foul and surface water pumping station; the reinforced concrete curb is shown in Fig. 10.27.

A larger caisson 64 m long and 27 m wide with an overall height of 19 m, sunk on land as an open well, was used for the substructure to a circulating water pumphouse in the Huntly power project.[29] The subsoil conditions consisted of 2 m thick silt overlying a depth of 50 m of pumiceous sand of high permeability interspersed with silt bands, and at 50 m a layer of hard silt of low permeability. Sheet pile cut-off walls were driven around the caisson periphery, which was built to full height before sinking. The sheet piling reduced excavation volume and improved crane access. Water jets cast into the walls were successfully used with sand pumps and airlifts to control excavation. The caisson was supported permanently on bearing piles driven to siltstone to avoid risk of horizontal movement due to liquefaction of the pumiceous sands under seismic conditions.

Pneumatic caissons

The pneumatic caisson, like the open caisson, is a four-sided box in steel and concrete but with the addition of an air deck so that the caisson bottom is like a diving bell. Compressed air excludes water from this bottom section and enables work under air to carry out the excavation in the dry. The air pressure balances the pore-water pressure at the cutting edge. The maximum depth to which pneumatic caissons can be sunk is therefore directly controlled by the maximum air pressure at which work can proceed. Some labour health laws permit a maximum pressure of 3·4 bar. This pressure balances an external piezometric head of 35 m and, unless



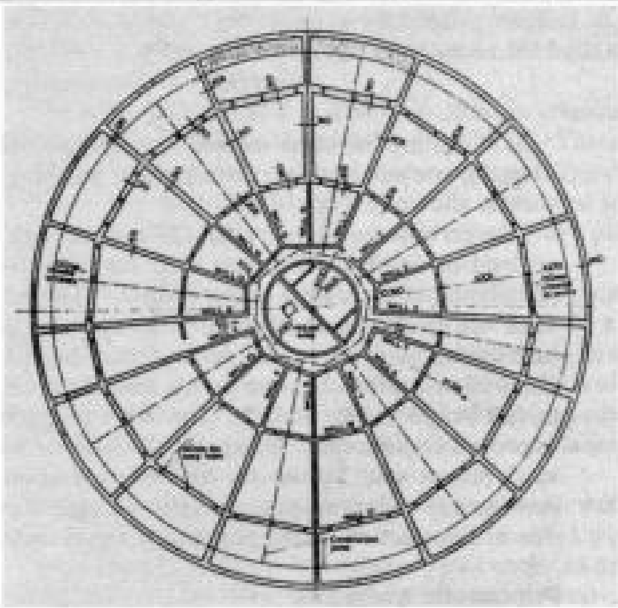


Fig. 10.26. Whickham caisson: cross-section and plan (Nisbet[28])

a reliable means of reducing the groundwater head is used, is the absolute limit for the pneumatic caisson working.

The use of high pressures limits the working hours in the caisson, and both labour and insurance costs become very high. From a health and safety point of view, the whole subject of working under compressed air for sustained periods has been of increasing concern in recent years. The use of pneumatic caissons has therefore declined for reasons other than just cost.

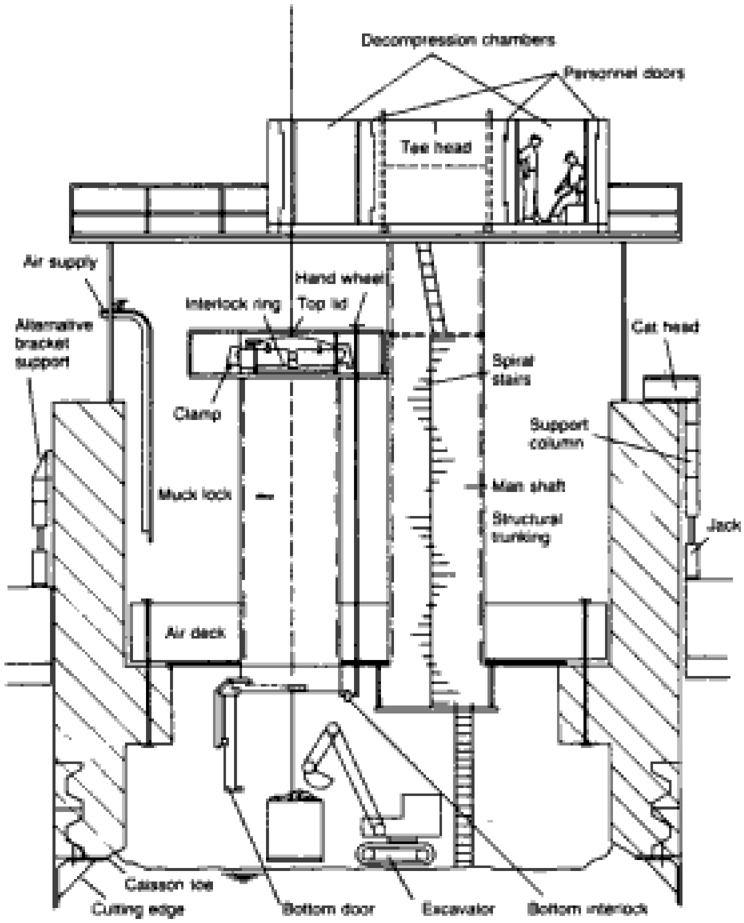


Fig. 10.29. Cairo waste water scheme: cross-section of full-depth caisson showing personnel, mud locks and excavation (Flint et al.[31])

with nozzle pressures in the range 20 to 40 N/mm^2 can be used to augment the work of hydraulic bursters to break boulders.

Figure 10.29 shows a cross-section through a caisson of later design used on the Cairo waste water system construction in the early 1990s.[31] Excavation, by a Smalley hydraulic excavator into 2·5 m^3 muck skips, took six to eight weeks for each shaft from application of compressed air to pouring the concrete plug. On contract 4 of the waste water scheme these shafts were sunk through typical Cairo subsoil conditions of fill overlying silts and sands with a high groundwater table, the excavation finishing in sand or gravel layers. The diameter of the finished structure varied from 6 to 10 m. The caissons were only used for the first 10 m after which a concrete segment-lined underpinning shaft, later lined with in situ reinforced concrete, was used. Air pressure varied with depth of the shafts, from 1·6 to 2·3 bar, and needed between 42 to 340 m^3/min free air delivery in the coarser soils.

The shafts are often in a figure-of-eight plan configuation, the muck shaft, usually 1 m in diameter, being placed next to the main access shaft. The shaft sections are usually 3 m lengths to faciliate extension heights to the caisson as sinking proceeds.

For safety reasons, separate air locks for workers and materials, except in the smallest caissons, are sited above maximum water level to allow workers to escape should air pressure fall and the caisson flood. For medium-sized caissons, say up to 100 m^2 in plan, it is usual to provide two muck locks and one man lock. For high pressures with long lock occupancy times, two man locks may be necessary

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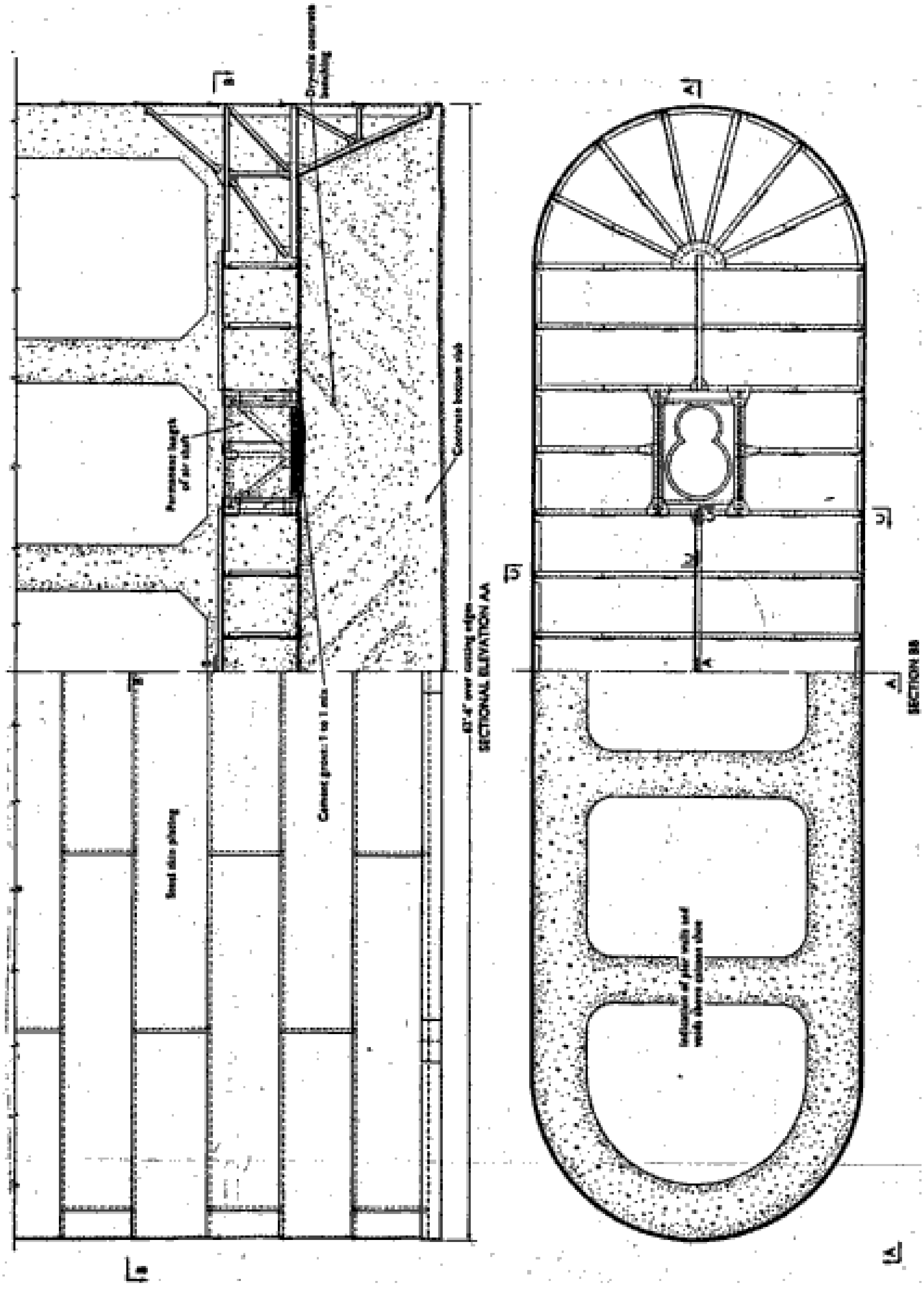

Fig. 10.31. Typical steel caisson for river pier showing deposition of concrete in working chamber (Wilson and Sully[32])

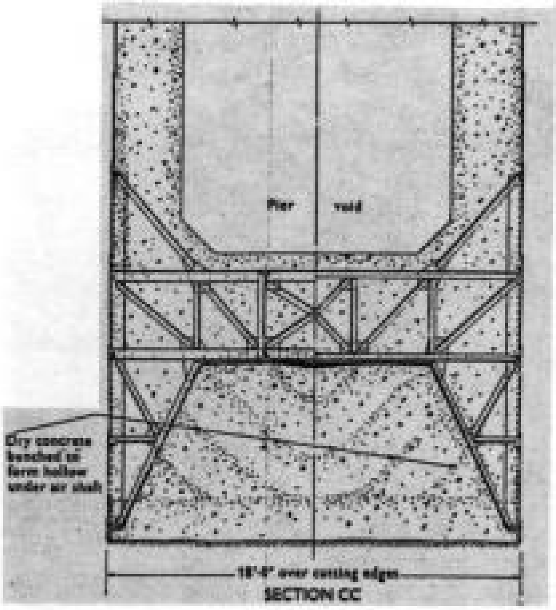

Fig. 10.31 (continued). Typical steel caisson for river pier showing deposition of concrete in working chamber (Wilson and Sully[32])

The cutting edge, made from stiffened steel plate, is vulnerable to buckling during caisson sinking and many problems can result. Wilson and Sully[30,32] recorded that the cutting edge shown in Fig. 10.32(c) was used through sand and on to rock on a pumphouse caisson at a steelworks. Considerable excavation was needed under the cutting edge, and pressure from rock not cleared sufficiently from the back of the cutting edge caused the steel plate to be bent inwards as the caisson dropped, causing the concrete to spill on the inside of the haunch and to damage the haunch reinforcement steel.

The cutting edge and haunch detail shown in Fig. 10.32(b) was designed for use in boulder clay, the slight outside slope of the haunch being provided to avoid pincer action in any uneven sinking. In the event, the detail proved less than successful; soft silt overlying the boulder clay and within the small angle of the haunching allowed the caisson to sink quickly and unevenly. Wilson and Sully described troublesome blows caused by loose soil falling and becoming lodged between the outside curb face and the excavated soil surface during sinking. This loose soil formed an inefficient seal and any excess air pressure within the chamber led to rapid air loss and water entry below the cutting edge.

