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Embedded retaining walls: guidance for economic design

Prepared under contract to CIRIA by Ove Arup and Partners International Limited in association with the University of Southampton and Bachy Soletanche Limited

A R Gaba	Ove Arup and Partners International Limited
B Simpson	Ove Arup and Partners International Limited
W Powrie	University of Southampton
D R Beadman	Ove Arup and Partners International Limited

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Summary

This report provides best practice guidance on the selection and design of vertical embedded retaining walls. It covers temporary and permanent cantilever, anchored, single and multi-propped retaining walls which are supported by embedment in stiff clay and other competent soils.

The report addresses the technical and construction issues relating to the selection of appropriate wall type and construction sequence to achieve overall economy. It clarifies areas of design ambiguity and presents a clear unambiguous method for the design of such walls.

Embedded retaining walls: guidance for economic design

Gaba, A R; Simpson, B; Powrie, W; Beadman, D R

Construction Industry Research and Information Association

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Ground engineering, ground investigation and characterisation, piling, soil-structure interaction

Reader interest	Classification	
Design, specification, construction, managers, clients and supervising engineers involved in civil and geotechnical works	AVAILABILITY	Restricted
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Note

Recent Government reorganisation has meant that DETR responsibilities have been moved variously to the Department of Trade and Industry (DTI), the Department for the Environment, Food and Rural Affairs (DEFRA), and the Department for Transport, Local Government and the Regions (DTLR). References made to the DETR in this publication should be read in this context.

For clarification readers should contact the Department of Trade and Industry.

Foreword

This report is the main output from CIRIA research project 629. It was prepared by Ove Arup and Partners International Limited in conjunction with the University of Southampton and Bachy Soletanche Limited.

The report was written by Mr A R Gaba and Dr B Simpson of Ove Arup and Partners International Limited, Professor W Powrie of the University of Southampton and Mr D R Beadman of Ove Arup and Partners International Limited (formerly of Bachy Soletanche Limited).

Following CIRIA's usual practice, the research project was guided by a steering group which comprised:

Mr J Findlay (Chairman)	Stent Foundations
Dr D Bush	Highways Agency
Dr D Carder	Transport Research Laboratory
Mr S Everton	Gibb
Mr R Fernie	Skanska
Prof R Mair	Cambridge University
Mr B McGinnity	London Underground Limited
Mr D McNicholl	Wardell Armstrong
Mr A O'Brien	Mott MacDonald Limited
Prof D Potts	Imperial College
Mr D Rowbottom	Corus
Dr H St John	Geotechnical Consulting Group
Dr J Wilson	W S Atkins
Mr R Yenn	TPS Consult
Dr N O'Riordan (corresponding member)	Rail Link Engineering

CIRIA's research manager for this project was Dr A J Pitchford.

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Mr A Pickles	Ove Arup and Partners International Limited
Mr J Smethurst	Southampton University
Professor J H Atkinson	City University
Dr D Borin	Geosolve Limited
Mr D Twine	Rail Link Engineering
Mr M J Puller	Malcolm Puller Associates
Mrs J Senior	Ove Arup and Partners International Limited
Mr J Taylor	Ove Arup and Partners International Limited

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Glossary

analysis	The process of breaking down a design into its constituent parts and of calculating the behaviour of each of those parts.
client	An organisation or individual using the services of construction professionals in order to invest in new building or construction work.
conceptual design	The identification of an appropriate design solution by qualitatively assessing the strengths and weaknesses of a range of possible design variants, without recourse to detailed analyses.
desk study	An examination of all existing information concerning a site (eg geological maps, previous borehole records, air photographs) to determine ground conditions and previous land use.
engineering judgement	A feel for the appropriateness of a situation, from the narrowest technical details to the broadest concepts of planning.
geotechnical adviser	A chartered engineer or a chartered geologist with five years of practice as a geotechnical specialist (Site Investigation Steering Group, 1993).
geotechnical engineer	A chartered engineer with at least one year of post graduate experience in geotechnics and a postgraduate qualification in geotechnical engineering or engineering geology, equivalent at least to an MSc or a chartered engineer with at least three years of postgraduate experience in geotechnics (Site Investigation Steering Group, 1993).
geotechnical risk	The risk posed to construction by the ground or groundwater conditions at a site.
ground investigation	The sub-surface field investigation, with the associated sampling, testing and factual reporting. See **site investigation**.
ground model	A conceptual model based on the geology and morphology of the site, and used to speculate on likely ground and groundwater conditions and their variability.
hazard	An event, process or mechanism that could affect the performance of an embedded retaining wall and prevent performance objectives from being met.

likelihood	The probability that an event will occur.
mitigation	The limitation of the undesirable effects of a particular event.
moderately conservative	A cautious estimate of soil parameters, loads and geometry. Worse than the probabilisitic mean but not as severe as a worst credible parameter value. Sometimes termed a conservative best estimate.
project manager	The individual or organisation responsible for managing a project.
risk	The combination of the probability and consequences of a hazard occurring.
risk assessment	A structured process of identifying hazards, their probability and consequence of occurring, and their likely impact on the performance of the retaining wall.
risk mitigation	Measures taken to either remove a hazard or to minimise the likelihood or consequences of it occurring to an acceptable level, including monitoring, and remedial action.
risk register	A list of risks arising from relevant hazards and the benefits of mitigating them.
rupture surface	The detachment surface on which differential movement occurs.
serviceability limit state	State of deformation of a retaining wall such that its use is affected, its durability is impaired or its maintenance requirements are substantially increased. Alternatively, such movement that may affect any supported or adjacent infrastructure, eg track, road or canal. See **ultimate limit state**.
site investigation	The assessment of the site, including desk study, planning and directing the ground investigation, and interpretation of the factual report.
ultimate limit state	State of collapse, instability or forms of failure that may endanger property or people or cause major economic loss. See **serviceability limit state**.
worst credible	The worst value of soil parameters, loads and geometry that the designer realistically believes might occur.

For further definitions and information, the reader is referred to technical dictionaries including; *Penguin dictionary of Civil Engineering* (Scott, 1991) and *Dictionary of Geotechnical Engineering* (Somerville and Paul, 1983).

Abbreviations

CDM	Construction (Design and Management) Regulations 1994
DETR	Department of Environment, Transport and Regions
DMRB	Design Manual for Roads and Bridges
FPS	Federation of the Piling Specialists
HA	Highways Agency
ICE	Institution of Civil Engineers
LUL	London Underground Limited
DPL	distributed prop load
SISG	Site investigation steering group
SLS	serviceability limit state
SPT	Standard Penetration Test
ULS	ultimate limit state
U100	102 mm diameter driven tube sample

1 Introduction

An earth retaining wall is required to withstand forces exerted by a vertical or near vertical ground surface. An *embedded* retaining wall is one which penetrates the ground at its base and obtains some lateral support from it. The wall may also be supported by structural members such as props, berms, ground anchors, slabs, etc, as applicable. It may be freestanding or it may provide support to a superstructure.

This report provides guidance on the selection and design of vertical embedded retaining walls. The report covers all types of embedded walls, see Figure 1.1.

Figure 1.1 *Wall types*

Economies in embedded retaining walls can be made by the selection of the appropriate wall type and support system for the envisaged construction sequence and long term use and the adoption of a clear unambiguous design method in conjunction with the application of best practice in appropriate ground investigation, laboratory and field testing, design analysis and the use of good quality case history data. This report provides guidance to achieve this.

1.1 BACKGROUND TO PROJECT

CIRIA Report 104 (Padfield and Mair, 1984) *Design of retaining walls embedded in stiff clays* has been hugely influential. Although strictly applicable to the design of singly propped or cantilever walls embedded in stiff overconsolidated clay, the principles presented in the report have been applied to a wide range of wall types and soils in the UK and overseas, including multi-propped embedded walls and even non-embedded walls. It has also formed the model for other documents providing design guidance such as the recently published BD 42/00 *Design of embedded retaining walls and bridge abutments* (Design Manual for Roads and Bridges, DMRB 2.1.2).

Since publication of CIRIA Report 104, several retaining wall design guidance documents and codes of practice relevant to embedded walls have been issued, most notably:

- BS 8002 (1994) *Code of practice for earth retaining structures*. The latest amendment to this code was issued in September 2001
- DD ENV 1997-1 Eurocode 7 (1995) *Geotechnical design Part 1 General rules*. This document is currently under revision
- BD 42/00 (Design Manual for Roads and Bridges, DMRB 2.1.2) *Design of embedded retaining walls and bridge abutments*
- British Steel (1997) *Piling Handbook* 7th edition
- CIRIA Special Publication 95 (1993) *The design and construction of sheet piled cofferdams*
- CIRIA report C517 (1999) *Temporary propping of deep excavations - guidance on design*
- CIRIA report 185 (1999) *The Observational Method in ground engineering - principles and applications*
- Geoguide 1 (Hong Kong Government, 1994) *Guide to retaining wall design.*

In addition to the above, many technical papers and text books have been published.

The above documents do not provide consistent and harmonious design advice and many omit detailed treatment of modern numerical analysis methods. The multiplicity of documents, and the guidance provided therein, results in confusion (which is costly to designers) and poor economy in construction (which is costly to constructors and clients). There is therefore a need for a coherent and authoritative document which collates the best ideas and experiences available in British practice and provides a clear path through the many alternative design approaches. This report aims to do this.

This research was commissioned by CIRIA in November 2000. The project was undertaken by Ove Arup and Partners International Limited with assistance from the University of Southampton and Bachy Soletanche Limited. The steering group which guided the work represented clients, consultants, contractors and academia. The authors consulted widely via a questionnaire, consultation workshop and literature searches.

1.2 OBJECTIVES OF REPORT

This report provides guidance on the design of embedded retaining walls. It aims to achieve economy in the resulting retaining structure and its support system while maintaining various levels of simplicity, as far as possible, but also facilitating more complex approaches where they give an advantage.

This report:

- provides best practice guidance for the design of embedded retaining walls consistent with recent research and current analytical techniques
- describes and compares existing design methods for such walls
- discusses available wall types and construction methods
- provides construction costs data to guide the reader in selecting wall types appropriate to particular requirements.

This report supersedes CIRIA report 104.

1.3 REPORT READERSHIP

This report is intended for use by those concerned with the selection, design and construction of embedded retaining walls. In addition to providing guidance to designers, it is intended that the report will also:

- provide background information on wall selection, construction methods and associated ground movements for clients and owners and their technical advisers
- present geotechnical principles and guidance to structural and geotechnical engineers and to students who wish to gain an appreciation of the issues relevant to the selection, design and construction of an embedded retaining wall
- act as a reference for more experienced geotechnical engineers.

1.4 APPLICABILITY OF REPORT

See also

2.2.2….Geotechnical categorisation of retaining walls

This report covers the design of temporary and permanent cantilever, anchored, single and multi-propped retaining walls which are supported by embedment in stiff clay and other competent soils. Its principles are applicable to a wide range of fine grained and coarse grained soils in the UK and overseas. The design of walls embedded in soft clay and those socketed into rock is beyond the scope of this report.

Typical British soils to which this report is applicable include London Clay, Oxford Clay, Gault Clay, Lias Clay, Atherfield Clay, Weald Clay, Barton Clay, Kimmeridge Clay, Lambeth Beds, Mercia Mudstone, and glacial tills. These soils have experienced high overburden pressures in their geological history which have caused them to consolidate to a dense state. Subsequent erosion of the upper soil horizons, or the removal of Quaternary ice cover, has resulted in significant unloading and swelling. These soils typically exhibit in situ moisture contents which are lower than they would have been if no overconsolidation had occurred. This geological history results in soils which:

- may be fissured
- have an in situ earth pressure coefficient, K_o, which is greater than unity
- have an undrained shear strength which is significantly greater than that of a normally consolidated soil at similar depth
- exhibit a peak shear strength at low strains and reduced shear strength at high strains.

Few walls are constructed entirely in stiff overconsolidated fine grained soils. Although the wall may be embedded in such soils, it is likely that it will also retain other soils, eg made ground, river gravels and other alluvial deposits. The principles presented in this report also apply to this common situation.

The report adopts the geotechnical categorisation proposed by EC7 (1995). It applies to the design of embedded retaining walls for geotechnical categories 1 (small and relatively simple structures) and 2 (conventional types of structures with no abnormal risks or unusual or exceptionally difficult ground or loading conditions). It does not specifically address the design requirements of walls for geotechnical category 3 (very large or unusual structures, structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas), although the general principles presented in this report will also apply to these structures. Geotechnical categories are defined and discussed in section 2.2.2.

Ground engineering requires a thorough knowledge and understanding of basic principles and the application of sound engineering judgement based on experience. This report is not a substitute for professional knowledge; if in doubt, seek appropriate advice.

1.5 ECONOMIC DESIGN

See also

Appendix J…Effect of method of analysis on economy

Economy can be achieved by:

- ensuring ease of construction and minimising construction duration
- optimising the use of materials
- applying appropriate design effort.

2.1.3.....Risk assessment and management

2.2.2....Geotechnical categorisation of retaining walls

3.........Construction considerations and wall selection

4.3.......Effect of method of analysis on economy

5.9.1....Temporary works design

6.........Design of wall

6.6.3....Steel sheet pile walls

The biggest economies will be available at the outset of a project during the selection of an appropriate method and sequence of construction, wall type and the optimisation of the temporary and permanent use of the retaining structure.

Achieving economy requires commitment and adherence to an approach which necessitates a holistic view to be taken of project requirements. **Whole life costs should be considered.** A robust design which minimises long term maintenance requirements may be appropriate in some circumstances. A design which minimises wall dimensions and material use but one which increases construction duration because it is difficult to build may not result in overall economy. It should be recognised that the designer cannot achieve the most cost effective solution in isolation of the client and the constructor.

The client, designer and constructor and, where appropriate, the architect and the quantity surveyor, should be involved as early as possible to:

- optimise the temporary and permanent use of the retaining structure (eg the adoption of one wall instead of two to serve both the temporary and permanent requirements) which is also compatible with long term maintenance requirements
- establish *appropriate* design and performance criteria for the retaining structure eg acceptable limits for wall deflection and associated ground movements, crack width criteria
- consider appropriate wall type
- consider appropriate method and sequence of construction to ensure buildability with minimum construction duration.

Initial ideas should be reviewed and alternatives explored prior to agreeing the preferred solution. It is important to involve individuals with appropriate qualifications and experience at all stages of the project and to maintain adequate continuity and communication between the personnel involved in data collection, design and construction. The involvement of the constructor at an early stage should minimise wasteful abortive work arising from design changes.

Where the design of a wall is governed by temporary works considerations, a risk based approach to design may result in significant savings. Use of the Observational Method may result in the most cost effective design solution for temporary and permanent works (section 5.9.1). However, in this circumstance, appropriate contractual arrangements should be in place to permit the necessary integrated interactive approach to be taken on site between design and construction.

A clear unambiguous design method which eliminates confusion will lead to savings in unnecessary design effort. The choice of analysis method can result in significant savings in the wall structure. For example, propped or anchored walls designed using soil-structure interaction methods will be shorter, and computed wall bending moments will be smaller, than those designed using limit equilibrium methods (section 4.3). Also, savings in material use of about 25% to 30% are possible if plastic design is applied to sheet pile walls (section 6.6.3). However, in the UK, it may not be possible to realise such savings as it is often necessary to adopt sheet pile sections which are of greater thickness than those determined from the analysis of the section in service in order to withstand the driving forces. **Procedures for higher wall categories can be used to justify more economic design**. For example, the design of walls to geotechnical category 2 can be used to justify a more economic design for structures which would otherwise be classified as category 1. The higher investigation and design costs of a geotechnical category 2 wall should be balanced against the potential savings in materials and construction over a geotechnical category 1 design.

1.6 REPORT LAYOUT

Figure 1.2 shows the principal design stages and the corresponding sections of this report. For readers seeking guidance on specific issues, Figure 1.3 provides a map indicating relevant report sections where key issues are discussed.

Figure 1.2 *Principal design stages and report layout*

Figure 1.3 *Key issues considered in report*

Details are presented in boxes, separately from the main text. Cross-referencing between sections is adopted throughout the report. Appendices provide background information, a commentary on current practices and a list of case history data.

The report is organised into eight largely self-contained chapters. In view of the anticipated wide readership (section 1.3), guidance is provided at the outset of each chapter regarding the target readership which is likely to gain most from the content of that chapter. For convenience, each chapter is concluded with a section which highlights the key points of guidance and recommendations provided therein. For clarity, these are also highlighted in bold within the main text of the report.

Chapter 1 states the objectives of the report and its applicability.

Chapters 2 and 3 provide guidance to assist the reader in the selection of the appropriate wall type and construction sequence. Some of the issues covered in these chapters are interrelated.

Chapter 2 provides guidance on the determination of the key wall design criteria, health and safety issues, risk assessment and management, site specific constraints and project specific requirements. Guidance is also provided on the estimation and effects of ground movements associated with wall installation and performance.

Chapter 3 reviews wall types and available construction sequences. The advantages and limitations of different construction sequences and wall types are compared and guidance is provided on the selection of the most appropriate sequence and wall type to satisfy particular site and project requirements. Construction cost data relating to wall types and construction methods are also presented.

Chapter 4 presents key principles of soil behaviour relevant to embedded retaining walls and the determination of earth pressures. Soil-structure interaction and methods for its analytical modelling are also discussed.

Chapter 5 provides guidance on the determination and selection of parameters for use in design calculations. Advice on the assessment of drained or undrained ground behaviour is given together with guidance on the selection of parameters appropriate for temporary works and permanent works design.

Chapter 6 provides best practice guidance on the geotechnical and structural design of the wall.

Chapter 7 provides best practice guidance on the design of propping systems, berms and anchors for lateral support to the wall.

Chapter 8 identifies areas of further work and research.

2 Design Considerations

See also

1.5…Economic design

3…...Construction considerations

This chapter is intended for the general reader: designers, constructors, clients and owners and their technical representatives. It assumes that the reader has some experience and knowledge of engineering, of design and construction, and of risk assessment and management.

This is the first of two chapters which deal with general issues of design and construction providing guidance on the selection of appropriate wall type and construction sequence for a project involving an embedded retaining wall. This chapter deals with design issues while chapter 3 deals with construction considerations. This chapter should be read in conjunction with chapter 3; some of the issues covered herein are interrelated (Figure 2.1).

Figure 2.1 *Decision paths for selection of appropriate wall type and construction sequence*

The cost of constructing a retaining wall is usually high compared with the cost of forming a battered slope. Therefore, the need for a retaining wall should be assessed carefully during design and efforts made to keep the retained height as low as possible. **Construction methods should be fully considered at the design stage, since different construction methods may require different detailed design approaches. An excavation cannot be made without causing ground movements. The chosen wall type and construction sequence should ensure that these movements, and their effects, remain within pre-defined limits. Such limits should not be unduly severe (sections 2.4.2 and 2.5).**

In addition to the above technical considerations, it is important to ensure that the design and construction procedures are safely undertaken and result in overall economy. **It is inappropriate to adopt advanced technological solutions which minimise the dimensions and material costs of the retaining structure while prolonging the design and construction periods and long term maintenance requirements resulting in increased overall costs. Similarly, it is inappropriate to take undue risks during construction to minimise construction duration to reduce costs. Balance is required. Whole life costs should be considered. A robust design which minimises long term maintenance requirements may be appropriate in some circumstances (section 1.5).**

This chapter:

- provides an outline of the roles and responsibilities of the various parties involved in an embedded retaining wall project, particularly in respect of health and safety and design
- stresses the requirement to adopt a risk based approach to design and construction management and provides guidance on the assessment of such risk
- defines geotechnical characterisation of retaining walls and identifies the issues relevant to the establishment of design requirements and performance criteria for the wall
- describes limit state design principles

- provides guidance on methods of predicting ground movements arising from wall installation and subsequent excavation in front of the wall
- provides guidance on methods of controlling ground movements
- outlines the principles of building damage assessment.

2.1 HEALTH AND SAFETY

2.1.1 Statutory requirements

See also

2.1.4….Contaminated land

A detailed discussion of safety legislation is beyond the scope of this report. However, general guidance on aspects of key safety legislation is provided below. Legislation relating to contaminated land is discussed in section 2.1.4. Legislation places duties of care on both the employer and the employee. The employer is required to provide safe access, a safe place of work and a safe system of work for employees. Employees are required to take reasonable care for the safety of themselves and others and to co-operate with the employer. The principal statute governing safety is the Health and Safety at Work Act (1974). This is an umbrella act which is enforced by the Health and Safety Executive (HSE) by means of a series of complementary construction regulations. Regulations pertinent to embedded retaining walls are listed below:

- for any type of building work and most types of civil engineering and construction works:
  - *The Construction (General Provisions) Regulations 1961*
  - *The Construction (Lifting Operations) Regulations 1961*
- for work over water: various dock regulations which come under the Factories Acts, Merchant Shipping Act, Coast Protection Act, Port and Harbours Regulations, etc
- *The Management of Health and Safety at Work Regulations* (1992) require all employers with five or more employees to make a suitable and sufficient assessment of the risks to employees and others from operations or undertakings. The regulations do not stipulate a particular method for undertaking such risk assessments

See also

Box 2.1

2.1.3….Risk assessment and management

- *The Construction, Design and Management (CDM) Regulations* (1994) require designers to consider the effects of a design on the health and safety of those who will carry out the construction, maintenance or demolition of the works. This also includes others, such as the general public, who may use the facility or be within its vicinity. The regulations require designers to alter, if possible, their design to avoid health and safety hazards, or where this is not reasonably practicable, to undertake risk assessment and mitigation. A recent amendment to the regulations (Construction (Design and Management) (Amendment) Regulations 2000) came into effect on 02 October 2000. This clarifies that the legal duties on designers to build safety into a design, apply not only to a design prepared by them personally, but also to a design prepared by an employee or other person under their control. A planning supervisor is appointed by the client to be responsible for health and safety throughout the design stages. For further information, the reader is referred to CIRIA report 145 (1995) *CDM Regulations - case study guidance for designers: an interim report*, particularly to chapter 10 therein which provides detailed guidance on the health and safety considerations necessary for a typical retaining wall project.

2.1.2 Contractual requirements

In addition to statutory legislation, the contracts between the various parties involved in a project will define their respective responsibilities. Methods of procurement are changing. Traditional arrangements involving a single consultant who designed and supervised all aspects of the work are being used less frequently (Clayton, 2001). Figure 2.2 shows the range of conditions of contract currently in use in the UK.

Figure 2.2 *Conditions of contract in use in UK construction (after Clayton, 2001)*

Design and construction involves a number of roles:

- client
- project / construction manager
- designer
- constructor.

There is no clear and unambiguous relationship between the above roles. The processes of design and construction are fragmented as specialist consultants and contractors are utilised under various forms of contractual arrangement and construction management. For example, a contractor undertaking a DBFO (Design, Build, Finance, Operate) contract can reasonably describe himself as the client for the work. He may also carry out some or all of the design, possibly in association with specialist subcontractors and subconsultants. To a subcontractor, the main contractor is the client. A designer may be employed by the client, by a contractor, by an engineering consulting practice, etc. The project manager's function may be fulfilled directly by the client, the designer or the constructor.

To ensure certainty of outcome in an increasingly fragmented construction environment, the following are essential:

- **good communication between all parties**
- **a team approach to problem solving**
- **an integrated total project process**
- **a risk based approach to design and construction management (section 5.9.1).**

Design is a continuous process, requiring regular review to ensure that the client's needs are being met. It is recommended that a lead designer is identified to oversee this process. Clear allocation of design responsibility is essential. Definition of the various roles is provided in Box 2.1, within a health and safety framework.

Box 2.1 *Typical roles in embedded retaining wall design (modified from Chapman et al, 2000).*

Client

The person who is promoting the development. Under the CDM regulations 1994 the client must select and appoint a competent planning supervisor (where so required by the CDM regulations) and principal contractor. The client must also be satisfied as to the competence of the designer. The client must be satisfied that adequate resources have been allocated for health and safety, and ensure that construction work does not start until the principal contractor has prepared a satisfactory health and safety plan. After project completion, it is the client's responsibility to ensure that the health and safety file is available for inspection.

Specifier

The person who identifies the requirements for the retaining wall and who produces documents to arrange its procurement. If the retaining wall forms part of a larger development, the specifier could be the architect or engineer designing the rest of the development.

Designer

The person who confirms that the chosen wall system has an acceptable margin against failure and who produces documents to communicate the design intent to a supplier via drawings and a written specification.

In most cases, the designer would also be the specifier or supplier (or a separate design consultant employed by the specifier or the supplier).

The CDM regulations have a wide definition of 'designer' which encompasses any person who carries on a business where designs are prepared or who arranges for any person under that organisation's control (including, where he is an employer, any employee of his) to prepare a design relating to a structure or part of a structure.

The 'designer' under the CDM regulations has specific key duties which include, so far as reasonably practicable, alerting clients to their duties; considering the hazards and risks which could arise during the construction and maintenance of the structure; designing to avoid or reduce the construction health and safety risks; making sure that the design includes adequate information on health and safety; and to co-operate with the planning supervisor and any other designers. Information on safety at the design stage should be given on drawings and specifications, etc, and the designer is responsible for passing this information to the planning supervisor so it can be included in the project's health and safety plan.

Checker

A person or organisation employed to confirm the adequacy of the designer's design and that it is accurately communicated by the specification and drawings. The checker may be part of the designer's team or from an outside body.

CDM Planning Supervisor

The planning supervisor (where required by the CDM regulations) has to co-ordinate the health and safety aspects of the project design and the initial planning. The co-ordination is to ensure, as far as is reasonably practicable, that designers comply with their duties, that a pre-tender health and safety plan is prepared before appointment of a contractor and that the project is notified to the Health and Safety Executive. The planning supervisor also has to ensure that the health and safety file is prepared and delivered to the client at the end of the project.

Principal Contractor

The principal contractor is responsible for developing and implementing the health and safety plan, arranging for competent contractors, the co-ordination of contractors, obtaining risk assessments from contractors and control of the general site safety - the health and safety file is compiled with information supplied by the principal contractor.

Constructor

The contractor who installs the wall on the site so as to construct a retaining system. The constructor should notify the specifier if conditions mean that the requirements of the specification cannot be complied with or if a situation is encountered which is different from that assumed in the design.

2.1.3 Risk assessment and management

See also

Appendix A...Example CDM risk assessment forms

Risks associated with the ground fall into two broad categories: those relating to safety and those relating to geotechnical and financial issues.

Safety

Risk assessment and management are statutory safety obligations (section 2.1.1). This is to ensure that safety is not degraded and that risks are maintained as low as is reasonably practicable (Perry *et al*, 2001).

Geotechnical and financial

Clayton (2001) defines *geotechnical risk* as *the risk to building and construction work created by the site ground conditions*. This is relevant to embedded retaining walls. Ground related problems can adversely affect project cost, completion times, profitability, quality and fitness for purpose and can cause environmental damage.

The reader is referred to *Managing Geotechnical Risk* by Clayton (2001) for more details and to CIRIA report 125 (Godfrey, 1996), the RAMP report (Institution of Civil Engineers and Institute of Actuaries, 1998) and the PRAM report (Simon *et al*, 1997) for more information and advice on general construction risk management.

There are four stages of risk management (Nicholson *et al*, 1999):

1. *hazard identification*: where hazards are identified and documented in a register via experience of similar projects, brainstorming, etc. For a category 2 wall, hazards should be identified from the findings of a comprehensive desk study (section 5.2.1) and from brain storming by a small group (typically 3 or 4) of experienced practitioners.

2. *risk assessment*: where the likelihood and consequence of each hazard are evaluated and combined to estimate the risk corresponding to each hazard.

3. *risk reduction*: where the hazards are eliminated, if possible, and the risks are reduced by a combination of design changes, procedural changes, etc.

4. *risk control*: where the risks are monitored, controlled and managed throughout the project.

Figure 2.3 shows the typical process of incorporating CDM requirements into the design process. Hazard identification, risk assessment and management techniques range from the qualitative to the relatively complex quantitative. Any method can prove useful provided it is appropriate and its limitations are recognised.

Figure 2.3 *CDM risk assessment*

In design and construction, common usage of the term *risk assessment* refers to a written description of: hazard identification, risk assessment and intended controls. The term is used loosely. The designer should identify project specific risks which a competent contractor would not be expected to routinely anticipate and control. Risk assessment serves two main functions:

- *comparative risk assessment*: to assist and document decision making. This is used to indicate the relative potential effect of design options on health and safety. Forms of the type presented in Appendix A (Forms A1 and A2) provide qualitative and simple quantitative example formats for recording such assessments. Subsequent events may cause decision making to be questioned but the designer will have recorded the basis of his decision on information available to him at that time. It is good practice for the designer to complete either Form A1 or A2 (or similar)
- *task risk assessment*: to allocate resources, justify and record measures to be taken. This demonstrates an awareness of hazards and the intended application of controls to minimise risk as far as reasonably practicable. Form A3 in Appendix A presents an example form for this purpose. **The designer should complete Form A3 (or similar) to satisfy CDM regulations.**

2.1.4 Contaminated land

See also

2.1.1….Statutory requirements

2.2.2….Geotechnical characterisation

3.3.4….Contaminated ground

5.2.1….Investigation of ground and groundwater

Contamination of the ground and groundwater can have significant adverse effects on projects, particularly in terms of cost and performance. The design and construction of an embedded retaining wall in such difficult ground conditions would fall into geotechnical category 3 (section 2.2.2).

In addition to the statutory and contractual requirements discussed in sections 2.1.1 and 2.1.2, the following primary legislation is relevant to work in contaminated land:

- control of substances hazardous to health regulations (1988)
- the *control of pollution act* (1974) and the *control of pollution (amendment) act* (1989)
- the water act (1989)
- the environmental protection act (1990)
- the water resources act (1991)
- the environment act (1995).

Contamination may be present in many forms (chemical, radioactive, biological, etc) and affect different media (soil, water, utilities, structures such as retaining walls, etc).

The assessment of a contaminated site requires the identification of risks due to the presence of contaminants, so that appropriate action can be taken.

The risk assessment of a potentially contaminated site requires information to characterise the contamination status. This information should be gathered by a process of site investigation relevant to category 3 walls as described in section 5.2.1.

2.2 DESIGN CONCEPTS

2.21 Design pre-requisites

The following should apply throughout the design process as a matter of good practice (after EC7, 1995):

- data required for design should be collected, recorded and interpreted
- structures should be designed by appropriately qualified and experienced personnel
- adequate continuity and communication should exist between the personnel involved in data collection, design and construction
- adequate supervision and quality control should be provided
- execution should be carried out according to the relevant standards and specifications by personnel having the appropriate skill and experience
- construction materials and products should be used as specified
- the structure should be adequately maintained
- the structure should be used in accordance with the purpose assumed in the design.

2.2.2 Geotechnical characterisation of retaining walls

See also
5.2.1….Site investigation

In this report, design requirements are established for three geotechnical categories; 1, 2 and 3. **The designer should assign a preliminary classification to the retaining wall structure according to geotechnical category prior to the geotechnical investigations.** The categories indicate the degree of effort required in site investigation and design. The category should be reviewed and changed (if necessary) at each stage of the design and construction process. Figure 2.4 shows the decisions required in assigning a category.

Figure 2.4 *Geotechnical categorisation (adapted from Simpson and Driscoll, 1998)*

Geotechnical categories are defined in EC7 (1995) as:

Category 1

Category 1 walls are small and relatively simple structures with the following characteristics:

- the retained height does not exceed 2 m
- the ground conditions are known from *comparable experience* to be sufficiently straightforward such that routine methods of design and construction can be employed
- previous experience indicates that a site specific geotechnical investigation will not be required
- there is negligible risk to property or life.

Comparable experience is defined in EC7 (1995) as *documented or other clearly established information related to the ground being considered in design, involving the same type of soil and for which similar geotechnical behaviour is expected, and* *involving similar structures. Information gained locally is considered to be particularly relevant.*

Category 2

Category 2 walls comprise conventional structures with no abnormal risks or unusual or exceptionally difficult ground or loading conditions. These walls require site specific geotechnical data (eg a desk study and ground investigation) to be obtained and analyses to be carried out.

The vast majority of embedded retaining walls fall into geotechnical category 2.

Category 3

Category 3 walls are structures or parts of structures which do not fall within the limits of geotechnical categories 1 and 2. These include large or unusual structures, structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas.

The general advice and methods contained in this report are applicable to category 3 walls, but specialist advice should be sought to ensure that the particular circumstances are adequately dealt with.

2.2.3 Geotechnical design process

See also

4....Analysis

5....Determination and selection of parameters

6....Design of wall

7....Design of support systems

Figure 2.5 *Elements of geotechnical design*

Figure 2.5 shows the five major elements necessary for geotechnical design:

- understanding the geological and hydrogeological setting of the site and its environs and the historical development of the site
- determination of ground stratigraphy and groundwater conditions
- understanding soil behaviour
- undertaking calculations and analyses
- applying empiricism based on sound judgement and experience.

Each of the above involves a distinct and rigorous activity. It is important to distinguish clearly between each. All five must be kept in balance; all are important.

Ground stratigraphy and groundwater conditions

In some instances, site specific data are not available and there is insufficient information to define ground stratigraphy and groundwater conditions accurately. This is not acceptable for the design of walls other than those which fall into geotechnical category 1.

Walls in geotechnical category 2 require *quantitative* geotechnical data, but routine procedures for field and laboratory testing and for design and construction. For such walls, **no amount of soil testing or analysis will compensate for a lack of knowledge about the ground stratigraphy and groundwater conditions.** Appropriate sound geotechnical advice should be obtained by the designer at an early stage.

See also

5.3…Assessment of drained/ undrained soil conditions

Soil behaviour

A knowledge of the principles of soil mechanics, ground fabric, permeability, stress history, and in situ strength and stiffness is essential in understanding soil behaviour. Of particular importance is the assessment of whether drained or undrained soil conditions will apply over the lifetime of a temporary structure or during the construction stages of a permanent wall in the short term. This is discussed further in section 5.3.

See also

4.2…Methods of analysis

Applied mechanics

The calculations and analyses undertaken should be appropriate for the geotechnical category within which the wall lies. As a minimum, these calculations and analyses should demonstrate that equilibrium is possible without overstressing materials.

Analytical methods and the computer software and hardware necessary for them to be carried out have developed rapidly over recent years. It is now possible to obtain solutions to many complex problems. The great advantage of these methods is that they can enable the designer to gain a better understanding of the behaviour of a soil-structure system such as an embedded wall. However, they should be used with discernment and scepticism. The designer should clearly understand the idealisations and assumptions made in the numerical modelling. To carry out such analyses requires specialist knowledge and experience in the use of the particular software application. Also, numerical analyses require high quality appropriate input data. If such data are not available, the results of the analyses should be treated with caution.

Software and hardware improvements will continue. This will enable larger, more complex problems to be analysed in the future. It is clear, however, that even with unlimited analytical power, the inherent uncertainties in the soil, the structure and the construction procedure are so significant that accuracy in the prediction of expected behaviour will never be achieved.

Judgement and experience

There is no substitute for the application of empiricism based on sound judgement and experience at every stage of the design process. The rationale of this process, including details of the experience upon which it is based, should be communicated and explicitly recorded as part of the design. This is essential and should not be overlooked. It is not sufficient to proceed simply on the basis of "in my experience…" with no further explanation but rather to document the information which complies with the definition of comparable experience in section 2.2.2.

2.3 DESIGN REQUIREMENTS AND PERFORMANCE CRITERIA

See also

1.5…Economic design

The design of an embedded retaining wall requires a holistic approach. This is a soil-structure interaction system where the retaining wall derives both loading and support, at least in part, from the ground. The wall transfers load from the retained ground so that it is resisted elsewhere in the soil mass and by the wall and its support system. The manner in which this transfer occurs depends upon the type of wall, the in situ stress state, strength and stiffness of the ground, the wall and the wall's support system and also on the method and sequence by which the wall and its support system are constructed. This requires a full understanding of the role of the wall in the overall structure and how it interacts with its support system. Figure 1.3 shows some of these considerations for a typical (geotechnical category 2) basement retaining wall.

It is important to establish, at the outset, the design requirements and performance criteria which the wall should satisfy. Clearly the wall should be stable and satisfy key performance criteria during construction and throughout its design life. Table 2.1 lists some of the key performance issues which should be considered at this stage.

Table 2.1 *Key performance considerations*

Site specific constraints	Project specific requirements
• Site location	• Wall design life
– proximity of adjacent buildings, services, roads, railways, etc?	– durability requirements?
– permissible limits for ground movements and wall deflection?	• Role of wall in overall structure
– delivery of materials to site?	• Wall watertightness requirements
• Site geometry	• Construction programme
– shape and dimensions?	
– site topography?	• Lateral support to wall
– working space for construction plant? Headroom restrictions?	– temporary requirements? – permanent requirements?
– is an embedded wall necessary?	
	• Vertical support from wall
• Ground and groundwater conditions	– temporary requirements?
– geology and hydrogeology?	– permanent requirements?
– soil stratigraphy, fabric and permeability?	
– soil strength and stiffness?	
– wall cut-off requirements?	
– temporary groundwater control measures?	
– wall provides drainage?	
– permanent groundwater control measures?	
– rising groundwater levels?	

In addition to the above, in many cases, project specific design specifications or code of practice requirements will define particular requirements (for example, crack width criteria for reinforced concrete walls) which the retaining wall should also satisfy.

2.4 LIMIT STATES

A description of limit state design philosophy and definition of ultimate (ULS) and serviceability limit states (SLS) is provided in the following documents:

- ENV 1991-1 Eurocode 1 (1994): *Basis of design and actions on structures. Part 1 Basis of design*
- BS 8002 (1994): *Code of practice for earth retaining structures:* section 3.1
- Simpson and Driscoll (1998): *Eurocode 7 a commentary:* section B2
- DD ENV 1997-1 Eurocode 7 (1995) *Geotechnical design: Part 1 General rules.*

Simpson and Driscoll (1998) define limit state design as a procedure in which attention is concentrated on avoidance of limit states, ie states beyond which the retaining wall no longer satisfies the design performance requirements. This relates to the possibility of damage, economic loss or unsafe situations. In limit state design, attention is directed to unexpected, undesirable and unlikely states in which the construction is failing to perform satisfactorily. This is done by taking pessimistic values for the leading parameters involved in the design, strengths, loads and geometric features, and checking that for these, the structure will not fail. The degree of pessimism associated with the selected parameters depends upon the severity, or consequences, of the particular limit state.

The following steps should be taken to verify the safety and serviceability requirements of a retaining wall:

- **list the performance criteria which the wall should satisfy**
- **list the limit states at which the various performance criteria will be infringed**
- **demonstrate that the limit states are sufficiently unlikely to occur.**

In compiling the list of required performance criteria and relevant limit states, the designer should consider the various design situations which can be foreseen during the construction and design life of the wall. The limit states should be shown to be sufficiently unlikely in each design situation.

2.4.1 Ultimate limit states

Ultimate limit states are those associated with collapse or with other similar forms of structural failure. They are concerned with the safety of people and the safety of the structure.

The following should be considered where relevant:

- loss of equilibrium of the structure or any part of it, considered as a rigid body
- failure by rotation or translation of the wall or parts thereof
- failure by lack of vertical equilibrium of the wall
- failure of a structural element such as a wall, anchor, wale or strut or failure of the connection between such elements
- combined failure in ground and in structural element

- movements of the retaining structure which may cause collapse of the structure, nearby structures or services which rely upon it
- failure caused by fatigue or other time-dependent effects.

It is important to note that the above do not mention what type of analysis should be used in studying the limit state, or whether the materials will be responding elastically or in a plastic mechanism. Rather, they are based entirely on the practical issues of degrees of danger, damage and, by implication, cost of repair. Thus, for example, if a structure supported by a retaining wall collapses because of wall displacement, an ultimate limit state has occurred despite the fact that the wall has merely deflected without forming a mechanism in the ground.

For embedded retaining walls, there are several possible ultimate limit states that may be reached. Some of these are shown in BS 8002 (1994), BD42/00 (DMRB 2.1.2) and EC7 (1995) and are reproduced in Figure 2.6.

Figure 2.6 *Ultimate limit state examples*

Wherever possible, the wall should be designed in such a way that adequate warning of danger (ie approaching an ultimate limit state) is given by visible signs. The design should guard against the occurrence of brittle failure eg sudden collapse without conspicuous preliminary deformations. Particular caution should be applied where this is not possible.

Case histories of embedded walls exhibiting "failure" are few. Malone (1982) reports six sheet pile wall collapses and two cases of gross movement of sheet pile walls. In these cases, the causes of failure were identified to be:

- inadequate support to the wall from the ground due to insufficient embedment
- buckling of the struts providing lateral support to the wall
- structural inadequacy of the connection between the strut and the wall
- inadequate foundations of raking struts
- over-excavation of the soil berm or its premature removal prior to installation of the struts.

Other case histories of sheet pile wall failures are provided by Sowers and Sowers (1967), Broms and Stille (1976), Daniel and Olsen (1982) and Rowe (1986).

Rowe (1986) describes the significant progressive outward movement of the toe of an anchored sheet pile wall due to the softening of the clay below the excavation in front of the wall from water seeping through the interlocks along confined permeable horizons in the ground. Inadequate understanding of the geological and hydrogeological conditions at the site was a significant contributory factor.

Problems experienced in reinforced concrete bored pile and diaphragm walls typically relate to difficulties in concreting leading to insufficient cover to reinforcement and lack of watertightness at joints.

The above indicates that **wall failures are seldom due to inadequacies of modern earth pressure theories or structural failure of the wall itself. They are most likely to be caused by:**

- **inadequate understanding of the geological and hydrogeological conditions**

- **poor design and construction *details* and poor standard of workmanship, particularly relating to support systems**
- **construction operations and sequences which result in earth pressures being different to those assumed in design**
- **inadequate control of construction operations, eg overexcavation of berms and formation, excessive surcharge loads from soil heaps and construction equipment.**

2.4.2 Serviceability limit states

See also

2.2.2….Geotechnical categorisation of retaining walls

6.4……SLS calculations

Serviceability limit states correspond to conditions beyond which specific service performance requirements are no longer met, eg pre-defined limits on the amount of water seepage, wall deflections.

The following should be considered where relevant:

- unacceptable wall deflections and associated ground movements
- unacceptable leakage through or beneath the wall
- unacceptable transport of soil grains through or beneath the wall
- unacceptable change to the flow of groundwater.

The permissible movements specified in design should take into account the tolerance of nearby structures and services to displacement. Limiting values should be assigned to allowable wall deflections and the movement of the ground adjacent to the wall. A cautious estimate of the distortion and displacement of the retaining wall and the effects on nearby structures and services should be made on the basis of comparable experience (as defined in section 2.2.2) and SLS calculations, where appropriate (section 6.4). This should include the effects of wall construction. The estimated displacements should not exceed the limiting values.

2.5 GROUND MOVEMENTS

Ground movements are of little consequence where there is nothing to be affected by them. However, in most urban locations where embedded retaining walls are likely to be installed, they can be of great importance. **The decision to set particular wall deflection and ground movement limits can be of significant economic importance. The setting of appropriate limits should be carefully considered.**

Measurements of wall deflections and ground movements and the use of numerical analysis over recent years have allowed better understanding of ground behaviour. The assessment of ground movements is not straightforward and much experience is required to make appropriate use of numerical analysis. It is therefore essential that optimum use is made of precedent in comparable conditions through the use of case history data.

This section discusses the principal sources of ground movements and methods for their prediction using case history data and the results of published numerical analysis. Principles of building damage assessment are also discussed.

2.5.1 Sources of ground movements

Ground movements arise from:

- construction of the wall
- excavation in front of the wall
- flow of water causing loss of ground and consolidation caused by changes in water pressures due to seepage through and/or around the wall.

Each of these is discussed separately below.

See also

2.5.2….Predictions of ground movements

Table 2.2

4.1.2….Effect of wall installation

Construction of the wall

The construction of walls may involve driving or boring piles or excavating panels into the ground. The former may cause vibrations and the latter involves loss of ground support. All construction processes make some noise.

The ground movements arising depend upon the ground conditions, the construction duration and methods used, and the quality of workmanship. **Damaging movements tend to be localised and caused by construction problems, for example unacceptable vibration, overbreak. The removal of obstructions and the excavation of guide trenches prior to wall construction may cause as much or more movement as wall installation itself.**

Driven piles: these can be installed by impact (drop, diesel and hydraulic hammers), vibration or by jacking methods. Ground vibrations due to piling may cause compaction of loose coarse grained deposits and may extend to considerable distances. Noise and vibration can be significantly reduced using jacking methods. Guidance on noise and ground vibrations caused by piling operations is provided in the following references:

- Hiller and Crabb (2000) *Groundborne vibration caused by mechanised construction works*
- BS 5228 (1992) *Noise control on construction and open sites. Part 4 Code of practice for noise and vibration control applicable to piling operations*
- BRE Digest 403 (1995) *Damage to structures from ground borne vibrations.*

King post walls: the installation of one pile every few metres will result in little ground movement. The installation of the infill panels may lead to more significant movements, particularly in coarse grained soils, depending on the installation method. These movements are difficult to quantify and depend on the standard of workmanship.

Bored pile walls: sequential construction of piles to form a wall causes the ground to move to take up support in adjacent ground or on the back of an adjacent pile. Movements are therefore confined to local areas around the piles, except where ground is “flowing” towards the pile as it is being bored. This can happen in pile boring below the water table in loose sandy deposits or soft clays during the construction of secant and contiguous pile walls. Construction in these conditions should proceed with great care and caution under good operational control ensuring that temporary casing is kept ahead of spoil removal and that a high water level or support fluid level is maintained within the bore.

Diaphragm walls: panels will normally be kept full of drilling support fluid to prevent collapse of trench sides. The magnitude of the ground movements will depend upon the margin of safety against trench instability which in turn is critically dependent upon the level of the support fluid relative to the groundwater level. For a diaphragm wall constructed by grab techniques, the support fluid level will inevitably drop as the grab is lifted out of the trench. The designer should not assume that the level can be maintained at platform level at all times and a 0.5 m reduction is realistic. Movements will also occur as the ground arches horizontally between adjacent panels and vertically after construction of each panel.

Case histories presenting measured ground movements arising from bored pile and diaphragm wall construction are discussed in section 2.5.2.

See also

2.5.2….Predictions of ground movements

Table 2.4

Excavation in front of wall

Ground movements arising from the effects of excavation in front of an embedded wall are influenced by:

- stress changes due to excavation
- soil strength and stiffness
- changes in groundwater conditions
- stiffness of the wall and its support system
- shape and dimensions of the excavation
- other effects such as site preparation works, installation of deep foundations, etc
- quality of construction workmanship.

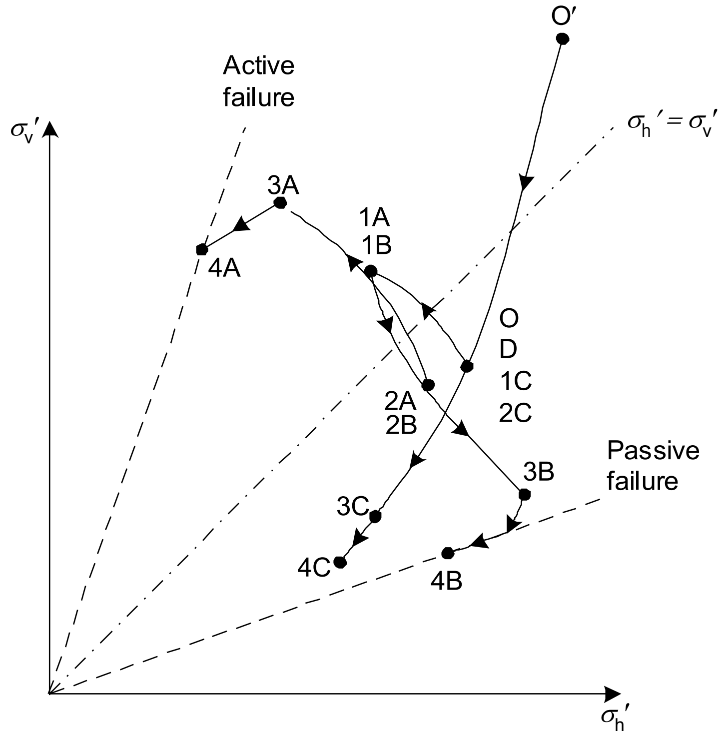
Good discussions of the above factors are provided in GCO publication 1/90 (Hong Kong Government, 1990) and Puller (1996).

The choice of wall type and excavation construction method are major influences on the magnitudes of wall and ground movements. **Inadequate workmanship and poor construction control are particularly significant contributory sources of ground movements.**

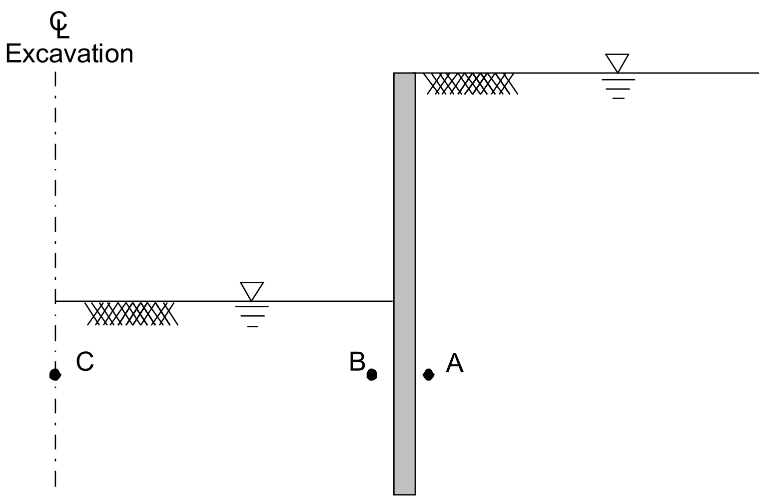
Overall ground movements (heave beneath the excavation, wall deflection and ground movements around the excavation) are influenced by the strength and stiffness of the ground. Movements are smaller in stronger, stiffer soils, such as dense coarse-grained soils and stiff clays compared to those in soft clays and loose sands.

Stiffer walls attract larger bending moments compared to more flexible walls in the same conditions (Potts and Day, 1990). The stresses imposed by the soil are free to redistribute through a more flexible wall, thus reducing the structural forces imposed on the wall. Although this may reduce bending moments, it occurs at the expense of larger wall and ground movements. This is acceptable only where such movements are tolerable. The designer needs a framework within which to consider this compromise between flexible wall types, which may be cheaper in themselves, and the necessary increase in propping or support levels (required to limit movements), which can increase construction costs through increased obstruction, materials and labour costs.

Box 2.2 *Stress paths for soil elements near an excavation retained by a cast in situ embedded wall*



0′ - 0 : Overburden removal after deposition (overconsolidation)
0 – 1 : Excavation for wall installation under support fluid
1 – 2 : Concreting of wall
2 – 3 : Excavation of front of wall
3 – 4 : Long term groundwater seepage conditions



The stress paths of four elements are considered:

- element A located immediately behind an embedded wall
- element B located immediately in front of an embedded wall
- element C located remotely in front of an embedded wall beneath the centre of the excavation
- element D remotely located behind an embedded wall.

It is assumed that pore water pressures are initially hydrostatic below an in situ groundwater level. After excavation, steady state seepage eventually develops from the initial groundwater level behind the wall to a groundwater level at formation level in front.

The geological stress history of the clay comprises deposition followed by the removal of overburden, resulting in an overconsolidated material with $\sigma'_h > \sigma'_v$ in situ. This is represented on the indicative stress path O'-O in the Figure above. There may also be reloading by superficial deposits. This is not shown on the Figure.

During wall installation, both elements A and B will be subjected to a reduction in lateral total stress as the excavation is made under support fluid, followed by an increase in lateral total stress as the concrete is poured (assuming uncased bores or panels). Field measurements (Symons and Carder, 1992) and centrifuge model tests (Powrie and Kantartzi, 1996) show that pore water pressures will fall during excavation under bentonite and recover back to approximately their in situ values during concreting. Indicative effective stress paths are shown in the Figure (0 – 1 under bentonite; 1 – 2 during concreting).

During excavation in front of the wall, the wall is likely to move forward into the excavation resulting in a reduction in horizontal total stress for the soil element A behind the wall. This will cause a reduction in pore water pressure behind the wall. In an overconsolidated clay, shear following yield will also generate negative excess pore water pressures (2 – 3 in the Figure). The long term steady state seepage pore water pressure behind the wall is less than the initial in situ hydrostatic value, but probably greater than the pore water pressure immediately following excavation so that the pore water pressure will probably increase in the long term as steady state conditions are approached (3 – 4). Thus overall (2 – 4), soil element A will experience a reduction in pore water pressure and a reduction in horizontal total stress at constant vertical total stress. These changes in pore water pressures and boundary stresses will result in an increase in vertical effective stress and a decrease in horizontal effective stress, bringing the soil element A towards the active condition.

During excavation, soil element B will experience a large reduction in vertical total stress, which will result in a large reduction in pore water pressure. Movement of the wall below formation level into the soil in front will tend to increase the horizontal total stress. These changes are likely to result in an increase in horizontal effective stress and a reduction in vertical effective stress during excavation (2 – 3 in the Figure). In the long term, pore water pressures will increase again as steady state seepage develops, reducing both vertical and horizontal effective stresses and bringing the soil towards passive failure (3 – 4).

These changes in stress from the initial, prior to excavation, stage are summarised in the Table below.

	Element A	**Element B**	**Element C**	**Element D(1)**
Vertical total stress during excavation	Constant	Decreases	Decreases	Unchanged
Horizontal total stress during excavation	Decreases	Decreases due to unloading. Increases due to wall movement	Decreases	Unchanged
Pore water pressure during excavation	Decreases	Increases	Decreases	See note(2)
Pore water pressure in the long term	Probably increases	Decreases	Decreases	See note(2)
Undrained shear strength in the long term	Probably decreases	Decreases	Decreases	Unchanged
Strain during excavation	Vertical compression	Vertical extension	Vertical extension	
Strain in the long term	Vertical compression	Vertical extension	Vertical extension	

Notes

1. Assumed to be located sufficiently remotely from the wall so as not to be affected by changes in soil stress due to excavation in front of the wall.
2. Depends on ground permeability.

Addenbrooke *et al* (2000) define a displacement flexibility number, Δ, as:

$$\Delta = EI/h^5 \quad (2.1)$$

where *EI* is the Young's modulus multiplied by the second moment of area of the wall section per metre length, and *h* is the average vertical prop spacing of a multipropped support system. By keeping Δ constant, a designer can consider various wall types and associated propping systems for the same absolute displacement.

In general, flexible walls with many props (smaller *h*) will give similar displacements to stiff walls with fewer props (larger *h*). This means that flexible walls (eg sheet pile walls) embedded in stiff clays and other competent soils can be adopted without significant increase in ground movements. However, the cost of the additional propping may outweigh the cost benefit of using a flexible wall and this may not result in an overall saving.

The plan extent, shape and depth of an excavation affect the magnitude of the associated ground movements. In many situations, embedded retaining walls form a closed box, or a more complex shape. In such cases the distribution of movement will be complex and the magnitude difficult to estimate without the benefit of comparable experience. This is discussed further in section 2.5.2.

Movements due to water

Figure 2.7 *Movements due to water flow (after Clough and O'Rourke, 1990)*

Figure 2.7 shows some potential water flow situations which can result in ground movements. Water movements in and around an excavation can occur:

- through flaws in an impervious wall
- by flow through a wall (eg contiguous bored pile wall, leakage through sheet pile interlocks)
- by flow under the wall
- by flow along boundaries between soils of different permeability
- by flow along the wall itself if the wall penetrates an underlying aquifer
- by dewatering.

If piezometric pressures in an aquifer underlying an excavation are not properly reduced, heaving of the base can occur, leading to loss of passive restraint to the wall. Piping may also occur in coarse-grained soils. A case history illustrating base instability in a cofferdam excavation is discussed by Preene *et al* (2000). Rowe (1986) describes a case history of piping failure at the base of an excavation in interbedded sand and clay horizons due to water leakage through sheet pile interlocks along confined horizons. Consolidation settlements may also occur in fine-grained soils due to groundwater lowering caused by a combination of the above effects.

2.5.2 Predictions of ground movements

See also

Appendix B… Ground movements and case history data

Figure 2.14…. Procedure for prediction of wall deflection

2.5.1…………Source of ground movements

2.5.3…………Control of ground movements

2.5.4…………Principles of building damage assessment

5.4.5…………Stiffness

Ground movements cannot be accurately predicted, but it is possible to estimate them based on either an empirical approach from field measurements or from analytical methods which are based on numerical models calibrated against comparable experience. This section considers the estimation of ground movements using empirical correlations based on case history field measurements. Analytical methods are considered in section 4.2.

The main reason for estimating ground movements is to assess possible damage to existing roads, buildings and services located close to the wall. The sensitivity of structures to ground movements depends on the type of structure (Burland and Wroth, 1975 and Burland, 2001). Differential movements are usually more important than total movements and horizontal movements are more damaging than vertical ones. Also, the pattern of movement is important: hogging is worse than sagging. The principles of building damage assessment are discussed in section 2.5.4.

Movements caused by wall installation and excavation in front of the wall are considered separately below.

Ground movements arising from wall installation

Guidance on ground vibrations caused by driven piling is provided in the references listed in section 2.5.1. There is little published data on ground settlements due to pile driving. Symons *et al* (1987) report ground surface settlements of about 50 mm immediately adjacent to 13 m long Larssen sheet piles driven in sand and gravel at Hatfield. Ground surface settlements were observed to reduce to zero at a lateral distance corresponding to the wall depth behind the wall. The observed pattern of settlement indicates that densification of the coarse grained soils may have occurred close to the piles due to vibration.

The magnitude of ground movements will depend upon the quality of workmanship. Large local ground movements can be expected where construction problems are encountered.

Ground movements arising from bored pile and diaphragm wall installation in stiff clays are summarised in the following publications and documents:

- Clough and O'Rourke (1990)
- Thompson (1991)
- Carder (1995)
- Carder *et al* (1997).

The case history data considered by the above are tabulated in Appendix B.

Figure 2.8 *Ground surface movements due to bored pile wall installation in stiff clay*

Figure 2.9 *Ground surface movements due to diaphragm wall installation in stiff clay*

Figures 2.8 and 2.9 show the combined data collated from Clough and O'Rourke (1990), Thompson (1991), Carder (1995) and Carder *et al* (1997) and can be used to estimate ground surface movements arising from the construction of bored pile and diaphragm walls embedded in stiff clays. Table 2.2 summarises the magnitude and extent of the monitored ground movements for walls installed under conditions of good workmanship. The data presented in Figures 2.8 and 2.9 are relatively limited, particularly measurements of horizontal movements for walls. Ground movement estimates based on Figures 2.8 and 2.9 and Table 2.2 should therefore be treated as indicative only. **At locations where such movements are of importance, appropriate instrumentation should be installed and the ground movements monitored accordingly.**

Table 2.2 *Ground surface movements due to bored pile and diaphragm wall installation in stiff clay*

Wall type	Horizontal movements		Vertical movements	
	Surface movement at wall (% of wall depth)	**Distance behind wall to negligible movement** (multiple of wall depth)	**Surface movement at wall** (% of wall depth)	**Distance behind wall to negligible movement** (multiple of wall depth)
Bored piles				
Contiguous	0.04%	1.5	0.04%	2.0
Secant	0.08%	1.5	0.05%	2.0
Diaphragm walls				
Planar	0.05%	1.5	0.05%	1.5
Counterfort	0.1%	1.5	0.05%	1.5

Notes:

1. Maximum surface movement occurs close to the wall and is calculated as a percentage of the pile depth/ diaphragm wall trench depth, as appropriate.
2. Extent of movement is calculated non-dimensionally by dividing by the pile depth/diaphragm wall trench depth, as appropriate

Ground movements arising from excavation in front of wall

Ground movements associated with excavations comprise “global” and “local” movements. Global movements are caused by elastic movements in the ground whilst local movements are concentrated and plastic and arise as the soil approaches its limiting strength. Movements induced by the excavation are made up of the response to the removal of lateral support to the sides of the wall and the response to the removal of the vertical load at the base of the excavation.

Figure 2.10 *Typical ground movement pattern associated with excavation stress relief*

Figure 2.10 shows typical movement patterns. The removal of the vertical stress from the base of the excavation in a homogeneous clay may result in movements over a large area around the excavation as a response to an undrained event. This results in vertical heave within the excavation and settlement outside, magnitudes being very dependent on the stiffness of strata beneath the excavation. It is difficult to measure short term undrained movements as these occur during excavation. In the long term, the sides of the excavation may be restrained from movement but the base and the ground outside may swell. These movements which extend further than the short term undrained movements may continue for a long time. Burford (1988) indicates that measured heave beneath the Shell Centre in London shows little sign of slowing down after 30 years. Carder and Darley (1998) report measurements of heave beneath the road carriageway at Bell Common over a period of 11 years. They report that the majority of the measured heave occurred within the first 4 years after bulk excavation and then continued at a more gradual rate thereafter. The magnitude and pattern of lateral ground movements will depend upon the stiffness of the ground, the wall and its support system. If the ground at the base of an excavation is rigid, the area influenced outside it will be small.

It is simplest to think of vertical and lateral movements separately in order to understand how different factors influence the magnitude and distribution of ground movements.

Vertical movements due to excavation stress relief

The magnitude of vertical movements due to excavation stress relief depend upon the plan extent and depth of the excavation and ground stratigraphy. This is project specific.

Methods of estimating short term ground heave and case histories of measured ground heave in well instrumented and monitored excavations in stiff clays are presented in Appendix B. Estimation of long term heave is much more uncertain due to lack of published field monitoring data. Available case history data indicate total heave to be much greater in magnitude than the corresponding settlement at similar stress changes. It is probably very dependent on the availability of water to allow swelling of the clay.

Monitored retaining wall deflections and associated ground movements

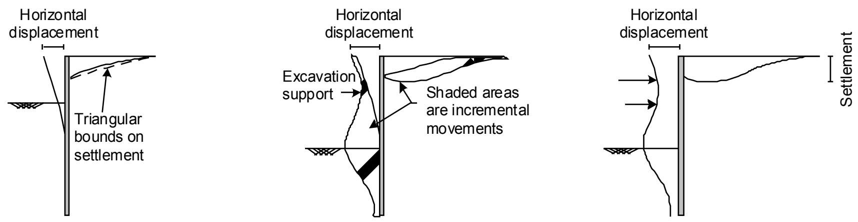
Many researchers have published measurements and patterns of wall deflections and ground movements arising from excavation in front of retaining walls, most notably:

- Peck (1969)
- Clough *et al* (1989)
- Clough and O'Rourke (1990)
- St John *et al* (1992)
- Carder (1995)
- Fernie and Suckling (1996)
- Carder *et al* (1997)
- Long (2001).

Lists of the case histories considered by the above researchers are provided in Appendix B. Box 2.3 shows typical movement profiles.

Box 2.3 *Typical movement profiles*

The general pattern of wall movement and adjacent ground deformation is shown below.



(a) Cantilever movement **(b)** Deep inward movement **(c)** Cumulative movement

During the initial stages of construction, soil may be excavated before the installation of support. In some cases, soil is excavated when the upper levels of support are not preloaded or lack stiffness to restrict inward movement. The wall deforms as a cantilever, and the adjacent soil settles such that vertical surface movements decrease with increasing distance from the edge of the excavation. Settlements during this stage of construction may be bounded within a triangular displacement distribution see (a) above.

When the excavation advances to deeper levels, upper wall movement is restrained by the installation of new support or the stiffening of existing support members. Deep inward movement of the wall occurs. This is shown as an incremental component of the total displacement, see (b) above. The combination of cantilever and deep inward components results in the cumulative wall and ground surface displacement profiles shown in (c) above. If deep inward movement is the predominant wall deformation, then settlements tend to be bounded by a trapezoidal pattern. If cantilever movement predominates, then settlements tend to follow a triangular pattern.

Figures (d) and (e) show the shape of wall deflections measured at the British Library, London.

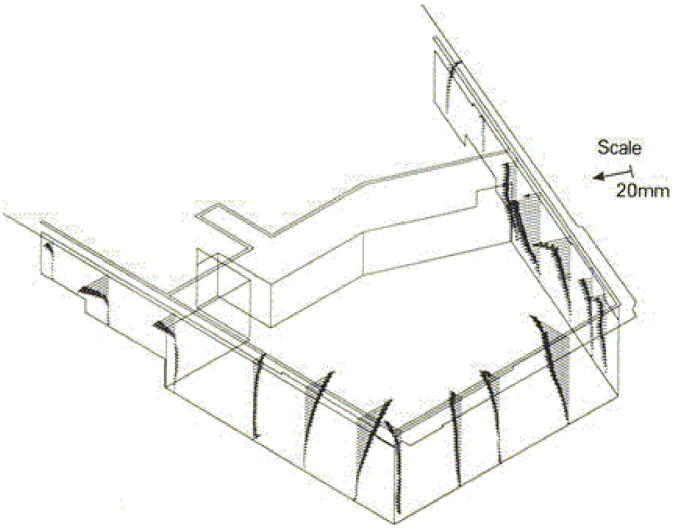


Figure (d) British Library: measured wall deflections - cantilever stage

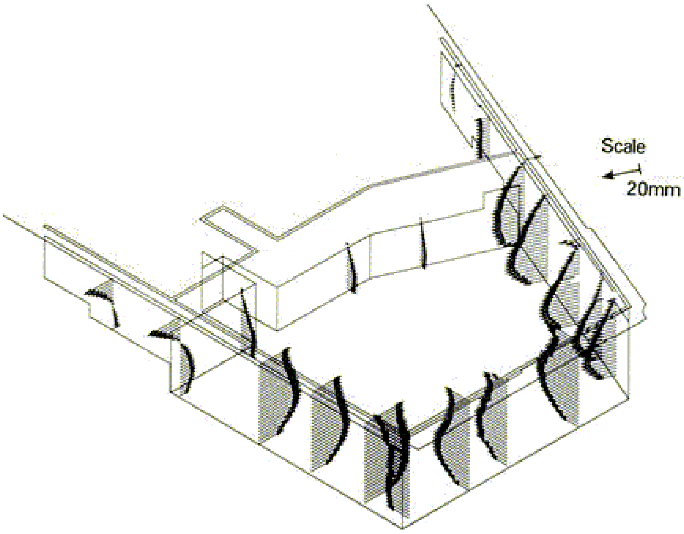


Figure (e) British Library: measured wall deflections - end of construction

Carder (1995) defined three categories of wall support stiffness and considered the performance of the walls comprising his database (Appendix B). These categories are defined in Table 2.3.

Table 2.3 *Support stiffness categories (Carder, 1995)*

Support stiffness	Description/examples
High	*Top down construction*, temporary props installed prior to permanent props at *high* level
Moderate	Temporary props of high stiffness installed prior to permanent props at *low* level
Low	*Cantilever walls*, temporary props of *low* stiffness or temporary props installed at low level

Table 2.4 summarises the magnitude and extent of the monitored ground surface movements due to excavation in front of bored pile, diaphragm and sheet pile walls wholly embedded in stiff clay under conditions of good workmanship. The case history data, upon which Table 2.4 is based, relate to excavations which range in depth from 8 m to 31 m, have a factor of safety against base heave in excess of 3 and where walls are wholly embedded in stiff clay.

Table 2.4 *Ground surface movements due to excavation in front of bored pile, diaphragm wall and sheet pile walls wholly embedded in stiff clays*

Movement type	**High support stiffness** (high propped wall, top down construction)		**Low support stiffness** (cantilever or low stiffness temporary props or temporary props installed at low level)	
	Surface movement at wall (% of max excavation depth)	**Distance behind wall to negligible movement** (multiple of max excavation depth)	**Surface movement at wall** (% of max excavation depth)	**Distance behind wall to negligible movement** (multiple of max excavation depth)
Horizontal	0.15%	4	0.4%	4
Vertical	0.1%	3.5	0.35%	4

Notes:

1. Maximum surface movement occurs close to the wall and is expressed as a percentage of maximum excavation depth in front of the wall.
2. Extent of movement is calculated non-dimensionally by dividing by maximum excavation depth.
3. Movements exclude those arising from wall installation effects.
4. Movements correspond to good workmanship and to walls wholly embedded in stiff clays retaining stiff clays or competent soils.
5. Movements will be greater where soft soils are encountered at formation level, see Appendix B.

Figure 2.11 *Ground surface movements due to excavation in front of wall in stiff clay*

Figure 2.12 *Ground surface settlement due to excavation in front of wall in sand*

Estimates of wall deflections (from the results of finite element analysis) are provided by Clough *et al* (1989) in terms of system stiffness (Figure 2.13).

Figure 2.13 *Maximum lateral wall movement vs system stiffness (after Clough et al, 1989)*

Figure 2.14 *Procedure for prediction of wall deflections and ground surface movements*

Estimates of wall deflections and associated ground surface movements should be made in accordance with the procedure shown in Figure 2.14. Case history based empirical methods of prediction are favoured in preference to the use of complex analyses, unless such analyses are first "calibrated" against reliable measurements of well monitored comparable excavations and wall systems. Table 2.4, in conjunction with Figure 2.11, can be used to estimate *ground surface movements associated with walls wholly embedded in stiff clay*. Figure 2.12 can be used for walls wholly embedded in sands. Preliminary estimates of *wall deflection* can be obtained from Figure 2.13 and from section B3 (Appendix B). This will depend upon the system stiffness, ρ_s, and the factor of safety against base heave. System stiffness is defined in Box 2.4. The reader is referred to CIRIA report C517 (1999), Appendix A4, for a good definition and explanation of base stability.

Available case history data indicate that the magnitude of horizontal wall deflection is almost totally dependent on the effectiveness of the support system. Long (2001) reports that where large wall deflections (greater than 0.3% of maximum excavation depth) were observed for walls wholly embedded in stiff soils, they were principally due to:

- movements associated with an initial cantilever stage at the beginning of the construction sequence
- an overly flexible retaining system
- creep of anchorages; and
- structural yielding.

With a high stiff prop installed early during excavation, the maximum horizontal wall movement is not likely to be measured at the wall top but at a depth of some 0.7 to 0.9 times the maximum excavation depth (Carder, 1995).

Box 2.4 *System stiffness*

System stiffness $\rho_s = EI/(\gamma_w h^4)$ after Clough *et al* (1989)

where *EI* is the Young's modulus multiplied by the second moment of area of the wall section, γ_w is the bulk unit weight of water and *h* is the average vertical prop spacing of a multipropped support system.

The above case history data indicate that for walls embedded in stiff soil with a factor of safety of 3 or more against base heave, wall deflections and associated ground movements are relatively insensitive to variation in wall thickness and stiffness provided the overall system stiffness is not significantly reduced. This means that economies in wall type and size can be achieved through the adoption of flexible walls (eg sheet pile walls) in stiff soils, without significant increase in ground movements. The cost of additional propping should be offset against the cost benefit of using a very flexible wall. It is important to note that increased propping can be counterproductive in reducing ground movements, since rates of construction may be slower, leading to increased movements.

Correlations between horizontal wall deflection and ground movements behind the wall

Where SLS calculations are carried out (section 6.4), these are typically undertaken using sub-grade reaction or pseudo-finite element software packages (such as WALLAP and FREW), for category 2 walls. These provide estimates of wall deflections only. Users of these software packages typically adopt soil parameters which have been determined from back-analysis of well instrumented and monitored case histories of comparable walls and excavations in similar ground conditions, using the same software. In such instances, there is a need to develop an understanding of how the computed lateral wall deflections correspond to associated ground movements behind the wall. However, only limited information is available for such correlations. The relationship between lateral wall deflections and vertical ground settlement behind the wall can be established from:

- case history data, from Figures 2.11 and 2.12 and Table 2.4 for walls embedded in stiff clay
- theoretical evaluation eg Milligan (1983)
- correlations with numerical analysis.

Milligan (1983) considered the development of ground movements around anchored sheet pile walls and introduced the concept of velocity fields. This approach gives an upper bound to the local concentrated magnitude of the short-term undrained plastic deformations behind the wall. He considered local short-term undrained deformation in clay behind flexible walls using a simple field of plastic deformation. Significant plastic movements are confined to a zone bounded by a line at 45˚ from the base of the wall to the ground surface (Figure 2.15).

Figure 2.15 *Simple field of plastic deformation (Milligan, 1983)*

Any point within this zone has equal components of displacement in the horizontal and vertical directions. The settlement profile at the ground surface is identical to the lateral displacement profile of the wall. Thus in Figure 2.15, all points along a particular β line have the same total displacement, controlled by the lateral displacement of the point at which the β line meets the wall. Provided the deflection of the wall can be calculated, the movements of all points in the ground within the zone may be determined.

For walls embedded in stiff soil, back-analysis of case history data relating to 16m and 18 m deep excavations at the YMCA, London, and New Palace Yard, London, respectively using the computer program FREW yields the approximate relationship shown in Figure 2.16. The case histories upon which this relationship is based comprise a top down construction sequence with diaphragm walls embedded in stiff clay retaining up to 10m thickness of coarse grained soils overlying the stiff clay. The relationship shown in Figure 2.16 should only be applied to excavations in comparable ground conditions with similar high support stiffness. Great caution should be exercised if this relationship is applied to situations which differ significantly from those applicable to the case histories upon which it is based.

Figure 2.16 *Relationship between analysed lateral (propped) wall deflections and predicted ground surface settlements in stiff soil*

Insufficient high quality data are currently available in the literature regarding the performance of walls. There is an urgent requirement for more case history data to be obtained to provide high quality measurements of the actual behaviour of different types of retaining wall installed in a range of different ground conditions. In particular, short term and long duration measurements of the vertical and horizontal movements of the wall and the ground around the wall are required to establish appropriate relationships between wall deflections, depth of excavation and ground movements behind and in front of the wall (not only at ground surface level but also with depth and proximity from the wall). Until such reliable measurements are available, relationships such as that show in Figure 2.16 can only be approximate and their applicability limited to very specific application only.

Corner effects

The shape of an excavation will affect the magnitude and distribution of ground movements around it. The corners of the excavation tend to restrict movement. Analyses undertaken to date allowing for 3-dimensional effects are site specific; the extrapolation of the results of such studies for general application is limited. Further work is necessary before reliable correlations can be established for general application. However, St John (1975) found that plane strain and axi-symmetric analyses gave similar vertical movements. Horizontal movements from the axi-symmetric analysis were some 50% of those computed in the plain strain model. Simic and French (1998) found that steel quantities in diaphragm wall reinforcement could be reduced by about 25% at the corners of the excavation they studied. Temporary prop loads measured across the corners of the Mayfair car park excavation in London indicate a 40% reduction within a horizontal distance from the corner equal to the excavation depth, compared to the central props where plane strain conditions predominated (Richards *et al*, 1999). Significant economies are clearly possible.

Ground movements arising from groundwater flow

Ground movements can occur from changes in the groundwater pressure regime around the excavation due to loss of ground and consolidation settlements in fine-grained soils. Consolidation settlements can be significant, particularly in soft compressible soils, and extend to a significant distance behind the wall.

2.5.3 Control of ground movements

Measures that can be adopted to minimise ground movements around and beneath an excavation are summarised below:

- ensure good workmanship throughout. Supports should be installed tight to the wall. The prop, and any packing between the prop and waling, should not rely on friction or adhesion between the prop end and waling to hold it in place
- ensure that the wall has adequate embedment in stiff strata for satisfactory vertical and lateral stability
- minimise the first stage excavation and install the first (stiff) support as early as possible in the construction sequence
- minimise the extent of the dig beyond the proposed support levels;
- minimise delays to the construction of the wall and its support system
- prevent deterioration of lateral support from a clay berm by blinding it or covering it with a waterproof membrane to maintain the berm's natural moisture content
- avoid over-excavation
- minimise removal of fines during dewatering
- minimise drawdown outside excavation.

The initial deflection, which takes place as the wall cantilevers after an initial excavation and before the first prop is installed can, on occasions, be the largest component of the overall wall deflection. It is also the form of displacement most likely to cause damage to nearby structures. **The early installation of a stiff first prop with a shallow first stage excavation is one of the best ways to reduce wall deflections.**

Clough and O'Rourke (1990) and Puller (1996) state that preloading of supports helps to limit movements. The same effect may also be obtained by using prestressed anchors. The preloading is applied by means of jacks at one end of the temporary props immediately after installation of the props and prior to any further excavation. However, in practice there is rarely the need to apply preload. The details at the ends of the props are increased in complexity and cost by the need to accommodate the jacks. The alternative of installing the props and packing tight against the wall is usually sufficient to control deflections.

The measures actually adopted at any given site will depend upon specific project requirements and site constraints.

2.5.4 Principles of building damage assessment

See also

Figure 2.14.....Procedure for prediction of wall deflection and ground movements

A three stage approach should be adopted for the assessment of potential damage to buildings located in the proximity of excavations supported by embedded retaining walls, see Figure 2.17.

Figure 2.17 *Procedure for building damage assessment*

Stage 1

Ground movements behind the retaining wall should be estimated as described in section 2.5.2 assuming greenfield conditions, ie ignoring the presence of the building or utility and the ground above foundation level. Contours of ground surface movements should be drawn and a zone of influence established based on specified settlement and distortion criteria. All structures and utilities within the zone of influence should be identified.

Stage 2

A condition survey should be carried out on all structures and utilities within the zone of influence before the start of the works on site. The structure or utility should be assumed to follow the ground (ie it has negligible stiffness) and hence the distortions and consequently the strains in the structure or utility can be calculated. The method of damage assessment should adopt the limiting tensile strain approach as described by Burland *et al* (1977), Boscardin and Cording (1989) and Burland (2001), see Table 2.5 and Figure 2.18.

Table 2.5 *Classification of visible damage to walls (after Burland et al, 1977, Boscardin and Cording, 1989; and Burland, 2001)*

Category of damage		Description of typical damage (ease of repair is underlined)	Approximate crack width (mm)	Limiting tensile strain ε_{lim} (%)
0	Negligible	Hairline cracks of less than about 0.1mm are classed as negligible.	< 0.1	0.0 - 0.05
1	Very slight	Fine cracks which can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.	< 1	0.05 - 0.075
2	Slight	Cracks easily filled. Re-decoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	< 5	0.075 - 0.15
3	Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15 or a number of cracks > 3	0.15 - 0.3
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15 to 25 but also depends on number of cracks	> 0.3
5	Very severe	This requires a major repair involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually > 25 but depends on number of cracks.	

Notes:

- In assessing the degree of damage account must be taken of its location in the building or structure.
- Crack width is only one aspect of damage and should not be used on its own as a direct measure of it.

Figure 2.18 *Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)*

Reinforced concrete framed structures are more flexible in shear than masonry structures and are consequently less susceptible to damage. Nevertheless, for the purposes of a stage 2 assessment of potential damage, all structures should be treated as masonry structures.

Box 2.5 *Procedure for stage 2 damage category assessment*

The following steps should be undertaken in making a stage 2 assessment of the damage to a structure:

(i) establish L and H for the structure (see Figure 2.18 (a) for definitions of L and H)

(ii) determine (L/H)

(iii) determine relationship between (Δ/L) and ε_h for the required (L/H) from Figure 2.18(b) for εl_{im} values from Table 2.5

(iv) estimate vertical and horizontal ground surface movements in the vicinity of the structure from Figure 2.14

(v) determine (Δ/L) and ε_h $(= \delta_h/L)$ where δ_h is the horizontal movement

(vi) estimate damage category from the relationship between (Δ/L) and ε_h established from step (iii) above.

If the estimated damage category is higher than that specified, a stage 3 assessment should be carried out.

Stage 3

A structural survey of the structure or utility should be carried out. Ground movement estimates should be refined and a soil-structure interaction analysis carried out allowing for the depth of structure foundations, 3-dimensional geometrical effects, non-linear ground characteristics and structural stiffness. The response of the structure should be assessed allowing for the actual conditions, materials and form of construction comprising the structure. It should be noted that the quality of workmanship in building construction can significantly affect the robustness of the building and its ability to tolerate movement.

2.6 KEY POINTS AND RECOMMENDATIONS

1. Design is a continuous process, requiring regular review to ensure that the client's needs are being met. There should be clear allocation of design responsibility between the various parties involved in a retaining wall project. The following are essential to ensure certainty of outcome:

   - good communication between all parties
   - a team approach to problem solving
   - an integrated total project process
   - a risk based approach to design and construction management.

   It is recommended that a lead designer is identified to oversee this process.

2. Risk assessment and management are statutory safety obligations under the CDM regulations. For a typical retaining wall project, completion of Forms A1 or A2 (or similar) is good practice. Completion of Form A3 will be sufficient to satisfy CDM requirements.

3. The designer should assign a geotechnical category to each retaining wall structure. The degree of effort required in site investigation and design should be appropriate

for the relevant category. Procedures of higher categories can be adopted to justify more economic design, where appropriate.

4. There is no substitute for the application of empiricism based on sound judgement and experience at every stage of the design process. The rationale of this process, including details of the experience upon which it is based, should be communicated and explicitly recorded as part of the design. This is essential.

5. Limit states are states beyond which the retaining wall no longer satisfies the design performance requirements. The following steps should be taken to verify the safety and serviceability requirements of a retaining wall:

   - list the performance criteria which the wall should satisfy
   - list the limit states at which the various performance criteria will be infringed
   - demonstrate that the limit states are sufficiently unlikely to occur.

   Wherever possible, the wall should be designed in such a way that adequate warning of distress (ie approaching an ultimate limit state) is given by visible signs. Design calculations and analyses should demonstrate that equilibrium is possible without overstressing materials.

6. Wall failures are seldom due to inadequacies of modern earth pressure theories or structural failure of the wall itself. They are most likely to be caused by:

   - inadequate understanding of the geological and hydrogeological conditions
   - poor design and construction *details* and poor standard of workmanship, particularly relating to support systems
   - construction operations and sequences which result in earth pressures being different to those assumed in design
   - poor control of construction operations, eg overexcavation of berms and formation, excessive surcharge loads from soil heaps and construction equipment.

7. The permissible movements specified in design should take into account the tolerance of nearby structures and services to displacement. The magnitude of ground movements is critically dependent upon the quality of workmanship and construction control during wall installation, subsequent excavation and construction of the wall's support system.

8. In general, flexible walls with many props will give similar displacements to stiff walls with fewer props.

9. Ground movements cannot be accurately predicted. It is essential that optimum use is made of precedent in comparable conditions through the use of good quality case history data. Case history based empirical methods of prediction are favoured in preference to the use of complex analyses, unless such analyses are first "calibrated" against reliable measurements of well monitored comparable excavations and wall systems. Wall deflections and ground movements should be estimated adopting the procedure shown in Figure 2.14.

10. Flexible walls with many props will give similar displacements to stiff walls with fewer props. However, the cost of the additional propping may outweigh the cost

benefit of using a flexible wall. Increased propping may also be counterproductive in reducing ground movements, since rates of construction will be slower, leading to increased movements.

11. The installation of a temporary or permanent prop of high stiffness at a high level early in the excavation sequence in combination with some of the measures listed in section 2.5.3 is the most effective way of controlling ground movements due to wall deflection.

12. A three stage approach should be adopted in the assessment of building damage arising from the effects of ground movements as shown in Figure 2.17. For the purposes of a stage 2 assessment, all structures should be treated as masonry structures; the method of damage assessment should adopt the limiting tensile strain approach described by Burland *et al* (1977), Boscardin and Cording (1989) and Burland (2001)

3 Construction considerations and wall selection

See also

1.5…..Economic design

2…….Design considerations

Major economies are possible at the scheme design stage by reviewing the construction method and type of wall to be used.

This chapter is intended primarily for the designer, but also the constructor, who is considering possible alternative solutions at an early stage in the design process to assist in the selection of an economic solution for the retaining wall and the construction sequence. Figure 3.1 illustrates the decision paths for the selection of appropriate wall type and construction sequence, emphasising the interaction between the design and construction considerations.

Figure 3.1 *Decision paths for selection of appropriate wall type and construction sequence*

The importance of reviewing the temporary and permanent conditions is emphasised to ensure that the solution takes account of the differing requirements for each stage of the wall's life. Construction methods are discussed for the excavation sequences from wall installation to the completion of the permanent works facilitated by the retaining wall. The characteristics of various different construction sequences are considered together with a review of different types of embedded retaining wall.

There may be many factors that affect the choice of construction sequence and type of retaining wall, not all of which relate to the basic design parameters. Personal preference and a history of successful projects using a specific approach often play a major part in the choice of methods. This chapter reviews the advantages and disadvantages of the various methods in order to guide the designer and the constructor away from unsuitable choices. The main issues covered in this chapter are:

- construction sequences appropriate for temporary and permanent works
- temporary and permanent support systems, props, berms, ground anchorages
- selection of appropriate construction sequence
- types of embedded retaining wall, including review of available wall types and associated construction methods and tolerances
- relative construction cost data for various embedded wall types
- wall selection.

3.1 CONSTRUCTION METHODS FOR SOIL SUPPORT

3.1.1 Construction sequence

The construction sequence from existing ground level should usually commence with the installation of the embedded retaining wall, assuming that any site preparation works have already been completed. Any excavation prior to wall installation, while reducing the depth of the wall, may involve additional temporary works, potential difficulties with plant access and extra ground movement. **The designer should consider the**

whole of the construction sequence, up to the completion of the permanent structure.

An efficient retaining wall design should avoid the need to design for particularly high section forces for isolated construction stages. This may not always be possible due to other constraints; for example, a large span between temporary supports during the construction stages may dominate the section design. A review of the support levels to reduce this span may enable a smaller section to be used. A balanced design should make full use of the section capacity at each construction stage.

See also

1.5.....Economic design

3.1.2...Construction requirements

5.5.....Determination of groundwater pressures

Temporary and permanent works

The use of the temporary retaining wall as the permanent works has the economic advantage of installing only one wall (Plates 3.1 and 3.2). When making this decision, the designer should consider the form of the permanent internal face and particularly the watertightness requirements. Where the wall is used to form a basement, guidance on the groundwater protection for various grades of basement usage is given in BS8102 (1990) and CIRIA report 139 (1995). This is discussed further in section 5.5. Table 3.7 identifies wall types which can act as permanent water retaining elements. Alternatively the structural capacity of the wall can be used to support the soil loads and any vertical loads from the permanent works, whilst a secondary wall (eg a reinforced concrete lining wall connected to the inside face of a contiguous piled wall) provides watertightness.

Plate 3.1 *Temporary and permanent works: Bristol underground car park. An example of the use of a sheet pile wall as the permanent wall, exposed and painted*

Some specified permanent works details may prevent the use of the temporary retaining wall as part of permanent works. For example a specified tanking membrane around the outside of the permanent structure will necessitate that the retaining wall only serves a temporary function in most circumstances. Any connections between the wall and internal slabs require careful consideration to ensure the buildability of the solution (section 3.1.2).

For an efficient design the wall capacity required for the permanent condition should also satisfy the requirements of the temporary condition, ideally avoiding the need to provide a stronger or stiffer wall for the temporary conditions. However it is important to have a holistic view of the whole project rather than just concentrate on the wall itself (section 1.5), because it may be preferable to provide an increased wall strength and stiffness specifically for the temporary conditions, to reduce the amount of temporary support. Where the temporary conditions are more onerous than the permanent, it may be possible to design more economically for a reduced durability in the short-term temporary condition. This may take the form of an increased allowable crack width or an increased allowable stress for the temporary case. These decisions should be taken for each project based on a full understanding of the design, cost and programme requirements of the options.

Cantilever wall

Figure 3.2 illustrates the sequence for a cantilever wall.

Figure 3.2 *Cantilever wall construction sequence*

Table 3.1 lists the advantages and limitations of adopting a cantilever wall. While this may be the simplest option, it may not be suitable because of unacceptable deflections during the temporary excavation stages. For deeper excavations the large depth and strength of the wall required to support the cantilevered excavation may make this an uneconomic option. Table 3.8 indicates the typical range of depths for various types of wall.

Table 3.1 *Cantilever wall*

Advantages	Limitations
• A simple construction sequence with no temporary propping to the wall.	• May be uneconomic for deeper excavations, (see Table 3.8).
• The permanent works are constructed in an open excavation free from the restrictions of working around temporary props.	• The deflections generated by the unpropped excavation may be unacceptable.
	• The depth and strength of the wall to ensure stability against overturning may be considerable.

A series of carefully defined construction stages, allowing propping of the wall to be inserted during a staged excavation sequence, is an alternative to a cantilevered solution to reduce the embedment depth, reduce the stiffness of the retaining wall and control the wall deflections. These may be categorised as either top down or bottom up construction sequences.

See also

2.5.3….Control of ground movements

Propped wall - top down sequence

A top down construction sequence is defined by the use of the permanent internal structure as the temporary propping to the retaining wall, cast in a 'top down' sequence. The higher level slabs are cast before the lower level slabs as the excavation progresses to act as horizontal frames for wall support. This process is shown in outline in Figure 3.3.

Figure 3.3 *Top down construction sequence*

Table 3.2 lists the advantages and limitations of adopting a top-down construction sequence. Excavation work takes place through openings in the permanent works beneath the previously cast slabs (Plate 3.3).

Plate 3.2 *Top down construction sequence*

For cut and cover schemes, the opportunity may exist for removal of the spoil and the supply of the construction materials from one or both ends of the site. The logistics of this operation should be carefully considered to avoid programme clashes between excavation routes and casting of the permanent slabs. The design and construction issues to enable use of a top down solution are:

- support for the vertical load of the permanent slabs in the temporary condition, which may be temporary piles, hangers or the permanent columns formed ahead of the excavation by means of piles, barrettes, etc. For short spans between opposing walls, it may be sufficient to connect the slabs to the walls and rely on the shear capacity of this connection to support the vertical load of the slab
- access for the removal of soil and the supply of materials, which may be through the ground floor and substructure slabs
- ventilation should be provided for the work below ground beneath permanent slabs and consideration should be given to safe methods of working

- a construction method for the excavation and construction of the substructure compatible with the available headroom and limited access.

The main advantage of top down construction is the ability to progress the superstructure construction at the same time as the substructure construction. There are a number of extra site planning and design issues, which should be addressed in this circumstance in addition to those noted above:

- sufficient vertical load capacity of the wall and the internal column supports (if applicable) to support the increasing superstructure load throughout the construction sequence
- access through the superstructure works for the substructure works. This can become a critical issue for small confined sites.

Common project requirements that necessitate the use of a top-down sequence are:

- the need to make an early start on superstructure construction
- the need to minimise ground movements.

In general, it is uneconomic to use a top down sequence to reduce the programme time to complete the superstructure unless there are more than two levels of substructure.

Table 3.2 *Top down construction*

Advantages	Limitations
• The superstructure construction can proceed at the same time as the substructure, provided the necessary vertical supports, generally piles, are in place.	• The excavation works and substructure construction are slower and more expensive due to the restrictions on the size of plant and the limited access.
• Temporary propping is replaced by the use of the permanent slabs.	• Holes may have to be left in the slabs to provide access for the subsequent excavation.
• Provides a stiff support system for the wall, minimising movement.	• Vertical support for the permanent slabs is required in the temporary condition.
	• The stiffer construction during the intermediate construction stages attracts higher loads into the permanent structure.

The vertical support to the permanent slabs during the temporary stages can be provided by temporary piles which are either subsequently removed or used as part of the permanent works after suitable surface preparation, to remove any overbreak for cast-in-place concrete piles for example. The most economic foundation option is a single pile supporting each column. The plunge column technique is frequently used. The placing tolerances of top down piles are shown in Table 3.3 depending on the degree of construction control. The table does not include any allowance for the rolling tolerances of the plunged steel beams, which can become critical when working to reduced tolerances.

Table 3.3 *Tolerances for top down piles and plunged columns*

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

Propped wall - bottom up sequence

A bottom up construction sequence is defined by the construction of the permanent works from the lowest level upwards, casting the foundation slab before the internal walls and slabs above. Figure 3.4 illustrates the sequence for a typical excavation with two levels of temporary props.

Figure 3.4 *Bottom up construction sequence*

Temporary props are likely to be heavy steel sections and the safety risks associated with their use should be considered (Plate 3.4).

Plate 3.3 *Bottom up construction sequence*

The safety issues are the risks to the workforce during the installation and removal of these elements together with the risk of accidental damage or unforeseen loading on the props during the excavation and construction operations. The sequential construction of the superstructure and the substructure minimises conflicts between different operations on site and normally results in a less congested critical path compared to a top down sequence.

Table 3.4 *Bottom up construction*

Advantages	Limitations
• Deflections are controlled by the use of propping to the wall. • Compared to a cantilever solution the wall strength, stiffness and depth may be reduced. • Sequential construction of the substructure and the superstructure.	• Compared to a cantilever wall, there are cost and programme penalties with the use of temporary props. • The propping impedes the final excavation and the construction of the permanent works.

An alternative means of achieving a bottom up construction sequence is by means of a circular cofferdam that relies on hoop compression to resist the external soil and water loads without the need for internal propping. The compression member can either be the retaining wall itself (eg a diaphragm or secant piled wall) or ring beams at suitable levels as the excavation progresses (Plate 3.4).

Plate 3.4 *Circular shaft under construction at Blackpool, Lancashire*

A circular cofferdam minimises the wall length for a fixed floor area and can provide an efficient solution where there is sufficient space to accommodate the circular plan area.

3.1.2 Construction requirements of temporary and permanent support system to retaining wall

See also

2.5…Ground movements

7…..Design of support systems

The designer should consider the whole construction sequence when designing an embedded retaining wall to ensure that the design satisfies the requirements of each stage and to minimise the overall construction costs. For example, it is not sufficient to consider the excavation down to formation level without reviewing how the permanent works will be constructed for a bottom up sequence. A cantilever or anchored wall results in no propping interfering with the internal works and therefore is likely to be the constructor's preferred solution. The wall may be either a temporary structure or a permanent wall. There should be sufficient system stiffness if ground movements are to be limited (section 2.5). Plate 3.5 shows various support systems to sheet pile walls at Thelwall Viaduct, Merseyside.

Plate 3.5 *Various support systems to sheet pile walls at Thelwall Viaduct, Merseyside*

See also

7.1…Temporary propping systems

Props

Design guidance for temporary propping is given in CIRIA report C517 (1999). This is discussed further in chapter 7.

Where propping to the retaining wall is required, it may be temporary works and should be removed in a defined manner, as the permanent works are constructed and are able to replace the role of the temporary props. Temporary props are usually made of steel (tubular props are often used due to their efficiency in compression) although concrete props are sometimes used, particularly as corner braces across the ends of excavations. For narrow width excavations, props can be supported by the perimeter retaining walls (Plate 3.6).

Plate 3.6 *Temporary props spanning full width of excavation for the Mayfair car park, London*

For wide excavations, vertical support may be required to the propping system. The prop removal sequence should be carefully considered to avoid the following problems, where possible:

- removal of the temporary props from beneath already constructed slabs
- the location of temporary props through internal walls or slabs, which may need to be cast prior to removal of the props.

CIRIA report C517 (1999) provides some guidance on prop removal.

See also

7.2…Berms

Berms

The use of a berm adjacent to the wall allows the excavation to proceed to a deeper level in the centre of the site, with the advantage that the major part of the excavation can be carried out unimpeded by props. The option of using raking props down to the formation level is shown in Figure 3.5.

Figure 3.5 *The use of a berm and raking props*

The berm is removed once the raking props are in place allowing the permanent structure to be completed, removing the props at a suitable stage (Plate 3.7).

Plate 3.7 *Berm with raking props at Canary Wharf, London*

As noted in section 2.4.1, the premature removal or over-excavation of a berm is one of the causes of failure identified by Malone (1982).

Figure 3.6 shows an alternative use of a berm in conjunction with construction of part of the permanent works.

Figure 3.6 *The use of a berm and a prop to the permanent structure*

The berm is removed once the permanent works are sufficiently advanced to provide support to the wall. A berm is only a realistic option where a lower level of permanent propping is required and usually for a wide excavation (Plate 3.8).

Plate 3.8 *Berm with low level permanent propping at Batheaston Bypass*

The design of the end prop support at formation level for a raking prop should be carefully considered to avoid unacceptable movement as the prop is loaded.

See also

7.3…Anchors

Ground anchorages

If a cantilever wall is not a suitable solution, the use of ground anchorages to provide the horizontal support to the retaining wall can be considered.

The main advantage of the use of ground anchorages is that the excavation remains unobstructed by propping (Plate 3.9). Table 3.5 lists the advantages and limitations.

Plate 3.9 *Anchored contiguous bored pile wall*

Table 3.5 *Ground Anchorages*

Advantages	Limitations
• Once installed, the excavation is free of any obstructions allowing for efficient construction of the permanent works. • The ground anchorage prestress may reduce wall deflection and settlement behind the wall, depending on the magnitude of the prestress.	• The time to install and stress the ground anchorages increases the excavation time. • The ground anchorages often extend outside the site boundaries and the necessary permissions are required. • The ground anchorages require de-stressing and occasionally removing at the end of construction.

Ground anchorages are greatly underused in the UK when compared with experience elsewhere in Europe. Their increased use may result in significant savings over propping schemes where programme time is available for the construction of ground anchorages and the space is available to locate them.

The following design issues should be considered:

- the ground anchorages will be prestressed to a percentage of their working load upon installation. The wall designer should allow for the effect of this preload
- the ground anchorages will usually be installed at an angle to the horizontal, imposing a vertical component of load to be resisted by the retaining wall. Depending on the fixing detail, a moment may also be induced in the wall
- a condition of the permission to install the ground anchorages beneath an adjacent owner’s property may be that the ground anchorages are removed at

the end of construction and the hole grouted up (there are proprietary systems, which allow the removal of the steel tension member)

- space is necessary outside the wall, free from services, to install the ground anchorages. Appropriate site investigation of this space is also required.

See also

7.1…Propping systems

Permanent slabs as props

Use of the permanent slabs of a basement or a cut and cover structure is a common way of providing the permanent support to a wall where the wall forms part of the permanent works. In addition to the prop load (section 7.1.3), the connection may also support vertical loads from the slabs. These may include the slab dead and live loads and, in the case of the base slab, any heave and ground water uplift loads.

The connections between the wall and the slabs may be costly and time-consuming to form, negating the advantages of using the temporary wall as part of the permanent works. The limitations of various connection details are given in Table 3.6.

Table 3.6 *Wall/slab connection types*

Connection type	Applicable wall types	Limitations
Welded bars	Sheet piles (see Figure 3.7 and Plate 3.10 for typical details).	• Site welding required.
Welded UB	Sheet piles and cast-in-place concrete walls where UB used as reinforcement.	• Site welding required.
Drilled-in bars	All concrete wall types.	• Often of limited capacity due to the inability to drill sufficient length to anchor large diameter bars. • Need to locate and avoid the wall reinforcement.
Couplers cast in the wall(1)	Diaphragm walls (see Figure 3.8 for typical details).	• Tolerance in vertical position of the diaphragm wall cage. (±100mm recommended) • Increased congestion(2).
Bend-out bars(1)	Diaphragm walls (see Figure 3.9 for typical details).	• Tolerance in vertical position of the diaphragm wall cage. (±100mm recommended) • Increased congestion(2). • Limitation on the size of the bars which can physically be bent out (16mm mild steel bars or occasionally 20mm).
Hinged joint(3)	All concrete wall types (see Figure 3.10 and Plate 3.11 for typical details of a hinged joint used at a project in London)	

Notes:

1. Couplers and bend-out bars are not usually recommended for cast-in-place piles due to the difficulties of ensuring their angular and vertical position.

2. The increased congestion may prevent the concrete flowing around the bars with honeycombing resulting where the concrete stresses are particularly high. The bars attached to the couplers and bend-out bars should not impede any concrete or tremie pipe used to place the concrete.

3. Hinged joints have been used on several road schemes in the UK to accommodate the heave of the underlying clay.

Figure 3.7 *Typical connection detail at sheet pile wall/concrete slab*

Plate 3.10 *Sheet pile wall/concrete slab connection at Bristol underground car park*

Figure 3.8 *Typical detail for couplers cast within a diaphragm wall panel*

Figure 3.9 *Typical details of bent out bars in diaphragm wall panel*

Figure 3.10 *Hinged slab: A406 North Circular Road, London*

Plate 3.11 *Hinged joint: A406 North Circular Road, London*

One solution to the above limitations is to avoid the need for a shear or moment connection between the slabs and the wall, designing the joint to resist only compression forces from the wall. There are various options for supporting the vertical loads on the slabs as an alternative to the shear connection to the wall:

- hangers may be used to carry the vertical load to support points above the slabs, often to the wall capping beam. The hangers may operate in the temporary and/or in the permanent condition (eg at the Copenhagen Metro deep stations, as described by Beadman and Bailey (2000))
- internal columns, designing the slab to cantilever out from the column position to the wall
- a permanent wall, cast against the temporary wall to provide the long term support for the vertical loads. This option may prove to be an economic alternative where it allows an economic temporary wall to be specified and avoids expensive wall/slab connections. However, in this circumstance, the designer should give careful consideration to how the loads will be shared between the temporary and permanent walls (Wharmby *et al*, 2001).

A further variation, where high uplift loads are applied to the structure from the base slab, is to form a corbel just above the base slab to support uplift loads. The corbel prevents any high fixed end moments from the base slab being transferred into the wall, which may be advantageous where the base slab edge support moments risk over-stressing the wall.

3.1.3 Selection of appropriate construction sequence

See also

2.3……….Design requirements and performance criteria

Table 2.1...Key performance considerations

The selection of the appropriate construction sequence involves many aspects of design and construction. The designer is aware of the design constraints and the contractor may have preferred ways to construct the works, based on equipment and propping availability or personal experience. It is difficult for a designer in isolation to appreciate the full picture and define the construction sequence. In selecting an appropriate construction sequence, the following issues should be considered:

- excavation depth and propping requirements
- deflection limits for the retaining wall
- use of the retaining wall as part of the permanent works

- sufficient space within the site boundaries to install the temporary and permanent wall
- permanent works details.

3.2 TYPES OF EMBEDDED RETAINING WALLS

3.2.1 Review of wall types

See also
Appendix C…Wall types

Wall types can be categorised by their material (usually steel or reinforced concrete) and by their installation method. In general steel walls are driven, vibrated or pushed into the ground without removal of any material. Reinforced concrete walls are created by first removing the ground and where necessary providing some form of temporary ground support in advance of placing the concrete and the reinforcement. This is a generalisation and some construction methods fall outside these definitions, but these techniques are relatively unusual and often costly.

The available wall types are described in detail in the *Specification for piling and embedded retaining walls*, (Institution of Civil Engineers, 1996) and summarised in Appendix C which illustrates some of the different wall types, demonstrating in particular the differences between contiguous and secant piled walls. For all wall types, ground conditions or environmental constraints may prevent their installation.

Table 3.7 *Wall types*

Wall type	Advantages	Limitations
Sheet Piles	• Provide an economic wall with a predictable surface finish. • No arisings to be removed. • Suitable as a water retaining wall.[1] • Can be used as both the temporary and the permanent wall.	• Maximum pile length approximately 30 m. • Potential declutching in coarse grained soils.
Combi wall	• Enables high capacity walls to be formed.	• Installation may be complex.
King post	• Can be installed around obstructions at isolated points.	• Not suitable to retain water in the long term. • Cannot be used for excavation below the groundwater table in coarse grained soils.
Contiguous pile	• The cheapest form of concrete piled wall.	• Not a water retaining solution. • Not a permanent solution in any soil due to the gaps between piles, unless a structural facing is applied.
Hard/soft secant	• Acts as a water-retaining temporary wall. • The use of soft piles enables the hard piles to be formed	• Not usually a permanent solution to retain water. • The soft pile mix is not significantly cheaper than concrete. The local

	using lower torque rigs than for hard/hard secant piles.	concrete plant is often unable to batch the soft material, so site batching is required. • The depth is limited by the verticality tolerance, which may determine the extent of the secanting (see Appendix C).
Hard/firm secant	• A permanent water retaining wall.[1] • The firm material for the primary (female) piles is either a standard concrete mix, retarded to reduce the strength when the secondary (male) piles are constructed or a reduced strength concrete mix.	• The depth is limited by the verticality tolerance, which may determine the extent of the secanting (see Appendix C).
Hard/hard secant	• A permanent water retaining wall.[1] • Installed using standard piling plant, with high torque rigs.	• The cutting of the hard primary (female) piles requires high torque rigs or oscillators. • The depth is limited by the verticality tolerance, which may determine the extent of the secanting (see Appendix C).
Diaphragm wall	• A permanent water retaining wall.[1] • Can be installed to great depths, provided the verticality tolerances can be accepted. • In some circumstances the face of the diaphragm wall can form the final finish subject to some surface cleaning and removal of protuberances. • Less joints compared to piled walls.	• Horizontal continuity is difficult to achieve between panels. • Cannot follow intricate plan outlines. • The installation equipment is extensive, requiring a large site area for accommodation of the support fluid plant, reinforcement cages and the excavation plant. • Disposal of the support fluid is costly.

Notes:

(1) Wall types are discussed in Appendix C

(2) Sheet pile walls, hard/firm secant pile walls, hard/hard secant pile walls or diaphragm walls may provide an acceptable level of water retention if a low grade (eg Grade 1, BS8102) of substructure/basement is required. For higher grades of space, structural facing walls and/or drained cavities should also be provided. Drained cavities should be designed to be kept free of water and adequately ventilated, to prevent penetration of water vapour into the substructure.

3.2.2 Wall construction methods and tolerances

The essential guide to the ICE specification for piling and embedded retaining walls, a joint publication between the Federation of Piling Specialists and the Institution of Civil Engineers (1999) contains some useful guidance for the designer on wall construction methods and tolerances. Table 3.8 shows typical applications and tolerances of embedded retaining walls.

Table 3.8 *Typical applications of embedded retaining walls (after Federation of Piling Specialists and Institution of Civil Engineers, 1999)*

Wall type (5)	Typical height range		Groundwater control		Verticality	
	Cantilever	Propped	Temp.	Perm.	Typ.(1)	Optimum.(1)
Sheet pile wall	to 5 m	4-20 m	Yes	Yes	1:75	1:100
Combi wall	to 10 m	5-20 m	Yes	No	n/a	n/a
King post	to 4 m	4-20 m	No	No	n/a	n/a
Contiguous pile	to 5 m	4-20 m	No	No	1:75	1:125
Hard/soft secant	to 5 m	4-20 m(2)	Yes	No(3)	1:75	1:125
Hard/firm secant (cfa)(4)	to 6 m	4-18 m	Yes	Yes	1:75	1:125
Hard/hard secant (cased)	to 6 m	4-25 m	Yes	Yes	1:75	1:200
Diaphragm wall (grab)	to 8 m	5-30 m	Yes	Yes	1:75	1:125
Diaphragm wall (mill)	to 8 m	5-50 m	Yes	Yes	1:200	1:500

Notes:

(1) Typical verticality is achievable without special measures under normal conditions. Optimum verticality is achievable with additional control measures. A higher degree of verticality may be possible. This should be discussed with the piling contractor.

(2) The depth to which hard/soft secant pile walls can provide water resistance is restricted by the construction tolerances of the boring rig and the groundwater pressure to be resisted. This type of wall is commonly used to resist groundwater flow to maximum depths of approximately 6 m, although up to 8 m head of groundwater has been retained.

(3) The long term resistance of the soft elements of hard/soft secant pile walls to groundwater flow relies on the wall remaining in a damp environment. Long-term water resistance is usually provided by additional works such as reinforced concrete lining walls, which transfer the groundwater load into the hard piles.

(4) Cfa, continuous flight auger piling techniques, require the cage to be pushed into the wet concrete, which limits the pile depth depending on the soil conditions. For example, coarse grained material above the ground water level can drain the free water from the concrete and induce a premature set, making the cage installation difficult.

(5) Wall types are described in Appendix C.

In specifying construction tolerances, the designer should consider the overall substructure retaining wall thickness. This may include:

- a guide wall, where required. This should be at least 1 m deep with a minimum width of 0.1 m at its narrowest part
- the embedded wall (diaphragm wall, contiguous/secant pile wall, sheet pile wall etc)

- an allowance for construction tolerances: position plus verticality plus an allowance for protrusions (for cast-in-place concrete walls)
- a facing wall, where required (eg for hard/soft secant pile walls)
- a drained cavity (say 0.15 m, but 0.05 m is possible in extreme circumstances)
- a blockwork wall (typically 0.15 m)
- an allowance for wall deflections, if applicable.

In addition there should be sufficient clearance from any adjacent buildings to install the wall (eg a typical bored piling rig requires a clearance of between 0.9 m and 1.2 m from the pile centreline and the face of the building). Building overhangs in particular should be considered. This gives a typical distance from the site perimeter to the inside face of the substructure wall of between about 1 m and 2 m for a concrete wall and approximately 0.75 m for a sheet pile wall. The project requirement will usually be to minimise this overall wall thickness.

3.2.3 Relative construction costs data for various embedded wall types

The use of the wall in the permanent condition in addition to the temporary case ensures that the wall is only formed once with the resulting cost and programme saving. **It is important to consider the full implications of the use of the temporary wall as the permanent wall to ensure a cost saving for the overall structure.** The following issues influence the overall cost:

- the space occupied by the retaining wall
- the watertightness criteria for the inner face of the permanent wall
- the use of an inner lining wall to form a drained cavity or to conceal the formed face of the embedded wall
- the connection details between the wall and the permanent slabs. For embedded walls, these details can be difficult and costly to form (section 3.1.2).

Table 3.9 gives an indication of the relative costs of various types of wall to assist in the choice of wall. It is stressed that this table can only be indicative and the designer is encouraged to discuss the choice of wall with the constructor. The table does not make any allowance for the effect of differing installation rates on the project.

Table 3.9 *Relative costs to assist choice of wall*

Wall type	Mobilisation factor (1)	Cost factor based on price per m² relative to a hard/soft secant piled wall (<650 mm) Equivalent thickness of concrete wall (mm)(2)				
		<650	650-800	850-1000	1050-1200	1250-1500
Sheet pile – temporary(3)	0.06	0.5	1.2	-	-	-
Sheet pile – permanent	0.03	1.0	1.8	-	-	-
King post wall	0.03	0.8	-	-	-	-
Cfa contiguous piled wall	0.04	0.5	0.6	0.7	-	-
Cfa hard/soft secant piled wall (low torque rigs)	0.06	1.0	1.1	-	-	-
Cfa hard/firm secant piled wall (high torque rigs)	0.07	1.1	1.2	-	-	-
Cased hard/hard secant piled wall (high torque rigs)	0.1	-	2.4	2.1	1.8(4)	-
Grab diaphragm wall	0.4	1.9	2.1	2.2	2.5	2.7
Reverse circulation mill diaphragm wall	0.8	2.3	2.5	2.6	2.9	3.1

The total cost is the sum of the mobilisation factor and the cost factor.

Notes:

(1) Mobilisation costs for each wall type are included on the basis that the wall area is 1500 m^2, namely a 15 m deep wall on a 100 m perimeter. Different wall areas may be assessed by a pro-rata of the mobilisation factor, eg mobilisation factor for a grab diaphragm wall for a wall area of 6000 m² = 0.4 x 1500 ÷ 6000 = 0.1.

(2) The sheet pile and king post walls have been chosen to match the strength of the equivalent concrete piled wall thickness.

(3) The temporary sheet pile costs are based on a resale value of the sheets and an extra mobilisation to extract the sheets.

(4) The cost of the hard/hard secant piled wall reduces slightly with increasing wall thickness due to the reduced number of piles per unit length.

The basis for the above cost comparisons is as follows:

- The costs are based on typical subcontract prices and do not include for any main contractor overheads such as site management, welfare facilities, site security etc.
- The costs do not include the construction of any facing or lining walls
- Costs for removal of spoil from the site are not included for the cast-in-place concrete wall
- Guide walls are included for all cast-in-place concrete walls except for the contiguous piled walls.

3.3 WALL SELECTION

The form of the wall affects the design parameters. The designer should determine the form of the wall before undertaking the detailed design because of the fundamental effect that the choice of wall can have on the design. It is difficult for the designer to determine the wall type without consideration of the practical aspects governing the installation of the wall. The final choice of wall type is often a compromise, satisfying several criteria:

- cost
- ground conditions, particularly the need to retain groundwater and the presence of obstructions, including any remains of archaeological interest, in the path of the wall
- the need to restrict ground movements to within acceptable limits
- extent of the site to accommodate construction plant. This is particularly important for the use of diaphragm walling techniques, with the need to accommodate support fluid mixing and storage facilities and reinforcement cages
- compatibility with the permanent works
- durability
- contaminated ground
- environmental issues
- speed of construction.

Some of these issues are discussed below.

3.3.1 Ground conditions and obstructions

See also

5.2.2….Ground stratigraphy and soil fabric

The ground conditions may dictate the type of equipment needed to install the wall, which is likely to affect the cost. Obstructions and boulders can prevent the installation of sheet piles and some types of reinforced concrete walls (eg cfa piles) without pre-treatment of the ground to remove or break up the boulders. This is often possible near the surface, but becomes more difficult at depths beyond 3-4 m. The presence of hard strata above the required toe level will necessitate special measures to ensure that the wall is installed, as tabulated below:

Table 3.10 *Measures for dealing with obstructions*

Type of wall	Installation measures	Potential issues
Sheet piles	Heavy hammers etc.[1]	Noise and vibration. Appropriate sheet pile section required.
	Preboring.	Additional settlement.
	Jetting (limited ability to deal with hard strata).	Additional settlement.
Cast-in-place piles (contiguous or secant piled walls)	Cased system rather than cfa, allowing chiselling etc.[2]	Longer installation programme with some vibration and noise.
Diaphragm walls	Chiselling for grabbed walls.[3]	Vibration, noise and overbreak.
	Reverse circulation mill.[4]	Expensive mobilisation. Inefficient to deal with fine grained material.

Notes:

1. Choice of an appropriate sheet pile section may enable obstructions to be pushed aside or broken up, particularly if used with the panel driving method of installation as described in the *Piling Handbook* (British Steel, 1997).
2. A cased pile system allows a range of drilling tools to be used within the casing to remove obstructions. The teeth on the casing are also used to drill through obstructions. This is a way of drilling through existing foundations, which may contain steelwork.
3. Diaphragm wall grabs are able to remove smaller boulders from the trench, although this may cause some overbreak where the boulder extends beyond the sides of the trench.
4. The reverse circulation mill requires a more extensive cleaning plant for the drilling fluid than for a grabbed diaphragm wall, to remove the spoil from suspension. On small sites, the cleaning plant may be located off site with the drilling fluid piped on and off the site.

3.3.2 Groundwater

See also

5.2……Investigation of ground and groundwater conditions

5.4.2….Soil permeability

5.5……Determination of groundwater pressures

Groundwater levels above excavation level usually dictate the need to install a wall that acts as a groundwater cut off. The need for a groundwater cut off is an important design decision with cost implications for the wall. The alternatives to the groundwater cut off are:

- temporary dewatering to lower the groundwater level, subject to due consideration being given to the potential resulting ground movements
- if the ground is sufficiently impermeable to allow temporary excavation with sump pumping only, a permanent wall could provide the groundwater cut off
- excavation is carried out underwater and part of the permanent works is placed under water either using tremie techniques or precasting
- a groundwater cut off is provided around the outside of the wall, allowing dewatering to take place around the wall.

3.3.3 Durability

See also

6.6.3….Steel sheet pile walls

6.6.4….Cast in place concrete

Durability will not usually be a concern for a temporary wall, unless the soil contains particularly aggressive contaminants. For use as part of the permanent works, the wall should satisfy the durability requirements specified for the permanent works in order to provide the required design life. For steel sheet piles, this is usually provided by means of an additional sacrificial thickness of steel to allow for the potential corrosion during the life of the wall (section 6.6.3). For concrete walls, the durability is satisfied by reference to the applicable code of practice for structural concrete (BS8110, BS 5400 or EC2 Part 1 for example). The durability requirement of the concrete is satisfied by means of a minimum cement content, maximum water cement ratios, minimum strength and minimum cover to the reinforcement, subject to an acceptable standard of workmanship on site (section 6.6.4).

3.3.4 Contaminated ground

See also

2.1.4….Contaminated land

2.2.2….Geotechnical characterisation

5.2.1….Investigation of ground and groundwater

The presence of contaminated ground creates additional safety risks for any wall system which involves removal of the ground. The handling and disposal of the arisings should be planned and carried out to minimise the risks to the site operatives and to the environment. The use of wall types which avoid soil removal (eg sheet piles) should be considered in these circumstances.

Certain aggressive chemicals prevent the use of cast-in-place concrete without the use of protection to the concrete such as sheet polyethylene or polychloroprene (BS 5328 Part 1, 1997, Table 7). This limits the choice of wall to sheet piles, although some of the cast-in-place systems could be adapted to satisfy these requirements (eg precast diaphragm wall panels).

3.3.5 Environmental issues

There is growing concern about the impact of construction on the environment and the designer and constructor should ensure that the issues have been properly considered. The choice of wall can affect the environment during the installation of the wall, during the wall's life and when the time comes to remove the wall. The issues to be considered are listed in Table 3.11:

Table 3.11 *Environmental issues throughout the lifecycle of the wall*

Life-cycle of wall	Environmental issues
Installation	Noise and vibration
	Number of vehicle movements associated with the wall construction
	Use of sustainable materials (for guidewall construction for example)
	Dust, gases and leachate from contaminated spoil, prior to disposal
	Disposal of any contaminated spoil
Working life	Effect on the local groundwater
End of life – removal	Ease of removal
	Reuse of materials.

3.4 KEY POINTS AND RECOMMENDATIONS

1. Major economies are possible at the scheme design stage by reviewing the construction method and type of wall to be used. The designer should consider the whole of the construction sequence, up to the completion of the permanent structure.

2. Subject to the permanent works details, there are economies to be gained by using the temporary wall as the permanent wall.

3. The constructor will always prefer a clear excavation with no propping to constrain the permanent works. This can be realised by the use of a cantilever wall or an anchored wall. The limitations of the use of a cantilever wall may include a substantial wall with unacceptable deflections during the excavation.

4. A propped wall with a top-down construction sequence provides a stiff support system with no temporary propping and is typically adopted where there is a need to:

   - make an early start on superstructure construction
   - minimum ground movements.

   In general, it is uneconomic to use a top down sequence to reduce the programme time to complete the superstructure unless there are more than two levels of substructure.

5. A bottom-up construction sequence is commonly adopted when the wall requires propping. Ground anchorages may be used as an alternative to temporary props to provide an obstruction free construction zone, but the programme time should be available for the construction of the ground anchorages and the space available to locate them. Where this is possible, use of ground anchorages may result in significant savings over schemes requiring propping.

6. Different wall types are discussed to identify their suitability in particular circumstances and relative costs are provided to allow a crude comparison to be made between the options

4 Analysis

See also

Appendix D...Soil mechanics

This chapter is intended primarily for geotechnical designers and students, and provides background information on lateral earth pressures and guidance on the methods of analysis that may be used in the design of an embedded retaining wall. It is assumed that the reader is familiar with the basic principles of soil mechanics essential to retaining wall design summarised in Appendix D, which are:

- the concepts of total, effective and shear stress
- the representation of the stress state within the cross sectional plane of a long retaining wall using the Mohr circle construction
- the distinction between undrained (short term) total stress analysis and drained (long term) effective stress analysis
- key aspects of soil behaviour relevant to embedded retaining walls, including the effect of stress history; soil strength; and soil stiffness
- the formulation of limiting (active and passive) lateral stresses (earth pressures), and basic limit equilibrium calculations for embedded retaining walls.

This chapter:

- discusses the evolution of earth pressures in overconsolidated soils
- provides guidance on the evaluation of limiting lateral earth pressures
- outlines the two main classes of analysis that may be used as the basis for design (limit equilibrium, based on conditions at collapse, and soil-structure interaction analyses which may be used either at collapse or to give an estimate of working conditions)
- discusses the application of these methods to Ultimate Limit State (ULS) and Serviceability Limit State (SLS) design calculations, providing guidance where appropriate for different structural forms.

4.1 EARTH PRESSURES

See also

4.1.5....Determination of limiting lateral earth pressures

Horizontal (lateral) stresses are usually described and quantified by means of a *lateral earth pressure coefficient, K*:

$$K = \sigma'_h / p'_v \qquad (4.1)$$

where p'_v is the effective overburden pressure and σ'_h is the horizontal (lateral) effective stress at the same point within the soil mass. The effective overburden pressure p'_v is given by

$$p_v' = \int_o^z \gamma dz + q - u \qquad (4.2)$$

where,

γ is the bulk density
z is the depth below ground surface
u is the pore water pressure
q is any uniform surcharge at ground surface.

Earth pressure coefficients are used to represent the state of stress in a soil mass. They are expressed as the ratio of horizontal effective stress to effective overburden pressure, p'_v, (rather than horizontal to vertical effective stress, σ'_v) because soil/wall friction makes the vertical effective stress in the soil adjacent to a retaining wall difficult to calculate.

4.1.1 In situ lateral stress

See also

5.4.3….In situ stress conditions

Unlike some other retaining walls, embedded walls retain predominantly natural ground. The pre-existing or *in situ* horizontal (lateral) earth pressure, possibly modified by the wall installation process, is therefore a potentially important consideration. The symbol K_o is used to denote the earth pressure coefficient describing the initial, *in situ* stress state in the ground before the wall is installed. The *in situ* earth pressure coefficient of a clay deposit depends, like its specific volume, on the geological stress history. Deposition (or burial under a glacier) corresponds approximately to one-dimensional consolidation, during which the horizontal and vertical effective stresses increase in proportion to each other (Figure 4.1). Clays may also become overconsolidated by desiccation (drying) on exposure to air, by vegetation or by freezing.

Figure 4.1 *Schematic stress history of an overconsolidated clay deposit*

On unloading, which might occur due to the erosion of overlying soil, re-saturation after desiccation, the melting of an overlying glacier or a rise in groundwater level, the horizontal effective stress, σ'_h, tends to remain "locked-in", decreasing proportionately less quickly than the effective overburden pressure, p'_v. Thus the *in situ* earth pressure coefficient K_o in an overconsolidated clay stratum is usually greater than unity. In heavily overconsolidated clays, a zone of undisturbed soil extending to a depth of several metres from the surface may be close to its limiting passive pressure as a result of the geological unloading process.

In situ lateral earth pressures for both normally consolidated (during deposition) and overconsolidated soils (following unloading) may be estimated using the equations and methods given in section 5.4.3.

4.1.2 Effect of wall installation

See also

Appendix E….Effects of wall installation
5.4.3…………In situ stress conditions

The process of installing a diaphragm or bored pile retaining wall in an overconsolidated soil may potentially be important in three respects:

1. Wall installation by boring or excavating panels may reduce the horizontal effective stresses close to the wall to below their *in situ* values. Wall installation by ground displacement methods (eg driving) may increase the horizontal effective stresses close to the wall.

2. During wall installation, the surrounding soil may be subjected to various stress paths involving lateral unloading and reloading. These define the recent stress history of the soil, which may influence the soil stiffness during bulk excavation in front of the wall (Powrie *et al*, 1998).

3. Ground movements during wall installation may require consideration in their own right (section 2.5.2). Experience (Thompson, 1991; Powrie and Kantartzi, 1996) indicates that ground movements due to the installation of a cast-in-place wall, under good workmanship conditions, in stiff ground where the water table is low are unlikely to be significant. Ground movements arising from wall installation where the ground is very soft and/or the water table is high, or workmanship is poor or local construction difficulties (eg obstructions in the ground) are encountered can be significant. For driven walls in coarse grained deposits, vibration induced settlement can also be significant.

Numerical analyses of the post excavation behaviour of the wall that do not take stress relief due to wall installation into account may overestimate wall bending moments and prop loads (eg Potts and Fourie, 1984; Powrie and Batten, 2000; Batten and Powrie, 2000).

It is difficult to give general guidance on the magnitude and extent of lateral stress reduction during wall installation, because this will depend on:

- the initial *in situ* earth pressure coefficient
- the soil properties and groundwater conditions
- the individual pile or panel geometry
- the detailed method (eg whether pile bores are cased, supported using support fluid or open) and sequence of construction.

Wall installation can be modelled in finite element analyses as described by Powrie and Batten (2000); Batten and Powrie (2000) and Ng *et al* (1995). In most cases, however, this degree of detail would be excessive, and the effects of wall installation will need to be taken into account empirically (section 5.4.3). Much uncertainty remains regarding the effects of wall installation. This is an area where further work and research is required to improve understanding.

The field data and finite element analyses reported in the literature and summarised in Appendix E suggest that the installation of a diaphragm wall in panels might be expected to reduce the *in situ* lateral earth pressure coefficient in an overconsolidated clay stratum by about 20%, and the installation of a bored pile wall by about 10%. It should be checked that the resulting pre-excavation lateral stress distribution is not less than that exerted by the wet concrete during construction. For a wall cast under support fluid, this is considered to be reasonably well represented by the hydrostatic pressure of wet concrete from the top of the wall to a critical depth $h_{crit} \sim H/3$, where H is the overall depth of the wall. For depths greater than h_{crit}, the rate of increase of lateral stress with depth is equal to the unit weight of the support fluid, ie:

$$\sigma_h = \gamma_c.z \qquad \text{for } z \leq h_{crit} \qquad (4.3)$$

$$\sigma_h = \gamma_c h_{crit} + \gamma_b (z - h_{crit}) \qquad \text{for } z \geq h_{crit} \qquad (4.4)$$

where σ_h is the horizontal total stress, γ_c is the unit weight of the concrete, γ_b is the unit weight of the support fluid, and z is the depth below the top of the wall (Lings *et al*, 1994).

With simple elastic soil-structure interaction analyses, which probably tend to underestimate the stiffness of the soil in lateral unloading, a lateral earth pressure

coefficient of unity prior to excavation is likely to give reasonably realistic bending moments and prop loads.

4.1.3 Limiting values

See also
4.1.5....Determination of limiting lateral earth pressures

When soil is removed from in front of an embedded retaining wall, the wall will usually move into the excavation. This will result in a reduction in the lateral stress in the soil behind the wall, eventually bringing it to the active condition in which the soil is at failure with the horizontal effective stress as small as it can be for the effective overburden pressure. In the soil that remains in front of the wall below formation level, the lateral stress increases until eventually a state of passive failure is reached, in which the horizontal effective stress is as large as it can be for the effective overburden pressure.

Approximations to these limiting pressures on a retaining wall may be calculated by considering either the stresses in the soil at failure or the equilibrium of an assumed sliding wedge. In the first, following Rankine (1857), a set of equilibrium stresses which do not violate the strength of the soil is studied and it is *assumed* that earth pressure increases linearly with depth. The limits which are calculated by this approach are sufficient for stability, but may be unnecessarily severe. They are therefore safe. In the second approach, following Coulomb (1776), the limits obtained are necessary for stability but may not be sufficient and could be unsafe. Coulomb's method only yields the total force on the wall: to estimate the equivalent pressure distribution on the wall, further assumptions are needed. Neither Rankine nor Coulomb consider the mode of wall deformation.

For a frictionless wall, the Rankine and Coulomb analyses give compatible results, see Figures 4.2 and 4.3.

Figure 4.2 *Rankine plastic equilibrium for a frictionless wall/soil interface translating horizontally*

Figure 4.3 *Coulomb's method to calculate limiting active force for a frictionless wall/soil interface translating horizontally*

Researchers such as Sokolovski (1965), Caquot and Kerisel (1948) and Kerisel and Absi (1990) have developed methods to account for realities such as wall friction, sloping ground surfaces and more complicated patterns of deformation. Most used failure mechanisms to determine the limiting force on the wall between the ground surface and any given point. Like Coulomb's original analysis, such methods give the limiting forces on a wall but do not give the equivalent pressure distribution explicitly, and produce limits that are necessary but may not be sufficient. However, the degree of refinement is such that these more recent theories may be considered to give accurate values for the limiting forces.

4.1.4 Wall friction and adhesion

See also
Appendix C....Review of current wall design
6................Design of wall

The interface between the soil and the wall is not frictionless. As a result of wall friction, the resultant force between the wall and the soil is inclined rather than normal to the wall.

Figure 4.4 *Effect of wall friction*

Figure 4.4 shows how a propped embedded retaining wall moves in response to the lateral loads applied to it. On the retaining (active) side, the soil slumps downward relative to the wall. On the excavated (passive) side, the soil heaves upward relative to the wall. These directions of relative soil/wall movement will tend to reduce K_a on the retained side and increase K_p in the soil in front of the wall. Both of these changes are beneficial to wall stability.

However, there are some cases in which the directions of relative soil/wall movement behind or in front of the wall may not be beneficial as conventionally assumed. These include:

- load bearing walls, which may move downward relative to the soil on both sides (the downward component of the load in an inclined anchor might have a similar effect)
- where an activity such as dewatering a compressible horizon or excavating an underlying tunnel results in the soil in front of the wall settling relative to the wall
- unusual loading conditions, for example a wall being used as a tension member.

Care should also be exercised in selecting passive earth pressure coefficients in zones where the wall is moving into the retained soil, eg above a prop just below the top of the wall (section 4.1.7), or below the pivot point in the case of an unpropped cantilever wall, where the direction of soil/wall friction is uncertain (Bica and Clayton, 1998).

Wall friction

The soil adjacent to a wall surface will generally have been disturbed by installation of the wall, and so will probably have no tendency to dilate. Hence, the *maximum* friction angle, δ_{max}, that can be mobilised against the surface of a wall, would be expected to be equal to the critical state angle of shearing resistance of the soil, ϕ'_{crit} (Bolton and Powrie, 1987; and Powrie, 1996 following Rowe, 1963). If the wall roughness is less than the typical particle size of the soil (D_{50}), lower values of δ will apply (Jardine *et al*, 1993). If the wall as very rough relative to the average particle size of the soil, forcing a rupture surface to develop within the undisturbed body of the soil rather than along the interface, some or all of the dilatant strength of the soil might be mobilised, giving an upper limit of ϕ'_{peak} (Subba Rao *et al*, 1998). In general, however, it is more conservative to take $\delta_{max} = \phi'_{crit}$. Rowe and Peaker (1965) show that the wall friction actually developed depends on the direction and magnitude of the movement at the soil / wall interface and that quite large movements might be required to develop full friction on the passive side. However, more recently, Subba Rao *et al* (1998) report results using shear box tests which show that small (less than 5 mm) relative movements can be sufficient to develop full friction at the soil/wall interface.

Walls formed from driven piles may require more careful consideration, particularly in overconsolidated clays where the large displacements at the soil/wall interface will probably have reduced the soil/wall friction angle to the residual strength of the soil.

Codes of practice have traditionally advocated the use of values of soil/wall friction angles δ that are somewhat less than the soil angle of shearing resistance, ϕ', (Appendix H). This is partly because if the peak angle of shearing resistance ϕ'_{peak} were used as a design parameter, a soil/wall friction angle of $\delta = \phi'_{peak}$ would be unrealistically high in most circumstances. The assumption $\delta < \phi'$ takes account of the "imperfect" roughness of the soil/wall interface. Furthermore, consideration of the vertical equilibrium of the wall may indicate that, in some circumstances, the wall friction angle δ may not attain the same value uniformly on both sides of the wall.

Wall adhesion

In total stress analysis, in stiff clays, the soil/wall adhesion, s_w, is often assumed to be a factor of approximately 2 smaller than the undrained shear strength of the soil s_u (ie $s_w = \alpha \times s_u$, where $\alpha \approx 0.5$). This is primarily to account for softening of the soil at the soil/wall interface during wall installation. Smaller values of wall adhesion may apply in particular circumstances (chapter 6).

Values of wall friction and adhesion for use in design calculations

Recommended limiting values of wall friction and adhesion for use in design calculations are given in chapter 6. In determining appropriate values, the following questions should be addressed:

- are the assumed directions of shear stresses at the soil/wall interface consistent with the expected relative soil/wall movements?
- can the wall be in vertical equilibrium under the actions of the assumed interface shear stresses, its own weight, any imposed load that the wall has to carry and a reaction force at the base?
- if designing to a particular code of practice, does the code rely on the use of a reduced soil/wall friction angle to provide some element of a factor of safety?
- how sensitive is the calculation to the assumed direction and values of soil/wall friction, particularly in zones where the direction of relative soil/wall movement is uncertain?

The effects of wall construction on the interface friction or adhesion between the soil and the wall should be taken into account.

For the particular circumstances of walls which support very large vertical loads at maximum excavation stage and which can settle relative to the soil under load, it is generally prudent to assume the limiting values for wall friction or adhesion given in section 6.3.1 over the embedded portion of the wall and zero friction or adhesion on the retained side above excavation level.

The calculation of lateral earth pressure coefficients taking into account the effects of shear stresses at the soil/wall interface is addressed in section 4.1.5.

4.1.5 Determination of limiting lateral earth pressures

See also

Appendix D....Soil Mechanics

Appendix F....Earth pressure coefficients

5.3...............Assessment of drained/undrained soil conditions

Earth pressures can be determined assuming either drained (effective stress) or undrained (total stress) conditions. The factors that must be taken into account in determining whether drained or undrained conditions apply are discussed in Appendix D and section 5.3.

Effective stress analysis

The effective horizontal active and passive earth pressure equations in generalised form are given by:

$$\sigma'_{a} = K_{a}\,p'_{v} - 2c'\sqrt{\{K_{a}.(1+(s'_{w}/c'))\}} \qquad (4.5)$$

$$\sigma'_{p} = K_{p}\,p'_{v} + 2c'\sqrt{\{K_{p}.(1+(s'_{w}/c'))\}} \qquad (4.6)$$

where: σ_a' is the effective active pressure acting at a depth in the soil

σ_p' is the effective passive pressure acting at a depth in the soil

p'_v is the effective overburden pressure ($p'_v = \int_0^z \gamma dz + q - u$)

γ is the bulk density (saturated density if below water level)

z is the depth below ground surface

u is the pore water pressure

q is any uniform surcharge at ground surface

c' is the cohesion (if any)

s'_w is the wall adhesion (if any)

K_a and K_p are earth pressure coefficients, the values of which depend on ϕ', δ, and β,

where: δ is the soil/wall friction

β is the slope of the soil surface.

In effective stress analysis, it is usual to adopt $s'_w = 0$.

The total horizontal active and passive earth pressures acting on the wall (which govern its structural behaviour) are given by:

$$\sigma_a = \sigma'_a + u \tag{4.7}$$

$$\sigma_p = \sigma'_p + u \tag{4.8}$$

where: u is the pore water pressure.

Published values of active and passive earth pressure coefficients K_a and K_p usually relate to the horizontal component of earth pressure. However in some cases, values relating to the resultant stress (which acts at an angle δ to the normal to the wall) are also given and care should be taken to ensure that the correct (horizontal component) values are used. Wall shear stresses may be obtained by multiplying the horizontal effective stress component σ'_p or σ'_a by the tangent of the angle of wall friction, tan δ.

Charts and equations of the horizontal components of earth pressure coefficients are given in Appendix F for the following cases:

- K_{ah} vs ϕ' for a vertical wall and backfill slopes β/ϕ' = -1, -0.75, -0.5, -0.25, 0, +0.25, +0.5, +0.75 and +1 for δ/ϕ = -1, -0.75, -0.67, -0.5, -0.33, 0, +0.33, +0.5, +0.67, and +1
- K_{ph} vs ϕ' for a vertical wall and backfill slopes β/ϕ' = -1, -0.75, -0.5, -0.25, 0, +0.25, +0.5, +0.75 and +1 for δ/ϕ' = -1, -0.75, -0.67, -0.5, -0.33, 0, +0.33, +0.5, +0.67, and +1

These have been calculated on the basis of the equations given in EC7 (1995), which are also reproduced in Appendix F, together with a definition sketch for the assumed inclinations of wall friction δ and slope angle β. The equations given in EC7 (1995) have been adopted to facilitate programming and calculations using spreadsheets. Simpson and Driscoll (1998) show that these equations give results generally close to those of Kerisel and Absi (1990). Exceptions occur for high values of ϕ' and (δ / ϕ'), for which EC7 (1995) is more conservative.

Total stress analysis

In total stress analysis, the generalised horizontal active and passive earth pressures are given by:

$$\sigma_a = \left(\int_0^z \gamma dz + q \right) - 2s_u \sqrt{\left(1 + \frac{s_w}{s_u}\right)} \tag{4.9}$$

and

$$\sigma_p = \left(\int_0^z \gamma dz + q \right) + 2s_u \sqrt{\left(1 + \frac{s_w}{s_u}\right)} \tag{4.10}$$

where:

σ_a is the total horizontal active earth pressure acting at a depth in the soil

σ_p is the total horizontal passive earth pressure acting at a depth in the soil

s_u is the undrained shear strength

s_w is the wall adhesion.

It is normal practice to apply limits on the value of s_w adopted in design (see chapter 6 and Appendix H). The term $2\sqrt{[1 + (s_w / s_u)]}$ is an approximation and should, in theory, not exceed 2.56.

4.1.6 Tension cracks

See also

5.5.1....Undrained conditions

5.9.1....Temporary works design

In theory, for a wall with no soil/wall adhesion and no available groundwater, the minimum active lateral total stress needed to support a clay soil in undrained conditions is negative to depths of $(2s_u - q)/\gamma$ and $2\,(2s_u - q)\,/\gamma$ by the Rankine and Coulomb analyses respectively (see Box 4.1), where s_u is the undrained shear strength, q is a uniform surface surcharge and γ is the unit weight of the soil. Rather than rely on a tensile stress acting across the soil/wall interface to help support the wall, it is usual to assume that a tension crack develops to depths below the retained surface where the calculated active lateral total stress is negative.

Box 4.1 *Theoretical depths of tension cracks by the Rankine and Coulomb analyses*

For a retaining wall where there is no soil/wall adhesion and no available groundwater, the theoretical depth of tension cracks by the Rankine and Coulomb analyses is given by:

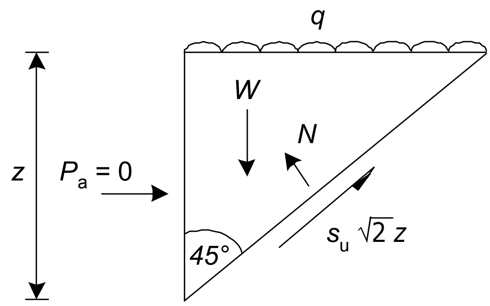
Rankine

The depth of tension cracks, z, is given by:

$$\int_{o}^{z} \gamma dz + q - 2\, s_u = 0$$

$$\therefore\ z = (2\, s_u - q) / \gamma$$

Coulomb



Resolving horizontally: $P_a = N \cos 45° - s_u \sqrt{2}\ z \sin 45° = 0$

$$\therefore\ N = s_u \sqrt{2}\ z$$

Resolving vertically: $W + qz \tan 45° = N \sin 45° + s_u \sqrt{2}\ z \cos 45°$

where,

$$W = ½\, \gamma z^2 \tan 45°$$

$$N = s_u \sqrt{2}\ z$$

$$\therefore\ z = 2\,(2\, s_u -) / \gamma$$

This is twice the depth of the tension crack derived from the lower bound stress field (Rankine) analysis.

