

# Steel Intensive Basements

E YANDZIO BSc, M Eng, CEng MIMarE

A R BIDDLE BSc, CEng, MICE

Published by:

The Steel Construction Institute  
Silwood Park  
Ascot  
Berkshire SL5 7QN

Tel: 01344 623345

Fax: 01344 622944

© 2001 The Steel Construction Institute

Apart from any fair dealing for the purposes of research or private study or criticism or review, as permitted under the Copyright Designs and Patents Act, 1988, this publication may not be reproduced, stored or transmitted, in any form or by any means, without the prior permission in writing of the publishers, or in the case of reprographic reproduction only in accordance with the terms of the licences issued by the UK Copyright Licensing Agency, or in accordance with the terms of licences issued by the appropriate Reproduction Rights Organisation outside the UK.

Enquiries concerning reproduction outside the terms stated here should be sent to the publishers, The Steel Construction Institute, at the address given on the title page.

Although care has been taken to ensure, to the best of our knowledge, that all data and information contained herein are accurate to the extent that they relate to either matters of fact or accepted practice or matters of opinion at the time of publication, The Steel Construction Institute, the authors and the reviewers assume no responsibility for any errors in or misinterpretations of such data and/or information or any loss or damage arising from or related to their use.

*Publications supplied to the Members of the Institute at a discount are not for resale by them.*

Publication Number: SCI P275

ISBN 1 85942 125 3

British Library Cataloguing-in-Publication Data.

A catalogue record for this book is available from the British Library.

# FOREWORD

This publication presents justification for selecting permanent steel sheet piling in basement construction. In many cases this has proven to be a more economic solution, than alternatives such as concrete diaphragm walls and contiguous secant pile walls, by saving construction time, ensuring continuous support, and enabling maximum use of site space for the building.

Traditionally, basement construction used a bottom-up method in which steel sheet piles have been used as a temporary external wall that was removed after an inner concrete construction was complete. Consequently, sheet piles have generally been considered to be “temporary” walling. However, on modern urban sites, basement construction methods have to ensure foundation support to existing adjacent buildings and that subsidence is not caused during construction. The permanent use of a steel sheet pile perimeter avoids any excavation or loosening of soils that can cause loss of support and subsidence damage.

More recent top-down construction methods favour use of embedded walls. The permanent use of sheet pile walls has been pioneered in Europe, particularly in Holland and Denmark, where they have successfully controlled severe problems with poor ground conditions and high water tables. This method of construction has now been adopted in the UK where it is increasingly being used for basements as the civil engineering industry appreciates its many advantages.

Permanent sheet piling is a narrow form of construction and can be installed close up to the boundary of the site, thereby maximising use of site space.

In the absence of any published design and construction guidance on sheet piling for basements, this publication was conceived and produced to help structural and geotechnical engineers take advantage of the use of permanent steel sheet piling.

The SCI has consulted its members during the preparation of the publication via the Steel Piling Group. Particular thanks is extended to: Ove Arup and Partners who have been the pioneer with this type of construction in the UK; Kvaerner Cementation for their help on sites and details of CEMLOC™ system; and Carrilion for allowing access to their site during basement construction.

SCI acknowledges the help and funding from Corus in the preparation of this Guide.



# CONTENTS

	<b>Page No.</b>
SUMMARY	viii
1 INTRODUCTION	1
1.1 The scope for use of steel in basements	1
1.2 Existing guidance on the design of basements	1
1.3 Scope of this publication	2
2 CASE STUDIES	3
2.1 Millennium Car Park, Bristol	3
2.2 Underground car park, The Hague, Holland.	5
2.3 Travelodge Hotel, Cardiff	6
2.4 Ford recycling facility, Bridgend, Glamorgan	8
2.5 Oslo City underground car park	10
2.6 Basement car park, Staines	11
2.7 Underground garage, Rapperswil, Switzerland	12
2.8 Underground garage, Bad Oeynhausen, Germany	14
2.9 Other steel intensive basements in Europe	15
3 BENEFITS AND COST COMPARISONS	17
3.1 Benefits of sheet piles in basements	17
3.2 Cost comparison study	18
3.3 Cost comparison for walls in Bristol Millennium underground car park	22
3.4 Aesthetics	22
3.5 Whole life cost	23
4 CONSTRUCTION METHODS FOR BASEMENTS	25
4.1 Factors affecting basement construction	25
4.2 Principal methods of constructing basements	25
4.3 Top-down construction	27
4.4 Bottom-up construction	29
4.5 Other construction methods	30
4.6 Reinforced concrete pile capping beams	31
5 DESIGN BASIS	33
5.1 Types of basement design	33
5.2 Requirements of the Building Regulations	33
5.3 Limit state design philosophy	34
5.4 Design standards	34
5.5 Geotechnical design	38

5.6	Structural design	39
5.7	Loading and design parameters	41
5.8	Behaviour and performance of retaining walls	41
5.9	Predicting ground movement	42
5.10	Data for design	44
5.11	Observational method	47
5.12	Design for driveability	49
5.13	Design for durability	50
6	DESIGN PROCEDURE FOR BASEMENTS	53
6.1	Design procedure	53
6.2	Activities in the design procedure	53
7	DESIGN OF CANTILEVER AND SINGLE-PROP WALLS	59
7.1	Simplistic representation of soil pressures	59
7.2	Water pressures	62
7.3	Wall stability and depth of embedment	62
7.4	Vertical load resistance	64
7.5	Design for lateral loading	68
8	DESIGN OF MULTI-PROP WALLS	71
8.1	Simple methods	71
8.2	Deformation methods	74
8.3	Vertical Load resistance	77
9	BASE STABILITY	79
9.1	Base heave	79
9.2	Design of base slab and foundations	82
9.3	Hydraulic instability in granular soils	83
10	DESIGN OF TEMPORARY SUPPORTS	87
10.1	Single-prop walls	87
10.2	Multi-prop walls	87
10.3	Temperature effects on props	90
10.4	Temporary support of retaining walls using soil berms	91
11	WATER-PROOFING	93
11.1	Introduction	93
11.2	The internal environment	93
11.3	Forms of water resisting construction	94
11.4	Prevention of water seepage through sheet pile walls	94
11.5	Waterproofing and design of reinforced concrete floors	98
11.6	Steel to reinforced concrete slab connection	100

11.7	Water seepage through steel sheet pile interlocks	104
12	<b>FIRE RESISTANCE AND FIRE PROTECTION OF BASEMENTS</b>	105
12.1	Fire concepts	105
12.2	The need for fire resistance	105
12.3	The Building Regulations	106
12.4	Approved Document B	106
12.5	Elements of structure	109
12.6	London Building Act 1939: London District Surveyors' Guide to Section 20 Buildings	109
12.7	Fire resistance requirements for sheet pile walls in basements	110
12.8	Compartmentation requirements in basements	111
12.9	Fire engineering solutions: Structural fire design - Design codes	111
12.10	Structural fire protection	111
13	<b>PILE DRIVING AND INSTALLATION</b>	117
13.1	Pile installation	117
13.2	Installation methods	118
13.3	Environmental factors: noise and vibration prediction	119
13.4	Selection of installation equipment	122
13.5	Installation ancillaries	124
14	<b>REFERENCES</b>	127
14.1	Numbered references	127
14.2	Regulations and Standards	134
APPENDIX A	Sheet pile types available from UK manufacturers	139
A.1	Sheet piles	139
A.2	Box piles	142
A.3	High modulus piles	143
APPENDIX B	CEMLOC™ System	145

# SUMMARY

This publication gives guidance on the design and installation aspects of permanent steel sheet piled walls for 'steel intensive' basement construction. This is particularly relevant to top-down construction methods where the sheet pile walls can be left in place and built into the final construction, being propped by cast in-situ concrete floor slabs, as the construction progresses deeper.

A lower whole life cost is obtained due to simpler and faster construction, less risk of subsidence during construction, reduced maintenance cost through less leakage, and ease of removal at the end of service life.

Several applications of this type of construction are presented as Case Histories. A detailed explanation of the advantages and cost benefits are included together with a general review of alternative construction methods.

The design basis is presented in some detail because of the current lack of appropriate design codes and guidance on this type of construction. It is followed by an explanation of the procedure involved in basement design.

A separate treatment of singly propped and multi-propped wall design is included. It permits the designer to choose a design analysis appropriate to the site at the various stages of construction and for comparison of wall types. An explanation of the recent advances in soil-structure interaction design methods that permit efficient design for flexible steel walls at all stages of construction is included. The important issue of ensuring base stability, and that of temporary design is covered.

Basement design requires careful attention to the degree of waterproofing needed to satisfy the intended use required by the client. Guidance is given on pre-installed sealants that have been developed to waterproof the joints of sheet piles permanently for the life of the structure.

The design of fire resistance and fire protection of basement construction is covered. Also, the issues concerning sheet pile installation are discussed.

## Utilisation de l'acier dans la construction en sous-sol

### Résumé

*Cette publication constitue un guide relatif aux aspects de dimensionnement et de mise en place de murs permanents en palplanches en acier pour la construction de sous-sols avec utilisation intensive d'éléments en acier. Ce type de construction "du haut vers le bas" utilise des murs en palplanches, qui sont laissées en place dans la construction finale, étançonnés par des dalles en béton coulé sur place, au fur et à mesure que la construction progresse en profondeur.*

*Le coût, intégré sur la vie de l'ouvrage, est réduit grâce à la rapidité et la simplicité du procédé, au moindre risque de subsidence durant la construction, au coût réduit de la maintenance lié à la diminution des infiltrations d'eau et la facilité de démantèlement en fin de vie.*



*Différentes applications de ce type de construction sont présentées à l'aide de réalisations concrètes. Une explication détaillée des avantages de la méthode est donnée ainsi qu'une comparaison à des méthodes alternatives.*

*Les bases du dimensionnement sont exposées en détail à cause du manque actuel de codes appropriés et de guides de dimensionnement consacrés à ce type de construction.*

*Les dimensionnements des murs à simple éتانçonnement et à éتانçonnements multiples sont traités séparément afin de permettre au concepteur de choisir la méthode de dimensionnement la plus appropriée compte tenu des particularités de l'ouvrage considéré. Un paragraphe est consacré à expliquer les progrès récents dans les méthodes prenant en compte l'interaction sol-structure, ce qui permet un dimensionnement performant des murs flexibles en acier à tous les stades de la construction.*

*Le dimensionnement de ce type de construction demande de traiter avec soin le degré d'imperméabilisation de la construction, en fonction des exigences du client. Des conseils sont donnés concernant les systèmes de scellement qui doivent permettre de rendre étanches les assemblages des palplanches en acier durant toute la vie de la structure.*

*Le dimensionnement à l'incendie de ce type de construction est également traité dans ce guide. Les problèmes liés à la mise en place des palplanches sont également évoqués.*

## **“Stahlintensive” Untergeschosse**

### **Zusammenfassung**

*Diese Publikation berät bei Entwurf und Einbau von permanenten Stahlspundwänden beim Bau von “stahlintensiven” Untergeschossen. Dies ist besonders relevant bei Bauverfahren, bei denen die Stahlspundwände im Untergrund belassen und in die endgültige Konstruktion integriert werden können, wobei sie im Zuge des fortschreitenden Bauprozesses durch Ortbetondecken abgestützt werden.*

*Es werden geringere Kosten über die gesamte Lebensdauer durch einfachere und schnellere Bauweise, geringeres Setzungsrisiko während der Bauzeit, geringere Unterhaltskosten durch weniger Leckagen und leichtes Entfernen am Ende der Lebensdauer erreicht.*

*Mehrere Anwendungsfälle dieser Bauweise werden als Fallbeispiele vorgestellt. Eine ausführliche Erklärung der Vorteile und Kostenvorteile ist enthalten, zusammen mit einem allgemeinen Überblick über alternative Bauverfahren.*

*Die Berechnungsgrundlage wird stellenweise vorgestellt, da es zur Zeit keine passenden Normen und Anleitungen zu dieser Bauweise gibt. Es folgt eine Erklärung der Vorgehensweise bezüglich der Berechnung von Untergeschossen.*

*Eine getrennte Behandlung der Berechnung von einfach und mehrfach gehaltenen Wänden ist enthalten. Sie erlaubt es dem Ingenieur, eine Berechnung zu wählen, die zur Baustelle und den verschiedenen Bauphasen paßt, und um Wandtypen zu*

*vergleichen. Eine Erklärung neuerer Fortschritte zur Berücksichtigung der Interaktion Boden-Tragwerk ist enthalten; sie ermöglichen eine wirtschaftlichere Berechnung von nachgiebigen Stahlwänden während der Bauphasen. Das wichtige Problem zur Gewährleistung der Stabilität des Unterbaus, auch während der Bauzustände, wird behandelt.*

*Die Berechnung des Unterbaus erfordert die sorgfältige Beachtung der benötigten Wasserdichtigkeit, um die gewünschte Nutzung durch den Kunden zu gewährleisten. Anleitungen werden zu voreingebauten Dichtungen gegeben, die entwickelt wurden, um die Verbindungen der Spundwände dauerhaft, während der gesamten Lebensdauer des Tragwerks, wasserdicht zu machen.*

*Die Berechnung der Feuerwiderstandsdauer und des Brandschutzes von Unterbaukonstruktionen wird behandelt. Auch die Probleme des Einbaus von Spundwänden werden diskutiert.*

## **Sótanos con uso intensivo de acero**

### **Resumen**

*Esta publicación se refiere a aspectos de proyecto e instalación permanente de tablestacas de acero para construcción de muros de sótano con uso intensivo de acero.*

*Ello es especialmente adecuado para métodos de construcción “top-down” en que las tablestacas pueden dejarse in-situ e incorporarse finalmente a la construcción quedando apuntaladas por forjados de hormigón in-situ conforme se va profundizando la excavación.*

*Debido a la rigidez y sencillez del método se obtiene un coste de vida más bajo, menor riesgo de subsidencia durante la construcción, menor coste de mantenimiento debido a filtraciones reducidas y facilidad de remoción al acabar la vida de servicio.*

*Como ejemplos prácticos se presentan diferentes aplicaciones de este tipo de construcción. Se incluye una explicación detallada de las ventajas y ahorros así como una visión general de métodos alternativos de construcción.*

*Las bases de proyecto se explican con algún detalle debido a la falta actual de códigos de buena práctica para esta tipología, y se da también una explicación de los métodos de cálculo para el proyecto de sótanos.*

*Igualmente se incluye un estudio del proyecto de muros con acodamiento sencillo o múltiple, que permite al proyectista escoger un método apropiado al emplazamiento en cada etapa constructiva así como la comparación de diferentes tipos de muros.*

*Se adjunta una explicación de los progresos recientes en interacción terreno-estructura que permiten un proyecto eficiente de tablestacas flexibles, incluyendo los importantes temas de estabilidad del fondo y proyecto evolutivo.*

*El proyecto de sótanos necesita que se contemple cuidadosamente el grado de impermeabilización necesaria para cumplir el uso decidido por el cliente. Se dan*

*consejos sobre sellantes pre-instalados que se han desarrollado para impermeabilizar las juntas de tablestacas durante toda la vida de la estructura.*

*Se trata asimismo la resistencia y protección al fuego del sótano así como los temas de implantación de las tablestacas.*



# 1 INTRODUCTION

## 1.1 The scope for use of steel in basements

Basements can provide valuable additional space on a given building footprint for many commercial activities such as storage and car parking without increasing the height of the building. In residential applications, basements provide valuable additional space that can be efficiently heated and which is particularly appropriate for games rooms, storage, laundry areas and parking.

In Europe, for many years now, permanent embedded steel sheet pile walls have been used in basements to help solve the problems caused by high water tables, to achieve minimum disturbance of fine grained alluvial soils, and to ensure ground stability during excavation. It has been found that the use and reliance on in-situ concrete diaphragm walling or contiguous bored concrete pile walls is not economic nor adequately safe in these conditions and more innovation is needed.

In the UK, basement construction procedures have been largely based on experience gained from construction in London. As the ground conditions in London (mainly London Clay) are not typical of other cities in UK, consideration is therefore required when applying construction methods, that are suitable primarily for excavation in London, elsewhere. There are opportunities to investigate and review basement construction elsewhere and to put forward alternative types of construction to suit the variety of soil conditions that exist.

Basements may be required to have 'waterproof' and 'vapour-proof' construction. To achieve that, traditional concrete walls will need to be 'tanked'; the cost of sealing treatments can be expensive. The new sealed clutch form of sheet piling offers watertightness at initial installation that leads to reduced cost of subsequent water-proofing measures.

## 1.2 Existing guidance on the design of basements

Publications and textbooks are available on the subject of basements that give general information on current practice and knowledge on related topics. Unfortunately, most of these relate to traditional cast in-situ concrete construction methods and most are also out of date in relation to the latest structural design concepts and methods.

Of the existing publications on the subject of basements or on topics relating to them, the following texts are noted and briefly reviewed.

### *Design and construction of deep basements*<sup>[1]</sup>

This document was published in 1975 and is concerned with the design and construction of deep basements. It was written as an advisory report and not as a Code of Practice. Consequently, most of the advice is in general form. The document is currently being rewritten and updated to include advances in construction techniques and knowledge. It is expected that a new version of the document will be published in 2001 or early 2002.

***Water-resisting basement construction - A guide - Safeguarding new and existing basements against water and dampness*** (CIRIA Report No. 139) <sup>[2]</sup>

This document was published in 1995 and updates the earlier CIRIA Guide 5, *Guide to the design of waterproof basements* <sup>[3]</sup> that was published in 1978. Report 139 complements BS 8102, BS 8007 and BS 8110 and takes account of the previous document<sup>[1]</sup>. The document also refers to the Building Regulations 1991 and to appropriate sections of BS 8007 and BS 8110.

A summary of CIRIA Report 139 is published as CIRIA Report 140 <sup>[4]</sup>.

***Deep excavation - A practical manual*** <sup>[5]</sup>

This publication contains both practical rules and details for the economic and efficient construction of deep excavations. The collected data and experience are present in the form of examples of design and solutions to construction problems.

None of the above documents cover the permanent use of steel sheet piling for basement walls.

### **1.3 Scope of this publication**

The object of this publication is to supplement the available general guidance on basement design with detailed information on the specific subject of steel intensive methods for construction and structural design of basements. To achieve this objective, information about a series of case studies on basements using steel piling has been collected and is presented in Section 2. These studies offer examples that will be of help to any designer contemplating the construction of a basement.

Clearly, cost is always an important consideration in choosing a structural configuration and material. Steel intensive basement construction offers cost savings over other forms of construction; this is illustrated by information presented in Section 3.

Design of basements depends very much on the method of construction. Section 4 explains the options for steel intensive basements and Sections 6 to 10 deal with particular aspects of design, following a design basis that is explained in Section 5.

Sections 11 to 12 considers water proofing, and fire resistance and protection. Section 13 deals with pile driving and installation. A summary of sheet pile types that are available in the UK and the CEMLOC<sup>TM</sup> system for plunged columns is included in the Appendices.

Regulations and Standards, referred to in this publication are listed in Section 14.2 and not given as numbered references in the text.

## 2 CASE STUDIES

This Section describes eleven projects that illustrate various applications of use of sheet piled walls and steel plunge piles (for internal columns) in steel intensive basements.

### 2.1 Millennium Car Park, Bristol

This underground car park located in the dock area of Bristol between two existing buildings is 80 m x 90 m by 6 m deep and provides 540 spaces for cars. The buildings, Lloyds Bank offices and Great Western Railway depot are only 8 and 12 m away. Work commenced in January 1998 to a 78 week programme. Ove Arup was the consultant responsible for design, with Carillion as the main contractor. Corus (formerly British Steel) provided the sheet piles (approximately 1300 tonnes) and Watson Steel Fabricators produced the steel tubular column piles. Sheet piling was installed by Commercial and Marine.



**Figure 2.1** *Bristol Millennium car park showing the aesthetic appearance of the sheet pile walls (see also front cover)*

The main considerations for the project were: simplicity in design and construction, the design life of the structure, aesthetics, speed of construction, the environmental impact, and the overall cost of the project.

Numerous options considering both top-down and bottom-up construction techniques were evaluated. Based on conceptual design studies, the permanent steel sheet pile wall and the reinforced concrete diaphragm wall options were found to be the most technically attractive. The top-down construction method using steel sheet piles for the car park walls was finally chosen because of the

lower overall cost and the saving in construction time. For further cost information see Section 3.3.

The sheet piles were driven in pairs (with the common interlock welded in the shop) to design penetration, to a positional tolerance well within specification; no pile trimming was required. Driving lengths were in the range 14 m - 17 m with approximately 2 m toe into the underlying rock strata (Mercia Sandstone). Most of the sheet piles were LX 32, with a small number of L6 piles in certain positions. Larssen box piles were also used, clutched into the adjacent sheet pile wall where vertical loads were particularly concentrated.

Top-down construction dictated that internal columns be installed. Following the installation of the sheet pile boundary walls, the grid of columns (7.8 m x 7.8 m) needed to be in-place to provide adequate support to the floor slab, as the underlying soil was removed. The columns and their foundations were installed using the 'plunge pile' installation method (see Section 4.1.1). Auger drilling into the Mercia Sandstone enabled a 1.2 m diameter reinforced concrete bored pile to be constructed. Once the concrete was tremied into the bentonite filled bored hole, a steel tubular column was 'plunged' into the wet concrete. Structural rather than piling tolerances both in plan and in verticality were achieved by the use of a template and plumb-bob arrangement. To provide adequate shear transfer between steel and concrete, shear studs were welded to the tubular column.

The floors of the car park consist of reinforced concrete flat slabs of varying thickness. The ground floor slab is 450 mm thick, the intermediate level floor slab 300 mm thick and the base slab 500 mm thick. As the ground floor slab needed to support 2 m of fill from landscape gardens above and to accommodate Highways Agency loading requirements, the thickness of the slab was noticeably more than a typical floor slab thickness of 300 mm. The additional thickness provided the necessary structural capacity for a more flexible approach to temporary works and the removal of the soil below ground level using larger 'moling holes'. Most significantly the slab could accommodate the loads from a tower crane without needing a steel grillage to spread the loads.

During excavation, dewatering of the ground at base level was required because the ground was highly permeable and the water table was only 1.5 m below ground level. Eight small pumps were used to lower the groundwater table evenly around the basement, and the rate of water flow was only 3 l/min due to the effectiveness of the embedded steel sheet pile walls. Once the floors of the basement were constructed, the sheet pile walls were made fully water tight by site welding of the remaining un-welded interlocks. Welding was a two-stage operation where water seepage was present. The first run sealed the clutches and then the main structural weld was placed over it. Sealing of the connection of the base slab to the steel sheet pile wall connection was effected using a proprietary grout sealant. As a controlled amount of water seepage can be tolerated in the concrete base of an underground car park, no waterproof membrane was used.

The exposed steel pile surfaces within the basement were sand blasted and painted with a two layer paint system comprising a basecoat and a topcoat. The internal columns were fire protected using an intumescent paint.



## 2.2 Underground car park, The Hague, Holland.

The underground car park was built as part of the extension to the head office of Siemens Netherlands NV offices in The Hague. The car park is rectangular in plan and consists of five levels, two of which are underground. The overall dimensions of the structure are 138 m long by 16 m wide on the surface and 32 m wide below ground. The deepest basement floor level is approximately 5.4 m below ground level with a usable height of 2.2 m between floors. Adjacent to the car park on one side is a railway running on an embankment, whilst on the other side there are adjacent buildings.



**Figure 2.2** *Underground car park , The Hague, Holland (TESPA)*

The soil strata at this location consisted of alternating layers of permeable and impermeable soils for the top 16 m, overlying very compact sand. As the ground water level was 1.5 m below ground surface, consideration had to be given to changes in drainage during construction and to the prevention of potential subsidence of the buildings adjacent to the car park. An aquifer providing drinking water was located in the strata where the underground car park was planned, hence it was stipulated by local authorities that drainage of groundwater during construction was to be restricted to less than 1.5 m<sup>3</sup>/hour. Since the aquifer was also under hydrostatic pressure, the design had to consider the possibility of ground heave in the lower impermeable layers. As a result of this requirement, the excavation depth was limited to 6 m below ground surface.

Two alternative designs were considered for the car park, one being a closed concrete structure and the other an open steel sheet piling structure. It was found that, on the basis of cost and speed of construction, the steel solution was 15% cheaper than the conventional concrete solution. This saving was achieved by using steel sheet piles as both temporary and permanent walls, thereby enabling a water-tight basement to be formed by toeing the sheet piles into an impermeable layer of clay above the aquifer. As there was the possibility of a slight amount of leakage along the interface between the piles and clay, a permanent drainage system was installed approximately 0.5 m below the deepest car park level.

The floor for the deepest level was designed as a thin concrete strut. This floor has a series of openings in it, in the form of small pavement sections set on sand. These holes allow water to pass through in the event of the car park flooding and prevent the floor becoming distorted due to water pressure in a flood situation.

As the railway embankment was very close to the car park, it was necessary to minimise subsidence during the construction phase, particularly when driving piles and when digging and draining the excavation. Piling therefore involved the use of low vibration installation methods. At locations where significant vertical loads from the structure above acted on the sheet piles, individual columns were formed using Larssen box piles. Impermeability of the sheet pile wall was obtained by applying a sealant to the interlocks in the factory before delivery.

Prior to driving the piles in the ground, holes were drilled in a line at 1.2 m centres, followed by the injection of a bentonite slurry into the hole. This pre-drilling was performed to reduce the vibration and subsidence effects on the surrounding ground and structures. A sealing product was applied at the factory to alternate interlocks; the un-sealed interlocks being positioned over the bentonite filled hole. Excavation of the soil within the confines of the sheet piles followed, with support to the wall being provided by temporary ground anchors and soil berms.

Fire protection measures were not considered necessary as the exposed steel surfaces were backed by water saturated soil, which formed a good thermal conductor.

Owing to the closeness of the sheet piling to the electrified railway, potential corrosion effects caused by stray electric currents from the rails had to be minimised. This was achieved by ensuring good conductivity between individual sheet piles and by welding steel rods to the piles to act as electrical earth pathways that would redirect stray electrical currents back to the rails.

Ventilation of the underground car park was achieved by sheet piled ventilation shafts that extend from the lowest basement level to the ground surface. Mechanical ventilation was also provided via small vertical circular ducts.

## **2.3 Travelodge Hotel, Cardiff**

The Imperial Gate Travelodge Hotel is situated in Cardiff city centre within the old docks area. The new hotel comprises a three storey building and a two level basement which is used as a bar and discotheque. The hotel building above ground level has a structural steel frame and reinforced concrete floors. The basement structure has steel sheet piles for the walls and reinforced concrete floors. The mezzanine basement level has an open central floor area, with the floor being supported by the sheet pile walls and by internal steel columns. The reinforced concrete basement slab is supported in part by the underlying soil and forms the horizontal boundary to the basement. The base slab is also connected structurally to the sheet pile walls.

The basement was constructed using the top-down construction method. Once the site was cleared, the sheet piles were driven to the required level. Owing to the closeness of the adjacent buildings and the stiff marls that are present, the chosen piling system was to drive single Larssen sections using an ICE 233 high

frequency vibrator. Once the sheet piles had penetrated past the upper marl, further piling was undertaken using an hydraulic drop hammer. Final penetration level was 10.5 m below ground level. Single piles were driven to reduce the energy required to drive the pile through the marl, as pile resistance would be less than that for two piles welded at the interlock (the most commonly used method). It was found that the maximum Peak Particle Velocity (PPV) was well within the limiting value recommended in BS 5228. Overall, approximately 150 m of Larssen LX32 high strength steel piles totalling some 300 tonnes were used in the development.



**Figure 2.3** *Permanent steel sheet pile wall and concrete pile cap supporting the building superstructure*

Once the outer walls of the basement were installed, it was necessary to install foundations that would support the basement floor as top down construction advanced. Plunge piles were installed at the locations of the internal columns of the building. Bored piles 1.2 m in diameter were augered to a depth of 13 m, concrete was tremied and 305 x 305 UC 198 steel columns were plunged into the wet concrete.

As the hotel is located in the old dock area, ground water level was high and set to rise further when the Severn estuary barrier is completed. Hence, the design of the basement needed to resist the high water pressures that act on the base slab and the walls.

Consideration had to be given to waterproofing the basement because it was to be used as a bar and discotheque. A high standard of waterproofing was required and this included prevention of vapour ingress. The steel sheet pile walls were made water tight by sealing each individual interlock. This was achieved by welding each interlock on-site, once excavation of the soil allowed access to the face of the pile. After excavation of the soil to base level, a fully water- and vapour-proof membrane was placed onto a prepared ground surface prior to the reinforced concrete base slab being cast. Prevention of potential leakage at the interface between the slab and the sheet pile wall was achieved by the presence of a proprietary tube system located within the depth of the reinforced concrete slab.



The tube system allows for pressure testing of the interface to ascertain whether any leakage is present. If leakage occurs, the interface is sealed by injecting grout into the tubing, which in turn is forced out of the tubing to fill any cracks and voids.

## 2.4 Ford recycling facility, Bridgend, Glamorgan

The Ford Motor Company, as part of the £340 m expansion in 1988, constructed a large underground recycling facility. To accommodate the necessary equipment and large holding tanks for fluids, a basement approximately 180 m long by 14 m wide and 9 m in depth was constructed. Ford had already installed a similar steel sheet pile wall basement at their works in Cologne and as that development was both simple and successful, Ford were keen to apply the same construction materials and principles again at Bridgend.



**Figure 2.4** *Panoramic view of the sheet pile walls and beams*

The driven pile approach however, was considered impractical at Bridgend on account of the extremely hard strata at the construction site. This could potentially lead to pile refusal when driving to the required depth. If driving were achieved, there would be a possibility that pile declutching would prejudice the integrity of the joints in what had to be a permanently waterproof structure.

Consequently, an open-cut approach was chosen, where bulk excavation of soil to a depth of 10 m took place. The process of pitching individual sheet pile sections around the perimeter of an excavated chamber and ensuring the interlocks were watertight would thus present no problems apart from the amount of site welding

involved. The extent of site work and the tight space constraints of the Bridgend site, led to the choice to carry out as much of the work as possible off-site.

Approximately 200 prefabricated walling panels of varying configurations (typical size being 9 m by 2.5 m), made from longitudinally welded piles were delivered to site to achieve the fast-track construction programme. The panels were designed to span between the roof and base slabs and to resist the loads from the adjacent soil, the ground water, and the loads from the roof structure.



**Figure 2.5** *Prefabricated panels being positioned onto concrete slab*

Concrete channel sections 500 mm wide were precast on site and installed around the perimeter of the 800 mm thick reinforced concrete slab. The channel section enabled the toes of the steel Larssen 6 sheet pile walls to be positioned accurately and provide a level base.

The slab was designed to prevent floatation of the basement. It also adequately fixed the walls by providing full lateral support to the wall toes. Bored cast in-situ tension piles were installed in the soil below the slab to provide anchorage and additional resistance against uplift.

A waterproof membrane was provided beneath the reinforced concrete slab, with external waterbars at all construction joints and a hydrophilic mastic seal around each tension pile where it penetrated the water proof membrane. To provide a water barrier between the steel wall and the concrete slab, a steel plate cut to the shape of the sheet pile was welded to the inside face of the sheet pile near the toe of the piles to act as a water bar; a hydrophilic seal was fixed to the top of the plate to provide addition sealing.

Conventional pile pitching techniques were used to interlock adjacent panels; prior to slab concreting, all site joints between panels were seal welded to ensure watertightness. Temporary wall bracing was progressively replaced by permanent horizontal beams at ground level that formed the ground floor level framework. The roof of the basement comprised a composite beam section using a reinforced concrete deck and high strength (S355) 610 x 305 UB beams spaced approx 2.5 m apart and a span of 14m. At locations where high forces were acting, Larssen box piles were chosen to increase both the bending and axial resistance.

The benefits realised during this project were:

- speed of construction
- a saving in construction duration of at least 2 months
- lower overall costs
- greater on-site safety
- ease of maintenance
- improved site management
- minimal land-take.

In addition, in an age of increasing environmental awareness, the steel sheet pile walls can be easily removed and recycled.

Further information can be found in a paper by Swan *et al.* <sup>[6]</sup>.

## 2.5 Oslo City underground car park

The construction of the biggest office block in Norway - 'The Oslo City' building, - took just 24 months to complete. This very short construction time was possible thanks to the use of steel sheet piles to form the six-storey underground car park. The sheet piles were used in permanent construction for both excavation support and to carry the vertical loads from the structure above.

Ground conditions at the site included soft to firm clay overlying bedrock; in light of this, a decision was made to use Z profile sheet piles where the toe level was above rock (17 m depth) and U profile sheet piles where rock penetration was required.

The U profile piles were supplied to site in pairs, where they were grouped into quadruple piles. Four steel tubes were then attached to each quadruple pile, and the tube ends were sealed with concrete. The sheet piles were driven to the rock horizon with vibratory techniques and then driven into rock using hydraulic plant. From the ground surface, drilling was carried out through the steel tubes into the rock. Steel shear pins were then inserted into the predrilled holes to anchor the piles to the rock, and grouted into place. The whole base area was then grouted with cement slurry to ensure a watertight construction.





**Figure 2.6** *Oslo City underground car park, Norway*

More than 700 anchors were installed as excavation progressed. When complete, the car park floors were used to stiffen the structure. Upon completion of the excavation, a reinforced local distribution beam was anchored into the rock prior to casting the concrete base.

An epoxy coating was applied to the visible portions of the steel sheet piling inside the building and a sprinkler system was installed to ensure compliance with the fire regulations.

## **2.6 Basement car park, Staines**

This project involved the construction of a new two-storey basement car park in ground conditions that were difficult because of a high water table.

The consultants Andrews, Kent and Stone, concluded that a sheet pile solution was the most practical and cost effective. Their investigation had to take account of the need for extensive temporary propping works due to the proximity of existing buildings to the site boundary. The proposal to line the basement with Larssen LX32 sheet piles in itself did not prevent water penetration. Water tightness was to be achieved by continuous welding of the joints after excavation and by provision of a fully drained cavity system, all of which added to costs.

During the tender period Kvaerner's engineers looked at various alternatives to the proposed design but came to the conclusion that sheet piling was the most economic method. The sheet piles were supplied with 'Haltlock<sup>TM</sup>' sealant. This involved welding an angle section to one clutch of each sheet pile, and filling the void with a bituminous compound, so that when driven, a permanent 'sealed' junction would be achieved.

The bid by Kvaerner (now Skanska UK Building) was successful and the installation was carried out using a silent piling technique.



**Figure 2.7** *Staines underground car park*

The basement is 8 m in depth and is 'L' shaped, with indents in the profile for staircases. Larssen LX32 sheet piles 14 m long were driven through the fill and Thames ballast into the London clay strata. The water table was only 2 m below ground level. The pile installation commenced six weeks after final detailing/scheduling. A total of 403 sheet piles were installed in four weeks using two hydraulic pressing rigs. Before driving of the sheet pile began, the pile line was pre-bored to 5 m with an auger piling machine in order to check for obstructions. Temporary works comprised eighty tons of steel in horizontal propping, which reacted onto the perimeter reinforced concrete capping beams.

The work was carried out during an extremely wet period in November/December 1998. As a result, the water table stayed at 2 m below ground level throughout the operation. Problems were experienced with disposal of water used in the 'jetting' process, but apart from that, the installation went exceedingly well.

After exposure of the face during the bulk excavation, a few minor leaks were experienced. These had occurred as a result of damage to the piles during handling and due to large flint obstructions in the ballast during driving. Leaks were repaired by welding on steel plates and filling them with a sealing compound. Overall, the method was very successful and proved to be a great asset to the project in terms of time, cost and programme.

## **2.7 Underground garage, Rapperswil, Switzerland**

This project for a multi-purpose building incorporated an underground garage between an existing underground structure, a street and an adjoining railway line. Strata encountered included soft, clayey to loamy silt. Ground water stood only a metre below the ground surface. These difficult ground conditions dictated a somewhat novel cofferdam construction method. It was decided to compartmentalise the structure in 7 m<sup>2</sup>, enabling excavation and subsequent underwater casting of the concrete floor in each compartment to be carried out in a single day. The partition walls also served to give an increased factor of safety



against uplift. The external wall of the cofferdam was designed to be rigid, using 23 m long U profiles and the internal walls were 18 m long Z profiles.



**Figure 2.8** *Construction of an underground garage, Rapperswil, Switzerland*

The use of this type of construction successfully overcame difficulties that had been encountered previously with other techniques. The sequence of construction was as follows:

- preliminary excavation down to ground water level
- vibratory installation of the internal walls and utilisation of a special template for the perimeter wall
- construction of the internal framing
- compartmentalised underwater excavation and underwater concrete base casting
- dewatering of the excavation
- removal of the internal walls to base level
- completion of foundations and concrete masonry.

A bituminous compound was applied to the pile locks prior to delivery which effectively sealed the complete wall against water intrusion. The sheet piling formed part of the permanent structure, thus minimising potential ground settlements.

The selected procedure enable successful construction without significant ground settlement problems to neighbouring properties.

## 2.8 Underground garage, Bad Oeynhausen, Germany

A two-storey underground garage with 300 parking spaces was constructed underneath the municipal bus depot of the city of Bad Oeynhausen in order to expand the city's parking facilities.

The external walls of the underground garage were constructed of steel sheet pile sections measuring 13 m to 17.6 m long. A total of 550 tonnes of steel piles were used.



**Figure 2.9** *Underground garage, Bad Oeynhausen, Germany*

The sections were secured with grouted single and double anchor bars and were designed to meet the following three objectives:

- provision of the external wall
- absorption of the high vertical loading (structure load and loading exerted by the bus depot)
- protection against uplift.

Stringent noise and vibration level limits had to be adhered to in view of the fact that the construction site was located in an urban centre. The decision was made to drive the sheet piles using a combined drilling and jacking technique. To achieve this, a graphite coating specially developed for this purpose was applied to the interlocks, to enhance the sliding action. After driving the sheet pile wall, the interlocks were welded to ensure that the wall was water tight. This was followed by sand blasting the wall and then applying a coat of epoxy resin to it. An additional sprinkler group was installed in front of the sheet pile wall in order to ensure adequate fire safety.

The garage is an economical, yet functional and aesthetically pleasing structure, with all of the advantages offered by sheet pile construction being optimally utilised.