The grain silo caisson curb detail shown in Fig. 10.32(a) was used for sinking predominantly through silty sand with some clay at deeper levels. Note the sloping inner face to the steel cutting edge instead of the usual horizontal lower face. The caisson sinking operation was quite successful.

Gerwick[26] pointed out that the cutting edge must meet the following requirements:

- (*a*) it must be strong and rugged to resist extremely high localized pressure, such as might be caused by a boulder
- (*b*) it must be designed to resist twisting, shearing, crushing and particularly the tendency to spread outward because of its sloping inner surface
- (*c*) its plates must be adequately stiffened and outside plates must be heavy
- (*d*) connection and splice details must be rugged and strong
- (*e*) there must be provision for ease of concrete placement inside the cutting edge, and provision for seal placement to avoid voids
- (*f*) there must be sufficient vertical diaphragms to make the cutting edge cross-section act as a whole
- (*g*) all cutting edges must be tied together in a rigid frame to resist distortion

pressure of 2·07 bar, requiring a maximum air consumption of approximately 113 m^3 per minute. Construction time for sinking was 20 weeks.

A considerable number of caissons were sunk for working shafts during the several phases of the Cairo waste water scheme. In some instances caissons were sunk in very narrow working sites in streets in the residential quarter of the city. Fig. 10.34 shows a particular example of a caisson site immediately adjacent to existing buildings. Originally designed, pre-contract, as underpinned caissons of limited depth, on some sections contractors resorted to full-depth caissons (Fig. 10.35). Flint *et al.*[31] concluded that the lack of any distress to very close existing

Fig. 10.34. Sinking of caisson in a narrow city street, Cairo waste water scheme (Flint et al.[31]*)*

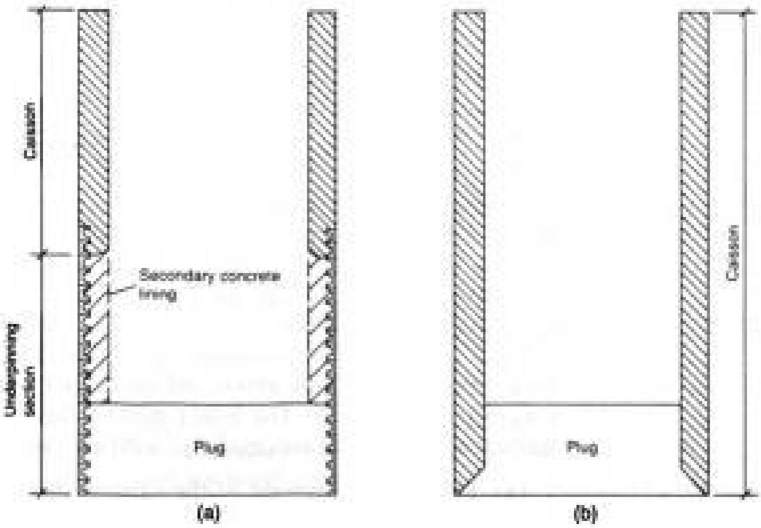

Fig. 10.35. Cairo waste water scheme, alternative methods of shaft construction: (a) Engineer's original design; (b) Contractor's design (Flint et al.[31]*)*

Table 10.2 Observed values of skin friction in caissons (Tomlinson[18])

Site	Type of caisson	Approximate plan size (m)	Soil conditions	Observed skin friction (kN/m²)	Reference	Remark
Howrah Bridge, Calcutta	Open well and pneumatic	55 × 25	Soft clays and silty sands	29	Howarth and Smith[14]	Measured value from Pneumatic caissons
Baton Rouge Bridge, River Mississippi	Open well		Caisson 1: Stiff clay.	38·3	Baine[21]	
			Tight clay grading to very sandy clay	40·7		
			Tight clay grading to very sandy clay with lubricating jets	31·0		
			Caisson 3: 9·5 m sand grading to gravel. 4·6 m clay and sand 12·2 m stiff clay 3 m clay and sand 2 m sand	40·5		
			Caisson 4: 14·3 m sand and clay. 16·1 m sand, sand and clay	35·2		
			Caisson 5: 8·5 m silt 29·9 m fine sand	53·8		
Lower Zambezi Bridge	Open well and pneumatic	11 × 6	Mainly sand	22·9	Howarth[23]	
Uskmouth Power Station	Pneumatic	50 × 33·5	12·2 m soft clay	55·0	Wilson and Sully[30]	
Grangemouth	Open well	13 × 13 19·5 × 10	Very soft clay Very soft clay	4·75 5·75–10·0	Murdock, discussion on paper by Pike and Saurin	
Kafr-el-Zayat	Pneumatic	15·5 × 5·5	Sand and silt	18·7–26·3	Hyatt and Morley	
Grand Tower Mississippi River	Open well	19 × 8·5	Medium fine sand and silt	Above W.L.:51 Below W.L:29·7	Newall[36]	Dewatering from wells shown to effect friction
Verrazano Narrows	Open well	69·5 × 39	Medium dense to dense sand and fine gravel	84·75–95·4	Yang[19]	At lowest stage of sinking to 40 m
Gowtami	Open well	9 × 6	9·1 m sand 13·7 m stiff clay 7·6 m sand	12·6	Ramayya	
New Redheugh Bridge, Newcastle	Pneumatic	11 m diameter	5·5 m depth below river bed. Hard boulder clay and dense sand at shoe	33	Mitchell[26]	North caisson. Reduction of friction of 20% assumed due to bentonite above point of injection.
			10 m depth. Dense sandy gravel and cobbles at shoe	38		
			12 m depth. Top of shoe in gravel, bottom in mudstone	36		
King Edward VII Bridge, Newcastle	Pneumatic	34·6 × 10·7	South caisson. Sand and gravel overlying shale.	31·6	Davis and Kirkpatrick[37]	
			Centre caisson 4·9 m sand and gravel overlying 3 m shale.	26·8		
			North caisson sand and gravel overlying shale	35·5		

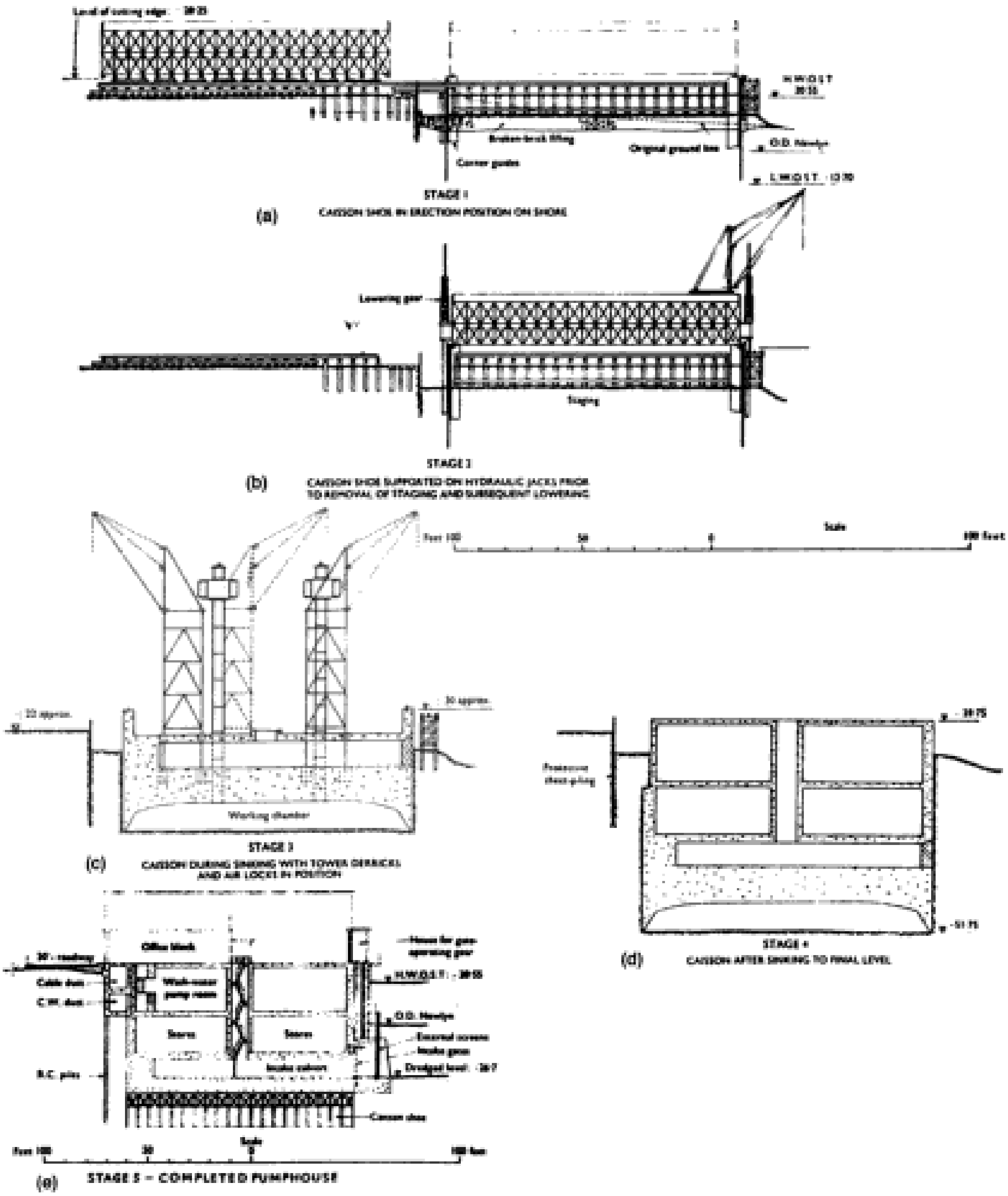

Fig. 10.36. Uskmouth Power Station, stages in erection and caisson sinking: (a) caisson shoe in erection position on shore; (b) caisson shoe supported on hydraulic jacks prior to removal of staging and subsequent lowering; (c) caisson during sinking with tower derricks and air locks in position; (d) caisson after sinking to final level; (e) completed pump house (Wilson and Sully[32])

The development of large diameter augered piles, called 'caissons' in North America, had enabled some caisson sinking underwater or under air to be replaced by more economical and safer means. In the 1980s major bridge crossings have tended to rely on foundation construction by augered large diameter piles rather

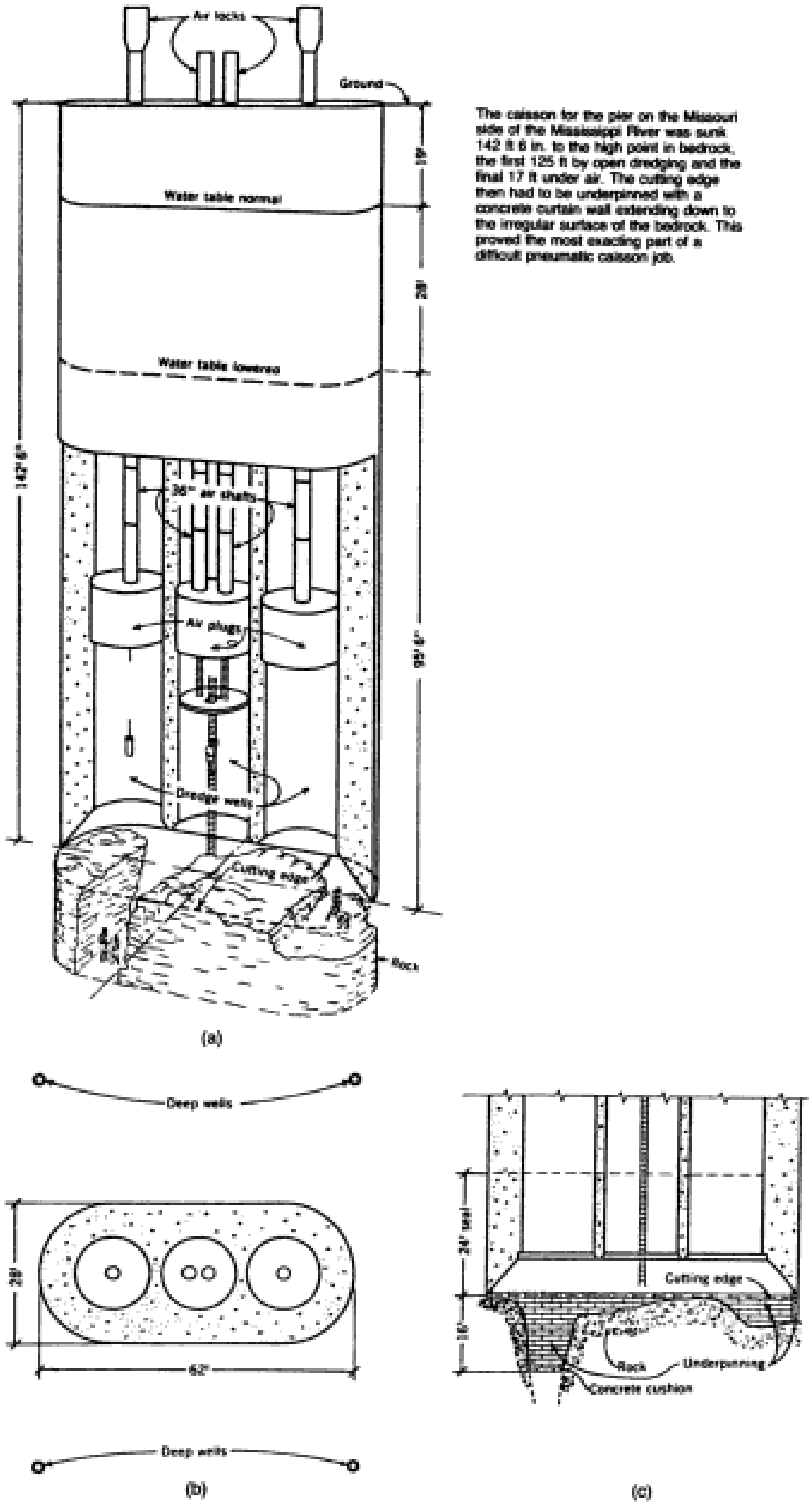

Fig. 10.37. Pneumatic caisson, Grand Tower Pipe Bridge, River Mississippi: (a) cut-away view; (b) horizontal cross-section; (c) arrangement of underpinning (Newell[36])