Determination of the actual depth of tension cracks is complex. In design, a shallow tension crack is more onerous for wall stability than a deep one. It is common to assume a tension crack depth derived from a lower bound stress field analysis (such as Rankine) in conjunction with a minimum equivalent fluid pressure (MEFP).

Where water is not expected, the depth of tension cracks should be assumed to be $(2s_u - q)/\gamma$. The total pressure acting on the retained side of the wall at any depth z (in metres) below ground level should be assumed to be given by a MEFP of 5 z kPa or by total stress analysis (Equation 4.9), whichever is greater, see Figure 4.5.

Figure 4.5 *Tension cracks: minimum total horizontal stress*

In the case of an embedded cantilever wall or where access to water is possible, consideration should be given to the possibility that a tension crack may flood. If this happens, the clay will be supported by the hydrostatic pressure of water in the tension crack, which may then open to a depth of $(2s_u - q)/(\gamma - \gamma_w)$, where γ_w is the unit weight of water. In such circumstances, in design, hydrostatic pressure of $\gamma_w z$ kPa (where z is depth in metres below the retained surface) should be adopted on the retained side of the wall to the depth where the total stress calculated from Equation 4.9 exceeds this value (Figure 4.5).

In the case of an embedded wall propped or anchored near or at the top, it is possible that the increase in lateral stress associated with the movement of the wall into the retained soil and/or stress redistribution onto a relatively stiff prop or anchor may prevent the ingress of surface water into a tension crack. Thus, **provided that a lateral stress greater than the hydrostatic pressure of water at the same level can be demonstrated over a minimum depth of 1m near the top of the wall, the possibility of a flooded tension crack developing behind a propped or anchored wall in a uniform homogeneous isotropic stratum of clay may be discounted**. Consideration would still need to be given to the possibility of water entering a tension crack, for example through:

- a sand parting or other more permeable horizon in the ground at a lower level
- preferential drainage paths which may have developed during wall installation (eg sheet piles driven through coarse grained soils into fine drained soils "dragging down" permeable soil behind the wall.

4.1.7 Factors affecting limiting lateral earth pressures

Ground stratigraphy

Within each stratum, the theoretical limiting active and passive pressures calculated using equations 4.5 to 4.10 will generally increase with depth. At the interface between two strata, the overburden pressure will take a single value but the lateral stress will change as a result of the different soil properties in each layer. In design analyses, the theoretical limiting lateral stresses should be used in each stratum as appropriate.

Adjacent highway traffic and railway vehicle loading

The effect of adjacent highway and railway loading should be determined from section 5.6.1, Tables 5.11 and 5.12.

Adjacent permanent vertical loading

A *uniform* surcharge acting over either the retained or the excavated soil surface is easily taken into account, as it simply increases the vertical total stress at every depth.

A *strip load* running parallel to the wall may be modelled using the procedure suggested by Pappin *et al* (1986), which is illustrated in Figure 4.6 (a) or by the 45° distribution approach proposed by Georgiadis and Anagnostopoulos (1998), which is illustrated in Figure 4.6 (b).

Figure 4.6 *Additional lateral effective stress acting on the back of a wall due to a strip load running parallel to it*

Finite element analyses (Georgiadis and Anagnostopoulos, 1998) show that a small lateral deformation of the wall significantly reduces lateral surcharge pressures and wall bending moments determined from elastic theory. **The use of elastic (Boussinesq) lateral stress distributions to model a strip surcharge is not recommended unless the wall is rigid with no deformation.**

A *line load* of magnitude Q_L (kN per metre run of the wall) may be considered to exert an additional lateral force of P_n per metre run, given by

$$P_n = Q_L.\sqrt{K_a} \qquad (4.11)$$

The pressure distribution is as shown in Figure 4.7 (adapted from Williams and Waite, 1993).

Figure 4.7 *Pressure diagram for a line load*

A *point load* or a line load of limited extent may be converted to an equivalent line load, and a surcharge of limited area to an equivalent strip load, using a method presented by Williams and Waite (1993). This is shown in Figure 4.8.

$$Q_L = Q_C/(2A+L) \qquad (4.12)$$

where Q_C is the concentrated load (kN), Q_L the equivalent line load (kN/m), A is the distance from the wall at which the load acts, and L is its lateral extent (which in the case of a point load will be zero).

Figure 4.8 *Concentrated and line load surcharges (after Williams and Waite, 1993)*

Sloping ground

Earth pressure coefficients for use when the ground in front of, or behind, the wall is sloping are included in Appendix F ($\beta/\phi' \neq 0$).

A difficulty may arise when the slope behind, or in front of, the wall is uneven or non-uniform. In these conditions, one of the following approximate methods may be used:

- adopt a uniform design slope that approximates or envelopes the actual ground profile. The error associated with an approximation to the actual ground profile may be difficult to quantify, while the adoption of a uniform envelope may be unduly conservative
- carry out a succession of Coulomb wedge analyses to determine the active lateral thrust at a number of different depths down the wall. This is discussed in more detail in the context of earth berms in Appendix I, and can be a long and complex process
- model the effect of the slope as a series of surcharges and proceed as outlined in Figure 4.5. This method does not model the active thrust within the slope above the top of the wall; this should be calculated and added as an additional force applied near the top of the wall.

Interacting walls in close proximity

In some circumstances, for example on either side of a highway or railway, walls may be located relatively close to each other. Finite element analyses carried out by Arup (unpublished) show that, for rough walls separated by a distance less than the wall embedment in homogeneous isotropic soil, this can result in significantly enhanced values of passive pressure coefficient. The analyses assumed homogeneous isotropic soil with an internal angle of shearing resistance of ϕ' with no apparent cohesion and zero angle of dilation. The walls were not vertically restrained. Interaction effects were found to increase with the soil angle of shearing resistance, ϕ', but became insignificant when the separation exceeded the wall embedment (Figure 4.9). Frictionless walls were unaffected at any spacing.

Figure 4.9 *Enhancement factor on passive earth pressure coefficient for rough walls in close proximity*

Based on these results the passive earth pressure coefficient, K_p*, due to interaction between walls is given by:

$$K_p^* = \xi K_p \qquad (4.13)$$

where:

ξ = enhancement factor from Figure 4.9

K_p = *passive* earth pressure coefficient for an isolated wall (ie no interaction effects).

The enhancement in K_p values presented in Figure 4.9 applies only for the conditions modelled in the finite element analysis. Further work and research should be carried out to model a combination of different ground and groundwater conditions to enable similar enhancement factors to be derived for more general application.

Props just below top of wall

Many singly-propped walls are propped just below, rather than exactly at the top. Also, real props are often one metre or more in depth. In these conditions, the top of the wall may tend to rotate backwards into the retained soil, leading ultimately to the development of passive rather than active conditions. However, the soil behind the wall is still moving downward relative to the wall, rather than upward as is the case in a conventional passive zone. The resulting downward shear stress on the back of the wall will tend to reduce, rather than enhance, the passive earth pressure coefficient.

The assumption of normal active conditions behind the wall, both above and below the prop, may be appropriate in limit equilibrium calculations for the design of walls having one level of props at a depth of up to one third of the retained height below the top of the wall. These calculations do not allow for the effects of soil-structure interaction and tend to overestimate wall bending moments and underestimate prop loads (section 4.3).

In soil-structure interaction analyses of propped or anchored walls where stress redistribution (section 4.2.2) is likely, reference to the appropriate charts in Appendix F shows that the coefficient of passive pressure relevant to a situation of this sort is likely to be about 1.

4.1.8 Factors potentially increasing earth pressures in SLS conditions

The earth pressures that actually develop around an embedded retaining wall will depend on how the initial stress state of the soil is changed by wall installation and the subsequent sequence of excavation and support. In most cases, design for ultimate and serviceability limit states using the procedures set out in chapter 6 will be adequate, but there are some circumstances in which further consideration of the possibility of enhanced lateral stresses under SLS conditions is appropriate. These are discussed below.

Compaction pressures

As embedded walls in stiff soils retain predominantly natural ground, the development of high lateral stresses due to compaction of the backfill in layers is unlikely to be relevant for this type of wall. However, compaction pressures should be considered where made ground is compacted against embedded walls or where the ground level adjacent to the wall is raised after construction of the wall. Theoretical treatments of compaction stresses are given by Broms (1971), Carder *et* al (1977) and Ingold (1979) for coarse grained soils, and by Carder *et* al (1980), Symons *et al* (1989) and Clayton *et al* (1991) for fine grained soils. Useful summaries are provided by Powrie (1997) for both coarse grained and fine grained soils, and in Geoguide 1 (Hong Kong Government, 1994) for coarse grained soils.

Long term pressures on walls in overconsolidated deposits

Designers are sometimes concerned about the possibility of the in situ lateral stresses becoming re-established against the wall, for example due to creep.

Long-term field measurements behind embedded walls retaining London Clay at Walthamstow, Hackney, Reading and Malden generally indicate a slight reduction in the measured lateral stresses near the wall over a duration of up to eight years following construction (Carder and Darley, 1998). From this, **it seems that for walls embedded in stiff overconsolidated clay, the long term total lateral earth *pressure* remains largely unchanged from that at the end of the construction period.**

4.2 METHODS OF ANALYSIS

4.2.1 Introduction

See also

4.3…Effect of method of analysis on economy

6…...Design of wall

Modern codes of practice generally require the designer to check the adequacy of the retaining wall and its supports against an ultimate limit state (ULS), eg global collapse, and a serviceability limit state (SLS), eg cracking of the concrete. For a retaining wall, this distinction can be particularly significant: in many real situations the loads imposed by the soil on the wall at the ultimate (collapse) limit state are smaller than those under working conditions, as under working conditions the strength of the soil is unlikely to be fully mobilised. However, the factors of safety introduced into ULS calculations lead, in most cases, to more severe structural action effects than those calculated for SLS.

Limit equilibrium

Traditional *limit equilibrium* methods of calculation are based on conditions at collapse, when the full strength of the soil is mobilised uniformly around the retaining wall. A factor of safety is applied to one or more of the parameters involved in the calculation to give the design geometry eg depth of wall embedment. Limit equilibrium calculations are usually based on simple linear lateral stress distributions; in reality the lateral stress distributions are different. They are better developed and more directly applicable for some structural forms (eg unpropped cantilever walls) than others (eg multi-propped walls and walls propped significantly below the top).

As limit equilibrium calculations are based on the soil strength, they do not in themselves give any indication of wall movements. Also, the use of factored limit equilibrium stress distributions to calculate SLS bending moments may lead to an overconservative design. For these reasons, together with the general availability of powerful computers, most retaining wall design is now carried out with the aid of computer software that enables the interaction between the wall and the soil to be considered.

Soil-structure interaction

Subgrade reaction and pseudo-finite element methods

In the simplest soil-structure interaction analyses, the wall is modelled as a beam and the soil as a series of horizontal springs (subgrade reaction method) or as an elastic continuum (pseudo-finite element method). The soil stiffness is characterised fairly crudely by means of spring stiffnesses, a modulus of subgrade reaction or the stiffness of the elastic continuum. Spring stiffnesses increasing with depth may be specified, and maximum and minimum spring forces (corresponding to the passive and active limiting stresses) imposed. A beam on springs (subgrade reaction) and pseudo-finite element (elastic continuum) analysis will calculate wall movements, bending moments and prop loads, but not ground movements around the wall. Props are generally modelled as springs or point loads and there may be some difficulty in representing real support conditions, especially where moment restraint is provided.

Although actual construction sequences can be modelled, it should be stressed that these are approximate, not exact methods or solutions. Their relevance to reality depends on the appropriate selection of design input parameters. These should be calibrated against reliable field measurements of well monitored comparable excavations and wall systems (section 2.5.2). Even then, the inherent approximations and the relative simplicity of these methods mean that the results obtained can only be considered to be approximate.

Finite element and finite difference methods

More complex soil-structure interaction analyses model the soil as well as the wall and its construction sequence explicitly, using finite element or finite difference techniques. In a finite element or finite difference analysis, it is possible to model:

- complex soil constitutive behaviour
- actual construction sequences
- structural and support details
- consolidation and groundwater effects.

Ground movements as well as wall movements, bending moments and prop loads are calculated, but may be of limited value unless a well developed soil constitutive model has been used and the results "calibrated" against reliable measurements of well monitored comparable excavations and wall systems (section 2.5.2).

Finite element and finite difference methods are "theoretically complete" solutions, yet are still relatively simple in their modelling of ground behaviour. These methods require the user to have significant and specific experience of the particular software package being used and experience of modelling the ground conditions and construction sequence envisaged. It is unlikely that two users of the same software, modelling the same problem, will obtain identical results.

Selection of method of analysis

The appropriate method of analysis to use in any given circumstances will depend on factors such as the complexity of the structure and the construction process, the information needed from the calculation, the input data available and the potential economic benefit from refining the analysis. For example, if the wall depth is governed by cut-off requirements or if a sheet pile wall section is governed by considerations of driveability, there may be little benefit in carrying out complex computations.

Similarly, there is little benefit to be obtained by using complex numerical analysis to reduce material costs of walls where there is little or no soil-structure interaction (eg cantilever walls), see section 4.3.

The most widely used methods of analysis are summarised in Table 4.1. Although some appear to give a large amount of design information, the reliability of this depends on the quality and suitability of the input data. Some of the more advanced numerical modelling techniques (finite element and finite difference) can be time consuming to set up and require considerable input data and appropriate operator knowledge and experience, and are unlikely to be cost effective in the design of a straightforward retaining wall. It is sensible to carry out some simple calculations as a check on more advanced methods, (eg wherever possible it is prudent to carry out simple limit equilibrium calculations with appropriate simplifying assumptions to obtain a conservative bound prior to carrying out complex finite element or finite difference analyses). **It is generally better to use a simple analysis with appropriate soil parameters than a complex analysis with inappropriate soil parameters.**

4.2.2 Limit equilibrium analysis

See also

4.2.1…Introduction

6……..Design of wall

A brief description of the main attributes of limit equilibrium analysis is given in section 4.2.1.

Determination of the lateral stress distribution acting on an embedded wall for use in an effective or total stress limit equilibrium analysis typically involves the steps given in Box 4.2.

Cantilever walls

Unpropped embedded walls rely entirely for their stability on an adequate depth of embedment: they are not supported in any other way. They will tend to fail by rotation about a pivot point near the toe, above which active conditions are developed in the retained soil and passive conditions in the restraining soil. The idealised stress distribution at failure is shown, together with the corresponding bending moments and implied wall deflections, in Figure 4.10.

Figure 4.10 *Idealised stress distribution for an unpropped embedded cantilever wall at failure; (a) effective stresses; (b) pore water pressures; (c) wall bending moment distribution; (d) wall deflection*

Table 4.1 *Advantages and limitations of common methods of retaining wall analysis*

Type of analysis/software	Advantages	Limitations
Limit equilibrium eg STAWAL ReWaRD	• Needs only the soil strength • Simple and straightforward	• Does not model soil-structure interaction, wall flexibility and construction sequence. • Does not calculate deformations. Hand calculations of deformations possible by relating mobilised strength, soil shear strain and wall rotation (rarely done); or through empirical databases • Statically indeterminate systems (eg multi propped walls), non-uniform surcharges and berms require considerable idealisation • Can model only drained (effective stress) or undrained (total stress) conditions • Two dimensional only • Results take no account of pre-excavation stress state
Subgrade reaction/beam on springs eg M SOILS WALLAP	• Full soil-structure interaction analysis is possible, modelling construction sequence, etc • Soil modelled as a bed of elastic springs • Soil-structure interaction taken into account • Wall movements are calculated • Relatively straightforward • Results take account of pre-excavation stress state	• Idealisation of soil behaviour is likely to be crude • Subgrade moduli can be difficult to assess • Two dimensional only • Berms and certain structural connections are difficult to model • Global effects not modelled explicitly • Ground movements around wall are not calculated
Pseudo-finite element eg FREW WALLAP	• Full soil-structure interaction analysis is possible, modelling construction sequence, etc • Soil modelled as an elastic solid with soil stiffness matrices calculated using a finite element program	• Two dimensional only • Limited to linear elastic soil model, with active and passive limits • Basic representation of pore water response • Berms and certain structural connections are difficult to model

	• Soil-structure interaction taken into account • Wall movements are calculated • Relatively straightforward • Takes account of pre-excavation stress state	• Global effects not modelled explicitly • Ground movements around wall are not calculated
Finite element and Finite difference eg SAFE (2D FE) PLAXIS (2D and 3D FE) CRISP (2D and 3D FE) FLAC (2D and 3D FD) ABAQUS (3D FE) DYNA (3D FE)	• Full soil-structure interaction analysis is possible, modelling construction sequence, etc • Complex soil models can represent variation of stiffness with strain and anisotropy • Takes account of pre-excavation stress state • Can model complex wall and excavation geometry including structural and support details • Wall and ground movements are computed • Potentially good representation of pore water response • Can model consolidation as soil moves from undrained to drained conditions • Can carry out two dimensional or three dimensional analyses	• Can be time consuming to set up, and difficult to model certain aspects eg wall installation • Quality of results dependent on availability of appropriate stress strain models for the ground • Extensive high quality data (eg pre-excavation lateral stresses as well as soil stiffness and strength) needed to obtain most representative results • Simple (linear elastic) soil model may give unrealistic ground movements • Structural characterisation of many geotechnical finite element and finite difference packages may be crude • Significant software specific experience required by user

These conditions are known as *fixed earth support*, because the depth of embedment has to be large enough to prevent translation or rotation of the toe.

Given the retained height h and the soil angle of shearing resistance, ϕ', the depth of embedment required just to prevent collapse, d, of a cantilever wall can be determined from the depth of the pivot point (about which the wall can be imagined to rotate) below formation level, z_p. The equations of horizontal and moment equilibrium can be used to find these two unknowns, so the system is statically determinate.

If the linear approximation to the steady-state pore water pressure distribution is used (section 5.5.2), the two equilibrium equations are simultaneous and quartic in the two unknowns, and can be solved either directly or by adopting an iterative solution such as that outlined by Bolton and Powrie (1987).

The inconvenience of the iterative solution in the days before personal computers led to the development of an approximation to the exact calculation, in which the resultant of the stresses below the pivot point is replaced by a single point force Q acting at the pivot (Figure 4.11).

Figure 4.11 *Approximate stress analysis for unpropped walls; (a) effective stresses; (b) pore water pressures; (c) check that the added depth can mobilise at least the required force Q*

Box 4.2 *Steps involved in a typical limit equilibrium analysis*

1. Identify key depths on each side of the wall, ie the retained and excavated soil surfaces, groundwater levels, and interfaces between different soil strata.

2. Identify which zones of soil adjacent to the wall would be in the active and passive conditions at failure, and hence the depths at which the stress state at failure changes between active and passive.

3. At each key depth, calculate the overburden pressure, p_v, and for an effective stress analysis the pore water pressure, u. The overburden pressure is given by the depth z below the relevant free surface multiplied by the average unit weight of the soil, γ, plus any uniform surface surcharge q ($p_v = \int_o^z \gamma dz + q$). The pore water pressure should be calculated from a suitable seepage analysis, eg a flownet or the linear seepage approximation (section 5.5.2).

4e. For an *effective stress analysis*, select values of wall friction based on the considerations listed in section 4.1.4 and chapter 6 and evaluate the appropriate active and passive earth pressure coefficients from section 4.1.5 and Appendix F.

4t. For a *total stress analysis*, select the values of wall adhesion based on the considerations listed in section 4.1.4 and chapter 6.

5e. For an *effective stress analysis*, calculate the effective overburden pressure ($\int_o^z \gamma dz + q - u$) and the horizontal effective stress using Equation 4.5 (active) or 4.6 (passive), using appropriate values of K_a and K_p from step 4 above.

5t. For a *total stress analysis*, calculate the horizontal total stress using Equation 4.9 (active) or 4.10 (passive). In a *total stress analysis*, increase the total horizontal stress to a minimum of 5 z kPa (where z is the depth in metres below the soil surface) for a dry tension crack, or to $\gamma_w.z$ kPa for a water-filled tension crack. Further details of the treatment of tension cracks are given in sections 4.1.6, 5.5.1 and 5.9.1.

6. The wall is required to be in horizontal and moment equilibrium under the combined actions of the effective stresses and pore water pressures *(effective stress analysis)* or the total stresses *(total stress analysis)*, prestressed anchor forces and prop forces. From the two equations of horizontal and moment equilibrium, two unknowns can be determined, from which all remaining earth pressures and structural action effects should be derived. For an unpropped wall, these are the depth of embedment and the point near the bottom of the wall at which active and passive pressures interchange. For a propped or anchored wall, assuming a "free earth" stress distribution (section 4.2.2), the two unknowns are the wall depth and the prop or anchor force.

   For a propped or anchored wall assuming a "fixed earth" stress distribution (section 4.2.2), the designer is required to impose a further requirement or assumption. This could be the level at which the wall bending moment is zero or maximum, the depth of the wall, the prop or anchor force or the maximum bending moment. This is discussed further in section 4.2.2..

7. Check vertical equilibrium.

   Steps 1-6 above are normally carried out by computer. Step 7 should be carried out separately.

The portion of the wall below the pivot does not feature in the approximate analysis. The two unknowns are now the depth to the pivot z_p and the equivalent point force Q. Solution is simpler in this case, since moments can be taken about the pivot, eliminating Q from the moment equilibrium equation. The value obtained for z_p is multiplied by an empirical factor, historically 1.2, to arrive at the overall depth of embedment, d. This factor of 1.2 is nothing to do with distancing the wall from collapse (ie it is **not** a factor of safety), but is necessary because the calculation is approximate. If the simplified procedure is used, then a check should be carried out to ensure that the added depth is sufficient to mobilise at least the calculated value of Q (Figure 4.11(c)).

To determine the design depth of embedment, the calculation indicated in Figure 4.10 should be carried out with the appropriate factors of safety, including any surface surcharges and allowance for overdig (chapter 6). Bending moments and shear forces, either at limiting equilibrium or for the design embedment depth with the specified factors of safety and modifications to geometry and loading applied, may be calculated from the appropriate equilibrium pressure distribution.

Powrie (1996) and Bica and Clayton (1998) argue that the stress distribution illustrated in Figure 4.10 gives a realistic estimate of the geometry of an unpropped cantilever wall at collapse (ULS), allowing for likely uncertainties in the soil angle of shearing resistance ϕ' and the direction and magnitude of the soil/wall friction angle δ (Figure 4.12, adapted from Bica and Clayton, 1998, for walls in dry sand).

Figure 4.12 *Normalised depths of embedment at failure (after Bica and Clayton, 1998)*

Embedded walls propped at the top

If the possibility of a structural failure of the wall or excessive movement of the props is discounted, an embedded wall propped at the top can only fail by rotation about the position of the prop. A simple, equilibrium effective stress distribution at failure is shown in Figure 4.13(a), and pore water pressures according to the linear seepage model (section 5.5.2) are shown in Figure 4.13(b). The resulting bending moment diagram and implied wall movements are shown in Figure 4.13(c) and (d).

Figure 4.13 *Idealised stress distribution at failure for a stiff wall propped rigidly at the top; (a) effective stresses; (b) steady state pore water pressures for a wide excavation where the differential water head dissipates uniformly; (c) wall bending moment distribution; (d) wall deflection*

The conditions giving rise to the effective stress distribution shown in Figure 4.13(a) are known as *free earth support*, because no fixity is developed at the toe. In this case, the two unknowns are the prop force P and the depth of embedment, d, required just to prevent failure. The depth of embedment, d, can be calculated by taking moments about the prop, and P then follows from the condition of horizontal force equilibrium. To determine the design depth of embedment, the calculation indicated in Figure 4.13 must be carried out with the appropriate factors of safety, as discussed in section 6.3.5.

For a number of reasons, the earth pressure distribution illustrated in Figure 4.13 may be less representative of what actually happens at collapse (the ULS) than Figure 4.11 for unpropped walls. In particular, real props are

- of finite depth
- likely to be located a small distance below the top, so that the wall above prop level may rotate back into the retained soil

- likely to provide a kinematic restraint that may inhibit the development of fully active conditions in the immediate vicinity, and is in any case not taken into account in the derivation of the lateral earth pressure coefficients likely to be used in analysis (Bolton and Powrie, 1987).

For these reasons, there may be a local increase in the lateral stress in the vicinity of the prop (compared with Figure 4.13), and a decrease in the lateral stress below it. This *redistribution* of lateral stresses would result in an increase in prop load and a reduction in wall bending moments in comparison with those obtained using the simple linear lateral stress distribution shown in Figure 4.13. As a result, a reduction in wall depth might be possible. This is discussed further in section 4.3 and chapter 6.

Some authors (eg Williams and Waite, 1993; British Steel, 1997) describe the use of a "fixed earth support" calculation for a propped wall. The idealised and simplified effective stress distributions are shown, together with indicative wall bending moments and deflections, in Figure 4.14.

Figure 4.14 *Fixed earth support effective stress distributions and deformations for an embedded wall propped at the top; (a) idealised stresses; (b) simplified stresses; (c) wall bending moment distribution; (d) wall deflection*

This stress distribution might correspond to a mechanism of failure involving the formation of a plastic hinge at the point of maximum bending moment. The fixed earth support analysis is unlikely to be appropriate for strong walls in clay soils whose embedment depths are governed by considerations of lateral stability. For such walls, the embedment depth calculated assuming fixed earth support conditions for a propped wall will be greater than that in a free earth support analysis. The fixed earth support analysis represents a very conservative lower bound for wall toe depth. There may be other reasons why the embedment depth of the wall is taken deeper than that required to satisfy lateral stability, eg to provide an effective groundwater cut-off or for adequate vertical load bearing capacity. In such circumstances, fixed earth conditions provide a more realistic basis than Figure 4.13 for the estimation of lateral stresses.

In the absence of a plastic hinge (which would define the wall bending moment at this point), both the idealised and the simplified stress distributions shown in Figure 4.14 are statically indeterminate. To calculate the prop force and the depth of embedment, the designer is free to introduce a further requirement or simplification. Williams and Waite (1993) suggest assuming that the point of contraflexure (ie where the bending moment is zero) occurs at the level where the net pressure acting on the wall is zero (Figure 4.14).

The stress distribution shown in Figure 4.14 would correspond to the "correct" failure mechanism for a propped or anchored wall where the prop or anchor yields at a constant load which is just sufficient to prevent failure. Such a system is statically determinate, provided that the prop or anchor yield load is known.

Walls propped at formation level

Figure 4.15 *Stress analysis for an embedded wall propped at formation level; (a) division of soil into zones; (b) idealised effective stress distributions (from Powrie and Li, 1991)*

A wall propped rigidly at formation level might be expected to rotate into the excavation, leading to active conditions in the retained soil above the wall and in the soil remaining in front of the wall (zones 1 and 3 in Figure 4.15), and passive conditions behind the wall below formation level (zone 2). However, as the depth of wall embedment is increased, the sense of wall rotation given by a simple limit equilibrium analysis reverses and the calculated strength mobilisation factor begins to decrease with increasing embedment depth (Powrie, 1985). This will not happen in reality, and for walls of deeper embedment such an analysis is overconservative.

The presumption towards active conditions in zone 3 takes no account of the tendency of excavation of the soil from in front of the wall to bring the soil in this zone to passive failure. Powrie and Li (1991) suggest that the opposing effects of wall rotation and vertical unloading may be accommodated by setting the earth pressure coefficient in zone 3 to unity (Figure 4.15b). This procedure is shown to give bending moments reasonably in agreement with those measured in centrifuge model tests and calculated in finite element analyses.

In reality, a wall propped at formation level is often supported in a different way (eg by a higher level temporary prop and/or earth berms) while excavation to formation level is carried out. In these circumstances, a careful consideration of the stability of the wall at each stage during construction will be required. In view of this and the fact that a simple earth pressure distribution is difficult to establish for limit equilibrium calculations, **it is recommended that soil-structure interaction analysis should be undertaken for the design of these walls.**

Retaining walls with a stabilising base

In some circumstances, a wall with a stabilising base (ie a platform extending a short distance in front of the wall with a rigid connection at formation level) can represent a more economic solution than either a rigidly propped wall or an unpropped wall of deeper embedment (see eg St John *et al*, 1993). The stabilising base works because the contact pressure between it and the excavated soil surface:

- gives rise to a restoring moment on the retaining wall
- increases the passive pressures in the soil in front of the wall by acting as a surcharge.

The degree of wall movement needed to mobilise both of these effects may be minimised by the use of temporary props to support the wall until the stabilising base has been cast and gained strength.

On the basis of a comparison with finite element analyses and centrifuge model tests, Daly and Powrie (1999) recommend the use of the limit equilibrium stress analysis shown in Figure 4.16 for this type of wall.

Figure 4.16 *Forces acting on a stabilising base retaining wall*

The bearing pressure on the underside of the stabilising base is calculated using conventional bearing capacity theory with zero friction between the stabilising base and the underlying soil, and the same mobilised angle of friction as used to calculate the lateral stresses on the wall. However, Daly and Powrie (1999) note that the calculation lacks rigour, and should therefore be confirmed by a more detailed soil-structure interaction analysis.

Finite element analyses by Powrie and Chandler (1998) suggest an optimum stabilising base width of about half the retained height.

Retaining walls with a stress relieving platform

If some excavation and/or fill is needed on the retained side of the wall, there may be an advantage in constructing a stress relieving platform, attached rigidly to the wall stem some distance below the top and protruding horizontally into the retained soil (Tsagareli, 1967; St John *et al*, 1993). The relieving platform will reduce bending moments in the wall by (a) applying a reverse moment at platform level, due to the weight of the soil on top of it, and (b) reducing vertical stresses in the retained soil below platform level. It is straightforward to take both of these effects into account in a limit equilibrium analysis. However, for maximum efficiency the platform should extend far enough into the retained soil to reduce vertical stresses adjacent to the wall, and there may need to be a void below it.

Multi-propped walls

In the permanent condition, embedded retaining walls are often propped at more than one level. Examples of this include underground car parks, basements and cut-and-cover tunnels, which may be propped by reinforced concrete floor slabs.

Furthermore, a multi-propped wall is likely to act in different ways at different stages during its construction (eg as an unpropped cantilever, an embedded wall propped at or near its top, or as an embedded wall with more than one prop). In investigating the design bending moments and prop loads, it is necessary to consider each stage of construction separately, including stages during which the wall is supported by temporary props, to determine the largest load in each part of the structure. The cumulative effect of the incremental changes in lateral stress and wall movement that occur during each stage of construction should also be considered in detail in a rigorous analysis of the final condition, see chapter 6. **It is recommended that soil-structure interaction analysis should be undertaken for the design of multi-propped walls.**

A multi-propped wall is a statically indeterminate structure, but a limit equilibrium calculation that may be used as an approximate check of the required depth of embedment is presented in section 6.3.5.

King post walls

A king post wall comprises a series of vertical soldier piles (king posts) installed into the ground at intervals, which support a retaining wall made up of horizontal laggings (section 3.2.1, Table 3.7; and Appendix C, section C3). This is a potentially very economical form of construction, but the movements associated with it can be relatively large.

The horizontal laggings may comprise steel, concrete or, more commonly, timber. The earth pressure to be resisted by the king posts and lagging will depend upon the stiffness of the support system. The design should address all stages of the excavation and support installation to the wall.

The design of the king post wall should ensure satisfactory overall stability of the wall and the individual king posts should be designed to resist the calculated lateral loads (Figure 4.17).

Figure 4.17 *King post wall design*

Overall stability

For cantilever walls, ULS limit equilibrium stability calculations should be based on values of lateral stress and load per metre length determined from the assumption of fully active lateral stresses in the retained soil with the factors of safety enumerated in section 6.1 together with the assumptions stated in section 6.3. The depth of embedment, z, of the king posts should be measured below the level of the unplanned excavation (section 5.7).

Where anchorages or props are installed to support the king posts, the king post piles and the walings should be designed to accommodate the situation where an individual anchorage fails to carry its full design load.

Lateral loading of king posts

The king posts should be designed as piles in lateral loading, with an ultimate net effective resisting force P'_u per metre length of

$P'_u = K_p.b.p'_v/s$ at embedment depths $z \leq 1.5\ b$ (4.14)

and

$P'_u = K_p^2.b.p'_v/s$ at embedment depths $z \geq 1.5\ b$ (4.15)

Where b is the king post width, s is the spacing of the king posts (s>3b), K_p is the passive earth pressure coefficient (K_p>3) defined as (1+sinϕ') / (1-sin ϕ'), and p'_v is the effective overburden pressure at depth z (Fleming *et al*, 1994).

The above expressions for P'_u are based on the work of Barton (1982) as reported by Fleming *et al* (1994) and are considered applicable for K_p values of between 3.0 and 5.3 (ie $30° \leq \phi' \leq 43°$).

In a total stress analysis, the ultimate net lateral resisting force per metre length P_u is given by Fleming *et al* (1994) as:

$P_u = [2 + (7z/3b)].s_u.b/s$ at embedment depths $z \leq 3\ b$ (4.16)

and

$P_u = 9.s_u.b/s$ at embedment depths $z \geq 3\ b$ (4.17)

Where b is the king post width, s is the spacing of the king posts (s>3b), and s_u is the undrained shear strength at embedment depth z.

Mobilisation of soil strength with wall displacement

In a normally consolidated soil, the *in situ* lateral earth pressure coefficient K_o may be close to the active limit. In these conditions, the stresses in the soil behind the wall fall to their active values after only a small movement of the wall. In front of the wall, larger movements than are acceptable under working conditions may be required for the stresses to rise to the passive limit. In these cases, the wall may, under working conditions, be in equilibrium under the action of active pressures in the retained soil, and lower-than-passive pressures in the restraining soil in front of the wall (Terzaghi, 1943; Rowe, 1952). This is the reasoning behind the design approach given in the former UK code of practice for retaining walls, CP2 (1951), which involved the reduction of the passive earth pressures by a factor, F_p, see Appendix G.

In an overconsolidated clay, the *in situ* lateral effective stress is likely to be closer to the passive limit than the active owing to the geological stress history (Skempton, 1961; Burland, *et al*, 1979). Although in such conditions the *in situ* lateral stresses are likely to reduce slightly during wall installation, the contrast between the pre-excavation stress states of normally and overconsolidated clays has led to concern that the assumption of fully-active conditions in the retained soil may be inappropriate for an overconsolidated clay at deformations small enough to be acceptable in service. Finite element analyses carried out by Potts and Fourie (1984) indicate that the assumption of fully-active conditions in the retained soil might seriously underpredict lateral stresses in the retained soil, and hence bending moments and prop loads. However, bending moments measured in centrifuge model tests (Bolton and Powrie, 1988), and in the field (Tedd *et al*, 1984; Carder and Darley, 1998) do not seem to evidence this concern in practice. This apparent discrepancy may be a result of the neglect in the analyses of stress relief due to wall installation, and/or the underestimation of the stiffness of the soil in lateral unloading.

Pressure redistribution and arching

It is well established that local variations in wall movement and rotation can, for propped or anchored walls, lead to non-linearities in lateral stress distributions. This redistribution of stress away from the linear-with-depth variations assumed in simple limit equilibrium analyses can be exploited to reduce design bending moments and wall depth if a soil-structure interaction analysis is carried out. Stress redistribution may occur due to:

- the kinematic restraint imposed by the prop, both under working conditions and at collapse
- wall flexibility.

Figure 4.18 *Reduction of lateral stress in the retained soil due to arching onto a rigid prop*

Rowe (1952) investigated the first of these by means of a series of large-scale model tests on anchored sheet pile walls of various stiffness, retaining dry sand. He found that for rigid props (or, in the case of his model tests, unyielding tie-back anchors), the horizontal stress distribution on the retained side of the wall was non-linear, with load "arching" onto the relatively stiff prop (Figure 4.18). This enabled a reduction in lateral stress at the mid-section of the wall, leading to a reduction in wall bending moment. If the wall is propped just below the top, the increase in lateral stress in the vicinity of the prop is likely to be more pronounced owing to the tendency of the upper part of the wall to rotate back into the retained ground.

In Rowe's tests, an outward movement at the anchor point of less than $H/1000$ (where H is the overall wall height) was sufficient to generate fully-active conditions, and a linear variation in lateral stress with depth behind the wall. With passive deadman anchors of the type modelled by Rowe, it is probable that movements at the anchor point of this order would occur in practice. However, modern support systems are likely to be somewhat stiffer, increasing the likelihood of stress redistribution of the type indicated in Figure 4.18.

The flexibility of an embedded retaining wall may affect both deformations and bending moments. For a propped or anchored wall of given overall height H and flexural stiffness EI (where E is the Young's modulus and I is the second moment of cross-sectional area of the wall), bending effects are most significant when the wall is supported at the top. In general terms, wall deformation occurs partly due to rigid body rotation (in the case of a propped or anchored wall, about the position of the prop or anchor), and partly due to bending (Figure 4.19).

Figure 4.19 *Components of wall displacements and definition of a stiff wall*

Rowe (1952) found that the lateral stress distribution in front of the wall under working conditions depended on the relative importance of the bending component of wall deformation, and hence on the stiffness of the wall which he quantified by means of a flexibility $\rho = H^4/EI$, where H is the overall wall height and EI its flexural stiffness.

If the wall was stiff, so that the deflection at the level of the excavated soil surface was of the same order as the deflection at the toe, the stress distribution in front of the wall under working conditions was approximately triangular. Measured bending moments were in agreement with those from a limit equilibrium calculation based on a fully-active triangular stress distribution behind the wall and a smaller-than-passive (ie factored) triangular stress distribution in front (Figure 4.20(a)).

Figure 4.20 *Stress distributions behind and in front of (a) stiff and (b) flexible embedded walls (after Rowe, 1952)*

If the wall was flexible, so that the deflexion at excavation level was significantly greater than at the toe, the centroid of the stress distribution in front of the wall under working conditions was raised (Figure 4.20b). This led to smaller anchor loads and bending moments than those given by the factored limit equilibrium calculation.

Rowe (1955) presented a design chart, giving the percentage reduction in bending moment due to wall flexibility, compared with a limit equilibrium calculation based on active pressures behind the wall and lower-than-passive pressures in front. Calculations using Rowe's approach were used successfully in the back analysis of an anchored sheet pile retaining wall on the A1(M) at Hatfield (Symons *et al*, 1987). However, the chart is not reproduced here as its general applicability is restricted because:

- Rowe's non-dimensional stiffness $m\rho$ involves an unusual definition of soil stiffness m, such that the modulus of subgrade reaction at a depth x for a wall of embedment d is given by mx/d. The soil stiffness parameter m, like the modulus of subgrade reaction, is non-fundamental and difficult to determine
- Rowe normalized his results with respect to the bending moments calculated in a limit equilibrium calculation with active pressures behind the wall and lower-than-passive pressures in front. For sheet pile walls in sand, in which the pre-excavation lateral earth pressure coefficient is low, this represents a reasonable upper bound. However, in an overconsolidated clay deposit in which the pre-excavation lateral earth pressures are likely to be relatively high, bending moments in a truly rigid wall may be greater than a calculation based on linear, active pressures behind the wall would suggest (Potts and Fourie, 1985). Thus the validity of the analysis based on fully active pressures in the retained soil as a benchmark for Rowe's chart probably depends on the initial lateral stresses being low
- Rowe's design chart is unlikely to cover walls of sufficient embedment for use in soils with low angles of shearing resistance and/or where the groundwater level is high
- modern propping systems tend to be more rigid than the deadman anchors used by Rowe, and may attract an increased load due to the kinematic restraints they impose.

A modern interpretation of Rowe's work is given by Potts and Bond (1994).

Some European codes allow for stress redistribution due to the kinematic restraint of the prop by means of rectangular, rather than triangular, stress distributions. This is discussed by Simpson and Powrie (2001), but is a practice that has not traditionally been followed in the UK.

In general, a designer wishing to take account of soil-structure interaction effects to achieve economies in design by taking account of stress redistribution should carry out a soil-structure interaction analysis as described in section 4.2.3.

4.2.3 Soil-structure interaction analysis

See also

4.1.2...Effect of wall installation

5.......Determination and selection of parameters for use in design calculations

6.......Design of wall

7.......Design of support systems

The main advantage of using something other than a limit equilibrium analysis is the ability to model soil-structure interaction and to predict wall defections and surrounding ground movements at least approximately by considering some or all of the following:

- soil conditions and behaviour (eg variation of stiffness with strain, effective stress and stress path; anisotropy; in situ earth pressures; consolidation from undrained to drained conditions)
- the wall and its support system
- the sequence of construction.

There is no point in using numerical analysis unless the level of detail to which results are obtained is really needed and appropriate input data are available. If a numerical analysis is to be carried out, the user of a particular package must understand the principles of the method and the data input requirements sufficiently to interpret and appreciate the limitations of the output. The main points that should be considered are summarised below.

Soil conditions and behaviour

- Is the in situ stress state known or can it be reliably estimated? How much will it affect the results required from the analysis?
- Will wall installation be modelled explicitly (unlikely), or will an empirical adjustment to the in situ stress state be made to account for its effects (sections 4.1.2 and 5.4.3)?
- Will the soil model include
  - a transition from elastic to plastic behaviour (essential)?
  - consolidation effects?
  - strain-dependent stiffness moduli?
  - stress path dependent moduli?
  - anisotropy?
- Are the required parameters available?
- How will the groundwater conditions (including any temporary dewatering) be modelled?
- How will the transition from short term (undrained) to long term (drained) conditions be handled?
- How will three dimensional geometrical effects (eg corners) be modelled?

- What other loads (eg from nearby structures, or for a loadbearing wall) need to be included and when?

Soil strength and stiffness must be appropriate for the range of stress and strain expected to apply during the analysis, especially with simple (linear elastic-perfectly plastic) models.

Wall support and sequence of construction

The wall will interact with its support system. In a typical cut and cover structure, the wall will interact with the base slab, the roof slab and intermediate props and slabs for both vertical and lateral stability. In finite element and finite difference analytical models, these effects are considered and analysed as part of the analysis. With subgrade reaction and pseudo-finite element techniques, appropriate assumptions in the input data and boundary conditions should be made to allow for these effects, eg the application of fixed end bending moments and rotational stiffness at slab/wall connections. Similarly, the effects of an earth berm will need to be modelled by the application of appropriate lateral stresses in a subgrade reaction or pseudo-finite element analysis (section 7.2).

In finite element, finite difference, subgrade reaction and pseudo-finite element analyses, the effects of construction sequence, which may result in the wall being supported in different ways at different excavation depths, can be modelled explicitly. In subgrade reaction or pseudo-finite element models, the magnitude of fixed end moments and rotational stiffness at the slab/wall and prop/wall interfaces should be compatible with the structural connection detail being modelled. In finite element and finite difference analyses, consideration should be given to the structural performance of prop/wall and slab/wall connections (eg pinned, butted or fully fixed).

Wall flexural stiffness

Appropriate values of the flexural stiffness of the wall (*EI*, where *E* is the Young's modulus and *I* is the second moment of cross sectional area) should be used at each stage of the analysis to model wall stiffness during construction and in the long term. The calculated load effects and wall deflection will depend upon the magnitude of the wall flexural stiffness adopted in analysis. The value of *EI* assumed should be appropriate for each construction stage and in the long term. For reinforced concrete walls, this should allow for the effects of cracking (due to wall flexure) and concrete creep and relaxation.

In subgrade reaction and pseudo-finite element analyses, it is necessary to input explicitly the wall flexural stiffness *EI* at each construction stage and in the long term. These should be determined as indicated in Table 4.2.

Reinforced concrete walls

For concrete walls, the value of *EI* should strictly be determined for the reinforced section. However, the approximation indicated in Table 4.2 is commonly adopted and has been found to be appropriate in conjunction with the design procedures presented in chapter 6.

For a reinforced concrete section, the value of *EI* changes over time, with creep and relaxation causing a ~50% reduction from the short-term uncracked value in the long term. The flexural stiffness EI of a concrete wall should therefore be calculated at each construction stage and in the long term.

As a rule of thumb, it is often appropriate to adopt 0.7E_oI and 0.5E_oI during the construction and long term stages respectively, where E_o is the uncracked short term Young's modulus of concrete (typically, E_o = 28 MPa) and I is the second moment of area of the reinforced concrete section as defined in Table 4.2.

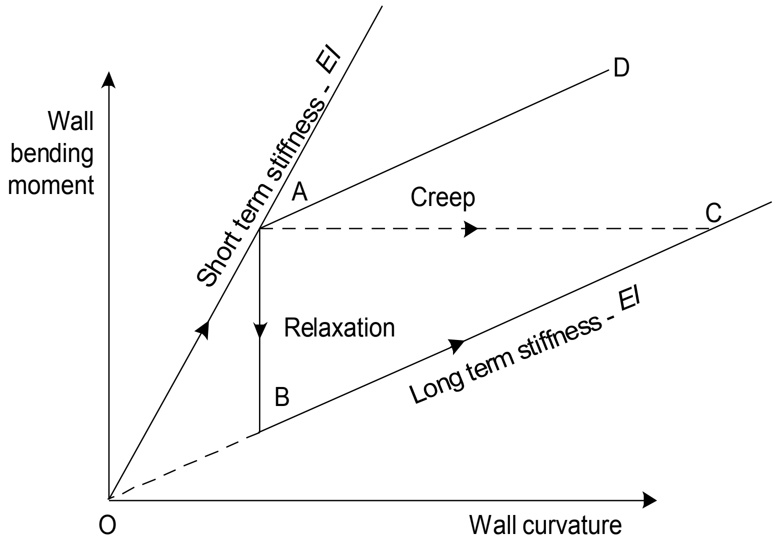
Table 4.2 *Values of Young's modulus E and second moment of cross sectional area I for various embedded retaining wall types*

Wall type	**Second moment of cross sectional area *I***	**Young's modulus *E***
Reinforced concrete diaphragm walls	$I = d^3/12$ m^4/m run, where d is the wall thickness in metres.	E = Young's modulus of concrete, making due allowance for creep and relaxation as discussed below
Reinforced concrete bored pile walls	$I = \pi D^4/64s$ m^4/m run, where D is the pile diameter in metres, and s is the spacing between piles for contiguous bored pile walls and hard/hard secant bored pile walls, or between hard piles for hard/soft and hard/firm secant bored pile walls.	E = Young's modulus of concrete, making due allowance for creep and relaxation
Steel sheet pile walls	I = second moment of area of sheet pile section	E = Young's modulus of steel comprising sheet pile section

The way in which the reduction in EI is applied in the analysis should be considered carefully: Box 4.3 shows how this should be done in a soil-structure interaction analysis. This approach is required in most available computer programs in which stiffness represents response to load increments only. The same approach may be used to model corrosion of steel sheet piles, in which I reduces with time.

Box 4.3 *Changing wall EI to allow for cracking, creep and relaxation of concrete*

The following shows the type of bending moment-curvature curve required for a change from short-term to long-term stiffness.



The high short-term stiffness on OA is required to drop to the lower long-term stiffness on line OBC. Consider an element of structure which in the short-term has been stressed to Point A. In the course of time, its state will move to be somewhere on line BC. If it is in a situation in which there is no change of strain during this change, stresses will simply relax and it will move to point B. If, on the other hand, the load on the element cannot change, it will creep and move to point C.

If an element is at point A and the only change made is to change the Young's modulus in the data, further behaviour will proceed along line AD. This does not represent creep or relaxation. The soil-structure interaction analysis should ensure that even if nothing moves, stresses will change from point A to point B. If these new stresses are no longer in equilibrium, the analysis should then indicate further strains such that the stress state will move up line BC.

Steel sheet pile walls

Values of *I* for steel Larssen (U-profile), Frodingham (Z-profile), box and high modulus piles are given in the British Steel (1997) *Piling Handbook* for the range of sections supplied by Corus.

The development of full section modulus in a sheet pile wall is based on the assumption that any two adjacent flanges are able to work together in bending.

Walls comprising Z-profile steel piles, which have their interlocks in the flanges (section C1, Appendix C), **develop the full section modulus of the combined wall (BS 8002, 1994).** It should be noted that with Z-profile piles, the effective section modulus will be reduced if the piles are allowed to rotate about a vertical axis during driving: as a rough guide, 5° of rotation will result in a 15% reduction in the combined section modulus (Williams and Waite, 1993). The designer should therefore ensure that construction tolerances compatible with the design assumptions are specified in this respect.

For walls made up of U-profile steel sheet piles, the connecting section incorporates an interlock which is located on the centre line or neutral axis of the wall. If the two piles are able to displace relative to one another along the interlock, then the full modulus of the combined sections will not be realised. These piles rely on the transfer of longitudinal shear stress between adjacent piles (by friction at the interlocks or clutches) to develop the full modulus of the combined section. It is likely that shear will be generated by surface irregularities, rusting, lack of initial straightness and soil particle migration into the interlocks during driving (Williams and Waite, 1993). **For walls made up of U-profile sheet piles, it is common for designers to assume the full combined modulus, except in circumstances where shear transfer may not be fully effective**, for example:

- piles forming cantilever walls
- piles cantilevering a significant distance above or below walings
- piles driven into and supporting silts and/or soft clay
- piles retaining free water over a part of their length
- piles which are prevented (eg by rock or obstructions) from penetrating to their required toe level.

In the above circumstances, it is common for designers to ensure that the U-profile sheet pile sections are connected together (by welding, pressing or other means) to ensure that the necessary shear resistance can develop to the extent that the full combined section modulus can be relied upon in design. Where piles are not connected together as described above, for use in the above applications, Williams and Waite (1993) report that the friction between the interlocks probably results in the development of at least 40% of the full section modulus.

Little is known about the effect of clutch slippage in sheet pile walls; significant further research is required in this area to improve understanding.

Axial stiffness of supports

In subgrade reaction and pseudo-finite element analyses, it is necessary to input explicitly the axial stiffness of any temporary or permanent props. The axial stiffness k (in kN/m per m run) of a prop should be calculated as follows:

$$k = AE\cos^2\alpha/Ls \qquad (4.17)$$

where E = Young's modulus of the material comprising the prop

A = cross-sectional area of the prop

L = effective length of the prop (typically the half-width of the excavation that the prop spans)

s = prop spacing

α = angle of inclination of the prop from the horizontal

If concrete slabs are used to support the wall (eg in a top down construction sequence), the calculated axial stiffness of the slab should be reduced to allow for any openings. For concrete slabs and props, the Young's modulus should be reduced to allow for the effects of creep and relaxation as described above for concrete walls.

4.3 EFFECT OF METHOD OF ANALYSIS

Appendix J compares the results obtained from the analysis of four generic retaining wall problems using the following commercially available software:

- limit equilibrium methods — STAWAL, ReWaRD
- subgrade reaction and pseudo-finite element methods — FREW, WALLAP
- finite element and finite difference methods — SAFE, FLAC

The problems analysed are defined in Figures J1 to J4. The assumptions made in the calculations and the results obtained are discussed in detail in Appendix J.

Figure 4.21 *Comparison of types of analyses: effective stress*

Figure 4.22 *Comparison of types of analyses: total stress*

Results are summarised in Figures 4.21 and 4.22. The main conclusions from this work are:

- in circumstances where there is little or no stress redistribution, eg cantilever walls, simple limit equilibrium calculations and soil-structure interaction analyses (subgrade reaction or pseudo-finite element methods and finite element or finite difference methods) are likely to give similar wall embedment depth and wall bending moments
- for propped or anchored walls where stress redistribution will occur, design by limit equilibrium calculations will result in longer walls with higher calculated wall bending moments compared to those obtained from soil-structure interaction analyses. Use of soil-structure interaction analyses may result in significant savings in wall material costs, depending upon project and site specific details
- where stress redistribution occurs, prop or anchor loads calculated from limit equilibrium methods will be smaller than those obtained from soil-structure interaction analyses. Prop or anchor loads solely obtained from limit equilibrium calculations may be significantly underestimated and should be treated with caution in design (section 7.1.3)
- in situations where the calculated prop loads are significantly different from those derived from experience of comparable construction (eg from the DPL method, see section 7.1.3), the designer should carefully investigate and understand the reasons for the calculated values. This will typically involve a detailed review of the assumptions made in the calculations and the carrying out of sensitivity analyses. The outcome of such investigations should enable the designer to adopt appropriate design values
- for walls embedded in soils where the total horizontal pressures near the base of the wall on the retained side are similar in magnitude to those on the restraining side, the results of calculations will be very sensitive to relatively small changes in pressures around the wall. The results of such calculations will also be influenced by node spacings in beam spring and pseudo-finite element models and mesh details in finite element and finite difference models. The designer should carry out sensitivity checks on the effects of such variations in the models adopted in the calculations.

4.4 KEY POINTS AND RECOMMENDATIONS

1. The principles of soil mechanics relevant to retaining wall design calculations are summarised in Appendix D.

2. Soil strength can be peak, critical state or residual. It is important to specify which of these a particular value of strength represents, and also to understand the limitations of the method of measurement. This is discussed in section 5.4.4.

3. Retaining wall analysis should generally be carried out in terms of effective stresses, using the angle of shearing resistance ϕ'. A total stress analysis using the undrained shear strength s_u may be used, but this is only valid in the short term while there is no significant dissipation of the negative pore water pressures induced on excavation. This is discussed in chapter 5.

4. Soil stiffness varies with strain, effective stress and overconsolidation ratio. This is discussed in section 5.4.5.

5. In an effective stress limit equilibrium analysis, lateral earth pressures are usually calculated from earth pressure coefficients giving the ratio of horizontal effective stress to the effective overburden pressure, p_v'. It is necessary to consider carefully the relative soil/wall movement (both horizontally and vertically) and the vertical equilibrium of the wall in assessing the shear stresses on the wall and selecting appropriate values of earth pressure coefficients. Appendix F provides equations and charts for the determination of earth pressure coefficients.

6. For walls subject to vertical applied loading, the magnitude and direction of the friction or adhesion assumed at the soil/wall interface should be appropriate for each construction stage and in the long term. For the particular circumstances of walls which support very large vertical loads at maximum excavation stage and which can settle relative to the soil under load, it is generally prudent to assume the limiting values for wall friction and adhesion given in section 6.3.1 over the embedded portion of the wall and zero friction or adhesion on the retained side above excavation level.

7. In total stress analysis, the following minimum total horizontal stress should be assumed on the retained side of the wall:

   - Where water is not expected

     MEFP = 5 z kPa or total stress calculated from Equation 4.9, whichever is greater (Figure 4.5)

   - flooded tension cracks

     hydrostatic pressure = $\gamma_w z$ kPa to the depth where the total stress calculated from Equation 4.9 exceeds this value (Figure 4.5).

   Where water is not expected, the total pressure acting on the retained side of the wall at any depth below the ground surface should not be less than 5 z kPa.

   For an embedded wall which is propped or anchored near its top, provided that a lateral stress greater than the hydrostatic pressure of water can be demonstrated over a minimum depth of one metre near the top of the wall on the retained side, the possibility of a flooded tension crack developing behind a propped wall in a uniform homogeneous isotropic stratum of clay may be discounted. The possibility of water entering a tension crack, for example through a sand parting or other more permeable horizon in the ground or through preferential drainage paths which may have developed during wall installation should be carefully considered.

8. Wall installation might cause a 10% reduction in the in situ lateral earth pressure coefficient for bored pile walls and 20% for diaphragm walls installed in overconsolidated clays, but it is difficult to give general guidance. Appendix E summarises the field data and finite element analysis reported in the literature regarding wall installation effects. With simple elastic soil-structure interaction analysis (where the pre-failure deformation of the soil is assumed to be linear), a pre-excavation lateral earth pressure coefficient of unity is likely to give reasonably realistic bending moments and prop loads for walls embedded in stiff overconsolidated fine grained soils.

9. For embedded walls in stiff clay, the long term earth *pressures* seem to remain largely unchanged from those at the end of the construction period.

10. It is generally better to use simple analysis with appropriate soil parameters than a complex analysis with inappropriate soil parameters.

11. Limit equilibrium analysis can be used with confidence to calculate the ultimate limit state (ULS) of unpropped embedded cantilever retaining walls. Walls propped or anchored at the top offer opportunities for stress redistribution away from the simple linear increase with depth assumed in a limit equilibrium calculation. In such circumstances, a shorter wall, smaller calculated wall bending moments and greater calculated prop or anchor loads will be obtained from the results of soil-structure interaction analysis compared with limit equilibrium calculations. Prop or anchor loads solely obtained from limit equilibrium calculations may be significantly underestimated and should be treated with caution in design (section 7.1.3). Limit equilibrium analysis can be used for the design of embedded walls where simple appropriate earth pressure distributions can be reliably established with confidence for use in such calculations. For wall types where this is not possible (eg walls which are singly propped at low level and multi-propped walls), limit equilibrium calculations are not recommended as the sole basis for the design of such walls and soil-structure interaction analysis should be carried out.

12. Stress distributions under working conditions are likely to differ significantly from ULS limit equilibrium distributions for walls other than unpropped embedded cantilevers. In these circumstances, a soil-structure interaction analysis for serviceability limit state (SLS) calculations is likely to be advantageous, provided that the required soil, structural and geometrical parameters can be determined with sufficient reliability and confidence. Specific guidance on the selection of structural properties for walls and props is given in section 4.2.3.

13. It is sensible to carry out some simple calculations as an approximate check on more advanced soil-structure interaction methods (eg wherever possible it is prudent to carry out simple limit equilibrium calculations with appropriate simplifying assumptions to obtain a conservative bound prior to carrying out complex finite element or finite difference analyses).

14. The value of wall *EI* assumed should be appropriate for each construction stage and in the long term.

For reinforced concrete walls, as a rule of thumb, it is often appropriate to adopt $0.7E_oI$ and $0.5E_oI$ during the construction and long term stages respectively, where E_o is the uncracked short term Young's modulus of concrete (typically $E_o = 28$ MPa) and I is the second moment of area of reinforced concrete section as defined in Table 4.2.

For steel sheet pile walls, the full section modulus of the combined wall can be assumed for Frodingham (Z-profile) piles, but with due allowance made for a reduction in the section modulus due to rotation about a vertical axis during driving: 15% reduction in the combined section modulus for 5° rotation.

For Larssen (U-profile) piles, the full section modulus of the combined wall can be assumed, except in circumstances where shear transfer may not be fully effective. For example where piles:

- form cantilever walls
- cantilever a significant distance above or below walings
- are driven into and support silts and/or soft clay
- retaining free water over a part of their length
- are prevented (eg by rock or obstructions) from penetrating to their required toe level.

For use in the above applications, piles should be connected together (by welding, pressing or other means) to ensure that the necessary shear resistance can develop to the extent that the full combined section modulus can be relied upon in design. Where piles are not connected together as described above, for use in the above applications, the designer should assume 40% of the full section modulus in design.

5 Determination and selection of parameters for use in design calculations

This chapter is intended for the geotechnical designer: consulting engineers, contractors and those involved in the design of temporary and permanent works. It assumes that the reader has some experience and understanding of the principles of engineering design and the requirements of input parameters for the associated analyses.

Figure 5.1 *Determination and selection of parameters for use in design calculations*

This chapter provides guidance on the determination and selection of parameters for use in design calculations and analyses. Figure 5.1 outlines the required process. The chapter provides guidance on:

- site investigation requirements
- the determination of ground stratigraphy, fabric and permeability and the assessment of drained or undrained ground behaviour
- the determination of the soil parameters relevant to retaining wall design
- the determination of groundwater pressure
- typical load cases
- circumstances where provision should be made for unplanned excavation of formation
- the selection of appropriate parameters for use in design and analysis with regard to temporary works and permanent works design.

The appropriate determination and selection of parameters for use in design calculations can lead to economies in wall materials and construction.

5.1 DESIGN PARAMETERS

See also

5.2.....Investigation of ground and groundwater conditions

5.3.....Assessment of drained/undrained soil conditions

5.4.....Determination of soil parameters

5.5.....Determination of groundwater pressures

Design parameters relate to:

- soil and groundwater conditions
- loading conditions, eg surcharges adjacent to the wall
- geometry, eg unplanned excavation allowance
- factors to ensure safety, to allow for uncertainty in material and soil properties, loading and calculation models and to ensure acceptable deformations.

The selection of parameters for use in design calculations should be based on careful assessment of the range of values determined for each parameter which might govern the performance of the retaining wall during construction and during its design life. The designer should ensure that he understands the input parameters required for the envisaged design calculations and specifies an appropriate site investigation to obtain them. In practice, site investigation is often done

5.6…..Loads
5.7…..Unplanned excavation of foundation
5.8…..Factors
5.9…..Selection of parameters

before the designer is appointed. In this eventuality, the designer should specify any additional site investigation required to achieve an economic design.

Table 5.1 lists the soil parameters which are typically required in relation to common methods of calculation adopted in retaining wall design.

Table 5.1 *Soil parameters required for various calculations / analysis methods*

Calculation method	Soil parameters						
	Bulk unit weight	In situ earth pressure coefficient	Soil shear strength				Soil stiffness parameters
			Ultimate limit state		Serviceability limit state		
	γ_b	K_o	Total stress s_u	Effective stress c', ϕ'	Total stress s_u	Effective stress c', ϕ'	
Limit equilibrium	√	×	√	√	√	√	×
Subgrade reaction / pseudo-finite element	√	√	√	√	√	√	√
Finite element / finite difference							
• elasto-plastic, Mohr-Coulomb models	√	√	√	√	√	√	√
• Non-linear stiffness models	√	√	(1)	(1)	(1)	(1)	(1)

Notes

(1) Special input parameters required depending upon analytical model adopted

Knowledge of the soil density (unit weight) and shear strength is *essential* in the design of an embedded retaining wall. In addition, the designer should also have a general appreciation of the following soil properties:

- classification and index properties, eg particle size distribution, moisture content, plasticity indices (for fine grained soils)
- soil permeability.

Knowledge of in situ stress conditions, particularly the value of the in situ earth pressure coefficient, K_o, and soil stiffness is essential in soil-structure interaction analyses.

For stiff overconsolidated soils, there are several different soil strengths; peak, critical state, residual and, drained or undrained. There is also a range of soil stiffnesses, depending on shear strain. For made ground and backfill materials, parameters for the determination of compaction and swelling pressures may be required. **In determining and selecting soil parameters, the first step is to decide which are appropriate for a particular analysis. Only then can the designer start to consider other issues such as reliability, selection of values for design and factors of safety.**

It may be appropriate to adopt different selected values for a parameter in different limit states and design situations. For example, in total stress analysis, the selected value of the undrained shear strength of the clay should consider the mechanisms or modes of deformation being considered for the wall. Different strengths will be required for a shear failure in fissured material depending upon whether the shear surface is free to follow the fissures or is constrained to intersect intact material. A range of values should be considered. These values should also allow for any softening due to potential changes in moisture content and the effect of excavation disturbance.

Many soil parameters are not true constants but depend upon factors such as stress and strain levels, mode of deformation, type of analysis, etc. Under working conditions while deformations are comparatively small, some or all of the soil will operate at below peak strength conditions. Under ultimate limit state conditions where deformations are comparatively large, the soil will operate beyond peak strength conditions and may dilate to approach critical state values (BS 8002, 1994).

The designer of an embedded wall in a stiff overconsolidated soil should decide which is the appropriate strength to use in particular circumstances. The residual strength might be appropriate where sliding along a pre-existing polished rupture surface represents a potential failure mechanism, but will in general be far too conservative in other situations. The choice is therefore usually between the peak and the critical state strength. In choosing which to adopt, the following points should be borne in mind:

- for a given soil, the critical state angle of shearing resistance, ϕ'_{crit}, is a constant over the range of stresses normally encountered in geotechnical engineering. Conversely, the development of a peak angle of shearing resistance, ϕ'_{peak}, depends on soil-structure and on the potential for dilation. The latter of these depends in turn on the soil density and the average effective stress during shear
- failure at the peak angle of shearing resistance is brittle. With continued post-peak deformation the soil strain softens, leading to the possibility of progressive failure. The factor of safety adopted in design should therefore ensure that displacements and strains will not be large enough to take the material into the post-peak range
- the onset of large deformations tends to occur when about 80% of the peak strength is mobilised. This applies to a wide range of soils
- in an overconsolidated soil which fails by rupture, the peak strength is easier to identify than the critical state
- at a given effective stress, denser soils (of a particular type) have both a higher stiffness and a higher peak strength. This is particularly relevant when retaining walls are designed by the application of a factor of safety to the soil strength. If critical state strengths are used in the collapse calculation, a higher factor of safety would be needed for a retaining wall in a loose soil than for an identical retaining wall in a dense soil, for the wall movements under working conditions to be the same.

5.2 INVESTIGATION OF GROUND AND GROUNDWATER CONDITIONS

A comprehensive treatment of ground investigation, laboratory and field testing is beyond the scope of this report.

5.2.1 Site Investigation

See also

2.2.2....Geotechnical categorisation of retaining walls

A site investigation appropriate to the geotechnical category of the retaining wall should be carried out. The ground conditions which may influence the decision about the geotechnical category should be determined as early as possible. For geotechnical category 2 walls, guidance on carrying out site investigations is provided in BS 1377 (1990). Special techniques may be required for geotechnical category 3 walls.

Category 1 walls

For category 1 walls, the minimum requirement is that all design assumptions should be verified at the latest during the supervision of the works (EC7, 1995).

Category 2 walls

For category 2 walls, the investigation should comprise:

- a desk study
- a site specific ground investigation.

Guidance on what should be considered in the desk study is provided in BS 5930 (1999), BS 8002 (1994), EC7 (1995) and Perry and West (1996). The reader is referred to these documents for more details. However, as a minimum, the following should be carefully considered:

- field reconnaissance including the examination of aerial photographs, neighbouring structures, services and excavations
- site and regional topography and geomorphology
- local and regional hydrology and hydrogeology and how these are likely to be affected by the proposed wall
- inspection of geological maps and memoirs
- inspection of historical maps charting previous developments at and around the site
- previous site investigations and construction experience at the site and its environs
- regional seismicity, if appropriate.

It is very important that the desk study is properly carried out as it identifies the hazards that will be investigated in the ground investigation. An inadequate desk study often leads to an inadequate ground investigation which increases the risks of time consuming and costly ground-related problems arising during construction.

The desk study should also address specific issues relevant to the form of retaining wall and associated support system likely to be adopted at the site. For example, issues such as satisfactory access to the site for plant, space for equipment for the desanding and re-circulation of support fluid, environmental aspects of the use of support fluid and noise and vibrations from sheet piling, licences for anchors, details of adjacent and nearby existing construction and foundations.

The number of ground investigation exploration points should be adequate to establish the ground conditions along the length of the wall and to ascertain the

variability in those conditions. The spacings between the exploration points and their depths will be site and project specific. **Exploration points should be spaced generally at 10 m to 50 m along the length of the wall and should extend to a depth which is at least 3 times the proposed retained height. Furthermore, if ground anchorages are proposed, the investigation should be of sufficient extent and depth to provide data for strata in which the anchorages will attain their bond lengths.** Category 2 walls require that a routine investigation is carried out comprising borings, in situ and laboratory tests. Where static cone penetration testing and / or other indirect methods are used, it is necessary to carry out borings to identify the ground conditions in which these methods are used. If the geological features of the site are well known, such borings may be omitted (EC7, 1995). Clayton *et al* (1993) review methods of ground investigation, in situ and laboratory testing for particular applications.

An important requirement in ground investigation is to establish the groundwater conditions at the site. This should include an investigation of the pore water pressure distribution including:

- observations of water levels in borings and piezometers and their fluctuations with time, preferably over the winter and spring months
- an assessment of the hydrogeology of the site, including investigation of any non-hydrostatic piezometric profiles and tidal variations. This should also include consideration of the likelihood of the long term rise or fall in groundwater levels in aquifers underlying the site
- observations of extreme levels of any free water which may influence the groundwater pressures.

Category 3 walls

This involves an investigation which is *additional* to that required for a category 2 wall and will normally involve highly specialised procedures, eg to provide appropriate input to complex numerical analyses. These may involve rotary coring, in situ determination of K_o, and the determination of the soil's non-linear stiffness-strain relationship using in situ pressuremeter testing and special laboratory testing techniques, eg bender element testing, local strain gauges attached to triaxial samples.

See also

2.1.4….Contaminated land
3.3.4….Contaminated ground

For sites where chemical contamination is suspected, the information necessary to characterise the contamination status should comprise:

- details of the historical setting of the site and the potential for the presence of contaminants
- identification of what may be affected by the contaminants, ie *receptors* (eg a retaining wall)
- information on the *pathways* by which contaminants could migrate or come into contact with receptors (including details of any physical characteristics of the site that may affect contaminant movement).

The results of the above should define all known aspects of the site that could impinge upon or affect the contaminant - pathway - receptor scenario and should be used to focus subsequent investigations. Further discussion regarding the strategy and implementation of such investigations is beyond the scope of this report; further details are provided in BS 10175 (2001).

The requirements for category 3 walls are very specific and specialist advice should be sought to ensure that the particular circumstances are adequately dealt with.

Procurement of site investigation

Guidance on methods of procurement is provided in SISG (1993). **Irrespective of the method of procurement of the site investigation, the designer of a category 2 wall should check that an adequate desk study and ground investigation has been carried out.** SISG (1993) sets out a decision making process which should be followed during a site investigation. This is reproduced in Figure 5.2 and shows that the client's Principal Technical Advisor (PTA) should appoint a Geotechnical Advisor (GA) to advise on the scope of the required ground investigation and its interpretation.

Figure 5.2 *Decision making process in site investigation (after SISG, 1993)*

It is bad practice to invite competition for the scope of the ground investigation as this rewards the smallest proposal with the least number of exploratory points thereby increasing the risks of unforeseen ground conditions during construction.

5.2.2 Ground stratigraphy and soil fabric

Stratigraphy

It is important that the designer fully understands the geological and hydrogeological conditions at the site, in particular:

- the regional geology of the site
- the geological processes and geological history which formed the geology of the site and its surrounding area
- likely variations in the stratigraphic conditions across the site
- the interbedded nature of the deposits including both the micro and macro fabric
- hydrogeological conditions.

Typical ground investigation techniques adopted in stiff overconsolidated soils to establish stratigraphy are listed in Table 5.2.

Table 5.2 *Typical ground investigation techniques used in stiff overconsolidated soils to establish stratigraphy*

Technique	Advantages	Limitations
Cable percussion	• Routine and low cost	• Poor stratigraphic identification
Cable percussion with consecutive sampling including splitting open and describing soil sequence and fabric	• As cable percussion	• Slow • Poor quality undisturbed samples
Rotary coring preferably using wireline techniques and coreliner	• Complete and suitable for most soil conditions • Excellent for soil profiling • No depth limitation	• Difficult to achieve full core recovery in soft / loose soils • More expensive than cable percussion
Static cone	• Good for soil profiling • Low cost and quick	• Hard strata or obstructions will prevent penetration • Depth limitation • Less sensitive to strata changes than piezocone
Piezocone	• Excellent for soil profiling	• Hard strata or obstructions will prevent penetration • Depth limitation • More sensitive to strata changes than static cone
Window sampling	• Cheap, portable equipment which gives complete but approximate soil profile	• Hard strata or obstructions will prevent penetration • Depth limitation • No in situ tests possible • Difficult to reliably install piezometers • Poorest sample quality of all of the above methods

Soil fabric

The in situ behaviour of a soil mass is influenced by details of the soil fabric. The designer should carefully inspect soil fabric:

- when considering and finalising the design stratigraphy
- when scheduling laboratory testing to ensure that the correct sample is tested and that the effect of fabric on sampling and on the subsequent test results can be understood.

The designer should seek answers to the following questions:

- has the soil profile, including fabric, been interpreted in terms of the geological history?
- is the reason for any soil cementing understood?
- have the potential effects of boring and soil sampling on the observed soil fabric been considered, eg the likelihood of desaturation, the potential for water content changes, separation on bedding planes and fissures, the representativeness of the samples?

Table 5.3 lists some effects of fabric on soil properties.

Table 5.3 *Effect of fabric on soil properties*

Soil type	Fabric	Effect on
Fine grained soils	• Fissuring, jointing	• Mass strength
	• Shear surfaces	• Anisotropy of strength
	• Partings	• Permeability
Coarse grained soils	• Grain shape and orientation	• Anisotropy of strength and stiffness
	• Cementing	• Peak shear strength

5.3 ASSESSMENT OF DRAINED/UNDRAINED SOIL CONDITIONS

See also Appendix D...Soil mechanics (D.2)

The circumstances and considerations which apply in determining whether drained or undrained soil conditions apply in design depend upon the rapidity with which drained conditions are approached. For fully saturated soil, undrained conditions are characterised by no change in volume although change in *shape* can occur. It is possible for such conditions to prevail approximately in the short term in low permeability soils such as high plasticity sedimentary clays where there is slow dissipation of excess water pressures and suctions generated by loading and unloading respectively. The magnitude of the water pressure at equilibrium is dominated by the detailed fabric of the soil. Undisturbed soil samples from boreholes should be split and logged and the soil fabric should be carefully inspected wherever possible in situ from within a pit or shaft. **The importance of obtaining an accurate record and gaining a good understanding of the soil fabric cannot be over-emphasised.**

Stiff overconsolidated clays are generally jointed, fissured and layered as a result of their depositional history. Although the permeability of the clay between the joints and discontinuities may be low the presence of small sandy or silty partings can have a disproportionate effect on the magnitude of the coefficient of permeability of the soil.

This influences the rate at which water may flow from one part of the soil mass to another and the rate at which water may be drawn into the soil. The assessment of whether undrained conditions are likely to prevail for any duration requires careful and thorough consideration of the following factors:

- soil stratigraphy and fabric. There is usually no dilation on pre-sheared surfaces. Thus, shearing can occur at constant volume under drained residual conditions. This should not be confused with undrained behaviour

- the mass in situ permeability of the soil (in the vertical and horizontal directions) which may be increased by the effect of high permeability horizons, or discontinuities and fissures that open up on unloading

- the proximity and likelihood of available water (potential sources include rainfall, surface run-off, natural water courses, leaking pipes, aquifers and sand horizons, wastewater from hosing down construction plant, etc)

- the soil stiffness (which affects the value of the coefficient of consolidation, c_v). This might be particularly high following a change in the direction of the stress path

- previous experience of construction in similar ground conditions.

The designer should evaluate drainage paths and assess the duration over which drained conditions may be restored. Sound judgement based on previous experience of construction in similar ground conditions should be applied in determining whether drained or undrained soil behaviour should be assumed in design. For example, undrained conditions are commonly assumed in the design of temporary works for excavations in London Clay for durations of up to 6 months. In contrast, in glacial clays the mass permeability may be relatively high due to the common occurrence of permeable horizons and channels. The assumptions of undrained conditions in glacial deposits requires particularly careful consideration and would often be inappropriate.

As a general guide, undrained conditions may be assumed in the short term where the mass in situ permeability of the ground is low (ie a coefficient of permeability of the order of 10^{-8} m/s or less, BS 8002, 1994). Where the mass permeability is not low, and in coarse grained soils, drained conditions should be assumed in design.

5.4 DETERMINATION OF SOIL PARAMETERS

Soil behaviour is influenced by the following:

- initial soil state: porosity, fabric, current effective stress state

- stress, time and chemical history which are embodied in the yield characteristics

- stress or strain path

- rate of shearing and drainage conditions.

The influence of initial stress state and of history diminish with increasing strains post-yield. Therefore, the required sample quality to measure small strain stiffness is different from that required to measure high strain drained shear strength. For laboratory measurement of pre-yield behaviour or undrained shear strength, the soil state and effects of history should be preserved. This requires no change in volume or effective stress. This ideal cannot be achieved.

Certain laboratory tests bear the full imprint of disturbance. The unconsolidated undrained triaxial test is a good example since no attempt is made to re-impose the in situ stresses and hence is particularly prone to sample disturbance. The effects of sample disturbance on laboratory determined values of undrained shear strength are discussed in section 5.4.4.

The effects of sample disturbance and limitations of many laboratory tests have contributed to poor predictions of in situ behaviour. This has contributed to an increase in in situ testing or at least to an integration of laboratory and in situ testing. A balanced view should be taken of the advantages and limitations of both types of tests so that they are appropriately included in a ground investigation.

There is no reason why a shear strength derived from an in situ vane, pressuremeter or cone test should coincide with that measured in a laboratory triaxial compression or simple shear test. For a soil of a given composition and deposition and post-deposition history, peak shear strength will be influenced by the initial effective stress state, by drainage during shear, by the stress path and the rate and direction of shear. These will vary between the different types of in situ and laboratory test and so, too, will the measured strength. The small strain stiffness behaviour will also be affected by the recent stress or strain history (in this context imposed by the sampling process). In view of stress-strain non linearity, comparisons are only meaningful if they are made at corresponding levels of strain.

In view of the above, soil parameters should be determined from a number of independent sources:

- directly from the results of in situ and laboratory tests
- from established empirical correlations between different types of in situ and laboratory tests and with the soil's grading and index properties
- from relevant published data and local and general experience
- wherever possible, from back analysis of measurements taken from comparable full-scale construction in similar ground conditions.

The above information should be carefully collated and assessed in the selection of soil parameters for use in design calculations.

The selected soil parameters should encapsulate the designer's expertise and understanding of the ground and should be based on both site specific information and a wider body of geotechnical knowledge and experience.

The references listed in Table 5.4 provide published data on typical properties of stiff overconsolidated soils commonly encountered in the UK.

Guidance is provided below on the determination of soil parameters most relevant to retaining wall design. Typical values are presented together with commonly adopted empirical correlations between test types and with the soil's grading and index properties. These typical values are necessarily very general and should not be used in design calculations other than for the purposes of preliminary sizing. **The design of retaining structures should be based on site specific information appropriate for the geotechnical category of the structure.** The typical values provided below are, however, useful for comparison purposes.

Table 5.4 *Relevant publications giving typical properties for stiff clays commonly encountered in the UK*

Stratum	Literature source
Atherfield Clay	Loveridge (2001)
Barton Clay	Marsland and Butler (1968)
Claygate Beds	Hubbard *et al* (1984)
Gault Clay	Lings *et al* (1991)
Glacial tills	Weltman and Healy (1978)
	Trenter (1999)
Kimmeridge Clay	O'Brien *et al* (1992)#
Lambeth Group	Hight *et al* (2001)
Lias Clay	Chandler (1972)
London Clay	Patel (1992)
	Burland and Kalra (1986)
	Simpson *et al* (1979)
Mercia Mudstone	Chandler and Forster (2001)
Oxford Clay	Pierpoint (1996)
Weald Clay	Loveridge (2001)
	O'Brien *et al* (1992)

Note: see Cripps and Taylor (1981) for general properties of the above soils.

5.4.1 Classification properties and unit weight

Classification tests

The results of classification testing are essential in understanding material characteristics and behaviour and are necessary in the interpretation of in situ and laboratory testing.

Table 5.5 lists the index properties which should be routinely determined for fine grained and coarse grained soils.

Table 5.5 *Index properties to be determined*

Property	Fine grained soil	Coarse grained soil
Water content	√	×
Liquid and plastic limits	√	×
Liquidity index	√	×
Bulk unit weight	√	√
Particle size distribution	√	√
Grain shape (angularity)	×	√
Voids ratio	√	√

Unit weight

Table 5.6 provides typical values for unit weight of soils in the absence of site specific test results.

Table 5.6 *Typical unit weight of soils and fills (from CIRIA SP95 and BS8002)*

Type of Material	Dry (kN/m^3)		Saturated (kN/m^3)	
	Loose	Dense	Loose	Dense
Gravel	16	18	20	21
Well graded sand and gravel	19	21	21.5	23
Coarse or medium sand	16.5	18.5	20	21.5
Well graded sand	18	21	20.5	22.5
Fine or silty sand	17	19	20	21.5
Rock fill and quarry waste	15	17.5	19.5	21
Brick hardcore	13	17.5	16.5	19
Slag fill	12	15	18	20
Ash fill	6.5	10	13	15
Topsoil	16	19	20	21
River mud	14.5	17.5	19	20
Silt	18		18	
Very soft clay	16		16	
Soft clay	17		17	
Firm clay	18		18	
Stiff clay	19		19	
Very stiff clay or hard clay	20.0-21.0		20.0-21.0	

5.4.2 Permeability

See also

5.2.2….Ground stratigraphy and soil fabric

5.5…….Determination of groundwater pressures

The coefficient of permeability of soil, k, varies over a very wide range of values from about 10^{-10} m/s for practically impervious clays to about 1m/s for clean gravels. A range of values for various soils is presented in BS 8004 (1986). This is reproduced in Figure 5.3 and shows that the mass permeability of fissured clays can vary over a wide range of values.

Figure 5.3 *Permeability and drainage characteristics of soils (after BS 8004, 1986)*

A detailed assessment of ground permeability is essential in the determination of drainage conditions and the evaluation of groundwater pressures which the retaining wall should be designed to withstand. **The assessment of groundwater conditions is one of the most important issues affecting the design and construction of embedded walls.** This is discussed further in section 5.5.

The permeability of the ground will affect the:

- selection of wall type and its depth and stability

- duration over which undrained conditions can be considered to apply in the design of temporary works in clays, see section 5.3
- rate of softening
- the need for temporary and permanent drainage
- assessment of time dependent consolidation or heave ground movements.

Methods of permeability determination

The ground may comprise several strata of differing soils varying in thickness and properties, both with depth and with plan position over the area of the site. Individual soil horizons often have different properties at different points within their mass, ie they are non-homogeneous and anisotropic. It is notoriously difficult to measure permeability with any accuracy. Nevertheless, there are a number of methods of measuring permeability in the field and in the laboratory. These are presented in Figure 5.3 and summarised below:

- full-scale pumping tests in the field
- rising and falling head tests in boreholes
- in situ piezocone tests (generally used in fine grained soils only)
- from grading curves relating to recovered representative soil samples
- from laboratory permeameter, oedometer or triaxial consolidation tests
- from flow conditions measured in boreholes and piezometers.

Whenever possible, in situ tests which measure the properties of a large ground volume are preferred. Values of permeability determined from laboratory tests will often be an order of magnitude smaller than those determined from field pumping tests.

The techniques for undertaking and interpreting the results of pumping tests and rising and falling head permeability tests in boreholes are presented in BS 5930 (1999). Methods of estimating permeability are discussed in detail by Preene *et al* (2000).

Empirical rules

These currently apply only to coarse grained soils with relatively small proportions of fine (silt and clay sized) particles. The approximate methods presented below should not be used for soils containing more than about 10% of fine particles. For these soils, other methods listed above should be used.

Based on experimental work with uniform clean sands, Hazen (1911) related the coefficient of permeability to the 10% particle size D_{10} obtained from grading tests:

$$k = C_1 D_{10}^2 \qquad (5.1)$$

Where C_1 is a constant typically varying between 0.007 and 0.017 (Preene *et al*, 2000), k is in m/s and D_{10} is in mm. To obtain very approximate values of the coefficient of permeability, it is usually sufficient to assume $C_1 = 0.01$ in the above expression. Hazen's expression was developed for uniform (uniformity coefficient, $C_u = (D_{60}/D_{10})$ <5 where D_{60} is the 60% passing particle size) filter sands and may give misleading answers if applied to other soil types.

An alternative approximate technique, appropriate for use in less uniform soils, such as sandy gravel, is the Prugh method. This is described by Powers (1992) and uses the D_{50} particle size, the uniformity coefficient, C_u and the relative density of the soil to estimate permeability.

Comparison of methods

Table 5.7 gives a qualitative assessment of the reliability, cost and ease of interpretation of in situ ground permeability using some common methods of permeability determination.

Table 5.7 *Comparison of common methods of permeability determination*

Test	Reliability	Cost	Ease of interpretation
Field pumping tests	very good	very high	routine to very sophisticated, depending on test type and ground conditions
In situ piezocone tests (in fine grained soils only)	good	moderate	routine
Variable head borehole tests	poor	low	routine
Tests in piezometers	poor	low	routine
Laboratory tests			
- direct*	poor	moderate	routine
- indirect#	very poor	low	routine

Key to table:

* tests using permeameter

\# derived from empirical correlations with grading, etc

Determination of permeability for use in design

In view of the difficulties of accurately determining the mass permeability of ground, **a number of the methods listed in Table 5.7 should be adopted in conjunction with a sound understanding of soil stratigraphy and fabric (section 5.2.2). Wherever possible, field pumping tests and piezocone testing should be carried out.** On projects where the costs of such testing is prohibitively expensive, or where the ground conditions are not appropriate for such testing, the mass permeability of the ground should be assessed from a careful consideration of the results of more than one of the remaining tests listed in Table 5.7. Laboratory permeameter testing should only be carried out on soil samples which are representative of in situ ground conditions.

5.4.3 In situ stress conditions

See also

4.1.1....In situ lateral stress

4.1.2....Effect of wall installation

5.4.4....Shear strength

The in situ stress state in the ground depends upon the depositional and erosional history of the soil. The coefficient of the in situ earth pressure, K_o, provides a means of evaluating the magnitude of the horizontal effective stress in undisturbed soil and, hence, the in situ stress conditions in the ground.

Normally consolidated soils

The value of K_o for a normally consolidated soil can be related to the drained angle of shearing resistance, ϕ', by the relationship:

$$K_{onc} = 1 - \sin\phi' \quad \text{(Jaky, 1944)} \qquad (5.2)$$

This relationship has been found to hold for normally consolidated sands and clays.

Overconsolidated coarse grained soils

Alpan (1967) proposed the following expression:

$$K_o = K_{onc}.\,(OCR)^{\lambda} \qquad (5.3)$$

where K_{onc} is the value of K_o for soil in a normally consolidated state, OCR is the overconsolidation ratio and λ is a factor which depends on the angle of shearing resistance, ϕ', of the soil and typically varies between 0.4 and 0.5.

Overconsolidated fine grained soils

For overconsolidated clays, several correlations have been published relating K_o to overconsolidation ratio, plasticity index and the shear strength ratio (s_u / σ_v') of the soil. These are listed below:

- Brooker and Ireland (1965) : see Figure 5.4
- Simpson *et al* (1979) : $K_o = K_{onc} \{ (s_u / \sigma_v') / (s_u / \sigma_v')_{nc} \}^{½}$ (5.4)
- Mayne and Kulhawy (1982) : $K_o = K_{onc}\, OCR^{\sin\phi'}$ (5.5)
- Shohet (1995) : $K_o = K_{onc} + ½ \{ (s_u / \sigma_v') / (s_u / \sigma_v')_{nc} - 1 \}$. (5.6)

where:

K_{onc} is the value of K_o for soil in a normally consolidated state (equation 5.2)

$(s_u / \sigma_v')_{nc}$ is the shear strength ratio for the soil in its normally consolidated state [$(s_u / \sigma_v')_{nc} \approx 0.11 + 0.0037PI$ (Skempton, 1957)]

(s_u / σ_v') and ϕ' are estimated from the results of the ground investigation or from typical values of s_u and ϕ' presented in section 5.4.4, where site specific data are not available.

Figure 5.4 *Correlation between the in situ coefficient of earth pressure and overconsolidated ratio for clays of various plasticity indices (after Brooker and Ireland, 1965)*

Figure 5.5 *Influence of stress history on K_o and σ_h' in a heavily overconsoldiated clay (after Burland et al, 1979)*

OCR can be estimated from the results of laboratory oedometer tests. However, high pressures are required for heavily overconsolidated clays. For such soils, Burland *et al* (1979) have calculated values of effective overburden pressure, effective horizontal stress, σ_h', and K_o for sites subject to erosion, erosion followed by surcharge and underdrainage (Figure 5.5). This shows that the value of K_o varies significantly in response to a change in the value of the effective overburden pressure whilst the value of the effective horizontal stress, σ_h', remains relatively unchanged. This is important in the context of retaining wall design where the designer is most interested in determining the value of the horizontal effective stress and is only interested in K_o as a means of calculating σ_h'. K_o values can typically range from near K_p at the top of the clay stratum, trending towards 1.0 at depth.

The value of K_o can also be obtained from measurement of soil suctions on undisturbed samples of clay in the laboratory. This provides an independent means of determining K_o which can be used as a check on other methods. Sample disturbance significantly affects the suction measured in clay samplers in the laboratory. Sample disturbance depends upon the size of the sample and the method adopted in recovering it from the ground. This is discussed in detail by Vaughan *et al* (1992) and Chandler *et al* (1992).

The process of drive-sampling with U100 samples (area ratio typically 25%) significantly increases the effective stress in tube samples of stiff overconsolidated clays. This increase in effective stress is due to the migration of pore water from the central parts of the sample to the sheared periphery. Chandler *et al* (1992) report measured increases in stress of 15-45 kPa in U100 samples recovered from shallow depths in weathered London Clay to 250-300 kPa in U100 samples recovered from very stiff unweathered London Clay. These increases overestimate undrained shear strength and suctions (and hence K_o).

Increasing the sample size and decreasing the area ratio of the cutting shoe of the sampler will reduce the effects of disturbance. Harrison (1991) compared suctions in samples of London Clay taken with thin wall samplers with suctions from U100 samples. These data show that, on average, the thin-walled samplers caused less disturbance, their suctions averaging 85 kPa less than those for U100 samplers.

Thus, where suction tests are to be carried out in the laboratory on samplers of overconsolidated clay, samples should be taken using pushed in thin wall samplers to reduce the effects of disturbance. They should be tested as soon as possible after sampling to minimise the effects of pore water migration and increasing effective stress. Any tubes which are damaged or distorted should not be subject to testing.

The soil suction within thin walled specimens should be measured using the filter paper technique as described by Chandler and Gutierrez (1986) and in the triaxial apparatus following procedures described by Burland and Maswoswe (1982).

Determination of K_o from in situ tests

The value of K_o can be derived using a self boring pressuremeter (Clarke, 1995) and spade total stress cells. However such techniques suffer from the effects of installation disturbance and significant effort and experience is required in interpreting the results. Nevertheless, they can be more reliable than laboratory determinations of K_o. There is much experience in the use of the self boring pressuremeter in soft and stiff clays (Wroth, 1982; Hawkins *et al*, 1990) but less so in sands (Fahey and Randolph, 1984; Whittle, 1991).

The self boring load cell pressuremeter (Carder and Bush, 2001; Darley *et al*, 1999; Darley *et al*, 1996) probably provides the best currently available method to determine the in situ lateral stresses in stiff overconsolidated clays. This device allows total stress and pore water pressures to be read directly and continuously from six load cells and piezometers equi-spaced around its circumference. Provided the test duration is sufficient to ensure that the stresses and pore water pressures around the device stabilise, the value of the in situ coefficient, K_o, can be determined.

Selection of *K* for use in design calculations

It is difficult to accurately assess the in situ stress in stiff overconsolidated soils. For this reason, the value of K selected for use in design calculations should be based on careful consideration of K_o values determined using one or more of the methods discussed above:

- from established correlations between K_o and soil index and strength properties
- from the stress-history approach described by Burland *et al* (1979)
- from suction tests carried out on thin walled samples
- from the results of in situ self boring pressuremeter, in situ self boring load cell pressuremeter and/or spade total stress cells
- from the back analysis of comparable construction in similar ground conditions as discussed in section 4.1.2.

The value of *K* adopted in design calculations should allow for the effects of wall installation (section 4.1.2). In general, it may be appropriate to adopt the following *K* values in simple elastic (ie where the pre failure deformation of the soil is assumed to be linear) soil-structure interaction analyses:

- **for normally consolidated soils: $K = K_{onc}$ from equation 5.2**
- **for overconsolidated fine grained soils: $K = 1.0$**
- **for overconsolidated coarse grained soils: $K = 1.0$ for walls installed by non-displacement methods (eg bored pile walls, diaphragm walls).**

5.4.4 Shear strength

See also

5.4.3....In situ stress conditions
5.9......Selection of design parameters

Fine grained soils

Undrained shear strength

The undrained shear strength of a clay is not a fundamental soil property. Different values may be measured in triaxial compression and extension and in in situ tests. Where, for temporary works design, the undrained shear strength of a clay is relied upon, it will be necessary to consider very carefully the effects of sampling and installation disturbance and the drainage paths present in the soil. Section 2.2.3 of BS 8002 (1994) provides a commentary on the considerations necessary in this respect.

Although published literature contains many references relating (s_u/σ_v') to *OCR*, liquidity index, etc (eg Ladd *et al*, 1977; Wroth, 1984), in practical terms, it is more convenient to measure the undrained shear strength of stiff overconsolidated clays directly in the laboratory or in situ using self boring pressuremeter tests. Hand shear vane and pocket penetrometer tests can be used to supplement these data. Hand shear vane tests in trial pits provide a rapid and economical method of measuring undrained shear strengths up to about 120 kPa. Pocket penetrometer results are less reliable but can be used to estimate undrained shear strengths up to 250 kPa.

Single stage unconsolidated undrained triaxial tests should be carried out on 100 mm diameter undisturbed soil samples to measure the undrained shear strength in the laboratory. Tests should be carried out as soon as possible after sampling to minimise the effects of sample disturbance (section 5.4.3). 38 mm diameter samples from clays which are highly fissured or which contain a high stone content

should not be relied upon to provide representative values of the undrained shear strength. Multistage testing should not be carried out. Unconfined tests should not be carried out. A confining pressure should always be applied and this should not be less than twice the total overburden pressure corresponding to the depth from which the sample was recovered.

In stiff overconsolidated clays of medium to high plasticity, the size of the sample has an effect on the measured value of undrained shear strength. Smaller samples give higher measured undrained shear strength with considerable scatter in results. This is likely to be due to the effects of sample disturbance which is more pronounced in small samples and partly the result of the presence of fewer fissures in smaller samples. Marsland (1971) shows that the undrained shear strength of London Clay varies with the ratio of sample size to the fissure spacing as shown in Figure 5.6.

Figure 5.6 *Influence of the ratio of sample size to the fissure spacing on the strength measured in laboratory tests (after Marsland, 1971)*

Suctions measured in samples are also related to sample size; suctions decrease with increasing sample size. Maguire (1975) showed that suctions measured in large samples (260 mm diameter) agreed closely with the estimated corresponding mean effective stress. Where suctions are measured in the laboratory, the measured suctions should be compared with the value of the mean effective stress estimated from the value of K_o determined from the methods described in section 5.4.3. If these suctions are significantly different from the mean effective stress, the measured undrained shear strength, s_u, should be treated with caution.

In stiff overconsolidated clays of low plasticity, eg glacial tills, the undrained shear strength measured in tube samples and remoulded samples at the same moisture content are in reasonable agreement. Sample disturbance appears not to influence the measured strength to the extent observed in more plastic clays. For such soils, correlations between liquidity index and undrained shear strength for remoulded clays provide a useful approximate check on soil test data (Figure 5.7). However, it should be noted that for such soils, the undrained shear strength is very sensitive to changes in moisture content. Also, the influence of stone content on undrained shear strength is important. The stone content should be taken into account in estimating the moisture content of the clay matrix.

Figure 5.7 *Correlation between undrained shear strength and liquidity index (after Skempton and Northey, 1952)*

The undrained shear strength of overconsolidated clays in the mass can be derived from empirical correlations with the results of sounding (ie static cone) and penetration tests (eg SPTs). The undrained shear strength, s_u, can be related to the static penetrometer cone resistance, q_c, as follow:

$$s_u = (q_c - \sigma_v) / N_k \qquad (5.7)$$

where N_k is a cone factor = 24 to 30, average 27, for stiff fissured marine clays and 14 to 22, average 18, for glacial tills (Meigh, 1987) and σ_v is the vertical total stress at the level of the test.

Stroud (1989) showed that:

$$s_u = f_1 N_{60} \qquad (5.8)$$

where f_1, depends upon the plasticity of the clay (Figure 5.8) and N_{60} is the Standard Penetration Test blowcount.

Figure 5.8 *Correlation between N_{60} value and undrained shear strength and plasticity index for insensitive clays (after Stroud and Butler, 1975)*

In the absence of site specific data, and for the design of category 1 walls, BS 5930 (1999) and BS 8004 (1986) provide approximate values of undrained shear strength, s_u, deduced from physical descriptions of soil samples. This is reproduced in Table 5.8.

Table 5.8 *Undrained shear strength of clays (after BS 8004, 1986)*

Consistency of clay BS 5930	Widely used	Field indications	Undrained shear strength s_u (kPa)
Very stiff	Very stiff or hard	Brittle or very tough	> 150
Stiff	Stiff	Cannot be moulded in the fingers	100-150
	Firm to stiff		75-100
Firm	Firm	Can be moulded in the fingers by strong pressure	50-75
Soft	Soft	Easily moulded in the fingers	20-40
Very soft	Very soft	Exudes between the fingers when squeezed in the fist	<20

Drained shear strength

The shear strength of fine grained soils in terms of effective stress (c', ϕ') can be determined from laboratory triaxial tests:

- consolidated drained triaxial tests in compression and extension
- consolidated undrained triaxial tests in compression and extension with pore pressure measurement.

The following considerations should apply in undertaking and interpreting the results of such tests:

- **tests should generally be carried out on 100mm diameter samples**
- **strain rates in drained tests should be sufficiently slow to ensure complete dissipation of excess pore pressure and equalisation of pore pressure**
- **consolidation pressures should be consistent with those anticipated as the working stress range.** For example, the *maximum* mean effective stress experienced by soil elements behind and in front of a 5m deep wall is unlikely to exceed about 75 kPa and the *average* stress behind the wall is about half of this value. The consolidation pressures should be chosen to represent a reasonable range of values which are appropriate for the wall being designed. **The designer should ensure that the testing laboratory provides reliable and accurate calibrations which are applicable over the range of consolidation pressures to be used in the testing. This is particularly important where very low effective consolidation pressures (less than about 50 kPa) are specified**

- when plotted on a graph of shear stress (τ) versus effective consolidation pressure (σ'), the failure envelope for most stiff overconsolidated clays is curved (Figure 5.9).

Figure 5.9 *Strength envelope for a given pre-consolidation*

Two approaches can be adopted for the determination of the effective stress parameters, c' and ϕ', from the test data:

– a zero value for c' is assumed and a *secant* ϕ' value is selected from data at effective stress levels at the upper limit of those applicable to the wall design (Figure 5.9). Moderately conservative, worst credible and most probable values (see section 5.9 for definitions) can be derived from appropriate lines drawn through the data. This approach will give ϕ' values which are relatively high in comparison with those commonly assumed for stiff plastic clays

– *tangent* parameters (c', ϕ') can be selected for the applicable stress range (Figure 5.9). By drawing appropriate lines through the data, moderately conservative, worst credible and most probable values of c' and ϕ' can be derived (see section 5.9 for more details). The tangent which defines the failure envelope is represented by:

$$\tau = c' + \sigma' \tan \phi' \qquad (5.9)$$

Equation 5.9 has no physical relevance: it is merely a fit to the data over the applicable stress range. c' is not necessarily indicative of a "real" cohesion (ie an ability to withstand shear stress at zero effective stress) and ϕ' is not the "true" angle of shearing resistance since, defined in this way, it may be smaller than the value of ϕ'_{crit} for the same soil.

- the significance of the cohesion intercept, c', should be carefully considered. In drained conditions, the shear strength of soils is principally frictional with c'=0. This holds for normally consolidated and remoulded fully saturated clays. Tests on undisturbed samples of overconsolidated clays and partially saturated clays (in which particles may be drawn together by surface tension meniscus effects) may exhibit some apparent cohesion intercept. If the test strain rate is too fast, large values of the cohesion intercept can be derived. The "real" cohesion in soil probably relates to bonding forces which have developed between clay particles over long periods of time undisturbed in the ground. Once broken, these bonds do not reform. The effects of remoulding due to wall installation and swelling of the clay due to excavation in front of it may generate sufficient strains for c' to be appreciably reduced. It is therefore likely that the value of c' mobilised along the failure surfaces behind and in front of the wall will be much smaller than the values derived from laboratory tests

- published effective strength parameters derived from the back analyses of slope failures may not be directly relevant to retaining walls due to differences in stress path direction. The mobilised strength of weathered clays is usually lower than unweathered clays. For many geologically old sedimentary clays, there is a gradual change from clay to very weak rock at depth. For example, clays such as Oxford Clay, Kimmeridge Clay, Weald Clay, etc, are more realistically described as very weak mudrocks at depth. This transition would usually be reflected by an increased mobilised strength, either increased ϕ' (if a secant fit approach was adopted) or increased c' (if a tangent fit approach was adopted). In addition, excavation in front of the wall will lead to swelling and increases in moisture content. This will be most pronounced near the excavation formation where volumetric strains are at a maximum and negligible at depth where volumetric strains are small. Because of the above two factors, it may be appropriate to select c' values which increase with depth if a tangent fit approach is adopted.

In the absence of reliable site specific laboratory test data, the values of the critical state angle of shearing resistance, ϕ'_{crit}, presented in Table 5.9 can be used in conjunction with $c'=0$.

Table 5.9 *ϕ'_{crit} for clay soils (after BS 8002, 1994)*

Plasticity index (%)	ϕ'_{crit} (degrees)
15	30
30	25
50	20
80	15

Coarse grained soils

The shear strength of coarse grained soils should be determined from the results of laboratory shear box or triaxial tests in accordance with the requirements of BS 1377 (1990). Tests can be carried out in small (60 mm square) or large (300 mm square) boxes. For the 60 mm shear box, it may be possible to obtain relatively undisturbed specimens from rotary coring techniques. If this is not possible, specimens should be reconstituted to approximately the in situ density. Tests should be carried out over the range of normal stresses likely to exist on the wall during its life (BS 8002, 1994). The displacements during testing should be sufficient to enable both the peak (ϕ'_{peak}) and critical state (ϕ'_{crit}) angles of shearing resistance to be determined directly. The drained shear strength can also be determined from isotropically or anisotropically consolidated drained or undrained triaxial tests in compression or extension. As with shear box tests, specimens can be obtained from rotary coring or reconstituted to in situ density. Unlike stiffness, the drained shear strength is less sensitive to alterations to the structure of the soil. The peak drained angle of friction obtained from laboratory triaxial test results, ϕ'_{pt}, can be correlated (Bolton, 1986) with relative density (D_r) and mean effective stress at failure (p'):

$$\phi'_{\text{pt}} - \phi'_{\text{crit}} = 3\, I_R \qquad (5.10)$$

where,

$$I_R = D_r\,(10 - \ln p') - 1 \qquad (5.11)$$

Relative density can be obtained from the results of SPT blowcounts, $(N_1)_{60}$, corrected for an effective overburden pressure of 100 kPa, as described by Skempton (1986). Stroud (1989) presents a relationship between ϕ'_{peak}, $(N_1)_{60}$, D_r and *OCR*. This is reproduced in Figure 5.10.

Figure 5.10 *Effect of overconsolidation on the relationship between $(N_1)_{60}$ and peak angle of friction ϕ'_{peak} (after Stroud, 1989)*

Based on the work of Bolton (1986), Stroud (1989) concludes that for a given material, **ϕ'_{crit} in plane strain is about 10% greater than that determined for triaxial loading.** This is important in retaining wall design where plane strain conditions are likely to predominate

The peak and critical state angles of shearing resistance for siliceous sands and gravels can be estimated from expressions presented in section 2.2.4 of BS 8002 (1994). These are reproduced below.

$$\phi'_{peak} = 30 + A + B + C \qquad (5.12)$$

$$\phi'_{crit} = 30 + A + B \qquad (5.13)$$

where, values of A, B and C can be obtained from Table 5.10.

Table 5.10 *Values of A, B and C for siliceous sands and gravels (after BS 8002, 1994)*

A − **Angularity**[(1)]	A **(degrees)**
Rounded	0
Sub-angular	2
Angular	4
B − **Grading of soil**[(2)]	B **(degrees)**
Uniform [$(D_{60}/D_{10})<2$]	0
Moderate grading [$2 \leq (D_{60}/D_{10}) \leq 6$]	2
Well graded [$(D_{60}/D_{10}) > 6$]	4
C − **Standard Penetration test blowcount**[(3)] N'	C **(degrees)**
$N' \leq 10$	0
$N' = 20$	2
$N' = 40$	6
$N'_{60} = 60$	9

Notes:

Angularity is estimated from visual description of soil

(1) Grading can be determined from the uniformity coefficient ($=D_{60}/D_{10}$), where D_{10} and D_{60} are particle sizes that in the sample, 10% of the material is finer than D_{10} and 60% finer than D_{60}.

(2) SPT N' blowcount corrected for effective overburden pressure as shown in Figure 5.11, BS 8002 (1994).

(3) Intermediate values of A, B and C may be obtained by interpolation.

Figure 5.11 *Derivation of N' from SPT blowcount N_{60} (after BS 8002, 1994)*

5.4.5 Stiffness

See also

2.5.2….Predictions of ground movements

5.4.3….In situ stress conditions

5.4.4….Shear strength

It is not good practice to rely upon a single method to determine soil stiffness but rather to use several different approaches. Current UK practice includes specialist in situ self boring pressuremeter testing, geophysical testing and specialist sampling and laboratory small strain stiffness measurement.

The self boring pressuremeter is probably the most robust means of determining soil stiffness at strains relevant for wall design across a broad range of overconsolidated clays and very weak rocks in the UK.

The stress-strain behaviour of soil is highly non-linear and soil stiffness decays with strain by orders of magnitude. Figure 5.12 shows a typical stiffness-strain curve for soil.

Figure 5.12 *Stiffness - strain behaviour of soil with typical strain ranges for laboratory tests and structures (after Atkinson and Sallfors, 1991 and Mair, 1993)*

At very small strains (about 0.001%), the stiffness is large; at strains close to failure, the stiffness is small (Atkinson, 2000). Atkinson and Sallfors (1991) identify three regions of a typical stiffness strain curve for soil. These are shown on Figure 5.12. Atkinson (2000) describes the different equipment and test procedures which are best able to measure stiffness in each region. Under working conditions, strains associated with retaining walls typically range between about 0.01% and 0.1% and lie in the small strain region. Laboratory testing methods appropriate for the determination of stiffness in this strain region require the use of local strain gauges (Jardine *et al*, 1984). Scholey *et al* (1995) provide a state of the art review on the measurement of soil stiffness using local gauges.

Soil stiffness for design of category 1 and category 2 walls

Serviceability limit state conditions

The performance of category 2 walls under working conditions, for displacement calculations, is usually analysed using a soil-structure interaction model which is based on subgrade reaction or pseudo-finite element or finite element or finite difference models adopting simple elasto-plastic or Mohr-Coulomb models. Wall deformations and soil strains are likely to be in the small strain region defined by Atkinson (2000). Due to difficulties in routinely measuring stiffness in the small strain region, stiffness values used in these programs are empirically derived from the back analysis of comparable excavations and wall systems. Furthermore, these stiffness values are used in conjunction with other soil parameters which are derived using methods described in sections 5.4.3 and 5.4.4. For example, in the FREW program, soil stiffness values corresponding to $(E_u/s_u) = 1000$ (and E' related to E_u in accordance with Equation B1, Appendix B), in conjunction with $E' = N_{60}$ (MPa) and $E' = 2N_{60}$ (MPa) for overlying normally consolidated and overconsolidated coarse grained soils respectively, generally provide conservative estimates for the deflections of walls embedded in stiff London Clay. It may be appropriate to vary (E_u/s_u) with depth as a simple way of reflecting stress-strain non linearity. Such an approach of **"calibration" against good quality case history data of comparable experience is essential for *any* soil-structure interaction model.**

As stated above, the self boring pressuremeter is probably the most robust means of determining soil stiffness at strains relevant for wall design. Recent research (Viggiani, 1992; Porovic, 1995; Viaggiani and Atkinson, 1995; Atkinson, 2000) enables the non-linear characteristics of soil stiffness to be readily derived from the results of small strain laboratory testing on high quality (thin walled pushed in or rotary core) samples and site specific geophysical tests (Matthews, *et al*, 2000) carried out in situ and on samples in the laboratory. These methods allow for the highly anisotropic nature of most stiff overconsolidated soils.

Where high quality site specific information (eg in situ self boring pressuremeter tests, geophysical tests, small strain laboratory tests) are not available, it may be appropriate to use relevant published data in conjunction with a "calibrated" soil-structure interaction model as described above.

Where refinements are made to the soil stiffness value assumed in a current soil-structure interaction model, then the model should be "re-calibrated" against good quality case history data of comparable experience to ensure that the results obtained from the model are reasonable. Further refinements may be necessary to the soil stiffness value and to the other soil parameters adopted in the model to ensure this.

Ultimate limit state conditions

Under ultimate limit state conditions, for collapse calculations, wall deformations are comparatively large. Under such conditions, soil strength is likely to be fully mobilised and soil stiffness will be of secondary importance. This is particularly applicable to cantilever and singly propped or anchored embedded walls. However, for multi-propped walls, where soil strength is not fully mobilised, soil stiffness may have a significant effect on ULS collapse calculations.

The magnitude of soil stiffness applicable to ULS calculations is difficult to estimate and will depend upon the applicable strains. **In general, it is often appropriate to adopt values of soil Young's moduli which are approximately 50% of those which are adopted in SLS calculations.**

Soil stiffness for design of category 3 walls

A full numerical analysis may be required. Where this is carried out, special in situ and laboratory tests may be necessary to determine the parameters required by the proposed numerical analysis. **The designer should specify precise details of these tests to obtain the necessary parameters.**

Further discussion is beyond the scope of this report.

5.5 DETERMINATION OF GROUNDWATER PRESSURES

See also

5.2.1….Site investigation

5.2.2….Ground stratigraphy and soil fabric

5.4.2….Soil permeability

6………Design of wall

The assessment of groundwater pressures acting on a retaining wall is very important. This should be based on an understanding of the ground stratigraphy and fabric (section 5.2.2), permeability (section 5.4.2) and the determination of the pore water pressure distribution from the ground investigation (section 5.2.1). Preene *et al* (2000) provide a detailed discussion of the considerations necessary in making this assessment.

In addition to the above, the designer should check that the following have also been considered:

- the proximity of sources of free water and the likelihood of such sources becoming available over the design life of the wall (section 5.2)
- the effects on the local hydrogeology of the site due to the construction of the wall, eg potential damming to natural groundwater flow patterns, long term rise in aquifer groundwater levels
- the effects of wall toe levels not reaching the target design levels due, for example, to obstructions, hard driving resistance
- the effects of drainage or dewatering during construction and during the lifetime of the wall
- changes to water pressures due to the growth or removal of vegetation
- changes to water pressures due to long term climatic variations

- the base stability of the excavation in front of the wall should be checked as being satisfactory, ie it does not fail by piping
- the vertical stability of the wall and the associated overall structure should be checked as being satisfactory, ie it does not fail by uplift or flotation.

Based on the above considerations, the designer should determine:

(i) water pressure and seepage forces which represent the most unfavourable values which could occur in *extreme* or *accidental* circumstances at each stage of the wall's construction sequence and throughout its design life. An example of an extreme or accidental event may be a burst water main in close proximity to the wall

(ii) water pressures and seepage forces which represent the most unfavourable values which could occur in *normal* circumstances at each stage of the wall's construction sequence and throughout its design life. Extreme events such as a nearby burst water main may be excluded, unless the designer considers that such an event may reasonably occur in normal circumstances.

The groundwater pressure corresponding to (i) above should apply to ULS considerations while (ii) should apply to SLS considerations. This is discussed further in chapter 6.

BS 8102 (1990) recommends the following water levels for walls which are required to achieve the watertightness requirements defined in Table 1, BS 8102:

- for basements not exceeding 4 m deep, a head of groundwater, three quarters of the full depth below ground (subject to a minimum of 1 m);
- for basements deeper than 4 m, a head of groundwater 1 m below ground level.

These water levels may be too onerous and may even be impossible in certain circumstances. It is recommended that groundwater pressures corresponding to (i) above be adopted to check compliance with the wall seepage performance levels defined in BS 8102 (1990).

Guidance on details required to achieve the BS 8102 watertightness requirements for bored pile, diaphragm wall and sheet pile walls is provided in CIRIA report 139 (1995).

5.5.1 Undrained conditions

See also

4.1.6..........Tension cracks

5.3..............Assessment of drained/ undrained soil conditions

Appendix D..Soil mechanics (D2)

Where undrained conditions are considered to prevail during the construction of the wall and the excavation in front of it (section 5.3), total stress considerations will apply (Appendix D). In such circumstances, there may be a tendency for tension cracks to develop in the retained soil (section 4.1.6). These tension cracks can be considered to be dry or flooded, depending upon the availability of water. There is significant uncertainty as to what depth of clay which may be affected by this. Tension cracks may be less likely behind a propped wall compared to a cantilever wall. **The designer should assume that tension cracks extend to the depths stated in section 4.1.6 together with the minimum total horizontal stress shown in Figure 4.5.**

In the case of an embedded cantilever wall or where water is expected to be present, eg from water bearing deposits overlying the stiff clay, hydrostatic pressure of $\gamma_w z$ should be adopted on the retained side of the wall to the depth shown in Figure 4.5. For an embedded wall propped or anchored near or at its top, the possibility of a flooded tension crack developing behind the wall may be discounted in a uniform homogeneous isotropic stratum of clay if it can be demonstrated that the lateral stress is greater than the hydrostatic pressure of water at the same level over a minimum depth of 1 m near the top of the wall and that water cannot enter the tension crack by other means (section 4.1.6).

Where water is not expected, the total pressure acting on the retained side of the wall at any depth below the ground surface should be assumed to be not less than 5 z kPa. In some circumstances, this requirement may appear to be too onerous. However, in view of the uncertainties regarding tension cracks, this is considered to be a prudent assumption and the designer should adopt it in his design calculations.

Assumptions relevant to temporary works design in undrained conditions are discussed in detail in section 5.9.1.

5.5.2 Drained conditions

See also

5.9.1….Temporary works design

5.9.2….Permanent works design

Where the ground fabric and permeability indicate that drained conditions should be designed for, effective stress considerations will apply. The designer should evaluate water pressure over the whole of the wall assuming steady state conditions at relevant stages of construction and over the lifetime of the wall. The use of flow nets to estimate water pressure in isotropic and anisotropic conditions is described by Williams and Waite (1993) who also consider the influence of the width of the excavation in front of the wall. Figure 5.13 shows some typical scenarios which illustrate the significant effect of anisotropy on groundwater pressures.

Figure 5.13 *Various steady state seepage flownets*

In reality, the ground will be anisotropic with a non-uniform coefficient of permeability, which may greatly affect the pore water pressure distribution. A commonly adopted simplified method of evaluating water pressure around an embedded retaining wall to a wide excavation in uniform isotropic conditions is shown in Figure 5.14 (a).

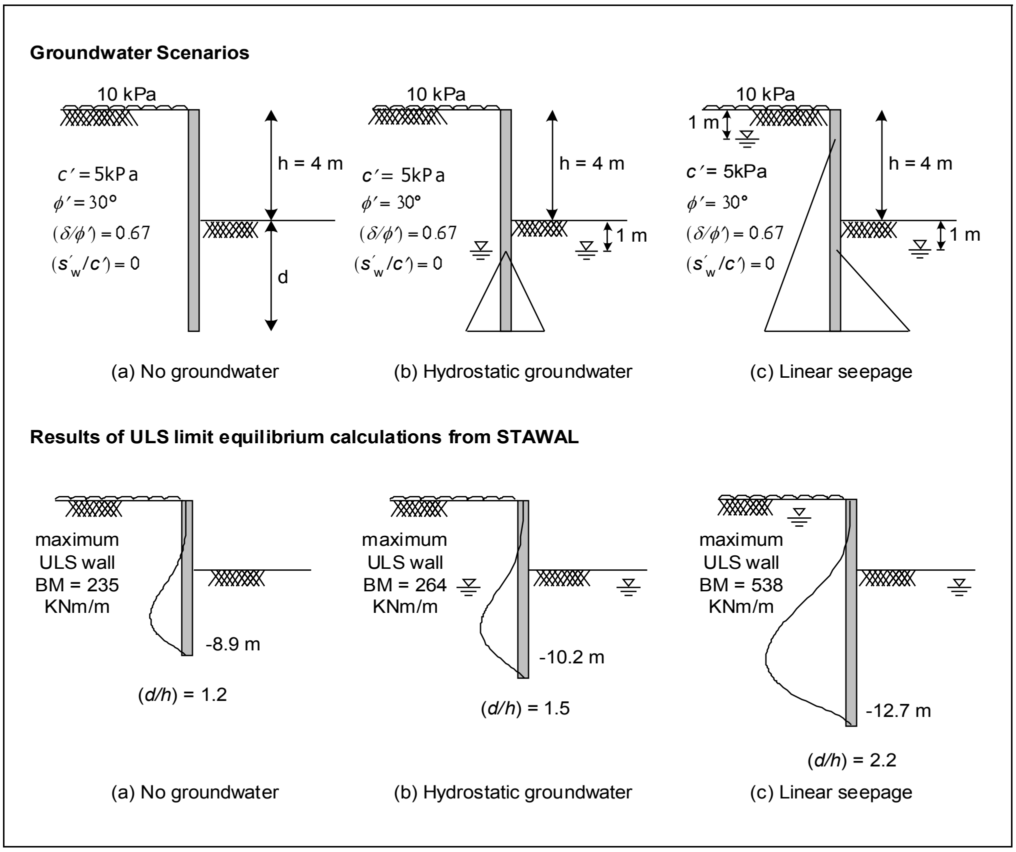
Figure 5.14 *Linear steady state seepage in uniform ground and the effect of excavation width (after Williams and Waite, 1993)*

This simplified method applies where the differential head of water across the wall dissipates uniformly along the length of the flow path adjacent to the wall and is sometimes referred to as *linear seepage*. This assumption tends to underestimate water pressures beneath narrow width excavations. In general, the expression in Figure 5.14 (a) should not be applied to excavations of widths which are less than 4 times the differential water head across the wall (BS 8002, 1994). At greater excavation widths, the assumption of linear seepage provides a good approximation to the pore water pressure distribution around the wall. For narrower excavations; in anisotropic ground conditions; and in situations where the design of the wall is sensitive to small changes in earth and water pressures around the wall (see section J5.2, Appendix J), water pressures should be evaluated by computer based analytical methods or by flow nets (Figure 5.14 (b)). Computer based methods, such as finite element analyses, are now readily available and should be preferred for their applicability to a wide range of ground conditions.

The groundwater regimes shown in Figures 5.13 and 5.14 are only applicable to impermeable walls. Where water flow or seepage is possible *through* a retaining wall (eg in the case of a contiguous bored pile wall, king post wall, seepage through the clutches of a sheet pile wall), appropriate boundary conditions (eg a zero phreatic surface at the line of a king post wall) should be assumed and the groundwater pressure applicable in a wall design should be determined from computer based seepage analysis or flownets.

The groundwater pressures assumed in design calculations will have a significant effect on the design of the retaining wall. Box 5.1 schematically illustrates the effect of groundwater on retaining wall design. Appendix J (Figures J13 and J14) shows that a relatively small change in the groundwater pressure can have a significant effect on retaining wall design.

Box 5.1 *Effect of groundwater pressure on retaining wall design*



Assumptions relevant to the design of temporary and permanent works in drained conditions are discussed in detail in sections 5.9.1 and 5.9.2 respectively.

5.5.3 Mixed undrained and drained conditions

See also

5.9.1….Temporary works design

It is possible for the retained side of the wall to be considered drained (eg due to unloading laterally and the likelihood of open fissures allowing water into the clay and the potential for consequential softening or due to the presence of water bearing silt and sand partings in the clay) while the restraining soil is considered to be undrained (eg due to an impermeable wall extending into impermeable ground thereby providing a cut-off to the groundwater recharge beneath the excavation on the restraining side). There is less likelihood of fissures opening and allowing water into the restraining clay due to its lateral confinement. In such circumstances, effective stress conditions may apply on the retained (active) side. Total stress conditions may apply in the restraining soil on the passive side and, possibly also, on the retained (active) side below the depth of the tension cracks.

The above is a very specific example used to highlight a scenario where mixed undrained and drained conditions may apply. This particular combination is quite often appropriate, though many other scenarios are possible. The designer should carefully consider the likely ground behaviour on each side of the wall before finalising his design assumptions.

5.6 LOADS

These comprise lateral loads and vertical loads applied to the wall.

5.6.1 Lateral loads

See also

4.1.7....Factors affecting limiting lateral pressures

4.1.8....Factors potentially increasing earth pressures in working conditions

5.6.3....Load case combinations

These are:

(i) derived from increased earth pressures due to vertical loading applied adjacent to the wall; and/or

(ii) forces applied directly to the wall, for example, from out of balance forces (a typical example is shown in Figure 1.3), impact forces, inertia forces due to seismic events.

The designer should carefully consider both sources (i) and (ii) above. Source (ii) will be project specific. Some guidance is provided below on typical values of loading relating to source (i).

The wall should be designed to allow for loading on the retained side which is appropriate under the particular circumstances where the wall is located. The designer should choose the appropriate loading based on careful consideration of the following:

- loads from adjacent roads, railways, buildings, etc
- loads due to construction activity, eg adjacent stockpiles, construction plant including cranes
- variations in ground surface levels.

The above loads should comprise permanent (dead) loads and temporary (live) loads. Section 5.6.3 provides guidance on load case combinations which should be considered in design calculations.

For flat ground and walls retaining heights greater than 3 m, it is recommended that a minimum surcharge of 10 kPa should be applied to the surface of the retained ground in design. For walls retaining less than 3m, this surcharge load may be reduced provided the designer is confident that a minimum surcharge of 10 kPa will not apply, during the life of the structure.

This is consistent with the requirements of the latest (September 2001) amendment to BS 8002 (1994); see section H2.5, Appendix H.

Highway traffic loading

Guidance is provided in BD 37/88 (DMRB 1.3) on how vehicle loading should be calculated. Normal traffic loading (HA) allows for all combinations of vehicles that normally occur on highways. HB loading corresponds to abnormal vehicle unit loading, eg industrial loads. The number of units of HB loading is usually specified by the Local Authority and depends on the particular usage of the road. Table 5.11 summarises typical loadings.

Table 5.11 *Highways live loadings (from BD 37/88)*

Type of loading	Equivalent surcharge (kPa)
HB (45 units)	20
HB (37.5 units)	16
HB (30 units)	12
HA	10

Railway vehicle loading

Railway universal loading (RU) allows for all combinations of vehicles currently running, or projected to run, on UK railway track. Railway light loading (RL) is a reduced loading for use only on tracks used solely by light rolling stock, eg LUL. Both are applied as a uniformly distributed surcharge over the area occupied by the tracks, ie sleeper end to sleeper end (Perry *et al*, 2001), see Table 5.12.

Table 5.12 *Railway live loading (from Perry et al, 2001)*

Type of loading	Equivalent surcharge (kPa)
R U loading	50
R L loading	30

Adjacent buildings

These should be represented by surcharge loads at the appropriate depths. Loads from adjacent strip foundations or walls should be represented by a line load or surcharge if they run parallel to the wall or by a concentrated load if normal to it. Pad foundations should be represented by surcharge loads of limited extent.

Adjacent stockpiles

These should be represented by surcharge loads of limited extent at the surface of the retained ground. Whenever possible, placing stockpiles adjacent to retaining walls should be avoided.

Construction plant

Loadings will be project specific. Heavy items of plant should be considered individually. For tracked cranes, the total weight of the crane plus the lifting tackle plus the heaviest load to be lifted should be assumed to be taken on one track with a uniform loading on the bearing length of the track. For lorry mounted or wheeled cranes and trucks, the manufacturer's figures for appropriate loadings should be used.

Sloping ground surface

The treatment of sloping ground is discussed in section 4.1.7.

Other loads

These will be project specific and may include loads arising from source (ii) above.

5.6.2 Vertical loads

See also

4.1.4….Wall friction and adhesion

These will be project specific and will depend upon the overall construction sequence adopted for the works. For example, in top down construction, downward loads on the retaining wall may be at their greatest at maximum excavation level in front of the wall. For a wall embedded in stiff clay, the wall may be required to resist uplift forces arising from the effects of ground heave to maintain overall vertical stability. **The designer should carefully consider the effects of vertical loading on the wall in selecting appropriate values for wall friction or adhesion in his design calculation,** (section 4.1.4).

5.6.3 Load case combinations

See also

6….Design of wall

7….Design of support systems

The designer should consider the following load case combinations in his design calculations:

- Construction stage load case

  This applies up to completion of all construction works on the site. The designer should take into account all loads associated with temporary works, construction loads, traffic loads, existing adjacent building surcharges, etc.

- Operational stage load case

  This applies from completion of all construction works on site to the end of the design life of the wall. The designer should consider all current loads and those which are anticipated to arise during the design life of the wall. He should also allow for long term effects such as changes to the groundwater regime and the effects of ground heave movements and pressures in the design of the wall.

- Accidental load case

  This is an extreme load case which can apply at any time during the construction and operational stages. It represents an extreme event such as an impact load on the wall, loss of a prop (partial support) to the wall, a rare flooding event, etc.

Appendix G...Comparison of lumped moment factors with factor on strength method in limit equilibrium calculations

The designer should ensure that the wall provides the required margin of safety against collapse and also provides acceptable performance under working conditions under the action of the construction stage and operational stage load cases. The designer should also check that the wall and its support system can satisfactorily accommodate the accidental load case without unacceptable movements and progressive failure.

The design method, appropriate parameters and factors of safety to be used in each of the above load case combinations are discussed in chapter 6.

5.7 UNPLANNED EXCAVATION OF FORMATION

See also

6….Design of wall

Foreseeable excavations such as service or drainage trenches in front of a retaining wall or dredging in front of a maritime wall structure are examples of *planned* excavations. *Unplanned* excavations arise as a result of unforeseen events.

In setting the planned excavation depth for use in design calculations, the designer should consider the likely tolerance within which the excavation level will be achieved.

The designer should ensure that allowance is made in ultimate limit state calculations for additional unplanned excavation in front of the wall. This should be taken as the *lesser* of:

- 0.5 m; or
- 10% of the total height retained for cantilever walls, or the height retained below the lowest support level for propped or anchored walls.

This is consistent with the general requirements of the latest (September 2001) amendment to BS 8002 (1994), see section H2.5 Appendix H, and EC7 (1995), see section H3 Appendix H.

The above provision for **unplanned excavation should not apply in serviceability limit state calculations.**

5.8 FACTORS

See also

The purpose of design calculations is to ensure:

- satisfactory safety and overall stability of the wall at the ultimate limit state; and
- acceptable deformation and performance at the serviceability limit state.

The purpose of factors is to allow for uncertainty in material properties, loading and calculation models and to ensure safety and acceptable performance.

6.1..............Design philosophy

There are many sources of uncertainty in design calculations. The effect, later in a calculation, of an uncertainty at an earlier point in a calculation is not easily foreseen and may be highly disproportionate (Simpson, 2000). Thus factors should be applied directly to the principal uncertainties in a design, rather than to quantities which are derived later in the calculation. **The number of factors should be as small as possible to minimise confusion and mistakes in calculations.** Therefore, uncertainties in groundwater pressure, geometric parameters (such as formation excavation levels and ground levels in general) and load case combinations should be covered by direct adjustment of these parameters rather than by factors applied elsewhere in the calculation. In geotechnical design, **soil strength is often the principal uncertainty; a factor should be applied to it**.

This approach of applying a factor to soil strength is incorporated in BS 8002 (1994), EC7 (1995) and BD 42/00 (DMRB 2.1.2). This is the recommended method for the design of embedded retaining walls. The soil strength is divided by a factor F_s as follows:

Effective stress

$$\tan \phi'_d = \tan \phi' / F_{s\phi'} \qquad (5.14)$$

$$c'_d = c' / F_{sc'} \qquad (5.15)$$

Total stress

$s_{ud} = s_u / F_{ssu}$ (5.16)

The factored *design* strength parameters (denoted by a subscript d above) are used to derive earth pressure coefficients which increase the earth pressures on the retained side and reduce the earth pressures on the restraining side as the value of F_s increases above unity.

This is discussed further in chapter 6.

Historically, in retaining wall design, moment equilibrium has been used directly or indirectly to ensure that restoring moments exceed overturning moments by a prescribed safety margin. This safety margin has been provided by a single lumped factor or by a factor on soil strength. A number of methods have been adopted. The factors typically adopted in these methods are reviewed and compared with the factor on strength method in Appendix G.

5.9 SELECTION OF DESIGN PARAMETERS

See also

5.9.1...Temporary works design

5.9.2...Permanent works design

6.3.....ULS calculations

6.4.....SLS calculations

6.5.....Progressive failure check

The designer should consider the following in selecting appropriate parameters for use in design calculations:

- geological and other background information, such as data from previous projects
- the variability of the determined values, including differences between the in situ conditions and the properties measured by field and laboratory tests
- the extent of the zone of ground governing the behaviour of the wall at the limit state being considered
- the effect of construction activities on the properties of in situ ground
- changes which may occur in the field due to variation in the environment or weather.

Uncertainty in the selection of soil strength, stiffness, loads and geometric parameters are of particular importance in retaining wall design. The risks of soil strength and stiffness being less or greater than assumed, or surcharge loads being greater, or of over-excavation or a rise in groundwater pressures occurring, influence the factor of safety appropriate for design.

Figure 5.15 *Design parameters - definition of terms*

Three design approaches are discussed in this report:

Design Approach A	**Moderately conservative** soil parameters, groundwater pressures, loads and geometry are selected and safety factors are applied. The term moderately conservative is a cautious estimate of the value relevant to the occurrence of the limit state (Figure 5.14). It is considered to be equivalent to *representative* values as defined in BS 8002 (1994) and to *characteristic* values as defined in EC7 (1995). This should not be confused with characteristic values (5% fractile) adopted in structural engineering for material properties. Box 5.2 provides guidance on determining a typical moderately conservative profile of soil undrained shear strength.
Design Approach B	**Worst credible** soil parameters, groundwater pressures, loads and geometry are selected and safety factors lower than those in Design Approach A are applied. The worst credible value is the worst which the designer can reasonably believe might occur - a value which is very unlikely. As a guide, Simpson *et al* (1979) state that it can be regarded as the 0.1% fractile (Figure 5.14). **Design Approach B is not appropriate for SLS calculations.**
Design Approach C	**Most probable** soil parameters, groundwater pressures, loads and geometry are selected and the safety factors of Design Approach A are adopted. Most probable values have a 50% probability of exceedance (Figure 5.14). **Design Approach C should only be used within an Observational Method process.** It should be used in conjunction with Design Approach B (to enable contingency measures to be developed for rapid implementation in the event that conditions actually encountered are less than the most probable). **It is not acceptable to proceed solely on the basis of Design Approach C.** The cost saving arising from the adoption of Design Approach C should be offset against the costs relating to the additional calculations to Design Approach B and those associated with the development of contingency measures and the additional monitoring and measurement systems necessary for the implementation of the Observational Method (section 5.9.1). A detailed description of the principles and application of the Observational Method is provided in CIRIA report 185 by Nicholson *et al* (1999).

Figure 5.14 defines the terms moderately conservative, worst credible and most probable.

Statistical methods can be used to derive parameters appropriate for use in each of the above design approaches. However, such methods should not be used uncritically. Any anomalous field or laboratory test result should be considered carefully in order to establish whether it is misleading or represents a real phenomenon which should be accounted for in design.

Simpson and Driscoll (1998) present an example of the typical considerations necessary to derive a moderately conservative design profile through available undrained shear strength data for London Clay. This is reproduced below in Box 5.2.

Box 5.2 *Moderately conservative profile of undrained shear strength (after Simpson and Driscoll 1998)*

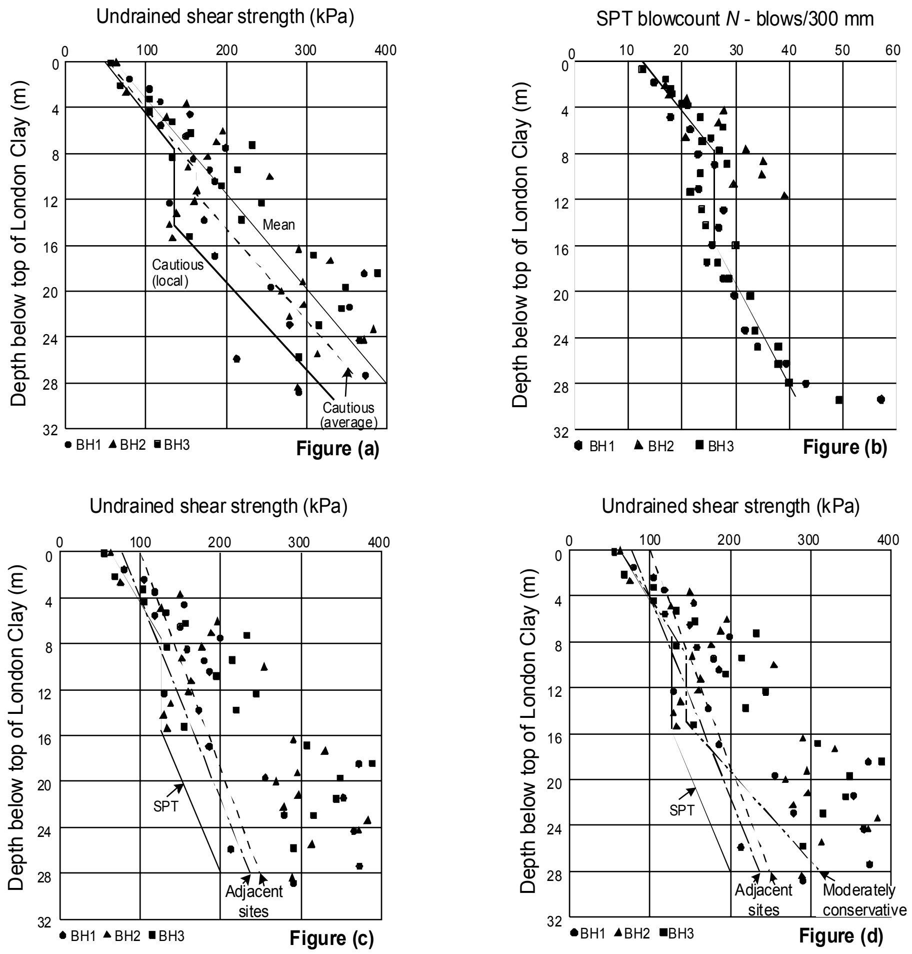


Figure (a) shows the results of a series of undrained shear strength measurements in London Clay. The measurements were made using unconsolidated undrained triaxial tests. A statistical mean line has been drawn through the data and it is clear that undrained strength increases with depth.

A moderately conservative line is required, and this should depend on how the values will be used - what is the limit mode being considered? For example, if the undrained strength is needed for calculations of ground movements around a retaining wall, a value such as the 'cautious (average)' value shown on the figure could be used. However for a problem in which failure might take place in a small zone of soil, a more cautious value - the 'cautious (local)' value should be adopted. From these boreholes, results from standard penetration tests were also available in Figure (b). In London Clay, Stroud's (1989) empirical correlation applies between standard penetration test blowcount and undrained shear strength results. However, if the mean line from the SPT results is transferred onto the undrained shear strength plot, as in Figure (c), it appears that the normal correlation does not work. In fact the measured undrained strengths are remarkably high: they are consistent with very low water contents, which were measured, but this might simply mean that the samples had dried out on the way to the laboratory, though there was no reason to suspect this. Figure (c) also shows lines representing mean values through data from other nearby sites, both for undrained shear strength, and SPT results. Stroud's (1989) correlation applies to these, and it is clear that the undrained strengths for the new site are remarkably high.

On the basis of these inconsistent data sets, what value should be used as moderately conservative undrained strength? The values measured in the triaxial tests should not be ignored, but the SPT results and the data from the adjacent sites should also affect the decision. The moderately conservative value proposed for these data is shown in Figure (d). This is less than the initial assessment in Figure (a), which was based on the triaxial results only, and is closer to a lower bound of this particular set of triaxial results.

Designers should follow this sort of process when trying to interpret real data. It may be that statistical methods could trace a similar logical sequence. However, this would require quite advanced methods and any statistical approach which failed to take account of the diverse array of data, typically available, would be harmful to the design process.

5.9.1 Temporary works design

See also

2.1.2..........Contractual requirements

2.1.3..........Risk assessment and management

4.1.6..........Tension cracks

5.5.1..........Undrained conditions

5.5.2..........Drained conditions

5.5.3..........Mixed undrained and drained conditions

Figure 5.15...Temporary works design assumptions

5.6.3..........Load case combinations

The wall should be designed for the construction stage and the accidental load cases (section 5.6.3).

As discussed in sections 2.1.2 and 2.1.3, the approach taken for **temporary works design is a matter of risk assessment, management and mitigation. It is very important that the roles and responsibilities of each of the parties (client / owner, designer, contractor, subcontractor, etc.) are clearly defined and fully understood by all parties in respect of the design and construction of the temporary (and permanent) works.**

The design of temporary works should be demonstrably robust. This robustness should be provided by identifying the hazards and risks and ensuring that the temporary works design adequately addresses them. The principal risks lie in predicting ground behaviour and dealing with parameter uncertainty. For example:

- will the ground respond in a drained or undrained manner, or a combination of both, during construction?
- if undrained, will tension cracks develop? Are these likely to be dry or full of water?
- what groundwater regime will apply during construction?
- what provision should be made to allow for deterioration in soil strength parameters due to the installation of the wall and its support system and disturbance due to excavation and partial drainage during the period of construction?
- what measures or controls will be applied to confirm that:
  - the loading conditions assumed in design will not be exceeded?
  - unplanned excavation of the formation will not occur?

- the actual support (props, anchors, berms, etc) to the wall will be as assumed in design?
- the actual wall performance (deflection, watertightness, etc) will remain within pre-defined limits?

The assumptions which are made in temporary works design in respect of the above depend upon how the associated risks are mitigated during construction. Risk mitigation is undertaken through close control of the process of design and construction of the temporary works on site. This requires an integrated interactive approach between design and construction on site with the ability to adapt the design quickly to suit changes in construction methods and vice versa. Appropriate contractual arrangements should be in place to permit this.

Figure 5.16 *Temporary works design assumptions*

In terms of risk, three possible sets of assumptions are possible for the design of temporary works in stiff clays:

- undrained conditions on both sides of the wall
- mixed undrained and drained conditions
- drained conditions on both sides of the wall with a steady state groundwater regime.

Each of these is discussed below.

Undrained conditions on both sides of the wall

The assumption of undrained conditions on both side of the wall is associated with significant risk. The appropriate mitigation of this risk requires significant control and vigilance on site to ensure that design assumptions are realised in practice. This is only feasible where the designer of the temporary works has either direct control over construction activities on site or has significant influence over such activities to enable:

- the design to be quickly modified to suit construction methods on site and vice versa
- contingency measures to be expeditiously implemented on site.

It is acceptable to proceed on the basis of Design Approach A in conjunction with comprehensive and robust instrumentation and monitoring of the works. The results of the monitoring should be regularly reviewed by the designer and should form the basis of design decisions during construction.