## 2.9 Other steel intensive basements in Europe

Although the use of permanent steel sheet pile walls for basements is a relatively new concept in the UK, in Europe this form of construction has been used on numerous projects. A total of 19 underground car park projects are listed in a brochure produced by the Technical European Sheet Piling Association (TESPA) literature; eight of the projects are in the Netherlands, five in Germany, two in Norway and one each in France and the UK. A selection of these are shown below:



*City Hall, Rouen*



*House for dentistry, Munster*



*World Trade Centre, Amsterdam*



*Congress Centre, Amsterdam*



*Court, Munster*



*Zwolsestraat, Scheveningen*





*Malieveld, Den Haag*



*Market square, Coesfeld*



*Munsterstrasse, Dusseldorf*



*KLM Amstelveen*



*VROM, Den Haag*



*Royal Christiana Hotel, Oslo*

## **3 BENEFITS AND COST COMPARISONS**

This Section discusses the technical and economic benefits, aesthetic considerations and whole life costing evaluations associated with the use of steel sheet piles for basements. A cost comparison study shows the economic attractiveness of steel sheet piles compared to the use of concrete diaphragm walls for underground structures (Section 3.2). The findings of the study are reinforced by an actual cost comparison exercise undertaken for the Bristol Millennium underground car park (Section 3.3).

### **3.1 Benefits of sheet piles in basements**

The use of steel sheet piles, box piles and high modulus piles provide a factory quality product of known structural integrity that fulfil the requirements for minimum construction time. Not only can piles be driven rapidly in the majority of soil types, they are also capable of being loaded immediately, which is a distinct advantage in fast-track construction projects.

By adopting sheet piling and/or box piling for basement walls, construction space is minimised and because the wall is installed prior to soil excavation, adjacent buildings can be more reliably supported, thereby preventing potential subsidence. Piles can be driven close up to a land boundary; this means that land usage can be maximised. As wall construction time is less than for concrete walls, associated site costs can be reduced.

In this environmentally conscious age, steel sheet piles have the added benefit because they can be easily extracted at the end of their purpose and can be re-used or re-cycled.

These advantages and others can be summarised as follows:

- Construction is significantly quicker than that for reinforced concrete walls.
- Permanent sheet piling is a narrow form of construction, which can be installed close up to the boundary of the site maximising usable site space.
- Steel sheet piles are suitable for all soil types.
- There is no requirement to excavate for wall foundations.
- There is no disturbance of existing ground unlike that for bored concrete piling.
- The steel components are factory quality as opposed to site quality.
- Steel sheet piles can easily be made aesthetically pleasing.
- Steel sheet piles can be placed in advance of other works.
- Immediate load-carrying capacity is present.
- Steel sheet piles can be used as curtain walling to contain the working site.
- They are a sustainable product as they are extracted easily and minimise waste.

## 3.2 Cost comparison study

A study was undertaken for Corus by Dearle and Henderson, Chartered Quantity Surveyors, to provide an independent assessment of the construction costs for steel sheet pile alternatives to diaphragm walling and secant pile walls. The research project on which it was based was undertaken by Potts and Day, Imperial College of Science, Technology and Medicine, London [7].

Traditionally, designers have assumed that only sheet pile walls with stiffness equivalent diaphragm walling, e.g. high modulus piling, would represent a structurally viable alternative. This research indicates that the ability of steel sheet piling to deflect under load, with a consequent redistribution of moments, does provide for a structurally viable alternative, whilst keeping overall wall deflection within acceptable limits.

Sketch drawings showing the form of the diaphragm wall and steel sheet pile solutions for three schemes which had previously been designed as either diaphragm walling or contiguous piling are shown in Figures 3.1 to 3.3. These schemes were:

- House of Commons underground car park (a multi-propped basement)
- George Green tunnel (a single-propped road tunnel)
- Bell Common tunnel (a double propped road tunnel).

The road tunnel construction projects are included because construction is similar to that for a basement, thus they provide supportive evidence for the use of steel sheet piles in underground structures.

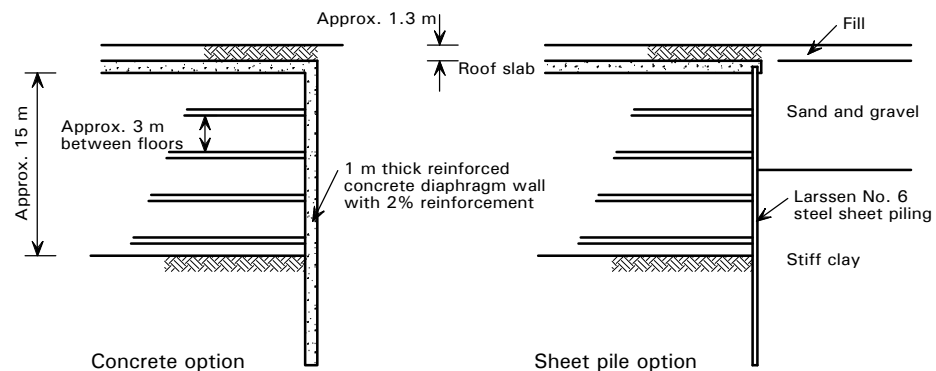
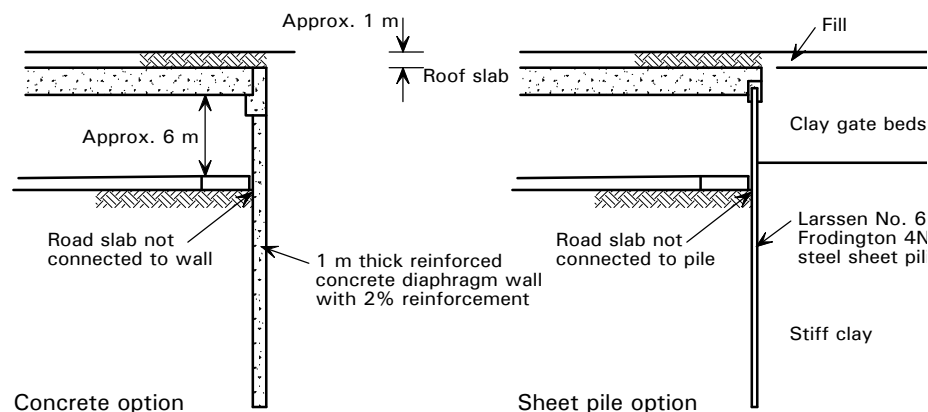
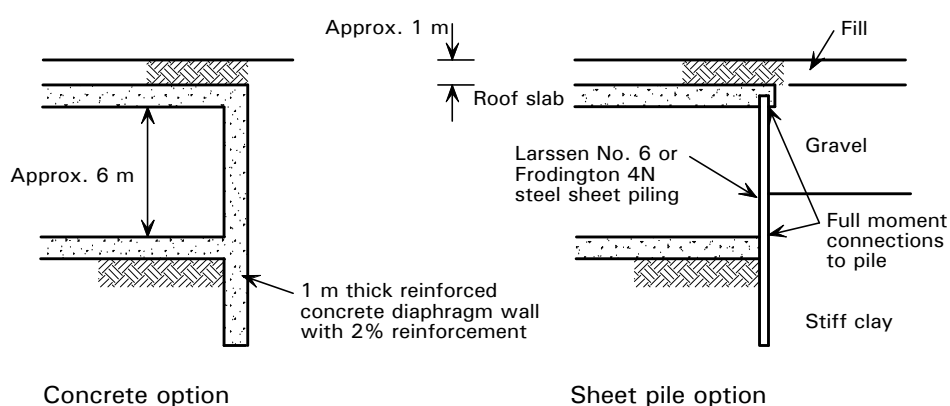


Figure 3.1 House of Commons underground car park



**Figure 3.2** *George Green single-propped road tunnel*



**Figure 3.3** *Bell Common double propped road tunnel*

### 3.2.1 Basis of costings

The costings were based on the following:

- Brief specification (as shown below).
- The method of procurement (would be bills of quantities under the ICE Form of Contract).
- A saving in contract period and consequent reduction in time related/preliminaries items.
- Installation of steel sheet piles using high frequency vibrator and pressing techniques to maintain noise and vibration comparable to diaphragm walling. Clearly, if these restrictions were not there, further significant savings could be achieved by conventional impact driving.

### 3.2.2 Wall alternatives considered

The following alternatives were considered for each scheme:

- Diaphragm wall finished 'as formed'.
- Diaphragm wall with blockwork facing.
- Diaphragm wall with reinforced in-situ concrete facing.
- Secant pile walls finished 'as formed'.

- Secant pile walls with blockwork facing.
- Secant pile walls with reinforced in-situ concrete facing.
- Corus sheet piles finished 'as formed'.
- Corus sheet piles with blockwork facing.
- Corus sheet piles with corrosion protection coating.
- Corus sheet piles with fire protective coating.

### 3.2.3 Specification

#### Diaphragm wall

For each of the three schemes, a 1.0 m thick wall with 2% steel reinforcement was assumed using 1.0 m x 1.0 m guide walls.

#### Steel sheet piling

For the House of Commons scheme, the use of Larssen 6 piles (138.7 kg/m) grade S355GP was assumed. For the other schemes both Larssen 6 and Fordingham 4N grade S355GP steel sheet piles were evaluated. Allowance was made for two stage driving of piles using high frequency vibrator and pressing techniques to maintain noise and vibration comparable to diaphragm wall operations.

#### Secant wall piling

The secant pile wall consisted of 1180 mm diameter bored piles at 1080 centres. Two types of reinforcement were considered, one type being steel bar reinforcement at 100 kg/m<sup>3</sup> and the other the use of a UB914x305x289 steel section.

The following alternatives were considered for each site:

Diaphragm walls and secant pile walls:

- finished 'as formed'
- blockwork facing
- 150 mm thick reinforced in-situ concrete facing.

Steel sheet piles:

- finished as formed
- 140 mm thick solid blockwork
- 150 mm thick reinforced in-situ concrete
- paint finish - Shot blast cleaning to steel piles and one coat high build isocyanate cured coal tar epoxy pitch
- one hour fire protective coating - Shot blast cleaning to steel piles and epoxy metallic zinc rich primer and intumescent spray paint.

### 3.2.4 Cost comparisons

The cost comparisons for the three schemes are presented in Tables 3.1 to 3.3. Each scheme has been individually expressed with the cost of all alternatives compared to a base of 1.00 for an 'as formed' diaphragm wall.



**Table 3.1** *Cost comparison for House of Commons multi-propped walls in underground car park*

<b>Method of construction</b>	<b>As formed</b>	<b>Blockwork facing</b>	<b>Concrete facing</b>	<b>Paint finish</b>	<b>Fire protected</b>
Diaphragm walling	1	1.05	1.08	n/a	n/a
Secant piling (100kg/m <sup>3</sup> )	0.81	0.85	0.88	n/a	n/a
Secant piling (UB914x305x289)	1.24	1.29	1.32	n/a	n/a
Larssen 6 steel sheet piles	0.77	0.82	n/a	0.84	0.87

**Table 3.2** *Cost comparison for George Green single-propped wall in road tunnel*

<b>Method of construction</b>	<b>As formed</b>	<b>Blockwork facing</b>	<b>Concrete facing</b>	<b>Paint finish</b>	<b>Fire protected</b>
Diaphragm walling	1	1.05	1.07	n/a	n/a
Secant piling (100kg/m <sup>3</sup> )	0.76	0.8	0.83	n/a	n/a
Secant piling (UB914x305x289)	1.22	1.26	1.29	n/a	n/a
Larssen 6 steel sheet piles	0.73	0.73	n/a	0.8	0.83
Frodingham 4N steel sheet piles	0.52	0.57	n/a	0.57	0.61

**Table 3.3** *Cost comparison for Bell Common double propped walls in road tunnel*

<b>Method of construction</b>	<b>As formed</b>	<b>Blockwork facing</b>	<b>Concrete facing</b>	<b>Paint finish</b>	<b>Fire protected</b>
Diaphragm walling	1	1.03	1.05	n/a	n/a
Secant piling (100kg/m <sup>3</sup> )	0.81	0.85	0.87	n/a	n/a
Secant piling (UB914x305x289)	1.2	1.24	1.25	n/a	n/a
Larssen 6 steel sheet piles	0.77	0.79	n/a	0.83	0.87
Frodingham 4N steel sheet piles	0.59	0.62	n/a	0.64	0.67

### Exclusions

The cost comparisons exclude the following:

Obstructions located within the ground.

Dewatering.

Professional fees.

Value Added tax.

Finance charges. On the basis that savings in time would accrue from the use of the steel sheet pile alternatives, there would be financial benefits resulting from an earlier return on investments.

### 3.2.5 Conclusion

Based on the information available, these comparisons indicate anticipated savings in the region of 25% to 40% for the equivalent finish by the use of steel sheet pile in lieu of diaphragm wall supported excavations.

## 3.3 Cost comparison for walls in Bristol Millennium underground car park

The main considerations for the project described in Section 2.1 were: simplicity in design and construction, the design life of the structure, aesthetics, speed of construction, the environmental impact, and the overall cost of the project. Based on the conceptual design studies, the permanent steel sheet pile wall and the reinforced concrete diaphragm wall options were found to be the most technically attractive. The top-down construction method using steel sheet piles for the underground car park walls was finally chosen because of simpler construction, the lower overall cost and the saving in construction time. A cost comparison of the two options is tabulated below.

**Table 3.4** *Comparison between the properties and performance of Larssen LX32 steel sheet piles and an 800 mm thick diaphragm wall*

Option	Steel sheet pile walls	Diaphragm Walls
Wall type	Larssen LX 32 pile	800 mm thick RC wall
Wall thickness	450 mm	1150 mm
Wall stiffness	$0.15 \times 10^6$ kNm <sup>2</sup> /m	$1 \times 10^6$ kNm <sup>2</sup> /m
Max horizontal deflection	56 mm	35 mm
Cost	£200 /m <sup>2</sup>	£325 /m <sup>2</sup>
Construction period	2.5 months	4.5 months

## 3.4 Aesthetics

In industrial basements or underground car parks where the basement walls can be left exposed, the crenellated appearance of sheet piles can be used to good effect by painting, without need to increase cost by specifying cladding. The strong relief of the sheet piles is increasingly found to be aesthetically acceptable to the public.

### **3.5 Whole life cost**

A lower whole life cost is obtained using steel intensive basements. The savings are due to: a faster construction period, maximum use of site area for building footprint, reduced maintenance cost from more effective waterproofing, and ease of wall and column removal at end of service life.

It is becoming increasingly recognised that useful building life is shorter than it was; periods of only 30-40 years are considered now instead of 60-80 years a few decades ago. In this new investment climate, the cost of returning the site for redevelopment is becoming an important factor.



## **4 CONSTRUCTION METHODS FOR BASEMENTS**

This Section discusses the construction methods that can be used for basements. Factors affecting construction are listed together with principal construction methods. Top-down construction and bottom-up construction techniques are introduced. In most cases embedded retaining walls, which can provide both temporary and permanent support, prove attractive both economically and technically. Other construction methods are also introduced.

### **4.1 Factors affecting basement construction**

Various factors influence the relative difficulty of basement construction and effectiveness of lateral support. These include:

- neighbours' legal rights
- location of the basement
- soil and groundwater conditions
- the proximity of existing structures and services
- proposed construction depth
- previous site usage
- proposed basement usage
- optimisation of available site volume.

The method of basement construction and design needs to overcome these constraints using the most economic techniques.

### **4.2 Principal methods of constructing basements**

There are three principal methods of constructing basements:

- a) in open excavations with sloping sides
- b) in excavations with temporary support to the sides
- c) in excavations supported by a permanent embedded wall constructed in advance of the main excavation.

#### **4.2.1 Open excavations**

For small and medium wall height, open excavations with soil side slopes for stability are the most economical form of construction for sites where there is ample adjacent space around the building footprint. Space has to be available to allow the sides of the excavation to be cut back to form a stable slope. Also, there must not be any problems associated with ground water flowing into the excavation (which could lead to erosion and slumping of the slopes). However, location of a basement seldom allow these open battered excavations, particularly in urban sites, where space is limited. As inner city land is expensive, basement construction is needed to maximise the most effective use of the site for the intended development.

## **4.2.2 Excavations with temporary support**

Construction in excavations with temporary support is suitable for sites where insufficient space is available around the excavation to form sloping embankments. A number of temporary support arrangements can be adopted, with steel sheet piling being one of the most favoured options. Normal methods of supporting these excavations include strutting or tie-back anchors.

Excavation and basement construction, with temporary external wall support, is only possible where subsidence of adjacent ground can be allowed when the temporary wall is removed. This is most unlikely on city sites and redevelopment in regeneration areas.

## **4.2.3 Excavations supported by a permanent embedded wall**

In excavations where support is to be provided by a permanent embedded wall constructed in advance of the main excavation, the essential feature is that the wall also forms part of the permanent structure. This method of construction in many cases is technically and economically very attractive. Although, historically, mainly concrete solutions have been used for permanent walls (i.e. reinforced concrete diaphragm walls, contiguous or secant bored pile walls), substantial economic benefits can, and have been gained, by using permanent steel sheet piling. This has been seen in the construction of steel intensive basements in Europe over the last 15 years and more recently within the United Kingdom.

As the extent of available working space at the rear of permanent retaining walls is likely to be smaller the nearer the basement site is to a city centre, and the majority of basements being considered are those in urban areas, soil support systems that incorporate both temporary and permanent support will prove most efficient in minimising the width of basement walls, and hence maximising basement volume.

In such sites, vertical peripheral support is required. The simplest form for sheeting or walling is to cantilever without propping. However, the extent of soil movement during and after soil removal, and the presence of services or roads at the extremities of the proposed basement boundary, may limit the use of cantilever walls.

As basement excavations increase in depth, excavation methods have to be more complex. With increasing depth, levels of propping need to be introduced, ranging from single-prop basements for shallower depths and/or good soil conditions to multi-level propping for deep basements. To accommodate these multi-level propped basements, techniques have been developed to reduce construction costs and duration to a minimum with the least amount of disruption taking place in surrounding urban areas. Based on these constraints, two steel intensive construction methods have been developed and are commonly used in practice. These construction methods are a top-down construction sequence and a bottom-up construction sequence. In both cases, the permanent walls use steel sheet piles. These construction methods and their relative advantages and disadvantages are discussed below.

### 4.3 Top-down construction

This method of basement construction uses the permanent works to control ground movement from the beginning of the excavation and to minimise or eliminate the need for temporary works. The works can be phased and sequenced such that only small amounts of soil are removed before lateral support is provided, thereby limiting ground movements. This construction method has provided major benefits in accelerating progress on the contract programme. The method also allows for construction of a superstructure simultaneously with the basement in some cases.

The elements of support and retention systems for steel intensive top-down construction include:

- permanent perimeter steel sheet retaining walls
- horizontal propping systems, i.e. either the floor slabs of the basement or steel frames or props
- foundations for basement columns.

Top-down construction sequence commences with the driving of the steel sheet piles to design penetration depth at the perimeter of the basement thereby forming the permanent retaining wall. Foundations for the columns of the basement are constructed next because the columns will provide support to the floors as top-down construction progress. Circular concrete bored and cast-in-place pile foundations are constructed. At the proposed foundation location, a hole is augered and, as the auger is removed, concrete is injected into the hole through the stem of the auger up to formation level of the basement slab. Almost immediately after the piles have been poured, and whilst the concrete is still wet, a steel column section is lowered and accurately positioned using a jig. When the concrete has set, a steel column and concrete pile foundation is formed. This procedure is repeated for all the basement column locations. This method is commonly called the ‘plunge’ column method.

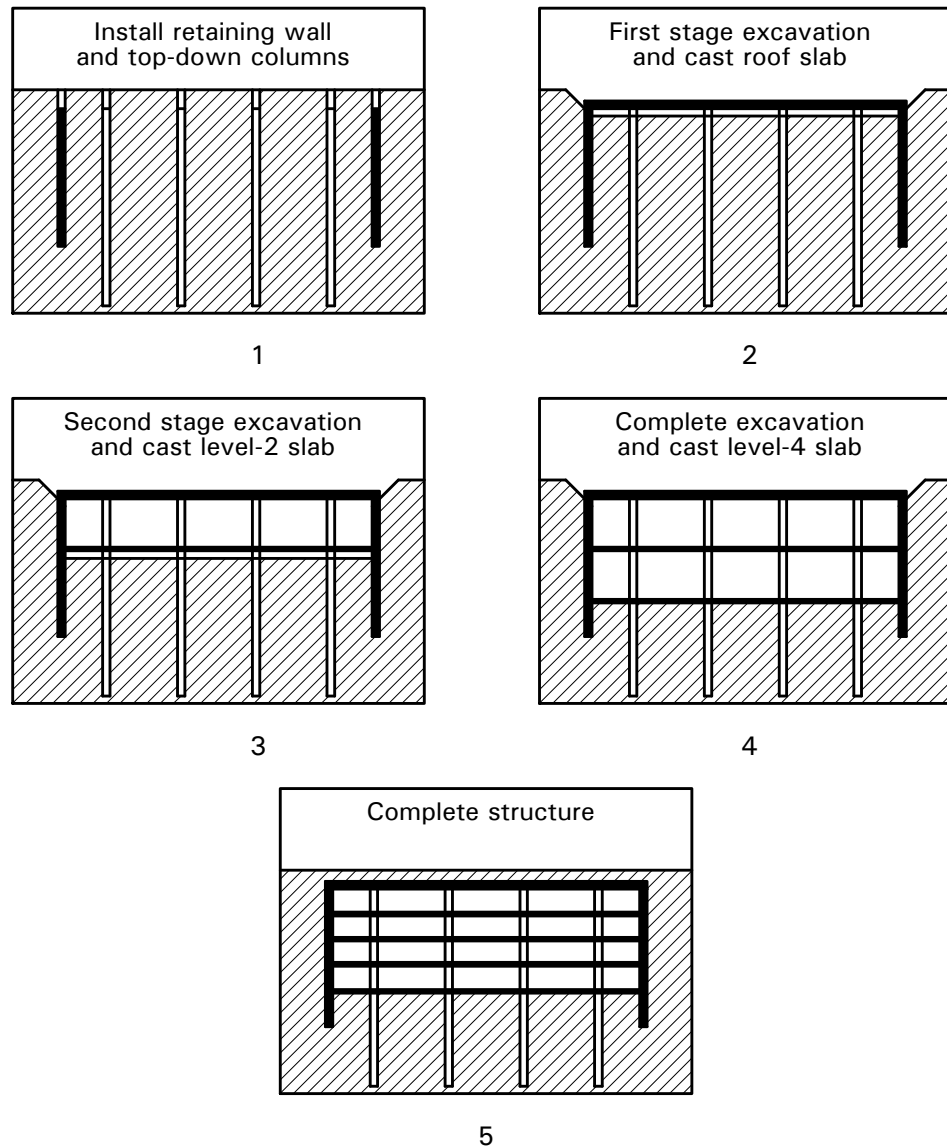
Although a number of methods have been used by contractors to install permanent columns for top-down construction, the ‘plunge’ column method has become the preferred choice owing to the substantial savings in time and cost. One such proprietary system using this ‘plunge’ column approach is the CEMLOC™ system patented by Kvaerner Cementation Foundations. See Appendix B for a more detailed description.

With the columns accurately positioned, the floor slabs are constructed. Design of the floors for the basement is conventional. The ground surface is prepared by levelling and compacting the soil to provide the base for the formwork of the ground floor slab. Concrete blinding and a polythene sheeting is placed on the blinding, thereby allowing for floor reinforcement to be placed and the reinforced concrete slab to be cast. After removal of the soil and the temporary base, a smooth concrete surface to the underside of the ground floor slab will be produced.

Using suitable openings in the ground floor slab, the soil can be excavated. Excavation is undertaken using construction plant with, where possible, a crane, although in certain circumstances, a conveyor belt can be installed. The soil is removed to the appropriate level for preparation of the base for the first intermediate floor slab and the process repeated. Where good soil conditions exist

it may be possible to excavate to a subsequently lower floor slab level, therefore creating a less confined environment for soil excavation. Finally, the base slab is constructed, either as a slab in contact with the soil underneath or as a suspended floor slab with a void below, if the upwards pressure of the soil is expected to be high.

The installation sequence for a top-down construction of a four-storey underground car park is shown in Figure 4.1.



**Figure 4.1** *Top-down construction*

Numerous advantages are inherent with top-down construction. These include:

- If a basement forms part of a multi-storey office building, top-down construction can take place whilst the building frame is constructed at the same time using bottom-up construction from the ground. This can shorten the construction programme by allowing above-ground construction to commence at an early stage.



- The resulting wall and adjacent soil movements can be controlled and kept to a minimum because the basement structure is supported laterally before significant excavation of the soil takes place. Hence top-down construction is very attractive where basements are to be built adjacent to any sensitive structures.
- Temporary construction is kept to a bare minimum, hence costs are reduced. Also deformations that would arise from temporary works and transfer of loads to the permanent works are avoided, hence movements are reduced.
- A large level site area is immediately available once the ground floor slab has been constructed.

Disadvantages are generally less onerous than the advantages that are gained. They include the following:

- Soil excavation can be difficult due to potential confinement created by the floor slabs above the area where soil is being removed. This can be minimised by planning sufficient access areas in the floor slabs.
- Ventilation systems are required to enable safe working conditions during soil removal.
- Positioning and extent of access holes in the floor slabs so as not to remove lateral support for the basement retaining walls can be restrictive.

#### **4.4 Bottom-up construction**

Bottom-up basement construction require:

- removal of as much spoil as possible by quick and economic methods, i.e. by direct access for excavation
- construction by conventional methods, with access from above
- limitation of wall and ground movement by temporary measures. Temporary support and retention systems are substituted for the weight and the pressure of the soil removed, until such time as the permanent works can become effective.

The elements of support and retention systems for bottom-up construction are:

- permanent or temporary retaining walls
- temporary restraint to retaining walls. Includes steel frames, horizontal tubular struts, inclined or raked props, ground anchorages and soil berms.

One possible bottom-up construction method includes the use of a permanent steel sheet pile wall and a temporary propping system. The construction sequence is shown in Figure 4.2.

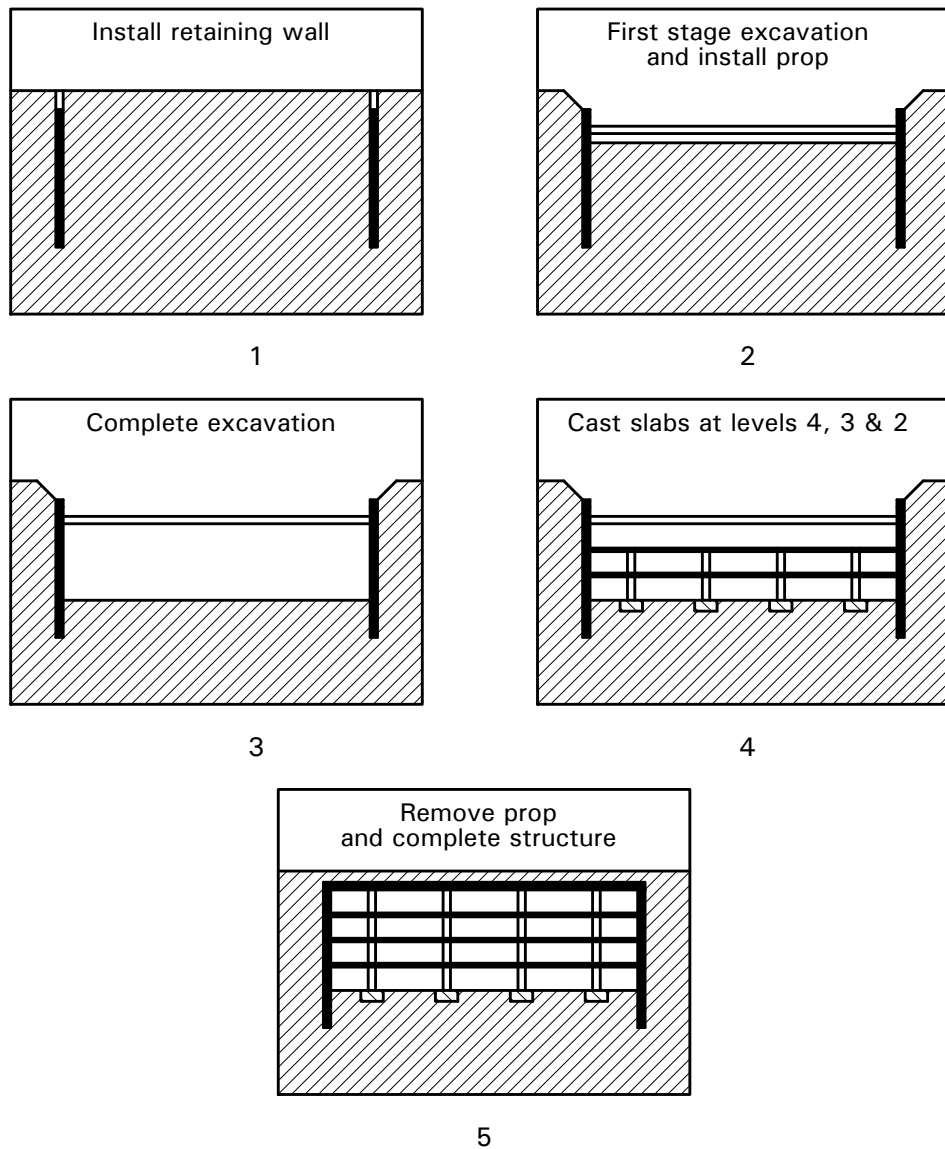
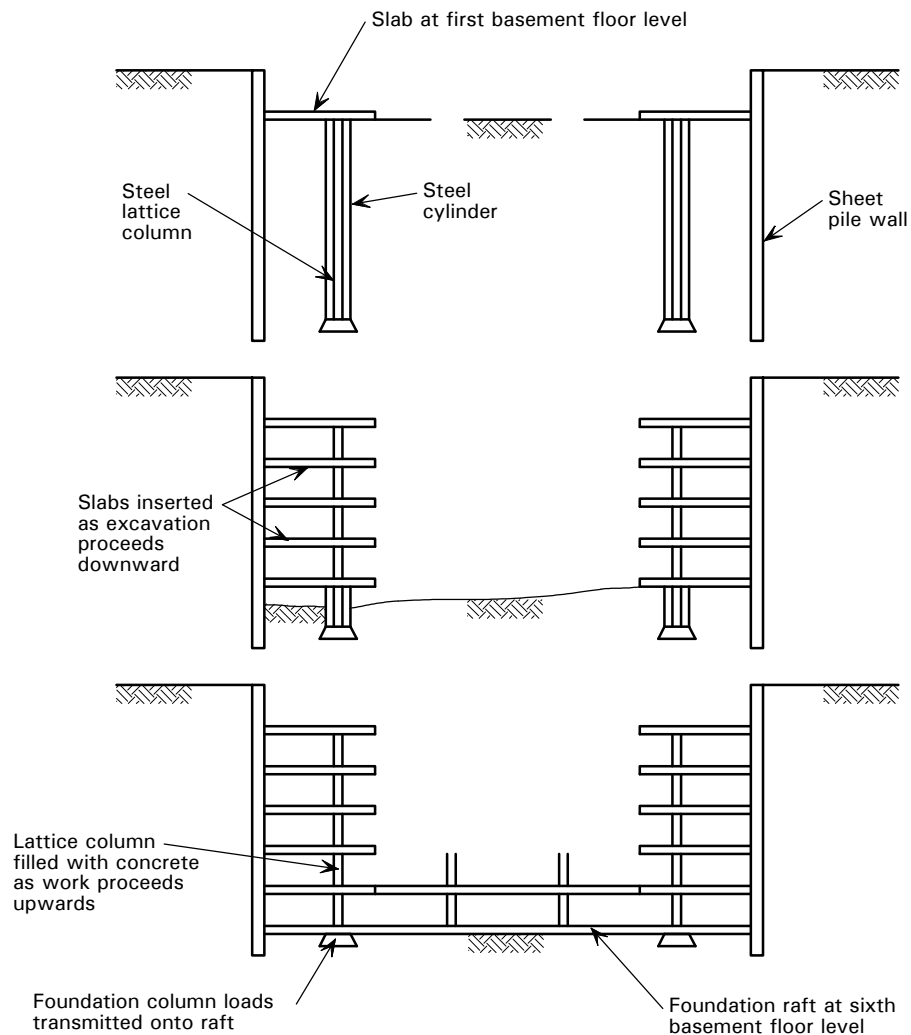


Figure 4.2 *Bottom-up construction*

## 4.5 Other construction methods

Other construction methods include combinations of top-down and bottom-up construction. One such scheme is presented in the publication *Design and construction of deep basements*<sup>[1]</sup>. In this scheme, the perimeter of the site is constructed using top-down construction and the centre is constructed in open excavation as if for bottom-up construction. See Figure 4.3.



**Figure 4.3** *Concurrent upward and downward construction (ISE publication)*

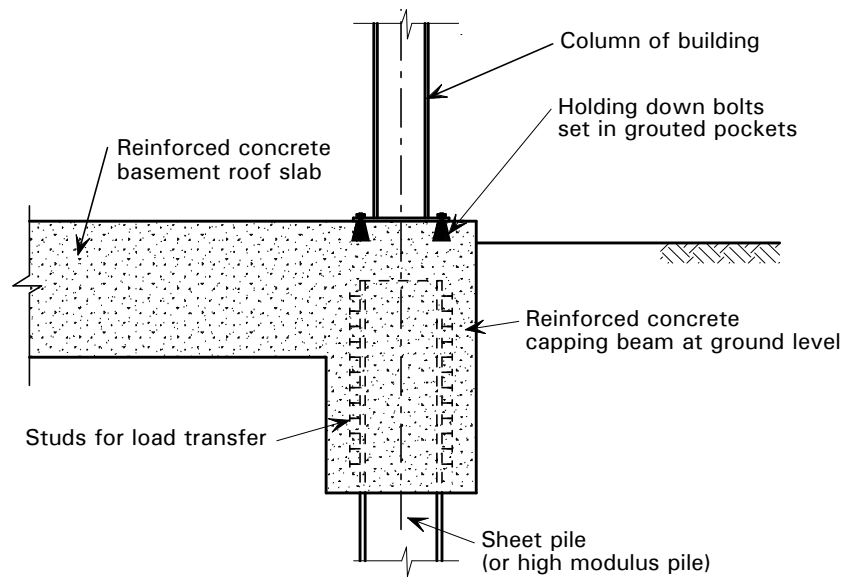
## 4.6 Reinforced concrete pile capping beams

The inclusion of pile capping beams to the tops of steel sheet piles and high modulus piles is quite common in practice (see Figure 4.4). Reinforced concrete pile capping beams are very practical because they accommodate any irregularities in the alignment and finished tops of sheet piles.

The functions of a reinforced concrete capping beam are:

- It transfers load from the superstructure to the sheet piles, thereby distributing the load effectively.
- It enables the interface detail (e.g. holding down bolts for a steel superstructure or starter bars for a reinforced concrete superstructure) to be attached.
- It accommodates the difference between nominal and as-built geometry of both the piles and the superstructure.

- It enables forces from individual props to be distributed over a number of sheet piles.



**Figure 4.4** *Reinforced concrete pile capping beam*

## 5 DESIGN BASIS

The design of basements involves geotechnical judgement. The degree of judgement depends on the complexity of the wall design and on the retained wall height (the latter affects the earth pressures to the square of wall height). The greatest need for judgement arises from the uncertainties in estimating earth and water pressures. There is much guidance available in text books and geotechnical references on that subject. However, a structural engineer needs guidance on when to perform a retaining wall design and when assistance from an experienced geotechnical engineer should be sought. This Section provides that guidance.

Compiling a design basis for basements is complex and must be carefully structured in order to be clear. This Section also includes a summary of the formal Codes and Standards and other key technical references together with guidance on procedures to be followed. The latter is a framework within which design problems can be solved to arrive at an appropriate steel intensive basement construction scheme that is relevant to the site.

### 5.1 Types of basement design

For small walls, of up to 3.5 m height, the soil pressures and consequent bending moments are not crucial to the choice of sheet pile section; the selection is governed mainly by installation and durability considerations. The simplified design procedures as given in BS 8002 and in the British Steel *Piling Handbook*<sup>[8]</sup> are sufficient to produce a safe and economic wall.

For medium wall heights, of 4 m to 6 m, the design bending moment from earth pressures becomes very important and geotechnical judgement is involved in deciding the soil design parameters. BS 8002 can be used (in conjunction with other Codes and Standards) by structural or geotechnical engineers to produce a safe wall, although the most economic construction method and overall design will be obtained by an experienced team of both structural and geotechnical engineers.

For deep basements of total excavated depth in excess of 6 m, geotechnical judgement is critical for selection of wall type, construction methods, propping and design of wall section. A team of experienced engineers is essential to produce a safe and economic basement construction. For the design of innovative steel intensive deep basement construction (which is likely to be the most economic type of construction), there is only limited experience available; this publication provides specific guidance that will supplement the general experience of specialists in foundation design.

### 5.2 Requirements of the Building Regulations

The type of basement construction can affect the internal habitable environment that is achievable because of the potential problems with water ingress and dampness. Sufficient knowledge is available to permit selection of the most appropriate construction type to satisfy the client's requirements.

The internal environment of the basement must at least satisfy the performance requirements of the Building Regulations 1991. These requirements, however, do not define how the performance is to be quantified or the measures necessary to achieve the required environment. Measures other than those specifically referred to in the Approved Documents C and F only relate to environments above drained groundwater level. Where construction is below ground water level, additional precautions will be necessary. Where an external wall or floor is subject to groundwater pressure, the Building Regulations refer to BS 8102:1990 *Protection of structures against water from the ground* for recommendations. Regulations and Standards referred to in Section 5 are listed in Section 14.2.

### 5.3 Limit state design philosophy

Limit state design philosophy is now generally accepted for structural design in UK and Europe. The old working stress design methods are being replaced by limit state methods.

The design philosophy considers two principal limit states:

Ultimate limit state (ULS)	collapse of all or part of the structure
Serviceability limit state (SLS)	a state, short of collapse, at which deformation, appearance or condition of the structure becomes unacceptable.

It is recognised that the loading conditions at ULS are normally more severe than at SLS and that most often it is the ULS considerations that govern design.

Geotechnical design has traditionally followed an essentially working stress philosophy that, in very many cases, is governed by limits on soil movement (deflections) rather than on strength.

Design of basements according to limit state philosophy has to recognise that the geotechnical design may be governed by SLS considerations (deflections) at the same time as the structural design of the walls and props is governed by ULS considerations (for strength). This is a new concept that will require time and experience to come to terms with. BS 8002 attempts to introduce the new philosophy, but it is not always clearly expressed and there is reluctance to use the Standard.