Table 10.3 Comparison between open well and pneumatic caissons (from Wilson and Smith[39])

Open well caissons	Pneumatic caissons
'Normal' heavy excavating plant only required. No restriction on excavation	Special compressed air plant required. All excavated material has to pass through air locks
No unusual precautions for personnel and normal rates of pay	Special medical precaustions, e.g. examination and rejection of unsuitable men — special care during and after decompression — provision of medical locks. Higher rates of pay
No limit of depth so far reached: 240 ft below water level at San Francisco Bay bridge through 200 ft of mud and soft deposits	Practical limit of depth, 120 ft below water level. In some countries restrictions on maximum air pressure
No men working inside caisson except occasional diver's inspection	Men work inside caisson during sinking
Obstacles may hold up sinking and may have to be removed 'blind' by blasting or other means	Obstacles clearly seen and the best method of dealing with them assured
A zone around the inside perimeter of the well (say 2 ft wide) not accessible to the grabs (this is not serious except for small wells)	Whole of the area beneath the caisson accessible to the excavators
Final cleaning up before concreting must be done underwater by grabs. If the depth is not too great, a diver can inspect	Foundation can be thoroughly cleaned and inspected before concrete is placed
Excavation is best suited to soft materials	Excavation can be done in any type of material

than resort to conventional caisson construction. Typical of these crossings was the Ajaokuta Bridge across the River Niger in the mid-1980s. Using a large jack-up barge and service barges for bentonite slurry and concrete supply, piles each 2·1 m in diameter were bored at each pier position using reverse circulation techniques. Due to the valleys in the rockhead below the river bed some piles were bored to a depth of 50 m in silty sands and gravels. Piling work is outside the scope of this book, but the use of piles of this size and capacity, with a maximum safe load of 1300 tonnes, augered under bentonite slurry with the minimum of casing from a large jack-up, provides an alternative to caisson construction on many jobs.

Morris and Hannant,[40] in discussion to their paper on 17th July Bridge in Baghdad, commented that all tenderers submitted alternative tenders based on large diameter piles from 1 to 2·5 m in diameter in addition to the contract design tender based on caissons. They added that at the time of tender invitation there was a shortage of local labour experienced in working in compressed air and because of this large diameter piles were economical. For peace of mind the authors said they would have preferred caissons, presumably because of scour risk.

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11 Soil movement due to deep excavations

In earlier times the task of the temporary works engineer was to design the peripheral soil support to a deep basement excavation to provide an adequate, but not over-generous, factor of safety against collapse. Risk was identified in terms of the adequacy of the structural strength of strutting, shoring, anchoring, sheeting or walling. Addressing the risk of excessive deformation of sheeting and bracing was frequently not a high priority. Now this has changed and the provision of deep basement accommodation on urban sites has raised to a new importance the serviceability design conditions of acceptable horizontal and vertical soil movement around and below the excavation. As basements are built to greater depths and building developments occupy greater plan areas, the problems of subsidence, heave and horizontal soil movement themselves become priorities. Insurers are no longer prepared to cover risks of property damage which can be recognized, from previous experience, as inevitable. This chapter addresses those factors which cause soil movement around an excavation, typically a large deep basement excavation, the measures which can be taken to alleviate soil movement, and the methods available to the designer to predict movement.

Factors that influence soil movement

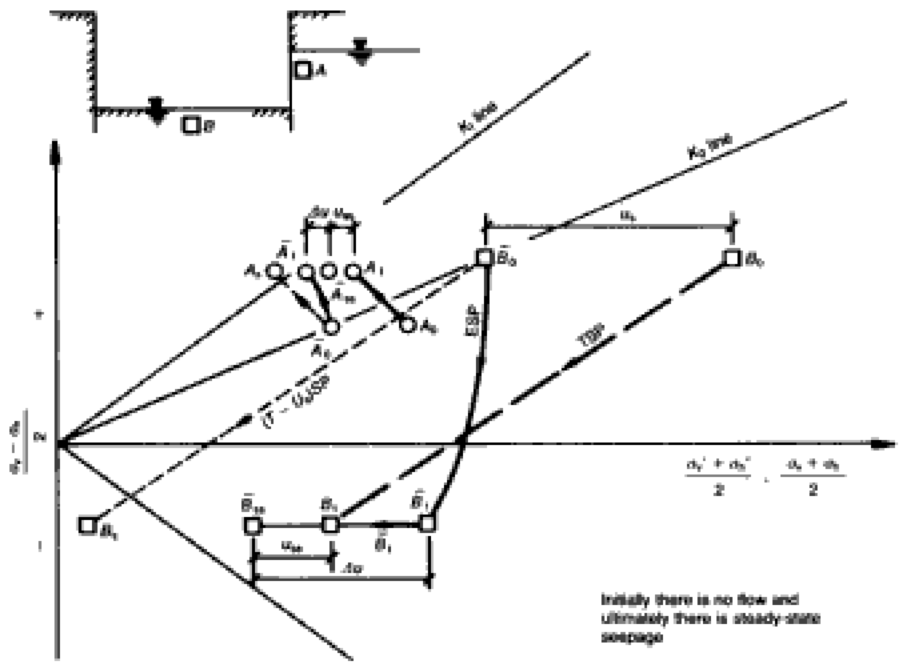
The principal factors which determine the extent of soil deformation have been listed for conditions in Hong Kong.[1] For wider geographical application the list of factors influencing soil deformation around a deep basement excavation is slightly longer:

(*a*) effects of stress changes within the subsoil
(*b*) dimensions of the excavation
(*c*) soil properties
(*d*) initial horizontal stresses within the soil
(*e*) groundwater conditions and changes to them
(*f*) stiffness of the sheeting and bracing system
(*g*) effects of pre-load in bracing and anchoring
(*h*) construction methods
(*i*) construction workmanship.

This list is not given in any order of priority since the importance of each factor varies from job to job. The list is examined in more detail below.

Effects of stress changes within the subsoil

The changes in stress which occur within two soil elements, one to the side (element A) and the other below a sheeted excavation (element B), are shown in Fig. 11.1. The stress paths are typical of those within a normally-consolidated clay. The reduction in total vertical and horizontal stress as excavation proceeds and the change in equilibrium pore-water pressure have important effects on soil deformation. The time-related changes in strain during excavation are themselves related to changes in effective stress as pore pressures attempt to normalize. In the consolidation process the rate of pore pressure dissipation is related to drainage



Stresses and strains for soil elements near an excavation	Soil element A	Soil element B
Initial (static) pore pressure u_0	$A_0\bar{A}_0$	$B_0\bar{B}_0$
Pore pressure at steady-state flow u_{ss}	$\bar{A}_1\bar{A}_{ss}$	$\bar{B}_1\bar{B}_{ss}$
Pore pressure upon unloading	Decreases	Decreases
Pore pressure during consolidation	Decreases	Increases
Strain upon unloading	Vertical compression	Vertical extension
Strain during consolidation	Vertical compression	Vertical extension
Undrained shear strength during consolidation	Increases	Decreases

Fig. 11.1. Stress paths for soil elements near an excavation (Lambe[2])

efficiency, that is, the permeability of the soil fabric and the readiness of moisture supply.

The vital factor influencing the horizontal movement of soil below excavation level, and therefore the magnitude and extent of vertical settlement, is the proximity of the unloading stress path of element B to the failure envelope. If the stress path $\bar{B}_1$ to $\bar{B}_{SS}$ is well within the effective stress failure envelop K_f, this shows that the yield itself is small and both heave and the resulting horizontal soil movement will also be small. Conversely, if the effective stress points for element B are close to the failure envelope, this indicates risk of excessive yield, local passive failure and high lateral movements.

Dimensions of the excavation

The plan shape, the plan area and the excavation depth all critically influence the extent and distribution of soil movement around and below a basement excavation in given soil conditions. The depth obviously affects movement; Tomlinson[3] referred to unavoidable inward movement in normally-strutted or anchored excavations of the order of 0·25% of excavation depth in soft clays and 0·05% in dense granular soils or stiff clays. It is usual to assume that the volume of horizontal soil movement at the sheeting within a unit length of excavation suport is approximately equal to the volume of vertical soil movement at ground level over the same unit length. As a rule of thumb, horizontal soil movements are likely to extend to a maximum lateral dimension of up to three times the excavation depth. The deformed soil profile therefore begins to take shape, although changes with time (due to pore-water pressure dissipation) and the effects of irregular plan shape complicate a simple assessment of settlement risk.

Soil properties

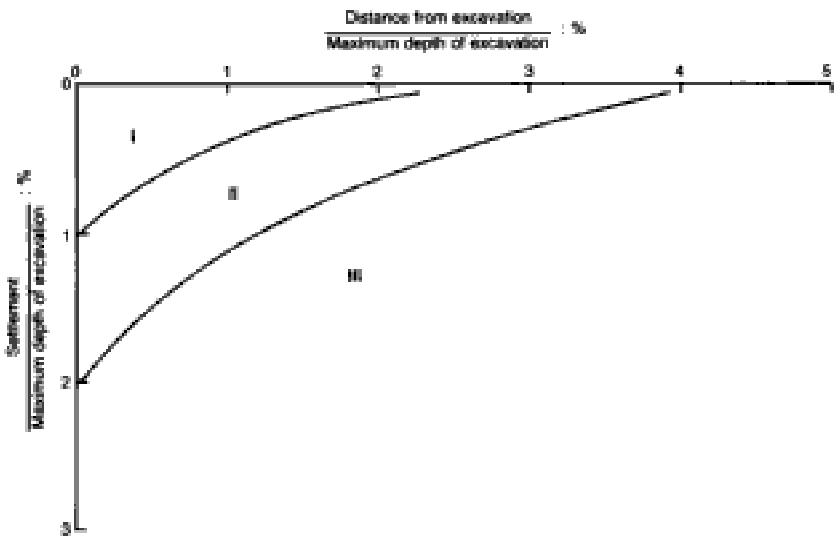
Soil properties were summarized by Peck.[4] Fig. 11.2, after Peck, shows smaller wall movements and ground settlements in stiffer soils, such as granular soils and stiff clays, than in softer soils such as soft and medium clays and loose silts.

Soil movements due to excavations in soft clays may prove to be embarrassingly large, particularly where the clays have been assumed incorrectly to be isotropic. Clough *et al.*[5] and Mana and Clough[6] showed the rate and magnitude of lateral wall movement both increase rapidly as the risk of base heave increases and the factor of safety against base failure reaches unity.

Overall deformation in terms of heave below the excavation and vertical settlement around it will depend on many factors including soil stiffness and, in weaker soils, soil strength. In weak clays and loose silts, yield in soil zones may result in providing passive resistance to peripheral sheeting or walling, with large movements resulting. From a practical viewpoint, in loose cohesionless soils with high piezometric pressure due to groundwater, excavation conditions may be close to quick conditions with risk of vertical soil subsidence and loss of ground between timbering, sheet piles or diaphragm wall joints. Soil and groundwater conditions, therefore, pre-empt all other factors as the prime critical risk of soil movement around deep excavations.

Initial horizontal stresses within the soil

Where high, locked-in horizontal stresses exist within soils, typically within over-consolidated clays, soil deformations surrounding excavations increase, even at relatively shallow depths. For soils with comparatively low values for coefficient of earth pressure at rest k_0, deformations are much less.[7]



Zone I — sand and soft to hard clay, average workmanship
Zone II — very soft to soft clay
Zone III — very soft to soft clay to a significant depth below bottom of excavation

The data used to derive the three zones were taken from excavations supported by soldier piles or sheet piles with cross-lot struts or tie-backs.