The Observational Method (Design Approach C in conjunction with Design Approach B, see section 5.9) is likely to result in the most cost effective design solution.

Implementation of the Observational Method requires:

- a non-brittle failure mechanism ie adequate warning of approaching an ultimate limit state by visible signs
- active involvement by all parties in the risk strategies inherent in the adoption of the Method. This should include regular pre-arranged review meetings with named key staff with clearly established specific responsibilities

- primary instrumentation and monitoring which is robust, reliable, simple and quick
- secondary instrumentation and monitoring which reliably confirms the results of the primary monitoring
- contingency plans (designed on the basis of Design Approach B) which are fully developed prior to the start of the work on site for implementation if monitoring indicates that conditions are worse than expected
- fully developed emergency plans for unexpected conditions, see Nicholson *et al* (1999).

The adoption of the Observational Method allows further savings to be made through progressive modification of the design. For example, in the case of a project involving a long retaining wall, observations from the initial construction phases can be fed back into the design of later phases.

The Observational Method should not be used where:

- there is insufficient time to fully develop and implement the contingency and emergency plans
- observations of actual wall performance would be difficult to obtain or are unreliable.

Tension cracks

Tension cracks should be assumed on the retained side of the wall in accordance with sections 4.1.6 and 5.5.1. For the conditions discussed in section 5.5.1, there are two situations; dry tension cracks and flooded tension cracks. Figure 5.16 (a) and (b) summarise the principal design assumptions in such conditions:

- if water is not expected, total stress analysis can be adopted to calculate the earth pressures acting on the retained and the restraining sides of the wall. On the retained side, negative values of calculated earth pressure should always be set to zero and a minimum horizontal total stress of 5 z kPa should be applied (sections 4.1.6 and 5.5.1). The pressures should be assumed to be given by the minimum horizontal total stress of 5 z kPa or by total stress analysis, whichever is greater, see Figure 5.16 (a)
- if water is expected to be present, full hydrostatic water pressure should be assumed on the retained side to the depth shown in Figure 5.16(b).

For an embedded wall propped or anchored near or at its top where the total stress on the retained side at the same level as the prop or anchor is greater than $\gamma_w z$ kPa over a minimum depth of 1m, the development of a flooded tension crack in a uniform homogeneous isotropic clay stratum may be discounted (sections 4.1.6 and 5.5.1). This will only be appropriate where the designer is confident that water cannot enter the tension crack by other means (section 4.1.6).

Softening of restraining soil

The value of the undrained shear strength, s_u, of the restraining soil should be assumed to be reduced to zero at excavation level rising to s_u at a depth of L to allow for excavation disturbance and dissipation of excess pore water pressure at excavation level, where:

$L = 0.5$ m	where there is no potential for groundwater recharge either at excavation level or within the soil
$L = \sqrt{(12 c_v t)}$	where recharge occurs at excavation level but with no recharge within the soil. c_v is the coefficient of consolidation and t is the elapsed time.

The above values of L are based on simple idealised analysis. The designer should satisfy himself that the value of L is appropriate for the particular circumstances of the project under consideration.

The value of s_u adopted on the restraining side should be judiciously selected to allow for the stress conditions likely to be applicable beneath the excavation (triaxial extension), the quality and reliability of the available data and the prospect of any softening during the duration of the temporary works. The commonly adopted reduction of 20% to 30% in the value of s_u in the restraining soil in London Clay may be too severe in some circumstances. The designer should consider all of the above factors in selecting an appropriate value for s_u. Previous experience of comparable construction should be used wherever possible.

Drained conditions on both sides of the wall

This is a low risk approach and may be appropriate where rapid softening and drainage of the ground on both sides of the wall is expected or where the designer of the temporary works may have little influence on activities on site. Design Approaches A or B can be adopted, as appropriate. Implementation of the Observational Method would not be appropriate where the designer has little influence on activities on site.

Effective stress analysis will apply on both sides of the wall in conjunction with the groundwater pressures discussed in section 5.5.2. This is illustrated in Figure 5.16 (c).

Mixed undrained and drained conditions

For the case where undrained total stress conditions are considered to apply on one side of the wall while drained effective stress conditions apply on the other, design Approaches A, B or the Observational Method can be adopted, depending upon the degree of involvement of the designer in the monitoring and management of construction activities on site.

For example, in the conditions discussed in section 5.5.3, undrained total stress conditions will apply in the restraining soil and below the drained soil on the retained side, while drained effective stress conditions will apply in the clay with permeable water bearing silt/sand horizons on the retained side in conjunction with the groundwater pressures discussed in section 5.5.3. This is illustrated in Figure 5.16 (d).

5.9.2 Permanent works design

If the permanent wall is utilised to provide temporary support during construction, then the design considerations discussed in section 5.9.1 will also apply to the permanent wall in the short term.

In the long term, fully drained conditions will prevail on both sides of the wall and effective stress analysis will apply, see Figure 5.15 (c). The groundwater pressures adopted in design should allow for long term effects such as changes due to long term climatic variations, rise in underlying aquifer water levels, etc. The wall should be designed for the operational and accidental load cases (see section 5.6.3).

5.10 KEY POINTS AND RECOMMENDATIONS

1. Design parameters relate to:

   - soil and groundwater conditions
   - loading conditions
   - geometry
   - factors to ensure safety, to allow for uncertainty in material and soil properties, loading and calculation models and to ensure acceptable deformations.

2. The designer should ensure that he understands the requirements of the input parameters for the envisaged design calculations.

3. Knowledge of the soil density (unit weight) and shear strength is *essential* in the design of an embedded retaining wall. An understanding of the in situ stress conditions and stiffness is also required for soil-structure interaction analysis.

4. A site investigation, which is appropriate to the geotechnical category of the retaining wall, should be carried out. Irrespective of the method of procurement of the site investigation, the designer should check that an adequate desk study and ground investigation has been carried out.

5. The designer should fully understand the geological and groundwater conditions at the site, in particular:

   - the geological process and geological history which formed the geology of the site and its surrounding area
   - the likely variations in the stratigraphic conditions across the site
   - the interbedded nature of the deposits including both the micro and macro fabric
   - hydrogeological conditions
   - the groundwater pressure profile at the site and extreme levels of any free water which may influence the groundwater pressures.

6. The designer should assess whether the ground conditions are likely to be drained or undrained in the short term. Use should be made of previous experience of construction in similar ground conditions. As a general guide undrained conditions may be assumed in the short term where the mass in situ permeability of the ground is low (ie a coefficient of permeability of the order of 10^{-8} m/s or less).

7. Soil parameters should be determined from a number of independent sources:

   - directly from the results of in situ and laboratory tests, making due allowance for the effects of installation and sample disturbance

- from established empirical correlations between different types of in situ and laboratory tests and with the soil's grading and index properties
- from relevant published data and local and general experience
- wherever possible, from back analysis of measurements taken from comparable full scale construction in similar ground conditions.

8. It is difficult to accurately assess the in situ stress in stiff overconsolidated soils. The value of K_o should be determined from one or more of the following methods:

   - established correlations between K_o and soil index and strength properties listed in section 5.4.3
   - the stress-history approach described by Burland *et al* (1979)
   - suction tests carried out on thin walled samples
   - in situ self boring pressuremeter, in situ self boring load cell pressuremeter and/or spade total stress cells
   - back analysis of comparable construction in similar ground conditions.

   The value of K adopted in design calculations should allow for the effects of wall installation. In general, it may be appropriate to adopt the following K values in simple elastic soil-structure interaction analyses (section 4.1.2):

   - for normally consolidated soils: $K = K_{onc}$ from equation 5.1
   - for overconsolidated fine grained soils: $K = 1.0$
   - for overconsolidated coarse grained soils: $K = 1.0$ for walls installed by non-displacement methods (eg bored pile walls, diaphragm walls).

9. There are several different soil strengths in stiff overconsolidated soils: peak, critical state, residual and, drained or undrained. The designer should decide which is appropriate for a particular analysis before considering other issues such as reliability, selection of values for use in design and factors of safety.

10. Single stage unconsolidated undrained triaxial tests should be carried out on 100mm diameter undisturbed soil samples to measure undrained shear strength in the laboratory. Unconfined tests should not be carried out. Multistage testing should not be carried out. Where suctions are measured in samples in the laboratory, and these are found to be significantly different from the estimated corresponding mean effective stress, the value of s_u should be treated with caution. Laboratory determination of s_u should be supplemented with in situ testing, eg static cone penetration testing and/or, standard penetration testing, etc.

11. The drained shear strength should be determined from laboratory consolidated drained and consolidated undrained triaxial tests and shear box tests (for coarse grained soils). Peak, critical state and residual angles of shearing resistance should be determined. Strain rates in drained tests should be sufficiently slow to ensure complete dissipation of excess pore pressure. Consolidation pressures should be chosen to represent a reasonable range of values which are appropriate for the wall being designed.

12. It is not good practice to rely upon a single method to determine soil stiffness but rather to use several different approaches.

The self boring pressuremeter is probably the most robust means of determining soil stiffness at strains relevant for wall design across a broad range of over-consolidated clays and very weak rocks in the UK.

The values of stiffness used in soil-structure interaction computer programs for SLS calculations should be derived from high quality site specific information (eg in situ self boring pressuremeter tests, geophysical tests, a small strain laboratory tests). The computer models should be "calibrated" against good quality comparable case history data. Where high quality site specific information is not available, it may be appropriate to use relevant published data in conjunction with a "calibrated" soil structure interaction model.

Under ULS conditions, for collapse calculations, wall deformations are comparatively large. Under such conditions, soil strength is likely to be fully mobilised and soil stiffness will be of a secondary importance. This is particularly applicable to cantilever and singly propped or anchored embedded walls. However, for multi-propped walls, where soil strength is not fully mobilised, soil stiffness may have a significant effect on ULS collapse calculations. The magnitude of soil stiffness applicable to ULS calculations is difficult to estimate and will depend upon the applicable strains. In general, it is often appropriate to adopt values of soil Young's moduli which are approximately 50% of those which are adopted in SLS calculations.

13. The groundwater pressures assumed in design calculations will have a significant effect on the design of a retaining wall. Groundwater pressures should be determined on the basis of the considerations listed in section 5.5. The designer should determine:

    (i) water pressures and seepage forces which represent the most unfavourable values which could occur in *extreme* or *accidental* circumstances at each stage of the wall's construction sequence and throughout its design life. An example of an extreme or accidental event may be a burst water main in close proximity of the wall

    (ii) water pressures and seepage forces which represent the most unfavourable values which could occur in *normal* circumstances at each stage of the wall's construction sequence and throughout its design life. Extreme events such as a nearby burst water main may be excluded, unless the designer considers that such an event may reasonably occur in normal circumstances.

    The groundwater pressure corresponding to (i) above should apply to ULS considerations while (ii) should apply to SLS considerations. This is discussed further in chapter 6.

14. In undrained conditions:

    - where water is not expected, the total horizontal pressure acting on the retained side of the wall at any depth below the ground surface should be assumed to be not less than 5 z kPa (where z is depth below ground level on the retained side of the wall)
    - where water is expected to be present, tension cracks should be assumed to be filled with water to their full depth. A minimum total horizontal pressure of $\gamma_w z$ should be adopted on the retained side of the wall to the depth shown in Figure 5.16(b). For an embedded wall propped or anchored near or at its top, the

possibility of a flooded tension crack developing behind the wall in a uniform homogeneous isotropic clay stratum may be discounted if it can be demonstrated that the lateral stress is greater than the hydrostatic pressure of water at the same level over a minimum depth of 1m near the top of the wall and that water cannot enter the tension crack by other means (section 4.1.6).

15. The wall should be designed for loading which is appropriate under the particular circumstances where it is located based on careful consideration of the following:

    - loads applied directly to the wall
        - vertically (eg from a top down construction sequence, inclined supports or anchors)
        - horizontally (eg out of balance forces)
    - loads from adjacent roads (Table 5.11), railways (Table 5.12), buildings and construction activities (section 5.6.1)
    - loads from variations in ground surface levels.

    For flat ground, and walls retaining heights greater than 3 m a minimum surcharge of 10 kPa should be applied at the surface of the retained ground. This surcharge may be reduced for walls retaining less than 3 m where the designer is confident that a minimum surcharge of 10 kPa will not apply during the life of the structure.

16. The designer should consider the following load case combinations in his design calculations:

    - temporary works and permanent works in the short term: construction stage load case and accidental load case
    - permanent works in the long term: operational stage load case and accidental load case.

17. The designer should ensure that allowance is made in ultimate limit state calculations for additional *unplanned* excavation in front of the wall. This should be taken as the *lesser* of:

    - 0.5m; or
    - 10% of the total height retained for cantilever walls, or the height retained below the lowest support level for propped or anchored walls.

    The provision for unplanned excavation should not apply in serviceability limit state calculations.

18. The purpose of factors is to allow for uncertainty in material properties, loading and calculation models and to ensure safety and acceptable performance. The number of factors should be as small as possible to minimise confusion and mistakes in calculation. Uncertainties in groundwater pressures, geometry and load case combinations should be covered by direct adjustment of these parameters rather than by factors applied elsewhere in the calculation. Soil strength is often the principal uncertainty and a factor should be applied to it. The factor on strength method should therefore be adopted in design.

19. Three design approaches are possible in temporary and permanent works design:

- Design Approach A — **Moderately conservative** soil parameters, groundwater pressures, loads and geometries are selected.
- Design Approach B — **Worst credible** soil parameters, groundwater pressures, loads and geometries are selected.
- Design Approach C — **Most probable** soil parameters, groundwater pressures, loads and geometries are selected.

Use of the Observational Method (Design Approach C in conjunction with contingency measures developed on the basis of Design Approach B) is likely to result in the most cost effective design solution for temporary works and permanent works which are used to provide temporary support during construction. For this to be feasible, appropriate contractual arrangements should be in place to permit the necessary integrated interactive approach to be taken on site between design and construction.

A more cautious, less economic, design solution will arise where the designer has little influence over activities on site.

20. In design calculations under undrained conditions, the value of the undrained shear strength, s_u, of the restraining soil should be assumed to be reduced to zero at excavation level rising to s_u at a depth of L to allow for excavation disturbance and dissipation of excess pore water pressure at excavation level.

where,

$L = 0.5$ m	where there is no potential for groundwater recharge either at excavation level or within the soil
$L = \sqrt{(12\, c_v t)}$	where recharge occurs at excavation level but with no recharge within the soil. c_v is the coefficient of consolidation and t is the elapsed time.

The above values of L are based on simple idealised analysis. The designer should satisfy himself that the value of L is appropriate for the particular circumstances of the project under consideration.

The value of s_u adopted on the restraining side of the wall should be judiciously selected to allow for the stress conditions likely to be applicable beneath the excavation, the quality and reliability of the available data and the prospect of any softening over the duration of the temporary works. The commonly adopted reduction of 20% to 30% in the value of s_u in the restraining soil in London Clay may be too severe in some circumstances. Previous experience of comparable construction should be used wherever possible.

6 Design of wall

See also

5….Determination and selection of parameters

7….Design of support systems

Appendix H...Review of current design methods to BS 8002, EC7 and BD 42/00

This chapter is intended for the geotechnical and structural designer: consulting engineers, contractors and those involved in the design of temporary and permanent works. It assumes that the reader has some experience of embedded retaining wall design and knowledge of the requirements of British codes of practice relevant to retaining walls.

The code of practice applicable to the design of retaining structures in the UK is BS 8002 (1994); the latest amendment to this code was issued in September 2001. There is no legal requirement that the design of a retaining wall has to strictly comply with the requirements of BS 8002 (1994). BS 8002 (1994) allows the designer to apply the results of research and to take advantage of special situations or previous experience in the design of retaining structures. The design of retaining walls supporting Highways Agency structures are required to comply with BD42/00 (DMRB 2.1.2) which has some mandatory requirements that are necessary for approval purposes. Project specific specifications may also impose their own design rules and procedures; these usually take guidance from relevant codes of practice and Eurocodes, such as EC7 (1995). The designer of a retaining wall in the UK should therefore be familiar with the requirements of BS 8002 (1994), EC7 (1995) and BD42/00 (DMRB 2.1.2). Appendix H describes and compares the methods currently advocated in BS 8002 (1994), EC7 (1995) and BD 42/00 (DMRB 2.1.2).

This chapter provides guidance on the geotechnical and structural design of the wall. Figure 6.1 outlines the design method recommended in this chapter. This is discussed in detail in section 6.2 and the design procedure is illustrated in a worked example in Appendix K.

Figure 6.1 *Design method*

6.1 DESIGN PHILOSOPHY

See also

2…..Design considerations

5.8...Factors

5.9...Selection of parameters

The limit state design philosophy described in chapter 2 should be adopted. Design calculations should satisfy the ultimate limit states (ULS) of wall stability and structural strength and the required serviceability limit states (SLS) by verifying satisfactory wall performance in respect of wall deflections, associated ground movement, wall watertightness criteria, etc. The designer should demonstrate that the exceedance of either ultimate or serviceability limit states is sufficiently improbable in the envisaged design situations.

The factor F_s should be applied on soil strength (section 5.8). The magnitude of F_s should be selected from Table 6.1. The soil *design* parameters derived therefrom should be used in conjunction with the groundwater pressures, loads and design geometries stated in sections 6.3, 6.4 and 6.5 for collapse (ULS) calculations, SLS calculations and the accidental design situation respectively.

Table 6.1 *F_s factors appropriate for use in design calculations*

Design Approach[3]	Ultimate limit states			Serviceability limit states		
	Effective stress		Total Stress	Effective stress		Total stress
	$F_{sc'}$	$F_{s\phi'}$	F_{ssu}	$F_{sc'}$	$F_{s\phi'}$	F_{ssu}
A: Moderately Conservative	1.2	1.2	1.5	1.0	1.0	1.0
B: Worst Credible	1.0	1.0	1.0	see note (4)	see note (4)	see note (4)
C: Most Probable	1.2	1.2	1.5	1.0	1.0	1.0

Notes

1. Design $\phi'_d = \tan^{-1} (\tan\phi' / F_{s\phi'})$

   Design $c'_d = c' / F_{sc'}$

   Design $s_{ud} = s_u / F_{ssu}$
2. The *design* strength parameters in note (1) above are used to derive earth pressure coefficients.
3. Refer to section 5.9 for a full definition of each design approach.
4. Not appropriate for SLS calculations.

6.2 DESIGN METHOD

See also

- 4.2…Methods of analysis
- 4.3...Effect of method of analysis on economy
- 4.4...Key points and recommendations
- 6.3…ULS calculations
- 6.4…SLS calculations
- 6.5…Progressive failure check
- 6.6…Structural design of wall
- 7…..Design of support systems

A clear unambiguous design method is essential in minimising confusion and maximising efficiency, thereby achieving economy in design. Figure 6.1 outlines the design method. Box 6.1 lists the sequential steps in the design process. Depending upon project requirements, it may not be necessary to carry out calculations for SLS and the accidental design situation/progressive failure check (sections 6.4 and 6.5).

In ultimate limit state (ULS) considerations, the designer should separately consider Design Approach A (moderately conservative soil parameters with $F_s = 1.2$ applied on c' and $\tan \phi'$ or $F_s = 1.5$ on s_u) and Design Approach B (worst credible soil parameters with $F_s = 1.0$) at the stage of selecting soil design parameters. By inspection, the more onerous of these two sets of parameters should be selected and *one* design calculation should be undertaken to calculate the wall embedment depth, wall bending moments, shear forces and prop loads for the assumptions listed in section 6.3. The wall design calculation should therefore adopt the more onerous of the soil design parameters derived from either Design Approach A or Design Approach B. Design Approach C should only be adopted as part of the Observational Method.

Where SLS calculations are required, the designer should carry out *one* design calculation to calculate the SLS load effects (wall bending moments, shear forces and prop loads) using unfactored (strictly $F_s = 1.0$) soil parameters for the assumptions listed in section 6.4. This calculation should be carried out using either Design Approach A or Design Approach C (if the Observational Method is adopted in design). Design Approach B is not appropriate for SLS conditions.

The designer should choose whether to carry out limit equilibrium calculations or soil-structure interaction analysis. Guidance is provided in sections 4.2 and 4.3 and is repeated here for clarity. In circumstances where there is little or no stress redistribution, eg cantilever walls, limit equilibrium calculations and soil-structure interaction analyses are likely to give similar wall embedment depths and wall bending moments. For propped or anchored walls where stress redistribution is likely, more economic design (shorter walls and smaller wall bending moments) will be obtained from the results of soil-structure interaction analyses. Soil-structure interaction analysis should be carried out where it is difficult to establish simple earth pressure distributions appropriate for use in limit equilibrium calculations (eg walls which are singly propped at low level and multi-propped walls) and where there is a requirement to undertake SLS analyses (eg for the determination of wall deflections and ground movements, compliance with specified crack width criteria for reinforced concrete walls, compliance with allowable stress criteria for steel sheet pile walls, if applicable).

It is sensible to carry out some simple calculations as an approximate check on more advanced methods (eg wherever possible it is prudent to carry out simple limit equilibrium calculations with appropriate simplifying assumptions to obtain a conservative bound prior to carrying out complex finite element or finite difference analyses).

In general, the designer should undertake the sequential steps listed in Box 6.1:

Box 6.1 *The design method: sequential steps*

1. establish site constraints: site access, constructability constraints, etc (section 2.3)
2. establish ultimate limit states (eg overall wall stability and structural strength) and serviceability limit states (eg limiting values for wall deflections, ground movements, wall watertightness, crack width criteria, allowable stress criteria and durability) from section 2.4
3. review ground and groundwater conditions (chapter 5)
4. select construction sequence and wall type (chapter 3)
5. assign geotechnical category to wall (section 2.2.2 and Figure 2.4)
6. determine design stratigraphy (section 5.2.2)
7. determine soil parameters, groundwater pressures, load case combinations and design geometry appropriate for ultimate limit state (ULS) calculations. Detailed guidance is provided in section 6.3
8. determine the minimum wall depth for vertical stability requirements, eg ensure that it has adequate load bearing capacity if vertically loaded and provides the required hydraulic cut-off or uplift resistance (section 6.3.5)
9. determine the direction and magnitude of the wall friction and adhesion required to satisfy vertical stability and ensure that this is consistent with the lateral stability calculations (section 4.1.4)
10. carry out collapse (ULS) calculations using limit equilibrium methods or soil-structure interaction analysis
11. adopt the *deeper* of the wall depth determined in steps 8 and 10 as the *design* wall depth
12. If the *design* wall depth is determined from step 8, adopt the *design* wall depth and carry out ULS limit equilibrium calculations or soil-structure interaction analysis to determine wall bending moment, shear force and prop load

    If the *design* wall depth is determined from step 10, review the direction and magnitude of the assumed wall friction and adhesion at the soil/wall interface to satisfy the requirements of step 9 and carry out ULS limit equilibrium or soil-structure interaction analysis to determine wall bending moment, shear force and prop load

13. Determine soil parameters, groundwater pressures, load case combinations, and design geometry appropriate for SLS calculations. This step may be omitted if SLS calculations are not required. Detailed guidance is provided in section 6.4

14. for the wall depth established in step 11, carry out SLS calculations using limit equilibrium methods or soil-structure interaction analysis in accordance with section 6.4.5 to determine SLS load effects (wall bending moment, shear force and prop or anchor loads). This step may be omitted if SLS calculations are not required (section 6.4)

15. determine wall deflections and ground movements from the results of SLS soil-structure interaction analysis (if undertaken) and empirical correlations with comparable case history data from Figure 2.14, chapter 2. This step may be omitted if wall deflections and ground movements are not of importance

16. check that the SLS load effects, wall deflections and ground movements determined in step 15 comply with the wall design requirements and performance criteria established in step 2 (eg check compliance with crack width criteria for reinforced concrete walls, compliance with allowable stress criteria for steel sheet pile walls, if appropriate)

17. carry out building damage assessment in accordance with the procedure outlined in Figure 2.18, chapter 2. This step may be omitted if not relevant

18. determine soil parameters, groundwater pressures, accidental load case and design geometry appropriate for a check against progressive failure. Detailed guidance is provided in section 6.5. This step may be omitted if not relevant

19. for the assumptions made in step 18, carry out calculations using limit equilibrium methods or soil-structure interaction analysis to confirm overall lateral stability. This step may be omitted if not relevant (section 6.5)

20. determine ULS wall bending moments (BM) and shear force (SF) appropriate for the structural design of the wall as the greater of:

    - BM and SF from step 12
    - 1.35 times the BM and SF values determined from step 14 (if undertaken)
    - BM and SF from step 19 (if undertaken).

21. determine SLS and ULS prop or anchor forces appropriate for the structural design of the prop or anchor from the above calculations using the procedure described in section 7.1.3.

Design steps 1 to 12 and step 20 should be undertaken for all embedded walls. Depending upon project requirements, some or all of steps 13 to 19 may be omitted particularly where calculations for SLS and the accidental design situation/progressive failure check are not undertaken. Detailed guidance on the circumstances under which SLS calculations and the accidental design situation/progressive failure check should be carried out is provided in sections 6.4 and 6.5. Design step 21 should be undertaken for all propped or anchored walls.

Further design and construction issues relating to sheet pile walls are considered in section 6.6.3.

Further design and construction issues relating to reinforced concrete walls are considered in section 6.6.4.

6.3 ULS CALCULATIONS

Collapse (ULS) calculations can be carried out using limit equilibrium methods or soil-structure interaction analysis. Their principal purpose is to determine the wall embedment depth and strength for overall stability. The strength of an embedded wall should be sufficient to use its full depth.

6.3.1 Soil design parameters

See also

4.1.4....Wall friction and adhesion

4.1.6.....Tension cracks

5.4.4.....Shear strength

5.5.1....Undrained conditions

5.9.1....Temporary works design

The soil *design* parameters which should be used to obtain earth pressures coefficients should be derived as follows:

Design Approach A

Effective stress analysis

Design $\phi' = \phi'_{d} = \tan^{-1} (\tan \phi'_{mc} / 1.2)$

Design $c' = c'_{d} = c'_{mc} / 1.2$

where:

ϕ'_{mc} = moderately conservative value of angle of shearing resistance available when the wall is on the point of, or approaching, ultimate failure. This could be peak, critical state or residual

c'_{mc} = moderately conservative value of apparent cohesion intercept (section 5.4.4).

The limiting value of wall friction, δ_{max}, should be taken to be:

$$\delta_{max} \leq k\ \phi'_{crit,\ mc}$$

where:

$\phi'_{crit,\ mc}$ = moderately conservative critical state angle of shearing resistance

k = 1.0 for *rough* concrete (eg concrete cast directly against soil) and for a rupture surface within the soil;

k = 0.67 for *smooth* concrete (eg precast concrete or concrete cast against formwork) and other smooth surfaces (eg steel) and for driven or jacked in walls.

The wall friction angle used in design calculations, δ_{d}, should be derived on the basis of the considerations listed in section 4.1.4.

The value of the design effective wall adhesion, s'_{wd}, should be taken as zero.

The earth pressure coefficients, K_a and K_p, used in design calculations should be derived from the charts / equations presented in Appendix F for the appropriate values of ϕ'_{d} and the ratio (δ_d / ϕ'_d). The values of k proposed above assume that earth pressure coefficients will not be derived by other, less conservative, means such as Coulomb (1776).

Total stress analysis

Design $s_u = s_{ud} = s_{umc}$ / 1.5

where,

s_{umc} = moderately conservative value of undrained shear strength, s_u.

The limiting value of wall adhesion, s_{wmax}, should be taken as:

$s_{wmax} = \alpha$ s_{ud}

where:

α = 0.5 in stiff clay (section 4.1.4). Smaller values of α may apply in particular circumstances eg steel sheet piles driven through overlying soft clay.

Tension cracks should be assumed on the retained side of the wall in accordance with sections 4.1.6, 5.5.1 and 5.9.1. The value of s_{ud} of the restraining soil should be assumed to be reduced to zero at excavation level rising to s_{ud} at depths of 0.5m and $\sqrt{(12 c_v t}$ for no recharge and full recharge at excavation level respectively, in conjunction with zero recharge from within the soil itself (section 5.9.1). The designer should satisfy himself that these depths are appropriate for the particular circumstances of the project under consideration. He should make further reductions in the value of s_{ud} on the restraining side of the wall if he considers that the ground and groundwater conditions are such that more significant softening may occur (section 5.9.1).

Design Approach B

Effective stress analysis

Design $\phi' = \phi'_d = \tan^{-1}$ ($\tan \phi'_{wc}$ / 1.0)

Design $c' = c'_d = c'_{wc}$/1.0

where:

ϕ'_{wc} = worst credible value of angle of shearing resistance. This will generally correspond to ϕ'_{crit} of the worst graded material which could possibly occur in sufficient quantity to govern the behaviour, but may be $\phi'_{residual}$ where a pre-existing polished rupture surface represents a worst credible failure mechanism

c'_{wc} = worst credible value of apparent cohesion intercept (section 5.4.4).

The limiting value of wall friction, δ_{max}, should be taken as:

$\delta_{max} \leq k\ \phi'_{crit,wc}$

where:

$\phi'_{crit, wc}$ = worst credible critical state angle of shearing resistance

k = the limiting values stated under Design Approach A.

The value of the design wall adhesion, s'_{wd}, should be taken as zero.

Total stress

Design $s_u = s_{ud} = s_{uwc} / 1.0$

where:

s_{uwc} = worst credible value of undrained shear strength, s_u.

The limiting value of wall adhesion, s_{wmax}, should be taken as:

$s_{wmax} = \alpha\, s_{ud}$

where:

α = 0.5 in stiff clay (section 4.1.4). Smaller values of α may apply in particular circumstances eg steel sheet piles driven through overlying soft clay.

The treatment of tension cracks, excavation disturbance and softening on the restraining side of the wall should be as for Design Approach A.

Design Approach C

This should only be used in design as part of the Observational Method (Nicholson *et al*, 1999).

Effective stress analysis

Design $\phi' = \phi'_d = \tan^{-1} (\tan \phi'_{mp} / 1.2)$

Design $c' = c'_d = c'_{mp} / 1.2$

where:

ϕ'_{mp} = most probable value of angle of shearing resistance available when the wall is on the point of, or approaching, ultimate failure. This could be peak, critical state or residual

c'_{mp} = most probable value of apparent cohesion intercept (section 5.4.4).

The limiting value of wall friction, δ_{max}, should be taken as:

$\delta_{max} \leq k\ \phi'_{crit, mp}$

where:

$\phi'_{crit, mp}$ = most probable critical state angle of shearing resistance

k = the limiting values stated under Design Approach A.

The value of the design wall adhesion, s'_{wd}, should be taken as zero.

Total stress analysis

Design $s_u = s_{ud} = s_{ump} / 1.5$

where:

s_{ump} = most probable value of undrained shear strength, s_u.

The limiting value of wall adhesion, s_{wmax}, should be taken as:

$s_{wmax} = \alpha \, s_{ud}$

where:

α = 0.5 in stiff clay (section 4.1.4). Smaller values of α may apply in particular circumstances eg steel sheet piles driven through overlying soft clay.

The treatment of tension cracks, excavation disturbance and softening on the restraining side of the wall should be on the basis of observations on site. In the absence of such observations, the guidance provided for Design Approach A should be adopted.

6.3.2 Groundwater pressures

See also

5.5...Determination of groundwater pressures

These should correspond to case (i) in section 5.5, namely, the worst credible groundwater pressures at each stage of the construction sequence and throughout the wall's design life.

6.3.3 Loads

See also

5.6...Loads

Calculations should be undertaken for the construction stage and operational stage load cases (section 5.6.3). The worst credible combination of loadings should be adopted for each of these load cases, excluding extreme or accidental events. For flat ground and walls retaining heights greater than 3 m, a minimum surcharge of 10 kPa should be applied to the surface of the retained ground. For walls retaining less than 3 m, this surcharge load may be reduced provided the designer is confident that a minimum surcharge of 10 kPa will not apply during the life of the structure (section 5.6.1).

6.3.4 Design geometry

See also

5.7...Unplanned excavation of formation

Unplanned excavation to the extent stated in section 5.7 should be assumed in design calculations. Allowances for disturbance and softening of the restraining soil in total stress analysis should apply below the underside of the unplanned excavation level (section 6.3.1).

6.3.5 Determination of wall embedment depth and load effects

See also

4.2...Methods of analysis

The wall toe level should be the *deeper* of that required to satisfy:

- load bearing capacity: see BS 8004 (1986) for guidance
- hydraulic cut-off and uplift: see Williams and Waite (1993) for guidance
- global stability
- lateral stability.

This section considers the determination of wall embedment depth for lateral stability only.

Wall toe level for overall lateral stability can be determined from limit equilibrium calculations or soil-structure interaction analysis.

Cantilever walls

Limit equilibrium calculations

Simple limit equilibrium calculations should be sufficient for the design of embedded cantilever walls where wall deflections and associated ground movements are not of importance.

These can be carried out assuming fixed earth support conditions as described in section 4.2.2. Alternatively, commercially available software packages such as STAWAL and ReWaRD can be used.

Soil-structure interaction analysis

This can be carried out using subgrade reaction (eg MSOILS, WALLAP), pseudo-finite element (eg FREW, WALLAP), finite element (eg SAFE, CRISP, PLAXIS) and finite difference (eg FLAC) software.

Walls singly propped near the top

Where an embedded wall is propped or anchored near its top, a shorter wall is likely to be obtained from the results of soil-structure interaction analysis compared with limit equilibrium calculations, due to the effects of stress redistribution (section 4.3). In such circumstances, prop or anchor loads calculated from limit equilibrium methods will be smaller than those obtained from soil-structure interaction analysis and should be treated with caution in design (section 7.1.3).

Limit equilibrium calculations

These can be carried out assuming either free earth or fixed earth support conditions as described in section 4.2.2. Alternatively, commercially available software packages (eg STAWAL, ReWaRD) can be used. Fixed earth conditions may be appropriate where the embedment depth of the wall is taken deeper than that required to satisfy lateral stability, eg to provide an effective groundwater cut-off or adequate vertical load bearing capacity.

Soil-structure interaction analysis

This can be carried out using the soil-structure interaction software packages mentioned for cantilever walls.

Walls singly propped at low level

The support may be considered to be at low level if the depth to support exceeds two thirds of the retained height.

Limit equilibrium calculations

A simple earth pressure distribution is difficult to establish (section 4.2.2) for limit equilibrium calculations. In practice, a wall propped at formation level is often temporarily propped while excavation is carried out to final formation level. A careful consideration of the stability of the wall at each stage during construction and in the long term will be required. **Soil-structure interaction analysis should be undertaken for these walls.**

Soil-structure interaction analysis

This can be carried out using the soil-structure interaction software packages mentioned for cantilever walls.

Multi-propped walls

Limit equilibrium calculations

A multi-propped wall is a statically indeterminate structure.

Approximate limit equilibrium methods can be used to determine wall embedment depth by assuming a plastic hinge and rotation about the lowest support level to the wall. A method of hand calculation assuming this mechanism is presented by Phillips *et al* (1993). This is reproduced as Figure 6.2. A drawback to this approximate approach is that the plastic moment capacity and the shear force capacity of the wall at the lowest support level are initially unknown and iteration is therefore required to achieve economy in design.

Figure 6.2 *Postulated failure mechanisms used to check toe stability (after Phillips et al 1993)*

Such a method will provide only an approximate solution which is dependent upon the simplifying assumptions made in the calculations. **There is significant soil-structure interaction in a multi-propped wall and therefore soil-structure interaction analysis should be carried out.** Nevertheless, the Phillips *et al* method provides an approximate check on the results of the soil-structure interaction analysis.

Soil-structure interaction analysis

The toe level of a multi-propped wall should be determined using the soil-structure interaction software packages mentioned above for cantilever walls. These programs allow every stage of the construction sequence to be modelled and analysed. With such analysis, the required toe level of the wall will correspond to that which allows the program to converge, regardless of any displacements which may be computed with soil design strengths as required by section 6.3.1. In practice, the displacement should not be so large that the geometry of the problem assumed in the model is no longer valid.

6.3.6 Load effects for structural design of wall

Guidance on this is provided in section 6.6.

6.4 SLS CALCULATIONS

SLS calculations should be carried out where:

- wall deflections and associated ground movements are of importance
- the wall is required to satisfy criteria which necessitate undertake SLS calculations eg crack width criteria for reinforced concrete walls, allowable stress criteria for steel sheet pile walls, if applicable.

6.4.1 Soil design parameters

Unfactored (strictly, F_s = 1.0) soil design parameters should be used to derive earth pressure coefficients for use in SLS calculations.

Design Approach B is not appropriate for SLS calculations.

6.4.2 Groundwater pressures

See also

5.5...Determination of groundwater pressures

These should correspond to case (ii) in section 5.5, namely, the most unfavourable values which could occur in *normal* circumstances at each stage of the construction sequence and throughout the wall's design life. Extreme events such as a nearby burst water main may be excluded, unless the designer considers that such an event may reasonably occur in normal circumstances.

6.4.3 Loads

See also

5.6...Loads

Calculations should be undertaken for the construction stage and operational stage load cases (section 5.6.3). The combination of loadings should correspond to those which the designer considers may apply under normal circumstances. Extreme or accidental events should be excluded. For flat ground and walls retaining heights of greater than 3 m, a minimum surcharge of 10 kPa should be applied to the surface of the retained ground. For walls retaining less than 3 m, this surcharge may be reduced provided the designer is confident that a minimum surcharge of 10 kPa will not apply during the life of the structure (section 5.6.1).

6.4.4 Design geometry

See also

5.7.....Unplanned excavation of formation

5.9.1...Temporary works design

No allowance should be made for unplanned excavation beyond the formation level expected in normal circumstances. This should include allowance for features such as temporary excavation for services, if these can reasonably be expected, and the likely tolerance within which the excavation level will be achieved (section 5.7). Allowances for disturbance and softening of the restraining soil in total stress analysis should apply below the underside of the formation level (section 5.9.1).

6.4.5 Determination of load effects and wall deflections

See also

4.3...Effect of method of analysis on economy

Load effects (wall bending moments, shear forces and prop or anchor forces) can be calculated using either limit equilibrium methods or soil-structure interaction analysis. Where used in the design of walls which are singly propped or anchored near the top, limit equilibrium methods will usually overestimate the wall bending moments and underestimate prop or anchor loads by comparison with soil-structure interaction analysis (section 4.3). **Prop or anchor loads obtained from limit equilibrium calculations may therefore be unconservative. Props should be designed in accordance with section 7.1.3.**

In general, for economy of final structure and where estimates of wall deflections and ground movements are required, it is recommended that soil-structure interaction analysis is carried out.

Cantilever walls and walls singly propped near the top

Limit equilibrium calculations

These should be appropriate for the design of such walls in circumstances where wall deflections and associated ground movements are not of importance (section 6.3.5).

The maximum bending moment and shear force acting on the wall and the prop or anchor load (if applicable) can be calculated at limiting equilibrium assuming unfactored soil parameters with the simplified earth pressure distributions shown in Figures 4.10, 4.13 and 4.14.

The actual design embedment depth calculated from section 6.3.5 will be greater than that required to give limiting equilibrium with unfactored parameters. This additional depth is not accounted for in the calculations. The maximum bending moment at limiting equilibrium is assumed to be approximately equal to that actually acting for the design configuration (ie the design embedment depth determined from ULS calculations). In a reinforced concrete wall, the reinforcement should not be curtailed at the point where the calculated bending moment is zero. It should be taken to the bottom of the wall using the approximate profile shown in Figure 6.3.

Figure 6.3 *Determination of wall bending moments in SLS conditions from limit equilibrium calculations*

The calculation of the SLS wall bending moment, shear force and prop or anchor load (if applicable) as above requires consideration of the pressures acting on the wall when it is in limiting equilibrium ($F_s = 1.0$). The wall under ULS conditions will have a deeper embedment corresponding to $F_s > 1.0$. The entire embedded depth of the wall should be considered in calculating the groundwater seepage pressure in the SLS condition.

Soil-structure interaction analysis

SLS load effects and wall deflections can be calculated using the soil-structure interaction software packages mentioned under section 6.3.5.

Walls singly propped at low level and multi-propped walls

Limit equilibrium calculations

It is difficult to establish, with confidence, simple earth pressure distributions which are appropriate for use in limit equilibrium calculations for the design of these walls. Consequently, the use of limit equilibrium calculations as the sole basis for the design of these types of walls is not recommended (sections 4.2.2. and 6.3.5). They can, however, be used to provide an approximate check on the results of more advanced soil-structure interaction, analysis (sections 4.2.1, 6.2 and 6.3.5).

Soil-structure interaction analysis

Every stage of the construction sequence should be modelled and analysed using a soil-structure interaction computer program to obtain SLS load effects and wall deflections.

6.4.6 Load effects for structural design of wall

The use of the SLS load effects calculated from section 6.4.5 in the structural design of the wall is covered in section 6.6.

6.5 PROGRESSIVE FAILURE CHECK

See also

2.1.3...Risk assessment and management

For walls which are critically dependent upon their support system to provide lateral stability and where there is a possibility of removal of part of this support under the action of an accidental load case, the designer should carry out a risk assessment (section 2.1.3) and, if possible, avoid the possibility of such an accidental occurrence by a combination of design changes, construction procedural controls, etc. If this does not adequately mitigate the risk, the designer should carry out calculations to show that progressive failure will not occur under such circumstances. The following assumptions should be made in this check.

6.5.1 Soil design parameters

As section 6.4.1.

6.5.2 Groundwater pressures

As section 6.4.2.

6.5.3 Loads

Calculations should be undertaken for the accidental load case (section 5.6.3).

6.5.4 Design geometry

As section 6.4.4.

6.5.5 Determination of load effects and wall deflections

As section 6.4.5.

The designer should check that the wall deflections do not exceed ULS criteria, if any.

6.5.6 Load effects for structural design of wall

The load effects calculated from section 6.5.5 should be treated as ULS values (section 6.6).

6.6 STRUCTURAL DESIGN OF WALL

The structural design of the wall should conform to the relevant code of practice for the particular material, namely BS8110 Part 1 (1997), BS 5400 Part 4 (1990) or EC2 Part 1 (DD ENV 1992-1-1: 1992) for reinforced concrete and BS5950 Part 1 (2000), BS 449 Part 2 (1969) or EC3 Part 5 (ENV 1993-5, 1998) for structural steelwork. The design of the structural members should allow for the loads generated by the temporary and permanent construction stages in addition to the installation method. For pushed, driven or vibrated sections, the installation stresses generated should also be considered. For concrete cast in-situ, into a pre-formed hole, the reinforcement detailing should allow for the method of placement of the reinforcement and the concrete.

6.6.1 SLS load effects

The SLS wall bending moments and shear forces calculated from section 6.4.5 should be used to check compliance with crack width criteria for reinforced concrete walls and allowable stress criteria for steel sheet pile walls, if applicable.

6.6.2 ULS load effects

The ULS wall bending moments and shear forces for use in the structural design of the wall should be obtained as the *greater* of:

- the values calculated from section 6.3.5
- 1.35 times the SLS values calculated from section 6.4.5, where SLS calculations are undertaken
- the values calculated from section 6.5.5, where an accidental design situation/progressive failure check is undertaken.

6.6.3 Steel sheet pile walls

See also

Appendix C...Wall types (C1)

Construction issues

For driven sheet piling, the forces induced during the driving process should not exceed the capacity of the section. It may be necessary to use a sheet pile section of greater thickness than required from the analysis of the section in service in order to withstand the driving forces. This is usually assessed on the basis of experience of driving sections into comparable soils and to similar depths; guidance is provided in *Piling Handbook* (British Steel, 1997) and in Clause 4.4.4 of BS 8002 (1994).

Durability

Steel corrosion rates are generally low and steel piling may be used for permanent works in an unpainted or unprotected condition. The degree of corrosion and whether protection is required will depend upon the working environment, which can vary along the length and depth of the pile and with time. Underground corrosion of steel piles driven into undisturbed natural soils which do not comprise peat and which are not chemically contaminated is negligible. This is attributed to the low oxygen levels present in such undisturbed soils. Corrosion rates are higher where steel piling is exposed to atmospheric conditions, fresh water and marine environments. BS 8002 (1994) and the *Piling Handbook* (British Steel, 1997) give corrosion rates for each of these natural environments. These are reproduced below in Table 6.2.

Table 6.2 *Corrosion rates for steel piling in natural environments (after BS 8002, 1994)*

Environment	Corrosion rate mm / side per year
Embedded in undisturbed soil	0.015 (maximum)
Exposed to atmosphere	0.035 (average)
Immersed in fresh waters	see note (1)
Exposed to marine environment	
– below bed level	0.015 (maximum)
– seawater immersion zone	0.035 (average)
– tidal zone	0.035 (average)
– low water zone	0.075 (average)
– splash zone	0.075 (average)

Note

(1) Fresh waters are variable. Corrosion losses in fresh water immersion zones are generally lower than for sea water.

For steel piles embedded in disturbed soils, peat or chemically contaminated ground, corrosion rates will be higher than natural undisturbed soil discussed above; protection systems should be considered in such conditions. The rate of corrosion will depend upon the aggressiveness of the ground. This will be very site specific and appropriate specialist advice should be obtained. Some guidance is provided in section 5 of BD 42/00 (DMRB 2.1.2) for steel piles for a design life of 120 years.

The effective life of unpainted or unprotected steel piling depends upon the combined effects of imposed stresses and corrosion. Guidance on measures to increase the effective life of steel piles is given in Clause 4.4.4.4.3.5 of BS 8002 (1994) and in chapter 3 of the *Piling Handbook* (British Steel, 1997). These include:

- use of a heavier section to allow for additional steel thicknesses as a corrosion allowance. Maximum corrosion seldom occurs at the same position as the maximum bending moment: thus, the use of a corrosion allowance can be a cost effective method of increasing effective life
- use of high yield steel; sheet pile sections are generally supplied in two grades of steel, S 270 GP and S 355 GP to BS EN 10248: 1995, with minimum yield strengths of 270 N/mm² and 355 N/mm² respectively (section C1, Appendix C). Although both types of steel have similar corrosion rates, the use of S 355 GP at S 270 GP stresses will allow an additional loss of permissible thickness to be sustained without detriment. This, in effect, builds in a corrosion allowance extending the effective life of the steel piling
- organic coatings; steel piles should be coated under shop conditions to achieve the required coating thickness in as few coats as possible
- concrete encasement; concrete encasement may be used to protect steel piles in marine environments. The concrete cope can be extended to below the lowest low water level to provide protection over the splash and tidal zones
- cathodic protection; the design and application of cathodic protection systems requires specialist advice.

Design

The analysis of the sheet pile section for the bending moments and shear forces imposed during the construction stages is carried out in accordance with the relevant structural code of practice (for example, BS5950-1 (2000) *Structural use of steelwork in building,* BS 449 (1969) *Part 2: Specification for the use of structural steel in building,* or EC3 (1998) *Design of steel structures - Part 5: Piling)*.

BS 5950-1 (2000) follows the limit state design philosophy adopted in this report; the load effects determined from section 6.6.1 and 6.6.2 should be used in the structural design of the wall.

The design of sheet pile walls to accommodate bending in accordance with BS 449 (1969) is based on elastic design principles with defined limitations on allowable stress. These allowable bending stresses are presented in Table 6.3.

Table 6.3 *Allowable bending stresses for steel sheet piling (after British Steel, 1997)*

Steel quality	Allowable permanent stress (N/mm²)	Allowable temporary stress (N/mm²)[1]
EN 10248: 1995 S270GP	180	200
EN 10248: 1995 S355GP	230	260

Notes

1. Higher allowable stress permitted for a temporary wall on the basis that a long term corrosion allowance will not be required and that the increased deflections will be acceptable under the short term loading.

Where the sheet pile wall is designed to comply with the above allowable stress criteria, SLS calculations (design steps 13, 14 and 16, see Figure 6.1 and Box 6.1) will be required to ensure compliance.

Figure 6.4 *Design of sheet pile walls to EC3, Part 5 (ENV 1993-5: 1998)*

EC3 Part 5 (ENV 1993-5: 1998) allows the full plastic material properties of the steel to be mobilised together with redistribution of earth pressures to achieve more economic design. Sheet pile walls are divided into four classes, as shown in Figure 6.4 and described below:

- Class 4; sections which fail due to local buckling within their elastic capacity
- Class 3; sections which reach their elastic moment capacity. The stress distribution across the section is elastic. The yield stress is reached in the extreme fibres of the section such that the elastic moment capacity, M_{el} is given by:

$$M_{el} = Z_e.f_y$$

where

Z_e is the section modulus of the wall and f_y is the yield stress

- Class 2: sections which take account of their full plastic moment resistance, M_{pl}, such that:

$$M_{pl} \approx 1.15\ M_{el}$$

- Class 1; sections which are designed *plastically* and which allow for moment redistribution due to rotation of the section. The moment capacity of the section, M_u, will depend on the ratio (b/t) where b and t are the width and thickness of the flange respectively.

According to Hartmann - Linden *et al* (1997) the ultimate limit state design of sheet piling can lead to considerable reductions in material use. For Class 2 design, this reduction is 15% to 20%. For Class 1 plastic design, the material reductions are about 25% to 30%. However, in the UK, it may not be possible to realise such savings as it is often necessary to adopt sheet pile sections which are of greater thickness than those determined from the analysis of the section in service in order to withstand the driving forces.

6.6.4 Cast-in-place Concrete

See also

Table 3.7

Construction issues

The method of constructing the concrete member below the ground can affect the structural design.

Continuous flight auger (cfa) piling

The use of cfa (continuous flight auger) piling, where the pile is bored and concreted in a single operation as the auger is drilled and extracted, restricts the depth and reinforcement density of the pile. The reinforcing cage is pushed into the wet concrete. Soil conditions that allow any free water to flow out of the concrete will induce a premature set in the concrete and prevent installation of the cage. High reinforcement densities, and particularly links, will also restrict the installation depth. A small vibrator attached to the top of the cage may ease the installation, but the limitations of this method should be recognised by the designer. Table 3.8 gives guidance on the typical depths which are achievable.

Bored piles and diaphragm walls

The concrete used for piles and diaphragm walls is not usually placed with the use of a vibrator and therefore should be self-compacting with the ability to flow around the reinforcement cage (refer to the *Specification for piling and embedded retaining walls,* Institution of Civil Engineers *et al*, 1996). When a drilling fluid is used to provide temporary support for the bore, the concrete should displace the support fluid. Good quality concrete should be ensured throughout the section and particularly in the cover zone between the reinforcement and the soil. It is important to note that the concrete has to flow out from the centre of the section through the reinforcement cage to the cover zone and it is this concrete which is usually important for maintaining the protection to the reinforcement in the long term.

BS EN 1536 (2000) provides guidance on the size of tremie pipes and annuli for different pile diameters and aggregate sizes.

Reinforcement detailing

Bundles and large diameter bars tend to be used more often in embedded wall concrete sections because of the need to provide a large clear space between bars to allow the concrete to flow into the cover zone. The reinforcement cage is fabricated above ground and then lifted and lowered into the bore, so it should be designed to allow for this lift. Long cages may need to be spliced over the bore due to lifting restrictions.

Pile cages

The *Essential guide to the ICE specification for piling and embedded retaining walls* (Federation of Piling Specialists and Institution of Civil Engineers, 1999) recommends that the minimum clear spacing between the vertical bars, or bundles of bars, should be 100 mm to ensure an adequate flow of concrete through the reinforcement cage. Multiple layers of reinforcement should be avoided.

Diaphragm wall cages

Space should be maintained around the tremie pipe positions to allow the tremie pipe to be installed and withdrawn without snagging on the reinforcement. Typically a minimum clear spacing of 500 mm is provided at the tremie positions. Large panels may require more than one set of tremie pipes to reduce the distance that the concrete has to flow within the section. The minimum link spacing required by the structural design codes may have to be compromised in order to insert the tremie pipe. The tremie pipe should be accommodated by providing the total number of links required by the code; adjusting the spacing locally to provide the 500 mm minimum clearance. There should be an unreinforced length of wall between adjacent panels to allow for tolerances in excavating the panels and to allow space for the joint detail and the waterbar where this is provided, typically 400 to 550 mm. The bar spacing of the vertical reinforcement should be reduced over the reinforced length of wall to allow for the unreinforced section, which may be up to 25% of the panel length for a single bite panel.

The *Essential guide to the ICE specification for piling and embedded retaining walls* (Federation of Piling Specialists and Institution of Civil Engineers, 1999) and BS EN 1538 (2000) give guidance on the clear spacing between bars to ensure an adequate flow of concrete through the diaphragm wall reinforcement cage. The final clear horizontal distance between vertical bars in a single layer should be 100 mm. This figure may be reduced to 80 mm over lap lengths (although this should be avoided wherever possible, maintaining the 100mm minimum clear spacing). Where two layers of reinforcement are required on a wall face, the bars in the inner layer should be aligned behind those in the outer layer in order to allow concrete to flow between them. The clear vertical distance between horizontal bars should be at least 200 mm where the clear distance between the vertical bars is 100 mm. The clear vertical distance between horizontal bars can be reduced to 150 mm if the spacing of the vertical bars is increased to give a clear window area of at least 0.02 m^2 between the horizontal and vertical bars or 0.16 m^2 over lap lengths. Where links are required the clear horizontal distance between legs of the links should be at least 150 mm.

EC2 (1992) detailing rules

EC2 (DD ENV 1992-1-1: 1992) is specifically for building structures and EC2 Part 6 for massive civil engineering structures. EC2 has strict rules regarding the use of bundles and large diameter bars, restricting the use of bundles to a maximum equivalent diameter of 55 mm and requiring all bars of 32 mm diameter and above to be joined using mechanical couplers rather than by means of lapping. Two 40 mm bars as a bundle have an equivalent diameter of 56.6 mm.

The design of reinforced concrete on the Copenhagen Metro project was carried out to the requirements of EC2. On this project, it was necessary to compromise the above criteria to the extent that 40 mm bars were lapped within the pile section and bundles of up to three 40 mm bars were used to satisfy a specified 0.2 mm maximum crack width criterion. This compromise was adopted for piles greater than 1050 mm diameter and diaphragm wall sections of 1000 mm thickness and greater (Beadman and Bailey, 2000).

Reinforced concrete design

Annex A of BD 42/00 (DMRB 2.1.2) provides detailed guidance on the design of reinforcement for cast-in-place retaining walls to the requirements of BS 5400: Part 4 (1990), as implemented by BD 24 (DMRB 1.3.1).

Rowe and Whittle (1997) The handbook to British Standard 8110: 1985. Structural use of concrete provides guidance on design to BS 8110 (1985).

Crack width Control

The specification of a maximum crack width in a reinforced concrete section is a SLS consideration and usually arises from concerns about durability, watertightness and aesthetics. Cost savings are possible if a pragmatic approach is taken to crack width control.

The location and orientation of a crack is more important than its size. Cracks of any size which are in line with the reinforcement, eg along the lines of the links, may cause corrosion, whereas cracks transverse to the reinforcement are unlikely to cause corrosion. Rowe *et al* (1987) indicate that crack widths of up to 0.5 mm may be acceptable from a durability viewpoint. Crack width control requirements and calculations principles to meet the requirements of BS 8110 (1997), BS 5400 (1990), BD 42/00 (DMRB 2.1.2) and EC2 (1992) are discussed in the sections below.

Durability

Durability resistance is determined by cover and mix design.

- BS 8110 Part 1 (1997): The wall should be designed for durability in accordance with Clauses 2.2.4, 2.4.7, 3.1.5, 4.1.5 and 6 of BS 8110 Part 1 (1997). The minimum cement content should be in accordance with section 3 of BS 8110 Part 1 (1997). Chemical analysis of the ground and groundwater should be made to assess its sulphate content and the concrete mix should be designed in accordance with the requirements of BRE Special Digest 1 (2001) C*oncrete in aggressive ground.* The concrete cover should be in accordance with Table 3.3 of BS 8110 Part 1 (1997). Walls subject to splashing or intermittent wetting by saline water should have adequate resistance to chloride attack and may need protection by a waterproof membrane.
- BS 5400 Part 4 (1990) and BD 42/00 (DMRB 2.1.2): Section A11 of BD 42/00 (DMRB 2.1.2) provides useful guidance on circumstances where crack width control is necessary from a durability perspective. Cracks located in wall sections which are permanently embedded in undisturbed non-aggressive ground will have little effect on the corrosion of reinforcement. In such conditions, the control of crack widths in accordance with the requirements of

Clause 5.8.8.2 of BS 5400 Part 4 (1990) as implemented by BD 24 (DMRB 1.3.1) should be restricted to sections of the wall *which are not classed as embedded as in section A11.1.1* of BD 42/00 (DMRB 2.1.2). The cover to the reinforcement over the embedded section of the wall in these conditions should be in accordance with Table 13 of BS 5400 Part 4 (1990) as implemented by BD 24 (DMRB 1.3.1). Where aggressive or very aggressive environments (as defined in section 5 of BD 42/00) exist, crack widths should be controlled in accordance with the requirements of Clause 5.8.8.2 of BS 5400 Part 4 (1990).

Watertightness

Any size of crack that passes through the section may let in water. However, wall flexure will normally cause a compression zone (provided it is not combined with significant tension) which will probably prevent water passage. BS 8007 (1987) provides guidance on how to keep the size of crack small in walls to tanks and reservoirs retaining water.

Aesthetics

If the main concern is the unsightliness of cracks, then a crackwidth of 0.2 mm may be appropriate. However, there are situations where leaks occur with smaller crack sizes where salts and deposits leak through and cause unsightly marks. It is therefore unlikely that a specification of crack width alone will be appropriate in such circumstances.

Crack width calculation principles

The crack width calculations recommended by BS 8110, BS 5400 and EC2 should be interpreted as follows for sections cast in the ground:

BS8110 Part 1 (1997)

For a typical pile section, the nominal cover to earth faces is 75 mm (BS8110 Part 1: 1997 Clause 3.3.1.4). The durability requirements are for a reduced cover, typically 35 mm for C35 (BS8110 Part 1; Table 3.3) concrete in moderate exposure (BS8110 Part 1; Table 3.2) conditions (non-aggressive soil). The crack width (BS8110 Part 2:section 3.8) should be assessed in accordance with a_{cr}, the distance from the point considered to the surface of the nearest longitudinal bar (BS8110 Part 1:Clause 3.8.2). This is customarily assessed at the nominal cover for durability, rather than at the 75 mm cover as shown in Figure 6.5.

Figure 6.5 *Pile cross section showing crack width calculation principles to BS 8110*

BS 5400 Part 4 (1990)

Guidance is provided in Annex A of BD 42/00 (DMRB 2.1.2).

EC2 Part 1 (1992)

To calculate the crack width under bending in accordance with EC2, the area of concrete in tension, A_{ct}, is the critical dimension (EC2 Part 1 (1992: Clause 4.4.2.2 (3)). If the concrete in the tension zone beyond the nominal cover is ignored, the area is as shown in Figure 6.6.

Figure 6.6 *Pile cross section showing crackwidth calculation principles to EC2 as applied on the Copenhagen metro project*

6.7 WORKED EXAMPLE

A worked example illustrating the steps outlined in section 6.2 for the design of a multi propped reinforced concrete retaining wall by pseudo-finite element methods (FREW) is presented in Appendix K.

6.8 KEY POINTS AND RECOMMENDATIONS

1. The limit state design philosophy described in chapter 2 should be adopted. Design calculations should satisfy the ultimate limit sates (ULS) of wall stability and structural strength and the required serviceability limit states (SLS). The designer should demonstrate that exceedance of ultimate and serviceability limit states is sufficiently improbable in the envisaged design situations.

2. A factor F_s should be applied on soil strength. The magnitude of F_s should be selected from Table 6.1.

3. A clear unambiguous design method is essential to achieve economy in design. The procedure shown in Figure 6.1, described in section 6.2 and illustrated in section 6.7 should be undertaken.

4. The wall toe level should be the *deepest* of those required to satisfy:

   - load bearing capacity
   - hydraulic cut-off and uplift
   - global stability
   - lateral stability.

5. The designer should choose whether to carry out limit equilibrium calculations or soil-structure interaction analysis. Guidance in this respect is provided in sections 4.2, 4.3, 6.2, 6.3 and 6.4. It is briefly repeated here for clarity.

   Limit equilibrium methods will suffice for the design of embedded cantilever walls where wall deflections and ground movements are not of importance. They can also be used in the design of walls which are singly propped or anchored at or near their top but, for these wall types, they are likely to overestimate wall depth and bending moments and underestimate prop or anchor loads by comparison with soil-structure interaction analysis (section 4.3). Prop loads calculated by limit equilibrium methods may be unconservative and should be treated with caution in design (section 7.1.3). Limit equilibrium calculations are not recommended as the sole basis of the design of embedded walls where it is difficult to establish, with confidence, simple earth pressure distributions which are appropriate for use in such calculations eg walls which are singly propped at low level and for multi-propped walls: soil-structure interaction analysis should be carried out for the design of such walls. Soil-structure interaction should also be carried out where wall deflections and ground movements are of importance.

   It is sensible to carry out some simple calculations as an approximate check on more advanced soil-structure interaction methods (eg wherever possible it is prudent to carry out simple limit equilibrium calculations with appropriate simplifying assumptions to obtain a conservative bound prior to carrying out complex finite element difference analyses).

6. In ULS considerations the designer should separately consider Design Approach A and Design Approach B at the stage of selecting soil design parameters. By inspection, the more onerous parameters should be selected and *one* design calculation should be undertaken to calculate wall embedment and maximum wall bending moments, shear forces and prop or anchor loads for the assumptions listed in section 6.3. The wall design calculation should adopt the more onerous of the soil design parameters derived from either Design Approach A or Design Approach B. Design Approach C should only be adopted as part of the Observational Method.

7. SLS calculations should be carried out where:

   - wall deflections and associated ground movements are of importance
   - the wall is required to satisfy criteria which necessitate the requirement to undertake SLS calculations, eg crack width criteria for reinforced concrete walls and allowable stress criteria for steel sheet pile walls, if applicable.

8. Where SLS calculations are required, the designer should carry out *one* design calculation to determine the SLS load effects (wall bending moments, shear forces, prop or anchor loads) using unfactored (strictly, $F_s = 1.0$) soil parameters for the assumptions listed in section 6.4. This calculation should be carried out using either Design Approach A or Design Approach C (if the Observational Method is adopted in design). Design Approach B is not appropriate for SLS calculations.

9. Where applicable, the designer should check that the SLS load effects, wall deflections and associated ground movements comply with the specified wall design requirements and performance and durability criteria (ie compliance with crack width criteria for reinforced concrete walls, allowable stress criteria for steel pile walls, etc.).

10. Where the risk of an accidental event which may result in the removal of part of the wall's support system cannot be adequately mitigated through a combination of design changes, construction procedural controls, etc, the designer should carry out calculations to show that progressive failure will not occur under such circumstances.

11. Where applicable, the designer should carry out *one* design calculation to determine load effects using unfactored (ie $F_s = 1.0$) soil parameters for the accidental design situation (section 6.5) to confirm that progressive failure of the wall and its support system does not occur. This calculation should be carried out using either Design Approach A or Design Approach C (if the Observational Method is adopted in design). Design Approach B should not be used for this calculation.

12. The SLS and ULS wall bending moments and shear forces for use in the structural design of the wall should be obtained as follows:

   - SLS load effects:
     - the values calculated from item 8 above
   - ULS load effects for use in the structural design of the wall should be obtained as the *greater* of:
     - the values calculated from item 6 above

- 1.35 times the SLS values calculated from item 8 above, where SLS calculations are undertaken
- the values calculated from item 11 above, where an accidental design situation/progressive failure check is undertaken.

13. The support system to the wall should be designed in accordance with the guidance provided in chapter 7.

7 Design of support systems

See also

5....Determination and selection of parameters

6....Design of wall

This chapter is intended primarily for geotechnical and structural designers involved in the design of the permanent and temporary support system to the wall. It:

- provides guidance on the design of permanent and temporary propping systems
- recommends a method by which earth berms may be represented in limit equilibrium and soil-structure interaction calculations using subgrade reaction and pseudo-finite element methods
- provides guidance on how to estimate wall deflections due to berm removal
- provides information on where guidance can be obtained for the design of ground anchors.

7.1 PROPPING SYSTEMS

Temporary props are used extensively in bottom-up construction sequences. Steel sections, typically tubular and box sections and universal columns, are commonly used.

Plate 7.1 *Propped steel sheet piles (courtesy of Corus)*

In top down construction, permanent propping is usually provided by concrete floor slabs; these are generally very stiff (except where significant openings are made in the slab) and of adequate capacity to support the loads arising. Consequently, this section concentrates on temporary propping.

The cost of the temporary propping system is usually small in comparison with the cost of the retaining wall. However, the delay and disruption to the excavation while the temporary props are installed may be of significant cost and such activities should be carefully considered and explicitly included in the overall programme for the construction works. While efficient and appropriate design of the propping system should be the aim, major reductions in overall construction costs will not be achieved, particularly if props are available for hire. Economy can be achieved by reducing the number of props or prop levels eg through the application of the Observational Method (section 5.9.1).

Temporary props are usually structurally over-designed. Failures are rare and are generally the result of poor detailing, misjudgement of ground conditions or accidents (Twine and Roscoe, 1999). The failure of a prop can lead to progressive collapse; this should be explicitly addressed in design and is considered further in section 7.1.3.

7.1.1 Design responsibility

See also

2.2.1....Statutory requirements

Clear allocation of design responsibility is essential.

Designers of props for a temporary wall, within which a permanent structure is to be built, should ensure that the performance requirements of the temporary wall are met (eg the temporary wall deflections do not impinge on the permanent works) and that the site operations are not unduly constrained. The designer should take account of how the permanent works will be constructed and the preferred method of prop removal.

2.1.2….Contractual requirements

2.1.3….Risk assessment and management

Where the permanent wall is utilised to provide temporary support during construction, the situation is more complex. The designer of the temporary propping system should consider the effects of load transfer on the permanent wall due to the installation and removal of the temporary props. In this instance, the designer of the permanent wall should inform the designer of the temporary props about the assumptions made regarding temporary propping in the design of the permanent works, eg propping levels and spacing, installation and removal sequence and prop stiffness.

Where the designer of the temporary props is unable to fully comply with the assumptions made by the designer of the permanent wall, the lead designer (section 2.1.1.) should coordinate the necessary interaction between the temporary works and permanent works designers.

7.1.2 Prop selection

In addition to the required axial load capacity, prop selection will be influenced by the following considerations:

- span
- ease of fabrication
- availability
- buildability.

7.1.3 Design of props

See also

6.3..…ULS calculations

6.4…..SLS calculations

6.5…..Progressive failure check

The design of the props will depend upon the analysis method adopted in the calculations for the design of the wall. Prop loads calculated from limit equilibrium calculations may be unconservative as the effects of soil-structure interaction are not included (section 4.3). In such circumstances, the calculated prop loads should be increased by 85% to allow for the effects of stress redistribution and arching behind the wall. Soil-structure interaction methods (undertaken as described in section 4.2) which allow stress redistribution to more realistically model the non-linear pressure profile behind the wall should provide calculated values of prop loads which are more representative of the particular project circumstances modelled. Irrespective of the type of analysis undertaken, the calculated prop loads should be checked for adequacy by comparing them with those derived from comparable experience, as defined in section 2.2.2. Wherever possible, this should be based on relevant reliable field measurements from case history data in comparable conditions. In the UK, the most appropriate way to do this is by use of the Distributed Prop Load (DPL) method.

In situations where the calculated prop loads are significantly different from those derived from experience of comparable construction (eg from the DPL method), the designer should carefully investigate and understand the reasons for the calculated values. This will typically involve a detailed review of the assumptions made in the calculations and the carrying out of sensitivity analyses. The outcome of such investigations should enable the designer to adopt appropriate values for use in the design of the props.

CIRIA report C517 (Twine and Roscoe, 1999) *Temporary propping of deep excavations* gives a detailed description of the principles and application of the DPL method for the determination of temporary prop loads. The report provides design guidance based on extensive field measurements of prop loads for flexible and stiff walls and for the range of ground conditions commonly encountered in the UK.

The essential features of the DPL method are described in Box 7.1 where the treatment of additional loading from temperature effects has been adapted from that presented in CIRIA report C517 to allow for the degree of restraint of the prop.

Box 7.1 *This distributed prop load design method (adapted from Twine and Roscoe, 1999)*

Design Method

Details are provided in CIRIA report C517 (1999).

The distributed prop load (DPL) method for calculating prop loads for propped temporary excavations is based on the back analysis of field measurements of prop loads relating to 81 case histories of which 60 are for flexible walls (steel sheet pile, king post walls) and 21 are for stiff walls (contiguous, secant, diaphragm walls). The case history data relate to excavations ranging in depth from 4 m to 27 m, typically 5 m to 15 m in soft and firm clays (soil class A, Table 1 below), 10 m to 15 m in stiff and very stiff clays (soil class B, Table 1 below) and 10 m to 20 m in coarse grained soils (soil class C, Table 1 below).

The method for determining prop loads is shown in Figure (a).

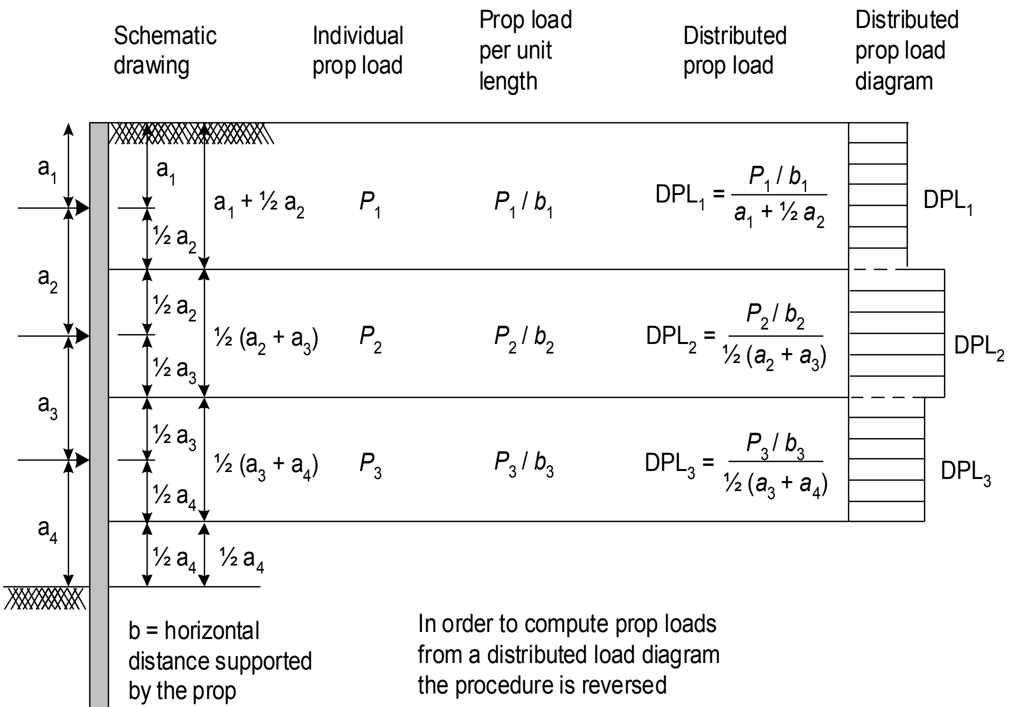


Figure (a)

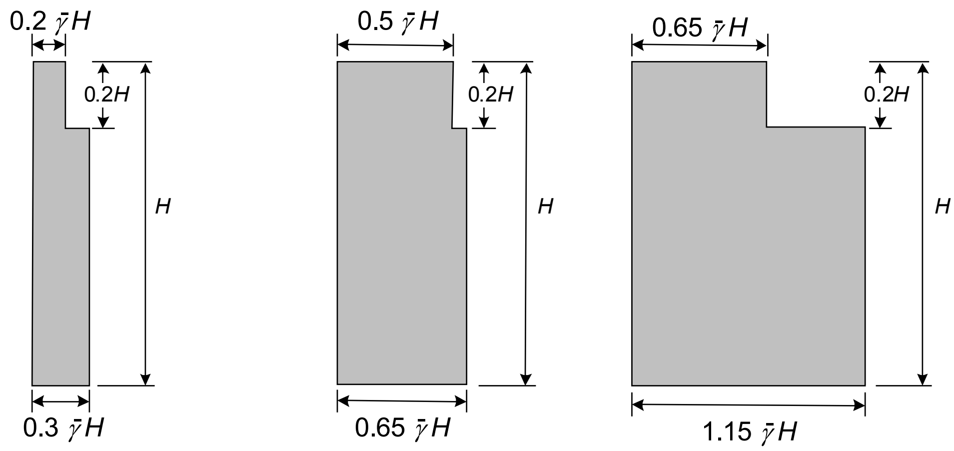
The data are classified on the basis of the type of ground retained by the excavation, see Table 1.

Table 1 *Classification of ground types*

Soil class	Description
A	Normally and slightly overconsolidated clay soils (soft to firm clays)
B	Heavily overconsolidated clay soils (stiff and very stiff clays)
C	Coarse grained soils
D	Mixed soils (walls retaining both fine grained and coarse grained soils).

The above classes are subdivided according to wall stiffness, ie flexible (F) walls and stiff (S) walls. Flexible walls retaining soft clay soil (class AF) have been further subdivided according to base stability conditions. Class C soils have been subdivided into dry and submerged.

Figure (b) shows the characteristic prop load diagrams for class A, B and C soils.



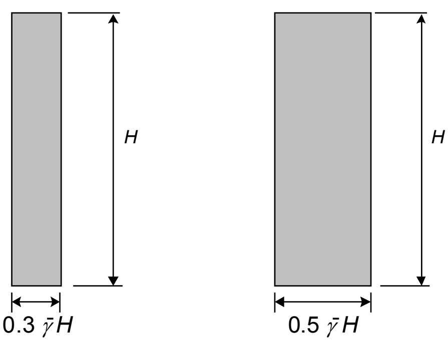
Class AF, Firm

Class AF, soft with stable base

Class AF, soft clay with enhanced base stability

For Class AS, tentatively as Class AF

<u>Class A soils (soft to firm clays)</u>



Class BF

Class BS

Notation :

γ = bulk unit weight of soil

γ' = effective unit weight of soil

$\bar{\gamma}$ = average unit weight of the soil over the retained height of the excavation, H

<u>Class B soils (stiff to very stiff clays)</u>

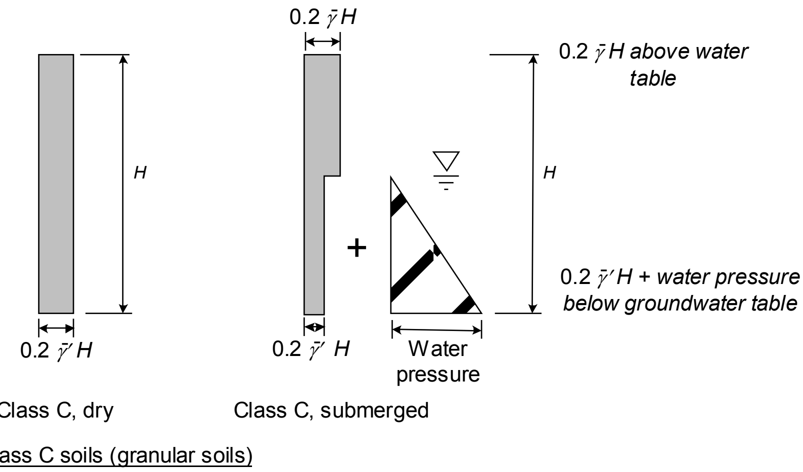


Figure (b)

Temperature effects on props

An increase or decrease in the temperature of a prop from its installation temperature will cause the prop to expand or contract according to the relationship:

$$\Delta L = \alpha.\Delta t.L$$

where:

ΔL = change in prop length

α = thermal coefficient of expansion for the prop material

Δt = change in prop temperature from the installation temperature

L = prop length.

If the prop is restricted or prevented from expanding freely, an additional load is generated in the prop. The magnitude of this additional load is:

$$\Delta P_{temp} = \alpha.\Delta t.E.A.\ (\beta/100)$$

where:

E = Young's modulus of the prop material

A = cross-sectional area of the prop

β = percentage *degree of restraint* of the prop
(70% for stiff walls in stiff ground and 40% for flexible walls in stiff ground).

The designer should select the appropriate value of β to suit particular project circumstances. Values other than those generally recommended above may be applied where the designer is confident that such values can be justified, eg on the basis of comparable experience (section 2.2.2).

The DPL method should be applied to excavations of similar depth and plan geometry and in ground conditions which are comparable to the case history data considered in CIRIA report C517. The designer should establish this prior to carrying out calculations of prop load using the DPL method.

The SLS and ULS prop forces for use with codes of practice for the design of the structural components of the support system should be determined as described below.

SLS prop load

SLS prop load (P_{SLS}) should be determined to be the *greater* of:

- the value calculated from section 6.4.5 if soil-structure interaction analysis is undertaken or 1.85 times the value calculated from section 6.4.5 using limit equilibrium methods
- that calculated from the DPL method (DPL_k) from Box 7.1 or determined on the basis of comparable experience as defined in section 2.2.2.

The designer should also ensure that the following serviceability limit state check is satisfied:

$$P_{SLS} + \Delta P_{temp} \leq P_E$$

where:

P_{SLS} = SLS prop load

ΔP_{temp} = additional prop load due to temperature change (Box 7.1)

P_E = load capacity of the prop at the elastic limit.

ULS prop load

ULS prop load (P_{ULS}) should be determined to be the *greater* of:

- 1.35 times the value of P_{SLS} as determined above
- the value calculated from section 6.3.5 if soil-structure interaction analysis is undertaken or 1.85 times the value calculated from section 6.3.5 using limit equilibrium methods
- the value calculated from section 6.5 using soil-structure interaction analysis or 1.85 times the value calculated from section 6.5 using limit equilibrium methods, where an accidental design situation/progressive failure check is undertaken
- 1.35 times DPL_k (from permanent actions) plus 1.5 times DPL_k (from variable actions)

The design of individual props should be robust. The designer should consider the implications of the accidental loss of a prop in one of two ways:

- incorporate the loss of a prop in the design of the wall support system in the calculations undertaken in section 6.5; or
- adequately mitigate the risk of accidentally damaging or removing a prop through design changes and robust construction management strategy.

The load transfer from the temporary props may induce high local stresses in the permanent works. Openings in the permanent works slabs may require temporary propping until the structure is completed, when the final load paths are established. The props and walings should also be designed for bending moment and eccentricity of loading, as appropriate.

7.1.4 Sway

The designer should allow for any imbalance in horizontal loading across the excavation. This imbalance will cause the entire retaining wall and its support system to sway towards the side with the lower external load. Figure 1.3 shows an example of this. Props may be inclined downwards from the high load side: the resulting vertical component of force should also be taken into account in design.

Sway will increase ground movements on one side and reduce them on the other side. The designer should ensure that the walls are designed to accommodate the associated pressures.

7.2 BERMS

Berms can be used to help stabilize embedded retaining walls and to reduce their movement. Easton and Darley (1999) describe four case histories where berms were successfully used for this purpose.

Figure 7.1 *Definition of berm geometry*

In given ground conditions, the degree of support offered by a berm will depend on the height *H*, the bench width *B* and the slope *S* (Figure 7.1). The maximum slope *S* will be governed by soil and groundwater conditions, while *H* and *B* may be limited by considerations of space and access. In soils of low permeability, the drainage conditions assumed in design and the length of time for which the berm is required to remain effective will be important. Most methods of representing the effect of a berm in a limit equilibrium or soil-structure interaction analysis using subgrade reaction and pseudo-finite element methods are semi-empirical even if conditions on site approximate to plane strain. If the berm is removed in sections along its length to allow permanent supports to be installed, a three dimensional analysis may be required to assess stability and wall deflection. The difficulty of analysis may explain why berms have often been used in conjunction with the Observational Method (Tse and Nicholson, 1993; Powrie *et al*, 1993; Gourvenec *et al*, 1997).

7.2.1 Common methods of modelling earth berms

See also

Appendix I…Analysis of berms

In a limit equilibrium or soil-structure interaction analysis using subgrade reaction or pseudo-finite element methods, it is necessary to make some assumptions about the lateral stresses exerted by the berm on the wall above formation level, and the influence of the berm on the lateral stresses in front of the wall below formation level, as these are not calculated explicitly in the analysis. Brief descriptions of the most common methods of representing an earth berm in a limit-equilibrium or soil-structure interaction analysis are given below.

(i) Equivalent surcharge approach

Figure 7.2 *Representation of a berm as an equivalent surcharge (after Fleming et al, 1994)*

The representation of a berm using the equivalent surcharge approach (Fleming *et al*, 1994) involves calculating the weight of the berm and equating it to an equivalent uniform surcharge, q^*. According to this approach, the equivalent surcharge is applied over a distance defined by the intersection with excavation level of a line emanating from the toe of the wall at an angle of (45° - ϕ'/2) to the horizontal (Figure 7.2). The lateral pressure exerted by the berm itself is neglected. This approach is very conservative, especially if the toe of the wall is deep.

(ii)Raised effective formation level approach

Figure 7.3 *Representation of a berm by means of a raised effective formation level (after Fleming et al, 1994)*

The raised effective formation level approach (Fleming *et al*, 1994) models the berm as if it caused a rise in excavation level in front of the wall (Figure 7.3). The original berm profile is reduced to a design berm profile with a slope of 1:3 but the base width of the berm, *b*, remains unaltered. The height of the design berm becomes *b*/3 and the increase in effective formation level is taken as one half of the design berm height, ie *b*/6. Any portion of the actual berm above the effective formation level and the design berm (shown shaded in Figure 7.3) is then treated as a surcharge acting at the revised formation level using the equivalent surcharge approach described in (i) above. The raised effective formation level approach takes some account of the lateral pressure exerted by the berm, but not fully, and the approach is conservative.

(iii) Coulomb wedge approach

The Coulomb wedge approach (NAVFAC, 1986) involves carrying out a series of Coulomb wedge analyses for a number of trial failure surfaces emanating from the toe of the retaining wall. Having determined the minimum passive thrust on the wall, some designers convert this into an equivalent triangular stress distribution on the front of the wall between the top of the berm (zero pressure) and the toe of the wall (maximum pressure). However, the modified limit equilibrium analysis presented in Appendix I shows that the assumption of a triangular stress distribution in this way will tend to overestimate the height of the centre of pressure.

In a finite element analysis, a berm can be modelled directly without prejudging its effect on the lateral stresses on the wall. However, careful thought should be given to the internal stability of the berm. For example, in an effective stress analysis in which the berm slope is steeper than the angle of shearing resistance of the soil, negative pore water pressures may need to be specified and maintained within the berm for the duration of the analysis (eg Gourvenec and Powrie, 2000; Powrie and Daly, submitted). If berm stability does depend on the maintenance of negative pore water pressures, it would be prudent to take steps to ensure that these can be relied on in practice – for example, blinding the surface of the berm with concrete or covering it with an impermeable membrane.

7.2.2 Recommended method of modelling earth berms

See also

7.2.1….Common methods of modelling earth berms

The main disadvantage of the approaches described in section 7.2.1 is that the lateral resistance provided by the berm is either ignored or treated empirically and/or incorrectly. Powrie and Daly (submitted) describe the results of a series of plane strain centrifuge model tests of embedded cantilever retaining walls of various embedment depths supported by berms of different sizes. Daly and Powrie (submitted) analyse the model tests using two of the three methods outlined in section 7.2.1 and the multiple Coulomb wedge analysis summarised in Appendix I, with the following results:

- for a wall of given embedment, the factor of safety is increased significantly if a larger berm is used. **Increasing the size of the berm is more efficient in enhancing wall stability than increasing the depth of embedment of a wall supported by a smaller berm**

- method (i), section 7.2.1, is very conservative, giving factors of safety between 15% and 25% less than the multiple Coulomb wedge analysis for the berm and wall geometries investigated. The degree of conservatism increases with increasing berm size and decreasing embedment depth
- method (ii), section 7.2.1, is less conservative than method (i), giving factors of safety between 5% and 11% smaller than the multiple Coulomb wedge analysis. The degree of conservatism increases with decreasing embedment depth, but is less sensitive to berm size.

It is recommended that for routine limit equilibrium analysis, the raised effective formation level approach indicated in Figure 7.3 is used to model the effect of an earth berm. Where there is scope for achieving economy with more design effort, the multiple Coulomb wedge approach summarised in Appendix I could be adopted in soil-structure interaction analysis using pseudo-finite element or subgrade reaction methods. Alternatively, a finite element or finite difference analysis could be carried out.

In any limit equilibrium analysis of a berm supported retaining wall, potential failure mechanisms that arise because of the wall and berm geometry and soil stratigraphy (eg possible sliding on a weak horizontal layer) should also be considered explicitly.

7.2.3 Deflections of walls supported by berms

See also

7.2.2….Common methods of modelling earth berms

7.2.3….Recommended method of modelling berms for more economic design

Methods based on limit equilibrium calculations do not provide guidance on the deflection of walls supported by berms in working conditions.

The main shortcoming of the analyses described in sections 7.2.1 and 7.2.2 is that they refer to conditions of plane strain. In other words, they assume that the berm remains intact over the entire length of the wall throughout the excavation and construction period. In reality, it may be necessary to remove the berm in sections so that the permanent support, eg in the case of a road cutting, a formation level prop can be installed.

Removal of a long berm in sections, is a three dimensional problem.

Wall movement due to the removal of a section of a long berm would be expected to increase in proportion to the length of the section removed. Gourvenec and Powrie (2000) carried out a series of three dimensional finite element analyses to investigate the effect on wall movements of the removal of sections of an earth berm supporting a long embedded retaining wall in overconsolidated clay. Details are provided in Appendix I and the results of their analyses are presented in Figure 7.4.

Figure 7.4 *Normalised wall top displacement at the centre of the unsupported section against degree of discontinuity β for different excavated bay lengths B*

The main general practical implications of this work are that further wall movement resulting from the removal of a number of berm sections simultaneously can be minimised by maintaining a separation of at least one to three times the length of the berm section removed (β = 25% to 50% in Figure 7.4, where $\beta = B / (B + B')$ and B' is the length of wall section supported by the berm and B is the length of the unsupported section).

Easton *et al* (1999) carried out three dimensional (3D) finite element analyses to develop a relationship between berm height and the effective uniform increase in formation level in front of the wall to give the same wall movement. Their analyses considered a berm supported retaining wall having the cross-sectional geometry shown in Figure 7.5, with berms of different height within the profile envelope indicated.

Figure 7.5 *Three dimensional finite element mesh, wall and excavation geometry and assumed soil parameters (after Easton et al, 1999)*

Along the wall, the berm was divided into three central bays, each 5 m in length, and two outer bays, each 30 m in length. Two sets of 3D finite element analyses were carried out. In one case, a construction sequence using berms of varying height was modelled. In the other, a construction sequence using a uniform effective excavation level (ie no berm) was modelled. In both cases, excavation to final formation level for permanent prop slab installation was carried out using the same "bay" sequence. The construction sequence listed in Box 7.2 was analysed.

Box 7.2 *Construction sequence analysed by Easton et al (1999)*

Stage 1: Excavation to final formation level in the centre with perimeter berms or to an effective ground level, as required, over a period of 30 days

Stage 2: Excavation to final formation level in the central (5 m wide) bay over a period of 30 days

Stage 3: Installation of the prop slab in the central bay in 1 day

Stage 4: Excavation to final formation level for bay over a period of 30 days

Stage5: Installation of prop slab for bay in 1 day

Stage 6: Excavation to final formation level at near bay over a period of 30 days

Stage 7: Installation of prop slab at near bay in 1 day

Stage 8: Excavation to final formation level in remaining bays and installation of prop slab over remaining bays over a period of 30 days

Consolidating element models were used for the soil strata. The assumed soil stiffness and permeability is listed in Figure 7.5. Non consolidating element models were used for the structural elements.

Charts presenting the results of Easton *et al*'s analyses are provided in Figure 7.6.

Figure 7.6 *Relationship between berm height and effective uniform ground level, (a) stiff to very stiff clay, ϕ' = 22°; (b) firm to stiff clay, ϕ' = 28°*

For ground conditions, wall and excavation geometries comparable to those assumed by Easton *et al* (1999), the designer can use Figure 7.6 to rationalise the design of soil berms in temporary works design. For any given wall depth, the design approach presented in section 6.3.5 can be adopted to calculate the effective uniform ground level above final formation level which is required to satisfy ULS wall stability. Cantilever wall deflections can then be assessed either empirically (section 2.5.2) or from SLS calculations (section 6.4.5). The height of berm above final formation level corresponding to the effective uniform ground level obtained from the ULS calculations can be obtained from Figure 7.6.

7.3 ANCHORS

As an alternative to propping systems within the excavation, passive deadman anchors or ground anchorages may be installed behind the wall on the retained side. A passive anchor typically comprises a tie which derives its resistance from a deadman (an anchor block or anchor back pile) to which it is connected (Figure 7.7).

Figure 7.7 *Passive anchor (after Williams and Waite, 1993)*

A ground anchorage consists of an anchor head, a free anchor length and a fixed anchor length which is bonded to the ground by grout (Figure 7.8).

Figure 7.8 *Sketch of a typical ground anchorage (after BS EN 1537, 2000)*

Ground anchorages are likely to be favoured in preference to passive deadman anchors where the groundwater level is above the tie rod level or where the ground level rises steeply behind the wall. In most other applications, where anchors are required at one level near the top of the wall, passive anchors will usually be more economical, provided that there is sufficient space available behind the wall. Ground anchorages have the advantage that they can be installed at more than one level. The advantages and limitations of ground anchorages are discussed in section 3.1.2, Table 3.5.

Irrespective of whether a passive deadman anchor or ground anchorages are installed behind the wall, it is important to carefully consider the locations of the critical failure planes for the whole support system to ensure satisfactory global stability. The distance behind the wall should be sufficient to position the anchors such that they extend beyond any such failure planes. Williams and Waite (1993) provide some guidance on this in respect of passive anchors; this is shown in Figure 7.7. Appendix D of BS 8081 (1989) provides useful guidance on ensuring overall stability where ground anchorages are installed.

Guidance on the design of passive deadman anchors is provided in Clause 4.6.5 of BS 8002 (1994) and in Williams and Waite (1993). The available passive resistance should be calculated on the basis of net available resistance ie passive pressure less active pressure. No advantage should be taken of any surcharge loading on the ground surface in front of the deadman, but surcharge loading immediately behind it should be allowed for in the calculations. Analysis of deflections should also include an assessment of the deflection of the deadman. It may be necessary to preload the tie bars and the deadman anchors to reduce the potential deflections.

Guidance on the design, specification and installation of ground anchorages is provided in the following documents:

- BS 8081 (1989) Code of practice for ground anchorages
- Hong Kong Government (1997) Model specification for prestressed ground anchors
- BS EN 1537 (2000) Execution of special geotechnical work - ground anchors.

Ground anchorages are greatly underused in the UK when compared with experience elsewhere in Europe. Their increased use may result in significant savings over propping schemes where programme time is available for the construction of ground anchorages and the space is available to locate them (section 3.1.2).

7.4 KEY POINTS AND RECOMMENDATIONS

1. The cost of a temporary propping system is usually small in comparison with the cost of the retaining wall. However, the delay and disruption to the excavation whilst the temporary props are installed may be of significant cost and such activities should be carefully considered and explicitly included in the overall programme for the construction works. Economy can be achieved by reducing the

number of props or prop levels eg through the application of the Observational Method (section 5.9.1).

2. Clear responsibility for the design of the propping system to the wall is essential. Where the permanent wall is utilised to provide temporary support during construction, the designer of the permanent wall should inform the designer of the temporary props about the assumptions made regarding temporary propping in the design of the permanent works. Where the designer of the temporary props is unable to fully comply with the assumptions made by the designer of the permanent wall, the lead designer (section 2.1.1) should coordinate the necessary interaction between the temporary works and permanent works designers.

3. The SLS prop load (P_{SLS}) should be determined as the *greater* of:

   - the value calculated from section 6.4.5 if soil-structure interaction analysis is undertaken or 1.85 times the value calculated from section 6.4.5 using limit equilibrium methods
   - that calculated from the DPL method (DPL_k) from Box 7.1, or determined on the basis of comparable experience (section 2.2.2).

   The designer should check that:

   $$P_{SLS} + \Delta P_{temp} \leq P_E$$

4. The ULS prop load (P_{ULS}) should be determined as the *greater* of:

   - 1.35 times the value of P_{SLS} determined from (3) above
   - the value calculated from section 6.3.5 if soil-structure interaction analysis is undertaken or 1.85 times the value calculated from section 6.3.5 using limit equilibrium methods
   - the value calculated from section 6.5 using soil-structure interaction analysis or 1.85 times the value calculated from section 6.5 using limit equilibrium methods, where an accidental design situation/progressive failure check is undertaken
   - 1.35 times DPL_k (from permanent action) plus 1.5 times DPL_k (from variable actions).

5. The designer should allow for any imbalance in horizontal loading across the excavation.

6. Earth berms represent an effective means of temporary support, for example, prior to the installation of permanent props at formation level. Increasing the size of the berm is likely to be more efficient in enhancing stability than increasing the depth of embedment of the wall.

7. Earth berms can be represented in plane strain limit equilibrium and simple soil-structure interaction analyses by means of the raised effective formation level approach (method (ii), section 7.2.1). This method is conservative, but adequate for routine design. In cases where more design effort will produce significant economy, the multiple Coulomb wedge analysis presented in Appendix I could be adopted in soil-structure interaction analysis using pseudo-finite element or subgrade reaction methods. Alternatively, a finite element or finite difference analysis could be carried out.

8. Removal of a long berm in sections, for example, to construct a permanent prop at formation level, is a three dimensional problem. Wall movement due to the removal of a section of a long berm increases in proportion to the length of the section removed. Further wall movement resulting from the removal of a number of sections simultaneously can be minimised by maintaining a separation of at least one to three times the length of the section removed (β = 25% to 50% in Figure 7.4).

9. Careful consideration should be give to the locations of the critical failure planes behind the wall to ensure satisfactory global stability of the whole support system. The distance behind the wall should be sufficient to position ground anchors such that they extend beyond any such failure planes.

10. Ground anchorages are greatly underused in the UK when compared with experience elsewhere in Europe. Their increased use may result in significant savings over propping schemes where programme time is available for the construction of ground anchorages and the space is available to locate them.

8 Areas of further work and research

1. The contractual environment within which embedded retaining walls are designed and constructed is fragmented. It is recommended that a lead designer is appointed to review and oversee all stages of the design and construction process to ensure that the client's requirements are met. This is essential to ensure consistency and certainty of outcome. This is not current practice in the UK and detailed consideration should be given to it for future implementation.

2. Significant cost savings can be achieved by adopting:

   - an appropriate method and sequence of construction, selection of wall type and optimisation of the temporary and permanent use of the retaining structure
   - a risk based approach to design and construction through the use of the Observational Method
   - an appropriate method of analysis. Soil-structure interaction analyses can result in economies of wall structure compared to limit equilibrium design methods (see also item 4 below).

   Ground anchorages are greatly underused in the UK. Their increased use may result in significant savings over propping schemes. This should be seriously considered.

3. Insufficient good quality data are currently available in the literature regarding the performance of walls. Case history data in the UK are mainly limited to:

   - bored pile and diaphragm walls installed in stiff clays
   - short duration measurements of wall deflections and, occasionally, ground surface movements behind such retaining walls
   - rare measurements of stresses around retaining walls.

   There is an urgent requirement for more case history data to provide high quality measurement of the actual behaviour of different types of retaining wall installed in a range of different ground conditions. In particular, short term and long duration measurements of:

   - stress changes and displacements in the ground due to the installation and subsequent performance of the retaining wall during its working life to better understand stress changes due to wall installation and in the long term
   - vertical and horizontal movements of the wall and the ground around the wall to establish appropriate relationships between wall deflections, depth of excavation and ground movements behind and in front of the wall (not only at ground surface level but also with depth and distance from the wall)
   - stress changes, prop or anchor loads, wall performance and ground movements around three dimensional (3D) excavation geometries to better understand 3D effects, eg corner effects, behaviour of berms.

   Greater reliance on more advanced computers and associated software (item 4 below) will inevitably increase the risk of erroneous results due to a lack of fundamental understanding. It is therefore essential that the above data are gathered, interpreted and understood in an ongoing manner.

4. Advances in computer software and hardware will continue. This will enable greater use of finite element (FE) and finite difference (FD) methods of analysis, particularly 3D FE and FD. This should lead to the development of more complex soil constitutive models on the basis of laboratory studies validated by field monitoring, particularly through the application of the Observational Method, and model testing, eg centrifuge testing. There is still much research to be undertaken in this area.

5. Much research work has been undertaken over recent years to understand better the small strain stiffness of soil. However, the stiffness-strain behaviour of the material comprising the wall is not well understood and requires further research. In particular:

   – the process of shear transfer and slippage at sheet pile interlocks/clutches and the associated effect on the mobilised section modulus needs significant further research

   – the relationship between uncracked/cracked and the short term/long term values of wall flexural stiffness (*EI*) for reinforced concrete walls requires further investigation.

   The analysis of soil-structure interaction has progressed sufficiently for the above to be a significant issue. This will become more urgent in view of the likely future trend in soil-structure interaction analyses (item 4 above).

6. The use of plastic methods of design for steel sheet pile walls can lead to significant savings in material costs. The development and application of such methods to routine design requires further work and research.

7. Research should be undertaken in developing new methods of construction to achieve overall economy and ease of construction, eg

   – the use of material other than steel, eg carbon or glass fibre, as reinforcement to concrete

   – the development of different construction sequences, eg installing slabs before excavation in a top down sequence.

8. Research should be undertaken to streamline methods of routine design to achieve economy, eg the development of simple rules on stress redistribution behind propped or anchored retaining walls for use in simple limit equilibrium calculations.

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Geotechnical design
Part 1 General rules
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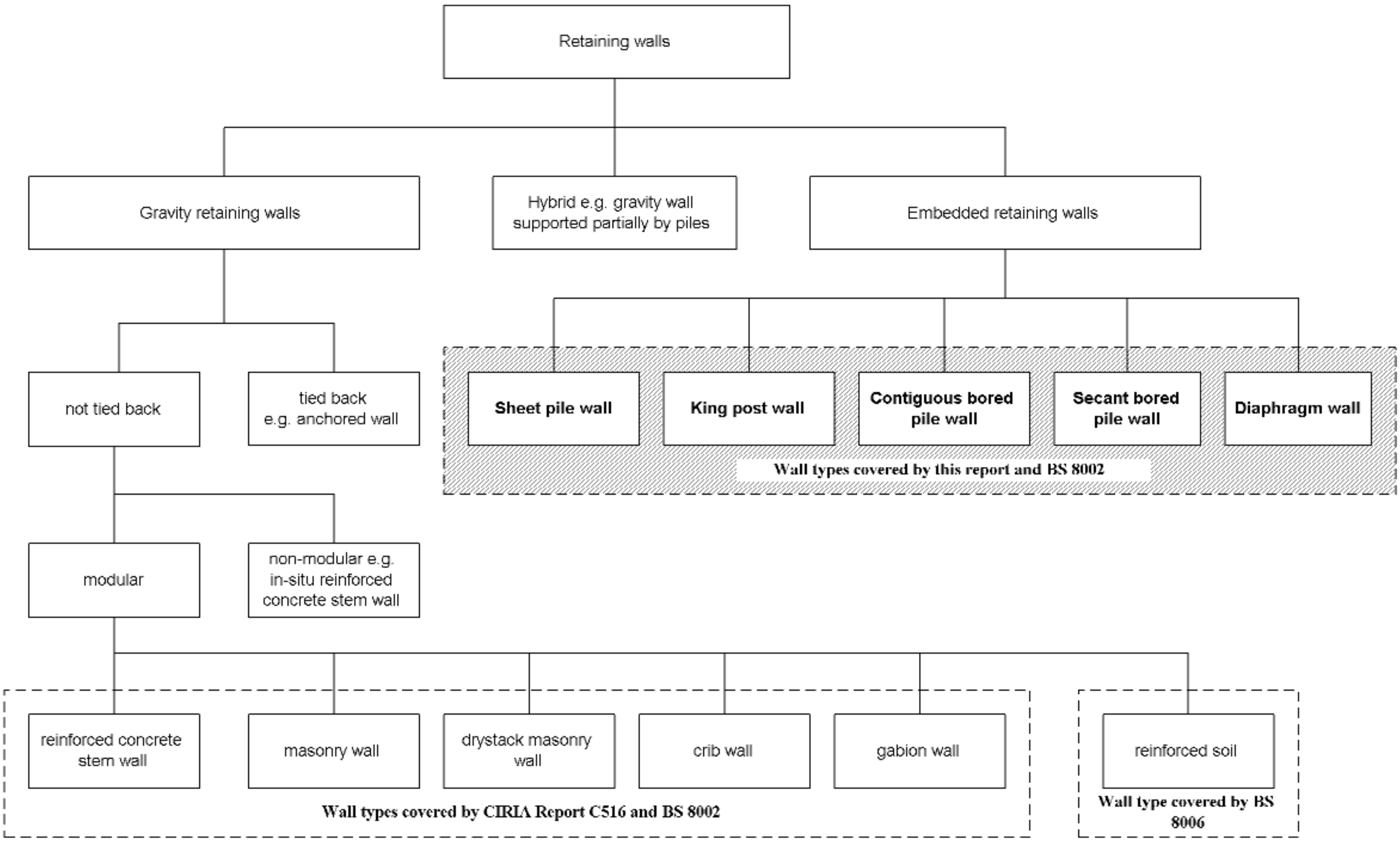


FIGURE 1.1 *Wall types*

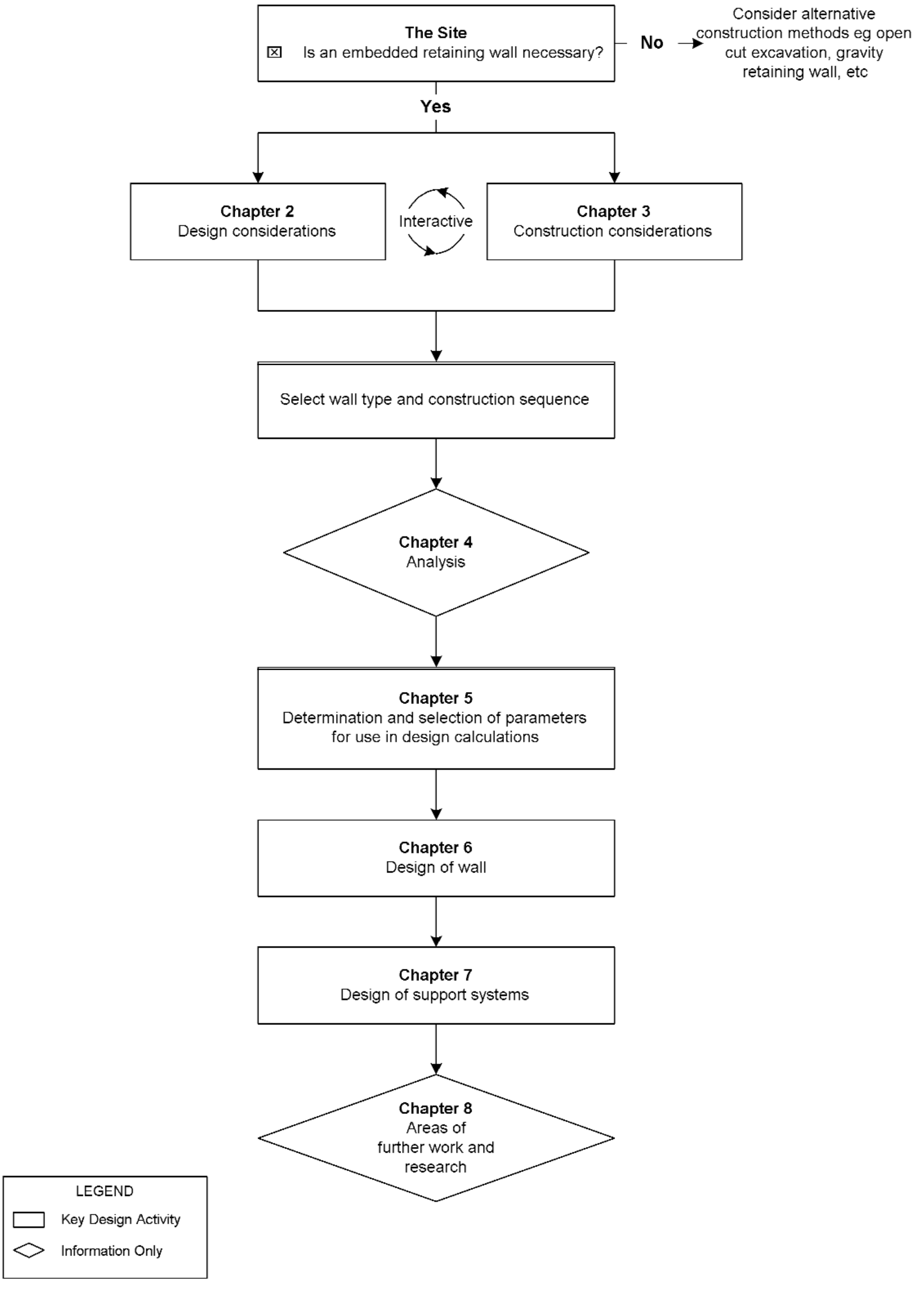


FIGURE 1.2 *Principal design stages and report layout*

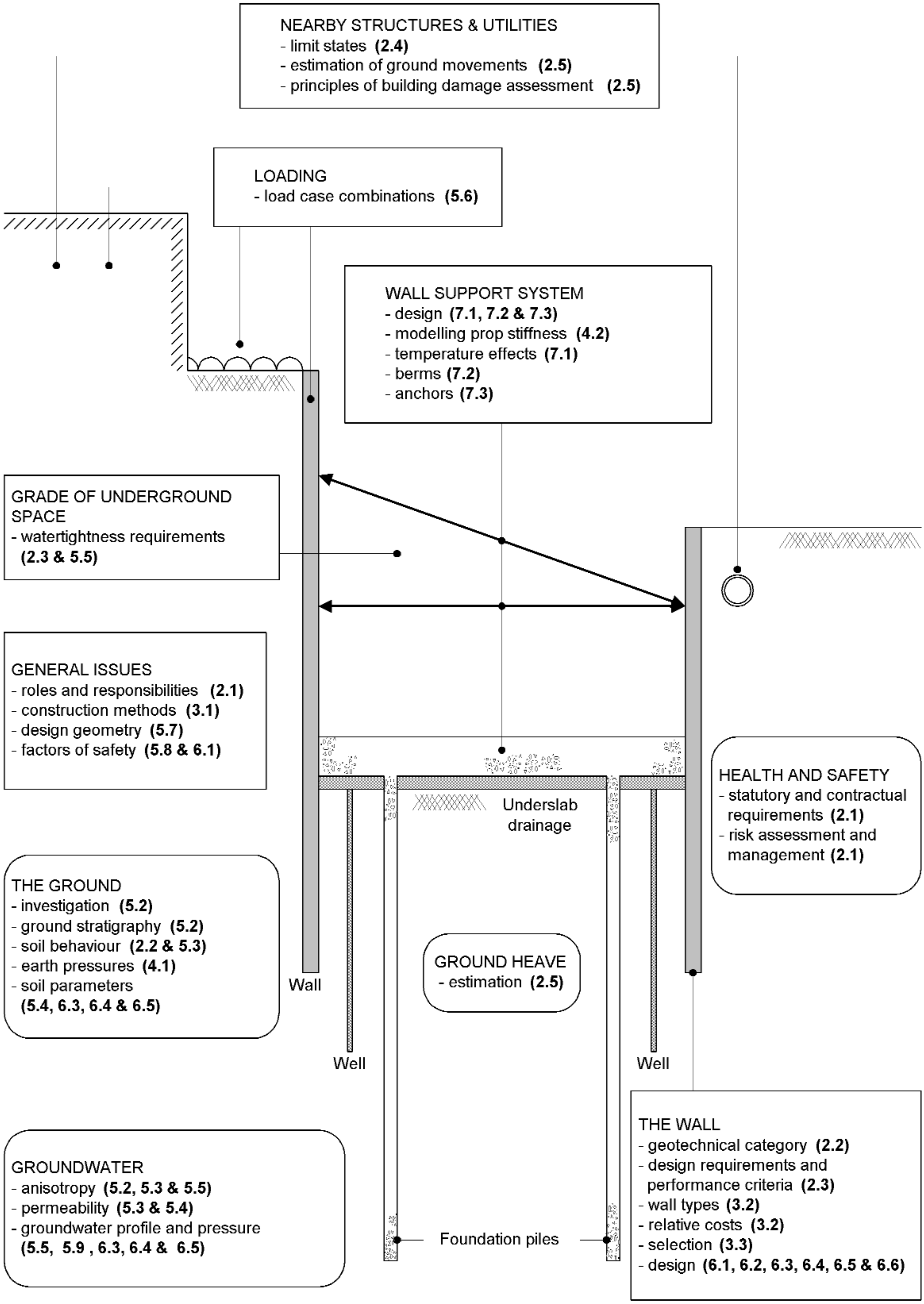


FIGURE 1.3 *Key issues considered in report.*

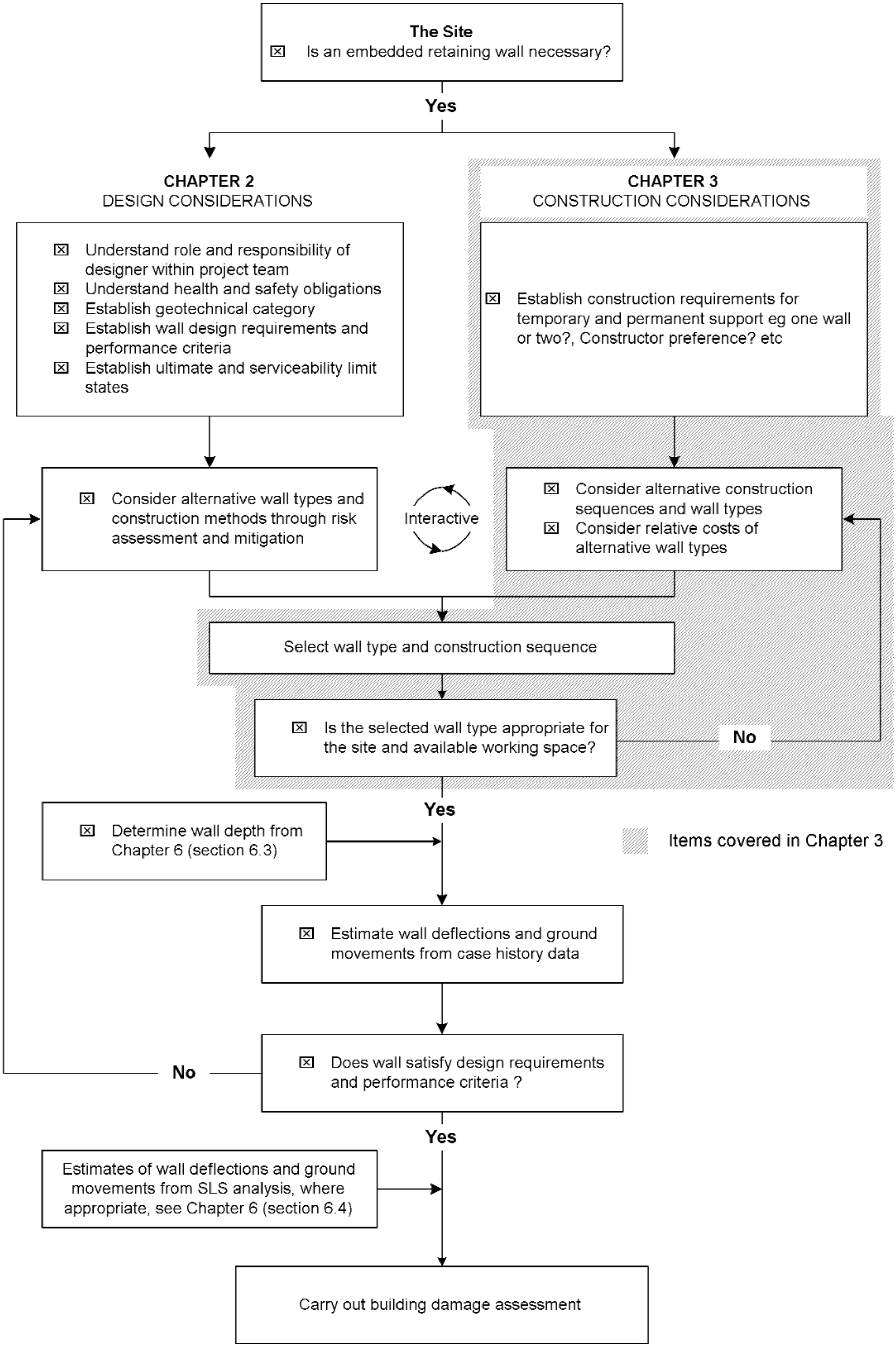


FIGURE 2.1 *Decision paths for selection of appropriate wall type and construction sequence*

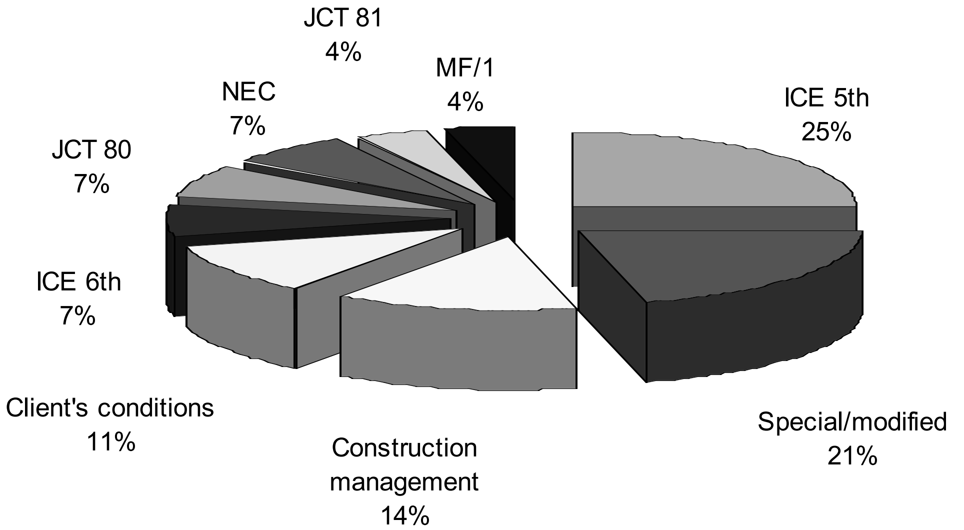


FIGURE 2.2 *Conditions of contract in use in UK construction (after Clayton, 2001)*

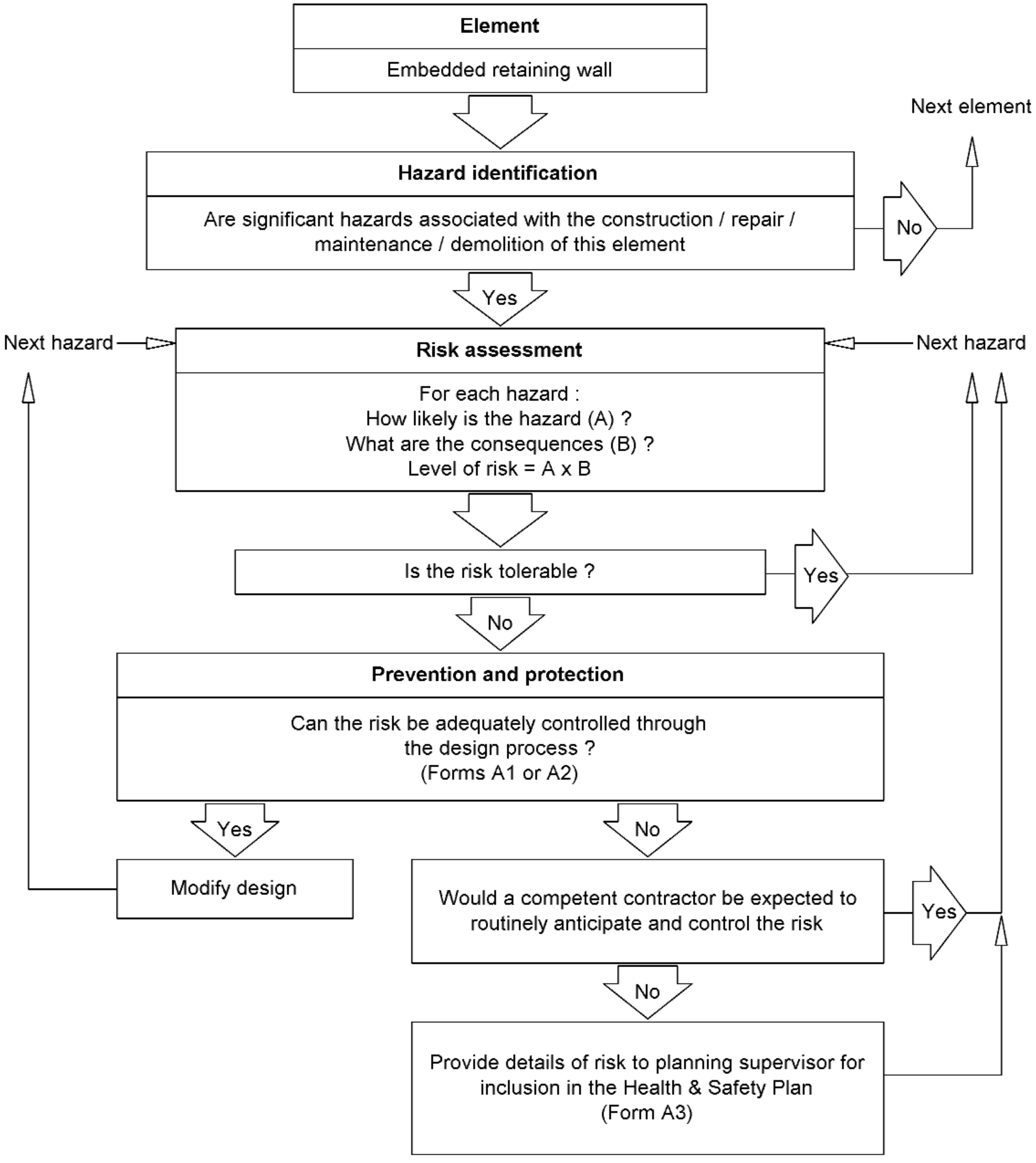


FIGURE 2.3 *CDM risk assessment*

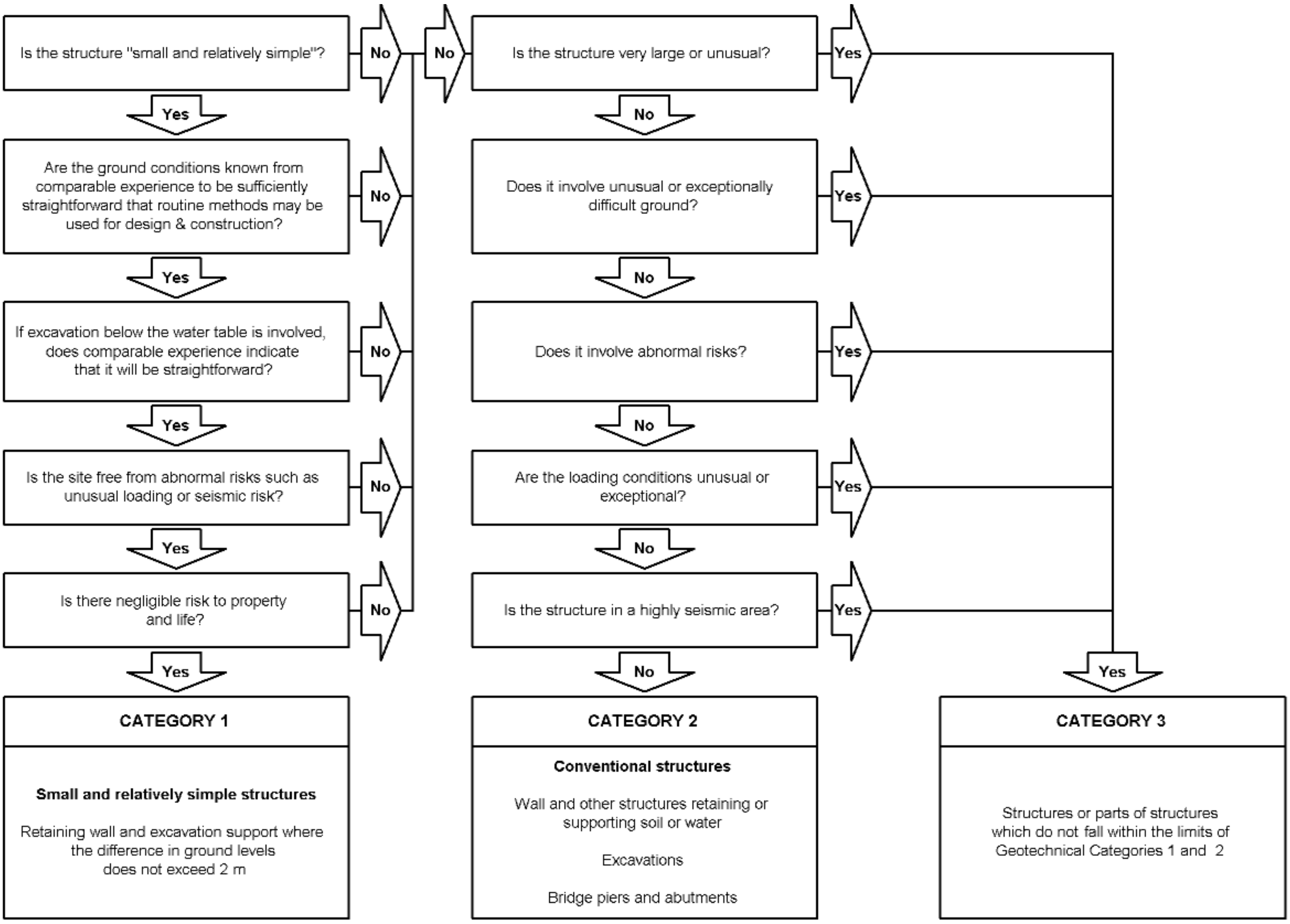


FIGURE 2.4 *Geotechnical categorisation (adapted from Simpson and Driscoll, 1998)*

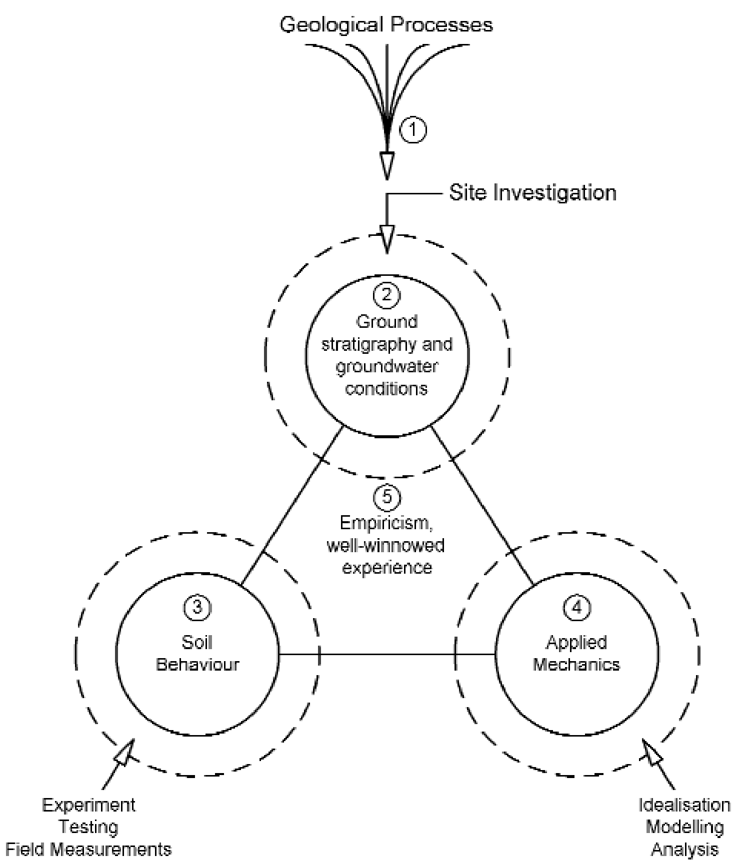
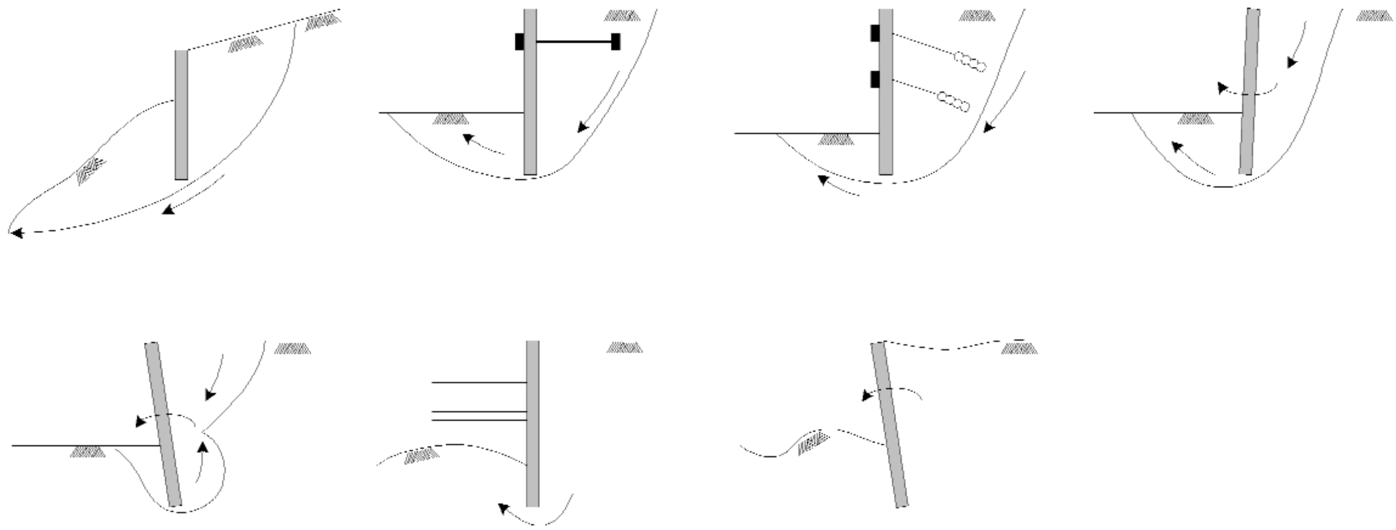
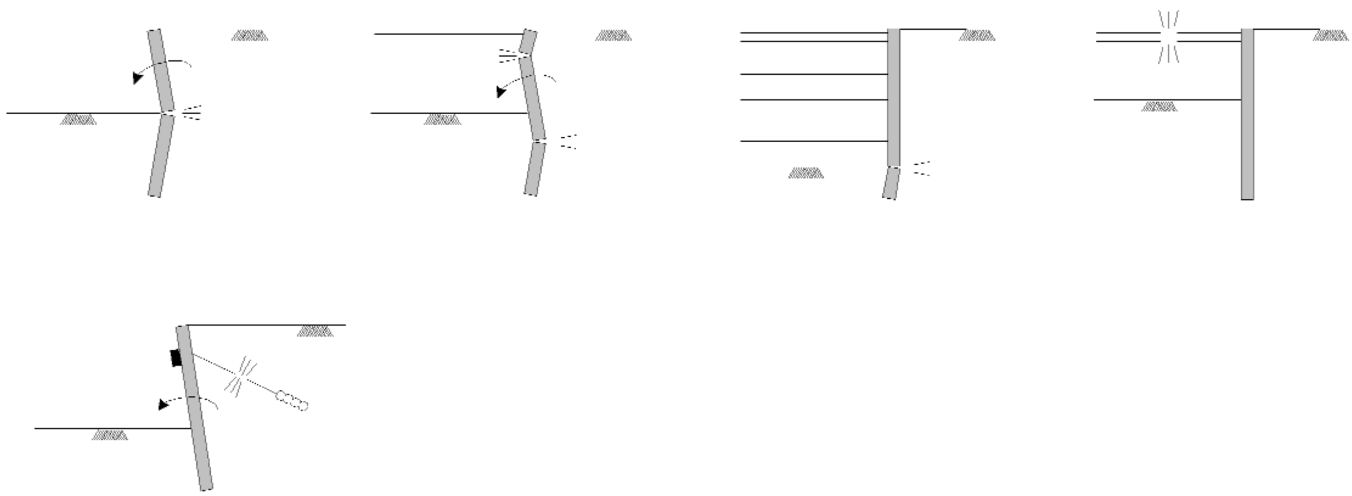


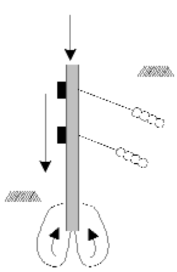
FIGURE 2.5 *Elements of geotechnical design*



(a) - *Rotational failure*



(b) - *Structural failure*



(c) - *Vertical failure*

FIGURE 2.6 *Ultimate limit state examples*

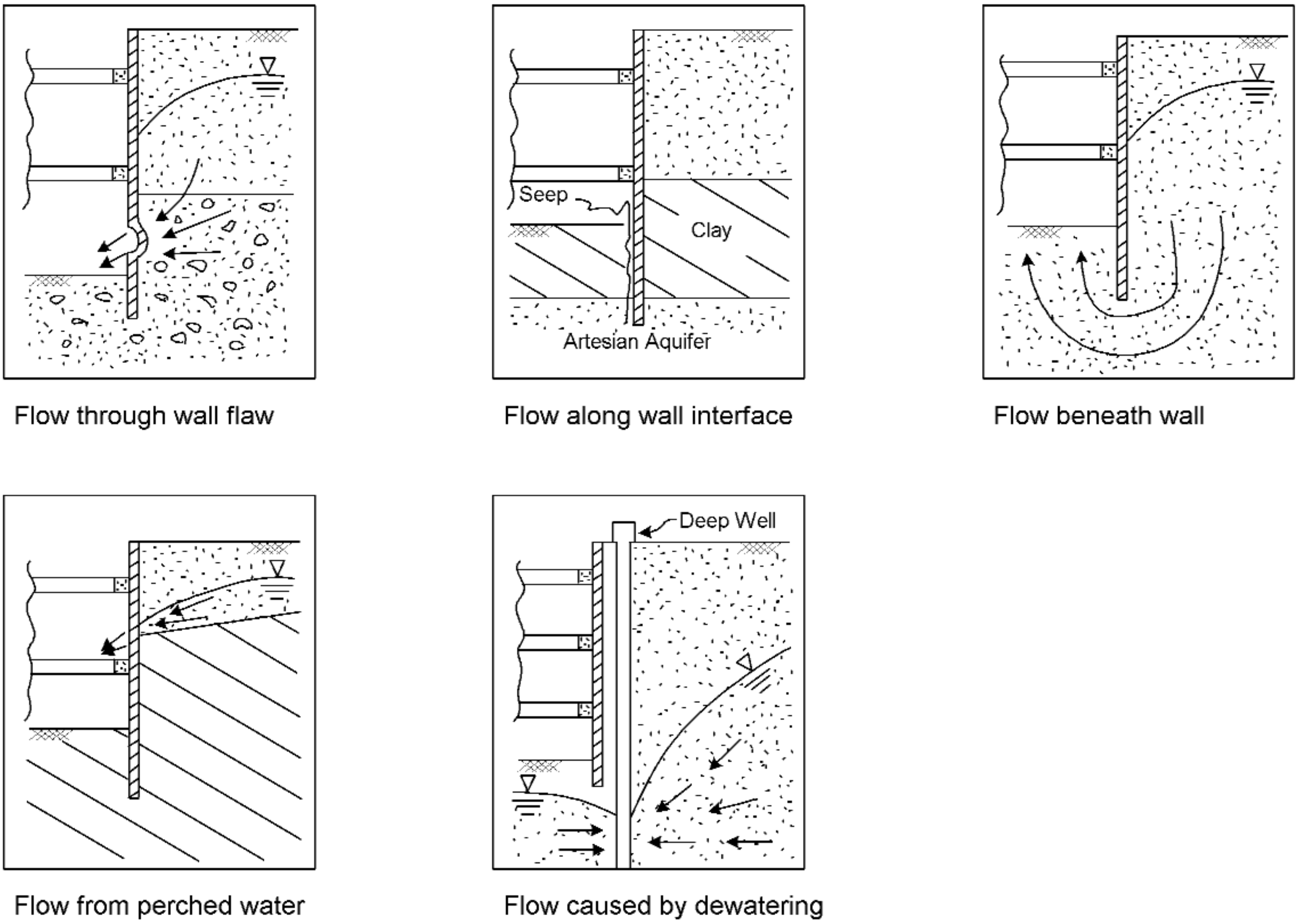
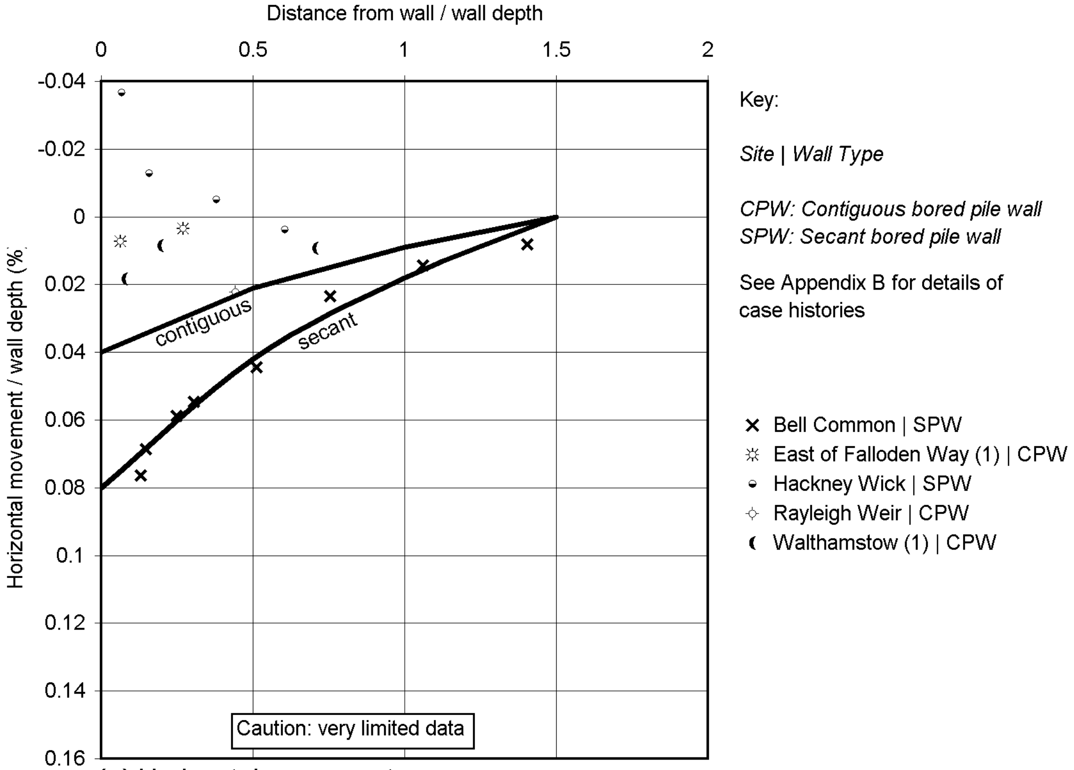
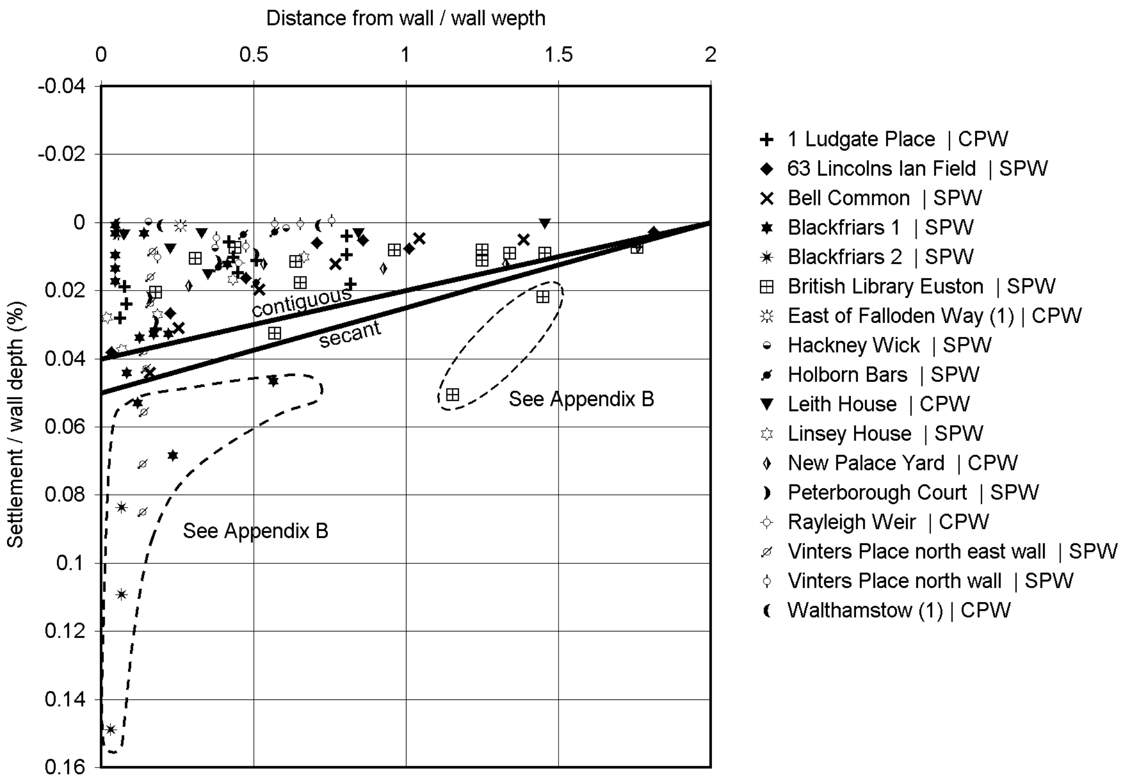