### 5.4 Design standards

The Approved Documents to the Building Regulations refer to numerous UK Standards and Codes of Practice. They also refer to some of the European standards (currently in the form of prestandards) introduced for both Design and Execution (Construction). These European Standards, known as Eurocodes, are not Codes of Practice; their role is slightly different.

Both types of Standards are discussed below.

### 5.4.1 National standards for design

There is no single Code of Practice for basement design, nor is there likely to be in the future. The principal British Standards associated with the design of basements are:

BS 8002	Code of practice for earth retaining structures
CP2 <sup>[9]</sup>	Code of practice No 2: Earth retaining structures
BS 8004	Code of practice for foundations
BS 5950	Structural use of steelwork in building
BS 8110	Structural use of concrete
BS 8007	Code of practice for design of concrete structures for retaining aqueous liquids
BS 8102	Code of practice for protection of structures against water from the ground

#### **BS 8002**

BS 8002 is a complete revision of the Civil Engineering Code of Practice No 2, which was issued by the Institution of Structural Engineers in 1951 on behalf of the Civil Engineering Codes of Practice Joint Committee.

The main changes from the IStructE code that were introduced by BS 8002 are:

- a) The recognition that effective stress analysis is the main basis for the assessment of earth pressures with total (undrained) stress analysis being important for some walls during or immediately following construction.
- b) The need to take account of the effect of movement (or lack of it) on the resulting earth pressures on the wall. The largest earth pressures which act on a retaining wall occur during working conditions (SLS). These earth pressures do not increase if the wall deforms sufficiently to approach failure conditions (ULS).

When SLS requirements for the geotechnical design are greater than the ULS requirements, this raises the question of which criterion should apply to the structural design of the wall. BS 8002 recommends that the moments and forces at SLS be used for the ULS design of the structure, although the most severe earth pressures on the wall occur at SLS. This is a conservative assumption because structural forces and bending moments due to earth pressure reduce as deformation increases (i.e. as the condition moves from SLS to ULS).

BS 8002 in its Foreword assumes that design of retaining walls is entrusted to chartered structural or chartered civil engineers who have sufficient knowledge of the principles and practice of soil mechanics as well as the principles and practice for the use of the appropriate structural materials. However, this code of practice does not restrict designers from applying the results of research nor from taking advantage of special situations or previous experience in the design of retaining structures.

### **BS 8004**

BS 8004 is applicable to the design and construction of foundations in general, which can be piled or shallow bearing foundations. BS 8004 is based on a working stress approach using lumped factors of safety and uses a design approach for foundations based on *moderately* conservative soil parameters, loads, and geometry.

### **CP2**

CP2 Earth retaining structures, although superseded by BS 8002, is still used in some temporary works design. *Total stress* design is applicable to cohesive soils where insufficient time has elapsed for the pore water pressures to be dissipated in the soil. This is a common situation in some temporary works design, particularly in clays, where undrained soil behaviour and soil properties are relevant.

### **BS 5950**

BS 5950: 2000 *Structural use of steelwork in building* provides a limit state approach to the design of buildings. The Standard covers the design, construction and fire protection of steel structures. It also provides specifications for materials, workmanship and erection.

### **BS 8110 and BS 8007**

Both BS 8110 *Structural use of concrete* and BS 8007 *Design of concrete structures for retaining aqueous liquids* are used to design the reinforced concrete floor slabs of a basement. Reference to two concrete design Standards are required because BS 8110 states that ‘water retaining structures.... are more appropriately covered by other codes’, in this case BS 8007. However, BS 8110 may be used in practice for the basement slabs for internal environments where tanking and drainage has been specified. Otherwise, the reinforced concrete floor slab will need to be designed using BS 8007.

One of the main consequences in using BS 8007 for the design of reinforced concrete slabs under water pressure is the noticeable increase in cost of construction. This is due to the significant additional steel reinforcement that is required in the slab owing to the stringent limiting crack width that is imposed by BS 8007.

### **BS 8102**

BS 8102 *Code of practice for protection of structures against water from the ground* provides recommendations for basements in which water pressures act on the basement structure. The document defines the grades of internal environments with regard to basement usage and performance level, and presents different acceptable forms of water resisting construction. Although still a current British Standard, much of the information contained in BS 8102 is now out of date. This is due to the technological advances made and the development of new forms of water resisting products and systems. The Standard does not cover the beneficial water resisting properties of sealed steel sheet piling products, which can be very effective as basement walls.



### **CIRIA Report 104**

CIRIA Report 104 (1984) <sup>[10]</sup> was adopted as an unofficial design standard before the publication of BS 8002 and is still widely used today, due to familiarity with the report and also concerns about BS 8002. CIRIA Report 104 does not address multi-prop walls, but its principle of factoring soil strength has been used in analyses of such walls by deformation methods.

The Report introduces both 'moderately conservative' and 'worst credible' values for soil parameters; the definitions are given in Section 5.10.2.

### **5.4.2 European standards**

Recently, numerous CEN 'prestandard' Eurocodes have become available. The Design Eurocodes provide a set of design principles and application rules that are deemed to satisfy the principles. They use limit state principles and a partial factor approach.

The Eurocodes use *characteristic strengths*. For materials such as steel and concrete, characteristic values can be defined in relation to values given in product standards on the basis of a probabilistic analysis of tests on samples.

The following Eurocodes may be applicable to the design of steel intensive basement design:

DD ENV 1997-1 Eurocode 7: Geotechnical design.

Part 1 General rules 1995

ENV 1992 Eurocode 2: Design of concrete structures

Part 1 *eneral rules for buildings*

Part 4 Liquid retaining and containment structures

ENV 1993 Eurocode 3: Design of steel structures

Part 1 General rules for buildings

Part 5 Design of steel structures - Piling

ENV 1994 Eurocode 4: Design of composite steel and concrete structures

Part 1 General rules and rules for buildings

To complement the Eurocodes, there are a number of CEN 'execution' Standards which relate to the construction of the works. The Standard most relevant to basements is:

EN 12063: Execution of special works - Sheet Pile Walls

### **5.4.3 Material standards for steel piles**

Steel sheet piling, including sections for box piles, is produced in accordance with BS EN 10248; the most typical grades being S270GP and S355GP with yield strength of 270N/mm<sup>2</sup> and 355 N/mm<sup>2</sup> respectively. In addition, higher strength steel sheet piles can be obtained to grade S390GP and S430GP (yield strength 390 N/mm<sup>2</sup> and 430 N/mm<sup>2</sup> respectively) subject to availability from the manufacturer.

Universal Beams (for High Modulus Piles) and other plates and sections are produced in accordance with BS EN 10025; grades S275 and S355 are commonly used. Note that while the S355 strength designations are the same, the difference between S270 and S275 should be noted.

Traditionally, impact toughness has not been considered a requirement for piling in the ground, so the grades S270GP and S355GP have no toughness testing requirement (although the Standard provides an option to require toughness testing). In the absence of specific guidance, it would seem reasonable to specify a toughness requirement equivalent to J0 quality (27J at 0°C) for the sheet piling and grade S275J0 or S355J0 for Universal Beams. As mentioned, the sheet piling material can be tested on request, and it is likely to meet the requirements of J0 quality without any special measures.

## 5.5 Geotechnical design

Limit state design of geotechnical structures, including propped embedded walls, is advocated in BS 8002 and in the prestandard Eurocode 7 Geotechnical Design. Limit states are load conditions at which the soil-structure system is on the point of failure or at which deformation exceeds the design limits specified. The principal limit states are generally classified into three types:

- Serviceability Limit States
- Ultimate Limit States
- Accidental Limit States.

To comply with serviceability limit state requirements, deformations of all elements in the soil-structure system under normal ‘working’ conditions must be satisfactory and must not cause deterioration of the materials.

The Serviceability Limit State may relate to:

- movement of ground or of a retaining structure that would cause damage to adjacent buildings
- movement of ground or of a retaining structure that would cause loss of performance of drainage or affect aesthetics of wall
- unacceptable leakage through or beneath the wall
- unacceptable transport of soil grains through or beneath the wall
- unacceptable change to the flow of groundwater
- structural durability.

To satisfy ultimate limit state requirements, it must be shown that for the worst combination of loading and material properties that can occur, there is an adequate margin of safety against collapse of any significant element of the soil-structure system.

For a satisfactory design, the occurrence of each limit state must have an acceptably low probability.

For retaining walls, the ultimate limit state may relate to:

- loss of overall stability
- failure of a structural element or failure of a connection between elements i.e. strut failure, bending stress and or shear failure in the sheeting
- combined failure through the ground and structural elements
- failure by forward rotation, translation, or lack of vertical equilibrium of the wall
- penetration failure
- toe failure
- foundation heave failure in soft clays
- hydraulic failure (i.e. piping in cohesionless soils with high external groundwater table)
- passive failure of soil below stage excavation level or final formation level.

A selection of these failure modes are shown in Figure 5.1.

Accidental limit states are a particular class of ultimate limit state where unintended loading is applied to a structure. Those relevant to basements and retaining walls include:

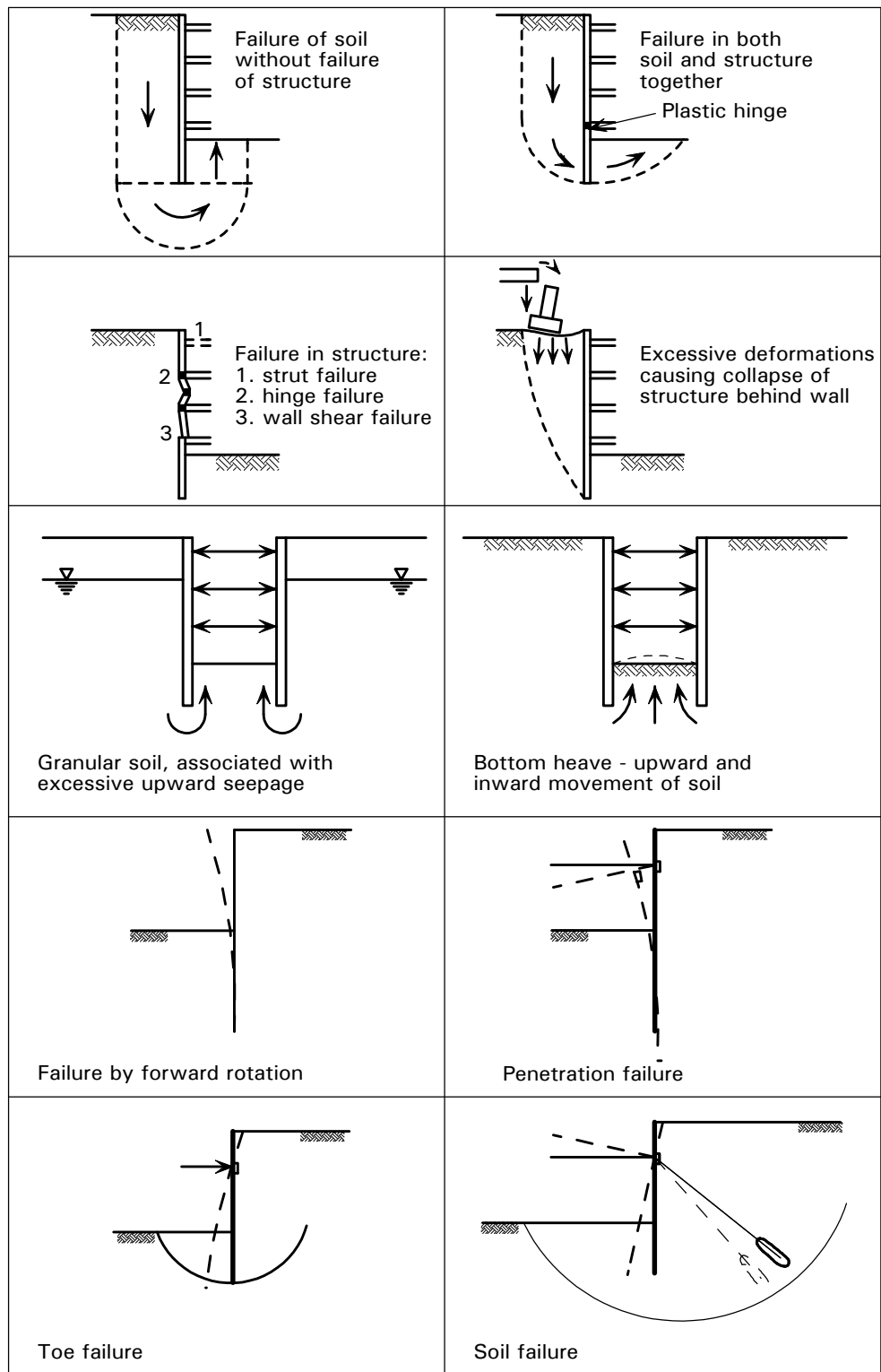
- surcharge overload
- temporary works damage
- burst water main.

## 5.6 Structural design

The structural design of the basement to a limit state philosophy can be performed based either on the use of British Standards, the Eurocodes or design guidance documents such as CIRIA Report 104 *Design of retaining walls embedded in stiff clays*<sup>[10]</sup>

If the geotechnical design of the basement has been based on the limiting equilibrium requirements of BS 8002 or CIRIA Report 104, it will be most appropriate to perform the structural design using British Standards. This includes design to the requirements of BS 5950-1 for the steel components of the basement such as the sheet pile wall, and if appropriate for the internal columns and temporary props, and BS 8110 for the design of the reinforced concrete components such as the basement floors.

Although the design factors in BS 8002 and CIRIA Report 104 are currently incompatible with those in BS 5950-1 and BS 8110, it is understood that in the case of BS 8002, this incompatibility will be resolved in Amendment No 2, which is to be issued in 2001. CIRIA Report 104 is also currently being revised and will also address the incompatibility.



**Figure 5.1** Possible modes of failure at Ultimate Limit State

Where it has been decided to undertake a limit state design of the basement to the requirements of the Eurocodes, the structural design of the basements can be undertaken using Eurocode 3 *Design of steel structures*, (in particular ENV 1993-5 *Piling*) for the sheet pile walls and Eurocode 2 *Design of concrete structures for the basement floors*.

It is important to remember that Eurocodes 2, 3 and 7 are currently pre-standards and as such will in all probability change as they progress to full Standard status. Care must therefore be taken in the use of the partial factors that apply at the time of the design.

Currently, BS 8002 and CIRIA Report 104 are not compatible for use with the Eurocodes.

## **5.7 Loading and design parameters**

Loading for each design case that is resisted by the retaining wall broadly comprises the following:

- soil weight
- earth pressures
- ground water and free water pressures
- seepage forces
- surcharge loads
- superstructure loads where appropriate (i.e. supporting building frame)
- horizontal and vertical loads on basement floors
- temperature.

Geometric parameters that are relevant include:

- level and slope of the retained ground surface
- levels of excavation
- characteristics of the geological model.

It is important that gross changes in levels are modelled directly and are not assumed to be included within any factor(s) of safety. For limit states, geometric parameters should represent the most unfavourable or worst credible values that could occur.

## **5.8 Behaviour and performance of retaining walls**

Practical experience with soils is required to judge appropriate parameters to model retaining wall behaviour during construction phases and in the final configuration. Uncertainties include:

- values of soil parameters
- rate of consolidation corresponding to the time taken for the soil to change from undrained to drained behaviour
- rate of softening on the passive side of the wall
- depth of soft and compressible deposits
- continuity, extent and pressures of water-bearing layers
- initial stress state in the ground
- pre- and post-construction water levels and drainage boundaries

- wall stiffness and sequence of construction
- existing and projected changes in superimposed loading.

These uncertainties can significantly affect the choice of support system, the installation method and the excavation sequence. There are no codes and standards which cover this judgement because of the complexity of interacting issues in addition to the geotechnical ones. Due to the inaccuracy of predicting earth pressure, there is a certain risk that has to be judged and this can either be accommodated by over-design or mitigated by on-site instrumentation and the monitoring of real wall behaviour (see Section 5.11 Observational Method). Over-design becomes a real penalty for medium to deep basements because it is uneconomic.

## **5.9 Predicting ground movement**

As basements are built to greater depths and building developments occupy greater plan areas, the effects of subsidence, heave and horizontal soil movement themselves become very important. Insurers are no longer prepared to cover risks of property damage that can be recognized, from previous experience, as inevitable; there is a need to predict ground movement by sound design and careful construction methods.

The following discussion addresses the factors which cause soil movement around a large basement excavation, the measures which can be taken to reduce it, and the methods available to predict it.

### **5.9.1 The effect of the method of construction**

The choice of construction method for the basement, either top-down or bottom-up, the technique used for walling or sheeting the basement periphery, and the period taken for each excavation stage, can all influence the extent of soil movement around the excavation.

However, the main causes of damage to adjacent buildings are from wall installation methods and problems associated with lowering of the groundwater level. Most wall movement and consequent soil movements tend to occur prior to the first prop being located. This is because the walls deflect as cantilevers until the first or top prop is inserted. The wall deflection can be reduced by:

- a) positioning the top prop as high as possible
- b) decreasing the vertical spacing between props (not applicable to top-down construction where the basement floors support the walls)
- c) using sheet piles with a greater stiffness.

Only minimal benefit is achieved if the stiffness of the propping system is increased or the props are pre-loaded (it is very difficult to push the wall back).

The top-down construction method is frequently used to reduce vertical soil settlements by using stiff concrete floors to successively prop the basement sheet pile walls. Wall and soil displacement can be minimised by regular propping at each storey height, the high stiffness of concrete floors and avoiding re-propping procedures. However, this may not be the case for all excavations. The

regularity of support provided by the floors to the exterior walling at each storey height may not necessarily provide support at optimum levels, especially where external surcharge loads are applied from, say, existing foundations to adjacent structures or where the height from the penultimate support to formation level must be minimized, (i.e where soft or weak strata exist immediately below final formation).

Some contractors prefer to construct the external walls and excavate to first basement floor level without casting the ground floor. This procedure loses some advantage of the top-down method in restricting ground movements. Potts and Fourie<sup>[11]</sup> concluded from the results of numerical analysis, that the use of temporary soil berms to support cantilevered external walls from ground floor to first basement floor level, partly reduces the additional settlement caused by excavation to first basement level.

For basements where soil conditions permit a cut-off against groundwater ingress, the top-down method of construction may prove attractive in reducing external soil settlement. The method has disadvantages, however, including:

- higher excavation costs (to remove soil from below basement floor construction)
- the risk of overall delay that can be caused by any local hold-up in a sequence of interdependent construction activities
- the constraints in terms of space and access for the numerous different activities on site at the same time.

Soil berms can be used to minimize lateral movements of the sheet pile wall at the periphery of an excavation, because the increase in vertical stress using a relatively small volume of soil is often sufficient to reduce lateral movements by 50%. Placement of props as the berms are removed in short lengths can minimise final lateral wall movement and vertical settlement of soil outside the excavation.

### **5.9.2 Methods of predicting ground movements**

Two methods are available to estimate ground movements. These are:

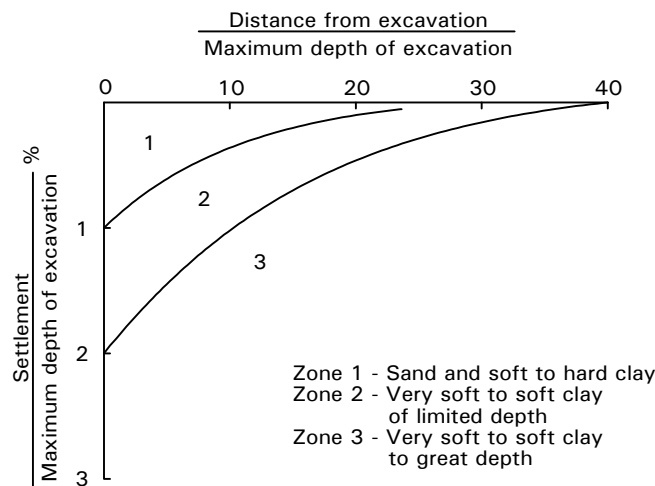
- a) using data from previous field observations and case histories
- b) prediction using numerical methods.

#### ***Field observations and case histories***

Settlements near excavations can be predicted from published measurement data pertaining to sites in similar soil conditions.

Ground movements depend on a number of factors, of which the most important are the height of the excavation and the prevailing soil type. Dimension-less settlement profiles can be used to determine the distribution of movement behind a wall.

Observations of vertical ground movements around a number of excavations in various soil types have been summarised in graphical form by Peck<sup>[12]</sup> and are illustrated in Figure 5.2.



**Figure 5.2** *Dimensionless settlement profiles (Peck)*

Where more detailed information is required on construction-induced ground movements, the databases produced by Clough and O'Rourke<sup>[13]</sup>, and St John *et al.*<sup>[14]</sup> can be used. Recently, this work has been extended by Fernie and Suckling<sup>[15]</sup>. These data are based on case histories for excavations up to 25 m in depth in sands, stiff clays, soft clays and overconsolidated clays.

The software analysis program for retaining wall design, ReWaRD<sup>® [16]</sup> uses the above databases to provide improved empirical expressions for wall and soil movements. Data for construction in sands, stiff clays and soft clays are included. For a more detailed background to the development of these expressions for soil and wall movement, reference should be made to the ReWaRD Reference Manual.

### ***Numerical methods***

The complex behaviour of the soil and structure during basement construction can be modelled much more realistically using finite element or finite difference methods. Two-dimensional plane strain solutions are more commonly undertaken using one of the many commercially-available computer programs. The software can model site conditions and enable sequential construction simulation as the excavation progresses. Displacements are solved implicitly within the mesh to provide prediction of horizontal and vertical movements. Further details of these modelling techniques can be found in Section 8.2.2.

## **5.10 Data for design**

The design of a retaining wall for a basement requires information on the physical conditions in the vicinity of the structure. This information includes the topography and layout of the site, the nature of the ground, the water conditions, and details of adjacent foundations. A site investigation enables this information to be collected and soil properties to be determined by testing.



### 5.10.1 Ground water

Information is required of ground water levels and seepage pressures at the site, including that on existence of any hydrostatic uplift pressures. Consideration should be given to potential changes in ground water level due to the presence of the proposed basement.

### 5.10.2 Soil properties

Data on the soil properties in respect of both strength and stiffness under both drained and undrained conditions may be required. Soil properties relating to strength include:

- saturated and unsaturated bulk densities (unit weight) and moisture content
- undrained ( $c_u$ ) and drained shear strength including angle of shearing resistance and cohesion intercept ( $\phi'$ ,  $c'$ )
- soil classification properties i.e. plasticity index, grain size, etc.

Stiffness related soil properties include:

- Young's Modulus
- Poisson's Ratio
- Coefficient of Horizontal Subgrade Reaction
- Over-consolidation ratio (OCR)
- Initial coefficient of earth pressure at rest ( $k_0$ )

Sources where relationships for Young's Moduli are given include Bowles<sup>[17]</sup> and Borin<sup>[18]</sup> and coefficients of horizontal subgrade reaction for typical soils are recommended by Terzaghi (1955)<sup>[19]</sup>.

It is important to be aware that appropriate parameters can vary dependent upon the mechanism or mode of deformation being considered. For sheet pile walls, strain levels and compatibility need to be considered in the assessment of strengths for the materials through which a presumed failure surface can occur. Ranges of values may also be required, particularly if the soil properties are likely to change across the site.

Both BS 8002 and CIRIA Report104<sup>[10]</sup> are particularly useful for reference on good practice in determining soils data for design and provide guidance on values of soil parameters.

#### ***Determination of design soil properties***

The determination of design values for soil properties from data obtained in a site investigation is the responsibility of the designer. However, to many this process can appear to be a 'black art'. This is because of the difficulty in finding practical advice that lays out the thought processes necessary to derive design values of soil parameters with an appropriate degree of conservatism. The subject is to a large degree empirical and it is not simple to convey engineering experience in design rules where there are so many factors involved.

Described below are the formalised procedures that need to be performed to obtain design values for soil properties. It should be noted that design values obtained will be different depending on the particular Code of Practice, Standard or design

guidance document that is being used for the design as the definitions of soil properties differ between these documents. For more clarity the differences are described below.

*Soil parameters based on CIRIA Report 104*

CIRIA Report 104 gives definitions for soil properties to be used in design, but data are presented only for stiff clays. The uncertainty involved in the selection of soil strength parameters for stiff clays is considered by stating two distinct definitions for soil properties. These two definitions are termed *moderately conservative* and *worst credible* soil properties.

The definitions given for these terms in CIRIA Report 104 are as follows:

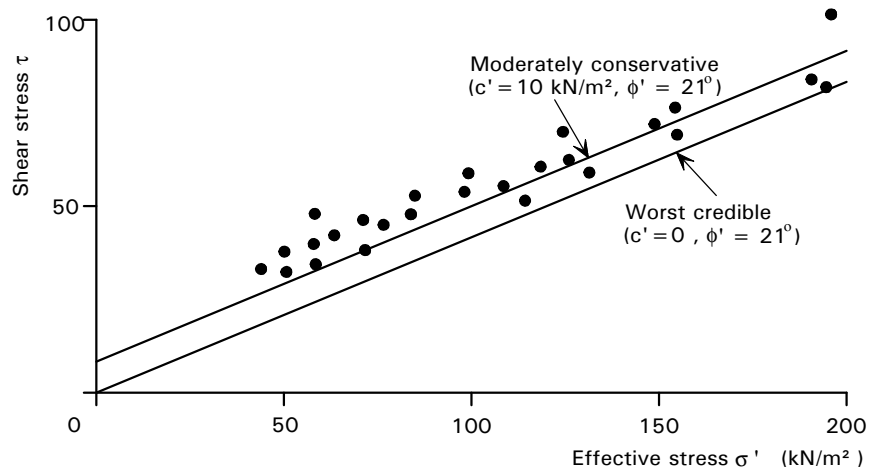
“A *moderately conservative* value for a soil parameter is defined as a conservatively best estimated value. It is the most commonly used value in practice by experienced engineers”.

“A *worst credible* value for a soil parameter is the worst value that the designer could realistically believe might occur and in most cases for retaining wall design is the most pessimistic value that is very unlikely to be any lower”.

*Moderately conservative* design parameters are a cautious assessment of the value of a parameter, worse than a probabilistic mean value (referred to as *most probable* (most likely to occur), in the ‘Observational Method’) but not so severe as the *worst credible value*.

Although the *moderately conservative* soil parameter is widely used, the *worst credible* soil parameter has the advantage in producing a lower bound solution for design.

The difference between the two, for a typical set of soil test results, is shown in Figure 5.3.



**Figure 5.3** Representation of moderately conservative and worst credible soil parameters

### *Soil parameters based on BS 8002*

In BS 8002 the default values for soil parameters are presented for both granular and clay soils. Soil properties are based on *representative* values and are defined as “conservative estimates of the properties of the soil as it exists in-situ”. In this context, *conservative* is defined further in BS 8002 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 and they tend towards the limit of the credible range of values.*”

Simpson and Driscoll<sup>[20]</sup> suggest that *representative* values are essentially the same as the *moderately conservative* values defined in CIRIA Report 104 and the *characteristic* values in ENV 1997-1.

BS 8002 requires that representative values of both the peak and critical soil strength parameters  $\phi_p'$  and  $\phi_{crit}'$  be obtained, as they provide a measure of the strength of the soil at different soil strains (at serviceability and ultimate limit states respectively). Further information is presented in both BS 8002 and in CIRIA Report 104.

Where in-situ parameters are obtained with confidence (results that show little variation), the *representative* value can be the mean value, e.g. soil density. Where greater variations occur and confidence is not as good, then the *representative* value will be a cautious assessment of the lower or upper bound of the acceptable data, dependent upon the purpose.

### *Soil parameters based on Eurocode 7*

Characteristic values of geotechnical parameters are fundamental to all calculations performed in accordance with the code. They are based on an assessment of the material actually in the ground and the way that material will affect the performance of the ground and structure in relation to a particular limit state (ENV 1997-1, Clause 2.4.3).

For materials such as steel and concrete, characteristic values can be defined from a probabilistic analysis of tests on samples. For soils, because of variability, this is not easily possible and hence in Eurocode 7, a ‘*characteristic value*’ is defined as a “*cautious estimate of the value affecting the occurrence of the limit state*”. In practice, the ‘*characteristic value*’ is broadly comparable with a ‘*Moderately Conservative*’ value as defined in the CIRIA Report 104.

## **5.11 Observational method**

The Observational Method involves making a best estimate of geotechnical behaviour in conjunction with the formulation of contingency plans for additional measures to be taken if the actual behaviour deviates from predictions by an unacceptable margin. In the construction industry, increasing emphasis is being placed on the value of the Observational Method (see Peck<sup>[21]</sup>) whereby immediate feedback from instrumentation monitoring of retaining wall behaviour is used to modify the design and construction procedures according to a pre-determined plan.

In geotechnical engineering, the current state of the art is such that predictions of wall and pile displacements are subject to a considerable degree of uncertainty. One of the reasons for this is the difficulty in predicting the soil response to

structural loading from a limited number of tests on soil samples, coupled with the general lack of correlation from monitoring of real basement structures.

The Observational Method is recommended in the ENV-1997-1 Eurocode 7. It states that if this method is to be used it is imperative that the following requirements are met before start of construction:

- Acceptable limits of behaviour are established.
- The range of possible behaviour lies within the acceptable limits.
- A monitoring plan is set up that shows whether the actual behaviour lies within the acceptable limits.
- A contingency plan is available if the actual behaviour is outside the acceptable limits.

For small- and medium-sized structures, the wall displacements will be small and the inherent uncertainties are normally catered for by adopting conservative values of soil properties, in design (see Section 5.10.2). For larger and more complex structures, however, any over-conservatism may lead to unacceptably high costs.

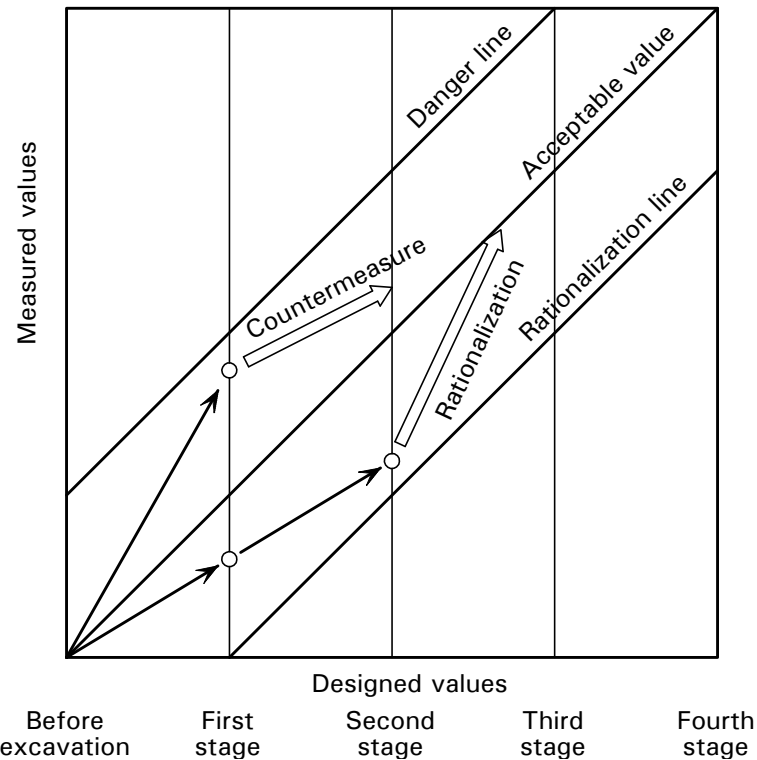
For further detailed explanation of the Observational Method the reader is referred to CIRIA Report 185 <sup>[22]</sup>.

An Observational Method applicable for basements uses the knowledge gained at the early stage of an excavation, to modify the excavation sequence and temporary support design, during later stages. The observations at the early stage of a basement excavation can be used to confirm the geology or the behaviour of the new structure or adjacent buildings. The information can be used to calibrate design parameters, hence allowing the excavation sequence to be modified before progressing to a deep level. This procedure was used for the construction of London Underground's Bermondsey Station, see Dawson *et al.* <sup>[23]</sup>.

The application of the Observational method for a multi-stage excavation has been undertaken by Ikuta *et al.* <sup>[24]</sup>. The project comprised a top-down construction of a deep basement in Tokyo. The method used was to define three "lines of behaviour", as shown in Figure 5.4. If the behaviour approached the "danger line", planned countermeasures would be instigated. If the behaviour approached the "rationalization line", subsequent stages of construction could be modified to make construction more cost and time effective.

The 'danger line' used by Ikuta is equivalent to the 'red trigger' as defined in CIRIA Report 185. This is normally governed by health and safety regulations, the damage criteria set by the owners of adjacent properties or the serviceability limit of the project structure at the final construction stage. The 'acceptable line' represents the desired factor of safety and is likely to be governed by health and safety regulations and economic criteria.

Several deep retaining wall structures have been built using the Observational Method; these include the A406 underpass at Neasden, the Limehouse Link tunnel and the A4/A46 Batheaston-Swainswick bypass, Bath.



**Figure 5.4** *The Observational Method of a deep basement construction in Tokyo (Ikuta et al., 1994)*

## 5.12 Design for driveability

A driveability assessment is required once a sheet pile section and length that is adequate for the predicted structural forces has been selected. It is important to check that the pile section chosen is capable of withstanding the rigours of driving and will reach the desired penetration in a condition suited to the application for which it is intended. The strength of the pile in driving is a function of its width (interlock spacing), the thickness of the pan and web and its moment of inertia. A term that takes all these factors into account is the section modulus (expressed as  $\text{cm}^3/\text{m}$ ) and relationships have been developed which relate this property to the density of the ground to be penetrated. Relationship tables are presented in the British Steel Piling Handbook. However it is important to recognise that these relationships should only be treated as the preliminary process in hammer sizing and that they are not intended to be a substitute for engineering experience or local knowledge.

Generally, the driving capability of sheet piles increases with the section modulus and can also be improved by specification of a higher grade of steel.

When the soil to be penetrated is hard, the pile section required for structural purposes will be lighter than that required for driving. This does not need to be a penalty as the additional steel can be considered as sacrificial, enhancing the corrosion performance of the steel piles.

### 5.12.1 Driveability of sheet piles in granular soils

Vibratory driving is the most effective means of installing piles in granular soils but this method may not be efficient when Standard Penetration Test N values exceed approximately 50 blows. In these conditions, it will be necessary to either treat the ground with pre-boring or water jetting or to adopt impact driving. Care must be taken in the design as changes to the soil properties adjacent to the pile wall may result from ground treatment measures.

### 5.12.2 Driveability of sheet piles in cohesive soils

Impact driving is the most efficient means of installing sheet piles in cohesive materials. When noise and vibration are issues, pile jacking techniques are available. In either case, it is essential that an appropriate pile section is adopted for the ground conditions present. If the section selected is too light, installation stresses may cause the pile to be damaged or become misaligned. A guide to selecting a pile section size in cohesive soil is given in BS 8002 and in the British Steel *Piling Handbook*. A table summarising the guidance is given in Table 5.1.

**Table 5.1** *Selection of sheet piles in cohesive soils - British Steel Piling Handbook*

Clay description	Minimum wall modulus (cm <sup>3</sup> /m)	
	Grade S270GP	Grade S355GP
Soft to firm	450	
Firm	600 - 700	450 - 600
Firm to stiff	700 - 1600	600 - 1300
Stiff	1600 - 2500	1300 - 2000
Very stiff	2500 - 3000	2000 - 2500
Hard ( $c_u > 200$ )	Not recommended	4200 - 5000

**Note.** The ability of piles to penetrate any type of ground depends on attention being given to good pile practice.

## 5.13 Design for durability

For design, it is important that the long-term performance of the structure is considered both in the choice of structural form and in the design of construction details. Failure to do so may result in maintenance problems requiring costly repair.

### 5.13.1 Corrosion allowances

The means for countering the effect of corrosion of steel piles are well developed. Guidance is given in the British Steel *Piling Handbook*.

BS 8002 considers that the end of the effective life of a steel sheet pile occurs when the loss of section, due to corrosion, causes the stress to reach the specified minimum yield strength. A pile section chosen for the in-service condition has to be adequate at its *end-of-design-life*, at which time the effective pile section will have been reduced by corrosion.

As the corrosion loss allowance varies along the pile according to the corrosion environment, the designer needs to be aware that the maximum corrosion may not

occur at the same level as the maximum forces and moments, and should allow for this accordingly.

Also, since redistribution of earth pressures may occur as a result of increased flexure of a corroded section, the *end-of-design-life* condition may be a critical design load case in the selection of the sheet pile section.

### 5.13.2 Corrosion and protection of steel piles

The design life requirements for proposed buildings and individual components or assemblies are defined in BS 7543 *Guide to durability of buildings and building elements*, where a building design life can range from 10 years for a building with a ‘short’ life to 120 years for civic and other high quality buildings. A basement structure must therefore comply with these requirements and be designed with sacrificial thicknesses applied to each surface, depending on the exposure conditions. The exposure conditions are based on the advice given in BS 8002: Clause 4.4.4.4.3 and are shown in Table 5.1.

**Table 5.2** *Sacrificial thicknesses for piling according to BS 8002*

Exposure zone	Sacrificial thickness (for one side of the pile only) mm/year
Atmospheric	0.035 (mean)
Continuous immersion in water or effluent	0.035 (mean)
In contact with natural soil	0.015 (max)
Splash and alternating wet/dry conditions	0.075 (mean)

The reduced (corroded) section properties can either be obtained by calculation or from the British Steel *Piling Handbook*.

Another way of allowing for sacrificial thickness is to use a higher strength steel than would be required if no corrosion were assumed (i.e. use steel grade S355GP, to BS EN 10248, in a wall designed for steel grade S270GP). This permits a greater loss of metal before stresses become critical.

It should be noted that the corrosion allowances apply to unprotected steel piles. Although it is generally cost effective to provide the sacrificial steel thickness, consideration can alternatively be given to the following corrosion protection options:

- Protective coatings, particularly in the exposed section of the pile.
- Cathodic protection in soil below the water table or in a marine environment.

Details of these options are given in the British Steel *Piling Handbook*.

#### ***Corrosion in fill or industrial soils***

Buildings can be constructed in areas of recent fill or *industrial soils*. Corrosion protection of the steel in contact with the fill material may be required, and this can be assessed by testing the material for pH and resistivity.