Fig. 11.2. Observed settlements behind excavations (Peck[4])

Groundwater conditions

The effects of groundwater on soil settlement are varied and occur at different stages of excavation. Where sheeting penetrates a cohesionless stratum but does not achieve a cut-off at depth, a steady groundwater seepage condition will develop whereby flow is established beneath the sheeting and upwards to the formation level of the excavation. This flow causes a decrease in groundwater pressure, an increase in effective stress and settlement outside the periphery of the excavation. At the same time passive resistance reduces due to the upward flow on the inside of the sheeting, and further horizontal movement occurs as sufficient passive resistance is mobilized. The establishment of a steady-state groundwater regime therefore causes both vertical and horizontal soil movement.

Where dewatering of sheeted excavations causes drawdown to the exterior groundwater table, again where the sheeters to the excavation do not make an adequate cut-off at full penetration, effective vertical soil pressure increases, resulting in vertical settlement. Since the drawdown is greatest near the excavation and reduces progressively with increasing distance from it, this settlement profile will be similar in shape to that due to relief of overburden by the excavation itself.

Stiffness of the support system

Parametric studies using Winkler spring or finite element soil–structure interactive programs and observations made on site show that the exterior ground settlement profile surrounding a sheeted excavation reduces as the stiffness of the sheeting and the bracing supporting it increase. The elastic stiffness of the bracing system appears to be most important. The vertical embedment of the sheeting beneath formation level will also materially alter the effective stiffness of the sheeting and influence external soil movement, both vertically and laterally.

A study of the effects of wall stiffness, bracing stiffness, vertical spacing of supports and embedment was reported by Goldberg *et al.*[8] A summary of the results is shown in Fig. 11.3 in which the stability number is plotted against the stiffness parameter. The data presented also suggest that sheeting stiffness and support spacing effectively influence external soil movements.

Experience over some years in temporary works design using a Winkler spring program has confirmed site observation that increasing strut stiffness decreases external soil movements, although less effectively at very high values of stiffness. These findings regarding the practical importance of sheeting and strutting stiffness are not confirmed by Clough and Davidson[9] nor by Tomlinson.[3] These authors

h vertical distance between support levels or between support level and excavation base
E_w modulus of elasticity of wall material
I_w movement of inertia of wall per unit length
γH overburden pressure
C_u undrained shear strength of the soil

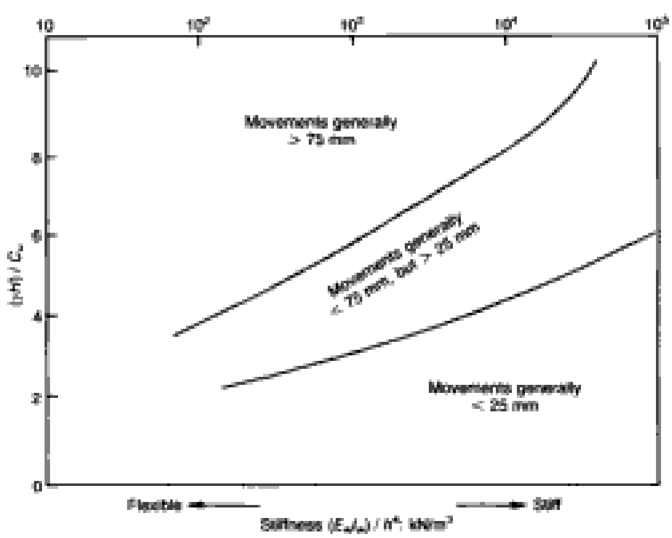


Fig. 11.3. Effects of wall stiffness and support spacing on lateral wall movements (Goldberg et al.[8])

site practices appear in reference 1. These include late installation of supports, over-excavation, poor pile driving and caisson construction, loss of water through holes for tiebacks and joints or sheet pile interlocks and diaphragm wall joints leading to loss of ground, remoulding and undercutting of clay berms, and excessive surcharge loads from spoil heaps and construction equipment. Many more items of inadequate workmanship or supervision standards which cause movement, subsidence or collapse can be added to the list. In particular, the lack of rigidity and tightness of shores and braces are important causes of wall and soil movement. Failure to provide or tighten wedges between walling and walings is a significant cause of movement and subsidence. Similarly, with Berlin walls failure to efficiently wedge horizontal laggings to vertical soldiers and ensure good uniform contact between soil and laggings is a direct cause of soil subsidence behind the wall. Peck[4] pointed out that the choice of detail of lagging connection to soldier could cause settlements adjacent to the excavation to vary widely; settlements adjacent to walls using the detail in Fig. 4.5(b) were three times those using the detail in Fig. 4.5(a).

Measures to reduce soil movement at the curtilage of a deep excavation

To ensure minimum soil movement horizontally and vertically, around and below a deep excavation of given dimensions in given soil conditions, several measures are necessary. Not all may prove to be financially worthwhile, but they are:

(*a*) provide a wall support which provides both temporary and permanent soil support
(*b*) make the sheeting or wall support flexurally stiff
(*c*) avoid installation vibration or other causes of loss of ground
(*d*) ensure the wall has adequate embedment in a stiff stratum
(*e*) ensure the wall receives support at frequent vertical centres and reduce these centres progressively with depth
(*f*) locate the lowest support near formation level
(*g*) make the bracing stiff in compression
(*h*) pre-load the bracing or pre-tension the ground anchors
(*i*) avoid delays in construction of either walling or bracing, avoid keeping diaphragm wall panels open for long periods and avoid delays in bracing works or anchor installation at each support level
(*j*) avoid any loss of ground by over-excavation or removal of fines during pumping
(*k*) avoid drawdown caused by dewatering outside the basement
(*l*) in weak soils, improve ground conditions below formation level to ensure adeaquate passive resistance inside the sheeting from soil with sufficient strength and high stiffness (such improvement could be made by localized jet grouting, vibro-compaction or vibro-replacement).

For deep basements, where soil conditions permit a cut-off against groundwater ingress, the top-downwards method of construction may prove attractive in meeting some of these criteria to reduce external soil settlement. The method has disadvantages, however, including high excavation costs to remove soil from below basement floor construction, the risk of overall delay caused by any local hold-up in a sequence of interdependent construction activities, and the problems in terms of space and access of several specialist firms working on site at the same time.

In shallower basements the use of top-downwards construction may be prohibitively expensive. In such circumstances, the risk of excessive settlements around the site will be minimized by the above methods. In particular, cantilevered walls and excessively high sheeting are a frequent cause of excessive soil movement outside the excavation and should be avoided where possible by propping the sheeting from temporary bases or from a previously constructed raft at the centre of the basement plan shape. Where walls are cantilevered at any stage individual piles or diaphragm panels within the basement wall should be connected by a stiff capping beam in reinforced concrete.

The use of soil berms to minimize lateral movements of walls or sheeters at the periphery of a deep excavation should be noted. The general consensus on the use of berms[11–13] is that the increase in vertical stress using a relatively small volume of soil is often sufficient to reduce lateral movements to walls or sheeters by 50% while the berm is left in place. If the berm is removed in short lengths while rakers or struts are placed, the final lateral movement, and thence the vertical settlement of soil outside the excavation, can be usefully reduced. Numerical studies by Clough and Denby[12] on an excavation in soft to medium clay showed a theoretical relationship (Fig. 11.4) between settlements behind sheet piles with berms and the stability number $\gamma H/c_{ub}$, where c_{ub} is the undrained shear strength at the base of the excavation for the condition after the berm has been removed and the rakers installed. The reduction in ground settlement increases as the stability number increases, and at high stability numbers increasing berm size leads to larger reduction in settlements. This apparent improvement may not be produced, however, where deep-seated movements occur at high-stability numbers with low-strength clays.

Burland *et al.*[13] showed the effect of a soil berm at one stage of a 16 m deep basement excavation in London. The peripheral soils were supported by a diaphragm wall with a depth of embedment 3 m below final formation level. The initial excavation was 10 m deep, at which depth a thick waling slab was cast on the exposed surface of the London clay. Excavation was then made to the full depth leaving a soil berm below the waling slab, as shown in Fig. 11.5. After the basement raft had been concreted in the central area the berm was removed in short lengths and the raft completed to support the wall. Observations of lateral movement of

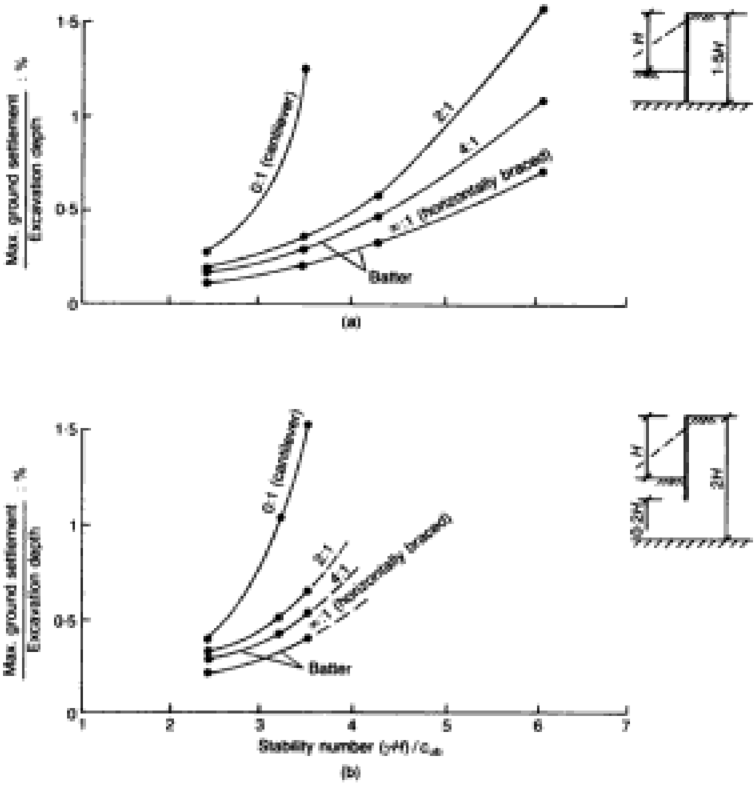


Fig. 11.4. Relationship between maximum ground settlement and stability number at the end of construction after berm excavation for berms of varying batter: (a) fully penetrating, fixed end MZ-27 sheet pile wall; (b) partially penetrating, free end MZ-27 sheet pile wall (Clough and Denby[12])