FIGURE 2.7 *Movements due to water flow (after Clough and O'Rouke, 1990)*

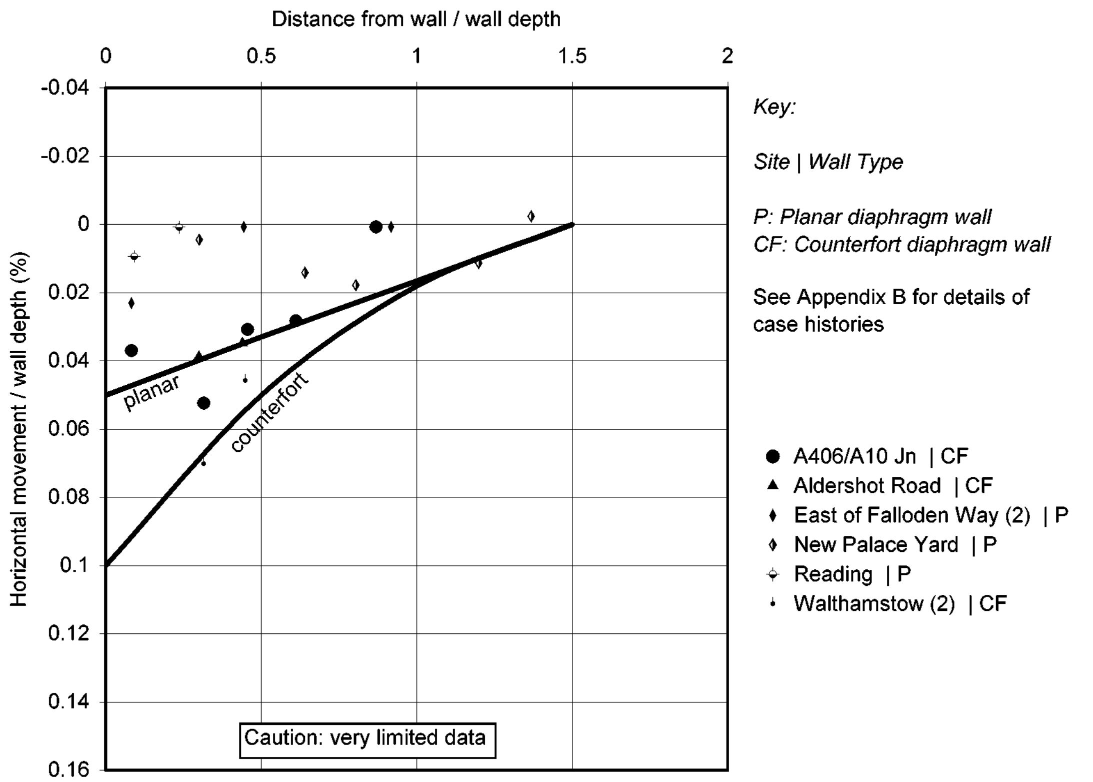


(a) Horizontal movements

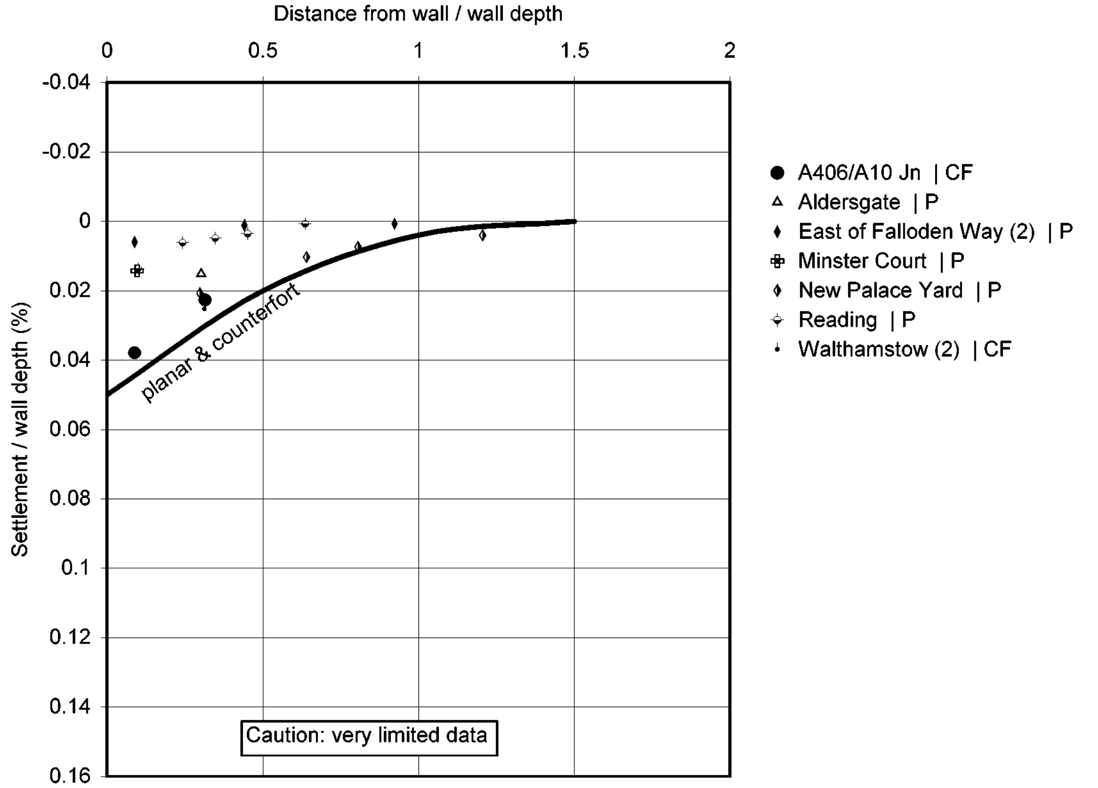


(b) Vertical movements

FIGURE 2.8 *Maximum ground surface movement during bored pile wall installation in stiff clay*



(a) Horizontal movements



(b) Vertical movements

FIGURE 2.9 *Maximum ground surface movement during diaphragm wall installation in stiff clay*

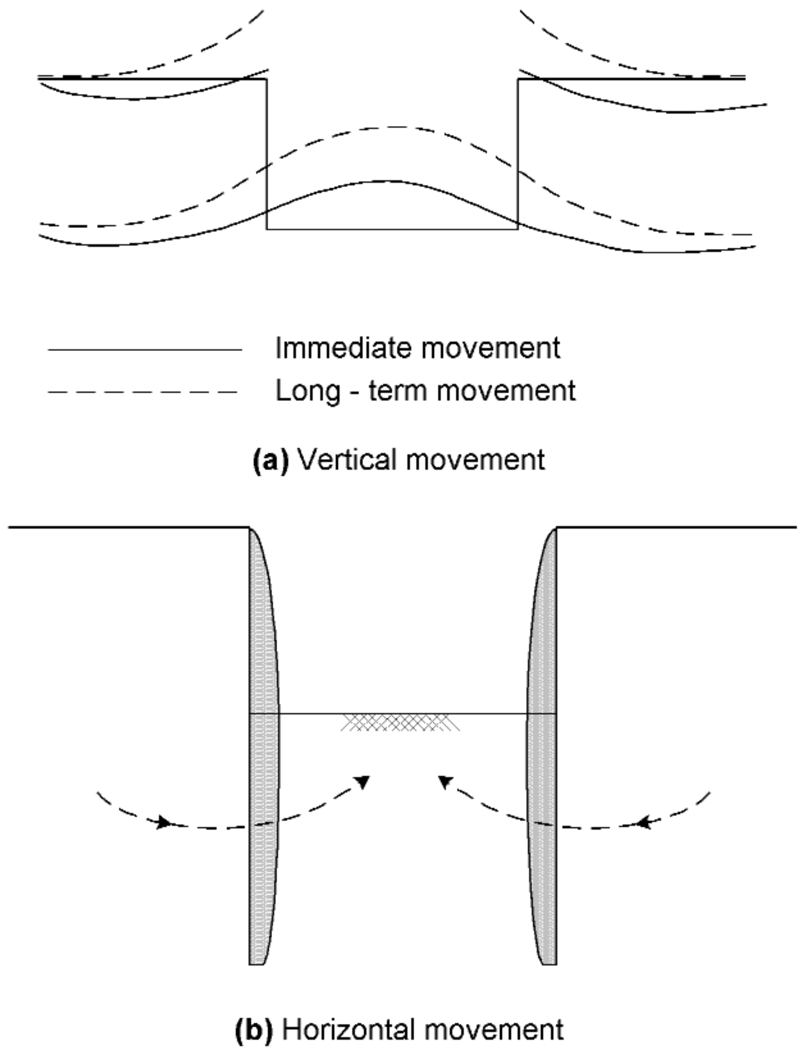


FIGURE 2.10 *Typical ground movement pattern associated with excavation stress relief*

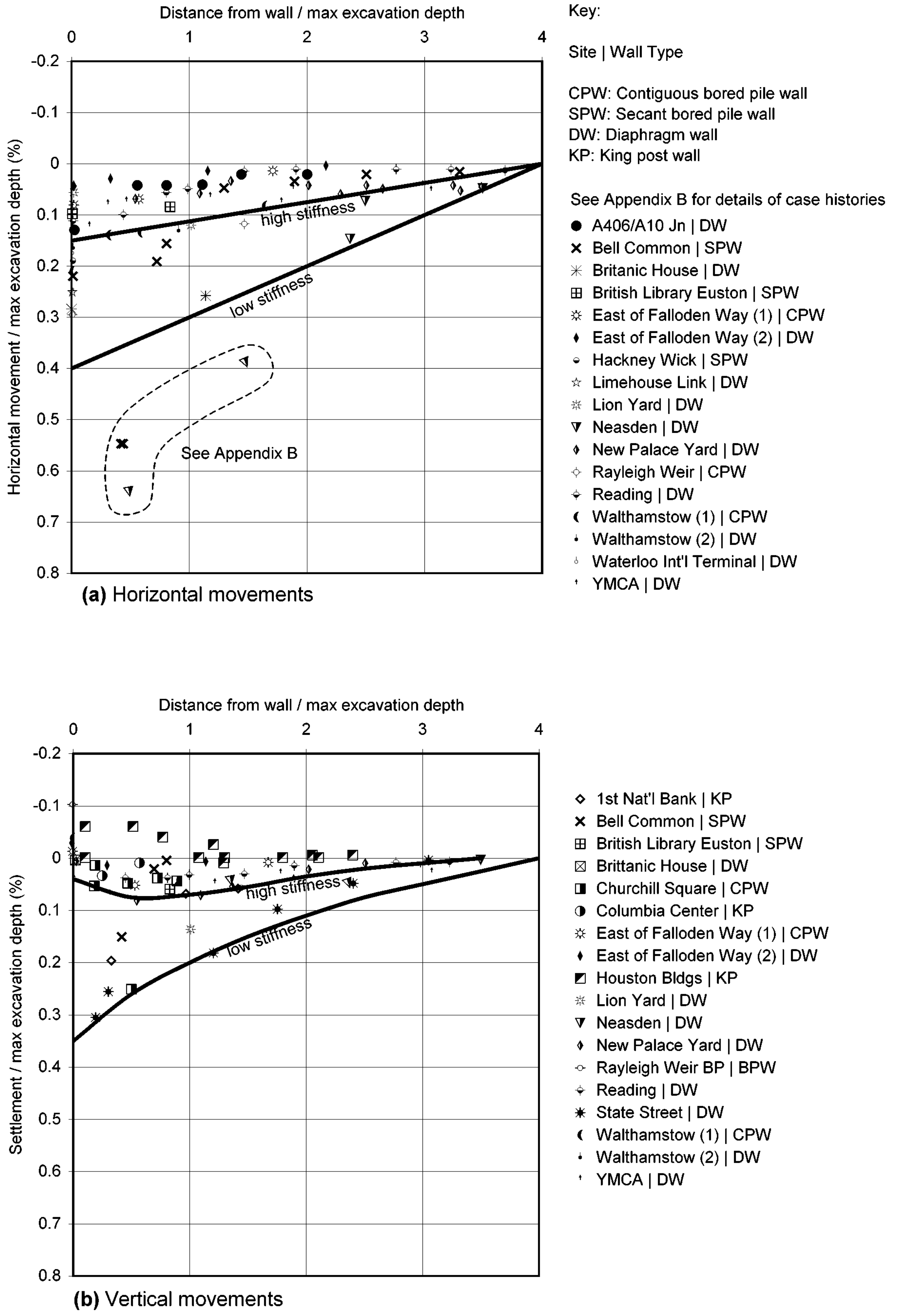


FIGURE 2.11 *Maximum ground surface movement due to excavation in front of a wall in stiff clay*

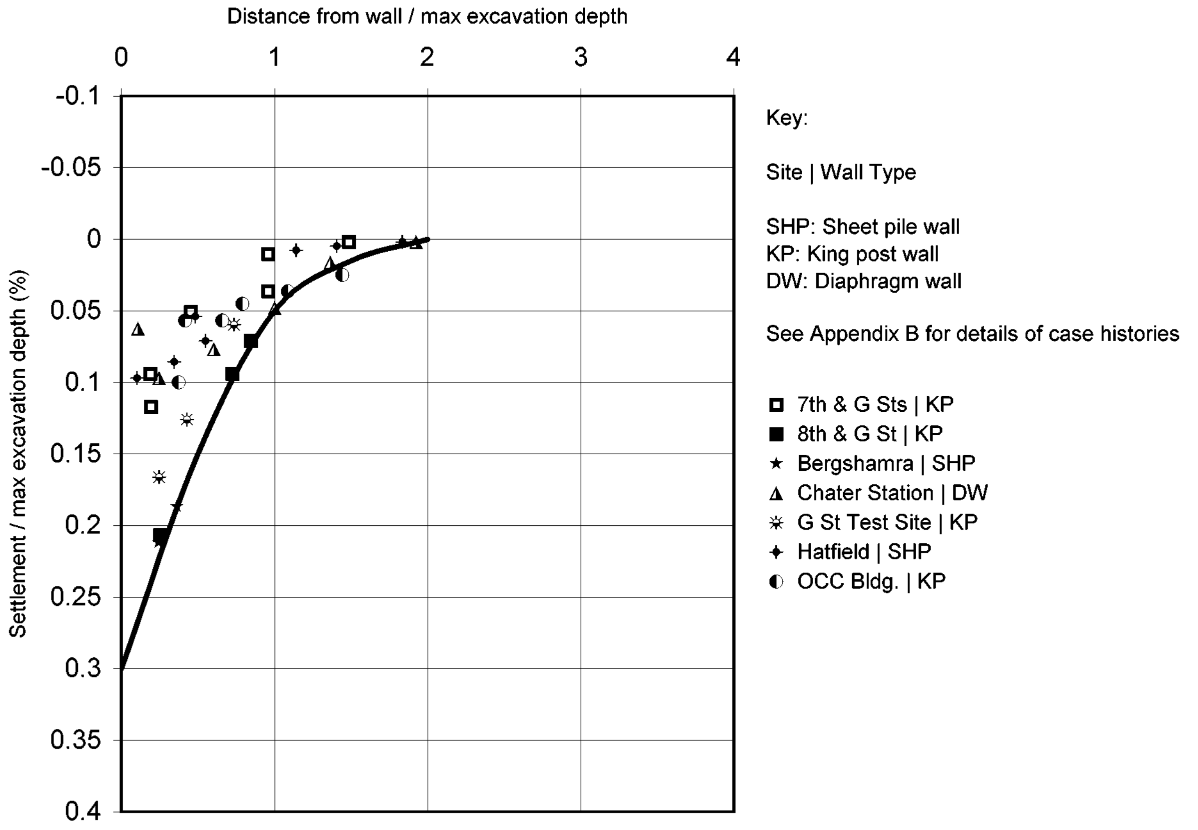


FIGURE 2.12 *Maximum ground surface settlement due to excavation in front of a wall in sand*

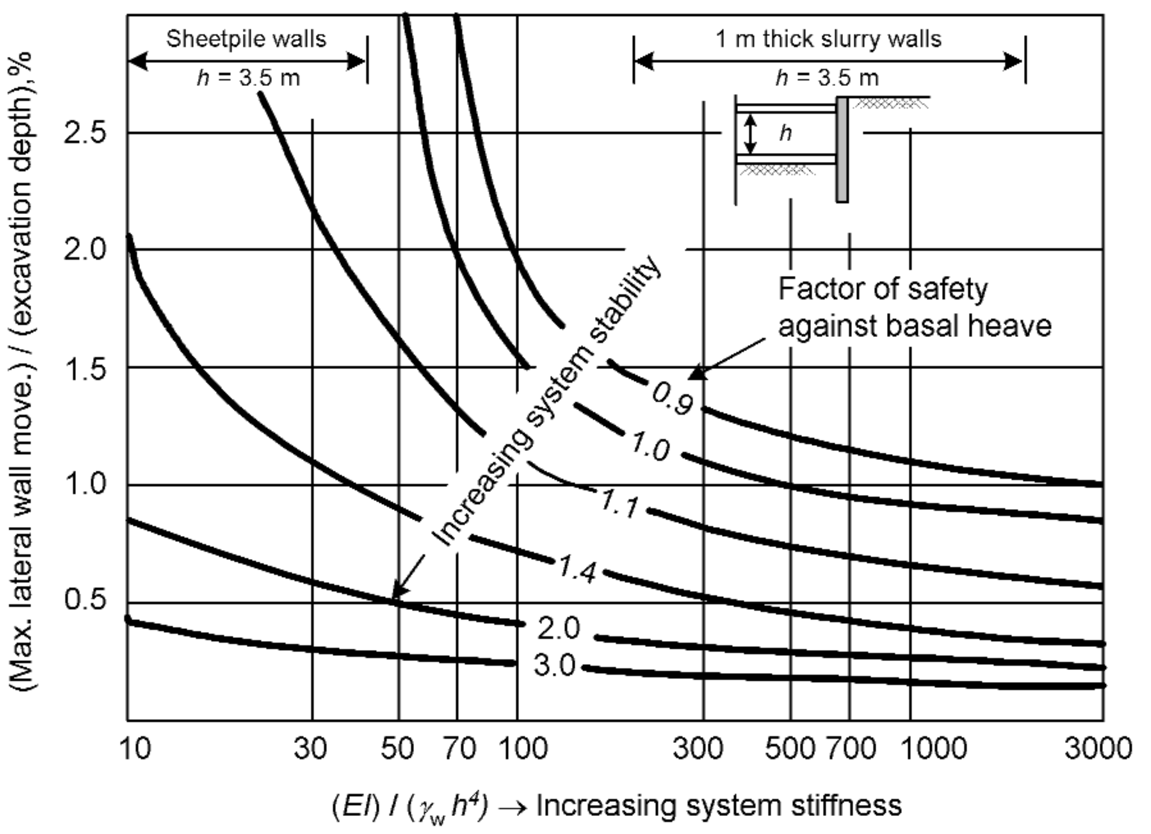


FIGURE 2.13 *Maximum lateral wall movements vs. system stiffness (after Clough et al, 1989)*

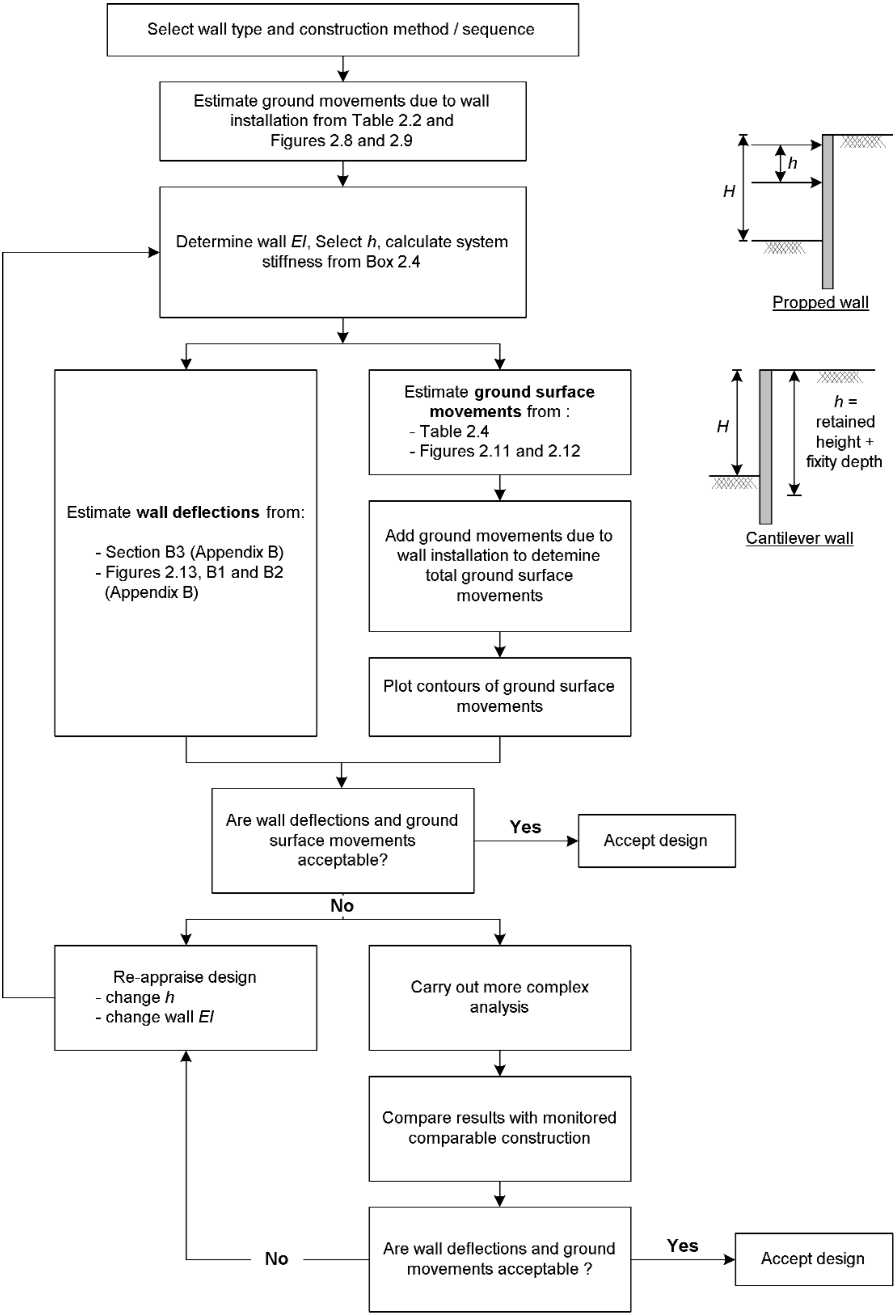


FIGURE 2.14 *Procedure for prediction of wall deflections and ground surface movements*

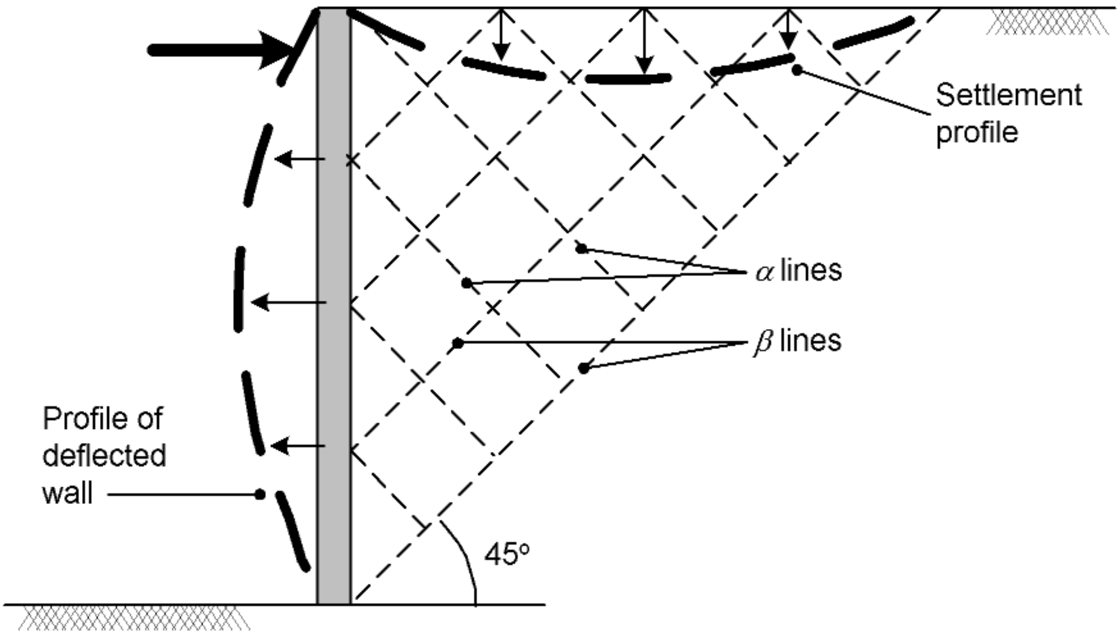


FIGURE 2.15 *Simple field of plastic deformation (Milligan, 1983)*

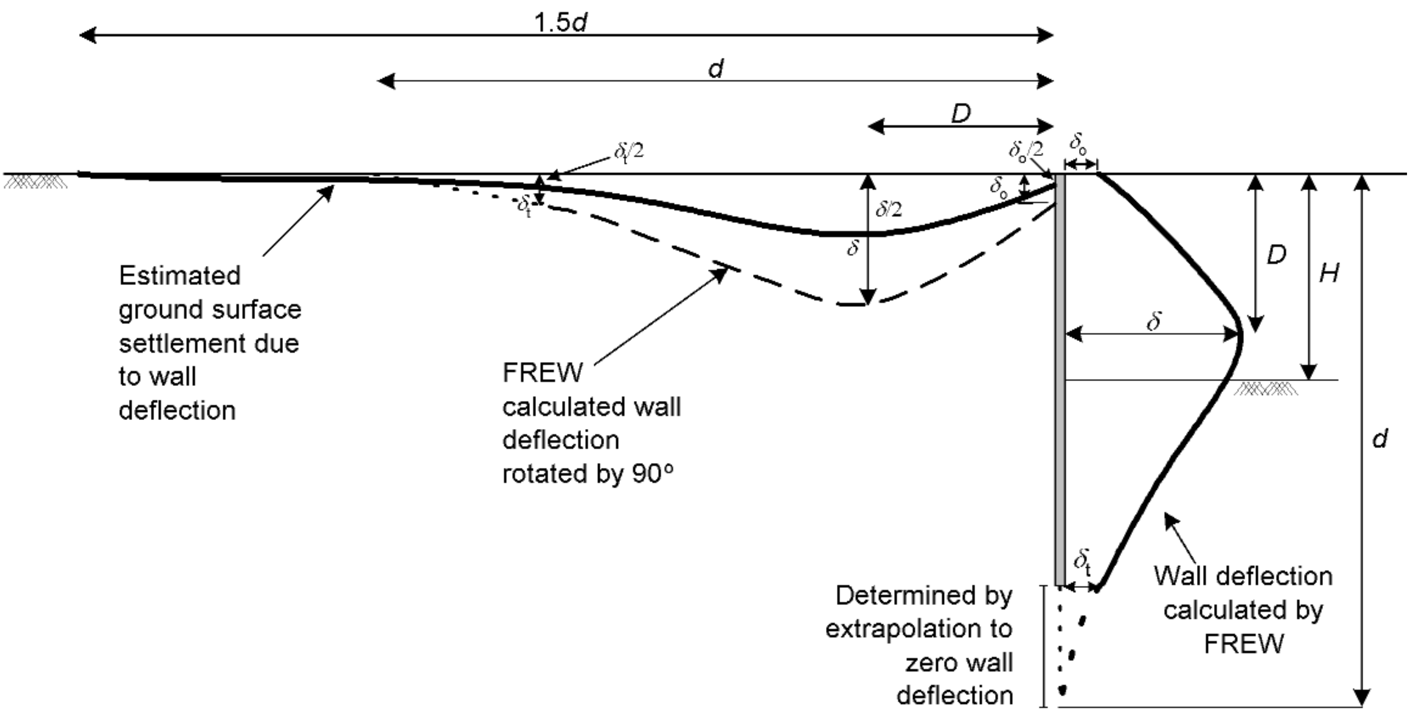


FIGURE 2.16 *Relationship between analysed lateral (propped) wall defections and predicted ground surface settlements in stiff soil (Arup unpublished)*

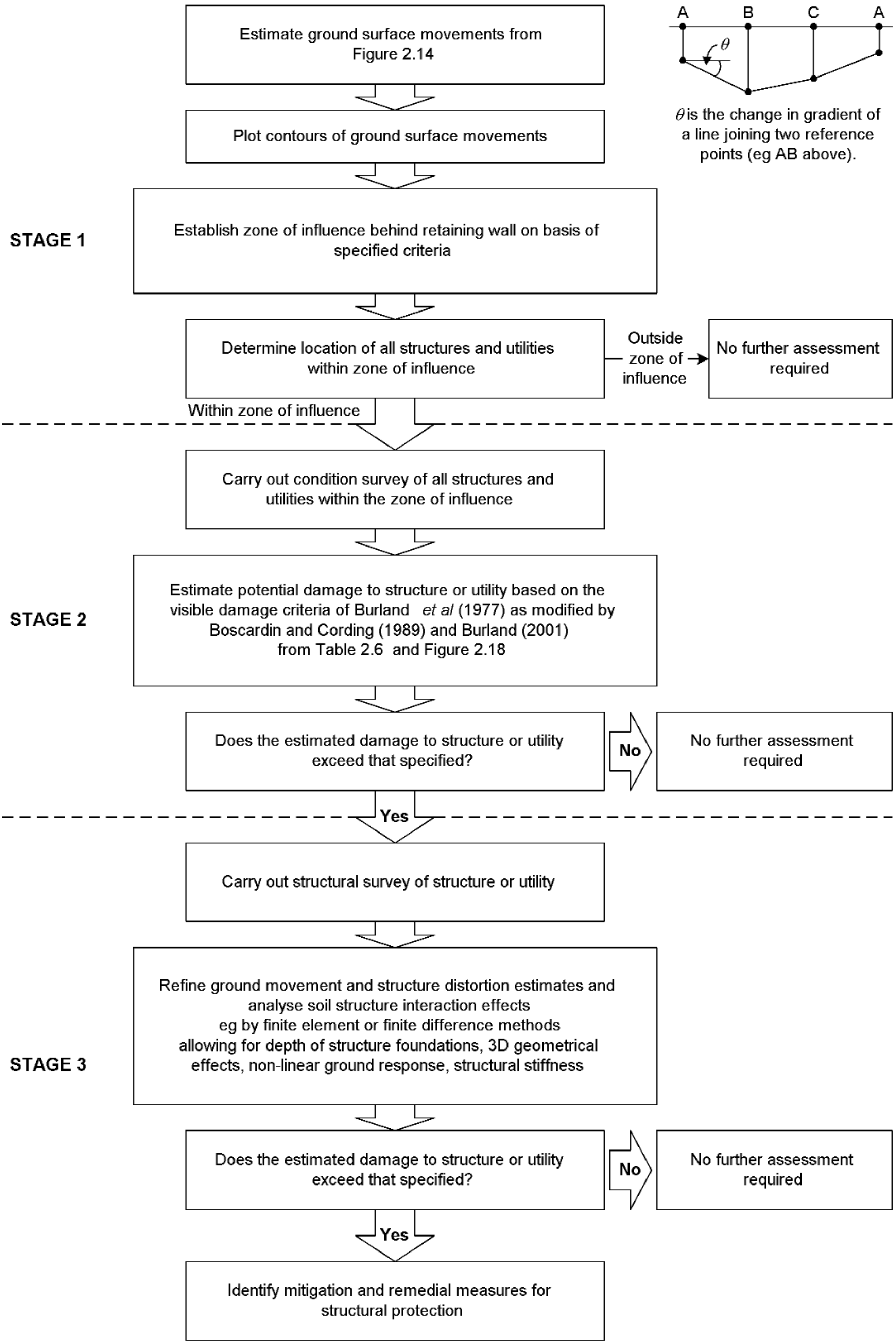
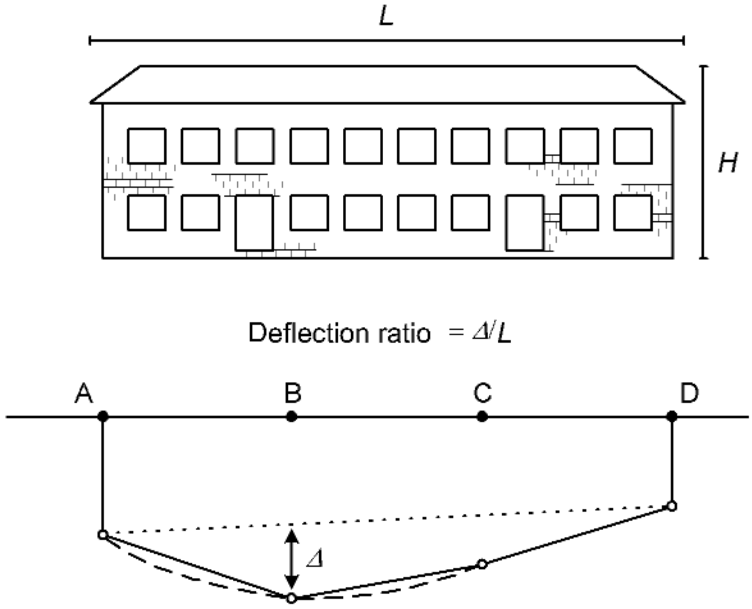
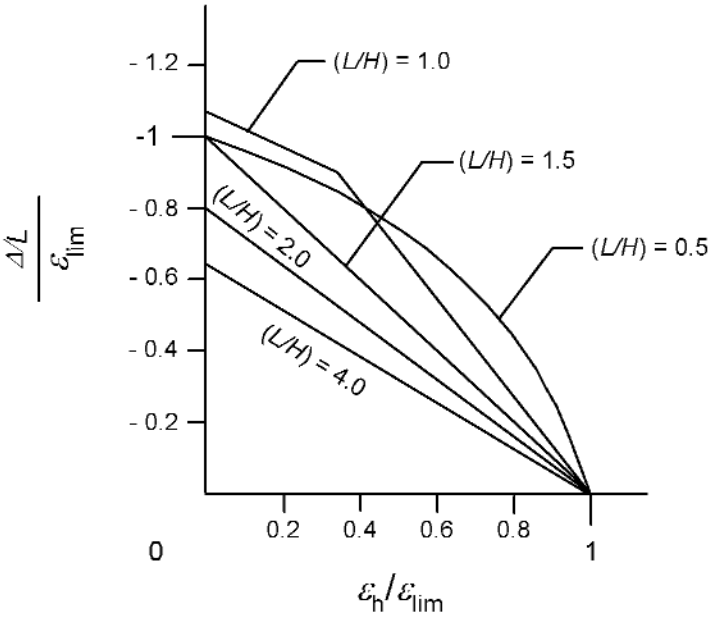


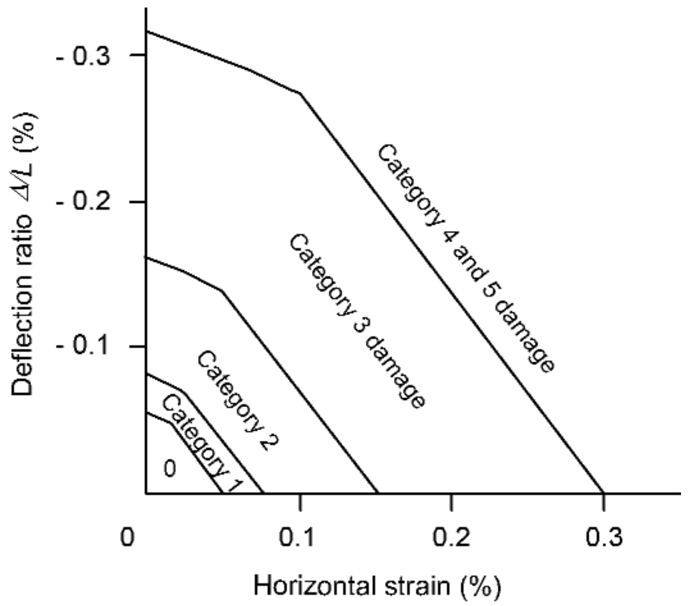
FIGURE 2.17 *Procedure for building damage assessment*



(a) Definition of deflection ratio.



(b) Influence of horizontal strain on Δ/L / ε_{lim} (after Burland, 2001)



(c) Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0 (after Burland, 2001)

By adopting values of ε_{lim} associated with the various damage catgories given in Table 2.6, Figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_{h} for a particular value of L/H Figure (c) shows such a diagram for (L/H) = 1.0.

FIGURE 2.18 *Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)*

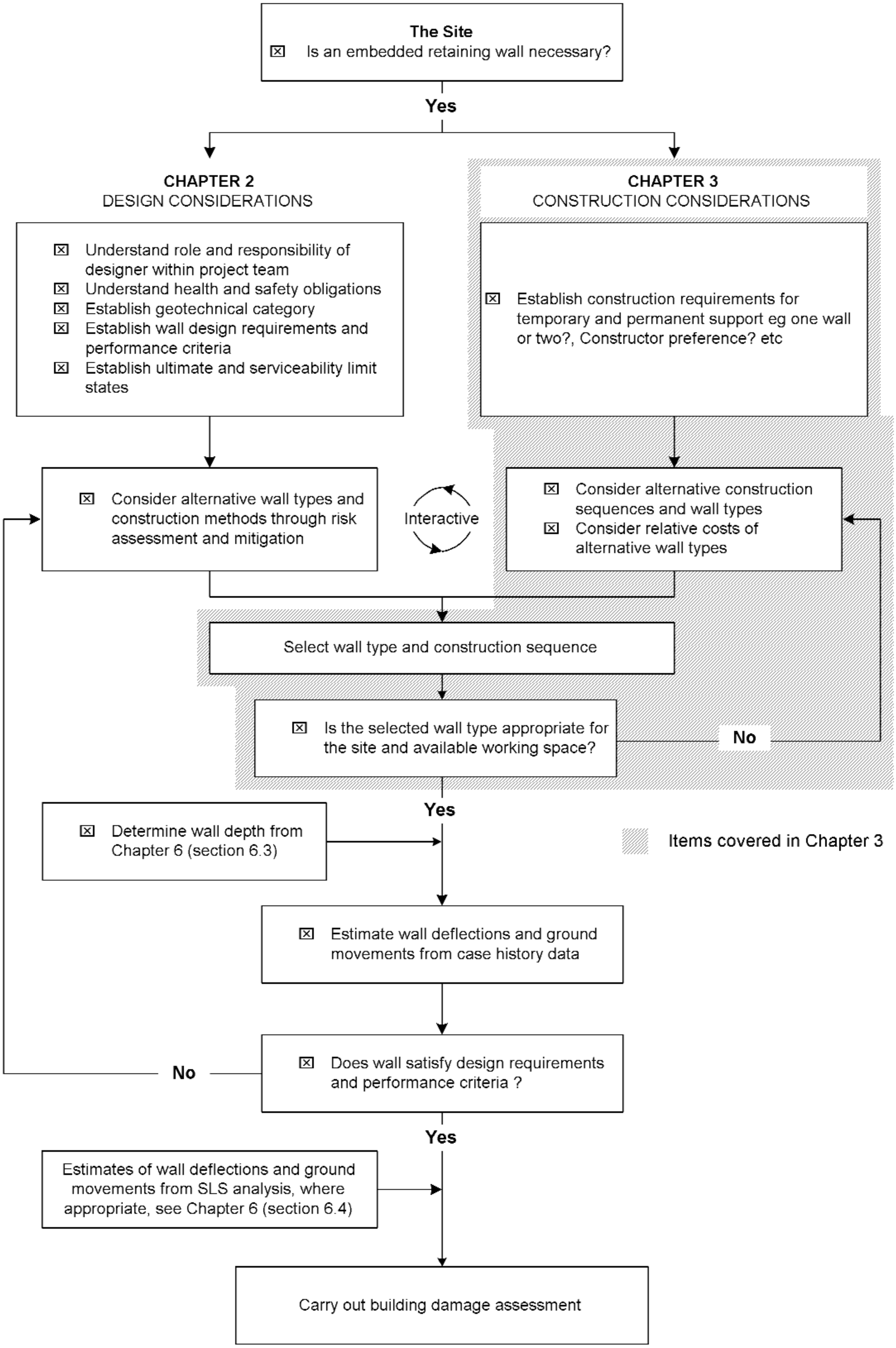


FIGURE 2.1 *Decision paths for selection of appropriate wall type and construction sequence*

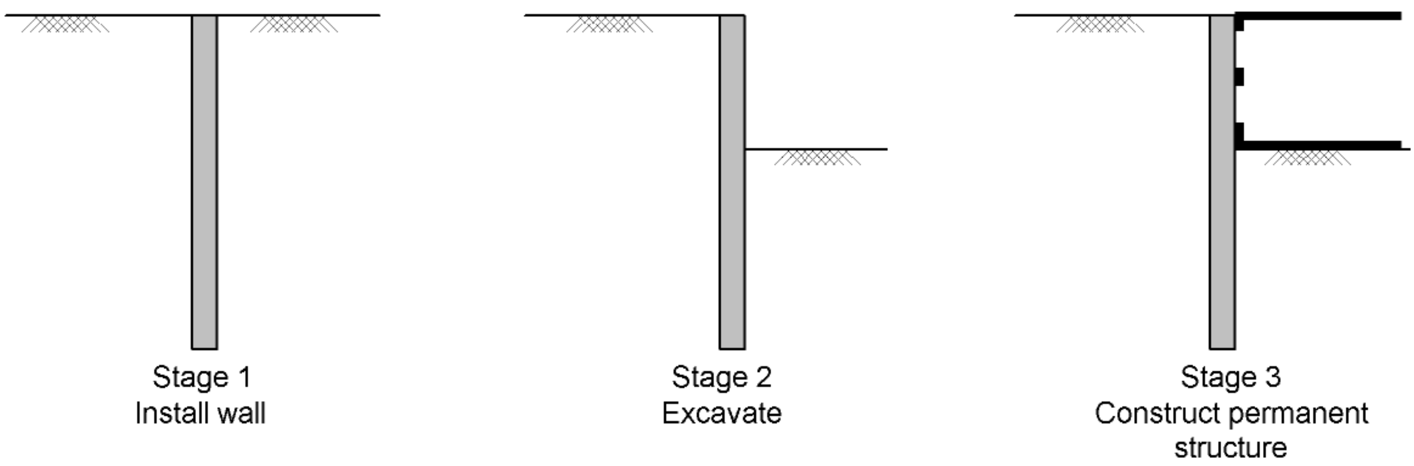


FIGURE 3.2 *Cantilever wall construction sequence*

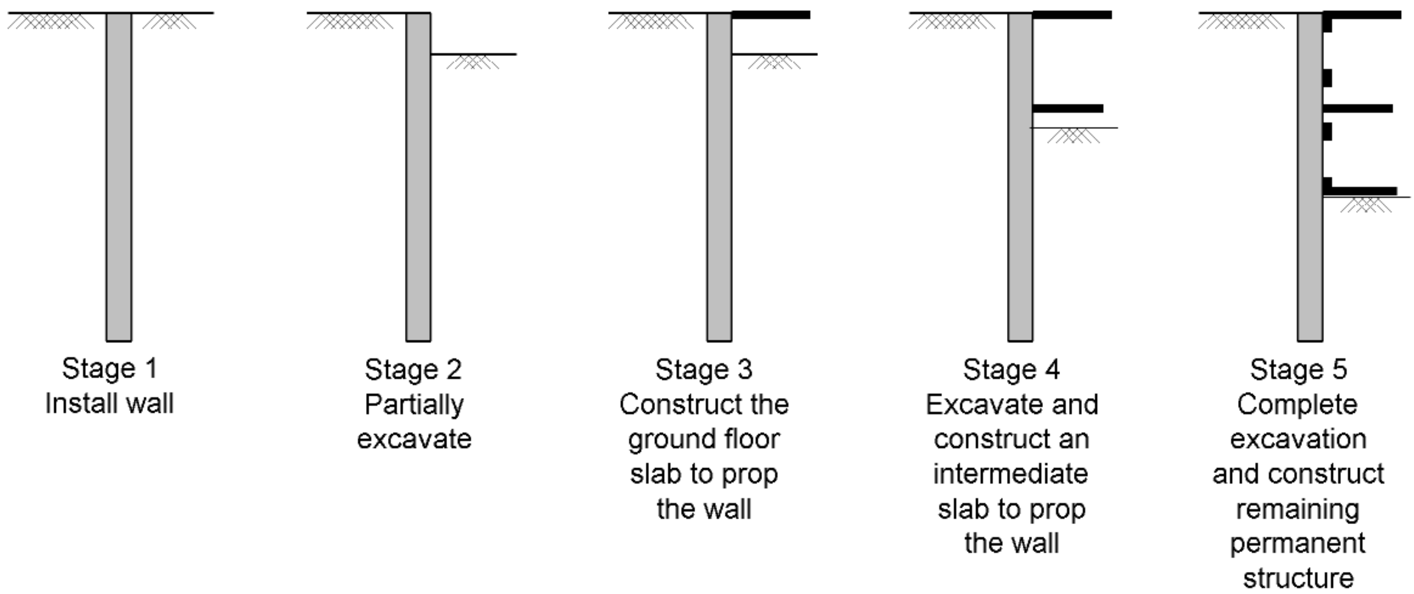


FIGURE 3.3 *Top down construction sequence*

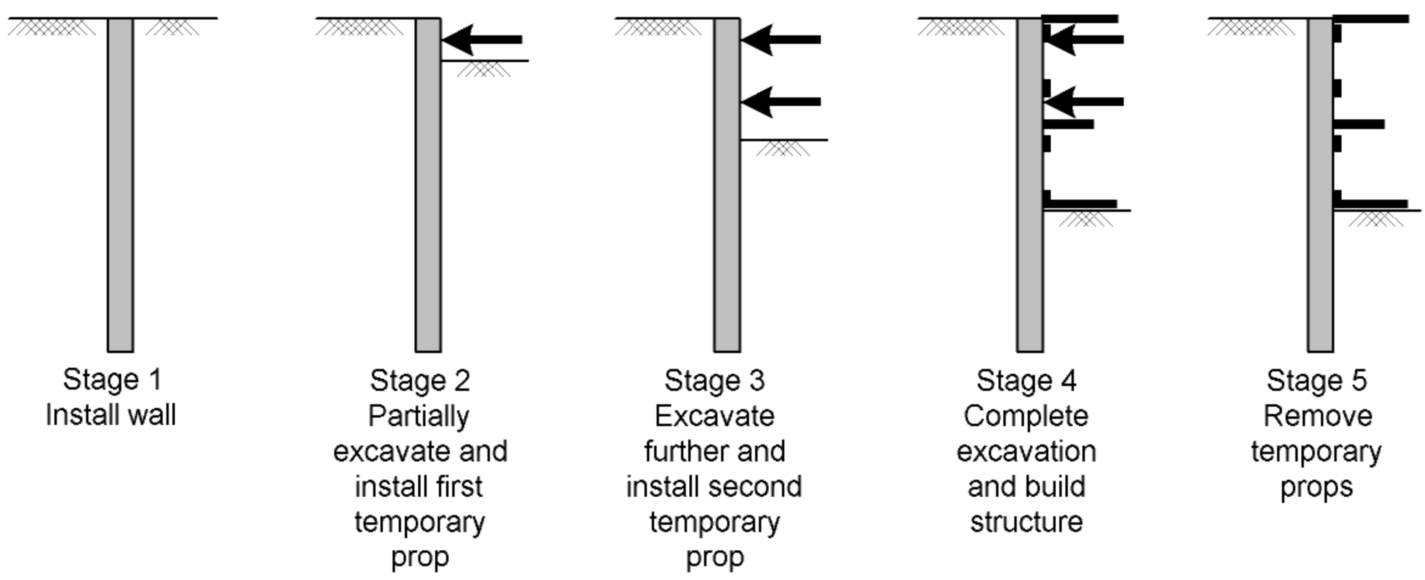


FIGURE 3.4 *Bottom up construction sequence*

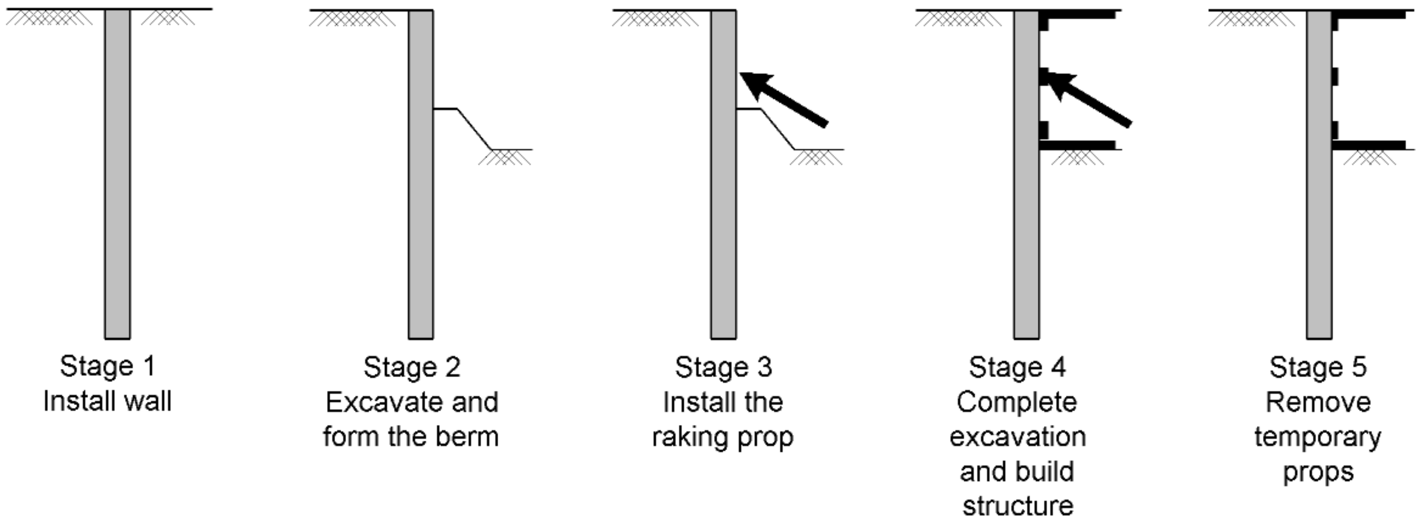


FIGURE 3.5 *The use of a berm and raking props*

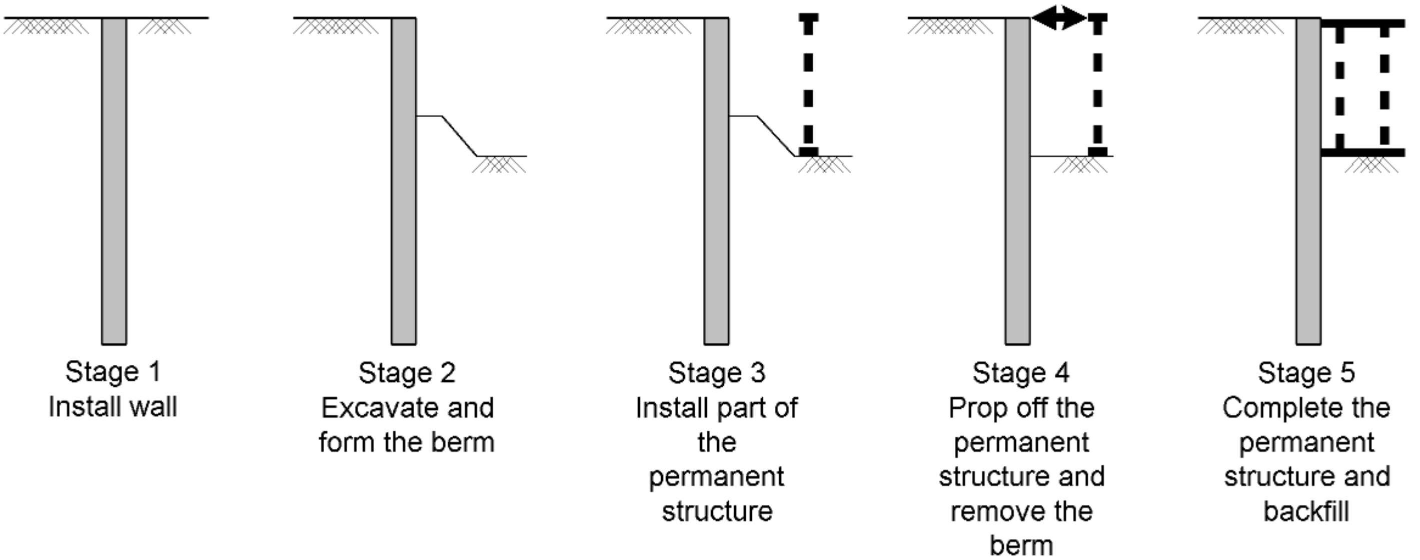


FIGURE 3.6 *The use of a berm and a prop to the permanent structure*

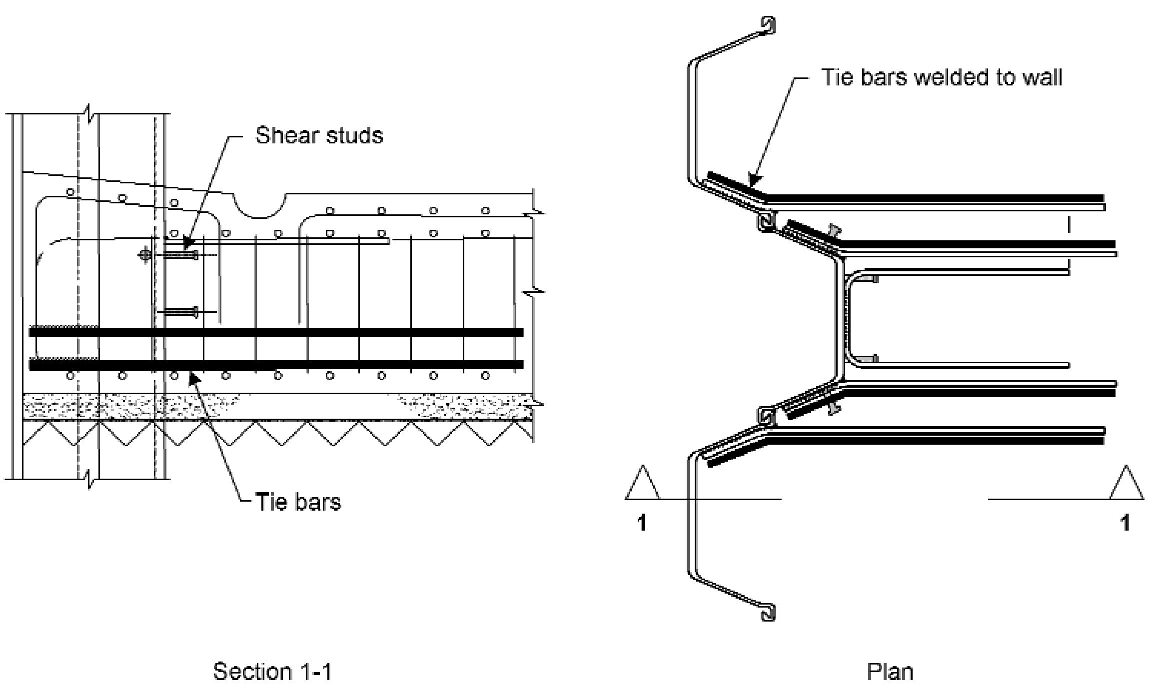
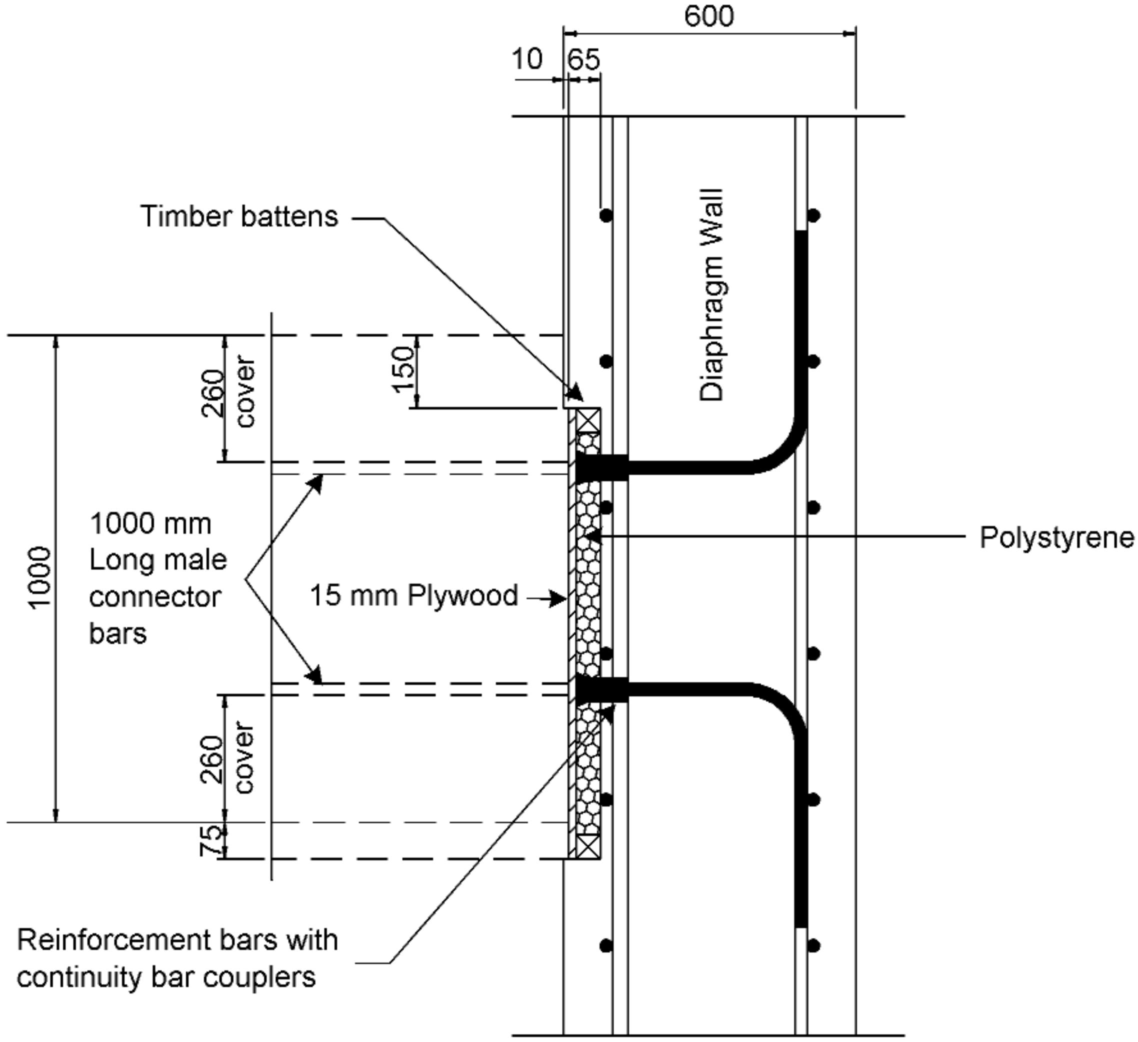


FIGURE 3.7 *Typical connection detail at sheet pile wall/concrete slab*



Sectional elevation through diaphragm wall showing the future slab connection

IGURE 3.8 *Typical detail for couplers cast within a diaphragm wall panel*

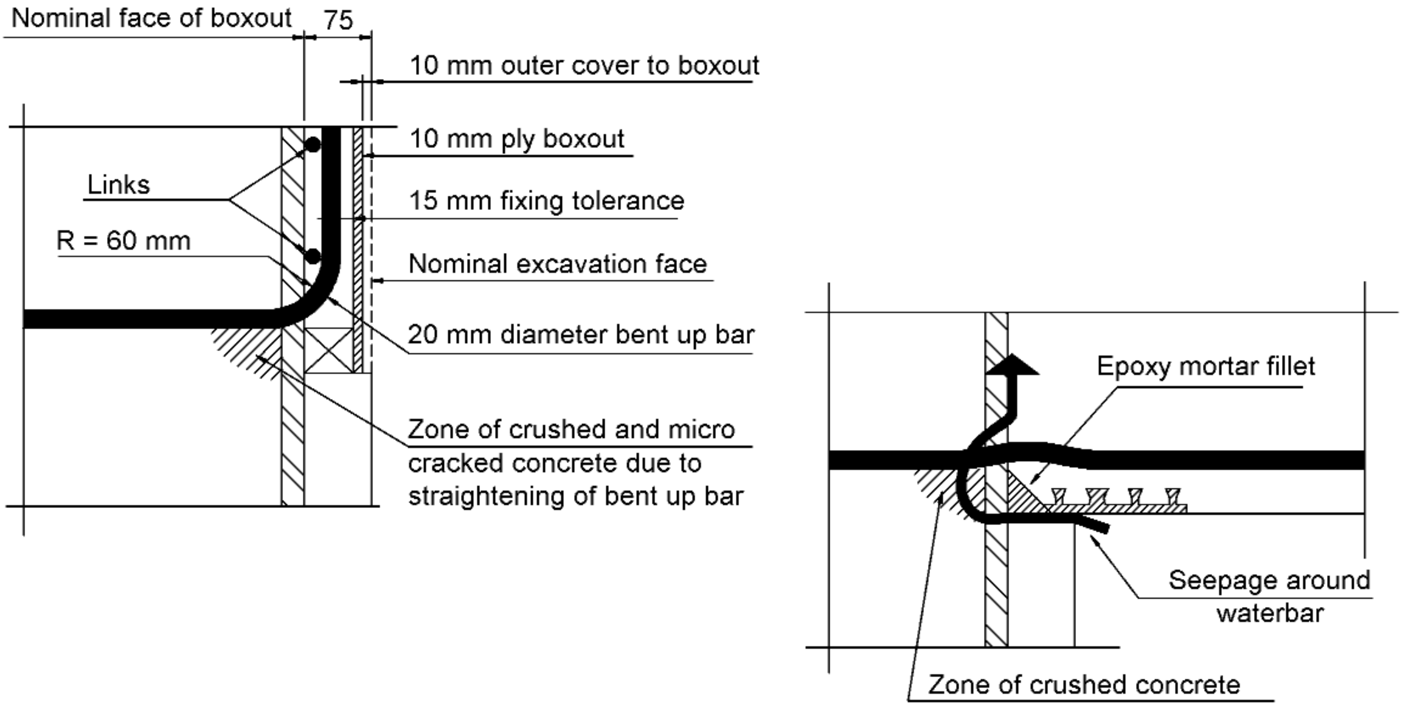


FIGURE 3.9 *Typical detail of bent out bars in diaphragm wall panel*

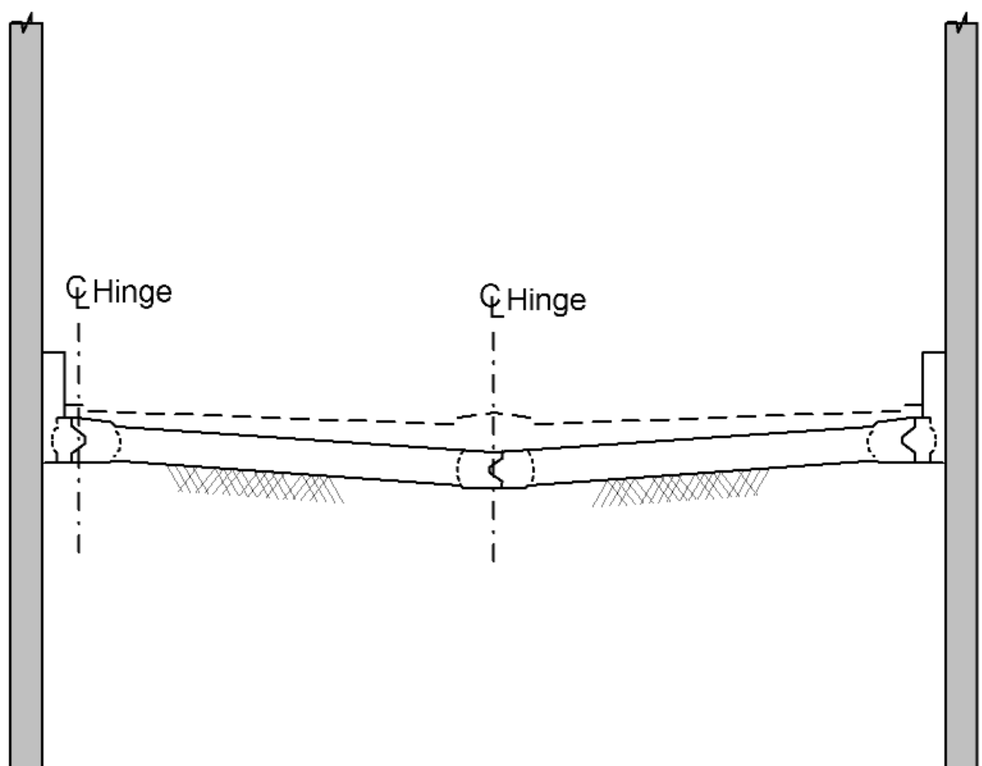


FIGURE 3.10 *Hinged slab : A406 North Circular Road, London*

PLATE 3.1 *Temporary and permanent works: Bristol underground car park. An example of the use of a sheet pile wall as the permanent wall, exposed and painted.*

PLATE 3.2 T*op down construction sequence.*

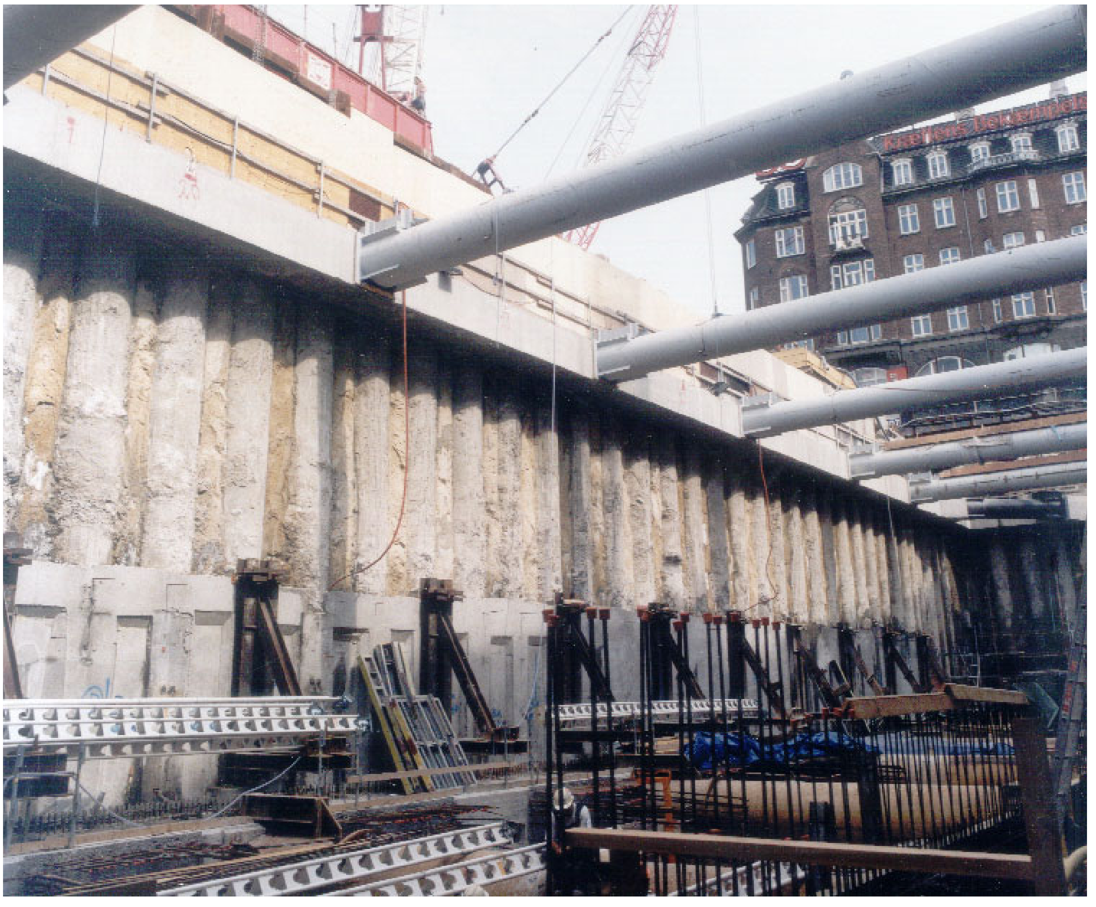


PLATE 3.3 *Bottom up construction sequence*

PLATE 3.4 *Circular shaft under construction at Blackpool, Lancashire*

PLATE 3.5 *Various support systems to sheet pile walls at Thelwall Viaduct, Merseyside*

PLATE 3.6 *Temporary props spanning full width of excavation for the Mayfair car park, London*

PLATE 3.7 *Berm with raking props at Canary Wharf, London*

PLATE 3.8 *Berm with low level permanent propping at Batheaston Bypass*

PLATE 3.9 *Anchored contiguous bored pile wall*

PLATE 3.10 *Sheet pile wall/concrete slab connection at Bristol underground car park*

PLATE 3.11 *Hinged joint: A406 North Circular Road, London*

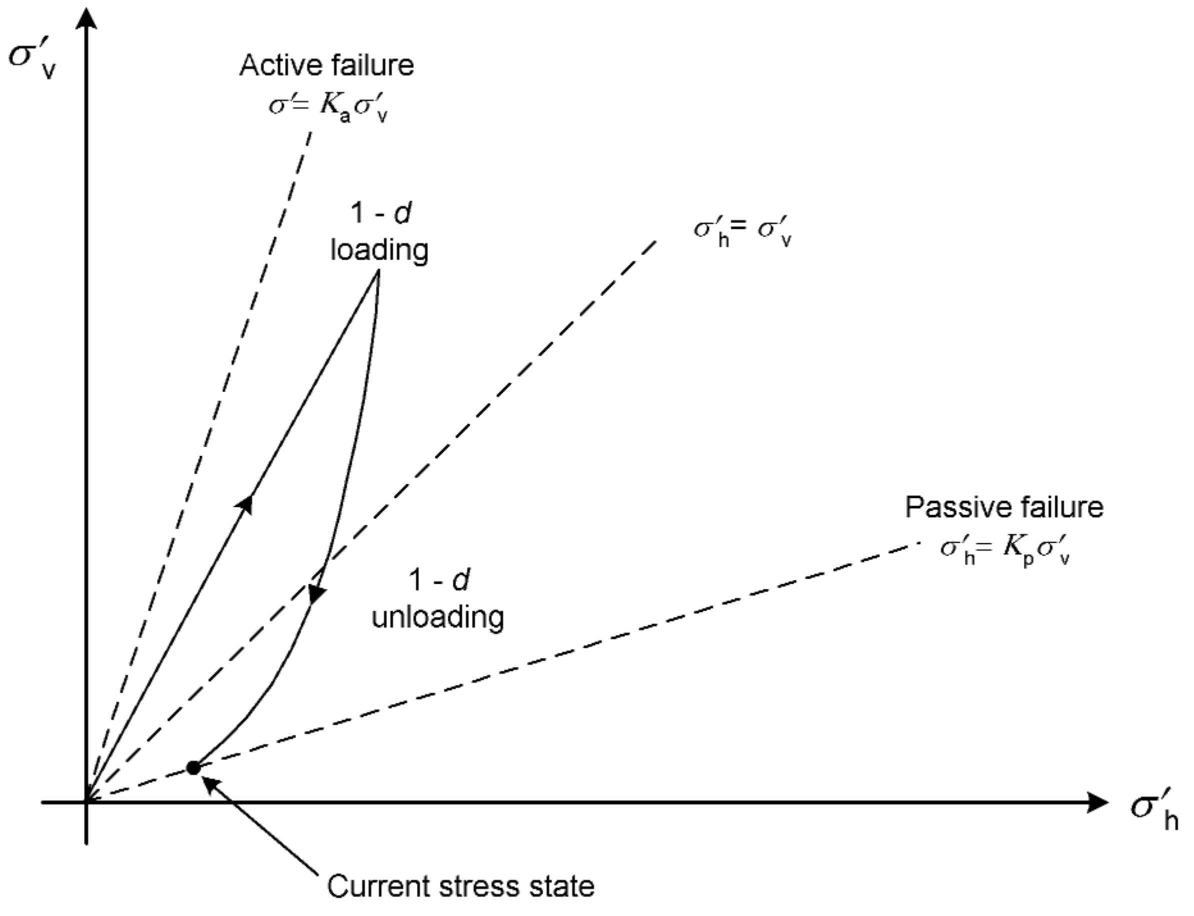


FIGURE 4.1 *Schematic stress history of an overconsolidated clay deposit*

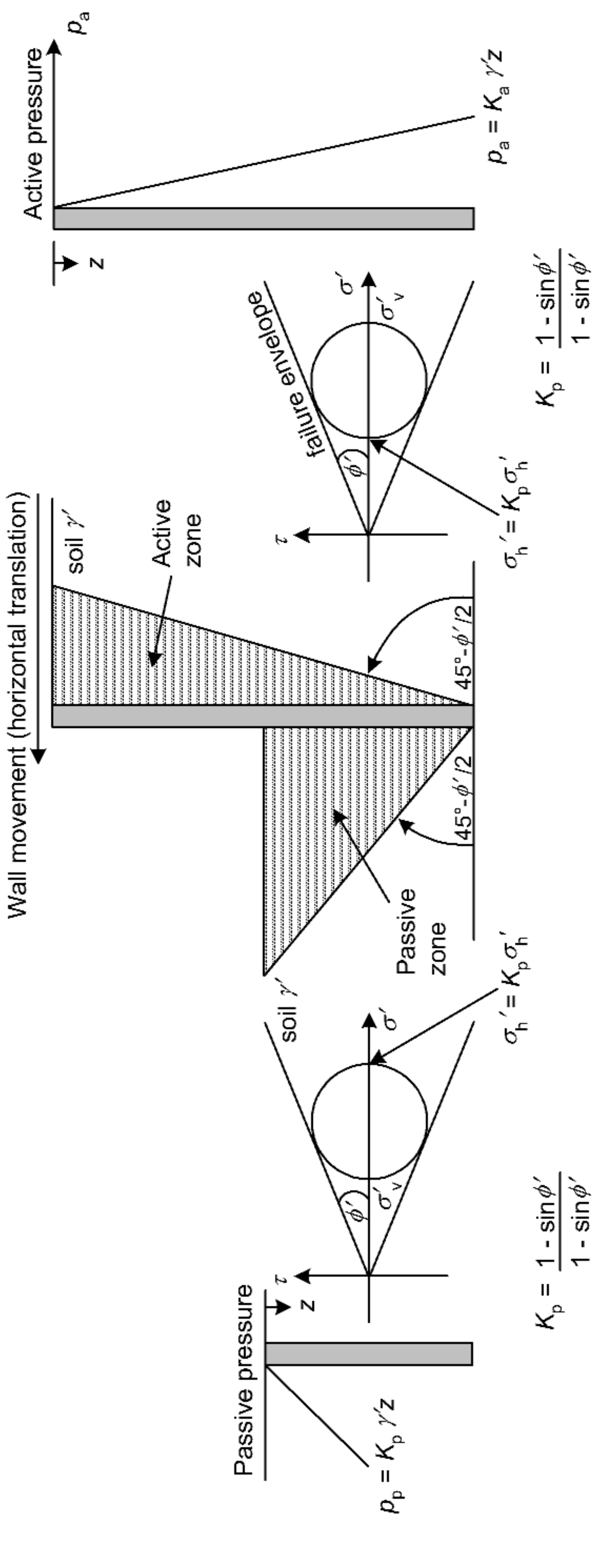
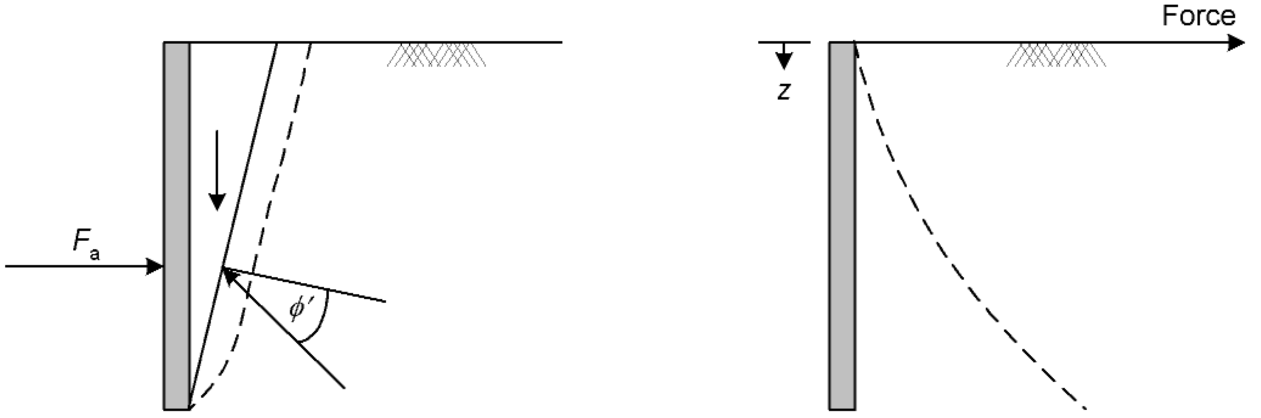


FIGURE 4.2 *Rankine plastic equilibrium for a frictionless wall or soil interface translating horizontally*

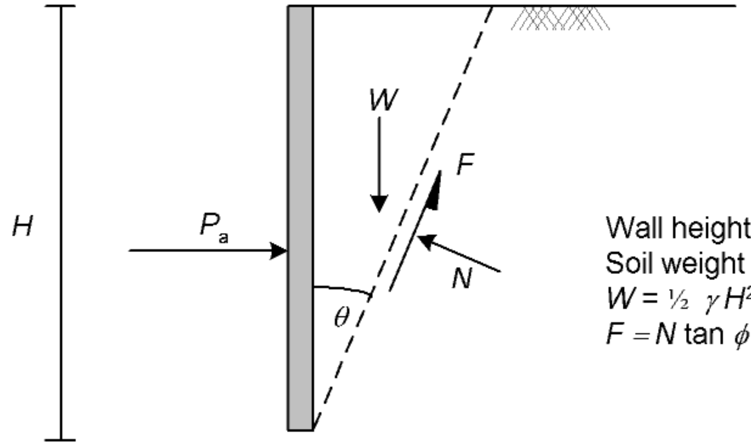


- wedge analysis to derive forces
- pressures by differentiation

$$F_{ap} = \tfrac{1}{2}\ K_{ap}\,(\gamma - \gamma_w)\,z^2 + \tfrac{1}{2}\ \gamma_w\,z^2$$

☒ same result as Rankine when no wall friction.
☒ generally unsafe ("upper") bound
☒ should not be used when wall friction significant (esp. passive).

Coulomb's theory for no wall friction:



Wall height H
Soil weight density γ
$W = \tfrac{1}{2}\ \gamma H^2 \tan\theta$
$F = N \tan\phi'$

Res. hor. $P_a = N\cos\theta - F\sin\theta$

$\qquad = N(\cos\theta - \sin\theta\tan\phi')$

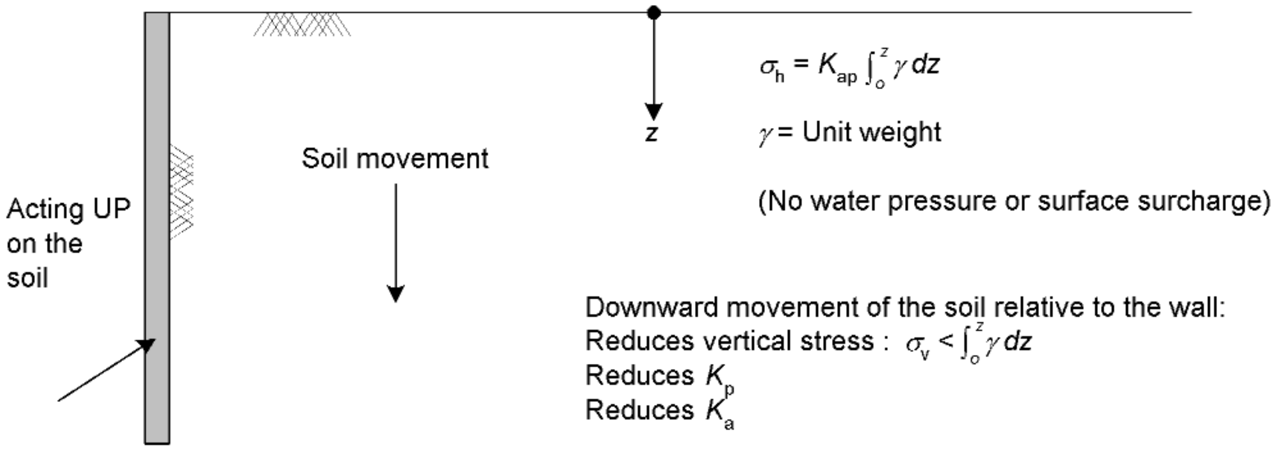
Res. vert. $W = \tfrac{1}{2}\gamma H^2 \tan\theta = N(\sin\theta + \cos\theta\tan\phi')$

Eliminate N and minimise $P_a \Rightarrow$

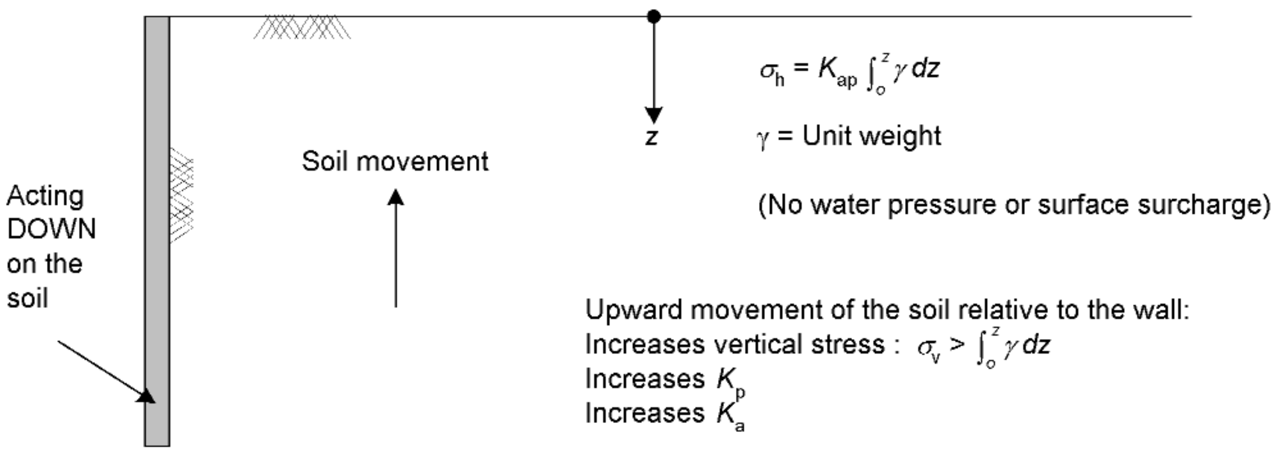
$\theta = 45° - \phi'/2 \qquad P_a = \tfrac{1}{2}\gamma H^2 K_a$ where $K_a = \dfrac{1 - \sin\phi'}{1 + \sin\phi'}$

The value of K_a is the same as that derived by Rankine (Figure 4.2) for a frictionless wall.

FIGURE 4.3 *Coulomb's method to calculate the limiting active force for a frictionless wall/soil interface translating horizontally*



(b) Wall friction: downward movement of the soil relative to the wall



(a) Wall friction: upward movement of the soil relative to the wall

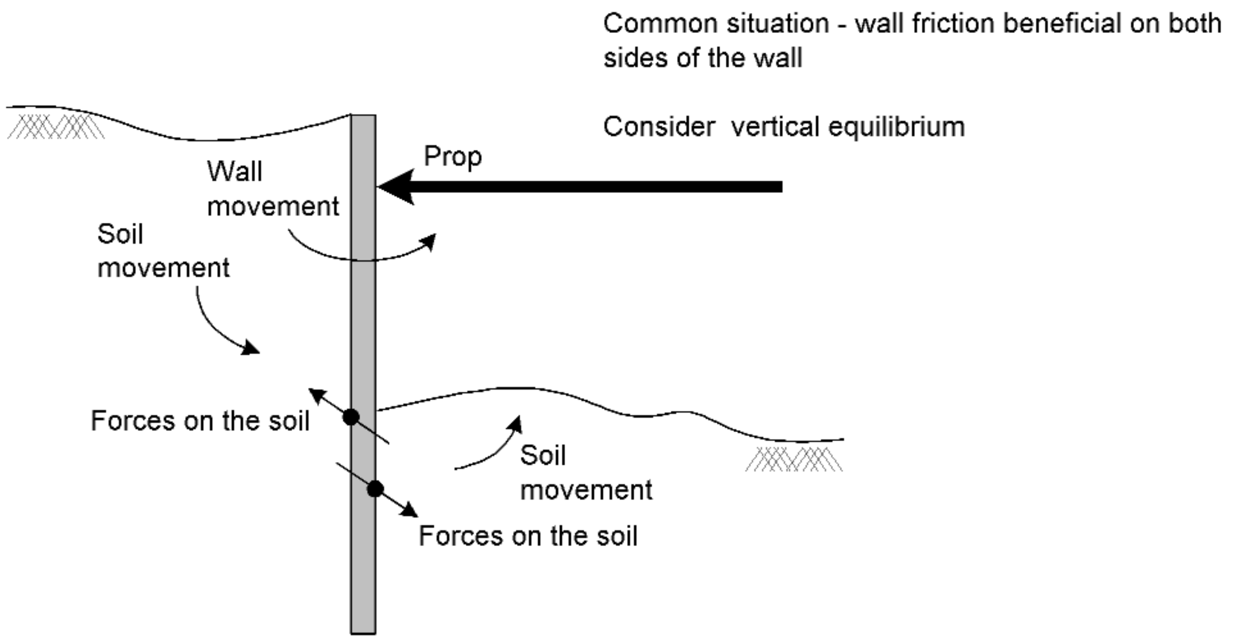


FIGURE 4.4 *Effect of wall friction*

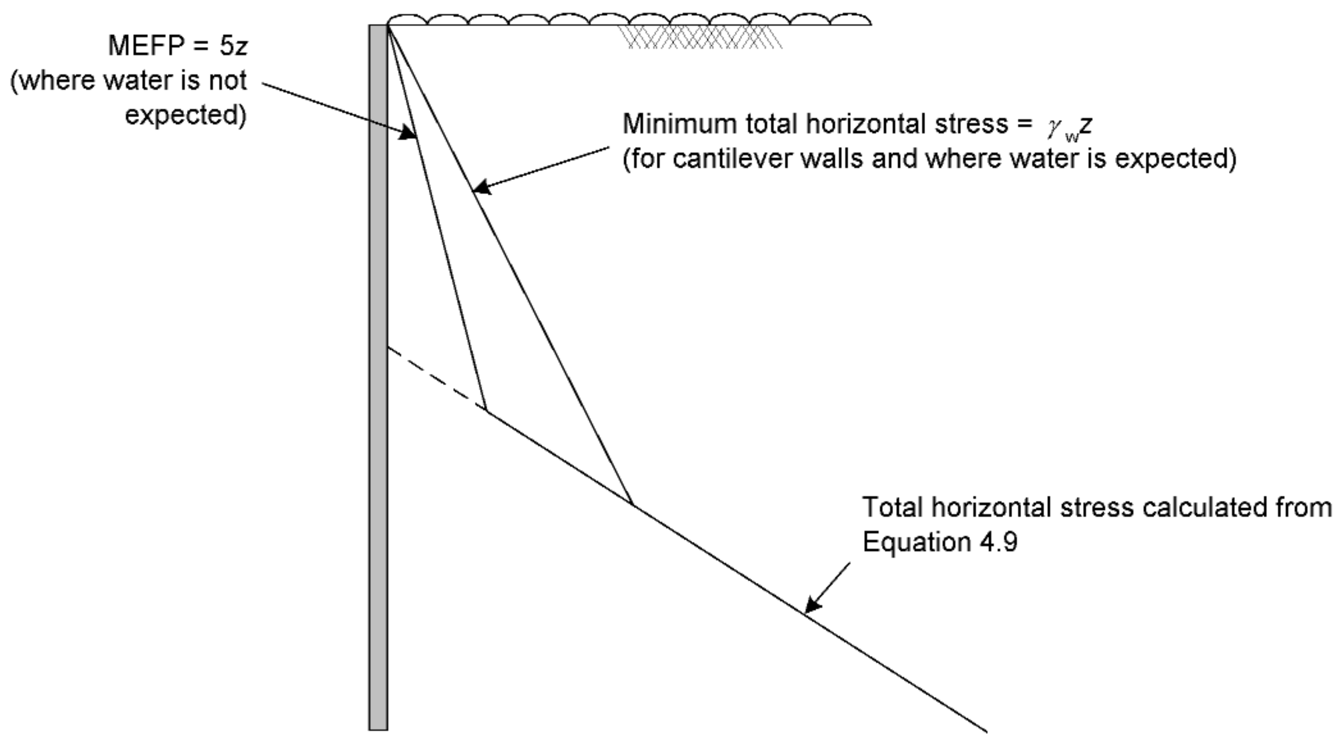
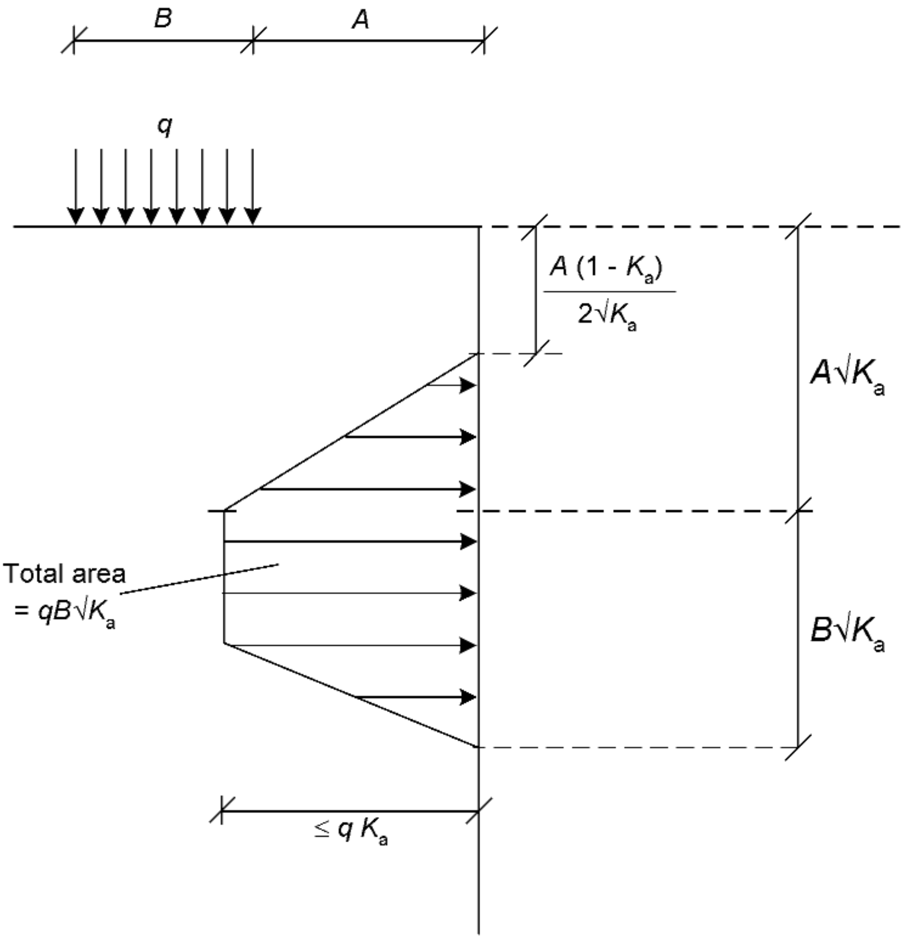
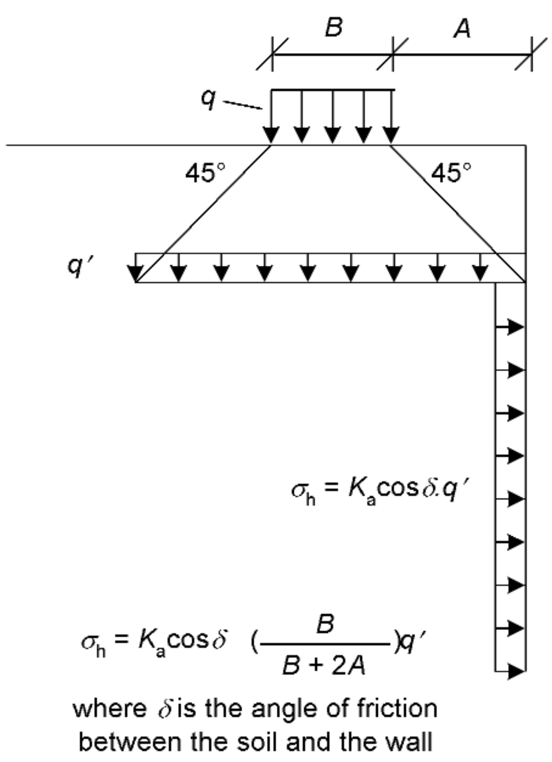


FIGURE 4.5 *Tension cracks : minimum total horizontal stress*



(a) Pappin *et al* (1986)



(b) Georgiadis and Anagnostopoulos (1998)

FIGURE 4.6 *Additional lateral effective stress acting on the back of a wall due to a strip load running parallel to it*

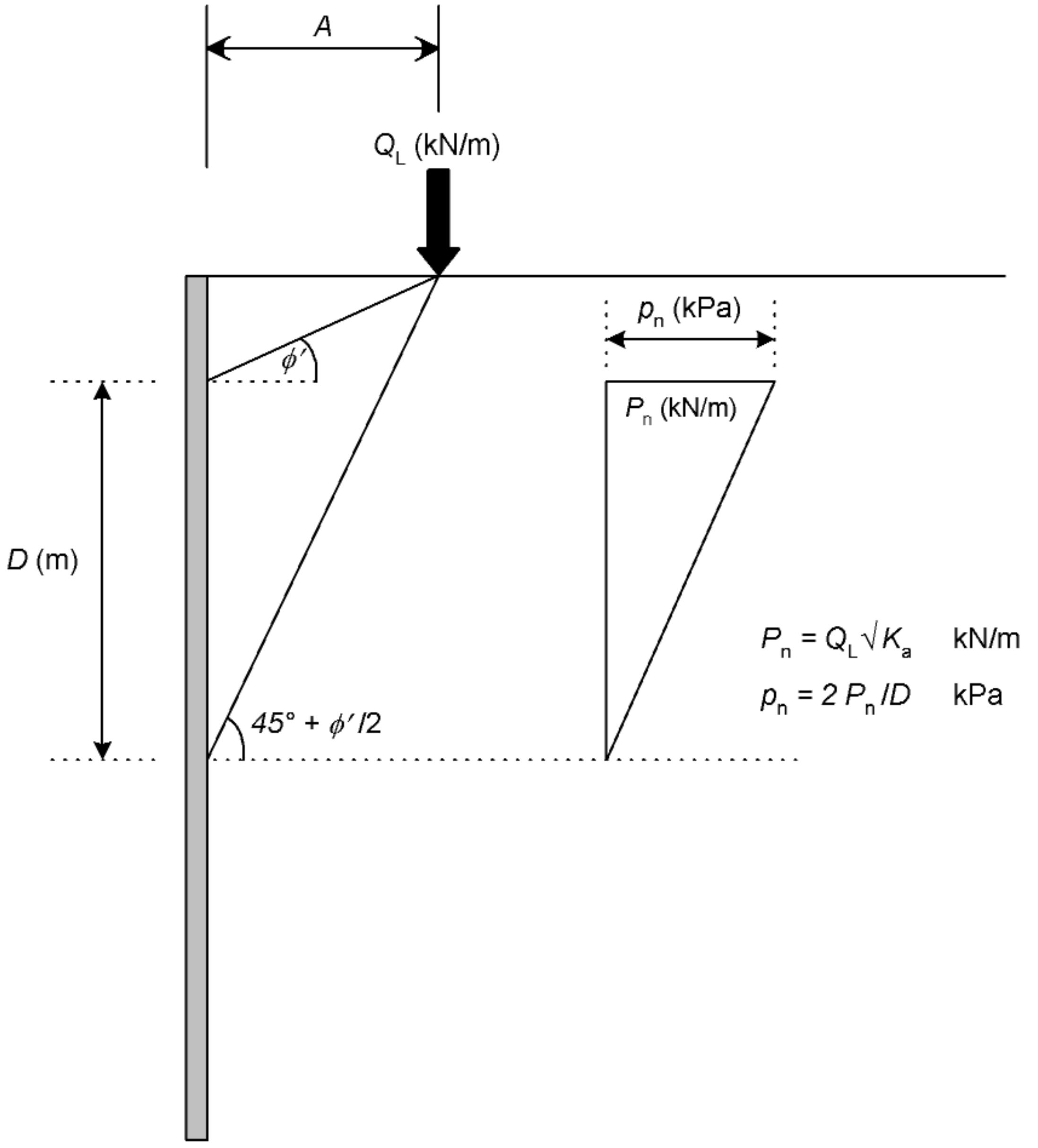


FIGURE 4.7 *Pressure diagram for a line load*

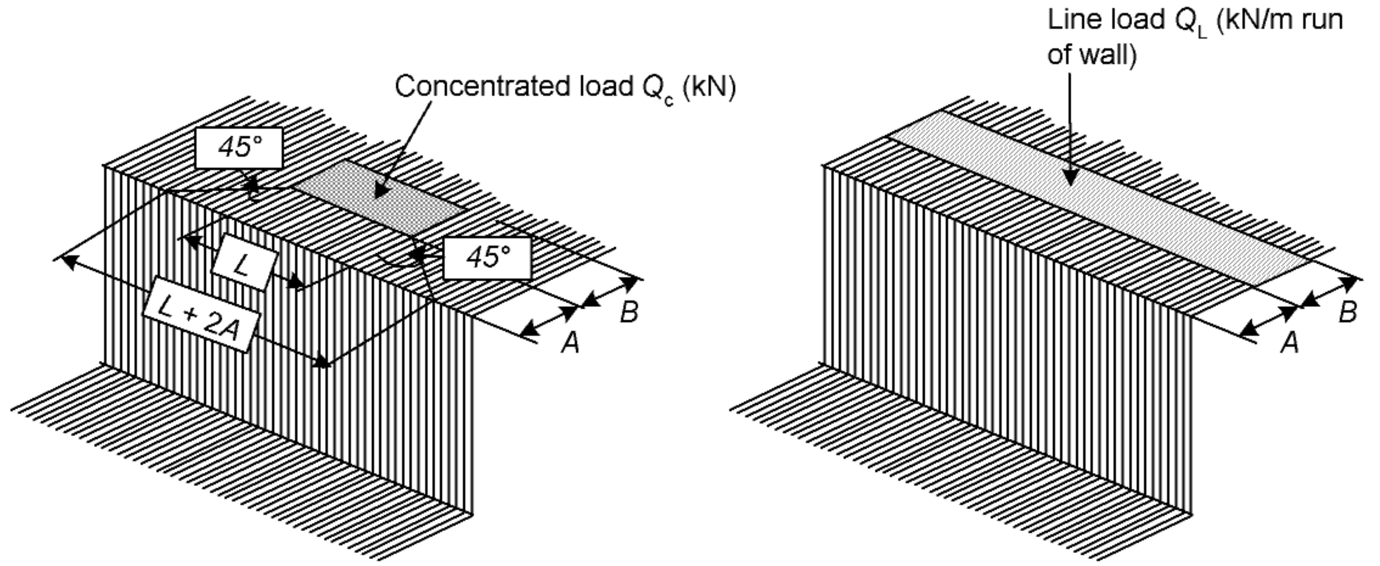
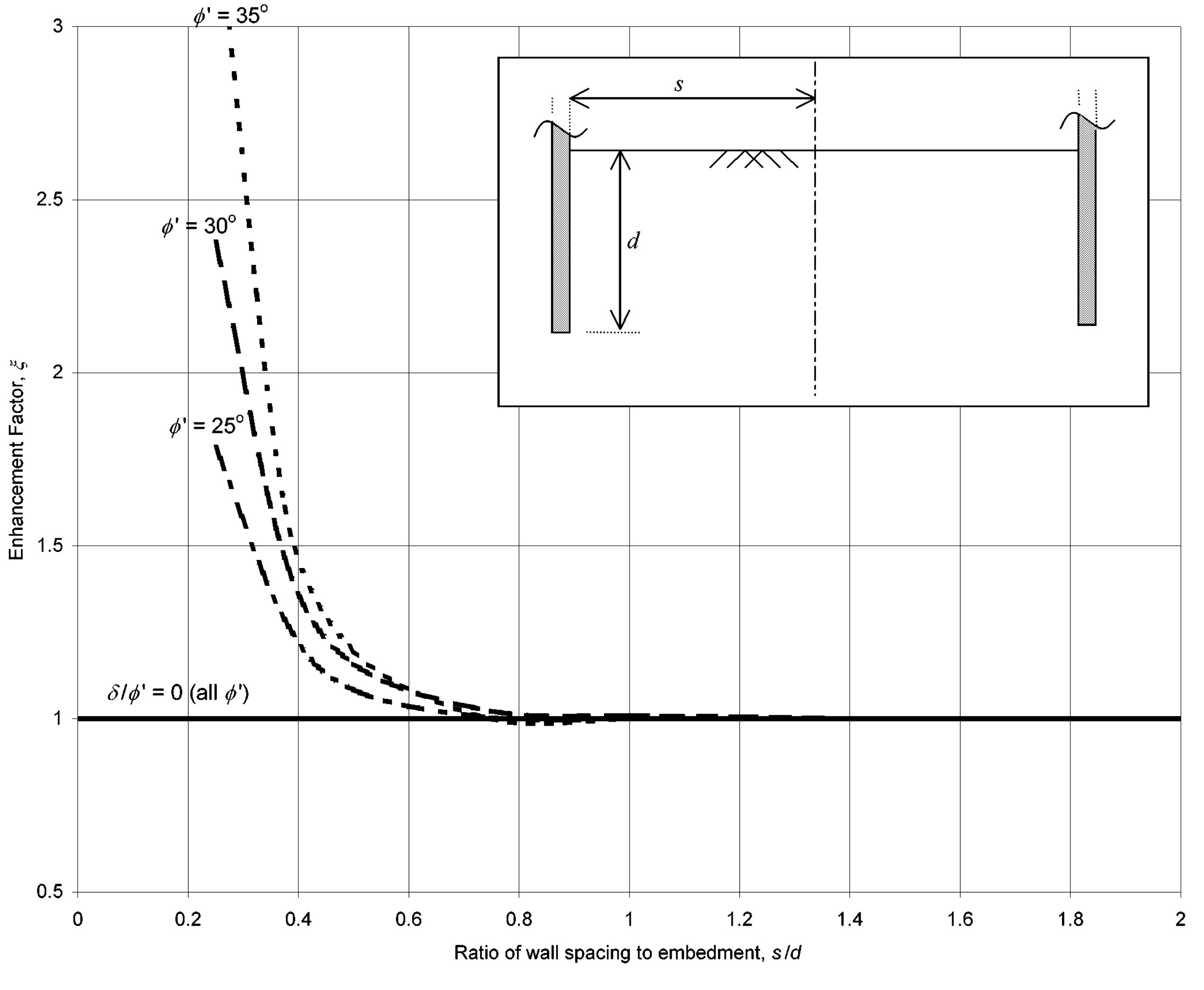


FIGURE 4.8 *Concentrated and line load surcharges (Williams and Waite, 1993)*



Notes:
(1) Homogeneous isotropic soil with no apparent cohesion and no dilation
(2) No groundwater
(3) Wall free to move vertically

FIGURE 4.9 *Enhancement factor on passive earth pressure coefficient for rough walls in close proximity*

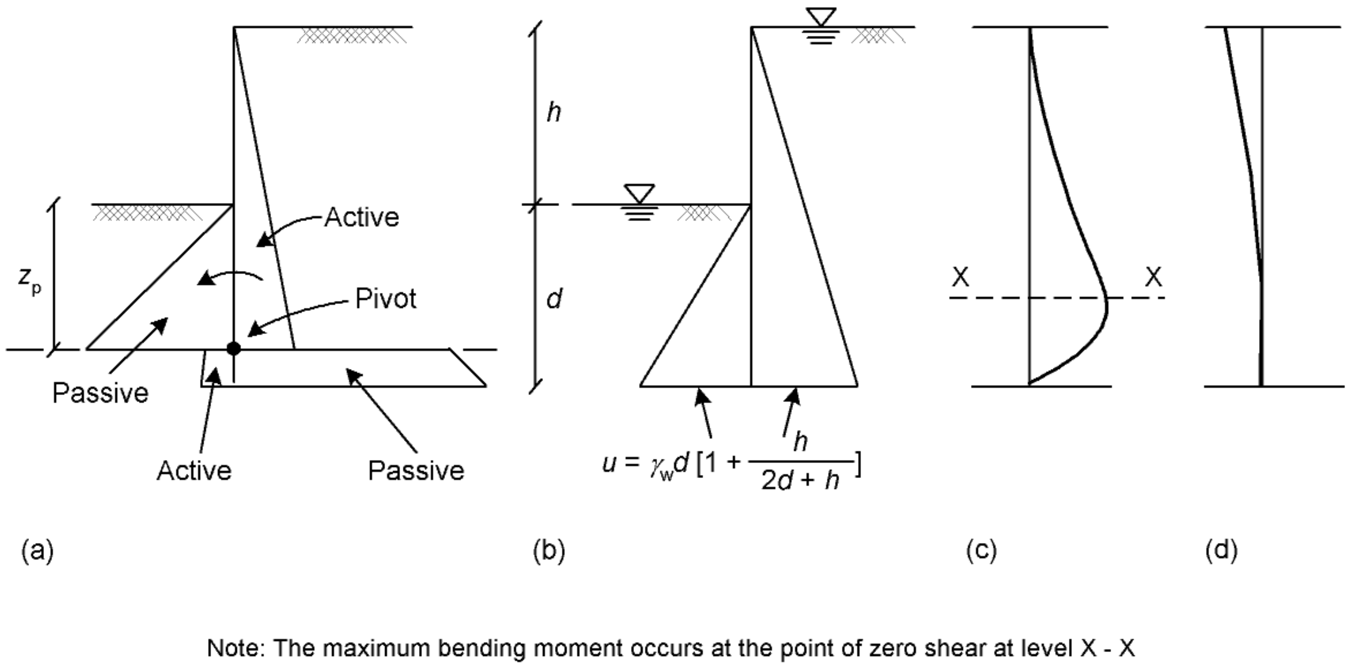


FIGURE 4.10 *Idealised stress distribution for an unpropped embedded cantilever wall at failure; (a) effective stresses; (b) steady state pore water pressures for a wide excavation where differential water head dissipates uniformly; (c) wall bending moment distribution; (d) wall deflection.*

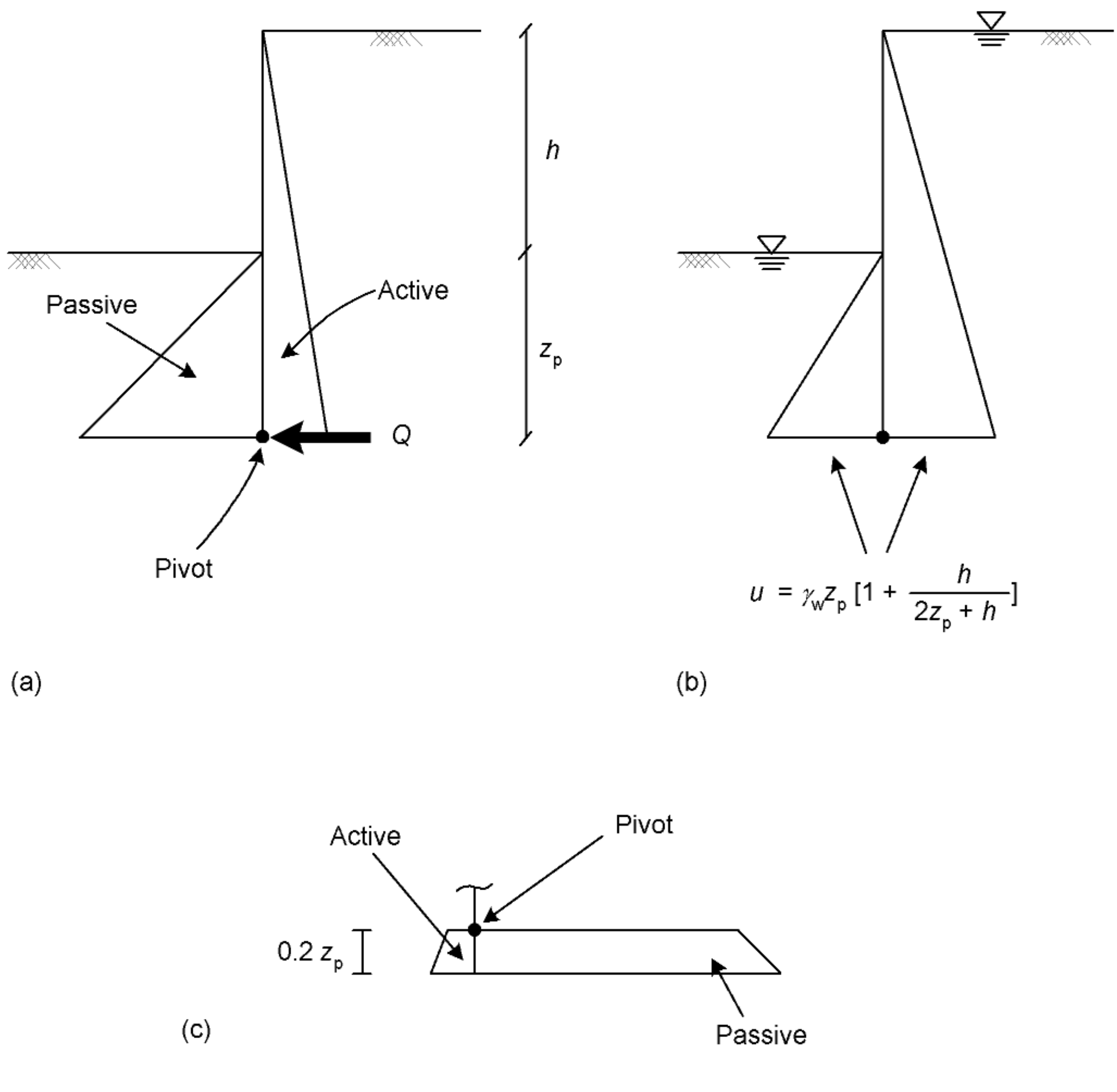
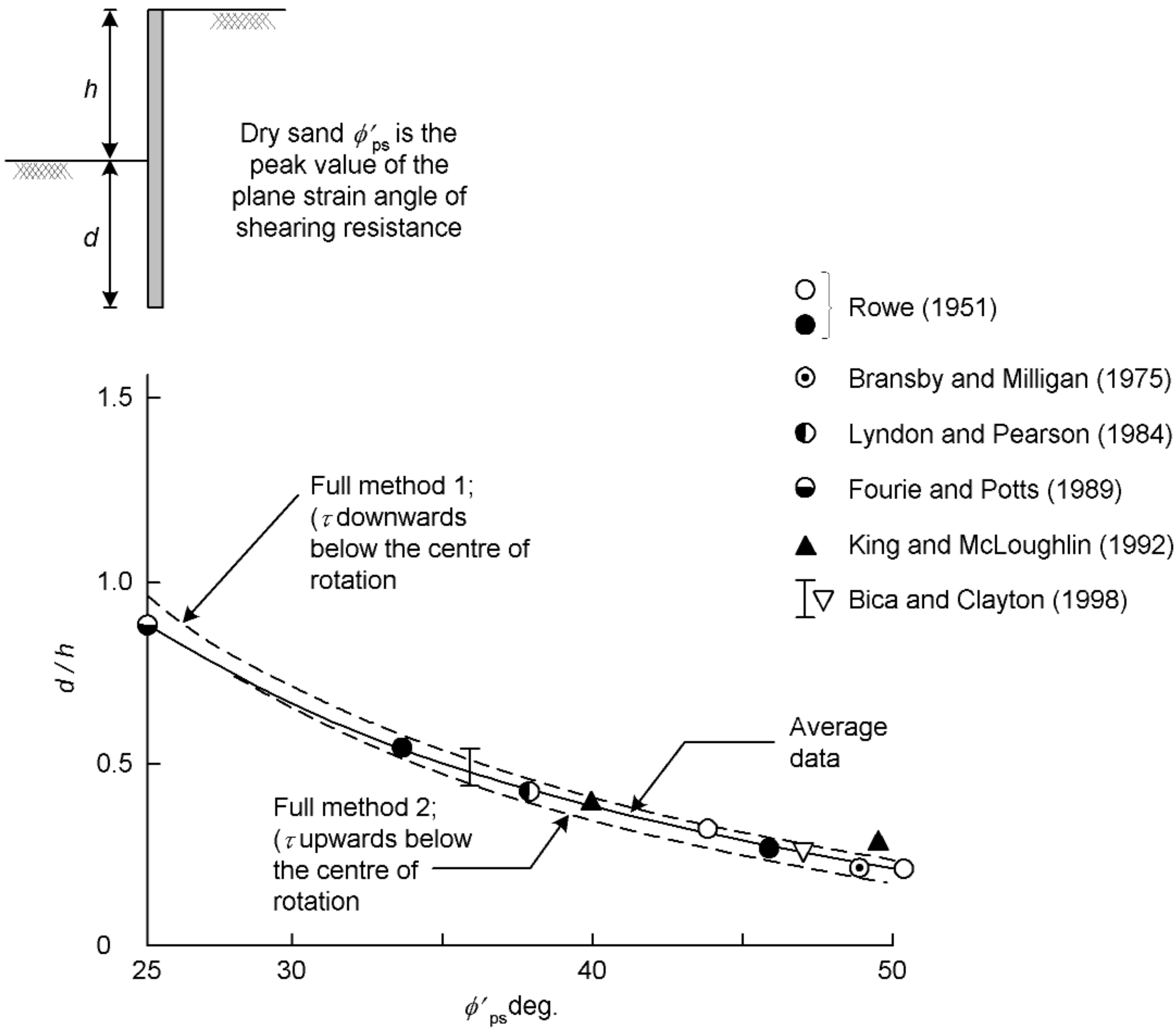


FIGURE 4.11 *Approximate stress analysis for unpropped walls; (a) effective stresses; (b) steady state pore water pressures for a wide excavation where the differential water head dissipates uniformly; (c) check that the added depth can mobilise at least the required force Q*



Note : Full method 1 assumes $\delta = 2\phi'/3$ with K_{ah} and K_{ph} determined using Caquot & Kerisel (1948). Full method 2 assumes wall friction downwards below the centre of rotation of the wall with K_{ph} at this location determined using Coulomb's method.

FIGURE 4.12 *Normalised depth of embedment at failure (after Bica and Clayton, 1998)*

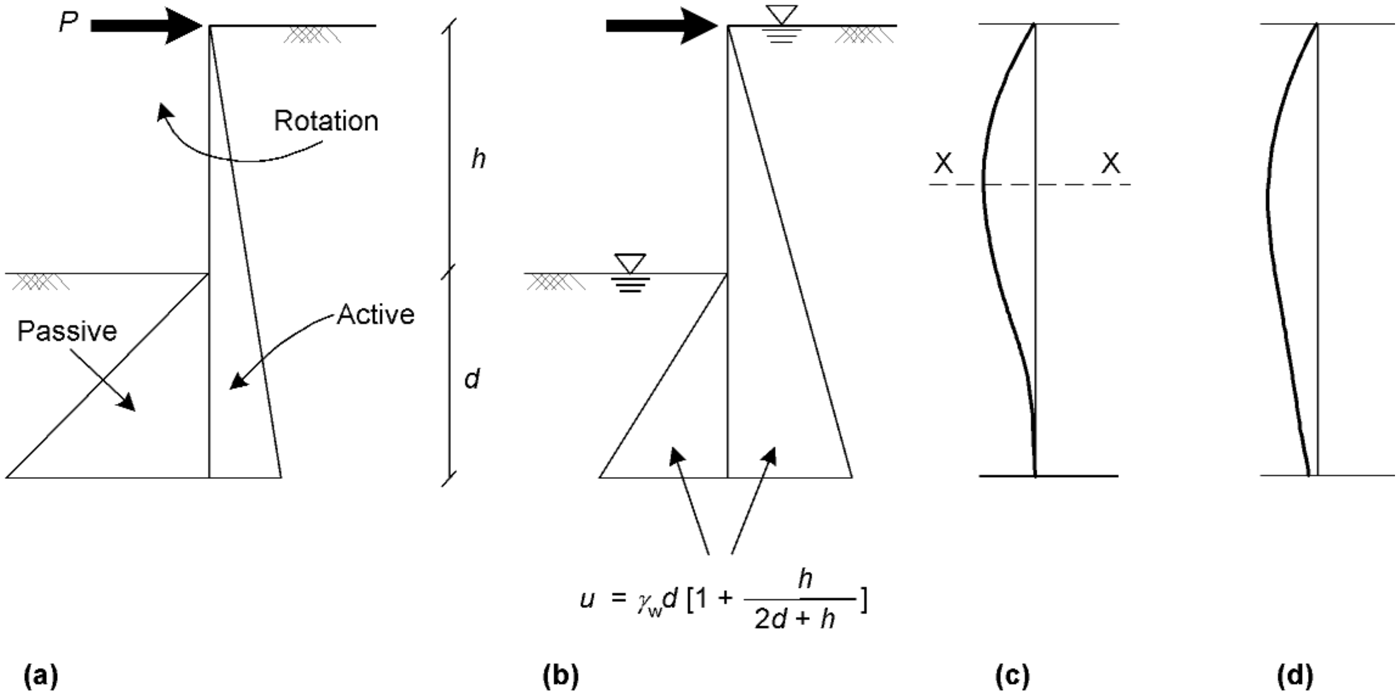
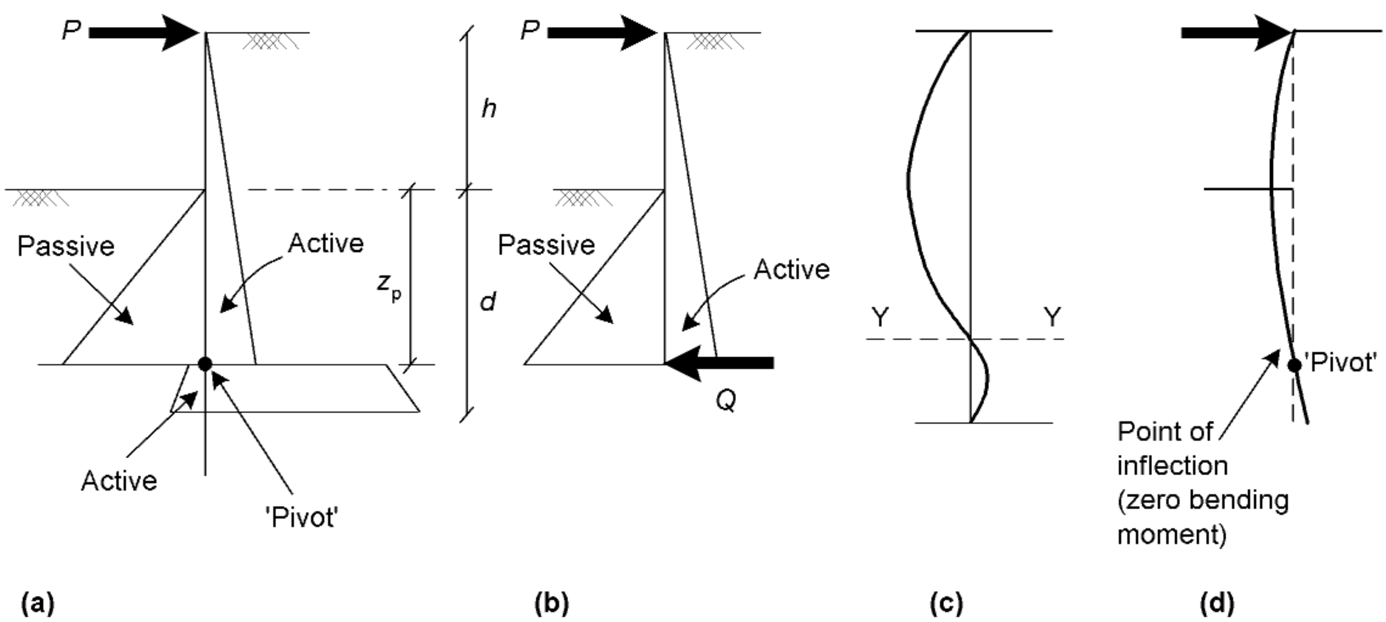


FIGURE 4.13 *Idealised stress distribution at failure for a stiff wall propped rigidly at the top; (a) effective stresses; (b) steady state pore water pressures for a wide excavation where the differential water head dissipates uniformly; (c) wall bending moment distribution; (d) wall deflection*



Note: The point of zero bending moment (at level Y - Y) is assumed to occur where the active and passive pressures balance ie the net pressure is zero (Williams and Waite, 1993).