The nature of in-situ fill soils can be variable, and a full soil analysis is required to assess the likely corrosion performance of steel in the environment. Soil tests to determine the pH of the soil should be in accordance with BS 1377-3 and as directed by the Contract to determine resistivity. Other tests may be relevant, and most of these are reviewed in CIRIA's series of reports on contaminated land (contact CIRIA for further details).

In a *controlled fill*, no special measures are required, and the same corrosion rates as in natural undisturbed soils can be assumed.

Corus (formerly British Steel) has undertaken significant research and development into corrosion of steel and corrosion protection. Further advice on corrosion assessment and protection can be obtained from Corus Construction Centre or from The Steel Construction Institute.

### **5.13.3 Corrosion and structural forces**

It is not immediately obvious whether the start of in-service life case or the end-of-design-life case will be the most critical for the structural design of retaining walls. At the end-of -design-life, the reduced stiffness of the corroded steel pile will permit increased deflection that will in turn reduce the soil pressures acting upon it (and therefore the induced moments and shears).

As the corrosion loss allowance varies along the pile according to the corrosion environment, the designer should consider that the maximum corrosion may not occur at the same level as the maximum section stresses.



## **6 DESIGN PROCEDURE FOR BASEMENTS**

This Section provides a procedure for the geotechnical analysis and structural design of steel intensive basements, following the design aspects explained in the Sections 7 to 13. Each aspect of analysis and design is addressed and a sequence in which these activities should be undertaken to design most effectively is presented.

### **6.1 Design procedure**

A well-structured procedure for analysis and design is important because it enables the designer to focus not only on the immediate activity at hand but also what is to be achieved in the overall design. Guidance given today tends to be less prescriptive than in the past. This allows the designer to think, rather than rigidly applying a set of prescribed instructions.

A structured procedure for the design of a basement is presented in the flow chart on Figure 6.1. It is a 'high level' flowchart, showing the main activities and the order in which they should be performed. No detailed breakdown is given of the activities, but a brief comment on each is given in Section 6.2, including reference to Sections of this publication that offer detailed advice.

### **6.2 Activities in the design procedure**

#### **6.2.1 Categorise basement type**

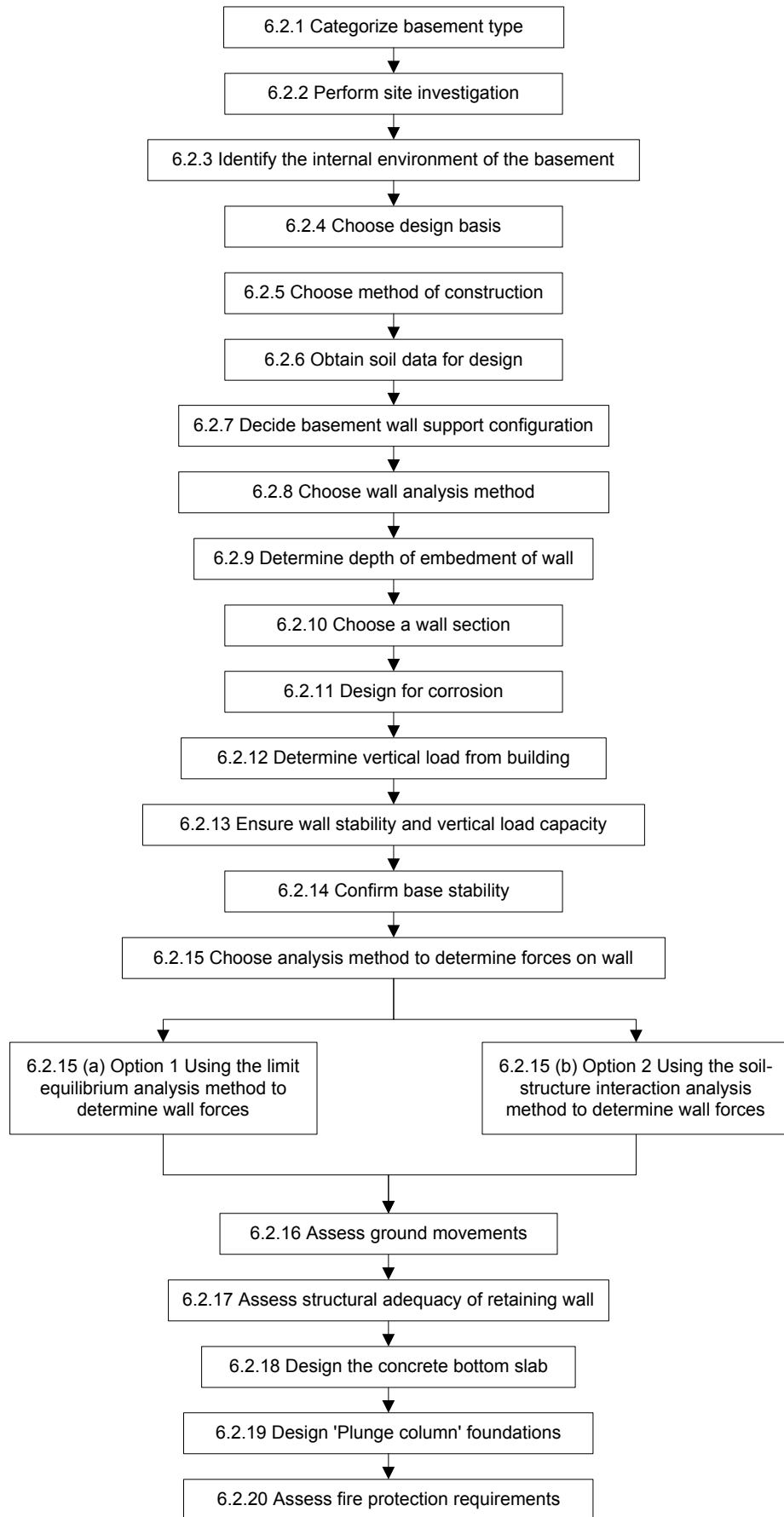
A basement may be required for a domestic dwelling, a commercial office building with a shallow or part basement only, or for a commercial complex with a deep basement (one which has two or more floors).

The structural engineer will therefore first have to identify the type of basement that is to be designed, as it will affect the complexity of the design and also the extent of input that will be required from a geotechnical engineer. Further information on basement types is given in Section 5.1.

#### **6.2.2 Perform site investigation**

It is necessary to consider the site location and how the physical factors such as soil behaviour and the water regime will affect construction of the basement and its support. The importance of the site investigation, cannot be stressed too highly.

The basement may be built on 'competent' ground where soil structure interaction effects do not pose any significant problem in design. In this case the geotechnical aspects of design may be performed by a structural engineer. However, if the basement is to be built on 'poor' ground where geological discontinuities, soil behaviour, or the water regime, create complexities in design, it is important that the structural engineer seeks assistance of an experienced



**Figure 6.1** *Design procedure for basements*

geotechnical engineer to critically analyse the findings of the site investigation and undertake the geotechnical design.

If the site investigation does not reveal any major geotechnical problems, the structural engineer can refer to Section 5.8 for general design advice on the behaviour and performance of retaining walls.

### **6.2.3 Identify the internal environment of the basement**

The internal environment of the basement must be identified in order to satisfy the performance requirements stated in the Building Regulations. Only once the internal environment is known, can the designer be able to define how this performance is to be quantified and how the measures necessary to achieve the internal environment will be implemented.

Timely identification of the internal environment of the basement is crucial; changes to the internal environment at a later stage are likely to have significant effect on the overall design of the basement.

Information regarding internal environments for basements and in particular their implication on providing water-resisting construction for the walls and the floors is discussed in Section 5.2 and Section 11.

### **6.2.4 Choose design basis**

In most cases, a Limit State design basis will be used to design a basement. The designer may refer to Section 5.3 and 5.4 for guidance on the selection of the most appropriate documents which will be consistent in providing the most efficient and safe basement.

### **6.2.5 Choose method of construction**

Various methods of construction can be used for basements. For basement construction in urban areas, the implications of each construction method considered will have to be investigated thoroughly. Before a method of construction is chosen it will be necessary to assess the effects that this construction will have on adjacent buildings. This primarily relates to potential damage that can occur to an adjacent building due to soil movements resulting from soil excavation and water regime changes.

In these situations, methods of construction that have proved superior are those that use permanent embedded retaining walls. In addition, methods where propping has been achieved by the use of basement floors have also proved successful, as temporary propping works have been minimised. This has resulted in the preferred use of 'top-down' construction methods as opposed to 'bottom-up' construction. Section 4 provides guidance on the methods available.

For further guidance actual basement case studies using steel sheet piles are presented in Section 2, with the technical and economic benefits of the use of steel piles and aesthetic considerations being discussed in Section 3.

For the larger and more complex basement projects where uncertainty in soils data can lead to over-conservatism and unacceptably high costs for the basement construction, the designer may consider the Observational Method for basement design and construction. This method is described in Section 5.11.

### **6.2.6 Obtain soil data for design**

The underlying importance of a thorough site investigation to obtain soil data is emphasised, and a detailed explanation relating to the selection and evaluation of soil parameters for design are given, in Section 5.10.

### **6.2.7 Decide basement wall support configuration**

Basement wall support configuration is dependent primarily on the depth of excavation, the soil properties and the water regime. Two wall support configurations are most appropriate for basements; one configuration being a wall supported by a cantilever or single-prop and the other a multi-propped wall. The choice of wall support configuration most appropriate for design can be made based on engineering judgement using experience gained on previous retaining wall designs, or by performing concept studies of each wall support configuration and deciding on merit which is the best configuration.

### **6.2.8 Choose wall analysis method**

Guidance relating to the analysis of single-propped walls is given in Section 7. Section 8 provides guidance for the analysis of multi-propped walls.

### **6.2.9 Determine depth of embedment of wall**

#### ***Single-propped walls***

The limit equilibrium methods that are commonly used to check overall wall stability and to calculate the required depth of embedment are presented in Section 7.3. These methods are based on the representation of simplistic soil pressure profiles as discussed in Section 7.1. Water regime and corresponding pressure estimation is discussed in Section 7.2

#### ***Multi-propped walls***

Due to the inherent redundancy of multi-propped wall configurations, overall wall instability does not occur. However, wall toe instability can occur, and must be prevented. The determination of the depth of penetration of the wall with an inherent and appropriate factor of safety against rotational failure requires that the lowest span of the wall is treated as a single-propped wall. Reference should be made to Section 7.3 to consider 'stability'.

### **6.2.10 Choose a wall section**

#### ***Standard section sizes***

Pile section sizes are listed in Appendix A. More detailed information is given in the British Steel *Piling handbook*. Information on material for steel sheet piles is given in Section 5.4.3.

#### ***Satisfying pile driving requirements***

Minimum size requirements for successful pile driving are presented in Section 13.4. Tables 13.2 and 13.3 are applicable for sheet piles in cohesionless soils and cohesive soils respectively. For High Modulus Piles and box piles, guidance is given in Section 13.4.3.

### **6.2.11 Design for corrosion**

Both uncorroded and corroded section properties need to be known to design a steel sheet pile wall. Corrosion allowances to be used in design are given in Section 5.11. Reduced section properties allowing for corrosion are either calculated or obtained directly from the British Steel *Piling handbook*.

### **6.2.12 Determine vertical load from building**

Any building superstructure loads acting on the wall should be obtained from the building structure analysis. Normally, unfactored loads for each of their individual components (i.e. dead, live load, etc.) need be obtained.

### **6.2.13 Ensure wall stability and vertical capacity**

The retaining wall has to resist vertical loads that act on the pile from the building superstructure. Comprehensive information regarding the vertical resistance and capacity of a retaining wall is given in Section 7.4. This aspect of design applies to both single-prop and multi-prop retaining walls.

Commonly, the easiest solution to satisfy both lateral stability and vertical capacity requirements is to increase the depth of embedment. If the depth of embedment cannot be increased, then an alternative pile configuration may be required.

### **6.2.14 Confirm base stability**

There are several modes of instability that have to be prevented. The modes are related to potential base heave or hydraulic failures. Both are discussed in Section 9.

### **6.2.15 Choose analysis method to determine forces on wall**

Two distinct methods can be used to determine structural forces acting on the wall and the prop(s). In the case of a single-prop retaining wall, the choice of method is made by the designer, taking into consideration the implications of Section 7.5. In the case of a multi-prop wall, reference should be made to Sections 7.5 and Section 8.

Where temporary propping is to be used for construction (predominantly for bottom-up construction), the reader is referred to Section 10 for guidance in determining prop forces.

#### **6.2.15 (a) Use the limit equilibrium method to determine wall forces**

##### ***Single-prop wall***

The limit equilibrium methods used to determine bending moments, shear forces, and prop forces are discussed in Section 7.5.2. The method includes modification factors that may need to be applied to structural forces acting on the wall to take into account the effects of method of construction, in-situ soil stress, and wall stiffness on structural forces. The internal forces in the retaining wall have to be determined both with uncorroded and with corroded section properties, as wall stiffness are different. Corrosion allowances are given in Section 5.11.1.

### ***Multi-prop walls***

The simplistic limit equilibrium methods that can be used to determine bending moments, shear forces, and prop forces are described in Section 8.1. As for single-prop walls, forces will have to be determined with uncorroded and corroded section properties. See Section 5.11.1.

### **6.2.15 (b) Use the soil-structure interaction method to determine wall forces**

The soil structure interaction methods used to determine bending moments, shear forces, and prop forces are discussed in Section 7.5.1 for single-prop walls and Section 8.2 for multi-prop walls. The construction sequence, in-situ stress, and wall stiffness are taken into account directly by the underlying theory.

As for the limiting equilibrium methods, the internal forces in the retaining wall have to be determined both with uncorroded and with corroded section properties, as wall stiffness are different. See Section 5.11.1. for corrosion allowance data.

### **6.2.16 Assess ground movements**

Estimates of wall displacements and ground movements can be made either by accessing databases containing information specific to field measurements during past construction projects, or by performing more complex analysis using computer assisted techniques, for example finite element methods. The reader is referred to Section 5.8.

### **6.2.17 Assess structural adequacy of retaining wall**

The structural adequacy of the wall and its props has to be checked for all loading combinations. See Section 5.6 for structural design aspects, Section 5.3 for limit state philosophy and Section 5.4 for information on design standards.

### **6.2.18 Design the concrete bottom slab**

The bottom slab of the basement should be designed not only to resist the loads that are acting on it but also to be resistant to water penetration. The level of resistance required depends on the proposed internal environment of the basement.

The designer is referred to Sections 9.2 and 11.7 for guidance on ground slab design. Typical construction details for slab to wall connections are shown in Section 11.8. For the structural adequacy of the basement slabs the reader is referred to Section 5.6.

### **6.2.19 Design slab supports**

For top-down construction where support to the floor slabs are by ‘plunge column’ foundations, see Section 4.1.1 for guidance on design aspects relating to ‘plunge columns’.

### **6.2.20 Assess fire protection requirements**

Fire protection requirements for basements are discussed in detail in Section 12.

## 7 DESIGN OF CANTILEVER AND SINGLE-PROP WALLS

This Section describes the methods that can be used to design cantilever and single-prop walls.

The design objective is to prevent the limit states being exceeded (at a limit state, the effects just equal the resistance, for economic design). The objective is realised by:

- determining an adequate depth of embedment of the wall to satisfy stability considerations and limit soil movements
- satisfying vertical load resistance requirements
- choosing a section size for the wall which will resist all the external loads that may act on it.

The design process involves first determining the loads on the wall (soil pressures, water pressures, vertical loads and surcharges, etc). Knowing the loads acting on the wall, the depth of embedment of the wall is calculated and the wall is checked to ascertain its capacity to resist vertical loads. Having determined the length of the wall calculations are performed to obtain the internal forces (bending moments, shear forces and forces) acting in the wall.

Both the simplified limiting equilibrium and the soil structure interaction design methods are discussed.

### 7.1 Simplistic representation of soil pressures

The limit equilibrium method assumes that the limiting pressures in the soil are fully mobilised. The method has been applied extensively to the analysis and design of embedded retaining walls for many years, and prior to the introduction of numerical techniques using computers, was the only method available. It is based on the classical theory of soil mechanics, where simplistic assumptions are made for the distribution of lateral earth pressures with depth.

The concept of an earth pressure coefficient  $k$  is used to describe the state of the stress in the soil. The earth pressure coefficient is defined as:

$$k = \frac{s'_h}{s'_v}$$

where:  $\sigma'_h$  is the effective horizontal stress  
 $\sigma'_v$  is the effective vertical stress.

There are three states of stress for a soil, the at-rest, and the limiting conditions of active and passive states, termed  $k_o$ ,  $k_a$ , and  $k_p$  respectively.

### 7.1.1 At-rest earth pressure profiles

In an undisturbed soil with a horizontal ground surface, the horizontal pressure at any depth is given by

$$s'_h = k_o s'_v$$

where  $k_o$  is the at-rest pressure coefficient

For normally consolidated soils that have not been subjected to removal of over-burden or to actions that have resulted in lateral straining of the ground,  $k_o$  can be obtained from an approximate expression developed by Jaky<sup>[25]</sup>,

$$k_o = 1 - \sin \phi'$$

where  $\phi'$  is the effective angle of shearing resistance of the soil.

For lightly over-consolidated clay, a coefficient  $k_{o,oc}$  can be obtained from the relationship given by the Canadian Geotechnical Society<sup>[26]</sup>.

For soils with complex stress histories, the distribution of  $k_o$  with depth should be investigated carefully (Burland *et al.*<sup>[27]</sup>). For example at shallow depths in a heavily over-consolidated clay,  $k_{o,oc}$  can approach the passive earth pressure coefficient  $k_p$  and values of  $k_o$  of 2 to 3 are common.

### 7.1.2 Active and passive earth pressure coefficients

Commonly quoted active and passive earth pressure coefficients  $k_a$  and  $k_p$  in codes of practice and design manuals include those from Kerisel and Absi<sup>[28]</sup>, Caquot and Kerisel<sup>[29]</sup>, and Sokolovsky<sup>[30]</sup>.

### 7.1.3 Lateral earth pressure representation

The analytical expressions that use the limit equilibrium approach assume that the stresses at limiting active and passive states increase linearly with depth. Figure 7.1 shows the assumed lateral earth pressure distribution for a cantilever and a propped embedded wall.

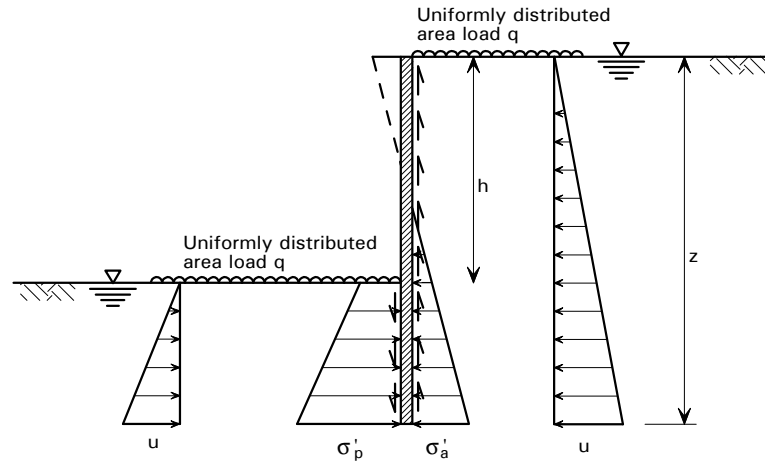
For long-term, drained effective stress analysis, the effective horizontal active and passive earth pressure equations are given in generalised form by:

$$\sigma'_a = k_a (\gamma z - u + q) - c' k_{ac}$$

$$\sigma'_p = k_p (\gamma z - u + q) + c' k_{pc}$$

where:  $\sigma'_a$  is the effective active pressure acting at a depth in the soil  
 $\sigma'_p$  is the effective passive pressure acting at a depth in the soil  
 $\gamma$  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 effective shear strength of the soil.





**Figure 7.1** *General earth pressure distribution*

$k_a$ ,  $k_p$ ,  $k_{ac}$ , and  $k_{pc}$  are earth pressure coefficients, the values of which depend on  $c'$ ,  $\phi'$ ,  $c_w$ ,  $\delta$ , and  $\beta$ ,

where:  $c_w$  is the wall adhesion  
 $\delta$  is the soil/wall friction  
 $\beta$  is the slope of retained surface.

See Figure 5.7 for diagrammatic representation of earth pressures.

The generalised form of  $k_{ac}$  and  $k_{pc}$  is obtained from BS 8002 or CIRIA 104.

The total horizontal active and passive earth pressures acting against the wall are given by:

$$\sigma_a = \sigma'_a + u \quad \text{and} \quad \sigma_p = \sigma'_p + u$$

where  $u$  is the pore water pressure.

For a short-term, undrained total stress analysis the generalised horizontal active and passive earth pressures are reduced to:

$$\sigma_a = (q + \gamma z) - c_u k_{ac}$$

$$\sigma_p = (q + \gamma z) + c_u k_{pc}$$

where:  $k_a$  is taken to be 1.0  
 $k_p$  is taken to be 1.0  
 $c_u$  is the undrained shear strength  
 $c_w$  is the wall adhesion.

The above is only a summary of the limiting equilibrium method. Further information can be obtained from CIRIA Report 104 or from most geotechnical text books.

## 7.2 Water pressures

Groundwater forces exerted on a retaining wall can often be greater than those from the soil. Careful consideration therefore needs to be given to the variation of water levels and pressures acting on each side of the wall. This applies to both the temporary case (i.e. during construction) and the permanent case. Consideration should also be given to the effects of water seepage, where water flows around the base of the wall into the excavation. This water seepage through the ground will affect the value of pore water pressures and may reduce significantly the value of passive resistance of the soil in front of the toe of the wall and increase the active soil load.

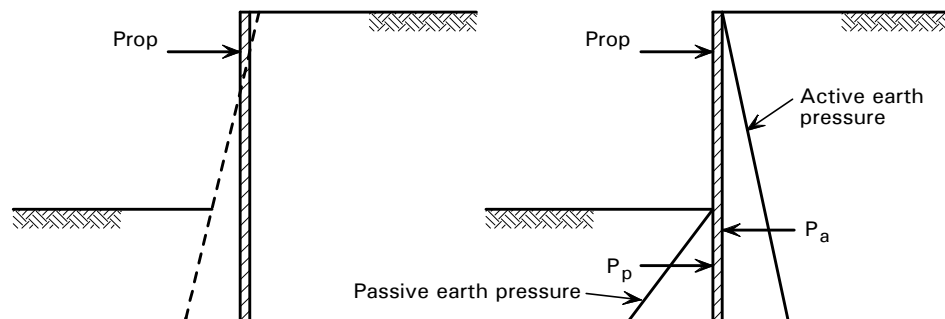
## 7.3 Wall stability and depth of embedment

For a retaining wall, the governing criterion for stability or security against overturning of the wall is one of moment equilibrium. Although other possible failures may occur, they are much less likely than that of overturning. In certain cases (particularly for waterfront structures or in sloping ground), a check should be made to ensure that a deep-seated slip plane (passing behind and below the wall) does not develop.

A minimum required depth of embedment for the wall may be obtained from the equation defining moment equilibrium; restoring moments should exceed overturning moments by a safety margin that is stated in the relevant Codes of Practice or Standards.

The method used by CIRIA 104 and BS 8002 to determine the depth of embedment for a cantilever wall assumes that the toe of the wall is fixed.

For a propped wall, it is assumed that there is sufficient embedment of the wall to prevent horizontal movement but rotation can still take place at the toe. Consequently, the wall is assumed to rotate as a rigid body about the prop. The wall/prop system is assumed to move far enough to develop active pressure in the retained soil (see Figure 7.2).



**Figure 7.2** *Free-earth boundary condition for a single-prop wall*

The required depth of embedment is determined by equating moments about the prop, assuming fully mobilised active and passive earth pressures (expressed as resultant forces  $P_a$  and  $P_p$ ), as shown in Figure 7.2. Consideration of horizontal equilibrium allows the necessary prop force to be calculated.

### 7.3.1 Design methods

For design, the moment equilibrium condition is used directly or indirectly to ensure that restoring moments exceed overturning moment by the required safety margin. This is achieved either by the use of partial factors (applied to the soil properties) in a limit state design method (often called the *factor on strength* method), or by use of a single or 'lumped' factor of safety which is applied to the bending moment (often called the *factor on moment* method). Both methods are applicable to single-prop walls.

#### ***Factor on strength method***

In this method, the soil strength parameters used to derive the earth pressure coefficients are reduced by dividing by appropriate factors. These can be partial factors for an ultimate limit state or factors that represent the soil strength to be considered for a serviceability limit state.

The factor on strength method is a consistent, logical, and reliable method that factors the parameters representing the greatest uncertainty. This is the preferred method adopted by CIRIA Report 104 and BS 8002. Caution is needed, however, as the calculation of embedment depth is sensitive to the chosen safety factor.

#### ***Factor on moment methods***

In these methods, the earth pressure distributions are calculated using the fully mobilised (unfactored) design soil strengths and the geometry determined such that restoring moments exceed overturning moments by a prescribed margin. This prescribed margin is obtained by a predefined *lumped factor*. Three principal empirical methods are available to determine an embedment depth, although each gives a different answer and behaves differently as parameters are varied.

The three methods are:

- Burland-Potts method
- Gross pressure method
- Net total pressure method.

For further information on the use of these methods the reader is referred to CIRIA Report 104.

### 7.3.2 Comparison of methods

A comparison of the factors of safety defined by the methods mentioned above, has been carried out by Potts and Burland<sup>[31]</sup>, Day and Potts<sup>[32]</sup>, and in CIRIA Report 104. Their comparisons revealed that there is no unique relationship between the results obtained by the different definitions of the lumped factor. The choice of method is largely one of convenience, and the lumped factor is related to the method used, provided that the methods are applied consistently.

With the recent introduction in the United Kingdom of BS 8002 and the publication of ENV 1997, the factor on strength method is being adopted more often for stability considerations. In addition, the Burland-Potts method has become popular. A comparison of the embedded retaining wall design using ENV 1997 and existing UK design methods has recently been published by Carder<sup>[33]</sup>.

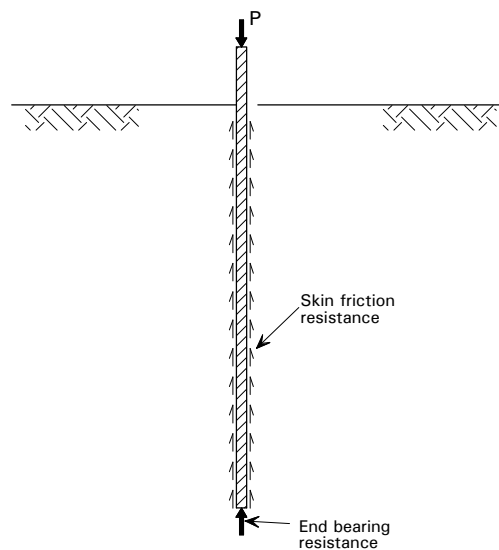
The designer should therefore consider all the options that are available, particularly noting the recent developments and introduction of new codes and standards. In all cases it is still appropriate to consider an alternative method of analysis as a check in stability design.

## 7.4 Vertical load resistance

For retaining walls that resist vertical loads from building superstructures, a method for predicting the axial capacity of sheet piles and bearing piles is needed. As BS 8004 is based on the lumped factor of safety approach and minimal information is given on steel sheet piles, it is recommended that the SCI publication *Steel bearing piles guide* <sup>[34]</sup> is used for design. The following comments are based on the advice in that publication.

### 7.4.1 Ultimate axial capacity and load transfer

A sheet pile subjected to a load parallel to its longitudinal axis will support that load partly by shear generated over its length, due to the soil-pile wall friction or adhesion, and partly by normal stresses generated at the base or tip of the pile, due to end bearing resistance of the soil (see Figure 7.3).



**Figure 7.3** *Wall friction and end bearing resistance against vertical loads*

The basic relationship is given in Eurocode 7 for the ultimate capacity  $R_c$  of the pile. This relationship assumes that the ultimate capacity  $R_c$  is equal to the sum of the wall friction capacity  $R_s$  and base capacity  $R_b$ , i.e.

$$R_c = R_s + R_b = q_s A_s + q_b A_b$$

where:  $q_s$  is the unit shaft friction value<sup>1</sup>  
 $A_s$  is the surface area of the pile in contact with the soil<sup>2</sup>  
 $q_b$  is the unit base resistance value  
 $A_b$  is the steel cross-section area of the base of the pile or plug cross-sectional area<sup>3</sup>.

Notes:

- <sup>1</sup> For a soil profile with more than one soil type, the average value of  $q_s$  over the length of the pile is taken.
- <sup>2</sup> See Section 7.4.4 for determination of surface area.
- <sup>3</sup> See Section 7.4.5 for determination of bearing area.

Numerous computer programs are available commercially to calculate the vertical capacity of piles.

For design on a limit state basis, the design ultimate capacity of a steel pile  $R_{cd}$  is given by:

$$R_{cd} = \frac{R_s}{\xi\gamma_s} + \frac{R_b}{\xi\gamma_b}$$

where:  $R_s$  is the ultimate shaft friction resistance  
 $R_b$  is the ultimate base resistance  
 $\gamma_s$  is the factor for shaft friction resistance  
 $\gamma_b$  is the factor for base resistance  
 $\xi$  is the material factor to take into account uncertainty of soil parameters determined on site or in the laboratory.

$\gamma_s$  and  $\gamma_b$  are partial factors for the resistance side of the limit state equation, and  $\xi$  is a material partial factor. These factors are not provided by BS 8002 or BS 8004 but are given in ENV 1997-1 Eurocode 7. In Eurocode 7, for driven piles:

$$\gamma_s = 1.3 \quad \gamma_b = 1.3 \quad \xi = 1.5$$

The design vertical capacity of the sheet pile-soil interface is adequate provided that:

$$\frac{P_{des}}{R_{cd}} \geq 1$$

where:  $P_{des}$  is the design magnitude of the axial load including all appropriate partial factors.

The unit skin friction and unit end bearing values for cohesive and cohesionless soils can be obtained from SCI publication *Steel bearing piles guide*.

## 7.4.2 Mobilisation of shaft friction on a retaining wall

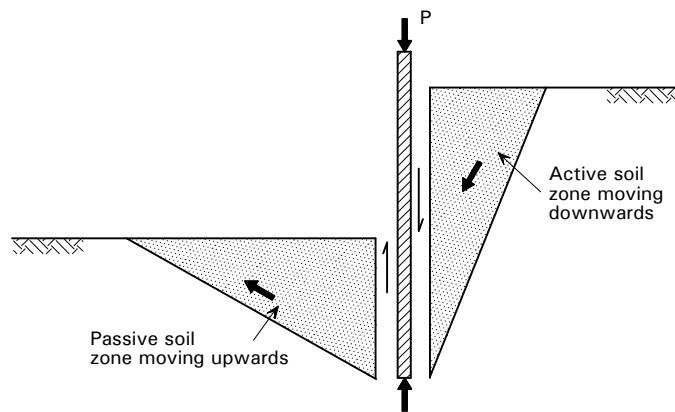
To design a retaining wall to resist axial load acting at the top of the wall, it is important that the overall behaviour of the retaining wall is considered. Although the design of the wall to resist axial load may be undertaken independently of the lateral loading case using the general method outlined above, the behaviour of the soil adjacent to the wall needs to be considered as the wall displaces laterally.

For an unpropped wall, as the wall deflects under lateral load the soil on the active or retained side of the wall moves down relative to the wall; on the passive

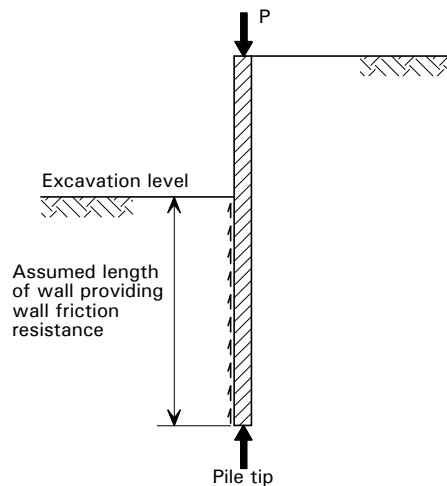
side, the displaced soil has to move upward (see Figure 7.4). If the wall itself displaces in a downward direction under the action of an axial load at the pile head, the shaft friction on the active side will diminish.

For simplicity, it may be conservatively assumed that wall friction resistance is mobilised only where the soil is displaced upwards, that is along the wall bounded between excavation level and the pile tip (see Figure 7.5). Thus only the side of the wall in contact with the passive soil zone is then considered in determining the contact area.

Where it is found that the depth of embedment required to achieve stability against overturning is insufficient to provide the required vertical resistance capacity, the depth should be increased as necessary to carry the vertical load. It may be assumed that any extra length of pile will have friction acting on both faces of the pile.



**Figure 7.4** *Generation of wall-soil friction by pile movement*



**Figure 7.5** *Length of sheet pile contributing to wall friction*

### **7.4.3 Determination of wall friction surface area**

#### ***Sheet piles and high modulus piles***

The surface area of sheet piles and high modulus piles can be obtained from manufacturers literature; this can generally be taken as the 'coated area' in the Corus data. As wall friction is assumed to act only on the passive zone of the soil, the area in which shaft friction acts is therefore half this value. This area per unit length is multiplied by the depth of embedment of the pile below excavation level over which shaft friction is mobilised.

#### ***Closed section and H piles***

The effective surface area for shaft friction resistance of box, tubular and/or H piles can be affected by whether or not a soil 'plug' is formed at the tip in the closed section.

If no plug is formed then the surface area is given by the summation of outside and inside shaft surface areas. If a plug is formed, the surface area is based on the outside surface only because skin friction on the inner surface is taken up in supporting the end bearing resistance. If there is uncertainty as to whether a plug does or does not form, the designer should consider both cases and adopt the one which produces the least resistance.

As for closed sections, the surface area of an H pile section depends on whether or not a soil 'plug' is formed at the tip. If no plug is formed at the tip of the pile, the surface area is given by the total surface area of the H section. If a plug is formed, the H pile is assumed to be a rectangular section with external dimensions of the H pile. For most UK applications for basements, plug formation is extremely rare.

For more detailed information refer to SCI Publication *Steel bearing piles guide*.

### **7.4.4 Determination of base resistance area**

#### ***Sheet piles***

The effective area at the tip of the sheet pile producing base resistance usually assumes that no soil plugging is present. In this case, the area is given by the cross-sectional area of steel.

#### ***Closed section and H section sheet piles***

For closed section sheet piles, the area to be used in the calculation of base resistance is the full cross-sectional area of the pile base comprising the pile wall and any soil plug. The calculated ultimate pile base resistance across the whole cross-section is compared with the internal soil plug skin friction plus the pile wall tip end bearing and the lesser is taken.

### **7.4.5 Buckling aspects of fully and partially embedded piles**

There are a number of simplistic analytical solutions available to determine the buckling behaviour of fully and partially embedded piles. See British Steel *Piling Handbook* or ENV 1993-5.

## 7.5 Design for lateral loading

A knowledge of the structural forces acting on a retaining wall involves determining bending moments, shear forces, and axial forces. The loads that cause these forces are due to earth and water pressures, surcharges acting on the surface of the soil, and possibly loads from the building superstructure.

Structural forces acting on retaining wall due to lateral forces are calculated using one of two methods. One method is to model soil-structure interaction effects using advanced analytical methods, while the second method is to use the simplistic distribution of earth pressures assumed in the limit equilibrium method is used. Both methods are adequate to determine conservative values of the structural forces in the retaining wall.

### 7.5.1 Soil-structure interaction method

The pressure distributions that occur in the design configuration of the wall under actual conditions can be modelled using the soil-structure interaction method (see Section 8.2). Structural forces and displacements are determined for the particular design situation considered, taking into account the stiffness of the retaining wall and the soil, the method of construction, and the initial stresses in the soil.

A soil-structure interaction method may be more appropriate where:

- the distribution and magnitude of soil movements need to be estimated
- the effects of construction stages on wall behaviour needs to be studied
- the influence of high initial in-situ at-rest soil stresses needs to be analysed
- wall /soil flexibility effects are to be modelled

These aspects cannot be analysed when using simplistic limiting equilibrium methods.

### 7.5.2 Limit equilibrium method

Bending moments and shear forces in the wall and forces in the prop due to pressures assumed in the limit equilibrium method can be calculated either manually or by computer software.

Where a limit equilibrium analysis is to be applied to determine structure forces, reference to CIRIA Report 104 should be used. Two methods are presented in CIRIA Report 104 to calculate structural forces acting on cantilever and propped retaining walls. They are termed the *ultimate conditions* method and the *working conditions* method. CIRIA Report 104 recommends that the *ultimate conditions* method is used.

To correct for the effects of in-situ stress state and wall stiffness, a measure of effective wall stiffness is needed. This is taken into account by use of a modified definition of Rowe's wall flexibility number  $\rho$ , as described by Potts and Bond<sup>[35]</sup>. They define the modified parameter  $\rho^*$  as:

$$\rho^* = \left( \frac{L^4 E_s^{av}}{E_w I} \right) = \rho E_s^{av}$$



where:  $\rho$  is the original Rowe's wall flexibility number <sup>[36] [37] [38]</sup>  
 $L$  is the full length of the wall  
 $E_s^{av}$  is the average stiffness of the soil over the full length of the wall  
 $E_w$  is the stiffness of the wall  
 $I$  is the inertia of the wall.

It is important that these modification factors for moments and forces in the steel pile are only applied to propped walls that have been analysed using the limit equilibrium free-earth support method.



## 8 DESIGN OF MULTI-PROP WALLS

Multi-prop walls are highly redundant structures and the degree of soil structure interaction can have a very significant effect on the distribution of forces and moments. The method of construction can have a large influence on the earth pressures acting on the wall. While numerical methods are more appropriate for the analysis of this type of structure, the simple and empirical methods can and do allow approximate solutions to be obtained.

For an accurate analysis of multi-prop walls, deformation methods should be used. These methods consider soil-structure interaction and calculate forces acting on the wall and supports and calculate wall deflections. With the advent of powerful desktop computers, these more complex methods of analysis are now widely available to practising engineers.

### 8.1 Simple methods

The simplistic limiting pressure methods do not satisfy all of the fundamental theoretical requirements to simulate soil-structure behaviour. In particular, they do not consider compatibility or the displacement boundary conditions, and hence the methods are only approximate. Although earth pressures acting against multi-propped walls are extremely difficult to predict, simple methods based on modified classical earth pressure distributions have been developed and used.