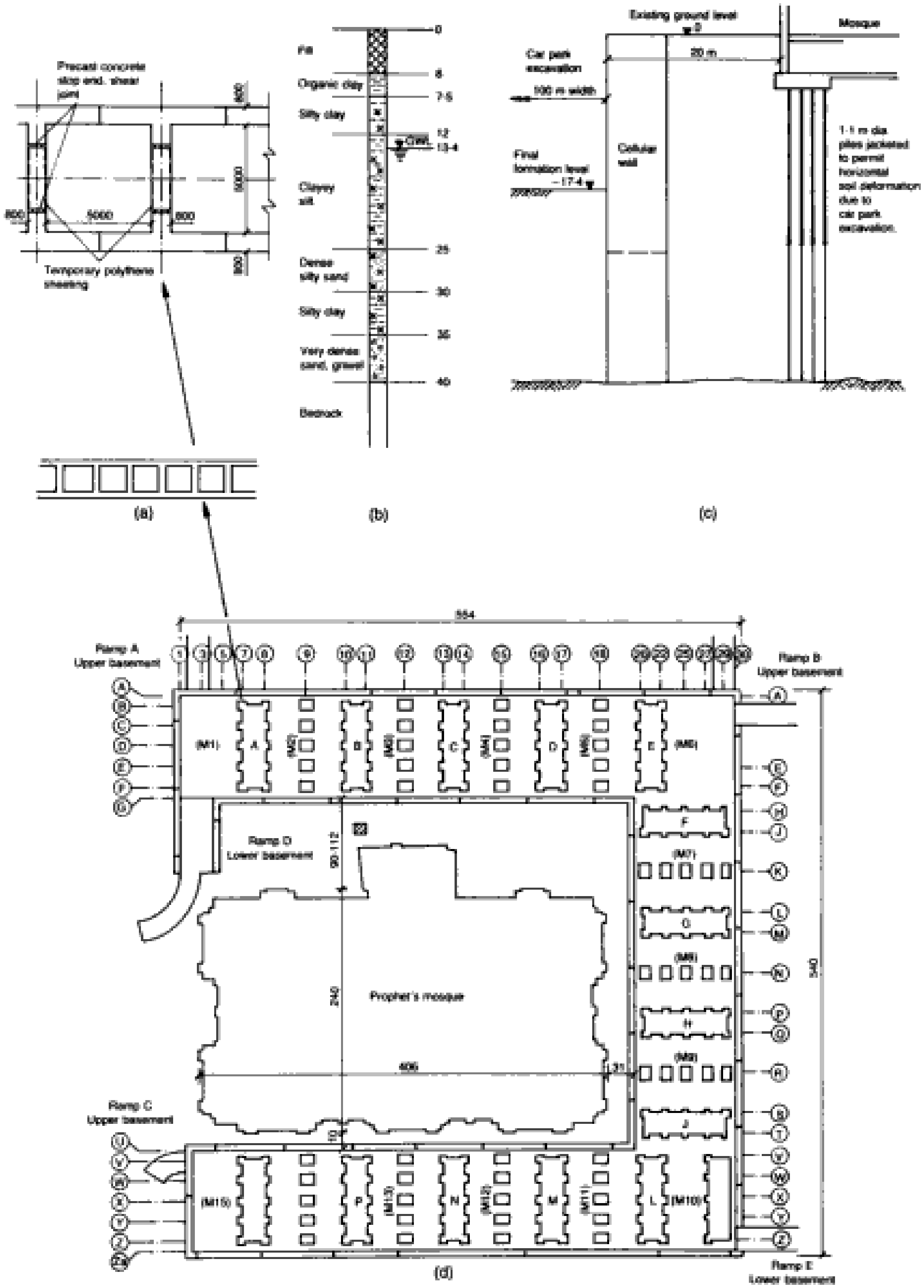
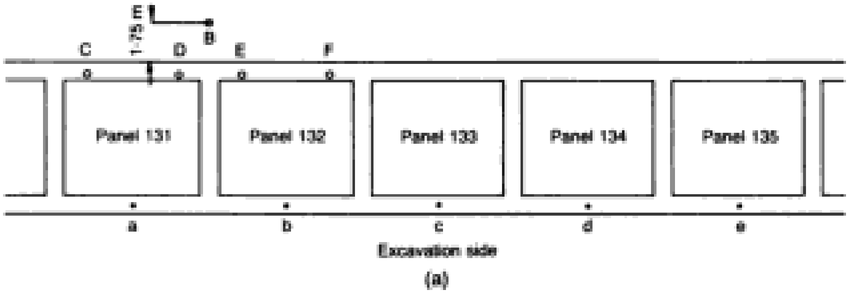
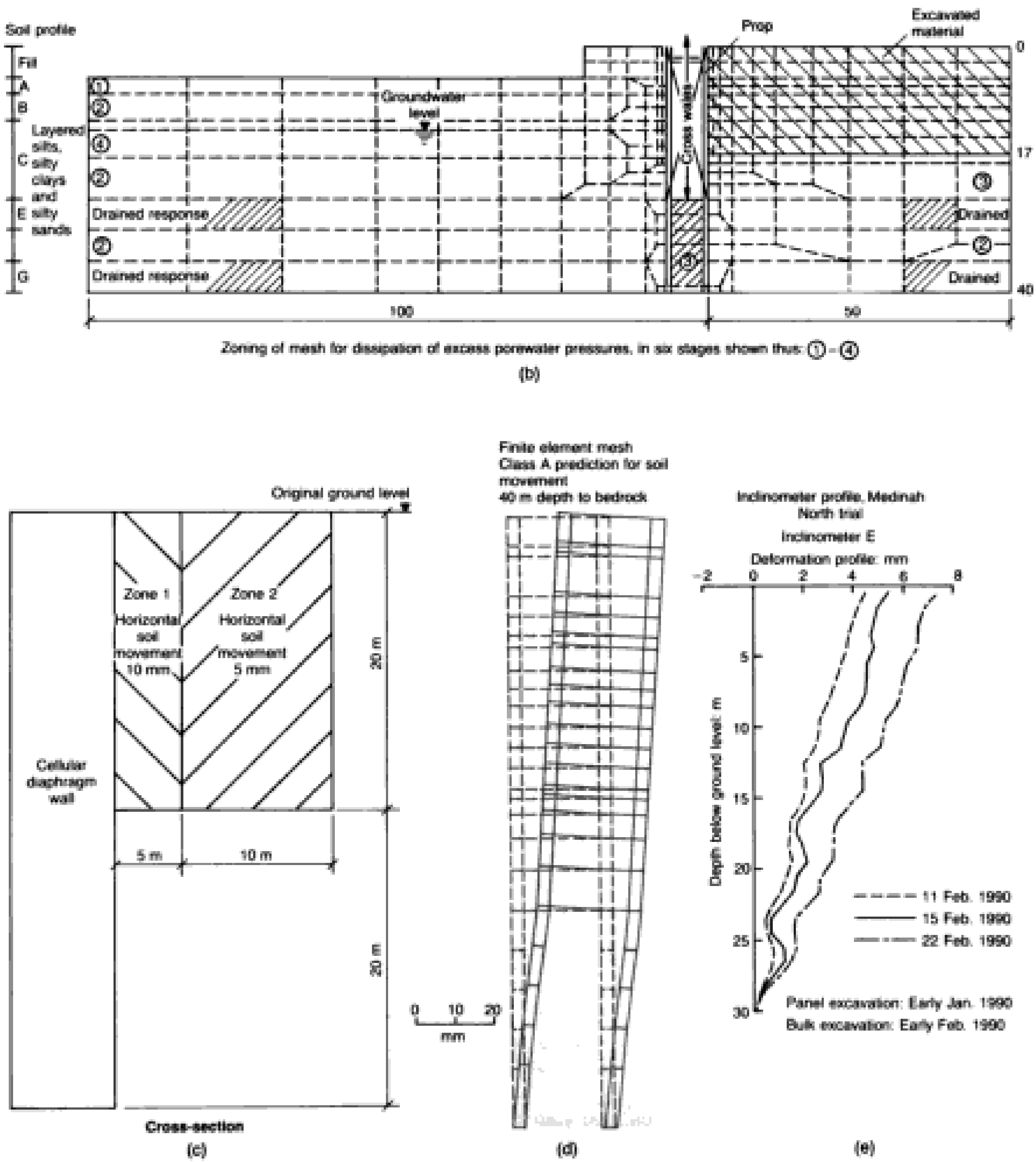


Fig. 11.6. Medinah car park: (a) plan of cellular wall construction; (b) design soil profile; (c) design cross-section; (d) key plan (dimensions are in m)

Fig. 11.7. Medinah car park: (a) key plan; (b) finite element mesh; (c) predicted horizontal soil movement due to panel excavation; (d) predicted horizontal soil movement due to bulk excavation; (e) observed horizontal soil movement at rear of wall by inclinometer E





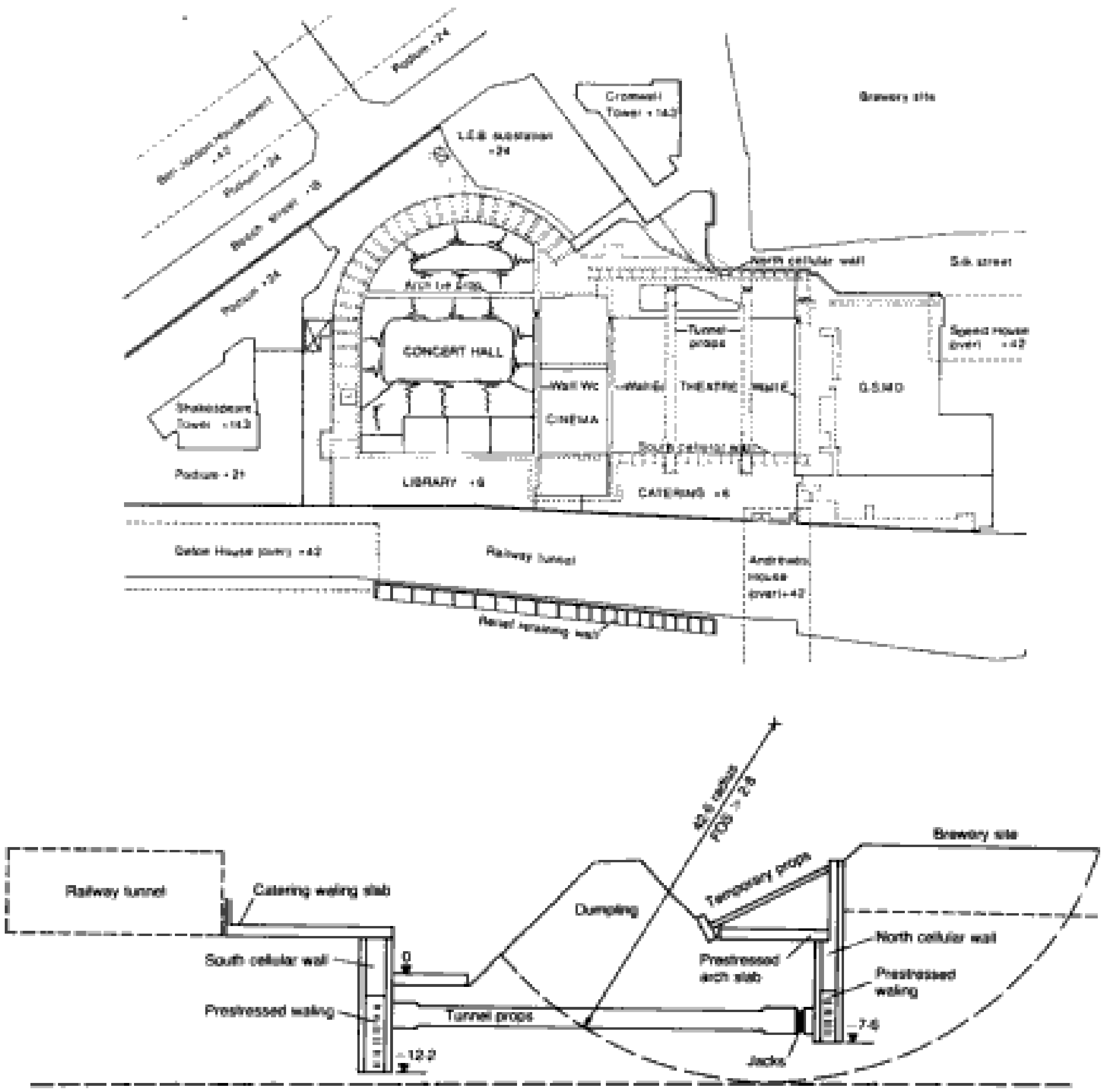


Fig. 11.8. Barbican Arts Centre: site plan and cross-section of site during excavation showing use of preloaded tunnel props (Stevens et al.[14]*)*

The proximity of nearby buildings required major temporary works of a different kind during the substructure construction of a large arts centre at the Barbican in London in the 1970s (Fig. 11.8).[14] Nearby tower blocks, although piled, were susceptible to tilt caused by soil movement during bulk excavation for the theatre, but even more importantly, analyses predicted excessive shear stresses in these piles if significant soil movement were allowed between the piles due to the excavation. Even though cellular diaphragm walls of considerable stiffness were designed at each side of the theatre basement, the predicted horizontal inward movement of these walls below formation level exceeded the maximum that the pile shear could withstand. To prevent this movement below formation level, two props were constructed in tunnels between the cellular walls and pre-loaded with thrusts up to 10 000 kN. The length of the north wall of the theatre exceeds 60 m, with a minimum height of 14 m. The wall was designed to be propped apart by the two diaphragm walls at the east and west sides of the theatre basement and at low level by the two pre-loaded tunnel props, as shown in Fig. 11.8. A horizontal waling was formed by the arch slab at the 6 m level spanning across the whole basement width. The wall itself spans horizontally across the low-level supports and vertically between the arch slab and the prestressed concrete beam within the wall at lower level. Measurements during construction showed that the north theatre wall was moved northwards by a maximum of 10 mm and the south wall moved southwards by 5 mm. The jacks were maintained in an operable state for one year after the basement excavation and were then stabilized by the exchange of hydraulic fluid with epoxy resin grout without loss of pressure. The jacks had therefore fulfilled their purpose and instead of soil movement towards the theatre excavation, the pre-loaded tunnel props caused small movement in the opposite direction. The essential point in this basement design was the risk, avoided by the use of the pre-loaded tunnel props, of progressive wall/soil movement below the excavated level of the theatre basement as it was dug out, the stiffness of the peripheral cellular walls below excavated level being insufficient to reduce it to acceptable levels without the action of the pre-loaded props.

Methods of predicting soil movement

Soil movement behind a supported excavation can be predicted empirically, semi-empirically, finite element or finite difference methods, or by other methods such as velocity fields.