FIGURE 4.14 *Fixed earth support effective stress distributions ans deformations for an embeddded wall propped at the top; (a) idealised stresses; (b) simplified stresses; (c) wall bending moment distribution; (d) wall deflection*

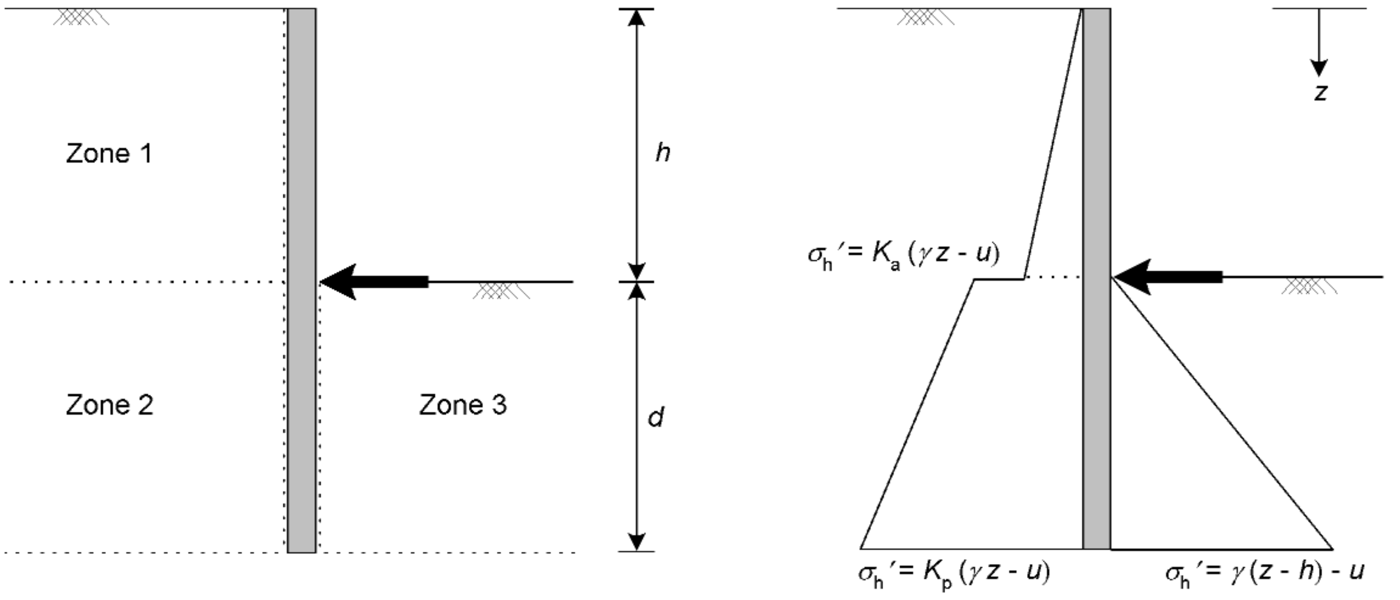


FIGURE 4.15 *Stress analysis for an embedded wall propped at formation level; (a) division of soil zones; (b) idealised effective stress distributions (from Powrie and Li, 1991).*

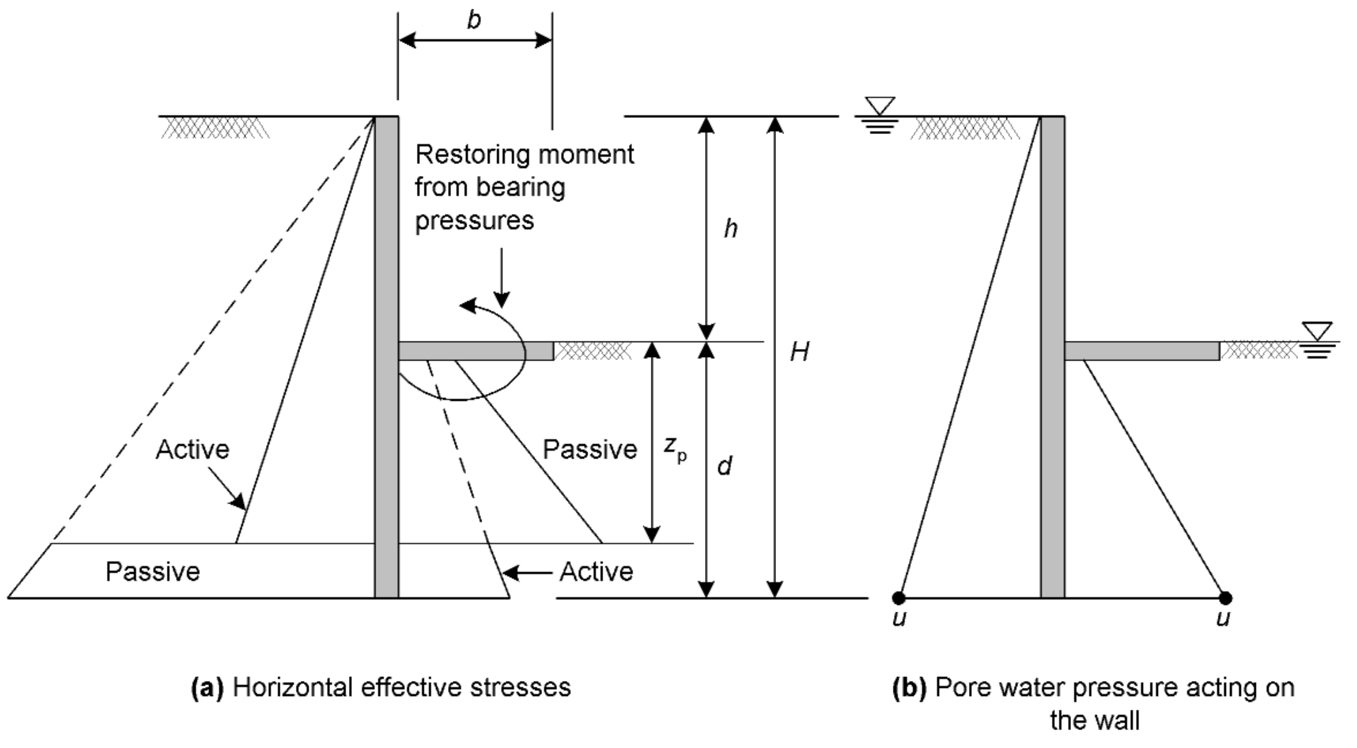
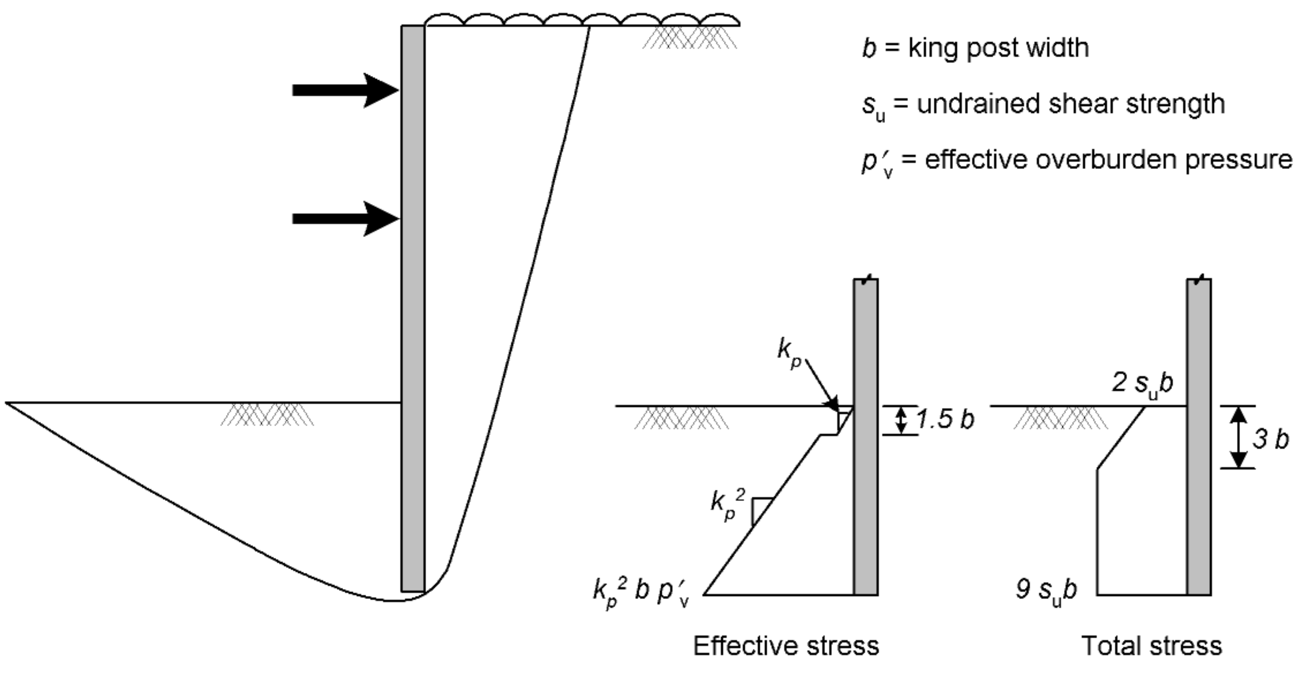


FIGURE 4.16 *Forces acting on a stabilising base retaining wall*



(a) Overall stability **(b)** Lateral loading of individual king posts

FIGURE 4.17 *King post wall design*

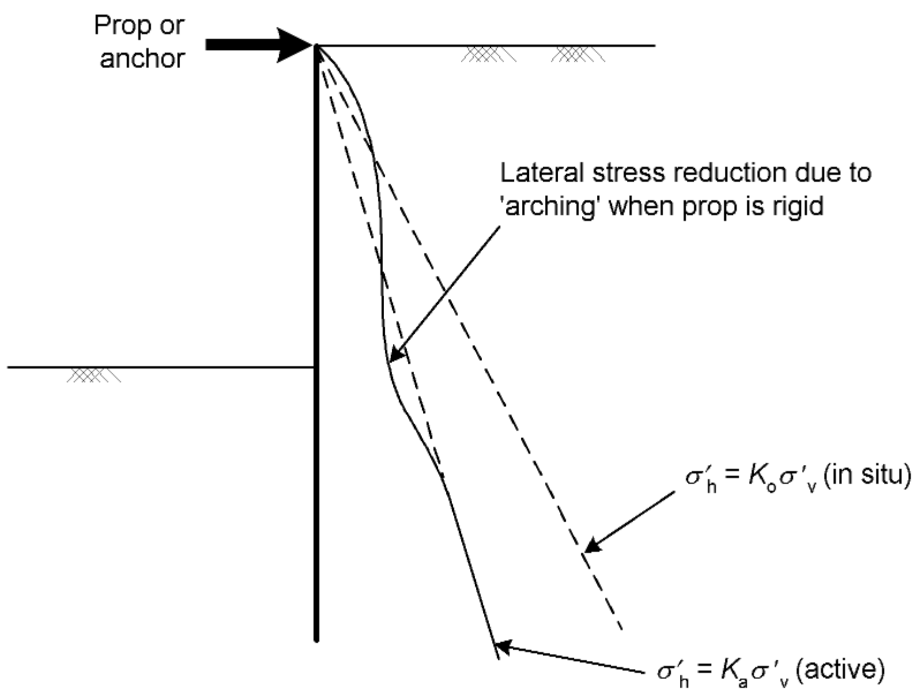


FIGURE 4.18 *Reduction of lateral stress in the retained soil due to arching into rigid prop*

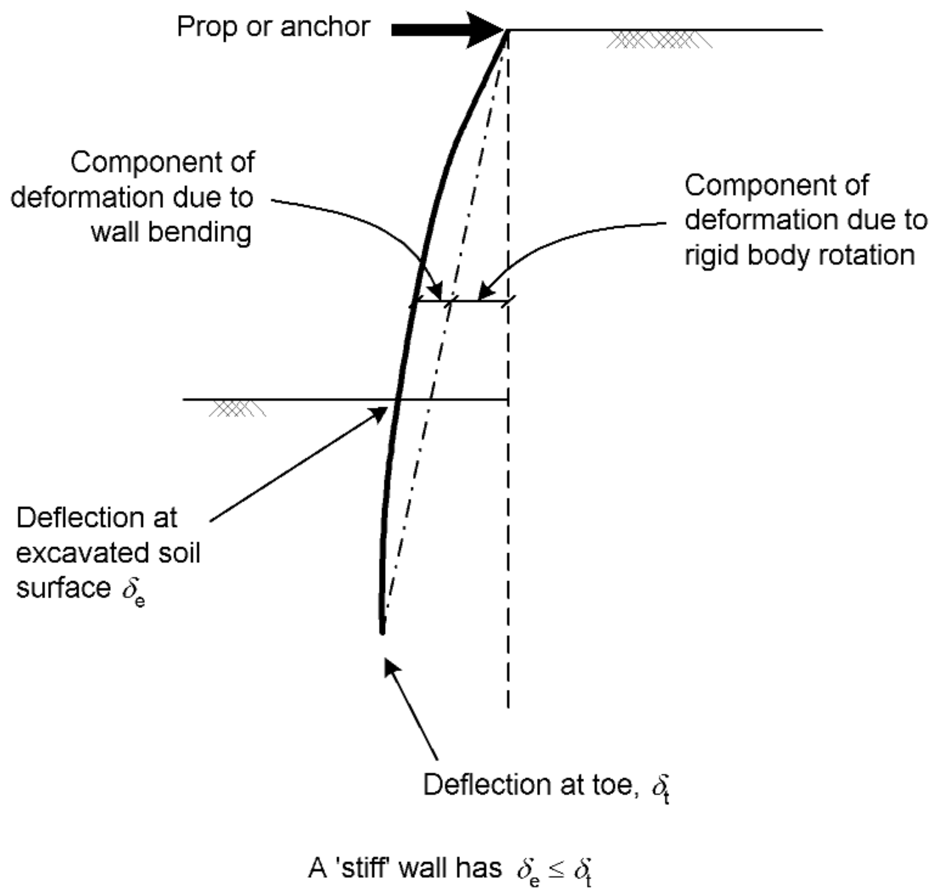


FIGURE 4.19 *Components of wall displacement and definition of a "stiff" wall*

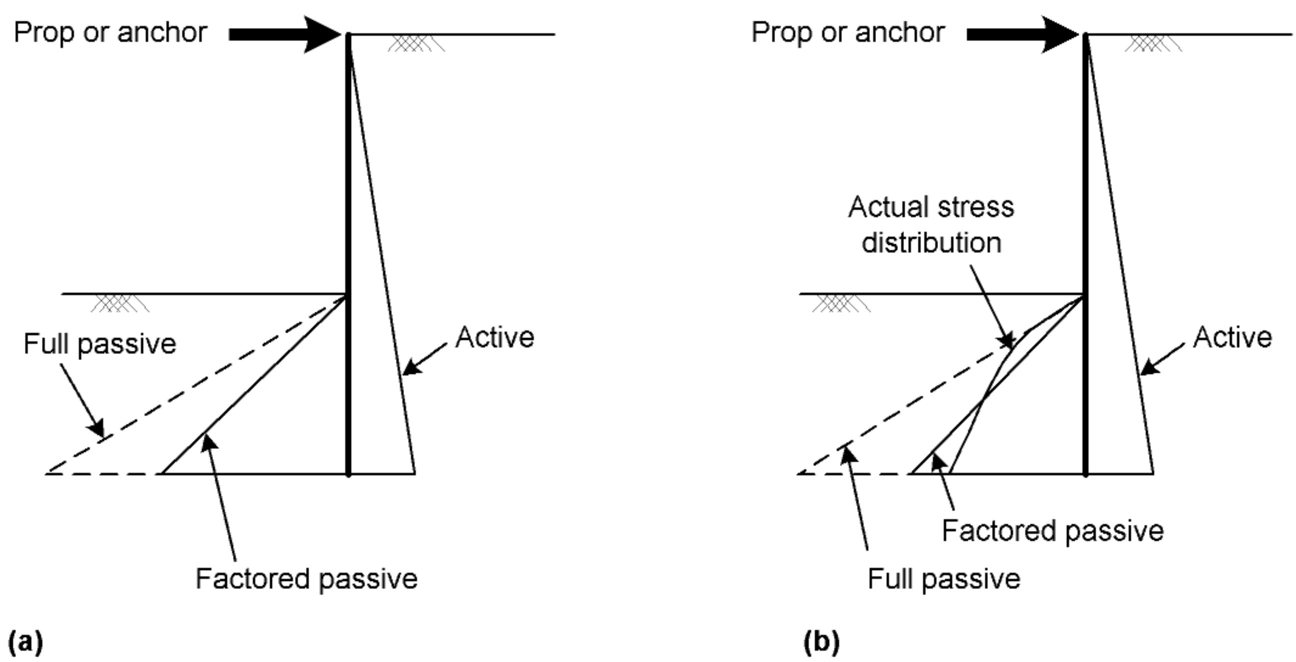
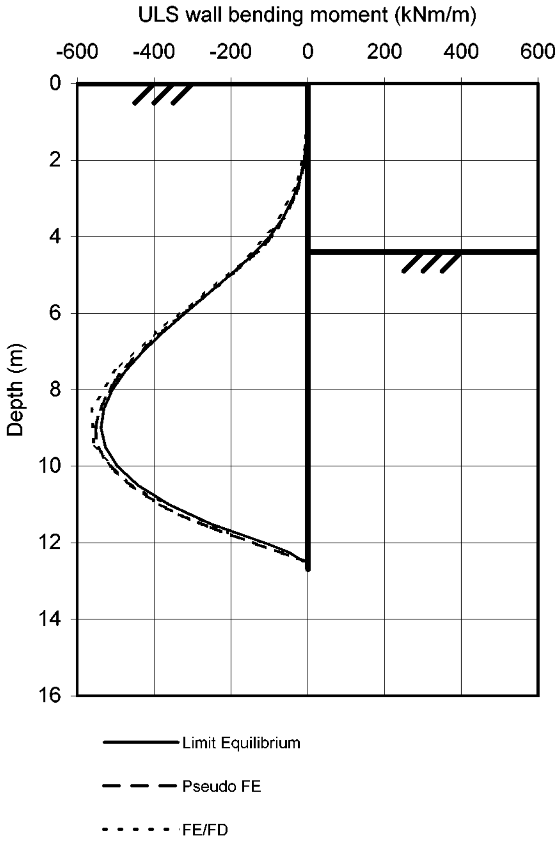
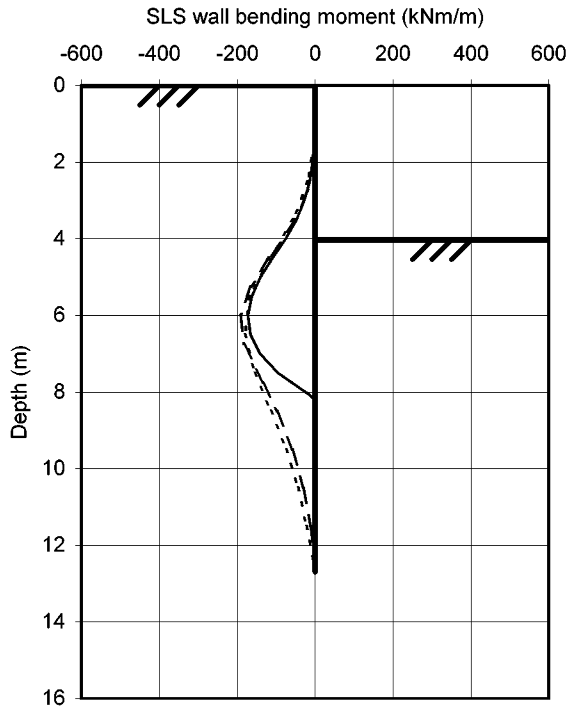


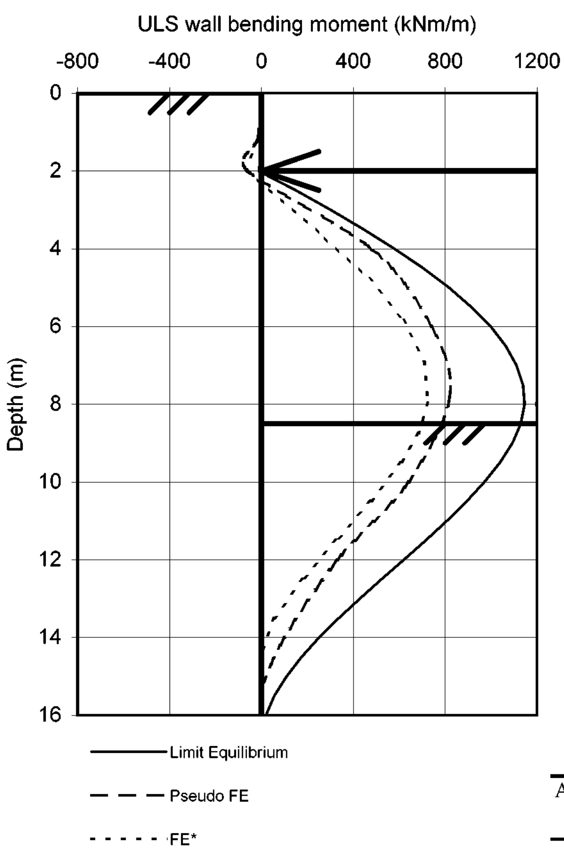
FIGURE 4.20 *Stress distributions behind and in front of (a) stiff and (b) flexible embedded walls (after Rowe, 1952)*

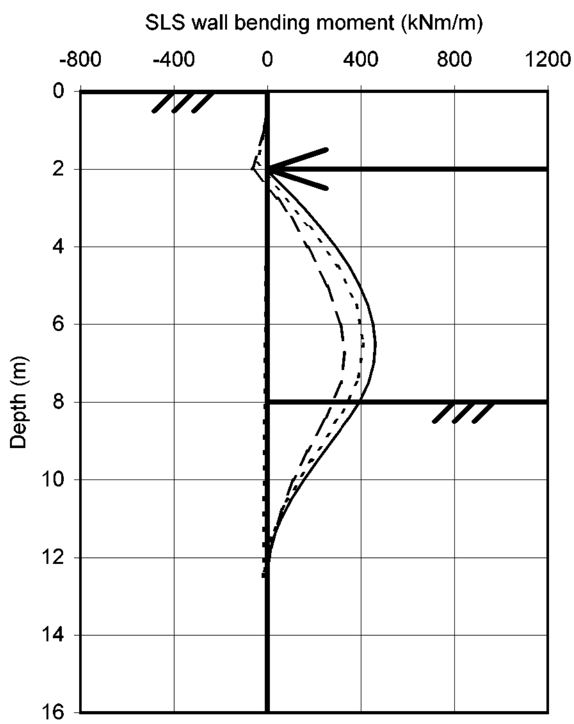




Analysis method	Average wall depth (m)
LE	13.3
Pseudo FE	12.8
FE/FD	12.5

Example 1: Cantilever Wall - effective stress analysis



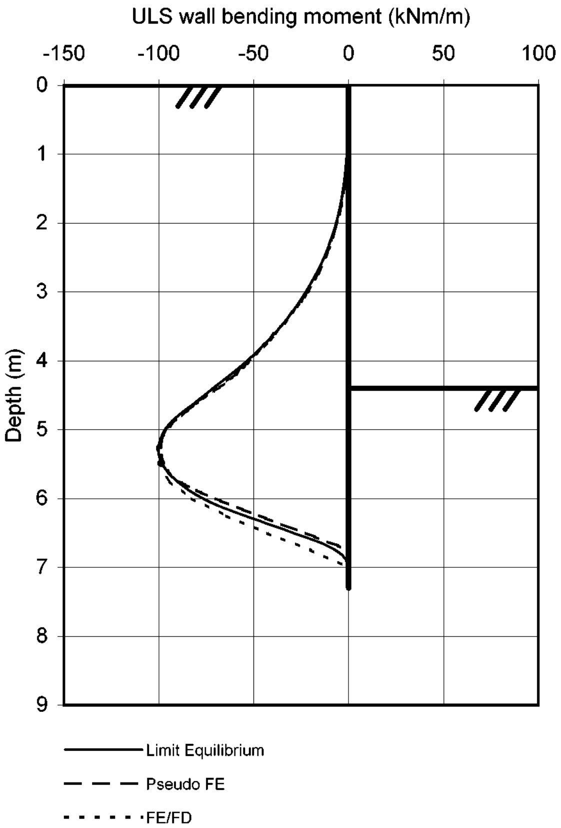


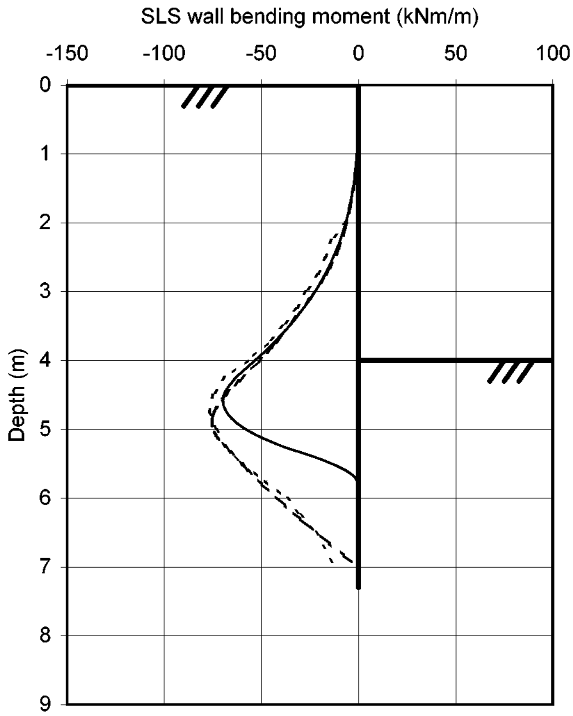
Analysis method	Average wall depth (m)	Average max ULS prop load (kNm/m)	Average max SLS prop load (kNm/m)
LE	16.7	339	182
Pseudo FE	15.3	444	242
FE *	14.0	380	220

* SAFE results with linear seepage (see Appendix J)

Example 2: Propped Wall - effective stress analysis

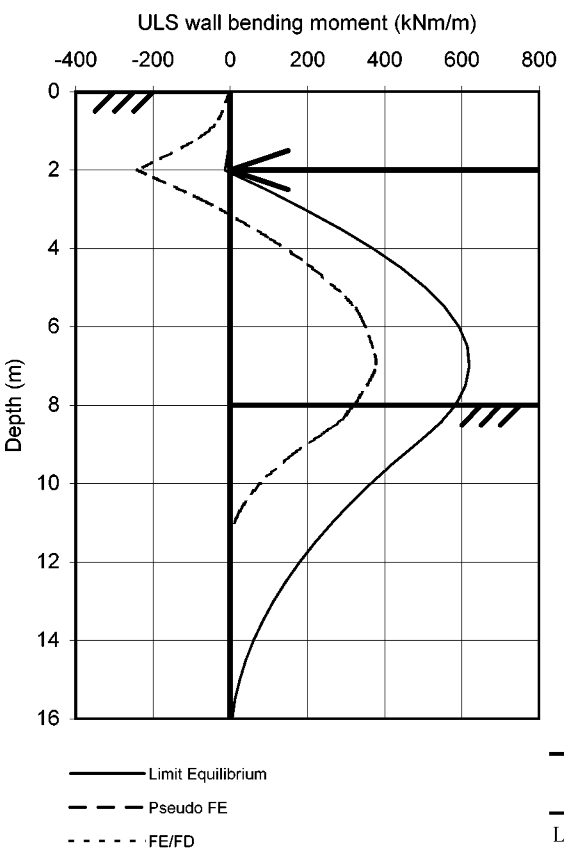
FIGURE 4.21 *Comparison of types of analyses - effective stress*

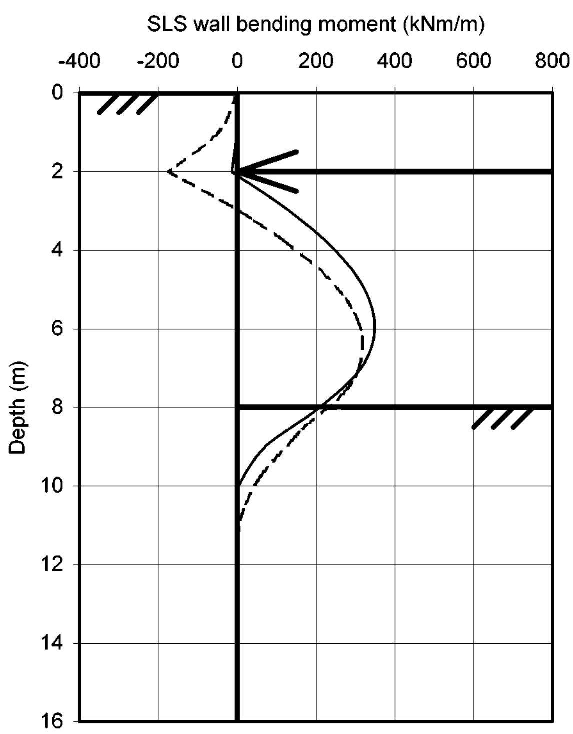




Analysis method	Average wall depth (m)
LE	7.2
Pseudo FE	7.1
FE/FD	7

Example 3: Cantilever Wall - total stress analysis





Example 4: Propped Wall - total stress analysis

Analysis Method	Average wall depth (m)	Average max ULS prop load (kNm/m)	Average max SLS prop load (kNm/m)
LE	16.9	241	165
Pseudo FE	10.8	466	365
FE/FD(1)	-	-	-

(1) SAFE and FLAC results not available (see Appendix J)

FIGURE 4.22 *Comparison of types of analyses - total stress*

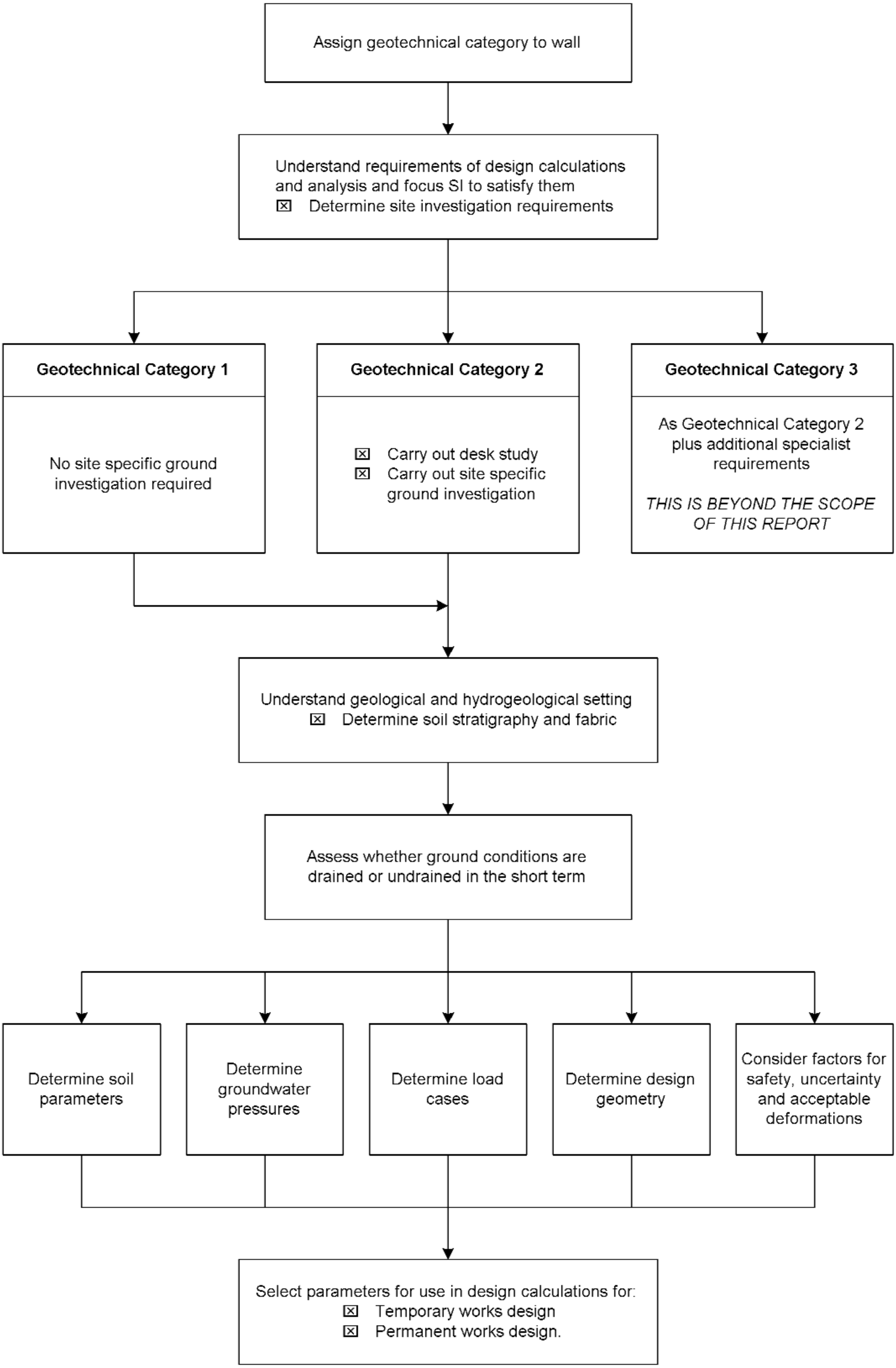


FIGURE 5.1 *Determination and selection of parameters for use in design calculations*

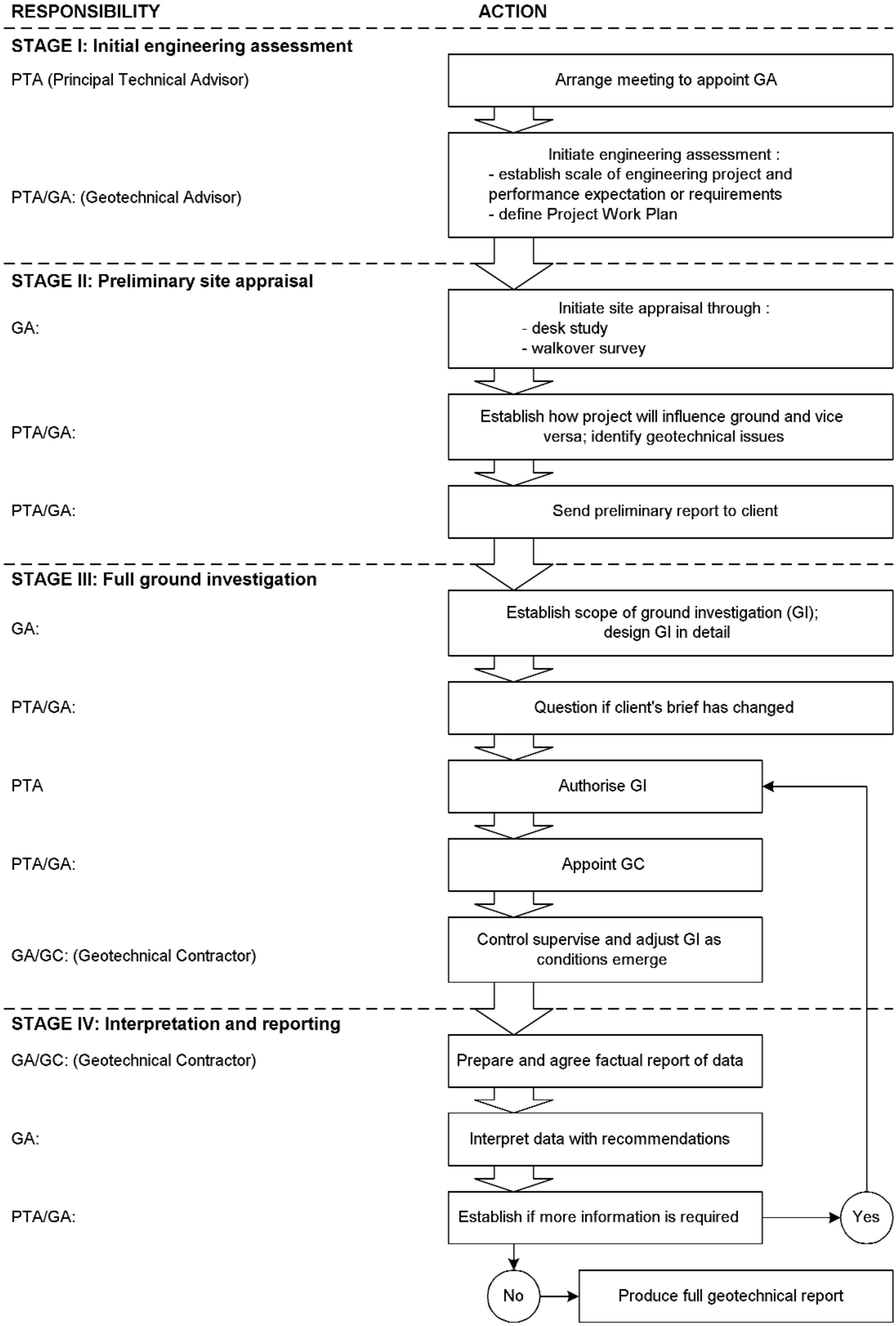


FIGURE 5.2 *Decision making process in site investigation (after SISG, 1993)*

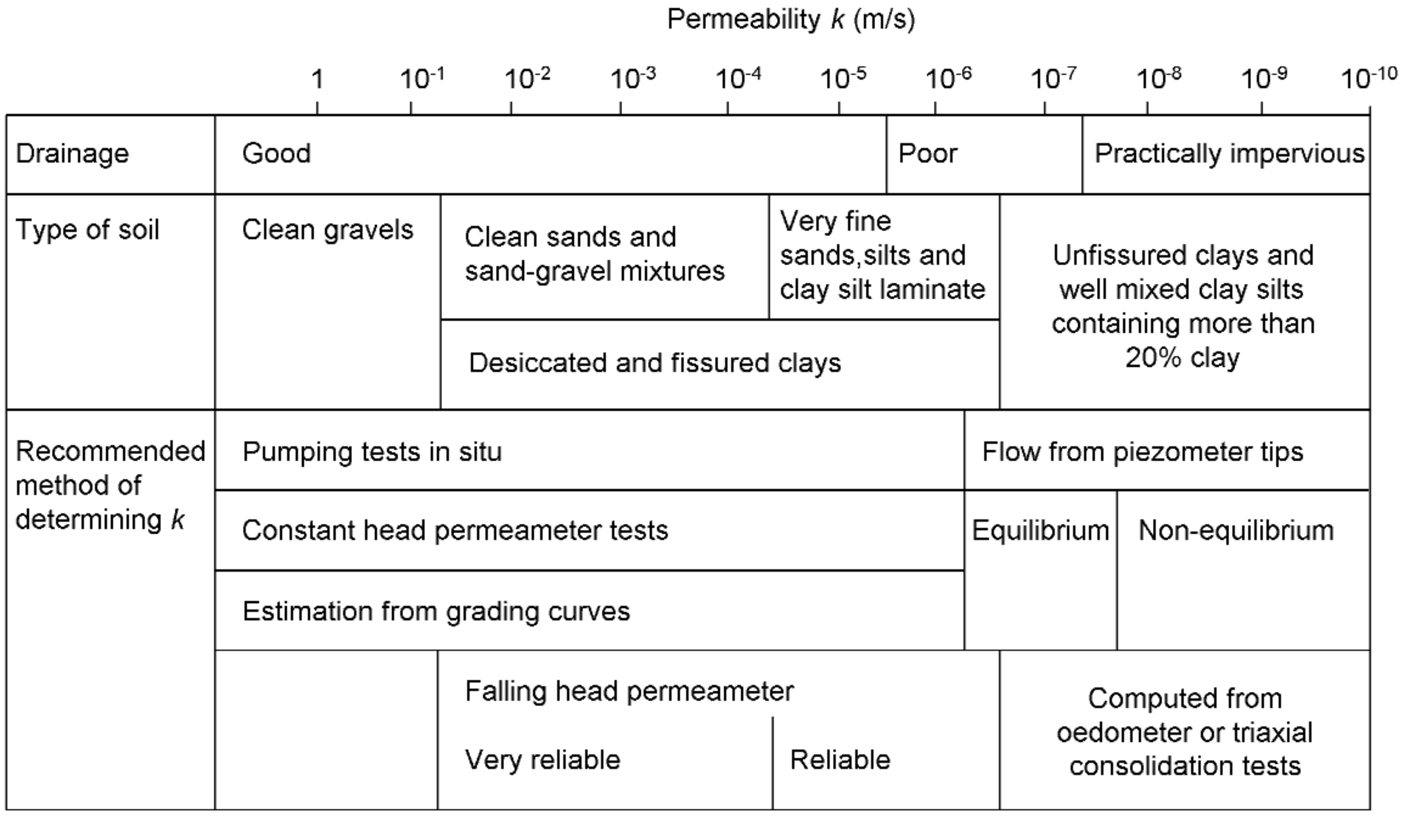


FIGURE 5.3 *Permeability and drainage characteristics of soils (after BS 8004, 1986)*

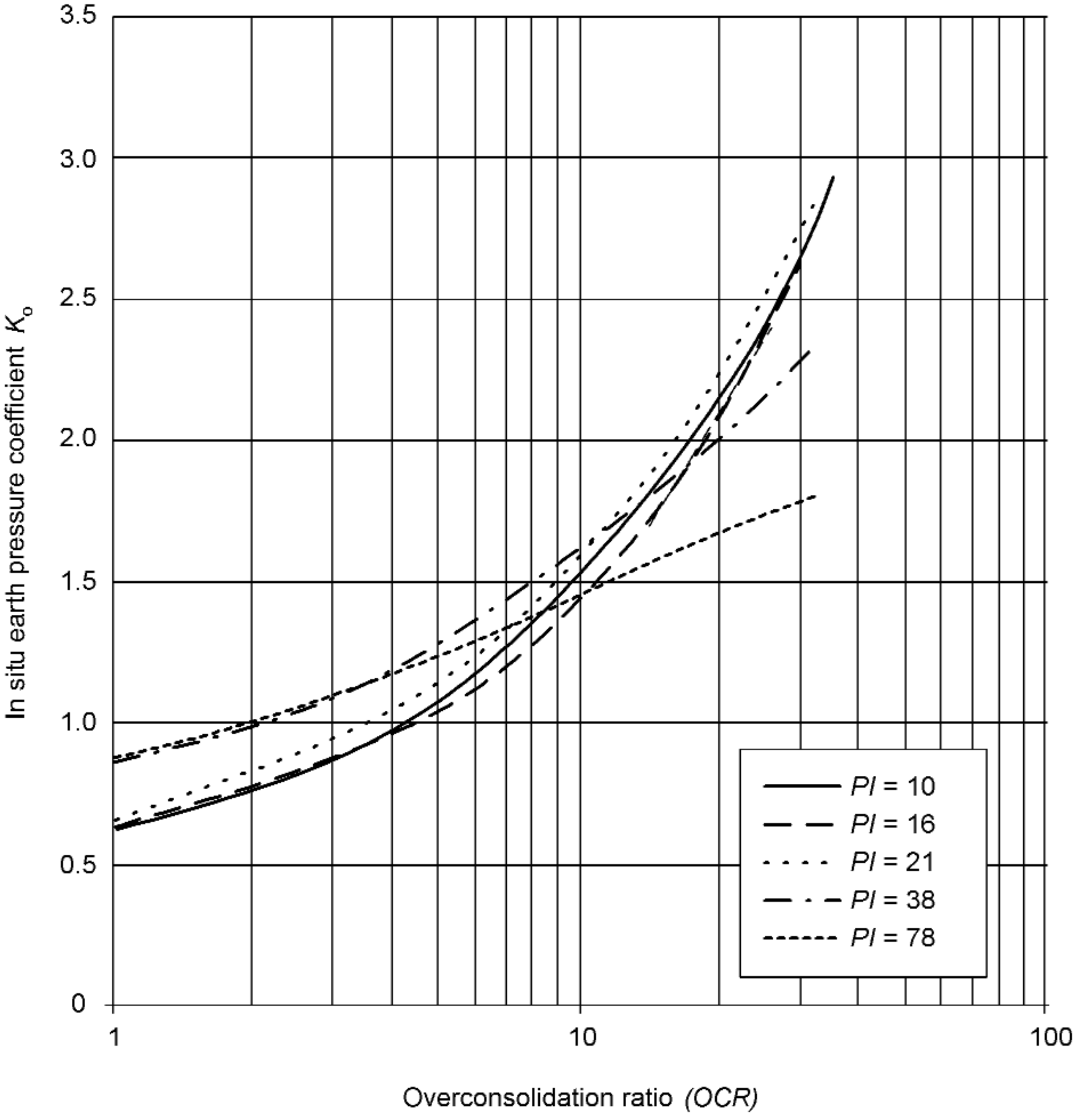


FIGURE 5.4 *Correlation between the in situ coefficient of earth pressure and overconsolidated ratio for clay of various plasticity indices (after Brooker and Ireland, 1965)*

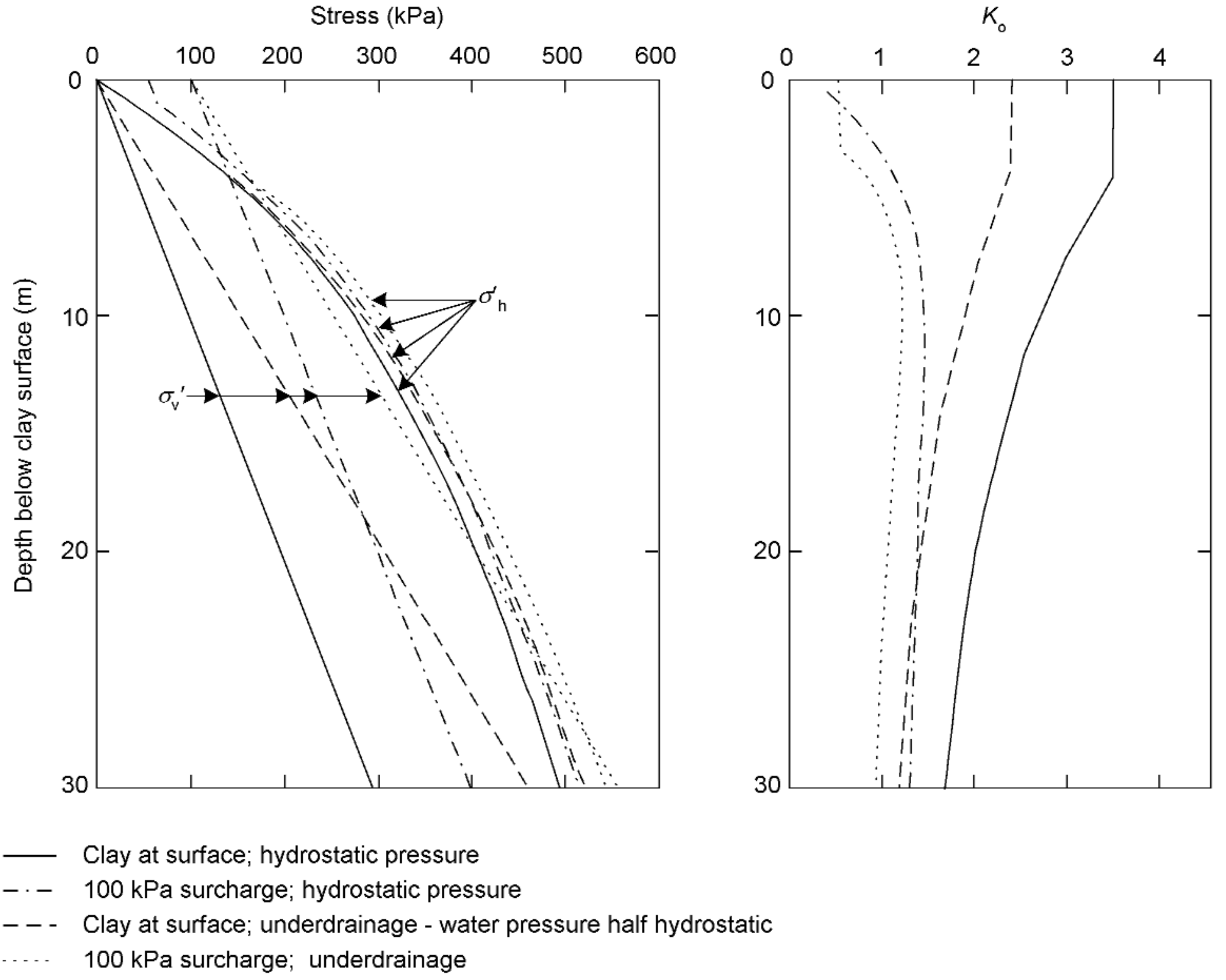


FIGURE 5.5 *Influence of stress history on K_o and σ_h' in a heavily over consolidated clay (after Burland et al., 1979)*

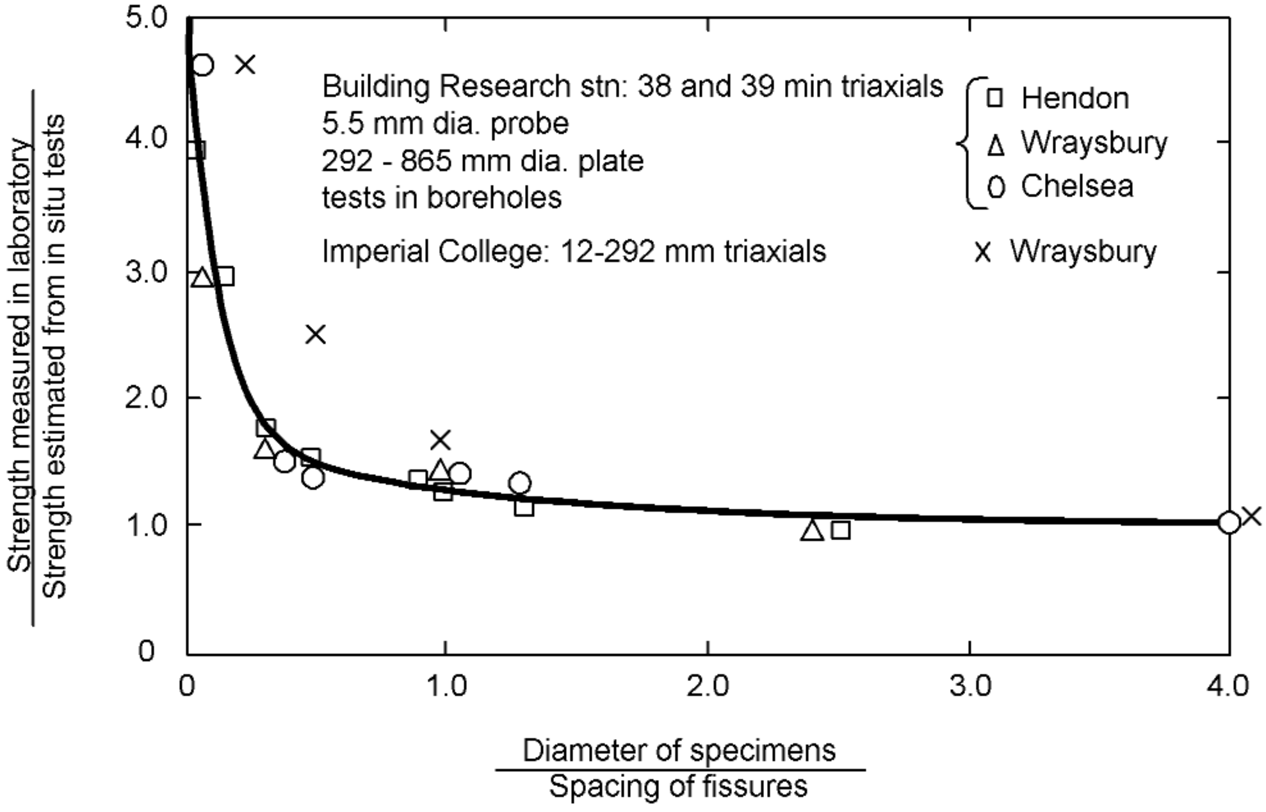


FIGURE 5.6 *Influence of the ratio of sample size to the fissure spacing on the strength measured in laboratory tests (after Marsland, 1971)*

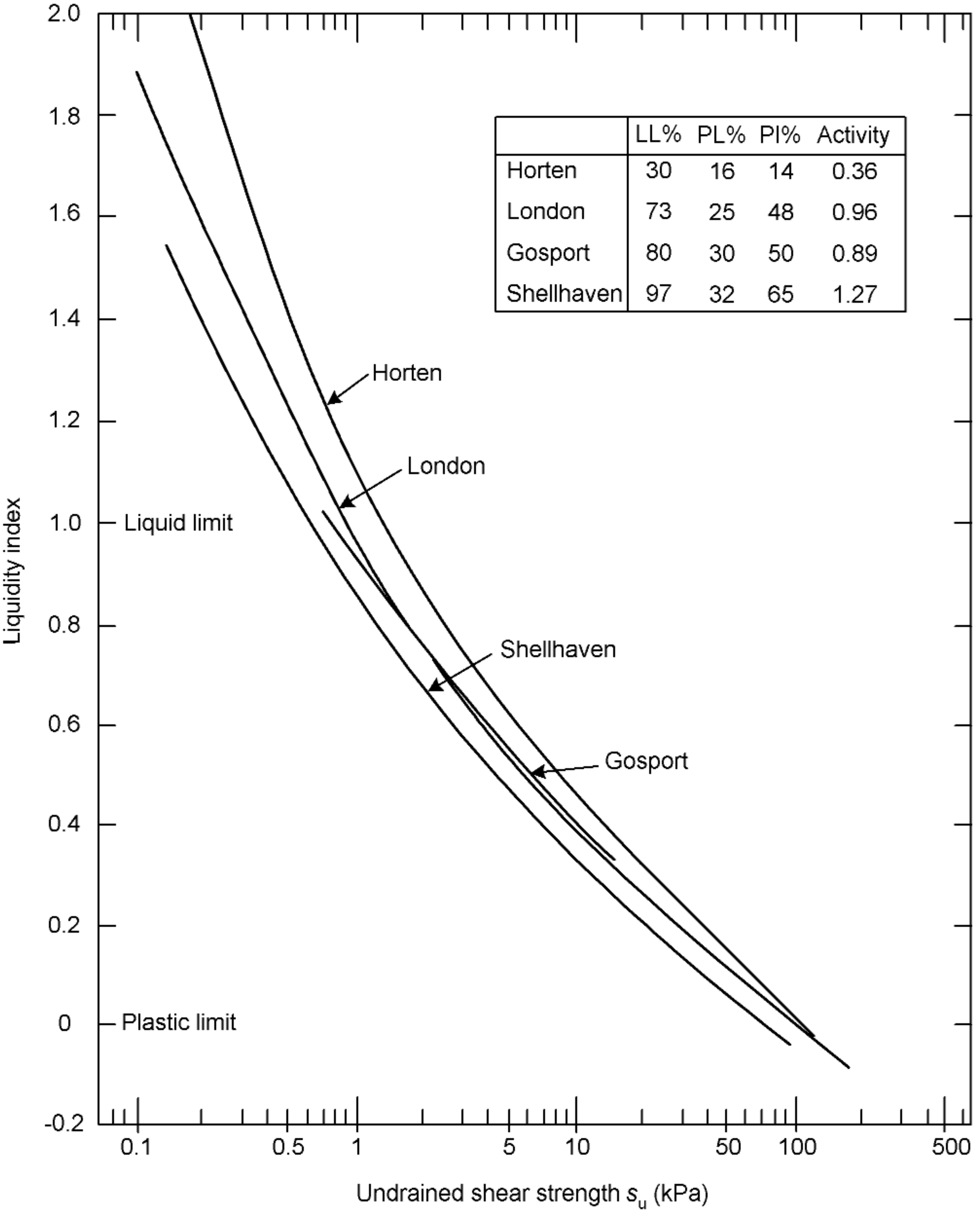


FIGURE 5.7 *Correlation between undrained shear strength and liquidity index (after Skempton and Northey, 1952)*

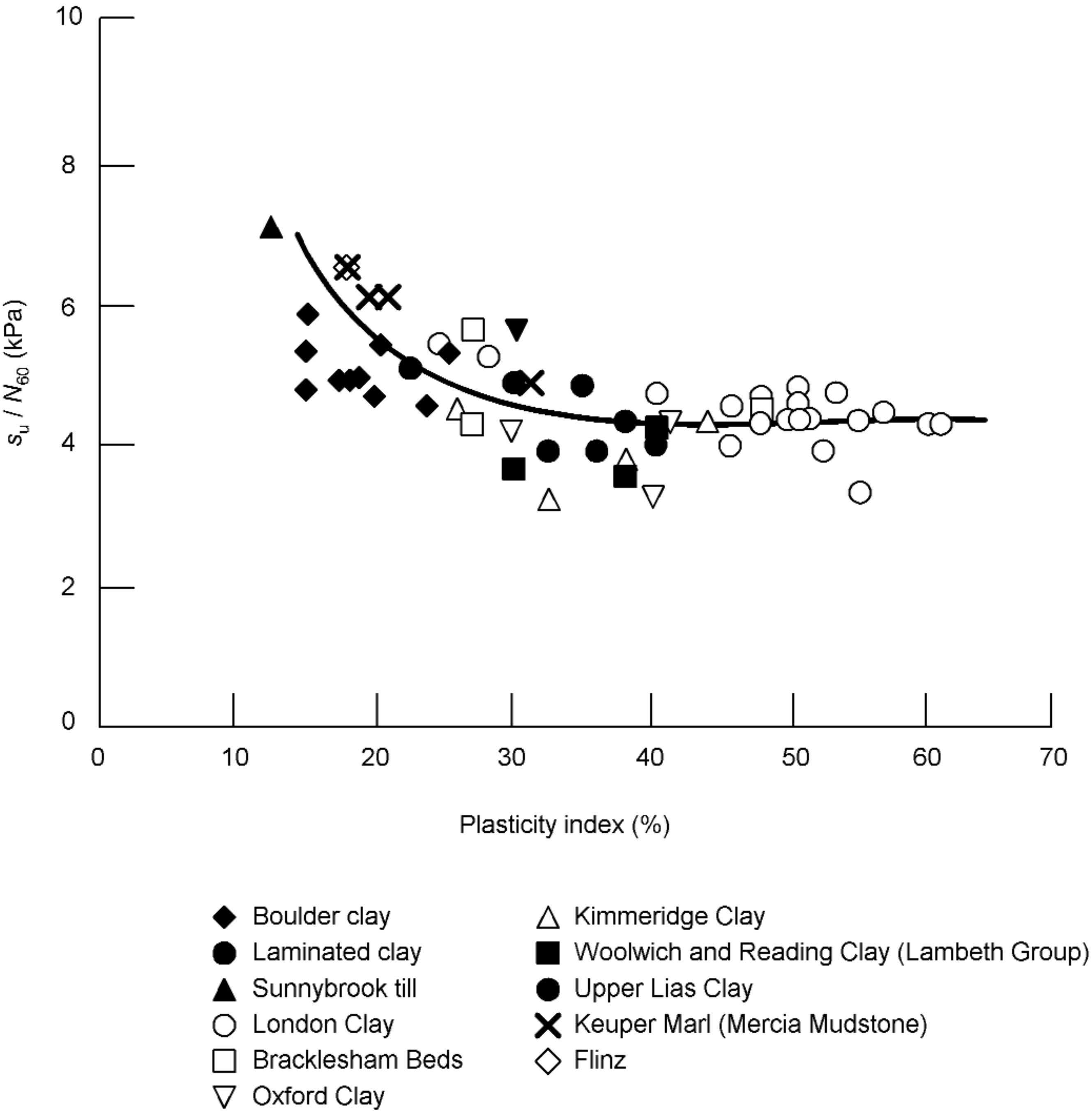


FIGURE 5.8 *Correlation between N_{60} value and undrained shear strength and plasticity index for insensitive clays (after Stroud and Butler, 1975)*

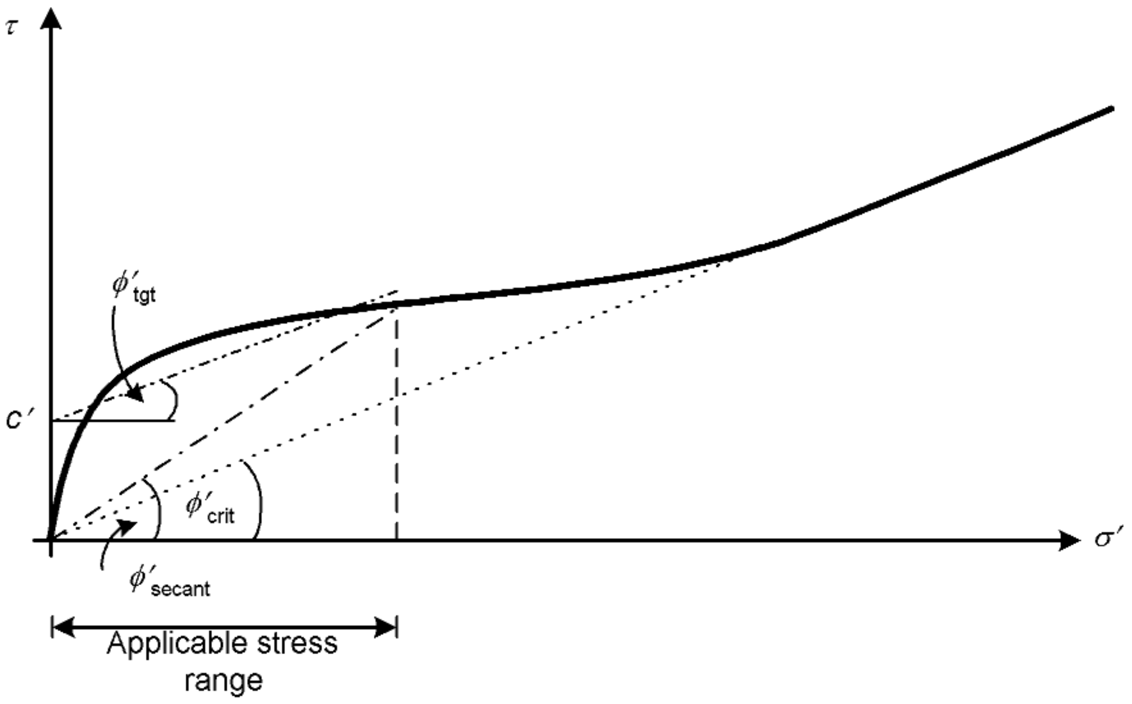


FIGURE 5.9 *Strength envelope for a given pre-consolidation*

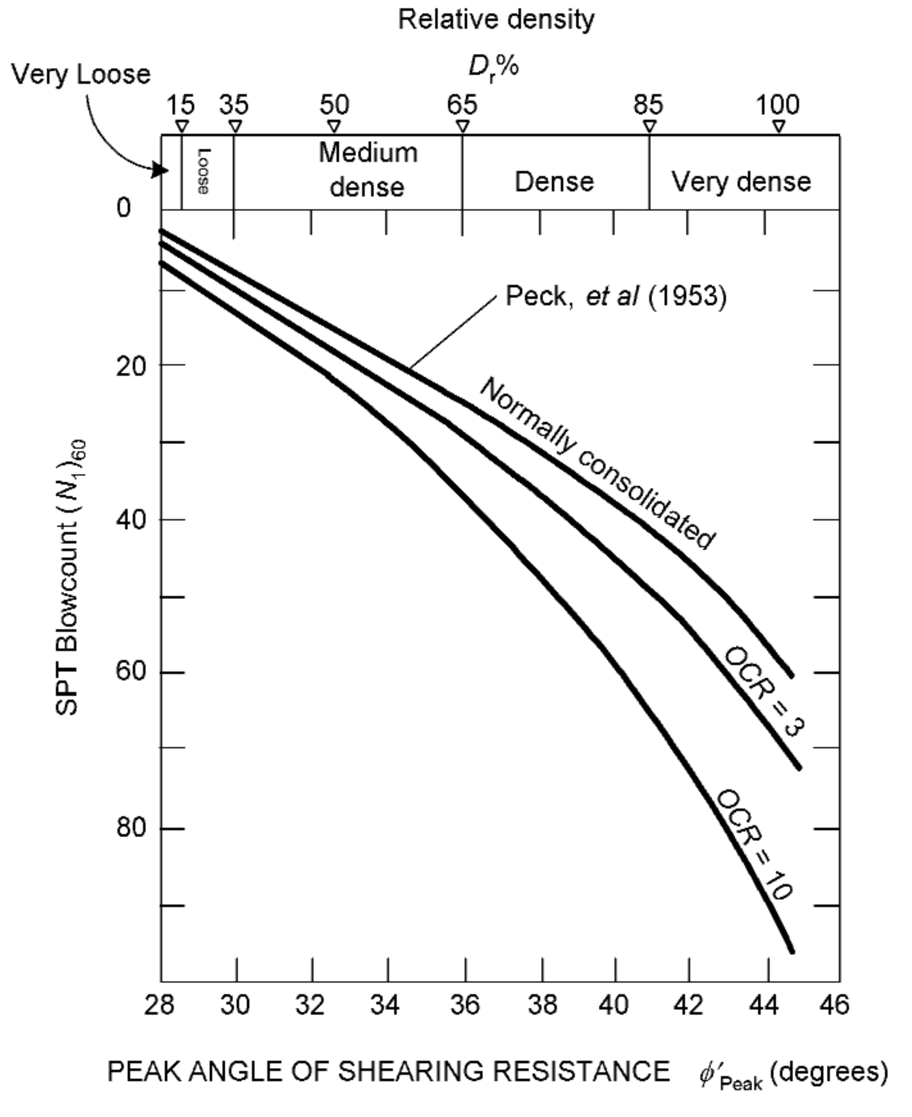


FIGURE 5.10 *Effect of overconsolidation on the relationship between* $(N_1)_{60}$ *and peak angle of friction* ϕ'_{peak} *(after Stroud, 1989)*

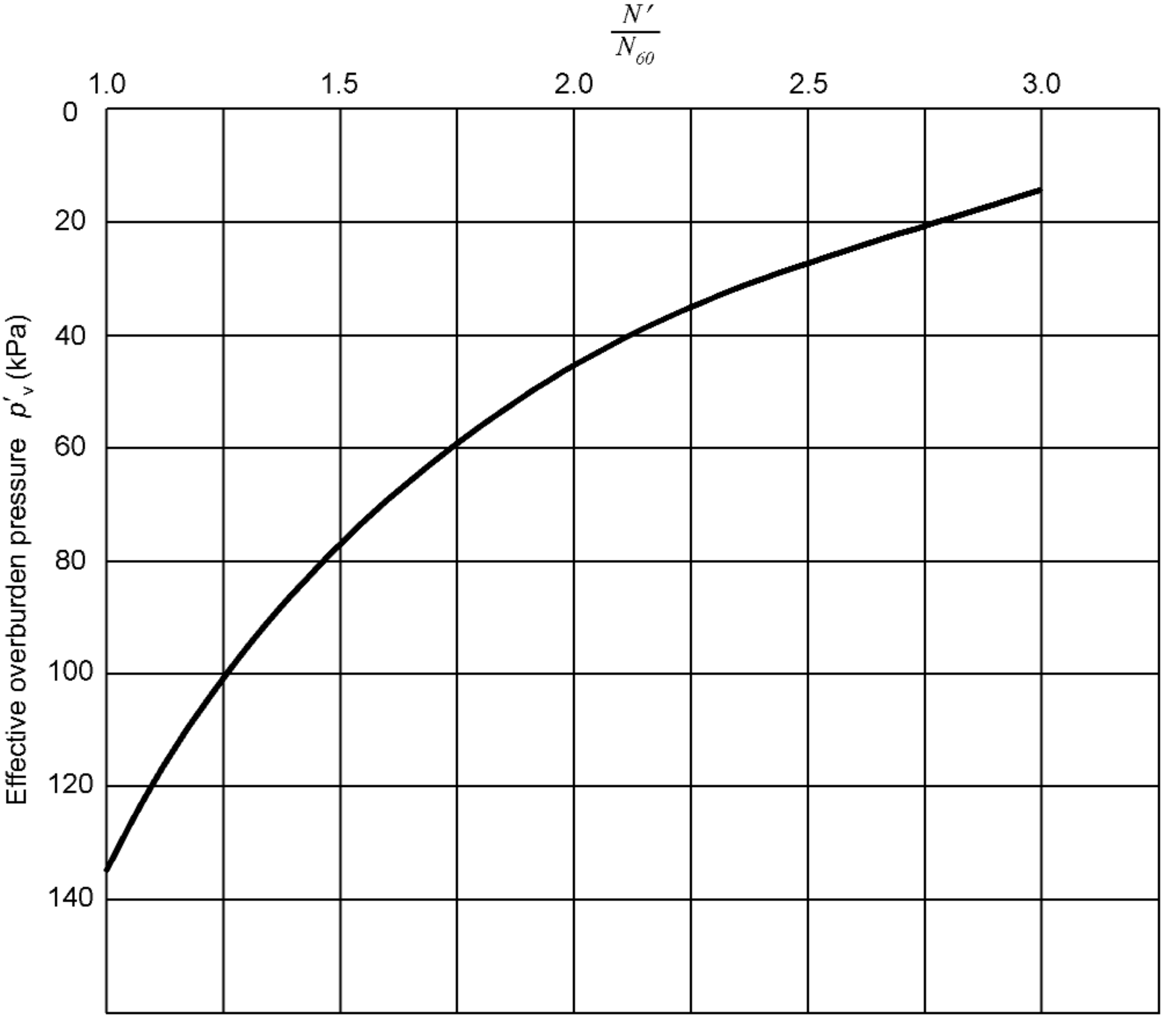


FIGURE 5.11 *Derivation of N' from SPT blowcount N_{60} (after BS 8002, 1994)*

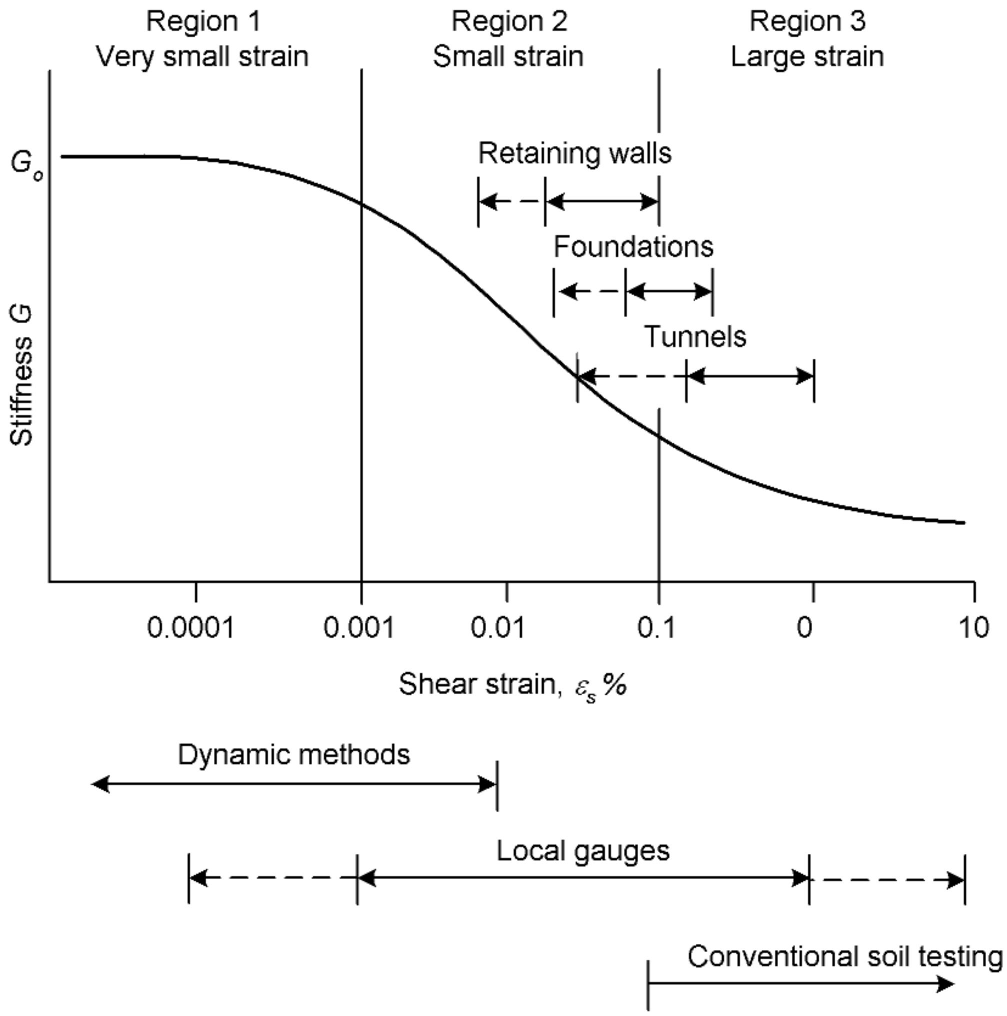
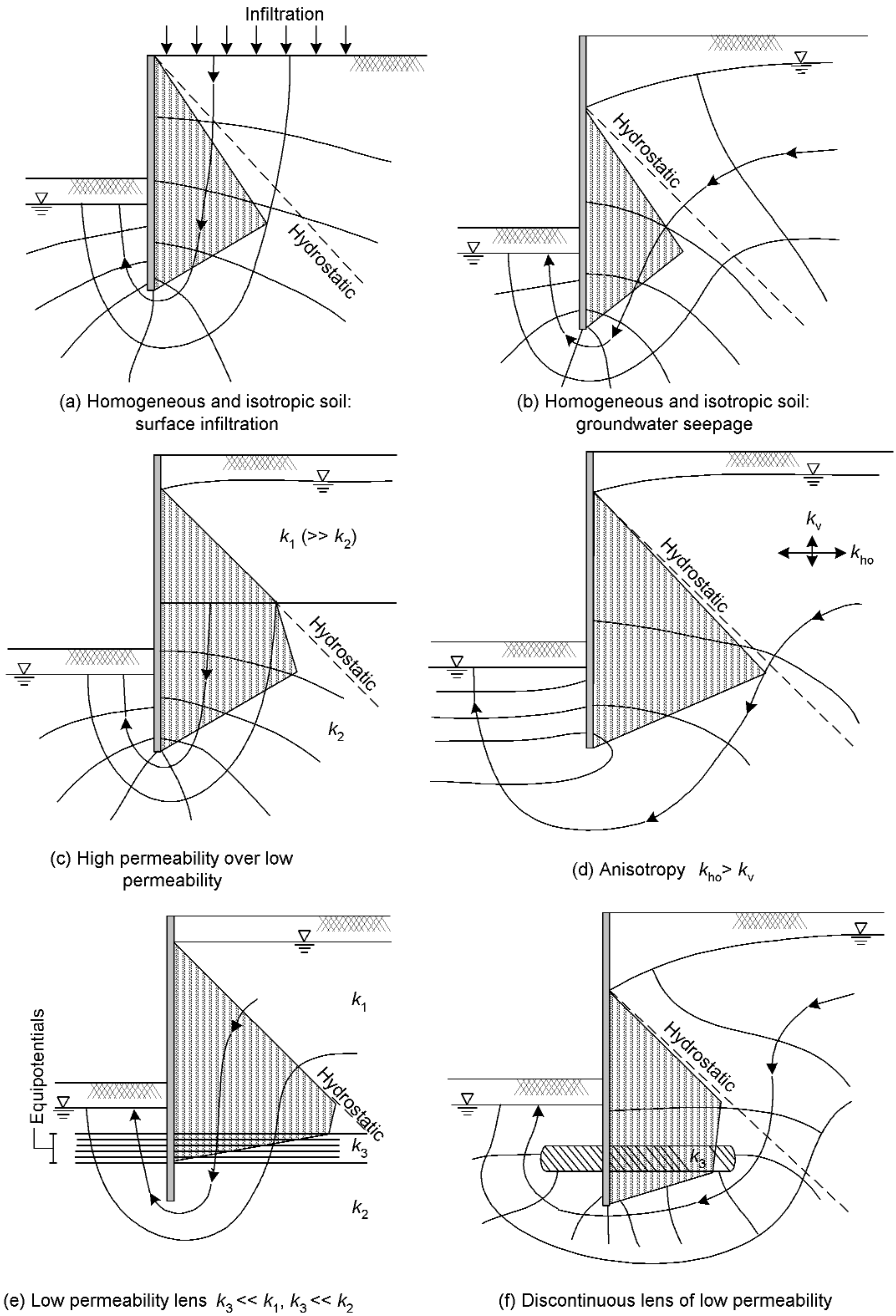
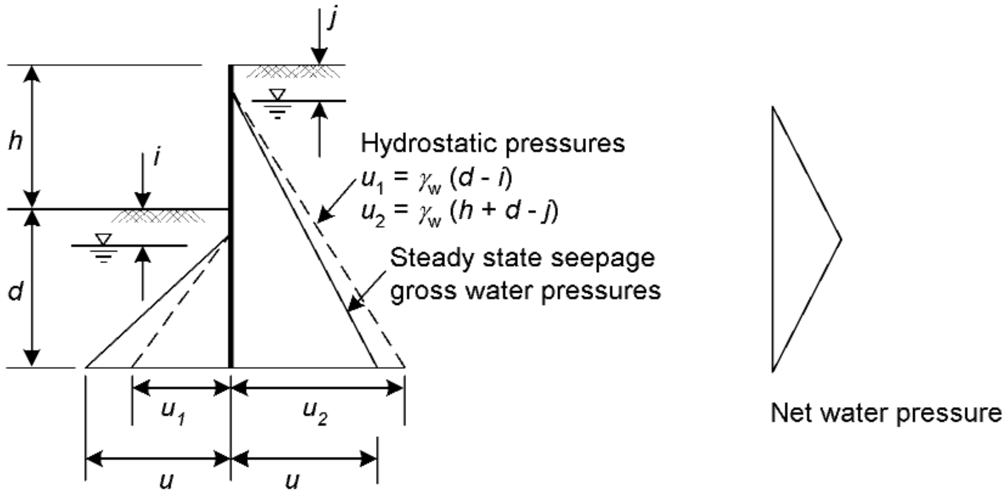


FIGURE 5.12 *Stiffness - strain behaviour of soil with typical strain ranges for laboratory tests and structures (after Atkinson and Sallfors, 1991 and Mair, 1993)*



Note : shading behind wall shows net water pressure

FIGURE 5.13 *Various seepage flownets for an impermeable wall*



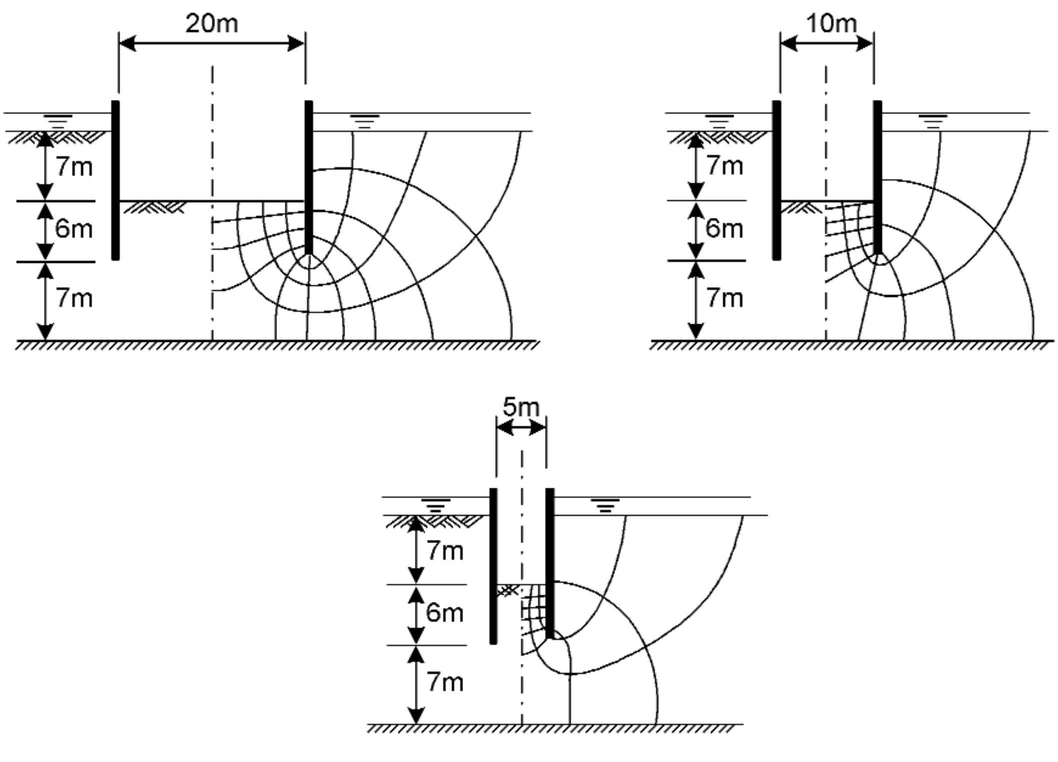
= Assumption (1)
Uniform dissipation of differential head along flow path adjacent to the wall

$$u = \frac{2(d + h - j)(d - i)}{2d + h - i - j}\,\gamma_w$$

= Assumption (2)
Average hydrostatic pressure at wall toe

$$u = \frac{u_1 + u_2}{2} = \gamma_w \left[d + \left(\frac{h - i - j}{2} \right) \right]$$

(a) Uniform ground : simplifying assumptions for calculating water pressures for steady state seepage conditions



(b) Uniform ground : effect of width of excavation on the flow net

FIGURE 5.14 *Linear steady state seepage in uniform ground and the effect of excavation width (after Williams and Waite, 1993)*

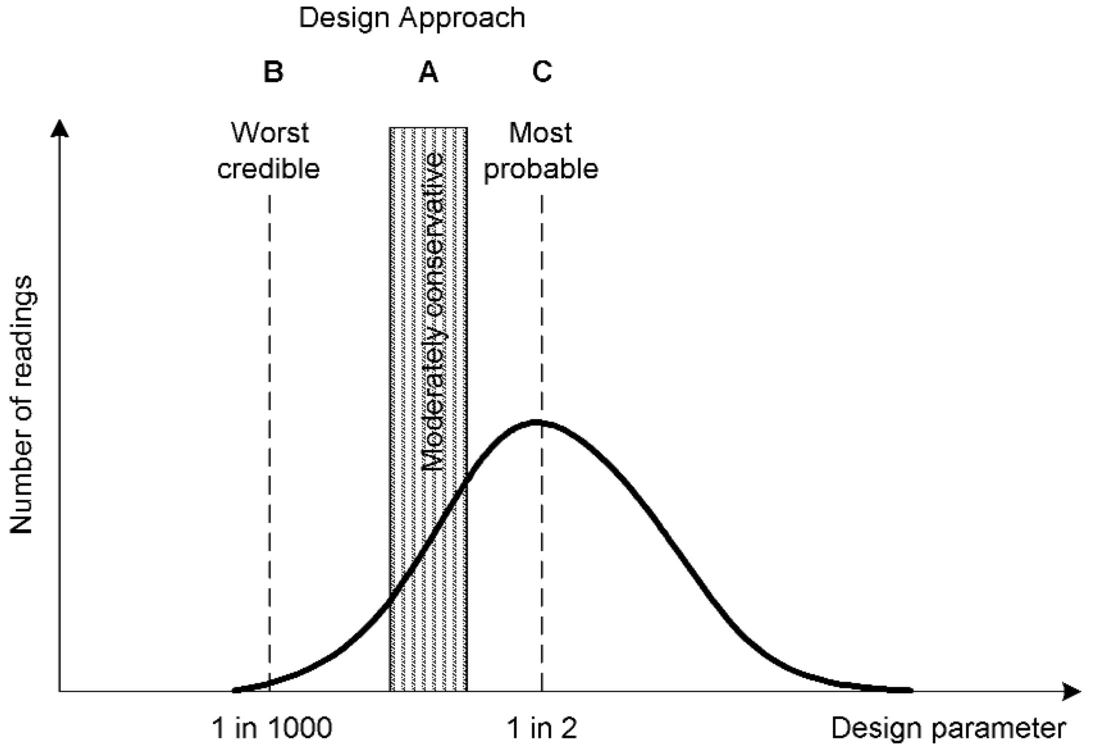
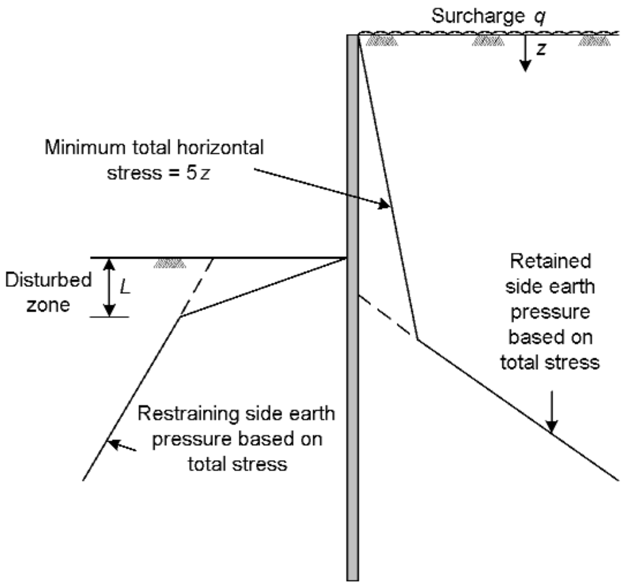
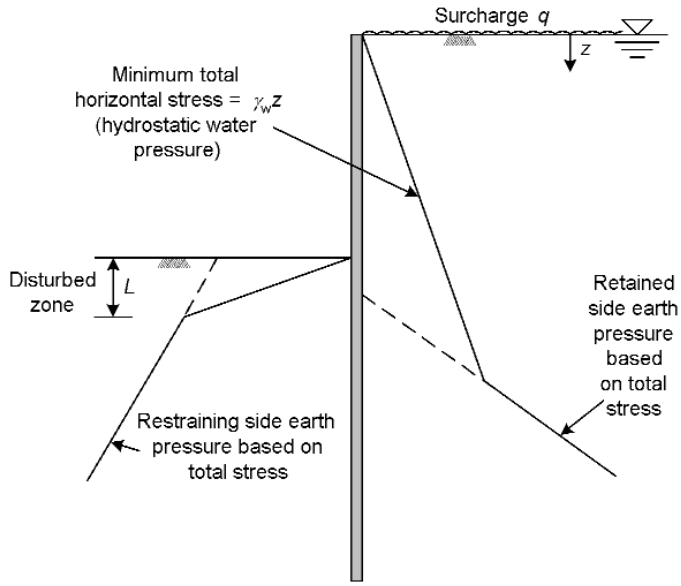


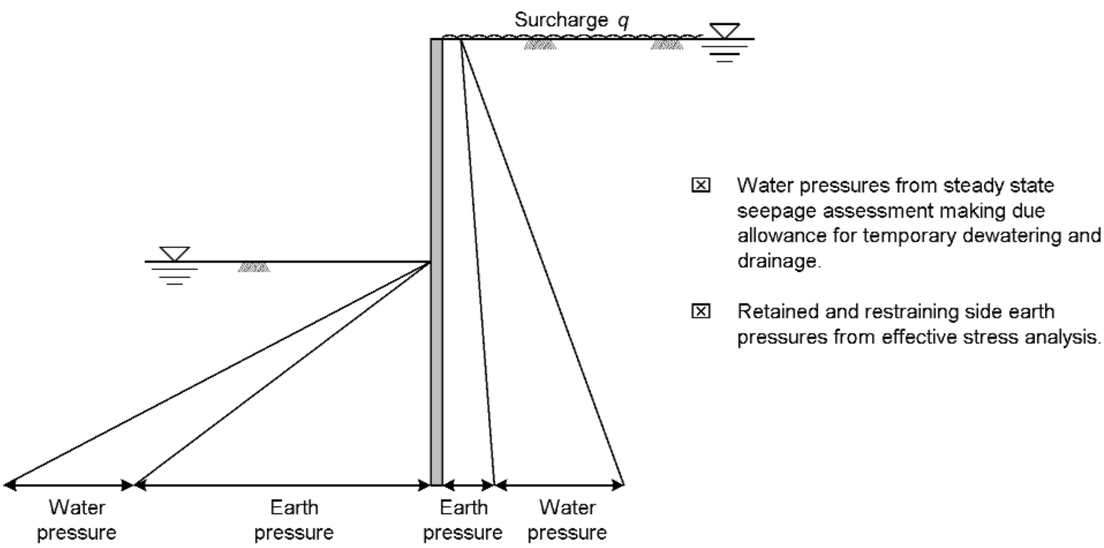
FIGURE 5.15 *Design parameters - definition of terms*



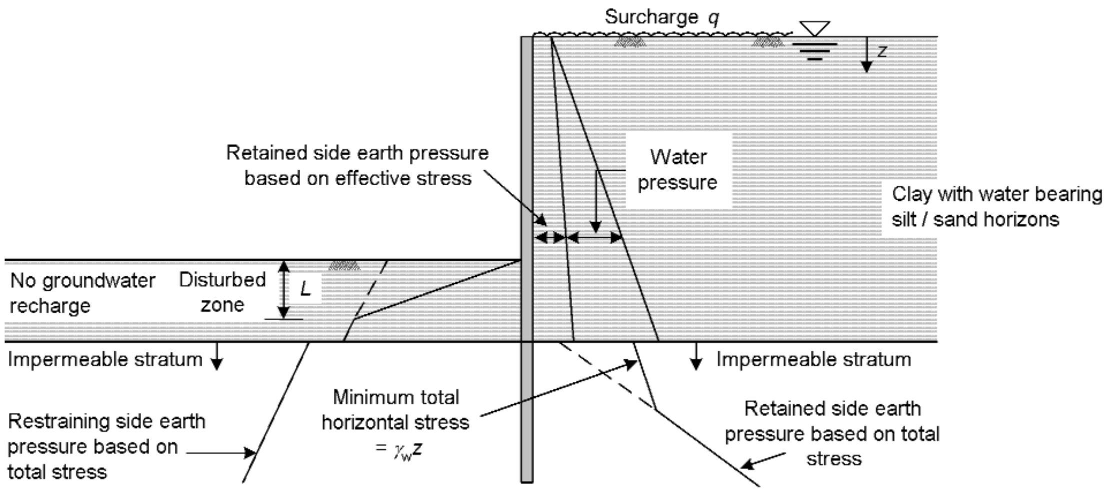
(a) *Undrained conditions : dry tension cracks*



(b) *Undrained conditions : flooded tension cracks*



(c) *Drained conditions*



(d) *Undrained / drained conditions*

FIGURE 5.16 *Temporary works design assumptions*

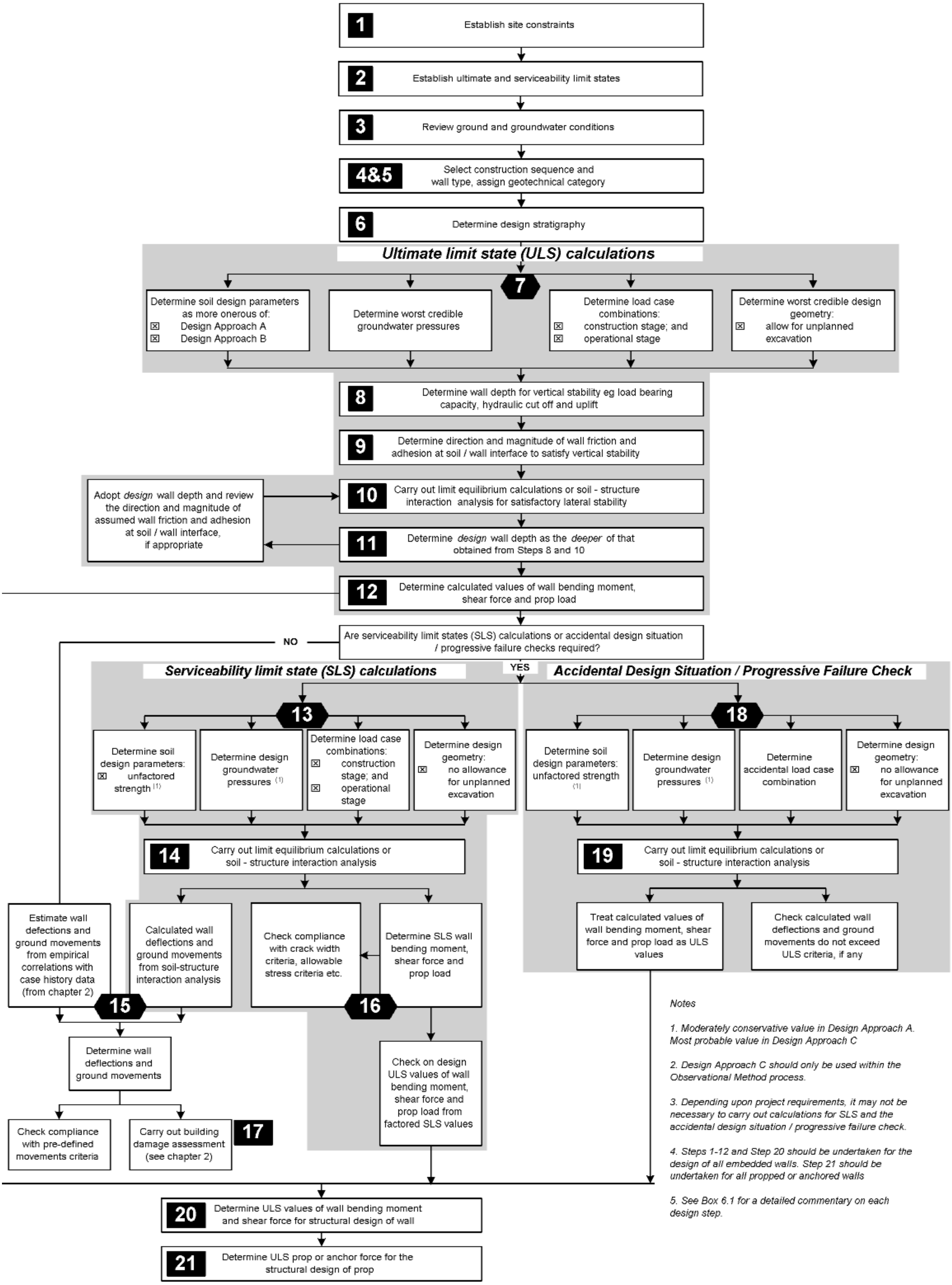
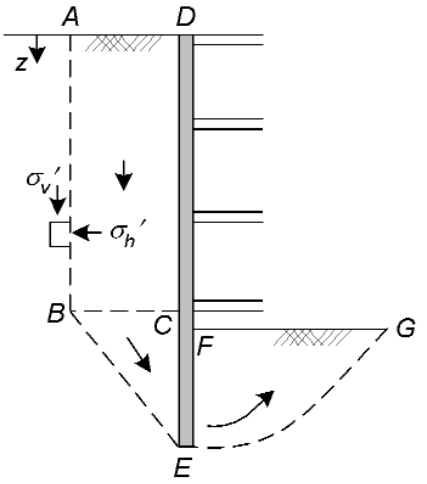
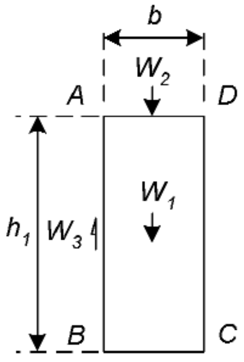


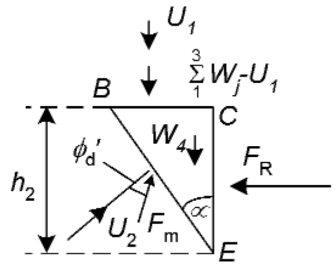
FIGURE 6.1 *Design method*

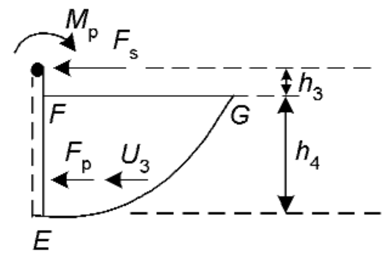




Notes:
1. Calculation to be done in terms of design angle of shearing resistance ϕ_d'

2. Pore water pressure u can be calculated assuming uniform loss of head around wall; assume u constant along any given horizontal plane

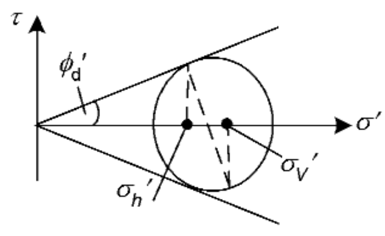




<u>Stresses along AB :</u>

assuming a vertical failure plane

$$\sigma_h' = \frac{1 - \sin^2\phi'_d}{1 + \sin^2\phi'_d}\,\sigma_v'$$



where $\sigma_v' = \gamma z - u$ and γ = bulk unit weight

z = depth below ground

u = pore water pressure

<u>Magnitude of forces :</u>

F_R = required force on BCE to maintain stability, calculated using "wedge" analysis; assumed to act at ½ h_2 below lowest strut

F_p = available "effective" passive resistance of EFG ($\bar{\sigma}_v'{}_p \,.\, K_p \,.\, h_4$); assumed to act at $^2/_3\,h_4$ below ground level

F_s = shear force capacity of wall immediately below lowest strut

M_p = plastic moment capacity of wall at lowest strut

Note : F_R and F_p are horizontal components of force

$W_1 = \gamma h_1 b$

W_2 = load applied on block ABCD from surcharges and structure

$W_3 = \int_0^{h_1} \sigma_h' \tan \phi'_d \, dz$

$W_4 = ½\, b\, h_2\, \gamma$

$$U_1 = \frac{u_B + u_C}{2}\,.\,b$$

$$U_2 = \frac{u_B + u_E}{2}\;\frac{h_2}{\cos\alpha}$$

$$U_3 = \frac{u_F + u_E}{2}\;h_4$$

U_3 assumed to act $^2/_3\,h_4$ below ground level

<u>Stability requirements;</u>

$$F_R \ngtr F_p + U_3 + F_s$$

$$F_R \,.\, ½\, h_2 \ngtr M_p + (F_p + U_3)\,.\,(h_3 + {}^2/_3\, h_4)$$

FIGURE 6.2 *Postulated failure mechanisms used to check toe stability (after Phillips et al, 1993)*

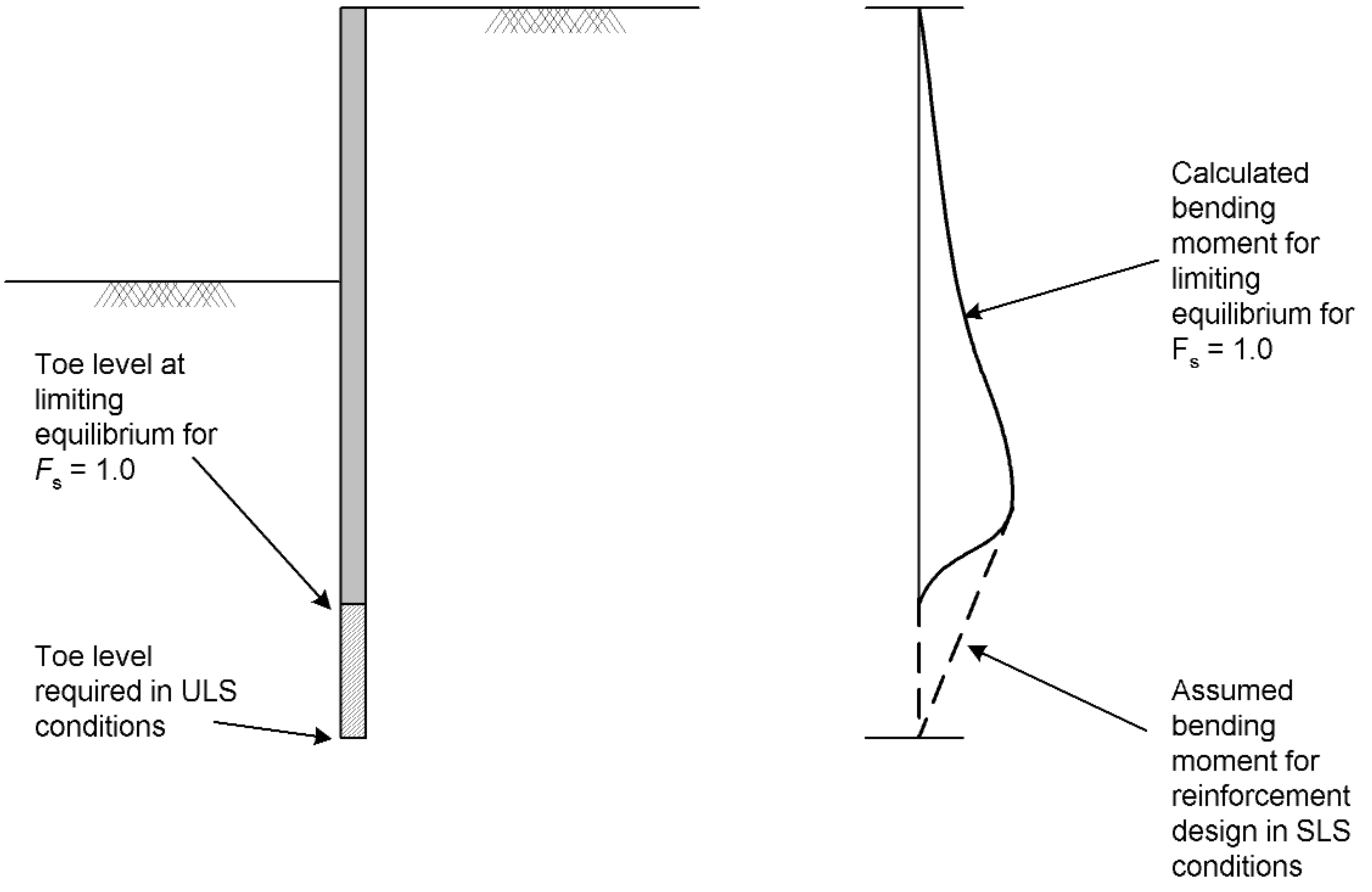


FIGURE 6.3 *Determination of wall bending moments in SLS conditions for limit equilibrium calculations*

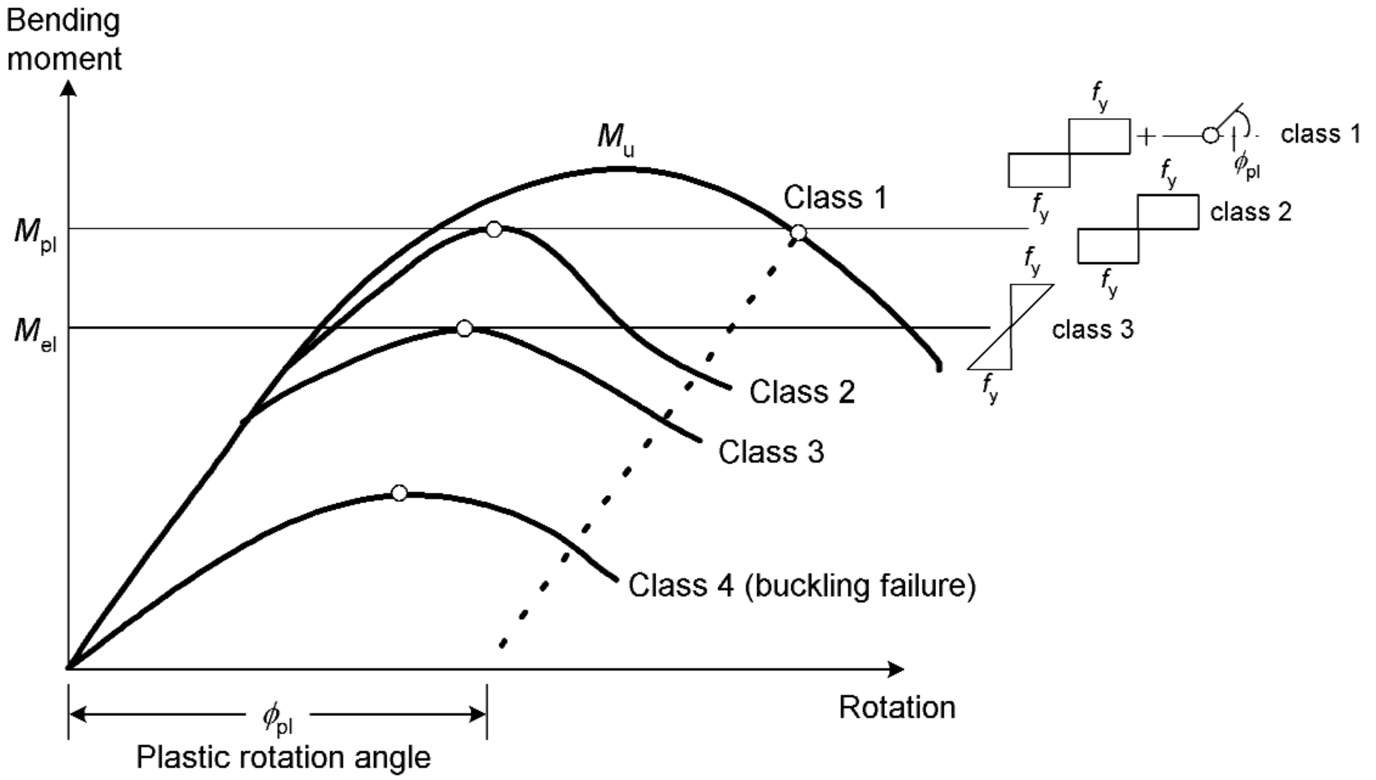


FIGURE 6.4 *Design of sheet pile walls to EC3, Part 5 (ENV 1993-5 : 1998)*

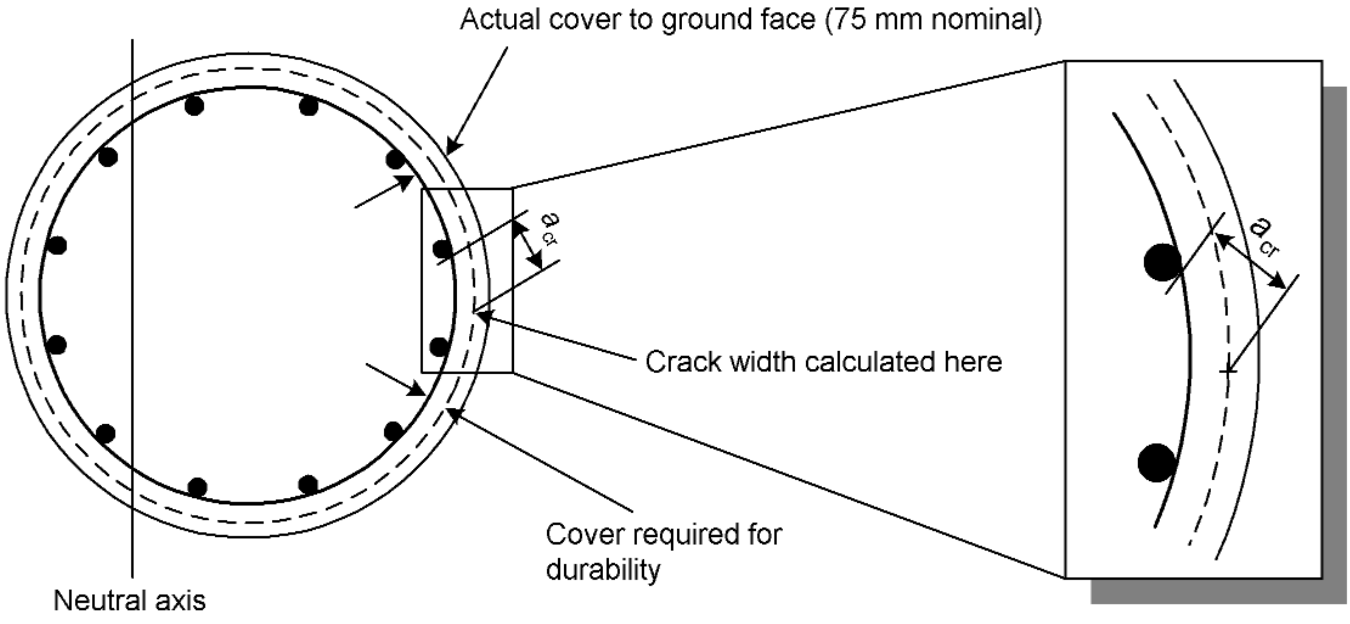


FIGURE 6.5 *Pile cross section showing crack width calculation principles to BS 8110*

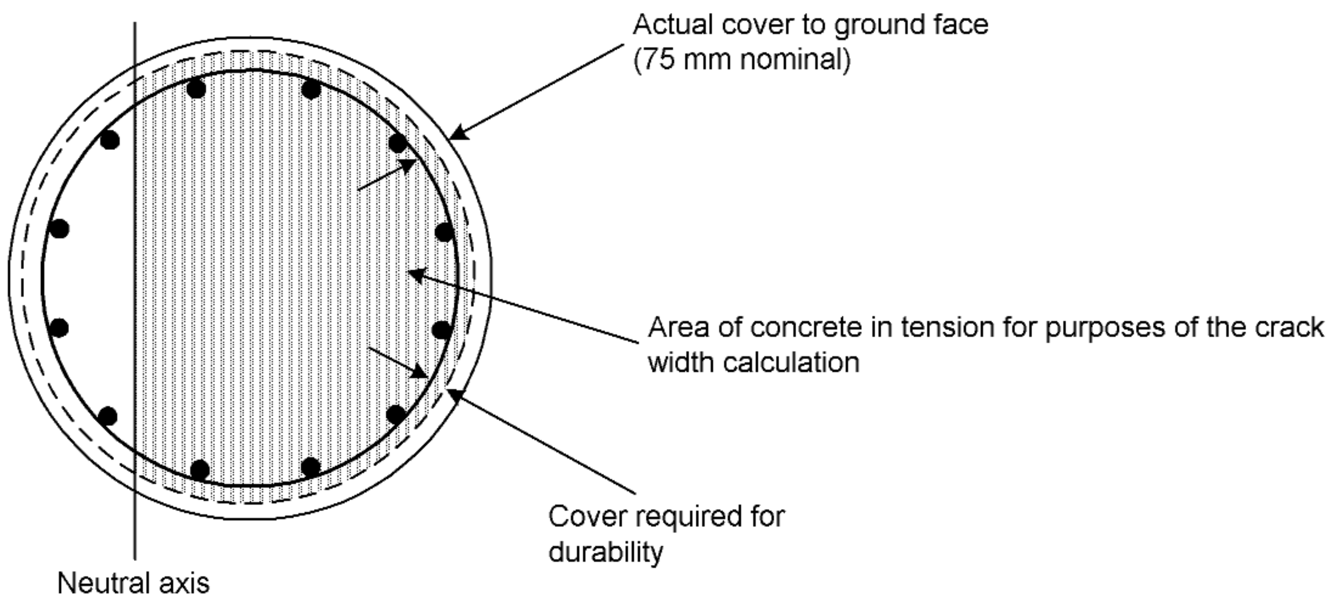


FIGURE 6.6 *Pile cross section showing crack width calculation principles to EC2 as applied on the Copenhagen Metro project.*

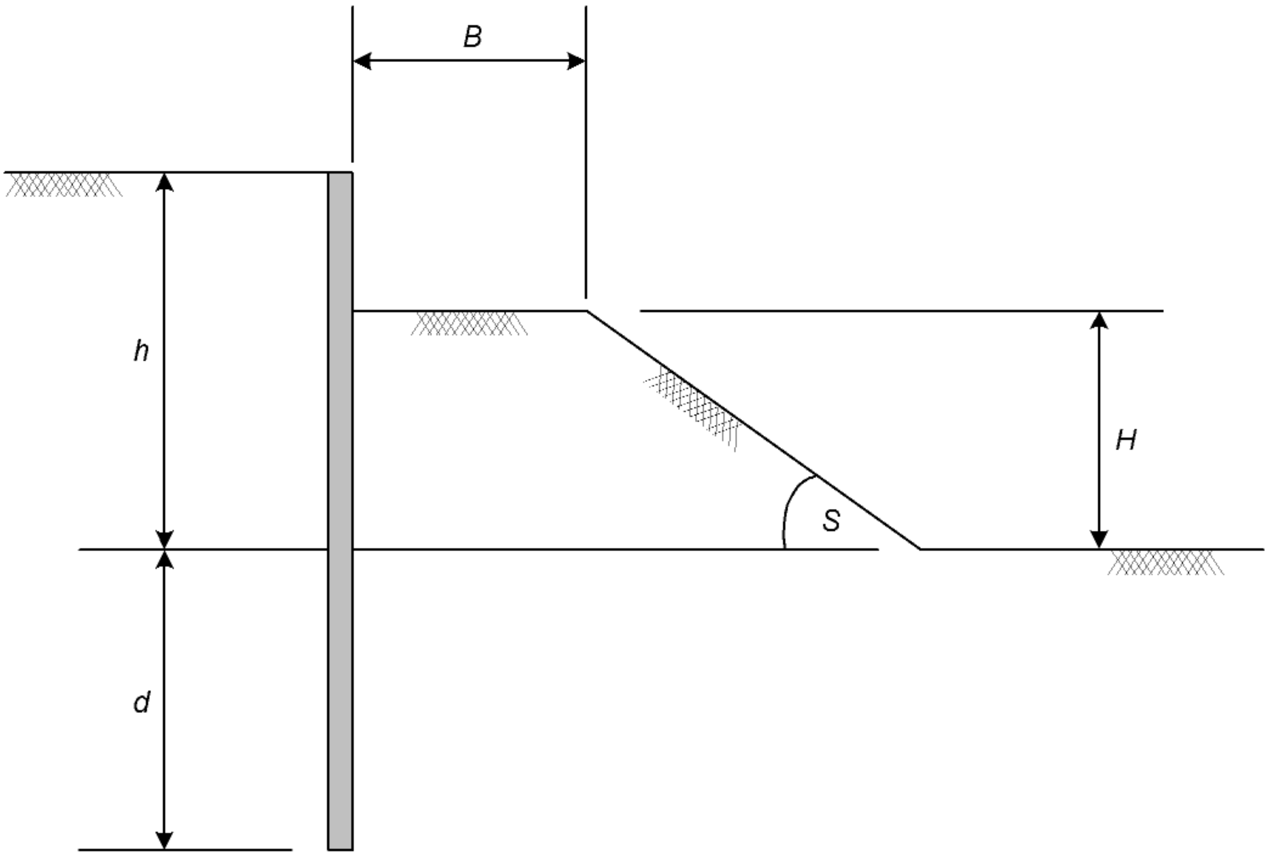


FIGURE 7.1 *Definition of wall and berm geometry*

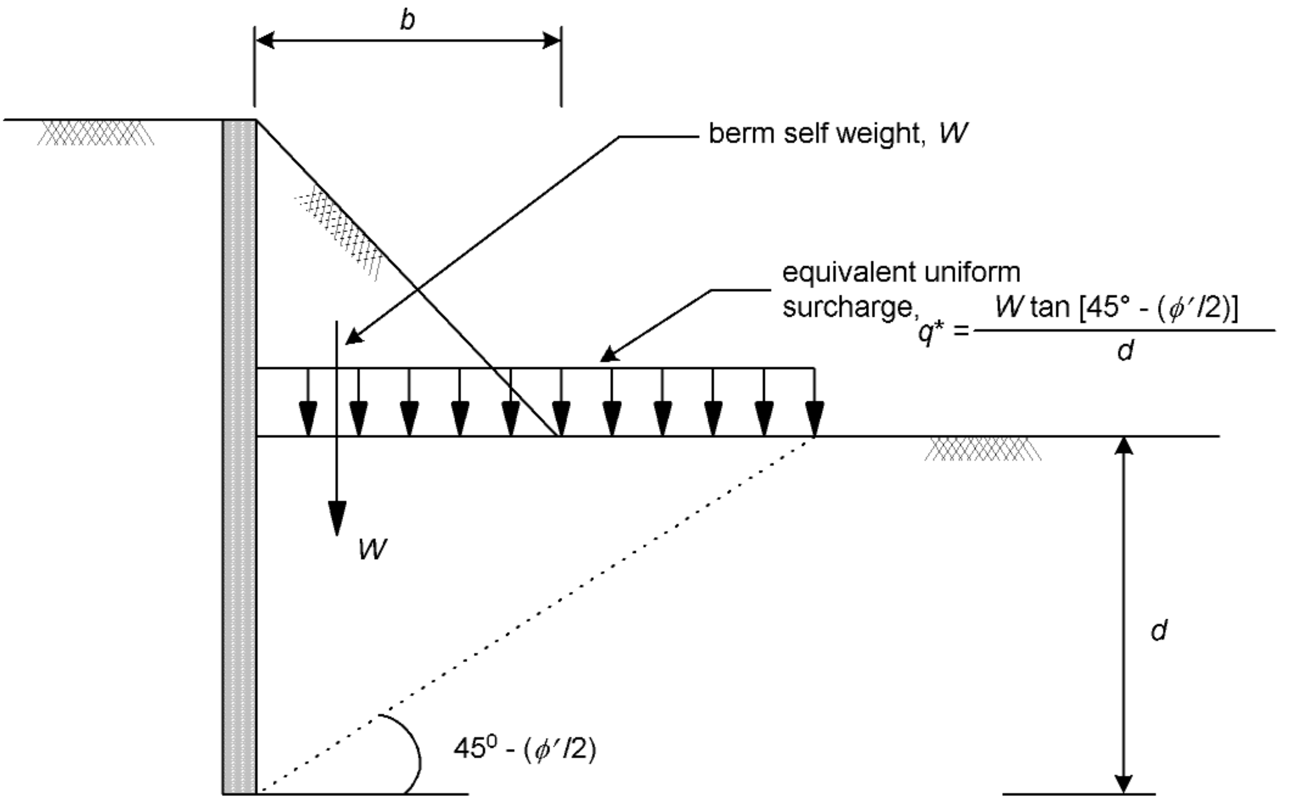


FIGURE 7.2 *Representation of a berm as an equivalent surcharge*

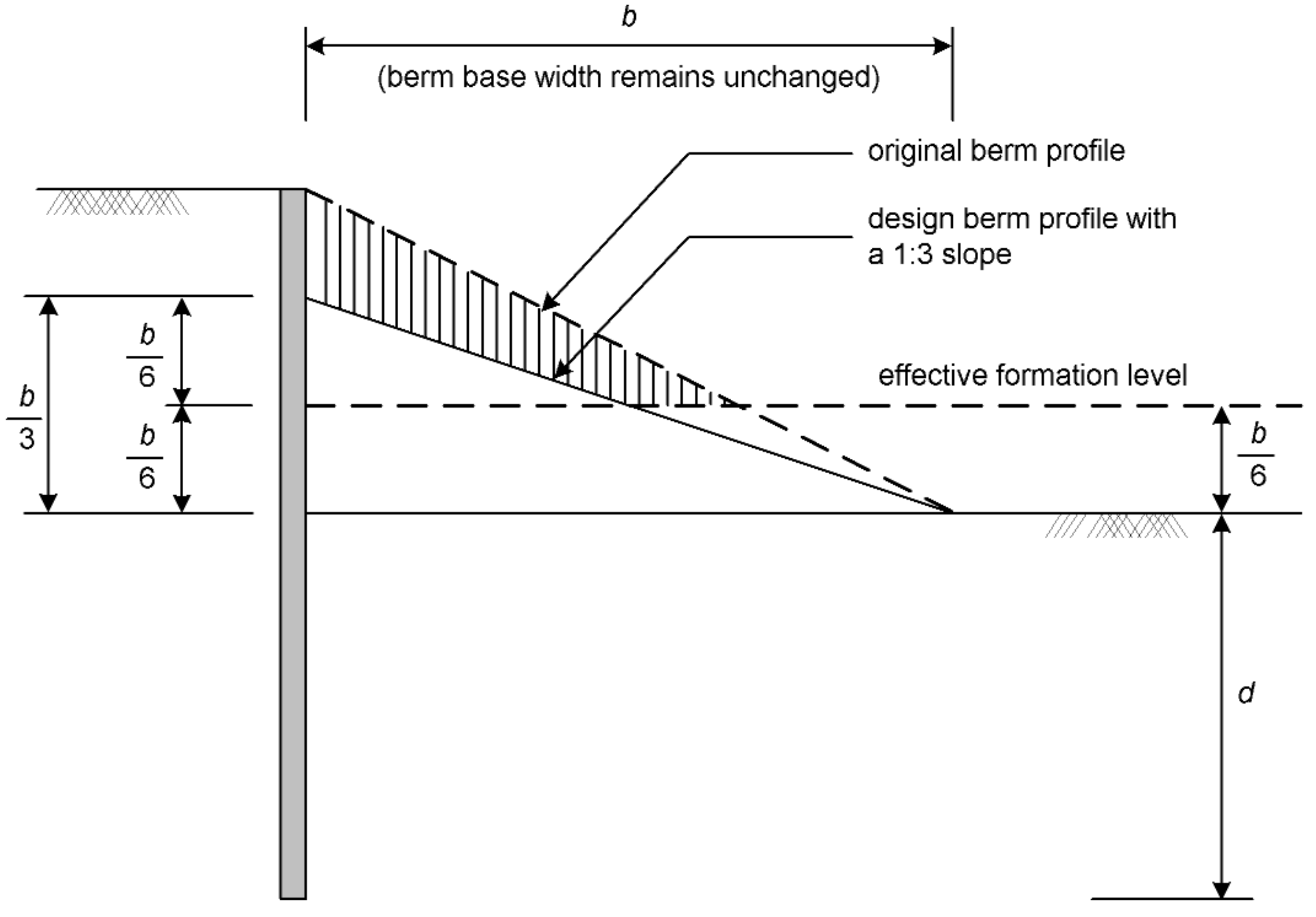
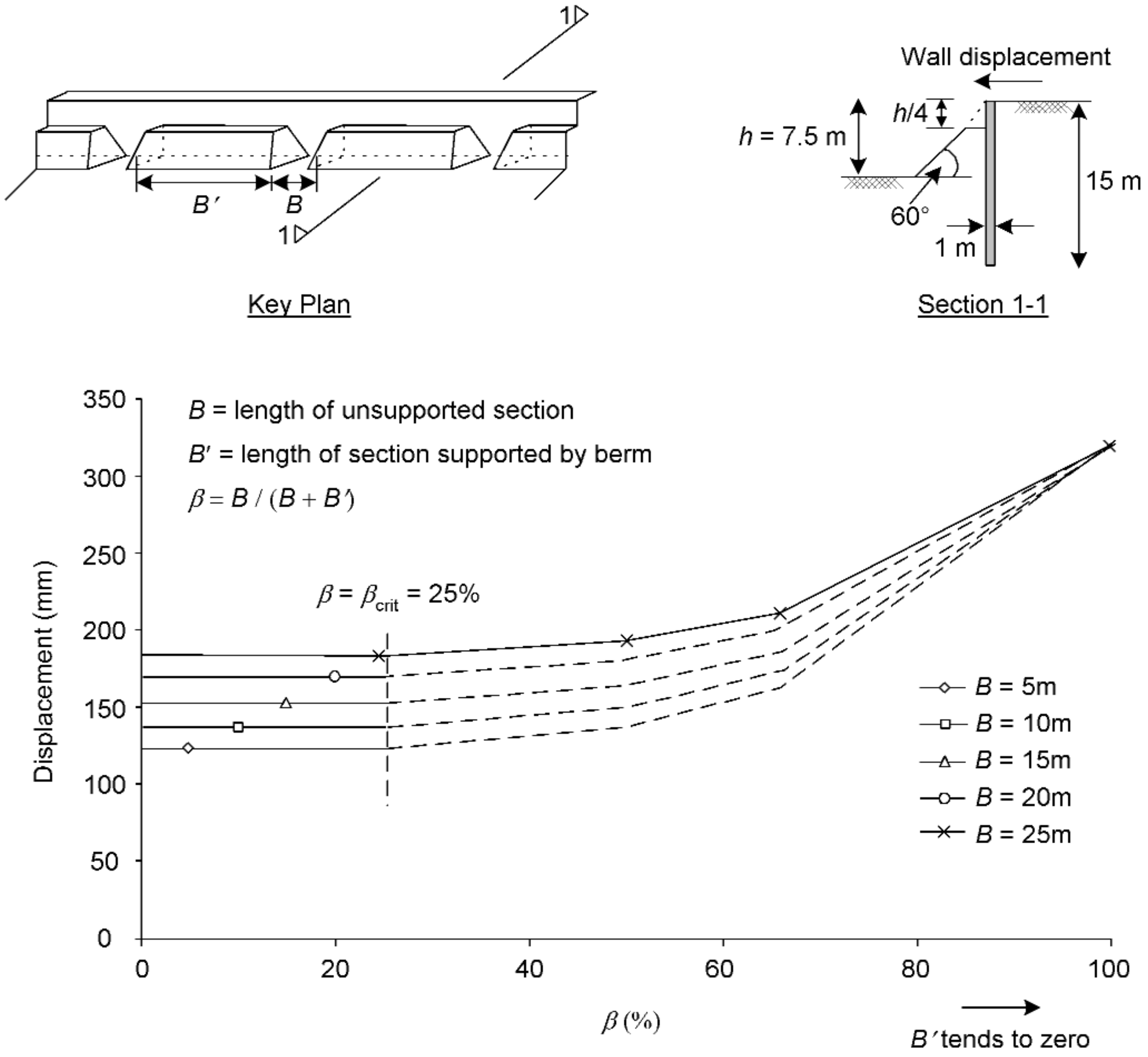
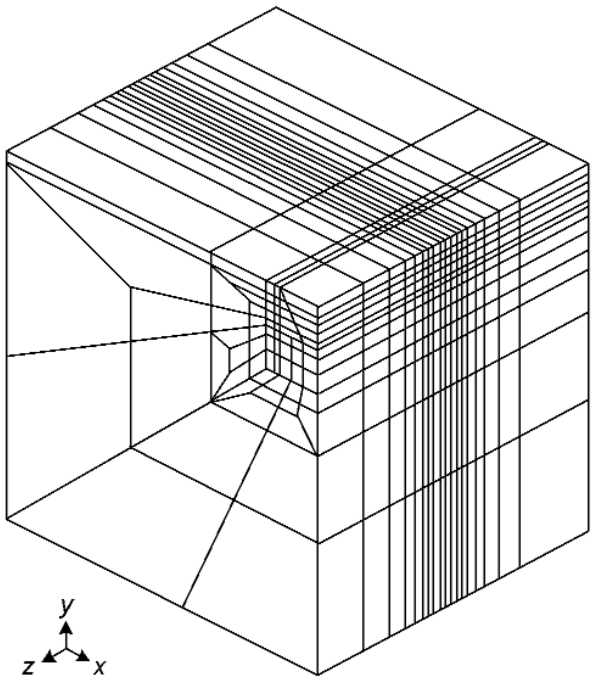


FIGURE 7.3 *Representation of a berm by means of a raised effective formation level (after Fleming et al, 1994)*

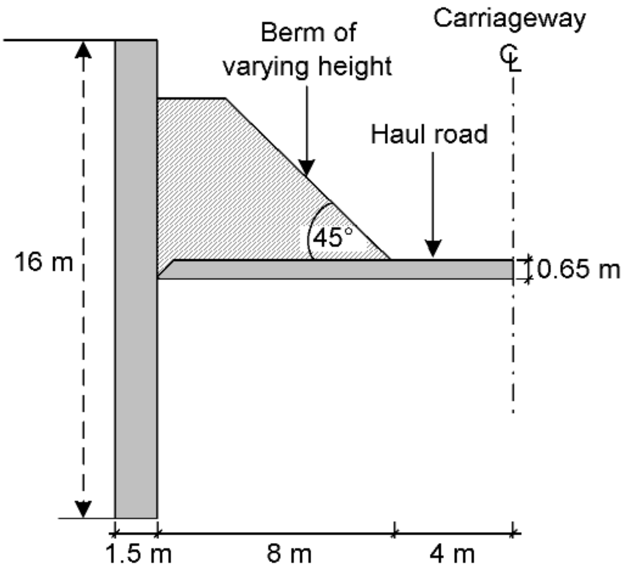


Note:The data points and solid lines represent confirmed findings, and the broken lines conjecture

FIGURE 7.4 *Normalised wall top displacement at the centre of the unsupported section against degree of discontinuity for different excavated bay lengths B. (The data points and solid lines represent confirmed findings, and the broken lines conjecture)*



Perspective view of 3D finite element mesh



Typical wall geometry

Soil parameters used in the analysis

Soil type	K_o	E'(MPa)	mE'(MPa/m)	ϕ'(°)	c'(kPa)	k_x(m/sec)	k_y(m/sec)
Stiff to very stiff clay	2.0	32	8.4	22	0 and 20	5×10^{-10}	1×10^{-10}
Firm to stiff clay	1.0	16	4.2	28	0 and 10	1×10^{-6}	1×10^{-7}

Notes
(1) Soil assumed as elastic perfectly plastic with Mohr Colomb failure criterion
(2) Retaining wall wished in place
(3) Groundwater table assumed at 1 m below existing ground level

FIGURE 7.5 *Three dimensional finite element mesh, wall and excavation geometry and assumed soil parameters (after Easton et al, 1999)*

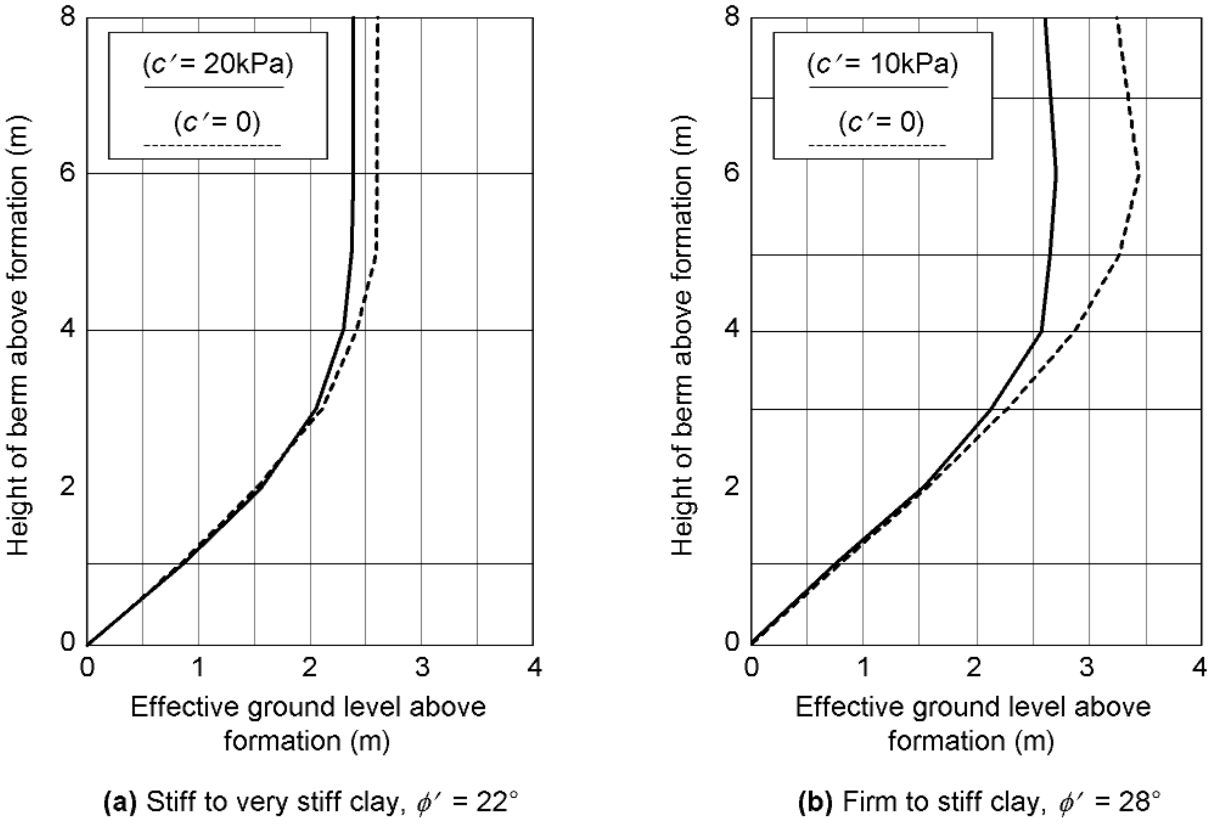


FIGURE 7.6 *Relationship between berm height and effective uniform ground level, (a) stiff to very stiff clay, ϕ' = 22°, (b) firm to stiff clay, ϕ' = 28° (after Easton et al, 1999)*

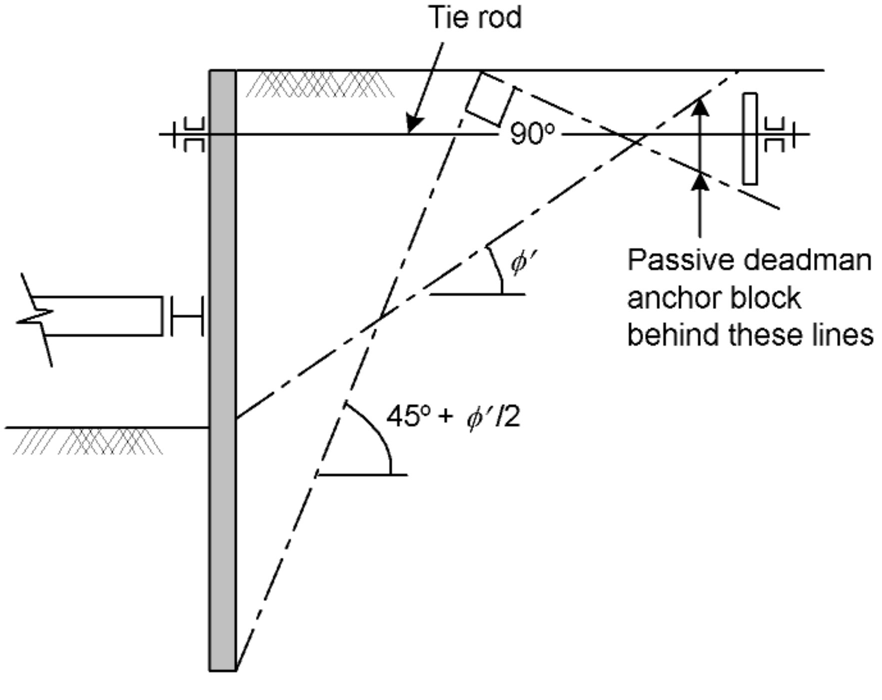


FIGURE 7.7 *Passive deadman anchor (after Williams and Waite, 1993)*

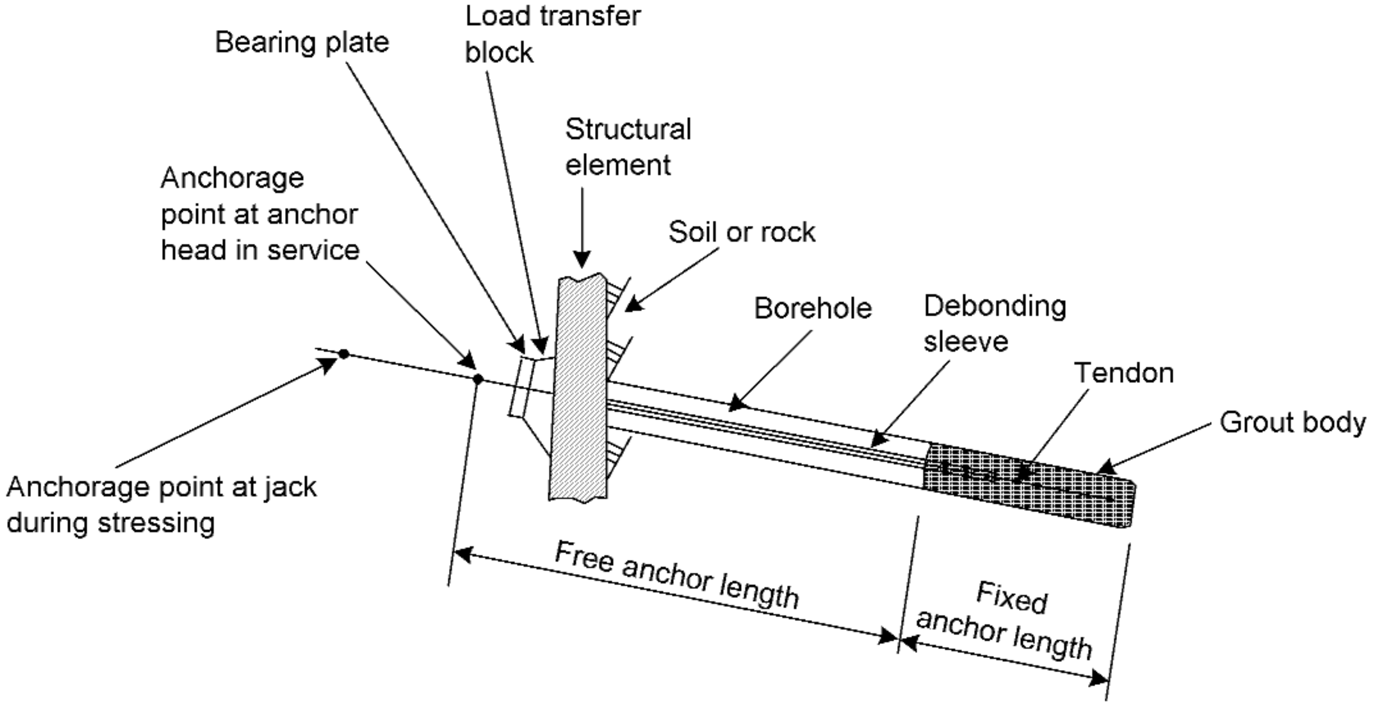


FIGURE 7.8 *Sketch of a typical ground anchorage, (from BS EN 1537, 2000)*

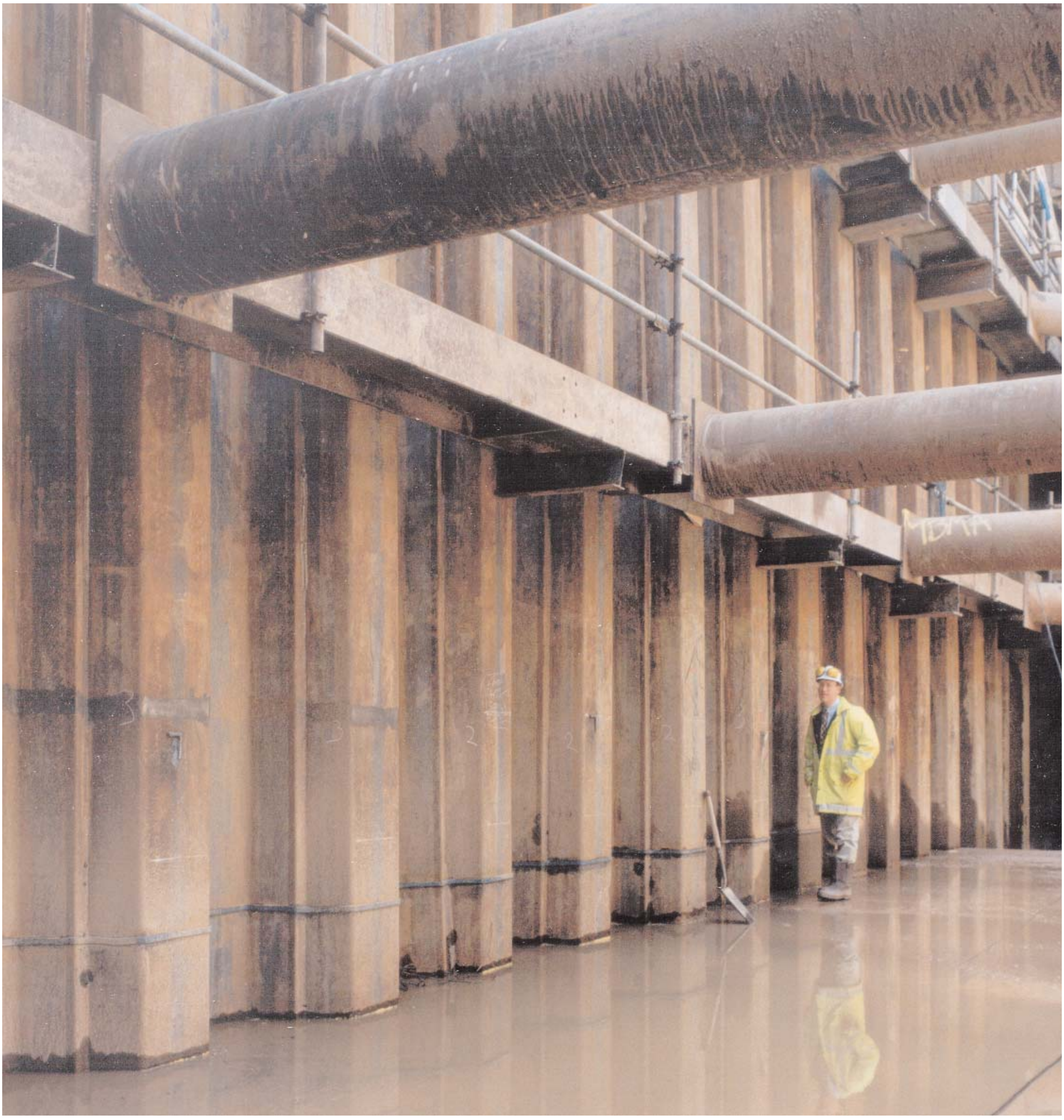


PLATE 7.1 *Propped steel sheet piles*

Appendix A Example CDM risk assessment forms

FORM A1 **RISK ASSESSMENT: DECISION JUSTIFICATION**

FORM A2 **RISK ASSESSMENT: RECORD OF SELECTION**

FORM A3 **RISK REGISTER**

Project	Job number
Design stage FEASIBILITY CONCEPT SCHEME DETAILED DESIGN OTHER	Assessment Ref
Topic	
Change/Alternative under consideration	

Date	Made by	Checked by

Risk Assessment :

	Yes	No
Will the alternative eliminate or reduce the risk without introducing an equally or more severe one ?		

….and check :

	Yes	No
Will the alternative be significantly easier and quicker to build ?		
Will the alternative be significantly less expensive to build and/or maintain ?		
Will the alternative be significantly less harmful to the environment ?		
Will the alternative be significantly safer and healthier for users ?		
Will the alternative be aesthetically acceptable ?		

Notes / Comment / Qualifications (if necessary) :

	Yes	No
Does the change/alternative need client approval ?		

Action to secure approval (if required) :

	Yes	No
Conclusion - do we make the change or not ?		

User Notes:

1. The greyed answers are those preferred to justify the alternative, but sensible weighting of certain issues may influence the decision to change.
2. If the Planning Supervisor disagrees on an important H and S issue, append copies of relevant correspondence to this form.
3. If the Client disagrees on an important H and S issue and insists that his/her view prevails, append copies of relevant correspondence to this form.

Form A2 *Risk assessment - record of selection*

Project	Job number
Design stage FEASIBILITY CONCEPT SCHEME DETAILED DESIGN OTHER	Assessment Ref
Topic	Hazard

Alternative A :

Assess the likelihood of harm from this hazard	H=3	M=2	L=1	Product	
Assess the likely severity of harm from this hazard	H=3	M=2	L=1		
Notes/Hazards arising					

Alternative B :

Assess the likelihood of harm from this hazard	H=3	M=2	L=1	Product	
Assess the likely severity of harm from this hazard	H=3	M=2	L=1		
Notes/Hazards arising					

Alternative C :

Assess the likelihood of harm from this hazard	H=3	M=2	L=1	Product	
Assess the likely severity of harm from this hazard	H=3	M=2	L=1		
Notes/Hazards arising					

Preferred Alternative: A B C

All risks should be reduced if it is reasonably practicable to do so.

Reason for Preferred Alternative:		
Date	Made by	Checked by

Form A3 *Risk register*

Register reference

Project	Job number
Package/Topic	Design stage

Remember: Avoid – Reduce – Control and communicate relevant information to others (CDM Regulation 13)

Date (+ initials)	**Description of Hazard and Risk Exposure**	**Mitigation of Risk (Potential or Achieved)**	**Further Action**	**by**	**Status** Active/closed

Example

Appendix B Ground movements and case history data

B1 GROUND MOVEMENTS DUE TO WALL INSTALLATION

B1.1 Overview

Ground movements arising from bored pile and diaphragm installation in stiff clays are summarised in the following publications/documents:

- Clough and O'Rourke (1990)
- Thompson (1991)
- Carder (1995)
- Carder *et al* (1997).

Figures 2.8 and 2.9 show the combined data from the above.

Some data fall outside the envelope of settlements arising from the construction of secant bored pile walls shown on Figure 2.8 (b). These data relate to walls constructed at:

- British Library, Euston, London
- Vintners Place north east wall, London
- Blackfriars, London.

Thompson (1991) states that the retaining wall at the British Library is very complicated and in many areas involved the construction of sheet piles around the wall prior to the construction of the secant bored piles. The settlements measured at the British Library site also included consolidation settlement due to local dewatering at this location. The sites at Blackfriars and Vintners Place were underlain by some 5 m to 7 m of sand and gravel deposits overlying stiff London Clay. Thompson (1991) reports that at these locations, these deposits comprised mainly sand with only a small proportion of gravel. The higher local settlement was probably due to ground disturbance arising from hydraulic imbalance in these deposits during pile boring. These examples illustrate the importance of good workmanship and effective control of construction operations to minimise ground movements. They also illustrate the localised nature of ground movements arising from construction problems.

Clough and O'Rourke's (1990) upper bound movements limit is likely to over-estimate ground movements arising from bored pile and diaphragm walls installed in stiff clay under conditions of good workmanship. The Clough and O'Rourke upper bound limit includes ground movements arising from diaphragm walls installed in soft alluvial deposits overlying completely decomposed granite in Hong Kong and walls installed in soft clay at Studenterlunden, in addition to walls installed in stiff clays.

B1.2 Case history data

Case histories relevant to wall installation effects are summarised in the following tables:

- Table B1.1, Clough and O'Rourke (1990)
- Table B1.2, Thompson (1991)
- Table B1.3, Carder (1995)
- Table B1.4, Carder *et al* (1997).

Table B1.1 *Clough and O'Rourke (1990)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Reference*
11	New Palace Yard	UK	Diaphragm Wall (1 m thick)	Top down, multi-propped	10 m of Made Ground/sand and gravel over London Clay		5.5	30	Burland and Hancock (1977); St John (1975); Simpson *et al* (1979)

Note: For details of source references, refer to Clough and O'Rourke (1990)

Table B1.2 *Thompson (1991)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Reference*
4	Bell Common (M25)	UK	Secant pile wall (1.18 m piles at 1.08 m centres)	Top propped	7 m Older Head and Claygate beds over London Clay		3	20	Tedd *et al* (1984); Symons and Tedd (1989)
11	New Palace Yard	UK	Diaphragm wall (1 m thick)	Top down, multi-propped	10 m of Made Ground/sand and gravel over London Clay		5.5	30	Burland and Hancock (1977); St John (1975); Simpson *et al* (1979)
17	Linsey House		Secant pile wall H/S 0.75 m CFA		0.9 m of Fill over 0.8 m of sand and gravel over London Clay		1.4	21	Arup Report (1989)
18	63 Lincolns Inn Field		Secant pile wall H/S 0.75 m CFA		0.9 m of Fill over 0.8 m of sand and gravel over London Clay		1.4	21	Arup Report (1989)
19	1 Ludgate Place	UK	Contiguous pile wall (0.75 m piles at 0.9 m)		3.4 m of Fill over 0.25 m of Sand and Gravel over London Clay		4.65	21.3	Arup Report (1989)
20	Holborn Bars	London, UK	Secant pile wall		4 m of Fill over 4 m of Sand and Gravel over London Clay		7.5	17	Arup Report (1988)
21	Peterborough Court	UK	Secant pile wall		2.8 m of Fill over 0.5 m of Sand and Gravel over London Clay		3	22	Arup Report (1988)
22	Leith House	UK	Contiguous pile wall (1.2 m)		4.3 m of Fill over 2.5 m Alluvium over 2.1 m of Sand and Gravel over London Clay			14.5	Arup Report (1988)
23	Vintners Place (N. Wall)	UK	Secant pile wall H/C (1.08 m)		7 m of Fill over 4 m of Sand and gravel over London Clay		6.8	30	Arup Report (1990)
24	Blackfriars 1	London, UK	Secant pile wall (1.2 m)		7.3 m of Fill over 1.5 m of Alluvium over 5m of sand and gravel over London Clay		4	22.5	GCG Report (1987)

Table B1.2 *Thompson (1991)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Reference*
25	Blackfriars 2	London, UK	Secant pile wall H/S (0.75 m)		7.3 m of Fill over 1.5 m of Alluvium over 5 m of sand and gravel over London Clay		4	11	GCG Report (1991)
26	Vintners Place (NE. Wall)	UK	Secant pile wall H/C (1.08 m)		6 m of Fill over 6 m of Sand and Gravel over London Clay		4.8	26	Arup Report (1990)
27	Minster Court	UK	Diaphragm wall (0.8 m)		5.6 m of Fill over 4.7 m of sand and gravel over London Clay		9.3	21.45	Arup Report (1988)
28	Aldersgate	UK	Diaphragm wall (1 m)		8 m of Fill over 4 m of Sand and Gravel over London Clay		10	33	Fernie *et al* (1991)

Note: For details of source references, refer to Thompson (1991)

Table B1.3 *Carder (1995)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Reference*
2	East of Falloden Way (A406) (DW)	UK	Diaphragm wall (5 x 1 m)	Cut and cover	21 m of Glacial Till over London Clay	Glacial till			Brookes and Carder (1996)
3	Walthamstow (CF)	UK	Diaphragm wall, counterfort, (4 x 0.8 m front, 3.2 x 1.5 m counterfort)	Low propped	London Clay to Surface	London Clay			Carder *et al* (1994)
4	Bell Common (M25)	UK	Secant pile wall (1.18 m Piles at 1.08 m centres)	Top propped	7 m Older Head and Claygate beds over London Clay		3	20	Tedd *et al* (1984); Symons and Tedd (1989)
5	Hackney Wick (A406)	UK	Secant pile wall (1.2 m Piles at 1.03 m centres)	Cantilever	5 m of Made Ground and terrace gravels over London Clay				Bennett *et al* (1996)
6	Walthamstow (CPW)	UK	Contiguous pile wall (1.5 m piles at 1.7 m centres)	Top propped	1.5 m of Made Ground over London Clay				Carswell *et al* (1993); Watson and Carder (1994)
7	A406/A10 Junction	UK	Diaphragm wall, counterfort, (4 m x 0.8 m front, 2.7 x 0.8 m counterfort)	Low propped	2.4 m of Made Ground / sand and gravel over London Clay				Carder *et al* (1991)
9	Rayleigh Weir	UK	Contiguous pile wall (1.5 m Piles at 1.7 m centres)	Low propped	3 m of Made Ground over London Clay				Darley *et al* (1994)
10	East of Falloden Way (A406) (CPW)	UK	Contiguous pile wall (1.5 m Piles at 2 m centres)	Cantilever	23 m of Glacial Till over London Clay				Brookes and Carder (1996)
11	New Palace Yard	UK	Diaphragm wall (1 m thick)	Top down, multi-propped	10 m of Made Ground/sand and gravel over London Clay			30	Burland and Hancock (1977); St John (1975); Simpson *et al* (1979)
15	Waterloo International Terminal	London, UK	Diaphragm wall (0.8 m thick)	Supported by 5 m berm during excavation of central area, temporary props, then permanent prop cast	8.5 m of Made Ground/alluvial clay/gravel over London Clay				Li *et al* (1992)

Note: For details of source references, refer to Carder (1995)

Table B1.4 *Carder et al (1997)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall Depth (m)*	*Reference*
347	Aldershot Road Underpass	UK	Counterfort Diaphragm Wall	Temporary props with permanent low prop	2 m Made Ground over 2 m sand and gravel over London Clay	London Clay		14.6	Carder *et al* (1997)

Note: For details of source references, refer to Carder *et al* (1997)

B2 GROUND HEAVE

B2.1 Estimation of ground heave movements

Beneath an excavation in clay, heave occurs immediately due to undrained deformation and subsequently as the clay draws in water and swells. Generally, undrained deformation implies shearing at constant volume. However, close to the excavated surface there may be insufficient suction to ensure this, leading to undrained expansion as the clay becomes unsaturated.

The simplest model of soil behaviour to use for estimating deformations is that of a linearly elastic isotropic material with elastic parameters E and ν (or G and k). Movements may be estimated by treating the excavation as a negative load as for an embankment with vertical sides (Poulos and Davis, 1980).

Now,

$$\frac{E'}{2\,(1+\nu')} = G = \frac{E_u}{2\,(1+\nu_u)} \qquad \text{(B.1)}$$

where, E' and E_u are the Young's modulus of elasticity of the ground under long term drained and short term undrained conditions respectively, ν' and ν_u are the values of Poisson's ratio under drained and undrained conditions and G is the shear modulus of the ground. For undrained conditions, $\nu_u = 0.5$ and for drained conditions, $\nu' = 0.1$ to 0.3, typically 0.2 for stiff clays.

For the reasons discussed in chapters 4 and 5, this is not a realistic model. Nevertheless, such a simple model when used in conjunction with appropriate values of stiffness obtained from its use in the back-analysis of well monitored excavations in similar conditions can provide a quick, convenient, but approximate means of estimating the order of magnitude of likely movements. Ho (1991) back analysed four well-instrumented and monitored excavations in London Clay using such a model (VDISP) and estimated:

$\left(E_u / s_u \right)$ = 410 - 465, typically 425

where s_u is the undrained shear strength of the London Clay below excavation level. This ratio of (E_u/s_u) can be used in elastic calculations to provide an approximate estimate of short term ground heave arising from the effects of excavation stress relief in London Clay. Additional heave may occur near the excavated surface owing to undrained expansion, as described above.

It is well established that the Young's modulus of a stiff, overconsolidated clay is highly non-linear and strain dependent. Simpson *et al* (1979), Atkinson and Sallfors (1991) and Mair (1993) show that the range of shear strains that can be expected around a strutted retaining wall in its working condition is typically 0.01% to 0.1%. Similarly, soil behaviour is also highly non-linear (Simpson 1992, Atkinson, 2000). Predictions of ground movements can be made using numerical analyses (eg finite element, finite difference methods) in conjunction with an appropriate soil constitutive model and one which incorporates the likely variation of the soil's stiffness with shear strain and appropriately models the initial in-situ stress conditions in the ground. However, care should be adopted in the use and application of the results of such analyses. It is important that the numerical model and the parameters adopted in analysis are first calibrated through the back-analysis of similar excavations in comparable ground conditions.

B2.2 Case history data

Table B2.1 lists relevant ground heave case histories.

Table B2.1 *Relevant ground heave case histories*

Site No	Site	Location	Soil stratigraphy	Stress Relief (kPa)	Reference
1	QEII conference centre	UK	10 m of London Clay over WR beds	145	Burland and Kalra (1986)
2	Cantley sugar silos	UK	6 m gravel over London Clay	130	Arup (1991)
3	St Pauls, London	UK	London Clay	147	Arup (1991)
4	New Palace Yard	UK	London Clay	370	Arup (1991)
5	Stock Exchange	UK	London Clay	177	Arup (1991)

B3 GROUND MOVEMENTS DUE TO WALL DEFLECTION

B3.1 Overview

Many researchers have published measurements and patterns of wall deflections and ground movements arising from excavation in front of retaining walls, most notably:

- Peck (1969)
- Clough *et al* (1989)
- Clough and O'Rourke (1990)
- St John *et al* (1992)
- Carder (1995)
- Fernie and Suckling (1996)
- Carder *et al* (1997)
- Long (2001).

Lists of the case histories considered by the above researchers are provided in section B3.2.

Peck (1969) summarised data on settlements behind walls. Three different zones were identified, depending upon ground conditions. Generally, where workmanship is average or above average and the soil conditions are not difficult, settlements are indicated not to exceed 1% of the excavation depth. From available case history data of walls embedded in sands and stiff clays, Clough and O'Rourke (1990) showed that the maximum horizontal and vertical ground movements behind these walls were typically less than 0.5% of the excavation depth. These data indicate that there is no significant difference between the maximum movements observed from a range of different wall types, suggesting that the stiff ground conditions may have had a significant effect on wall behaviour.