To perform a simple analysis to determine the forces and bending moments acting on the wall, some sweeping assumptions have to be made. Using as a basis the extreme approach where it is assumed that active conditions act on the back of the wall and passive conditions act on the front of the wall, Liao and Neff (1991) <sup>[39]</sup> introduced mobilised earth pressure coefficients to improve the earth pressure distributions and make them more 'realistic'. See Figure 8.1.

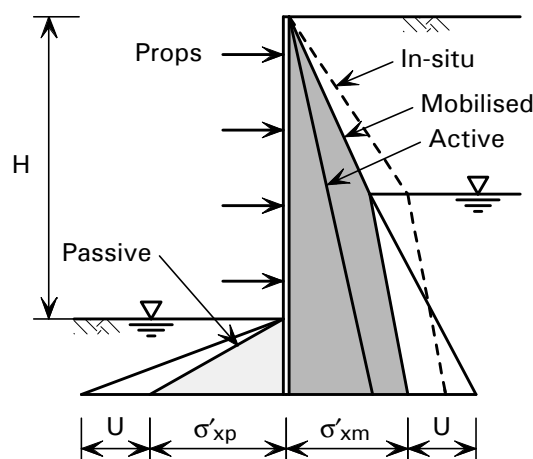


Figure 8.1 Mobilised earth pressure diagram

For this approach, the modified horizontal effective stress acting on the back of the wall is given by:

$$s'_{xm} = K_m s'_v$$

where:  $K_m$  is the mobilised earth pressure coefficient  
 $s'_v$  is the vertical effective stress in the field.

The soil on the on the front side of the wall is assumed to be in a state of passive failure with

$$s'_{xp} = s'_p$$

The mobilised earth pressure coefficient,  $K_m$  is defined as:

$$K_m = K_a + m(K_o - K_a) \geq 1.3 K_a$$

where:  $K_a$  and  $K_o$  are the soils's active and at-rest earth pressure coefficients, respectively  
 $m$  is the mobilisation factor ( $0 \leq m \leq 1$ ).

The Canadian Foundation Engineering manual (1985)<sup>[40]</sup> suggests the following possible values for  $m$ :

If the wall movements can be tolerated, or where the foundation of adjacent buildings extend to below the wall toe,  $m=0$  (i.e.  $K_m = K_a$ ).

If foundations of buildings or services exist behind the wall at a horizontal distance  $x$  away (where  $H/2 \leq x \leq H$ ),  $m=0.5$  (i.e.  $K_m = 0.5(K_a + K_o)$ ).

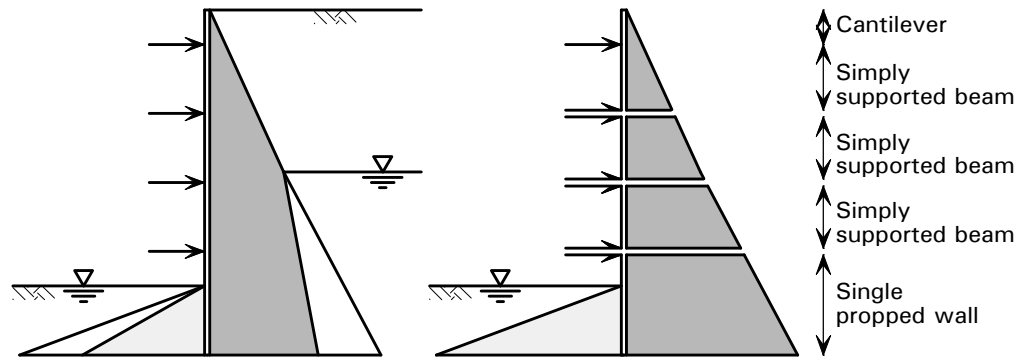
If foundations of buildings exist behind the wall at shallow depth at a horizontal distance  $x$  away (where  $x < H/2$ ),  $m=1.0$  ( $K_m = K_o$ ).

In the above,  $H$  is the retained height.

Having estimated the earth pressures acting on the wall it is possible to calculate the wall bending moments and support forces. Unfortunately, the structure is statically indeterminate and a number of further assumptions need to be made. This has resulted in numerous approaches. The two most commonly used methods are the hinge method and the continuous method.

### 8.1.1 Hinge method

This method allows the structure to be analysed at successive stages of construction, modelling the retaining wall at a number of construction stages when additional supports are introduced. Hinges (positions of zero bending moment) are assumed to occur at all prop levels except the first. The spans between the props are designed as simply supported beams loaded with the earth and water pressures (most texts that describe this approach advise using  $K_a$  earth pressures, i.e.  $m=0$ ). The span between the lowest prop and excavation level is designed as a single-propped embedded wall with its appropriate earth and water pressures. See Figure 8.2.



**Figure 8.2** Hinge method for multi-prop walls

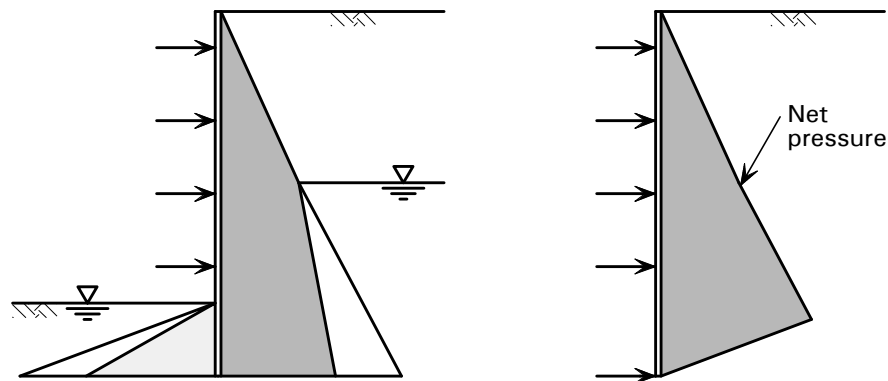
The analysis of structures using this method is carried out on a stage by stage basis with excavation carried out to sufficient depth to enable the next level of support to be installed. It is therefore possible that the support loads and bending moments calculated for a given stage of excavation are exceeded by those from a previous stage and it is important that the highest values of calculated force and bending moments are used for design purposes.

Using this method, it is possible to calculate a depth of penetration needed to give a factor of safety against rotational failure, as the lowest span is treated as a singly propped wall and can therefore be analysed as such. This can be a comfort to designers, as the calculations show that a given factor of safety has been achieved. However, this should only be considered as an indicative value, as the remainder of the piled wall has been ignored and failure will not be in the form assumed.

This method is mentioned in both BS 8002 and the British Steel *Piling Handbook* and is included in the ReWaRD analysis software.

### 8.1.2 Continuous beam method

In this method the wall is assumed to act as a continuous beam supported at the prop positions and by an additional fictitious support located at the point below excavation level where the net total earth pressure on the back of the wall drops to zero. Only positive net total pressures are considered in this analysis. The wall below the position of the fictitious prop is ignored. See Figure 8.3.



**Figure 8.3** Continuous beam method for multi-prop walls

The problem is statically indeterminate and a numerical procedure in which the beam is either represented by finite difference or finite elements is usually adopted in order to obtain a solution. It is possible to treat the props as either rigid or give them a prescribed stiffness (beam on elastic foundation). The method yields the distribution of shear force and bending moment in the wall and the prop loads.

The analysis is performed at each stage in the construction sequence, i.e.:

Stage 1 Excavate to depth allowing erection of the first level of props  
(i.e. a cantilever wall)

Stage 2 Continue excavating to allow erection of second level of props  
(i.e. a single-prop wall)

Stage 3 Further excavation to allow erection of third level of props  
(i.e. a multi-prop wall)

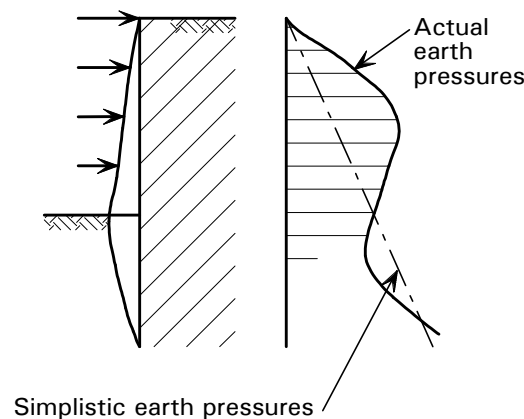
and so on, including backfilling and removal of/or alterations to the level of any props as the construction of the permanent works proceeds. It is important to remember that the prop loads and bending moments during earlier stages may be greater than those at later stages and should be used for the design of the structural members.

This method is described in detail by Tamaro and Gould (1992) <sup>[41]</sup>.

## 8.2 Deformation methods

Deformation methods using a soil-structure interaction approach can produce a much more realistic representation of the behaviour of a retaining wall by taking into account wall and soil stiffness, in-situ soil stresses, and the load distribution capability of the soil and wall continuum.

Deformation methods predict the earth pressure distribution that acts on the design configuration of the wall. As the stiffnesses of the wall and the soil are modelled, the earth pressure profiles predicted using these methods compare favourably with actual earth pressures. Figure 8.4 shows a typical earth pressure profile for a multi-prop wall obtained from a soil-structure interaction analysis.



**Figure 8.4** *Horizontal earth pressure distribution for a sheet pile wall*

There are also other methods of increasing complexity, where the soil is modelled by boundary element, finite difference, and finite element numerical approximations.

For design, the method chosen should be one where the level of simplicity is consistent with providing adequate results.

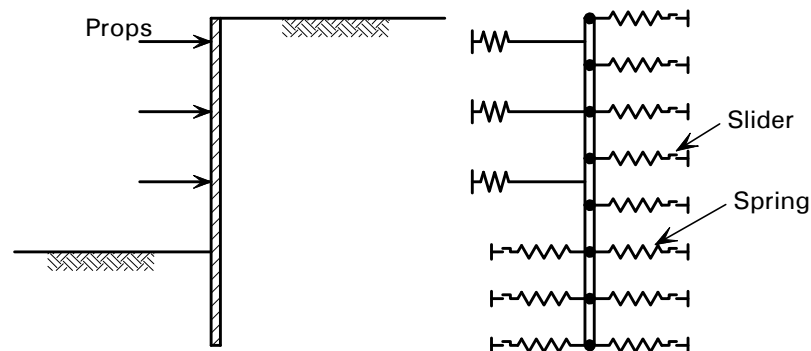
### 8.2.1 Beam on elastic foundation methods

The simplest of the soil-structure interaction methods is the Winkler<sup>[42]</sup> spring model (beam on elastic foundation) where the soil is modelled as a spring.

The assumption of a beam or slab on an elastic foundation has found application in numerical analysis of sheet piles. Power series, finite differences, distribution, and discrete element methods are employed for the solution of the governing differential equations. In each case the elastic foundation is assumed to generate reactive pressure proportional to the deflection (Winkler's hypothesis). The soil response is usually characterised by a spring constant, which is related to the coefficient of subgrade reaction. The coefficients of horizontal subgrade reaction recommended by Terzaghi<sup>[43]</sup> normally are used.

The subgrade reaction approach is commonly used for soil-structure interaction because of the ease with which it can be applied; various methods are available. Commonly, the soil mass is modelled as a series of isolated horizontal springs, or as springs with some form of interconnection (see Figure 8.5).

The initial, at rest, values on each side of the wall are allowed to reach equilibrium in a series of iterations using numerical methods such as finite elements or finite differences, until earth pressures lie, between at rest values and active values on the retained side of the wall, and between at rest and passive values on the excavated side. The effect of temporary soil berms is seldom modelled accurately, if at all, in these programs. Where soil stiffness is input in terms of subgrade reaction values, there is often a lack of user confidence in the selection of accurate values. Programs which are based on the use of elastic interaction factors often have difficulty in accurately applying wall friction/adhesion. Props, both stressed and unstressed, are simulated by additional springs of the required stiffness and at the appropriate level.



**Figure 8.5** Spring model for analysis of retaining wall

In most situations the bending moments and shear forces internal to the wall obtained from the Winkler method are insensitive to the values of the spring stiffness chosen and used in the analysis. However, computed support forces in

a prop are greater in magnitude than those obtained from the empirical or simplistic methods. Also, prediction of wall deformations using the deformation methods can only be regarded as rough estimates. Wherever possible, checks need to be made to compare field measurements with those obtained from the analyses.

There are a number of commercially-available numerical analysis software products that are commonly in use by design practices. Two of the most well known are WALLAP, and FREW.

### **8.2.2 Computer methods based on continuum models**

Computer methods that are based on continuum models include finite element, boundary element and finite difference numerical approximations. The principal advantages of these methods are that they include the ability to model wall and soil deformation and stress in a realistic sequence of operations that follow actual construction stages.

The analyses show both immediate deformation and time-dependent changes related to pore pressure equalization. Pre-judged failure modes are not needed because these are all revealed by the analysis. Use of low strain values of soil stiffness are essential in such approaches.

As computer technology progresses, the application of these methods to three-dimensional problems as well as to more routine two-dimensional problems are becoming more universal.

#### ***Finite Element (FE) methods***

Geotechnical FE packages offer a number of different constitutive models that range from simple elastic models to highly sophisticated elasto-plastic strain-hardening/softening models. The choice of model is closely linked with the selection of appropriate soil parameters and how much of the complexity has to be modelled to ensure a realistic result.

Computer software packages which have been widely used for the design of excavation support systems in the UK, include the finite element program CRISP-90 and the Imperial College Finite Element Package (ICFEP).

#### ***Finite difference analysis***

In the finite difference method (FDM), materials are modelled in zones, which are defined within a gridwork; each zone has a prescribed stress-strain behaviour (for example, elastic or plastic). The method uses the basic equations of motion and a time-stepping process to calculate accelerations, velocities and displacements of the zone mass. The strains obtained are then used in a constitutive law, to determine the corresponding stresses.

#### ***Boundary element analysis***

The boundary element method (BEM) is a numerical method for solving boundary value problems governed by differential equations. Typically, the BEM links boundary stresses to boundary displacements, and only the boundary of the domain needs to be discretized, not the interior. This results in a smaller system of equations and noticeable savings in computing time. The method is particularly suited to three-dimensional foundation problems.



### **8.3 Vertical Load resistance**

See Section 7.4.



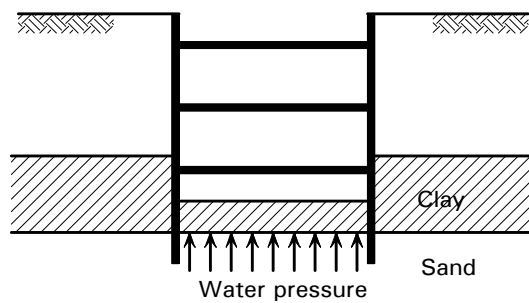
## 9 BASE STABILITY

There are several possible modes of instability that can occur in supported excavations, these include:

- base heave
- hydraulic failures.

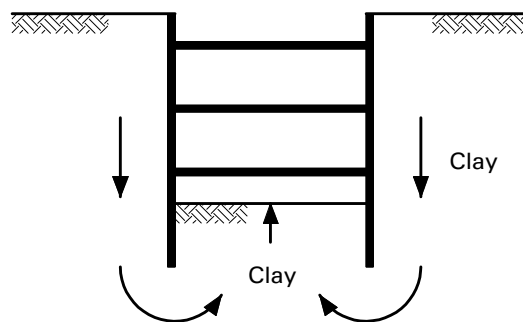
### 9.1 Base heave

One form of base heave arises as a result of excess pore water pressure in underlying soil layers. If there is a thin layer of clay overlying sand or gravel that has a sufficiently high pore water pressure, then the clay can be forced into the base of the excavation. A simple calculation comparing the weight of the thin clay layer to the pore water pressure beneath it, will indicate the potential of failure in this manner. See Figure 9.1.



**Figure 9.1** *Base heave resulting from excessive water pressures*

Another form of base heave arises if the soil at the base is not strong enough to support the stress imposed by the soil adjacent to the excavation. In this case the base of the excavation will fail and the soil will be forced upward into the excavation. This results in large movements in the adjacent ground. The depth of excavation at which base heave occurs is called the critical depth,  $D_c$ . See Figure 9.2.



**Figure 9.2** *Base heave due to weight of adjacent soil*

As this type of failure can occur during construction and before any base slab is installed, analysis is usually performed using the undrained shear strength,  $c_u$ . There are two methods of calculating the critical depth of excavation. One method

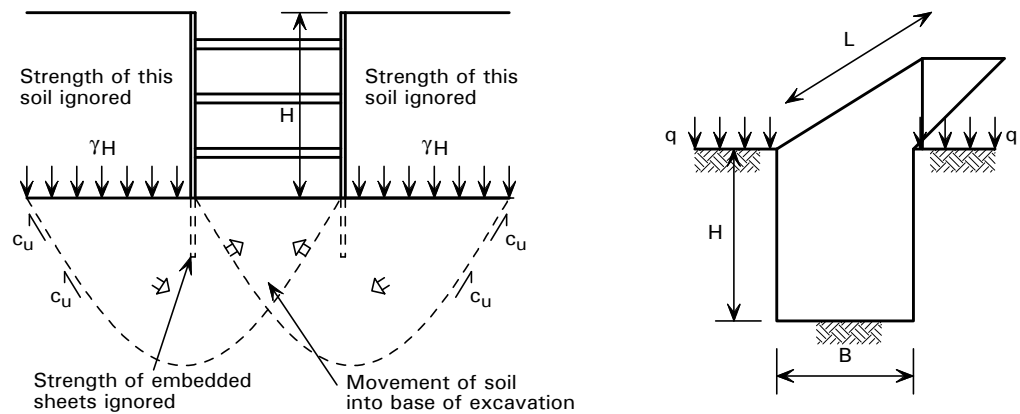
is by Terzaghi [44] and the other is the method developed by Bjerrum and Eide [45]. More recently, improved methods have been proposed that take into account the stabilising effect of the embedded wall.

The most commonly used method is the Bjerrum and Eide method as it has been shown that Terzaghi's method is only reliable if:

- the width of the excavation is large compared with its depth
- the clay is comparatively homogeneous with no stiff (weathered) upper layer.

### 9.1.1 Bjerrum and Eide method

In this approach, the bottom heave problem is considered to be the reverse of the standard foundation failure problem. The shear stresses mobilised in the soil are a result of the material removed from the excavation. See Figure 9.3



**Figure 9.3** *The mechanics of base heave*

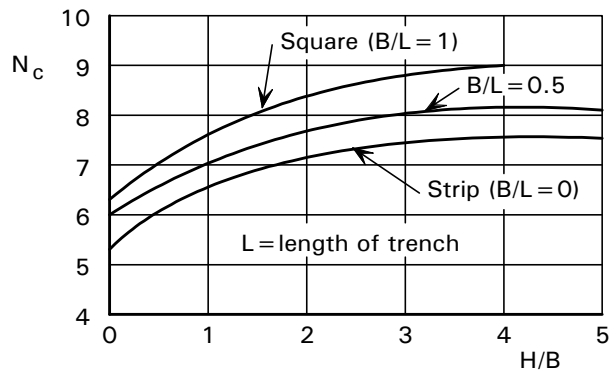
Based on foundation analysis and noting that the shear stresses in the soil are mobilised in the opposite sense to those under a loaded foundation, the factor of safety against bottom heave is given by:

$$F = \frac{N_c c_u}{(gH + q)}$$

where:  $N_c$  is a bearing capacity coefficient dependent on the shape and depth of the excavation

$q$  is the surcharge acting

$N_c$  can be found from tabulated values in most codes of practice and design manuals for foundation. See Figure 9.4.



**Figure 9.4** Bearing capacity factors (Bjerrum and Eide 1956)

At failure,  $F$  is equal to

To avoid plastic 1.0 and the critical depth is given by:

$$D_c = \frac{N_c c_u}{g} + \frac{q}{g}$$

To avoid plastic yielding of the soil, and so minimize ground movements, the factor of safety should exceed 2.5 - 3.0, so that the mobilised bearing capacity factor is less than 3.14 (Peck, 1969) <sup>[46]</sup>.

This approach is conservative as it does not account for the reinforcing effects of wall penetration below the base of excavation. O'Rourke (1992) <sup>[47]</sup> proposed an improvement of the Bjerrum and Eide method to account for the flexural capacity of the wall extending below the excavation. Factors of safety determined by this method are in better agreement with the observed performance of excavations approaching base failure.

O'Rourke's expressions are based on a dimensionless stability number  $N_{OR}$ . Various expressions for  $N_{OR}$  are given for different end conditions of the wall. See O'Rourke (1992).

The effect of wall stiffness, depth of embedment and thickness of clay layer on base stability has also been considered by Goh <sup>[48]</sup> using finite element analyses. The factor of safety proposed by this approach is given by:

$$F_{base} = \frac{c_u N_h}{g H} m_t m_d m_w$$

where:

- $\gamma$  is the unit weight of the soft clay
- $H$  is the depth of excavation
- $N_h$  is the bearing capacity factor as a function of  $H/B$
- $B$  is the width of excavation
- $\mu_t$  is the multiplying factor, which is a function of  $T/B$
- $T$  is the thickness of soft clay beneath the base of the excavation
- $\mu_d$  is the multiplying factor, which is a function of  $D_e/T$
- $D_e$  is the depth of embedment of the wall
- $\mu_w$  is the multiplying factor, which is a function of  $D_e/T$ , wall stiffness and  $T/B$ .

See Goh for details and for charts of  $N_h$ ,  $\mu_t$ ,  $\mu_d$ , and  $\mu_w$ .

### ***Means of ensuring base stability***

If the depth of the proposed design excavation results in unacceptably low factors of safety on base stability, measures can be taken to extend the depth of excavation. These include:

- a) Extend the retaining wall to a competent stratum (where possible), so that the wall prevents the retained soil from being displaced into the excavation. See Figure 9.5 (a). This method results in higher prop loads.
- b) Dig the excavation as a series of smaller excavations with a reduced plan area. This approach changes the aspect ratio of the foundation, resulting in an increased bearing capacity factor  $N_c$ , and factor of safety. See Figure 9.5 (b).
- c) Excavate under water or bentonite mud to reduce the unloading of the excavation. This involves flooding the excavation and is often undesirable. See Figure 9.5 (c).
- d) Increase the soil strength prior to excavation by freezing, grouting or in-situ mixing. See Figure 9.5 (d).
- e) Reduce the effective excavation depth by removing soil adjacent to main excavation. See Figure 9.5 (e).

## **9.2 Design of base slab and foundations**

In clay soils, the choice of foundation has to allow for the possible long term build-up of water pressures and the potential heave of clay due to the reduced vertical effective stress below excavation level while providing adequate support for the internal structure. A piled foundation is required for a top-down method of construction. This is achieved by the use of the bored pile and plunge column foundation system, as explained in Section 4.1.1. For a bottom-up construction method, pad foundations can be used to support the internal columns of the basement.

The potential build-up of water pressure beneath the base slab formed by bottom-up construction may require tension piles or vertical ground anchors to prevent flotation of the structure in the long term (i.e. to compensate for the insufficient dead weight within the structure and the backfill). An alternative approach is to provide drainage beneath the base slab by means of land drains laid to falls and bedded in pea gravel. The drain feeds a sump, fitted with pumps to lift water up to the local surface drainage system.

To prevent large heave forces on the underside of the slab, an effective void can be formed beneath the base slab by use of polystyrene slabs as compressible filler material. It is important to note that if a drained void beneath the base slab is used, the magnitude of long term settlements around the excavation could be significantly increased. This may be unacceptable where basements are to be built adjacent to sensitive structures. The alternative is to construct a stiff heavily reinforced base slab and a retaining wall with a deep toe designed to provide long term support for the external soil.

Where pad foundations are used to support internal columns, they can also provide support for suspended base slabs. In these cases it is necessary to assess long and short term heave movements and the effects these have on the relative movement of the pads and the base slab.

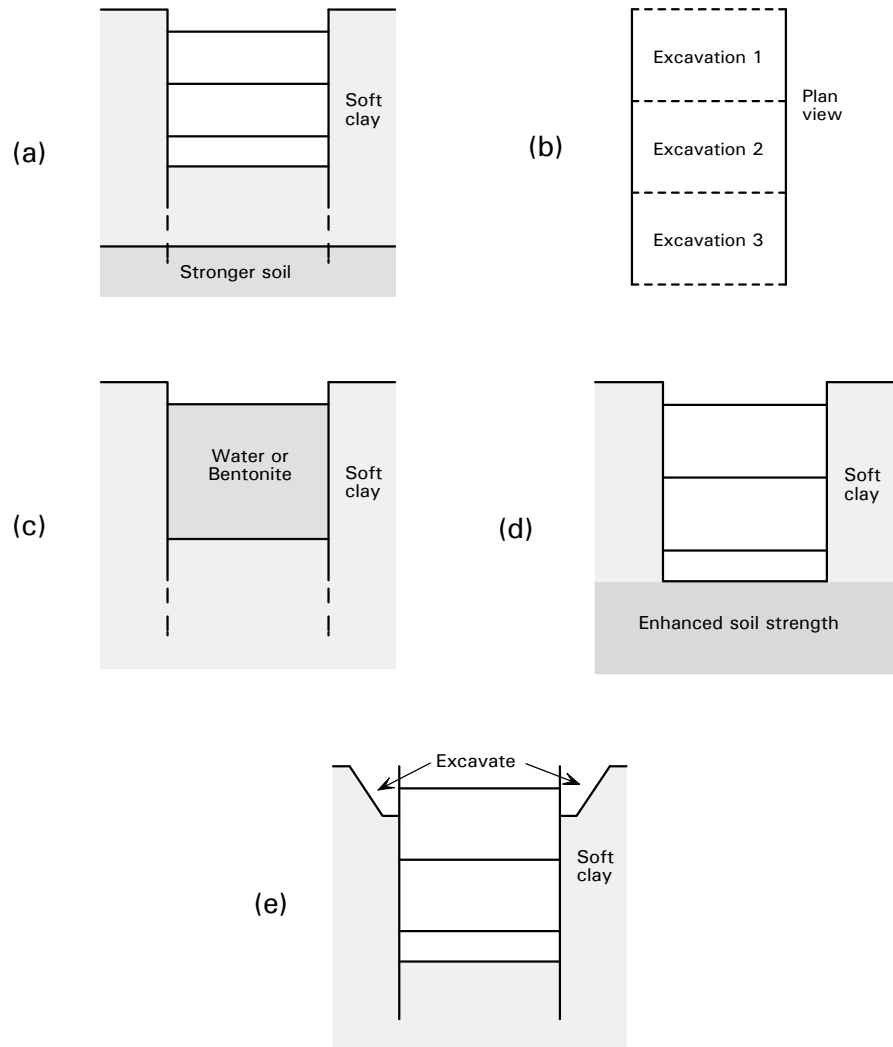
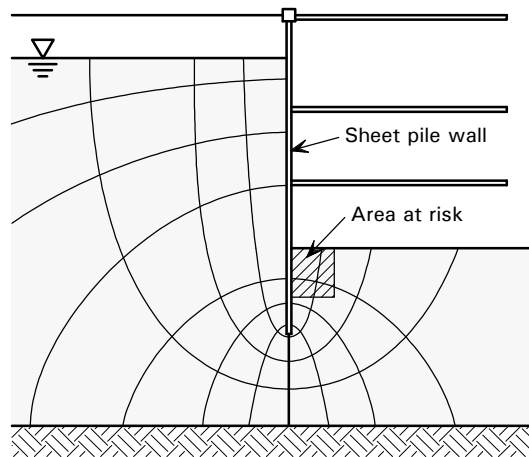


Figure 9.5 Methods to prevent base heave

### 9.3 Hydraulic instability in granular soils

Where groundwater exists above the base of the excavation, and where the toe of the wall does not penetrate into an impermeable layer, flow will occur under the wall and upward through the base of the excavation. The result is loosening of the bottom soils, which may cause collapse of the wall and loss of the bearing capacity for building foundations.

The most effective control is dewatering, but in deep excavations it may be more economical to penetrate the wall to a depth sufficient to intercept the potential flow lines with high heads of water (Figure 9.6.) It should be noted that during construction (before the slab is cast), corners of basements are at the greatest risk from piping.



**Figure 9.6** *Piping in soil*

The analysis of seepage and the determination of flow lines are obtained using flow-net construction techniques.

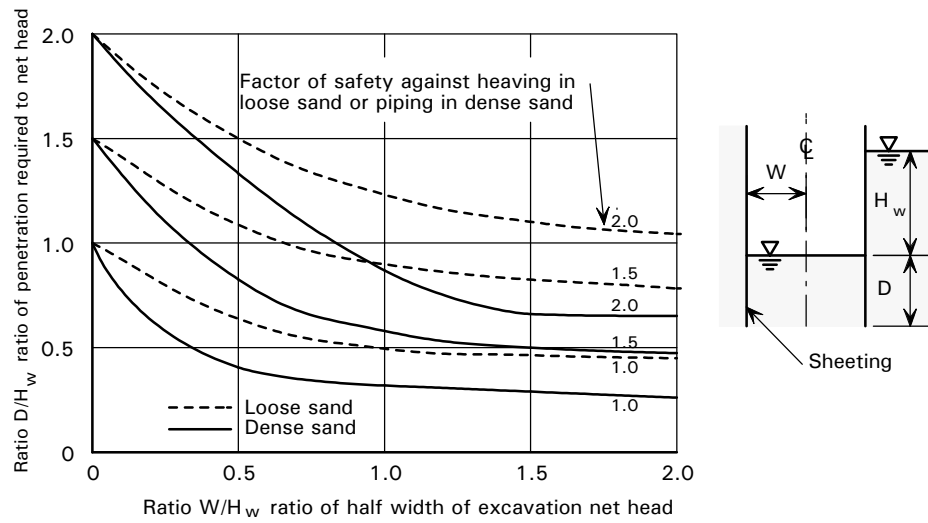
The penetration depth required to prevent piping is a function of the pressure head between the water table and the excavation bottom (net head), the ratio of the excavation width to net head, and the soil conditions below excavation bottom. Charts have been produced for estimating depths for various conditions. The more common conditions and criteria for estimating sheeting penetration for various factors of safety against heave or piping in isotropic sands are shown in Figure 9.7. These are design charts based on NAVFAC - DM7 (1982) <sup>[49]</sup>.

Typically wall penetration sufficient to give a safety factor of 1.5 to 2.0 against piping is considered appropriate to avoid this failure.

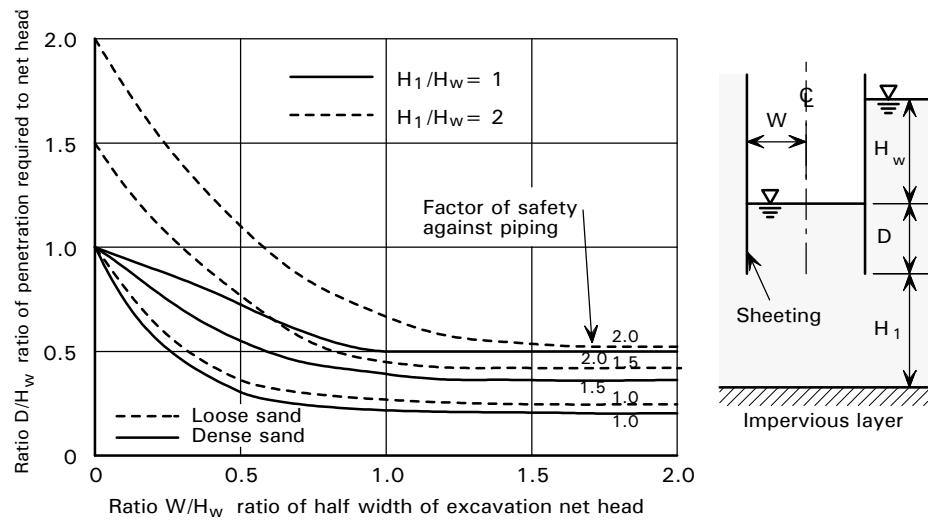
For those situations where it is not possible to obtain a suitable depth of sheet pile penetration, or where the required depth is clearly uneconomical, the following measures may be used:

- Well points from original surface level, to reduce the water level in the excavation. This is acceptable for homogeneous soil conditions or where permeability decreases with depth.
- Pressure relief wells in the base of the excavation. These are suitable where a thin impermeable layer of soil overlies permeable soil relatively close to the base of the excavation.
- A filter layer in the base of the excavation. This provides weight and prevents the upward movement of soil particles with the inflowing water.





(a) penetration required for sheet piles in dense sand of infinite depth



(b) penetration required for sheet piles in dense sand of limited depth

**Figure 9.7** Penetration of sheet piling required to prevent piping in sand (NAVFAC-DM7, 1982)



## 10 DESIGN OF TEMPORARY SUPPORTS

This Section provides guidance for the design of temporary props and soil berms. Temporary props are used extensively where bottom-up construction is undertaken. For top-down construction, very stiff propping is provided by the concrete floors, however, in some cases temporary propping may also be used. Steel sections, which include tubulars, box sections and universal columns, are the most common forms of temporary propping. In certain cases soil berms may be used either on their own or with temporary props.

### 10.1 Single-prop walls

For the design of a single-propped wall, limit equilibrium methods are usually adequate to determine the support force reliably. These methods are described in Section 7.

### 10.2 Multi-prop walls

Numerous empirical methods, based on field measurements, have been developed to determine forces on temporary props acting on multi-prop walls. One such method is the 'pressure envelope' method. The most frequently used 'envelopes' are those of Terzaghi and Peck<sup>[44]</sup>, subsequently modified by Peck<sup>[46]</sup>. This method is included in BS 8002.

A more recent method to predict prop loads is presented in CIRIA Report C517 *Temporary propping of deep excavations*<sup>[50]</sup>. The method is called the *Distributed prop load* method.

#### 10.2.1 Peck's pressure envelope method

Peck's envelopes provide an empirical way of estimating maximum prop loads for multi-prop walls. The envelopes were derived from measurements of strut loads in real excavations. The field observations consist of measurements of loads carried by the props at one or more vertical cross-sections of an excavation. Since reliable direct measurements of the earth pressure against the walls have rarely been made, the magnitude and distribution of earth pressure must be inferred from the prop loads.

It is important to note that the Peck's pressure diagrams do not represent actual earth pressure or its distribution with depth, but load envelopes from which prop loads can be evaluated. For this reason the pressure calculated in this manner is called the *apparent earth pressure*. If the apparent earth pressure is known, the corresponding prop loads can be computed by following the reverse procedure.

As failure of an individual prop increases, the load on adjacent props progressive failure of the system may be initiated. Each prop should therefore be designed for the maximum load to which it may be subjected to and consequently design of the prop system should be based on the envelope of all the apparent pressure diagrams obtained from the measured prop loads. Peck therefore considered all the field data and obtained standard apparent pressure distributions for use with appropriate safety factors in design.

The design earth pressure diagrams for different soil conditions can be found in most foundation text books.

The conditions for the satisfactory application of the Peck method are:

- the depth of excavation must be greater than 6 m
- the sand is assumed to be drained (i.e. use *effective stresses*)
- clay is assumed to be undrained (i.e. use *total stresses*)
- bottom stability must be checked separately
- envelopes for clay are applicable for short term conditions, therefore they may not give realistic estimates of prop loads in the long term.

There are a number of difficulties associated with the use of this method to real soil conditions; these include, how to:

- classify clays and sands
- treat groundwater
- select the correct value of undrained shear strength for a soil profile with varying undrained shear strength
- treat soil profiles with layers of sand and clay and to treat silt.

All of these problems need careful judgement by the individual designer.

Pressure envelope methods are not used to determine forces acting on sheet pile walls. Bending moments and shear forces acting on the wall are more suitably obtained by either simplistic methods, based on a classical pressure distribution (see Section 8.1), or by deformation methods (See Section 8.2).

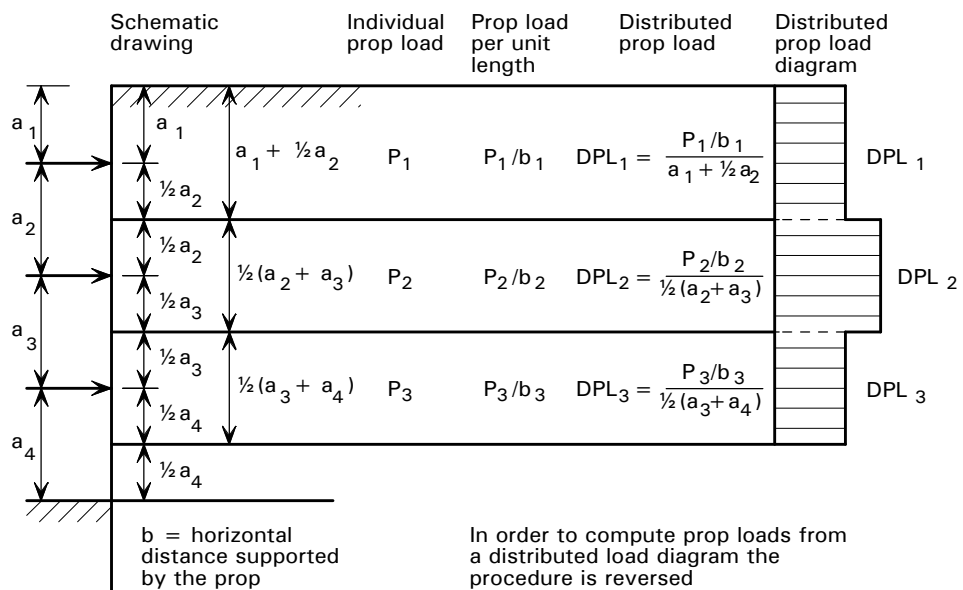
Although prop loads using Peck's pressure envelop method can be computed by hand, the computation procedure can readily be automated.

### **10.2.2 Distributed prop load method**

In 1999, an improved method to predict prop loads based on Pecks's 'pressure envelope' approach was published in CIRIA Report C517. This publication provides guidance on design based on the interpretation of extensive field measurements of prop loads for flexible and stiff walls and for the range of ground conditions commonly encountered in the United Kingdom. The method is the called the Distributed prop load method.

Peck's method has been renamed to the distributed load method owing to the term apparent pressure having misled some people into thinking that Peck's envelopes represent the actual earth pressures acting on the wall.

The method for determining prop loads is shown in Figure 10.1.