Empirical methods

The risk of settlements in the vicinity of proposed deep excavations can be assessed, in broad terms, from published data from sites in similar soil conditions. The most useful records include those published by Peck,[4] O'Rourke *et al.*[15] and others.[16–18]

Peck's work is summarized in Fig. 11.9(a) showing vertical settlement (as a percentage of excavation depth) against distance from the excavation (plotted non-dimensionally as a ratio of excavation depth). Peck used this plot to draw attention to the distances from the cut at which settlement occurs, and to the experience that settlements in plastic clays were likely to be greater than in cohesive soils and stiff clays. Both immediate and consolidation settlements are included in the settlement data in Fig. 11.9. It should be noted that in very soft to soft clays, settlements as great as 0·2% of the excavation depth can occur at distances of three or four times the depth. The critical influence of excavation depth on vertical settlement in shown in Fig. 11.9(b) for basements in Chicago soils, generally supported by sheet piling with small embedment and cross-lot strutting, or more usually with rakers. The upper 5 m of soil in downtown Chicago consists of fill and sand underlain by a soft clay, becoming stiffer with depth until hardpan is met at 23 m. The single-storey basements shown therefore do not penetrate into the soft clays and the recorded settlements were probably caused by the caisson construction on which the basements were founded rather than by the basement excavation. The care required in extrapolating data obtained from one set of soil conditions to another site with an inexact match of soil conditions is self-evident. The data published by Peck serve only to show the order of settlement and the extent to which such settlements are likely to occur in soft clays.

O'Rourke *et al.*[15] published settlement data for excavations supported by soldier piles and horizontal laggings with cross-lot strutting, in dense sand and interbedded clays in Washington, DC (Fig. 11.10). In these conditions, maximum settlements of the order of 0·3% of excavation depth were recorded immediately to the rear of the sheeting and extended up to twice the excavation depth laterally from the rear of the excavation. O'Rourke *et al.* also published records of settlement

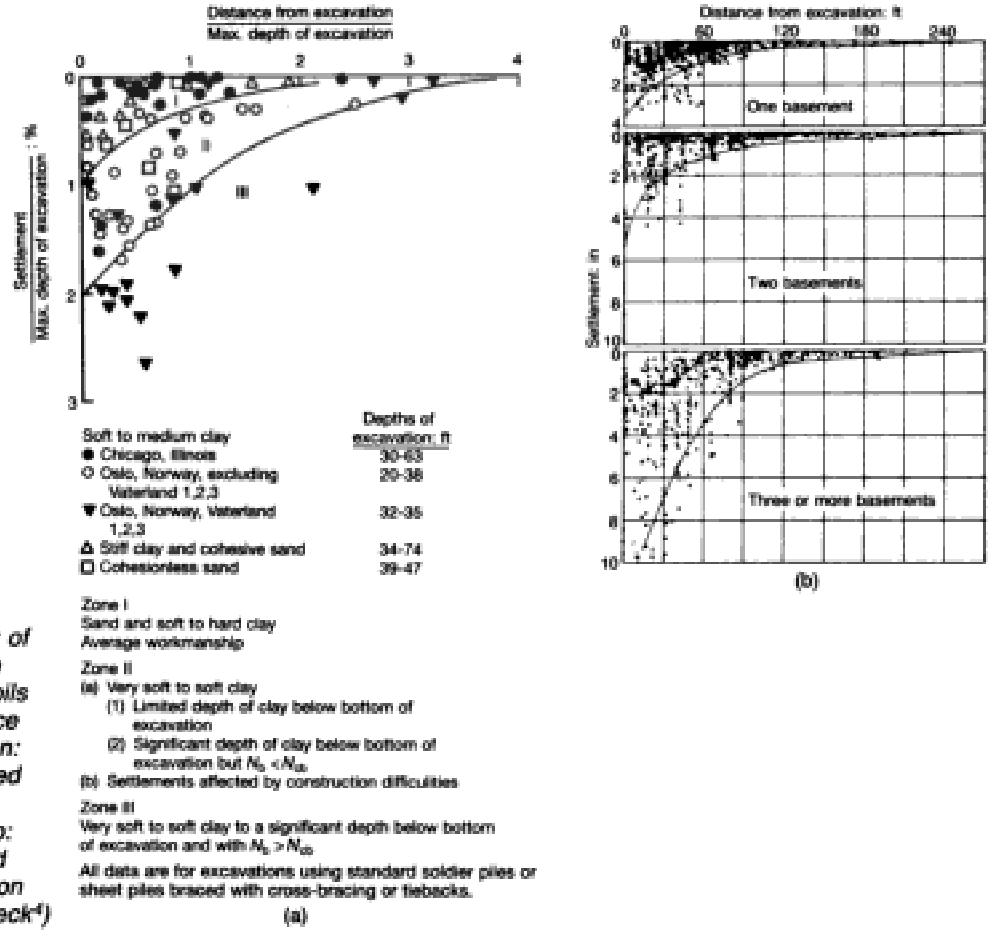


Fig. 11.9. (a) Summary of settlements adjacent to open cuts in various soils as a function of distance from edge of excavation: (b) settlement associated with foundation construction in Chicago: summary of results and settlements as a function of excavation depth (Peck[4])

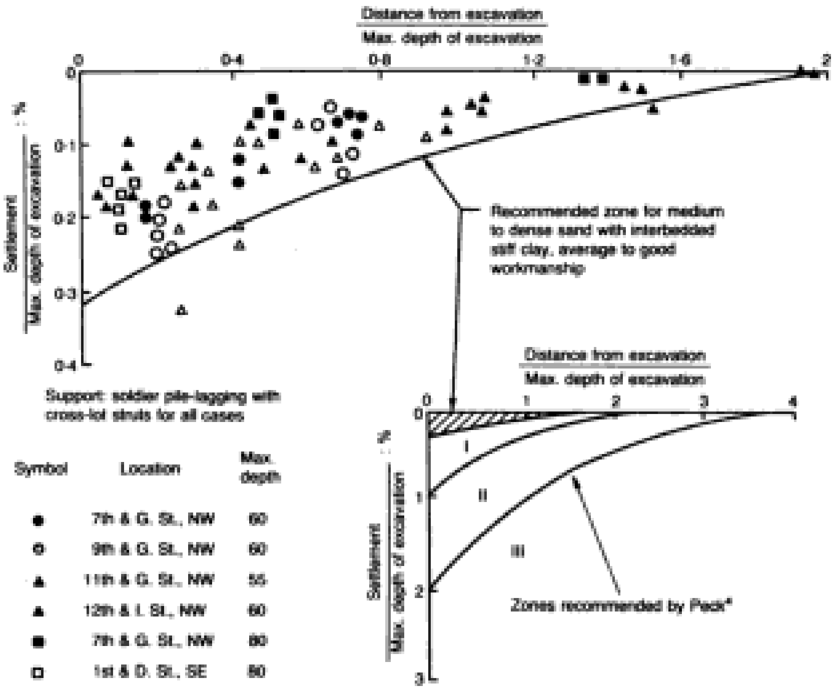


Fig. 11.10. Summary of measured settlements adjacent to strutted excavations in Washington, DC (O'Rourke et al.[15])

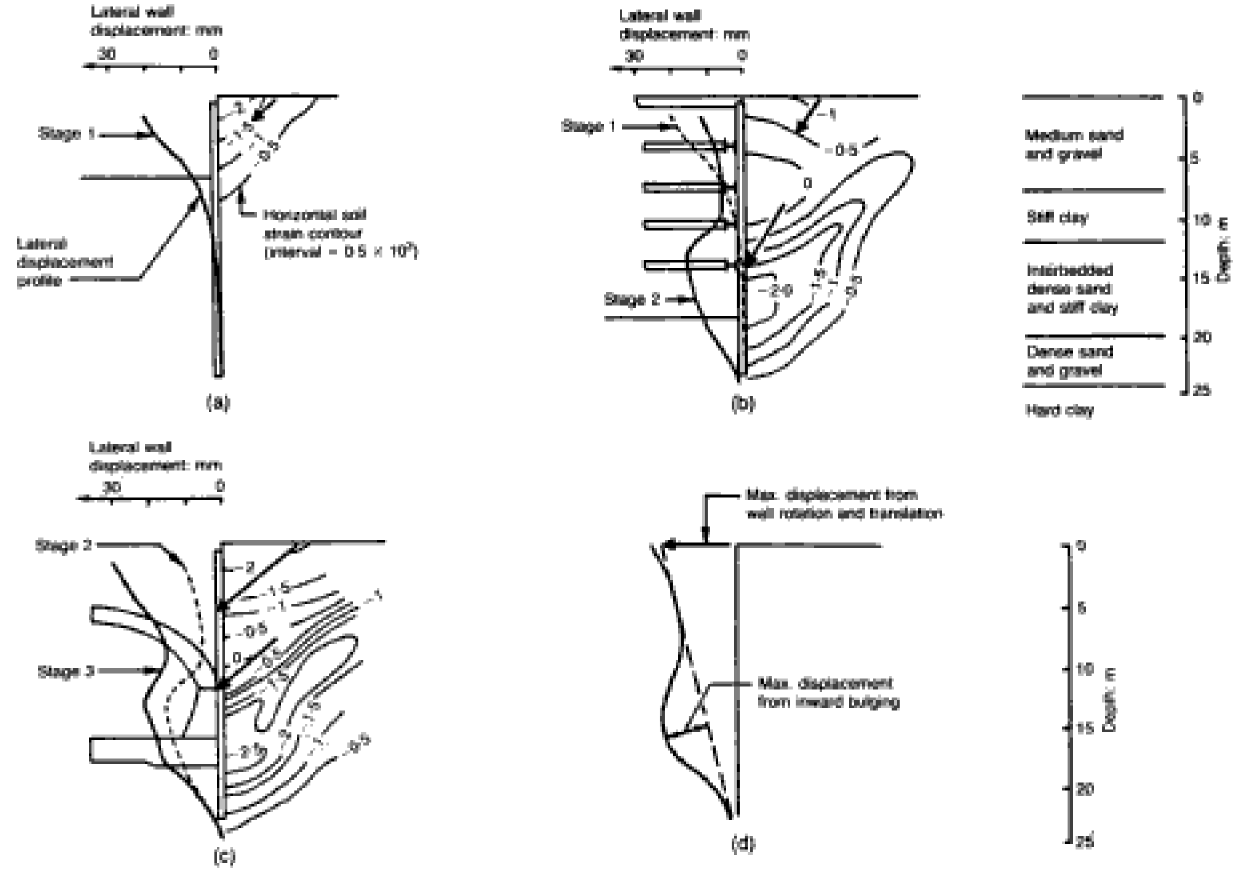


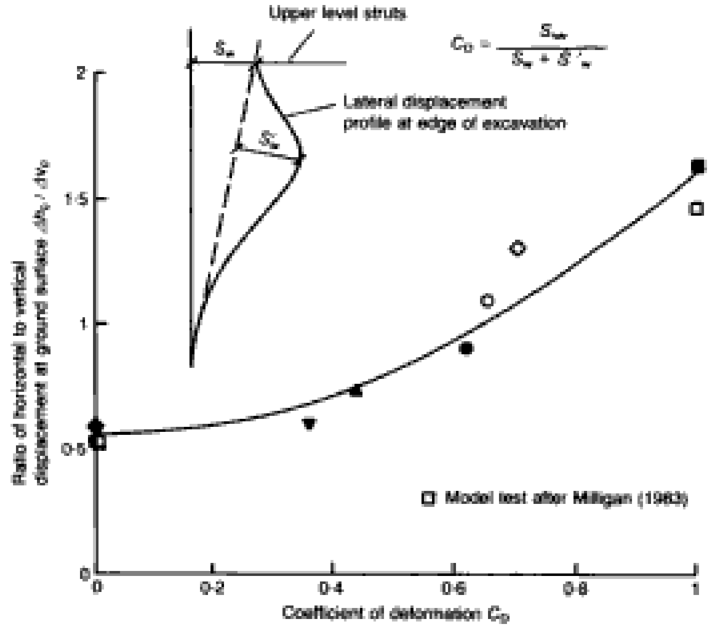
Fig. 11.12. Horizontal strains as measured at successive stages of strutted excavations: (a) stage 1, initial excavation; (b) stage 2, excavation to subgrade; (c) stage 3, removal of struts; (d) principal components of wall movement (O'Rourke[10]). (Bold arrows show movement vectors)

and the maximum value of settlement, at the rear of the sheeting, is of limited use on its own.

Clough *et al.*[19] and Mana and Clough[6] examined data from sheet pile walls and Berlin walls in clays supported by cross-lot struttings with either free end or fixed end support. The results (in Fig. 11.14) show the relationship between maximum lateral wall movement and the factor of safety against basal failure by heave. Lateral movements of sheeting are shown to increase very rapidly below a factor of safety of 2.

Mana and Clough[6] produced an empirical relationship between maximum ground settlement and maximum wall movement from data in varied overall ground conditions in clays in San Francisco, Oslo and Chicago (Fig. 11.15). Perhaps the limited conclusion to be drawn from this plot is that maximum vertical settlements appear most likely to be equivalent to maximum horizontal displacements in clays.