Clough and O'Rourke (1990) present envelopes of maximum horizontal movements and settlements at ground surface behind different wall types embedded in sand, stiff to hard clay and soft to medium clay. The surface displacements and distance from the wall are expressed as ratios of the maximum excavation depth (*H*) and the distribution of settlement is shown as a proportion of the maximum settlement behind the wall.

Carder (1995) defined three categories of wall support stiffness and considered the performance of the walls comprising his database (section B3.2). These categories are defined in Table 2.3. Carder considered the measured horizontal and vertical movements at ground surface associated with bored pile and diaphragm walls falling into each of the support stiffness categories. He found little difference in wall performance between bored pile walls and diaphragm walls in each of the support stiffness categories.

Observed maximum wall deflections (δ_{hmax}) relating to excavations in London Clay are presented by St John *et al* (1992), see Figure B1.

Figure B1 *Observed maximum lateral wall deflections for excavations in London Clay (after St John et al, 1992)*

For top down construction (high support stiffness), St John *et al* (1992) indicate maximum (δ_{hmax}) values of < 0.2% *H* (typically 0.15% *H*, where *H* is the maximum excavation depth) and *typical* values of about 0.4% *H* for cantilever (low support stiffness) walls.

Fernie and Suckling (1996) considered the horizontal deflections of bored pile walls, diaphragm walls and sheet pile walls in terms of system stiffness as defined in Box 2.4. They found that for a factor of safety against base heave of greater than 3, the lateral deflection of walls wholly embedded in stiff soils was less than 0.3% *H* and that the *typical* value of δ_{hmax} for walls wholly embedded in London Clay in top down conditions was about 0.15% *H*.

Carder *et al* (1997) present wall deflections and ground movements relating to bored pile and diaphragm walls caused by wall installation in stiff clay and subsequent excavation in front of walls which were permanently propped at final formation level only. These correspond to the moderate stiffness category defined by Carder (1995).

Figure 2.11 shows the combined data collated from Clough and O'Rourke (1990), Carder (1995) and Carder *et al* (1997) for surface movements arising from excavation in front of bored pile, diaphragm and sheet pile walls embedded in stiff clays. The data are presented in terms of the high and low support stiffness categories, as defined by Carder (1995).

It is clear from Figure 2.11 (a) that data from two sites fall outside the envelope of movements relating to low support stiffness walls. These data relate to walls constructed at:

- Bell Common
- Neasden.

At both of these sites unusual site specific circumstances dictated the measured ground movements. At Bell Common, temporary sheet piles supporting an adjacent 3.5m deep excavation were propped against the permanent secant pile wall. At Neasden, block movement of London Clay resulted in horizontal displacement of the wall and its tie back anchor supports.

Figure 2.12 shows monitored ground surface settlements arising from excavation in front of walls embedded in sand. The data have been taken from Clough and O'Rourke (1990). Settlements due to wall installation have been excluded. Figure 2.12 indicates maximum surface settlement of less than 0.3% of maximum excavation depth immediately adjacent to the wall decreasing to zero settlement at a lateral distance of some 2.0 times the maximum excavation depth.

Long (2001) extended the database to some 300 case histories (section B3.2) and considered the horizontal deflections of walls embedded in a stiff stratum, but retaining varying amounts of soft ground, against system stiffness. Long found that, for a factor of safety against base heave of about 3 or more, the measured maximum wall (δ_{vmax}) deflections varied depending upon the proportion of soft ground retained by the wall, see Figures B2 (a) - (d). Significant scatter was observed in the data, probably reflecting, in part, local construction problems and variable quality of workmanship.

Figure B2 *Normalised maximum wall deflection versus system stiffness (after Long, 2001)*

Long (2001) reports that where large wall deflections (greater than 0.3% *H*) were observed for walls wholly embedded in stiff soils, they were principally due to:

- movements associated with an initial cantilever stage at the beginning of the construction sequence
- an overly flexible retaining system
- creep of anchorages; and
- structural yielding.

Table B3.1 lists the *average* values of δ_{hmax} as a percentage of maximum excavation depth (*H*) for cases where the observed values of δ_{hmax} were less than 0.3% *H* for walls wholly embedded in stiff soils.

Table B3.1 *Average δ_{hmax} values due to excavation in front of walls embedded in stiff soil for data where δ_{hmax} < 0.3% H (after Long, 2001)*

Wall support	δ_{hmax} (%*H*)	
	$s<0.6H$ stiff soil at dig level	$s>0.6H$ stiff soil at dig level
Top down	0.16%	No data

Notes:

H = maximum excavation depth

s = thickness of soft ground overlying stiff ground

The average δ_{hmax} value of 0.16% H in Table B3.1 is consistent with the findings of St John *et al* (1992) and Fernie and Suckling (1996).

Long (2001) reports average δ_{hmax} values of about 0.4% *H* for cantilever walls wholly embedded in stiff soils (Figure B2). This is consistent with observations reported by St John *et al* (1992) and Carder (1995) for low support stiffness category walls (Figure 2.11a and Table 2.4).

Long (2001) also considered maximum ground surface *settlements* δ_{vmax} behind walls wholly embedded in stiff soils. Average values of δ_{hmax} were found to be < 0.2% *H* for top down (high support stiffness) walls. This is greater than the value of 0.1% *H* reported by other observers (Table 2.4). It is possible that Long's data represent total settlements which include settlements due to wall installation. Long shows that for walls embedded in a stiff stratum with a large factor of safety against base heave that retain a significant thickness of soft soil (>0.6*H*) and have soft soil at formation level, wall deflections and associated ground settlements may increase significantly compared to the case where stiff ground exists at formation level (Figure B2c).

The above indicate that, for walls wholly embedded in stiff soil with a factor of safety of 3 or more against base heave, wall deflections are relatively insensitive to variation in wall thickness and stiffness provided the overall system stiffness is not significantly reduced. This means that economies in wall type and size can be achieved through the adoption of flexible walls (eg sheet pile walls) in stiff soils, without significant increase in ground movements.

B3.2 Case history data

Case histories detailing the ground movements due to wall deflection are listed in the following tables:

- Table B3.1, Clough and O'Rourke (1990)
- Table B3.2, Carder (1995)
- Table B3.3, Fernie and Suckling (1996)
- Table B3.4, Carder *et al* (1997)
- Table B3.5, Long (2001).

Table B3.1 *Clough and O'Rourke (1990)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
1	Neasden Lane Underpass	London, UK	Diaphragm wall (4.57 x 0.6 m)	Anchored	0.5 m of Made Ground over 30 m London Clay	London Clay		13	8	5	1.15	Sills *et al* (1977); Simpson *et al* (1979); St. John (1975); Carswell *et al* (1991)
11	New Palace Yard	UK	Diaphragm wall (1 m thick)	Top down, multi-propped	10 m of Made Ground/sand and gravel over London Clay		5.5	30	18	12		Burland and Hancock (1977); St John (1975); Simpson *et al* (1979)
12	YMCA	London, UK	Diaphragm walls (0.6 m thick)	One anchor, propped by floors top down	7.5 m of Made Ground and Gravel			18	16	2	3.1	St John (1975) Burland *et al* (1979)
29	Churchill Square	Edmonton, Canada	Contiguous pile wall (0.91-1.07 m diameter)	Top down, multi-propped	4.5 m of fill and lake edmonton Clay over 13 m of Glacial till over 5.5 m of Saskatchewan sands over Cretaceous bedrock			28	15	13	7	Eisenstein and Medeiros
30	Columbia Center	Seattle, Washington, USA	Kingpost, 4 m intervals, wood lagging 10-15 cm thick	Multi-propped	11 m of v. stiff clay over 3 m of sand and gravel over 5 m glacial till over interbedded sands, silts, and clays		36	37	31	6	1.7	Grant
31	Houston Buildings	Houston, USA	Kingpost	Anchored	10 m stiff clay over interbedded stiff clay and fine sandy silt	Interbedded stiff clay and fine sandy silt		19.2	16.7	2.5	3.1	Ulrich (1989)
32	1st National Bank	Seattle, Washington, USA	Kingpost, no lagging	Multi –propped	3.04 m of Fill, over 4.87 m of v. stiff silty clay over 11 m of silty sand over 6.4 m of sand over 9.7 m of silty clay over 4.6 m of glacial till			25.22	23.7	1.52	2.7	Shannon and Strazer (1970)
33	OCC Building	Washington, USA	Kingpost	Multi –propped	2 m of Fill over 22 m of Pleistocene terrace deposits of clay and sand over Cretaceous Potomac formation			22	18.3	3.7	3.9	Ware *et al*
34	Chater Station	Hong Kong	Diaphragm wall	Multi -propped top down	9 m of fill over 8 m of marine deposits over 27 m of decomposed granite over granite rock		2	37	27	10	6	Davies and Henkel
35	Hatfield	UK	Sheet pile wall	Anchored, 1 high anchor	3 m of fill over 2 m of Gravel over 2.1 m of sandy gravel over 5.7 m of sand over 2 m of till over 8 m of Gravel/sand over Chalk		8	13	9.3	3.7		Symons *et al* (1999)
36	Bergshamra	Stockholm	King post, wooden lagging	Anchored, using tie rods and rockbolts	1 m of fill over 0.6 m of clay over 7.8 m of moraine over bedrock		1	10.4	10.4	0	2.5	Stille (1976)

Table B3.1 *Clough and O'Rourke (1990)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
37	7th and G sts	USA	Kingpost, lagging	Multi-propped	7.3 m of sand and gravel over 4.6 m of stiff clay over 6.4 m of Dense sand and interbedded stiff clay over 5.2 m of dense sand and gravel over 4 m of hard clay over 10.7 m of dense clayey sand over schistose Gneiss						1.83	O'Rourke *et al*
38	G st test site	USA	Kingpost, lagging	Multi-propped	7.3 m of sand and gravel over 4.6 m of stiff clay over 6.4 m of Dense sand and interbedded stiff clay over 5.2 m of dense sand and gravel over 4 m of hard clay over 10.7 m of dense clayey sand over schistose Gneiss						1.83	O'Rourke *et al*
39	8th and G st	USA	Kingpost, lagging	Multi-propped	7.3 m of medium to dense sand and gravel over 4.6 m of stiff clay over 6.4 m of Dense sand and interbedded stiff clay over 5.2 m of dense sand and gravel over 4 m of hard clay over 10.7 m of dense clayey sand over schistose Gneiss						1.83	O'Rourke *et al*
15	Waterloo International Terminal	London, UK	Diaphragm wall (0.8 m thick)	Supported by 5 m berm during excavation of central area, temporary props, then permanent prop cast	8.5 m of Made Ground/alluvial clay/gravel over London Clay							Li *et al* (1992)
16	Reading (A329(M))	UK	Diaphragm wall (3.66 x 1.22 m)	Cantilever	3 m of Made Ground and terrace gravel over London Clay							Burland *et al* (1979); St John (1975); Carder and Symons (1989)

Note: For details of source references, refer to Clough and O'Rourke (1990)

Table B3.2 *Carder (1995)*

Site No.	*Site*	*Location*	*Wall Type / Dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
1	Neasden Lane Underpass	London, UK	Diaphragm wall (4.57 x 0.6 m)	Anchored	0.5 m of Made Ground over 30 m London Clay	London Clay		13	8	5	1.15	Sills *et al* (1977); Simpson *et al* (1979); St. John (1975); Carswell *et al* (1991)
2	East of Falloden Way (A406) (DW)	UK	Diaphragm wall (5 x 1 m)	Cut and cover	21 m of Glacial Till over London Clay	Glacial till						Brookes and Carder (1996)
3	Walthamstow (CF)	UK	Diaphragm wall, counterfort, (4 x 0.8 m front, 3.2 x 1.5 m counterfort)	Low propped	London Clay to surface	London Clay						Carder *et al* (1994)
4	Bell Common (M25)	UK	Secant pile wall (1.18 m piles at 1.08 m centres)	Top propped	7 m Older Head and Claygate beds over London Clay		3	20				Tedd *et al* (1984); Symons and Tedd (1989)
5	Hackney Wick (A406)	UK	Secant pile wall (1.2 m piles at 1.03 m centres)	Cantilever	5 m of Made Ground and terrace gravels over London Clay							Bennett *et al* (1996)
6	Walthamstow (CPW)	UK	Contiguous pile wall (1.5 m piles at 1.7 m centres)	Top propped	1.5 m of Made Ground over London Clay							Carswell *et al* (1993); Watson and Carder (1994)
7	A406/A10 Junction	UK	Diaphragm wall, counterfort, (4 m x 0.8 m front, 2.7 x 0.8 m counterfort)	Low propped	2.4 m of Made Ground / sand and gravel over London Clay							Carder *et al* (1991)
8	Limehouse Link	London, UK	Diaphragm wall (4.2 x 1 m)	Tunnel portal, cantilever	6 m of Made Ground and terrace gravel over London Clay							Moran and Laimbeer (1994)
9	Rayleigh Weir	UK	Contiguous pile wall (1.5 m piles at 1.7m Centres)	Low propped	3 m of Made Ground over London Clay							Darley *et al* (1997)
10	East of Falloden Way (A406) (CPW)	UK	Contiguous pile wall (1.5 m piles at 2m Centres)	Cantilever	23 m of Glacial Till over London Clay							Brookes and Carder (1996)
11	New Palace Yard	UK	Diaphragm wall (1 m thick)	Top down, multi-propped	10 m of Made Ground/sand and gravel over London Clay		5.5	30	18	12		Burland and Hancock (1977); St John (1975); Simpson *et al* (1979)
12	YMCA	London, UK	Diaphragm walls (0.6 m thick)	One anchor, propped by floors top down	7.5 m of Made Ground and Gravel			18	16	2	3.1	St John (1975); Burland *et al* (1979)
13	Lion Yard	Cambridge, UK	Diaphragm wall (8.5 x 0.6m)	Top down, multi-propped	Gault Clay overlain by 3 m of Made Ground and Gravel							Lings *et al* (1991)

Table B3.2 *Carder (1995)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
14	Britanic House	London, UK	Diaphragm wall (0.8 m thick)	Supported by berm during excavation of central area, temporary struts, then floor cast	2.5 m of sand and gravel over London Clay							Cole and Burland (1972); Burland *et al* (1979)
15	Waterloo International Terminal	London, UK	Diaphragm wall (0.8 m thick)	Supported by 5m berm during excavation of central area, temporary props, then permanent prop cast	8.5 m of Made Ground/alluvial clay/gravel over London Clay							Li *et al* (1992)
16	Reading (A329(M))	UK	Diaphragm wall (3.66 x 1.22 m)	Cantilever	3 m of Made Ground and terrace gravel over London Clay							Burland *et al* (1979); St John (1975); Carder and Symons (1989)

Note: For details of source references, refer to Carder (1995)

Table B3.3 *Fernie and Suckling (1996)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
286	Norwich	UK		Multi prop		Upper chalk			18		2.57	Grose
287	Japan			Multi prop		Stiff clay			13.75		6.88	Maruoka
288	Singapore			Multi prop		Soft clay			13		3.25	Wallace
289	Croydon	UK		Multi prop		London clay			11.4		5	Brooks
290	Holborn	UK		Single prop		London clay			11		8.65	Ward
291	Waterloo	UK		Single prop		Medium gravel			5.78		5.8	Li
292	Barnsley	UK		Single prop		Coal meas			10		7.5	Curtis
293	Japan			Multi prop		Soft clay			17.1		4.3	Sato
294	Detroit	USA		Single prop		Soft clay			7		11.2	Abedi
295	Lyon	France		Multi prop		Stiff clay			8		3.2	Kastner
296	Italy			Multi prop		Soft clay			5.5		7.8	Rampello
297	Buffalo	USA		Multi prop		Dense sand and gravel			11		3.67	Peck
298	Ontario	USA		Multi prop		Very dense sand			15.2		3.04	Bauer
299	Zurich	Switzerland		Single prop		Silt/sand			15.5		13.7	Gysi
300	Oakland	USA		Multi prop		Stiff soil			19		3.8	Peck
301	Seattle	USA		Multi prop		Stiff soil					7.64	Peck
302	Chicago	USA		Multi prop		Soft clay			13.4		4.46	Gnaedinger
303	London	UK		Single prop		London clay			8		5.2	Hodgson
304	Chelsea	UK		Multi prop		London clay			13		4.33	Corbett
305	Guildhall	UK		Multi prop		London clay			6.5		3	Littlejohn
306	Vauxhall	UK		Multi prop		London clay			14.5		3.63	Littlejohn
307	YMCA	UK		Multi prop		London clay			16		10	St.john
308	Charing Cross	UK		Multi prop		London clay			11		2.75	Wood
309	New Palace Yard	UK		Multi prop		London clay			18.5		3.08	Burland
310	A329	UK		Cantilever		Woolwich and Redding Beds			6.9		9.7	Carder
311	Humber Bridge	UK		Multi prop		Kimer clay			29.5		4.92	Busbridge
312	Oslo	Norway		Multi prop		Soft clay			10		1.67	Peck
313	New York	USA		Multi prop		Rock			18.5		3.7	Saxena
314	Houston	USA		Multi prop		Stiff clay			18.3		6.1	Peck
315	Brittanic House	UK		Multi prop		London clay			14		4.67	Cole
316	Neasden	UK		Multi prop		London clay			8		2	Sills
317	Aldersgate	UK		Multi prop		London clay			23		3.3	Fernie
318	Chingford	UK		Single prop		London clay			8		9.2	Carswell
319	Swindon	UK		Cantilever		Kimer clay			4.5		6.3	Fernie and Suckling (1996)
320	Mark Lane	UK		Multi prop		London clay			7		3.5	Fernie and Suckling (1996)
321	Mark Lane	UK		Single prop		London clay			5.5		6.3	Fernie and Suckling (1996)
322	Benwell Rd	UK		Cantilever		London clay			4.5		6.3	Fernie and Suckling (1996)
323	Manchester	UK		Cantilever		Sandstone			5		7	Fernie and Suckling (1996)
324	Manchester	UK		Cantilever		Sandstone			5		7	Fernie and Suckling (1996)
325	Manchester	UK		Single prop		Sandstone			7		8	Fernie and Suckling (1996)

Table B3.3 *Fernie and Suckling (1996)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
326	Edinburgh	UK		Cantilever		Coal meas			6		8.4	Fernie and Suckling (1996)
327	JLE 111	UK		Single prop		London clay			8		8.2	Fernie and Suckling (1996)
328	Eastbourne	UK		Single prop		Alluvial clay			11		17.6	Fernie and Suckling (1996)
329	Eastbourne	UK		Single prop		Alluvial clay			14		22.4	Fernie and Suckling (1996)
330	Bell Common	UK		Single prop		London clay			9		8.9	Tedd
331	A1(m)	UK		Single prop		Medium sand			9.3		10.6	Symons
332	Cambridge	UK		Multi prop		Gault clay			10		3.3	Ng
333	Singapore			Multi prop		Stiff clay			12.9		2.58	Tan
334	Switzerland			Multi prop		Moraine			17.15		4.29	Huder
335	Chicago	USA		Multi prop		Soft/medium clay			8		2	O'Rourke
336	Chicago	USA		Multi prop		Soft/medium clay			9		3	O'Rourke
337	Washington	USA		Multi prop		Hard clay			17		11	O'Rourke
338	Washington	USA		Multi prop		Hard clay			25		6.25	O'Rourke
339	Chicago	USA		Multi prop		Soft/medium clay			8		2.67	O'Rourke
340	Chicago	USA		Multi prop		Soft/medium clay			8.5		2.83	Clough
341	Boston	USA		Multi prop		stiff clay			19		3.8	Clough
342	San Fransisco	USA		Cantilever		Soft clay			10		23.5	Clough
343	San Fransisco	USA		Multi prop		Soft clay			11		10	Clough
344	San Fransisco	USA		Multi prop		Soft clay			22		2.75	Clough
345	Victoria embankment	UK		Multi prop		London clay			18		6	Stjohn
346	San Fransisco	USA		Multi prop		Soft clay			13.7		3.43	O'Rourke

Note: For details of source references, refer to Fernie and Suckling (1996)

Table B3.4 *Carder, Press et al (1997)*

Site No.	*Site*	*Location*	*Wall Type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
347	Aldershot Road Underpass	UK	Counterfort diaphragm wall	Temporary props with permanent low prop	2 m Made Ground over 2 m sand and gravel over London Clay	London Clay		14.6	10	4.6	4	Carder *et al* (1997)

Note: For details of source references, refer to Carder *et al* (1997)

Table B3.5 *Long(2001)*

Site No.	Site	Location	Wall Type / dimensions	Construction sequence / support system	Retained soil stratigraphy	Restraining soil	Depth to water table (m)	Wall depth (m)	Excavation depth (m)	Embedment depth (m)	Support spacing (m)	Reference
40	Croydon		Secant	Multi-prop	2 m of soft soil above hard soil	London clay		11.4			5	Brooks and Spence (1992)
41	Holborn		Secant	Single prop	3.5 m of soft soil above hard soil	London clay		11			8.65	Ward (1992)
27	Minster Court		Diaphragm	Props and berm	5 m of soft soil above hard soil	London clay		9			7.3	Tse and Nicholson (1992)
43	Britannic Hse		Diaphragm	Props and berm	1 m of soft soil above hard soil	London clay		14			4.67	Cole and Burland (1972)
44	Chelsea		Diaphragm	Props and berm	4 m of soft soil above hard soil	London clay		13			4.33	Corbett *et al*. (1975)
45	Walthamstow		Secant	Multi-prop	1.4 m of soft soil above hard soil	London clay		7.9			7.4	Watson and Carder (1994)
46	Barbican		Diaphragm	Multi-prop	2 m of soft soil above hard soil	London clay		16			4	Stevens *et al*. (1977)
47	Charing Cross		Diaphragm	Multi-prop	5 m of soft soil above hard soil	London clay		11			2.75	Wood and Perrin (1984)
48	John Lewis KUT		Diaphragm	Props and berm	2.5 m of soft soil above hard soil	London clay		12			3.8	Long (1989)
49	Victoria Emb		Secant	Props and berm	6 m of soft soil above hard soil	London clay		18			6	St. John *et al*. (1992)
50	London		Diaphragm	Single prop		London clay		8			5.2	Fernie and Suckling (1996)
51	Guildhall		Diaphragm	Single prop		London clay		6.5			3	Fernie and Suckling (1996)
52	Vauxhall		Diaphragm	Multi-prop		London clay		14.5			3.63	Fernie and Suckling (1996)
53	Chingford		Secant	Single prop		London clay		8			9.2	Fernie and Suckling (1996)
54	Mark Lane		Contiguous	Multi-prop		London clay		7			3.5	Fernie and Suckling (1996)
54	Mark Lane		Contiguous	Single prop		London clay		5.5			6.3	Fernie and Suckling (1996)
56	JLE HI		Diaphragm	Single prop		London clay		8			8.2	Fernie and Suckling (1996)
57	Malden Way		Contiguous	Single prop		London clay		7.5			5.5	Symons and Carder (1991)
58	Bermondsey		Diaphragm	Multi-prop		Woolwich and Reading beds		19.5			6	Dawson *et al*. (1996)
59	Canada Water		Secant	Multi-prop	7 m of soft soil above hard soil	Woolwich and Reading beds		17			5.7	Powrie and Batten (1997)
60	Humber Bridge		Diaphragm	Multi-prop		Kimmer clay		24.5			4.92	Busbridge (1974)
61	Cambridge		Diaphragm	Multi-prop	2 m of soft soil above hard soil	Gault clay		10			3.3	Ng and Lings (1995)
62	Channel Tunnel		Sheet piles	Multi-prop		Gault clay		6.5			4	Young and Ho (1994)
63	Lyon		Diaphragm	Multi-prop		Stiff clay		8			3.2	Kastner and Ferrand (1992)
64	Switzerland		Diaphragm	Multi-prop		Morraine		17.2			4.29	Huder (7)
65	Dublin-Jervis		Secant	Single prop	3 m of soft soil above hard soil	Glacial till		9.7			8.5	Long (1997)
66	Dublin-Clarend		Soldier pile	Single prop	1 m of soft soil above hard soil	Glacial till		6.2			5	Long (1997)
67	Dublin-MandS		Sheet piles	Single prop	3 m of soft soil above hard soil	Glacial till		7.2			6	Long (1997)
68	MBTA. Boston		Diaphragm	Multi-prop		Stiff clay		15.2			3.36	Becker and Haley (1990)
69	Oakland		Sheet piles	Multi-prop		Stiff clay		19			3.36	Peck (1969)

Table B3.5 *Long(2001)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
70	Houston		?	Multi-prop		Stiff clay		18.3			6.1	Peck (1969)
71	Seattle		Sheet piles	Multi-prop		Stiff clay		23.8			2.64	Peck (1969)
72	West. Station, Seattle		Contiguous	Multi-prop		Stiff clay		15.2			3.8	Borst *et al.* (1990)
73	Pion. Square, Seattle		Secant	Multi-prop		Stiff clay		21.9			5.5	Borst *et al.* (1990)
74	Washington		Piled	Multi-prop		Hard clay		17			11	O'Rourke (1992)
74	Washington		Sheet piles	Multi-prop		Hard clay		25			6.25	O'Rourke (1992)
74	Washington		Contiguous	Multi-prop	4.5 m of soft soil above hard soil	Stiff clay		15			7.5	Eisenstein and Medeiroz (1983)
77	Houston-Exxon		Soldier piles	Multi-prop		Stiff clay		16.2			6.5	Ulrich (1989a)
78	Houston-1Shell		Contiguous	Multi-prop	3 m of soft soil above hard soil	Stiff clay		18			6	Ulrich (1989a)
79	Houston-Cokex		Sheet piles	Multi-prop		Stiff clay		17.1			3.5	Ulrich (1989a)
80	Tiong Bahru		Soldier piles	Multi-prop	1.5 m of soft soil above hard soil	Stiff clay		15.3			4.6	Leonard *et al.* (1987)
81	Singapore CE II		Soldier piles	Multi-prop		Stiff clay		9			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multi-prop		Stiff clay		9			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multi-prop		Stiff clay		9			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multi-prop		Stiff clay		10.2			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multi-prop		Stiff clay		10.5			4	Wong *et al.* (1997)
86	Singapore CE U		Soldier piles	Multi-prop		Stiff clay		11.5			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multi-prop		Stiff clay		11.5			4	Wong *et al.* (1997)
86	Singapore CE U		Sheet and H	Multi-prop		Stiff clay		10			4	Wong *et al.* (1997)
86	Singapore CE U		Sheet and H	Multi-prop		Stiff clay		10			4	Wong *et al.* (1997)
86	Singapore CE U		Sheet and H	Multi-prop		Stiff clay		10			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		10			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		11			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		11.5			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		11.5			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		12			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		17			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		17			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		11			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		11			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		11.5			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		12			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		12			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		12.5			4	Wong *et al.* (1997)
86	Singapore CE U		Contiguous	Multi-prop		Stiff clay		13.5			4	Wong *et al.* (1997)
86	Singapore CE U		Contiguous	Multi-prop		Stiff clay		13.5			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		13.5			4	Wong *et al.* (1997)
86	Singapore CE U		Contiguous	Multi-prop		Stiff clay		20			4	Wong *et al.* (1997)
86	Singapore CE U		Contiguous	Multi-prop		Stiff clay		21.5			4	Wong *et al.* (1997)
86	Singapore CE U		Diaphragm	Multi-prop		Stiff clay		13.5			4	Wong *et al.* (1997)
86	Singapore CE U		Diaphragm	Multi-prop		Stiff clay		14.5			4	Wong *et al.* (1997)
111	Waterloo		Diaphragm	Props+berm	2 m of soft soil above hard soil	Medium gravel		5.78			5.8	Li *et al.* (1992)
112	Eastbourne		Diaphragm	Multi-prop		Gravel/sand		11			7	Fernie *et al.* (1996)

Table B3.5 *Long(2001)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
113	Buffalo		Sheet piles	Multi-prop		Dense Sand and Gravel		11			3.67	Peck (1969)
114	Ontario		Sheet piles	Multi-prop		Very Dense sand		15.2			3.04	Bauer (7)
115	Zurich		Diaphragm	Single prop		Silt/sand		15.5			13.7	GYSI (7)
116	Lyon-P Kieb		Sheet piles	Single prop	3.5 m of soft soil above hard soil	Sandy gravel		6.75			6.75	Kastner (1982)
117	Lyon-R Ney		Diaphragm	Multi-prop	6.5 m of soft soil above hard soil	Sandy gravel		9.95			3.1	Kastner (1982)
118	Lyon-S. Gam.		Diaphragm	Single prop	3 m of soft soil above hard soil	Sandy gravel		10.7			5.2	Kastner (1982)
119	Karlshrue		Sheet piles	Single prop		Sands		5			3.75	Josseaume *et al*. (1997)
120	Maas, Rotterdam		Sheet and tubes	Single prop		Silts/sands		21			17	Bakker and Brinkgrieve (1991)
121	Cairo Metro		Precast	Multi-prop	6 m of soft soil above hard soil	Clay/sands		10			3.3	El-Nahhas and Eisenstein (1989)
122	Lisbon-Carlos		Diaphragm	Multi-prop	5.5 m of soft soil above hard soil	Clay/sands		13.8			2.76	Mattos-Fernandes (1985)
123	Hannover		Secant	Multi-prop		Sand, marl, silt		11.5			2.6	Rizkallah and Vogel (1992)
123	Hannover		Secant	Multi-prop		Sand, gravel, clay		21.9			6.6	Blümel and Wemheuer (1980)
125	Duisburg		Soldier pile	Multi-prop		Gravel, sand, silt		23			4.8	Hettler *et al*. (1997)
126	Offenbach		Secant	Multi-prop		Clay		19.3			6.4	Krajewski *et al*. (1997)
127	Lubeck		Secant	Multi-prop		Silt		9.6			4.25	Rodatz *et al*. (1996)
128	Salsburg		Soldier piles	Single prop		Gravel, clay		10			5	Breymann (1992)
129	Bruckmuhl		Soldier piles	Single prop		Sand, marl		14.1			7	Viendenz (1984)
130	Sao Paulo, ES1		Soldier pile	Multi-prop		Residual soil		9			3.6	Massad (1985)
131	Sao Paulo, ES2		Soldier pile	Multi-prop		Residual soil		19			3.8	Massad (1985)
132	Argyle Station, HK		Diaphragm	Multi-prop	7 m of soft soil above hard soil	Residual soil		18.7			6.2	Morton *et al*. (1980)
133	Manchester		Contiguous	Single prop		Sandstone		7			8	Fernie and Suckling (1996)
134	New York		Piled	Multi-prop		Rock		18.5			3.7	Saxena (7)
135	Han River, Seoul		Secant/H	Multi-prop	13.5 m of soft soil above hard soil	Weathered rock		25			2:78	Choi and Lee (1998)
136	YMCA, London		Diaphragm	Multianch	1 m of soft soil above hard soil	London clay		16			10	St. John (1975)
137	Neasden		Diaphragm	Multianch	1 m of soft soil above hard soil	London clay		8			2	Sills *et al*. (1977)
138	Oresund-Sydh		Soldier pile	Single anch	5 m of soft soil above hard soil	Boulder clay		10			7	Hess *et al*. (1997)
139	Copenhagen		Tubes/H	Single anch	1 m of soft soil above hard soil	Stiff clays		11			5.5	Duc Long and Bredenberg (2000)
140	Lisbon-DD Ave.		Soldier pile	Multianch	2.6 m of soft soil above hard soil	Stiff clay		17			3.25	Correia and da Costa Guerra (1997)
141	Lisbon-Colom		Soldier pile	Multianch	4 m of soft soil above hard soil	Stiff clay		14			3.25	Correia and da Costa Guerra (1997)
142	Lisbon-ivens		Soldier pile	Multianch		Stiff clay		9			3.25	Correia and da Costa Guerra (1997)
143	Colomb., Seattle		Piled	Multianch		Stiff clay		37			1.75	Grant (1985)/

Table B3.5 *Long(2001)*

Site No.	Site	Location	Wall type / dimensions	Construction sequence / support system	Retained soil stratigraphy	Restraining soil	Depth to water table (m)	Wall depth (m)	Excavation depth (m)	Embedment depth (m)	Support spacing (m)	Reference
144	University Street		Contiguous	Multianch		Stiff clay		18.3			2.6	Borst *et al.* (1990)
145	Station, Seattle				m of soft soil above hard soil							
71	Seattle		Soldier pile	Multianch		Stiff clay		23			1.5	Winter (1990)
147	Houston-Herm		Contiguous	Multianch		Stiff clay		9			3	Ulrich (1989b)
148	Houston-Bank		Contiguous	Multianch	3m of soft soil above hard soil	Stiff clay		16.8			3.35	Ulrich (1989b)
149	Houston-FCB		Contiguous	Multianch		Stiff clay		9.1			8	Ulrich (1989b)
150	Houston-Smith		Contiguous	Multianch		Stiff clay		15.5			3	Ulrich (1989b)
151	Houston-Texas		Contiguous	Multianch	3 m of soft soil above hard soil	Stiff clay		16			3.2	Ulrich (1989b)
152	Houston-Cullen		Contiguous	Multianch		Stiff clay		8.2			7	Ulrich (1989b)
153	Houston-321		Contiguous	Multianch		Stiff clay		9.1			3	Ulrich (1989b)
74	Washington		Soldier	Multianch	3 m of soft soil above hard soil	Stiff clay		15.2			3	Ware *et al.* (1973)
155	State Trans, Boston		Diaphragm	Multianch		Stiff clay		9.1			3.36	Becker and Haley (1990)
156	60 State St., Boston		Diaphragm	Multianch		Stiff clay		9.1			3.36	Becker and Haley (1990)
157	Davis Square, Boston		Diaphragm	Multianch		Stiff clay		17.1			3.36	Becker and Haley (1990)
158	1 Memorial, Boston		Diaphragm	Multianch		Stiff clay		8.2			3.36	Becker and Haley (1990)
159 160	Harvard Square, Boston		Diaphragm	Multianch		Till		15.7			5.2	Hansmire *et al.* (1989)
159	Harvard Square,		Diaphragm	Multianch		Till		15.7			6.2	Hansmire *et al.* (1989)
160	Boston				m of soft soil above hard soil							
163	Boston		Soldier pile	Multianch	2 m of soft soil above hard soil	Stiff clay		18.9			2.7	Houghton and Dietz (1990)
164	Salt Lake City		Soldier pile	Multianch	3 m of soft soil above hard soil	Stiff clay		12.5			3.2	Cliendo *et al.* (1990)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		12.2			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		12.5			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		12.5			4	Wong *et al.* (1997)
86	Singapore CE U		Soldier piles	Multianch		Stiff clay		16			4	Wong *et al*, (1997)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		15			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		17			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		18.5			4	Wong *et al.* (1997)
81	Singapore CE II		Soldier piles	Multianch		Stiff clay		19			4	Wong *et al.* (1997)
173	A1(M)		Sheet piles	Single anch		Sand		9.3			10.6	Symons (7)
35	Hatfield		Sheet piles	Single anch	3 m of soft soil above hard soil	Gravels		9.3			6.8	Symons *et al.* (1988)
175	Lisbon		Diaphragm	Multianch	5.5 m of soft soil above hard soil	Sand		13.8			2.75	Fernandes (1985)
176	Paris-R Gau		Sheet	Single anch	4 m of soft soil above hard soil	Sand/gravel		12.3			8.3	Gignan (1984)
177	Paris-13e		Diaphragm	Multianch		Sands		17.4			4.35	Josseaume and Stenne (1979)
178	Calais		Diaphragm	Multianch	4.5 m of soft soil above hard soil	Sands		24			8	Delattre *et al.* (1995)
179	Le Havre		Diaphragm	Single anch	9 m of soft soil above hard soil	Sand/gravel		16.5			10.5	Maquet (1981)
180	Geneva, Le Mail		Diaphragm	Multianch	4 m of soft soil above hard soil	Sand/gravel		14.8			2.5	Monnet *et al.* (1994)

Table B3.5 *Long(2001)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
181	Berlin P Platz DB		Diaphragm	Single anch	3 m of soft soil above hard soil	Sands		18			15	Triantafyllidis *et al*. (1997)
182	Berlin-Hofgarten		Diaphragm	Multianch	3 m of soft soil above hard soil	Sands		17			2.83	Nussbaumer (1998)
183	Berlin		Soldier piles	Multianch		Sands		18.5			0.75	Triantafyllidis (1998)
183	Berlin		Secant	Multianch		Sands		12.3			3.6	Weibenbach and Gollub (1995)
185	SONY, Berlin		Diaphragm	Multianch	5 m of soft soil above hard soil	Sand/gravel		14.3			3.58	Kudella and Mayer (1998)
186	Salzburg		Soldier piles	Multianch		Gravel, clay		11			4	Breymann (1992)
186	Salzburg		Soldier piles	Multianch		Gravel, clay		11.5			4.9	Breymann (1992)
188	Urreiting		Soldier piles	Multianch		Gravel, sand		18.3			5.5	Naderer (1988)
189	Fr'hafen		Secant	Multianch		Gravel, clay		12.5			2.9	Ostermayer (1983)
190	Wien		Soldier piles	Multianch		Sand, marl		18.5			2.9	Pötscher *et al*. (1984)
191	Dusseldorf		Soldier piles	Multianch		Sand, gravel		19.8			3.5	Ulrichs (1980)
192	Dusiburg		Soldier piles	Multianch		Sand, gravel		20			2.9	Ulnchs (1982)
193	Grauholz, Switzerland		Contiguous	Multianch		Sands/silts		17			2.4	Steiner and Werder (1991)
194	Johannsburg, South		Soldier piles	Multianch		Firm silt		18.3			3.05	Day (1990)
195	Africa											
196	Milwaukee		Deep soil mix	Multianch		Sands		5			1.67	Anderson (1998)
197	Norwich		Contiguous	Multianch	1.2 m of soft soil above hard soil	Chalk		18			2.57	Grose and Toone (1992)
198	Dartford		Barrettes	Multianch		Chalk		9			2	Wood *et al*. (1989)
199	Bell Common		Secant	Top down	4 m of soft soil above hard soil	London clay		9			8.9	Tedd *et al*. (1984)
11	New Palace Yard		Diaphragm	Top down	2 m of soft soil above hard soil	London clay		18.5			3.08	Burland and Hancock (1977)
201	British Library		Secant	Top down	3 m of soft soil above hard soil	London clay		24.4			5	Simpson (1992)
202	Nat Gal Ext		Secant	Top down	4.2 m of soft soil above hard soil	London clay		10			7	Long (1989)
28	Aldersgate		Diaphragm	Top down	8 m of soft soil above hard soil	London clay		23			3.3	Fernie *et al*. (1991)
204	Limehouse		Diaphragm	Top down	4 m of soft soil above hard soil	Woolwich and Reading beds		16			4	Stevenson and De Moor (1994)/De Moor and Stevenson (1996)
205	P0 Square Boston		Diaphragm	Top down	4 m of soft soil above hard soil	Till		23.4			3.3	Whittie *et al*. (1993)
206	HKandS Bank, HK		Diaphragm	Top down	5 m of soft soil above hard soil	Decomposed. granite		16			4	Humpheson *et al*. (1986)
207	Charter Station, HK		Diaphragm	Top down	14 m of soft soil above hard soil	Decomposed granite		26			4	Davies and Henkel (1980)
208	Hong Kong		Soldier piles	Top down		Decomposed granite		28			4.7	Triantafyllidis (1996)
209	Rowes Whr, Boston		Diaphragm	Top down	5 m of soft soil above hard soil	Stiff clay		16.8			3.36	Becker and Haley (1990)
210	75 State St., Boston		Diaphragm	Top down	3 m of soft soil above hard soil	Stiff clay		19.8			3.36	Becker and Haley (1990)
211	125 Sum. St., Boston		Diaphragm	Top down		Stiff clay		18.3			3.36	Becker and Haley (1990)
186	Salzburg		Soldier piles	Top down		Gravel, clay		11.5			3.8	Breymann (1992)

Table B3.5 *Long(2001)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
190	Wien		Soldier piles	Top down		Sand, silt		24			6.3	Fross and Klenovec (1992)
121	Cairo Metro		Diaphragm	Top down	3.5 m of soft soil above hard soil	Clay/sands		10			5	El-Nahhas and Eisenstein (1989)
215	Singapore River		Diaphragm	Multi-prop	20 m of soft soil above hard soil	Stiff clay		29			3.3	Gronin *et al.* (1991)
216	Singapore Havelock		Diaphragm	Multi-prop	16.5 m of soft soil above hard soil	Stiff clay		16.5			4	Gronin *et al.* (1991)
217	Singapore CBD		Sheet piles	Multi-prop	14 m of soft soil above hard soil	Stiff clay		14.7			2.1	Broms *et al.* (1986)
81	Singapore CE II		Diaphragm	Multi-prop		Stiff clay		10			4	Wong *et al.* (1997)
86	Singapore CE U		Sheet and H	Multi-prop		Stiff clay		10			4	Wong *et al.* (1997)
81	Singapore CE II		Sheet and H	Multi-prop		Stiff clay		11			4	Wong *et al.* (1997)
86	Singapore CE U		Contiguous	Multi-prop		Stiff clay		14			4	Wong *et al.* (1997)
86	Singapore CE U		Arbed	Multi-prop		Stiff clay		15			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		17			4	Wong *et al.* (1997)
86	Singapore CE U		Sheet and H	Multi-prop		Stiff clay		17			4	Wong *et al.* (1997)
86	Singapore CE U		Arbed	Multi-prop		Stiff clay		17.5			4	Wong *et al.* (1997)
81	Singapore CE II		Contiguous	Multi-prop		Stiff clay		18.5			4	Wong *et al.* (1997)
86	Singapore CE U		Arbed	Multi-prop		Stiff clay		19			4	Wong *et al.* (1997)
228	Singapore multistory		Diaphragm	Multi-prop	17.3 m of soft soil above hard soil	Clayey sand		17.3			2.88	Lee *et al.* (1998)
229	Singapore interchange		Contiguous	Multi-prop	15 m of soft soil above hard soil	Stiff clay		20			4	Vuillemin and Wong (1991)
230	Singapore canal		Sheet	Single prop	6 m of soft soil above hard soil	Stiff clay		6.5			5	Wong and Chua (1999)
231	Bangkok B		Diaphragm	Multi-prop	15 m of soft soil above hard soil	Stiff clay		15.5			5.1	Balasubramaniam *et al.* (1991)
232	Bangkok D		Diaphragm	Multi-prop	14 m of soft soil above hard soil	Stiff clay		16			3.2	Balasubramaniam *et al.* (1991)
233	Oslo Gronland 1		Sheet	Multi-prop	11.5 m of soft soil above hard soil	Rock		11.5			4.5	NGI (1962a)
234	Oslo Tech School		Sheet	Multi-prop	6 m of soft soil above hard soil	Rock		6			2	NGI (1962b)
235	Oslo Telephone		Diaphragm	Multi-prop	18.5 m of soft soil above hard soil	Rock		18.5			6.1	DiBiagio and Roti (1972)
236	Oslo Olav Kyrres		Sheet	Multi-prop	18.5 m of soft soil above hard soil	Rock		18.5			2	Karlsrud and Myrvall (1976a, b)/Karlsrud (1986)
237	Oslo City		Sheet	Multi-prop	19 m of soft soil above hard soil	Rock		19			2.5	Bruskeland (1991)
238	Nieuw Maas		Diaphragm	Multi-prop	19.5 m of soft soil above hard soil	Sand		19.5			6	Van Tol and Brassinga (1991)
239	Japan 1		Sheet	Multi-prop	12 m of soft soil above hard soil	Stiff clay		13.75			6.88	Fernie and Suckling (1996)
240	Sheung Wan HK		Diaphragm	Multi-prop	19 m of soft soil above hard soil	CDG		30			2.73	Fraser (1992)
86	Singapore CE U		Diaphragm	Multianch		Stiff clay		12			4	Wong *et al.* (1997)
242	Quai Gloria France		Diaphragm	Single anch	9 m of soft soil above hard soil	Sands		12			10	Vezinhet *et al.* (1989)

Table B3.5 *Long(2001)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
243	Hartford, Conn.		Soldier pile	Single anch	13.5 m of soft soil above hard soil	Dense gravels		7			5	Murphy *et al*. (1975)
244	UOB Singapore		Diaphragm	Multi-prop	30 m of soft soil above hard soil	Soft clay		13			2.6	Wallace *et al*. (1992)
245	H'Fok A Singapore		Sheet piles	Multi-prop	19 m of soft soil above hard soil	Soft clay		7.3			1.83	Davies and Walsh (1983)
246	CTC Singapore		Sheet piles	Multi-prop	37 m of soft soil above hard soil	Soft clay		12			2	Lee *et al*. (1985)
247	Somerset Singapore		Diaphragm	Multi-prop	10 m of soft soil above hard soil	Peats/silts		15.2			3.8	Leonard *et al*. (1987)
248	MOE 12 Singapore		Sheet piles	Multi-prop	24 m of soft soil above hard soil	Soft clay		6.8			1.7	Tan *et al*. (1985)
249	MOE 19 Singapore		Sheet piles	Multi-prop	12 m of soft soil above hard soil	Soft clay		6.4			1.6	Tan *et al*. (1985)
250	Singapore Bugis		Diaphragm	Multi-prop	30 m of soft soil above hard soil	Soft clay		18.3			2.29	Hulme *et al*. (1989)
217	Singapore CBD		Sheet piles	Multi-prop	17 m of soft soil above hard soil	Soft clay		15			2.5	Broms *et al*. (1986)
252	Singapore Parking		Diaphragm	Single prop	12 m of soft soil above hard soil	Soft clay		9.5			5.9	Vuillemin and Wong (1991)
253	Taiwan Airline		Contiguous	Multi-prop	9 m of soft soil above hard soil	Firm clay		9.6			2.4	Ou *et al*. (1993)
254	Taiwan Power		Diaphragm	Multi-prop	15 m of soft soil above hard soil	Firm clay		16.2			3.24	Ou *et al*. (1993)
255	Taiwan Quen M		Diaphragm	Multi-prop	8 m of soft soil above hard soil	Firm clay		10.7			2.68	Ou *et al*. (1993)
256	Taiwan Tax		Sheet	Multi-prop	8 m of soft soil above hard soil	Soft clay		7.65			1.91	Ou *et al*. (1993)
257	Taiwan Formosa		Diaphragm	Multi-prop	20 m of soft soil above hard soil	Soft clay		18.45			2.64	Ou *et al*. (1993)
258	Taiwan Cathay		Diaphragm	Multi-prop	12 m of soft soil above hard soil	Firm clay		21			2.63	Ou *et al*. (1993)
259	Bangkok A		Diaphragm	Multi-prop	15 m of soft soil above hard soil	Soft clay		9.8			3.1	Balasubramaniam *et al*. (1991)
260	Bangkok C		Diaphragm	Multi-prop	12 m of soft soil above hard soil	Soft clay		18.5			4.6	Balasubramaniam *et al*. (1991)
231	Bangkok B		Sheet	Multi-prop	12 m of soft soil above hard soil	Soft clay		7.2			1.8	Balasubramaniam *et al*. (1991)
262	Oslo Vaterland 1		Sheet	Multi-prop	16 m of soft soil above hard soil	Soft clay		11			2	NGI (1962f)
263	Oslo Vaterland 2		Sheet	Multi-prop	16 m of soft soil above hard soil	Soft clay		11			2	NGI (1962g)
264	Oslo Studenterlu		Diaphragm	Multi-prop	37 m of soft soil above hard soil	Soft clay		16			5.3	Karlsrud (1981, 1983, 1986)
265	Oslo Jerbanetorget		Diaphragm	Multi-prop	35 m of soft soil above hard soil	Soft clay		10			5	Karlsrud (1981, 1983)
266	Oslo Bank of Norway		Diaphragm	Multi-prop	18 m of soft soil above hard soil	Soft clay		16			3.2	Roti and Friis (1985)

Table B3.5 *Long(2001)*

Site No.	*Site*	*Location*	*Wall type / dimensions*	*Construction sequence / support system*	*Retained soil stratigraphy*	*Restraining soil*	*Depth to water table (m)*	*Wall depth (m)*	*Excavation depth (m)*	*Embedment depth (m)*	*Support spacing (m)*	*Reference*
267	Eastbourne 1		Diaphragm	Single prop	15 m of soft soil above hard soil	Soft clay		11			10	Fernie and Suckling (1996)
268	Eastbourne 2		Diaphragm	Single prop	15 m of soft soil above hard soil	Soft clay		14			13	Fernie and Suckling (1996)
269	Pietrafitta, Italy		Sheet	Multi-prop	20 m of soft soil above hard soil	Soft to firm clay		5.5			7.8	Rampello *et al.* (1992)
270	Chicago		Sheet	Multi-prop	15 m of soft soil above hard soil	Soft clay		13.4			4.46	Fernie and Suckling (1996)
271	Inland Steel Chicago		Sheet	Multi-prop	19 m of soft soil above hard soil	Soft clay		11			2	Flaate (1966)
272	Sewage Tr. Tokyo		Steel pipe pile	Multi-prop	30 m of soft soil above hard soil	Soft clay		26			4.3	Tominaga *et al.* (1985)
273	Osaka A		Diaphragm	Multi-prop	25 m of soft soil above hard soil	Soft clay		20.6			3	Tamano *et al.* (1996)
274	Japan 2		Steel pipe pile	Multi-prop	20 m of soft soil above hard soil	Soft clay		17.1			4.3	Fernie and Suckling (1996)
275	Lake zone, Mexico		Diaphragm	Multi-prop	20 m of soft soil above hard soil	Soft clay		15.7			2.62	Auvinet and Organista (1998)
276	Shanghi-Jin Mao		Diaphragm	Multi-prop	36 m of soft soil above hard soil	Soft/firm clay		19.65			3.93	Zhao *et al.* (1999)
277	Shanghi-Heng Long		Diaphragm	Multi-prop	29 m of soft soil above hard soil	Soft/firm clay		18.2			3.64	Zhao *et al.* (1999)
278	Shanghi		Diaphragm	Multi-prop	24 m of soft soil above hard soil	Soft/firm clay		17.85			3.57	Onishi and Sugawara (1999)
279	River Wall M'Boro		Sheet	Single anch	12 m of soft soil above hard soil	Soft clay		9.5			4.25	Baggett and Butting (1977)
280	Detroit		Sheet	Single anch	10 m of soft soil above hard soil	Soft clay		?			11.2	Fernie and Suckling (1996)
281	TP, Bogota		Diaphragm	Multianch	34 m of soft soil above hard soil	Soft clay/silt		16			3.75	Maldonado (1998)
282	Newton Singapore		Diaphragm	Top down	12 m of soft soil above hard soil	Soft clays		14.5			3.63	Nicholson (1987)
283	Taiwan Chi Ching		Diaphragm	Top down	15 m of soft soil above hard soil	Soft clays		13.9			2.78	Ou *et al.* (1993)
284	Taiwan Far East		Diaphragm	Top down	24 m of soft soil above hard soil	Firm clay		20			3.33	Ou *et al.* (1993)
285	Oslo Christiana		Sheet	Top down	23 m of soft soil above hard soil	Soft clay		9.6			3	Finstad (1991)

Note: For details of source references, refer to Long (2001)

B3.3 Corner effects

The shape of an excavation will affect the magnitude and distribution of ground movements around it. This is illustrated by contours of ground surface settlements measured for excavations in stiff London Clay at New Palace Yard (Burland and Hancock, 1977) and in coarse grained soils at Yen Chow Street, Hong Kong (Lui and Yau 1995), see Figure B3.

Figure B3 *Ground surface settlement contours at New Palace Yard, London (after Burland and Hancock, 1977)*

The corners of the excavation tend to restrict movement. A number of researchers have considered this effect, notably St John (1975), Ou *et al* (1996), Ou and Shiau (1998), Simic and French (1998) and Lee *et al* (1998). Two dimensional and 3-dimensional numerical analyses have been carried out to study details of behaviour. Due to the specific nature of such studies, their extrapolation for general application is limited. However, some useful findings have been made. These are discussed below.

St John (1975) compared the results of finite element analyses assuming plane strain and axi-symmetric conditions for an unsupported excavation which was square in plan with depth equal to one third of the length of one side. He assumed uniform elastic soil with initial hydrostatic groundwater pressures. The results of the analysis, which are also reported by Burland *et al* (1979), indicated a significant reduction in the horizontal ground movements towards the corners of the excavation. The plane strain and axi-symmetric analyses gave similar vertical movements. The horizontal movements from the axi-symmetric model were some 50% of those computed in the plane strain model.

Figure B4 *Plan of the Hai-Hua building (after Ou et al, 1996)*

Ou *et al* (1996) report a study related to the excavation for the Hai-Hua building, Taipei, shown in plan in Figure B4. They carried out a 2-dimensional finite element analysis of the main section and a 3-dimensional analysis of the corner section of the building, and developed correlations for wall deflections between the 2-dimensional and 3-dimensional results. The ground conditions were principally firm clays and the parameters were developed for a Duncan and Chang model (elastic-plastic material described by a hyperbolic relationship). At inclinometers positions I1, I2, I3, on the long sides of the excavation, it was found that the 2-dimensional plane strain analysis gave good agreement with field measurements, but that 3-dimensional effects were important at inclinometers I4 and I5. The measured deflections at I4 and I5 were some 50% and 40% of the movements computed by the 2 dimensional plane strain analysis. Ou *et al* developed correlations between the 3-dimensional and 2-dimensional analyses to enable predictions to be made of wall deflections near corners. These correlations worked well for the project they studied.

Simic and French (1998) used a 3-dimensional analysis of an underground station box, formed in diaphragm walls, to seek savings in reinforcement when comparing results with plane strain analysis. They concluded that steel quantities could be reduced by about 25% for the project they studied, mainly because the walls near the corners of the excavation were computed to be less heavily loaded.

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Appendix C Wall types

C1 SHEET PILES

The British Steel (1997) *Piling Handbook* contains information on the various available sections and steel grades supplied by Corus. The two different sections are Larssen and Frodingham as shown on Figures C1 and C2, distinguished by the position of the clutch. The Larssen sections are generally preferred for temporary works due to their ease of stacking, driving and transportation. Frodingham sections are more often specified for permanent works, particularly where there is a need to retain water. With the clutch located on the tension and compression faces of the Frodingham sections, there is less chance of clutch slippage during service and less risk of water leakage along the interlock compared to Larssen sections.

Figure C1 *Larssen sheet pile sections and properties (after Corus, 2001)*

Figure C2 *Frodingham sheet pile sections and properties (after Corus, 2001)*

The Steel Construction Institute (SCI, 2001) *Steel Intensive Basements* describes several methods to inhibit water seepage through sheet pile walls, concentrating on sealing the interlocks. The systems include non-swelling sealants, hydrophilic (water swelling) sealants, combination systems and welded interlocks. It is important to note that for the systems that rely on pre-applied sealants, the integrity of the vertical joints is very much dependent on the driving conditions and the installation techniques used. Site welding is the only impervious solution, but cannot be carried out below the excavation level except for piles driven in pairs, where the joint between the pair can be welded full length before installation. Combination sealant systems use both mechanical seals and sealing compounds. Advice should be sought from piling manufacturers or specialist piling contractors on particular applications.

The issue of clutch slippage is discussed further in section 4.2.3. Durability and corrosion resistance of sheet pile wall sections is discussed in section 6.6.3.

Plate C1 *Sheet pile wall installation, Portsmouth Harbour (courtesy of Corus)*

Two different steel strengths are available for sheet pile sections, S270GP and S355GP (BS EN 10248:1995). The 270 and 355 designations refer to the minimum yield strength, and are similar to BS 4360:1986 grades 43A and 50A.

Where higher stiffness and strength are required, high modulus sections are available, combining sheet pile sections with universal beam sections as shown on Figure C3.

Figure C3 *High modulus sheet piles (after Corus, 2001)*

C2 COMBI WALL

A combi wall is a combination of tubular sections and sheet pile clutches, enabling a continuous retaining wall to be formed of high capacity tubular sections.

C3 KING POST WALL

Isolated steel beams or posts are installed along the line of the wall, either driven into place or placed in bored cast-in-place piles at centres typically between 1 and 3 m. The space between the posts is filled in over the retaining height as the excavation proceeds using either the traditional solution of timber railway sleepers (with the king post piles at 2.4 m centres), precast concrete elements, in-situ or spray concrete. King post walls are not suitable for retaining coarse grained soils below the water table and the excavation process relies on some short term cohesion in the retained material to avoid any significant over-break and collapse. Control of displacements in the retained material is very dependent on the workmanship and the ability of the retained soil to be self-supporting in the short term (section 2.5.1).

C4 CONTIGUOUS PILE

A contiguous pile wall consists of bored cast-in-place concrete piles along the line of the wall as shown in Figure C4. Construction is by means of cfa or traditional bored piling rigs with temporary casings as required. The dimension of the gap between the piles can be varied to suit site dimensions and the specific ground conditions within a typical range of 50 to 150 mm. Note that the use of cfa piles limits the depth of installation of the reinforcement cage as noted in Table 3.8. The maximum achievable wall depths for traditional bored piles is limited by the kelly bar length for kelly rigs, typically 34 to 55 m depending on the particular rig. Verticality tolerances should be considered to ensure either that the potential gap between piles does not increase unacceptably with depth and that the piles do not overlap. In practice it is unusual for pile walls to be constructed in excess of 25 m depth, but isolated piles may be extended deeper to provide vertical load capacity.

Figure C4 *Contiguous pile wall*

Contiguous pile walls are not suitable for retaining water bearing coarse grained soils and are usually only specified as temporary walls. A permanent wall may be created with a structural facing applied to the piles to fill in the gaps. This may either take the form of a structural concrete facing wall, tied as necessary to the contiguous piles, or sprayed concrete, which fills the gaps to beyond the pile centreline to form a positive key between the contiguous piles and the spray concrete. Typical diameters and pile spacing are shown on the Table C1, although the use of diameters greater than 1200 mm is unusual.

Table C1 *Contiguous pile wall - typical diameters and spacing*

Diameter mm	Spacing mm	Diameter mm	Spacing mm	Diameter mm	Spacing mm
300	400	900	1000	1800	1900
450	550	1050	1150	2100	2200
600	700	1200	1300	2400	2500
750	850	1500	1600		

Plate C2 *Contiguous bored pile wall (courtesy of Bachy Soletanche Limited)*

C5 HARD/SOFT SECANT

A hard/soft secant pile wall consists of overlapping piles as shown on Figure C5. The primary (female) piles are cast first and consist of a soft pile mix, typically cement and bentonite or cement, bentonite and sand with a characteristic compressive strength of between 1 and 3 N/mm^2. They are unreinforced.

Figure C5 *Hard/soft secant pile wall* *(see page containing Figure C4)*

The secondary (male) piles are subsequently installed to intersect the primary (female) piles as shown. Cfa piling rigs are often used to install the wall, but traditional bored piling rigs may be used. As for contiguous pile walls, the reinforcement depth may be limited by the installation technique. The depth of the hard/soft secant wall is limited by the ability to control the verticality tolerances in order to maintain the secanting. The male and female piles may have a different diameter and may extend to different depths (Beadman and Ward, 1998). For small projects it may be preferable to retain the same diameter for both the male and female piles to avoid either duplicating piling rigs or the time taken to change the drilling diameter for a single rig operation. When ground conditions allow, the female piles may be curtailed after penetrating fine grained strata below water bearing coarse grained strata. Soft piles have been used to retain up to 8m head of groundwater. Typical diameters are shown in Table C2:

Table C2 *Hard/soft secant pile wall - typical diameters and spacing*

Diameter mm		Spacing[1]	Diameter mm		Spacing
Male	Female	mm	Male	Female	mm
450	450	600	900	600	1100
600	600	800	1200	600	1400
750	750	1000	1200	750	1450

Note

1. The gap between the male piles should not exceed 40% of the diameter of the soft piles.

The soft pile mix is not usually a permanent solution to retain water, due to the shrinkage and cracking characteristics of the mix when it dries out. Should the wall be a permanent structure, then the choice of soft pile material should be made with great care and the hydrological setting of the structure should be such that there is a high confidence that the soft piles will remain saturated throughout their life. Alternatively a structural wall can be applied to the face of the hard/soft secant piles to reinforce the soft piles in the long term.

Plate C3 *Hard/soft secant pile wall as temporary works at North Greenwich Station, London (courtesy of Bachy Soletanche Limited)*

C6 HARD/FIRM SECANT

A hard/firm secant pile wall shown in Figure C6. The female pile has a characteristic compressive strength of 10-20 N/mm² which is retarded to reduce the strength of the mix while the male piles are drilled between the female piles. Typically the characteristic strength of the female pile mix is specified as a 56-day strength rather than the more usual 28-day strength. This enables such walls to be installed with cfa piling rigs, using less energy to drill the softer female piles than for a hard/hard secant. The female piles may be installed to a lesser depth as noted above for the hard/soft secant. The female piles are typically unreinforced. The spacing of the male piles is calculated by ensuring that a minimum overlap of say 25 mm is provided at the maximum functioning depth of the wall when the most onerous verticality tolerances are applied.

Figure C6 *Hard/firm secant pile wall (see page containing Figure C4)*

The maximum functioning depth may be the excavation depth or a greater depth if the wall is required to provide water cut off at a greater depth. Typical pile spacings are shown in Table C3 and tend to lie between the spacing for a hard/soft wall and a hard/hard wall.

Table C3 *Hard/firm secant pile wall - typical diameters and spacing*

Diameter mm male and female	Spacing mm
600	900
750	1150

C7 HARD/HARD SECANT

The male and female piles of a hard/hard secant pile wall are both cast with full strength concrete and both may be reinforced. The female piles are cast first, followed by the male piles, which are formed by drilling into the female piles using casing rotated by an hydraulic rig or an oscillator. A thick walled casing, typically 40 mm thick, is used to resist the high torque generated during the cutting process. As shown in Figure C7, the female pile reinforcement should be detailed and placed to avoid being cut during the installation of the male piles.

Figure C7 *Hard/hard secant pile wall (see page containing Figure C4)*

Either a rectangular cage is used, carefully spaced and orientated in the pile bore before concreting, or a steel UB section is used. The limiting depth for hard/hard secant pile walls is about 25 m and is limited by the piling rig's ability to rotate the casing. The verticality tolerances should be specified to ensure that secanting is maintaining over the required functioning depth of the wall and to avoid the possibility of cutting into the female pile cage.

Table C4 *Hard/hard secant pile wall-typical diameters and spacing*

Diameter mm Male and female	Spacing mm
750	650
880	760
1180	1025

Plate C4 *Hard/hard secant pile wall and an example of top-down construction (courtesy of Bachy Soletanche Limited)*

Hard/hard secant pile walls may be used for circular shafts with no waling beams, using hoop compression to resist the ground and groundwater forces. The width of the interlock for the most onerous tolerances should be designed at maximum excavation level to ensure that it is capable of transmitting the compression forces.

C8 DIAPHRAGM WALLS

Diaphragm walls are formed by sequenced excavation under a support fluid followed by the lowering of the prefabricated reinforcement cage and concreting using tremie pipes to displace the fluid. Excavation is carried out by grab, using chisels to break up obstructions, or by reverse circulation mills to cut through harder materials.

Reverse circulation mills rely on the spoil from the trench being held in suspension by the support fluid, which is pumped out and cleaned to extract the spoil. This requires an extensive site installation for the handling of the fluid. Fine grained material may block the cutters and reduce the efficiency of the cutter. The disposal of the support fluid can be costly as it is treated as a contaminated material and requires disposal in a suitably licensed site.

The reinforced concrete for diaphragm walls may be post-tensioned or precast panels may be used, pretensioned if required. The precast panels are lowered into the fluid filled trench and sealed into the ground with in-situ concrete or grout, tremied into position. Precast panels provide a preformed high quality surface finish. Unfortunately, the weight of the panels often makes this solution impractical.

Figure C8 *Diaphragm wall panel and joint (see page containing Figure C4)*

Figure C8 indicates a typical diaphragm wall panel. Panel widths are a function of the grab and cutter widths available and are typically 600, 800, 1000, 1200 or 1500 mm wide. Contractors offer different joint systems, some of which enable a vertical waterbar to be included in the joint between panels as shown. The number of bites to excavate the panel and the grab or cutter length determines the panel length. A typical grab length is 2.8 m, but may reduce to say 2.2 m. The potential variations are shown in Table C5.

Table C.5 *Diaphragm wall - typical panel widths*

Number of bites	Panel length[3] m Minimum	Maximum	Excavation sequence
1	W[1]	W	1
2	W+T[2]	2xW-T	1 2
3	2xW+T	3xW-2xT	1 3 2

Notes

1. W is the grab length available (typically 2.8 m)
2. T is the grab width, 600, 800, 1000, 1200 or 1500 mm.
3. The stability of the bentonite filled temporary trench should be checked for adequacy (Huder, 1972).

There are additional limitations on the length of the starter panel (the first panel to be constructed in a sequence) and on the closure (or final) panel. The detailed planning of the site will often dictate where the starter and closure panels need to be located and this should be discussed with the constructor. For T panels or corner panels, the dimensions should be discussed with the constructor.

The maximum depth of the diaphragm wall is limited by the depth of reach of the mill, which relates to the length of hose available to connect the bentonite pumps at the mill end to the surface. For a rope operated grab, the limit is the length of the rope, which may be considerable. Diaphragm walls have been constructed to depths of 120 m. The designer should consider verticality tolerances. There are also practical difficulties with installing deep reinforcement cages, including the need for splices, the length of cage to be lifted and the overall weight of the cage.

Plate C5 *Diaphragm wall construction at Canary Wharf Station, London (courtesy of Bachy Soletanche Limited)*

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Appendix D Soil mechanics

This Appendix summarises the basic principles of soil mechanics essential to retaining wall design, which are:

- the concepts of total, effective and shear stress
- the representation of the stress state within the cross sectional plane of a long retaining wall using the Mohr circle construction
- the distinction between undrained (short term) total stress analysis and drained (long term) effective stress analysis
- key aspects of soil behaviour relevant to embedded retaining walls, including the effect of stress history; soil strength; and soil stiffness.

D1 STRESS ANALYSIS

D1.1 Introduction

The loads and forces applied to a solid body (such as a soil mass) are distributed within the body as stresses. Provided that there are no inhomogeneities that might interrupt the transfer of stress, it is usually assumed that the stresses vary smoothly and continuously throughout the body, which is then described as a continuum. In general, any plane within a solid body in a general state of stress will be subject to both shear stresses (acting parallel to it) and normal or direct stresses, acting perpendicular to it.

In soil mechanics, it is usual to take compressive direct stresses and strains as positive. This is in contrast to structural mechanics, in which tensile direct stresses and strains are conventionally taken as positive. The "compression positive" sign convention is adopted in soil mechanics because soils are essentially particulate materials, which cannot sustain tensile stresses unless the particles are cemented together. Stresses in soil mechanics are therefore almost always compressive. A stress *increment*, however, can be tensile provided that the cumulative stress remains compressive. Tensile strains are also permissible, again provided that the cumulative stress remains compressive.

D1.2 Principal stresses

In a three-dimensional body under a general state of stress, there will be three orthogonal planes on which stress acts. These three planes are known as *principal planes*, and the normal stresses which act on them are the *principal stresses*. By definition, the shear stress associated with a principal stress is zero, and a principal plane is a plane on which there is zero shear stress. The largest principal stress is termed the *major principal stress*, the smallest principal stress is the *minor principal stress*, and the remaining principal stress is the *intermediate principal stress*.

D1.3 Plane strain

Many retaining walls are long in comparison with their height. This means that any cross-sectional plane must be identical to any other in all respects, including the stresses acting on and within it. Considerations of equilibrium and symmetry further require that the plane of the cross-section is a principal plane with zero strain.

The cross sectional plane is almost always the plane on which the intermediate principal stress acts, so that the major and minor principal stresses are contained within it. As the failure of soil is governed by the major and minor principal stresses, analysis of a retaining wall that is long in comparison with its other dimensions need consider only the plane of the cross section. Also, the longitudinal (intermediate) principal stress takes up whatever value is necessary to ensure that the strain in the longitudinal direction is zero. Thus all deformation takes place within the cross-section of the structure – a condition known as *plane strain*. In reality, geotechnical structures are of finite extent and there are bound to be differences in geometry and/or soil conditions along their length. Nonetheless, the plane strain assumption is a useful and usually a reasonable approximation that is made in nearly all simple retaining wall analyses.

D1.4 Total and effective stress

A saturated soil comprises two distinct material phases, the soil particles and the pore water, which have very different shear strengths (ie ability to resist shear). The shear strength of the pore water is zero, while the soil skeleton can resist shear partly as a result of particle interlocking and structure, but mainly because of interparticle friction. Because the strengths of the soil skeleton and the pore water are so different, it is necessary to consider the stresses acting on each phase separately.

As the pore water cannot take shear, all shear stresses must be carried by the soil skeleton. The normal total stress (denoted σ) applied to a soil element may be separated into the *effective stress* (σ') carried by the soil skeleton and the *pore water pressure* (u, measured relative to atmospheric pressure), using the *principle of effective stress* (Terzaghi, 1936). For a saturated soil,

$$\sigma' = \sigma - u \qquad \text{(D.1)}$$

The effective stress σ' controls the (volumetric and shear) stiffness, strength and failure of the soil. However, the equilibrium of a retaining wall is governed by the total stress acting on it:

$$\sigma = \sigma' + u \qquad \text{(D.2)}$$

Non saturated soils, containing air in the pores as well as water, are more complicated and are beyond the scope of this report. They can generally be disregarded in design of embedded walls, for which most soils are either saturated or dry.

D1.5 Mohr circle of stress

The normal and shear stresses σ and τ acting on a plane in a given direction within the cross-sectional plane will depend on the orientation of the plane with respect to the major and minor principal stress directions (Figure D1).

Figure D1 *Normal and shear stresses acting on an imaginary plane within the cross-section plane*

If the plane is perpendicular to either the major or the minor principal stress, the shear stress acting in the direction of the plane will be zero. In general, however, there will be a shear stress acting along the plane, the magnitude of which increases as the plane is rotated away from the direction of the planes of principal stress.

The stress state within a plane containing any pair of principal stresses is most conveniently represented by means of a graphical construction known as the *Mohr circle of stress* (Figure D2).

Figure D2 *Mohr circles of stress*

This is a circle, plotted on a graph of shear stress τ against normal stress σ. The circle may be plotted for either total normal stresses, σ, or effective normal stresses, σ'. The total and effective shear stresses are the same, as all shear stress must be carried by the soil skeleton.

The Mohr circle passes through the points representing the major and the minor principal stresses, whose coordinates are (σ'_1, 0) and (σ'_3, 0) (for effective stresses) and (σ_1, 0) and (σ_3, 0) (for total stresses) respectively. The centre of the circle of effective stress is at ([$\sigma'_1+\sigma'_3$]/2, 0), and the centre of the circle of total stress is at ([$\sigma_1+\sigma_3$]/2, 0). Recalling that $\sigma = \sigma' + u$ (where u is the pore water pressure), the centres of the circles of effective and total stress are separated by a distance equal to u along the normal stress axis. $(\sigma'_1+\sigma'_3)/2$ is the average of the major and minor principal effective stresses, and is conventionally given the symbol s'. Similarly, $(\sigma_1+\sigma_3)/2$ is the average of the major and minor principal total stresses, and is given the symbol s.

The radius of the circle of effective stress is $[\sigma'_1-\sigma'_3]/2$, while the radius of the circle of total stress is $[\sigma_1-\sigma_3]/2$. These are identical, because the pore water pressure u is cancelled out in the subtraction of the two principal effective stresses. $[\sigma'_1-\sigma'_3]/2$ (or $[\sigma_1-\sigma_3]/2$) is equal to the maximum shear stress acting within the principal plane represented by the Mohr circle, and is conventionally given the symbol t.

The stresses acting on a line at an angle θ anticlockwise from the plane on which the major principal stress acts are found by drawing a line from the centre of the Mohr circle to the circumference, which makes an angle 2θ (measured anticlockwise) with the normal stress (σ or σ') axis. The stress state on the line (in effective stress terms, [σ', τ]) is given by the point where this diameter meets the circumference of the circle (Figure D2).

The Mohr circle of stress shows that, unless the major and minor principal stresses are equal, there must be some shear stress acting somewhere within the plane under consideration. The maximum shear stress within the plane is equal to the radius of the Mohr circle, $[\sigma'_1-\sigma'_3]/2 = [\sigma_1-\sigma_3]/2$. It occurs at angles of $\pm 90°$ to the normal stress axis on the Mohr diagram, indicating that in reality, the shear stress is largest on planes that are at $\pm 45°$ to the planes on which the major and minor principal stresses act.

The maximum ratio of shear to normal effective stress on any plane within the sample, $(\tau/\sigma')_{max}$, occurs where the tangents through the origin touch the circle. If the soil has no cohesion and is at shear failure, this occurs at angles of $\pm(90° \pm \phi')$ to the normal stress axis on the Mohr diagram, indicating that in reality, the shear stress is largest on planes that are at $\pm(45°+\phi'/2)$ to the plane on which the major principal stress acts, and $\pm(45°-\phi'/2)$ to the plane on which the minor principal stress acts.

D2 DRAINED AND UNDRAINED CONDITIONS

One of the key differences between the engineering behaviour of clays and that of sands is the rate at which the effective stresses within the soil respond to a loading or unloading, or a change in pore water conditions, at a boundary. The application of a load to the surface of a saturated soil results initially in an increase in the pore water pressure. This gives rise to a hydraulic gradient, in response to which pore water flows out of the soil and the soil compresses. As the water flows out of the soil, the pore water pressures gradually move to their long-term equilibrium values and the soil deforms in the time-dependent process of *consolidation*. The time taken for consolidation to occur decreases with increasing soil stiffness (because the ultimate volume change is reduced) and increasing permeability (because the pore water can flow more easily through the soil skeleton).

The consolidation behaviour of a soil is characterised by the *consolidation coefficient,* c_v,

$$c_v = k.M_o/\gamma_w \qquad \text{(D.3)}$$

where k is the coefficient of permeability of the soil in the direction of drainage, M_o is the one-dimensional modulus (constrained modulus) in the direction of compression or swelling, and γ_w is the unit weight of water.

In sands, which are often both stiff and permeable, consolidation is effectively instantaneous. Volume changes are small, and pore water pressures move rapidly to their equilibrium long term or "*drained*" values. The term "drained" is used to indicate that the pore water pressures have reached their long-term, steady-state values. As the steady-state pore water pressures can be calculated by means of an appropriate seepage analysis such as a flownet, the effective stresses can be determined and the behaviour of a sand can always be analysed using effective stress parameters. In clays, consolidation (or swelling) in response to a change in boundary loading or water pressure conditions can take years or even decades.

The changes in pore water pressure that occur in a clay during consolidation or swelling can significantly affect the stability of a geotechnical structure. In general, unloading processes will promote swelling and softening of the soil (tending to make failure more likely in the long-term), while loading processes will cause long term consolidation (so that failure is more likely in the short term, immediately after loading). Thus in clays, it is usually necessary to investigate separately the possibility of failure in both the short term (ie immediately after loading/unloading) and in the long term.

The long-term calculation must be carried out in terms of effective stresses and pore water pressures, and is often referred to (as indicated above) as a *drained* analysis. To carry out the short-term calculation in terms of effective stresses would present a problem, as the non-equilibrium pore water pressures following loading/unloading and during consolidation/swelling - and hence the effective stresses - in a clay soil are difficult to predict. However, it is possible to carry out an analysis in terms of total stresses, using a different strength criterion, for deformations that occur rapidly in comparison with the time it takes for changes in specific volume to occur. Such an analysis is termed an *undrained* analysis, as the underlying assumption that there is no volume change is equivalent to assuming that there is no drainage of pore water into or out of a saturated soil.

Excavation in front of an embedded retaining wall in a stiff clay is in itself an unloading process. It is likely to generate negative excess pore water pressures, so that stability would be expected to be more critical in the long term than in the short term. However, it also brings about changes in the hydraulic boundary conditions (for example, a lowering of the groundwater level in front of the wall) that make the long-term equilibrium pore water pressures different from the initial groundwater regime. In addition, there may a degree of vertical reloading as basement floors are constructed, so each case must be assessed on the basis of its own facts.

While soil behaviour is governed by the effective stress, the wall structure will respond to the total stress acting on it so changes in pore water pressure and effective stress must both be considered in retaining wall design.

D3 STRESS HISTORY

The current volumetric state (void ratio, specific volume or water content) of a clay is related to its previous loading or unloading history. The geological stress history of a stiff clay deposit is likely to comprise one dimensional compression, as further material was deposited on top, followed by one dimensional swelling as overlying material was removed by erosion. For example, it is estimated that the London Clay at Bradwell, Essex, has been subjected to an overburden approximately 1450 kPa greater than that at present (Skempton, 1961).

The stress history of a typical overconsolidated clay deposit may be represented on a graph of the specific volume v against the natural logarithm of the vertical effective stress ln σ'_v, (Figure D3), and on a graph of σ'_h against σ'_v, as shown in Figure 4.1.

Figure D3 *Schematic stress history of an overconsolidated clay*

During deposition, there is a unique, straight-line relationship between the specific volume v (=1+e) and ln σ'_v. This is known as the *one-dimensional normal compression line*. During first compression, deformations occur as a result of (a) particle distortion and (b) particle slip or breakage, as the soil skeleton rearranges itself to support the increased stress. The first of these is recoverable on unloading, but the second is not. Thus on unloading (and also on subsequent reloading should it occur), the soil response in terms of the change in specific volume for a given change in vertical effective stress will be much stiffer. In unloading and reloading, the soil will follow a hysteresis loop on the graph of v against ln σ'_v which is usually idealised as a straight line. Unlike the normal compression line, there is no unique unload/reload line: an unload/reload line can begin from any point on the normal compression line at which the soil starts to be unloaded.

A soil whose state lies on the normal compression line has never before been subjected to a vertical effective stress higher than the current value. Such a soil is termed *normally consolidated*. A soil which has previously been consolidated to a higher vertical effective stress than that which currently acts is *overconsolidated*, with an overconsolidation ratio (*OCR*) given by

$$OCR = \sigma'_{v(\mathrm{max\,prev})}/\sigma'_{v(\mathrm{current})} \qquad \text{(D.4)}$$

A rise in the groundwater level will reduce the vertical effective stress, resulting in overconsolidation without the need for overburden removal.

The overconsolidation ratio (Equation D.4) is a crude but important indicator of the stress state of a clay, in relation to its previous stress history. An overconsolidated clay is likely to be much stiffer than a normally consolidated clay at the same σ'_v, and stiffness generally increases with overconsolidation ratio. A further important distinction is that heavily overconsolidated clays (ie with a high *OCR*) will tend to dilate when sheared, while normally consolidated or lightly overconsolidated clays (ie with an *OCR* of less than about 3) will tend to compress. The tendency of heavily overconsolidated clays to dilate when sheared will lead to the development of a peak strength, which is unlikely to be apparent in a normally consolidated or lightly overconsolidated material. When sheared in undrained conditions, in which volume change is prevented, heavily overconsolidated clays will generate reduced pore water pressures which may go negative (ie suction), while lightly overconsolidated or normally consolidated clays will generate increased pore water pressures.

D4 SOIL SHEAR STRENGTH

Soil strength may be defined in terms of effective stress as an *angle of shearing resistance* ϕ' and a further component, generally small, known as *effective cohesion*, c'. In terms of total stress, for an undrained analysis of a clay, soil strength may be defined as an *undrained shear strength* s_u.

D4.1 Effective stress

When sheared (and, in the case of a clay, allowed to drain during shear), a loose or lightly overconsolidated soil will gradually compress until it reaches a *critical state*, at which shearing may continue without any further change in shear stress τ, normal effective stress σ' and specific volume v. A dense or heavily overconsolidated soil will initially compress and then dilate to achieve the critical state (Figure D4).

Figure D4 *Typical stress-strain data for a loose (lightly overconsolidated or normally consolidated) soil and for dense (heavily overconsolidated) soil*

For a loose soil, the critical state is relatively easy to identify, and if the sample does not rupture may reasonably be determined on the basis of measurements made at the boundaries of a test specimen, assuming that the sample deforms as a continuum with uniform stresses and strains.

To define fully the state of a soil, three variables are required quantifying the specific volume v, the shear stress τ and the normal effective stress σ'. Critical states are combinations of these three variables at which steady, continued shear deformation can take place. On a three-dimensional plot of specific volume against shear stress against normal effective stress, the critical states form a unique line known as the critical state line. On a graph of shear stress τ against normal effective stress σ' on the plane of maximum stress ratio, the critical state line is straight with equation

$$\tau = \sigma'.\tan\phi'_{crit} \qquad (D.5)$$

On a graph of specific volume v against the logarithm of the normal stress, ln σ', the critical state line is straight as indicated in Figures D5 and D6.

Figure D5 *Critical state line*

Figure D6 *Undrained state paths for clay samples having the same specific volume (a) v against ln σ'; (b) τ against σ'. Sample A is heavily overconsolidated; sample B is lightly overconsolidated*

For a dense or heavily overconsolidated soil, the stress-strain behaviour is more complex. The shear stress rises to a *peak*, at or near which a rupture surface develops. The shear stress then falls rapidly, and failure is brittle. Once the rupture has formed, it governs the overall behaviour of the soil element being tested. Compression between the ends of a triaxial test sample is due to relative sliding along the rupture surface rather than a uniform, continuous axial strain. The axial load that the sample can sustain depends on the stress state of the soil in a thin rupture zone, which is likely to soften and swell preferentially and be very different from the remainder of the sample. It is therefore inappropriate to convert loads and displacements measured at the boundaries of the test specimen to equivalent uniform stresses and strains after a rupture surface has developed.

Figure D7 *Ring shear test data for undisturbed London Clay (from Bishop et al, 1971)*

Figure D7 shows data from a ring shear test on an undisturbed sample of unweathered London Clay. As shearing continues after the peak, the clay particles in the rupture zone gradually become aligned with the direction of shear, resulting in a gradual polishing and loss of strength on the rupture surface. The angle of shearing resistance ϕ' (or the maximum stress ratio τ/σ') falls through the critical state, until eventually a *residual strength* is reached.

When quoting a value of soil strength, it is important to state whether it refers to the peak, critical state, or residual. A further point is that the peak angle of shearing resistance ϕ'_p occurs when the stress ratio τ/σ' is greatest, which in a conventional triaxial compression test (in which both the shear stress and the normal effective stress increase with the axial load) is unlikely to coincide with the peak shear stress.

Effective stress strength parameters are often determined by carrying out three conventional triaxial compression tests on similar samples at different cell pressures; plotting Mohr circles of effective stress at peak stress ratio; and drawing the best fit tangent which defines a failure envelope of the form

$$\tau = c' + \sigma' \tan \phi'_{tgt} \qquad \text{(D.6)}$$

where ϕ'_{tgt} is the slope of the failure envelope and c' its intercept with the shear stress axis.

If the data used to plot the Mohr circles of stress represent critical states, the slope of the failure envelope will be equal to the critical state angle of shearing resistance ϕ'_{crit} and the failure envelope will pass through the origin giving $c' = 0$. This is because the only component of soil strength still operating at the critical state is that due to interparticle friction, so that if the effective stress is zero, then so is the shear strength.

The treatment of peak strength data in this way should be approached with caution, because in these circumstances:

- Equation D.6 has no direct physical interpretation: it is merely a fit to the data over the range for which data are available. c' is not necessarily indicative of a real cohesion (ie an ability to withstand shear stresses at zero effective stress), and ϕ'_{tgt} is not a true angle of shearing resistance, as shown by the fact that when defined in this way it may be smaller than the value of ϕ'_{crit} for the same soil
- the approach takes no account of differences in stress history and specific volume between individual samples, which would be expected to alter the potential for dilation and hence the peak strength achieved
- the scatter in the values of c' and ϕ'_{tgt} obtained from similar sets of samples can be very wide (Muir Wood, 1990).

A more satisfactory interpretation of peak strength data may be obtained by normalising the values of shear stress and normal effective stress on the plane of maximum stress ratio with respect to the equivalent consolidation pressure σ'_e (the value of σ' on the normal compression line at the current sample specific volume). In this way, the dependence of peak strength on stress history and specific volume or water content is to some extent taken into account, and the scatter in the results is reduced. (This is demonstrated by Muir Wood (1990) using triaxial test data). When plotted on a graph of τ/σ'_e against σ'/σ'_e, the peak strength data should lie on a straight line of equation

$$\tau/\sigma'_e = h_e(g + \sigma'/\sigma'_e) \qquad \text{(D.7)}$$

known as the Hvorslev line. The Hvorslev line is limited at its left hand end by the point where it forms a tangent to a Mohr circle of stress passing through the origin (as a result of the inability of the soil to carry tension). At its right hand end, the Hvorslev line intersects the critical state line, which represents a sample insufficiently overconsolidated to generate a peak strength (Figure D8).

Figure D8 *Normalised peak and critical state*

The key point is that to understand the peak strengths of a clay, it is necessary to take account of both the normal effective stress and the specific volume at failure, since both of these will influence the potential for dilation and hence the peak strength actually developed.

A bonded or cemented soil may have a capacity to carry a shear stress at zero normal effective stress. Otherwise, the peak strength is probably best defined by the line joining the stress state at the peak to the origin on the Mohr diagram, giving a stress-dependent peak angle of shearing resistance

$$\phi'_{\text{peak}} = \tan^{-1} (\tau/\sigma')_{\text{peak}} \qquad \text{(D.8)}$$

For most soils, the difference between the critical state and the peak angles of shearing resistance is related to the angle of dilation, ψ. It is important to determine the peak strength over a stress range applicable to the retaining wall under consideration. This is discussed in section 5.4.4.

D4.2 Total stress (undrained shear strength)

The critical state model illustrated in Figure D5 describes the state of a soil at failure in terms of the specific volume and the effective stresses (shear and normal). It has already been mentioned that clays will consolidate or swell in response to a change in loading or in hydraulic boundary conditions, during which time the pore water pressures - and hence the effective stresses - are difficult to predict.

The critical state framework provides an alternative way of investigating the strength of a clay soil in the short term, assuming that the clay is sheared quickly in comparison with the time it takes for changes in specific volume to occur. Shear failure at constant specific volume must follow a horizontal path on the graph of v against ln σ' from the initial condition to the critical state, see Figure D6(a). The position on the critical state line is fixed by the specific volume of the sample as sheared: this also defines the shear stress at undrained failure, see Figure D6(b). The effective stress state on the critical state line is independent of the external changes in stress (loading or unloading) that cause failure. The pore water pressure will take up the difference between the total and effective normal stresses, but the shear stress at failure is a function of the specific volume alone.

Figure D9 *Mohr circles representation of undrained shear strength failure criterion in terms of total stresses for shearing at constant specific volume*

Figure D9 shows that, for a sample of clay sheared undrained to the critical state, there is only one possible Mohr circle of effective stress at failure. The radius of this Mohr circle - and hence its position on the σ' axis, given that it must touch the failure envelope $\tau = \sigma' \tan\phi'_{\text{crit}}$ - depends on the specific volume of the clay as sheared. The position of the Mohr circle of total stress on the normal stress axis depends on the applied loading, with the pore water pressure (which may be negative or positive) being equal to the distance between the centres of the circles of total and effective stress (Figure D9). However the radius of the Mohr circle of total stress must be the same as the radius of the Mohr circle of effective stress. Thus the envelope to all possible Mohr circles of total stress is given by:

$$\tau_{\text{max}} = \pm s_{\text{u}} \qquad \text{(D.9)}$$

where τ_{max} is the maximum shear stress within the sample, and s_u is the *undrained shear strength* of the clay.

The s_u model for failure in terms of total stresses represents a very special case, and is applicable only to clay soils sheared at constant volume. A further problem is that the value of undrained shear strength can be particularly sensitive to sample size, sample disturbance and the test apparatus used, as discussed in sections 5.4 and 5.4.4

The undrained shear strength of a clay of a given specific volume is usually determined in the laboratory by carrying out unconsolidated-undrained triaxial tests at different cell pressures. When the Mohr circles of total stress are plotted from such tests, they should all have the same radius s_u *provided that the samples all had the same initial specific volume*. Their common tangent should be horizontal, intercepting the τ axis at $\tau = s_u$.

If the common tangent is not horizontal, the inference is that the samples as tested did not have the same specific volume at failure or that the tests were not fully undrained. This would occur if the samples had been allowed to consolidate to equilibrium at the test cell pressure before the start of shear (ie the tests carried out were consolidated-undrained, rather than unconsolidated-undrained). In this case, the test results provide an indication of the increase in s_u with decreasing water content, due to increasing initial stress, perhaps corresponding to increasing depth in the field. Alternatively, the samples may not have been fully saturated. In this case, the air is compressed and dissolved in the pore water as the cell pressure is increased, so that changes in sample volume and void ratio occur without the passage of pore fluid (air or water) into or out of the sample. The notion that test results with an inclined tangent may be described in terms of an "undrained friction angle ϕ_u" and an "undrained cohesion intercept c_u ", is fundamentally flawed.

Multi-stage tests, in which a single sample is brought to failure at three successively greater cell pressures, are not recommended because the later stages are effectively tests on a damaged sample, and so give erroneous results.

D4.3 Drained or undrained conditions?

As excavation in front of an embedded retaining wall is an unloading process, the stability of the wall would often be expected to be more critical in the longer term as the negative excess pore water pressures induced on excavation dissipate and the clay tends to soften and swell. In the short term, the negative excess pore water pressures maintain the average effective stress s' ($=\frac{1}{2}[\sigma'_1+\sigma'_3]$) temporarily high, so that the Mohr circle of effective stress is initially well within the failure envelope (Figure D10).

Figure D10 *Failure after dissipation of negative excess pore water pressures induced on excavation*

As the negative excess pore water pressures dissipate, the clay swells and the average effective stress s' is reduced. The shear stress required to maintain the stability of the excavation remains approximately constant, and when the Mohr circle of effective stress touches the effective stress failure envelope, the soil will fail (Figure D10).

Although a permanent wall must be designed on the basis of an effective stress analysis for the anticipated long-term pore water pressure conditions, a more economical design will usually result if it can be assumed that substantially undrained conditions will prevail over the lifetime of a temporary structure. The key question is whether this assumption is reasonable. Unfortunately, the answer is extremely complex, and will depend on factors which even in the same soil may vary from site to site. These factors, which are discussed in section 5.3, are practically unquantifiable. Clearly, an undrained analysis should never be used on its own for permanent works, and even for temporary works it is advisable to err on the side of caution.

Softening is likely to occur first near the retained and excavated soil surfaces, which may be taken into account by using effective stress analysis or reduced undrained shear strengths in these zones. The depth L below a free surface to which the soil is affected by softening after an elapsed time t may in some circumstances be estimated approximately as:

$$L = \sqrt{(12c_v t)} \tag{D.10}$$

where c_v is the coefficient of consolidation (see eg Bolton, 1991). This assumes that the free surface acts as a recharge boundary, but that there is no source of recharge within the soil itself. As the horizontal coefficient of permeability of a soil is often an order of magnitude or so greater than the vertical permeability, lateral recharge may often be more significant than vertical recharge in the field. This is discussed further in sections 5.4.2 and 5.5.

D5 SOIL STIFFNESS

Calculations based on soil strength can be used to assess the stability of a wall, but not on their own to estimate wall and soil movements under working conditions. To do this, a stress-strain relationship for the soil is needed (Figure D11).

Figure D11 *Soil stiffness definition*

The simplest approach is to describe the pre-failure behaviour of the soil as linear elastic. This is not a realistic model because the "stiffness" of a clay (defined either as a tangent stiffness, $d\sigma/d\varepsilon$, or as a secant stiffness $\Delta\sigma/\Delta\varepsilon$, where $\Delta\sigma$ and $\Delta\varepsilon$ represent changes of generalised stress and strain from a defined starting point depends on:

- the average effective stress, p' $(=[\sigma'_1+\sigma'_2+\sigma'_3]/3)$
- the overconsolidation ratio
- the stress path being followed, particularly in relation to the recent stress history.

The typical variation of soil stiffness with continued shearing is discussed in section 5.4.5.

The maximum shear strain increment in the soil around an embedded retaining wall with acceptably small deflections is likely to be in the order of 0.1% (Atkinson and Sallfors, 1991; Mair, 1993). This can be used, in association with a stiffness-strain curve such as that described in section 5.4.5, to estimate a suitable soil stiffness profile for use in analysis. Usually, the soil stiffness must be allowed to vary with depth to account for the effects of increasing average effective stress and decreasing overconsolidation ratio.

It is generally accepted that with a judicious choice of stiffness parameters, numerical analyses (eg finite element or finite difference) using a linear elastic-plastic soil model can lead to reasonable estimates of wall movements and bending moments (eg Burland and Kalra, 1986; Powrie *et al*, 1999). However, the calculation of realistic ground movements requires the use of a more complex soil model that better represents the degradation of stiffness with strain indicated in section 5.4.5 (eg Jardine *et al*,1986; Simpson, 1992; Stallebrass and Taylor,1997; Atkinson, 2000). It is important to check, however, that the computed stresses do not take the soil beyond the strain range for which the stiffness parameters are chosen.

The stress changes associated with wall installation on the stiffness of the soil during subsequent excavation in front of the wall are a further consideration, see section 4.1.2.

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Appendix E Effects of wall installation

This Appendix summarises recent research into the effects of diaphragm wall and bored pile wall installation in overconsolidated clays, the results of which have been used to inform the general guidance given in section 4.1.2.

E1 CONCRETE DIAPHRAGM AND BORED PILE WALLS

Recent investigations into the effects of diaphragm and bored pile wall installation on lateral earth pressures have been reported by:

- Tedd *et al* (1984): field data from Bell Common retaining wall
- Powrie (1985): elastic stress analysis
- Gunn and Clayton (1992) and Gunn *et al* (1993): plane strain and axisymmetric finite element analyses
- Symons and Carder (1993): field data from three sites)
- Ng *et al* (1995): finite element analyses, Lion Yard, Cambridge
- Page (1995): centrifuge model tests
- de Moor (1994): 2D plan view finite element analyses of the installation of a number of diaphragm wall panels in sequence
- Ng and Yan (1998): 3D finite difference analysis of the installation of a single diaphragm wall panel
- Gourvenec and Powrie (1999): 3D finite element analyses of the installation of nine diaphragm wall panels in sequence
- Ng and Yan (1999): 3D finite difference analyses of the installation of three diaphragm wall panels in sequence
- Powrie and Batten (2000): axisymmetric analysis of a single bored pile.

During installation of an *in situ* concrete retaining wall, the soil adjacent to the wall is likely to be unloaded laterally (eg to the hydrostatic pressure of the fluid used to support the panel or pile bore), before being re-loaded when the concrete is poured. Powrie (1985) argued that this process would reduce the lateral earth pressure coefficient in an overconsolidated deposit to a value somewhat greater than one. At Bell Common (Tedd *et al*, 1984), the earth pressure coefficient in the London Clay fell during installation from 1.7 at a depth of 9 m and 1.5 at a depth of 12 m to approximately 1. However, in the Claygate Beds at a depth of 6 m, the initial lateral earth pressure coefficient of 1 was substantially unaffected.

In a finite element analysis of the installation of a single pile, Powrie and Batten (2000) calculated reductions in lateral earth pressure coefficient in the range 28 – 37% in strata where the initial lateral earth pressure coefficient was 1 or more, but an increase in the lateral earth pressure coefficient of up to 20% where the initial value was only 0.5. However, this analysis is only of limited value as it did not consider the effects of building adjacent panels. Generally, the degree of stress relief would be expected to increase with increasing wall panel length. Thus the lateral stress relief associated with a single diaphragm wall panel might be expected to be greater than for a single bored pile wall. However, the effect of installing subsequent piles or panels is less obvious.

Three dimensional finite element analyses by Gourvenec and Powrie (2000) confirm that the degree of lateral stress relief during wall installation decreases with panel length. They also indicate that the reduction in lateral stress that occurs during the installation of a single panel is to some extent reversed during the subsequent installation of the adjacent panels.

Figure E1 *Earth pressure coefficient profiles 1 m behind the centre of the primary panel during construction of the wall: 3D analysis with 5 m panels*

Figure E1 shows the changes in lateral earth pressure coefficient at a distance of 1 m from a panel resulting from the installation of that panel (panel 1), the adjacent panels (panels 2 and 3), and a further 3 panels on each side (giving a total of 9 panels). It may be seen from Figure E1 that the installation of a diaphragm wall in panels each 5 m long and 15 m deep resulted in overall reductions in lateral earth pressure coefficient from ~2.5 to ~2 at the top and ~1.4 to ~1 at the toe at a distance of 1 m from the line of the wall. In Gourvenec and Powrie's analyses with 5 m long, 15 m deep panels, wall installation had little effect on lateral earth pressure coefficients at distances in excess of 5 m from the wall (Figure E2).

Figure E2 *Earth pressure coefficient profiles normal to the centre of the primary panel following completion of the wall: 3D analysis with 5 m panels*

It should be noted that Gourvenec and Powrie's results may have been influenced by a relatively coarse finite element mesh (ie there were insufficient elements) and the locations at which pressures were noted may have been unrepresentative. Consequently, the reductions in earth pressure coefficient shown in Figures E1 and E2 may have been overestimated.

Symons and Carder (1993) describe field monitoring of the effects of wall installation at three sites (A, B and C) involving embedded walls installed in London Clay. Earth and water pressures were measured using push-in spade shaped pressure cells each fitted with an integral high air entry pneumatic piezometer. The pressure cells were installed at locations in front of and behind the proposed retaining wall up to 3 months prior to the construction of the wall. During installation, the blade of each cell was carefully oriented to measure the lateral stress acting towards the retaining wall. Empirical corrections were necessary to the measured values of earth pressure to allow for disturbance caused by the cell.

A 24 m deep contiguous bored pile wall (comprising 1500 mm diameter bored piles at 1700 mm spacings) was constructed at Site A. The spade cells were located 1.5 m from the wall. The ground conditions comprised up to 3 m depth of Made Ground overlying 3 m thickness of Claygate Beds overlying London Clay. A decrease in the average earth pressure coefficient, K, of about 10% was observed at this site.

Diaphragm walls were installed at sites B and C. T-shaped panels (consisting of a 4.0 m by 0.8 m front section and a 2.7 m by 0.8 m counterfort) were installed to a depth of 13.5 m at Site B. Planar panels (4 m in length by 0.6 m wide) were installed to a depth of 15 m at Site C. The ground conditions at Site B comprised London Clay overlain by 2.4 m thickness of gravel and Made Ground. At Site C, 6.9 m thickness of soft alluvial deposits overlay 2.2 m of gravel above the London Clay. The spade cells were installed 1.5 m from the T-shaped panels at Site B. Spade cells were not installed at Site C but changes in porewater pressures due to wall installation were measured in piezometers installed 1 m from the wall. The spade cells at Site B indicated a decrease in the average earth pressure coefficient of about 20% in the London Clay. The piezometers at Site C indicated reductions in porewater pressure during panel excavation under bentonite with a subsequent increase to above the pre-construction values after concreting. These porewater pressures then gradually stabilised to their pre-wall construction values.

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Appendix F Earth pressure coefficients

F.1 NUMERICAL PROCEDURE FOR CALCULATING EARTH PRESSURE COEFFICIENTS

The equations presented below are taken from EC7 (1995) Annex G. They have been simplified to account only for vertical walls, with a vertical surcharge on the retained side. The following symbols are used in the equations:

ϕ'	angle of shearing resistance of soil (degrees)
δ	wall/soil friction angle (degrees)
β	angle of ground surface to horizontal (degrees)

The coefficient of *horizontal* earth pressure, K_h is given by:

where,

$$K_h = \cos^2 \beta \cdot \left(\frac{1 + \sin\phi' \sin(2m_w + \phi')}{1 - \sin\phi' \sin(2m_t + \phi')} \right) \cdot \exp(2\nu \tan\phi') \qquad (F.1)$$

$$m_t = \frac{\cos^{-1}\left(-\frac{\sin\beta}{\sin\phi'} \right) - \phi' - \beta}{2} \qquad (F.2)$$

$$m_w = \frac{\cos^{-1}\left(\frac{\sin\delta}{\sin\phi'} \right) - \phi' - \delta}{2} \qquad (F.3)$$

$$\nu = m_t + \beta - m_w \qquad (F.4)$$

and

m_t, m_w and ν have units of degrees. However, ν must be converted into *radians* before substitution into equation F.1.

For calculation of *active* earth pressure coefficients, the angle of shearing resistance of the soil and the wall/soil friction angle must be entered as *negative* values. For calculation of *passive* earth pressure coefficients *positive* angles should be used.

For both active and passive earth pressure coefficients the value of β is *positive* for a ground level which *increases* with distance from the wall.

These equations have been used to derive the horizontal earth pressure coefficient charts given in Figures F1 to F9.

Figure F1 *Active and passive earth pressure coefficients, (β / ϕ') = -1*

Figure F2 *Active and passive earth pressure coefficients, (β / ϕ') = -0.75*

Figure F3 *Active and passive earth pressure coefficients, (β / ϕ') = -0.5*

Figure F4 *Active and passive earth pressure coefficients, (β / ϕ') = -0.25*

Figure F5 *Active and passive earth pressure coefficients, (β / ϕ') = 0*

Figure F6 *Active and passive earth pressure coefficients, (β / ϕ') = 0.25*

Figure F7 *Active and passive earth pressure coefficients, (β / ϕ') = 0.5*

Figure F8 *Active and passive earth pressure coefficients, (β / ϕ') = 0.75*

Figure F9 *Active and passive earth pressure coefficients, (β / ϕ') = 1*

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Appendix G Comparison of lumped moment factors with factor on strength method in limit equilibrium calculations

G1 TRADITIONAL DESIGN METHODS FOR EMBEDDED WALLS

These are described in Annex B, BS 8002 (1994). The depth of embedment has been traditionally determined from a ULS stability assessment based on limiting equilibrium methods of analysis in which failure is postulated and a factor of safety applied to ensure that such a failure does not occur. This is done in three ways:

(a) by using a multiplying factor to increase the embedment depth of that required for limiting equilibrium - the *factor on embedment* method. This method is not recommended and is not considered further

(b) by applying a *factor on soil strength*, see section 5.8

(c) by factoring the gross or net pressures on the wall by a *lumped factor*. Moment equilibrium is used to ensure that restoring moments exceed overturning moments by a prescribed safety margin. With this approach:

$$\text{Factor of safety, F} = \frac{\text{restoring moments}}{\text{overturning moments}}$$

With such lumped factor methods, scaling down the moments due to passive earth pressures on the restraining side of the wall provides a factor of safety against overturning. There are three different methods and they are summarised below.

Figure G1 *Methods of assessing the ratio of restoring moments to overturning moments*

Gross pressure, F_p

The method is described in CP2, (1951). Unmodified limiting earth pressures are calculated against a retaining wall, see Figure G1(a). To ensure the factor is not applied to water pressures, the net active water pressure is calculated. A wall embedment depth is then selected to satisfy:

$$\boxed{\text{Moment of passive earth pressure}} \div F_p = \boxed{\text{Moment of active earth pressure}} + \boxed{\text{Moment of net active water pressure}}$$

Net pressure, F_{np}

This method is described in the British Steel (1997) *Piling Handbook*. The limiting active and passive pressures, including water pressure, are modified to give a net pressure distribution. The net pressures are the unshaded pressure profiles shown on Figure G.1(b). Wall embedment is then selected to satisfy:

Moment of net passive pressure (earth + water)	$\div F_{np} =$	Moment of net active pressure (earth + water)

Available passive resistance, F_r

This method was derived from an analogy with a simple bearing capacity problem. It is described in detail by Burland *et al* (1981). The method evaluates modified earth pressure distributions and a net active water pressure distribution. Figure G1(c) shows typical modified earth pressure distributions. For effective stress analysis the pressure distributions may be derived by drawing a vertical line from the level of the excavation on the limiting active pressure diagram and shading any pressures above that value. This shaded area is then subtracted from the passive resistance to give the modified pressure distribution. The modified pressures are shown by the unshaded areas in front and behind the wall in Figure G1(c). Where a value of c' is used and for total stress analysis, the designer should refer to Burland *et al* (1981) to derive the relevant pressure distributions. Wall embedment is then selected to satisfy:

Moment of modified passive earth pressure	$\div F_r =$	Moment of modified active earth pressure	+	Moment of net active water pressure

G2 FACTORS

Table G1 lists the magnitude of the ULS factors commonly adopted in the design of permanent works for the methods described in items G1 (b) and G1 (c) above. The factor on strength method, F_s, is included for comparison purposes. The F_s factors listed in Table G1 are those recommended by BS 8002 (1994).

Table G1 *Commonly adopted ULS factor values*

Method	Effective stress analysis			Total stress analysis
	$\phi' \leq 20°$	$20° < \phi' \leq 30°$	$\phi' > 30°$	all s_u
Factor on strength, F_s (BS 8002, 1994)	1.2 (1)	1.2 (1)	1.2 (1)	1.5 (2)
Gross pressure, F_p (CP2, 1951)	1.5	1.5-2.0	2.0	2.0
Net pressure F_{np} (British Steel, 1997)	2.0	2.0	2.0	N/A
Net available passive resistance F_r (Burland *et al* 1981)	2.0	2.0	2.0	2.0

Notes

(1) Applied to representative (moderately conservative) values of c' and $\tan \phi'$.

(2) Applied to representative (moderately conservative) values of s_u.

G3 COMPARISON OF METHODS

Comparison of the methods described in section G1 (c) with the factor on strength method has been reported by Burland *et al* (1981) and by Carder (1998). This Appendix extends this work.

A number of design situations were assumed with uniform soil on both sides of the wall. Drained (effective stress analysis) and undrained (total stress analysis) conditions were assumed with a range of surcharge loads on the retained side. A cantilever wall and a wall which was singly propped near its top were considered (Figure G2). No groundwater and full linear seepage conditions were assumed. Figure G2 also shows the range of variables considered.

Figure G2 *Wall geometry for general analysis case and range of variables*

For each of the above design situations, the wall embedment depth was determined using limit equilibrium analysis adopting the factor on strength method with $F_s = 1.2$ and 1.5 applied to moderately conservative values of tan ϕ' and s_u respectively. For each calculated embedment depth, the corresponding factors of safety were derived using the Gross Pressure (F_p), Net Pressure (F_{np}) and Available Passive Resistance (F_r) methods and compared with values which are commonly adopted using these methods (Table G1). In this way, the relative economy of using the factor on strength method can be compared with the more traditional limit equilibrium methods of determining wall embedment depth.

Results are summarised in Table G2 and Figures G3, G4 and G5.

Table G2 *Summary of equivalent factors of safety*

Analysis case	Wall type	Ground conditions	Water conditions	Equivalent factors of safety			
				F_p	F_{np}	F_r	F_s
1.	Cantilever	Drained	Linear seepage	1.35 - 1.6 for 20°<ϕ'<40°	1.85 - 2.0 for 20°<ϕ'<40°	1.4 - 1.65 for 20°<ϕ'<40°	1.2
2.	Cantilever	Drained	Dry	1.35 - 1.85 for 20°<ϕ'<40°	1.85 - 2.1 for 20°<ϕ'<40°	1.45 - 1.9 for 20°<ϕ'<40°	1.2
3.	Cantilever	Undrained	N/A	1.6 for s_u>75kPa H<6.0 m	N/A	1.6 for s_u>75kPa H<6.0 m	1.5
4.	Propped	Drained	Linear seepage	1.2 - 1.7 for 15°<ϕ'<40°	2.45(2) - 2.55(2) erratic	1.4 - 1.9 for 15°<ϕ'<40°	1.2
5.	Propped	Drained	Dry	1.25 - 1.9 for 15°<ϕ'<40°	2.35 - 2.5 erratic	1.4 - 1.9 for 15°<ϕ'<40°	1.2
6.	Propped	Undrained	N/A	1.5 for s_u>75kPa H<6.0 m	N/A	1.5 for s_u>75kPa H<6.0 m	1.5

Notes

(1) Surcharge, q = 10 kPa applied on retained side

(2) Erratic values. Dependent upon geometry, eg prop position

(3) Results insensitive to retained height in drained conditions and for H<6.0 m in undrained conditions

(4) Results insensitive to variation in surcharge loading from zero to 25 kPa

Figure G3 *Equivalent F_r, values for F_s = 1.2*

Figure G4 *Equivalent F_{np}, values for F_s = 1.2*

Figure G5 *Equivalent F_p, values for F_s = 1.2*

For the range of parameters and design situations considered, the following conclusions are drawn:

- In drained (effective stress) conditions, a wall embedment depth corresponding to $F_s = 1.2$ is equivalent to F_p and F_r values which are smaller than those which are commonly adopted with these methods. Thus, adopting the factor on strength method with $F_s = 1.2$ applied to moderately conservative values of tan ϕ' will lead to a shorter wall
- In drained (effective stress) conditions, F_{np} values significantly in excess of 2.0 are derived for embedment depths corresponding to $F_s = 1.2$ for singly propped walls. Furthermore, F_{np} values vary with the proximity of the prop to the top of the wall. Use of $F_{np} = 2.0$ in conjunction with moderately conservative soil strength parameters for propped walls may result in an inadequate level of safety. If this method is used, eg due to the explicit requirements of project specifications, worst credible soil parameters should be adopted in design calculations. The results of such calculations should be compared with the factor on strength method as a confirmatory check on adequacy
- In undrained (total stress) conditions, a wall embedment depth corresponding to $Fs = 1.5$ is equivalent to F_p and F_r values which are smaller than those which are commonly adopted with these methods. Thus, adopting the factor are strength method with $F_s = 1.5$ applied to moderately conservative values of s_u will lead to a shorter wall.

The use of the factor on strength method with $F_s = 1.2$ (effective stress) and $F_s = 1.5$ (total stress) is recommended in BS 8002 (1994). Experience gained over recent years indicates that walls designed on this basis have not exhibited any increased problems of instability. The use of these factors is therefore recommended in this report.

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BRITISH STEEL (1997)
Piling Handbook Seventh edition

BS 8002: 1994
Code of practice for earth retaining structures British Standards Institution 1994 amended 2001, London

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The overall stability of free and propped embedded cantilever retaining walls Ground Engineering, 14 No. 5, 28-38

CARDER, DR (1998)
A comparison of embedded and conventional retaining wall design using Eurocode 7 and existing UK design methods TRL Report 320, TRL Limited, Crowthorne

CP2 (1951)
former UK code of practice for retaining walls superseded by BS8002 (1994)

Appendix H Review of current wall design methods to BS 8002 (1994), EC7 (1995) and BD42/00 (2000)

H1 UK CODES OF PRACTICE AND DESIGN STANDARDS

Section 1.1 of the report lists the documents which provide guidance on the design of embedded retaining walls in the UK. Principal amongst these are the following:

- BS 8002 (1994) *Code of practice for earth retaining structures*. The latest amendment to this code was issued in September 2001.
- DD ENV 1997-1 Eurocode 7 (1995) *Geotechnical design*. This document is currently under revision.
- BD42/00 (Design Manual for Roads and Bridges, DMRB 2.1.2) *Design of embedded retaining walls and bridge abutments*
- CIRIA report 104 (Padfield and Mair, 1984) *Design of retaining walls embedded in stiff clays*
- British Steel (1997) *Piling Handbook* 7th edition.

In this Appendix, the methods advocated in BS 8002 (1994), EC7 (1995) and BD42/00 (DMRB 2.1.2) are described and compared.

H2 BS 8002 (1994)

A general introduction to BS 8002 (1994) is given in Akroyd (1996) and its design approach is described by Bolton (1996).

H2.1 Applicability

When published in 1994, BS 8002 (1994) was primarily applicable to small to medium sized earth retaining structures with a retained height of up to approximately 8 m. The latest revision of BS 8002 (September 2001) amends the applicability of the code to walls with a retained height of up to about 15 m. The code requires specialist advice to be sought with regard to the design and construction of larger structures and for those where movement of the retained soils requires close control.

H2.2 Design philosophy

The code of practice adopts the philosophy of limit state design. It is based on the use of limit equilibrium methods of retaining wall design. This method adopts theoretical limiting earth pressures without practical proof that they would co-exist at the wall's ultimate limit state. As BS 8002 is based on this approach, it cannot define uniquely the limit states that are to be used for design. This is seen in its definition for limit state design in that *the safety and stability of the retaining wall may be achieved, whether by overall factors of safety, or partial factors of safety, or by other measures* (Clause 3.1.2, BS 8002). Owing to this lack of precision in definition, BS 8002 refers both to partial factor based limit state codes of practice such as BS 8110 (1985), BS 5400 (1990) and also to BS 449 Part 2 (1969) which is based on the use of lumped factors of safety.

H2.3 Soil design parameters

BS 8002 uses an approach based on soil and groundwater parameters which tend towards worst credible values to develop an adequate margin of safety. A mobilised soil strength is advocated for use in design at the serviceability limit state (so as to limit retaining wall movements), because BS 8002 states that the most severe earth pressures that can credibly occur on a retaining wall, occur at that limit state. The design soil strength is obtained by dividing the *representative* peak strength by a mobilisation factor, M, or by adopting the *representative* critical state strength of the soil, whichever is the smaller.

The *representative* soil strength is defined in Clause 1.3.17 of BS 8002 as a *conservative estimate of the mass strength of the soil. The value is determined from reliable site investigation and soil test data* or, in the absence of such data, from typical values presented in chapter 5 of this report. Clause 1.3.2 of BS 8002 defines conservative values *as values of soil parameters which are more adverse than the most likely values. They may be less (or greater) than the most likely values. They tend towards the limit of the credible range of values*. This definition means that representative values are essentially the same as the moderately conservative values defined in section 5.9 of this report.

BS 8002 (1994) states 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 mobilisation factor, M of not less than 1.5. For designs using effective stress strength, the value of M is 1.2 to reduce horizontal wall movements to 0.5% of wall height. For the calculation of wall equilibrium and its structural components, BS 8002 (1994) requires design to be based on the *lesser* of (i) and (ii) below:

(i) Representative *peak* strength of the soil divided by a mobilisation factor:

- In effective stress terms:

$$\text{Design } \phi' = \phi'_{d} = \tan^{-1}\left(\frac{\tan \phi'_{\text{representative peak}}}{1.2}\right)$$

$$\text{Design } c' = c'_{d} = \frac{c'_{\text{representative}}}{1.2}$$

- In total stress terms:

$$\text{Design } s_u = s_{ud} = \frac{s_{u\ \text{representative}}}{1.5}$$

The value of the design wall adhesion, s_w is discussed further in section H2.5.

(ii) Representative *critical state* strength of the soil.

This applies only in terms of effective stress analysis.

$$\text{Design } \phi' = \phi'_{d} = \phi'_{\text{representative crit}}$$

$$\text{Design } c' = c'_{d} = \text{zero}$$

H2.4 Design groundwater pressure

Clause 3.2.2.3 of BS 8002 requires that the groundwater pressure assumed in design should correspond to *the most onerous that is considered to be reasonably possible*. This definition means that the design water pressure is equivalent to the worst credible value defined in section 5.9 of this report. This should allow for events such as potential damming to natural groundwater flow patterns and burst water mains adjacent to the retaining structure if the designer considers that these are reasonably possible over the design life of the wall. The assessment of groundwater pressures for use in design should be based on a consideration of the factors listed in section 5.5 of this report.

H2.5 Principal assumptions in design calculations

Loads

A minimum surcharge of 10 kPa is applied to the surface of the retained soil (Clause 3.3.4.1, BS 8002). All loads and surcharges are applied in an unfactored manner. The latest revision of BS 8002 (September 2001) permits the requirement for this minimum surcharge to be relaxed for walls retaining less than 3 m of earth, provided the designer is confident that a minimum surcharge of 10 kPa will not apply during the life of the structure.

Unplanned excavation

Additional unplanned excavation is applied in front of the wall. The latest revision of BS 8002 (September 2001) requires this to be the *lesser* of:

- 0.5 m; or
- 10% of the total height retained for cantilever walls, or of the height retained below the lowest level for propped or anchored walls.

The designer is permitted to adopt values which are more adverse than the above in *particular critical or uncertain conditions* and also to adopt smaller values than the above *where adverse conditions are beyond reasonable doubt*. The terms *critical* or *uncertain conditions, adverse conditions* and *reasonable doubt* are not defined in BS 8002. It is considered that, in general, the above additional unplanned excavation should be adopted in calculations to BS 8002.

Foreseeable excavation, eg service or drainage trenches in front of the retaining wall, are treated as planned excavation (Clause 3.2.2.2, BS 8002).

Passive softening

Clause 3.2.3 of BS 8002 (1994) requires the designer to allow for changes in loading associated with the construction of a retaining wall which may result in changes in the strength of the ground in the vicinity of the wall. This clause also notes that an inherent assumption in total stress analysis is that *there will be no change in the soil strength as a result of the changes in load caused by the construction.* This allows the designer to assume, where appropriate, that the undrained shear strength is not reduced beneath an excavation.

Wall friction and adhesion

- In effective stress analysis:

  The limiting value of wall friction in design, δ_d, is taken to be the *lesser* of (a) and (b) below.

  (a) the representative strength determined from large shear box testing (Clause 2.2.8, BS 8002) using the same mobilisation factors as for the adjacent soil (Clause 3.2.6, BS 8002). In the absence of such tests, the representative effective strength is:

  $\delta_d \leq \phi_{crit}$ for soil, for rough surfaces with a texture coarser than that of the median particle size

  $\delta_d = 20°$ for smooth surfaces with a texture finer than that of the median particle size.

  The effective adhesion, s_w', is taken as zero.

(b) 75% of the design shear strength:

$$\tan \delta_d = 0.75 \tan \phi'_d$$

where:

$$\tan \phi'_d = \frac{\tan \phi'_{\text{representative peak}}}{1.2}$$

The effective adhesion, s_w', is taken as zero.

- In total stress analysis:

  Design adhesion, $s_{wd} = 0.75\ s_{ud}$

  where:

$$s_{ud} = \frac{s_{u\ \text{representative}}}{1.5}$$

ie $\quad s_{wd} = 0.5\ s_{u\ \text{representative}}$

The designer should consider soil-structure interaction to ensure that the assumed values of wall friction and adhesion will be mobilised. In some circumstances, wall friction and adhesion can be reduced or eliminated altogether (eg if a geotextile membrane exists adjacent to the wall) or even reversed (eg vertical load applied to the wall), see Clause 3.2.6, BS 8002.

H2.6 Design method

The design earth pressures are active and passive derived from the *design* soil strengths using the appropriate earth pressure coefficient (K_a and K_p). Simple active and passive diagrams (eg Figure 4 of BS 8002) are obtained from design earth pressures and used to determine a wall length which satisfies vertical, horizontal and moment equilibrium (ie there is no further "overall" factor of safety). Overall stability (eg slip failure mechanisms beneath the wall) is also checked using the same factored soil strengths. The design values of lateral earth pressure are intended to give an overestimate of the earth pressure on the retained (active) side and an underestimate of the resisting earth pressures on the restraining (passive) side for small deformations of the structure as a whole in the working state. In some cases, the design earth pressure is greater than the pressure obtained from the coefficients of earth pressure, eg from compaction effects, swelling clays, wall rotating back into the retained ground - see Clause 3.1.9, BS 8002. Where these pressures occur, they will govern the wall design and the designer should ensure that the wall is designed to accommodate them.

H2.7 Compatibility with structural design codes

The earth pressures to be used in structural design should be determined as described in section H2.6 above. These pressures are considered to be the most unfavourable that are likely to occur and they are considered to occur under working conditions (Clause 3.2.7, BS 8002).

BS 8002 (1994) states that the earth pressures which occur at ultimate limit state are less than those which occur under working conditions. Clause 3.2.7, BS 8002, states that *the earth pressures at structural serviceability limit state and ultimate limit state will be similar for relatively rigid walls, such as mass gravity walls, because displacement criteria will be similar.* No specific mention is made of embedded walls or flexible walls. Clause 3.2.7, BS 8002, also states that *the strength of the structural sections of the retaining wall may be determined using either permissible stress methods or by the methods used in partial factor structural codes.* No further guidance is provided in BS 8002 regarding design to structural codes. The concept that ULS earth pressures will be less than or, at least, similar to SLS earth pressures does not comply strictly with the assumptions made in structural design calculations to BS 8110, BS 5400 and BS 5950 which assume that load effects at ultimate limit state are much greater than those at serviceability limit state.

BS 8110 and BS 5400 require partial safety factors, γ_f, of 1.4 and 1.5 respectively to be applied to serviceability loads on structures arising from earth and water for the ultimate limit state. BS 8002, on the other hand, implies that γ_f for such loads should be 1.0. This inconsistency causes confusion amongst structural designers of retaining walls.

A proposal by Beeby and Simpson (2001) for an amendment to the partial safety factors applied to soil pressures derived from BS 8002 in structural design to BS 8110 recommends:

$\gamma_f = 1.0$ when unplanned excavation is included in design to BS 8002

$\gamma_f = 1.2$ when no unplanned excavation is assumed in design.

The situation regarding structural design to BS 5400 using load effects derived from BS 8002 is unclear. However, the design of retaining walls supporting Highways Agency structures is required to comply with BD42/00 (DMRB 2.1.2), see section H4.

H3 EUROCODE EC7 (1995)

DD ENV 1997-1 Eurocode 7 *Geotechnical Design* was issued in 1995 by the British Standards Institution as a draft pre-standard.

Simpson and Driscoll (1998) describe the general background to understanding the use of Eurocode 7 together with a detailed clause by clause commentary and worked examples illustrating the application of EC7 (1995) to engineering design problems.

At the time of compiling this report, a further draft of EC7 was in preparation. This Appendix considers the requirements of the current version of EC7 (DD ENV 1997-1: 1994). This is generally referred to as EC7 (1995) in this report.

H3.1 Applicability

EC7 (1995) is part of a series of Eurocodes which are intended to be used by engineers in general building and civil engineering design. According to Simpson and Driscoll (1998), *they are particularly relevant on projects involving international co-operation or competition, especially on publicly funded work, where it may become a legal requirement to accept designs which satisfy the Eurocodes.* A full list of Eurocodes is provided in Simpson and Driscoll (1998). The Eurocodes are an inter-dependent and consistent set of documents. For example, with respect to embedded retaining wall design, a sheet pile wall will be designed using EC7 and EC3 (*Design of steel structures*) whilst a concrete wall will be designed using EC7 and EC2 *(Design of concrete structures)*. Each Eurocode is supplemented by a *National Application Document* (NAD) of the country in which construction is to take place. In the UK, EC7 and the UK NAD are published together by the British Standards Institution.

According to Simpson and Driscoll (1998), *under the rules of CEN (Comité Européen de Normalisation), codes published by national standards institutions that conflict with the principles of the Eurocode must normally be withdrawn when a CEN standard is produced at EN status. However, an exception has been made for Eurocodes, allowing a period of coexistence of national standards with the EN for some years after publication of the EN. For EC7 a particular problem arises in deciding which British standards are in conflict with the EN. For example, BS 8002: 1994 contains material on the design of retaining walls which is not included in EC7. The same applies to other codes such as BS 8004, BS 8006, BS 5930, etc. The British codes provide much advisory information, which is informative rather than obligatory, so they are somewhat different in nature from the Eurocodes; it might therefore be questioned whether they conflict with the Eurocode*. EC7 (1995) has not yet achieved EN status. The design of embedded walls in the UK need not comply with the requirements of EC7 (1995). Nevertheless, it is important for designers to be aware of the requirements of EC7 (1995) as this may achieve EN status in the future.

H3.2 Design philosophy

EC7 (1995) adopts limit state design principles with a partial factor approach. It requires the selection of a geotechnical category for the structure or part of a structure under consideration, see Clause 2.2.2. EC7 (1995) adopts a two stage design procedure:

- establish *design situations*; and
- demonstrate that limit states will not be exceeded in the *design situations.*

Limit states are checked by considering *design situations* which are *sufficiently severe and so varied as to encompass all conditions which can be foreseen to occur during the execution and use of the structure* (Clause 2.3, EC1, 1994). Design situations are categorised as persistent, transient and accidental and the limit states which are relevant to these various situations may vary. For example, for an accidental situation which involves exceptional circumstances, the structure may be required merely to survive without collapse, in this case, serviceability limit states would not be relevant (Simpson and Driscoll, 1998).

For design by calculation EC7 (1995) does not restrict the means by which the designer demonstrates that limit states will not be exceeded in the design situations. It allows, for example, design by prescriptive measures (Clause 2.5, EC7, 1995) and by the adoption of the Observational Method (Clause 2.7, EC7, 1995).

For design by calculation, EC7 (1995) identifies three ULS design cases (A, B and C) for which each design should be verified, as relevant. EC7 (1995) makes it clear that all structures must satisfy all three cases, both structurally and geotechnically. Case A primarily relates to buoyancy problems, where water uplift forces comprise the main unfavourable action, and so would not be relevant for the vast majority of retaining wall designs. Case B is often critical to the design of the strength of structural elements. It is not applicable to problems where there is no strength of structural materials involved. Case C is generally critical for geotechnical stability of the structure and it is therefore often critical for the sizing of structural elements in geotechnical problems such as retaining wall design. For embedded walls, case C often gives the most severe bending moment and case B may be critical for prop loads and shear forces in the wall.

The partial factors that should be applied in each of the above cases are shown in Table H1. It is important to note that the partial factors on actions are multipliers and increase the applied forces (actions) while those on ground properties are dividers and reduce the ground strength.

Table H1 *Partial factors (γ_f and γ_m) - ultimate limit states in persistent and transient situations (after EC7, Table 2.1)*

Case	Actions			Ground Properties		
	Permanent		Variable	$\tan\phi'$	c'	s_u
	Unfavourable	Favourable	Unfavourable			
Case A	1.00	0.95	1.50	1.1	1.3	1.2
Case B	1.35	1.00	1.50	1.0	1.0	1.0
Case C	1.00	1.00	1.30	1.25	1.6	1.4
	Design action = action x factor			Design property = property/factor		

For accidental design situations, all numerical values of partial factors for actions and material strengths should be taken equal to unity.

H3.3 Soil design parameters

Design values of ground properties X_d should be derived from the *characteristic* value X_k (EC7, Clause 2.4.3) using the equation:

$$X_d = X_k/\gamma_m$$

Where γ_m is the relevant partial factor from Table H1.

EC7 Clause 2.4.3 (5) states that, when selecting *characteristic* values for soil properties, they should represent a ‘cautious estimate’ of the values which will govern the behaviour of the ground. This definition means that characteristic values are essentially the same as the moderately conservative values defined in section 5.9 of this report.

For ultimate limit states:

- Cases A and C: soil strength is factored in accordance with Table H1
- Case B: soil strength is unfactored (strictly, factor = 1.0)

For serviceability limit states, the factor applied to soil strength is unity.

H3.4 Design groundwater pressures

The considerations discussed in section 5.4 should form the basis of the assessment of groundwater pressures for use in design.

For ultimate limit states (ie cases A, B or C), *the design values for water pressures and seepage forces shall represent the most unfavourable values which could occur in extreme circumstances* (Clause 2.4.2 (10), EC7, 1995). This is equivalent to the worst credible value defined in section 5.9 of this report with due allowance for effects such as potential damming to natural groundwater flow patterns, burst water mains in close proximity to the wall, etc. This corresponds to case (i), section 5.5 of this report.

For serviceability limit states, the *design values shall be the most unfavourable values which could occur in normal circumstances* (Clause 2.4.2 (10), EC7, 1995). This is equivalent to the worst credible value defined in section 5.9 of this report but with no allowance for extreme events such as an adjacent burst water main, etc, unless the designer considers that such events may reasonably occur in normal circumstances. This corresponds to case (ii), section 5.5 of this report.

H3.5 Principal assumptions in design calculations

Loads

There is no minimum surcharge. The designer is free to adopt loadings which are appropriate for the specific circumstances of the project.

For ultimate limit states, the following factors are applied to loads (surcharges):

- Case B

  factor = 1.0 [(1)] on permanent unfavourable loads

  factor = 1.1 [(2)] on variable unfavourable loads

  factor = 1.0 on favourable loads

  Notes:

  (1) This is (1.35/1.35) as a factor of 1.35 is applied to case B calculated load effects

  (2) This is (1.50/1.35) as a factor of 1.35 is applied to case B calculated load effects

- Case C

  factor = 1.0 on permanent unfavourable loads

  factor = 1.3 on variable unfavourable loads

factor = 1.0 on favourable loads

The above factors are derived from Table H1 and Clause 2.4.2 (17), EC7 (1995).

Unplanned excavation

For ultimate limit states, a minimum *additional* unplanned excavation is applied in front of the wall. This is taken as the *lesser* of:

- 0.5 m; or
- 10% of the total height retained for cantilever walls, or the height retained below the lowest support level for propped or anchored walls.

For serviceability limit states, unplanned excavation is not included.

Passive softening

In total stress analysis, there is no explicit requirement for the designer to reduce the value of undrained shear strength beneath an excavation in front of the wall to allow for the effects of disturbance and softening due to partial drainage. Instead, EC7 (1995) Clause 8.3.3 (1) requires the designer to consider the *variation in soil properties with time and space*. Any allowance for passive softening is therefore a matter for the designer based on previous experience of comparable construction in similar ground conditions.

Wall friction and adhesion

- In effective stress analysis

  The limiting value of wall friction, δ, is taken to be:

  $$\delta \leq k\ \phi'_{\mathrm{crit}}$$

  where

  k = 1.0 for *rough* concrete (eg concrete cast against soil)

  k = 0.67 for *smooth* concrete (eg precast concrete or concrete cast against formwork) or sheet piling supporting sand or gravel.

  The effective adhesion, s_w', is taken as zero.

  Strictly the above values are for sands and gravels, but there seems to be no reason for this apparent restriction. Simpson and Driscoll (1998) note that it may be difficult to satisfy the condition of vertical equilibrium if $\delta = \phi'$ is used on both sides of the wall.

- In total stress analysis

  No specific guidance is provided in EC7 (1995) in respect of wall adhesion, except in Clause 8.5.1 (4) which states that adhesion, $a = 0$ immediately after driving a steel sheet pile in clay under undrained conditions.

In selecting the values of wall friction and adhesion to be used in design, the designer should consider the roughness of the wall, relative vertical movements and the need to preserve vertical equilibrium.

H3.6 Design method

The simplest approach to the use of EC7 (1995) for retaining wall design is to solve first for case C which uses factored design strength for soils but which leaves permanent (dead load) actions unfactored and variable (live load) actions factored with a factor of 1.3. This usually gives the depth of retaining wall that is required. Where the strength of structural elements is important, case B then needs to be re-checked where soil strengths are not factored but actions (including earth pressures calculated for newly placed fill) are factored. Forces from soil such as earth weight and earth pressure can in virtually all cases be considered as permanent while surcharges can be considered as variable. This initial design should then be checked again to confirm that cases A, B and C have all been satisfied.

ULS load effects (bending moments, shear forces and prop loads) for use in the structural design of the wall are obtained as the *greater* of those determined from:

- case C analysis
- 1.35 x case B load effects.

For verification of serviceability limit states, partial factors of unity are used for all ground properties and permanent and variable actions (Clause 2.4.2 (18), EC7, 1995). This yields SLS load effects.

The design of the structural elements is then carried out using compatible codes (eg EC2, 1992 for reinforced concrete design and EC3 (1998) for steel structures such as sheet piling).

H4 BD 42/00 (DMRB 2.1.2)

Retaining structures supporting or adjacent to highways are usually required to be designed to the appropriate Highways Authority Departmental Standard in the Design Manual for Roads and Bridges (DMRB). This means that all such retaining structures should be designed to comply with the requirements of BD 42/00 *Design of embedded retaining walls and bridge abutments.* This standard forms part 2 of section 1 *(substructure)* of volume 2 of *Highway Structures*: *Design (substructures, special structures and materials).*

H4.1 Applicability

The scope of BD42/00 (DMRB 2.1.2) includes cantilever and singly and doubly propped anchored walls embedded in stiff clay, firm clay and granular soils. BD 42/00 (DMRB 2.1.2) only applies to the design of walls forming the permanent works. It is not applicable to temporary works design. For permanent walls which are subject to loading during construction, Clause 2.12 of the standard requires the design to *accommodate the intended method and sequence of construction with separate checks on the adequacy of the design made at each stage.*

In common with BS 8002 (1994), BD 42/00 (DMRB 2.1.2) is based on limit equilibrium methods of analysis. The use of numerical analytical methods, although permitted by the standard, is specifically excluded from its scope.

The standard does not apply to the determination of vertical load capacity of the wall. Clause 1.9 of BD 42/00 (DMRB 2.1.2) states that *design for vertical load capacity of the walls and individual piles shall be in accordance with BS 8004 Foundations as implemented by BD 74.*

The standard also provides general guidance on the use of the Observational Method and the use of soil berms for temporary support.

H4.2 Design philosophy

BD 42/00 implements relevant parts of BS 8002 (1994) Annex B (*traditional design methods for embedded walls*) and CIRIA 104 (Padfield and Mair, 1984) but adopts limit state principles compatible with BS 5400 (1990). It recommends that the factors and methods listed in Table 5 of CIRIA report 104 are used to determine wall embedment and structural strength. This table is reproduced as Table H2. In addition to these methods, BD 42/00 (DMRB 2.1.2) also permits the use of the "net total pressure" method described in Appendix G and shown in Figure G1 (b). The use of this method is, however, qualified to apply *only in predominantly granular soil conditions, provided worst credible soil parameters are used in the calculations. A factor of safety of not less than 2 is required. The design shall be checked against at least one of the recommended methods in CIRIA 104* (Clause 3.1 (iv), BD 42/00). The use of lumped and partial factors is permitted. As discussed in Appendix G, use of $F_{np} = 2.0$ in the design of propped walls may result in an inadequate level of safety. The results of such calculations should be compared with the factor on strength methods as a confirmatory check on adequacy.

BD 42/00 (DMRB 2.1.2) assumes that the adoption of the factors listed in Table H2 and $F_{np} \geq 2$ in coarse grained soils *will normally satisfy the serviceability limit state of deformation* (Clause 3.1 (ii), BD 42/00). Nevertheless, BD 42/00 (Clause 3.3) requires the designer to carry out an assessment of the likely ground movements *from relevant field data and from experience of similar structures in similar ground conditions.* Furthermore, the designer is required to carry out a deformation analysis *for both the construction and working load stages* (Clause 3.4, BD 42/00). No guidance is given as to how this deformation analysis should be undertaken.

H4.3 Soil design parameters

Moderately conservative and worst credible soil parameters are adopted in design, as appropriate, in conjunction with the methods and factors listed in Table H2.

H4.4 Design groundwater pressures

There is no specific Clause in BD 42/00 (DMRB 2.1.2) which is solely devoted to the groundwater pressures to be assumed in design. There are, however, references to groundwater pressures in a number of Clauses:

- during construction, Clause 2.12 requires the wall design to allow for the *ingress of water into tension cracks in clay soils formed in the retained ground at the soil structure boundary*. The Clause notes that *a water filled crack can theoretically extend to a very much greater depth than a dry crack*
- Clauses 2.13 and 3.10 require the wall to be designed for the new groundwater flow pattern arising from the construction of the wall and any long term changes in pore water pressures

- Clauses 3.1 (i) and 3.6 require the use of worst credible groundwater pressures in consideration of the ultimate limit state
- at the serviceability limit state of structural elements, Clause 3.10 requires the use of *the greatest pressures and loads likely to act on the structure during its design life*, taking into account *any long term changes in pore water pressures*.

From the above and the intention of the standard to implement relevant parts of BS 8002 (1994) Annex B and CIRIA report 104 compatible with BS 5400 (1990), it is considered that BD 42/00 (DMRB 2.1.2) requires that worst credible groundwater pressures be adopted in design. These water pressures should be the same as those applied in BS 8002 (1994), see section H2.4.

Table H2 *BD 42/00 (DMRB 2.1.2) recommended methods for determining a stable wall geometry in stiff clays (after CIRIA report 104, Table 5)*

Method		Design Approach A		Design Approach B	
		Recommended range for moderately conservative parameters (c', ϕ', or s_u)		Recommended minimum values for worst credible parameters (c' = o, ϕ')	
		Temporary Works	Permanent Works	Temporary Works	Permanent Works
1. 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
	Total stress*	2.0	-		-
2. Strength factor method F_s	Effective Stress	1.1 to 1.2 (usually 1.2 except for ϕ'>30^o when lower value may be used)	1.2 to 1.5 (usually 1.5 except for ϕ'>30^o when lower value may be used)	1.0	1.2
	Total stress*	1.5	-	-	-
3. 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
	$\phi' \geq 30^o$	1.5	2.0	1.0	1.5
	ϕ'= 20-30^o	1.2 to 1.5	1.5 to 2.0	1.0	1.2 to 1.5
	$\phi' \leq 20^o$	1.2	1.5	1.0	1.2
	Total stress*	2.0	-	-	-
4. 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
	Total stress*	2.0	-	-	-

* Total stress factors are speculative, and they should be treated with caution.

Notes:

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.

2. Only methods 2 (strength factor method) and 3 (CP2 method) should be used to calculate the stability of walls which are singly propped near excavation level or have a stabilising base near excavation level. Methods 1 (factor on embedment) and 4 (Burland-Potts method) do not apply to these types of walls.

H4.5 Principal assumptions in design calculations

Loads

Unfactored highway loading as given in BD 37/88 (DMRB 1.3) is applied at relevant points and at appropriate levels in combination with unfactored permanent and other loads (eg parapet loads). The partial load factors for earth pressure given in BD 37/88 (DMRB 1.3) are not applied with these load combinations. Live load surcharge is applied on the retained side of the wall, as described in Clause 5.8.2 of Appendix A of BD 37/88 (DMRB 1.3).

Unplanned excavation

There is no explicit statement in BD 42/00 (DMRB 2.1.2) regarding how much provision (if any) the designer should make for additional unplanned excavation. Clause 2.12 states that *allowance shall be made in the design.... for any over excavation that may be involved in the construction*, while Clause 3.6 requires the worst credible geometry to be considered to calculate ULS load effects (wall bending moments, shear forces and prop loads) for use in structural design to BS 5400 (1990). No such reference to worst credible geometry is made in Clause 3.10 which deals with the calculation of SLS load effects.

From the above, it is concluded that unplanned excavation complying to the requirements of BS 8002 (1994) should apply at ULS while any provision for unplanned excavation at SLS is left to the discretion and judgement of the designer.