**Figure 10.1** Method for calculating the distributed prop load (CIRIA Report C517)

The distributed prop load diagrams presented in CIRIA Report C517 are based on 81 case histories of which 60 are for flexible walls and 21 are for stiff walls. Flexible walls comprise steel sheet pile and king post/soldier pile walls, whilst stiff walls include contiguous, secant and diaphragm concrete walls. The load magnitudes given are characteristic values (see section 5.2.2) and not maximum values as given by Peck's apparent earth pressure distributions. These characteristic values can be used in limit state design as appropriate partial factors can be applied for the ultimate and serviceability limit states. The data presented is classified on the basis of type of ground retained by the excavation. These classes are shown in Table 10.1.

**Table 10.1** Classification of ground types

Class	Description
A	normally and slightly overconsolidated clay soils (soft and firm clays)
B	heavily overconsolidated clay soils (stiff and very stiff clays)
C	granular/cohesionless soils
D	mixed soils (walls retaining both cohesive and cohesionless soils)

Further subdivision is made for Class A and B soils according to wall type based on wall stiffness, i.e. flexible (F) walls and stiff (S) walls. Flexible walls retaining soft clay soil (Class AF) have been further subdivided according to base stability conditions into "stable" and "enhanced stability" cases. Walls in granular soils (Class C) are subdivided into "dry" and "submerged" cases.

Table 10.2 is an extract taken from CIRIA Report C517, which shows the characteristic distributed prop load magnitudes for Class A, B and C soils, to be used to determine characteristic prop loads for flexible walls (applicable to steel sheet pile walls) shown in Figure 10.1.

**Table 10.2** *Characteristic distributed prop load magnitudes for Class A, B and C soils.*

Soil class		Characteristic DPL/ $\gamma H$		Limitations
		0-0.2D/H	0.2D/H - H	
AF	Firm clay	0.2 $\gamma H$	0.3 $\gamma H$	
	Soft clay with stable base	0.5 $\gamma H$	0.65 $\gamma H$	
	Soft clay with enhanced base stability	0.65 $\gamma H$	1.15 $\gamma H$	Refer to CIRIA Report C517
BF		0.3 $\gamma H$	0.3 $\gamma H$	
C	Dry	0.2 $\gamma H$	0.2 $\gamma H$	
	Submerged	0.2 $\gamma H$ + water pressure	0.2 $\gamma H$ + water pressure	

It is important to note that the DPL diagrams assume that the bottom of the excavation is a prop, i.e. the soil in front of the toe of the wall is assumed to support the wall between the base of the excavation and half way towards the lowest prop, as well as the earth pressures from the retained ground over the embedment length. The engineer should check that the wall embedment is sufficient to satisfy this assumption with an appropriate factor of safety on the passive pressures or soil strength appropriate to the allowable movement of the toe of the wall. Refer to method proposed by Goldberg *et al.* (1976)<sup>[51]</sup>. This is likely to be satisfied unless the wall has a lower factor of safety on overall lateral stability.

For full details of the distributed prop load method the reader should refer to CIRIA Report C517.

### 10.3 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 usual relationship:

$$\Delta L = \alpha \Delta t L$$

where:  $\Delta L$  is the change in prop length  
 $L$  is the prop length  
 $\Delta t$  is the change in temperature from the installation temperature  
 $\alpha$  is the thermal coefficient of expansion for the prop material, typically  $1.2 \times 10^{-5}$  per  $^{\circ}\text{C}$  for steel.

If the prop is restricted or prevented to expand freely, an additional load is generated in the prop. For a fully restrained prop in which no expansion occurs, the magnitude of this load is given by:

$$\Delta P = \alpha \Delta t EA$$

where:  $E$  is Young's modulus of the prop material  
 $A$  is the cross-sectional area of the prop.

If the degree of restraint of the prop allows some expansion, lesser prop loads due to temperature effects will result. For flexible steel sheet pile walls, the degree of restraint is typically between 10-25 % but can reach 40%. If the prop is subjected to a decrease in temperature, the load in the prop will be reduced.

The envelopes produced by Peck (1969) are based on maximum measured loads and include some temperature effects.

It is not usual for deformation methods of analysis to include temperature effects, although this is within the scope of most of the available methods. Temperature effects are normally added to the predicted prop loads after the analysis is complete.

No common approach exists in the UK for the design of props loaded as a result of increases in prop temperature above the installation temperature. When the effects of temperature are explicitly considered, it may be prudent to follow the guidance given in BS 5400-2 if no other guidance is available. Typically a small temperature range is chosen, say 10°C, with the assumption that the prop is to be completely restrained. If a larger temperature range is adopted, say 30°C, the assumption in this case may be that only 50% of the fully restrained value is attained.

## 10.4 Temporary support of retaining walls using soil berms

CIRIA Report 104 summarises the current design methods employed in assessing the performance of soil berms for the temporary support of retaining walls during construction. These methods are based on either empirical solutions or numerical analyses.

The two most successfully applied empirical methods are those proposed by Fleming *et al.* (1992)<sup>[52]</sup>. They are:

- The method where the berm is considered as an increase in the effective ground level on the passive side of the wall. The design height of the berm is restricted to a third of the berm width and the increase in the effective ground level is taken to be one half of the design height of the berm.
- The method where the weight of the berm is converted to an effective surcharge acting at the final excavation level on the potential passive failure zone.

These empirical methods generally are over-conservative as they fail to consider the lateral resistance provided by the soil berm. Also no quantitative information is provided on the performance of soil berms at the serviceability limit state where working loads act.

Methods which involve numerical calculations include:

- The use of a slope stability solution, where the forces from the wall and the weight of the berm are included.

- The use of a Coulomb wedge analysis for a selection of potential failure surfaces starting from the toe of the wall. The passive force acting on the front of the wall is assumed to comprise a triangular stress distribution acting between the top of the berm and the toe of the wall.

As each of the methods mentioned above have their limitations, research has been undertaken with the aim of improving these procedures. Three dimensional finite element analyses together with factor of safety calculations have been carried out by Easton *et al.* <sup>[53]</sup> to develop less empirically based methods of incorporating soil berms into temporary works design. This has resulted in design aids being produced in which charts are used to rationalise the design of soil berms and produce economy in their use. These charts enable suitable soil berm sizes to be chosen to provide the required factor of safety.

For further information regarding the design of soil berms using these improved methods, the reader is referred to TRL Report 398 *Design guidance on soil berms as temporary support for embedded retaining walls*<sup>[53]</sup>.



# 11 WATER-PROOFING

## 11.1 Introduction

Water-proofing is an essential requirement for all new basements. No matter how dry the soil around a building appears to be, there will always be ground moisture even above the water table. This will be supplemented by rainwater soaking into the ground from drains and any surface flooding. Without precautions, water will seep from the surrounding ground through any joints or cracks in walls and floors of a basement by a combination of flow under pressure and capillary action. Water vapour transfer may also occur from ventilated cavities in lined walls. Concrete wall construction for basements generally brings a long-term maintenance problem of drainage pumping to remove this leakage water. Intrinsically permeable materials such as concrete or masonry will leak water into the basement throughout its life. Steel sheet pile walls are intrinsically impermeable and should have no seepage problems, provided that the clutches are sealed.

Unfortunately, the available guidance on water-proofing of basements relates to the problems of concrete wall construction and does not cover the best solution of incorporating permanent steel sheet piled walls for a waterproof construction.

This Section is intended to complement the Code of Practice BS 8102: *Code of practice for the protection of structures against water from the ground* :1990 with an emphasis on providing up-to-date information on water proofing techniques specific to basements with steel sheet pile walls.

Guidance is given on:

- the requirements of the internal environment appropriate for specific uses
- the evaluation of external ground conditions
- the protection against seepage groundwater
- the water and vapour resistance of individual elements that comprise the structure to achieve required resistance.

## 11.2 The internal environment

For any part of the basement which is subject to groundwater pressure, the Approved Documents to the Building Regulations refer to BS 8102 for recommendations. BS 8102:1990 defines four 'grades' as a guide to the level of protection to suit basement use. In addition to the grade numbering defined in BS 8102, CIRIA Report 140 introduces terms for these grades. The four grades are:

### *Grade 1 'Basic utility'*

Basement usage is for car parking; plant rooms (excluding electrical equipment) and workshops. The performance level allows for some seepage and damp patches are tolerated.

**Grade 2 ‘Better utility’**

Basement usage is workshops, plant rooms (requiring drier environment) and retail storage areas. The performance level allows no water penetration but high humidity is tolerated.

**Grade 3 ‘Habitable’**

Basement usage is ventilated residential and working areas including offices, restaurants and leisure centres. The performance level is a dry environment.

**Grade 4 ‘Special’**

Basement usage for archives and stores requiring a controlled humidity environment. The performance level is a carefully controlled totally dry environment.

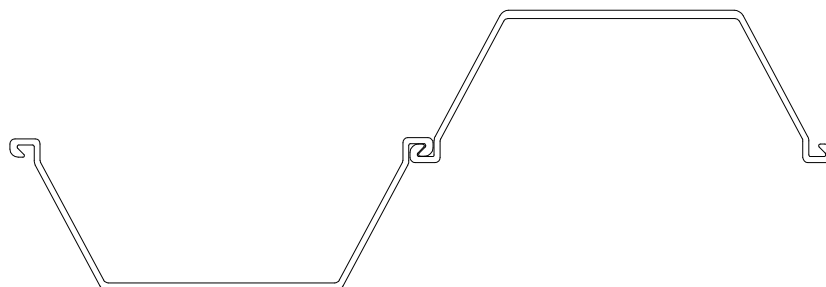
### 11.3 Forms of water resisting construction

BS 8102: 1990, which was written with concrete/masonry construction in mind, contains recommendations for either minimising or preventing the entry of water to the inner surfaces of basements concrete walls and identifies different types of construction to achieve this. Use of permanent steel sheet pile walls is not covered in BS 8102 but the type of construction is effectively a *structurally integral protection* system.

Frequently, a combination of protection types are proposed using new proprietary products which have been developed since the Code was published. The new clutch sealants described below are such products.

### 11.4 Prevention of water seepage through sheet pile walls

Steel sheet pile walls comprise a sequence of interconnected sheet pile sections which are joined together at the interlocks (Figure 11.1). Water seepage can only occur at the interlocks as steel plate is totally impervious to water seepage. However, if the gaps in the interlocks are filled, the wall can be made fully water tight.



**Figure 11.1** *Interlocking sheet piles*

To provide an adequate measure of water resistance to steel sheet pile basement walls, vertical and horizontal sealing systems have to be applied. Vertical sealing systems are applied to the interlocks of steel sheet pile walls, whilst horizontal

sealing systems prevent water ingress at the junction between the steel sheet pile wall and the base slab.

The following Sections provide information and guidance relating to the construction of water resistant steel sheet pile walls, the generic products and techniques that can be used and the performance that can be expected.

### 11.4.1 Vertical Sealant Systems

Numerous sheet pile clutch interlock sealing systems are available. These include:

- Non-swelling sealants
- Hydrophilic (water swelling) sealants
- Combination systems
- Welded interlocks.

The applicability of each the above types of systems is governed by a number of different parameters:

- driving conditions (both the nature of the soils and the method of installation);
- water pressure requirements
- permeability requirements
- durability requirements.

It is important to note that for the systems that rely on a pre-applied sealant, the integrity of the vertical joints is very much dependent on the driving conditions and the installation techniques used. Advice should be sought from piling manufacturers or specialist piling contractors on particular applications.

A guide to the relative performance of each of the sealant types is given in Table 11.1. The performance is, of course, dependent on the effective application of the sealants and appropriate installation techniques.

**Table 11.1** *Relative performance of sealant types*

Description	Permeability Class <sup>1</sup>	Applications	Installation Technique <sup>2</sup>		
			Percussive	Vibro	Silent
Water swelling paste	C	Temporary Works	A	A	A
Hot casting compound	B	Temporary Works	A	G	G
Flexible epoxy resin	A	Permanent or Temporary Works	A	NR	NR
Protected system	B	Permanent or Temporary Works	A	G	G
Site welding	I	Permanent Works	P	P	P

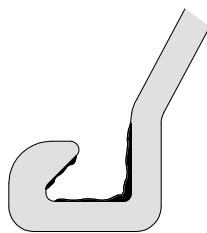
1. Permeability class gives an indication of the achievable levels of permeability the system can achieve. Class I= Impervious. Class A= less than  $10^{-8}$  m/s, Class B =  $10^{-6}$  to  $10^{-8}$  m/s, Class C = greater than  $10^{-6}$  m/s. The level of permeability achieved will depend upon the soil conditions, pile section, water head and the quality of the installation.
2. Installation technique refers to the most suitable method of installing the piles containing the sealant. P=Post installation sealant system, A=Acceptable method, G=Guidance should be sought regarding the best practice, NR = Not recommended.

### ***Non-swelling sealants***

There are a number of non-swelling sealants available for sealing sheet pile clutches. These include hot casting bituminous and vegetable oil based compounds in addition to a range of flexible resins (which are epoxy based). Hot casting compounds become viscous once heated having a treacle-like consistency which can be poured into pile interlocks. Flexible epoxy resins have a rubbery consistency and are pumped into the interlocks.

### ***Water swelling products***

Water swelling or hydrophilic sealants are urethane polymer products that, once in contact with water, expand to many times their original volume. They generally come in paste form and are brushed into pile interlocks. See Figure 11.2.



**Figure 11.2** *Hydrophilic sealant on the inside of the pile interlock*

### ***Application of pre-applied sealants***

The application of pre-applied sealants should be made under controlled conditions. Prior to application, the steel must be dry, clean and free from corrosion. This is usually best achieved under factory conditions immediately prior to delivery to site. If the piles are to be supplied in pairs, the intermediate interlocks can either be welded or treated with any one of the pre-applied sealants before being crimped.

### ***Installation of pre-applied sealants***

When installing piles containing an interlock sealant system, special attention is required, as sealants can cause an increase in the friction in the interlocks during pitching and driving. This means that the piles could experience some resistance during pitching and, in extreme circumstances, could cause draw down of the adjacent pile (if not held in place).

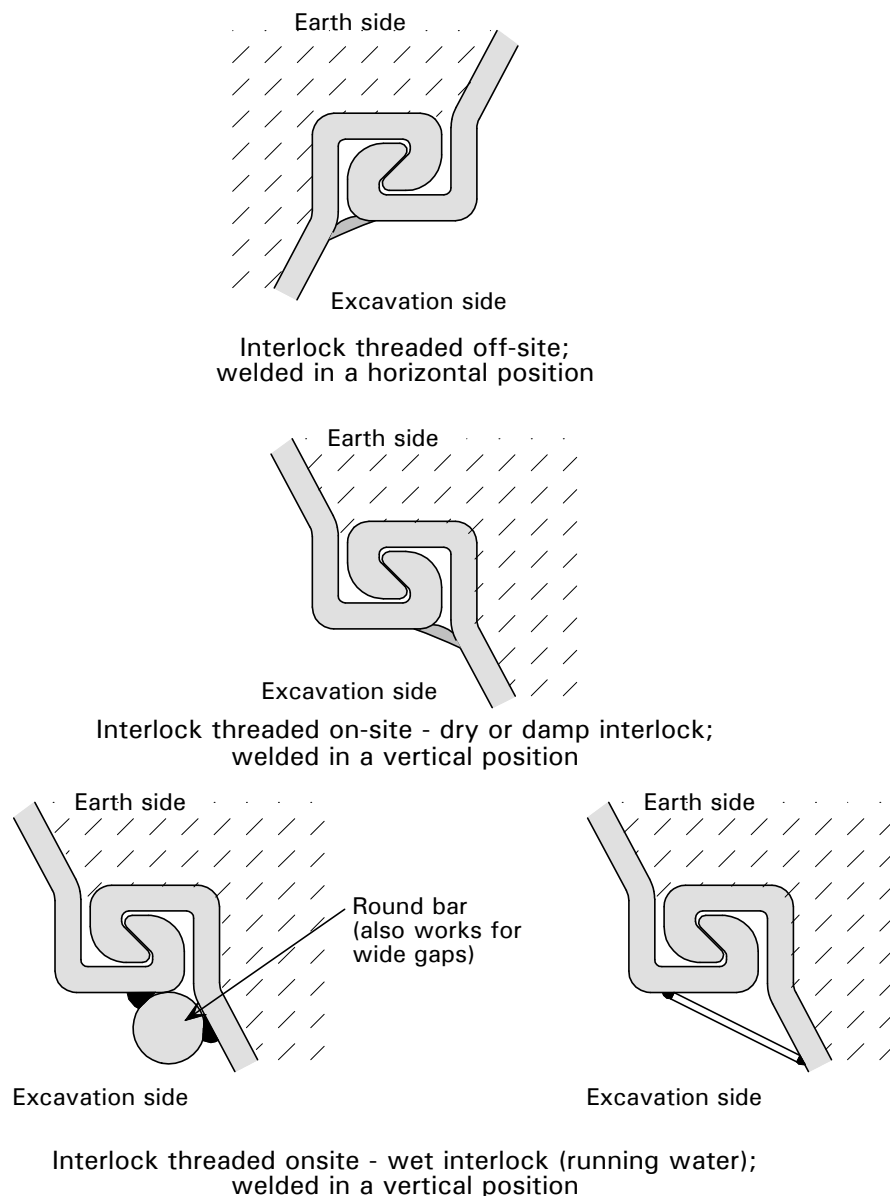
The heat generated by vibro driving may cause sealants to burn or decompose. If refusal is encountered it is recommended that vibro driving is stopped and the pile is driven to level with an impact hammer.

To ensure good joint integrity, it is important to control the alignment of the piles in the horizontal and vertical planes such that the sealant remains intact as the piles are driven. Any procedure that aids alignment of the piles (e.g. panel driving and driving in pairs) will also help to maintain the integrity of the sealant during installation.

### **Welding**

One way of achieving a water-tight seal is to weld the interlocks of steel sheet pile walls. The majority of electric arc welding processes are acceptable for sealing the interlocks of steel sheet piling threaded in the workshop or on-site. If the piles are to be driven individually, each of the interlocks will have to be welded in-situ. However, if the soil conditions are such that piles can be driven in pairs or triples, welding can be shared between the workshop and on-site. Intermediate pile interlocks can be welded in the workshop, whilst the remaining alternate pile interlocks can be welded once all the piles have been driven and the soil excavated. Various welding procedures can be used, a number of which are shown below.

There are two distinct disadvantages in using site welding when compared to pre-applied sealants: (i) it is much more time consuming and (ii) on site welding can only seal the exposed portion of the piles (pre-applied sealants achieve a seal over the entire length of pile). See Figure 11.3.



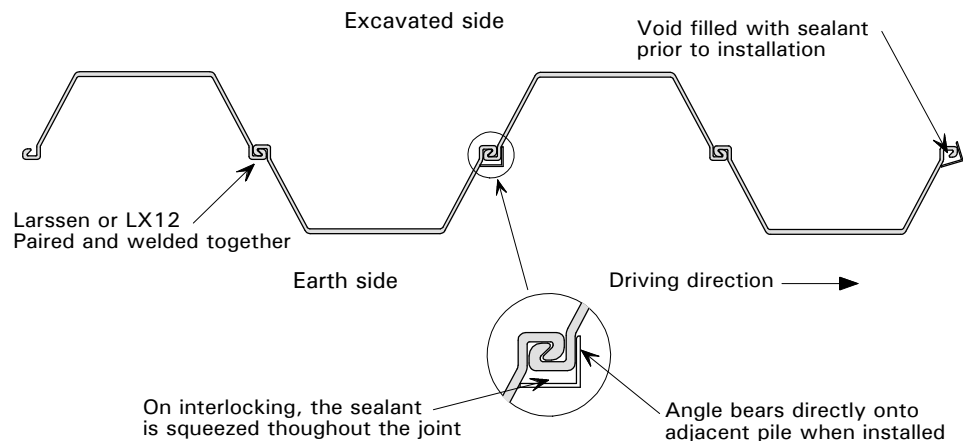
**Figure 11.3** *Methods for providing welded pile clutches*

The quality of welding is dependent on weld surface preparation and how dry the joint is. Also, proper pile driving practice must be undertaken using adequate pile section sizes for driving otherwise pile deviation can take place which may lead to difficult welding conditions.

### **Combination sealant systems**

Combination sealant systems use both mechanical seals and sealing compounds. One such system is the CS4200 (Figure 11.4) protected system (formerly known as Haltlock) from Corus, which has been successfully used on numerous projects.

The CS4200 system comprises a standard Corus Larssen or LX sheet pile, factory fitted on one side with a special steel angle section. The steel angle is welded to the clutch of one pile and a sealant compound is pumped into the remaining void filling the pile clutch. When an interlocking pile is introduced, the driving action causes this angle to displace and effectively 'bite' the incoming flange whilst, at the same time causing the sealant to diffuse throughout the joint.



**Figure 11.4** *The Corus CS4200 system showing the welded angle and the bituminous sealant*

## **11.5 Waterproofing and design of reinforced concrete floors**

Joints, and cracks emanating from joints, are the most vulnerable part of the structure to water ingress. Such ingress can often require expensive and time consuming remedial work to put right. In concrete joints most of the movement and shrinkage occurs during the initial six to twelve months. However, movement continues during the life of the structure due to structural and thermal movements, settlement and heave. As a result, joints and cracks can open to provide water paths into and around the structure. This is often compounded by inadequate preparation of the joint and poor workmanship during concreting.

The real problem is that these weaknesses only become visible once the joint is subject to hydrostatic pressure. BS 8007 recommends that water retaining structures are filled to test water tightness before completion. However, critical water-excluding structures, including basements constructed to BS 8007, are rarely subjected to hydraulic testing. Leaks only become apparent when the dewatering

system is removed or later in the lifetime of the structure, when the water table rises. Solutions exist to overcome leakage problems as outlined below.

### **11.5.1 Design**

The elements of design considered in this Section relate to reinforced concrete floor slabs that directly affect the water resistance of a basement, namely material properties, cracking, construction joints, movement joints, waterproofing treatments and drainage. Only a summary is provided on design of reinforced concrete slabs in this Section. More detailed information can be obtained from CIRIA Report 139.

BS 8110 states that '*water retaining structures...are more appropriately covered by other codes*'. BS 8007 provides recommendations for the design and construction of normal reinforced and prestressed concrete structures used for the containment or exclusion of aqueous liquids but '*does not cover .. the damp-proofing of basements*'. BS 8102 provides '*guidance on methods of dealing with and preventing the entry of water from surrounding ground into a building below ground level*'.

#### ***Properties of materials***

The properties of materials govern the permeability of the basement to water and vapour and also the durability of the construction. For a reinforced concrete slab, the design of the concrete mix is important, as a compromise must be made between the conflicting requirements of strength, high workability, high aggregate/cement ratio (to minimise early thermal and shrinkage cracking), low water/cement ratio (to ensure low permeability), and economy.

BS 8110 covers good design practice for durability in some detail, while BS 8007 is more concerned with the permeability of the concrete, and gives guidance on concrete specification.

BS 8007 recommends designing for severe exposure conditions with not less than 40 mm cover to the reinforcement and grade C35A concrete (i.e. concrete with a 28-day characteristic compressive cube strength of 35 N/mm<sup>2</sup>). It is stated that this classification is not in accordance with BS 8110, as high 28-day strengths may, with some types and proportions of constituent materials, lead to undesirably high cement contents. A reduction in water/cement ratio may be achieved by the use of plasticisers. Spacing of reinforcement (to control cracking) is important, and it is strongly advised that the design recommendations for area and spacing are followed.

#### ***Cracks***

It is important to control the width and occurrence of cracking in concrete structures. Early thermal cracking caused by restrained contraction or warping of the concrete as it cools immediately after pouring is generally more serious and occurs more frequently than cracks due to applied loads or resulting from shrinkage or thermal differentials in service. The guidelines to limit cracking are given by BS 8110 and are provided in the form of detailing rules deemed to satisfy the provisions in Part 1, and a calculation method with tables to assist in the estimation of the early thermal crack width in Part 2. A calculated maximum crack width of 0.3 mm is generally recommended.

BS 8007, specifically related to water-excluding structures, recommends limiting calculated maximum crack widths to 0.2 mm or 0.1 mm, according to defined circumstances.

BS 8102 deals more generally with the overall design of basements, and expects that membranes for tanked protection should be capable of accommodating cracks in structures of up to 0.6 mm. The document also assumes that concrete structures will be within the serviceability crack width limit specified in either BS 8110 or BS 8007 (according to the environmental grade, determined by its usage). It specifically recommends that Grade 1 basements should have calculated crack widths not exceeding 0.3 mm, in accordance with BS 8110: Part 2. Grades 2, 3 and 4 basements should comply with BS 8102 (calculated crack widths not exceeding 0.2 mm).

### ***Construction joints***

Detailed consideration of the position and treatment of construction joints is covered by BS 8110. This is not developed much further by BS 8007, which states 'It is not necessary to incorporate waterstops in properly constructed construction joints'. BS 8102 recommends the use of waterstops.

### ***Movement joints***

BS 8110 gives guidance on the calculation of movement in a structure and on where to provide joints, in addition to descriptions of contraction, expansion, hinged and settlement joints. BS 8007 introduces the partial contraction joint discusses the spacing of the joints, and fully describes joint fillers, waterstops and joint-sealing compounds. BS 8102 states that allowance should be made in tanking details for substrate movement.

### ***'Waterproofing' treatments'***

Waterproofing' treatments are discussed in detail in BS 8102. Although general good practice is given on preparing a structure for tanking, the different types of materials that are described are not comprehensive as recent developments are not included. The designer should contact appropriate waterproofing manufacturers for details of their latest products.

### ***Drainage***

BS 8110 recognises the need for exposed surfaces to be freely drained. Drainage is not covered by BS 8007. General good practice is given in BS 8102.

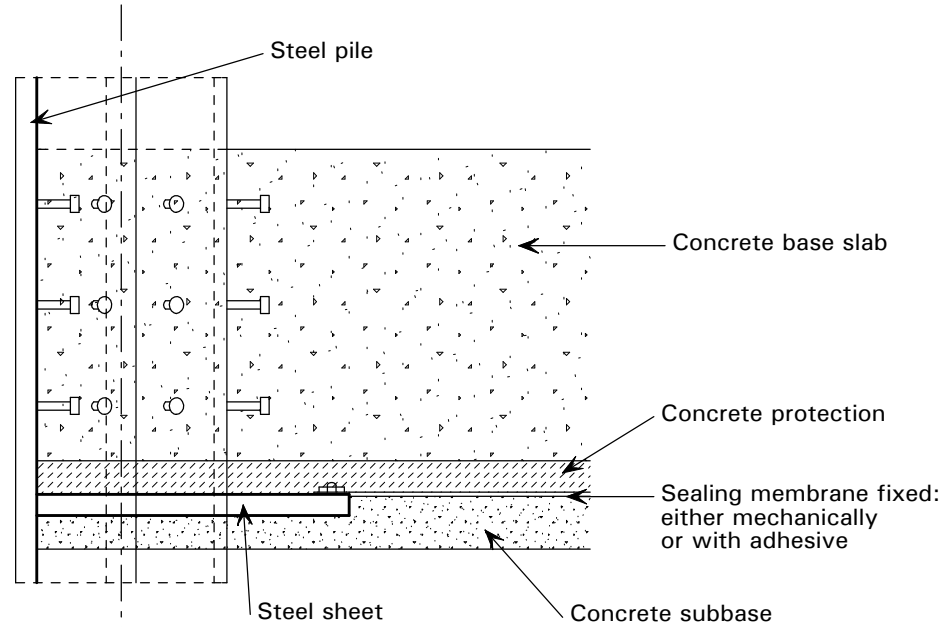
## **11.6 Steel to reinforced concrete slab connection**

Sealing to form a water-tight connection between two types of dissimilar construction materials are required at the interface between a steel sheet pile wall and an external reinforced concrete floor slab. Both 'passive' and 'active' waterproofing systems are available, with the former gaining more market share owing to its distinct advantages.



### 11.6.1 Older 'Passive' waterproofing systems

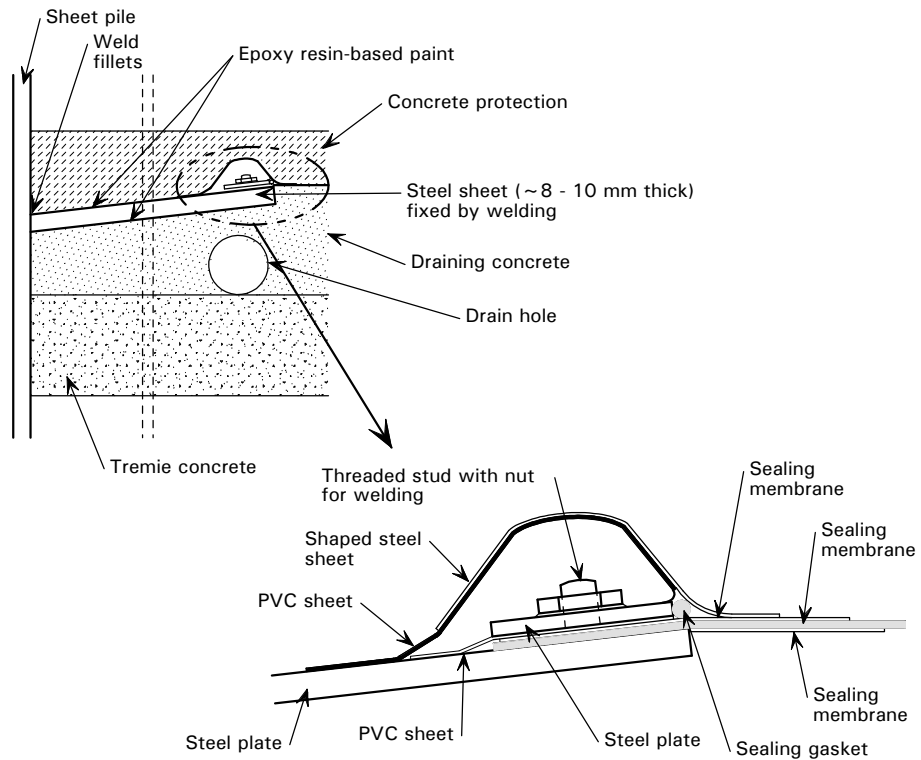
One form of wall/slab connection using conventional passive waterproofing where low to average loads act is shown in Figure 11.5.



**Figure 11.5** *A passive sealing system at the interface between the steel sheet pile wall and the reinforced concrete base slab*

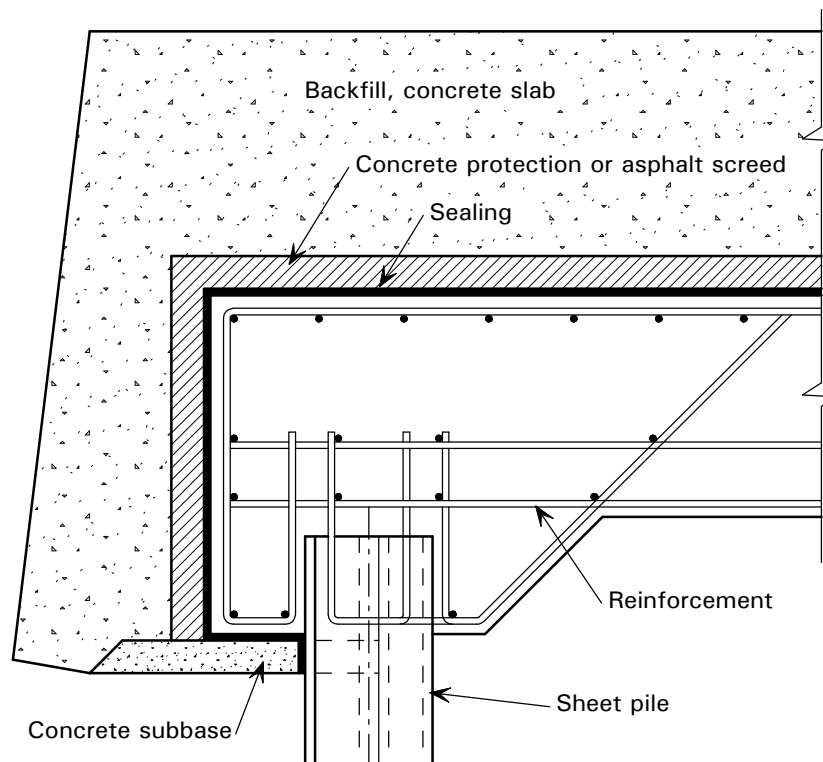
To provide an adequate barrier to water ingress through the reinforced concrete slab, a membrane sheet is laid on top of the concrete blinding sub-base. This membrane is fixed mechanically (or alternatively with adhesive) to a steel plate ('puddle' plate), at the corresponding elevation of the proposed membrane. A concrete protective layer is then applied to protect the membrane during the working. The 'puddle' plate is cut to the profile of the sheet pile and welded to the sheet pile, hence the membrane can be terminated at the edge of the plate to provide an effective waterproof seal. The steel plate itself increases significantly the length of the potential water seepage path at the steel-concrete interface. To prevent any water seepage through any potentially poorly welded connection, a hydrophilic sealant can be applied to the top of the steel plate at the sheet pile-steel plate interface.

Where larger loads act on the base slab, a more appropriate connection detail can be used as shown in Figure 11.6



**Figure 11.6** *Pile-slab connection detail for transmitting large loads*

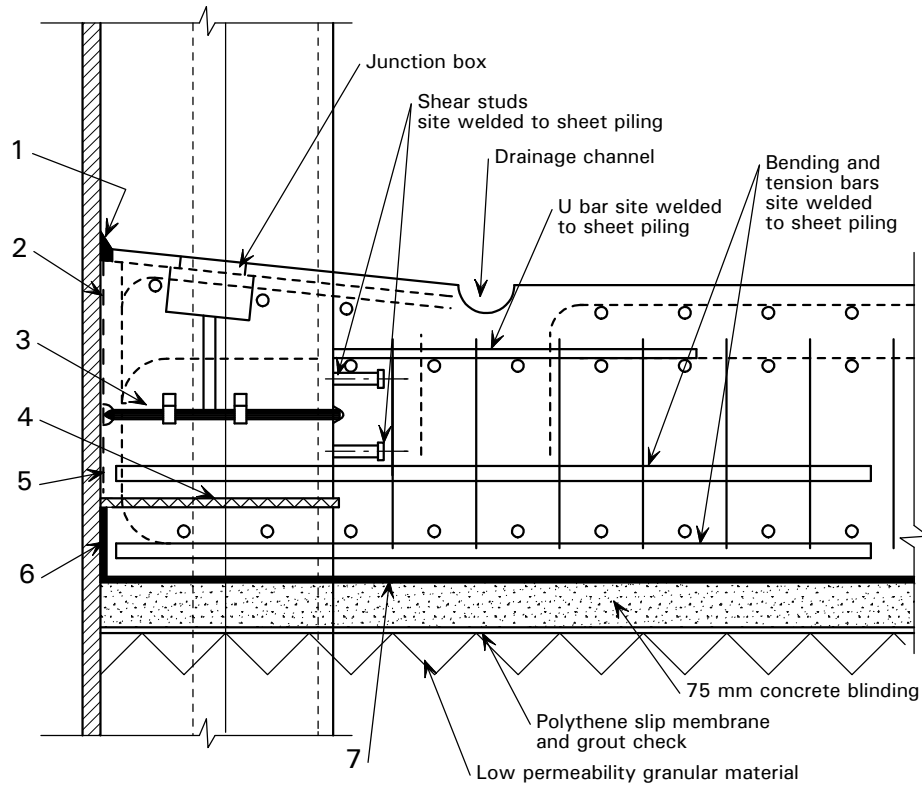
A typical detail for the ground floor slab and the top of the sheet pile wall is shown in Figure 11.7



**Figure 11.7** *A possible connection between top of sheet pile wall and the ground floor slab*

### 11.6.2 Recent advances and 'active' waterproofing

Due to recent technological advances in waterproofing products, active systems have greatly improved the water tightness of steel to concrete connections and are becoming the preferred option. Unlike the conventional passive method shown in the figures above, an injection system will remain active after the concreting of the base. See Figure 11.8.



- 1 10 mm wide x 10 mm deep chase formed in top of slab with pourable sealant.
- 2 100 mm wide adhesive waterproofing tape membrane with permanent mechanical bond to concrete, bonded to sheet pile at construction joint location.
- 3 Hose injection waterproofing system clipped to face of sheet piles.
- 4 20 mm x 5 mm hydrophilic waterstop bonded to sheet piling.
- 5 Double sided self adhesive rubber/bitumen waterproofing membrane securely bonded to sheet pile, water bar and hydrophilic strip.
- 6 P.V.C. waterbar returned 125 mm.
- 7 P.V.C. waterbar with co-extruded hydrophilic elements at construction joints.

**Figure 11.8** *Waterproofing a steel sheet pile to concrete base slab connection using an active injection system*

The injection system provides a conduit to the inside faces of the connection without drilling or damage to the concrete itself. This conduit allows the watertight integrity of the connection to be proven by pressurised water injection and provides for the sealing of any identified leaks. The penetrating resin is injected under pressure to seal the joint and any emanating cracks, to prevent water ingress and tracking along the joint. The integrity of the joint is proven once pressure can be maintained. The injection resin cures to form an elastic non-shrink filler for lifetime protection.

In parallel with the advances made in injection systems, advances have also been made in the waterproofing of the concrete slab. High density polyethylene (HDPE) sheeting are now available which physically isolate the slab structure from the surrounding ground. The sheet comprises three layers: the HDPE layer which prevents the ingress of water, vapour and gas; a layer of special conformable adhesive; and a white protective coating. These layers work together to form a microscopic integral seal to wet concrete poured against it, hence it is unaffected by settlement of the substrate. This permanent bond prevents migration of water between the membrane and the concrete, eliminating a common cause of leaks. Importantly, it does not require protection and can be trafficked immediately after laying. As it is durable and easy to handle, liquid membrane products can be used to sealed lap joints and seal the sheet membrane to the steel sheet pile. As there is no requirement for the 'puddle' plate (as shown in Figure 11.7), the costly and time consuming welding operation can be omitted.

In basements where a controlled amount of water seepage can be tolerated e.g. in an underground car park, the waterproof membrane may be omitted (see Bristol Millennium Car Park case study, Section 2.1).

## **11.7 Water seepage through steel sheet pile interlocks**

Recently, research has been undertaken by Delft Geotechnics to develop a consistent methodology that will enable the magnitude of water seepage that takes place through the interlocks of steel sheet piles to be quantified. This method is included in EN 12063 *Execution of special geotechnical work - Sheet pile walls*.

If this analysis method is to be used, it is important to consider the effects of the following:

- frequency of clutches
- tightness of clutches
- driving tolerance
- clutch geometry

and how they may affect the confidence of the results.