Clough[18] had earlier summarized empirical data on anchored sheeting and walls (Fig. 11.16) for subsoils varying from sands and silts to stiff clays and shales to soft clays. Most values of maximum movement remain below 1% of excavated depth and no significant variation is shown with soil type. Clough suggested that the maximum reduction in soil movement using prestressed anchors was achieved with prestress forces obtained from ground pressures slightly greater than those advised by Terzaghi and Peck.[20]



Symbol	Max. depth: m	Support	Soil	Location
●	18	Soldier piles and lagging, five strut levels	Sand and stiff clay	Washington, DC
▲	13	Soldier piles and lagging, three levels of struts and rakers	Soft to medium clay	Chicago
▼	8	Sheet pile, three raker levels	Soft to medium clay	Chicago
■	13	Slurry wall, two levels of tiebacks and rakers	Soft to medium clay	Chicago
◇	9	Soldier piles and lagging, two raker levels	Soft to medium clay	Chicago
○	8	Sheet pile, two raker levels	Soft to medium clay	Chicago
◆	14	Sheet pile, two levels of struts and rakers	Soft to medium clay	San Francisco

Fig. 11.13. Ratio of horizontal to vertical soil movement as function of the coefficient of deformation (O'Rourke[10])

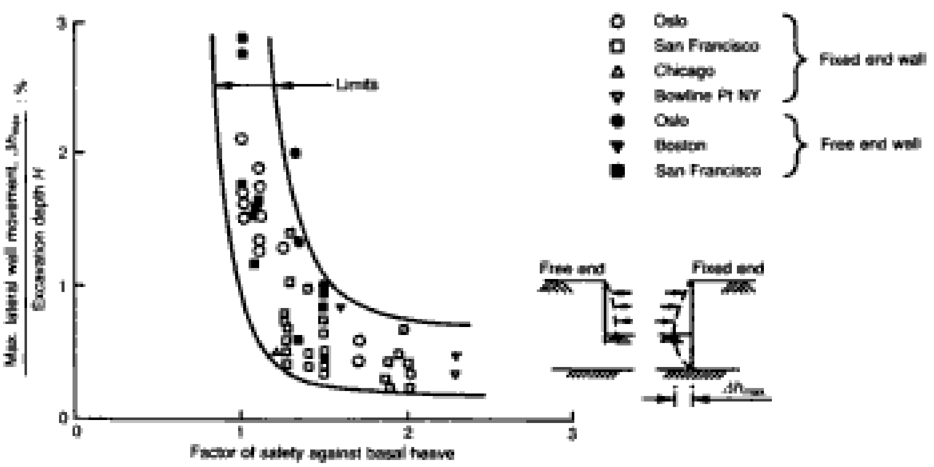


Fig. 11.14. Empirical relationship between the factor of safety against basal heave and non-dimensional maximum lateral wall movement (Clough et al.[19])

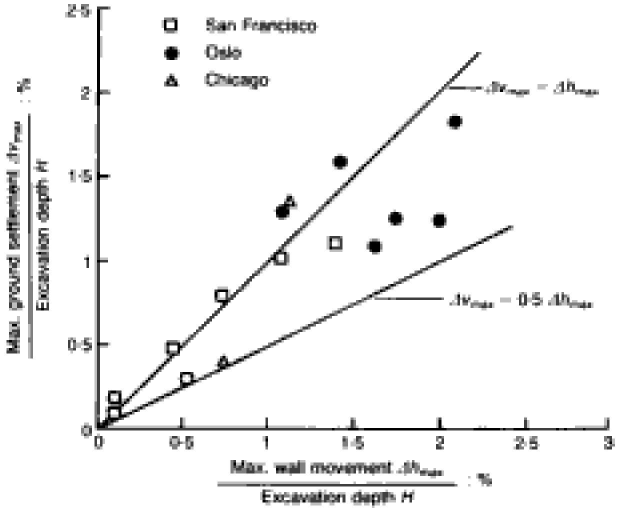


Fig. 11.15. Empirical relationship between maximum ground settlement and maximum lateral wall movements (Mana and Clough[6])

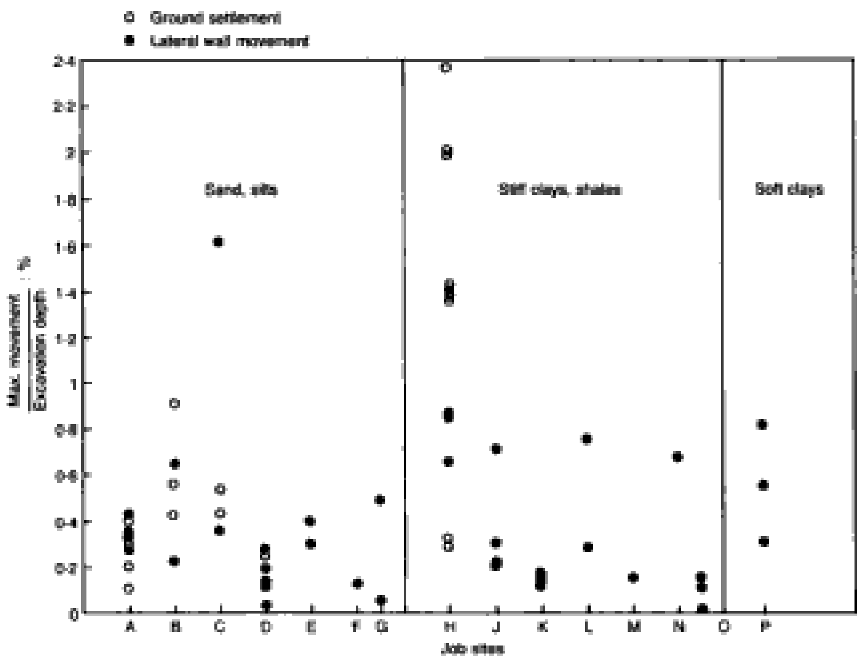


Fig. 11.16. Observed movements of anchored wall systems for varying soil types (Clough[18])

Semi-empirical methods

Several methods have been devised which enable the settlement profile at the rear of the wall supporting a deep excavation to be calculated from empirical relationships determined for the lateral movement of the wall. Caspe[21] published a method of analysis which related the settlement profile to the deflected shape of the wall. In this method:

(*a*) there is a surface behind the wall which defines the limit of soil deformation due to the excavation

(*b*) a variation in horizontal strain in the soil between this no-strain surface and the wall is assumed

(*c*) at all locations, vertical strain is assumed to be related to horizontal strain by Poisson's ratio ν.

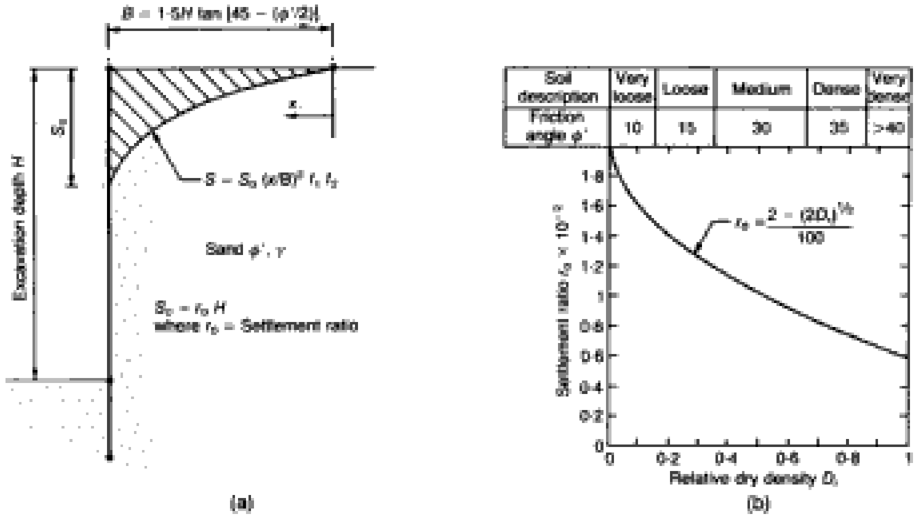


Fig. 11.17. Semi-empirical method to estimate settlement in sands: (a) ground settlement adjacent to wall; (b) variation of settlement ratio with soil properties (Bauer[23])

Factor	Workmanship				Factor	Construction difficulty		
	Excellent	Good	Average	Poor		None	Average	Severe
f_1	0·8	0·9	1	1·1	f_2	1	1·02	1·05

Others have commented that for plane strain conditions this last assumption is incorrect and the relationship between vertical and horizontal strain should be expressed by the ratio $\nu/(1-\nu)$. Caspe's method was altered by Bowles[22] to take this into account, with reasonable agreement between the calculated settlement profile and site measurement.

A further semi-empirical method devised by Bauer[23] is shown in Fig. 11.17. This method, applicable to excavation in sands, was claimed to show reasonable fit of settlement profiles with site movements, although the calculated width of settlement influence appears to limit the lateral extent of this zone to less than the excavated depth for practical values of ϕ.

Finite element and finite difference methods

Numerical methods using finite elements or finite differences allow a soil–structure analysis. As mathematical tools these methods provide convenient two-dimensional plane strain solutions (three-dimensional soil-structures solutions will increasingly become available as computing resources improve) and use commercially-available programs (such as CRISP) or in-house programs developed by academic or professional organizations (such as ICFEP from Imperial College, London). The methods attempt to address all theoretical requirements with boundary conditions that realistically model the site problem and incorporate, for instance, a stage-by-stage simulation as the excavation progresses, including time-related aspects such as dissipation of excess pore pressure. Displacements are the primary unknown solved by the methods, so prediction of horizontal displacement and vertical settlements fall conveniently to this solution.

Finite element packages generally offer the user a choice of constitutive models which range from simple elastic models to sophisticated non-linear elastic–plastic models. The final choice of model will depend the accuracy required of the prediction and the availability of appropriate input data, particularly with regard to soil parameters. Some of the issues facing the designer in the choice of constitutive model were raised by Woods and Clayton[24] and included two items related to soil stiffness: linearity and small-strain behaviour. Although the solution, using the simple linear model, has been available for many years, it has always been appreciated that most natural soils are of non-linear nature, even at the very low strain values which occur in wall deformation and settlement profile prediction. In addition, the use of finite element programs to predict movement around excavations has been shown generally to exaggerate deformation unless soil stiffness at very small strain volumes is used in the analysis. To obtain these values, specific measurement procedures have been designed for use in the triaxial test. Even so, choice of a suitable average operational strain level is necessary, particularly for soils from previously undeveloped areas. Where the excavation is near previous sites where measures have been made, back analysis will provide appropriate soil stiffness parameters, providing the excavation and subosil conditions are similar. Good agreement between the use of small strain non-linearity to predict settlement behind a strutted excavation and field behaviour was described by Jardin *et al.*[25]

In addition to the use of appropriate soil stiffness parameters, the quality of prediction will depend on the selection of accurate k_0 values for the particular site. The method of back analysis on its own may not prove sufficiently dependable to obtain these values because of the relatively large variations in at-rest pressures within relatively small distances.

The finite element method was developed by Mana and Clough[6] to formulate a design method for estimating wall deformation and the settlement trough for a strutted excavation in soft to medium clays without resorting to use of a finite element program for a particular design problem. Their procedure was as follows:

(*a*) At each construction stage where prediction of movement is needed, calculate the minimum factor of safety against basal heave using Terzaghi's method.

(*b*) Estimate the maximum wall movement Δh_{max} from the relationship between factor of safety against basal heave and maximum wall movement, shown in Fig. 11.18. Approximate maximum ground settlement Δv_{max} can be estimated by assuming that Δv_{max} lies within the range 0·6 to 1·0 Δh_{max}.