Passive softening

In total stress analysis BD 42/00 (DMRB 2.1.2) (Clauses 7.9, 8.7 and 9.8) requires the designer to adopt appropriate undrained shear strength (s_u) parameters *to account for effects such as softening*. BD 42/00 (DMRB 2.1.2) refers the designer to CIRIA report 104 for more guidance. CIRIA report 104 recommends that for excavations in London Clay, the value of s_u should be reduced to zero at excavation surface increasing to 70% - 80% of its undisturbed value at a depth of 1m below excavation level. For other overconsolidated stiff clays, CIRIA report 104 states that any allowance for passive softening is a matter for the designer based on previous experience of comparable construction in similar ground conditions.

Wall friction and adhesion

There is very little guidance on this matter in BD 42/00 (DMRB 2.1.2). Clause 1.9 notes that *vertical loading on the wall might reduce or reverse the direction of wall friction and result in increased earth pressure on the retained side*. For walls propped at the top embedded in heavily over-consolidated clays ($K_o \geq 1.5$), Clause 3.10 (iii) requires the design to be based on a *limit equilibrium analysis assuming full passive pressures with full wall friction acting on the excavated side*. There is no other reference to wall friction or adhesion in BD 42/00 (DMRB 2.1.2).

The intent of BD 42/00 (DMRB 2.1.2) is to implement the relevant parts of BS 8002 (1994) Annex B and CIRIA report 104. BS 8002 Annex B does not provide guidance on the limiting values of wall friction or adhesion to be assumed in design other than to state that the *wall friction, δ_m, and adhesion, s'_{wm}, should be determined by maintaining a constant value of the ratios (δ_m / ϕ'_m) and (s'_{wm} / c'_m)* equal to the assumed values of (δ / ϕ') and (s'_w / c') (section B4, Annex B, BS 8002, 1994). In the absence of specific guidance, it is considered that the recommendations of CIRIA report 104 (Padfield and

Mair, 1984) should generally apply in this respect, except for walls which fall within the remit of Clauses 3.10 (iii) and 7.12 of BD 42/00 (DMRB 2.1.2).

- In effective stress analysis

  The limiting value of wall friction, δ, is taken to be:

  $(\delta / \phi') \leq k$

  where:

$k = 0.67$	on the retained (active) side
$k = 0.5$	on the excavated (passive) side
$\phi' = \phi'_{crit}$	this is considered to be appropriate in the absence of specific guidance in CIRIA report 104.

  The effective adhesion, s'_w, is taken as $(s'_w / c') \leq 0.5$.

- In total stress analysis

  $(s_w / s_u) \leq 0.5$

  where:

$s_w \leq 50$ kPa	on the retained (active) side
$s_w \leq 25$ kPa	on the excavated (passive) side.

H4.6 Design method

Guidance on the determination of wall embedment (overall stability) and load effects (wall bending moments, shear forces and prop loads) for use in structural design to BS 5400 (1990) is provided with regard to wall type. The following wall types are considered in BD 42/00 (DMRB 2.1.2):

- cantilever walls and walls propped at the top
- walls with a single prop or stabilising base near excavation level
- walls which are doubly propped.

Each of these wall types is discussed further below.

Cantilever walls and walls propped at the top

(a) Wall embedment

Wall embedment is determined using moderately conservative parameters (design approach A) in accordance with one of the four methods listed in Table H2 (Clause 3.1 (i), BD 42/00). The wall embedment is checked for adequacy using worst credible parameters (design approach B) for the same method (Clause 3.1 (i), BD 42/00). This can be achieved in one set of calculations if the factor on soil strength, F_s, method is used by comparing the design values from design approaches A and B and adopting the worst of the two. A further check is then carried out using at least one of the other methods listed in Table H2 (Clause 3.1 (i), BD 42/00).

(b) Load effects for structural design

ULS load effects for use in the structural design of the wall are calculated from a limit equilibrium analysis assuming worst credible soil parameters, groundwater and geometry. The resulting ULS bending moments and shear forces are multiplied by a partial load factor, $\gamma_{fl} = 1.0$, and then by the appropriate value of γ_{f3} from BS 5400: parts 3, 4 and 5 (as implemented by BD 13, BD 24 and BD 16). A partial load factor, $\gamma_{fl} = 2.0$, multiplied by the appropriate value of γ_{f3} for concrete or steel, is applied to the calculated prop loads to determine the prop load for structural design.

BD 42/00 (DMRB 2.1.2) recognises that the earth pressures mobilised in the retained ground at small deformations under working conditions corresponding to the SLS may exceed the earth pressures mobilised at larger ground deformations corresponding to the ULS. Therefore, to avoid underdesign of the structural elements, BD 42/00 (DMRB 2.1.2) requires the designer to calculate load effects at the SLS to check the adequacy of the structural capacity. This is done by a limit equilibrium analysis assuming worst credible soil parameters and groundwater but not the worst credible geometry. The resulting bending moments and shear forces are multiplied by the following partial factors, γ_{fl}, given in Table H3 and then by the appropriate value of γ_{f3} as described above to obtain the ULS design load effects:

Table H3 *Partial factors γ_{fl}*

Wall type	Partial factor, γ_{fl}
Cantilever walls in all soils	1.3
Walls propped at the top in soils other than heavily overconsolidated clays ($K_o < 1.5$)	1.0
Walls propped at the top in heavily overconsolidated clays* ($K_o > 1.5$)	1.0

* earth pressures acting on the retained side in the long term are determined from Clause 3.10 (iii) of BD 42/00 (DMRB 2.1.2)

With respect to props, a partial load factor, $\gamma_{fl} = 1.5$, multiplied by the appropriate value of γ_{f3} for concrete or steel, as appropriate, is applied to the calculated SLS prop loads to obtain the ULS design axial force.

Walls with a single prop or a stabilising base near excavation level

(a) Wall embedment

This is determined adopting moderately conservative parameters (design approach A) in accordance with methods 2 (factor on strength method) and 3 (CP2 method) listed in Table H2 together with the factors associated with temporary works. BD 42/00 (DMRB 2.1.2) argues that the temporary construction stages are likely to govern the design of these types of walls. However, this may not always be the case and the designer should ensure that the wall embedment is adequate for short term and long term conditions.

(b) Load effects for structural design

These are determined from a limit equilibrium analysis using methods 2 and 3 of Table H2 together with the factors associated with temporary works. The load effects resulting from this analysis are multiplied by factors γ_{fl} and γ_{f3} as discussed in the section dealing with cantilever walls and walls propped at the top, except that the earth pressures acting on the retained side in the SLS are determined from Clause 7.12 of BD 42/00 (DMRB 2.1.2) which requires a K value of 1.0 to be adopted for walls retaining heavily overconsolidated clays ($K_o > 1.5$).

The designer should note that the above approach may be unconservative. It is recommended that for these types of walls, the designer should carry out both ULS and SLS calculations in accordance with the design method described in chapter 6 of this report as a confirmatory check on adequacy. The load effects used in structural design should be derived from section 6.6 of this report.

The design of the permanent structural prop near excavation level needs to take both axial forces and bending moments (from heaving of the underlying overconsolidated clay) in the long term (Clause 7.13, BD 42/00).

Walls which are doubly-propped

(a) Wall embedment

As described for walls with a single prop near excavation level.

(b) Load effects for structural design

As described for wall with a single prop near excavation level.

Clause 8.5 provides guidance on five methods of analysis for the design of temporary and permanent props:

(i) pressure envelope semi-empirical method (Terzaghi and Peck, 1967; BS 8002, 1994)

(ii) staged excavation - limit equilibrium method of analysis with hinges introduced at each prop position below the top prop so that each wall span is analysed as a simply supported beam (Williams and Waite, 1993; BS 8002, 1994)

(iii) as (ii) but assuming the wall acts as a continuous beam which is supported at some point below formation level (Tamaro and Gould, 1993)

(iv) numerical analysis closely modelling the construction sequence (Richards and Powrie, 1995)

(v) empirical method of design based on field measurements of prop loads (Twine and Roscoe, 1999).

BD 42/00 (DMRB 2.1.2) recommends that prop forces assessed from methods (i), (ii) and (iii) above should be increased by 25% for the upper prop and 15% for the lower prop to allow for soil arching and stress redistribution. **It is recommended that the designer determine prop loads in accordance with the method described in section 7.1.3 of this report.**

H4.7 Comparison of methods

The following references provide a comparison of the design of embedded walls using BS 8002, EC7 and BD 42 (1994 issue):

- TRL Report 320 (Carder, 1998) *A comparison of embedded and conventional retaining wall design using Eurocode 7 and existing UK design methods*
- Simpson and Driscoll (1998) *Eurocode 7: a commentary* Appendix 3 for a comparison of BS 8002 and EC7 design methods.

Table H4 summarises the principal assumptions made and the factors adopted in the design of retaining walls to BS 8002 (1994), EC7 (1995) and BD 42/00 (DMRB 2.1.2). For comparison purposes, only the factor on strength method from BD 42/00 has been considered as this is consistent with BS 8002 and EC7 and is the preferred method for the design of embedded retaining walls, see section 5.8 of this report.

It is important not to mix the assumptions and factors from the various codes and standards:

- BS 8002 (1994) requires the consideration of representative peak and critical state strength parameters. Both of these cases should be considered at the stage of selecting design parameters. The more onerous parameters should be selected and *one* design calculation should be undertaken
- by contrast, EC7 (1995) requires the consideration of three design cases A, B and C. Case A is usually not critical to the design of retaining walls. Two design calculations (cases B and C) are therefore usually required and the worst case is adopted in design
- BD 42/00 (DMRB 2.1.2) requires the consideration of moderately conservative and worst credible soil parameters (except for walls with a single prop or a stabilising base near excavation level) described above.

Table H4 *Comparison of retaining wall design assumptions and factors*

DESIGN[7] STANDARD / METHOD	SOIL PARAMETERS[8] TO WHICH FACTORS ARE APPLIED		FACTOR ON STRENGTH: SERVICEABILITY LIMIT STATE (SLS) c'	SLS s_u	SLS $\tan\phi'$	ULTIMATE LIMIT STATE (ULS) c'	ULS s_u	ULS $\tan\phi'$	PRINCIPAL ASSUMPTIONS IN DESIGN CALCULATIONS
BS8002 (1994) amended 2001	Representative peak strength		1.2	1.5	1.2	1.2	1.5	1.2	**Applicability** • BS 8002 applies to walls up to 15 m high **Groundwater** • Most onerous groundwater pressures that are considered "reasonably possible" **Surcharge** • Minimum surcharge of 10kPa on retained side. Lower values where walls retain less than 3 m and designer confident that a minimum surcharge of 10 kPa will not apply during life of structure **Unplanned excavation** • Lesser of (i) and (ii): (i) 0.5 m; or (ii) 10% of total height retained for cantilever walls or height retained below the lowest support level The designer can adopt more adverse or smaller values to suite particular circumstances **Limiting wall friction** • Values depend on vertical equilibrium requirements. Adopt *lesser* of (i) and (ii): (i) value determined from large shear box tests modelling soil/wall interface or, in the absence of such data: $\delta = \phi'_{crit}$ rough surface: texture coarser than median particle size $\delta = 20°$ smooth surface: texture finer than median particle size (ii) $\tan\delta = 0.75 \tan\phi'_d$ where, $\tan\phi'_d = \tan\phi'_{representative} / 1.2$ **Limiting adhesion** • $s'_w = 0$ • $s_{wd} = 0.75\, s_{ud}$ where, $s_{ud} = s_u / 1.5$ for wall displacements < 0.5% of wall height
	Representative critical state strength		N/A[3]	N/A	1.0	N/A	N/A	1.0	
EUROCODE 7 DD ENV 1997-1: 1995	Characteristic soil strength	SLS	1.0	1.0	1.0	N/A	N/A	N/A	All loads are unfactored **Applicability** • EC7 is not a full standard. It is a pre-standard **Groundwater** • ULS: most unfavourable which could occur in extreme circumstances • SLS: most unfavourable which could occur in normal circumstances **Surcharge** • No requirements for minimum surcharge but unfavourable loads are multiplied by a factor. This factor is 1.0 in SLS **Unplanned excavation** • ULS: *lesser* of (i) and (ii) in BS 8002 above • SLS: no explicit requirement for unplanned excavation. **Limiting wall friction** • Values depend on vertical equilibrium requirements. $\delta \le \phi'_{crit}$ for rough concrete (eg cast against soil) $\delta \le 0.67\,\phi'_{crit}$ for smooth concrete (eg precast concrete, cast against formwork), or steel sheet piling in sand or gravel **Limiting adhesion** • No specific guidance except $s_w = 0$ immediately after driving a steel sheet pile in clay
		ULS Case B	N/A	N/A	N/A	1.0	1.0	1.0	Unfavourable surcharge loads are factored: - Permanent: 1.0 - Variable: 1.1
		ULS Case C	N/A	N/A	N/A	1.6	1.4	1.25	Unfavourable surcharge loads are factored: - Permanent: 1.00 - Variable: 1.30
BD 42/00 (DMRB 2.1.2) (FACTOR ON STRENGTH METHOD)	Moderately conservative parameters	ULS (for wall stability check only)	N/A	N/A	N/A	1.1-1.5[1]	1.5	1.1-1.5[1]	**Applicability** • BD 42/00 (DMRB 2.1.2) applies to cantilever, singly and doubly propped walls embedded in stiff clay, firm clay and coarse grained soils. All structures designed for a 120 year design life **Groundwater** • As BS 8002: most onerous that are "reasonably possible" over the design life of the structure **Surcharge loading** • Live load surcharge is applied on the retained side in accordance with Clause 5.8.2, Appendix A, BD 37/38 (DMRB 1.3). Relevant highway loadings are unfactored **Unplanned excavation** • No specific guidance. See section H4.5 **Limiting Wall Friction** • Values depend on vertical equilibrium requirements • No specific guidance. See section H4.5 **Limiting adhesion** • No specific guidance. See section H4.5
	Worst credible parameters	ULS (for wall stability check only)	N/A[3]	N/A	N/A	N/A	N/A	1.2	
		SLS[4] load effects	N/A[3]	N/A	1.0[2]	N/A	N/A	N/A	
		ULS[5] load effects	N/A[3]	N/A	N/A	N/A	N/A	1.0	

Notes:

1. Permanent works during construction : $F_s = 1.1 - 1.2$ usually 1.2 except for $\phi' > 30°$
   Permanent works in the long term : $F_s = 1.2 - 1.5$ usually 1.5 except for $\phi' > 30°$ when 1.2 may be used
2. Except in heavily overconsolidated clays with $K_o \ge 1.5$ where Clause nos. 3.10 (iii) and 7.12 of BD 42/00 (DMBR 2.1.2) apply
3. $c' = 0$ for worst credible and critical state parameters
4. Provision for unplanned excavation at designer's discretion
5. With unplanned excavation as BS 8002
6. Design value for wall friction and adhesion will depend upon consideration of soil-structure interaction and vertical equilibrium of the wall
7. See sections H2-H4 for details of BS 8002 (1994), EC7 (1995) and BD 42/00 (DMRB 2.1.2)
8. See section 5.9 for explanation of representative, characteristic, moderately conservative and worst credible

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Appendix I Analysis of berms

I.1 MULTIPLE COULOMB WEDGE LIMIT EQUILIBRIUM ANALYSIS

I1.1 Total stress analysis

A modified limit equilibrium stress analysis for berm supported walls which, as well as providing a reasonable estimate of the lateral stresses exerted by the berm, enables the factor on soil strength to be determined for any given wall or berm geometry and soil strength properties, is presented by Daly and Powrie (submitted). This analysis is based on the undrained shear strength of the soil, and is therefore applicable only to clay soils in the short term, before any significant drainage or negative excess pore water pressure dissipation occurs.

In the modified limit equilibrium approach, the distribution of passive resistance provided by the berm is represented more closely than in any of the methods described in section 7.2.1 by carrying out Coulomb wedge analyses over the entire depth of the wall rather than just at the toe. The multiple Coulomb wedge method is also used to represent berms in some soil-structure interaction computer programs, and is summarised in the flow diagram given in Figure I1.

Figure I1 *Modified equilibrium analysis approach: multiple Coulomb wedge analysis (undrained) in terms of total stresses for a retaining wall supported by an earth berm*

A typical limit equilibrium analysis involves the following steps:

(1) sub-divide the wall into nodes at (say) 1 m intervals over its depth

(2) assume a point of rotation at a depth $h+z_p$ below original ground level (97.5% of the total wall height $h+d$ below original ground level represents an appropriate initial starting point

(3) carry out Coulomb wedge analyses in front of the wall at each of the nodes at and above the pivot point to determine the plane slip surface at each location offering the least resistance to failure

(4) calculate an equivalent earth pressure distribution in the restraining soil in front of the wall, at and above the pivot point, by dividing the increase in resistance between successive sliding wedges by the distance between successive nodes

(5) carry out Coulomb wedge analyses in front of the wall at each of the nodes at and below the pivot point and calculate an equivalent active earth pressure distribution by dividing the increase in the force between successive sliding wedges by the distance between successive nodes.

Behind the wall, standard active (above the pivot) and passive (below the pivot) pressures are assumed. Conventional limit equilibrium stress analyses, based on the fixed earth support method for free cantilever walls (assuming short-term undrained conditions) are then carried out. For a given berm geometry, retained height h and depth of wall embedment d, the unknown quantities are the mobilised undrained shear strength $s_{u\,mob}$ and the depth below formation level to the point of rotation z_p. These can be determined from the conditions of horizontal force and moment equilibrium. Simplifying the calculation by replacing the active and passive pressures below the pivot point with a net resultant force and applying an empirical factor of 1.2 to the calculated depth of embedment, will lead to a conservative solution, as shown by Bolton and Powrie (1987).

I1.2 Effective stress analysis

In principle, a multiple Coulomb wedge calculation in terms of effective stresses could be carried out. However, such an approach is as yet unvalidated and may be unconservative because the critical passive slip surfaces may not be planar.

I.2 GENERAL FINITE ELEMENT ANALYSIS OF AN EARTH BERM REMOVED IN SECTIONS FROM IN FRONT OF A LONG RETAINING WALL

Gourvenec and Powrie (2000) carried out a series of three dimensional finite element analyses to investigate the effect on wall movements of the removal of sections of an earth berm supporting an embedded retaining wall in overconsolidated clay. The results showed that:

- removal of a section of an earth berm will result in localised displacements in the vicinity of the unsupported section of the wall, the magnitude and extent of which increase with the length of the berm section removed and with time following excavation of the berm
- wall movements during removal of a berm in sections can be minimised by reducing the width of the sections removed; and
- a number of sections can be removed simultaneously without increasing wall movements, provided successive unsupported sections are separated by a sufficient length of intact berm.

For a wall along which bays of length B are excavated simultaneously at regular intervals separated by sections of intact berm of length B', the degree of discontinuity β may be defined by the ratio of the excavated length to the total length ie $\beta = B/(B+B')$ (see Figure 7.4). Then, for a given wall and berm geometry, ground conditions and time period there is a critical degree of berm discontinuity, β_{crit}, such that:

- if the degree of discontinuity β of a berm supported wall is less than, β_{crit}, displacements increase linearly with the length of the unsupported sections
- if β exceeds β_{crit} then displacements become a function not only of the length of the unsupported section but also of the degree of discontinuity and increase more rapidly with continued increases in β.

In practical terms,

- when a number of sections along the berm are removed simultaneously, as may occur with construction of a long retaining wall in bays, the sections removed should be separated by a section of intact berm between one and three times as long as the section removed (ie β=25 to 50%). This is because the maximum wall movement (at the centre of the unsupported section) begins to increase with β above β=25%, and the minimum wall movement (at the centre of the supported section) increases with β when β>50%;

- if minimisation of wall movements is critical, the length of the unsupported bays should be as small as possible, as the additional wall movement (compared with the case of an intact berm) increases in proportion to the length of berm section removed.

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Appendix J Effect of method of analysis

J1 METHODS OF ANALYSIS

These range from simple limit equilibrium calculations to complex numerical analyses of soil-structure interaction using finite element or finite difference methods and are discussed in detail in section 4.2 of the report. Methods of analysis fall into three broad categories:

- limit equilibrium
- subgrade reaction or pseudo-finite element
- finite element or finite difference.

In this Appendix, four generic problems are solved using two commercially available software packages from each of the above categories to highlight, in general terms, the potential pitfalls and the benefits which can be realised from using increased complexity of analysis. It is not the purpose of this Appendix to make detailed comparisons between the software packages considered or to draw definitive conclusions regarding the merits or limitations inherent in each of the above analysis categories. This would require a much broader study than the one undertaken herein. It is also not the purpose of this Appendix to illustrate, in detail, the application of the design recommendations made in chapter 6, although these have been adopted. Appendix K provides a step by step guide to the application of the design method recommended in chapters 6 and 7.

J2 SOFTWARE PACKAGES

The following commercially available software packages were used:

• limit equilibrium	-	STAWAL (version 17.7.2) ReWaRD (version 2.5)
• subgrade reaction and pseudo finite element	-	FREW (version 17.7.2) WALLAP (version 4.10)
• finite element and finite difference	-	SAFE (version 17.5.18) FLAC (version 3.4)

J3 PROBLEMS ANALYSED

Four simple cases were considered, as listed below. Examples 1, 2 and 3 were based on examples B3, C3 and B4 in CIRIA report 104 (Padfield and Mair, 1984).

- Effective stress analysis
  - example 1: cantilever wall (Figure J1)
  - example 2: singly propped wall (Figure J2)

Figure J1 *Example 1 - cantilever wall: effective stress analysis*

Figure J2 *Example 2 - propped wall: effective stress analysis*

- Total stress analysis
  - example 3: cantilever wall (Figure J3)
  - example 4: singly propped wall (Figure J4)

Figure J3 *Example 3 - cantilever wall: total stress analysis*

Figure J4 *Example 4 - propped wall: total stress analysis*

J4 ASSUMPTIONS AND ANALYSIS PROCEDURE

ULS and SLS calculations were carried out for each of the four examples listed in section J3 using each of the software packages listed in section J2.

The subgrade reaction and pseudo-finite element models of WALLAP were used to analyse all four examples. As expected, the results from these two sets of analyses were found to be almost identical for each of the cantilever wall cases (examples 1 and 3). The subgrade reaction model does not allow for the effects soil arching, which would be expected to occur in the propped wall case illustrated in example 2. Consequently, the results of the pseudo-finite element model in WALLAP have been used hereafter together with the results obtained from FREW to indicatively represent the pseudo-finite element analysis category in making comparisons with the results obtained from limit equilibrium and finite element and finite difference methods.

The moderately conservative soil parameters shown in Figures J1 to J4 were adopted in the calculations together with the design assumptions listed in sections 6.3 and 6.4 of this report. Factor on strength, F_s, values of 1.2 and 1.5 were applied in effective stress and total stress calculations respectively. Table J1 summarises the principal assumptions made in the calculations.

J4.1 Limit equilibrium calculations

ULS and SLS calculations were carried out using STAWAL and ReWaRD for the assumptions listed in Table J1. Results are discussed in section J5.

J4.2 Pseudo-finite element calculations

These were carried out using FREW and WALLAP and required further assumptions to be made about:

- construction sequence
- in situ stress conditions; and
- soil, wall and prop stiffness.

The following construction sequence and additional assumptions were made:

Cantilever wall (examples 1 and 3)

- Stage 0: install wall
  - lateral stress coefficient, K = 1.0
  - Young's modulus of soil = 48,000 kPa (example 1)
    = 60,000 kPa (example 3)

– wall EI = 469,000 kNm²/m. This corresponds to a hard/hard secant bored pile wall (750 mm pile diameters at 650 mm spacing). The value of EI was taken as $0.7E_oI$ (section 4.2.3)

- Stage 1: excavation to final formation level
  – excavation to 4.4 m depth in ULS analysis and 4.0 m depth in SLS analysis
  – soil and wall stiffness as stage 0 above.

Table J1 *Principal assumptions made in calculations*

Parameters	Effective stress analysis				Total stress analysis			
	Example 1 Cantilever wall		Example 2 Propped wall		Example 3 Cantilever wall		Example 4 Propped wall	
	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
Factor on strength, F_s	1.2	1.0	1.2	1.0	1.5	1.0	1.5	1.0
Moderately conservative soil parameters								
c' (kPa)	5	5	5	5	N/A	N/A	N/A	N/A
ϕ' (degrees)	25	25	25	25	N/A	N/A	N/A	N/A
s_u (kPa)	N/A	N/A	N/A	N/A	60	60	60	60
Design Soil parameters								
c'_d (kPa)	4.2	5	4.2	5	N/A	N/A	N/A	N/A
ϕ'_d (degrees)	21	25	21	25	N/A	N/A	N/A	N/A
(δ/ϕ')	0.67	0.67	0.67	0.67	N/A	N/A	N/A	N/A
s'_w (kPa)	0	0	0	0	N/A	N/A	N/A	N/A
s_{ud} (kPa)	N/A	N/A	N/A	60	40	60	40	60
(s_{uw}/s_u)	N/A	N/A	N/A	N/A	0.5	0.5	0.5	0.5
Unplanned excavation (m)	0.4	0	0.5	0	0.4	0	0.5	0
Groundwater conditions	linear(1) seepage	linear(1) seepage	linear(1) seepage	linear(1) seepage	dry(2) tension cracks	dry(2) tension cracks	flooded(2) tension cracks	flooded(2) tension cracks

Notes

1. See section 5.5.2 and Figure 5.14 (a) for more details. Steady state groundwater seepage pressures were computed directly by the finite element and finite difference programs.
2. See Figures 5.16 (a) and 5.16 (b) for more details.

Singly propped wall (examples 2 and 4)

- Stage 0: install wall
  – lateral stress coefficient, K = 1.0
  – Young's modulus of soil = 48,000 kN/m² (example 2)

= 60,000 kN/m² (example 4)

  - wall EI = 469,000 kNm²/m. This corresponds to a hard/hard secant pile wall (750 mm pile diameters at 650 mm spacing). The value of EI was taken as $0.7E_oI$ (section 4.2.3)

- Stage 1: excavate to 1 m below prop level
  - excavation to 3.3 m depth in ULS analysis and 3.0m depth in SLS analysis
  - soil and wall stiffness as stage 0 above
- Stage 2: install prop and excavate to final formation level
  - install prop. Prop stiffness, k = 100,000 kN/m/m
  - excavate to 8.5 m depth in ULS analysis and 8.0 m depth in SLS analysis
  - soil and wall stiffness as stage 0 above.

The wall depth was determined iteratively as the minimum depth where the FREW and WALLAP programs converged in the ULS analysis. This wall depth was then adopted in the SLS analysis. Results of the ULS and SLS calculations are discussed in section J5.

The width of the excavation in examples 1 to 4 was modelled to be 32 m in the pseudo-finite element and finite element and finite difference analyses.

J4.3 Finite element and finite difference calculations

The same mesh was adopted in the SAFE and FLAC analyses. The wall was modelled by beam elements in the FLAC analysis and by conventional elements (ie the thickness of the wall was modelled) in the SAFE analysis. Interface elements were introduced between the wall and the soil to ensure that the (δ/ϕ') and (s_{uw}/s_u) ratios and the tension crack assumptions listed in Table J1 were correctly modelled.

In the analysis of examples 1 and 2, steady state groundwater seepage pressures were computed directly by the finite element and finite difference programs.

J5 RESULTS

Results from the above analyses are summarised in Figures J5 to J12. These show:

- wall depths
- profiles of:
  - mobilised total horizontal pressures
  - wall bending moments.
- tabulated maximum ULS and SLS values of wall bending moment, shear force, prop load and maximum SLS wall deflection.

J5.1 Cantilever walls

Effective stress analysis (example 1)

ULS calculations

Results are presented in Figure J5.

(i) Wall depths and bending moments determined from limit equilibrium methods are, in general, similar to those determined from soil-structure interaction analyses (pseudo-finite element, finite element and finite difference methods), see also Figure 4.21.

(ii) The beneficial effects of shear mobilised at the soil/wall interface and at the base of the wall (in the FE analysis) are explicitly considered in finite element and finite difference methods. However, for the problem considered, such effects were largely negated by the slightly greater water pressures computed in the finite element and finite difference models in comparison to the linear seepage pressures adopted in the limit equilibrium and pseudo-finite element models.

(iii) The wall depths calculated from the two limit equilibrium programs were slightly different. This is because the version of the program ReWaRD which was used in the calculations adopted the approximation described in section 4.2.2 (and shown in Figure 4.11) where the portion of the wall below the pivot is represented as a point force, Q, and the depth to the pivot is increased by a factor of 1.2 to obtain the overall required depth of embedment. STAWAL assumes the pressure distribution shown in Figure 4.10 and iterates to find the wall embedment at which horizontal and moment equilibrium is satisfied. This illustrates the effect of the different assumptions which can be made about fixity at the base of cantilever walls (section 4.2.2).

(iv) The slight differences in wall depths computed in the pseudo-finite element models were due to the iterative process of optimising the wall depth to obtain satisfactory numerical convergence in these programs. Such differences, which are sensitive to details of the particular model adopted in the calculations (eg node spacings), are not unusual and are to be expected.

SLS calculations

Results are presented in Figure J6.

Similar wall bending moments were obtained from limit equilibrium calculations and soil-structure interaction analyses. The approximate profile shown in Figure 6.3 should be applied to the SLS wall bending moment calculated by limit equilibrium methods.

Total stress analysis (example 3)

ULS calculations

Results are presented in Figure J9.

Wall embedment depths and calculated maximum wall bending moments are generally similar for all three categories of analysis (see also Figure 4.21).

SLS calculations

Results are presented in Figure J10.

Calculated maximum wall bending moments are generally similar for all three categories of analysis. The approximate profile shown in Figure 6.3 should be applied to the SLS wall bending moment calculated by limit equilibrium methods.

J5.2 Propped walls

Effective stress analysis (example 2)

ULS calculations

Results are presented in Figure J7.

(i) The results of the soil-structure interaction analyses indicate stress redistribution and arching behind the wall. This is not modelled in the limit equilibrium calculations.

(ii) Shorter wall depths, smaller maximum wall bending moments and higher maximum prop loads are computed in the soil-structure interaction analyses compared with values calculated by limit equilibrium methods. This is a consequence of the computed pressure profiles. In pseudo-finite element models, these pressures will depend upon particular rules of stress redistribution adopted in such models. As these models adopt different rules, some difference in results would be expected as a consequence.

(iii) The results of the finite element and finite difference analyses are affected by the higher groundwater seepage pressures computed in these analyses in comparison with the linear seepage pressures adopted in the limit equilibrium and pseudo-finite element models.

In order to investigate the relative effects of this, further finite element analyses were carried out using the program SAFE assuming linear seepage groundwater pressures in the model. Results are presented in Figures J13 and J14.

Figure J13 *Example 2 – singly propped wall: effective stress - ULS calculations. Effect of groundwater pressures*

Figure J14 *Example 2 – singly propped wall: effective stress - SLS calculations. Effects of ground water pressures*

Figure J13 shows a 1 m reduction in the computed wall depth and significant reductions in the maximum values of wall bending moment and prop load. The magnitude of these reductions (due to a relatively small change in the total horizontal pressure around the wall) is due to the fact that, for the problem under consideration, near the base of the wall, the total horizontal pressures on the retained side are similar in magnitude to those on the restraining side. This is typical of soils with a low angle of shearing resistance, ϕ'. In such circumstances, the results of calculations would be expected to be very sensitive to small variations in pressures around the wall and to be influenced by node spacings in beam spring and pseudo-finite element models and mesh details in finite element and finite difference models. The designer should carry out sensitivity checks on the effects of such variations in the models adopted in the calculations.

(iv) For the same groundwater pressure assumptions (ie linear seepage), the results of the finite element analysis indicate a shorter wall, smaller computed wall bending moments and a computed maximum prop load which lies between the values calculated by limit equilibrium and pseudo-finite element methods (Figure 4.21).

SLS calculations

Results are presented in Figure J8.

Smaller maximum wall bending moments were obtained from the results of the pseudo-finite element analyses compared with limit equilibrium calculations. The higher bending moments obtained from the finite element and finite difference analyses were due to the greater groundwater pressures computed in these analyses in comparison with the linear seepage pressures adopted in the limit equilibrium and pseudo-finite element models, as discussed in section J5.2 (iii). Figure J14 shows lower computed maximum values of wall bending moment and prop load for the linear seepage assumption compared with the computed seepage analysis. For the same groundwater assumptions (ie linear seepage), Figure 4.21 shows the values of the maximum wall bending moment and prop load computed in the finite element analysis to lie between those calculated by limit equilibrium and pseudo-finite element methods.

The approximate profile shown in Figure 6.3 should be applied to the SLS wall bending moment calculated by limit equilibrium methods.

Total stress analyses (example 4)

Results from the SAFE and FLAC analyses were not available at the time of compiling this report. Current versions of these programs cannot reliably model the opening and then the closing of tension cracks behind a propped wall. For this reason, the results of the SAFE and FLAC analyses are not reported for this example.

It should be noted that for the particular geometry of this example and the assumption of a constant value of undrained shear strength for the clay, stability could not be achieved in STAWAL and ReWaRD for the assumption of $(s_w/s_u) = 0$ on the retained side of the wall. For problems of this nature, the designer should be very careful in selecting appropriate values of (s_w/s_u).

Calculations were carried out assuming $(s_w/s_u) = 0.5$ to enable a comparison to be made between the different types of analysis considered.

ULS calculations

Results are presented in Figure J11.

(i) Significantly shorter wall depths, smaller maximum wall bending moments and much higher prop loads are computed in the pseudo-finite element analyses compared with limit equilibrium methods. This is due to significant stress redistribution behind the wall in the soil-structure interaction analyses.

(ii) The prop loads computed in the pseudo-finite element analyses are significantly greater than those calculated in the limit equilibrium calculations. The calculated prop loads should be compared with those derived from experience of comparable construction (eg from the DPL method, see section 7.1.3). In situations where the calculated prop loads are significantly different from those derived from experience of comparable construction, the designer should carefully investigate and understand the reasons for the calculated values. This will typically involve a detailed review of the assumptions made in the calculations and the carrying out of sensitivity analyses. The outcome of such investigations should enable the designer to adopt appropriate design values.

SLS calculations

Results are presented in Figure J12.

Similar values of maximum wall bending moments were obtained from limit equilibrium calculations and soil-structure interaction analyses. Significantly greater prop loads were obtained from the soil-structure interaction (pseudo-finite element) analyses compared to those obtained from limit equilibrium calculations, due to the effects of stress redistribution.

The approximate profile shown in Figure 6.3 should be applied to the SLS wall bending moment calculated by limit equilibrium methods.

J6 CONCLUSIONS

1. In circumstances where there is little or no stress redistribution, eg cantilever walls, limit equilibrium calculations and soil-structure interaction analyses are likely to give similar wall embedment depths and wall bending moments.

2. For propped or anchored walls where stress redistribution will occur, design by limit equilibrium calculations will result in longer walls with higher calculated wall bending moments compared with those obtained from soil-structure interaction analyses. Prop or anchor loads calculated from limit equilibrium methods will be smaller than those obtained from soil-structure interaction analyses. Thus, prop loads solely obtained from limit equilibrium calculations may be significantly underestimated and should be treated with caution in design (section 7.1.3).

   In situations where the calculated prop loads are significantly different from those derived from experience of comparable construction (eg from the DPL method, see section 7.1.3), the designer should carefully investigate and understand the reasons for the calculated values. This will typically involve a detailed review of the assumptions made in the calculations and the carrying out of sensitivity analyses. The outcome of such investigations should enable the designer to adopt appropriate design values.

3. For walls embedded in soils where the total horizontal pressures near the base of the wall on the retained side are similar in magnitude to those on the restraining side, the results of calculations will be very sensitive to relatively small changes in earth and water pressures around the wall. The results of such calculations will also be influenced by node spacings in beam spring and pseudo-finite element models and mesh details in finite element and finite difference models. The designer should carry out sensitivity checks on the effects of such variations in the models adopted in the calculations.

Appendix K Worked example

This appendix presents a complete worked example that demonstrates each of the design steps outlined in section 6.2.

STEP 1 ESTABLISH SITE CONSTRAINTS (SECTION 2.3).

The example involves the construction of a 16m deep, 80m long, 40m wide, 3 storey basement. Four levels of support are provided by three, 0.5m thick, permanent concrete slabs and a 1m thick base slab in the long term. The geometry of the retaining wall and its supports are shown in Figure K1.

Figure K1 *Worked example geometry*

Site and project specific constraints are listed below:

- site is located in an urban setting
- there are nearby buildings
- there is insufficient space on site for bentonite recirculation plant
- programme constraints require the superstructure construction to start before completion of the basement
- permanent drainage will be provided beneath the base slab to prevent long term build up of water pressure.

STEP 2 ESTABLISH LIMIT STATES (SECTION 2.4)

ULS criterion:

- avoid the ultimate limit state of collapse and structural failure of the wall and its support system.

SLS criteria:

- there is to be no running water down the face of the wall
- project specification limits maximum ground surface settlement around the excavation to 25 mm.

STEP 3 REVIEW GROUND AND GROUNDWATER CONDITIONS (CHAPTER 5)

Ground conditions comprise sandy made ground overlying medium dense to dense gravel over stiff overconsolidated clay (Figure K1). The water table lies in the gravel, at a depth of 7 m.

The moderately conservative and worst credible soil parameters are given in Table K1.

Table K1 *Summary of ground parameters*

	Made ground		Gravel		Clay	
	Moderately Conservative	Worst Credible	Moderately Conservative	Worst Credible	Moderately Conservative	Worst Credible
ϕ' (degrees)	25	23	35	32	25	21
c' (kPa)	0	0	0	0	5	0
s_u (kPa)	N/A	N/A	N/A	N/A	$75 + 5z$	N/A
γ_b (kN/m^3)	18		19		20	
k (m/sec)	10^{-6}		10^{-4}		10^{-9}	
ν'	0.3		0.25		0.2	
K_o	0.6		0.4		1.0	
ν_u	N/A		N/A		0.5	

Note: values of K_o obtained from section 5.4.3.

The clay is homogeneous with a coefficient of permeability, $k < 10^{-8}$ m/s. Therefore, it is assumed to behave in an undrained manner during the excavation period and in a drained manner in the long term. As specified in the SLS criteria, the wall will be impermeable and will provide a groundwater cut-off in the clay. There is no source of groundwater recharge at or below excavation level, ie there are no water bearing permeable horizons that could provide drainage paths within the clay. This means there will only be excavation disturbance to a depth of 0.5m below formation level (section 5.9.1). No further softening of the clay is assumed during excavation on the restraining side.

STEP 4 SELECT CONSTRUCTION SEQUENCE AND WALL TYPE (CHAPTER 3)

Top down construction with internal stanchions (plunge columns) to support the floor slabs will be appropriate. This will allow concurrent construction of the superstructure and the substructure to meet the overall programme objectives. The basement will be excavated in 4 stages corresponding approximately to the four levels of support in the excavation (Figure K2). At each stage of excavation, the ground will be excavated to 500 mm below the underside of the slab to allow for a soffit shuttering system. The slab will be cast and once an appropriate concrete strength has been achieved, excavation will continue beneath. The final excavation level will be to 500 mm below the underside of the base slab to allow for the permanent drainage system.

Figure K2 *Top down construction sequence*

A hard/hard secant bored pile wall comprising 880 mm diameter bored piles at 760 mm spacings will be adopted (Appendix C).

STEP 5 ASSIGN GEOTECHNICAL CATEGORY TO WALL (SECTION 2.2.2 AND FIGURE 2.4)

The wall is geotechnical category 2.

STEP 6 DETERMINE DESIGN STRATIGRAPHY (SECTION 5.2.2)

The design stratigraphy is shown in Figure K1.

STEP 7 DETERMINE SOIL PARAMETERS, GROUNDWATER PRESSURES, LOAD CASE COMBINATIONS AND DESIGN GEOMETRY APPROPRIATE FOR ULTIMATE LIMIT STATE (ULS) CALCULATIONS (SECTION 6.3)

The moderately conservative soil parameters listed in Table K1 are factored by $F_s = 1.2$ (effective stress parameters) and $F_s = 1.5$ (total stress parameters). These are presented in Table K2 together with the worst credible parameters from Table K1. The more onerous of these two values will be adopted in the ULS calculations. These are highlighted in bold in Table K2.

Table K2 *Soil design parameters - ULS calculations*

	Made Ground		Gravel		Clay	
	Factored moderately conservative	Worst credible	Factored moderately conservative	Worst credible	Factored moderately conservative	Worst credible
ϕ'_d (degrees)	**21**	23	**30**	32	21	**21**
c'_d (kPa)	**0**	0	**0**	0	4.2	**0**
s_{ud} (kPa)	N/A	N/A	N/A	N/A	**50+3.3z**	N/A
γ_b (kN/m^3)	18		19		20	
k (m/sec)	10^{-6}		10^{-4}		10^{-9}	
ν'	0.3		0.25		0.2	
K_o	0.6		0.4		1.0	
ν_u	N/A		N/A		0.5	

The wall will be designed using the computer program FREW. This pseudo-finite element soil-structure interaction program requires the wall flexural stiffness (EI), soil Young's modulus and prop stiffness to be input to the analysis.

Wall I $= \pi D^4/64.s$

$= 0.039$ m^4/m

Young's modulus of concrete, E_o = 28 MPa in the short term.

Wall EI during construction = 0.7 E_oI (section 4.2.3)

= 7.6 x 10^5 kNm²/m

Wall EI in the long term = 0.5E_oI (section 4.2.3)

= 5.5 x 10^5 kNm^2/m

Soil Young's modulus, E_{ULS} = E_{SLS}/2 (section 5.4.5)

where E_{SLS} is the Young's modulus of the soil adopted in SLS calculations (see step 13).

Thus, E_{ULS} = 5 MPa	in made ground
25 MPa	in gravel
38 + 2.5z MPa	in clay (undrained)
30 + 2z MPa	in clay (drained)

Prop stiffnesses were derived from Equation 4.17, section 4.2.3.

An allowance for unplanned excavation in accordance with section 5.7 was made at each stage of excavation. The excavation depths in the ULS calculations were therefore 2.75 m (0.25 m unplanned excavation), 7.2 m (0.495 m unplanned excavation), 11.75 m (0.5 m unplanned excavation) and 17.0 m (0.5 m unplanned excavation).

The groundwater is hydrostatic from 7 m below ground level. However, in flood conditions the groundwater may rise to 2 m below ground level, and this is the condition which correspond to case (i) section 5.5 and will be used in the ULS calculations in the short term. In the long term linear seepage is assumed in accordance with Figure 5.13(a), as the width of the excavation is greater than 4 times the differential water head across the wall.

During excavation, the clay is assumed to behave in an undrained manner, and inside the excavation at construction stages 4 and 5 (Figure K2), the clay is assumed to be disturbed to a depth of 0.5m, as stated in step 3 above.

A minimum 10 kPa surcharge is applied to the ground surface on the retained side (section 6.3.2).

STEP 8 DETERMINE THE MINIMUM WALL EMBEDMENT FOR VERTICAL STABILITY, WATER CUT-OFF ETC. (SECTION 6.3.5)

Since the basement is to be constructed top-down, vertical loads will be applied to the wall both during construction and in the long term. Based on these vertical loads a separate calculation shows that a minimum wall embedment of 4.5 m is required at construction stage 5 (Figure K2). This embedment will also provide satisfactory long term vertical stability allowing for effects of ground heave, etc.

STEP 9 DETERMINE THE DIRECTION AND MAGNITUDE OF THE WALL FRICTION AND ADHESION (SECTION 4.1.4)

Since the wall is to support significant vertical loads from the superstructure and the substructure floor slabs, friction and adhesion at the wall-soil interface should be carefully assessed at each excavation stage (section 4.1.4). Because of relative movements between the wall and the soil, in the circumstances of this example, at construction stage 5 (Figure K2) it is reasonable to apply the rule of thumb stated in section 4.1.4, namely, assume zero wall friction and adhesion in the retained soil above excavation level and limiting soil/wall adhesion, $(s_{uw}/s_u) = 0.5$ below final excavation level (Figure K3).

Figure K3 *Limiting wall friction and adhesion: wall supporting large vertical loads*

Values of earth pressure coefficients are derived from Appendix F for appropriate values of wall friction at each construction stage. These coefficients are input to the FREW analysis.

STEPS 10, 11 AND 12

CARRY OUT COLLAPSE (ULS) CALCULATIONS TO DETERMINE WALL DEPTH FOR OVERALL LATERAL STABILITY AND CALCULATE VALUES OF MAXIMUM WALL BENDING MOMENT, SHEAR FORCE AND PROP / ANCHOR LOAD.

Using the ULS soil design parameters determined in step 7, FREW analysis was carried out. This analysis indicates that the embedment required for lateral stability is less than the 4.5 m minimum embedment required for vertical loading (see step 8). Thus, an embedment of 4.5 m will be used in the ULS and SLS analyses. For this embedment, the maximum ULS values of wall bending moments and shear force and prop forces calculated by FREW are summarised below:

Max prop force in P1	222	(kN/m)
Max prop force in P2	686	(kN/m)
Max prop force in P3	1860	(kN/m)
Max prop force in base slab	1284	(kN/m)
Max wall shear force	978	(kN/m)
Max positive wall bending moment	+543	(kNm/m)
Max negative wall bending Moment	-2048	(kNm/m)

Figure K4 shows the computed ULS wall bending moment envelope.

Figure K4 *Calculated ULS and SLS wall bending moment envelopes*

STEP 13

DETERMINE SOIL PARAMETERS, GROUNDWATER PRESSURES, LOAD CASE COMBINATIONS, AND DESIGN GEOMETRY APPROPRIATE FOR SLS CALCULATIONS (SECTION 6.4)

Table K3 summarises the SLS soil design parameters. These are taken from the moderately conservative soil parameters in Table K1.

Table K3 *Soil design parameters - SLS calculations*

	Made Ground	Gravel	Clay
ϕ' (degrees)	25	35	25
c' (kPa)	0	0	5
s_u (kPa)	N/A	N/A	75 + 5z
γ_b (kN/m^3)	18	19	20
k (m/sec)	10^{-6}	10^{-4}	10^{-9}
ν'	0.3	0.25	0.2
K_o	0.6	0.4	1.0
ν_u	N/A	N/A	0.5

From previous experience and back analysis of case histories in comparable ground conditions using FREW, the Young's modulus of the soil in SLS conditions:

E_{SLS} =	10 MPa	in made ground
	50 MPa	in gravel
	$75 + 5z$ MPa	in clay (undrained)
	$60 + 4z$ MPa	in clay (drained)

Wall flexural stiffness and prop stiffnesses are as derived in step 7.

The following are also assumed in SLS calculations (section 6.4):

- no unplanned excavation (ie excavation levels are those shown in Figure K2)
- 10 kPa minimum surcharge on the surface of the retained soil
- 0.5 m disturbance in the undrained clay below excavation level
- Ground water at 7 m depth on the retained side.

STEP 14 CARRY OUT SLS CALCULATIONS TO DETERMINE SLS LOAD EFFECTS - WALL BENDING MOMENTS, SHEAR FORCE AND PROP LOADS (SECTION 6.4.5)

The analysis was carried out with FREW using the wall geometry determined at step 8 and the parameters and assumptions outlined in step 13. The maximum SLS values of wall bending moments and shear force and prop loads calculated by FREW are summarised below:

Max prop force in P1	136	(kN/m)
Max prop force in P2	380	(kN/m)
Max prop force in P3	603	(kN/m)
Max prop force in base slab	1111	(kN/m)
Max wall shear force	586	(kN/m)
Max positive wall bending moment	+515	(kNm/m)
Max negative wall bending moment	-521	(kNm/m)

Figure K4 shows the computed SLS wall bending moment envelope.

STEP 15 DETERMINE WALL DEFLECTIONS AND GROUND MOVEMENTS (CHAPTER 2)

Figure K5 shows the calculated SLS wall deflections and the associated estimated ground surface settlements from Figure 2.16.

Figure K5 *Estimated ground surface settlement from computed SLS FREW wall deflection*

Ground surface settlements due to wall installation are estimated from Figure 2.8 (b) for a secant bored pile wall. These are shown in Figure K6 (a).

The ground surface settlements predicted from the SLS FREW wall deflections (Figure K5) are compared, in Figure K6 (b), with the settlements derived from Figure 2.11 (b) case history data for high stiffness walls. The total estimated ground surface settlements due to wall installation and wall deflection are shown in Figure K6 (c).

Figure K6 *Estimated ground surface settlement due to wall installation and deflection*

STEP 16 CHECK COMPLIANCE WITH THE WALL DESIGN REQUIREMENTS AND PERFORMANCE CRITERIA

Total estimated ground surface settlements behind the retaining wall are less than the specified 25 mm.

STEP 17 CARRY OUT BUILDING DAMAGE ASSESSMENT IN ACCORDANCE WITH THE PROCEDURE OUTLINED IN (FIGURE 2.18, CHAPTER 2) FOR USE IN STRUCTURAL DESIGN

Building damage assessment should be carried out for all structures located in the zone of influence indicated by Figure K6 (c).

STEP 18 DETERMINE SOIL PARAMETERS, GROUNDWATER PRESSURES, ACCIDENTAL LOAD CASE AND DESIGN GEOMETRY APPROPRIATE FOR A CHECK AGAINST PROGRESSIVE FAILURE (SECTION 6.5)

In this case the permanent slabs are installed during the top-down sequence. A risk assessment has been carried out and construction procedural controls will ensure that the support to the permanent slabs will be maintained at all stages of construction. Therefore the accidental load check of losing a prop does not need to be made.

STEP 19 CARRY OUT ACCIDENTAL LOAD CASE CALCULATIONS

N/A.

STEP 20 DETERMINE ULS WALL BENDING MOMENTS (BM) AND SHEAR FORCE (SF) FOR USE IN STRUCTURAL DESIGN

The ULS wall bending moments and shear forces for use in the structural design of the wall are highlighted in bold in the table below:

Table K4 *ULS values of maximum wall bending moment and shear force*

	Maximum wall bending moment (kNm/m)		Maximum wall shear force (kN/m)
From step 12	**-2048**	+543	**978**
1.35 x step 14	**-703**	+695	791
From step 19	N/A	N/A	N/A

STEP 21 DETERMINE ULS PROP FORCES (SECTION 7.1.3)

SLS prop loads are determined from the higher of the SLS calculations (step 14) and the DPL method. A summary of these results is given in Table K5, with the critical SLS values highlighted in bold:

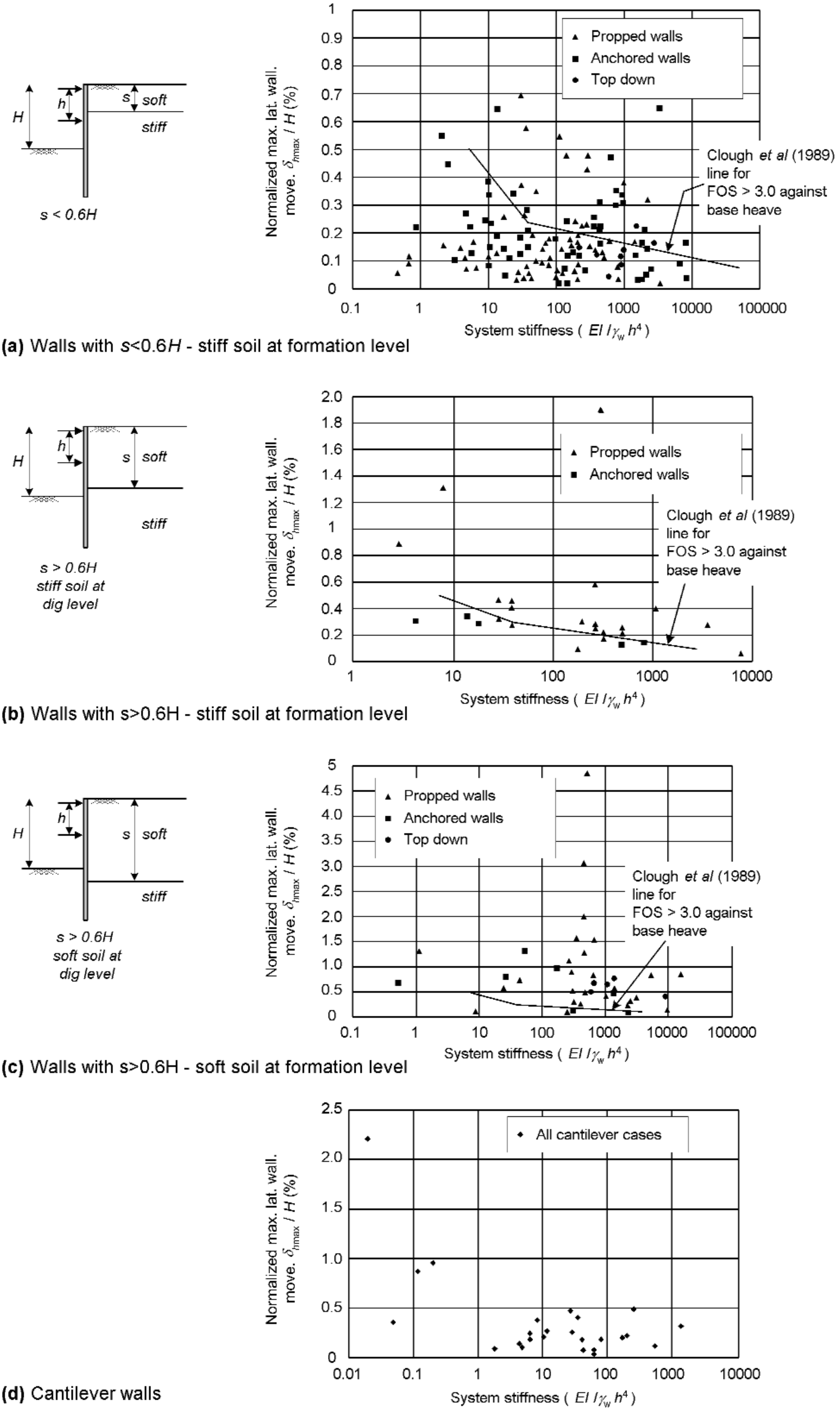
Table K5 *SLS prop forces*

	SLS calculations (Step 14) kN/m	**DPL method kN/m**
Prop P1	138	**251**
Prop P2	**385**	246
Prop P3	**606**	376
Base Slab	**878**	NA

ULS prop loads are highlighted in bold below:

Table K6 *ULS prop forces*

	ULS calculations (step 12) kN/m	**1.35 x SLS calculations (step 14) kN/m**	**1.35 x DPL method kN/m**
Prop P1	222	184	**339**
Prop P2	**686**	513	332
Prop P3	**1860**	814	508
Base Slab	1284	**1500**	NA



(a) Walls with s<0.6H - stiff soil at formation level

(b) Walls with s>0.6H - stiff soil at formation level

(c) Walls with s>0.6H - soft soil at formation level

(d) Cantilever walls

FIGURE B1 *Normalized maximum wall deflection versus system stiffness (after Long, 2001)*

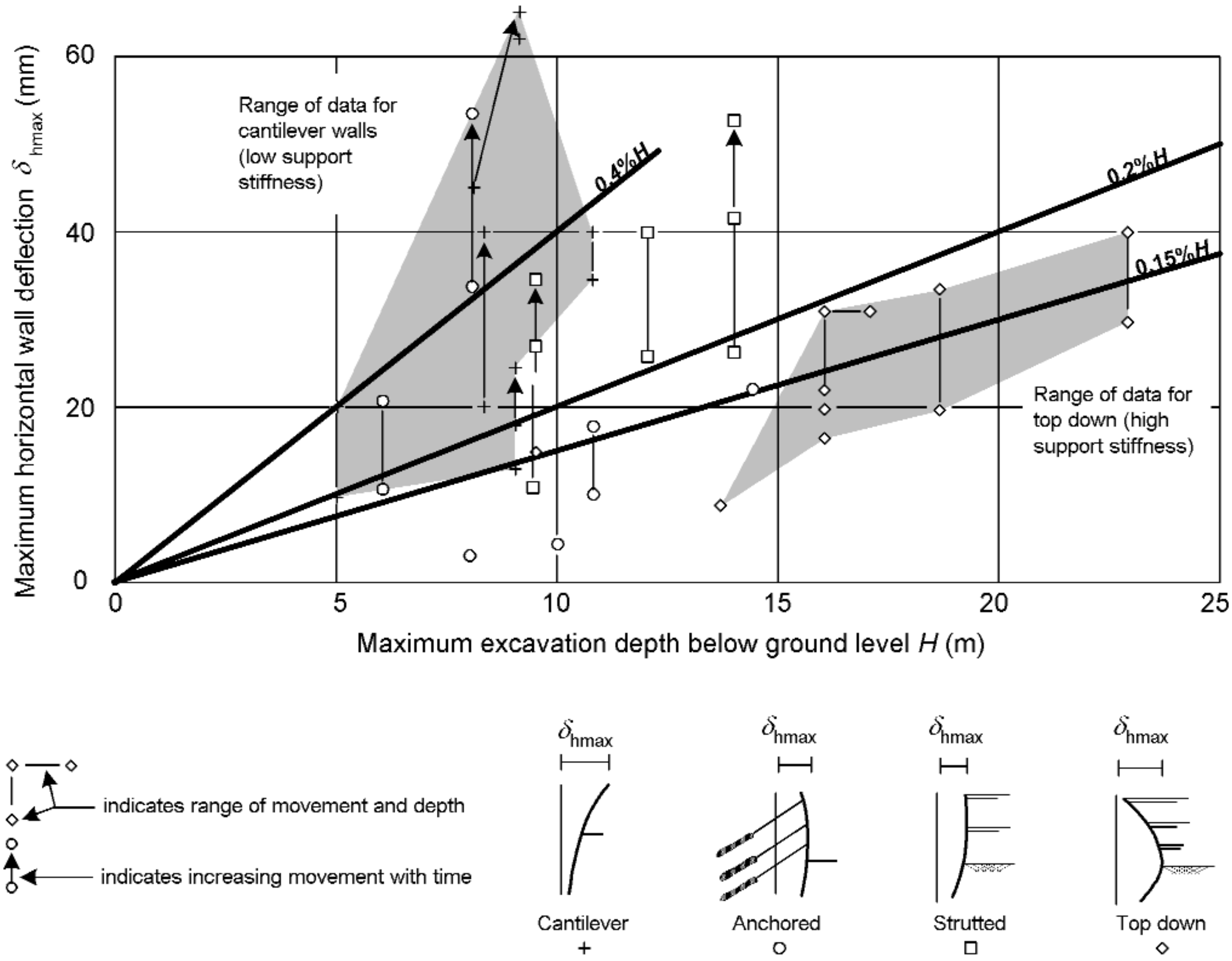


FIGURE B2 *Observed maximum lateral wall deflections for excavations in London Clay (after St John et al, 1992)*

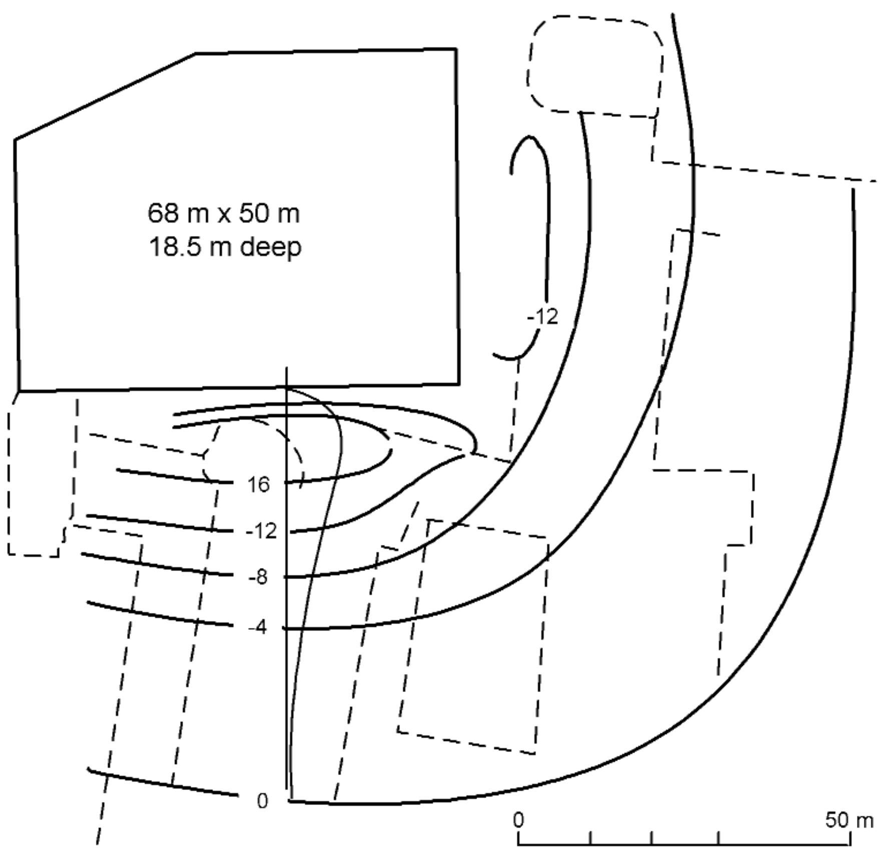


FIGURE B3 *Ground surface settlement contours at New Palace Yard, London (after Burland and Hancock, 1977) Effect of excavation geometry on ground surface settlements*

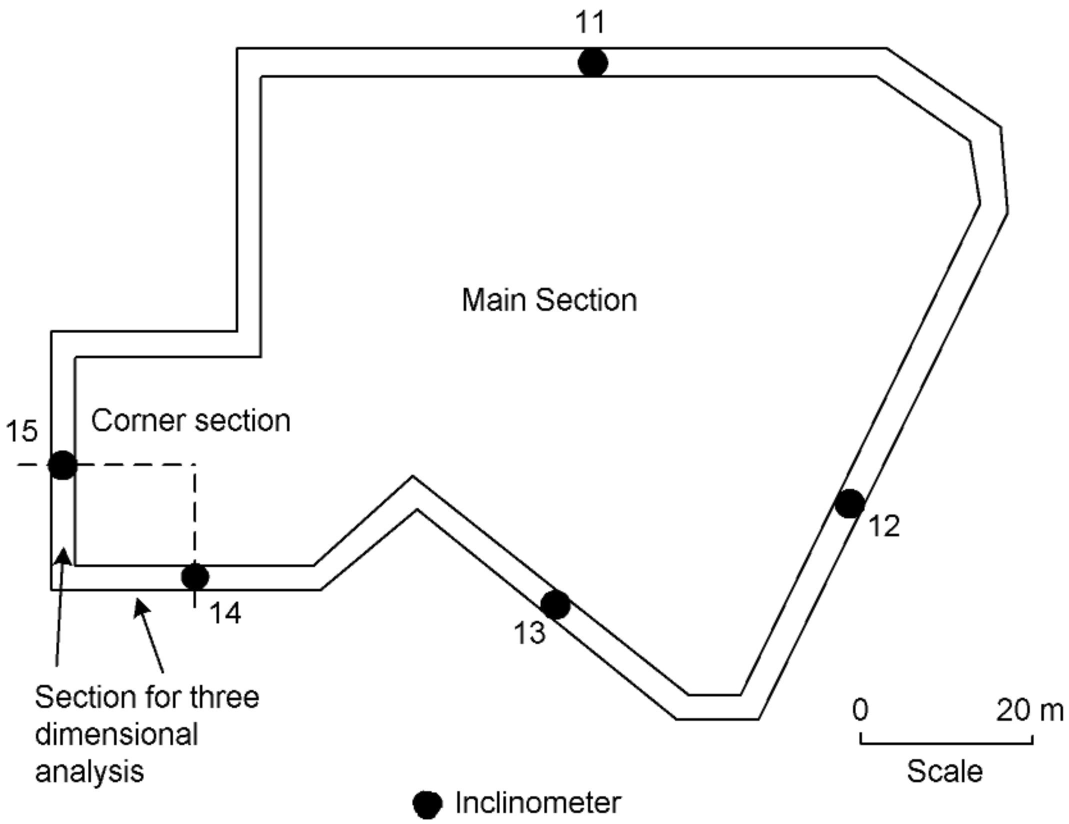
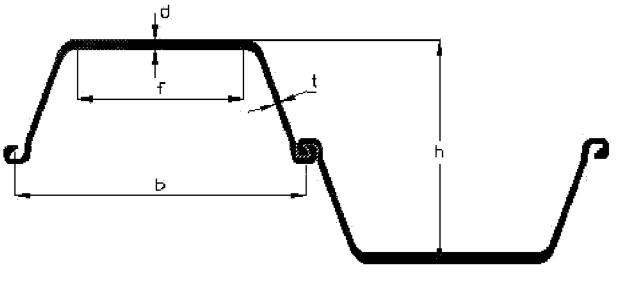


FIGURE B4 *Plan of the Hai-Hua building (after Ou et al., 1996)*

Section	Width b mm	Height h mm	Flange d mm	Web t mm	Flat of pan, f mm	y bar mm	CS area cm^2/m	Mass kg/m^2	I_{xx} cm^4/m	Section modulus Elastic cm^3/m	Section modulus Plastic cm^3/m	Coating m^2/m
LX8	600	310	8.2	8	250	84.5	116	91.0	12863	830	980	2.54
LX 12	600	310	9.7	8.2	386	99.1	136	106.5	18727	1208	1345	2.72
LX12d	600	310	10	8.33	386	99.5	139	108.8	19217	1240	1379	2.72
LX 12d 10	600	310	10	10	382.3	92.7	155	121.5	19866	1282	1434	2.74
LX 16	600	380	10.5	9	365	117.8	157	123.5	31184	1641	1853	2.91
LX 20	600	430	12.5	9	330	130.1	177	138.7	43484	2023	2298	3.00
LX 20 d	600	450	11.2	9.7	329.9	129.2	179	140.5	45197	2009	2313	3.07
LX 25	600	460	13.5	10	351.2	141.1	202	158.3	57656	2507	2845	3.15
LX 25 d	600	450	15	11	325.5	137.6	212	166.7	57246	2544	2920	3.05
LX 32	600	460	19	11	340	149.7	243	190.7	73802	3209	3636	3.11
LX 32 d	600	450	21.5	13	319.7	143.2	269	210.8	75325	3348	3845	3.03
LX 38	600	460	22.5	14.5	336.5	146.6	298	234.0	87511	3805	4370	3.11
6W	525	212	7.8	6.4	332.6	61.7	109	85.3	6508	614	672	2.54
20Wd	525	400	11.3	10	332.6	118.0	196	153.7	40574	2029	2310	3.26
GSP 2	400	200	10.5	8.6	265.9	61.4	157	123.5	8756	876	965	2.85
GSP 3	400	250	13.5	8.6	269.8	75.2	191	150.3	16316	1305	1439	3.08
GSP 4	400	340	15.5	9.7	259	106.4	242	190.3	38742	2279	2579	3.53
6 (42)	500	450	20.5	14	329.45	141.8	339	266.0	94755	4211	4804	3.64
6 (122)	420	440	21.99	14	249.5	129.6	371	291.7	92115	4187	4815	3.92
6 (131)	420	440	25.4	14	249.6	133.7	396	311.2	101598	4618	5301	3.90
6 (138.7)	420	440	28.6	14	250.8	136.9	419	329.3	110109	5005	5743	3.89

(a) Section properties per metre run of wall

Section	CS area cm^2	Mass kg/m	I_{xx} cm^4	Elastic Modulus cm^3	Coating m^2
LX8	69.6	54.6	2746	263	1.52
LX 12	81.4	63.9	3239	272	1.63
LX12d	83.2	65.3	3302	276	1.63
LX 12d 10	92.8	72.9	3943	342	1.64
LX 16	94.4	74.1	5620	404	1.75
LX 20	106.0	83.2	8154	531	1.80
LX 20 d	107.4	84.3	9183	601	1.84
LX 25	121.0	95.0	10485	635	1.89
LX 25 d	127.3	100.0	10241	636	1.83
LX 32	145.7	114.4	11614	668	1.86
LX 32 d	161.1	126.5	12164	727	1.82
LX 38	178.9	140.4	14075	814	1.87
6W	57.1	44.8	1253	153	1.34
20Wd	102.8	80.7	7060	499	1.71
GSP 2	62.9	49.4	1135	140	1.14
GSP 3	76.6	60.1	2253	226	1.23
GSP 4	97.0	76.1	4585	355	1.41
6 (42)	169.4	133.0	13407	789	1.82
6 (122)	156.0	122.5	12545	781	1.65
6 (131)	166.5	130.7	12971	787	1.64
6 (138.7)	176.2	138.3	13285	791	1.63



(b) Section properties of individual piles

FIGURE C1 *Larssen sheet pile section properties (after Corus, 2001)*

Section	Width	Height	Flange t	Web t	Flat of pan	f1	f2	Area / m	Mass / m	Mass / sm	Inertia / m	Elastic	Plastic	Coating /pile	Coating /sm
	mm	mm	mm	mm	mm	mm	mm	cm^2/m	kg/m	kg/m^2	cm^4/m	cm^3/m	cm^3/m	m^2	m^2/m^2
1 BXN	476	143	12.7	12.7	303	77	122	170	63.4	133.2	4947	692	859	1.16	2.44
1 N	483	170	9.0	9.0	334	107	142	126	48.0	99.4	6072	714	831	1.22	2.53
2 N	483	235	9.7	8.4	337	91	146	145	54.8	113.5	13641	1161	1333	1.34	2.78
3 NA	483	305	9.7	9.5	341	90	148	166	62.7	129.8	25710	1687	1937	1.48	3.06
4 N	483	330	14.0	10.4	326	75	128	218	82.7	171.2	39869	2415	2787	1.52	3.16
5	426	311	17.1	11.9	345	87	119	302	101.0	237.1	49329	3171	3683	1.51	3.56

Note: All values are per metre run of wall

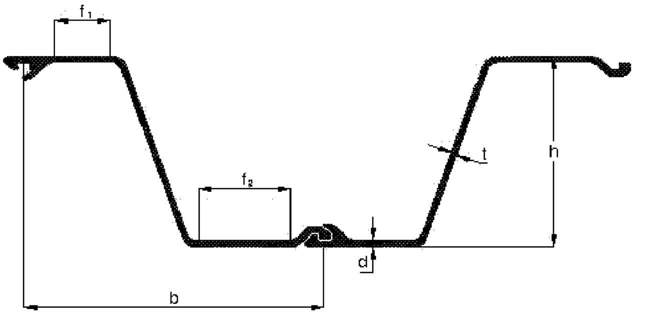
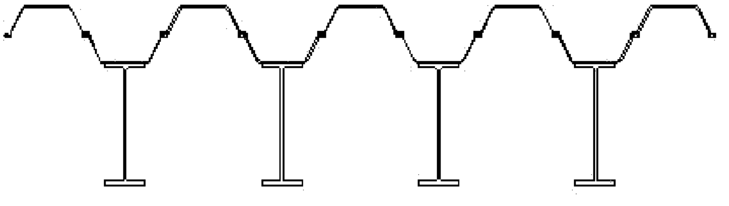
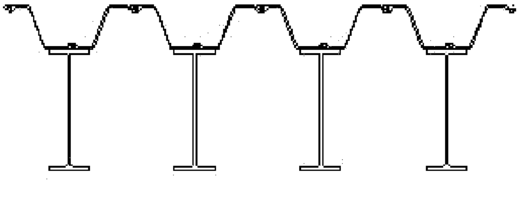


FIGURE C2 *Frodingham sheet pile section properties (after Corus, 2001)*



(a) Larssen high modulus piling



(b) Frodigham high modulus piling

FIGURE C3 *High modulus sheet piles (courtesy of Corus, 2001)*

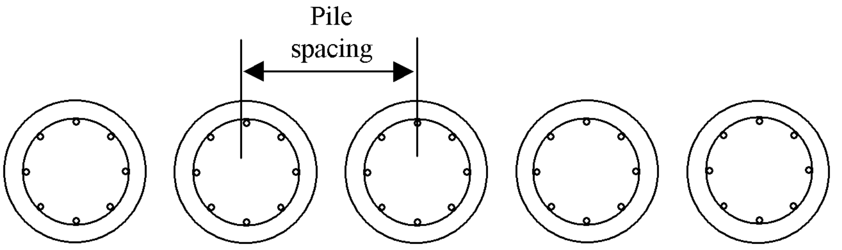


FIGURE C4 *Contiguous pile wall*

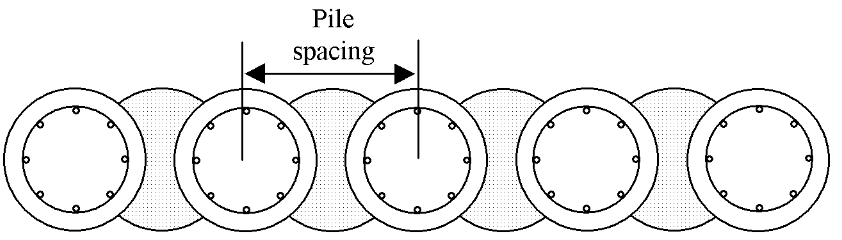


FIGURE C5 *Hard/soft secant pile wall*

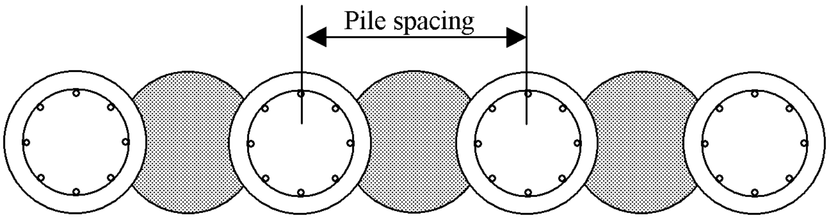


FIGURE C6 *Hard/firm secant pile wall*

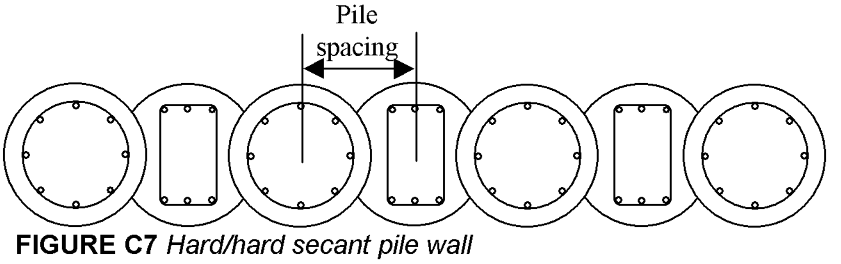


FIGURE C7 *Hard/hard secant pile wall*

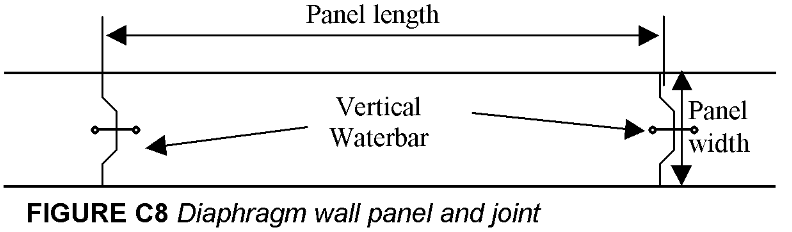


FIGURE C8 *Diaphragm wall panel and joint*

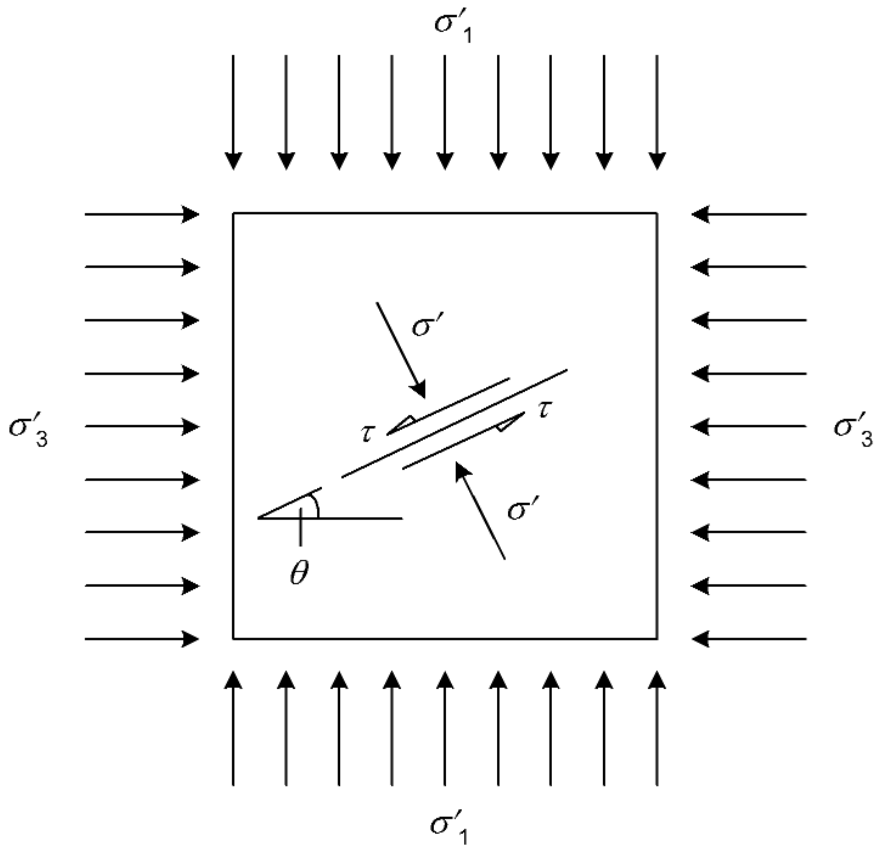
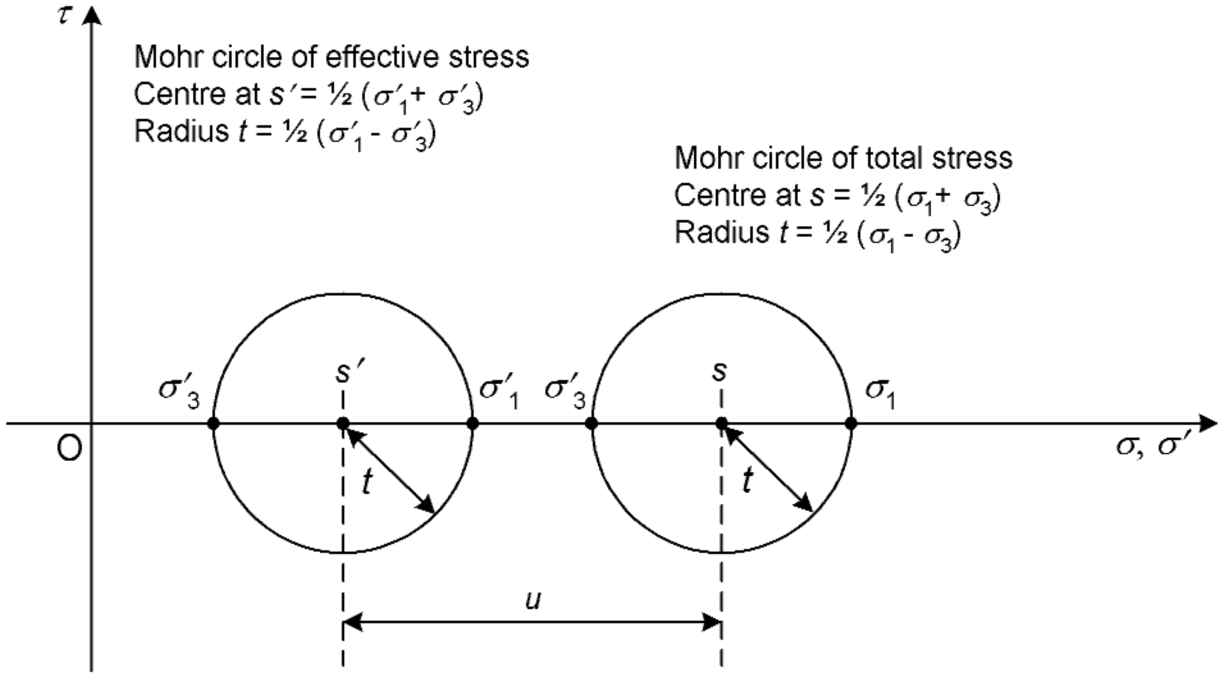
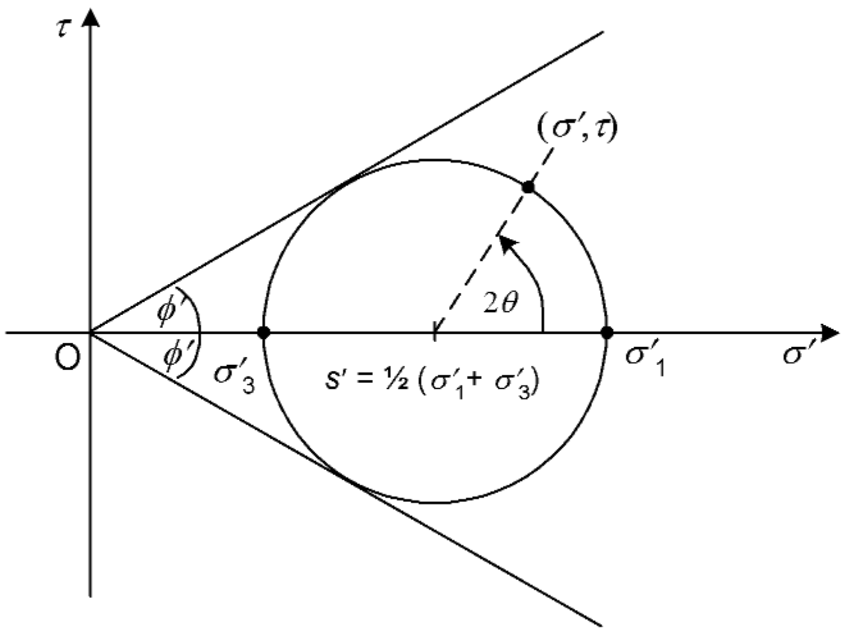


FIGURE D1 *Normal and shear stresses acting on an imaginary plane within the cross section plane*



(a)



(b)

FIGURE D2 *Mohr circles of stress*

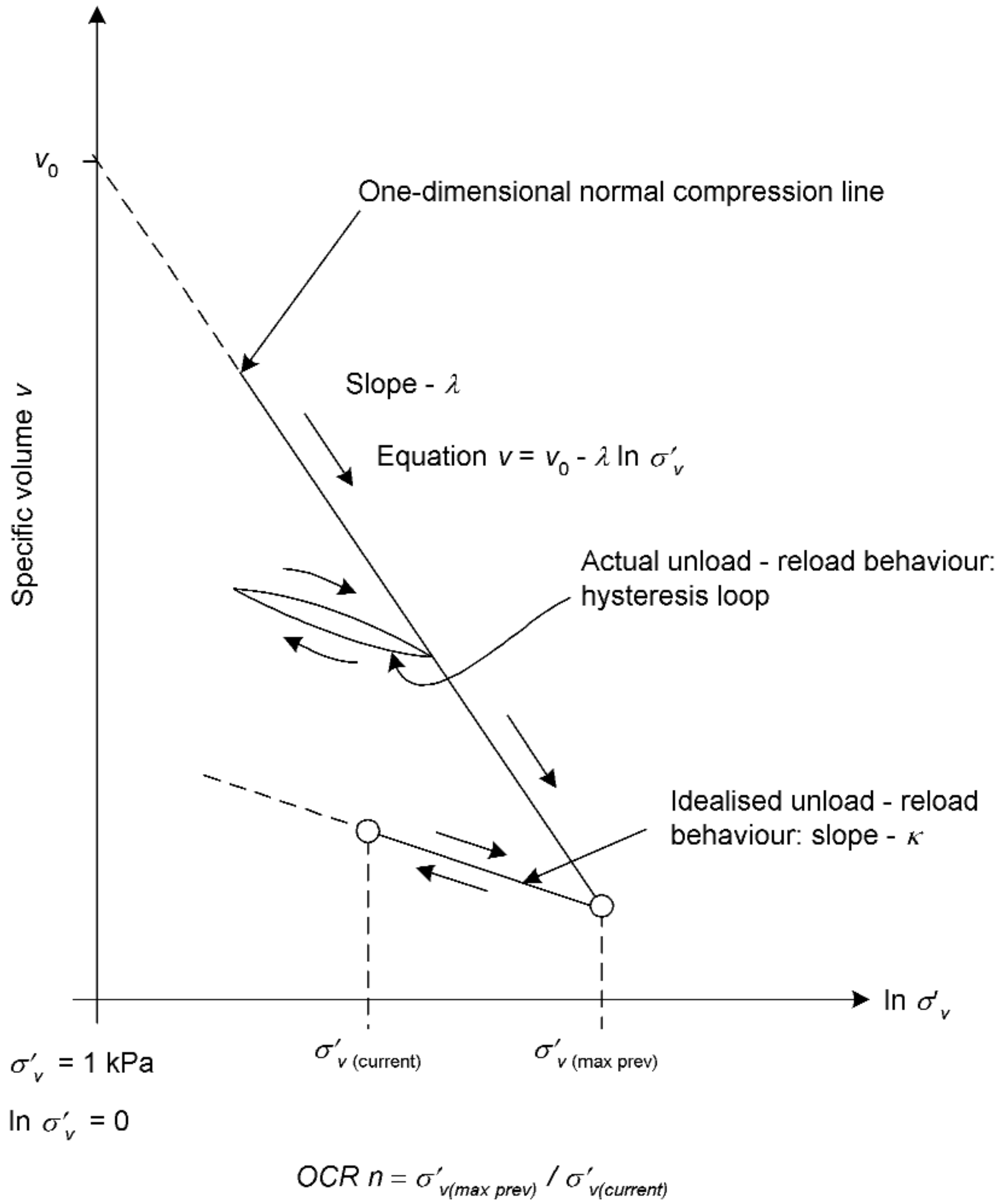


FIGURE D3 *Schematic stress history of an overconsolidated clay*

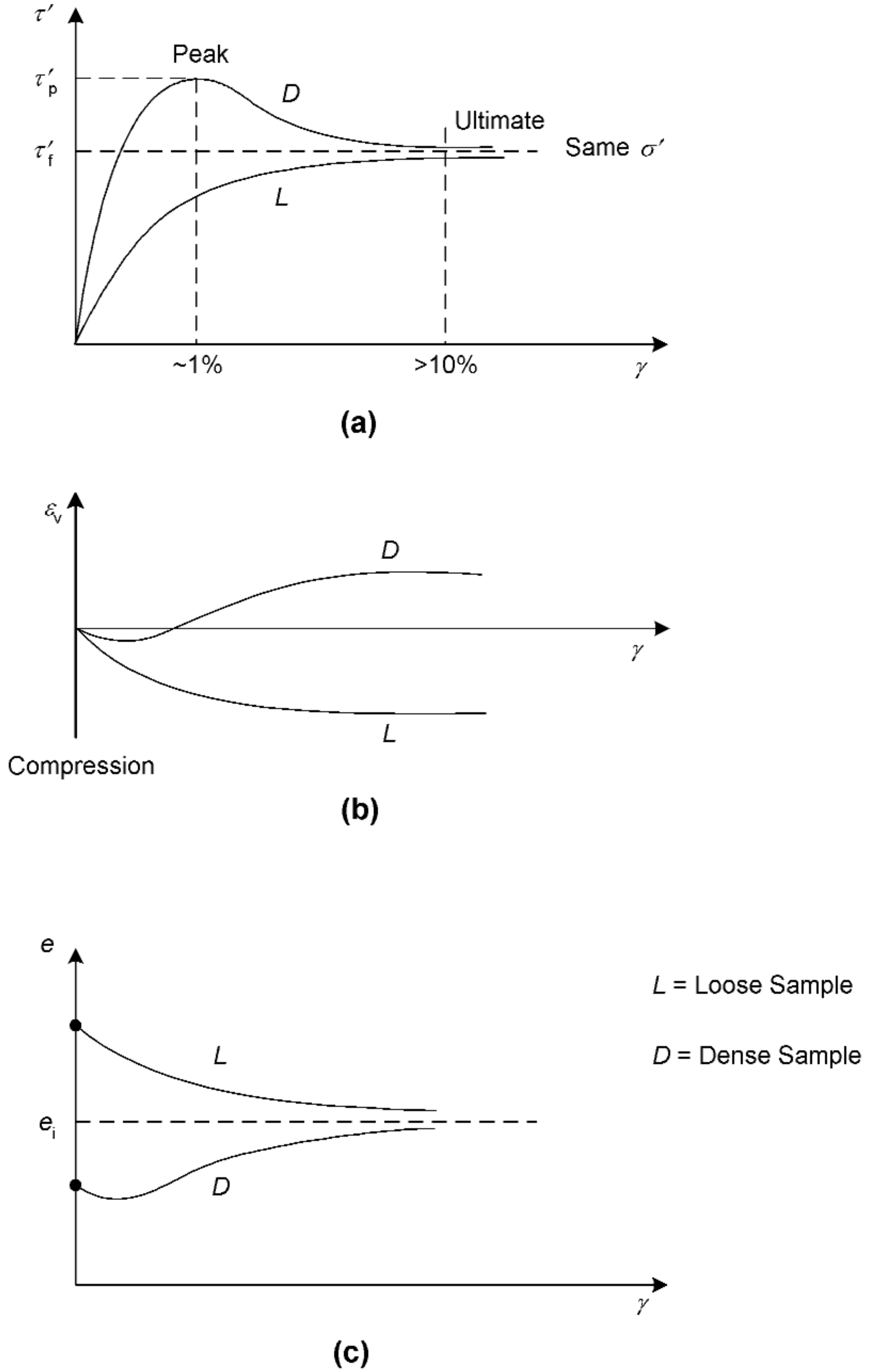


FIGURE D4 *Typical stress-strain data for a loose (lightly overconsolidated or normally consolidated) soil and for dense (heavily overconsolidated) soil*

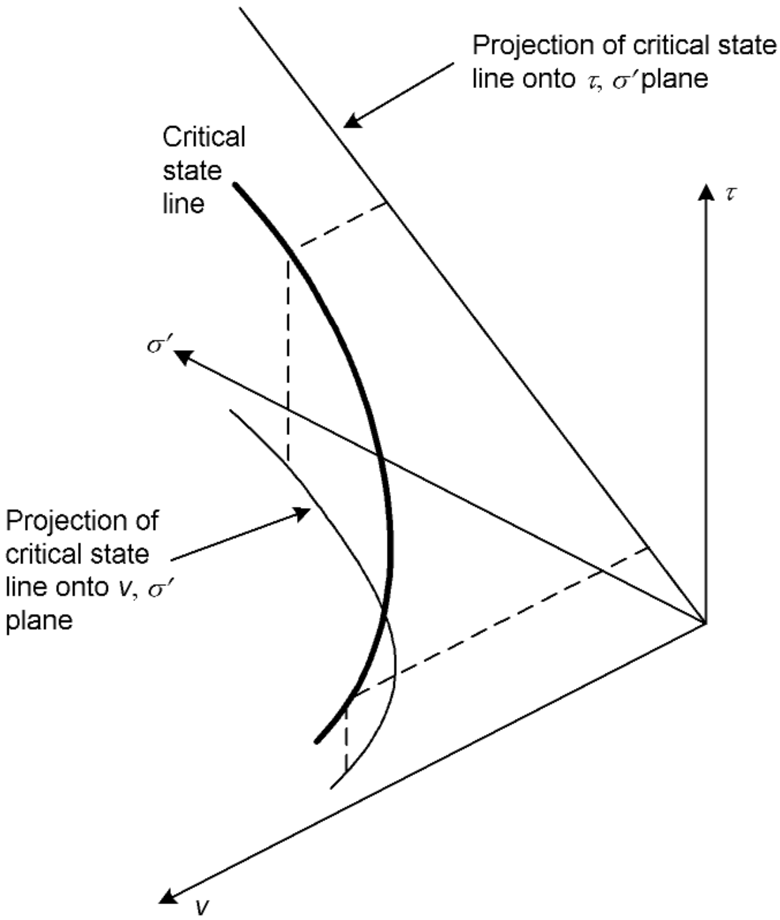


FIGURE D5 *Critical state line*

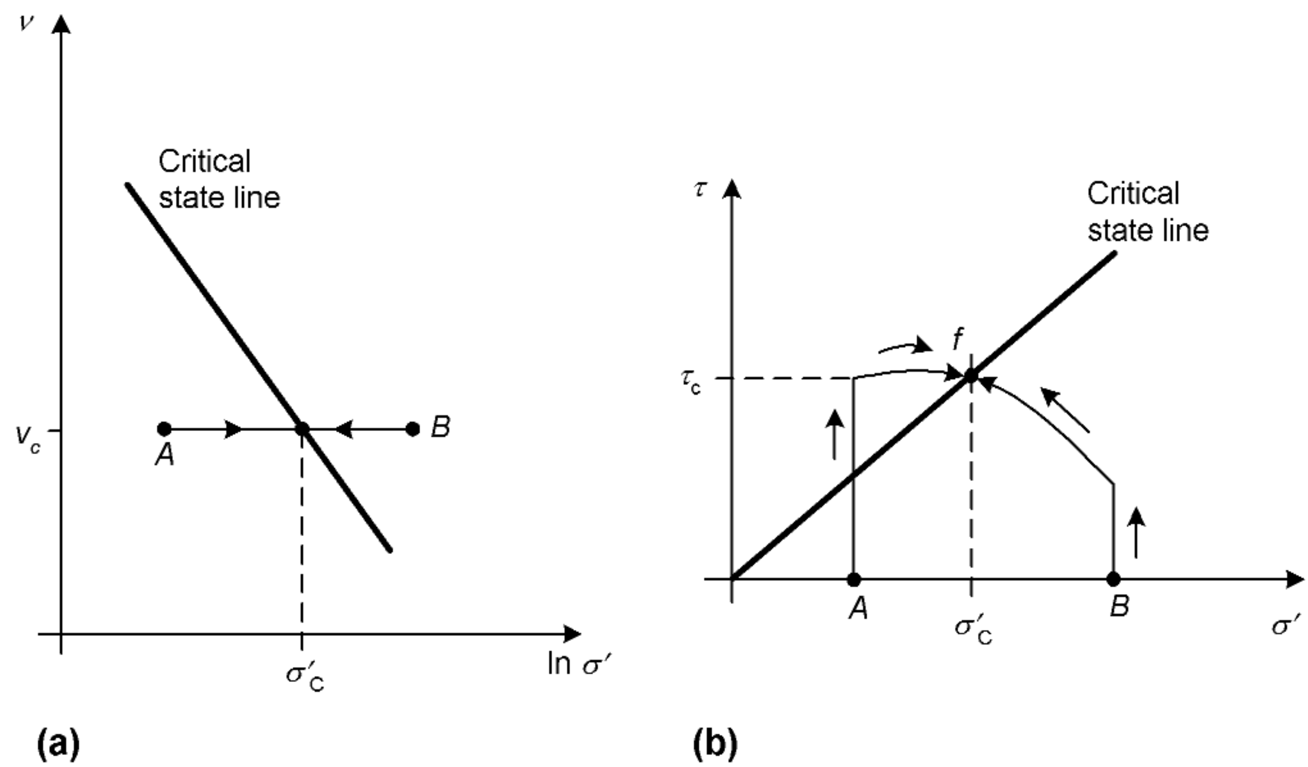


FIGURE D6 *Undrained state paths for clay samples having the same specific volume (a) ν against ln σ', (b) τ against σ'. Sample A is heavily overconsolidated; sample B is lightly overconsolidated*

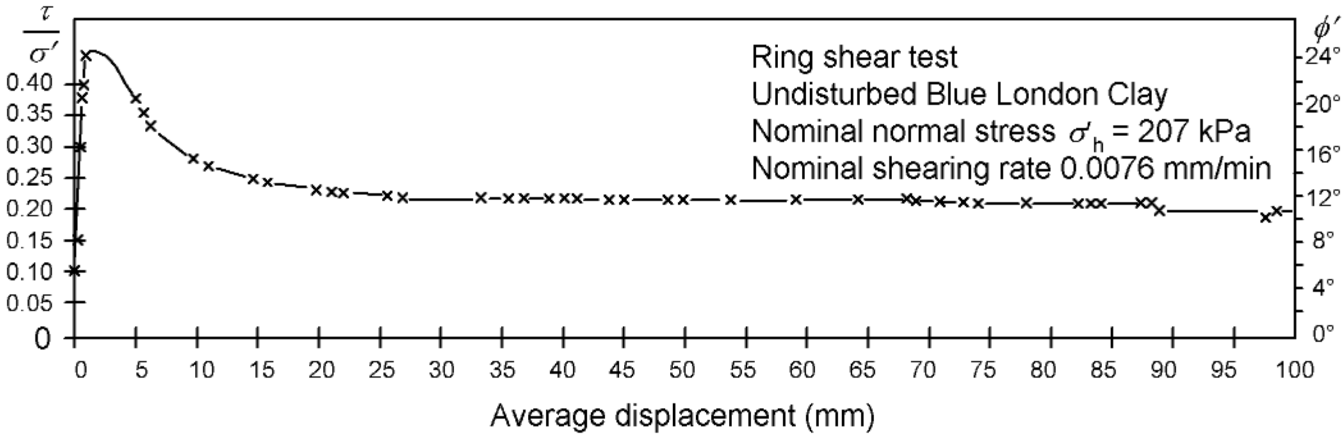


FIGURE D7 *Ring shear test data for undisturbed London Clay (from Bishop et al, 1971)*

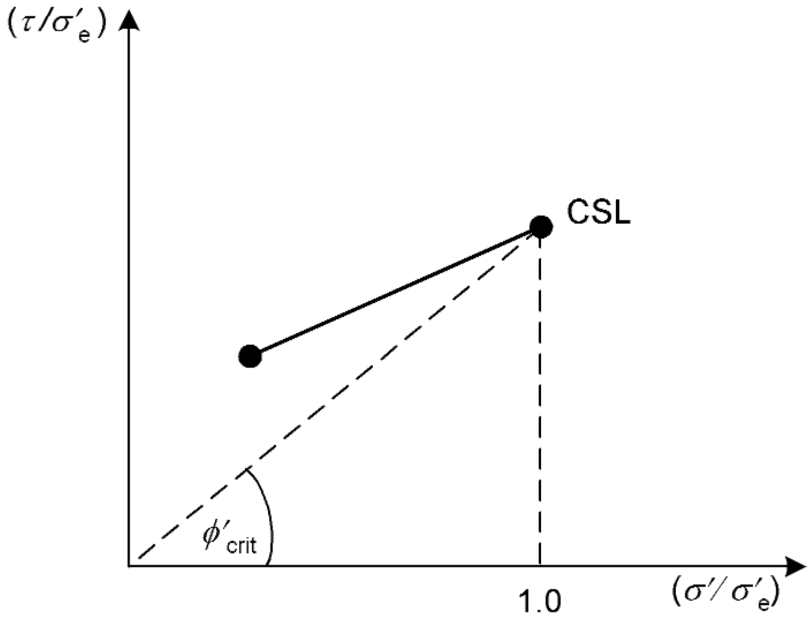
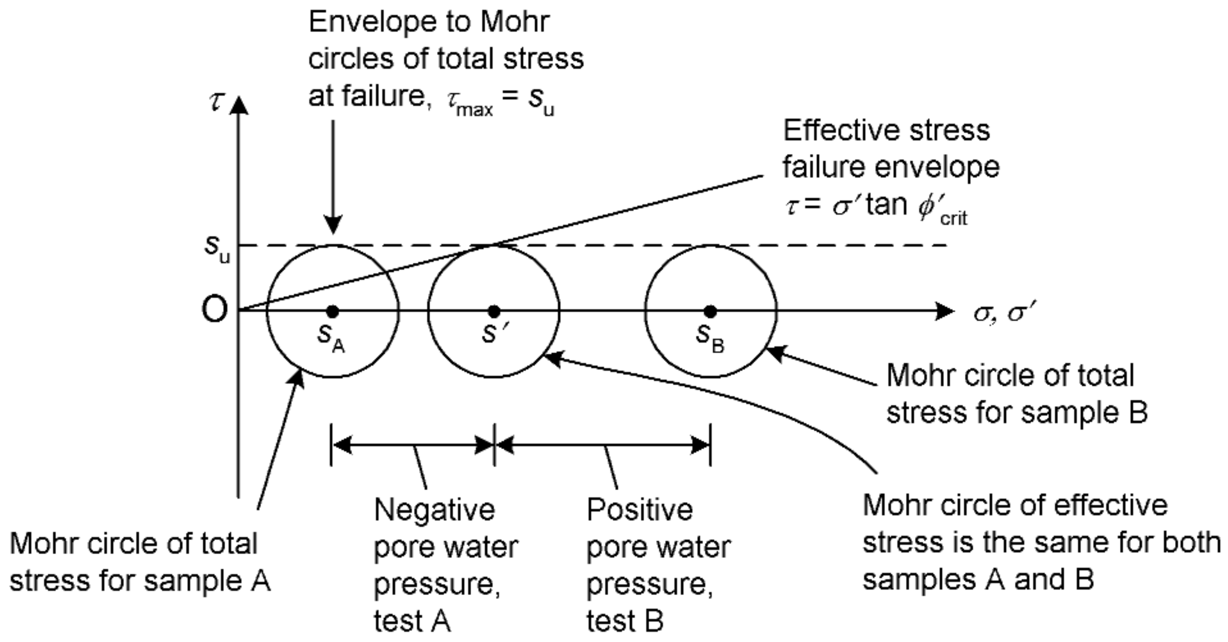


FIGURE D8 *Normalised peak and critical state*



Note : see Figure D6 for undrained state paths for clay samples A and B

FIGURE D9 *Mohr circles representation of undrained shear strength failure criterion in terms of total stresses for shearing at constant specific volume*

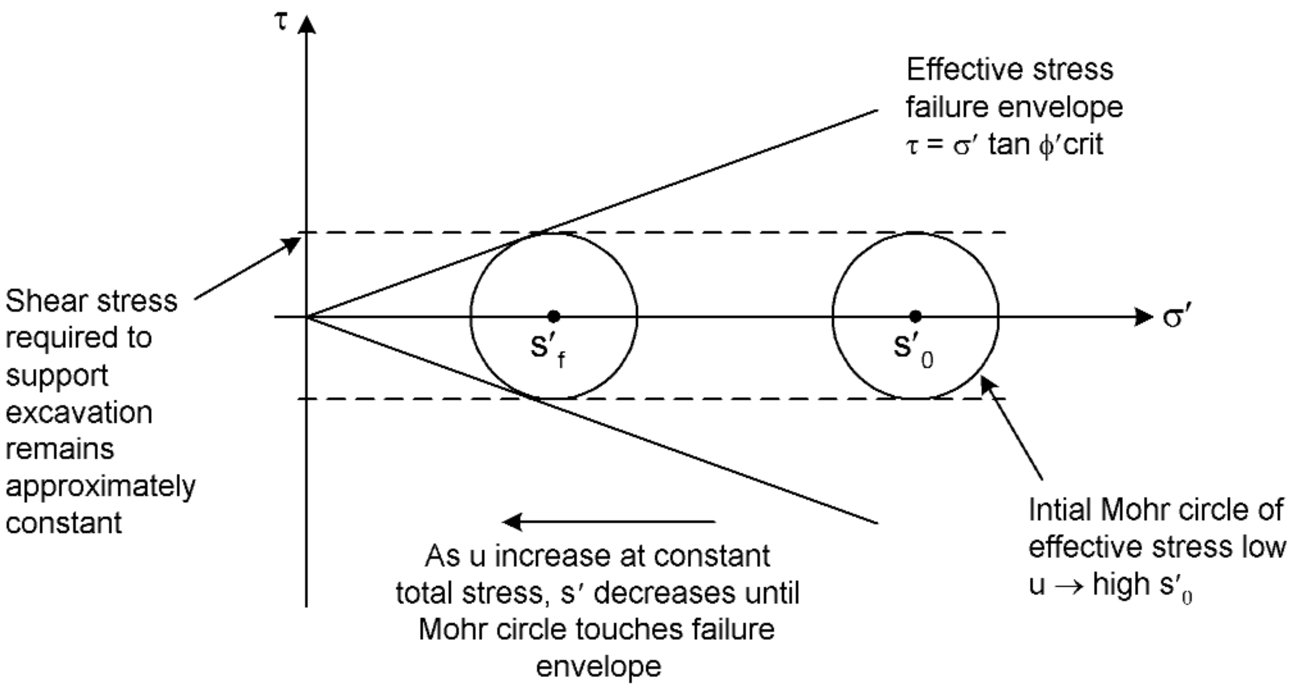


FIGURE D10 - *Failure after dissipation of negative excess pore water pressures induced on excavation*

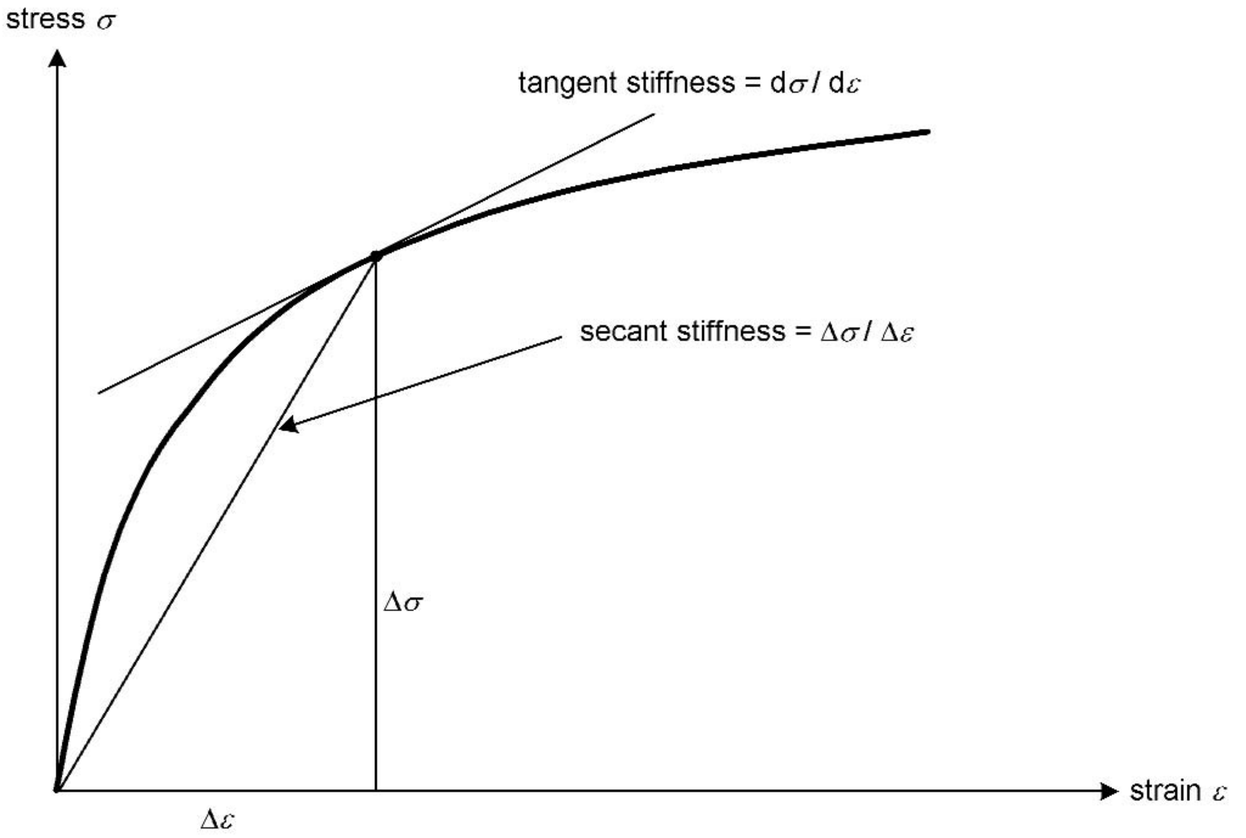


FIGURE D11 *Soil stiffness definition*

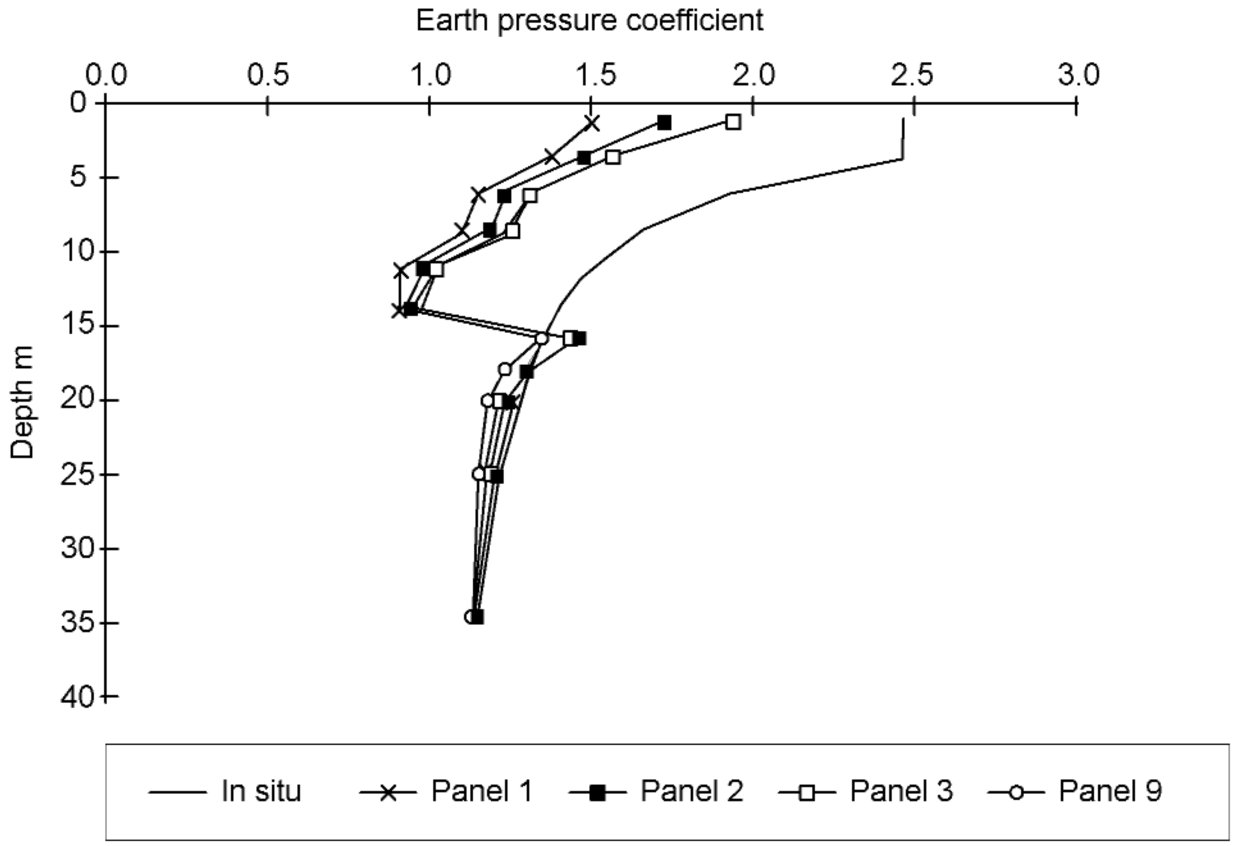


FIGURE E1 *Earth pressure coefficient profiles 1 m behind the centre of the primary panel during construction of the wall: 3D analysis with 5 m panels*

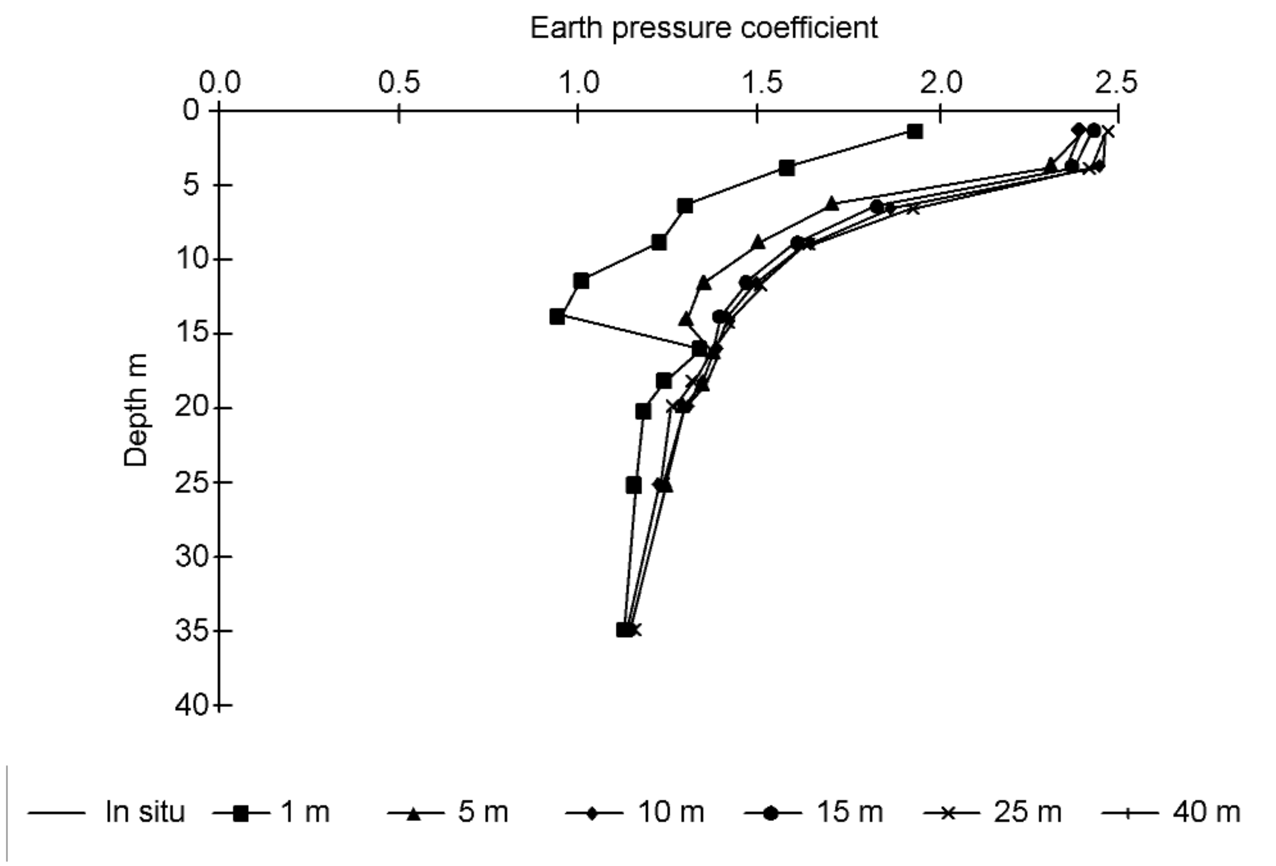
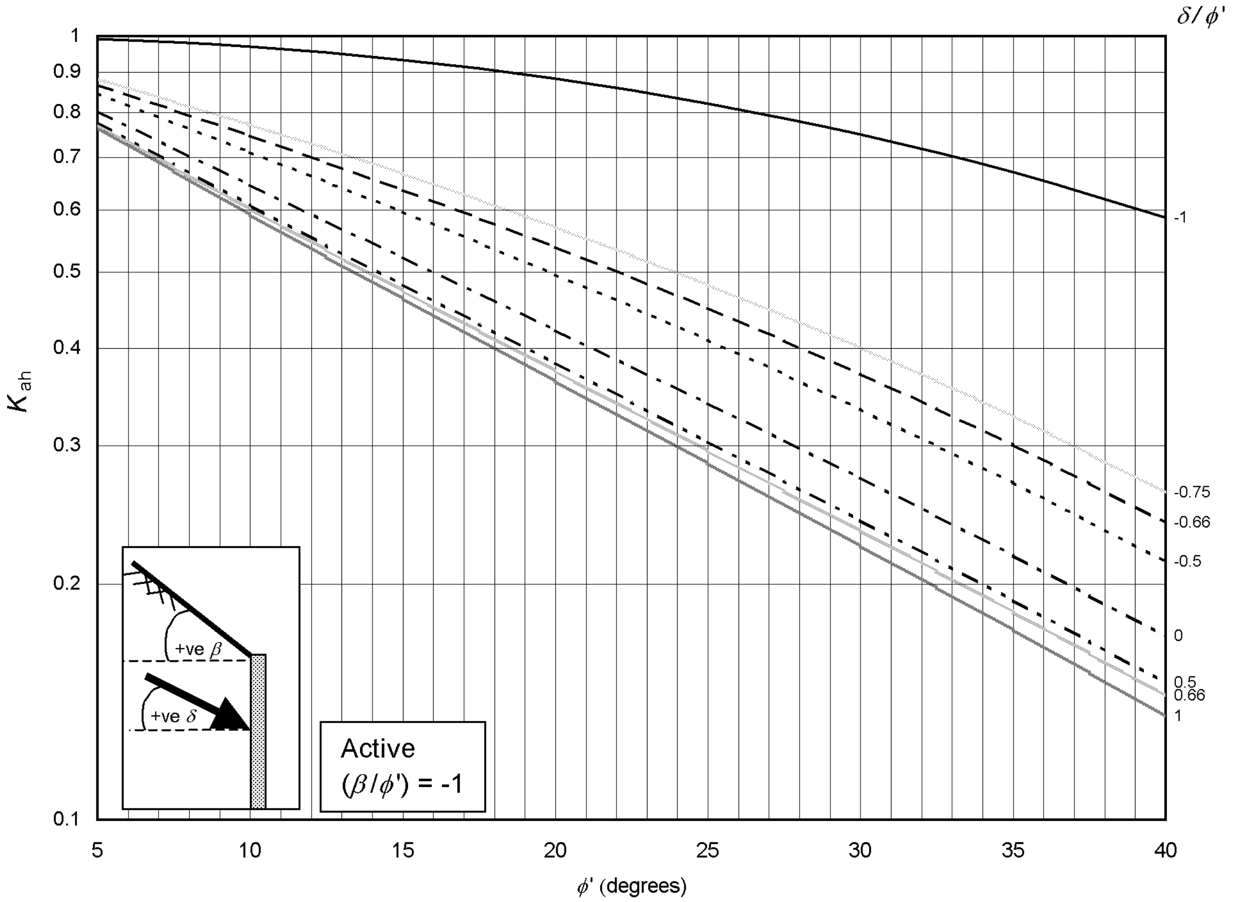


FIGURE E2 *Earth pressure coefficient profiles normal to the centre of the primary panel following completion of the wall: 3D analysis with 5 m panels*



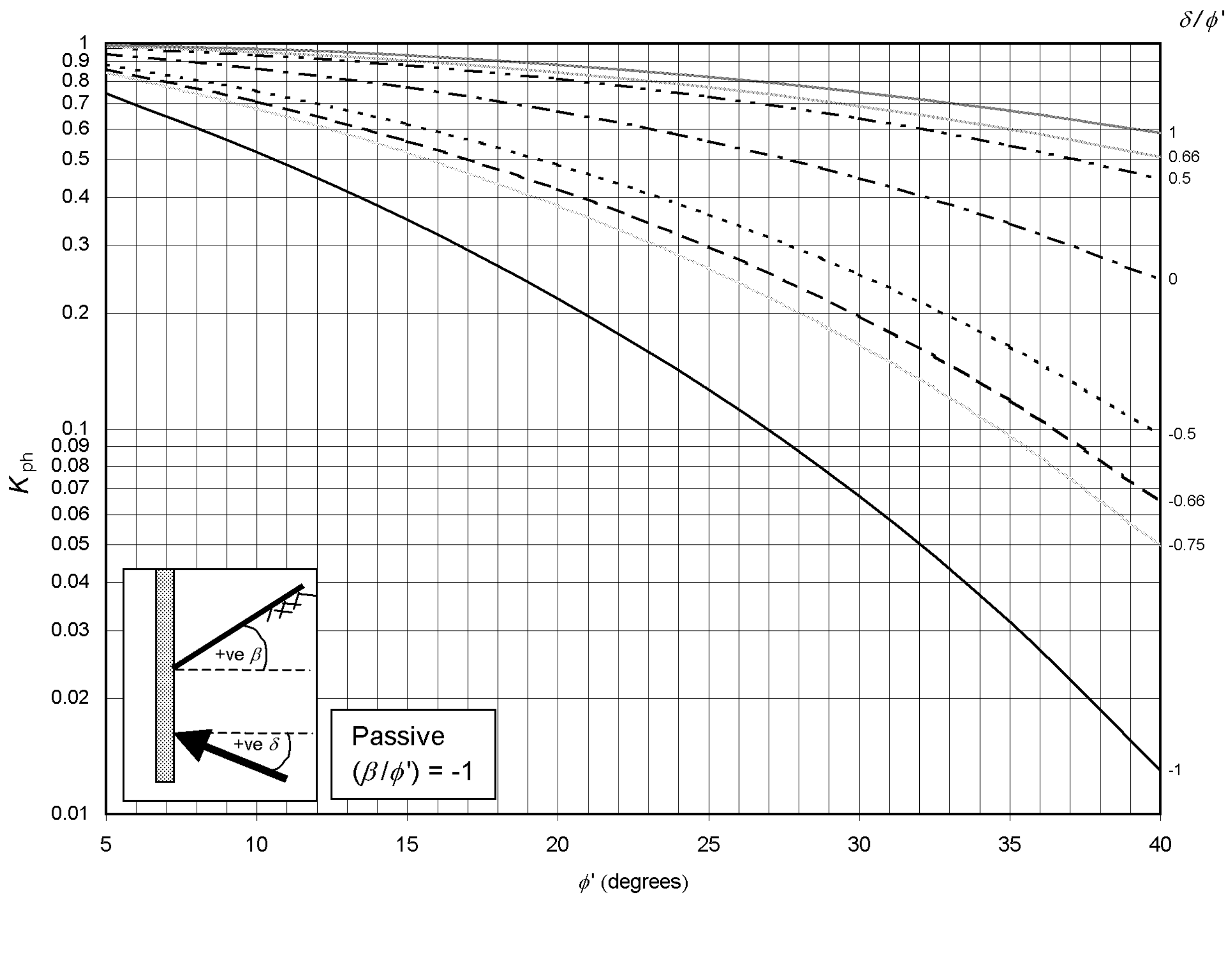
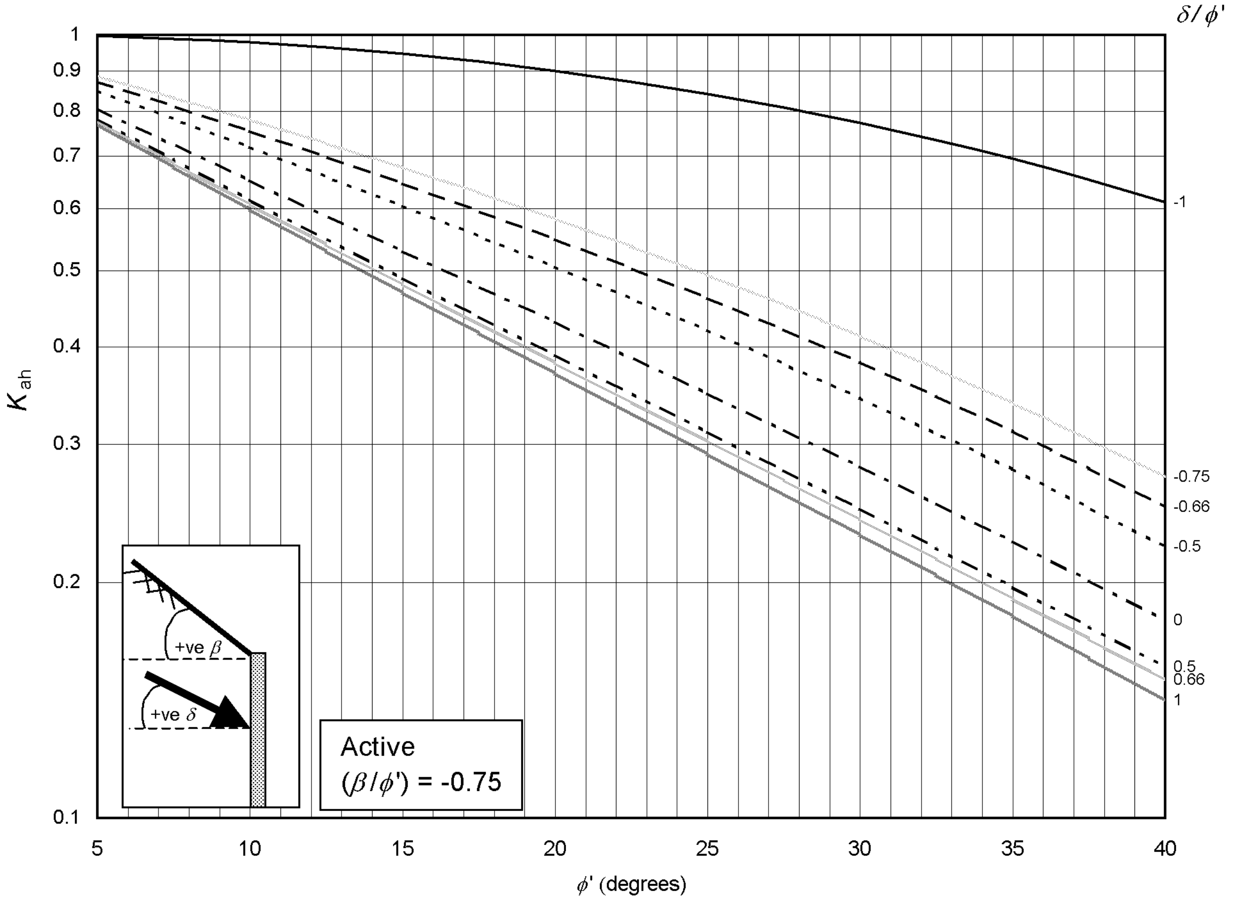


FIGURE F1 *Active and passive earth pressure coefficients, (β/ϕ') = -1*



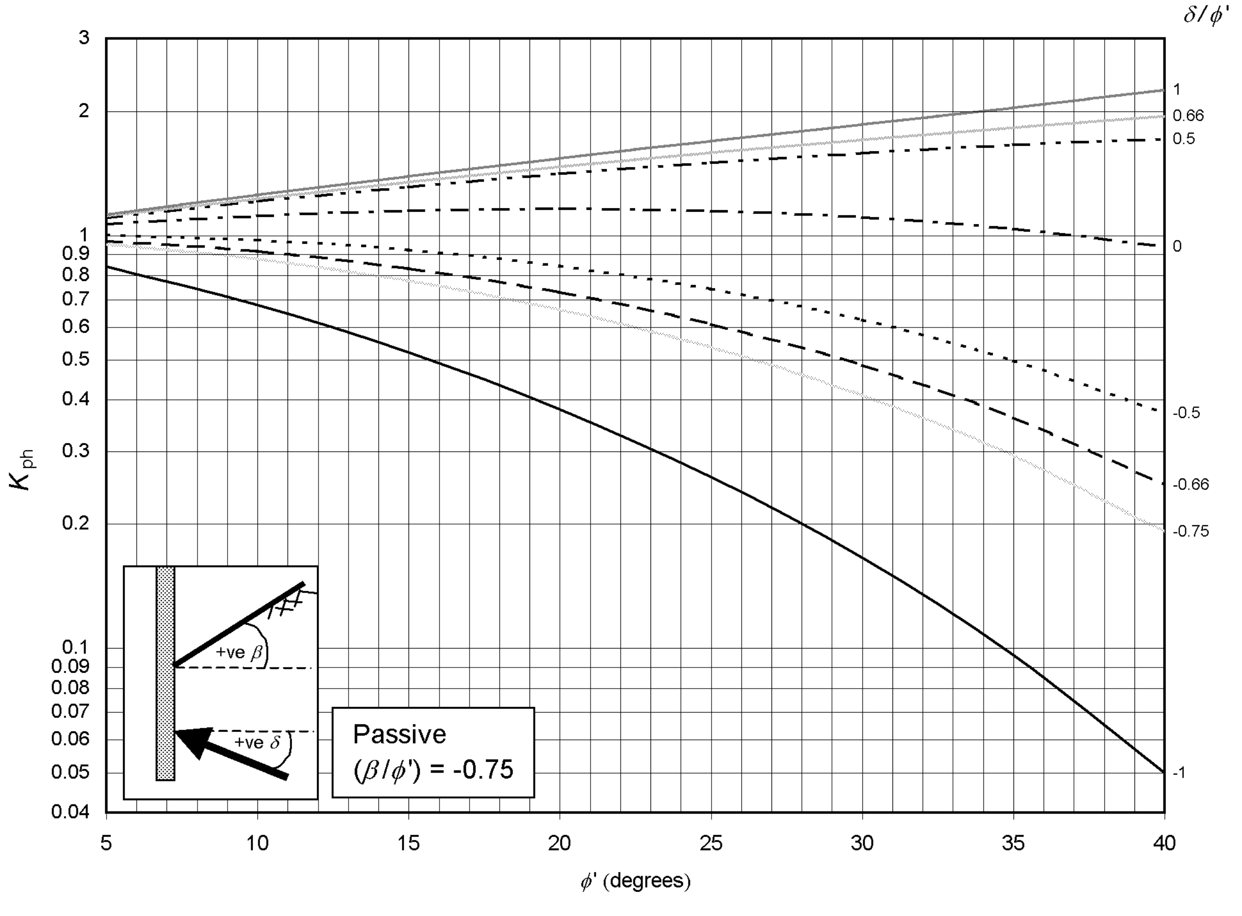
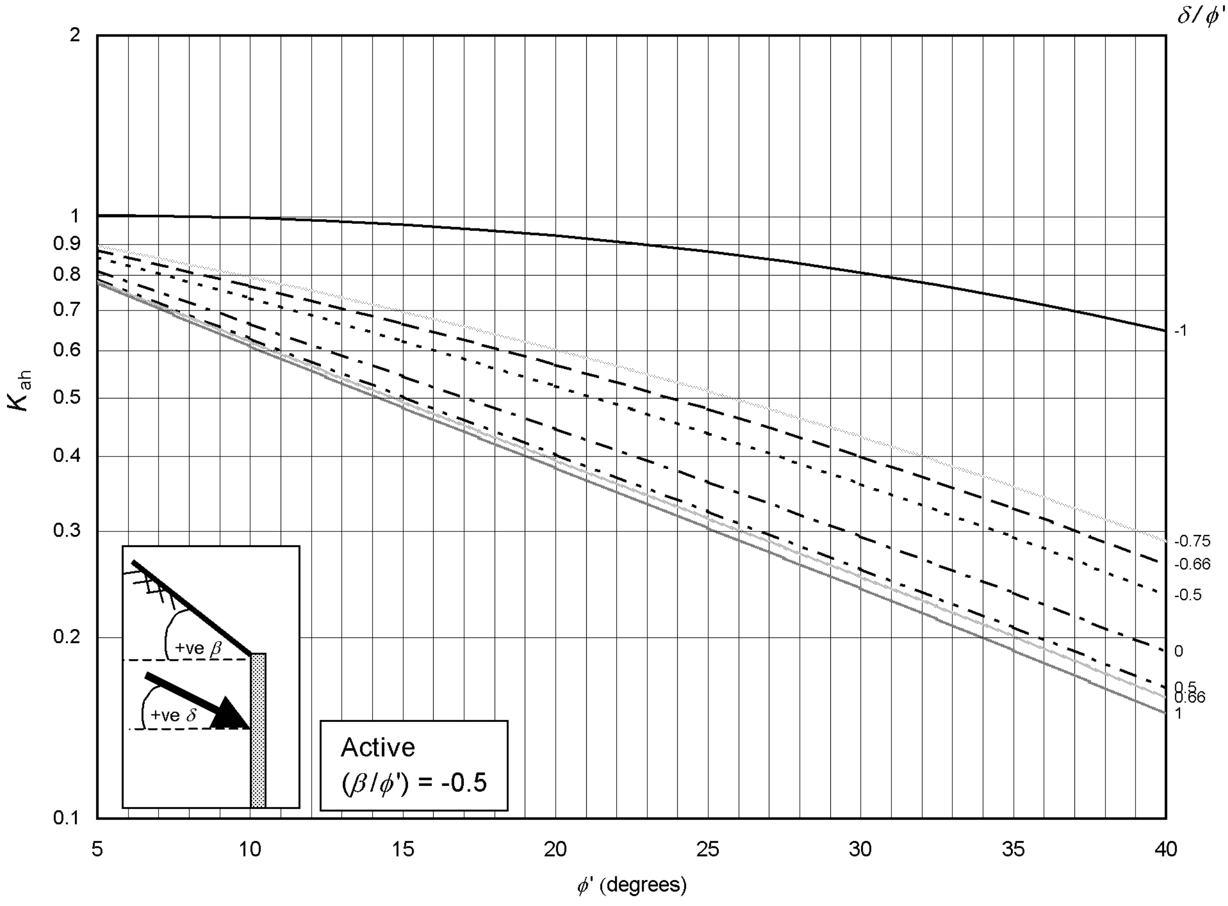


FIGURE F2 *Active and passive earth pressure coefficients, (β/φ') = -0.75*



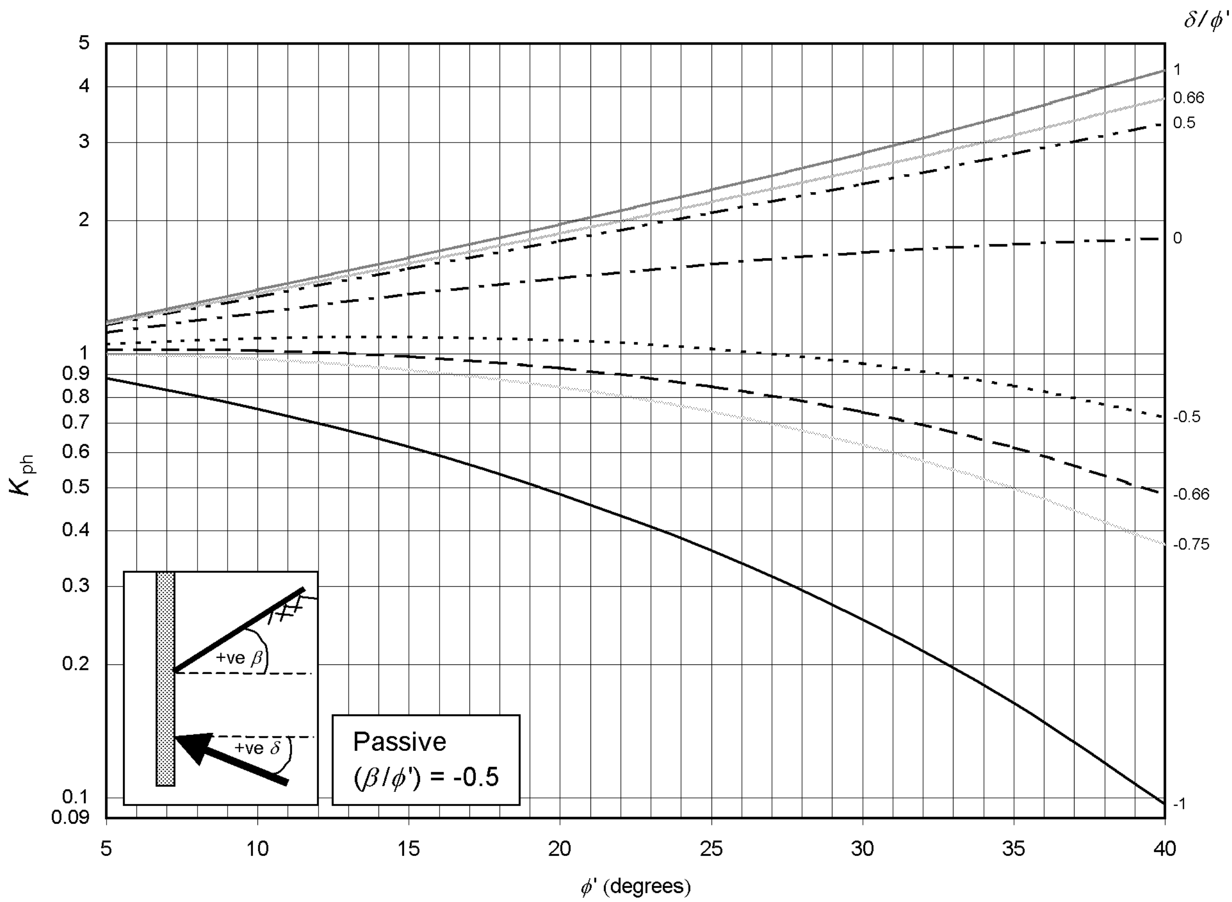
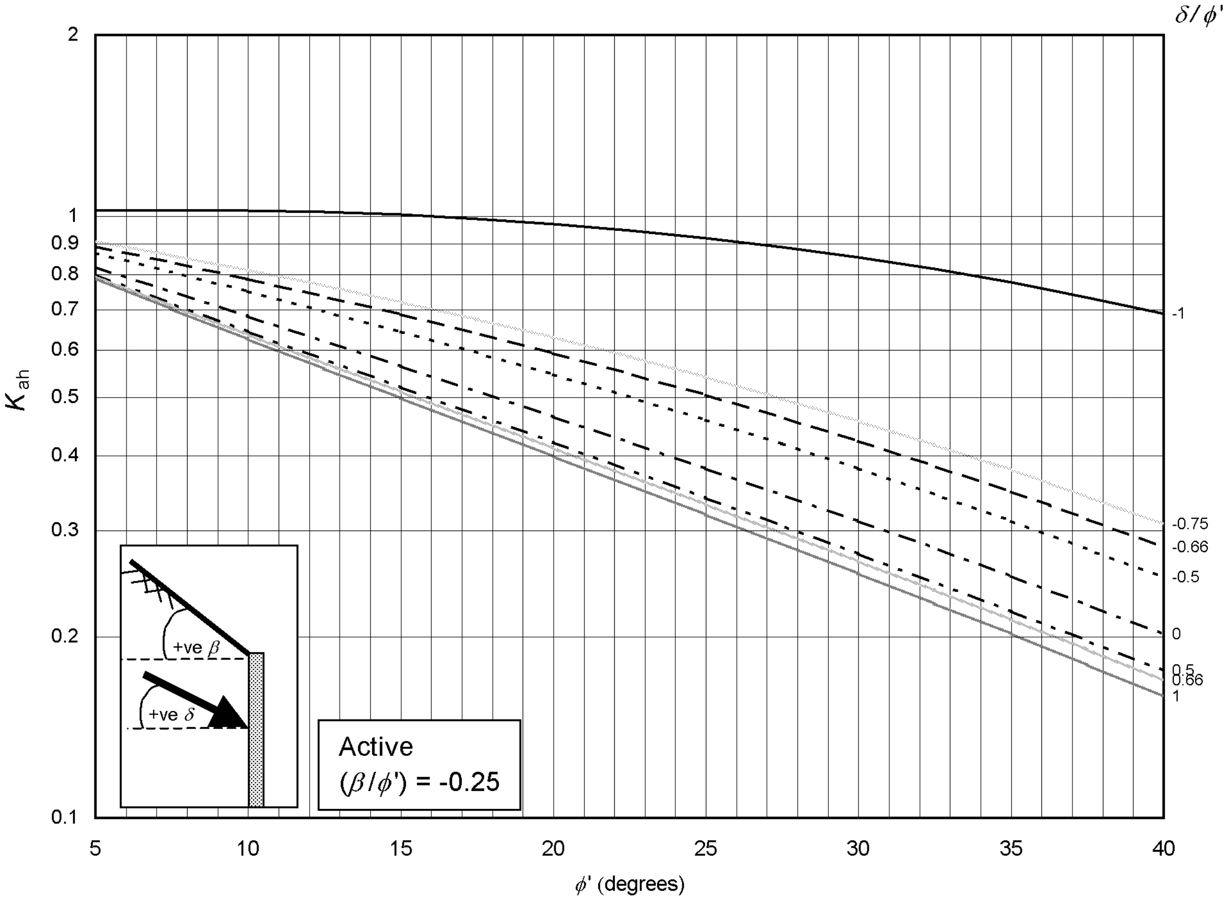