Further information on the method can be obtained from Sellmeyer <sup>[54]</sup>.

## **12 FIRE RESISTANCE AND FIRE PROTECTION OF BASEMENTS**

The effects of fire need to be considered in the design of basements. This section provides information necessary to help an engineer to understand the principles of fire protection provision for steel sheet piling in basements.

### **12.1 Fire concepts**

Fire resistance is used to characterise the performance of elements of construction in fire.

The standard definition of fire resistance is "the ability of a component or construction to satisfy for a stated period of time, some or all of the appropriate criteria specified in the relevant part of BS 476". The three basic criteria are load bearing capacity, integrity and insulation.

Time is generally used as the measure for defining performance in fire. However, it is important to recognise that fire resistance time is the time to failure in a standard (BS 476 or equivalent) test. The time taken for a phenomenon to occur in a test may possibly relate to the time of occurrence of that phenomenon in a building fire but it is not necessarily a reflection of the time a structure will resist fire in a building. In general terms, all that can be assumed is that an element which performs better than another in a test is likely to perform better than the other in an actual fire in a basement. For example, 60 minutes fire resistance does not mean that a building will be stable for 60 minutes in a real fire. However, the contrary may be also true, an element of structure with 60 minutes fire resistance may remain stable for far longer than 60 minutes if the standard fire is more severe than the real fire.

Escape time is a notional time that is used as the basis for defining the physical characteristics and acceptability of different building forms when considering the speed for people to evacuate the building. It should not be confused with the real time that it takes for people to escape the building: that time is a function of a wider range of human factors such as threat, perception, familiarity and motivation.

### **12.2 The need for fire resistance**

An element of structure needs to have fire resistance primarily to satisfy the life safety requirements of The Building Regulations.

Fire resistance may also be required in order to:

- protect property
- meet the safety objectives of other legislation
- meet the requirements of insurers
- meet specific client requirements.

## 12.3 The Building Regulations

Provision for structural fire resistance of buildings is embodied in Part B of Schedule 1 to the Building Regulations, 1991. The Regulations themselves do not state fire resistance standards. Instead the legal requirements are expressed in functional terms, the functional requirement for structural performance in fire being that:

- B3(1) The building shall be designed and constructed so that, in the event of fire, its stability will be maintained for a reasonable period.
- B3(2) A wall common to two or more buildings shall be designed and constructed so that it adequately resists the spread of fire between buildings.
- B3(3) To inhibit the spread of fire within the building, it shall be subdivided with fire-resisting construction to an extent appropriate to the size and intended use of the building.
- B3(4) The building shall be designed and constructed so that the unseen spread of fire and smoke within concealed spaces in its structure and fabric is inhibited.

Demonstration of compliance with the functional requirement may be achieved by one of the following:

- Compliance with the guidance in Approved Document B.
- Compliance with local regulations such as the Local Building Acts (Amendment) Act 139.
- Meeting defined objectives set out as part of a Fire Safety Engineering strategy.

## 12.4 Approved Document B

In England and Wales the most common source of information on what constitutes a reasonable period is Approved Document B to the Building Regulations 1991 (2000 Edition). This document interprets the requirements of the Building Regulations and contains detailed provisions for the maintenance of safety in fire. These are intended to provide guidance for some of the most common building situations. In practice they are used in the majority of buildings.

The underlying philosophy of the Approved Document is to contain the fire within the compartment of origin for sufficient time to allow the building's occupants who are able to escape, the opportunity to do so via the available escape routes, and to allow the fire brigade personnel the opportunity to rescue others. The maximum allowable size of such fire compartments relates to risk to life and therefore to the use and occupancy of the building.

Approved Document B is subdivided into five sections:

- B1 Means of warning and escape
- B2 Internal fire spread (Linings)
- B3 Internal fire spread (Structure)
- B4 External fire spread
- B5 Access and facilities for the fire service

Approved Document B offers some general guidance as to how in the view of the Secretary of State, the various functional requirements may be satisfied. This is only a view, as the final arbiters on matters of law are the courts. It is neither mandatory nor deemed to satisfy and contains the following statement:

"There is no obligation to adopt any particular solution contained in an Approved Document if you prefer to meet the requirements in some other way."

The document goes on to suggest other means to demonstrate compliance by stating that:

"Fire safety can provide an alternative approach to fire safety. It may be the only viable way to a satisfactory standard of fire safety in some large and complex buildings and in buildings containing different uses. Fire safety engineering may also be suitable for solving a problem with an aspect of the building design which otherwise follows the provisions of the document".

#### **12.4.1 Fire resistance requirements in Approved Document B**

The required fire resistance of an element of structure in a basement is a function of the purpose of the basement and also its depth.

Table A2 of Approved Document B sets out the minimum periods of fire resistance for elements of structure. These are summarised in Table 12.1. For basements, the fire resistance of elements of structure are based on the depth of the basement, whilst for superstructure it is the height of the building. Fire resistance for the building is given as the basement may be part of a multi-storey building above ground. If this is the case, the floor at ground surface level will be required to have the maximum of the fire resistance required of the basement and that of the superstructure.

**Table 12.1** *Minimum periods (minutes) of fire resistance (based on Approved Document B Table A2)*

Purpose of building	Minimum periods (minutes)					
	Basement storey (§) including floor over		Ground or upper storey			
	Depth (m) of a lowest basement		Height (m) of top floor above ground, in a building or separated part of a building			
	> 10	≤ 10	≤ 5	≤ 18	≤ 30	> 30
1. Residential (domestic)						
a. flats and maisonettes	90	60	30*	60** †	90**	120**
b. and c. dwelling houses	not relevant	60	30*	60@	not relevant	not relevant
2. Residential:						
a. Institutional §	90	60	30*	60	90	120#
b. Other residential	90	60	30*	60	90	120#
3. Office:						
- not sprinklered	90	60	30*	60	90	not permitted
- sprinklered (2)	60	60	30*	30*	60	120#
4. Shop and Commercial:						
- not sprinklered	90	60	60	60	90	not permitted
- sprinklered (2)	60	60	30*	60	60	120#
5. Assembly and recreation:						
- not sprinklered	90	60	60	60	90	not permitted
- sprinklered (2)	60	60	30*	60	60	120#
6. Industrial:						
- not sprinklered	120	90	60	90	120	not permitted
- sprinklered (2)	90	60	30*	60	90	120#
7. Storage and other non-residential						
a. any building or part not described elsewhere:						
- not sprinklered	120	90		90	120	not permitted
- sprinklered (2)	90	60	30*	60	90	120#
b. car park for light vehicles						
- Open sided car park (3)	n/a	n/a	15* +	15* +	15* +	60
- Any other car park	90	60	30*	30*	90	120#

§ The floor over a basement (or if there is more than one basement, the floor over the topmost basement) should meet the provisions for the ground and upper storeys if that period is higher.

\* Increased to a minimum of 60 minutes for compartment walls separating buildings.

\*\* Reduced to 30 minutes for any floor within a maisonette, but not if the floor contributes to the support of the building.

§ Multi-storey hospitals designed in accordance with the NHS Firecode documents should have a minimum 60 minutes standard.

# reduced to 90 minutes for elements not forming part of the structural frame.

+ Increased to 30 minutes for elements protecting the means of escape.

† Refer to para 8.10 regarding the acceptability of 30 minutes in flat conversions.

@ 30 minutes in the case of 3 storey dwelling houses, increased to 60 minutes minimum for compartment walls separating buildings.

Notes:

1 Refer to Table A1 for the specific provisions of test.

2 "Sprinklered" means that the building is fitted throughout with an automatic sprinkler system meeting the relevant recommendations of BS 5306 Fire extinguishing installations and equipment on premises, Part 2 Specification for sprinkler systems; i.e. the relevant occupancy rating together with the additional requirements for life safety.

3 The car park should comply with the relevant provisions in the guidance on requirement B#, Section 12.



## 12.5 Elements of structure

In Building Regulations the term 'elements of structure' is applied to main structural elements such as structural frames, floors and walls. Compartment walls are treated as elements of structure although they are not necessarily loadbearing. Only members which are classed as 'elements of structure' are required to have fire resistance. 'Elements of structure' as defined in Approved Document B, Appendix E may be:

- Members forming part of the main structural frame of a building.
- Loadbearing walls or loadbearing parts of walls.
- A floor.
- Some galleries.
- A compartment wall.

Loadbearing elements may or may not have a fire separating function. Similarly, fire separating elements may or may not be loadbearing.

There are circumstances where guidance indicates that structural members may not need fire resistance. These exclusions usually include:

- A structure that only supports a roof, unless the roof performs the function of a floor (e.g. as a means of escape or as a car park) or is essential for the stability of a wall which requires fire resistance.
- The lowest floor of a building.
- A platform floor.
- Some galleries.

Basement roofs may be treated as elements of structure because in many cases they:

- are essential for the stability of an external wall which needs to have fire resistance.
- serve the function of a floor i.e. a loaded ground level floor or as a means of escape.

Fire resistance requirements will therefore apply. See also Section 12.4.1.

## 12.6 London Building Act 1939: London District Surveyors' Guide to Section 20 Buildings

Buildings located within the inner London area are subject to the requirements of the London Building Act 1939. Within this act, the precautions against fire in buildings is covered by Section 20. This section ensures that "proper arrangements will be made and maintained for lessening so far as is reasonably practicable danger from fire in buildings."

The development of functional regulations in 1985 led to the revision of Section 20 via Statutory Instrument 1936, Building and Buildings, The Building ( Inner London ) Regulations 1985. This revision led to the inclusion of a number of additional clauses by which the District Surveyor may impose conditions for

provision and maintenance of fire alarms, automatic fire detection systems, effective means of removing smoke etc. It also set out that the revised Section 20 applied to buildings over 25 m when the area of the building exceeds 930 m<sup>2</sup>.

In 1990 the London District Surveyors' Association published *Fire safety guide, No. 1: Fire safety in Section 20 Buildings*<sup>[55]</sup>. This document contains detailed information on fire resistance requirements within the inner London area. In theory, it is for guidance only but in practice, it tends to have the status of legislation when used by District Surveyors.

Within the guide, Section 4.02, Fire Resistance of Elements of Structure, sets out the following:

1. Elements of structure should have a four hour standard of fire resistance within multi-storey buildings used for bulk storage of flammable or combustible liquids or solids (e.g. boiler room and oil storage areas). This is reinforced on Part 3 of the guide, Special Fire Risk Areas, Section 2, Clause B, which states, "Where oil storage adjoins, or is within a building, it should be enclosed by walls and a roof of non combustible construction having a fire resistance of not less than four hours."
2. In all other cases the elements of structure should have the standard of fire resistance to meet B2/3/4.

Clause 2 above makes reference to B2/3/4. This is Approved Document B. See Section 12.4.

Chapter 3, Special fire risk areas, Section 3.08, Underground car parks makes special mention of basement car parks. It differentiates between small and other underground car parks where 'small' means under 500 m<sup>2</sup> in floor area. The main provisions as they apply to structural precautions are as follows:

- The car park should be separated from any other part of the building by non-combustible construction having not less than four hours fire resistance.
- Small car parks should be separated from any other part of the building by non-combustible construction having not less than two hours fire resistance.
- All supporting members thereto (i.e. having a separating function) should have a similar fire resistance.
- The elements of structure within the car park together with any necessary compartment walls and floors should not have less than two hours fire resistance.
- The car park should be subdivided into compartments so that each storey forms a separate compartment and no compartment exceeds 14000 m<sup>3</sup> in extent.

## **12.7 Fire resistance requirements for sheet pile walls in basements**

The fire resistance requirements for elements of structure outlined in the previous Sections exist primarily in order to ensure life safety by preventing structural collapse and also to ensure that compartmentation remains intact to prevent the spread of fire, smoke and hot gases.

On this basis, it is recommended that, where collapse of a steel sheet piling retaining wall in a basement will not result in additional structural collapse or allow the passage of hot gasses into other compartments, the steel will not require protection. Where the collapse of the steel sheet piling retaining wall in a basement will result in additional structural collapse and thus create a risk to life and to the integrity of the building's compartmentation, the steel will require protection, but only in the locality of the load path. The latter event will probably occur where the sheet piling has a function other than simple earthwork retention, e.g. it also has to take vertical load from the structure. If the piling has to take load from the structure but its collapse will not precipitate additional collapse, a case could still be made for not protecting it.

## **12.8 Compartmentation requirements in basements**

In large basements, compartment walls used to divide space into compartments to protect particular hazards, will need to comply with fire resistance requirements, even when not load bearing.

Provision of compartmentation in basements in non-residential buildings is covered in Approved Document B, Section 9.20. Maximum sizes of fire compartments are given in Table 12 of Approved Document B. Only the floor of the ground storey needs to be a compartment floor if the lower basement is at a depth of not more than 10 m. All basement storeys need to be separated by compartment floors if any storey is at a depth greater than 10 m.

## **12.9 Fire engineering solutions: Structural fire design - Design codes**

Fire safety engineering can provide an alternative approach to fire safety. (See Section 12.4). It can be seen as an integrated package of measures designed to achieve the maximum benefit from the available methods for preventing, controlling or limiting the consequences of fire. In terms of structural stability, fire safety engineering is aimed at adopting a rational scientific approach which ensures that fire resistance/protection is provided where it is needed and expense is not incurred needlessly to provide an illusion of safety. Fire safety engineering is most effective where it can be demonstrated that the prescriptive requirements of documents, such as Approved Document B or the London District Surveyors' Association *Fire safety guide, no. 1: Fire safety in Section 20 buildings*, are not appropriate to the risk in the basement and can be reduced. A fire safety engineering assessment should, wherever possible, be carried out alongside the initial design.

The development of fire engineering solutions will be supported in 2002 by the publication of BS 7974, the new British Standard for fire safety engineering.

## **12.10 Structural fire protection**

Various generic and proprietary fire protection systems are used to protect structural steelwork. Manufacturers and/or specialist contractors offer comprehensive information on characteristics of materials, test results, advice about suitability for particular applications and installation procedures. A list of

manufacturers and installers is available from the UK trade association, The Association for Specialist Fire Protection (ASFP). However, not all manufacturers and applicators in the UK are members of the ASFP and this list is therefore not inclusive.

The thickness of fire protection material required to satisfy a specific fire resistance period can be selected from authoritative material performance data sheets published by the manufacturers or from the 'Yellow Book'<sup>[56]</sup>. This is very much out of date now, although it does still contain some good information on the theory behind fire protection. A new edition is in production.

### **12.10.1 Generic fire protection materials**

Guidance on the fire performance and use of generic materials such as concrete etc is presented in the Building Research Establishment publication *Guidelines for the construction of fire resisting structural elements*<sup>[57]</sup> and in Appendix 2 of the 'Yellow Book'.

### **12.10.2 Proprietary fire protection materials**

The most common types of fire protection systems available can be classed into three main product groups

- boards and blankets
- sprays
- intumescent coatings

Boards, blankets and sprays are non-reactive whilst intumescent coatings are reactive.

Most proprietary fire protection materials are designed for use in dry internal environments, i.e. Category C1 environments as defined in ISO 12944 Part 2. For other environments, for example underground car parks, advice should be taken from the fire protection manufacturers as to their product's suitability.

#### ***Boards and blankets***

Blankets, semi-rigid and rigid boards are used as dry forms of fire protection installed in-situ as either profile or boxed protection. Base materials include ceramic fibres, calcium silicate, rock fibre, gypsum and vermiculite. The principal advantages of using boards are:

- rigid boards offer a clean, boxed appearance which may be pre-finished or suitable for further decoration
- application is dry and usually does not have a significant effect on other trades
- boards are factory manufactured hence thickness can be guaranteed
- boards can be applied on unpainted steelwork.

However, fitting around complex details may be difficult and board systems may be slower to apply than other methods.

Blanket or flexible fire protection systems have been developed as a response to the need for a cheap alternative to sprays but without the adverse effects on construction program often associated with wet application.

Board systems are the most popular type of fire protection in the UK. They are widely used where the protection system is in full view. Specifiers should be aware that board systems that can provide an aesthetic finish will be more expensive than purely functional systems. Up to 240 minutes fire resistance can be provided.

### ***Sprays***

Sprays are cement or gypsum based materials containing mineral fibre, expanded vermiculite, expanded perlite and/or other lightweight aggregates or fillers. The principal advantages of sprays are:

- spray protection can usually be applied for less than the cost of the cheapest board. As the cost of spray material is low compared to that of getting labour and equipment on site, costs do not increase in proportion to resistance times
- it is easy to cover complex details
- some materials may be used in external or corrosive environments
- some materials may be applied on unpainted steelwork.

However, sprays are not visually appealing and as it is a wet trade, significant knock on effects on the construction program can result in an increase in the overall cost construction or prolonged construction duration. Difficulty may also be encountered when reinstating the protection if it is necessary after services installation.

Sprays are generally the least expensive form of fire protection. These materials can provide up to 240 minutes fire resistance.

### ***Intumescent coatings***

Intumescent coatings can be classified as either 'thin film' or 'thick film (mastics)'. Thin film intumescent coatings account for the majority of systems used in general construction, whilst thick film intumescent coatings are commonly used for the heavy industrial petrochemical and offshore oil and gas industries.

Thin film intumescent coating systems are similar in appearance to conventional paints and are applied either by airless spray, brush or roller, and either on-site or off-site. General guidance on the selection and use of intumescent coatings can be found in BS 8202-2, whilst guidance on the use of offsite application of intumescent coatings is given in SCI publication *Structural fire design: Off-site applied thin film intumescent coatings*<sup>[58]</sup>.

The principal advantages of thin film intumescent coatings are:

- the shape of the underlying steel can be expressed
- it is attractive and decorative finishes are possible
- complex details are easily covered.

However, typical application costs are higher than sprays, it is a wet trade and most intumescent coatings are economic only up to 60 minutes fire resistance. It is possible to achieve up to 120 minutes fire resistance but this is likely to be an expensive alternative. Potential problems may also occur in corrosive environments (i.e. anything other than C1) and a maintenance programme and guarantees may be required

### **12.10.3 Inherently fire resistant forms of steel construction**

Unprotected steel can be used in basements if collapse does not compromise compartmentation or structural stability. This is particularly relevant to sheet piling used purely to retain earthworks and not carrying other loads. (See Section 12.7). Particular forms of construction can also be used to provide significant levels of fire resistance without the use of applied protection.

#### ***Columns***

Fully exposed I or H steel sections can achieve 30 minutes fire resistance in certain limited cases where the loading is low or the section is large. This is generally impractical in basements. However, there are a number of ways of achieving fire resistance for columns without applied fire protection.

#### ***Blocked-in columns***

Placing concrete blocks between the flanges of Universal Columns can increase the fire resistance to 30 minutes. See BRE Digest 137<sup>[59]</sup>.

#### ***Web-infilled columns***

Unreinforced concrete filled between the flanges of columns can achieve 60 minutes fire resistance. The concrete is contained between web stiffeners and held in place with shear connectors. See SCI Technical Report *Fire resistance of web-infilled steel columns*<sup>[60]</sup>.

#### ***Partially encased columns***

The use of reinforced concrete between the flanges of columns can achieve 120 minutes fire resistance. See EC4-1-2 *Structural fire design* for further information.

#### ***Concrete filled structural hollow sections***

Filling a hollow section with reinforced concrete can provide up to 120 minutes fire resistance, whilst for concrete which is unreinforced, 60 minutes fire resistance is achievable. See SCI publication: *The design of steel framed buildings without applied fire protection*<sup>[61]</sup>, Corus publication: *Design manual for concrete filled columns*<sup>[62]</sup> and ENV 1994-1-2 Structural fire design.

#### ***Floor systems***

Fully exposed I or H steel sections acting as beams can achieve 30 minutes fire resistance in certain limited cases where the loading is low or the section is large. This is generally impractical in basements as the fire ratings required are generally greater than 30 minutes.

Various floor systems can be used for the floors of basements. The choice of a particular floor system is dependent on the functionality of the basement and the

floor loading and also the method of construction. A number of these floor systems incorporate ways of achieving fire resistance for beams with reduced, or without, applied fire protection.

#### *Composite Floors*

Many basements are designed using composite slabs with shallow decking supported by steel beams. Generally these beams will require fire protection, however a great many secondaries can be left unprotected if the floor is designed according to the guidance provided in the SCI Publication *Fir safe design: Anew approach to multi-storey steel-framed buildings*<sup>[63]</sup>. Some compensating features, such as increased mesh size, may be required.

#### *Fabricated Slimflor beams*

Slimflor beams do not generally require fire protection for fire resistance periods up to 60 minutes. For higher fire resistance periods boards or intumescent coatings are usually applied to the exposed bottom flange only. Thin coat intumescent coatings specifically tested on Slimflor sections are available<sup>[64]</sup><sup>[65]</sup>.

#### *Rectangular hollow section (RHS) Slimflor edge beam floors*

RHS Slimflor beams do not generally require fire protection for fire resistance periods up to 60 minutes. For higher fire resistance periods boards or intumescent coatings are usually applied to the exposed bottom flange only. Thin coat intumescent coatings specifically tested on RHS *Slimflor* sections are available<sup>[66]</sup>.

#### *Slimdek system using Asymmetric beams (ASB) floors*

Slimdek beams do not generally require fire protection for fire resistance periods up to 60 minutes. For higher fire resistance periods, or where the web of the beam is penetrated by service holes, boards or intumescent coatings are usually applied to the exposed bottom flange only. Thin coat intumescent coatings specifically tested on Slimdek sections are available<sup>[67]</sup>.

#### *Shelf angle floor beam floors*

It is possible to achieve up to 60 minutes fire resistance without added fire resistance using a SAFB. This can be achieved by increasing the area of the beam protected by the floor slab. In practical terms however it is only economically possible where the beam is relatively lightly loaded. Where fire protection is required, the thickness is calculated on a section factor based on the exposed perimeter and the full cross-sectional area<sup>[68]</sup>.

#### *Partially encased composite beam floors*

The use of reinforced concrete between the flanges of beams can achieve 180 minutes fire resistance. See ENV 1994-1-2 *Structural fire design* for further information.





# 13 PILE DRIVING AND INSTALLATION

This Section describes sheet pile driving installation methods and equipment, driving analysis methods, construction tolerances, and the environmental implications of sheet pile driving.

## 13.1 Pile installation

Sheet piles can be installed by a variety of methods and equipment. There are three types of installation equipment which operate by impact, vibration or by jacking. Each has particular advantages and disadvantages, and the final choice is, in most cases, a balance between speed and economy of installation. A further deciding element is the increasing concern for noise and vibration control to which the industry has responded with the development of new installation techniques and equipment.

### 13.1.1 Steel pile installation tolerances

Information on tolerances that are achievable using commonly available pile driving equipment and methods is quoted in the Institution of Civil Engineers publication *Specification for piling and embedded retaining walls*<sup>[69]</sup> and specifications issued by the Federation of Piling Specialists<sup>[70]</sup>, the CEN Standard EN 12063 *Execution of special geotechnical works - Sheet-pile walls*, and TESPA publication *Installation of steel sheet piles*<sup>[71]</sup>.

Table 13.1, is included in the TESPA publication and represents tolerance levels for sheet piling which should not be too onerous to achieve but will give results that are visually acceptable - an important feature for permanent exposed sheet piling.

**Table 13.1** *Steel pile driving tolerances (TESPA)*

Type of pile and method of driving	For panel drive method
Deviation normal to the wall centre line at pile head	± 50 mm Dependent on equipment used
Finished level deviation from a specified level:	
of pile head, after trim	± 20 mm
of pile toe	± 120 mm
Deviation from specified inclination measured over the top 1 m of wall:	
normal to line of piles	± 1%
along line of piles	± 0.5%

Accuracy of alignment will also be affected by pile stiffness, the driving equipment, and the experience of the workforce. Use of pile guide frames, which are often formed from Universal Beams aligned with their webs horizontal, will ensure that good alignment of the sheet piles is achieved.

## 13.2 Installation methods

Sheet piles for use in permanent construction must be installed very accurately as the finished structure must not only be structurally sound but visually acceptable. The method of installation selected must be capable of achieving that accuracy in plan position and verticality during the 'driving' operation. There are two fundamentally different methods of installation available, namely

- panel driving
- pitch and drive.

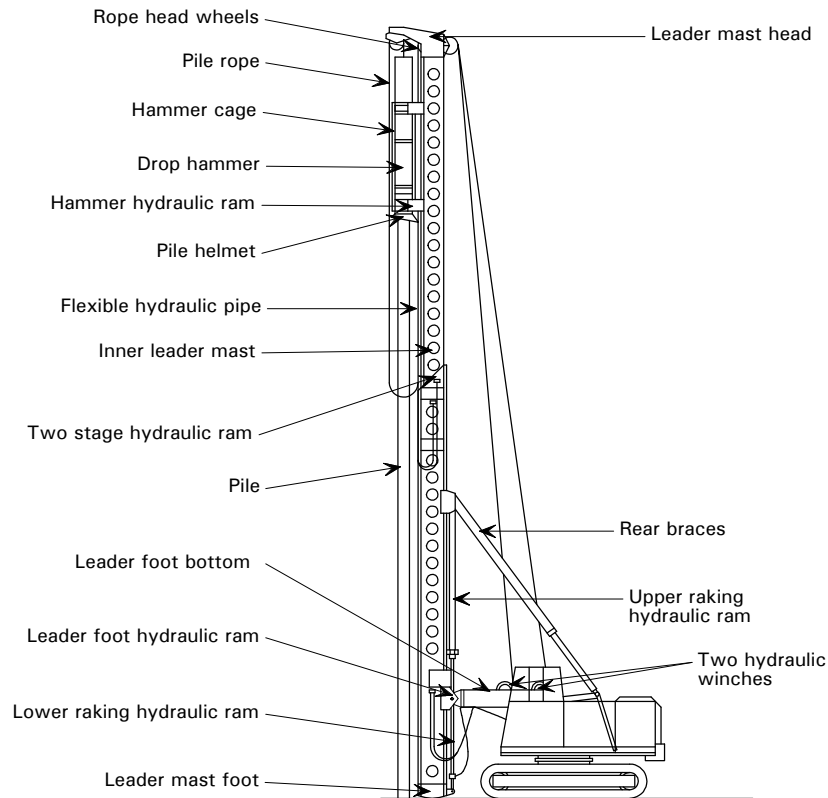
A hybrid installation method may also be used in appropriate conditions which adopts pitch and drive initially and panel driving to complete the process. This system may be beneficial where long piles are involved.

Panel driving requires guide frames (on at least two levels) to be erected before installation begins. Once these frames have been correctly positioned, the first pair of piles should be carefully pitched, plumbed and partly driven to form a guide for adjacent piles. The remaining panel of piles (usually 10 - 20 m run of wall per panel) is then pitched and interlocked. The last pair of piles pitched into the panel are then partially driven, followed by partial driving of the rest of the panel, working back towards the first pair in the panel. Care must be taken not to allow any departure from the vertical. The top guide walings can then be removed and all but the last pair of piles in the panel are driven to level; these will form the guide pair for the commencement of the next panel. This process is repeated panel by panel. This method has particular advantages if obstructions are encountered during installation. The remainder of the panel can be completed and the piles that are obstructed will then have support from neighbouring piles during any attempt to drive through the obstruction. If this is not successful, panel driving can continue whilst the obstruction is removed, minimising delays to construction.

Panel driving allows difficult driving conditions to be more capably dealt with and will generally result in better vertical alignment.

As the name suggests, pitch and drive entails the pitching of each pile or pair of piles and driving to finished level before repeating the operation. The plant required for this usually comprises a crane or specialist lifting machine with a leader attachment. The pile/s are winched into the leader and guided by restraints at the pile head and at a second position usually near to the base of the leader, ensuring that verticality is maintained throughout the installation. The advantage of this method is that speed of installation is maximised. However it would not be considered a suitable technique for installing long piles, where the ground conditions may give rise to hard driving or where obstructions are likely to be encountered during the drive.

As each driving operation is carried out, with the trailing clutch interlocked with the previously driven pile but the leading lock free, there can be a tendency for the piles to lean in the direction of wall progress. Only with care and frequent monitoring and correction can this problem be avoided. This method is well suited to the installation of bearing piles and king pile systems.



**Figure 13.1** *A steel pile driving leader rig*

### 13.3 Environmental factors: noise and vibration prediction

Increasing attention has been directed to environmental factors with regard to driven piles in recent years. Although the duration of the piling contract may be short in comparison with the whole contract period, noise and vibration perception may be more acute during the piling phase. Human perception is very intolerant of noise and vibration or shock transmitted through the ground, and tolerance requires careful prior education of the public. Efforts made to advise the public and to plan the precise times of driving carefully, can reassure those likely to be affected in the vicinity of a pile installation, and can result in the necessary cooperation.

In the UK, the Control of Pollution Act (1974) provides a legislative framework for, amongst other things, the control of construction site noise. The Act defines noise as including vibration and provides for the publication and approval of Codes of Practice, the approved code being BS 5228. Part 4 of the Code deals specifically with piling noise. This Code was revised in 1992 to include guidance on vibration.

Two relevant documents include the TRRL Research Report RR53 *Ground vibration caused by civil engineering works* <sup>[72]</sup>, and the British Steel publication *Control of vibration and noise during piling* <sup>[73]</sup>.

BS 6472 deals specifically with evaluation of human exposure to noise and vibration in buildings.

### 13.3.1 Noise from piling operations

Pile driving is perceived to be an inherently noisy operation because impact based methods of installation have historically been used. Typical data on noise levels produced by piling operations have been published by CIRIA Report No. 64 *Noise from construction and demolition sites - Measured levels and their prediction* <sup>[74]</sup>. These are discussed and interpreted in CIRIA Report PG9 *Noise and vibrations from piling operations* <sup>[75]</sup>. It is important to note that the modern pile installation equipment, such as the noise free pile jacking machines were not included as they post date these publications.

Impact driving of steel sheet piling is often noisy because the operation involves steel to steel contact. In areas where severe restrictions are placed on noise levels, pile vibratory or jacking equipment should be adopted. Such machines emit a different frequency and lower level of noise, which may be acceptable. Also, recent advances in noise reduction technology ensure that the auxiliary power pack emits negligible noise.

### 13.3.2 Ground vibrations caused by piling

It is widely recognised that noise and vibration, although related, are not amenable to similar curative treatment. In the main, noise from a site is airborne and consequently the prediction of noise levels is relatively straightforward, given the noise characteristics and mode of use of the equipment. On the other hand, the transmission of vibration is determined largely by site soil conditions and the particular nature of the structures involved. General guidance can be derived from the study of case histories of similar situations. Useful references on the subject of ground vibrations are provided by CIRIA Technical Note 142 *Ground-borne vibrations arising from piling* <sup>[76]</sup>, the publication *Dynamic ground movements - Man-made vibrations in ground movements and their effects on structures* <sup>[77]</sup>, BRE Digest No. 403 *Damage to structures from ground-borne vibration* <sup>[78]</sup>, and the references given in Section 13.3.1.

Prediction of peak-to-peak acceleration or velocity in real situations is not straightforward. Firstly, the energy transfer to soil is poorly understood and attenuation of high-frequency components is rapid. Secondly, the response of various forms of construction in adjacent inhabited buildings to ground vibrations is difficult to predict, and some structural details, e.g. floor spans that resonate, may lead to a magnification of the effect. The most widely accepted of these criteria are based on the peak particle velocity or the energy intensity of the vibrations induced in the soil adjacent to the foundations of a building. Empirical guidelines have been drawn up using these criteria to define various levels of damage. The recommendations given in DIN 4150 <sup>[79]</sup> and BS 5228 are listed in Tables 13.2 and 13.3.

**Table 13.2** *Maximum allowable peak particle velocity (DIN 4150)*

Description	Maximum velocity (mm/s)
Ancient ruins and listed buildings	2
Buildings with existing defects	5
Undamaged buildings in technically good condition	10
Strong buildings and industrial buildings	10-40

**Table 13.3** *Maximum allowable peak particle velocity (BS 5228)*

Description	Maximum velocity (mm/s)
<b>Residential buildings</b>	
- Intermittent vibration	10
- Continuous vibration	5
<b>Industrial and commercial buildings</b>	
Light and flexible	
- Intermittent vibration	20
- Continuous vibration	10
Heavy or stiff buildings	
- Intermittent vibration	30
- Continuous vibration	15
<b>Ancient ruins and listed buildings</b> <sup>[80]</sup>	

When considering reasonable limits for ground vibrations, the ambient background level of vibration should be assessed. In built-up areas, heavy traffic can cause surprisingly high intensities of vibration, and peak-to-peak velocities exceeding 3 mm/s have been recorded at a distance of 10 m from a road.

The use of empirical limits on velocity or acceleration in specifications and contracts necessitates the use of field instrumentation to observe the actual induced vibrations.

In general, human perception of vibrations occurs at levels that are low in comparison with the thresholds of risk for structural damage. BS 6472 sets out tables for vibrations in various types of accommodation for vibrations in the range 1 to 80 Hz. The vast majority of piling operations currently in use give rise to vibrational energy within this range.

Various expedients may be adopted to reduce the intensity of ground vibrations caused by piling.

Steel piles have low displacement and cause less ground disturbance than full displacement piles but further reduction of vibration can be obtained by preboring. As steel piling takes very little time to install and is an appropriate construction method for most soil types, it has advantages to the contractor.

Public irritation and objections to noise and vibration from piling installation can be minimised through cooperation gained by prior notice and careful advice and explanation by the contractor.

### 13.4 Selection of installation equipment

A wide variety of plant is available to facilitate the economical installation (and extraction for any temporary piles) of all types of conventional piling. The choice of piling plant must be considered well in advance of actual driving operations to ensure the best results from the outset and tables are included below which may assist in plant selection. They do not list every piece of plant available for economical pile installation but give an indication of the equipment type, which ground characteristics they are suited to, and other considerations relevant to their use.

**Table 13.4** *Plant type selection for granular soils (Piling Handbook<sup>[8]</sup>)*

Plant type	Density SPT 'N' Value			
	Loose	Medium dense	Dense	Very dense
	0-10	11 - 30	31 - 50	51 and over
Small Vibro's	A	B	B	D
Large Vibro's	B	A	B	C
Small Drop/Hyd Drop	A	A	B	C
Large Drop/Hyd Drop	C	B	A	A
Air hammers	A	A	C	D
Diesel hammers	C	B	A	A
Pushing Techniques	B	B	C	D
Vibro Pushing	A	B	B	C

A = Most suitable, B = Suitable, C = Not ideal, D = Not suited

**Table 13.5** *Plant type selection for cohesive soils (Piling Handbook<sup>[8]</sup>)*

Plant type	Cohesion $c_u$ kN/m <sup>2</sup>			
	Soft 0-45	Firm 46 - 80	Stiff 81 - 150	Very stiff 151 and over
Small Vibro's	C	D	D	D
Large Vibro's	B	C	D	D
Small Drop/Hyd Drop	A	B	B	C
Large Drop/Hyd Drop	C	A	A	A
Air hammers	A	B	C	D
Diesel hammers	A	A	A	B
Pushing Techniques	A	A	A	B
Vibro Pushing	B	B	C	D

A = Most suitable, B = Suitable, C = Not ideal, D = Not suited

**Table 13.6** *Plant type selection environmental issues (Piling Handbook)*

Plant type	Noise output	Vibration output	Vibration type
Small Vibro's	Low	Low	Cont
Large Vibro's	Medium	High	Cont
Small Drop/Hyd Drop	Medium	Medium	Int
Large Drop/Hyd Drop	High	High	Int
Air hammers	High	Low	Int
Diesel hammers	High	High	Int
Pushing Techniques	V Low	V Low	N/A
Vibro Pushing	Medium	Medium	Cont

**Key:**

V Low	Acceptable in all circumstances
Low	Acceptable in all but extreme circumstances
Medium	Borderline when occupied facade is close
High	Not acceptable close to occupied or susceptible facades
Cont	Continuous vibration during drive
Int	Intermittent vibration from each hammer blow

Note : Further guidance is available from Corus Construction Centre

**13.4.1 Hydraulic hammers**

Hydraulic hammers are available in a wide range of sizes to suit the pile to be driven. They operate effectively in both granular and cohesive soils and their efficiency and reliability have resulted in universal adoption of this type of hammer, at the expense of diesel and air hammers, where impact driving is necessary.

The energy imparted to the pile is governed by the size of the moving weight and the height of drop selected by the user. This permits the hammer to deliver low energy at the start of the drive when the pile is most vulnerable to misalignment and maximum energy at the end when it is fully supported.

Although described as an impact hammer, hydraulic drop hammers can be fitted with noise suppressing shrouds to reduce environmental impact.

**13.4.2 Vibratory devices**

When soils are predominantly granular and of low to medium density, a vibratory driver will be the most efficient and effective device for installing sheet piles. These devices incorporate a series of contra rotating eccentric masses within a housing which is clamped to the head of the pile causing it to vibrate in the vertical plane. During the driving process, the induced vibration effectively fluidises the ground adjacent to the pile reducing its bearing capacity and shaft friction. This in turn allows the pile to move downwards under its own weight plus that of the vibrodriver.

Vibratory driving is not effective in cohesive soil other than that which is very soft.

Vibrations introduced to the ground will be of uniform amplitude and being generated in the range 20 to 40Hz are above the natural frequency of most buildings. However, there has been concern that the vibrations caused during the start up and run down phases pass through the natural frequencies of the ground and adjacent buildings and, although of short duration, the vibration levels increase dramatically. The introduction of variable frequency and amplitude machines has successfully addressed this issue and further developments have seen the introduction of high frequency vibrators which cause the pile to resonate resulting in extremely high rates of installation in appropriate soils. These high frequency vibrators are also very appropriate for urban areas where adjacent structures are close to the piling line. In all cases, sound propagation is low.

### **13.4.3 Jacking systems**

The most recent development in pile installation technology is pile jacking which is a noise and vibration free form of pile installation. Installation is effected by mobilising the resistance between buried piles and the soil and using it as a reaction to push against to install another pile. This method of installation is particularly effective in cohesive soils where the skin friction developed is reasonable but end bearing resistance is relatively low. Monitoring of the hydraulic pressures during the installation process gives the operator a good understanding of the soil resistance encountered which can be used to verify design assumptions.

Although not ideally suited to granular soils, techniques for pile installation in such soils have been developed in conjunction with jetting or pre augering to reduce the driving resistance.

It is essential that a robust section is adopted when jacking techniques are to be used as the machines can impart substantial forces into the piles and too light a section may buckle instead of driving, or may deviate off line due to lack of stiffness.