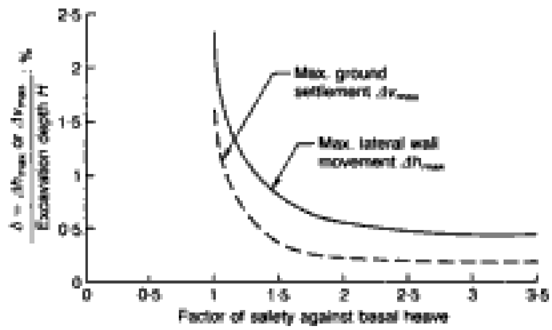


Fig. 11.18. Analytical relationship between maximum lateral wall movement and factor of safety against basal heave (Mana and Clough[6])

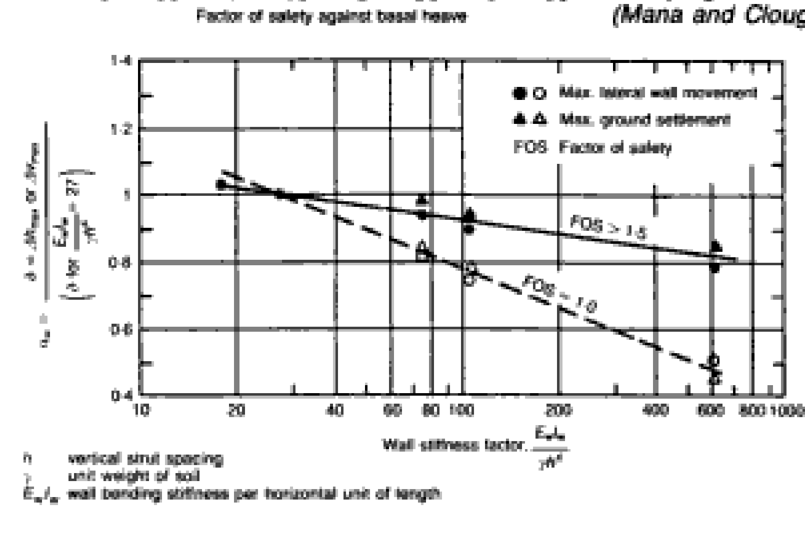


Fig. 11.19. Effect of wall stiffness on maximum lateral wall movement and maximum ground settlement (Mana and Clough[6])

S_s strut stiffness per horizontal unit of length
h vertical strut spacing
γ unit weight of soil

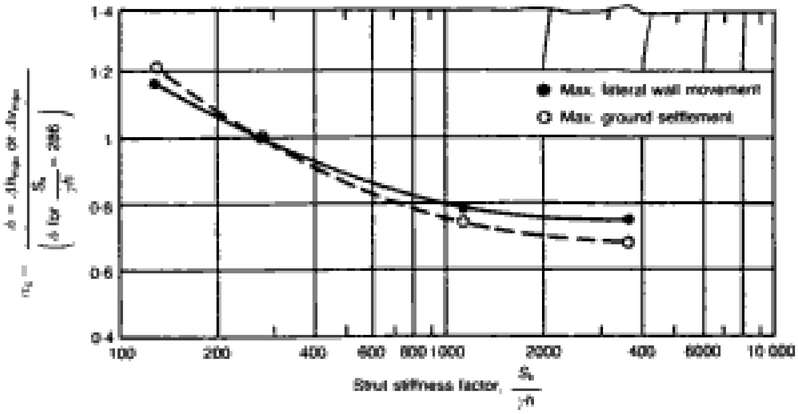


Fig. 11.20. Effect of strut stiffness on maximum lateral wall movement and maximum ground settlement (Mana and Clough[6])

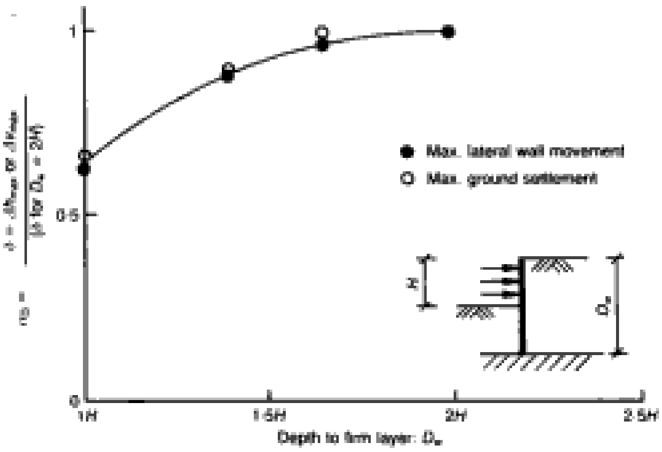


Fig. 11.21. Effect of depth to firm layer on maximum lateral wall movement and maximum ground settlement (Mana and Clough[6])

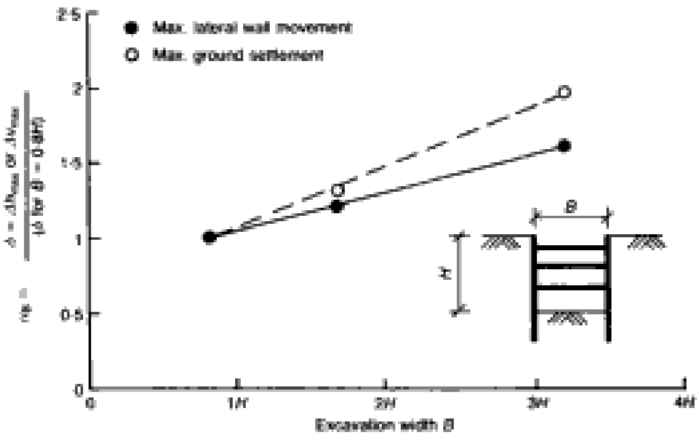


Fig. 11.22. Effect of excavation width on maximum lateral wall movement and maximum ground settlement (Mana and Clough[6])

(*c*) Based on the wall stiffness factor, the strut stiffness factor, the depth to a firm soil layer and the excavation width B, determine the influence coefficients α_w, α_s, α_D, α_B, using Figs 11.19–11.22.
(*d*) Determine the influence coefficient for the design strut pre-loading α_P using Fig. 11.23.
(*e*) Determine the modulus multiplier influence coefficient α_m from Fig. 11.24.
(*f*) Using the value of Δh_{max} from step (*b*) and the influence coefficients determined in stages (*c*)–(*e*), calculate a revised value for the maximum lateral movement from

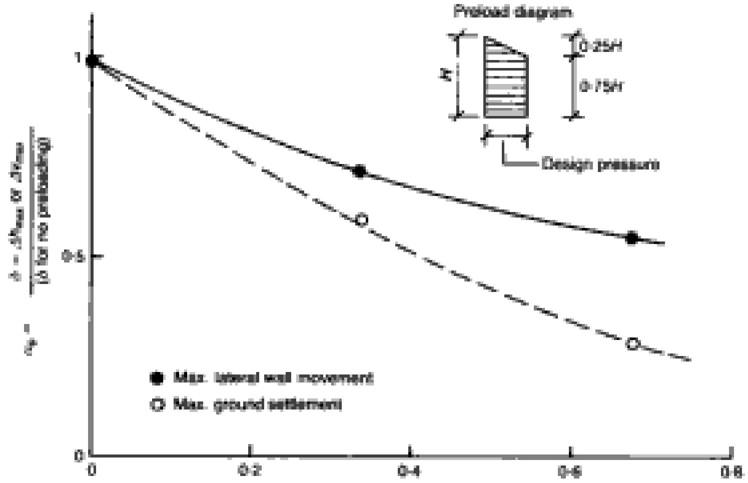


Fig. 11.23. Effect of strut preload on maximum lateral wall movement and maximum ground settlement (Mana and Clough[6])

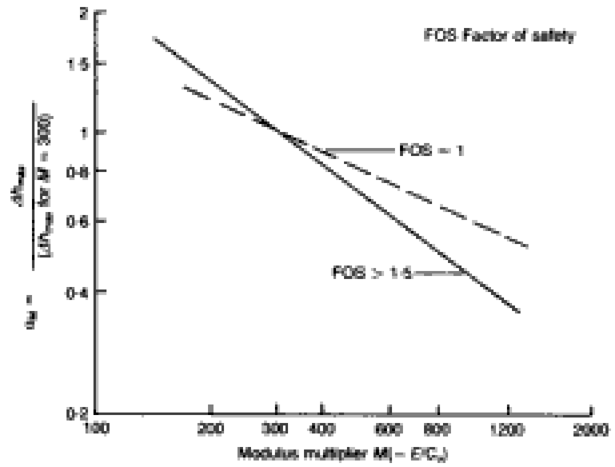


Fig. 11.24. Effect of modulus multiplier on maximum lateral wall movement and maximum ground settlement (Mana and Clough[6])

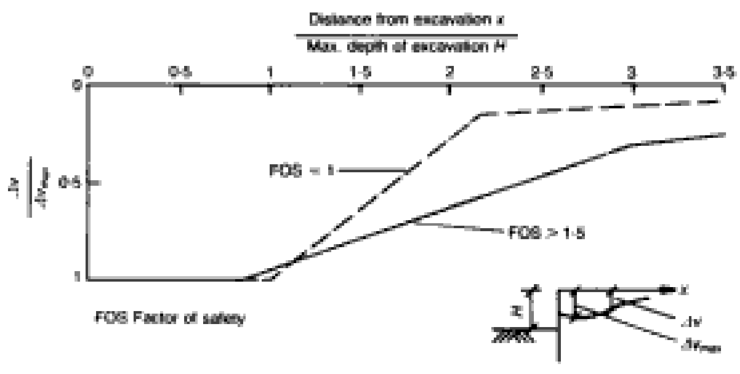


Fig. 11.25. Envelopes to normalize ground settlement profiles (Mana and Clough[6])

$$\Delta h^{*}_{\max} = \Delta h_{\max}\, \alpha_{w}\, \alpha_{s}\, \alpha_{D}\, \alpha_{B}\, \alpha_{p}\, \alpha_{m} \tag{84}$$

(*g*) Revise the estimate of $\Delta h_{\max}$ using the relationship $\Delta v^{*}_{\max}$ = 0·6 to 1·0 times $\Delta h^{*}_{\max}$.

(*h*) Plot the ground settlement profile using the calculated value $\Delta v^{*}_{\max}$ and the profile shown in Fig. 11.25.

This method can be used for walls supported by anchors provided the anchors themselves are embedded in a mass of soil or rock which is materially beyond the movement zone.

Other predictive methods

A method originally developed by Roscoe[26] and developed by James *et al.*[27] and Serrano[28] uses stress and strain fields for increments of structural deflexion and load. In a series of iterative steps, stress and strain fields are produced which comply with all the parametric values for a particular problem. More recently Maruoka *et al.*[29] presented a predictive method for vertical settlement in cohesive soils based on the deformed shape of the wall and patterns of zero extension lines in the adjacent soil. They concluded that strain fields consisting of straight lines and circular areas could be used with a rigid body spring model and finite element analysis to give reasonably accurate settlement profile predictions. This method, however, requires the initial prediction of the deformed shape of the wall due to the excavation, and while a solution based on a Winkler spring model could be used to do this, any inaccuracy in this prediction would presumably be reflected in the accuracy of the final settlement profile. A simpler solution based on the kinematics of a mechanism involving soil and wall mass is referred to in reference 30.

Soil movement during diaphragm wall installation

The soil deformations considered in this chapter so far have been due to unloading of the soil surrounding the deep substructure during bulk excavation. The soil structure, however, undergoes several stress changes during installation of the peripheral walling prior to bulk excavation. These changes may be due to dynamic stresses set up by the driving of sheet piles, or the relief of stress due to augering of piles or the excavation of diaphragm wall panels. Each change in in-situ soil stress has an associated volume change and a resulting vertical settlement. The stress changes and settlements which result from the installation of diaphragm wall panels at the periphery of a deep excavation deserve special comment. Until recently, soil movements associated with diaphragm wall installation had been assumed to be very small and limited in lateral extent, but experience with the initial sections of the Island Metro Line in Hong Kong indicates otherwise, and the causes for this difference should be noted.

Stress changes near a diaphragm wall panel excavation occur as a result of unloading due to panel excavation, recharging with bentonite slurry and the subsequent fluid pressures from liquid concrete. (The in situ stresses from the concreting operation were measured in tests by Reynaud[31] which emphasized the relatively high pressures caused by concreting relative to soil values of K_0.) The soil stress changes due to diaphragm wall panel installation are therefore relatively complex and do not stem only from the excavation operation.

The extent of soil movement due to excavation of diaphragm wall panels depends on soil properties, groundwater levels, panel width and the length of time between excavation and concreting. There are limited records of in situ measurements of soil movement due to panel installation: those due to Uriel and Oteo[32] in Seville, Spain; Farmer and Attewell[33] in London; and Humpheson *et al.*,[34] Davis and Henkel,[35] Morton *et al.*[36] and Stroud and Sweeney[37] in Hong Kong, should be referred to.

Measurements on the sites in London and Seville within stiff over-consolidated clays confirmed the general opinion of diaphragm wall specialist firms that soil movements are generally small and reduce rapidly at short distances from the panel, say equal to the panel length. Measurements made by the Author at panel excavations in lightly overconsolidated silty clay and silty sand washdown soils in Medinah, Saudi Arabia, also confirm this view. The movements caused by panel excavation in Hong Kong, as described in chapter 9, were substantial. Measurements made at these sites are compared in Table 11.1. The soil conditions in Hong Kong are materially different from those in London, Seville and Medinah. In the Chater station excavation in Hong Kong, fill and marine deposits to a total depth of 15 m

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Index

DEEP EXCAVATIONS

A practical manual

MALCOLM PULLER

This book assembles the practical rules and details for the economical and efficient execution of deep excavations. Although formal codes of practice may be produced with the same intention, they may also constrain innovation and ingenuity if they become accepted as the *only* basis of acceptable foundation engineering design. The collected data and experience in this book present examples of design and solutions to construction problems which dispel the conception of design by regulation.

Thomas Telford

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