FIGURE F3 *Active and passive earth pressure coefficients, (β / φ') = -0.5*



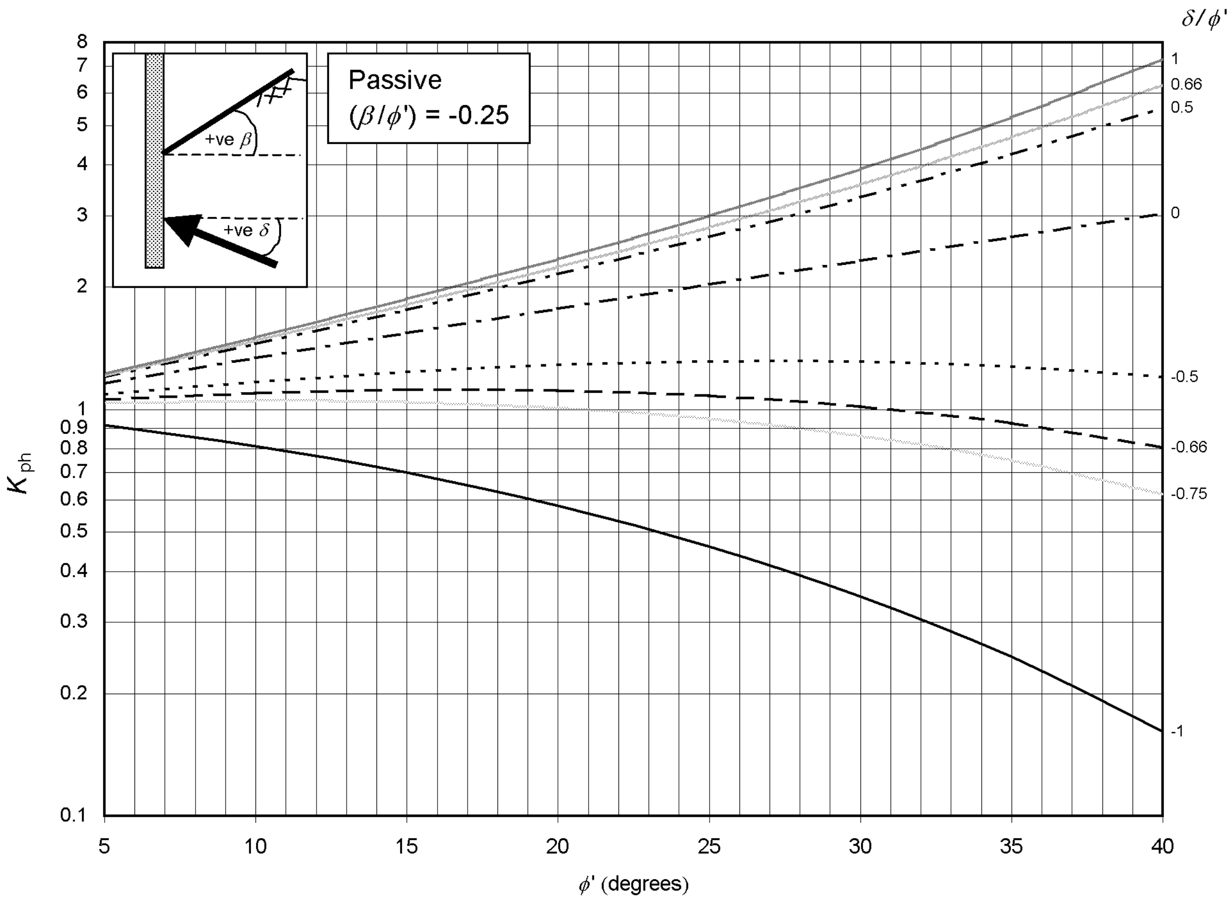
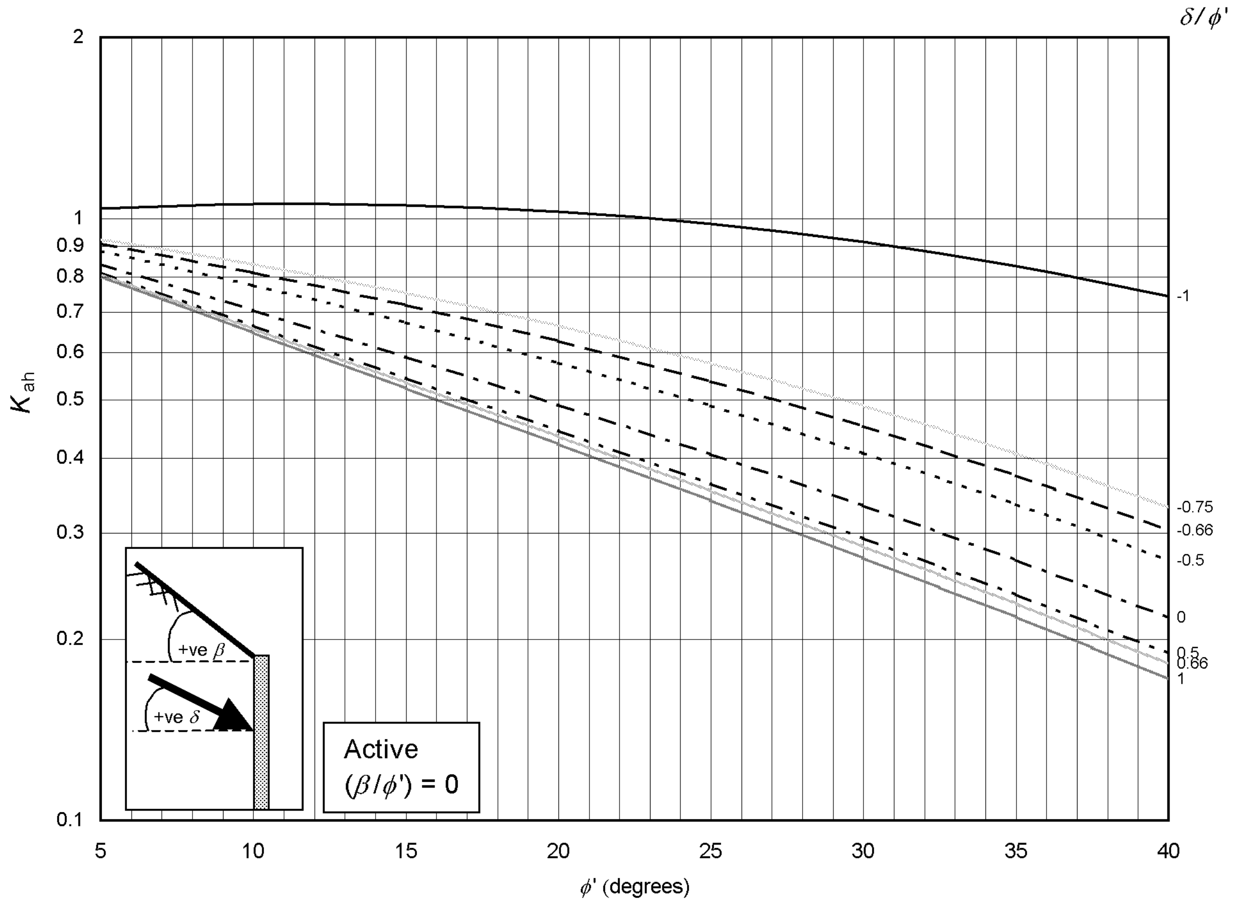


FIGURE F4 *Active and passive earth pressure coefficients, (β/ϕ') = -0.25*



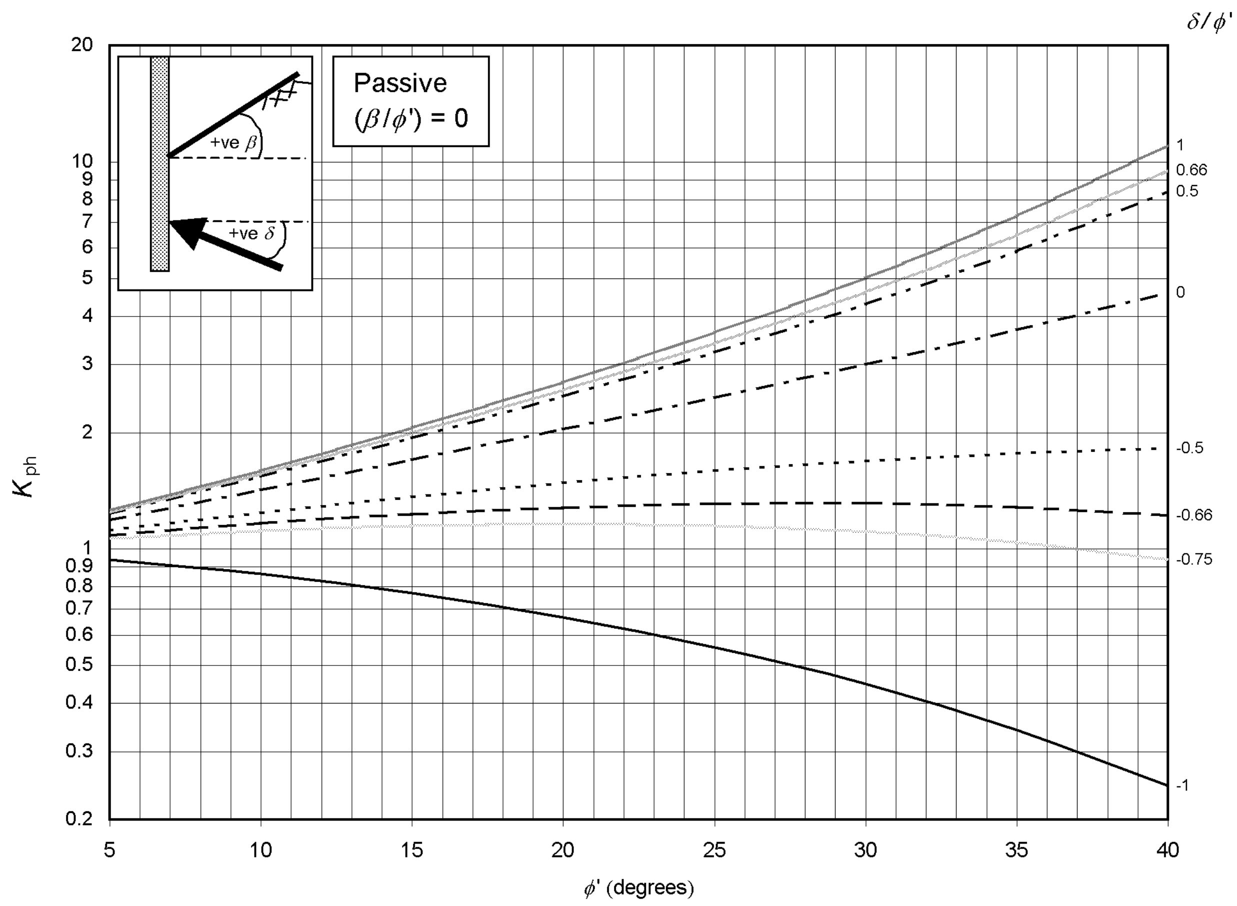
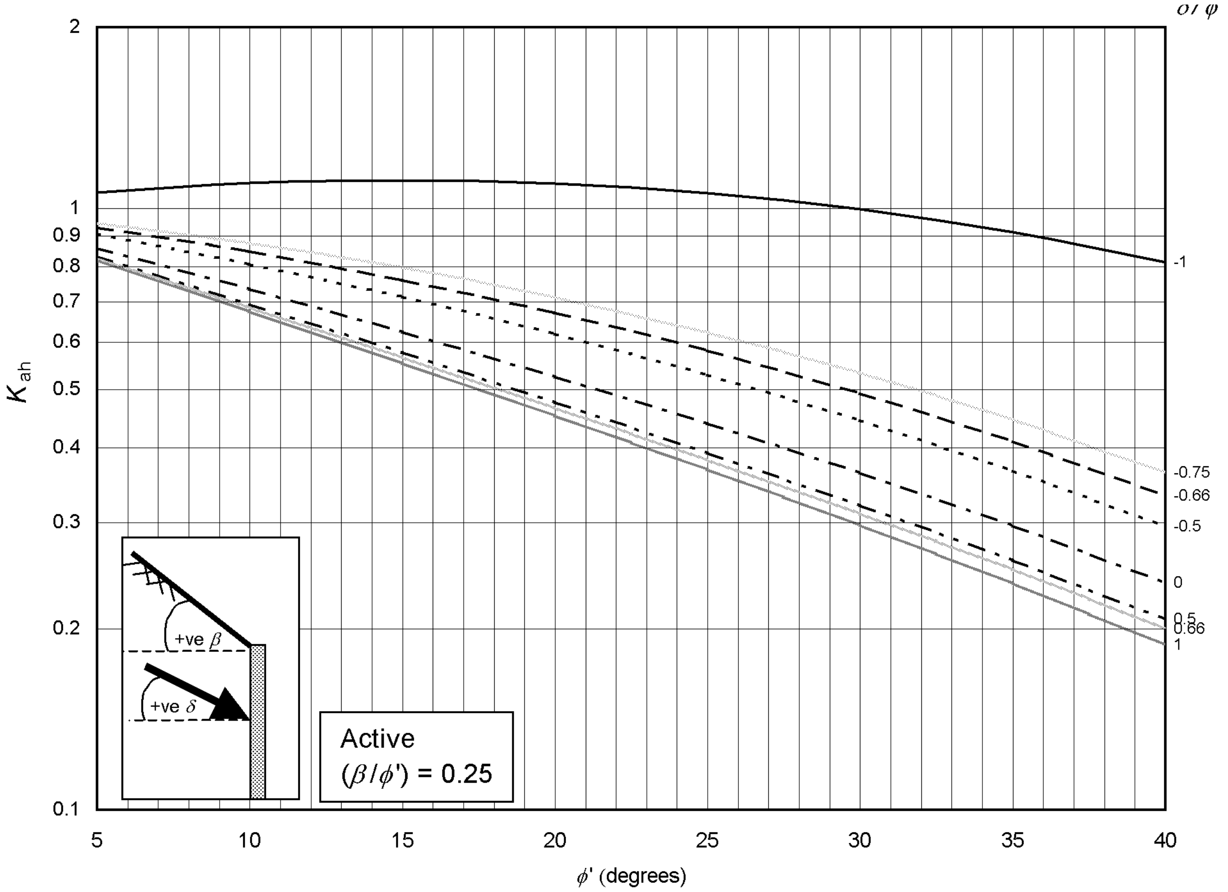


FIGURE F5 *Active and passive earth pressure coefficients, (β/ϕ') = 0*



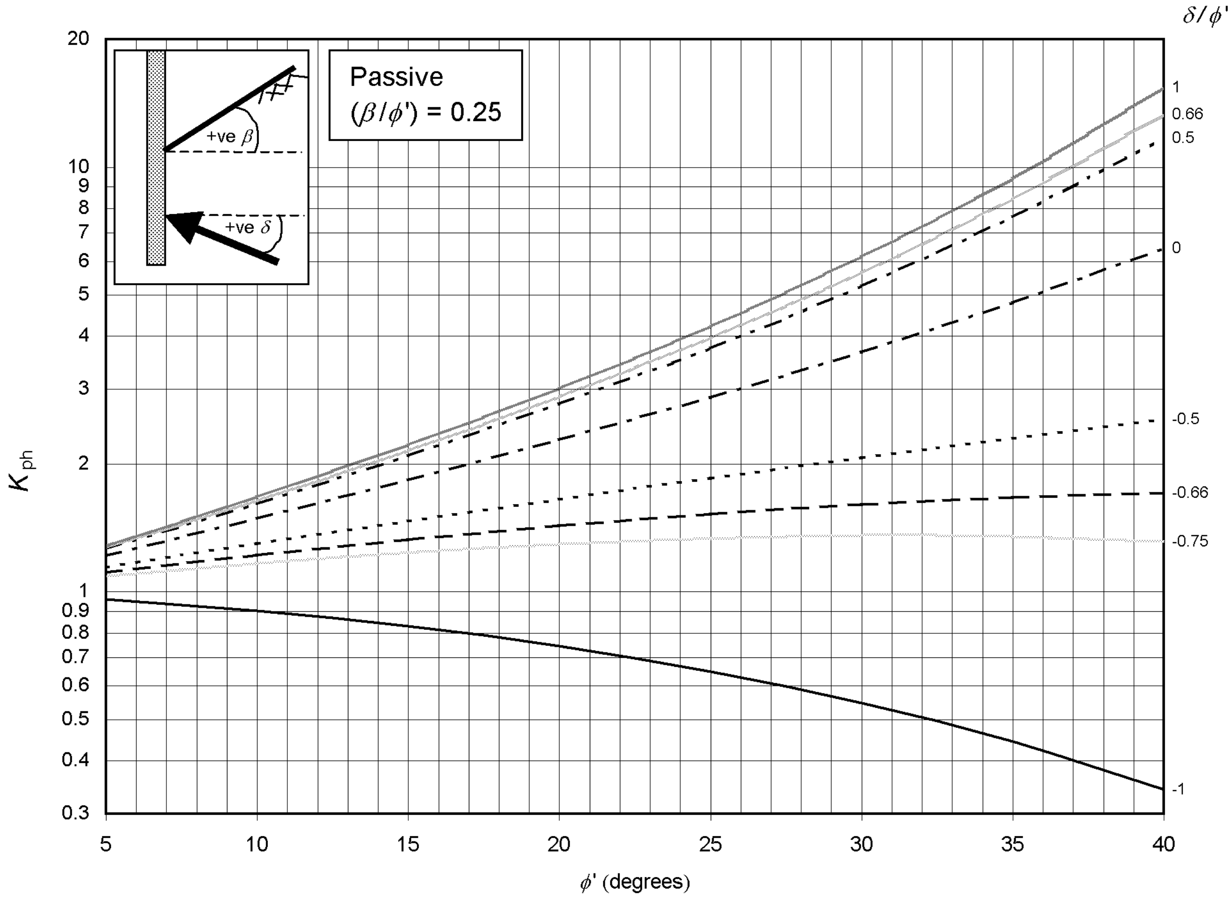
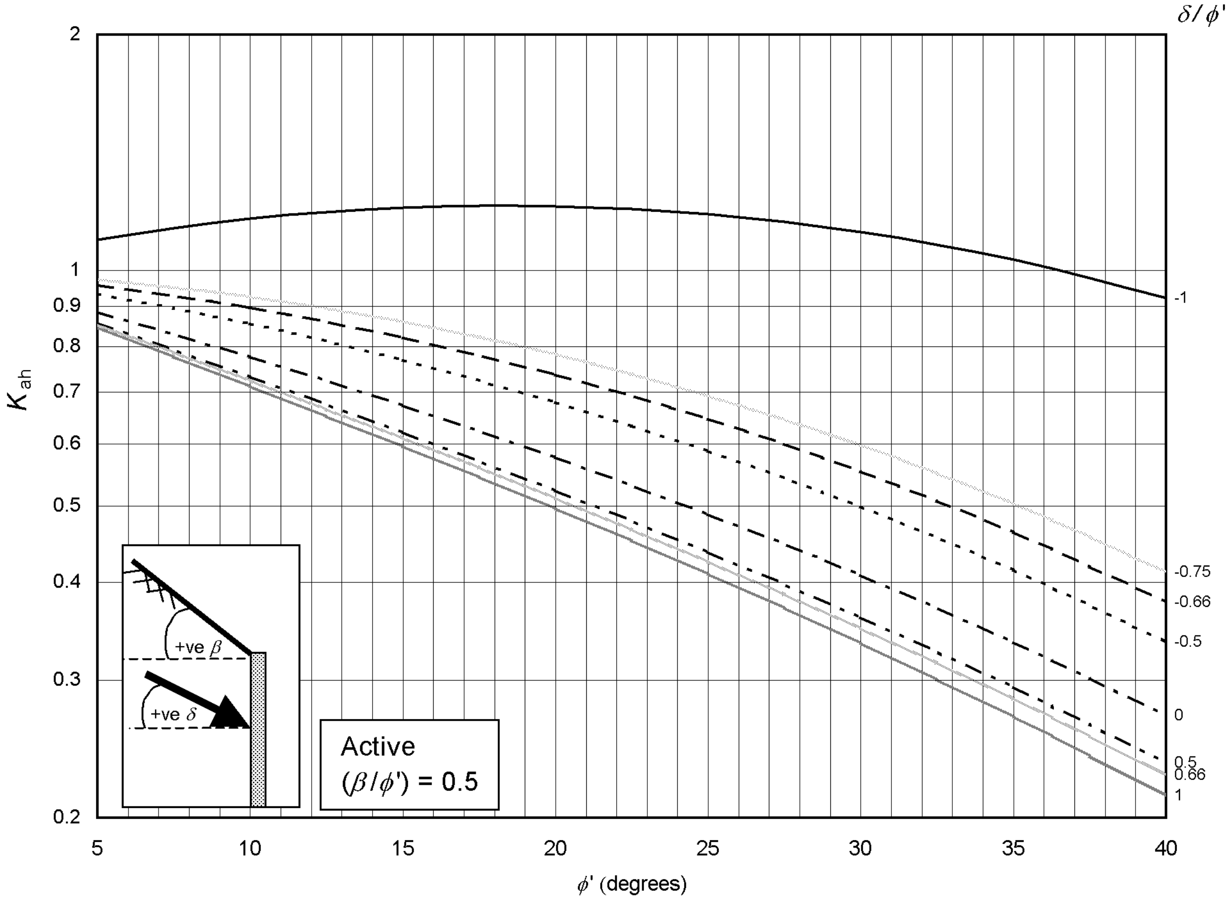


FIGURE F6 *Active and passive earth pressure coefficients, (β / φ') = 0.25*



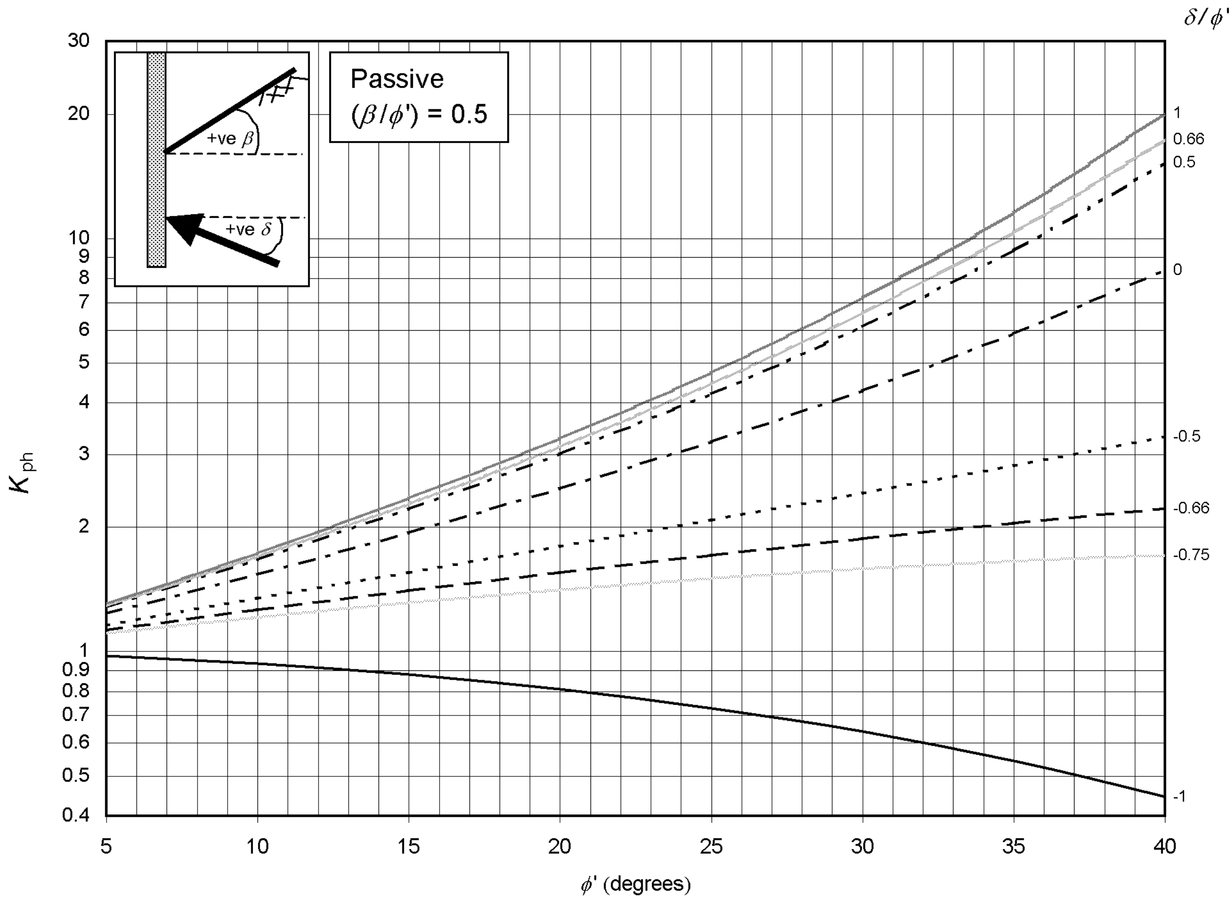
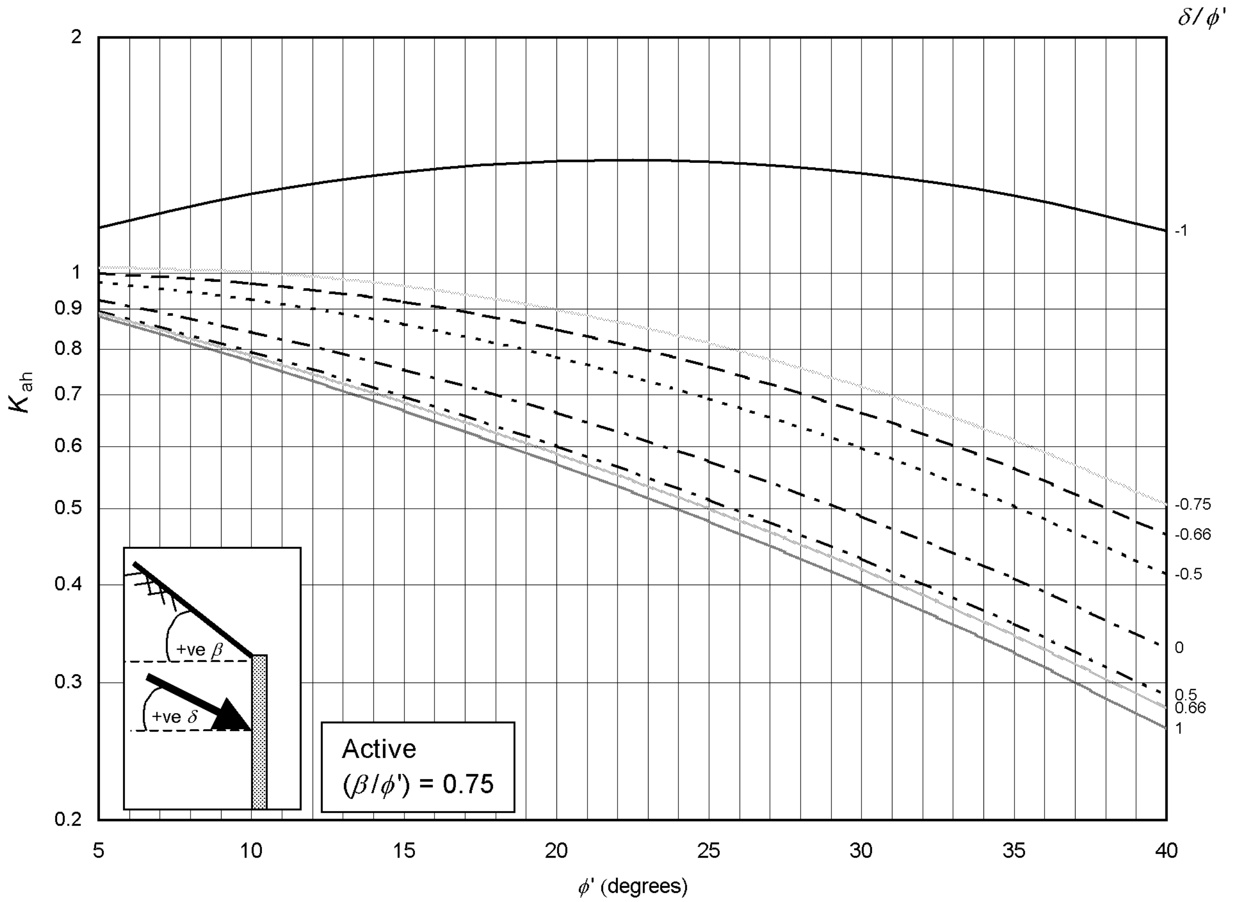


FIGURE F7 *Active and passive earth pressure coefficients, $(\beta/\phi') = 0.5$*



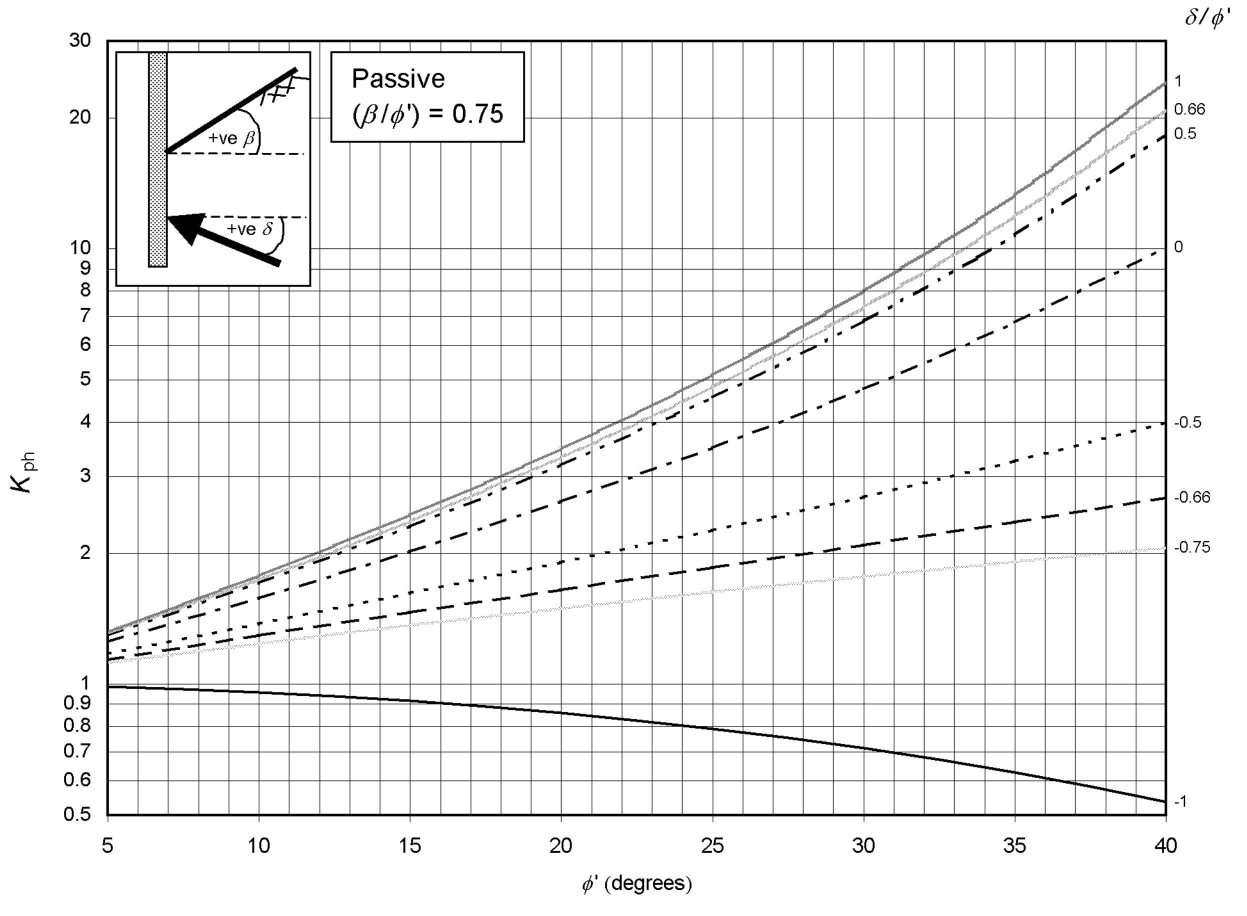
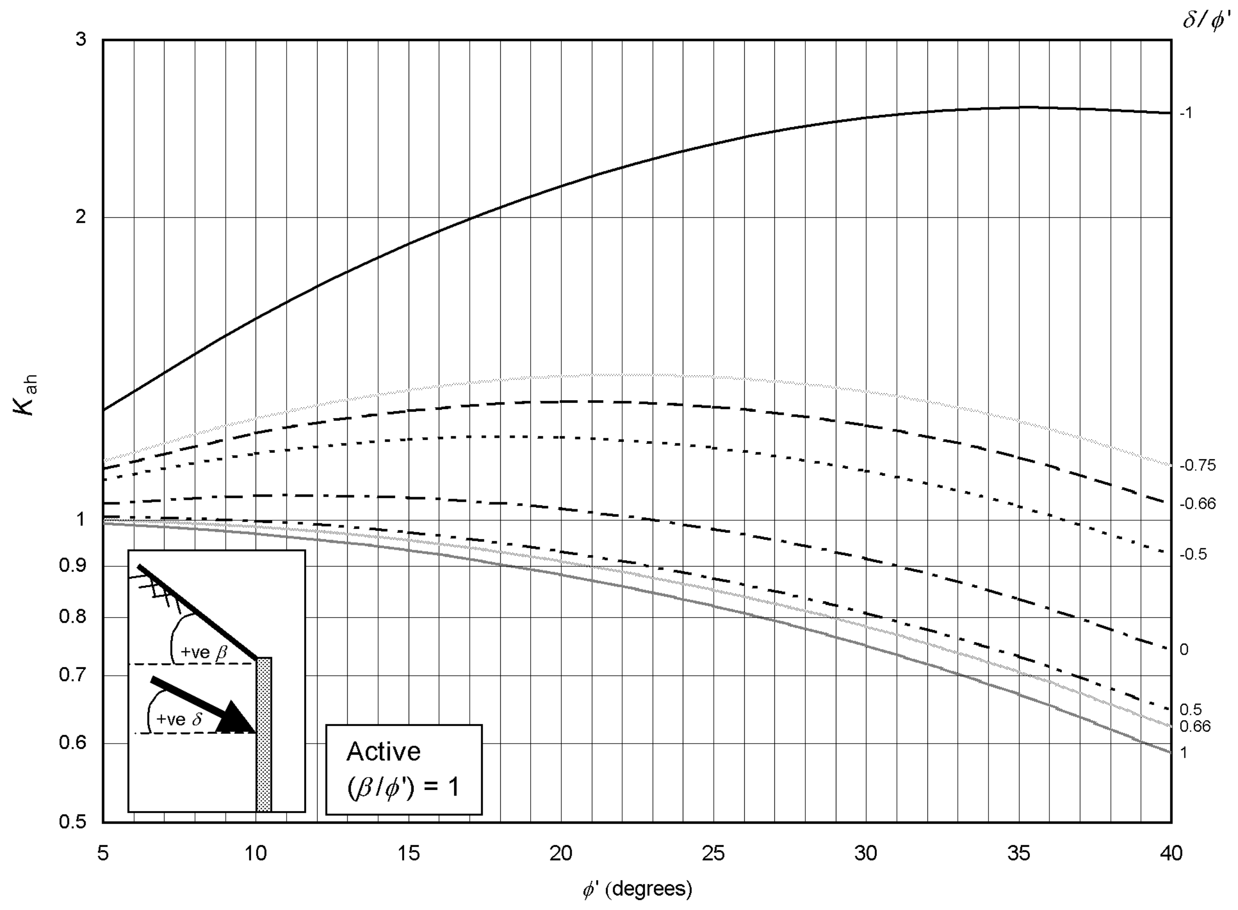


FIGURE F8 *Active and passive earth pressure coefficients, (β/ϕ') = 0.75*



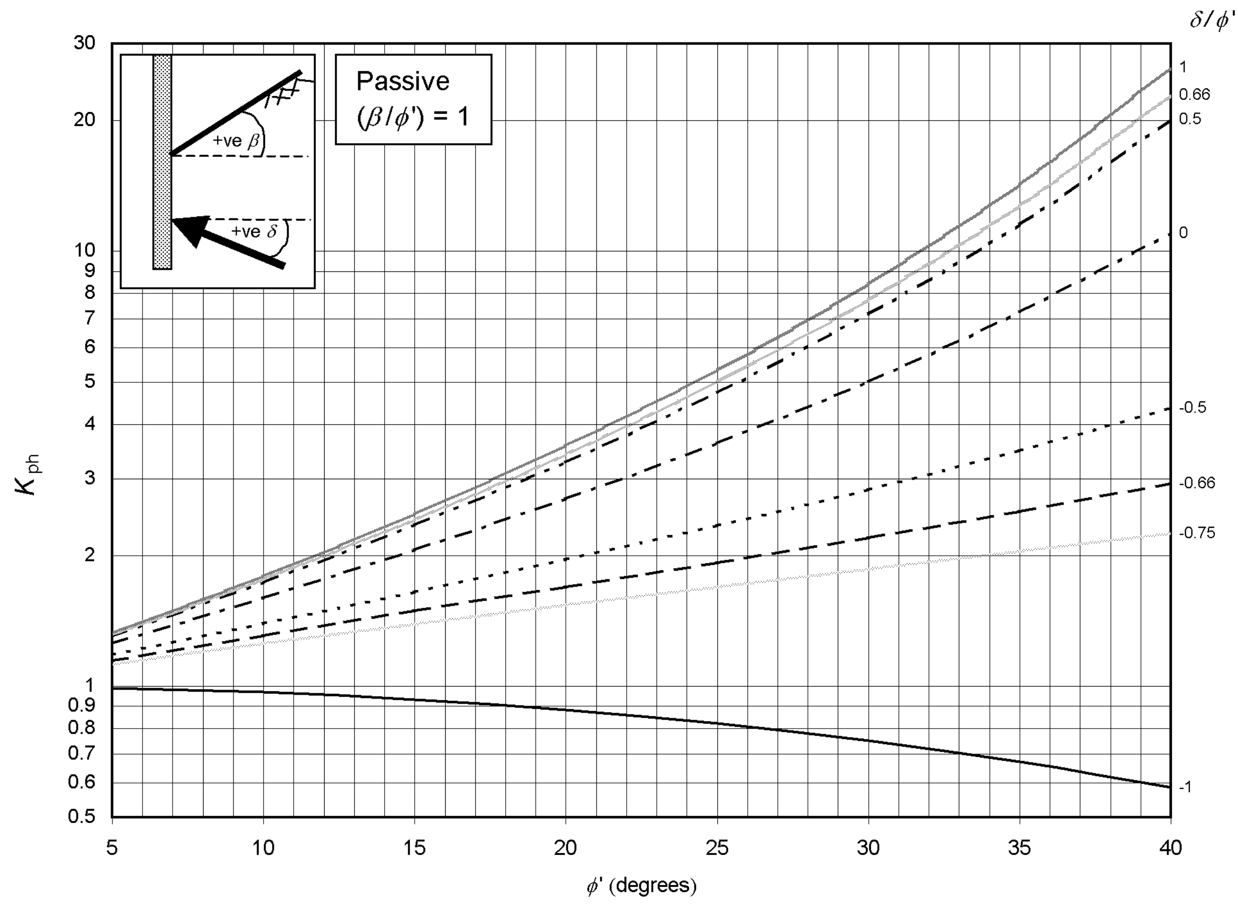
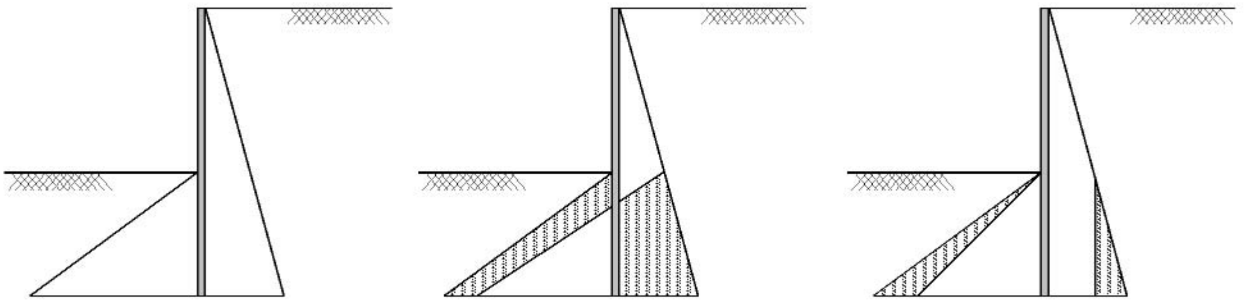


FIGURE F9 *Active and passive earth pressure coefficients, $(\beta/\phi') = 1$*

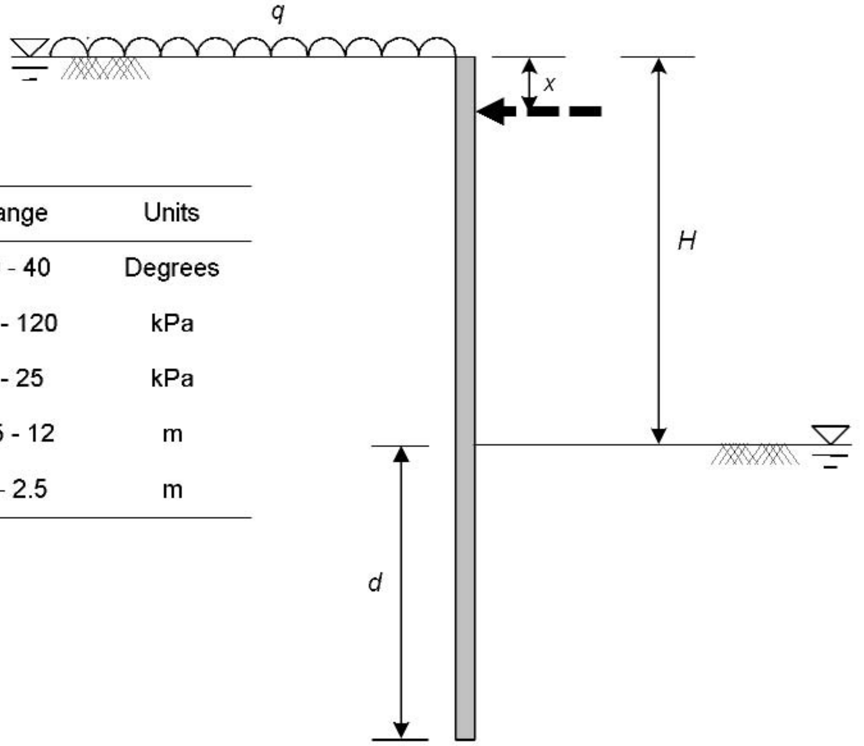


(a) Gross pressure method (after CP2, 1951)

(b) Net total pressure method (after British Steel, 1997)

(b) Net available passive resistance (after Burland *et al*, 1981)

FIGURE G1 *Methods of assessing the ratio of restoring moment to overturning moments*



Variable	Range	Units
ϕ'	20 - 40	Degrees
s_u	60 - 120	kPa
q	0 - 25	kPa
H	4.5 - 12	m
x	0 - 2.5	m

FIGURE G2 *Wall geometry for general analysis case and range of variables*

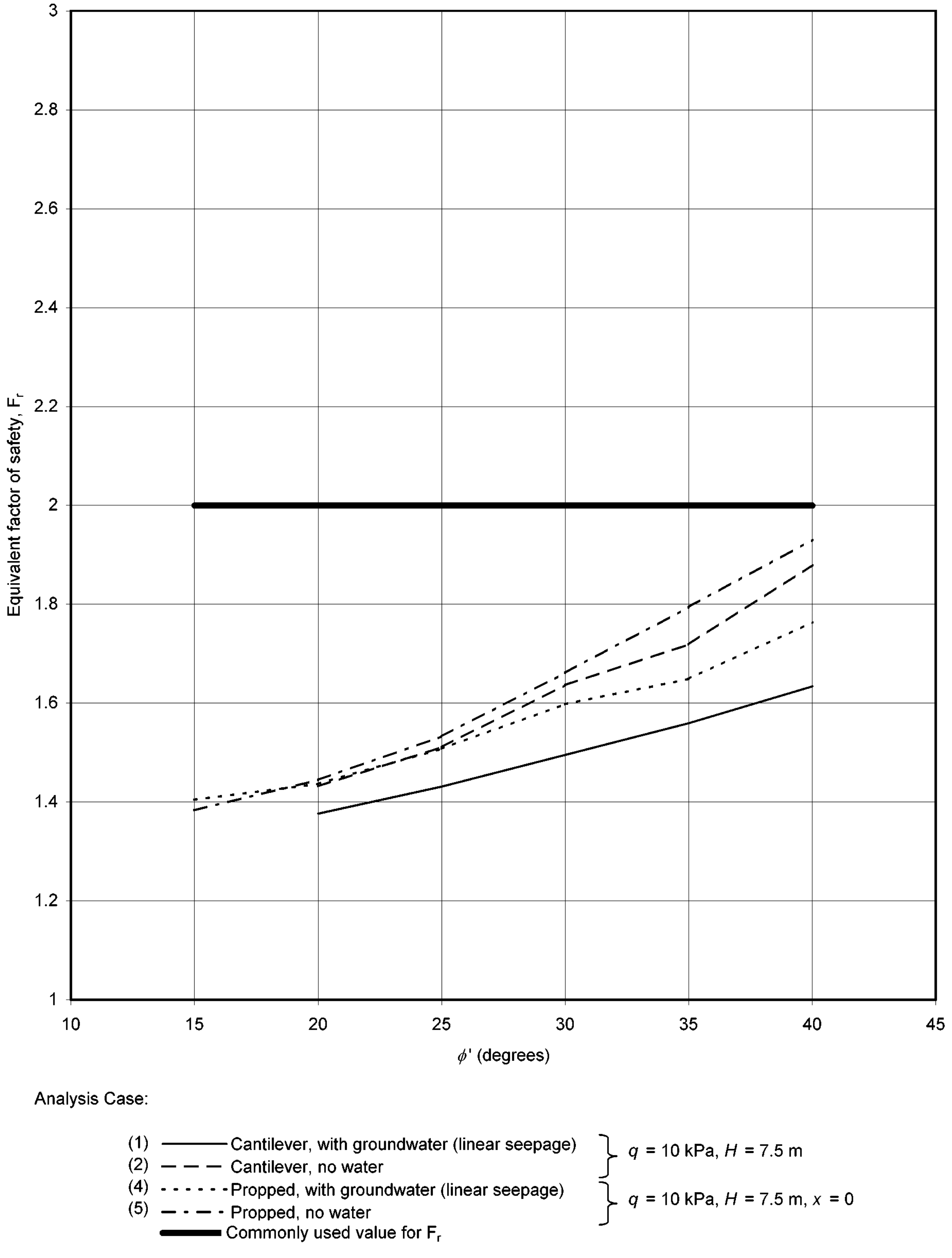


FIGURE G3 *Equivalent F_r values for F_s = 1.2*

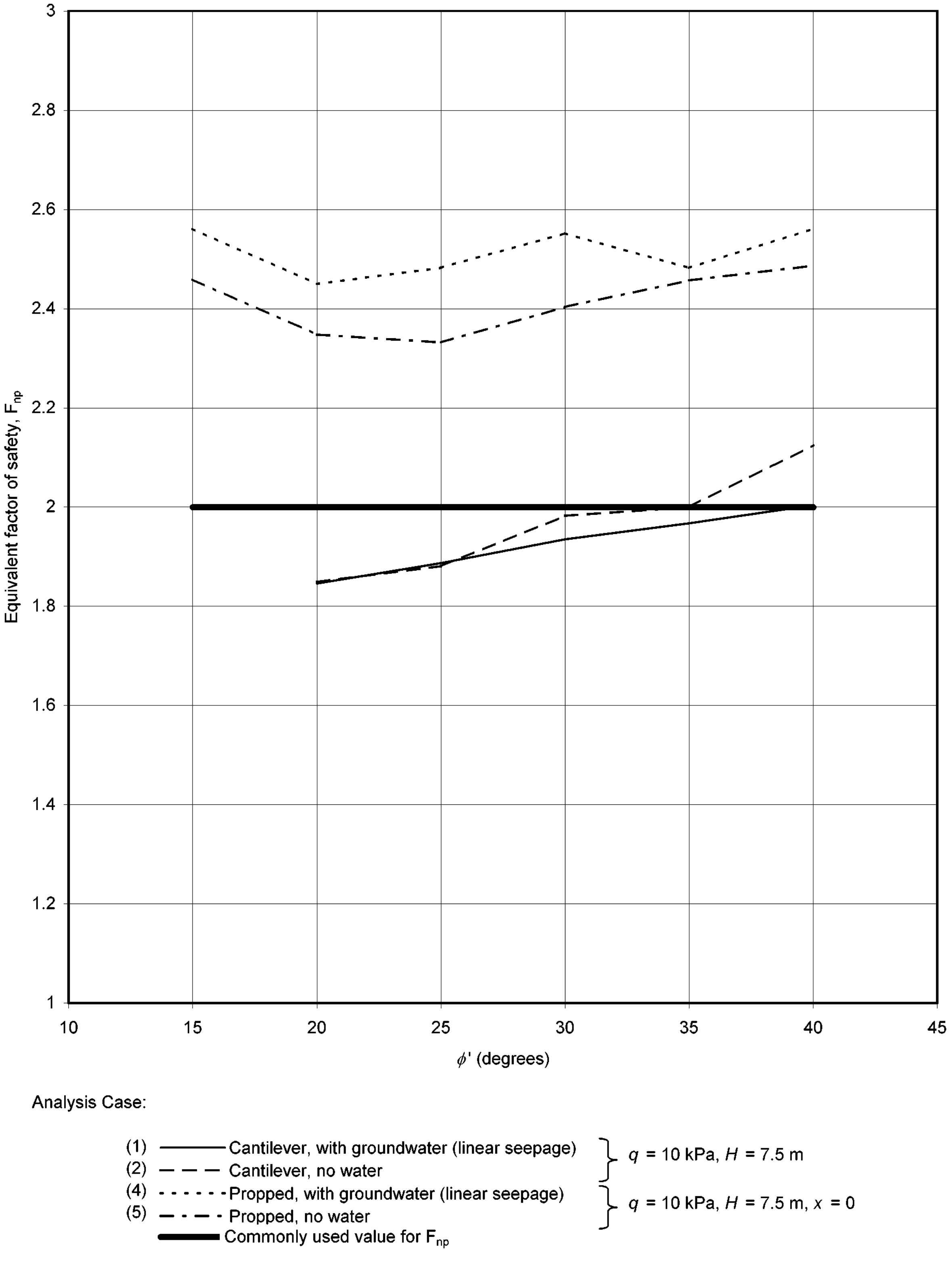


FIGURE G4 *Equivalent F_{np} values for F_s = 1.2*

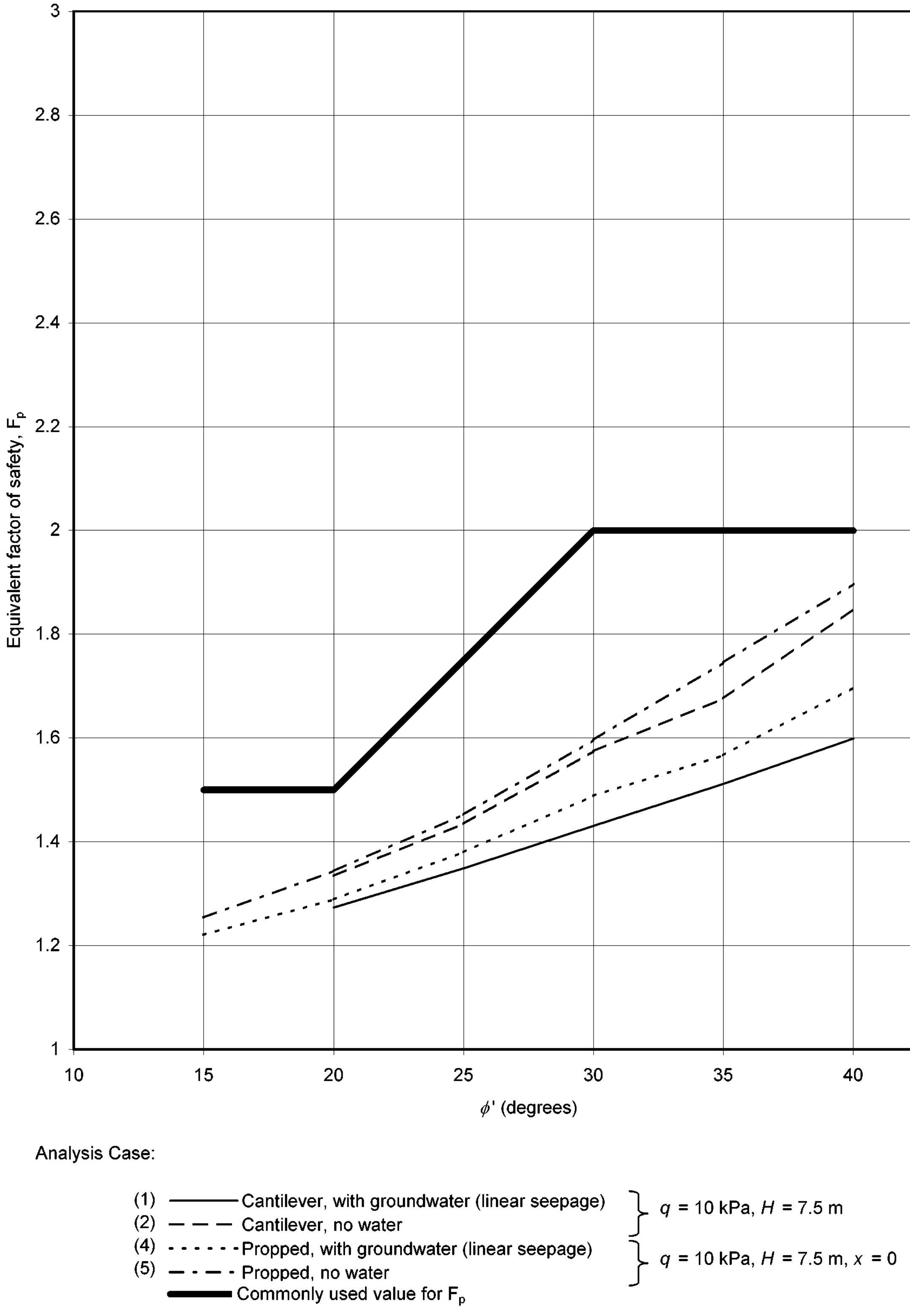


FIGURE G5 *Equivalent F_p values for F_s = 1.2*

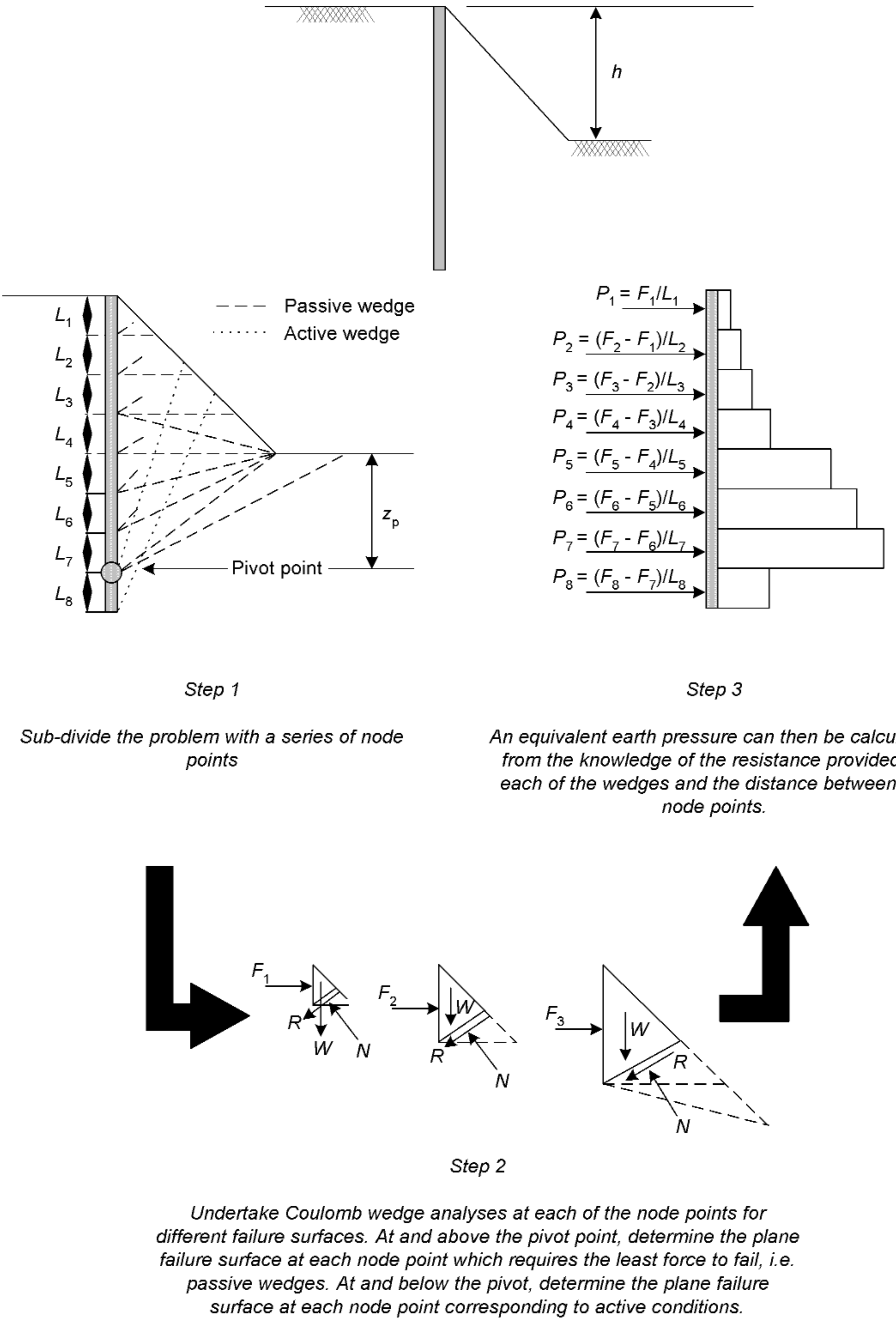


FIGURE I.1 *Modified equilibrium analysis approach: multiple Coulomb wedge analysis (undrained) in terms of total stresses for a retaining wall supported by an earth berm*

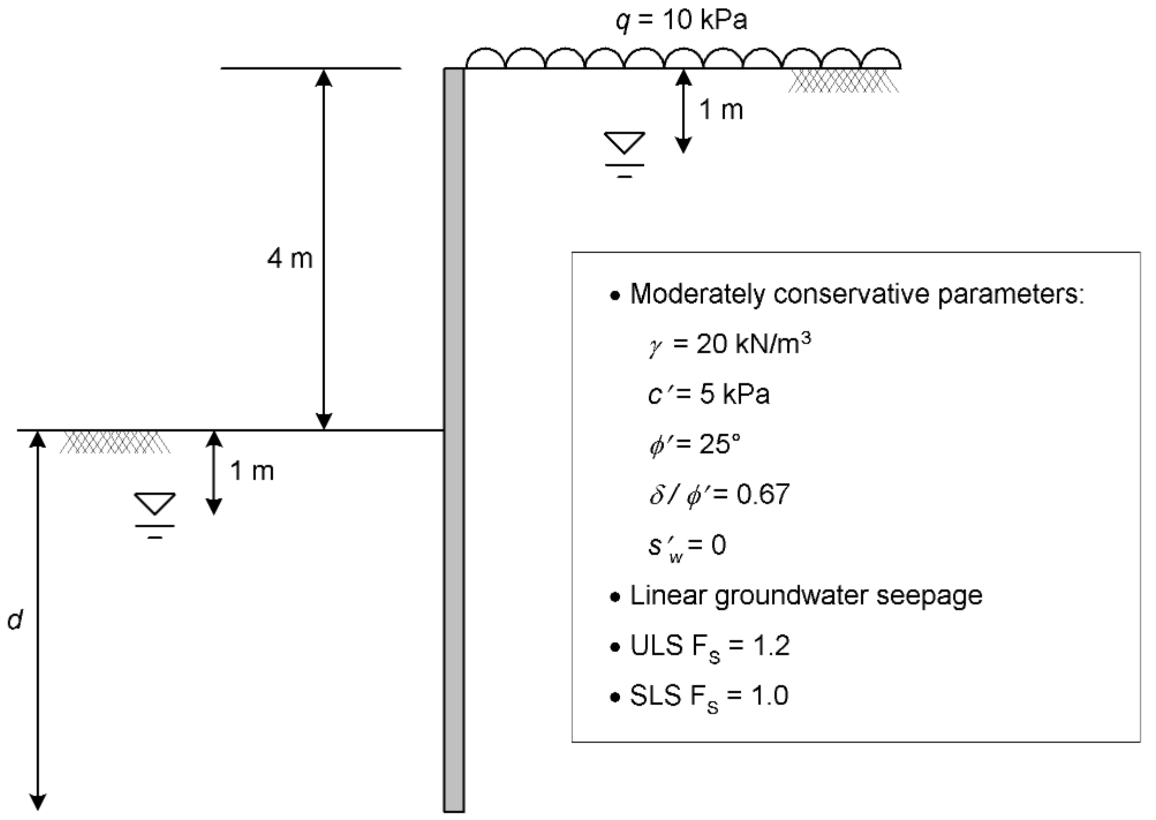


FIGURE J1 *Example 1 - cantilever wall: effective stress analysis*

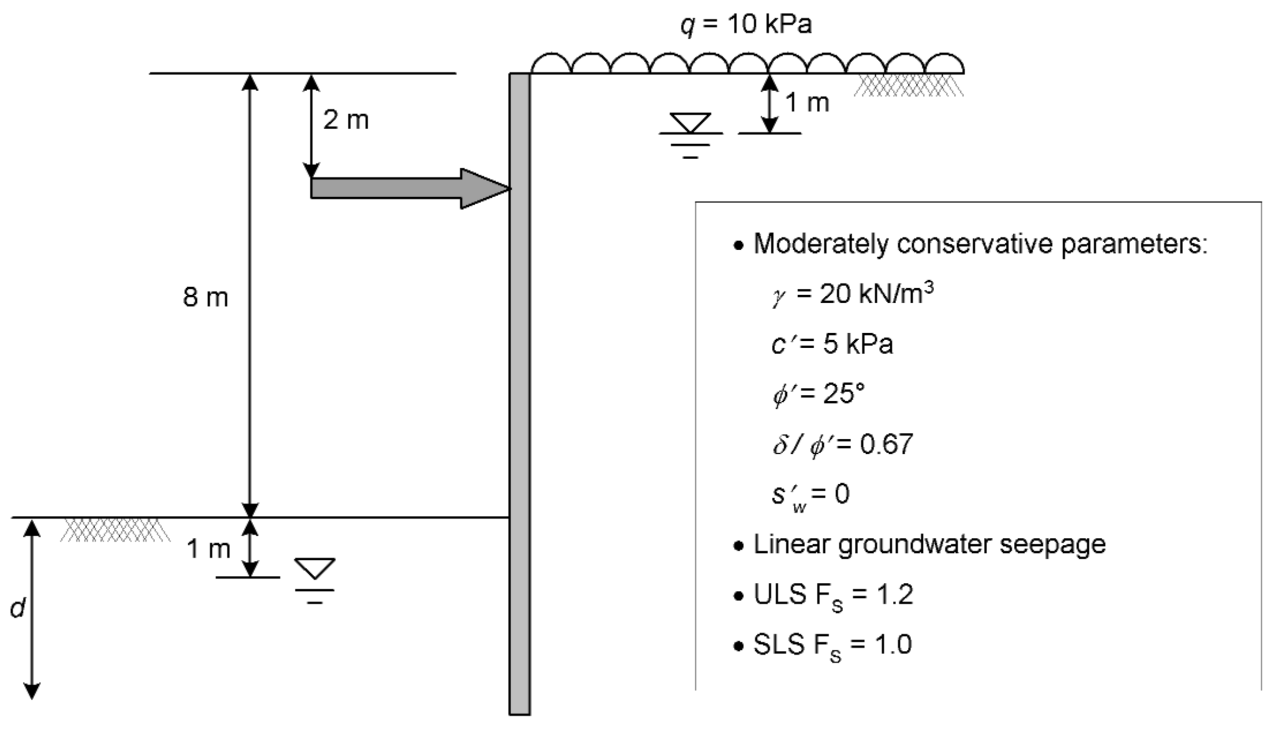


FIGURE J2 *Example 2 - propped wall: effective stress analysis*

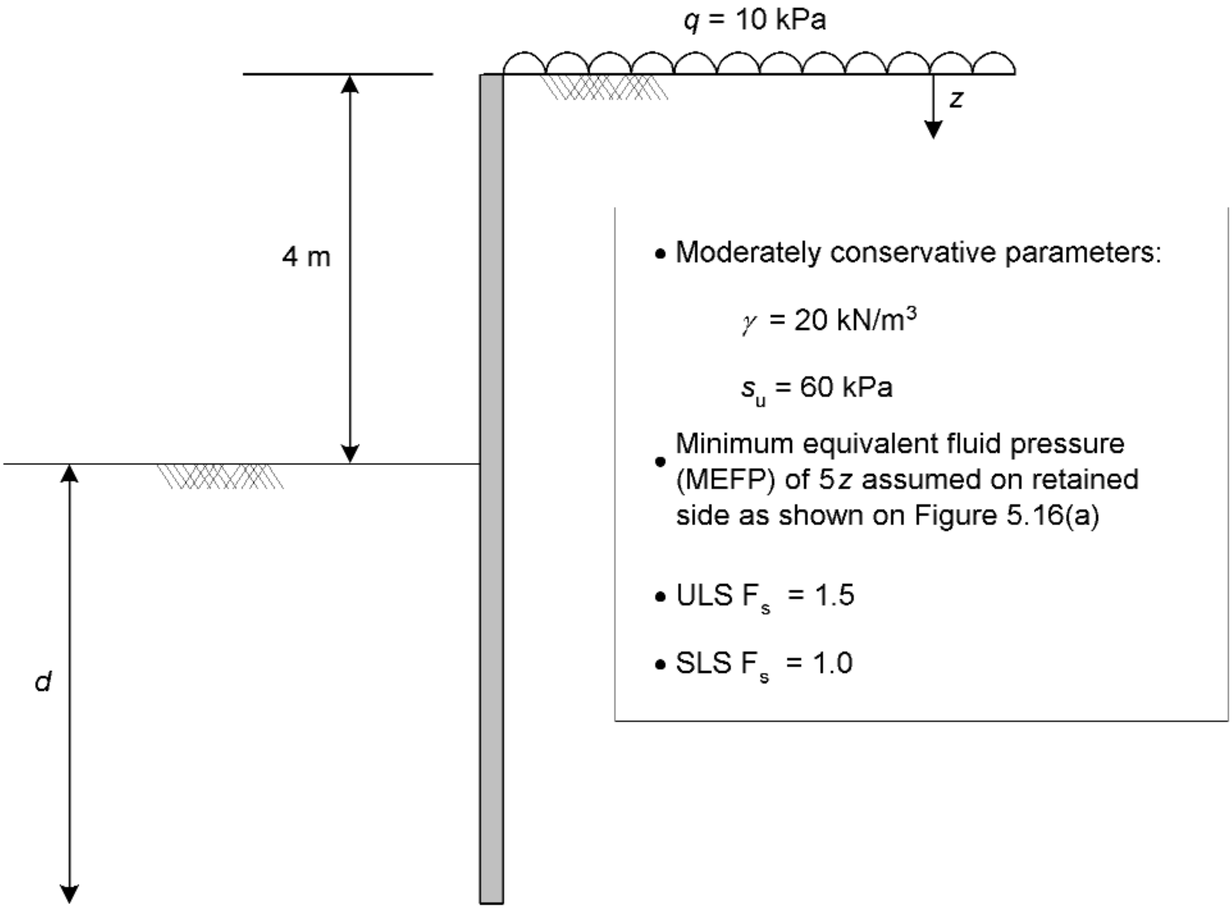


FIGURE J3 *Example 3 - cantilever wall: total stress analysis*

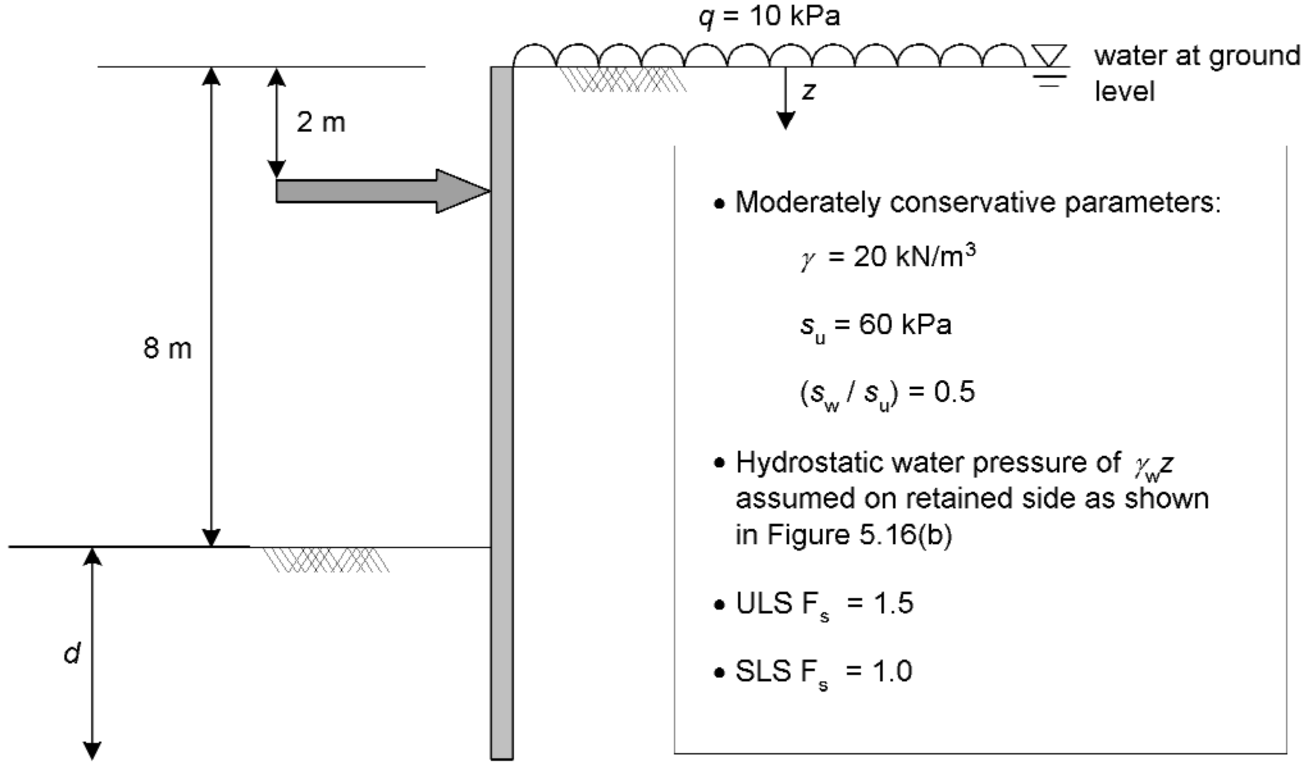
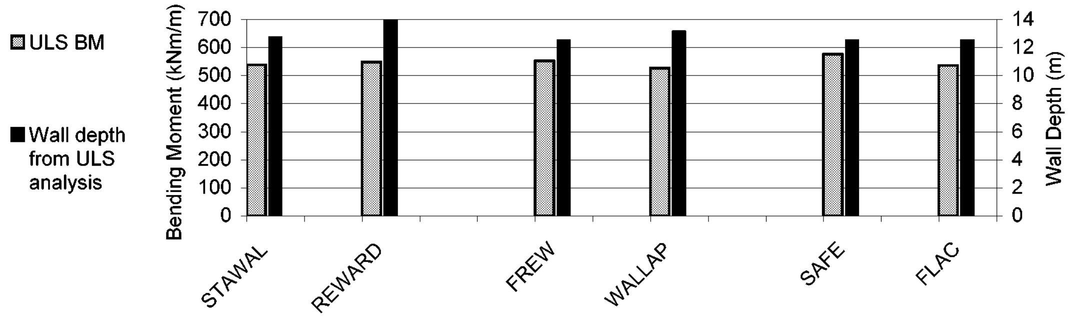
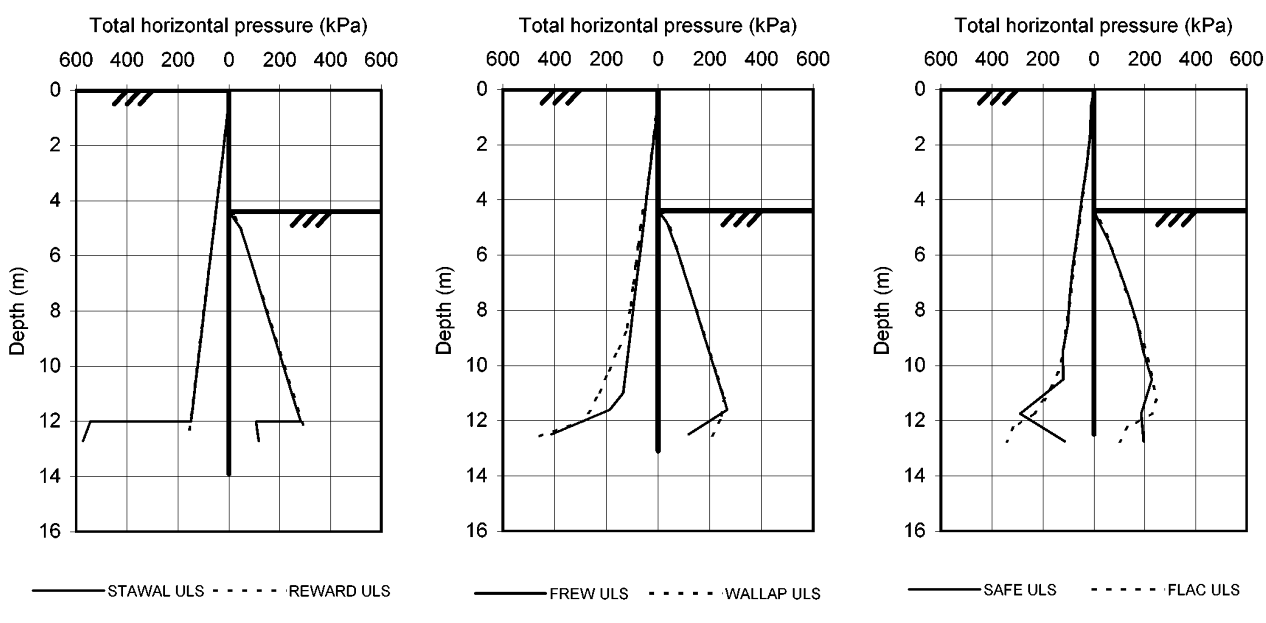


FIGURE J4 *Example 4 - propped wall: total stress analysis*

Cantilever wall Effective stress (see Figure J1)	Limit equilibrium STAWAL ULS	 REWARD ULS	Pseudo-finite element FREW ULS	 WALLAP ULS	Finite element/difference SAFE ULS	 FLAC ULS
Wall depth (m)	12.7	13.9	12.5	13.1	12.5	12.5
Max wall BM (kNm/m)	538	548	552	526	576	536
Max wall SF (kN/m)	312	369	250	173	232	216
Max wall deflection (mm)	n/a	n/a	n/a	n/a	n/a	n/a





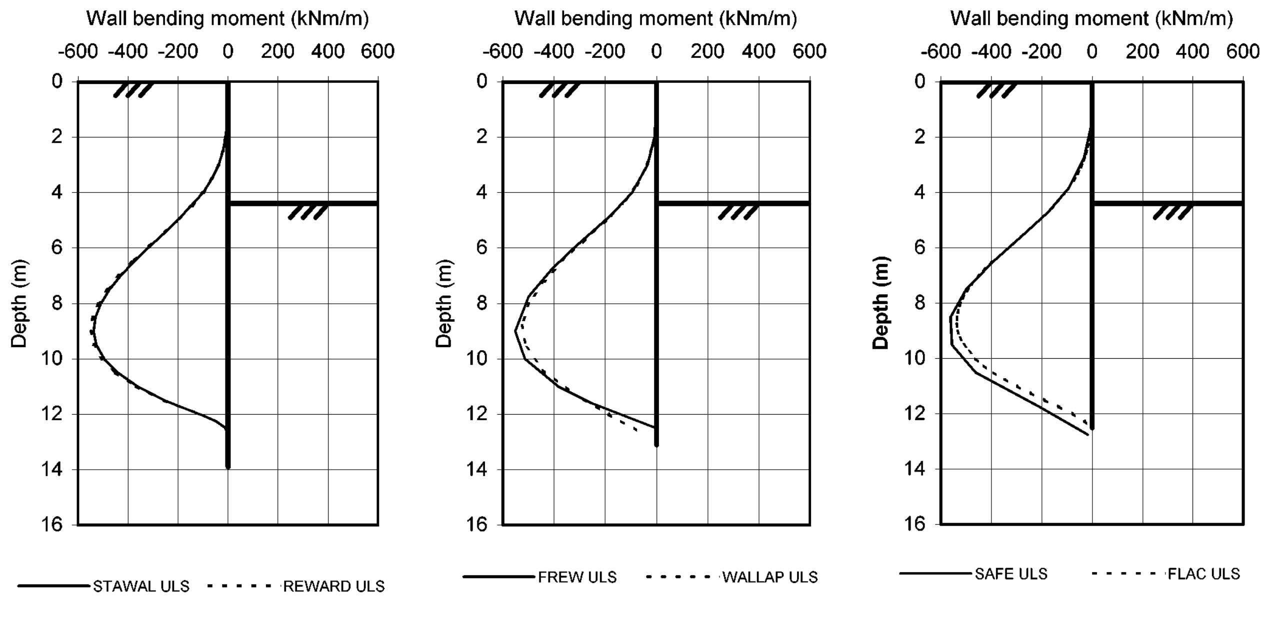
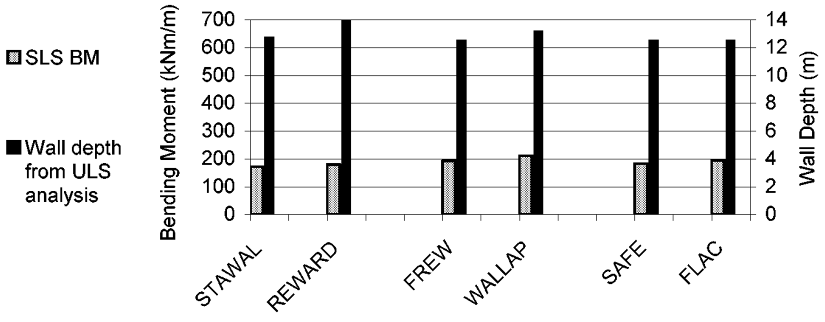
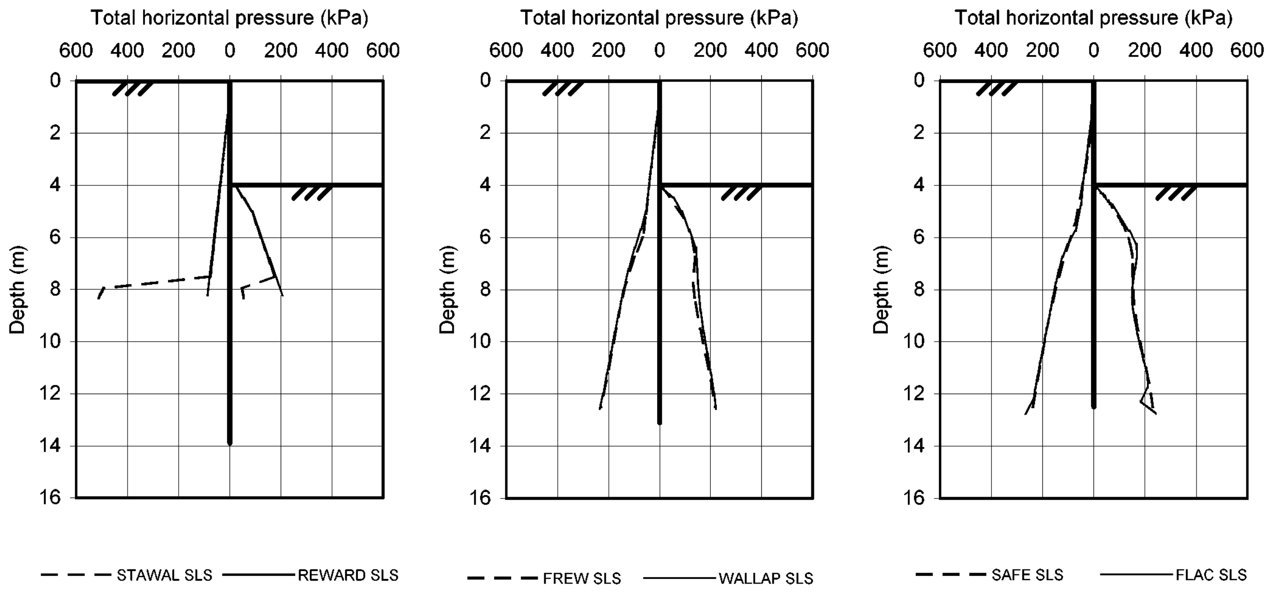


Figure J5 *Results for example 1 - cantilever wall: effective stress analysis - ULS calculations*

Cantilever wall	Limit equilibrium				Pseudo-finite element				Finite element/difference			
Effective stress	STAWAL		REWARD		FREW		WALLAP		SAFE		FLAC	
(see Figure J1)	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
Wall depth (m)	12.7	8.3	13.9	9.1	12.5	12.5	13.1	13.1	12.5	12.5	12.5	12.5
Max wall BM (kNm/m)		172		180		192		211		183		194
Max wall SF (kN/m)		165		196		67		75		74		76
Max wall deflection (mm)		n/a		n/a		22		49		15		15





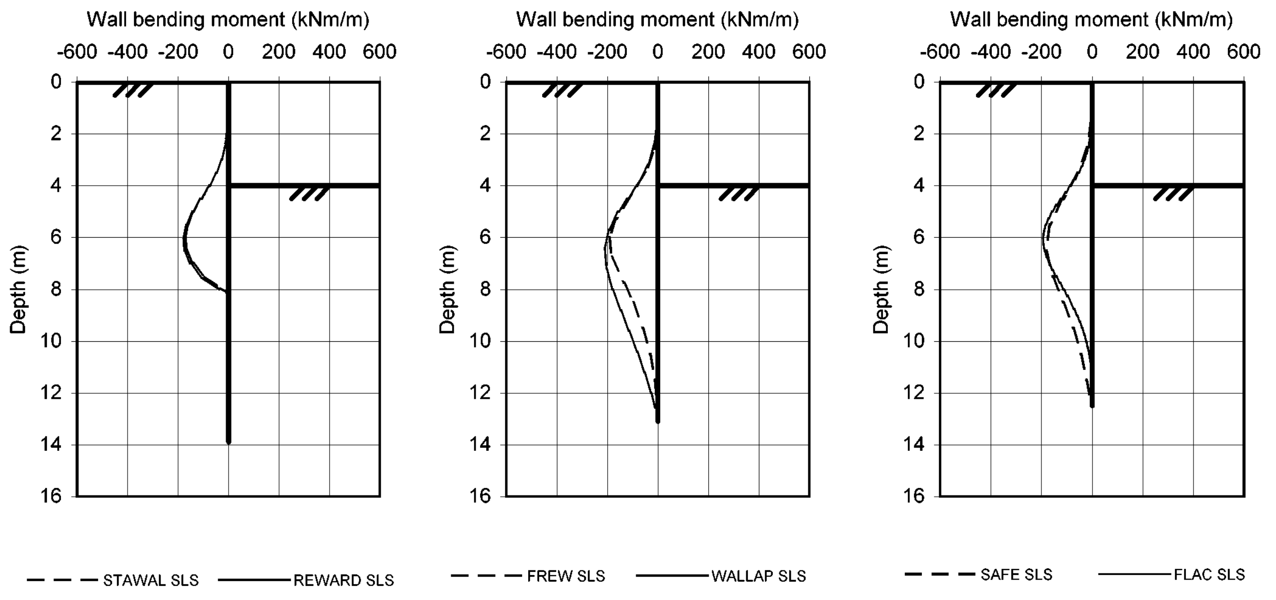


Figure J6 *Results for example 1 - cantilever wall: effective stress analysis - SLS calculations*

Propped wall Effective stress (see figure J2)	Limit equilibrium STAWAL ULS	 REWARD ULS	Pseudo-finite element FREW ULS	 WALLAP ULS	Finite element/difference SAFE ULS	 FLAC ULS
Wall depth (m)	16.7	16.6	14.4	16.2	15.0	15.5
Max wall BM (kNm/m)	1146	1157	764	910	944	836
Max wall SF (kN/m)	322	324	368	367	362	343
Max wall deflection (mm)	n/a	n/a	n/a	n/a	n/a	n/a
Maximum prop load (kN/m)	338	340	444	444	518	469

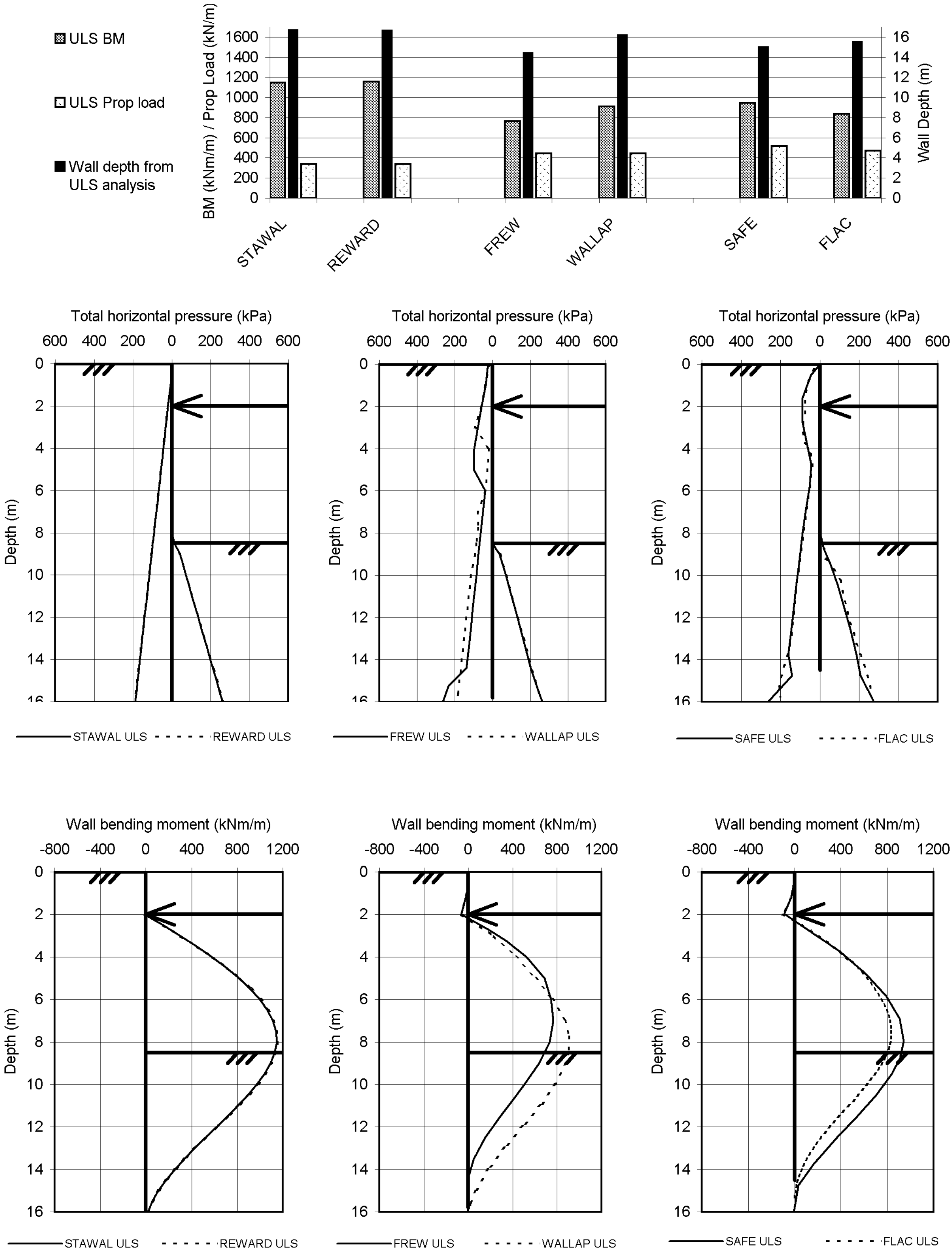


FIGURE J7 *Results for example 2 - singly propped wall: effective stress analysis - ULS calculations*

Propped wall Effective stress (see figure J2)	Limit equilibrium STAWAL ULS	 SLS	 REWARD ULS	 SLS	Pseudo-finite element FREW ULS	 SLS	 WALLAP ULS	 SLS	Finite element/difference SAFE ULS	 SLS	 FLAC ULS	 SLS
Wall depth (m)	16.7	12.4	16.6	12.3	14.4	14.4	16.2	16.2	15.0	15.0	15.5	15.5
Max wall BM (kNm/m)		461		459		345		340		473		470
Max wall SF (kN/m)		170		170		174		160		217		232
Max wall deflection (mm)		n/a		n/a		18		25		19		24
Maximum prop load (kN/m)		182		181		248		235		356		387

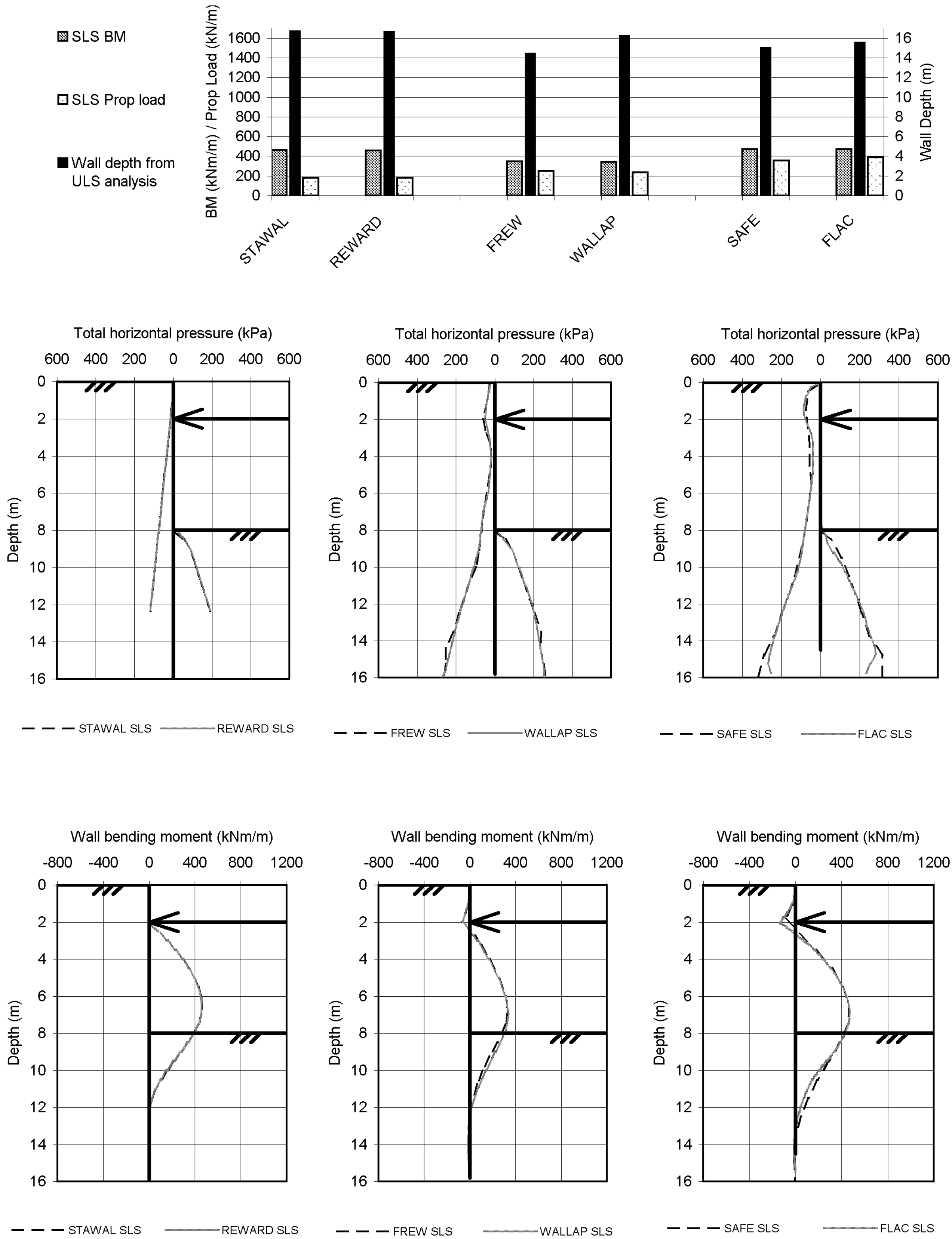


FIGURE J8 *Results for example 2 - singly propped wall: effective stress analysis - SLS calculations*

Cantilever wall Total stress (See Figure J3)	Limit equilibrium STAWAL ULS	 REWARD ULS	Pseudo-finite element FREW ULS	 WALLAP ULS	Finite element/difference SAFE ULS	 FLAC ULS
Wall depth (m)	7.0	7.3	6.8	7.3	7.0	7.0
Max wall BM (kNm/m)	98	100	99	99	105	101
Max wall SF (kN/m)	120	144	52	67	53	91
Max wall deflection (mm)	n/a	n/a	n/a	n/a	n/a	n/a

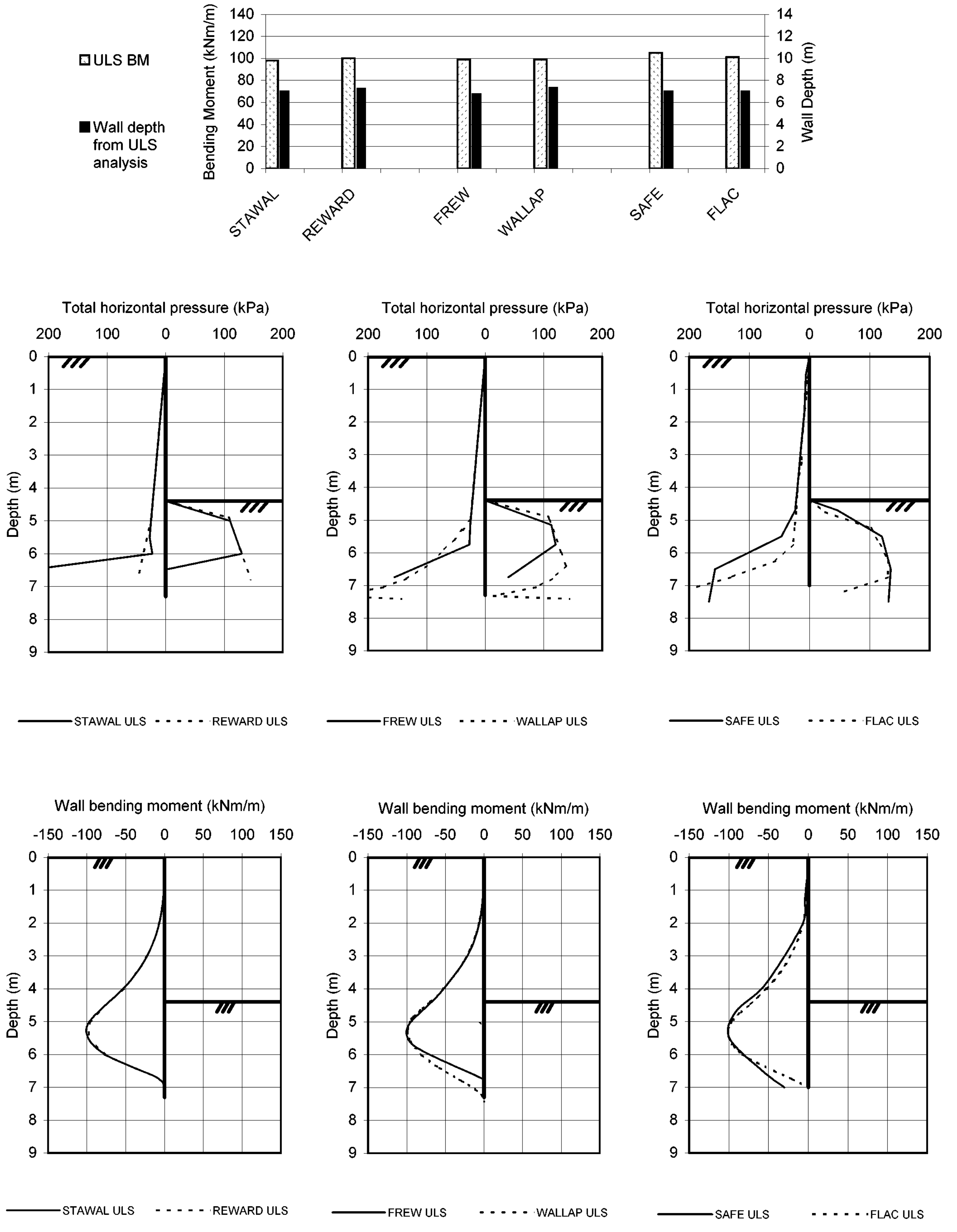


FIGURE J9 *Results for example 3 - cantilever wall: total stress analysis - ULS calculations*

Cantilever wall Total stress (See Figure J3)	Limit equilibrium STAWAL ULS	 SLS	 REWARD ULS	 SLS	Pseudo-finite element FREW ULS	 SLS	 WALLAP ULS	 SLS	Finite element/difference SAFE ULS	 SLS	 FLAC ULS	 SLS
Wall depth (m)	7.0	5.8	7.3	5.9	6.8	6.8	7.3	7.3	7.0	7.0	7.0	7.0
Max wall BM (kNm/m)		70		70		69		79		81		71
Max wall SF (kN/m)		115		143		33		43		58		47
Max wall deflection (mm)		n/a		n/a		20		28		22		18

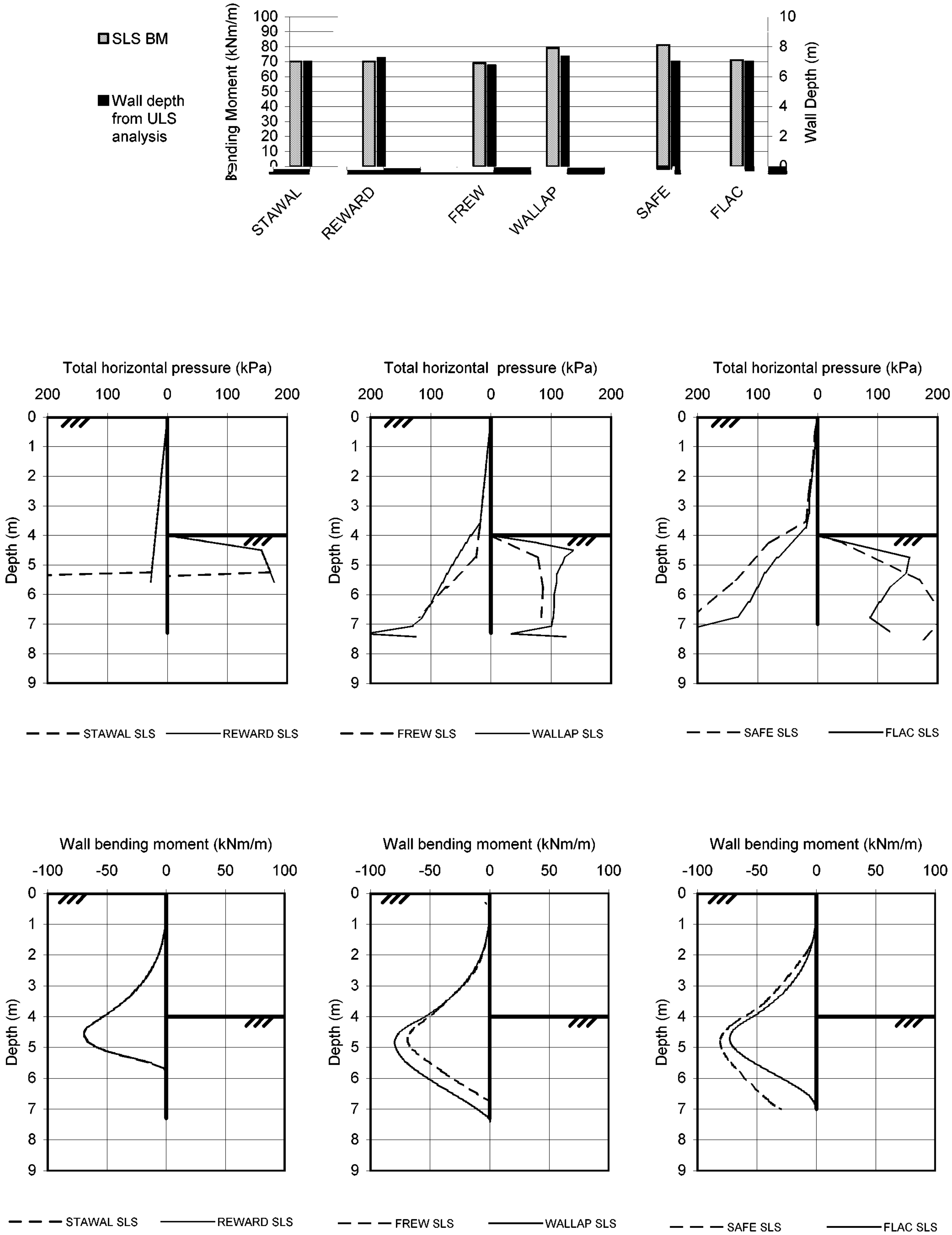


FIGURE J10 *Results for example 3 - cantilever wall: total stress analysis - SLS calculations*

Propped wall	Limit equilibrium		Pseudo-finite element		Finite element/difference	
Total stress	STAWAL	REWARD	FREW	WALLAP	SAFE	FLAC
(See Figure J4)	ULS	ULS	ULS	ULS	ULS	ULS
Wall depth (m)	16.7	17.0	10.6	10.9		
Max wall BM (kNm/m)	619	656	375	361		
Max wall SF (kN/m)	218	223	271	236		
Max wall deflection (mm)	n/a	n/a	n/a	n/a		
Maximum prop load (kN/m)	238	243	507	425		

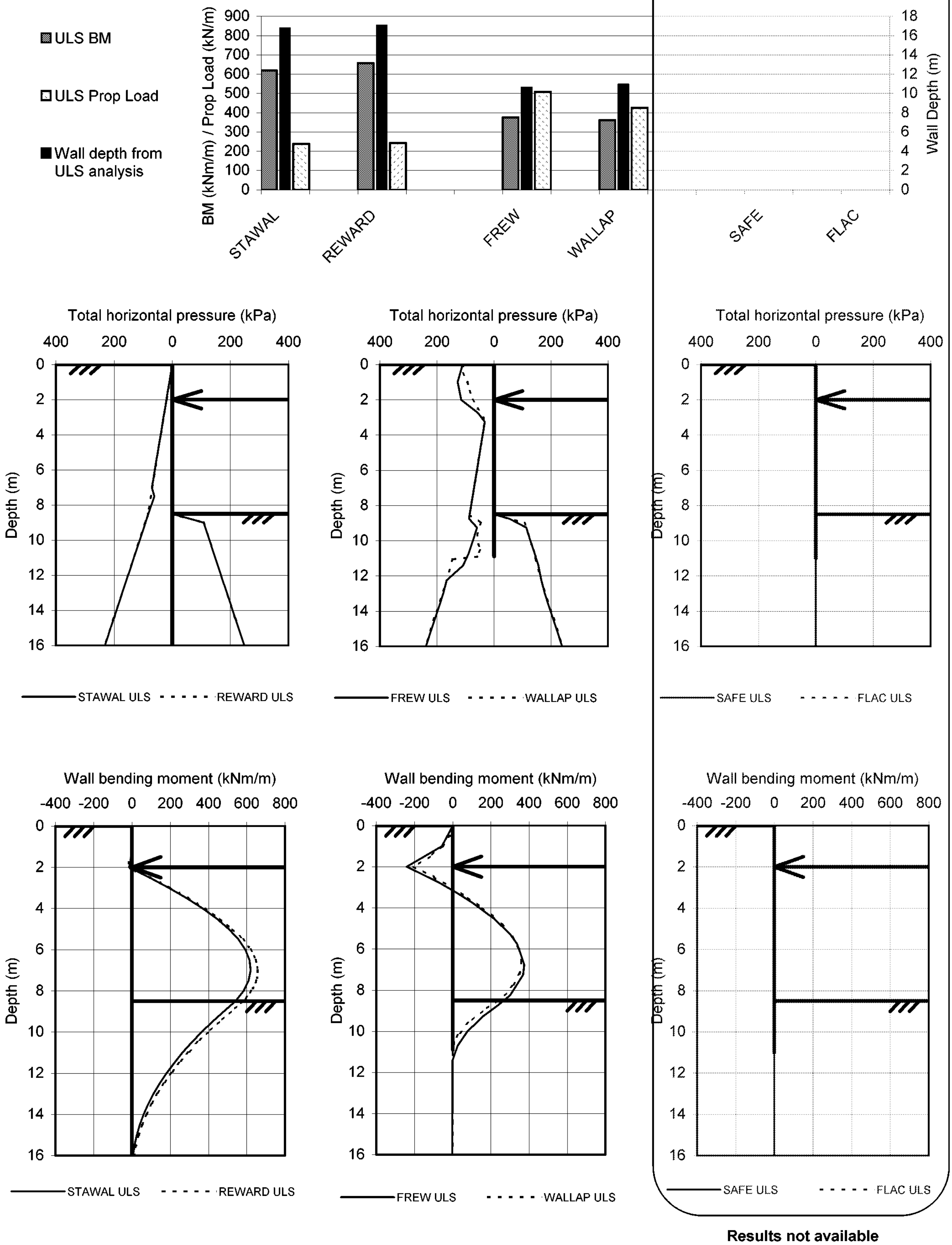
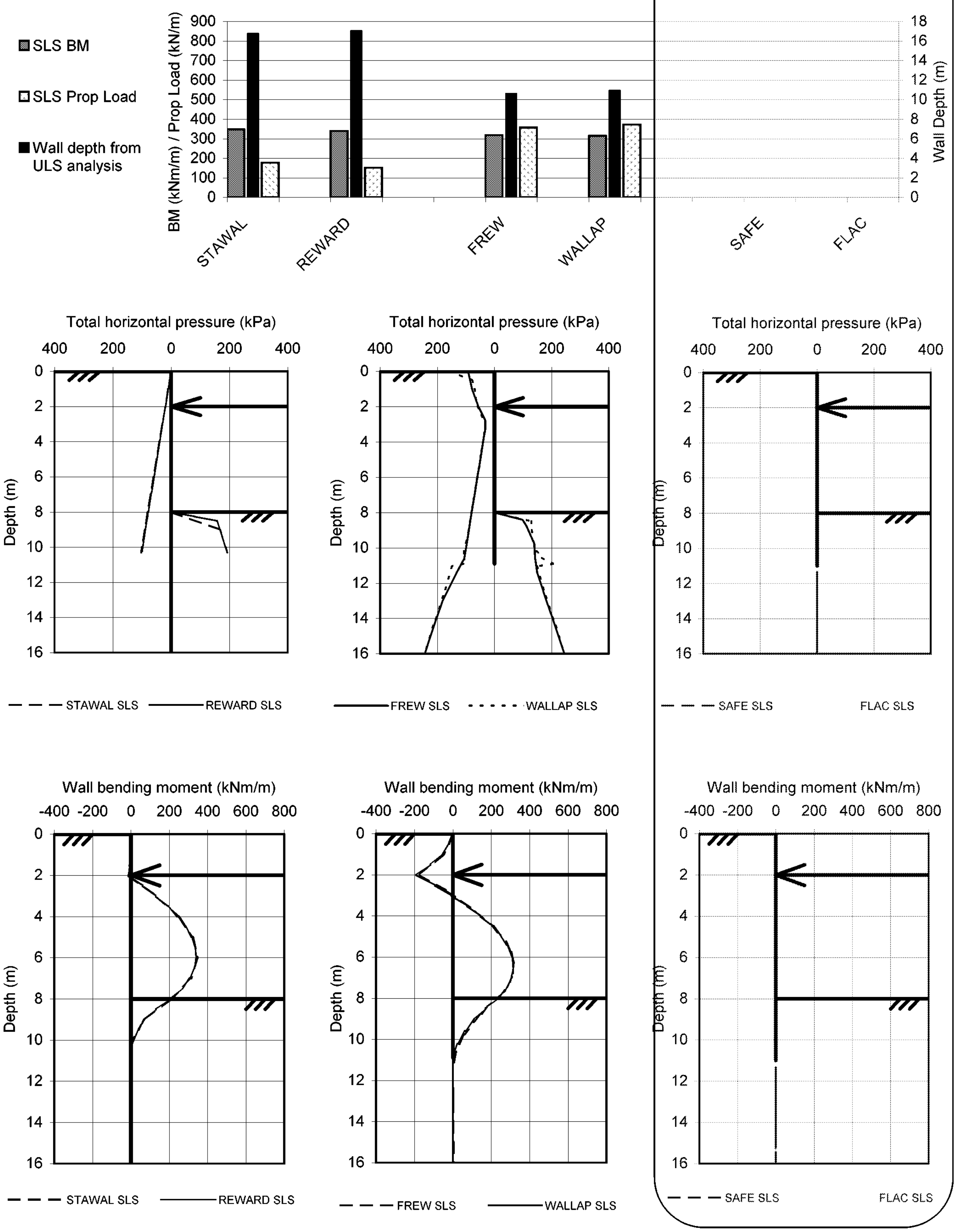


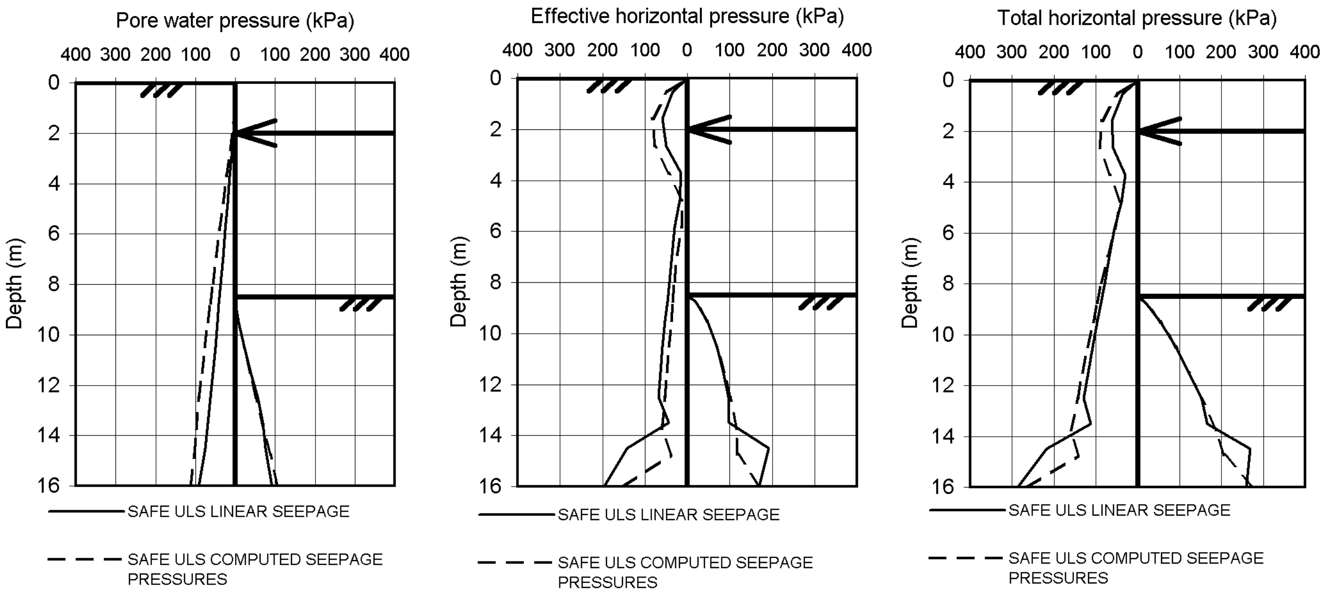
Figure J11 *Results for example 4 - singly propped wall: total stress analysis - ULS calculations*

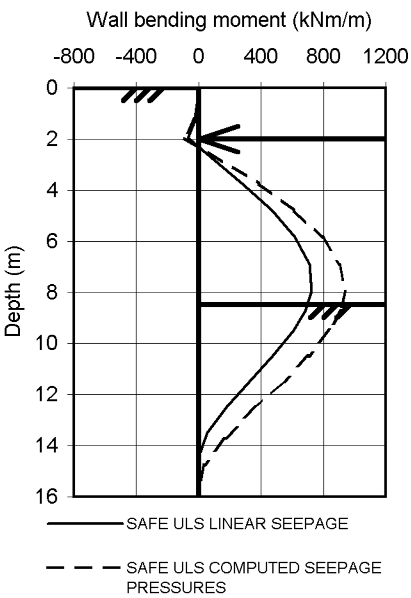
Propped wall Total stress (See Figure J4)	Limit equilibrium STAWAL ULS	SLS	REWARD ULS	SLS	Pseudo-finite element FREW ULS	SLS	WALLAP ULS	SLS	Finite element/difference SAFE SLS	FLAC SLS
Wall depth (m)	16.7	10.3	17.0	9.4	10.6	10.6	10.9	10.9		
Max wall BM (kNm/m)		348		340		318		315		
Max wall SF (kN/m)		157		132		207		210		
Max wall deflection (mm)		n/a		n/a		23		25		
Maximum prop load (kN/m)		177		152		357		373		



Results not available

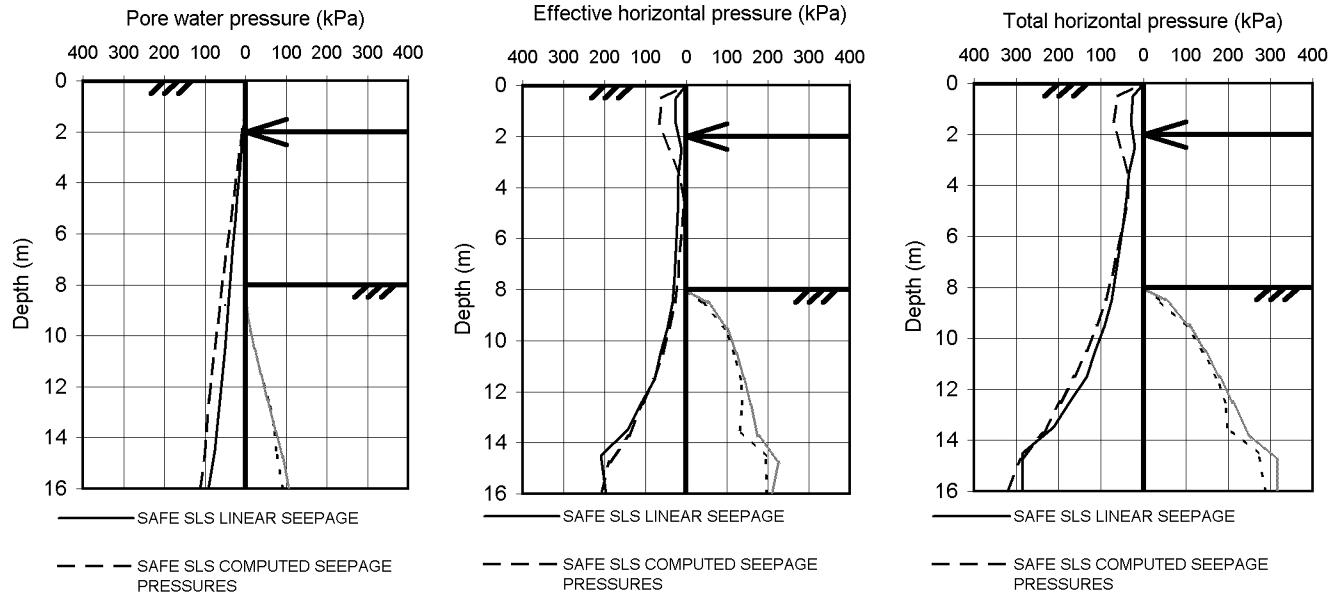
Figure J12 *Results for example 4 - singly propped wall: total stress analysis - SLS calculations*

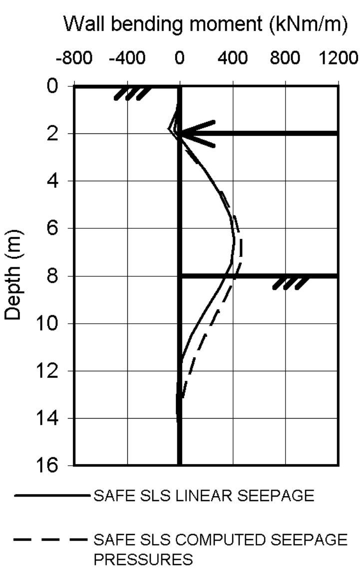




Propped wall		**SAFE**
Effective stress	**Linear seepage**	**Computed seepage pressure**
(see Figure J2)	**ULS**	**ULS**
Wall depth (m)	14.0	15.0
Max wall BM (kNm/m)	734	944
Max wall SF (kN/m)	256	361
Max wall deflection (mm)	N/A	N/A
Maximum prop load (kN/m)	380	518

FIGURE J13 *Results for example 2 - singly propped wall: effective stress analysis - ULS calculations. Effects of ground water pressures*





Propped wall			**SAFE**	
Effective stress	**Linear seepage**		**Computed seepage pressure**	
(see Figure J2)	**ULS**	**SLS**	**ULS**	**SLS**
Wall depth (m)	14.0	14.0	15.0	15.0
Max wall BM (kNm/m)		412		460
Max wall SF (kN/m)		169		216
Max wall deflection (mm)		14		20
Maximum prop load (kN/m)		220		356

FIGURE J14 *Results for example 2 - singly propped wall: effective stress analysis - ULS calculations. Effects of ground water pressures*

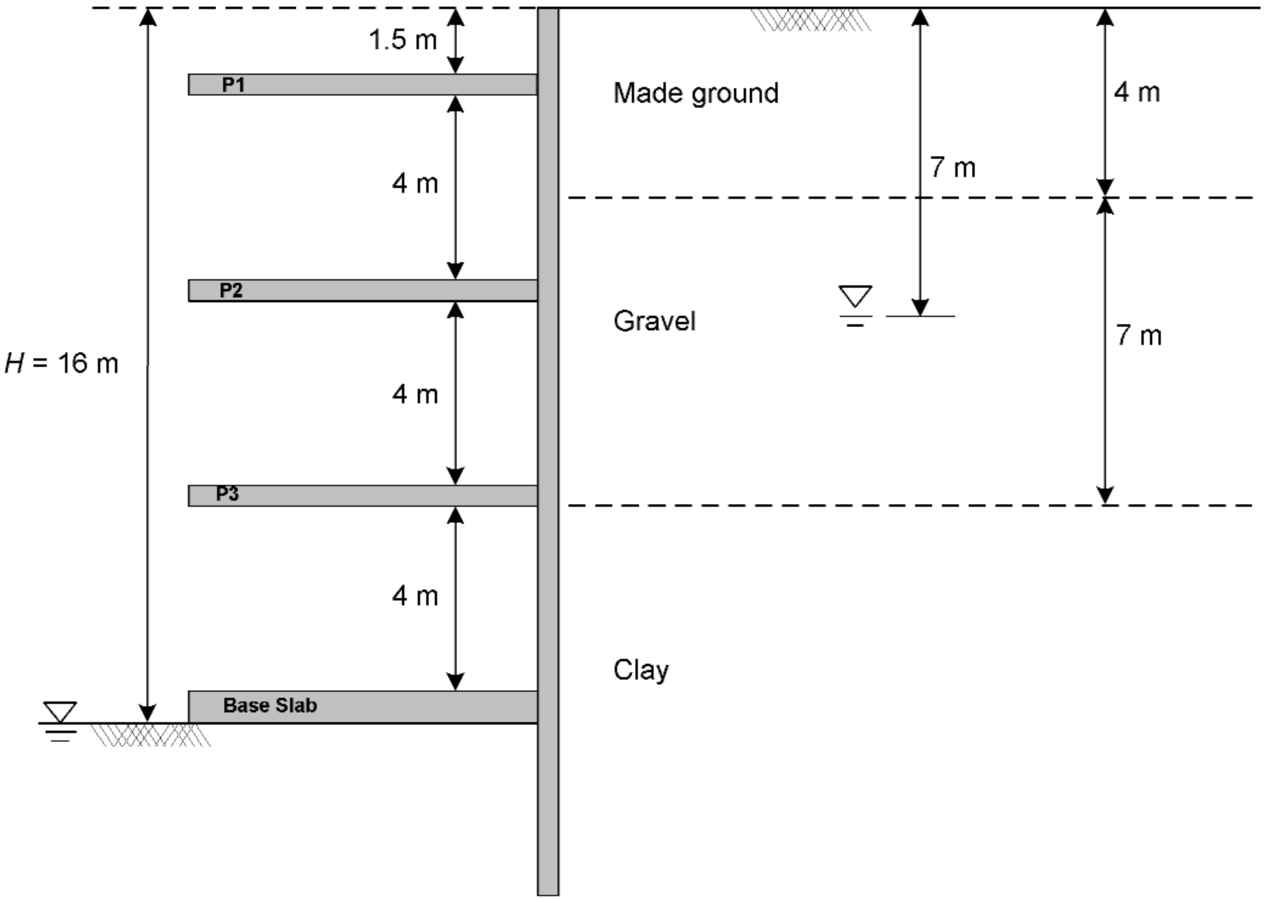
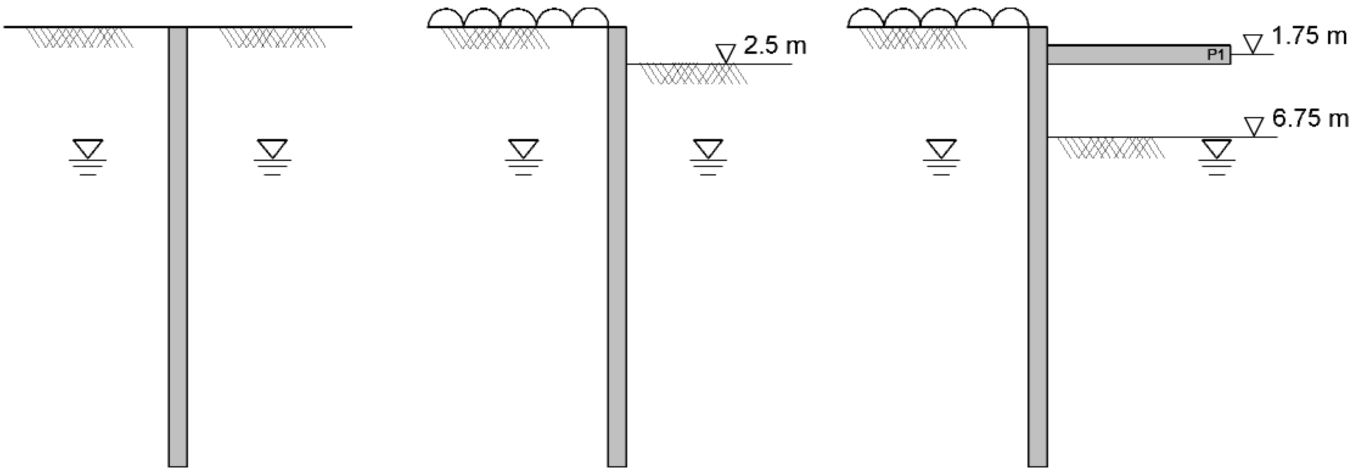


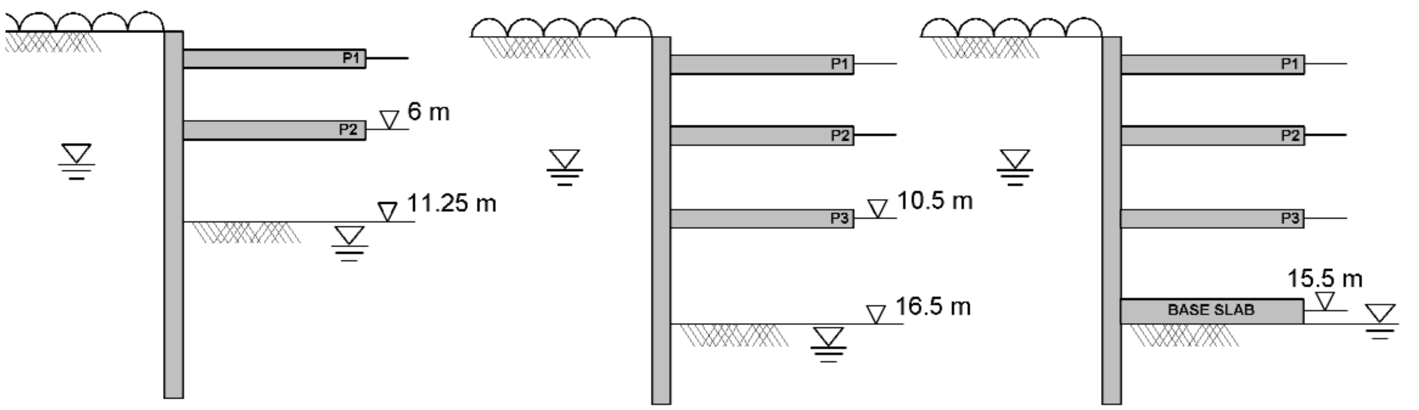
FIGURE K1 *Worked example geometry*



Stage 1: Install wall

Stage 2: Apply a 10 kPa surcharge outside the wall and excavate to 2.5 m depth

Stage 3: Install (500 mm thick)slab at 1.75 m depth and excavate to 6.75 m depth



Stage 4: Install (500 mm thick) slab at 6 m depth, dewater the gravel and excavate to 11.25 m depth

Stage 5: Install (500 mm thick) slab at 10.5 m depth and excavate to 16.5m depth

Stage 6: Install (1 m thick) base slab at 15.5 m depth with permanent drainage beneath. Clay drains in long term, with water at base slab soffit level.

FIGURE K2 *Top down construction sequence*

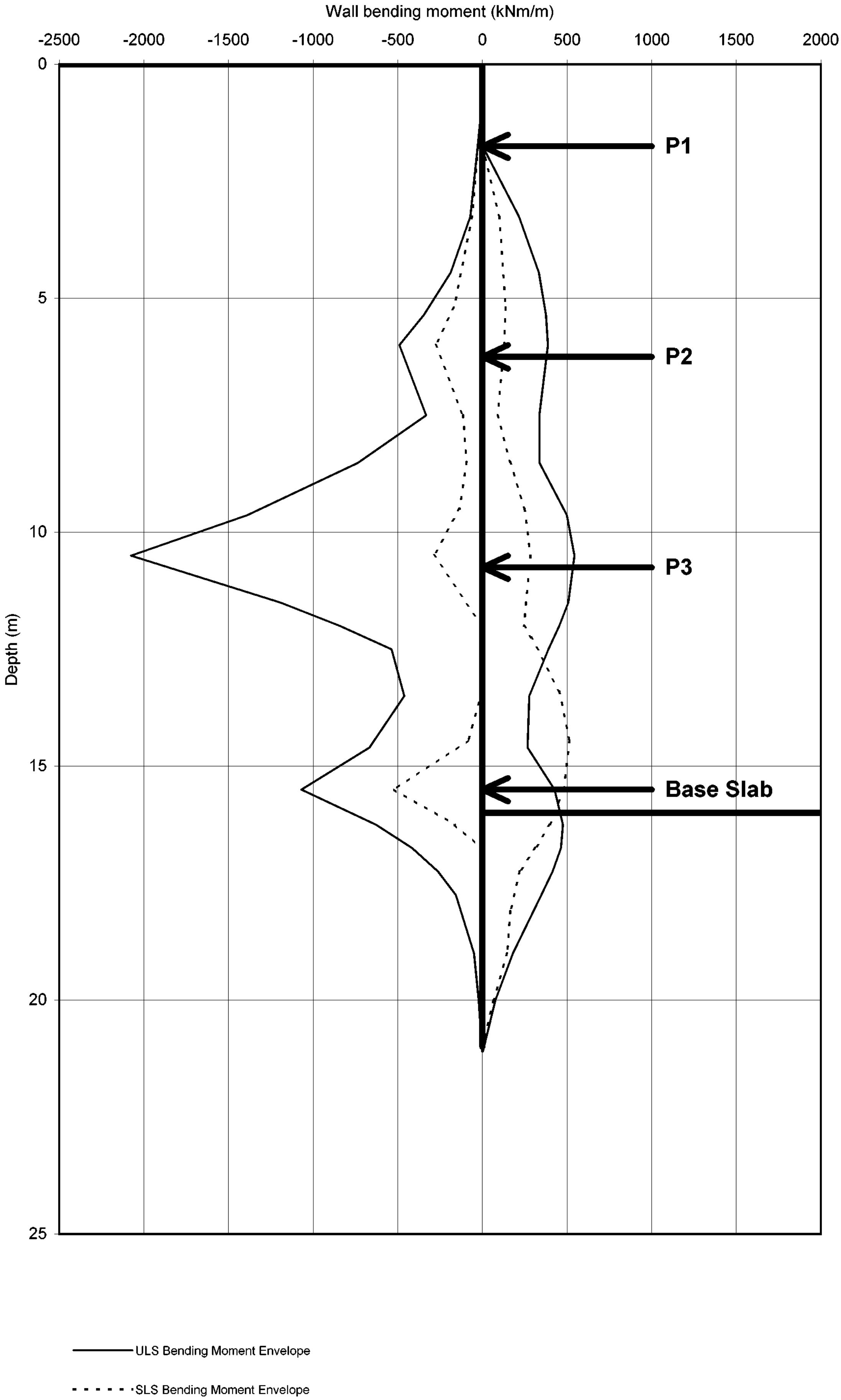


FIGURE K4 *Calculated ULS and SLS wall bending moment envelopes*

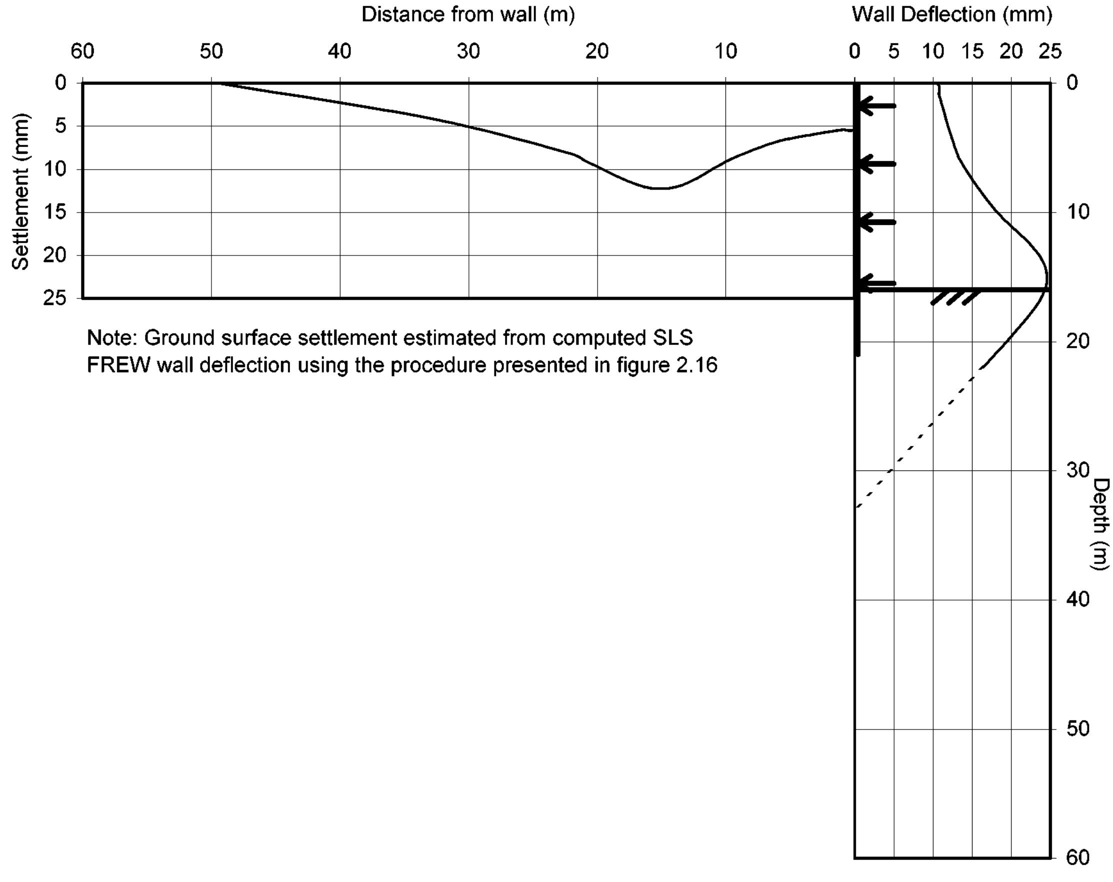
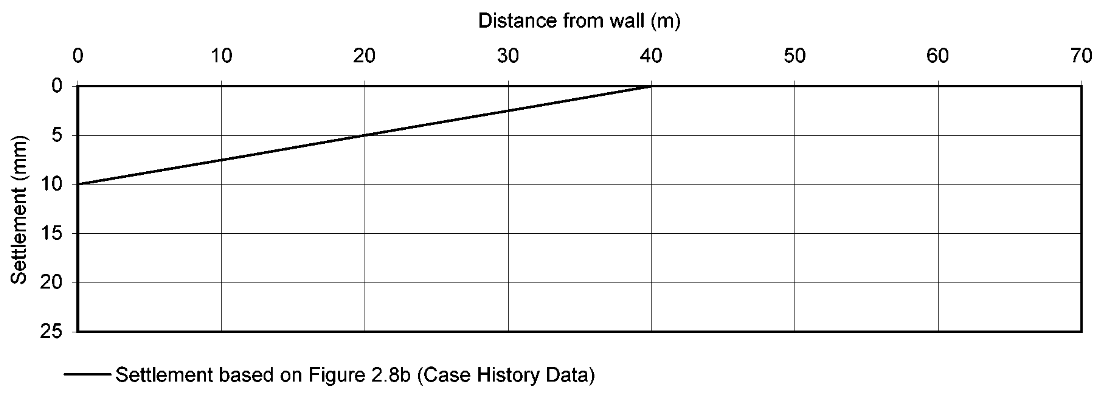
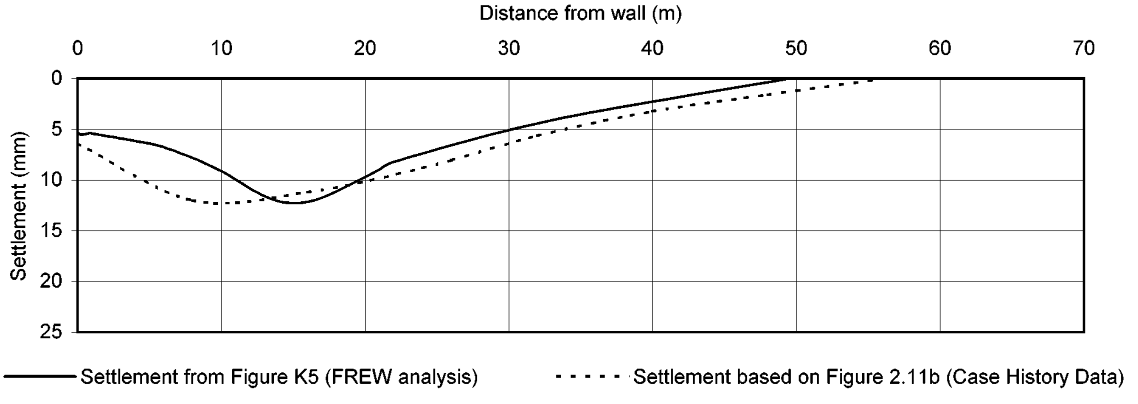


FIGURE K5 *Estimated ground surface settlement from computed SLS FREW wall deflection*



(a) Estimated ground surface settlement due to wall installation



(b) Estimated ground surface settlement due to wall deflection

PLATE C.1 *Sheet pile wall installation, Portsmouth Harbour (courtesy of Corus)*

PLATE C.2 *Contiguous bored piled wall (courtesy of Bachy Soletanche Limited)*

PLATEC.3 *Hard/soft secant pile wall as temporary works at North Greenwich Station, London (courtesy of Bachy Soletanche Limited)*

PLATEC.4 *Hard/soft secant pile wall and an example of top down construction (courtesy of Bachy Soletanche Limited)*

PLATE C.5 *Diaphram wall construction at Canary Wharf Station, London (courtesy of Bachy Soletanche Limited)*