This form of installation can be used to install sheet piles at a distance of approximately 750 mm from an existing structure to the wall centre line, if required.

## **13.5 Installation ancillaries**

### **13.5.1 Water jetting**

Driving in sand, silty sand, fine sandy gravel, and similar non-cohesive soils can usually be assisted by jets of water at high pressure directed into the ground alongside and below the pile. In some cases jetting is so effective that ordinary driving can almost be dispensed with, but at least the last metre of driving should be carried out without jetting, so that the lower part of the pile is in undisturbed soil.

It is best to have a pipe on either side of the pile as a single jet pipe can cause the pile to go off line. Occasionally a single pipe is positioned on the centre of each pile with a special nozzle opening at the same level as the pile toe. The water from the jetting pump is led through a flexible hose into pipes of 40 mm to 50 mm diameter, terminating in a nozzle or fishtail of slightly smaller cross-sectional area. It is not necessary to attach the jetting pipe to the pile, it often helps to surge it into the ground slightly ahead of the pile.



A possible serious disadvantage of jetting is that it may cause disturbance to the soil in the vicinity of the pile being installed, to nearby piles which have already been installed, or to adjacent existing structures.

### **13.5.2 Ground treatment**

Where the section choice is dominated by driving considerations, rather than moment capacity, it may be cost effective to consider pre-treatment of the ground to allow a lighter section choice for drivability. This can take a number of forms depending on the ground conditions and other site restrictions and includes:

- Pre-auguring along the proposed pile line: This can be carried out either in advance of the piling works or alongside part driven piles and allows hard ground to be broken up or extracted and replaced with a material more suitable for driving piles into.
- Driving of a heavier section along the proposed piling line: Installation and subsequent extraction of a driveable pile section along the piling line can be used to break up the ground prior to driving the working pile.



## 14 REFERENCES

**Note:** References to papers and other publications that are referenced by number in the text, are listed in Section 14.1.

Regulations and other Standards have not generally been given numbered references in the text; these are listed in Section 14.2.

### 14.1 Numbered references

1. THE INSTITUTION OF STRUCTURAL ENGINEERS  
Design and construction of deep basements  
Institution of Structural Engineers, 1975
2. CONSTRUCTION INDUSTRY RESEARCH INFORMATION ASSOCIATION  
Water-resisting construction - A guide - Safeguarding new and existing basements against water and dampness  
CIRIA Report 139, 1995
3. CONSTRUCTION INDUSTRY RESEARCH INFORMATION ASSOCIATION  
Guide to the design of waterproof basements  
CIRIA, Guide 5, February 1978 (reprinted 1984)
4. CONSTRUCTION INDUSTRY RESEARCH INFORMATION ASSOCIATION  
Water-resisting basements construction - A guide -Safeguarding new and existing basements against water and dampness, Summary Report  
Guide to the full Report (139)  
CIRIA Report 140, 1995
5. PULLER, M.  
Deep excavations - A practical manual  
Thomas Telford, 1996
6. SWAN, M., WRIGHT, G., and ROWBOTTOM, D.  
'Just-in-time' construction at Ford  
The Structural Engineer, Vol. 76, No. 13 7 July 1998
7. POTTS, D.M. and DAY, R. A.  
Use of sheet pile retaining walls for deep excavations in stiff clay  
Proceedings of the Institution of Civil Engineers, Part 1 1990, 899-927
8. BRITISH STEEL SECTIONS, PLATES & COMMERCIAL STEELS  
Piling handbook, Seventh Edition  
BS SP & CS, 1997  
(British Steel has now changed its name to Corus)

9. INSTITUTION OF STRUCTURAL ENGINEERS  
Code of Practice No 2: Earth retaining structures  
ISE, 1951
10. CONSTRUCTION INDUSTRY RESEARCH AND INFORMATION  
ASSOCIATION  
CIRIA Report 104: Design of retaining walls embedded in stiff clays  
CIRIA 1984
11. POTTS, D.M. and FOURIE, A.B.  
A numerical study of the effects of wall deformation on earth pressures  
International Journal of Numerical and Analytical Methods in  
Geomechanics, Vol. 10, No. 3, 1986, 383-404
12. PECK, R.B.  
Deep excavations and tunneling in soft ground  
Proceedings of the 7th International Conference on Soil Mechanics and  
Foundation Engineering, Mexico, 1969 pp 225-290
13. CLOUGH, W.G. and O'ROURKE, T.D.  
Construction induced movements of in-situ walls  
Proceedings of the Speciality Conference on Design and Performance  
of Earth Retaining Structures, Cornell, ASCE, Geotechnical Special  
Publication No. 25, edited by P. C. Lambe and L. A. Hansen,  
1990, pp 439-470
14. ST JOHN, H.D., HARRIS, D.I., POTTS, D.M. and FERNIE, R.  
Design study for a cantilever bored pile retaining wall with relieving  
slabs  
Proceedings of the Conference on Retaining Structures, Cambridge,  
ICE, Thomas Telford, London, 1992
15. FERNIE, R. and SUCKLING, T.  
Simplified approach for estimating lateral wall movement of embedded  
walls in UK ground  
In: Pre-print Volume of Proceedings of International Symposium on  
Geotechnical Aspects of Underground Construction in Soft ground, City  
University, London, 1996
16. BOND, A.  
ReWaRD<sup>®</sup> 2.5 (*Software package - the Retaining Wall Research and  
Design program*)  
Geocentrix Ltd, Banstead, 2000 (Tel: +44 (0)1737 373963)
17. BOWLES, J.E.  
Foundation analysis and design, Fourth Edition  
McGraw-Hill, 1988
18. BORIN, D.L.  
WALLAP anchored and cantilevered retaining wall analysis program:  
User's manual (Version 4)  
Geosolve, London, 1997

19. TERZAGHI, K.  
Evaluation of coefficients of subgrade reaction  
Geotechnique, Vol. 5, pp 297-326, 1955
20. SIMPSON, B, and DRISCOLL, R.  
Eurocode 7 a commentary  
Arup BRE, 1998
21. PECK, R.B.  
Advantages and limitations of the Observational Method in applied soil mechanics - Ninth Rankine Lecture  
Geotechnique, Vol. 19, pp 171-187, 1969
22. NICHOLSON, D., TSE, C. and PENNY, C.  
Ciria Report 185: The Observational Method in ground engineering: principles and applications  
Construction Industry Research and Information Association, 1999
23. DAWSON, M.P., DOUGLAS, A.R., LINNEY, L., FRIEDMAN, M. and ABRAHAM, R.  
Jubilee Line Extension, Bermondsey Station Box, design modifications, instrumentation and monitoring  
Proceedings of the Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, City University, London, 15-17 April, pp 49-54, 1996
24. IKUTA, Y., MARUOKA, M., AOKI, M. and SATO, E.  
Application of the Observational Method to a deep basement excavated using the top-down method  
Geotechnique, Vol. 44 (4), pp655-664, 1994
25. JAKY, J.  
The coefficient of earth pressure at rest  
Journal of the Society of Hungarian Architects and Engineers, Vol. 78 (22), pp 335-358, 1944
26. CANADIAN GEOTECHNICAL SOCIETY  
Canadian foundation engineering manual, Second Edition  
Canadian Geotechnical Society, Ottawa, pp 456, 1985
27. BURLAND, J.B., SIMPSON, B. and ST JOHN, H.D.  
Movements around excavations in London Clay  
Proceedings of the 7th European Conference on Soil Mechanics and Foundation Engineering, Brighton, Vol. 1, pp 13-29, 1979
28. KERISEL, J. and ABSI, E.  
Active and passive tables, Third Edition  
A. A. Balkema, Rotterdam, 1990
29. CAQUOT, A. and KERISEL, J.  
Tables for calculation of passive, active pressure and bearing capacity foundations (translated from French)  
Gauthier-Villars, Paris, 1948

30. SOKOLOVSKY, V.  
Statics of soil media  
Butterworth, 1960
31. POTTS, D.M. and BURLAND, J.B.  
A parametric study of the stability of embedded earth retaining structures  
TRRL Supplementary Report 783  
Transport and Road Research Laboratory, Crowthorne, 1983
32. DAY, R.A. and POTTS, D.M.  
A comparison of design methods for propped sheet pile walls  
The Steel Construction Institute, 1989
33. CARDER, D.R.  
A comparison of embedded and conventional retaining wall design  
using Eurocode 7 and existing UK design methods  
TRL Report 320  
Transport and Road Research Laboratory, Crowthorne, 1998
34. BIDDLE, A.R.  
Steel bearing piles guide (P165)  
The Steel Construction Institute, 1997
35. POTTS, D.M. and BOND, A.J.  
1004/Calculation of structural forces for propped retaining walls  
Proceedings of the 13th International Conference on Soil Mechanics  
and Foundation Engineering, New Delhi, India, 1994
36. ROWE, P.W.  
Anchored sheet pile walls  
Proceedings of the Institution of Civil Engineers, Part 1, 1952
37. ROWE, P.W.  
A theoretical and experimental analysis of sheet pile walls.  
Proceedings of the Institution of Civil Engineers 7, pp 32, 1955
38. ROWE, P.W.  
Sheet pile walls in clay  
Proceedings of the Institution of Civil Engineers 7, pp 629, 1957
39. LIAO, S.S.C. and NEFF, T.L.  
Estimating lateral pressures for design of excavation support.  
Proceedings of the Speciality Conference on Design and Performance  
of Earth Retaining Structures, Cornell, ASCE, Geotechnical Special  
Publication No 25, pp 489-509, 1991
40. CANADIAN GEOTECHNICAL SOCIETY  
Canadian foundation engineering manual, Second Edition  
Canadian Geotechnical Society, Ottawa, 1985
41. TAMARO, G.J. and GOULD, J.P  
Analysis and design of cast-in-situ walls (diaphragm walls),  
Proceedings of the Conference on Retaining Structures, Cambridge,  
ICE, Thomas Telford, London, 1992

42. WINKLER, E.  
Die Lehre von Elastizität und Festigkeit  
Prague, 1867
43. TERZAGHI, K.  
Evaluation of coefficients of subgrade reaction  
Geotechnique, Vol. 5, pp 297-326, 1955
44. TERZAGHI, K. and PECK, R.B.  
Soil mechanics in engineering practice, 2<sup>nd</sup> Edition  
John Wiley, New York, 1967
45. BJERRUM, L. and EIDE, O.  
Stability of strutted excavations in clay.  
Geotechnique, 6, pp 32-47, 1956
46. PECK, R. B.  
Deep excavations and tunnelling in soft ground, Proc. 7<sup>th</sup> Int. Conf. SM  
and FE, Mexico, 1969
47. O'ROURKE, T. D.  
Base stability and ground movement prediction for excavations in soft  
clay.  
Proceedings of Institution of Civil Engineers Conference on Retaining  
Structures, Cambridge. Thomas Telford, London, 1992
48. GOH, A. T. C.  
Estimating base heave stability for braced excavations in clay  
Journal of Geotechnical Engineering, American Society of Civil  
Engineers, Vol. 120,(8), 1994
49. NAVFAC-DM7 (1992)  
Design Manual: Soil Mechanics, Foundations and Earth Structures  
US Dept. of the Navy, Washington DC., 1992
50. CONSTRUCTION INDUSTRY RESEARCH INFORMATION  
ASSOCIATION  
CIRIA Report C517: Temporary propping of deep excavations -  
guidance on design  
CIRIA, 1999
51. GOLDBERG, D.T., JAWORSKI, W.E. and GORDON, M.D.  
Lateral support systems & underpinning. Volume 1: Design and  
construction. Volume 2: Design fundamentals. Volume 3: Construction  
methods  
Federal Highway Administration Reports FHWA-RD-75-128, p312;  
FHWA-RD-75-129, p237; FHWA-RD-75-130, p465, 1976  
(National Technical Information Service No PB257210, PB257211,  
PB257212)
52. FLEMING, W.G.K. , WELTMAN, A.J., RANDOLPH, M.F. and  
ELSON, W.K.  
Piling Engineering 2<sup>nd</sup> Edition  
Blackie Academic and Professional, 1992

53. EASTON, M.R., CARDER D.R. and DARLEY, D.  
Design guidance on soil berms as temporary support for embedded retaining walls. TRL Report 398  
Transport Research Laboratory, 1999
54. SELLMEIJER, J.B.  
Steel sheet pile seepage resistance  
Fourth International Landfill Symposium, Cagliari, Italy, 1993
55. Fire Safety Guide, No. 1: Fire Safety in Section 20 Buildings  
London District Surveyors' Association, 1990  
(Available from LDSA Publications, PO Box 266, Bromley, BR2 9ZN)
56. Fire protection for structural steel in buildings, Revised 2<sup>nd</sup> edition  
Association of Specialist Fire Protection Contractors and Manufacturers Ltd, The Steel Construction Institute and The Fire Test Study Group, 1992
57. MORRIS, W.A.  
Guidelines for the construction of fire resisting structural elements  
Building Research Establishment, 1988
58. THE STEEL CONSTRUCTION INSTITUTE  
Structural fire design: Off-site applied thin film intumescent coatings.  
Part 1: Design guidance; Part 2: Model specification (P160)  
Edited by: E.Yandzio, J Dowling and G Newman  
SCI, 1996
59. BUILDING RESEARCH ESTABLISHMENT  
BRE Digest 317: Fire resistant steel structures: Free standing blockwork-filled columns and stanchions  
BRE, 1986
60. NEWMAN, G.M.  
Technical Report: Fire resistance of web-infilled steel columns (P124)  
The Steel Construction Institute, 1992
61. BAILEY, C.G., NEWMAN, G.M. and SIMMS, W.I.  
The design of steel framed buildings without applied fire protection (P186)  
The Steel Construction Institute, 1999
62. Design manual for concrete filled columns  
Corus, 1992
63. NEWMAN, G.M., ROBINSON, J.T. and BAILEY, C. G.  
Fire Safe Design: A new approach to multi-storey steel-framed buildings (P288)  
The Steel Construction Institute, 2000
64. MULLETT, D.L  
Slimflor design and construction (P110)  
The Steel Construction Institute, 1992



65. MULLETT, D.L. and LAWSON, R.M.  
Design of *Slimflor* fabricated beams using deep composite decking (P248)  
The Steel Construction Institute, 1993
66. MULLETT, D.L.  
Design of RHS *Slimflor* edge beams (P169)  
The Steel Construction Institute, 1997
67. LAWSON, R.M., MULLETT, D.L. and RACKKHAM, J.W.  
Design of asymmetric *Slimflor* beams using deep composite decking (P175)  
The Steel Construction Institute, 1997
68. NEWMAN, G.M  
Technical Report: Fire resistance of shelf angle floor beams to BS 5950 Part 8 (P126)  
The Steel Construction Institute, 1993
69. INSTITUTION OF CIVIL ENGINEERS  
Specification for piling and embedded retaining walls  
ICE, 1996
70. FEDERATION OF PILING SPECIALISTS  
Specification for steel sheet piling  
FPS, London, 1991
71. TECHNICAL EUROPEAN SHEET PILING ASSOCIATION  
Installation of steel sheet piles  
TESPA, L-2930 Luxembourg, 1993
72. TRANSPORT AND ROAD RESEARCH LABORATORY  
Ground vibration caused by civil engineering works  
TRRL Report RR53, Crowthorne, 1986
73. BRITISH STEEL  
Control of vibration and noise during piling, 1997
74. CONSTRUCTION INDUSTRY RESEARCH AND INFORMATION ASSOCIATION  
CIRIA Report 64: Noise from construction and demolition sites - Measured levels and their prediction  
CIRIA, London, 1977
75. WELTMAN, A.J.  
Noise and vibrations from piling operations  
DOE and CIRIA Piling Development Group  
Construction Industry Research and Information Association Report PG9, 1980
76. HEAD, J.M. and JARDINE, F.M.  
Ground-borne vibrations arising from piling  
Construction Industry Research and Information Association Technical Note 142, 1992

77. SKIPP, B.O.  
Dynamic ground movements - Man-made vibrations in ground movements and their effects on structures  
Blackie, Glasgow and London, pp 381-434, 1984
78. BUILDING RESEARCH ESTABLISHMENT  
Damage to structures from ground-borne vibration  
BRE Digest No. 403, 1995
79. DEUTSCHES INSTITUT FUR NORMUNG  
DIN 4150: Part 3: Structural vibration in buildings: Effects on structures DIN, 1986 (draft 1997 not yet translated into English)
80. WATTS, G.R.  
Case studies of the effects of traffic induced vibrations on heritage buildings. Research Report 156  
Transport and Road Research Laboratory, Crowthorne 1988

## 14.2 Regulations and Standards

### **REGULATIONS**

The Building Regulations 1991 (SI 1991 No. 2768)  
The Stationery Office, 1991

The Building Regulations 1991, Approved Documents:

B2/3/4 Fire Spread 2000

Cl/2/3/4 Site preparation and contaminants 1992

C4 Resistance to weather and ground moisture 1992

D1 Cavity insulation 1985

F1 Means of ventilation 1990

F2 Condensation 1990

H1 Sanitary pipework and drainage 1990

H3 Rainwater drainage 1990

M Access for the disabled 1992

DEPARTMENT OF THE ENVIRONMENT  
Chapter 40 United Kingdom Control of Pollution Act  
The Stationery Office, 1974

### **BRITISH STANDARDS INSTITUTION**

BS 4 Structural steel sections

BS4-1: 1993 Specification for hot-rolled sections

BS 449 Specification for the use of structural steel in building

BS449-2: 1969: Metric units

BS 476 Fire tests on building materials and structures  
BS 476-4: 1970 Non-combustibility test for materials  
BS 476-4:1987 Method for determination of the fire resistance of elements of construction (general principles)  
BS 476-21:1987 Methods for determination of the fire resistance of loadbearing elements of construction

BS 1377 Methods of test for soils for civil engineering purposes  
BS 1377-3:1990 Chemical and electro-chemical tests

BS 5228 Noise control on construction and open sites  
BS 5228-4:1992 Code of practice for noise and vibration control applicable to piling operations

BS 5306 Fire extinguishing installations and equipment on premises  
BS 5306-0:1986 Guide for the selection of installed systems and other fire equipment

BS 5400 Steel, concrete and composite bridges  
BS 5400-2:1976 Specification for loads

BS 5950 Structural use of steelwork in building  
BS 5950-1:2000 Code of practice for design. Rolled and welded sections  
BS 5950-2:1992 Specification for materials, fabrication and erection: hot-rolled sections  
BS 5950- 3.1:1990 Design in composite construction. Code of practice for design of simple and continuous composite beams  
BS 5950-4:1994 Code of practice for design of composite slabs with profiled steel sheeting  
BS 5950- 6:1995 Code of practice for design of light gauge profiled steel sheeting  
BS 5950-7:1992 Specification for materials and workmanship: cold formed sections  
BS 5950-8:1990 Code of practice for fire resistant design

BS 6349 Maritime structures  
BS 6349-1:2000 Code of practice for general criteria  
BS 6349-2:1988 Design of quay walls, jetties and dolphins

BS 6472:1992 Guide to evaluation of human exposure to vibration in buildings (1 Hz to 80 Hz)

BS 7543:1992 Guide to durability of buildings and building elements, products and components.

BS 8002:1994 Code of practice for earth retaining structures

BS 8004:1986 Code of practice for foundations BSI, 1986

BS 8007:1987 Code of practice for design of concrete structures for retaining aqueous liquids

BS 8102:1990 Code of practice for protection of structures against water from the ground

BS 8110 Structural use of concrete

BS 8110-1:1997 Code of practice for design and construction

BS 8110-2:1985 Code of practice for special circumstances

BS 8202 Coatings for fire protection of building elements

BS 8202-1:1995 Code of practice for the selection and installation of sprayed mineral coatings

BS 8202-2:1992 Code of practice for the use of intumescent coating systems to metallic substrates for providing fire resistance

BS EN 10025:1993 Hot rolled products of non-alloy structural steels. Technical delivery conditions

BS EN 10248 Hot rolled sheet piling of non alloy steels

BS EN 10248-1:1996 Technical delivery conditions

BS EN 10248-2:1996 Tolerances on shape and dimensions

BS EN 10249 Cold formed sheet piling of non alloy steels.

BS EN 10249-1:1996 Technical delivery conditions

BS EN 10249-2:1996 Tolerances on shape and dimensions

DD ENV 1994-1-1: Eurocode 4 Design of composite steel and concrete structures. General rules and rules for buildings (together with United Kingdom National Application Document 1994)

DD ENV 1997-1: 1995 Eurocode 7 Geotechnical design. General rules (together with United Kingdom National Application Document)

BS EN 12699:2001 Execution of special geotechnical work. Displacement piles

### ***EUROCODES***

#### ***EUROPEAN COMMITTEE FOR STANDARDIZATION***

ENV 1991: Eurocode 1 Basis of design and actions on structures

ENV 1991-1:1994 Basis of design, 1994

ENV 1992 Eurocode 2 Design of concrete structures

ENV 1992-1-1:1994 General rules and rules for buildings

ENV 1992-1-2:1995 Structural fire design

ENV 1993 Eurocode 3: Design of steel structures

ENV 1993-1-1:1998 General rules and rules for buildings

ENV 1993-1-2:1995 Structural fire design

ENV 1993-1-5:1998 Piling

ENV 1994 Eurocode 4 Design of composite steel and concrete structures

ENV 1994-1-1:1992 General rules and rules for buildings

ENV 1994-1-2:1994 Structural fire design

ENV 1997 Eurocode 7 Geotechnical design

ENV 1997-1:1994 General rules

EN 12063:1999 Execution of special geotechnical work - Sheet Pile Walls

EN 12699:2001 Execution of special geotechnical works - Displacement piles

### ***INTERNATIONAL STANDARDS ORGANISATION***

ISO 12944: Paints and varnishes. Corrosion protection of steel structures by protective paint systems

ISO 12944: Part 2:1998 Classification of environments

### ***DIN STANDARDS***

DEUTSCHES INSTITUT FUR NORMUNG

DIN 4150: Part 3: Structural vibration in buildings: Effects on structures DIN, 1986 (draft 1997 not yet translated into English)



## **APPENDIX A Sheet pile types available from UK manufacturers**

Steel piles suitable for basement walls are available in several profiles of varying stiffness and shape to suit different purposes and differing degrees of propping. Most commonly, these can be classed as steel sheet piles, high modulus piles, and box piles. The type of pile profile that is most appropriate for a basement depends on the height of the basement wall, the distance between lateral restraints and the magnitude of the axial and lateral loads.

### **A.1 Sheet piles**

Two steel sheet profiles, designated as 'U' and 'Z', are the most common forms available. In the UK, Larssen (U profile) and Frodingham (Z profile) sections are commonly used and each type has its own characteristics which, in certain situations, can influence the choice.

Sheet pile sections are generally supplied in two grades of steel, S270GP and S355GP to BS EN 10248:1995, with minimum yield strengths of 270 N/mm<sup>2</sup> and 355 N/mm<sup>2</sup> respectively. Other grades are available on request.

## A.1.1 Larssen sections

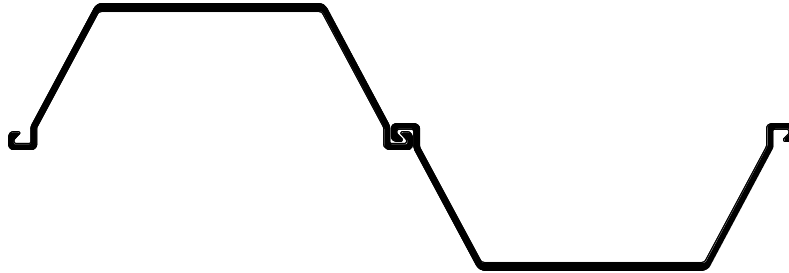


Figure A.1 A Larssen sheet pile

Table A.1 Dimensions and properties for LX and Larssen Sections

Section	b	h	d	t	f	Sectional Area of wall	Mass		Moment of inertia	Section Modulus
	mm	mm	mm	mm	mm		kg/m	kg/m <sup>2</sup> of wall		
LX8	600	310	8.2	8.0	250	116.0	54.6	91.0	12863	830
LX12	600	310	9.7	8.2	386	136.0	63.9	106.5	18727	1208
LX12d	600	310	10.0	8.3	386	139.0	65.3	108.8	19217	1240
LX12d10	600	310	10.0	10.0	382	155.0	72.9	121.5	19866	1282
LX16	600	380	10.5	9.0	365	157.0	74.1	123.5	31184	1641
LX20	600	430	12.5	9.0	330	177.0	83.2	138.7	43484	2023
LX20d	600	450	11.2	9.7	330	179.0	84.3	140.5	45197	2009
LX25	600	460	13.5	10.0	351	202.0	95.0	158.3	57233	2488
LX25d	600	450	15.0	11.0	326	212.0	100.0	166.7	57246	2544
LX32	600	460	19.0	11.0	340	243.0	114.4	190.7	73802	3209
LX32d	600	450	21.5	13.0	320	269.0	126.5	210.8	75325	3348
LX38	600	460	22.5	14.5	337	298.0	140.4	234.0	87511	3805
Larssen:										
6W	525	212	7.8	6.4	333	109.0	44.8	85.3	6508	614
20Wd	525	400	11.3	10.0	333	196.0	80.7	153.7	40574	2029
GSP2	400	200	10.5	8.6	266	157.0	49.4	123.5	8756	876
GSP3	400	250	13.5	8.6	270	191.0	60.1	150.3	16316	1305
GSP4	400	340	15.5	9.7	259	242.0	76.1	190.3	38742	2279
6 - 42	500	450	20.5	14.0	329	339.0	133.0	266.0	94755	4211
6(122)	420	440	22.0	14.0	250	371.0	122.5	291.7	92115	4187
6(131)	420	440	25.4	14.0	250	396.0	130.7	311.2	101598	4618
6(138.7)	420	440	28.6	14.0	251	419.0	138.3	329.3	110109	5005



## A.1.2 Frodingham sections

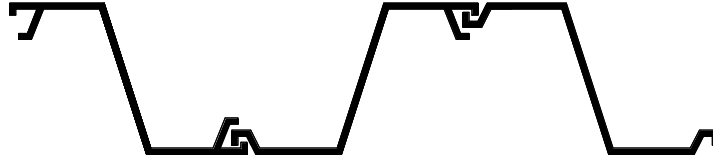


Figure A.2 A Frodingham sheet pile

Table A.2 Dimensions and properties for Frodingham Sections

Section	b	h	d	t	f1	f2	Sectional Area of wall	Mass		Moment of inertia	Section Modulus
	mm	mm	mm	mm	mm	mm		kg/m	kg/m <sup>2</sup> of wall		
1BXN	476	143	12.7	12.7	77	122	170	63.4	133.2	4947	692
1N	483	170	9.0	9.0	107	142	126	48.0	99.4	6072	714
2N	483	235	9.7	8.4	91	146	145	54.8	113.5	13641	1161
3NA	483	305	9.7	9.5	90	148	166	62.7	129.8	25710	1687
4N	483	330	14.0	10.4	75	128	218	82.7	171.2	39869	2415
5	426	311	17.1	11.9	87	119	302	101.0	237.1	49329	3171

## A.2 Box piles

Box piles are formed by welding two or more sheet pile sections together. See Figure A.3. Both Larssen and Frodingham sheet piles can be used. They can be introduced into a line of sheet piling at any point where local heavy loads are to be applied or can be used in a continuous manner if stiffer walls than sheet pile walls are required. The appearance of the wall is unaffected by the box piles.

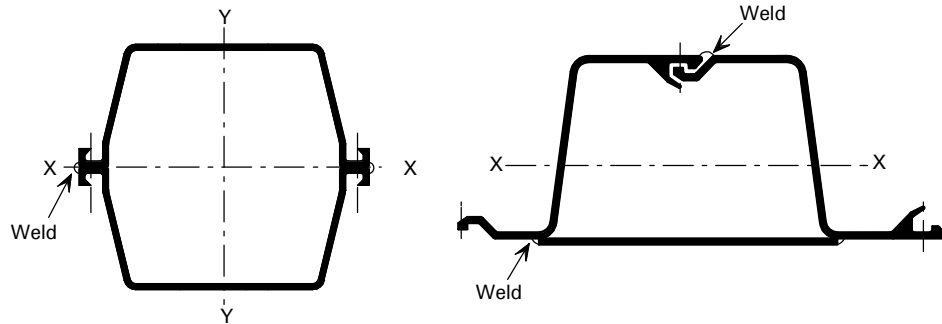


Figure A.3 Types of box piles

Table A.3 Properties for selected Larssen box piles

Section	Section Modulus cm <sup>3</sup>	
	XX axis cm <sup>3</sup>	YY axis cm <sup>3</sup>
LX25	3424	3257
LX32	4377	3544
LX38	5271	4374
6-42	4920	3902

Table A.4 Properties for Frodingham 4N box piles

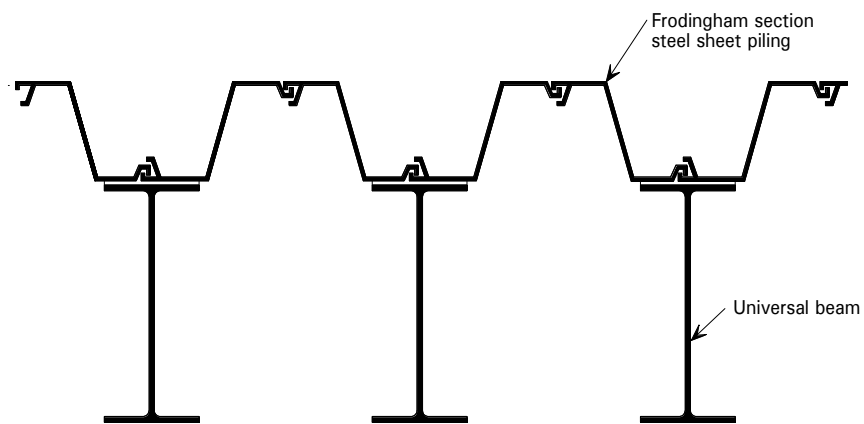
Section	Elastic section modulus cm <sup>3</sup>	
	Plated box	Double box
Frod 4N	2662	5805

### A.3 High modulus piles

**Table A.5** *Dimensions and properties for selected Frodingham 4N high modulus piles*

Universal beam		Centres of UB's mm	Mass		Combined moment of inertia cm <sup>4</sup> /m	Elastic section modulus cm <sup>3</sup> /m
Serial size mm	Mass kg/m		kg/m	kg/m <sup>2</sup>		
533 x 210	101	966	267	276	259478	4832
610 x 305	147	966	314	326	397108	7198
762 x 267	176	966	338	350	584576	9026
838 x 292	194	966	359	372	732365	10621
914 x 305	253	966	419	433	1005797	14254
914 x 419*	388	966	522	540	1353126	21435

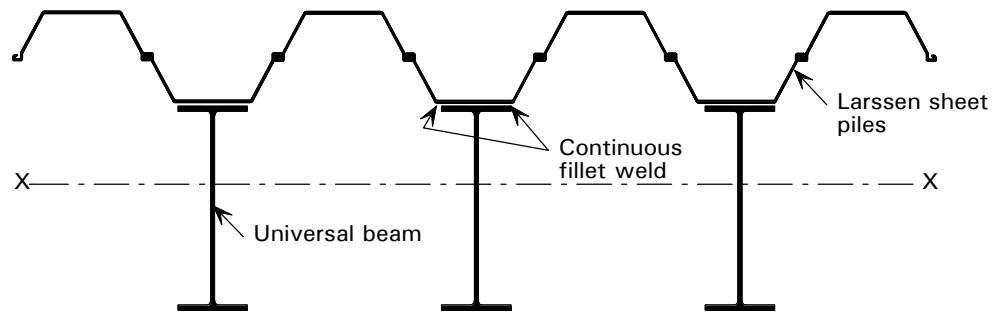
\* Denotes beam section with one flange reduced to 310 mm to facilitate fabrication



**Figure A.4** *Frodingham high modulus piles*

**Table A.6** *Dimensions and properties for selected LX20 high modulus piles*

Universal beam		Centres of UB's mm	Mass		Combined moment of inertia cm <sup>4</sup> /m	Elastic section modulus cm <sup>3</sup> /m
Serial size mm	Mass kg/m		kg/m	kg/m <sup>2</sup>		
686 x 254	125	1200	208.4	243	200426	3945
762 x 267	147	1200	230.1	261.1	267505	4918
838 x 292	176	1200	259.1	285.3	363085	6273
914 x 305	253	1200	336.6	349.8	558248	9453
1016 x 305	222	1200	305.2	323.7	555491	8673
1016 x 305	487	1200	570.2	544.5	1132123	16938

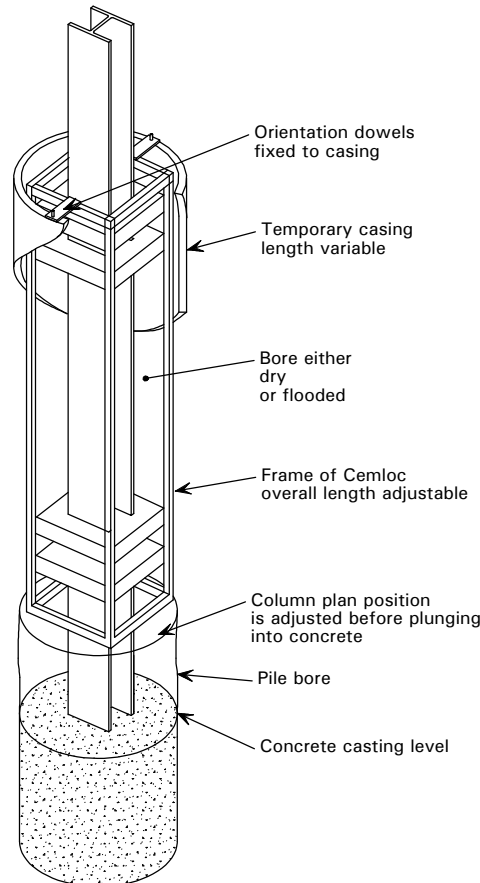


**Figure A.5** *Larssen high modulus piles*

Further details and properties for the full range of sheet pile sections can be obtained from the Corus Construction Centre.

## APPENDIX B CEMLOC™ System

The CEMLOC™ system, patented by Kvaerner Cementation Foundations, has been specifically developed to accurately place plunged columns into piles. Figure B.1 shows a schematic of the system.

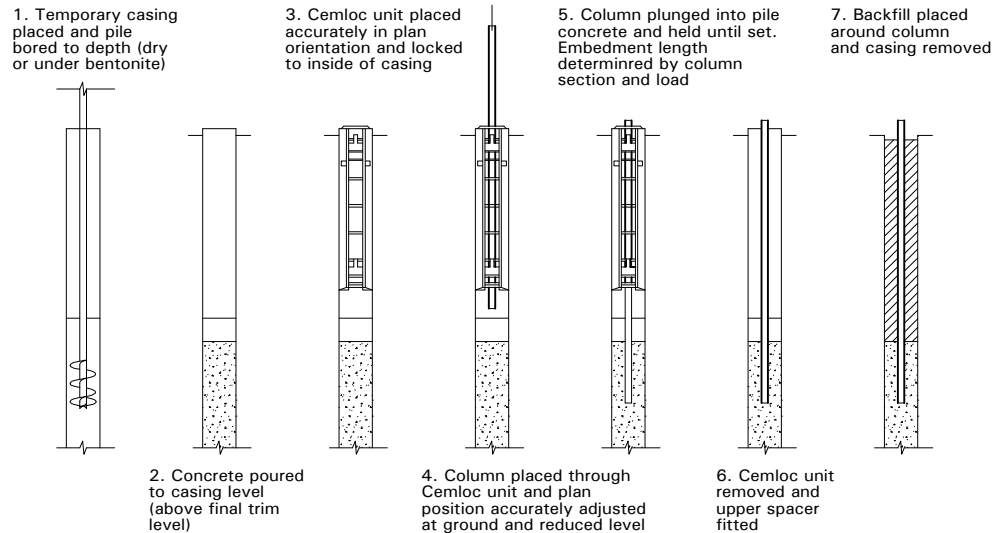


**Figure B.1** *The CEMLOC™ system Kvaerner Cementation Foundations*

Standard techniques are used for constructing a rotary bored pile. After the pile bore has been completed, the reinforcement is installed. One of two different techniques are used, depending on circumstances. In one technique the concrete is placed by tremie, to the low casting level. The CEMLOC™ jig is lowered into the temporary casing and the correct orientation achieved by means of locating dowels. The unit is then locked onto the inside of the casing using hydraulic rams near the head and near the base of the jig. The column is lowered through the jig and held with its base above the cast concrete level. The plan location of the column, at ground level and reduced level, is adjusted by the steering system and confirmed by surveying techniques. The column is then plunged into the fresh concrete. Alternatively where protrusions on the flanges (shear studs) are required, along the embedment length, and/or at basement floor levels, the column can be accurately positioned first and then the concrete placed by pump line or tremie.

For both techniques, the jig is left in position until the concrete has gained sufficient strength. It is then withdrawn and the annulus around the column is

filled with granular material. In certain circumstances, layers of weak concrete fill may be placed to give additional lateral support. Finally, the temporary casing is withdrawn. The installation sequence is shown schematically in Figure B.2.



**Figure B.2** *Installation sequence for Cemloc plunge column system*

This approach has many advantages, including a faster construction cycle, no requirement for a permanent casing to ground level and no requirement to prepare the pile cap. Also there is no special requirement for safety equipment and procedures since the works are carried out at ground level. The tolerances expected for fabricated structural steel can be achieved, resulting typically in levels of accuracy of  $\pm 10$  mm in plan position and 1:600 verticality.

The plunge column system allows the load to be transferred from the column to the pile via the embedment length of steel column/concrete pile. As no definitive design procedures to date have been presented in British Standards and little but not directly relevant information is given in ENV 1994-1, numerous design methods have been postulated by contractors and consultants. Kvaerner Cementation Foundations base their design on bond between steel section surface and the concrete and do not consider protrusions (such as shear studs), as their tests did not indicate any benefit in load transfer where protrusions are used. Kvaerner Cementation Foundation postulate the embedment length is given by:

$$L_e = \frac{2W}{L_p \times 0.35\sqrt{f_{cu}}}$$

where:

- $W$  is the unfactored column load
- $L_p$  is the perimeter length of column section
- $f_{cu}$  is the characteristic strength of concrete.

The maximum length  $L_e$  is limited to 5 m. If this length does not generate sufficient bond area, then additional flanges are welded to the steel section.



